ASPIRE®

THE CONCRETE BRIDGE MAGAZINE

SPRING 2025

www.aspirebridge.org

Palmer Engineering

Providing civil engineering services across the Appalachian region for 55 years

U.S. ROUTE 395 NORTH SPOKANE CORRIDOR SPOKANE RIVER CROSSING
Spokane, Washington

MOUNT VERNON VIADUCT
San Bernadino, California

Presorted Standard
Postage Paid
Lebanon Junction, KY
Permit No. 567

Photo: Palmer Engineering.

Photo: Knife River Prestress.

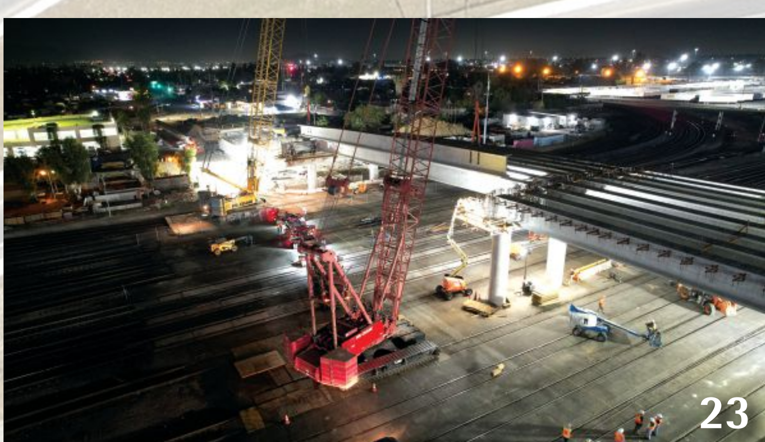


Photo: Dany Schimpf Photography.

CONTENTS

Features

Departments

Photo: Palmer Engineering.

CORRECTION

A reader has noted an error in "Camber Variability in Prestressed Concrete Bridge Beams," which was published in the Spring 2015 issue of ***ASPIRE***®. On page 39, the eccentricity of prestress force ***e*** is missing in the second sentence following Eq. 1. That sentence should be as follows:

> The curvatures ϕ_1 and ϕ_2 are equal to Pe/EI for Sections 1 and 2 at the location at which prestress is effective at the end of the girder (dimension ***a***) and at midspan, respectively. The values of ***P***, ***e***, ***E***, and ***I*** are the prestress force, its eccentricity, the concrete modulus of elasticity, and the cross section moment of inertia, respectively, at the section considered for the curvature calculation.

Advertisers' Index

EDITORIAL


Photo: PCI.

More Greek for You to Consider

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

In an editorial written several years ago, I discussed the Greek concept of *Philotimo*, which is essentially the idea of doing what is right (see the Spring 2020 issue of *ASPIRE*®). To my surprise, that article was republished by the National Society of Professional Engineers' *PE Magazine*, reaching a larger engineering audience—and for that I am grateful.

Today, it seems appropriate to share another Greek word, *Phronē*, with all of you. *Phronē* means "to intensely interest oneself in." There are myriad topics that require our attention and, in some cases, a spirit of *Phronē*. Two topics that have recently grabbed my attention are new, larger loadings from truck platoons and the concrete industry's shift toward the use of portland-limestone (Type IL) cement. In this issue of *ASPIRE*, you will find another article in the series by Dr. Jay Puckett on truck-platoon loading, and a recent *PCI Journal* article[1] by Dr. R. Douglas Hooten (University of Toronto) and Dr. Kyle A. Riding (University of Florida) provides an enormous amount of detail about the impacts of Type IL cement on strength gain and other concrete properties. I think you will find these papers to be very interesting. Dr. Riding previously published a high-level article on the topic of portland-limestone cement in the Winter 2022 issue of *ASPIRE*. Today, we know much more.

In their *PCI Journal* article, Hooten and Riding explain the societal concerns about environmentalism that are motivating the cement and concrete industries to take action. They mathematically show how the use of Type IL cement can reduce a structure's carbon footprint by 10%, and they address previous concerns regarding short- and long-term prestress losses. Hooten's and Riding's leadership in this area is highly commendable.

The challenges of embracing a spirit of *Phronē* and remaining engaged with today's developing workforce continually resonate with me. How do I, and how do we, remain "current" and stay informed about relevant topics? The advantage of being "seasoned" in this profession is the perspective one gains about questions like this. Although engineering techniques change and evolve, our reliance on theory, calculations, and results remains a foundational principle.

When I was first becoming established in the industry, my interactions with senior bridge engineers—even when they seemed skeptical about my work—were very helpful. When the engineers senior to me covered my calculations with red and yellow pencil marks, it felt like they were challenging me, asking, "Why did you do this?" Some old-timers were troubled just by the presence of younger engineers, and the atmosphere they created as they marked up my plans and calculations was not particularly encouraging of dialogue. I really wanted to ask to see *their* calculations, but that type of question wasn't welcome. Still, I learned from their open and implied critiques of my work.

Now that I am a senior engineer, I try to set a different tone. I seek opportunities to encourage others and to share my thoughts. Those of you who know me just shake your heads, knowing that I rarely hold my thoughts to myself. That's true, but seasoning and perspective allow for a great deal of reflection and *Phronē*. I continue to find our industry intensely interesting, and its progress requires our engagement and attention.

In my Winter 2025 *ASPIRE* editorial, "Stay Vigilant and Innovate," I discussed the need to stay engaged with suppliers and remain vigilant about the quality of concrete materials. I hope that you have signed up for a National Concrete Bridge Council webinar or in-person course so that you can collaborate with the experts in our industry.

I also hope that you will find a concrete bridge topic that drives your interest and immerse yourself in it. Then, once you realize your *Phronē*, I hope you will share your discoveries with your bridge squad at your place of work or at a professional engineering society meeting in your area.

Reference

1. Hooten, R. D., and K. A. Riding. 2025. "Type IL Cement Use in Precast, Prestressed Concrete." *PCI Journal* 70 (2): 22–36. https://doi.org/10.15554/pcij70.2-04.

Editor-in-Chief
William N. Nickas • wnickas@pci.org

Managing Technical Editor
Dr. Richard Miller

Technical Editors
Monica Schultes, Angela Tremblay, Dr. Krista M. Brown

Program Manager
Trina Brown • tbrown@pci.org

Associate Editor
Thomas L. Klemens • tklemens@pci.org

Copy Editor
Elizabeth Nishiura

Layout Design
Walter Furie

Cover
Palmer Engineering designed a cost-effective bridge with a profound skew to carry U.S. Route 20 over the Norfolk Southern railroad in Ashtabula County, Ohio. Photo: Palmer Engineering.

Ad Sales
Scott Cunningham • scunningham7@aol.com
(678) 576-1487 (mobile)
(770) 913-0115 (office)

Reprints
lisa scacco • lscacco@pci.org

Publisher
Precast/Prestressed Concrete Institute
Bob Risser, President

If you need to update your contact information with us or have a suggestion for a project or topic to be considered for *ASPIRE*, please send an email to info@aspirebridge.org.

Postmaster: Send address changes to *ASPIRE*, 8770 W. Bryn Mawr Ave., Suite 1150, Chicago, IL 60631. Standard postage paid at Chicago, IL, and additional mailing offices.

ASPIRE (Vol. 19, No. 2), ISSN 1935-2093, is published quarterly by the Precast/Prestressed Concrete Institute.

https://doi.org/10.15554/asp19.2

Lightweight Concrete Provides Solutions for More Efficient Construction Operations

Lightweight concrete is most often used to provide more efficient bridge designs. But lightweight concrete can also be used to improve construction operations as illustrated in an article in this issue of *ASPIRE* about the spliced concrete girder bridge carrying the North Spokane Corridor (NSC) over the Spokane River. For this bridge, lightweight concrete was used to reduce the weight of the haunched pier segments to improve in-plant girder handling and truck hauling.

For the NSC project, the maximum fresh density of the lightweight concrete was limited to 128 lb/ft^3. Lightweight concrete mixtures, including the one used in the NSC project, are usually proportioned using lightweight coarse aggregate and conventional sand. However, if the density required to achieve the design or construction objectives is not this low, a mixture often referred to as a "specified density mix" can be used where lightweight and normal weight coarse aggregates are blended to achieve any density between lightweight concrete and normal weight concrete.

Lightweight concrete haunched pier girder segment for the North Spokane Corridor Bridge over the Spokane River. Photo credit: Jordan Pelphrey.

Another example of using lightweight concrete to address construction issues is the Kentucky River Bridge at Gratz, KY, that currently holds the record for the longest precast concrete spliced girder bridge span in the U.S. at 325 ft (see Winter 2011 issue of *ASPIRE*). The project used a lightweight concrete mixture with an unreinforced concrete density of 125 lb/ft^3 for the drop-in segments that "reduced the weight and made the girders easier to maneuver to and from the barges during loading and erection." Pairs of girders were barged to the site and erected using strand jacks.

More information on using lightweight concrete for bridges is available on the ESCSI webpage: www.escsi.org

CONTRIBUTING AUTHORS

Dr. Atorod Azizinamini is the Vasant Surti Professor of Civil Engineering at Florida International University and director of IBT/ABC-UTC.

Dr. Mark A. Finlayson is an associate professor of Computer Science at Florida International University, specializing in natural language processing and cognitive science.

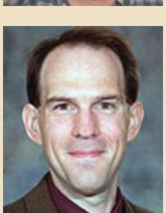
Dr. David Garber is a senior structural engineer with the Federal Highway Administration's Resource Center providing concrete bridge expertise to internal and external stakeholders.

Dr. Benjamin Graybeal is the Federal Highway Administration's team leader for bridge engineering research.

Dr. Rafic G. Helou is a research structural engineer with the Federal Highway Administration's Office of Infrastructure Research and Development.

Dr. Jay Puckett is a professor in the Durham School for Architectural Engineering and Construction at the University of Nebraska–Lincoln.

Jacob T. Rausch is a precast concrete engineer at Coastal Precast Systems in Wilmington, N.C.



Dr. Joshua Steelman is an associate professor in the Department of Civil and Environmental Engineering at the University of Nebraska–Lincoln.

CONCRETE CALENDAR

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

March 30–April 2, 2025
ACI Concrete Convention
Sheraton Centre Toronto
Toronto, Ontario, Canada

April 7–10, 2025
CRSI Spring Business and Technical Meeting
Marriott Marquis Atlanta
Atlanta, Ga.

April 15–16, 2025
NCBC Prestressed Concrete Bridge Seminar: Concepts for Extending Spans
Hyatt Regency Columbus
Columbus, Ohio

April 28–May 2, 2025
PTI Certification Week
Salt Lake City Marriott
University Park
Salt Lake City, Utah

May 4–7, 2025
PTI Convention
Sheraton Phoenix Downtown
Phoenix, Ariz.

May 31–June 6, 2025
AASHTO Committee on Bridges and Structures Meeting
Dallas, Tex.

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

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

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

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

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

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

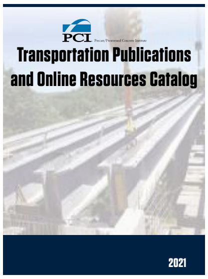
October 20–25, 2025
PTI Certification Week
Atlanta Marriott Northeast/Emory
Atlanta, Ga.

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

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

PCI Now Offers eLearning Modules

28 Courses on Design and Fabrication of Precast, Prestressed Concrete Bridges



Download the *Transportation Publications and Online Resources Catalog* at https://www.pci.org/PCI_Docs/Design_Resources/Transportation_Resources/2021%20Transportation_Catalog.pdf

PCI eLearning Courses

For information on how to use PCI's eLearning site, follow this link: https://youtu.be/Pbrlz4lflw8

PCI eLearning is useful for engineers at all stages of their careers. Professors may require students to take eLearning courses to learn more about specific topics, and it is suggested that novice and mid-level-experienced engineers take in numerical order the T100 courses, and then the T500 and T510 courses. The remaining courses focus on specialized areas. Although more experienced engineers may elect to skip topics in eLearning courses, they can refresh their knowledge by reviewing specific modules and may wish to take the tests to earn PDHs or LUs.

T100 series course is based on Chapters 1 through 9 of *PCI Bridge Design Manual*, 3rd ed., 2nd release (MNL-133).

T200 series courses are based on the *State-of-the-Art Report on Full-Depth Precast Concrete Bridge Deck Panels* (SOA-01-1911).

T310 series course is based on MNL-133 Chapter 11.

T450 series courses are based on MNL-133 Chapter 10. T710 series course is based on MNL-133 Chapter 18.

T500 and T510 series courses are based on the *Bridge Geometry Manual* (CB-02-20).

T520 series courses are based on *Recommended Practice for Lateral Stability of Precast, Prestressed Concrete Bridge Girders* (CB-02-16) and *User Manual for Calculating the Lateral Stability of Precast, Prestressed Concrete Bridge Girders* (CB-04-20).

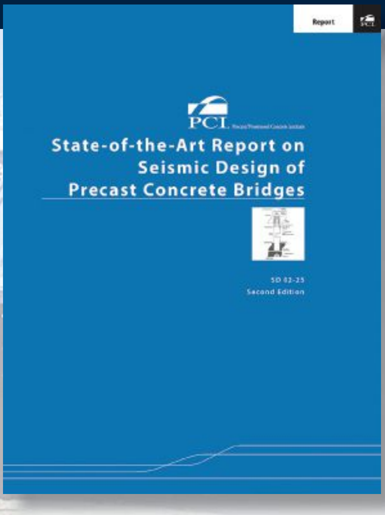
T350 series courses are based on the *Curved Precast Concrete Bridges State-of-the-Art Report* (CB-01-12), *Guide Document for the Design of Curved, Spliced Precast Concrete U-Beam Bridges* (CB-03-20), and MNL-133 Chapter 12.

AASHTO THE VOICE OF TRANSPORTATION

PCI Precast/Prestressed Concrete Institute

Now Available!

Bridge Design Manual Chapter 15: State-of-the-Art Report on Seismic Design of Precast Concrete Bridges (SD 02-25)



The *Bridge Design Manual*, Fourth Edition, Chapter 15 includes many new examples, which reference the latest research in seismic design. Chapter 15 was also reorganized, with background information moved to an appendix, to simplify its use.

Download the free PDF from the PCI Bookstore: www.pci.org/SD02-25

Bridge Design Manual Chapter 20: Precast Prestressed Concrete Piles (BM 20-25)

The *Bridge Design Manual*, Fourth Edition, Chapter 20 is updated to comply with the *AASHTO LRFD Bridge Design Specifications,* Ninth Edition, and the recommendations and commentary of ANSI/PCI 142-24, *Specification for Precast, Prestressed Concrete Piles*. Example problems are presented in accordance with both documents. The PCI Pile Interaction-Diagram Spreadsheet (PCI PD-15) is used to construct axial load-and-moment interaction diagrams.

Download the free PDF from the PCI Bookstore: www.pci.org/BM20-25

A simple log-in to the PCI website is all that is needed to download these free resources.

PCI Precast/Prestressed Concrete Institute

Standardization of Segment Shape

Improving the cost-effectiveness of small precast concrete segmental superstructures

by Jacob T. Rausch, Coastal Precast Systems

Two relatively small, three-span concrete segmental bridges were recently constructed along the Blue Ridge Parkway in western North Carolina: the Blue Ridge Parkway Bridge over Interstate 26 (I-26) and the Blue Ridge Parkway Laurel Fork Bridge replacement. Each bridge consists of precast concrete box girders fabricated at Coastal Precast System's facility in Wilmington, N.C.

Coastal Precast fabricated 62 segments for the I-26 project and 56 segments for the Laurel Fork Bridge replacement. These quantities are significantly smaller than those required for most precast concrete segmental bridge projects, where the number of segments often ranges into the hundreds. While casting a small number of segments may seem simpler than casting many, small projects like the Blue Ridge Parkway bridges pose their own unique challenges. (See the Summer 2024 issue of *ASPIRE*® for a Project article about the Blue Ridge Parkway Bridge over I-26.)

One of the biggest challenges on small projects stems from the lack of economy of scale. For large projects, each casting cell can be used to cast hundreds of segments. In contrast, the casting cell on the I-26 project was initially used to cast only 62 segments. Purchasing and setting up a casting cell is a significant investment, costing hundreds of thousands of dollars. That cost is the same whether the form is used hundreds of times or 62 times. Using the form fewer times increases the cost per segment.

To reduce that cost per segment, a simple (basic) form that has few accessories and functions can be purchased. For example, instead of hydraulic operation of the form, manual turnbuckles can be used, and instead of using multiple individual bulkhead forms that can be swapped out quickly for each variable-depth bulkhead joint, the bulkhead form can be removed, modified with shims and filler sections, and reinstalled.

Another way to reduce the formwork cost per segment is to use the same casting cell for all segment types. On most projects, separate casting cells are used for nontypical segments such as pier and abutment segments. These segments contain diaphragms that require different formwork. These nontypical segments also need two bulkheads when they are cast first because there are no segments to match cast against.

On large projects, additional casting cells may also be used to cast constant-depth segments and variable-depth segments separately. On the small I-26 project, a single casting cell was used to cast all 62 segments—56 variable-depth typical segments (ranging from 8 to 16 ft tall), four pier segments, and two expansion abutment segments.

While using the same casting cell for all segment types reduces the cost of the form, it approximately doubles the casting cycle time. Because one simplified form was used on the I-26 project, two segments could be cast per week on average. However, erection of precast concrete segments at the project site was at a pace of two segnments or more per day. Therefore, a large inventory of segments needed to be amassed to maintain the erection schedule and avoid costly delays. For this project, the longer cycle time was factored into the schedule at the beginning of the project. The additional cost during production (more labor to operate forms and longer casting cycle) was compared against cost of the cheaper form, and because of the small number of segments to be cast, the savings were greater using the simplified form.

With the help of COWI's Jerry Pfunter, the project's construction engineer (who became the new engineer of record), the precast concrete segment producer modified the shapes of the segments as part of a redesign for the Laurel Fork design-bid-build project to allow reuse of the casting cell from the I-26 project. However, due to the initial differences in the designs of the two bridges, the shapes could not be made identical. To mitigate differences in the shapes, adjustments were made on the Laurel Fork project: the bottom slab haunch

In the background, the construction of the precast concrete segmental Blue Ridge Parkway Bridge over Interstate 26 in western North Carolina is partially complete. All Photos: Coastal Precast Systems.

Casting of a pier segment at the precast concrete manufacturer's facility for the Interstate 26 project.

A variable-depth typical precast concrete segment for the Laurel Fork project is in position for the next segment to be match cast against it in the adjacent casting cell.

was eliminated, the thickness of the top slab was reduced, and the anchor blocks were modified to match specifications used in the I-26 project and the existing formwork. Unfortunately, the joint widths and heights varied between the two projects, so the bulkhead, mandrel, soffit table, and wing form required significant changes. However, the supporting structure of the casting cell was reused without significant modification.

Reusing the segmental casting cell from the I-26 project on the Laurel Fork project saved two months in startup time for the Laurel Fork project. A typical lead time, from ordering forms to casting the first segment, is about five months. After the design of the revised shape was completed, it took three months to modify the forms. This time frame would have been even shorter if the original segment shapes had been more similar.

The cost-effectiveness of using the same shape across multiple projects can be seen with prestressed concrete girders, which are economical in part because the initial form cost can be spread out over numerous projects. Because girder forms are standard shapes used across multiple jurisdictions, they can be used to cast thousands of girders. Therefore, a particular project with only one or two spans does not have to absorb the entire cost of the form.

Typically, segmental casting cells are only used for one project, so the cost cannot be allocated over other projects. If the use of standardized shapes becomes more commonplace, that will open the door for precast concrete segmental superstructures to be an economical option for more projects.

The *AASHTO-PCI-ASBI Segmental Box Girder Standards for Span-by-Span and Balanced Cantilever Construction*[1] are available for download from the American Segmental Bridge Institute (https://asbi-assoc.org/resources). While these standard drawings do not cover spans greater than 200 ft, the basic shape can be used for taller segments. These standard drawings can also be applied to variable-depth segments to extend spans. Using details from those drawings, including web wall slope and thickness, slope and length of cantilever wing base, width of core, and radius at top of web wall, precast concrete manufacturers could invest in typical, multiuse casting cells that could be used on multiple projects. For additional information on the AASHTO-PCI-ASBI segmental box girder standards, please refer to articles by Freyermuth[2] and Figg.[3]

Formwork is modified between the casting of segments for the Blue Ridge Parkway Bridge over Interstate 26 and the Laurel Fork Bridge replacement. In this photo, the width is being reduced for the nearly 3-ft narrower section. The form skin was removed, and I-beams were added to narrow the structure. The form skin was then replaced.

References

1. American Association for State Highway and Transportation Officials (AASHTO), Precast Concrete Institute (PCI), and American Segmental Bridge Institute (ASBI). 2000. *AASHTO-PCI-ASBI Segmental Box Girder Standards for Span-by-Span and Balanced Cantilever Construction*. Austin, TX: ASBI. https://asbi-assoc.org/wp-content/uploads/2023/07/Box-Girder-Segments-PCIASBIEnglish.pdf.
2. Freyermuth, C. L. 1997. "AASHTO-PCI-ASBI Segmental Box Girder Standards: A New Product for Grade Separations and Interchange Bridges." *PCI Journal* 42 (5): 32–42. https://doi.org/10.15554/pcij.09011997.32.42.
3. Figg, E. C. 1997. "Proposed AASHTO Standards for Segmental Bridges Represent a Growing Market for the Precast Concrete Industry." *PCI Journal* 42 (5): 30–31. https://doi.org/10.15554/pcij.09011997.30.31.

Artificial Intelligence Methods Can Assist Bridge Engineers

by Dr. Atorod Azizinamini and Dr. Mark Finlayson, Florida International University

Artificial Intelligence (AI) has the potential to provide bridge engineers, especially newer engineers entering the field, with practical, hands-on tools for making better decisions. The Innovative Bridge Technology/Accelerated Bridge Construction University Transportation Center (IBT/ABC-UTC), a U.S. Department of Transportation–funded center, is developing these kinds of AI tools. This article provides background on AI and two examples of the technologies being used in bridge engineering.

What Exactly Is Artificial Intelligence?

Put simply, AI is the art, science, and engineering of creating machines that can do—or assist humans in doing—intelligent things. The term "intelligent" is somewhat elusive and difficult to define, but it traditionally refers to cognitive tasks such as perceiving, learning, classifying, abstracting, reasoning, or acting.[1] As an academic and applied pursuit, AI is subdivided into many areas, including machine learning, natural language processing, multiagent systems, planning, knowledge representation, computer vision, human-machine interaction and teaming, and robotics.

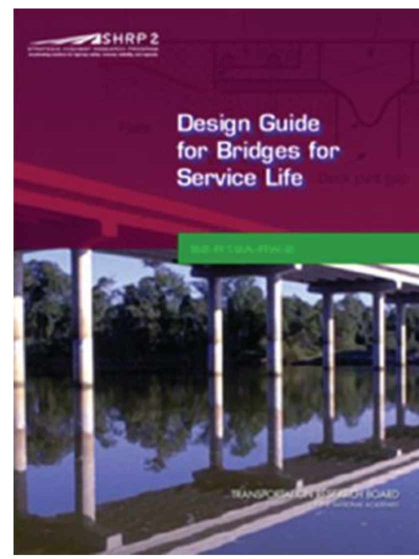
In the popular imagination, AI is a brand-new area that will result in the complete replacement of people in their jobs, with potentially disastrous consequences for the workforce. In reality, AI as a scientific and engineering field is more than 75 years old, and for decades AI systems have been deployed in ways large and small across many industries, making countless people more productive, efficient, and capable than they would have been without AI. Many AI researchers believe that the best use of AI technologies is to assist people in their work, not replace them. With this vision in mind, researchers at IBT/ABC-UTC are developing new AI-enabled technologies for bridge engineers. Two examples of these technologies—BridgeGPT and detecting corrosion of steel strands in external tendons—are presented herein.

BridgeGPT

IBT/ABC-UTC has initiated research to develop an AI tool to assist bridge professionals, called BridgeGPT, which will be similar to ChatGPT, except that BridgeGPT is devoted to bridge engineering.[2] ChatGPT is a type of large language model (LLM) that is trained on vast amounts of text and images, mainly drawn from the internet, to create "conversational" text in response to queries or prompts. The size and diversity of its training data allow ChatGPT to respond to a wide variety of queries; however, when it is asked to focus on a specialized or technical domain, such as bridge engineering, its answers can range from unhelpfully vague to dangerously incorrect. The vision for BridgeGPT is that the system will be trained using verified information drawn from sources used by bridge engineers.

At IBT/ABC-UTC, the first step in developing BridgeGPT is a small one: a module devoted to service-life design of bridges, a topic that was the focus of the Transportation Research Board's second Strategic Highway Research Program (SHRP2) R19A project.

The main deliverable of that project was the 2013 report *Design Guide for Bridges for Service Life*.[3] This document provided the foundation, information, and methodology for development



of the American Association of State Highway and Transportation Officials' (AASHTO's) *Guide Specification for Service Life Design of Highway Bridges*.[4] The SHRP2 R19A report and other project materials, as well as the AASHTO guide specification, contain a wealth of information that can quantitatively predict the service lives of bridge components and subsystems. The 2013 report begins with an overview of the philosophy used for the design of bridges for service life and contains flowcharts guiding the user to the application of the information presented in the document. Each of the report's 11 chapters includes additional flowcharts and information for service-life design of specific bridge components.

The volume of information developed on service-life design may seem overwhelming to bridge engineers who wish to apply it. This is where AI can help. Researchers and system developers can use the guides and specifications

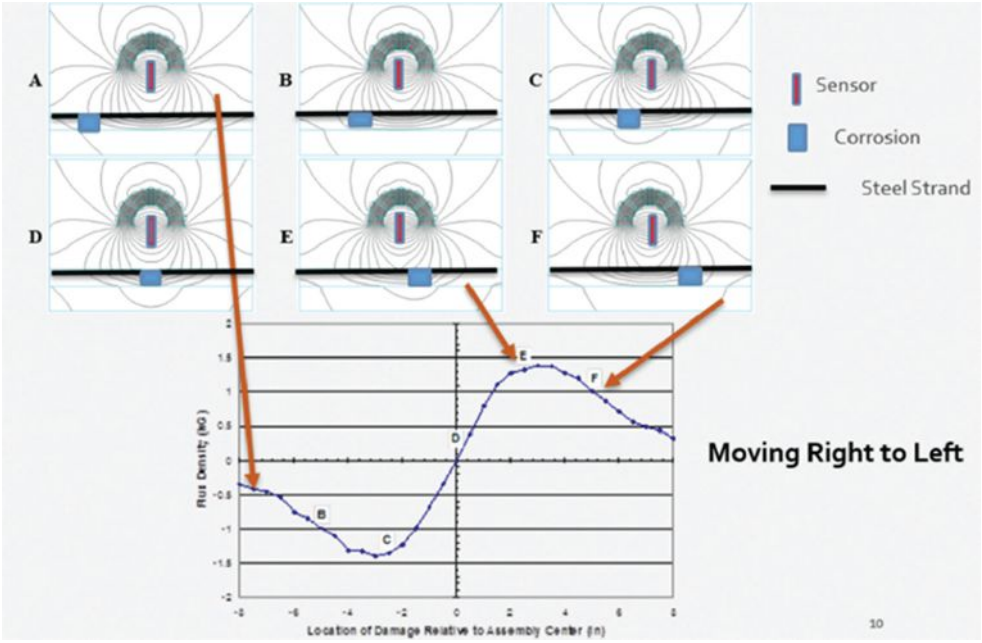


Figure 1. Results from the experimental residual magnetic flux leakage (MFL) method where a sinusoidal graph indicates the location of steel strand section loss. Figure: Florida International University.

just described to train an off-the-shelf, general LLM to specialize in the bridge engineering domain. Such a system could then respond to prompts related to design for service life, including answering factually based questions, finding relevant sections from reference materials, and integrating information drawn from several different documents. While we envision that the first version of BridgeGPT will focus specifically on service-life design, the architecture of BridgeGPT is such that additional knowledge can be included in the future, expanding the utility of BridgeGPT to other areas of bridge design and engineering.

Detecting Corrosion of Steel Strands in External Tendons

The Florida Department of Transportation and IBT/ABC-UTC are sponsoring the development of a technology to inspect external tendons in concrete segmental bridges for corrosion. The residual magnetic flux leakage (MFL) method is used to identify the sections of external tendons with steel section loss because of corrosion. In this method, an MFL device automatically runs across a tendon duct and develops a signal that is relayed to computer software for plotting. The presence of steel section loss coincides with a signal that has a sinusoidal shape (Fig. 1).

In general, the MFL method requires a trained eye to evaluate the signals and locate the regions of the tendon with steel section losses. In this project, machine learning (a type of AI) is used to analyze the data and assist the user in locating the regions of the tendon with potential steel section loss. To achieve this objective, various tests are being conducted to obtain signals from the MFL equipment. Signals from numerous experiments—each having different numbers and locations of steel wires within the strands removed—are obtained. These data are then used to train the system and develop machine learning algorithms that can help the user identify the regions along the tendons with potential steel section losses. The technology developed is quite accurate, capable of identifying steel section losses of less than 1%, regardless of the location of steel corrosion.

Conclusion

In the two examples provided, AI will not replace bridge engineers. Instead, it will provide valuable tools to facilitate decision-making processes and help engineers navigate through large amounts of data.

References

1. Martinez, D. R., et al. 2019. *Artificial Intelligence: Short History, Present Developments, and Future Outlook, Final Report*. Lexington, MA: Massachusetts Institute of Technology Lincoln Laboratory. https://www.ll.mit.edu/r-d/publications/artificial-intelligence-short-history-present-developments-and-future-outlook.
2. Innovative Bridge Technology/Accelerated Bridge Construction University Transportation Center. n.d. "Development of Architecture for Generative Pre-trained Transformer (GPT) Inspired Model for Bridge Engineering with Application to Service Life Design, Called BridgeGPT." Accessed February 13, 2025. https://abc-utc.fiu.edu/research-projects/development-of-architecture-for-generativepre-trained-transformer-gpt-inspired-modelfor-bridge-engineering-with-application-toservice-life-design-called-bridgegpt.
3. Transportation Research Board Strategic Highway Research Program (SHRP 2). 2013. *Design Guide for Bridges for Service Life*. S2-R19A-RW-2 Washington, DC: National Academies Press. https://doi.org/10.17226/22617.
4. American Association of State Highway and Transportation Officials (AASHTO). 2020. *Guide Specification for Service Life Design of Highway Bridges*. Washington, DC: AASHTO.

EDITOR'S NOTE

For further information on magnetic flux-based nondestructive evaluation for post-tensioned tendons see the Federal Highway Administration's 2022 report FHWA-HRT-23-005 (https://highways.dot.gov/sites/fhwa.dot.gov/files/FHWA-HRT-23-005.pdf).

PERSPECTIVE

Normalization of Deviance: Ethics Lessons from NASA

by Gregg Freeby, American Segmental Bridge Institute

In the Spring 2020 issue of *ASPIRE*®, William Nickas, editor-in-chief of *ASPIRE* and I coauthored "Why Didn't They Just Close the Road?" about the ethical dilemmas faced by state bridge engineers when public safety is at stake. After that article, I thought my time in the writer's hot seat was done, but William and I have continued the ethics conversation, and recently facilitated some ethics discussions as part of the National Concrete Bridge Council's *Concrete Bridge Seminar: Concepts for Extending Spans*. The topic I've covered in those sessions is "Lessons from NASA," and I'd like to share those lessons with you.

I have been a bridge engineer since the late 1980s. I never worked at NASA, but when I was 5 years old, the United States put the first man on the moon in 1969. That was when my fascination with the space program took root, and that fascination continues to this day. My wife would probably tell you my fascination borders on obsession. There are very few books or documentaries about the space program that I haven't read or seen. I've read the biographies and memoirs of astronauts, both living and dead, and I've read official mission reports of some of the events I will be covering here. I am no expert on the space program, but I do have a deep appreciation for the history of NASA and what the bridge industry can learn from the space agency's experiences.

Space Shuttle Columbia

On February 1, 2003, the space shuttle Columbia disintegrated over Texas during reentry, killing all seven crew members. Why did this happen? Normalization of deviance is why. To understand normalization of deviance and what it has to do with the space shuttle Columbia, we have to rewind and look at the NASA organization. Some would argue that NASA represents the finest group of engineers, technicians, and other professionals ever assembled. However, I believe that many of you work for organizations that rival NASA in terms of talent and commitment. As excellent as the NASA engineers are, they are human and they do make mistakes.

NASA, the Early Days

NASA got off to a rocky start in the late 1950s and early 1960s with several epic rocket failures, but it didn't take long before the agency hit its stride. Let's look at some of NASA's early accomplishments.

On May 5, 1961, Alan Shepard became the first American to complete a suborbital flight. The United States had finally put a man in space. The mission was a success by all measures. On July 21, 1961, astronaut Gus Grissom flew a similarly successful suborbital mission. His capsule, the Liberty Bell 7, sank to the bottom of ocean a short time after splashdown, but Grissom was out of the capsule by the time that happened and was never really in any danger. Just seven months later, on February 20, 1962, John H. Glenn became the first American to orbit the Earth during the three-orbit Mercury–Atlas 6 mission. That mission was also a success and was followed by 13 more manned space missions of varying lengths and crews. There were a few hiccups, but nothing of great significance. The proof was in the pudding: NASA's safety program was working—or so it seemed.

Apollo 1

Then we come to January 27, 1967. On that day, Grissom, Ed White, and Roger Chaffee were doing a live test of their Apollo 1 capsule at the Cape Canaveral Launch Complex 34. It was a day that would go horribly wrong, not only for the three astronauts but also for

Photo: NASA.

NASA. As the crew sat in their capsule, frustrated with radio communication problems among the various facilities, an electrical spark occurred within their spacecraft. This spark in the 100% pure oxygen environment set materials inside the pressurized capsule on fire, creating an inferno that swiftly engulfed the entire crew compartment. By the time rescuers were able to open the access hatch, the entire crew had suffocated. How could this tragedy happen at an organization with some of the best and brightest engineers and technicians in the world? How could this happen for an organization whose safety program was working, as evidenced by 16 successful manned missions and countless tests? To answer these questions, we need to review those previous successful missions and tests.

During the Mercury and Gemini programs, which preceded the Apollo 1 program, engineers and technicians were vigilant about fire prevention inside capsules. Materials were rigorously tested for flammability, and their use was prohibited if they posed a fire risk: if it burns, it does not fly. But as more successful missions were completed and more live tests were performed, this prohibition was informally relaxed. Over time, mission after mission, flammable materials were allowed into the capsule as astronauts and technicians added Velcro and netting to keep small items in place. By the time of Apollo 1, Velcro was used extensively throughout the capsule and the capsule was described by one engineer as nearly wall-to-wall Velcro. Why was this change allowed? Why would NASA put known flammable materials into a 100% pure oxygen environment? The explanation is simple: The use of flammable materials was allowed in the Apollo 1 capsule because nothing bad had happened before. The previous Mercury and Gemini capsules also had pressurized oxygen environments, and they never had a fire.

This acceptance of flammable materials didn't happen overnight. Over time, the rule prohibiting such materials was relaxed, and the informal norm eventually became "We've always done it that way." This slow erosion of rules or practices has a name: "normalization of deviance."

Another issue with Apollo 1 that contributed to the tragic outcome originated during Grissom's suborbital mission in Liberty Bell 7. The Liberty Bell 7 capsule sank because the hatch opened too early. To prevent the hatch from opening prematurely, engineers designed the Apollo command module door to open inward, without consideration for the potential safety consequences. After the fire, it was discovered that White had tried to use the emergency procedure for opening the door, but the pressure of hot air in the capsule made it impossible to open the door.

The Apollo Program Continues

After the Apollo 1 fire, NASA removed the flammable materials from the redesigned command module. They also eliminated the 100% pure oxygen environment and redesigned the escape hatch to open outward. After a delay of more than a year, Apollo missions resumed. Crews on Apollo 7 through 10 flew successful missions of various designs and durations, culminating in the Apollo 11 moon landing in 1969. That historic event was followed by five more successful lunar landings and the establishment of a space station. Even the well-known Apollo 13, despite challenges, returned home safely.

Space Shuttle Challenger

On January 28, 1986, a date that is probably familiar to many, the space shuttle Challenger was lost 73 seconds into its flight and all seven crew members died. What happened? Most analysts will say that cold O-rings that did not seal properly caused the tragedy. This explanation is incomplete. On the one hand, it is true that the physical cause of the failure was damaged O-rings that allowed superheated gases to escape from the solid rocket booster and burn a hole in the larger exterior liquid fuel tank. The resulting explosion of the tank caused the orbiter to become unstable and break apart. On the other hand, this type of O-ring failure was a known problem. In 14 of 24 space shuttle missions prior to the Challenger explosion, there was evidence that the O-rings had been burned. In fact, O-ring failure, regardless of the temperatures at launch, was identified as a problem after just the second shuttle launch. However, the shuttle had flown successfully at low temperatures, and the air temperatures on January 28 were just a little colder than during the last really cold flight. Since nothing bad happened in the past, safety warnings about O-ring risks in cold weather were disregarded.

Back to Columbia

Now let's go back to the first NASA tragedy mentioned in this article—the disintegration of the space shuttle Columbia on reentry and the death of its seven crew members—and consider what happened in that event. During Columbia's launch, insulating foam came off the external fuel tank and struck and critically damaged the heat shield on the orbiter's left wing. As a result, the damaged heat shield was unable to protect the underlying wing from superheated gases generated by the friction of reentry. The gases burned through the wing, causing the orbiter to become unstable and break apart.

When foam hit the wing during Columbia's launch, did anyone know about it? Yes. Was foam coming off the tank and striking the orbiter during launch a common occurrence? Yes. This type of incident had happened on many missions. Were people concerned about this issue? Yes. NASA personnel did tests to evaluate the risks, and it was concluded that chunks of foam large enough to cause significant damage just couldn't happen. Foam had broken off on many missions and seemed to be just an annoyance. In fact, it was so common that when those who tried to raise the alarm that somehow this recent episode with Columbia was different, no one listened. After all, at this point the shuttle program had flown more than 100 missions without the foam causing a major issue. Nothing bad had ever happened.

The term "normalization of deviance" was coined by sociologist Diane Vaughan. In her book *The Challenger Launch Decision: Risky Technology, Culture, and Deviance at NASA*,[1] Vaughan defines the term as "the gradual process through which unacceptable practices and standards become acceptable. As the deviant practice is repeated without catastrophic consequences, it becomes the social norm of the organization." To put it another way, normalization of deviance occurs when people become so accustomed to a deviation that they no longer consider it abnormal.

NASA Gets It Right

On June 5, 2024, astronauts Suni Williams and Butch Wilmore launched from Cape Canaveral in Florida aboard the Boeing Starliner. Although the launch was successful, five thrusters failed as the Starliner docked at the International Space Station. The cause of the failure was later identified through ground tests as heat buildup, but NASA could not fully understand why the thrusters malfunctioned. Therefore, NASA officials decided that it would be too risky to return the craft to Earth with astronauts aboard. The previous experiences with Challenger and Columbia were cited as contributing to this decision. Ultimately, Starliner returned unmanned on September 7, 2024, without incident. It would appear that NASA has learned from its history and decided that the "nothing bad has ever happened" approach isn't always valid.

What Have We Learned?

What does all of this NASA history have to do with our work in the concrete bridge industry? Everything. I assume that you work at an organization that resembles NASA in many ways: The engineers, technicians, and other professionals at your organization are among the best and brightest. Your organization also has a long history of success, where bad things rarely happen. You have a robust safety program and quality control/quality assurance (QC/QA) processes with policies and procedures that, when followed, yield amazing results. But also like NASA, you can't allow your past success to lull you into believing you're always doing things correctly. Have you tolerated relaxed safety procedures because nothing bad has ever happened in the past? Are all your QC and QA processes being followed, or do you skip a few steps here and there since you've never found errors in someone's work in the past?

We must always be mindful of the potential for normalization of deviance. I suspect there are probably things we have always done in a particular way that we shouldn't be doing but, because nothing bad has ever happened, we've informally decided that the status quo must be okay. "We've always done it that way!" isn't just a phrase we use to be critical of bureaucratic processes—it's also a potential killer.

Reference

1. Vaughan, D. 2015. *The Challenger Launch Decision: Risky Technology, Culture, and Deviance at NASA.* Chicago, IL: University of Chicago Press.

Gregg A. Freeby is the executive director of the American Segmental Bridge Institute and past chair of the National Concrete Bridge Council.

FOCUS

Palmer Engineering

Palmer Engineering has provided civil engineering services across the Appalachian region for 55 years

by Monica Schultes

Palmer Engineering designed a cost-effective bridge with a profound skew to carry U.S. Route 20 over the Norfolk Southern railroad in Ashtabula County, Ohio. The unique design and orientation of the beams leaves the prestressed concrete beams exposed beyond the limits of vehicular and pedestrian traffic. Photo: Palmer Engineering.

Palmer Engineering Company, incorporated in 1969, is a consulting engineering firm specializing in transportation, structures, surveying, land development, and water resources. Headquartered in Winchester, Ky., Palmer Engineering employs more than 120 professionals across five states. The firm maintains four offices in Kentucky, three in Ohio, two in Florida, and one each in West Virginia and Tennessee. Some offices specialize in a particular subfield, but all offer a wide range of services, including civil engineering, surveying, environmental, infrastructure design, water resources, bridge design, bridge inspection, roadway design, traffic engineering, and utility engineering.

The Palmer Way

Palmer Engineering offers comprehensive bridge engineering services, including design of new structures, in-service bridge inspection, load rating, and maintenance. Everyone on their team of structural engineers is experienced in both new design and bridge inspection. "That is the Palmer way. We want our people in the field and involved in designing and maintaining our infrastructure. There is a great reward in that connection," says Dr. David Deitz, vice president at Palmer Engineering.

"That is the Palmer way. We want our people in the field and involved in designing and maintaining our infrastructure. There is a great reward in that connection."

Deitz credits the company's longevity and success in the region to their unique understanding of the critical facets of resilient structures. While many engineering firms specialize, Palmer Engineering is well rounded, which complements the needs of their region's, and the country's, aging infrastructure. Their bridge work is primarily with departments of transportation, often in conjunction with highway projects.

Bridge Inspection

In addition to designing new structures, Palmer Engineering works with transportation agencies to maintain their inventory of bridges. The firm performs inspections and load ratings of existing structures and will develop repair plans, if necessary. Much of that work involves visual inspection of every type of bridge, from small culverts to major river crossings. For example, in Palmer Engineering's work with the Kentucky Transportation Cabinet (KYTC), some projects start with a detailed bridge inspection to document deterioration, cracks, or excessive corrosion. In some cases, these findings lead to the development of a set of plans to repair the structure. Those plans might include a deck overlay, component retrofits, or crack repair. The inspection findings are also sometimes used to refine bridge load ratings, which are needed to document a structure's load-carrying capacity.

Many Palmer Engineering inspectors are certified by the Society of Professional Rope Access Technicians, which means they can safely perform rope-access inspections from heights. All primary Palmer bridge inspectors also maintain Remote Pilot Certificates from the Federal Aviation Administration. Deitz notes the value of drones in their inspection work. "This month, our crews inspected 30 small-to-medium bridges, and because our inspectors always have a drone with them, the drones supplement their efforts on the ground. Drones provide a better look at site conditions because even the smallest inspections often require a

The Kennedy Interchange—also known as "Spaghetti junction"—is the confluence of three interstates and intertwines with local access roads between downtown Louisville, Ky., and the Ohio River. Palmer Engineering was part of the project team that designed the reconstruction of the interchange. Photo: Palmer Engineering.

better view. Our drones are just another tool in the toolbox."

In 2024, Palmer Engineering inspected twin bridges carrying State Route 2 over the Huron River and an adjacent estuary in Erie County, Ohio. Each of the bridges is composed of 27 spans, with a total length of approximately ½ mile. To enhance the inspection, Palmer performed a scan of the bridge decks with a drone-mounted infrared camera operated by Palmer's experienced drone pilots. The inspection is the first phase of a bridge rehabilitation that will evaluate deterioration of the concrete decks, piers, and girders.

Emergency Work

Palmer Engineering's breadth of experience in design and inspection is called upon in emergency situations. In November 2024, the firm provided immediate support to the Ohio Department of Transportation after a large outdoor fire under the Ohio approach to the Daniel Carter Beard Bridge on Interstate 471, which connects Cincinnati to Newport, Ky. This fire led to the closing of the southbound lanes of the interstate, and the firm was part of the project team that designed shoring towers to stabilize the approach spans until more permanent repairs could be installed.

In 2022, localized flooding in eastern Kentucky prompted KYTC to engage Palmer Engineering for emergency bridge inspections. "These bridges were rarely, if ever, overtopped in the past, but after the storm, some were under several feet of water," says Deitz. "Some structures had been completely washed away." Immediately after the event, Palmer inspected more than 60 bridges in support of KYTC's efforts to document the condition of every structure in the area. The firm then worked with KYTC to design 11 replacement bridges in three months. "We were called out after the storm event to support inspection efforts and then to design and replace some of these critical bridges immediately," explains Deitz. "We designed one bridge within a week to get it reopened. Our firm is local, so we were committed to helping."

In November 2020, a truck hauling potassium hydroxide and diesel fuel crashed into a jackknifed truck and caught fire on the northbound deck of the Brent Spence Bridge. The structure is a vital crossing of Interstate 71 and Interstate 75 over the Ohio River between Covington, Ky. and Cincinnati, Ohio. Palmer Engineering and three other consulting firms joined KYTC inspectors to assist in the immediate inspection of the bridge. Because chemical fire temperatures exceeded 1500°F, traditional inspection methods were not initially feasible; under these conditions, drones were an important inspection tool.

Workforce Development

Like most firms in the industry, Palmer Engineering is seeking innovative ways to find and retain highly skilled employees. Hiring and keeping a highly skilled workforce is challenging, says Deitz. "There is a real shortage of engineers, so we work with the University of Kentucky and other universities to find young talent." This outreach program feeds into the Palmer Engineering intern program. "We look ahead to future graduates who will make a good fit with our team," says Deitz. For example, to prepare engineering students to enter the workforce, the University of Cincinnati has a mandatory co-op program in which students alternate semesters in the classroom with semesters working full time. Palmer Engineering maintains a relationship with the program and employs a few students each semester.

"We take an active role in hiring interns and making sure they're a good fit for us and we are a good fit for them before they graduate," says Deitz. Palmer Engineering supports young engineers as an investment in

Precast concrete deck panels that are full-depth and full-width are placed on box beams spanning Harrods Creek. The deck panels cantilever beyond the existing spandrel walls on each side to widen the existing crossing while maintaining its historic profile. Photo: Stantec.

Part of the Corridor Q project, this bridge carrying U.S. Route 460 over Marrowbone Creek and Kentucky Route 195 features twin nine-span precast concrete I-beam structures with 180-ft-tall piers. The structure is supported on tapered, hollow piers. Photo: Palmer Engineering.

their future. Their support of intern and co-op programs helps strengthen the firm's structural engineering pipeline while also helping young engineers innovate and grow in their careers.

The firm also fosters a mentoring culture among its employees. More-seasoned engineers make a concerted effort to mentor younger professionals during projects, giving them opportunities to gain experience in each phase of the work. "I think our other principals would agree that we are sharing our philosophies and experiences," says Deitz. It comes down to making an investment to teach innovative approaches and provide a variety of experiences. "If the young engineers have not yet designed a side-by-side concrete box-beam bridge or are unfamiliar with tall concrete piers, then we include them and provide that opportunity, so everyone is well rounded."

Project Highlights

Ashtabula County, Ohio, Bridge Replacement

When a 1940s-era, 23-span reinforced concrete slab bridge in Ashtabula County, Ohio, needed to be replaced due to age and deterioration, Palmer Engineering prepared the design plans. The bridge carries U.S. Route 20 over the Norfolk Southern rail line, where the skew between the road and railroad is approximately 72 degrees.

For this project, the owner sought a cost-effective bridge that would increase vertical clearance while also accommodating two future tracks adjacent to the existing rail lines. To address the challenges, the project team designed and constructed an innovative single-span bridge with prestressed concrete I-beams oriented perpendicular to the centerline of the railroad tracks rather than parallel to the roadway centerline. The parts of the superstructure within the roadway limits are supported on stub abutments behind mechanically stabilized earth walls; beyond those limits, the bridge is supported on piers.

To reduce construction costs, a splayed framing plan with relatively wide beam spacing was used at the acute corners. This strategy eliminated beam lines and minimized substructure lengths. For additional cost savings, the bridge deck extends only in the footprint of the roadway and sidewalks, leaving significant portions of the beams exposed. The final layout uses 33 beams, with the 100-ft interior beams perpendicular to the abutments and spaced at a constant 13 ft. The 10 beams on each side of these central beams are splayed, with spacings ranging from 6 to 16 ft, and spans ranging from 100 to 141 ft.

Tall Piers and Seismic Challenges in Kentucky

Eastern Kentucky has some of the tallest bridges in the eastern United States due to its rugged mountainous terrain and steep slopes. Many project sites in the region pose unusual challenges for designers. Mobility difficulties were acknowledged in the Appalachian Regional Development Act of 1965, which designated highway corridors in the region for specific infrastructure funding. Over the past 25 years, Palmer Engineering has been responsible for the design of more than 25 miles of highway in eastern Kentucky. This part of the Appalachian regional highway system has more than 30 major bridges, several with concrete pier heights in excess of 200 ft.

Recently, Palmer Engineering was responsible for designing the entire 16-mile Kentucky portion of the Appalachian Development Highway System's Corridor Q (U.S. Route 460). That corridor runs from U.S. Route 23 in Kentucky to Interstate 81 in Virginia. The firm's experience designing bridges in mountainous areas helped them work around constraints such as the potential for subsidence associated with an abandoned coal mine. Construction is ongoing, and the project should be complete in 2025.

Tall concrete piers in the mountains can be a very impressive feature. A representative design on the Corridor Q project consists of hollow cast-in-place concrete columns with 1-ft 6-in.-thick walls that are a consistent 16 ft wide transversely and begin at 6 ft thick longitudinally just below the pier cap, widening at a tapered ratio of 40 to 1 in the longitudinal direction as the column approaches the ground elevation. Palmer Engineering has designed similar piers for other projects in the region. Rock is typically close to the surface, so many of the piers are supported on spread footings.

History of Palmer Engineering

Ralph Palmer and Dick Nunan founded Palmer Engineering in 1969 in Winchester, Ky., as a surveying enterprise. The firm later moved into highway design and grew from there, expanding into structural and environmental engineering services. Palmer's vision to add more in-house services led the business to become what it is today. Today, the firm has more than 120 employees, including 60 professional engineers, engineers-in-training, and professional land surveyors.

Ralph Palmer graduated from the University of Kentucky with a degree in civil engineering. He retired in 2006, but his fundamental belief that business should be based on honesty and integrity is still part of the culture of Palmer Engineering. The firm has registered professionals specializing in transportation engineering, traffic engineering, structural engineering and inspection, surveying, and environmental design. The firm intends to continue to be a small firm and operate like the family business that Palmer envisioned. His son currently works in the survey division.

However, in situations where there is significant overburden, end bearing piles or large-diameter drilled shafts are used for the foundations. Given the height of the piers required, Palmer Engineering made special considerations for creep, slenderness, and shrinkage of the concrete, which are not specifically considered in conventional pier design.

Palmer Engineering also works in western Kentucky, which, in contrast to eastern Kentucky, is mostly flat. For example, the firm partnered with Michael Baker International for the replacement of the western Kentucky bridges carrying U.S. Route 68/Kentucky Route 80 over Kentucky Lake and Lake Barkley. A concern on this project was seismic activity, as western Kentucky is located in the New Madrid seismic zone. "As part of our work," Deitz says, "we performed rigorous time-history analysis of the approach structures across the two large lakes."

Deitz notes that the firm has the expertise to address the engineering challenges it has encountered in Kentucky and elsewhere. "Palmer Engineering has performed detailed seismic analysis and has designed some of the tallest piers in the Eastern United States," explains Deitz. "We can manage large and challenging projects despite our small size. We pride ourselves in our technical capabilities and ability to undertake complex projects."

"Palmer Engineering has performed detailed seismic analysis and has designed some of the tallest piers in the Eastern United States."

As part of the inspection of the twin bridges carrying State Route 2 over the Huron River and an adjacent estuary in Erie County, Ohio, Palmer Engineering performed a scan of the bridge decks with a drone-mounted infrared camera operated by experienced drone pilots. The scan helped to document the condition of the 27-span precast, prestressed concrete girder bridges. Photo: Palmer Engineering.

Kennedy Interchange, Louisville, Ky.

The Kennedy Interchange in downtown Louisville, Ky., is the junction of Interstates 64, 65, and 71. Because of the interchange's complicated geometry and required diverging, weaving, and merging movements, it has been called "Spaghetti Junction." Reconstruction of Kennedy Interchange was part of a larger project that included construction of 41 permanent bridges and aimed to improve mobility across the Ohio River downtown and on the east side of Louisville. Palmer Engineering was part of the design-build team, led by Walsh Construction, for the project.

Palmer Engineering's responsibilities included the final design of 15 bridges, including flyovers, interstate-to-interstate ramps, and local access ramps. The superstructures vary depending on the site constraints and include prestressed concrete I-girders and box beams, among others. Substructures include integral end bents, semi-integral breast wall abutments, expansion abutments with backwalls, and expansion joints. Palmer Engineering also provided surveying services. "All of the structures had challenging geometry, including severe skews, wedge-shaped spans, horizontal curves, and sharp vertical curves," recalls Deitz. "This was a good example of the use of conventional simple-span precast concrete girders made continuous for live load despite unique shapes, layout, and skews."

Widening the Historic Harrods Creek Bridge

In Louisville, Ky., Palmer Engineering served as the specialty engineer for a challenging project to widen an existing one-lane bridge to two lanes while preserving the historic character of the original span across Harrods Creek. The structure, which was deteriorating rapidly, was eligible for the National Historic Register. Facing a mandate to keep the original concrete arch, the project team pursued a unique solution to conceal a new structure inside the arched spandrels of the existing bridge. The concept would not transfer any new loads onto the existing arch. Precast, prestressed concrete box beams were selected to maintain the structure's narrow profile while adding the necessary deck width to meet current design standards. Palmer Engineering worked with the precast concrete manufacturer to design specially fabricated full-width, full-depth precast concrete bridge deck panels. The existing bridge deck had to be widened by more than 10 ft, so the new deck cantilevers outside the existing spandrel walls on each side. The prestressed concrete box beams had to be shallow in depth to meet the geometric constraints of the project, and it was important to install them quickly so that the structure could reopen in time for the beginning of the new school year.

Stewardship

Deitz emphasizes that Palmer Engineering intends to continue to grow as a firm and maintain their multifaceted team of professionals. "We are vested in the benefits of prolonging the life of our infrastructure," says Deitz. In addition to performing inspections, the firm helps owners implement maintenance programs, which are an important part of extending the service lives of bridges. Their work on transportation projects puts Palmer Engineering on the frontline as stewards of these important assets.

PROJECT

U.S. Route 395 North Spokane Corridor Spokane River Crossing

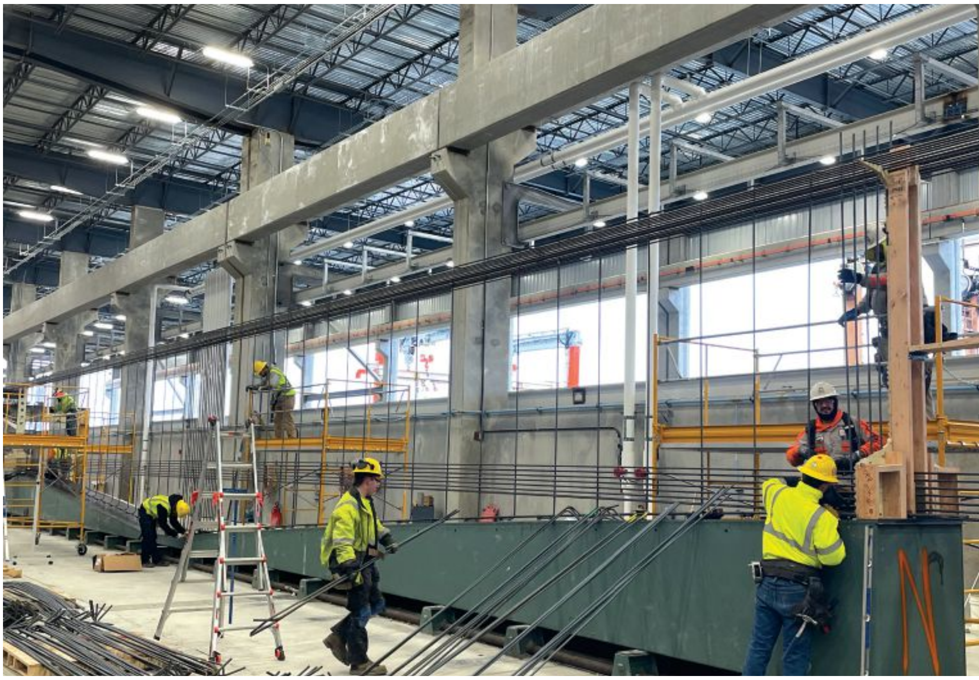
Connecting Washington One Segment at a Time

by Jordan Pelphrey, Pelphrey Prestress Partners; Dusty Andrews, Knife River Prestress; and Mathew Rochon, Washington State Department of Transportation

The 10.5-mile U.S. Route 395 (U.S. 395) North Spokane Corridor (NSC) is part of the Washington state highway system. When completed, it will be a north/south limited-access highway with a dedicated pedestrian and bike path that connects Interstate 90 on the south end to U.S. Route 2 and U.S. 395 on the north end.[1] The highway is expected to significantly improve safety and traffic flow through north Spokane. This project has been in the works for more than 50 years. Substantial research planning, legislation, and public input were required to gain approval for the corridor.

As part of the overall NSC project, the U.S. 395 NSC Spokane River Crossing project includes construction of two bridges that cross the Spokane River and connect two other NSC projects together: the Sprague Avenue to Spokane River Phase 1 project, which is located to the south of the Spokane River and is adjacent to Spokane Community College, and the Spokane River to Columbia project, which is located north of the Spokane River.

The Washington State Department of Transportation (WSDOT) has committed to a community engagement process for the entirety of the NSC project. Early stages in the process established a corridor theme unique to the Spokane area. In addition, neighborhood-specific aesthetics were established where



Reinforcement is being installed at the precasting facility for the prestressed concrete variable-depth girder segment. The 98-ft-long, lightweight prestressed (pretensioned and post-tensioned), haunched concrete girder segments weigh 198,400 lb and vary in depth from 8 ft 8½ in. at the ends to 12 ft at the center. Photo: Knife River Prestress.

profile

U.S. ROUTE 395 NORTH SPOKANE CORRIDOR SPOKANE RIVER CROSSING / SPOKANE, WASHINGTON

BRIDGE DESIGN ENGINEER: Washington State Department of Transportation Bridge and Structures Office, Olympia, Wash.

OTHER CONSULTANTS: Precast concrete specialty engineers: Pelphrey Prestress Partners, Grandville, Mich., and Williams & Works, Grand Rapids, Mich.

PRIME CONTRACTOR: Kuney Construction, Spokane, Wash.

READY-MIXED CONCRETE SUPPLIER: Central Pre-Mix, Spokane Valley, Wash.

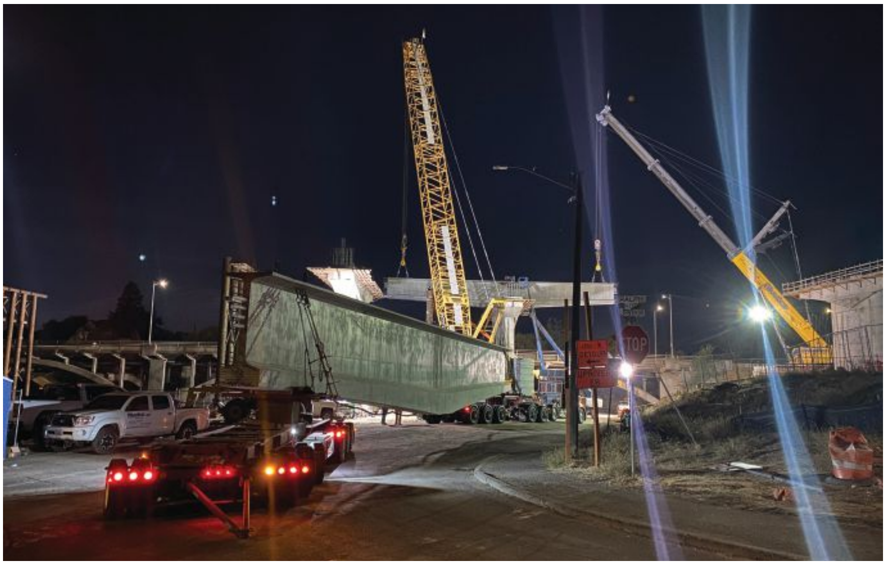
PRECASTER: Knife River Prestress, Newman Lake, Wash.—a PCI-certified producer

POST-TENSIONING CONTRACTOR: Structural Technologies/VSL, Wheat Ridge, Colo.

The cast-in-place concrete pier is a two-column structure with decorative precast concrete panels to give the illusion of a single-column pier. The panels will receive a base coat of gray pigmented sealer, and some areas of the concrete texture will be enhanced with accent colors. Photo: Kuney Construction.

View from beneath the erected WF83G pretensioned concrete girders. Photo: Kuney Construction.

adjacent communities expressed an interest in unique local themes, as well as discrete locales for all respective projects. The Spokane River Crossing project received considerable attention because of the river's significance throughout Spokane's history. The structure's aesthetics and design needed to reflect its importance and context in relation to that of other river crossings throughout the city. At this site in particular, the adjacent Greene Street Bridge's historic spandrel arch design was relevant. Because the new bridge was to be significantly elevated relative to Greene Street, the intent was to provide a form that would complement the Greene Street Bridge's spandrel arch without overshadowing it.



Delivery of a prestressed concrete girder segment. Photo: Knife River Prestress.

WASHINGTON STATE DEPARTMENT OF TRANSPORTATION, OWNER

OTHER MATERIAL SUPPLIERS: Formwork (new for variable-depth segments): Helser Industries, Tualatin, Ore.; lightweight aggregate: Stalite, Salisbury, N.C.

BRIDGE DESCRIPTION: Two 1211-ft-long precast, prestressed, and post-tensioned concrete girder vehicular bridges

STRUCTURAL COMPONENTS: 70 WF83G bulb-tee precast, pretensioned concrete girders; 28 WF83PTG precast, post-tensioned concrete end segments; 14 WF83PTG precast, post-tensioned concrete drop-in center segments; 28 WF83PTG precast, post-tensioned lightweight concrete haunched girder segments; 8-in.-thick cast-in-place concrete deck; cast-in-place concrete pier caps, crossbeams, closure pours, columns, drilled shafts, and footings

BRIDGE CONSTRUCTION COST: $91 million


Erection of the haunched girder segments. Photo: Knife River Prestress.

An extensive type, size, and location study was performed in conjunction with the community engagement process. Several structure types were considered. A haunched, precast concrete bridge option provided the form and function needed for the crossing and was ultimately chosen as the solution. Additionally, other creative precast concrete components—including decorative panels for the faux pier walls, end caps, and column medallions—were used in the project. The cast-in-place (CIP) concrete piers comprise a two-column structure with decorative precast concrete infill panels, precast in the field by the contractor, that give the illusion of a single-column pier. Aesthetic treatments on these surfaces consist of standardized items from the NSC aesthetic theme, such as the sunburst pattern, nature themes near the Spokane River, and other custom concrete finishes developed through community engagement by a local artist.

Superstructure

The project features two 1211-ft-long precast, prestressed spliced concrete girder vehicular bridges, with one carrying northbound traffic and the other carrying southbound traffic. The precast, prestressed concrete girders are a combination of pretensioned and post-tensioned elements. Each bridge consists of eight spans of concrete girders. Spans 1, 2, 3, 7, and 8 consist of seven lines of precast, pretensioned concrete wide-flange bulb-tee girders (WF83Gs). These girders are 83 in. deep and weigh up to 163,000 lb, with spans ranging from 113 ft 4 in. to 148 ft 4 in. Spans 4, 5, and 6 consist of seven lines of girder segments, with each line composed of five precast, post-tensioned concrete girder segments (WF83PTGs): two end segments, two haunched concrete girder segments, and one center drop-in segment. Spans 4, 5, and 6 comprise 540 ft of the total bridge length, with end spans 4 and 6 measuring 155 ft each and center span 5 measuring 230 ft.

The overall bridge width for each bridge is 55 ft 9 in., with a roadway width of 50 ft. The superstructure includes three

AESTHETICS COMMENTARY

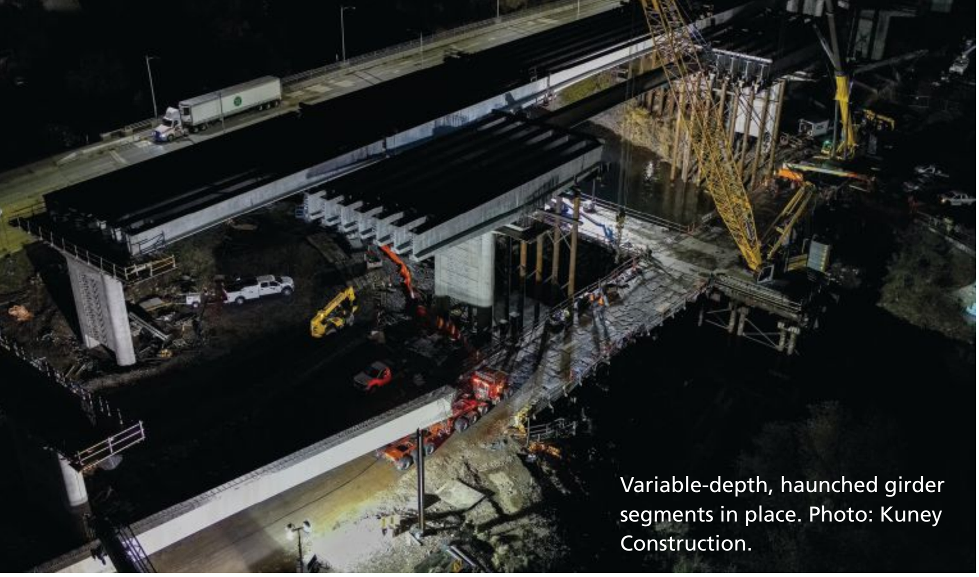
by Frederick Gottemoeller

The designers of the U.S. Route 395 North Spokane Corridor Spokane River Crossing were wise enough to recognize that they already had in hand a rare asset, a tradition of community stewardship for the river. That tradition meant that the community had already decided how to treat public structures that affect the river. So, the designers set up a dialogue with the community to work out exactly how that tradition would affect the new Spokane River Bridge.

Other designers worry that these kinds of efforts could increase costs. In fact, such efforts usually decrease costs. They head off community controversies and the resulting delays. If a project is delayed, especially a large project such as the Spokane River Bridge, the inflationary cost increase of even a few months' delay can eat up the supposed savings gained by a more conventional solution.

One decision that resulted from the community-outreach effort was to respect the adjacent historic Greene Street Bridge. That prompted the selection of longer-than-usual main spans that open views to the Greene Street Bridge and the river itself. Then, for structural reasons, the use of long spans prompted the designers to deepen the continuous girders using variable-depth, haunched girder segments over the piers. That creates visual evidence of where the forces are concentrated and how the structure is responding. Most observers appreciate that understanding. Finally, putting the soffits of the girders and haunched girder segments on continuous parabolic curves gives the bridge graceful, curved lines that complement the arches of the Greene Street Bridge.

But the designers didn't stop there. They also took advantage of the community's tradition in the design of the details, such as the pier wall panels, end caps, and column medallions. That gives the structure a consistent aesthetic theme that matches the aesthetic theme of the rest of the corridor. It also creates additional visual interest, which nearby observers will certainly enjoy. With the Spokane River Crossing, the designers and builders have created a civic asset that Spokane will long appreciate.



Variable-depth, haunched girder segments in place. Photo: Kuney Construction.

Prestressed concrete drop-in segments suspended from the haunched girder segments whose ends are temporarily supported by falsework before post-tensioning is installed. Photo: Kris Brown.

12-ft-wide lanes, a 4-ft-wide shoulder on one side, and a 10-ft-wide shoulder on the other. Other bridge components include an 8-in.-thick CIP concrete deck, CIP concrete pier caps with radiused bottoms, and CIP crossbeams, closure pours, columns, drilled shafts, and footings. The bridge exteriors include specialized finishes, with the exterior face of the girder, the CIP deck, and part of the CIP railing receiving a pigmented sealer, and the other part of the CIP railing receiving a fractured basalt finish with a concrete-staining treatment. Utilities are carried across the river by means of a hanger support system located between adjacent girders.

The design of the structure was based on HL-93 loading in accordance with the ninth edition of the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*,[2] with an allowance for 35 lb/ft^2 of future wearing surface. All work on the project is in accordance with the project's special provisions and WSDOT's 2023 *Standard Specifications for Road, Bridge, and Municipal Construction* and amendments.[3]

Precast Concrete Segments and Post-Tensioning (Spans 4–6)

The haunched girder segments were designed using lightweight concrete to keep the weight below 200,000 lb, which was a target weight based on plant lifting capabilities and truck hauling limitations. The lightweight aggregate was readily available from the supplier but was procured months ahead of production to allow shipping to be a backhaul for the trucking company and save costs. The concrete mixture design was developed specifically for the project, and some initial testing was completed before the bid to confirm the required strengths and unit weight could be met. The maximum specified unit weight of the lightweight concrete, including reinforcement, was 138 lb/ft^3.

These haunched segments had a design length of 98 ft, and each weighs 198,400 lb. The segments vary in depth from 8 ft 8½ in. at the ends to 12 ft at the center. They include 32 permanent 0.6-in.-diameter pretensioning strands. Each strand in every segment was initially tensioned to 43,900 lb. To help control the stresses from hanging the center drop-in segments from the haunched girder segments during erection, 18 of the pretensioning strands were located in

The girder lines of the southbound bridge are erected and ready for installation of post-tensioning. Photo: Knife River Prestress.

the top flange. All segments had a design concrete compressive strength at transfer of 7 ksi. The haunched segments had a 28-day concrete strength of 9.8 ksi.

The end segments are 103 ft 3 in. long and use normalweight concrete; each weighs 155,000 lb. These segments vary in depth from 6 ft 10⅝ in. at the lower end to 8 ft 7¼ in. at the higher end. They include 18 permanent and 2 top temporary 0.6-in.-diameter pretensioning strands and have a 28-day concrete strength of 9.5 ksi. The end segments include the four post-tensioning anchorages located only at the lower, or 6 ft 10⅝-in.-deep, end.

The center drop-in segments are 128 ft long, use normalweight concrete, and weigh 190,100 lb. They vary in depth from 8 ft 7¼ in. at the ends to 6 ft 10⅝ in. at the center. They include 20 permanent and 2 top temporary 0.6-in.-diameter pretensioning strands. The 28-day concrete strength for the drop-in segments was 10.5 ksi.

The bottom flange of each of the segments follows a parabolic curve. The segments' design includes four post-tensioning ducts with fourteen 0.6-in.-diameter post-tensioning strands per duct, for a total of 56 post-tensioning strands in each segment line. Special reinforcing bar jigs were developed to ensure that the ducts were supported in the correct locations throughout the production process.

Each segment included pretensioning strands to control stresses during lifting, handling, shipping, and erection. In the center drop-in and end segments, the pretensioning strands were designed to follow the profile of the parabolic bottom flange. To achieve the parabolic shape, the bottom strands were chorded along the bottom form soffit at 12-ft intervals. To handle the force from chord points, special holdup devices were designed and installed in the bottoms of the segments.

Special formwork was designed to produce the parabolic shape. The formwork used to produce the haunched girder pieces was nearly 17 ft tall and needed a special form stand that was designed to support the pieces of the formwork's side rail. All girder segments were cast inside the precasting facility and transferred to outside storage when completed. This process helped ensure quality control during production, a period of more than nine months.

Construction

The project was advertised to bidders in December 2022 and awarded to the contractor in February 2023. Construction began in May 2023. Erection of the southbound post-tensioned prestressed concrete girder segments began on October 14, 2024, and was completed on October 24, 2024. The haunched segments were set first, followed by the end segments and then the drop-in segments. The same order was followed for the northbound girder lines, starting on October 27, 2024, and finishing on November 7, 2024. Initially, three or four girders were set each night; by the end of the project, seven girders were being set each night. The Spokane River Crossing phase of the NSC project is slated for completion in late 2025.

Conclusion

When construction is finished, this bridge will improve north/south travel in Spokane and bring the entire North-South Corridor project one step closer to completion. The structure will create the opportunity for adjacent commercial and industrial development, improve user safety, and reduce collisions.

References

1. Washington State Department of Transportation (WSDOT). n.d. "North Spokane Corridor." Accessed December 27, 2024. https://wsdot.wa.gov/construction-planning/major-projects/north-spokane-corridor.
2. American Association of State Highway and Transportation Officials (AASHTO). 2020. *AASHTO LRFD Bridge Design Specifications*. 9th ed. Washington, DC: AASHTO.
3. WSDOT. 2023. *Standard Specifications for Road, Bridge, and Municipal Construction*. M 41-10. Spokane, WA: WSDOT. https://wsdot.wa.gov/publications/manuals/fulltext/M41-10/SS2023.pdf.

Jordan Pelphrey is director of engineering at Pelphrey Prestress Partners. Dusty Andrews is engineering director at Knife River Prestress. Mathew Rochon is state bridge and structures architect for the Washington State Department of Transportation.

New Mount Vernon Viaduct—Third Time's the Charm

by Claus Frederiksen, COWI, and Benjamin Turner, Dan Brown and Associates

The new Mount Vernon Viaduct is a vital project currently under construction, with an expected completion date in late 2025. This new structure over 3rd Street and a large rail yard operated by Burlington Northern Santa Fe (BNSF) Railroad in San Bernadino, Calif., will replace the existing bridge, which has been deemed structurally deficient and functionally obsolete. The City of San Bernardino, the owner, in collaboration with co-owner San Bernardino County Transportation Authority (SBCTA), awarded a design-build contract in 2020.

Background

The history of the original bridge and the challenges it faced over the years provide crucial context for the current viaduct project. When the original structure was built in 1934 as a replacement for a 1907 steel viaduct, the project team attempted to salvage much of the existing steel from the original structure. While this reuse of materials seemed resourceful at the time, it contributed significantly to the structural issues that led to the viaduct's deterioration and poor safety ratings.

By 2004, the viaduct showed clear signs of distress, with cracks appearing in the steel girders that necessitated immediate intervention. After temporary shoring was installed to support the compromised steel bents, the bridge remained operational for passenger vehicles and pedestrians. However, the restrictions placed on larger vehicles, such as buses and trailer trucks, highlighted the structure's changing functionality.

New Bridge

The new bridge is designed for a 75-year design life and in accordance with the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*,[1] the California Department of Transportation's (Caltrans') *California Amendments to the AASHTO LRFD Bridge Design Specifications*,[2] and American Railway Engineering and Maintenance-of-Way Association's *Manual for Railway Engineering*,[3] including seismic, roadway, and railway specifications. Moreover, the design incorporates features that accommodate bicycles, pedestrians, and accessibility requirements of the Americans with Disabilities Act, significantly enriching the experience for all users of the bridge. Critical project objectives are to improve the alignment

After cracks appeared in the steel bents of the existing bridge, temporary shoring was needed and only passenger vehicles and pedestrians could be accommodated. Photo: San Bernadino County Transit Authority.

profile

MOUNT VERNON VIADUCT, SAN BERNADINO, CALIFORNIA

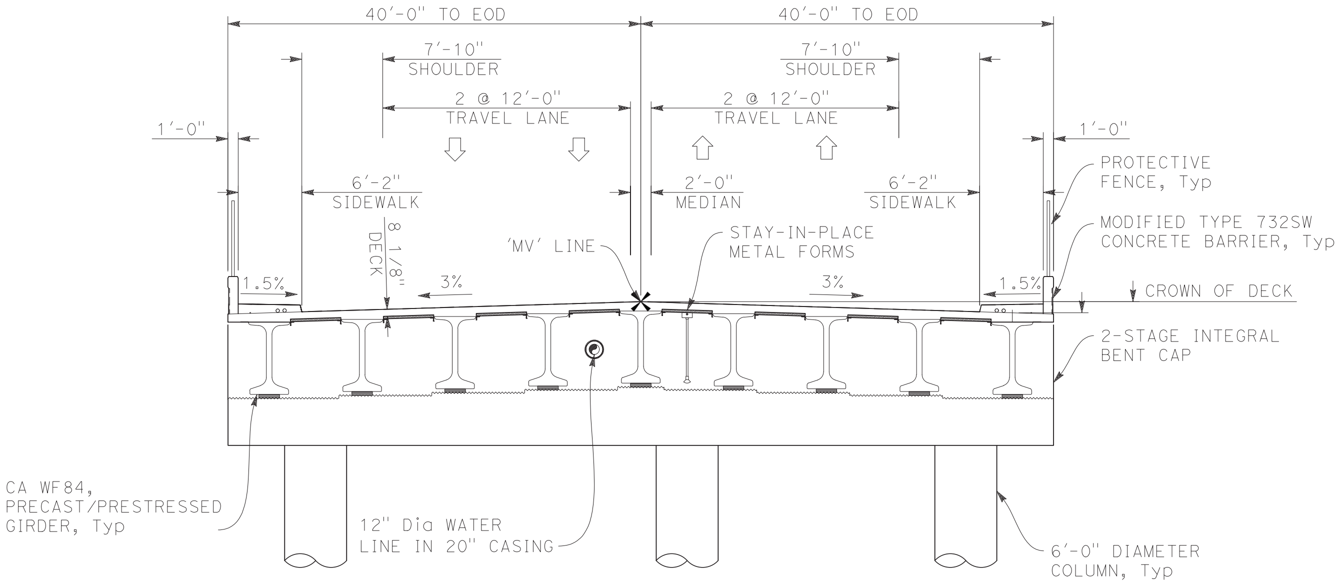
BRIDGE DESIGN ENGINEER: COWI North America, Oakland, Calif.

OTHER CONSULTANTS: Foundation designer: Dan Brown & Associates, Knoxville, Tenn.; civil engineer: Hernandez Kroone & Associates, San Bernadino, Calif.

PRIME CONTRACTOR: Traylor-Granite Joint Venture, Long Beach and Watsonville, Calif.

CONCRETE SUPPLIER: Robertson Ready Mix, San Bernadino, Calif.

PRECASTER: Girders and partial-depth panels: Con-Fab California LLC, Shafter, Calif.—a PCI-certified producer



New bridge cross section for spans 1 and 2 of the Mount Vernon Viaduct. Figure: COWI.

at the southern end of the bridge and to raise the vertical clearance by approximately 2 ft, thereby meeting the standard minimum vertical clearance of 24 ft for BNSF railways, 24 ft 6 in. for Metrolink regional rail lines, and 15 ft for local roads at the 3rd Street overcrossing. To achieve this increased clearance without drastically deviating from the profile of the existing bridge, the project team chose precast, prestressed concrete girders based on their efficiency and an outstanding span-to-depth ratio of roughly 25:1.

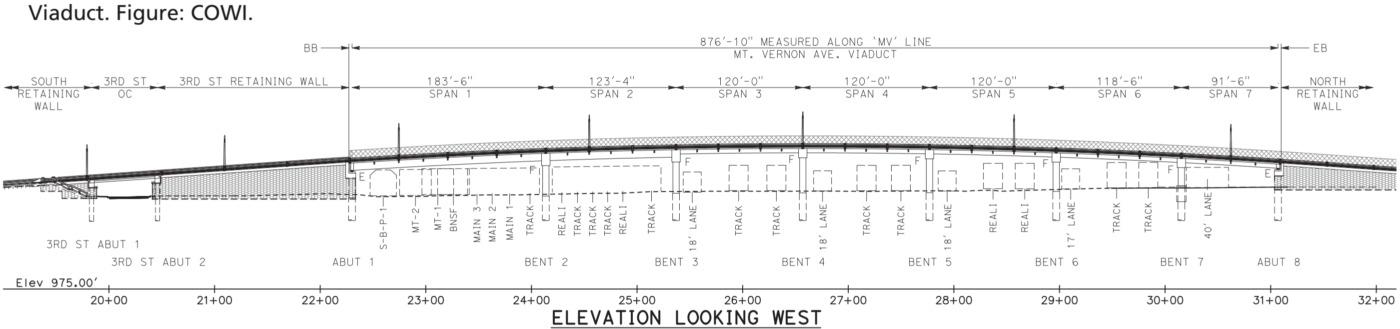
The seven spans of the Mount Vernon Viaduct cross over four Metrolink regional rail lines and 18 BNSF lines. The BNSF lines consist of three mainline tracks, six storage tracks, and nine intermodal tracks that are constantly active, with trains being assembled around the clock. Ensuring minimal disruption to the ongoing operations of both Metrolink and BNSF was a key requirement for the project, and precast, prestressed concrete girders were the ideal solution because they could be erected without the use of falsework in the busy rail yard.

The existing bridge consisted of four 10-ft travel lanes with no median or shoulder, and 3.5-ft-wide sidewalks on both sides. In contrast, the new Mount Vernon Viaduct will feature four 12-ft travel lanes, a 2-ft median, 7-ft 10-in. shoulders, and 6-ft 2-in. sidewalks in each direction. This upgrade will enhance vehicular movement and significantly improve pedestrian safety and accessibility.

Innovative Solutions

The design-build team introduced several Alternative Technical Concepts (ATCs) during the proposal phase, one of which involved replacing approximately 185 ft of the overall bridge length with an earth-filled precast concrete retaining-wall system from the 3rd Street overcrossing to the Metrolink right-of-way. The addition of the wall effectively split the viaduct

Profile of the new structure shows the one-span 3rd Street crossing, the retaining-wall system, and the seven-span Mount Vernon Viaduct. Figure: COWI.



CITY OF SAN BERNADINO AND SAN BERNADINO COUNTY TRANSPORTATION AUTHORITY, OWNERS

BRIDGE DESCRIPTION: 3rd Street overcrossing: 62-ft 6-in. single-span precast, prestressed concrete girders with cast-in-place concrete deck. Mount Vernon viaduct: seven-span bridge with spans ranging from 91 ft 6 in. to 183 ft 6 in. (center-to-center of piers); precast, prestressed concrete girders; and cast-in-place concrete deck.

STRUCTURAL COMPONENTS: 3rd Street overcrossing: eight 63-ft 10-in.-long, 28-in.-deep NEXT E girders with a 4½-in.-thick cast-in-place concrete deck. Mount Vernon Viaduct: spans 1 and 2: eighteen CA WF84 precast, prestressed concrete girders decked with stay-in-place metal deck forms and 8⅛-in.-thick cast-in-place concrete deck; spans 3–7: thirty-five CA WF48 precast, prestressed concrete girders with 258 partial-depth precast concrete panels and 9-in.-thick cast-in-place concrete deck.

BRIDGE CONSTRUCTION COST: \$105 million (approx. \$1400/ft^2)

Setting of a prestressed concrete Northeast Extreme Tee (NEXT) girder at the 3rd Street overcrossing. The shallow 28-in.-deep NEXT girder is used to provide the 15-ft minimum vertical clearance. Photo: Dany Schimpf Photography.

into two distinct structures: the single-span 3rd Street overcrossing and the seven-span Mount Vernon Viaduct. This ATC improved project aesthetics with a landscape buffer replacing rundown areas that were frequently targeted by graffiti. The reduction in total elevated structure reduced the project's initial cost and minimized future maintenance.

Foundation Design Challenges

The location of the bridge presents geotechnical challenges. Situated near both the San Andreas and San Jacinto fault lines, the structure must be designed for very high seismic demands, including a peak ground acceleration of approximately 0.9 *g*, a peak spectral acceleration close to 2.0 *g* at 0.3 seconds, and a spectral acceleration of about 1.7 *g* at 1.0 second.

The design uses Caltrans Type I column-to-pile connections,[4] which are less common than Type II shafts using an oversized foundation. This choice enabled maximum design efficiency, but it also involved substantial analysis and design efforts to meet the desired seismic performance. The design also required special attention to the detailing of the shaft-to-column steel reinforcing cage splice. Notably, the Type I detail eliminated the need for a separate column cage, which would have been cast into the pile and required temporary bracing/guying. With the Type I detail, the pile reinforcement was simply spliced to the column bars using couplers at the same time the forms were installed, thereby minimizing disruption in the rail yard.

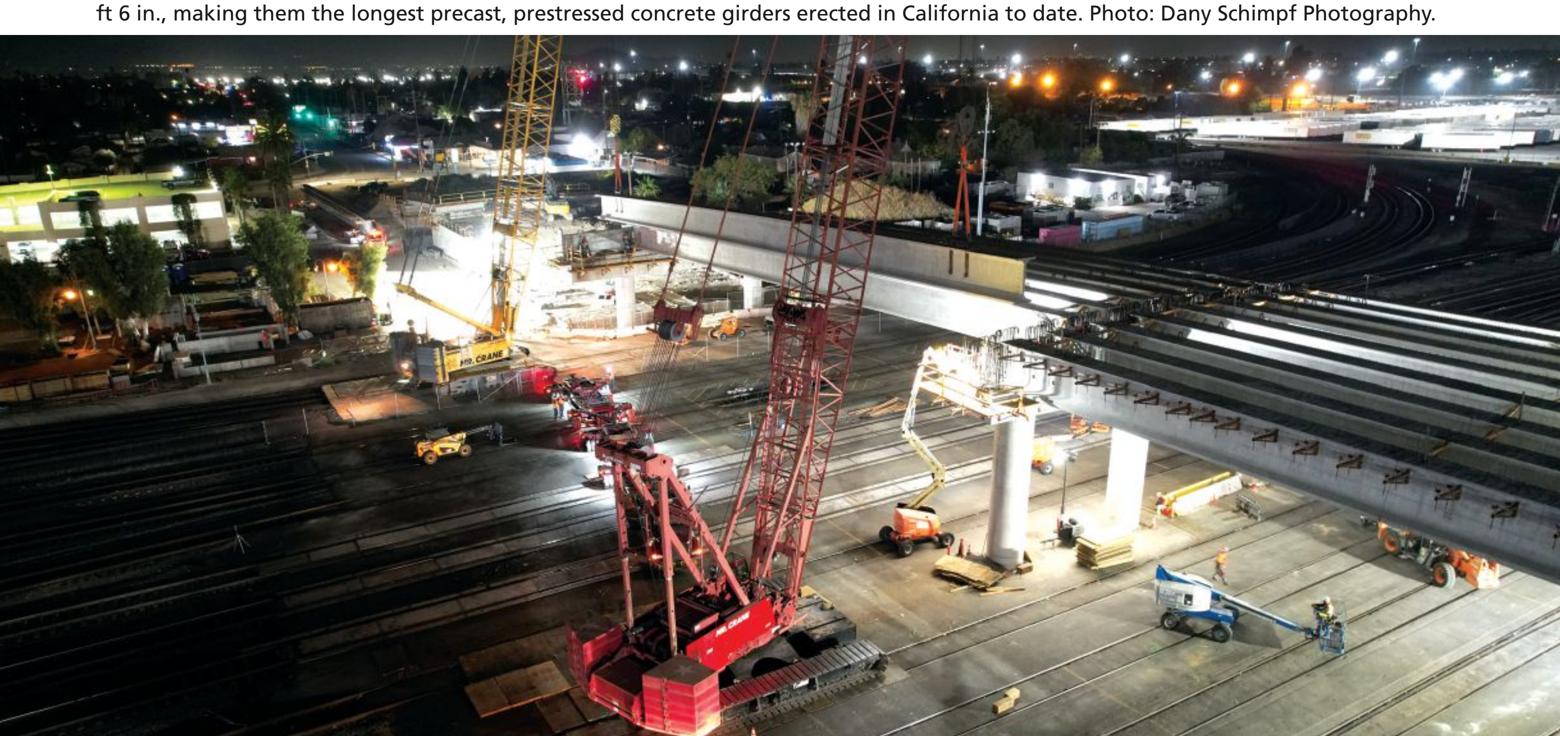
The design-build team opted to use drilled shafts instead of the cast-in-steel-shell piles (driven pipe piles filled with reinforced concrete) specified in the final report of the project. This decision was critical for both safety and practicality, as it allowed for construction work to be conducted close to operational rail lines while minimizing unnecessary vibrations. Specifically, the team used full-depth temporary casings advanced and withdrawn by an oscillator to facilitate the installation of the shafts with minimal impact to adjacent rail lines. This methodology enhanced safety and expedited the foundation installation process.

A significant challenge arose with the design of bent 7, the last interior bent on the northern end of the bridge, which was considerably shorter than the other interior bents due to the bridge profile's fall. This height differential led to high seismic demands on the short, stiff columns. To address this, the effective column length was increased by using a below-grade isolation casing, which reduced the bent stiffness to align more closely with the other bents and lowered the seismic design load on the shorter columns.

Superstructure Design

As noted previously, the use of the ATC to replace a portion of the bridge with a precast concrete retaining-wall system

Setting of a California WF84 (CA WF84) girder on span 1 of the new Mount Vernon Viaduct. The longest CA WF84 girders measure 182 ft 6 in., making them the longest precast, prestressed concrete girders erected in California to date. Photo: Dany Schimpf Photography.



Detail of an original steel corbel on the old bridge at the sidewalk (left) and a rendering of the precast concrete corbel detail for the new viaduct (right). Photo (left): San Bernadino County Transit Authority. Figure (right): COWI.

resulted in a superstructure design that is for two distinct bridges. The design for the 3rd Street overcrossing uses low-profile Northeast Extreme Tee (NEXT) girders, the first implementation of such precast, prestressed concrete sections in California. The NEXT E girders selected are only 28 in. deep and double as the formwork for the 4½-in. cast-in-place topping slab. Using these low-profile girders was critical to provide the 15-ft vertical clearance without requiring significant changes to the existing profile that would alter the southern roadway tie-in.

Of the seven spans in the Mount Vernon Viaduct, the two longest spans use California WF84 (CA WF84) girder sections with forty-eight 0.6-in.-diameter straight strands. The longest single span measures 182 ft 6 in., making it the longest single precast, prestressed concrete girder erected in the state to date. Another distinctive aspect of the long girders was the use of a secondary pour on the top flange, which followed a parabolic profile from ½ in. at the ends to 4 in. at midspan. The secondary pour was completed in the casting yard with the girders shored, which prevented any deflection from the added concrete of the secondary pour at the time of placement, mitigated differential camber, enhanced the section properties of the girders, and reduced deflection from the

The new seven-span Mount Vernon Viaduct bridge and the one-span 3rd Street crossing use precast, prestressed concrete girders. Photo: Dany Schimpf Photography.

The California WF84 girders are transported to the site using specialized hauling equipment to accommodate the weight and length of the large girders. Photo: Dany Schimpf Photography.

cast-in-place deck. The remaining five spans of the viaduct, ranging in length from 91 ft 6 in. to 120 ft, use CA WF84 girders with eighteen 0.6-in.-diameter harped strands. These shallower girders did not incorporate a secondary pour.

To achieve the low depth-to-span ratios chosen to maximize clearance, the girder designs used a 28-day concrete strength of 10,000 psi. Fabrication occurred at a precast concrete facility in Shafter, Calif., and the girders were transported to the site using specialized hauling equipment during nighttime hours to minimize disruption to rail service. The design-build team also erected all the girders during nighttime closures to avoid disruption of Metrolink and BNSF operations. In total, girder erection took approximately two months: the 3rd Street girders were set in a single 10-hour shift and the girders for the viaduct were all set within one month.

The spans with CA WF84 precast, prestressed concrete girders use stay-in-place metal deck forms and an 8⅛-in.-thick cast-in-place concrete deck. The spans with CA WF48 precast, prestressed concrete girders use partial-depth precast concrete panels and a 9-in.-thick cast-in-place concrete deck. The stay-in-place forms and partial-depth precast concrete panels significantly minimize the amount of forming required over the rail yard and resulted in a reduced schedule as compared to traditional forming for cast-in-place concrete decks. Because the girder ends were cast integrally into the bent caps, the design eliminated the need for bearings within the rail yard. This drastically reduced the need for future access for inspections in the rail yard—a significant step to minimize the scale of ongoing maintenance efforts.

Aesthetic Considerations

As the new viaduct was designed, the project team and stakeholders carefully considered the historical significance and aesthetics of the structure being replaced. Caltrans and the California State Historic Preservation Office established a memorandum of agreement emphasizing the importance of preserving the character-defining features of the original structure. This collaborative effort aimed to ensure that the new viaduct would be visually compatible with the adjacent Santa Fe Depot and would incorporate several aesthetic elements of the original structure. Features such as the existing stairwell on the south side of the bridge, along with specific lighting and railing details, were identified as critical components worthy of preservation.

Conclusion

The new Mount Vernon Viaduct represents the third generation of this critical transportation link. Because the project team seamlessly blended lessons learned from the original structures with state-of-the-art engineering and design practices that address current seismic and safety codes, this new crossing is poised to serve the San Bernardino area for decades to come. As the construction progresses toward completion, the new Mount Vernon Viaduct is set to fulfill its role as a vital connector between two significant areas of San Bernardino, facilitating safe and efficient transportation routes for both vehicles and pedestrians alike. Achieved through careful planning, innovative design, and proactive engagement with historic preservation efforts, the project symbolizes a forward-thinking approach to infrastructure development that honors the past while looking toward a sustainable future.

References

1. American Association of State Highway and Transportation Officials (AASHTO). 2012. *AASHTO LRFD Bridge Design Specifications*. 6th ed. Washington, DC: AASHTO.
2. California Department of Transportation (Caltrans). 2014. *California Amendments to the AASHTO LRFD Bridge Design Specifications*. 2012 6th ed. Sacramento, CA: Caltrans.
3. American Railway Engineering and Maintenance-of-Way Association (AREMA). 2019. *Manual for Railway Engineering*. Lanham, MD: AREMA.
4. Caltrans. 2019. *Seismic Design Criteria*. Version 2.0. Sacramento, CA: Caltrans.

Claus Frederiksen is the head of discipline, roads and major crossings west, for COWI. Benjamin Turner is a senior engineer for Dan Brown and Associates.

A Crack Is Not a Crack: End Region Cracking in Prestressed Concrete Components

by Dr. Oguzhan Bayrak, University of Texas at Austin

This article, which is the fourth in this series, addresses the cracking that may occur in the end regions of prestressed concrete components. Within this context, we will address pretensioned and post-tensioned components separately.

Pretensioned Concrete Components

When I think about the end regions of pretensioned concrete girders, I am reminded of the historical evolution of standard girder types in the United States. Since the inception of pretensioning technology, the criticality of the complex stress state that occurs within the end regions of pretensioned concrete girders was understood. At early stages, end blocks and special reinforcement details were employed to ensure proper transfer of the pretensioning force to surrounding concrete and to interior portions of the beam, away from the end regions. When sufficient experience and knowledge were gained, the use of end blocks was eliminated as owners began taking advantage of sections such as AASHTO standard girder shapes. Meanwhile, through advances in materials science, the tensile strength of pretensioning strand was increased from 250 to 270 ksi. The stress-relieving process used in the early days of strand technology gave way to a better method (strain tempering) intended to reduce relaxation-related losses. With the advance of strain tempering, low-relaxation strands were introduced. More recently, ½-in.-diameter strands, which were once used routinely in highway construction, have been largely replaced by 0.6-in.-diameter strands. Current research continues to push those boundaries by exploring the use of 300 ksi material, as well as 0.7-in.-diameter strands.[1] With a typical 2-in. spacing of strands, these changes amount to a greater prestressing force within the bottom flanges of the girders, and where harped strands are used, in the web as well. It is worth noting that with the introduction of 0.6.-in.-diameter strands, a similar increase in the prestressing force was introduced to the typical 2-in. grid by increasing the force per strand compared with 0.5-in.-diameter strands.

Parallel to developments in prestressing reinforcement technology, concrete materials science also advanced. Chemical admixtures, supplementary cementitious materials, and microfine aggregates all helped improve particle packing and reduced water use in modern concrete mixtures. As a result, high-performance, high-strength concretes became widely available. With higher-performing materials, the quest to optimize sections for better structural performance led to the variety of different bulb tees that we see in the industry today. Such sections commonly have large bottom flanges to accommodate the large number of strands. These advances also helped concentrate additional prestressing forces in bottom flanges, allowing for longer span capabilities for a given beam section with a given depth.

An unintended consequence of increasing the pretensioning force within the end region of pretensioned girders and optimizing the concrete section is the end-region cracking that we sometimes see in modern pretensioned girders. **Figure 1** shows one such example. The *CEB-FIP Model Code 1990*[2] identifies various types of cracks that may form in the end regions of pretensioned girders (**Fig. 2**). As we can see in Fig. 2, the complex state of stress within approximately one component depth *h* from the end of a pretensioned girder causes multiple effects. Let us examine spalling stresses first. As part of a thought exercise, let's make a horizontal cut between the top flange of a pretensioned girder and the web. Let us also imagine that our beam is made from an ordinary kitchen sponge so that we can envision exaggerated displacements. When we apply a compressive force at the bottom flange (that is, squeeze the bottom flange inward), the girder will camber upward. As the girder cambers upward, the top flange will separate and move away from the web at the location where we introduced the horizontal cut. Of course, in an actual beam, there is no cut; therefore, we must restore the integrity of the beam such that deformations of the top flange can be made compatible with the rest of the beam. In an actual concrete beam (Fig. 1), cracking occurs and engages the vertical reinforcement. Such cracks are called "spalling cracks" or "compatibility cracking" in the terminology used in the Model Code 1990. The cracks near the top flange of the beam and upper portions of the web in Fig. 1 are examples of spalling cracks.

Bursting cracks occur as the concentrated force transferred into the concrete section starts to spread in the transverse direction. Let us recall that the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[3] specifies that transfer length is 60

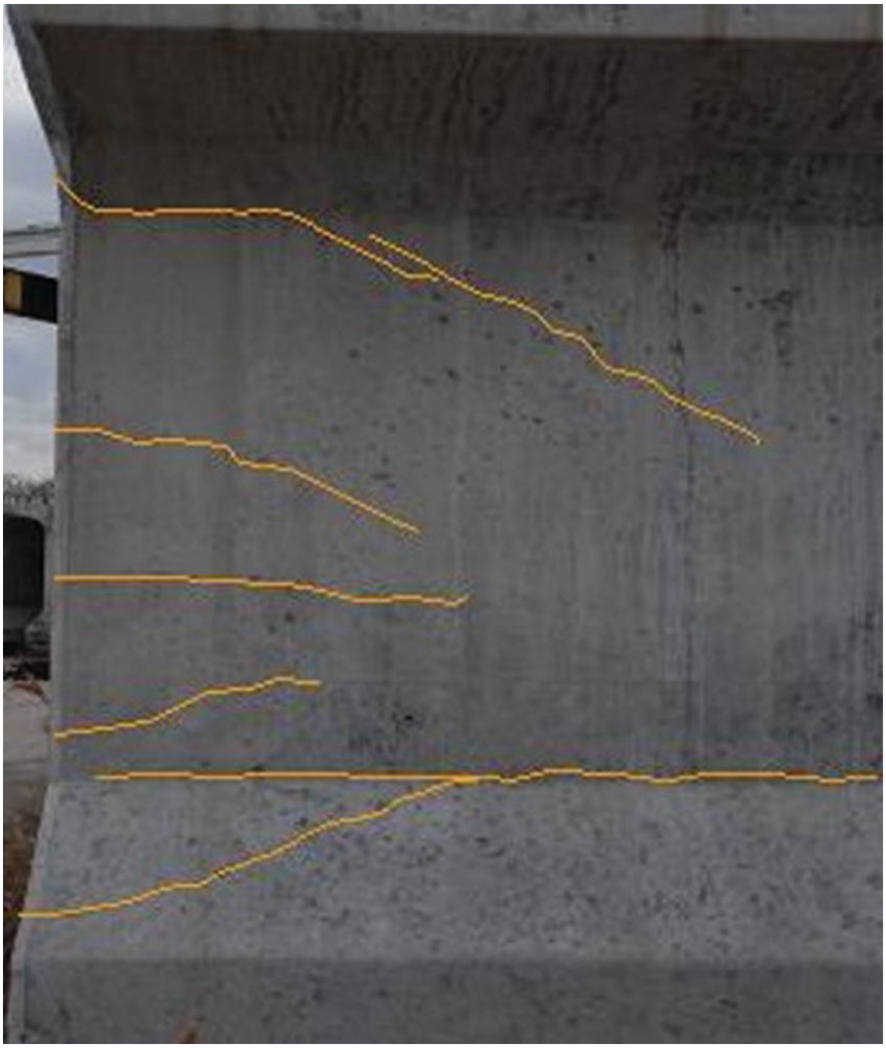


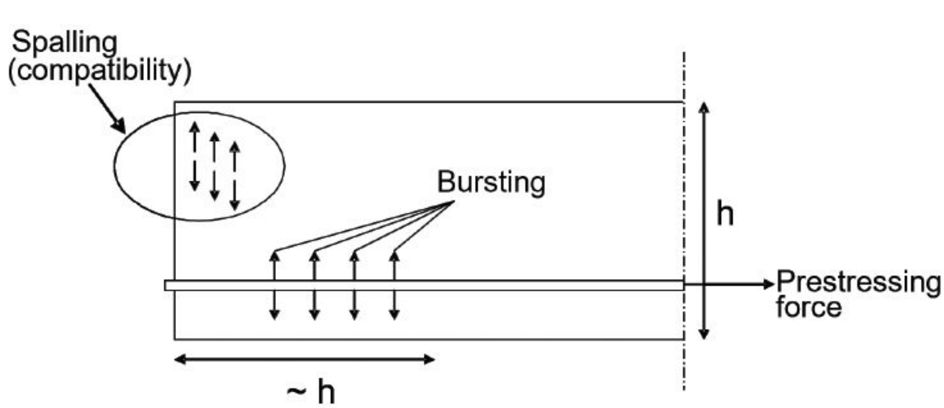
Figure 1. End-region cracking (enhanced) in a pretensioned concrete I-girder. Photo: Matt O'Callaghan and FSEL Researchers.[4]

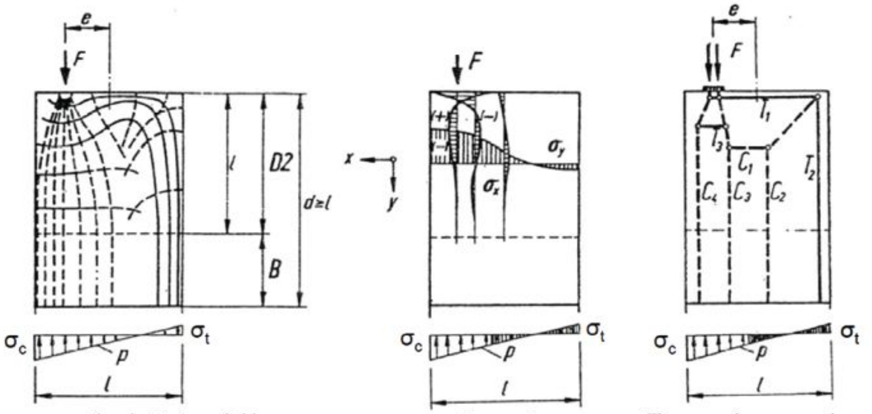
times the strand diameter. For 0.6-in.-diameter strands, this distance is 36 in. At a section approximately 2.5 to 4 ft from the end of a beam, we would see the highest transverse forces that can be attributed to bursting effects. These cracks typically form within the bottom flange and in close proximity to the bottom flange–to–web interface. Both the vertical reinforcement that crosses the bottom flange–to–web interface and the confinement reinforcement we provide within the bottom flange of pretensioned girders help us address the demand imposed by large prestressing forces in modern bulb tees.

The confinement reinforcement provided within the bottom flange also helps in tackling the Hoyer effect–related transverse stresses within the very end of a beam, over the transfer length. The Hoyer effect can be visualized as the tendency of the wires that form a seven-wire strand to expand back to their original (unstressed state) diameter and their original orientation as the strands are wedged in the surrounding concrete at prestress transfer. This effect is the primary mechanism by which the force transfer occurs, in addition to the mechanical interlock that stems from the helical nature of the circumferential wires that surround the king wire in a seven-wire strand.

Of all the cracks shown in Fig. 1, the horizontal crack at the bottom flange–to–web interface has been a primary cause for concern for my research team at the University of Texas at Austin. More specifically, we have focused our attention on nontraditional shear failure modes in a number of studies, starting with an experimental study conducted at the Phil M. Ferguson Structural Engineering Laboratory, *Tensile Stresses in the End Regions of Pretensioned I-Beams at Release*.[4] We have researched the additive nature of stresses imposed by reinforcing bars that cross the end-region cracks and those imposed by the shear stresses caused by the dead and live loads as they are transferred to the supporting bents. With all strength and serviceability considerations, we concluded that bundling some bars with traditional shear reinforcement in the end regions of the bulb tees would be a good detailing practice to (a) materialize the shear strength calculated by the shear design provisions based on modified compression field theory in the AASHTO LRFD specifications, and (b) keep the end-region crack widths to a minimum to ensure long-term durability. Today, several states employ similar details where several sets of stirrups are bundled with additional or special reinforcement in the end regions, with the ultimate goal of reducing transverse stresses in those bars and limiting the widths of the cracks that form in the end regions at prestress transfer.

Figure 2. Schematic of spalling and bursting stresses in the end region of a pretensioned concrete component.[2,4]





The elastic stress field that forms when a concentrated load is applied

Stresses in the local zone

The general zone near the post-tensioning anchorages can be designed and detailed using strut-and-tie models (STM) like the one shown here

Figure 3. The stress immediately adjacent to a concentrated load on a member are complex. Source: Figure C5.8.2.7[1-3] from the *CEB-FIP Model Code 1990*.[2,5]

What Do the AASHTO LRFD Specifications Say?

Article 5.9.4.4.1 of AASHTO LRFD specifications is devoted to splitting resistance in pretensioned concrete components. That section states the following:

> *The factored splitting resistance of pretensioned anchorage zones provided by reinforcement in the ends of pretensioned beams shall be taken as:*
>
> $$P_r = f_s A_s \qquad (5.9.4.4.1\text{-}1)$$
>
> *where:*
>
> f_s = *stress in steel not to exceed 20.0 ksi*
>
> A_s = *total area of reinforcement located within the distance* h*/4 from the end of the beam (in.²)*
>
> h = *overall dimension of precast member in the direction in which splitting resistance is being evaluated (in.)*
>
> *For pretensioned I-girders or bulb tees,* A_s *shall be taken as the total area of the vertical reinforcement located within a distance of* h*/4 from the end of the member, where* h *is the overall height of the member (in.).*
>
> *For pretensioned solid or voided slabs,* A_s *shall be taken as the total area of the horizontal reinforcement located within a distance of* h*/4 from the end of the member, where* h *is the overall width of the member (in).*
>
> *For pretensioned box or tub girders,* A_s *shall be taken as the total area of vertical reinforcement or horizontal reinforcement located within a distance* h*/4 from the end of the member, where* h *is the lesser of the overall width or height of the member (in.).*
>
> *For pretensioned members with multiple stems,* A_s *shall be taken as the total area of vertical reinforcement, divided evenly among the webs, and located within a distance* h*/4 from the end of each web.*
>
> *The resistance shall not be less than four percent of the total prestressing force at transfer.*
>
> *The reinforcement shall be as close to the end of the beam as practicable.*
>
> *Reinforcement used to satisfy this requirement can also be used to satisfy other design requirements.*

In accordance with Article 5.9.4.4.1, we must pay attention to providing special end-region reinforcement through the web, as close to the beam end as possible. We must appreciate the fact that the 20-ksi limit on the reinforcement stress is placed as a serviceability limit to control the widths of the cracks that may form within the end regions. In a beam that does not have sufficient reinforcement to meet the required quantity of reinforcement specified in Article 5.9.4.4.1, we can expect to see end-region cracks that are wider than those implied with a 20-ksi stress limit. Put simply, if we provide one-half of the reinforcement required by Eq. (5.9.4.4.1-1), we can expect to see end-region cracks that are approximately twice as wide. While this ratio will not directly translate to structural capacity lost due to wider cracks, it may have adverse implications for the durability of the pretensioned girders, depending on the environmental exposure conditions.

In addition to the previously discussed requirements, Article 5.9.4.4.2 of the AASHTO LRFD specifications covers confinement reinforcement design considerations that reflect decades' worth of experience gained through construction and structural testing of pretensioned girders. The confinement reinforcement requirements of AASHTO LRFD specifications are as follows:

> *For the distance of 1.5*d *from the end of the beams other than box beams, reinforcement shall be placed to confine the prestressing steel in the bottom flange. The reinforcement shall not be less than No. 3 deformed bars, with spacing not exceeding 6.0 in. and shaped to enclose the strands.*
>
> *For box beams, transverse reinforcement shall be provided and anchored by extending the leg of stirrup into the web of the girder.*

Some states and designers find it practical to pair the confinement reinforcement with the shear reinforcement, ultimately resulting in a greater quantity of confinement reinforcement. Such decisions made by local jurisdictions reflect local experiences that complement AASHTO LRFD specifications and further enhance the performance of pretensioned girders.

Post-tensioned Concrete Components

Elements of the previous discussion also apply to post-tensioned concrete components, but there are distinctive aspects of behavior in post-tensioned components that warrant additional discussion. While post-tensioning technology provides opportunities that cannot be materialized by pretensioned components, the technology also provides challenges that are rooted in specific aspects of behavior that require separate treatment. With this in mind, let us examine a portion of a 1987 paper by Schlaich et al.[5] **Figure 3** illustrates the elastic stress field that forms when a concentrated load is applied (*F* in Fig. 3). The dashed lines illustrate the compression trajectories, and the solid lines signify the tension fields that develop under the load application. The stress trajectories in the left side of the figure indicate the complex nature of the stresses immediately adjacent to the bearing device. This region, also known as the local zone, is best studied through testing conducted by, or on behalf of, the post-tensioning system suppliers. Design provisions in the AASHTO LRFD specifications do not provide explicit guidance for detailing the local zone. That said, this region is typically reinforced with spiral reinforcement. Confinement offered by the spiral reinforcement facilitates the transfer of high levels of compressive stresses into the post-tensioned concrete component. Engaging transversely positioned confining reinforcement by applying longitudinal compressive stresses in this area may result in cracking within the immediate vicinity of the anchorage device. The general zone near the post-tensioning anchorages can be designed and detailed using strut-and-tie models (STM) like the one shown on the right side of Fig. 3. As can be seen in that STM, the post-tensioning force spreads, moving away from the anchorage point, creating a transverse tension field (T_3 in Fig. 3). This bursting effect is similar to the one discussed earlier for the pretensioned sections. Edge tension force T_1 and longitudinal tension force T_2 are additional tensile forces that must be resisted by the reinforcement provided within these regions. To resist T_1, T_2, and T_3 shown in Fig. 3, typical unfactored post-tensioning forces may cause stress or strain levels in ordinary reinforcement that will be in the linear elastic range of the material response but large enough to exceed typical concrete cracking strains. In other words, if the tie reinforcement is to meaningfully engage and reinforce the general zone, we expect to see some cracking. While these cracks are expected due to bursting, spalling (edge tension), or longitudinal tension effects, their widths and propensity can be controlled by using a sufficient quantity of well-detailed reinforcement in compliance with the AASHTO LRFD specifications.

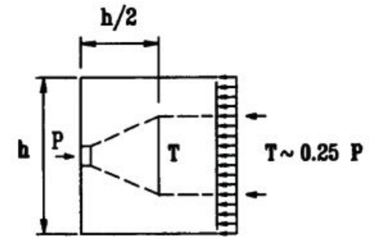
The actual structural details used and the geometry of concrete within and ahead of an anchorage zone play important roles in determining the bursting as well as edge tension effects, especially when there are discontinuities that disturb the flow of forces. In the AASHTO LRFD specifications, Commentary C5.8.4.5.1 includes a discussion that clearly highlights this fact, as follows:

> *Discontinuities, such as web openings, disturb the flow of forces and may cause higher compressive stresses, bursting forces, or edge tension forces in the anchorage zone. Figure C5.8.4.5.1-1* [reproduced herein as **Fig. 4**] *compares the bursting forces for a member with a continuous rectangular cross-section and for a member with a noncontinuous rectangular cross-section. The approximate equations may be applied to standard I-girders with end blocks if the longitudinal extension of the end block is at least one girder height and if the transition from the end block to the I-section is gradual.*

Figure 4 shows that the presence of an opening can increase the bursting effects by as much as a factor of 2 or more compared with a section without an opening. While this comparison may be viewed as an extreme case, we must recall the fact that the actual geometry of a concrete component must be properly considered in our STM selection to appropriately detail the general zones.

Conclusion

In a Professor's Perspective published in the Spring 2015 issue of *ASPIRE*®, I focused on Fritz Leonhardt's classic textbook, *Prestressed Concrete Design and Construction*.[6] Within the context of that article, I addressed the "ten commandments for the prestressed concrete engineer" by borrowing terminology from Professor Leonhardt. I would like to finish this article with his fifth recommendation: "Provide ordinary reinforcing bars transverse to the direction of prestressing force within the transfer length." After all these years, much research, and many success stories, those first principles in concrete construction remain as important as ever.



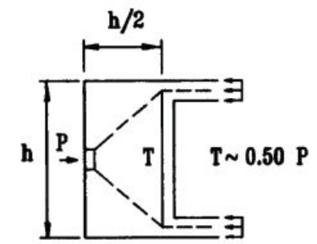


Figure 4. Effect of an opening in the anchorage zone. Source: Figure C5.8.4.5.1-1 from the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*.[3]

References

1. National Cooperative Highway Research Program (NCHRP). 2022. *Use of 0.7-in. Diameter Strands in Precast Pretensioned Girders*. NCHRP Research Report 994. Washington, DC: National Academies Press. https://doi.org/10.17226/26677.
2. Euro-International Concrete Committee (CEB) and International Federation for Prestressing (*fip*). 1990. *CEB-FIP Model Code 1990*. Lausanne, Switzerland: *fib* (International Federation for Structural Concrete).
3. American Association of State Highway and Transportation Officials (AASHTO). 2020. *AASHTO LRFD Bridge Design Specifications*. 9th ed. Washington, DC: AASHTO.
4. O'Callaghan, M.R., and O. Bayrak. 2008. *Tensile Stresses in the End Regions of Pretensioned I-Beams at Release*. Technical Report IAC-88-5DD1A003-1. Austin, TX: Center for Transportation Research, Bureau of Engineering Research, University of Texas at Austin. https://fsel.engr.utexas.edu/pdfs/IAC885DD1A003-1.pdf.
5. Schlaich, J., K. Schäfer, and M. Jennewein. 1987. "Toward a Consistent Design of Structural Concrete." *PCI Journal* 32 (3): 74–149. https://doi.org/10.15554/pcij.05011987.74.150.
6. Leonhardt, F. 1964. *Prestressed Concrete Design and Construction*. 2nd ed. Berlin, Germany: Wilhelm Ernst & Sohn.

Concrete Segmental Bridges—

Preliminary Design Approximations for Cross-Section Dimensions

by R. Kent Montgomery, GM2 Associates

This article, which is the second in a series discussing preliminary design approximations, discusses determining preliminary dimensions for cross sections of post-tensioned concrete segmental bridges. These approximations apply to precast concrete and cast-in-place (CIP) cross sections.

As noted in the previous article in this series (see the Winter 2025 issue of *ASPIRE*®), the detailed final design demands will be needed before a structure's design can be approved. However, the preliminary approximations presented herein provide a good starting point for determining cross-section dimensions that typically result in an optimal final design.

The longitudinal design behavior of a concrete segmental bridge typically governs the web thickness. After the basic dimensions of the bridge have been determined, a good preliminary approximation for the web thickness b_w is to provide a ratio of 0.36 to 0.39 in.^{2} of web area (based on theoretical vertical webs) per square foot of deck span area (**Fig. 1**). The preliminary web width of a cross section is approximated as

$$b_w = \frac{\alpha WL}{nh}$$

where

b_w = thickness of one web, in.
α = constant between 0.36 and 0.39
W = deck width, ft
L = span length, ft
n = the number of webs
h = section depth at the pier, in.

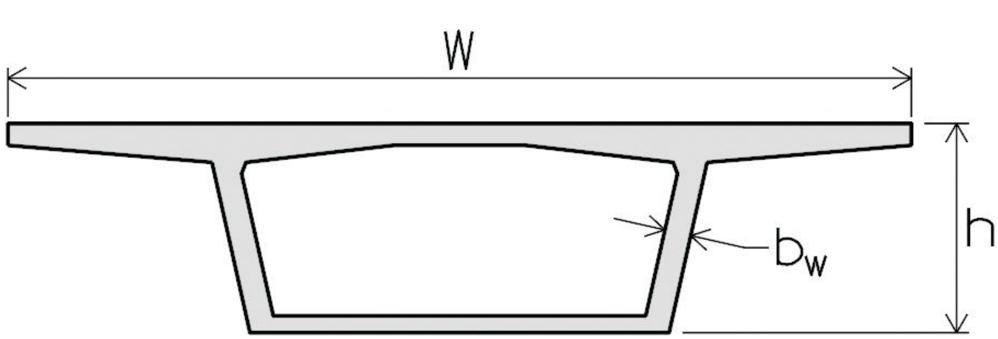
For example, if $\alpha = 0.36$ is selected for a 316-ft span with a deck width of 53 ft, two webs, and a structure depth of 16.5 ft (198 in.) at the pier:

$$b_w = \frac{\left(0.36\frac{\text{in.}^2}{\text{ft}^2}\right)(53\text{ ft})(316\text{ ft})}{2(198\text{ in.})} = 15.2\text{ in.}$$

Round b_w to 15 in. for advancement to final design. For box girders with two webs, each web will be the same thickness. For box girders with three webs, the thickness $n \times b_w$ is the total width of all three webs. For three-web sections, an indeterminate shear flow analysis is necessary in the final design to determine how much shear each web resists. For preliminary design, it can typically be assumed that the interior web will resist 40% of the shear and each exterior web will resist 30%. The individual web thicknesses can be proportioned accordingly from the total thickness nb_w. The minimum web thickness b_w for each web should not be less than 10 in. for preliminary design.

There is a possibility to further optimize the design for constant-depth structures by linearly tapering the thickness of each web from a maximum at the top to a minimum at the bottom slab. In this case, the web thickness, as determined previously, would be the thickness at the neutral axis. The reason for the tapering is that there can be significant bending near the top of the web due to deck loadings, but there is little bending near the bottom slab. The web thickness at the bottom slab should not be less than the minimum bottom-slab thickness or 8 in. Note that the minimum web width should be used in shear calculations using modified compression field theory in accordance with the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*.[1] For variable-depth bridges, it is recommended to use constant web widths along the height of the webs.

Figure 1. Basic dimensions of a concrete segmental cross section are used for approximating web width. All Figures: R. Kent Montgomery.



The longitudinal design behavior typically governs the bottom-slab thickness t_{bs}. The previous article discussed determining the bottom-slab thickness near the piers for structures constructed using the balanced-cantilever method. If internal continuity tendons are present, the minimum bottom-slab thickness near midspan will likely be determined by the thickness required to accommodate post-tensioning ducts between the mats of reinforcement. If there are no internal tendons in the bottom slab (for example, in span-by-span construction), the bottom-slab thickness should not be less than 7 in. and the ratio of the bottom slab width to the bottom slab thickness b_{bs}/t_{bs} should not exceed 35. These limits also apply to balanced-cantilever construction but will likely not govern over the thickness required to accommodate post-tensioning ducts. In accordance with Article 5.6.4.7.2c of the AASHTO LRFD specifications, the compressive stress limit is reduced by the ϕ_w factor for b_{bs}/t_{bs} greater than 15 (**Fig. 2**). Thickening the bottom slab to decrease the compressive stress to meet the reduced stress limit may be required.

Transverse behavior usually sets the top-slab dimensions. The ratio of the cantilever wing length to interior span length (**Fig. 3**) should typically be set between 0.30 and 0.55, with 0.42 being optimal for transverse design. Cantilever wings that are longer or shorter than the optimal length have been used to achieve other design aims, such as achieving the required gore area geometry.

Top-slab thicknesses can be set as shown in **Fig. 4**. Figure 4 is for single-cell sections, but the principles can be extended to multiple-cell sections. The 9-in. minimum wing tip dimension is to accommodate transverse post-tensioning anchors in accordance with the AASHTO LRFD specifications. Designs for bridges without transverse post-tensioning, such as light-rail transit bridges, can use a minimum 8-in. wing-tip dimension. Haunch lengths for an interior transverse span should be at least 20% of the span, but they may be significantly longer to accommodate cantilever tendon ducts for balanced-cantilever bridges (**Fig. 5**). The haunch length for the cantilever wing is typically set such that the constant-depth portion of the wing does not exceed $LH_c = 10T_c$, unless a longer haunch length is required to fit tendon ducts.

The slope of the web can be used to set the desired bottom-soffit width. For example, highly sloped webs can be used to achieve a narrower bottom soffit and

Figure 2. Compressive stress limits are used to approximate the thickness of the bottom slab.

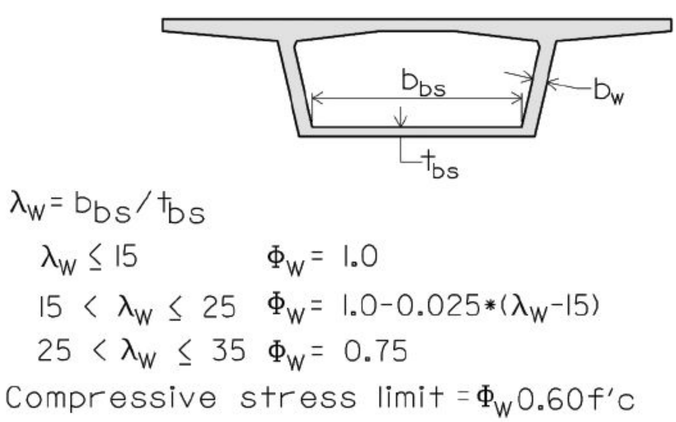


Figure 3. Variables for preliminary approximation of top-slab transverse proportions.

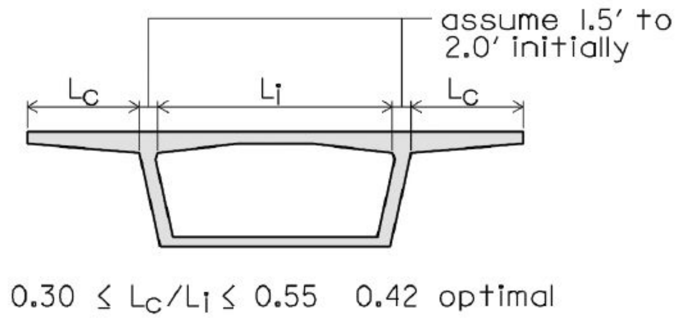


Figure 4. Minimum top-slab thicknesses at various locations in the cross section.

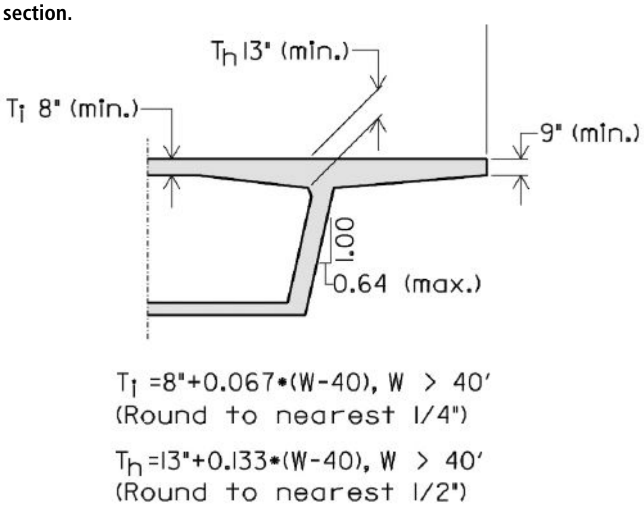
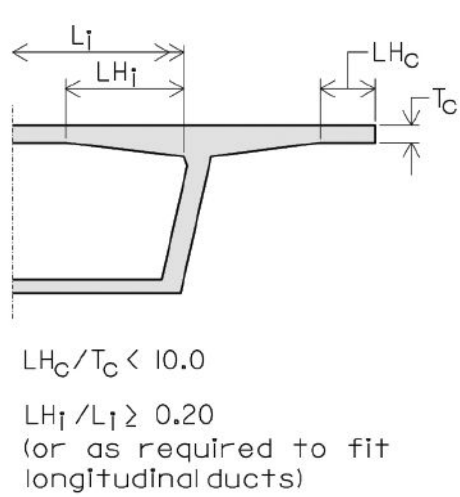


Figure 5. Preliminary haunch dimensions.



smaller pier cap dimension for wider decks. However, the web slope, usually defined on the outside surface of the web, typically should not exceed 0.64:1.00 (Fig. 4). Conversely, larger vertical web slopes (lower horizontal run to vertical rise) can be used to achieve the minimum soffit width to avoid bearing uplift due to overturning moments.

The cross-section dimensions should also consider constructability. The cross section should be dimensioned and detailed for simplicity. Casting cells should be as easy to use as possible such that one typical segment can be cast per day for each casting cell. The casting cell typically comprises five main forming surfaces (**Fig. 6**):

- Core form
- Web/wing form
- Soffit form
- Bulkhead
- Transverse bulkhead

The core dimensions shown in Fig. 6 are essentially the portion of the cross section formed by the core form and the web/wing form close to the cantilever root. Every effort should be made to keep the core dimensions constant for all segments. The core form can be extended down the web for casting variable-depth components. As long as the upper portion of the core form remains unchanged, this adjustment can be accomplished relatively easily and is not considered a change to the core dimensions.

Variable widths can most easily be handled by moving the transverse bulkheads, but this solution has limitations as the length of the cantilever wing is limited by flexural capacity. The width of the core form should be carefully selected to accommodate the required range of widths, and compromises to the optimal ratio of the cantilever wing length to interior span length at the extreme widths (maximum and minimum) may be needed. Using a variable-width core form adds significant complexity to the casting cell and should generally be avoided.

Variable depths are typically accommodated relatively easily using these methods:

- The web portion of the web/wing form is fabricated to accommodate the deepest cross section but remains unchanged from segment to segment.
- Removable panels are used at the bottom of the core form, with enough variation in the panels to accommodate the variable depth requirements.
- The soffit form is fabricated such that its height is adjustable (the core and web/wing forms are stationary). This allows the depth of the segments to vary. Due to the slope of the webs, the bottom-soffit width also varies, and this variation is accommodated by variable extensions from the main soffit form/table.
- The bottom portion of the bulkhead must also be adjusted, typically by variable panels.

These options are feasible if the web slope is kept constant. Detailing variable-slope webs adds significant complications to a design, including having to warp the web forms, and should be avoided when possible. While the modifications described in the previous list are readily achievable when casting one segment per day in each casting cell, they present necessary complications to the casting cell. Therefore, when allowed by short-to-medium span lengths, constant-depth spans are preferred.

Variable crown points can be handled by adjusting the top-slab thickness to achieve the desired cross slopes, while allowing the core form to remain unchanged. This philosophy works when the variation in the top slab results in reasonable thicknesses. If the variation is in the shoulders, it is possible to fabricate a casting cell where the wing form rotates relative to the web form.

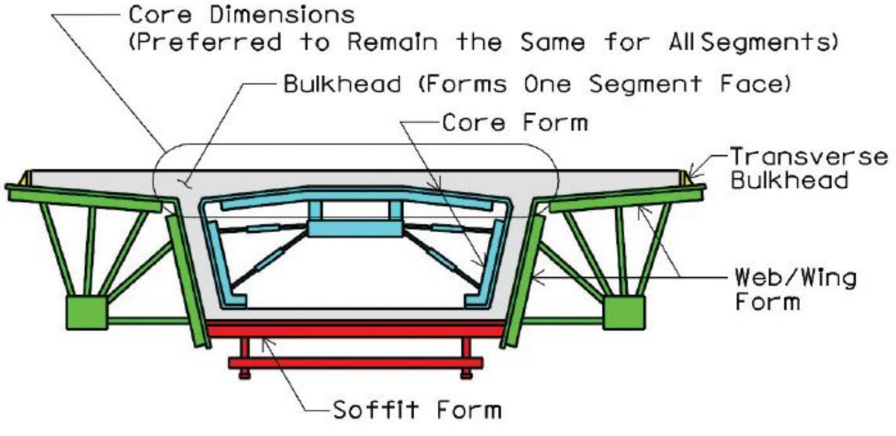
The information in this article, together with the insights from the previous article, help the design team set preliminary superstructure cross-section dimensions. The next steps in a good preliminary superstructure design are to determine the amount of post-tensioning, tendon layouts, anchorage locations, and the resulting diaphragm dimensions and deviator locations. These post-tensioning design parameters for the next steps can be determined with simplified preliminary analysis procedures that avoid complex time-dependent models. This approach generates a sound preliminary design as a starting point for a final design, with minimal changes needed during the final design process. The next article in this series will present these simplified methods for preliminary analysis.

Reference

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

R. Kent Montgomery is a senior technical manager for complex bridges at GM2 Associates in Denver, Colorado. He serves as the chair of the Design Subcommittee of the American Segmental Bridge Institute's Technology and Innovation Committee.

Figure 6. The five main forming surfaces and the core dimensions in a typical casting cell.



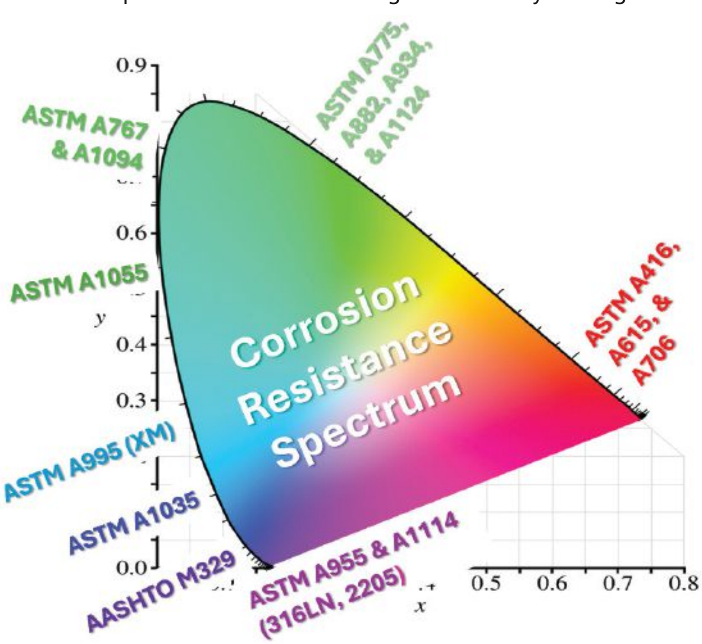
Corrosion-Resistant Steel Reinforcement for Concrete Structures: Defining Resiliency Using the Color Spectrum

by Steven Nolan, Florida Department of Transportation, Dr. Salem Faza, CMC, and Dr. Samuel McAlpine, Allium Engineering Inc.

Following the *ASPIRE*® Winter 2025 article "Corrosion-Resistant Fiber-Reinforced Polymer Reinforcement for Concrete Structures," and the *ASPIRE* Spring 2020 article "Epoxy-Coated Prestressing Steel Strand," we continue the discussion with this article about corrosion-resistant steel reinforcement. Corrosion resistance for steel reinforcement should be viewed as a spectrum of options with a broad range of durability performance resulting from an equally broad range of steel chemistry, microstructure, and coating or cladding systems.

It is worth noting that corrosion of steel bars embedded in a concrete matrix is significantly different from general steel corrosion under atmospheric conditions. The highly alkaline internal environment of a concrete member is initially compatible with conventional carbon steel bars such as those conforming to ASTM A615[1] or ASTM A709,[2] and it has contributed to the long-term success and economy of concrete structures for more than a century. However, time and exposure conditions can eventually change the internal concrete environment, significantly reducing the initially high pH (typically ranging from 12.5 to 13.5) via atmospherically induced concrete carbonation and matrix dissolution, or accelerated steel corrosion potential catalyzed by chemical ion ingress. The rate of steel reinforcement deterioration is significantly influenced by external and internal concrete conditions, exposure cycling, and—perhaps most importantly—time.

Figure 1. Comparing the relative economic and performance strengths of various types of reinforcing steel is analogous to this depiction of how visible colors of light relate to one another. The x-axis represents the indexed perception of value, and the y-axis is the life-cycle cost analysis rating. Figure: Adapted by Steven Nolan from https://commons.wikimedia.org/wiki/File:CIExy1931.svg.



External conditions include average and extreme exposure temperatures, the pH (acidity or alkalinity) of atmospheric and surface moisture, chemical ion content (chloride and sulfate), and even microbiologically induced corrosion. Internal factors include the chemistry of the concrete binder (portland cement, pozzolans, and supplementary cementitious materials), matrix porosity and interstitial void connection structure, the degree of moisture saturation, aggregate types, and the aggregates' susceptibility to alkali reactions (alkali-aggregate reactions and alkali-silica reactions), which induce matrix cracking.

We have broadly classified steel reinforcement on a spectrum of available corrosion resistance based on coatings, constituent metallic alloy chemistry, and the crystalline microstructure created during the rolling, quenching, and tempering production processes. The color palette or nonlinear spectrum in **Fig. 1** illustrates the classification of reinforcement from "rusty red" to "shimmering violet."

On the red, or "rusty," end of this spectrum is conventional carbon steel reinforcement (ASTM A416,[3] ASTM A615,[1] and ASTM A706[4]). The violet or more "durable" end of the corrosion-resistance spectrum includes highly alloyed or duplex stainless steel (ASTM A955[5] and ASTM A1114[6]) and titanium alloy bars (ASTM B1009[7]) used for near-surface mounting.[8] In the middle of the spectrum are yellow, orange, and green zones for reinforcement with various barrier coatings or lower-alloyed steel reinforcement options such as epoxy-coated (ASTM A775,[9] ASTM A882,[10] ASTM A934,[11] and ASTM A1124[12]), galvanized (ASTM A767[13] and ASTM A1094[14]), dual-coated epoxy-zinc (ASTM A1055[15]), and low-nickel molybdenum-free austenitic stainless steel (ASTM A955[5]).

Developments over the last quarter century have introduced and refined what might be considered a "blue" zone of steel reinforcing options, including low-carbon chromium steel (ASTM A1035[16]), other lower-chromium alloyed steels (AASHTO M334[17]) and stainless clad carbon steel (AASHTO M329[18]). While the ultimate durability of a concrete structure is influenced by many factors, the selection of an appropriate reinforcement type for the intended purpose and expected service life is paramount to satisfying both resilience and sustainability objectives.

Types of corrosion-resistant steel reinforcing bars in the blue zone are not as well known among engineers as products in other zones of the spectrum. ASTM A1035 bars (ChromX 2000, 4000, and 9000 series) have a specific layered microstructure that contributes to both strength and corrosion resistance (**Fig. 2**). These bars come in a range of alloy contents and corresponding corrosion resistance, and they exhibit high tensile strengths of 100 and 120 ksi. While a 100-ksi maximum is currently permitted for design under specific conditions, such as in low-seismicity zones, in the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[19] and the American Concrete Institute's *Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary (ACI 318R-19)*,[20] even the 80-ksi design limit can provide

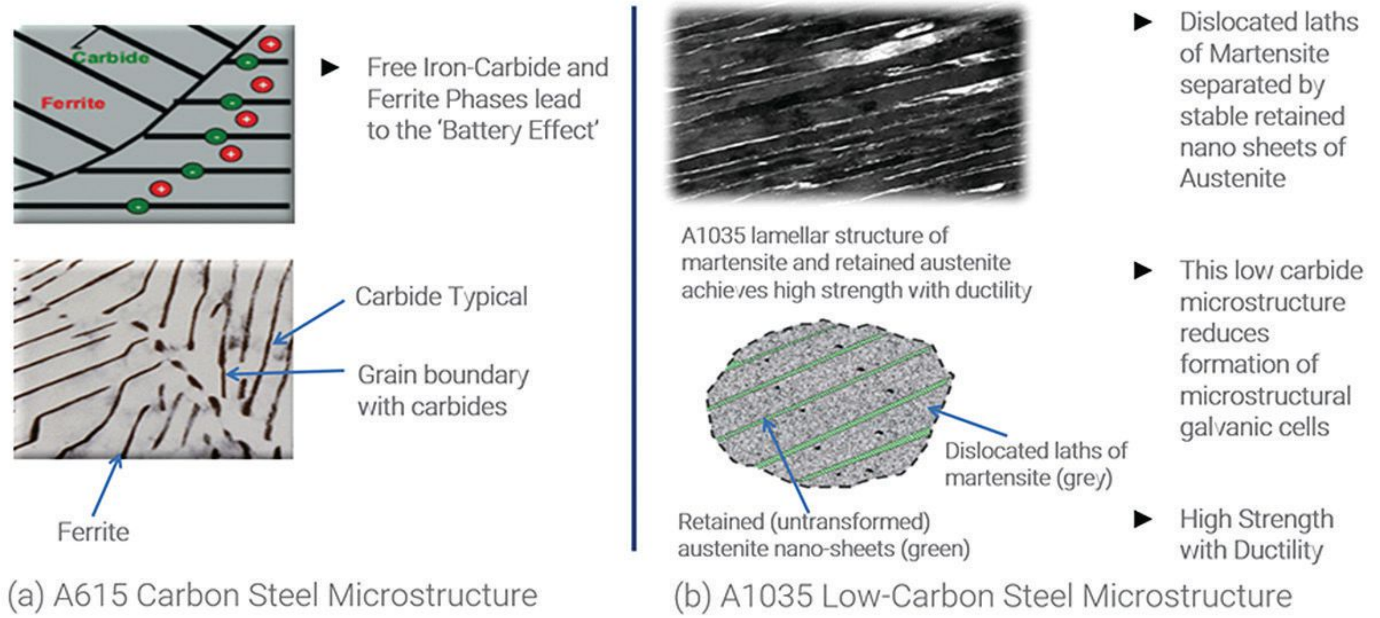


Figure 2. Comparison of steel microstructure for carbon steel (ASTM A615[1]) and low-carbon chromium steel (ASTM A1035[16]). Figure: CMC.

structural efficiency without compromising ductility, assuming that service limit state conditions can still be satisfied.

Perhaps the least-known product type in the blue zone is stainless steel–clad reinforcement. This is a metallic composite material that combines a corrosion-resistant outer layer of high-chromium and nickel stainless steel alloy (such as UNS S31653) with a carbon steel core (equivalent to ASTM A615[1]). This type of reinforcement was introduced to North America in 2001 from the United Kingdom; a newer generation and domestically manufactured version that meets AASHTO M329[18] and U.S. federally funded procurement sourcing restrictions is now available.[21] The corrosion-resistance of stainless steel–clad reinforcement[22] is comparable to that of solid stainless steel (ASTM A955[5]), provided that the metallurgically bonded coating remains unbreeched, whereas the bulk mechanical properties and structural design are comparable to conventional carbon steel reinforcement (**Fig. 3**). Compared with solid stainless steel reinforcement, stainless steel–clad reinforcement is substantially less expensive to produce and has a significantly lower global warming potential (GWP). There is a minor risk of some localized corrosion at the cut bar ends or in isolated spots subjected to extreme impacts during construction, if those areas are not subsequently sealed appropriately. This risk demotes stainless steel–clad reinforcement from a potential violet status to blue; however, when robust quality control is in place at the jobsite, this type of reinforcement may warrant an indigo status.

Production of the current version of stainless steel–clad reinforcement differs significantly from production of the earlier U.K.-based product line for the patented composite steel. Production of the former version involved filling a stainless steel tube with carbon steel (either shavings or a solid billet), sealing the end of the tube, and hot-rolling the composite piece into deformed steel reinforcement. For the current version, a U.S.-based company has developed a novel manufacturing approach using laser deposition to build up a metallurgically bonded stainless steel layer on the outside of a carbon steel billet, thereby creating a composite stainless steel–clad billet, which can then be hot-rolled into deformed stainless steel–clad reinforcement at commercial steel reinforcing bar rolling mills. Initial installations of this stainless steel–clad reinforcement are underway in California and Florida after successful completion of the National Cooperative Highway Research Program Innovations Deserving Exploratory Analysis Project 240.[23] This material is a cost-effective and scalable option, especially for highly aggressive environments resulting from deicing or marine salt exposure,[24] with the advantage of using familiar structural design practice and having ductility equivalent to that of ASTM A615 reinforcement.

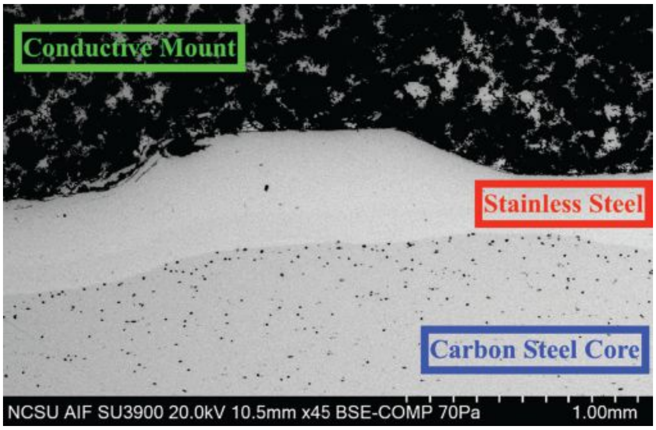
Conclusion

A spectrum of steel reinforcement options is commercially available and standardized in the United States. Given the broad range of options, design professionals are obliged to familiarize themselves with both the benefits and limitations of each product type to best serve their client-owner interests and be good stewards of our concrete infrastructure. In the context of national and international cement and concrete road maps for decarbonization by 2050,[25–28] resilience (durability, robustness, and self-healing), and sustainability (GWP, adaptability, and reuse), design guidance should be refined to balance both economic and environmental costs. Here, the use of corrosion-resistant steel reinforcement along with smart concrete choices may be a pivotal advancement as we all strive to be environmental stewards.

References

1. ASTM International. 2024. *Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement*. ASTM A615/ASTM A615M-24. West Conshohocken, PA: ASTM International.
2. ASTM International. 2024. *Standard Specification for Structural Steel for Bridges*. ASTM A709/A709M-24. West Conshohocken, PA: ASTM International.
3. ASTM International. 2024. *Standard Specification for Low-Relaxation, Seven-Wire Steel Strand for Prestressed Concrete*. ASTM A416/A416M-24. West Conshohocken, PA: ASTM International.

Figure 3. Electron microscope image of a longitudinal cross section of a stainless steel–clad reinforcing bar. The cladding is shown across a surface deformation. Photo: Allium Engineering Inc.



4. ASTM International. 2024. *Standard Specification for Deformed and Plain Low-Alloy Steel Bars for Concrete Reinforcement*. ASTM A706/A706M-24. West Conshohocken, PA: ASTM International.
5. ASTM International. 2020. *Standard Specification for Deformed and Plain Stainless Steel Bars for Concrete Reinforcement*. ASTM A955/A955M-20c. West Conshohocken, PA: ASTM International.
6. ASTM International. 2020. *Standard Specification for Low-Relaxation, Seven-Wire, Grade 240 [1655], Stainless Steel Strand for Prestressed Concrete*. ASTM A1114/A114M-20. West Conshohocken, PA: ASTM International.
7. ASTM International. 2024. *Standard Specification for Titanium Alloy Bars for Near Surface Mounts in Civil Structures*. ASTM B1009-24. West Conshohocken, PA: ASTM International.
8. American Association of State Highway and Transportation Officials (AASHTO). 2020. *Guide for Design and Construction of Near-Surface Mounted Titanium Alloy Bars for Strengthening Concrete Structures*. AASHTO NSMT-1. Washington, DC: AASHTO.
9. ASTM International. 2022. *Standard Specification for Epoxy-Coated Steel Reinforcing Bars*. ASTM A775/A775M-22. West Conshohocken, PA: ASTM International.
10. ASTM International. 2020. *Standard Specification for Filled Epoxy-Coated Seven-Wire Steel Prestressing Strand*. ASTM A882/A882M. West Conshohocken, PA: ASTM International.
11. ASTM International. 2022. *Standard Specification for Epoxy-Coated Prefabricated Steel Reinforcing Bars*. ASTM A934/A934M. West Conshohocken, PA: ASTM International.
12. ASTM International. 2024. *Standard Specification for Textured Epoxy-Coated Steel Reinforcing Bars*. ASTM A1124/A1124M-24. West Conshohocken, PA: ASTM International.
13. ASTM International. 2024. *Standard Specification for Zinc-Coated (Galvanized) Steel Bars for Concrete Reinforcement*. ASTM A767/A767M-24. West Conshohocken, PA: ASTM International.
14. ASTM International. 2020. *Standard Specification for Continuous Hot-Dip Galvanized Steel Bars for Concrete Reinforcement*. ASTM A1094/A1094M-20. West Conshohocken, PA: ASTM International.
15. ASTM International. 2022. *Standard Specification for Zinc and Epoxy Dual-Coated Steel Reinforcing Bars*. ASTM A1055/A1055M-22. West Conshohocken, PA: ASTM International.
16. ASTM International. 2024. *Standard Specification for Deformed and Plain, Low-Carbon, Chromium, Steel Bars for Concrete Reinforcement*. ASTM A1035/A1035M-24. West Conshohocken, PA: ASTM International.
17. AASHTO. 2021. *Standard Specification for Uncoated, Corrosion-Resistant, Deformed and Plain Chromium Alloyed, Billet-Steel Bars for Concrete Reinforcement and Dowels*. AASHTO M334M/M334-17. Washington, DC: AASHTO.
18. AASHTO. 2011. *Stainless Clad Deformed and Plain Round Steel Bars for Concrete Reinforcement*. AASHTO M329M/M329. Washington, DC: AASHTO.
19. AASHTO. 2024. *AASHTO LRFD Bridge Design Specifications*. 10th ed. Washington, DC: AASHTO.
20. American Concrete Institute (ACI). 2019. *Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary (ACI 318R-19).* Farmington Hills, MI: ACI.
21. Office of Management and Budget. 2023. "Buy America Preferences for Infrastructure Projects." 2 CFR Part 184. https://www.ecfr.gov/current/title-2/part-184.
22. Ogunsanya, I. G., V. Strong, and C. M. Hansson. 2020. "Corrosion Behavior of Austenitic 304L and 316LN Stainless Steel Clad Reinforcing Bars in Cracked Concrete." *Materials and Corrosion* 71(7): 1066-1080. https://doi.org/10.1002/maco.201911221.
23. McAlpine, S. 2023. *Stainless Steel Coated Rebar for Chloride Resistant Concrete Highway and Bridges*. Final Report, National Cooperative Highway Research Program Innovations Deserving Exploratory Analysis Project 240. Washington, DC: Transportation Research Board. https://onlinepubs.trb.org/onlinepubs/IDEA/FinalReports/Highway/NCHRP240.pdf.
24. AASHTO. 2020. *Guide Specifications for Service Life Design of Highway Bridges*. AASHTO NSMT-1. Washington, DC: AASHTO.
25. Global Cement and Concrete Association (GCCA). 2021. *Concrete Future: The GCCA 2050 Cement and Concrete Industry Roadmap for Net Zero Concrete*. London, UK: GCCA. https://gccassociation.org/concretefuture/wp-content/uploads/2021/10/GCCA-Concrete-Future-Roadmap-Document-AW.pdf.
26. Portland Cement Association (PCA). 2021. *Roadmap To Carbon Neutrality*. Washington, DC: PCA. https://www.cement.org/wp-content/uploads/2024/05/Roadmap_Jan2024.pdf.
27. Innovation, Science and Economic Development Canada. 2022. *Roadmap to Net-Zero Carbon Concrete by 2050*. Ottawa, ON: Canada Department of Industry. https://ised-isde.canada.ca/site/clean-growth-hub/en/roadmap-net-zero-carbon-concrete-2050.
28. Lorenz, E., M. VanGeem. 2025. "Lessons Learned from Various Embodied Carbon Reduction Strategies for Concrete in the United States." In *4th* fib *International Conference on Concrete Sustainability*, edited by J. A. O. Barros, V. M. C. F. Cunha, H. S. Sousa, J. C. Matos, and J. M. Sena-Cruz.. Cham, Switzerland: Springer. https://doi.org/10.1007/978-3-031-80724-4_18.

Steven Nolan is a senior structures design engineer for the Florida Department of Transportation in Tallahassee. Dr. Salem Faza is the engineering manager at CMC. Dr. Samuel McAlpine is the cofounder and CTO of Allium Engineering Inc.

Reliability-Based Service III Evaluation for Prestressed Concrete Girder Bridges Under Platoon Loads

by Dr. Jay Puckett and Dr. Joshua Steelman, University of Nebraska

This article is the fourth in a series. In the Winter 2024 issue of *ASPIRE®*, we introduced a potential strategy to safely allow truck platoons to increase truck weights based on live-load factor calibration and reliability principles. The premise of this work is that trucks will become more intelligent and will be capable of driving long distances autonomously. With such intelligence, trucks in a platoon can likely report their axle weights and spacings and control their relative headways.

In the Summer 2024 issue of *ASPIRE*, we outlined how reliability indices are used for a simple-span concrete T-beam bridge. Then, in the Winter 2025 issue, we presented the designs (optimal number of strands) for one- and two-span bridges of four span lengths, using six different scenarios involving loss models and Service III limit state live-load factors (0.8, 1.0), with gross or transformed sections. Limited data were presented in that article; however, complete results are provided elsewhere.[1–3] These designs provided the basis for our reliability analysis.

In the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*,[4] Service III is not formally calibrated, and load and resistance factors are based on judgment and experience. A goal of our research was to determine the implied index $\beta_{implicit}$ for current Service III designs, which was a significant task. We would then use $\beta_{implicit}$ as a benchmark for calibrating platoon permit load factors. Once estimated, our calibration task was relatively straightforward, and we can now determine the load factors for permitting these loads.

In this article, we build on the previous discussion and address the determination of $\beta_{implicit}$ and the probability of cracking during a bridge's service life. The analysis assumptions are the same as would be used for design (that is, 75-year design life, HL-93 loading, and AASHTO load factors). Refer to our article in the Winter 2025 issue of *ASPIRE* for definitions of variables and discussion of the reliability of different design options.

Probabilities of Exceeding Tension Limits

Monte Carlo simulation (MCS) was implemented to determine dead load, live load, and resistance according to distributions based on nominal values and statistical parameters.[1–3] As one example of optimal design using the AASHTO LRFD specifications, **Fig. 1** presents probability density functions (PDFs) for Service III with tensile stress limited to $f_t = 0.0948\sqrt{f'_c}$ (κ_{eva} = 0.0948) for a 120-ft simple-span bridge designed using AASHTO LRFD specifications published after 2005 (*Post*), γ_L = 1.0 with elastic gains from live load included in the precompression stress (*Post-1.0-Gains*).

The mean resistance is close to the nominal resistance for evaluation in Fig. 1. However, the mean load is greater than the mean resistance.

Figure 1. Probability density function for evaluating the Service III limit state at f_t (κ = 0.0948) for 120-ft simple-span bridges designed by using *Post-1.0-Gains*. All Figures: University of Nebraska.[1,2]

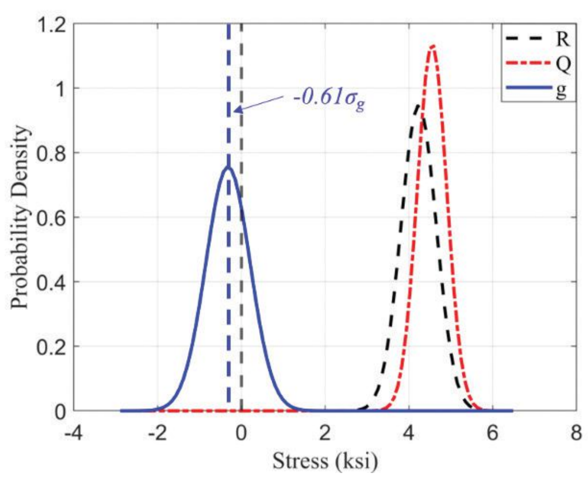
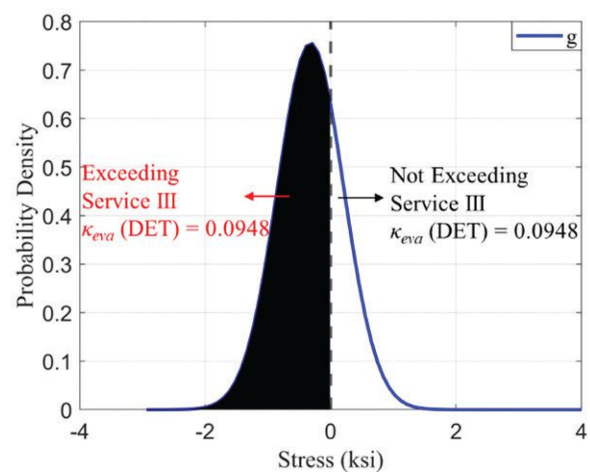


Figure 2. Probability density function showing the probability that Service III f_t (κ_{eva} = 0.0948) will be exceeded during the service life for 120-ft simple-span bridges designed by using *Post-1.0-Gains*.



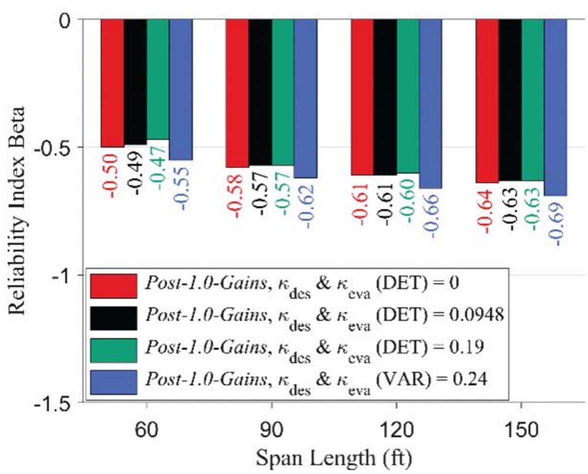


Figure 3. Reliability index β of exceeding tension limits for simple-span bridges designed using the *Post-1.0-Gains* method.

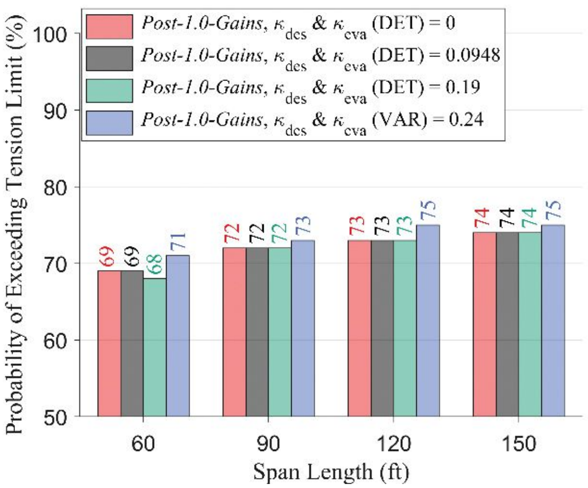


Figure 4. Probabilities of exceeding the tension limit for simple-span bridges designed using the *Post-1.0-Gains* method.

The $\beta_{implicit}$, representing the number of standard deviations between the mean for the limit state function, g = *resistance* – *load effect* = $R - Q$, and the failure threshold, is $\beta_{implicit} = -0.61$. A negative value indicates that the probability of exceeding the Service III limit state is more than 50%. **Figure 2** further examines the PDF for g. Figure 2 depicts the probability that Service III f_t (κ_{eva} = 0.0948) will be exceeded during the service life. The figure shows about 72% of the 120-ft simple-span bridges designed with f_t (κ_{eva} = 0.0948) are expected to exceed the Service III limit state during their service lives, even though they are optimally proportioned to satisfy Service III design criteria. Note that the computed negative values are a "head-spinner" for reliability experts and were not accepted without careful verification.

Figures 3 and **4** expand the data to include all span lengths and reasonably represent much more extensive data.[3] Bar colors indicate design and reliability analysis assumptions. "DET" indicates the use of a deterministic value for f_r, and "VAR" indicates the use of statistical properties. Observations drawn from Fig. 3 and Fig. 4 include the following:

- $\beta_{implicit}$ was consistently about –0.60 for bridges designed using *Post-1.0-Gains*, was not prominently sensitive to the tension stress limit f_t (κ), and was slightly sensitive to span length. This consistency is expected, as the permitted tensile stresses are the same for both the optimal design and the reliability analysis.
- Figure 4 presents corresponding probabilities of exceeding tension limits. Probabilities are greater than 50% for all cases. The average probability is approximately 73%.
- Considering f_t (κ = 0.0948), the f_{eval} of *Post-1.0-Gains* in Fig. 4 can be used as a reference case because it provided consistent reliability indices regarding span lengths and f_t.

Cracking Probability During Service Life

A negative $\beta_{implicit}$ implies that optimally designed bridges for the Service III limit state are more than 50% likely to violate tensile stress limits under current design live loads at some point during their 75-year service lives. However, exceeding the service limit does not necessarily mean they experience flexural cracking in the precompressed tensile zone. **Figure 5** presents graphs for the likelihood of cracking at modulus of rupture f_r (κ_{eva} = 0.24) for a 120-ft simple-span bridge designed using *Post-1.0-Gains*. This 120-ft simple-span bridge is assumed to be designed with f_t (κ = 0.0948).

Figure 5 shows that the mean resistance is slightly larger than the mean load—the corresponding cracking reliability index is +0.08. The total area under the blue "g" line in **Fig. 6** represents all possible outcomes, and the shaded area represents the proportion of cracking cases. The shaded area in Fig. 6 is smaller than that in Fig. 2, reflecting that cracking is less likely than exceeding a limit state set to a tension stress limit less than the theoretical concrete cracking strength. Figure 6 indicates that about 47% of the 120-ft simple-span bridges are expected to crack during their 75-year service lives.

To further evaluate $\beta_{cracking}$ in bridges, bridges designed using a typical f_t (κ = 0.0948) were evaluated over a range of f_r (κ). The moduli of rupture were assumed equal to $0.24\sqrt{f_c'}$, $0.30\sqrt{f_c'}$, and $0.37\sqrt{f_c'}$ to investigate how cracking probability varied. The maximum value considered here is approximately the same as the upper-bound concrete cracking stress specified for minimum reinforcement in Article 5.6.3.3 of the AASHTO LRFD specifications.

Figure 7 presents the cracking reliability index and corresponding cracking probability for the *Post-1.0-Gains*, *Post-0.8-No-gains*, and *Approx.-0.8-No-gains* methods. These results are typical of present practice. The figure provides results for the typical modulus of rupture $0.24\sqrt{f_c'}$ and the additional cases for moduli of rupture of $0.30\sqrt{f_c'}$ and $0.37\sqrt{f_c'}$. The cracking probability graphs show that the higher, and perhaps more likely, f_t decreases the probability of cracking significantly, perhaps to a level more closely matching in-service observations and inspections. However, it is important to keep in mind that the higher level of cracking stress in Article 5.6.3.3 of the AASHTO LRFD specifications reflects the upper tail of a concrete cracking strength probability distribution, so using this value as the mean concrete cracking strength is not strictly consistent with test data. On the other hand, using the higher value can be considered to approximately represent increasing concrete strength over time.

Summary of Key Points

We observed the following key points from our reliability study. Note that only limited data are provided here. Please see Steelman et al.[1,2] for expanded results.

- $\beta_{implicit}$ values of −0.60 and −1.20 imply a 73% to 88% probability of exceeding the Service III limit during service life.
- $\beta_{implicit}$ was approximately −1.20 when averaged across all considered span lengths for bridges designed using the following loss methods:
  - Post-2005 loss method with elastic gains and using γ_L = 0.8
  - Approximate loss method with elastic gains and using γ_L = 0.8
- A target of $\beta_{implicit}$ = −0.60 is recommended to evaluate girder bridges for platoon loading at Service III. A more liberal value of −1.20 may be acceptable if it is supported by observations of adequate performance in service for girders designed considering elastic gains and using γ_L = 0.8.
- Bridge cracking probabilities range between 10% and 51% when bridges are designed using the code-specified service load, indicating that optimally designed bridges may crack during their service life, despite the nominal expectation that cracking will be avoided by using tensile stress limits less than the modulus of rupture. A nominal change of f_r (κ = 0.24) to f_r (κ = 0.37) produces an average increase of β equal to 0.64 and an average decrease in cracking probability of 22%. $\beta_{Cracking}$ and cracking probability change approximately linearly as a function of nominal moduli of rupture and can be preliminarily estimated using an assumed value.
- Bridges designed using the Pre-2005 loss method (Article 9.16.2.1 of the *AASHTO Standard Specifications for Highway Bridges*[5]) without elastic gains and using γ_L = 1.0 possessed the highest cracking reliabilities, which indicates that heavy platoon loads pose lower risks to these bridges than to other bridges designed using more-recent prestress loss estimation methods.

Negative Reliability?

The Service III limit state has not been formally calibrated and is based on decades of experience. Strength limit states have much higher reliability indices and lower exceedance probabilities. However, an open question remains about the likelihood of exceeding the service limits using the same rules as the governing design, which we determined is likely. We also determined that the probability that the girders would crack during the service life is lower but also likely.

Although it may seem odd at first, negative reliability is just a number that predicts a likelihood of occurrence. Over 75 years, concrete may crack, pretension closes the cracks, and reasonable performance is likely observed. So, how often can this cycle occur without adverse effects if we permit platoons (or any other heavier loads)? This is an open question. Another item for discussion is whether evaluation load levels should be based on shorter return periods, such as two years, or another period based on the inspection frequency, actual weigh-in-motion data, long-term strength increases (bias), and other factors. Using such assumptions will lower the probability of exceeding the limit state or modulus of rupture.

Takeaway

The probability that a prestressed concrete girder will crack during its service life seems to be relatively high; however, little work that explores the repeated load effect between cracking and yielding of reinforcement has been done. A better understanding of the effect of heavier loading due to possible platoon permits brings us back to the moment-curvature relationship and repeated loading presented in the first article in this series (*ASPIRE* Winter 2024).

Service III typically controls the required number of strands in a design. The findings explored in this series raise questions about whether it is time to rethink the Service III design process related to concrete cracking, and/or a new limit state related to a percentage of strand yielding.

Figure 5. Probability density functions for evaluating cracking at modulus of rupture f_r (κ = 0.24) for 120-ft simple-span bridges designed using the *Post-1.0-Gains* method.

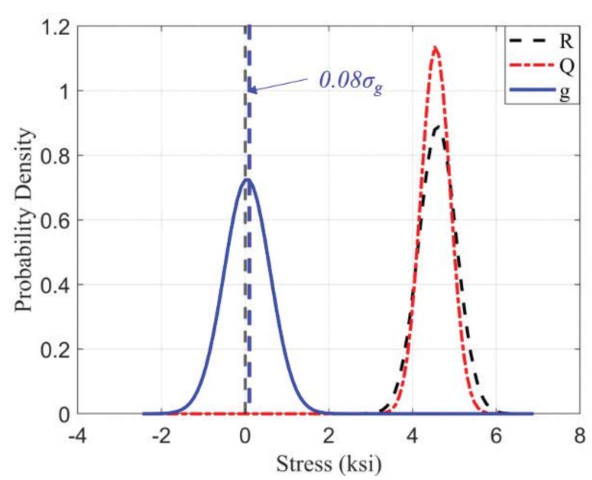
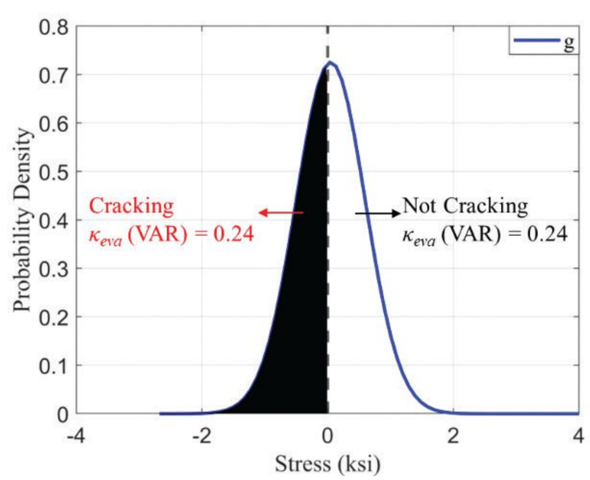


Figure 6. Probability density function for the likelihood of cracking at modulus of rupture f_r (κ = 0.24) for a 120-ft simple-span bridge designed using *Post-1.0-Gains*, representing all possible outcomes. The shaded area represents the proportion of cracking cases.



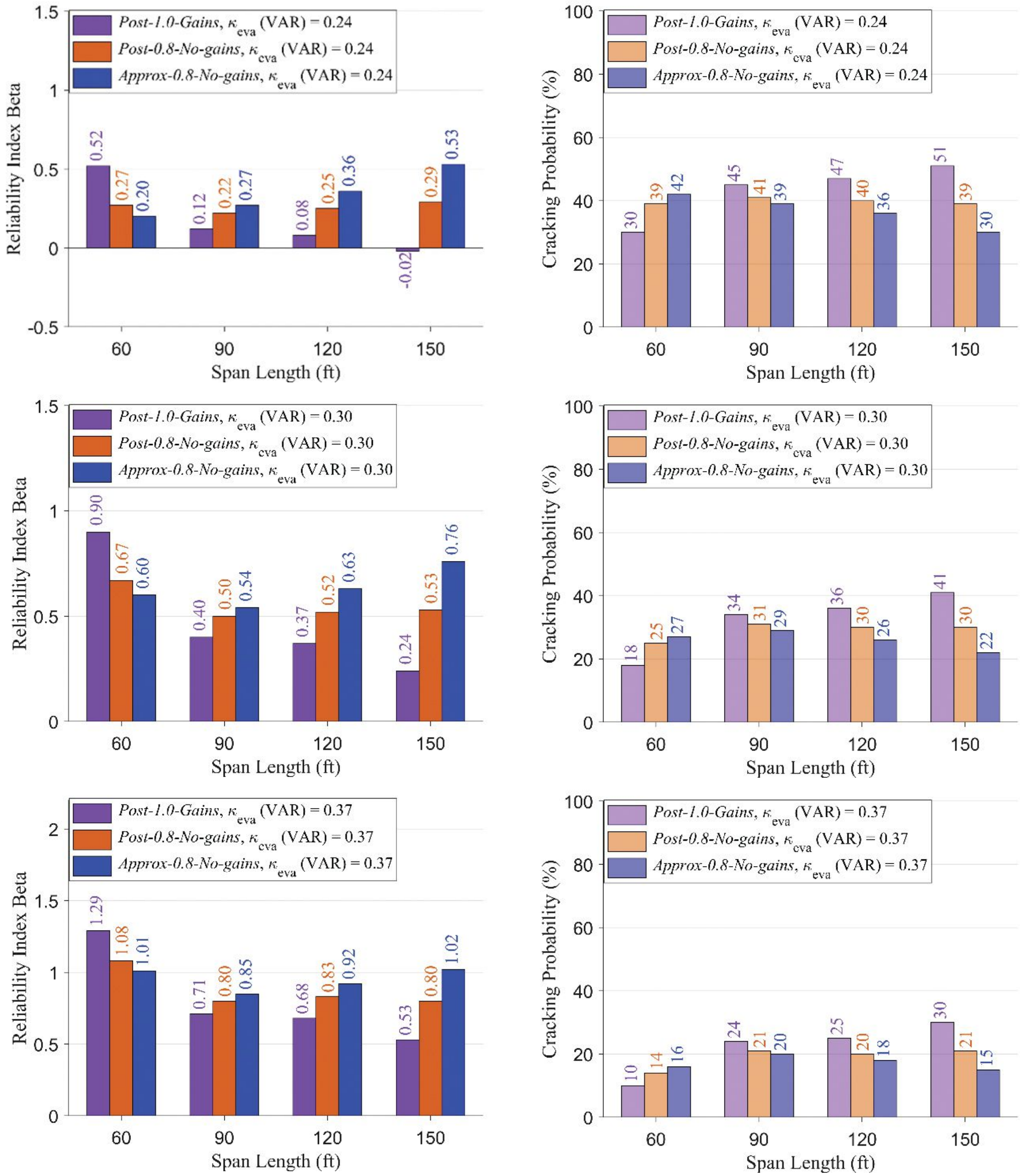


Figure 7. Reliability index $\beta_{cracking}$ and cracking probabilities for simple-span bridges designed using *Post-1.0-Gains*, *Post-0.8-No-gains*, and *Approx.-0.8-No-gains* methods and different values for f_r (κ = 0.24, 0.30, and 0.37).

References

1. Steelman, J. S., J. A. Puckett, D. G. Linzell, and B. Yang. 2021. *Truck Platooning Effects on Girder Bridges*. Final report no. SPR-1(20) M030. Lincoln: Nebraska Department of Transportation.
2. Steelman, J. S., J. A. Puckett, D. G. Linzell, and B. Yang. 2023. *Truck Platooning Effects on Girder Bridges*: Phase II. Final report no. SPR-FY22(011). Lincoln: Nebraska Department of Transportation.
3. Yang, B., J. S. Steelman, J. A. Puckett, and D. G. Linzell. 2024. "Reliability-Based Service III Evaluation for Prestressed Girder Bridges Under Platoon Loads" *Transportation Research Record*. 2678 (7): 515–536. https://doi.org/10.1177/03611981231208200.
4. American Association of State Highway and Transportation Officials (AASHTO). 2020. *AASHTO LRFD Bridge Design Specifications*. 9th ed. Washington, DC: AASHTO.
5. AASHTO. 2002. *AASHTO Standard Specifications for Highway Bridges*. 17th ed. Washington, DC: AASHTO.

STATE

Washington

The Washington State Department of Transportation addresses unique challenges through the use of innovative design, materials, and programs

by Mathew Rochon, William J. Miller III, Anthony Mizumori, Richard Brice, and Amy C. Leland, Washington State Department of Transportation

Known for its rugged Pacific coastline and the Cascade Mountains, Washington state is a study in contrasts when it comes to climate and geography. The state has the largest ferry operation in the United States as well as 8421 bridges carrying public roads that are subject to the Federal Highway Administration inspection requirements; 3345 of these bridges are state owned.[1]

WSDOT employs innovative programs and a creative workforce. The agency pilots new materials and designs to help advance efforts to create and maintain a transportation system that meets safety, reliability, resiliency, accessibility, and affordability requirements.

Innovative Programs

WSDOT completes roughly half of the state's design work in-house. From this work, the agency has acquired unique insight regarding the complexities of bridge design, construction, and asset management. They understand aspects of construction and material availability. The agency considers not just the current design and construction stages but also the longevity of the structure, and how it will serve the community for years to come.

WSDOT, in partnership with the Texas Department of Transportation, created PGSuper (www.wsdot.wa.gov/eesc/bridge/software), which is open-source software to design and load rate precast, prestressed concrete girder bridges in accordance with the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[2] and WSDOT criteria. The agencies also developed PGSplice, an open-source program for the design of precast, post-tensioned concrete spliced-girder bridges. Girders are evaluated for stress and stability during handling and transportation as well as strength limit state requirements. WSDOT maintains both programs to accommodate the state's particular challenges, including strategies to mitigate issues related to top flange fit-up (differential camber), camber predictions, and stability.

The colors and textures on the East Trent Bridge over the Spokane River reflect community input about the structure's aesthetics. On every bridge project, the Washington State Department strives to meet the needs of the community and fulfill a "visual level of service." All Photos: Washington State Department of Transportation.

Lightweight Concrete to Extend Spans

Washington state is home to some of the longest precast, prestressed concrete girders in the United States. WSDOT experts are actively involved in the development of guidelines that address the design of bridge girders for lateral stability during handling and transportation. WSDOT has implemented practices that are in the latest edition of the AASHTO LRFD specifications, as well as those that are being included in the forthcoming second edition of the Precast/Prestressed Concrete Institute's (PCI's) *Recommended Practice for Lateral Stability of Precast, Prestressed Concrete Bridge Girders*.[3]

The successful use of mega-girders in Washington is facilitated by the use of lightweight concrete. The record-breaking, limit-pushing girders were made possible by lightweight aggregates. Now WSDOT frequently uses lightweight concrete in its bridge designs to extend span length. Limiting the weight of the girders is critical to ensure components can be hauled to the jobsite by truck. Because western Washington is a high-seismic region, lightweight concrete also helps reduce the bridge mass and substructure earthquake loads.

Puyallup River Bridge

One recent project over the Puyallup River involved the state's longest prestressed concrete girders (223 ft long). These massive girders support southbound Interstate-5 (I-5) traffic in Tacoma over the Puyallup River and adjacent rail lines. To meet the handling and shipping requirements, lightweight concrete was used for a modified WSDOT WF100G girder section with a widened top flange to improve stability. (For

The longest prestressed concrete girders in Washington state are installed on southbound Interstate 5 Puyallup River Bridge in Tacoma. These girders measure 223 ft in length and almost 9 ft in height, and each weighs more than 246,500 lb.

For the North Spokane Corridor project, local artists helped identify community-based themes. In the Euclid Undercrossing, images evoking native culture and natural elements from the surrounding community are embedded in the concrete abutment.

more information on this project, see the Fall 2019 issue of *ASPIRE*®.)

The Puyallup River Bridge is part of the larger Tacoma/Pierce County corridor program, which is adding high-occupancy vehicle lanes and other operational improvements along I-5, State Route 16, and State Route 167 (SR 167). The second stage of construction, on the SR 167/I-5 to the SR 509 New Expressway phase of the project, began in the summer of 2022. That project also includes structures that use both lightweight concrete and long-span girders. WSDOT coordinates with local precast concrete manufacturers and haulers to determine the logistics of transporting these large components to the site and installing them. Route investigation for hauling typically includes checks on horizontal clearances during turning movements, vertical clearances to overhead structures and utilities, and rotational stiffness requirements for the hauling equipment.

Galvanized Reinforcement in a Coastal Environment

For the crossing of U.S. Route 101 over Steamboat Creek, which is in a coastal environment, WSDOT used galvanized reinforcement in the precast, prestressed concrete spliced girders, and the concrete deck and barriers. This pilot project in Jefferson County near the Pacific Coast used post-tensioned, spliced prestressed concrete girders. This was the first WSDOT project to use plastic ducts in precast, prestressed concrete girders as standard practice. The ducts, which are made from high-density polyethylene, are intended to protect tendons from corrosion and provide adequate space for post-tensioning tendons and hardware. The post-tensioning ducts were grouted with prepackaged Class C cementitious grout, which is WSDOT's standard practice.

In addition to the galvanized reinforcement throughout the bridge, WSDOT included other corrosion-protection measures to withstand the marine exposure. WSDOT recently adopted the requirements of the Post-Tensioning Institute's PTI Performance Level 2 for post-tensioned bridges to enhance the service life of these structures. Based on the success of this galvanized reinforcing bar pilot project, the option will be considered for other projects, such as the U.S. Route 395 North Spokane Corridor.

Precamber Geometry

Prestressed concrete girder bridge systems continue to evolve. One advancement is the use of precambered girders. Building an intentional vertical curve (precamber) into girders is an effective method to accommodate challenges with vertical alignment and bridge geometry. However, precamber has design and fabrication challenges that must be addressed. WSDOT engineers, working with local fabricators, documented these challenges and a solution in the May–June 2020 issue of *PCI Journal*.[4] Several bridges have been constructed with precambered girders for vertical clearance. This method can also be used when there is a potential for sag in long-span girders.

Modest spans with steep vertical curves lend themselves to the use of precambered precast concrete girders. The practice reduces the concrete placed for the deck and improves aesthetics. It is also helpful when there is a vertical clearance requirement underneath the structure. One project example is Thorne Lane bridge over I-5 in Lakewood, Wash., where precambered girders were used to meet

To improve stability during handling, hauling, and erection, the top flange of a lightweight concrete girder for the Puyallup River Bridge was widened to 5 ft 1 in. and ten 0.6-in.-diameter pretensioned temporary top strands were placed in the top flange.

A culvert under State Route 9 through Gribble Creek in Skagit County is replaced with a new precast concrete culvert as part of the statewide fish passage imperative. This project required only a five-day road closure.

The replacement of the existing bridge deck on the Interstate 90 Vantage Bridge aims to maintain the structural integrity and extend the life of the bridge. While allowing traffic on adjacent lanes, construction is currently underway using precast concrete bridge deck panels and ultra-high-performance concrete in the longitudinal and transverse keyway joints.

The crossing of U.S. Route 101 over Steamboat Creek used spliced, post-tensioned precast concrete girders and was the first Washington State Department of Transportation project to use plastic ducts in prestressed concrete girders. The high-density polyethylene ducts are intended to protect tendons from corrosion. Shown here is a closure joint before the duct couplers, additional reinforcement, and post-tensioning are installed and concrete cast.

the vertical clearance requirements over the interstate and adjacent Sound Transit railroad.

The Importance of Aesthetics

Communities play an important role in determining bridge aesthetics. People expect bridges to provide durable, resilient, safe, and economical transportation while also reflecting the communities they serve. WSDOT has a dedicated state bridge and structures architect on staff with a team that provides oversight and guidance for bridge aesthetics that suit the state's diversity.

Color palette and formliner technology are used to imbue concrete with different colors and textures. In Spokane, the SR 290 East Trent Bridge over the Spokane River is one example of striving for a harmonious and balanced solution. Context is critical—a concrete finish suitable for Seattle may be out of place in Spokane. Many state highways incorporate tribal arts into the aesthetics of supporting structures and abutments.

The I-405 corridor team developed guidelines for aesthetics to define their image across current and future projects. They use four color palettes as well as distinct column shapes and custom concrete finishes to develop their own corridor identity.

The new North Spokane Corridor connects north and south areas of Spokane. The corridor encompasses several locales with unique community identities, and it has both isolated interchanges and elevated structures contiguous to neighborhoods, business districts, and the Spokane Community College. To create a unifying aesthetic for the project, WSDOT asked local artists to assist the agency in identifying community-based themes for the structure. Images based on those themes were then cast into the concrete.

Complete Streets

Community engagement is important for all projects, regardless of their size. Since 2022, the "Complete Streets" requirement has directed WSDOT to provide access for multimodal users, including pedestrians, bicyclists, and public transportation users. For the many Washingtonians who do not drive, WSDOT incorporates input from landscape architects and urban planners to make sure that structures are contextually relevant and fit into the surrounding landscape.

Ultra-High-Performance and High-Early-Strength Concrete

WSDOT uses ultra-high-performance concrete (UHPC) primarily to fill joints between precast concrete components. On the San Poil River and Swauk Creek bridge deck girder projects, the longitudinal sides of the top flanges were joined with UHPC. Also on these projects, two mats of no. 5 reinforcing bars at 6-in. spacing were extended from each flange and joined with a noncontact lap splice.

The Coastal 29 progressive design-build project is the first WSDOT project to use UHPC to connect adjacent slab-girder bridges. Those adjacent slabs are connected with a single mat of reinforcement with a noncontact lap splice in the joint. For the deck replacement of the I-90 Vantage Bridge over the Columbia River, WSDOT is using UHPC to connect precast concrete deck panels while allowing traffic to remain on adjacent lanes of the bridge during construction.

The PGSuper software was recently updated to support the design of UHPC pretensioned bridge girders for the recently published *AASHTO Guide Specifications for Structural Design with Ultra-High-Performance Concrete*.[5]

High-early-strength concrete (HESC) is being researched by WSDOT for rapid bridge deck overlay preservation. Bridge engineers are investigating the use of calcium sulfoaluminate (CSA) cement as a permanent bridge overlay material to speed up construction time. Researchers at the University of Washington are evaluating prepackaged, manufactured products for the potential use of HESC for long-term durability. While these products are used by WSDOT highway maintenance crews for fast repair and short-term fixes, they have not been used for large repair projects. CSA cement shows the potential to help crews work within tight construction windows and reduce the greenhouse gases associated with portland cement.

Fish Passage Imperative

Washington has 4037 highway crossings on fish-bearing waters. Of those, 2074 are documented fish passage barriers, with 1531

For the replacement of the San Poil bridge, the Washington State Department of Transportation used wide-flange deck girders with a thicker than standard top flange. The prestressed concrete girders eliminated the need for deck formwork. Ultra-high-performance concrete was used in the top-flange connections between adjacent WF39DG girders.

culverts blocking upstream habitat. Over the last several years, WSDOT has accelerated work to comply with the requirements of a U.S. District Court injunction to correct impediments to fish migration. To date, WSDOT has completed nearly 500 fish passage barrier corrections, restoring more than 570 miles of potential salmon and steelhead habitat. WSDOT is using several strategies to facilitate the effort to accomplish this daunting task by 2030. Specifically, the agency is bundling fish passage projects by geographic regions; using progressive design-build contracts to expedite delivery; and creating standard details and drawings for these types of projects.

The fish passage program requires the replacement of hundreds of structures. Precast concrete and accelerated bridge construction methods offer economical and efficient options to quickly replace the structures that block fish migration. Given the urgent need for standard plans, WSDOT sought input from local precast concrete fabricators regarding constructability. The standard plans include buried structure split boxes, three-sided buried structures, and precast reinforced concrete retaining walls. Implementation of the new standards generated policy changes regarding hydraulic design, load distribution, seismic design, and traffic control. Ultimately the standard drawings will streamline the design and construction process and improve efficiencies. (See the Spring 2022 issue of *ASPIRE* for a Concrete Bridge Technology article about WSDOT's precast concrete buried structures for fish passage.)

WSDOT is tasked with the critical mission of maintaining infrastructure, restoring habitat, connecting communities, and saving lives. It achieves that goal through the use of innovative materials, design, research, and programs.

Artistic touches to the concrete substructure were added to the skyway portion of North-South Corridor (U.S. Route 395) near the Spokane Community College and Spokane River structures.

References

1. Federal Highway Administration, Washington Division. 2024 "Bridge Program." Accessed October 31, 2024. https://www.fhwa.dot.gov/wadiv/programs/bridge/inspect.cfm.
2. American Association of State Highway and Transportation Officials (AASHTO). 2020. *AASHTO LRFD Bridge Design Specifications*. 9th ed. Washington, DC: AASHTO.
3. Precast/Prestressed Concrete Institute (PCI). Forthcoming. *Recommended Practice for Lateral Stability of Precast, Prestressed Concrete Bridge Girders*. 2nd ed. CB-02. Chicago, IL: PCI.
4. Brice, R., S. J. Seguirant, A. Mizumori, and B. Khaleghi. 2020. "Fabrication and Design of Precambered Precast, Prestressed Concrete Bridge Girders." *PCI Journal* 65 (3): 64–77 https://doi.org/10.15554/pcij65.3-02.
5. AASHTO. 2024. *Guide Specifications for Structural Design with Ultra-High-Performance Concrete*. Washington, DC: AASHTO.

Mathew Rochon is an architect and engineer for the Washington State Department of Transportation (WSDOT) Bridge & Structures Architectural Services, Olympia, Washington. William J. Miller III is a WSDOT bridge design engineer. Anthony Mizumori is a WSDOT bridge engineer. Richard Brice is the WSDOT Bridge Design Technology Unit manager. Amy C. Leland is the WSDOT State Bridge Design Engineer.

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://asbi-assoc.org/wp-content/uploads/2023/07/Box-Girder-Segments-PCIASBIEnglish.pdf
The Perspective on page 6 highlights efficiency in concrete segmental box-girder design and fabrication. This is a link to the *AASHTO-PCI-ASBI Segmental Box Girder Standards for Span-by-Span and Balanced Cantilever Construction*. The use of standardized shapes could open the door for precast concrete segmental superstructures to be an economical option for more projects.

https://rosap.ntl.bts.gov/view/dot/68881/dot_68881_DS1.pdf
This is a link to download the University of Nebraska–Lincoln report *Truck Platooning Effects on Girder Bridges: Phase II – Service*. The Professor's Perspective on page 38 is the fourth article in a series dedicated to investigating the statistical reliability indices that should be used to accommodate truck platoons.

https://www.pci.org/PCI/PCI/Project_Resources/Project_Profile/Project_Profile_Details.aspx?ID=23546
The project to widen River Road over Harrods Creek Bridge, which is mentioned in the Focus article about Palmer Engineering on page 14, won the PCI Design Award for best rehabilitated bridge in 2011. Palmer Engineering served as specialty engineer for this challenging project to widen an existing one-lane bridge to two lanes while preserving the historic character of the original span across Harrods Creek. This link includes the PCI Design Award profile of the project, with additional photos and an overview video about the bridge.

https://wsdot.wa.gov/construction-planning/major-projects/north-spokane-corridor
https://storymaps.arcgis.com/collections/6e4dffc6e5d34705b34a1eadb325403a
These links provide background information about the North Spokane Corridor (NSC) project, as well as photos and maps of the project. The U.S. Route 395 Spokane River crossing bridge is part of the larger NSC project and is featured in the Project article on page 18. Community engagement and corridor theming were priorities for the Washington State Department of Transportation throughout the project.

https://www.youtube.com/watch?v=uWxwBzTEgEA
A video showing the construction progress of the Mount Vernon Viaduct replacement is available at this link from the San Bernardino County Transportation Authority. The new structure, which spans over 3rd Street, four Metrolink regional rail lines, and 18 Burlington Northern Santa Fe railroad lines, is featured in the Project article on page 23. The project team chose precast, prestressed concrete girders for this project based on their efficiency and an outstanding span-to-depth ratio.

https://fdotwww.blob.core.windows.net/sitefinity/docs/default-source/research/reports/fdot-bdv29-977-45-rpt.pdf?sfvrsn=e8b04e2f_2
Information about research on the magnetic flux leakage method for damage detection in post-tensioning tendons can be found at this link. The Perspective article on page 8 discusses how artificial intelligence and machine learning can help engineers analyze data and locate damage using the output from this method.

https://www.fhwa.dot.gov/resourcecenter/teams/structures-geotechnical-hydraulics/Structural_Design_UHPC_Workshop_Manual.pdf
The Federal Highway Administration's (FHWA's) *Structural Design with UHPC Workshop Manual* is available at this link. The FHWA article on page 50 focuses on structural design using ultra-high-performance concrete (UHPC).

https://nationalconcretebridge.org/seminars
The Perspective on page 10 discusses the ethical implications of the normalization of deviance. The article is derived from ethics lessons shared by the author in the National Concrete Bridge Council's (NCBC's) Concrete Bridge Seminar: Concepts for Extending Spans. This is a link to NCBC's upcoming seminars.

Design for Torsional Effects

by Dr. Oguzhan Bayrak, University of Texas at Austin

Complex geometry, as well as complex loading conditions that we see in our bread-and-butter bridges, can result in torsional moments (or torques) being applied to concrete components. One of the topics that generates questions from our bridge design community involves requirements in the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[1] that apply to torsional designs. This article is devoted to providing a succinct discussion of this topic.

Let us begin our discussion by considering under what circumstances the AASHTO LRFD specifications deems torsional moments to be negligibly small. According to Eq. (5.7.2.1-3) in those specifications, if the factored torsional moment T_u is less than 25% of the factored torsional cracking moment $0.25\phi T_{cr}$, torsional effects do not need to be investigated. In such circumstances, torsional effects cause only a very small reduction in shear capacity or flexural capacity. More specifically, the following requirements of the AASHTO LRFD specifications can be used as a starting point to assess whether we need to consider torsion in structural designs:

Torsional effects shall be investigated where:

$$T_u > 0.25\phi T_{cr} \qquad (5.7.2.1\text{-}3)$$

For solid shapes:

$$T_{cr} = 0.126K\lambda\sqrt{f'_c}\frac{A_{cp}^2}{p_c} \qquad (5.7.2.1\text{-}4)$$

For hollow shapes:

$$T_{cr} = 0.126K\lambda\sqrt{f'_c}\,2A_o b_e \qquad (5.7.2.1\text{-}5)$$

in which:

$$K = \sqrt{1 + \frac{f_{pc}}{0.126\lambda\sqrt{f'_c}}} \leq 2.0 \qquad (5.7.2.1\text{-}6)$$

where:

T_u = *applied factored torsional moment (kip-in.)*
T_{cr} = *torsional cracking moment (kip-in.)*
A_o = *area enclosed by the shear flow path, including any area of holes therein (in.^{2})*
A_{cp} = *area enclosed by outside perimeter of concrete cross-section (in.^{2})*
p_c = *length of outside perimeter of the concrete section (in.)*
f'_c = *compressive strength of concrete for use in design (ksi)*
f_{pc} = *unfactored compressive stress in concrete after prestress losses have occurred either at the centroid of the cross-section resisting transient loads or at the junction of the web and flange where the centroid lies in the flange (ksi)*
b_e = *effective width of the shear flow path taken as the minimum thickness of the exterior webs or flanges comprising the closed box section (in.). b_e shall be adjusted to account for the presence of ducts.*
ϕ = *resistance factor specified in Article 5.5.4.2*
λ = *concrete density modification factor as specified in Article 5.4.2.8*

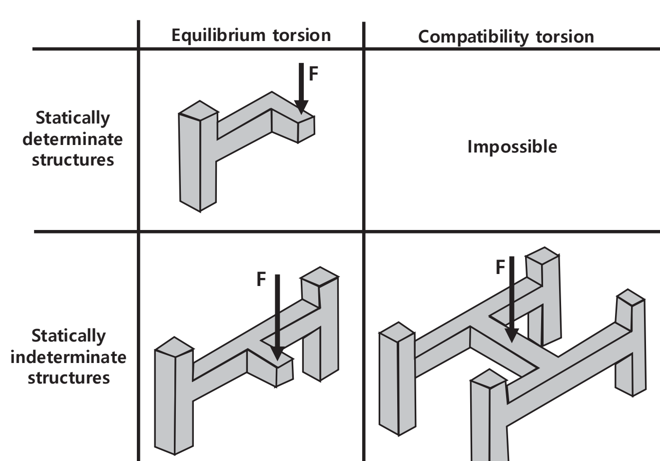
b_e defined above shall not exceed A_{cp}/p_c, unless a more refined analysis is utilized to determine a larger value.

The effects of any openings or ducts in members shall be considered. K shall not be taken greater than 1.0 for any section where the stress in the extreme tension fiber, calculated on the basis of gross section properties, due to factored load and effective prestress force after losses exceed $0.19\lambda\sqrt{f'_c}$ in tension.

When calculating K for a section subject to factored axial force, N_u, f_{pc} shall be replaced with $f_{pc} - N_u/A_g$. N_u shall be taken as a positive value when the axial force is tensile and as a negative value when it is compressive.

For cases in which torsional effects are significant, the next step in design involves a thorough understanding of equilibrium and compatibility torsion. This understanding has design implications, as will be discussed in the later sections of this article. In statically determinate structures, consideration of torsional effects becomes particularly significant. To study equilibrium torsion, let us consider, for example, a hammerhead pier supporting spans of different lengths up station and down station. The beam reactions applied on the hammerhead pier cap could differ significantly and subject the cap to torsional effects, such as shown in the upper left corner of **Fig. 1**.

Figure 1. Illustrations of equilibrium and compatibility torsions. Source: *AASHTO LRFD Bridge Design Specifications* Fig. C5.7.2.1-1.[1]



If the applied torque exceeds the threshold torque $0.25\phi T_{cr}$, designing the cap for torsional effects would be mandatory. Importantly, the factored torsional resistance shall be greater than or equal to the factored torsional moment acting on the member (that is, $\phi T_n \geq T_u$). At the strength limit state, failing to provide sufficient torsional strength would result in torsional cracking, and the torsional cracks would establish a torsional hinge and allow the cantilever portion of the cap to rotate freely. The plastic mechanism that forms in this example would lead to structural collapse.

In a statically indeterminate case, we may or may not be dealing with equilibrium torsion. For the case shown at the bottom left corner of Fig. 1, formation of torsional hinges at the faces of the column would lead to a plastic mechanism and total loss of equilibrium (that is, collapse). In this case, factored torsional resistance must be greater than or equal to the factored torque, like the previous case we discussed. The details of those calculations are discussed in upcoming sections of this article.

Let us next examine the compatibility torsion case that is shown as the bottom right corner of Fig. 1. Compatibility torsion is the type of torsion that is rooted in keeping the deformations of adjacent elements compatible in their deformed state. Let us envision a case in which torsional hinges form at the faces of the supporting columns. In this scenario, the twist experienced in the ends of the beams will relieve the applied torque. Therefore, rather than resisting the full factored torque applied at the column faces, "twisting" of the beam ends can be permitted, and the beams can be relieved from the applied torsional moments. To this end, commentary for Article 5.7.2.1 of the AASHTO LRFD specifications states:

> *It is not necessary to design for compatibility torsion as long as the other load paths are properly designed for the redistributed forces. Consideration should be given to the aesthetic issues that may be caused by cracking that could be associated with not designing for compatibility torsion.*

Furthermore, the code portion of Article 5.7.2.1 states:

> *In a statically indeterminate structure where significant reduction of torsional moment in a member can occur due to redistribution of internal forces upon cracking, the applied factored torsional moment at a section,* T_u, *may be reduced to* ϕT_{cr}, *provided that moments and forces in the member and in adjoining members are adjusted to account for the redistribution.*

In the aforementioned case, we must expect to see torsional cracks to allow for redistribution of the forces and create an opportunity for the loads to look for alternative load paths. This redistribution of forces is coupled with the need to design the section for ϕT_{cr}.

To complete this discussion, let us take a look at provisions in the AASHTO LRFD specifications for calculating torsional resistance, for cases in which we must perform a complete torsional design. Article 5.7.3.6.2 indicates:

> *The nominal torsional resistance shall be taken as:*

$$T_n = \frac{2A_o A_t f_y \cot\theta}{s}\lambda_{duct} \qquad (5.7.3.6.2\text{-}1)$$

where:

A_o = *area enclosed by the shear flow path, including any area of holes therein (in.²)*

A_t = *area of one leg of closed transverse torsion reinforcement in solid members, or total area of transverse torsion reinforcement in the exterior web and flange of hollow members (in.²)*

θ = *angle of inclination of diagonal compressive stresses as determined in accordance with the provisions of Article 5.7.3.4 with the modifications to the expressions for* v *and* V_u *herein (degrees)*

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

λ_{duct} = *shear strength reduction factor as defined in Eq. 5.7.3.3-5*

Note that the transverse steel area for torsion A_t is defined differently from the transverse steel area for flexural shear A_v. A common error is to confuse the two definitions.

Torsion increases the demand on shear reinforcement. In addition, torsion increases the demand on longitudinal reinforcement. To that end, Article 5.7.3.6.3 in the AASHTO LRFD specifications states:

> *The provisions of Article 5.7.3.5 shall apply as amended, herein, to include torsion. At least one bar or tendon shall be placed in the corners of the stirrups.*
>
> *The longitudinal reinforcement in solid sections shall be proportioned to satisfy Eq. 5.7.3.6.31:*

$$A_{ps}f_{ps} + A_s f_y \geq \frac{|M_u|}{\phi d_v} + \frac{0.5N_u}{\phi} + \cot\theta\sqrt{\left(\left|\frac{V_u}{\phi} - V_p\right| - 0.5V_s\right)^2 + \left(\frac{0.45p_h T_u}{2A_o\phi}\right)^2} \qquad (5.7.3.6.3\text{-}1)$$

> *In box sections, longitudinal reinforcement for torsion, in addition to that required for flexure, shall not be less than:*

$$A_\ell = \frac{T_n p_h}{2A_o f_y} \qquad (5.7.3.6.3\text{-}2)$$

where:

p_h = *perimeter of the centerline of the closed transverse torsion reinforcement for solid members, or the perimeter through the centroids of the transverse torsion reinforcement in the exterior webs and flanges for hollow members (in.)*

A_ℓ *shall be distributed around the outermost webs and top and bottom slabs of the box girder.*

In summary, torsional effects impose additional demands, and therefore require additional reinforcement, both in longitudinal and transverse directions. This conclusion can be best visualized in the space truss model (also known as the tubular truss model) shown in **Fig. 2**. A given torsional moment results in an increased demand on one face of a typical structural component, thereby increasing the demand on the torsional reinforcement on that face. However, typical structural components are reinforced symmetrically in view of the force

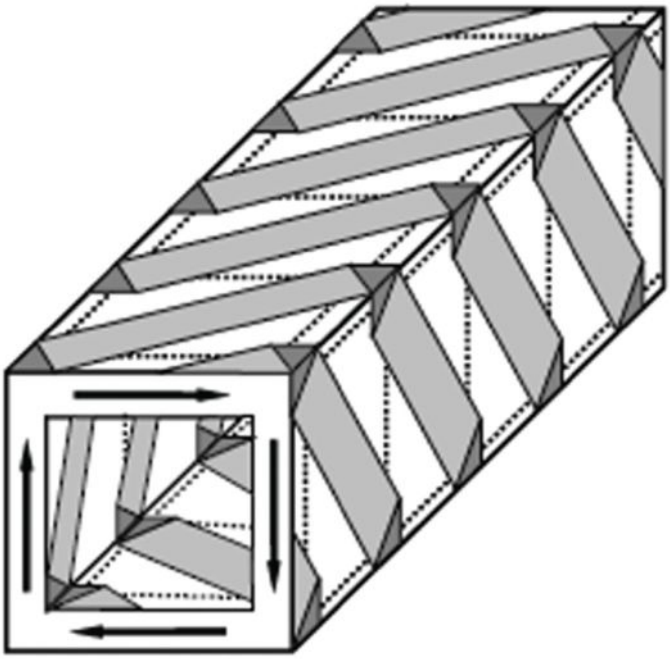


Figure 2. Strut-and-tie model of a space truss subjected to a torsional moment. Source: *Anchorage of Headed Reinforcement in CCT Nodes*.[2]

flow that is discussed in Fig. 2. Notably, in the terminology of the AASHTO LRFD specifications, all the nodes shown in this strut-and-tie model (STM) are smeared nodes. What is very clear is that the formation of a helical compression field in three-dimensional space requires ties in the longitudinal and transverse directions. Notably, the previously discussed torsional design requirements from the AASHTO LRFD specifications are based on the model in Fig. 2. In other words, should we choose to design for torsional effects using an STM, we would reach a design equivalent to what is explicitly required by the previously discussed provisions.

Understanding the implications of compatibility and equilibrium torsions is a key starting point for torsional design. Comparing the applied factored torsional moments to a member to the threshold torque is a necessary second step. Following these initial steps, additional decisions can be made to complete the structural design for torsional moments.

References

1. American Association for State Highway and Transportation Officials (AASHTO). 2020. *AASHTO LRFD Bridge Design Specifications*. 9th ed. Washington, DC: AASHTO.
2. Thompson, M.K., M.J. Young, J.O. Jirsa, J.E. Breen, and R.E. Klingner. 2003. *Anchorage of Headed Reinforcement in CCT Nodes*. Center for Transportation Research Report 1855-2. Austin, TX: University of Texas at Austin.

Dr. Oguzhan Bayrak is a chaired professor at the University of Texas at Austin, where he serves as the director of the Concrete Bridge Engineering Institute.

Structural Design with Ultra-High-Performance Concrete

by Dr. David Garber, Dr. Rafic G. Helou, and Dr. Benjamin Graybeal, Federal Highway Administration

Ultra-high-performance concrete (UHPC) is a portland cement–based composite material that includes supplementary cementitious materials (SCMs), fine sand, water, steel fibers, high-range water-reducing admixture, and other admixtures as needed. The rheology of UHPC can be customized to be anywhere from self-consolidating (ideal for casting connections and components in formwork) to thixotropic (ideal for casting bridge deck overlays on cross slopes and grades).

UHPC is concrete with enhanced mechanical properties. In accordance with the American Association for State Highway and Transportation Officials' (AASHTO's) recently released *Guide Specifications for Structural Design with Ultra-High Performance Concrete*,[1] a concrete mixture must have steel-fiber reinforcement and achieve the following characteristics to be classified as a UHPC material:

- A minimum compressive strength f'_c of 17.5 ksi
- A minimum effective cracking strength $f_{t,cr}$ of 0.75 ksi
- A minimum crack localization strength $f_{t,loc}$ equal to or greater than the effective cracking strength $f_{t,cr}$
- A minimum crack localization strain $\varepsilon_{t,loc}$ of 0.0025

UHPC is much more durable than conventional concrete. The chloride ion penetrability of UHPC, measured in accordance with ASTM C1202,[2] can be as low as 50 coulombs (compared to 1500 to 2500 coulombs for conventional concrete). The relative dynamic modulus (RDM) of UHPC, measured in accordance with the standard test method for freeze-thaw resistance (formerly ASTM C666[3]) can be greater than 95% (compared to RDMs of 75% to 80% for conventional concrete). These properties demonstrate that UHPC has very low permeability and is highly resistant to freezing and thawing.

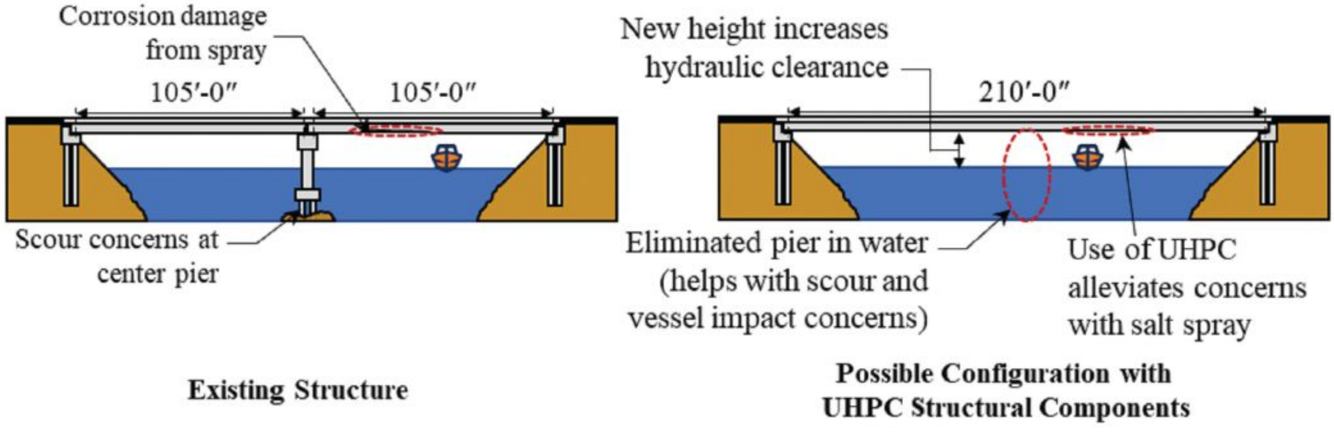
To date, UHPC has been primarily used for connections between prefabricated concrete elements and for preservation and repair activities (including deck overlays, link slabs, and beam end repairs).[4] However, AASHTO's *Guide Specifications for Structural Design with Ultra-High Performance Concrete* now enable owners and engineers to design structural components, leveraging the full potential of UHPC.

Benefits of UHPC Structural Components

UHPC structural components can be designed to be more efficient and have higher strengths than conventional concrete components. Therefore, engineers can leverage UHPC to design superstructures with several structural benefits, including the following:

- Decreased superstructure weight. When UHPC is used instead of conventional concrete, superstructures can be constructed with lighter sections and wider beam spacing, which can result in significantly lighter superstructures. As a result, it may be feasible to reuse a substructure in situations where reuse was not previously possible, and substructure costs may be reduced.
- Shallower superstructure depths. UHPC superstructures can be shallower than conventional concrete alternatives for similar span lengths, which can help state departments of transportation and other agencies alleviate

Figure 1. Side-by-side comparison of a conventional structure and a hypothetical structure where ultra-high-performance concrete structural components can be used to eliminate the center pier and increase hydraulic clearance. All Figures: Federal Highway Administration.



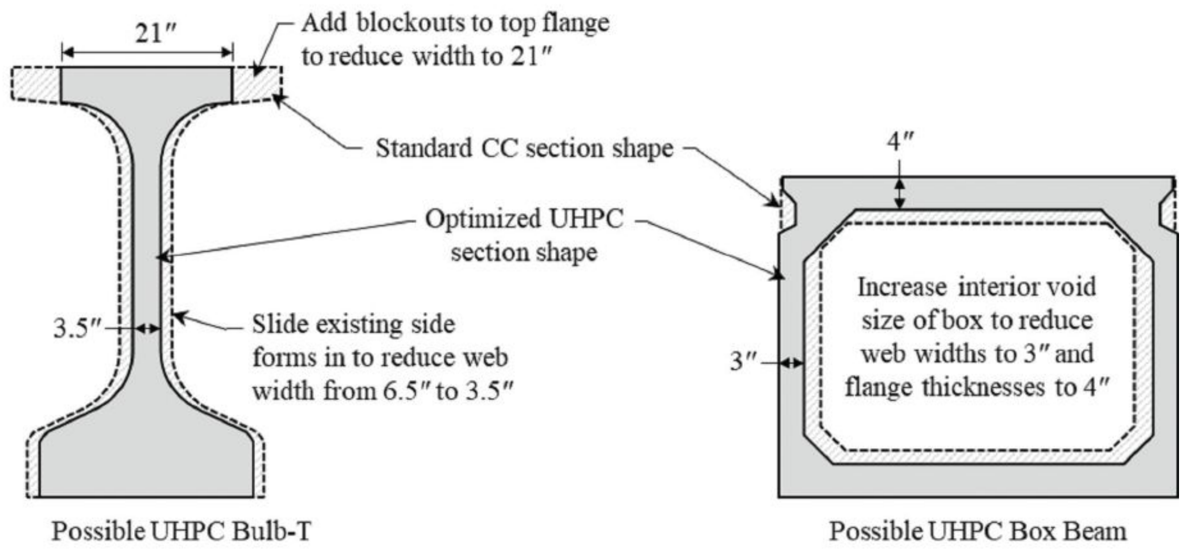


Figure 2. Ultra-high-performance concrete (UHPC) section shapes are optimized by modifying the existing formwork for conventional concrete (CC) standardized shapes.

challenges involving low vertical clearance and bridge collisions. Bridge owners could also benefit from greater hydraulic clearance.

- Increased span lengths. UHPC superstructures can be designed to have longer spans than conventional concrete alternatives at similar section depths. As a result, it may be possible to eliminate interior piers in waterways (**Fig. 1**) or shoulder piers that are immediately adjacent to traffic. These design changes can extend a bridge's service life and potentially resolve other structural integrity and safety concerns.

Garber et al.[5] quantified these benefits in several case studies.

Figure 2 illustrates that cross sections for UHPC components can be optimized to be more than 25% lighter than conventional concrete components—even considering UHPC's higher unit weight—and they can span lengths more than 25% longer than those spanned by conventional concrete sections of similar depths.[1,6,7] These optimized section shapes can be constructed with simple modifications to formwork used for standard conventional concrete shapes.

UHPC section depths can be more than 12.5% shallower than the depths of conventional concrete components for similar span lengths, and UHPC superstructures can be designed to be 41% lighter for a 120-ft span and 68% lighter for a 30-ft span.[5]

UHPC structural components will also have longer service lives than conventional concrete alternatives, leading to lower service-life costs for UHPC structures.[8] An extended service life can be beneficial for structures on routes with high volumes of average daily traffic, routes in remote areas with long detours, or bridges of high importance (for example, adjacent to hospitals). Many other benefits to using UHPC structural components are summarized in Garber et al.[7] and Graybeal and Helou.[6]

Design of UHPC Structural Components

In many ways, the design of UHPC structural components mirrors the design of conventional concrete. An engineer who understands the process and procedures of designing a conventional concrete structural component will be able to understand the procedures for designing a UHPC structural component. Some of the differences in the design process are highlighted in this section.

One of the primary benefits of UHPC is its post-cracking strain capacity in tension. The design of UHPC structural components requires an engineer to return to the fundamentals of engineering mechanics and use strain-based design approaches. The strain-compatibility approach is used for calculating the nominal flexural resistance (with or without axial force), where a linear strain profile across the section depth is used with idealized stress-strain material relationships and equilibrium principles to calculate the neutral-axis depth and associated moment and curvature values.[6,7,9] (See the "Strain Compatibility Primer" articles in the Fall 2024 and Winter 2025 issues of *ASPIRE*® to learn more about the strain-compatibility approach.) An alternate strain-based shear design approach was developed for AASHTO's *Guide Specifications for Structural Design with Ultra-High Performance Concrete* based on the same concepts as the modified compression field theory developed for conventional concrete (see El-Helou and Graybeal[10]).

Many other aspects of the design of UHPC structural components are similar to the design of conventional concrete components. However, there are slight differences, including prestress loss calculations (with different equations for calculating creep and shrinkage), service stress checks (with different tensile stress limits), principal stress checks in the web (required for all UHPC components), and fatigue stress checks (required for embedded steel in UHPC components). Additionally, slenderness effects and girder stability may need to be considered for the longer, more slender members that are made possible with UHPC. More details on the design of UHPC structural components can be found in Garber et al.,[7] Murphy and Bayrak,[11] and Graybeal and Helou.[6]

Available Resources and Workshop

The Federal Highway Administration (FHWA) has recently published several resources to assist state transportation agencies, engineers, contractors, and other stakeholders in implementing UHPC structural components. These include the following reports:

- *Structural Design with UHPC Workshop Manual* (FHWA-RC-24-0006)[7]
- *Structural Design with Ultra-High Performance Concrete* (FHWA-HRT-23-077)[6]
- *Possible Framework for Using the Strut-and-Tie Method (STM) with Ultra-High Performance Concrete (UHPC)* (FHWA-RC-24-0004)[12]
- *Section Shapes for Short-Span UHPC Bridges* (FHWA-RC-24-0009)[5]

FHWA has also developed a one-day workshop to help with the implementation of UHPC structural components. The workshop builds on basic knowledge of reinforced and prestressed concrete bridge design, and introduces and explains aspects of analysis and structural design that are unique for UHPC structural components. The learning objectives for the workshop include the following:

- Identify when using UHPC will be advantageous.
- Describe the differences between conventional concrete and UHPC related to the design of structural elements.
- Analyze and design UHPC structural elements using *Structural Design with Ultra-High Performance Concrete* (FHWA-HRT-23-077).[6]

State departments of transportation interested in implementing UHPC structural components in the near future may contact David Garber (david.garber@dot.gov) for more information on the workshop.

References

1. American Association of State Highway and Transportation Officials (AASHTO). 2024. *Guide Specifications for Structural Design with Ultra-High Performance Concrete*. Washington, DC: AASHTO.
2. ASTM International. 2022. *Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration*. ASTM C1202-22e1. West Conshohocken, PA: ASTM International.
3. ASTM International. 2015. *Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing (Withdrawn 2024)*. ASTM C666/C666M-15. West Conshohocken, PA: ASTM International.
4. Federal Highway Administration (FHWA). 2024. "Deployments of UHPC in Highway Bridge Construction. Interactive Map." https://highways.dot.gov/research/structures/ultra-high-performance-concrete/deployments.
5. Garber, D., R. Helou, and B. Graybeal. 2024. *Section Shapes for Short-Span UHPC Bridges*. FHWA-RC-24-0009. Washington, DC: FHWA.
6. Graybeal, B., and R. Helou. 2023. *Structural Design with Ultra-High Performance Concrete*. FHWA-HRT-23-077. McLean, VA: FHWA. https://highways.dot.gov/sites/fhwa.dot.gov/files/FHWA-HRT-23-077.pdf.
7. Garber, D., R. Helou, and B. Graybeal. 2024. *Structural Design with UHPC Workshop Manual*. FHWA-RC-24-0006. Washington, DC: FHWA. https://www.fhwa.dot.gov/resourcecenter/teams/structures-geotechnical-hydraulics/Structural_Design_UHPC_Workshop_Manual.pdf.
8. Haber, Z., M. McDonagh, A. Foden, S. Sadasivan, and B. Graybeal. 2023. *Ultra-High Performance Concrete (UHPC) Overlays: An Example of Lifecycle Cost Analysis*. FHWA-HRT-23-012. McLean, VA: FHWA. https://highways.dot.gov/sites/fhwa.dot.gov/files/FHWA_HRT-23-012.pdf.
9. El-Helou, R. G., and B. A. Graybeal. 2022. "Flexural Behavior and Design of Ultrahigh-Performance Concrete Beams." *Journal of Structural Engineering* 148 (4). https://doi.org/10.1061/(ASCE)ST.1943-541X.0003246.
10. El-Helou, R. G., and B. A. Graybeal. 2023. "Shear Design of Strain-Hardening Fiber-Reinforced Concrete Beams." *Journal of Structural Engineering* 149 (2). https://doi.org/10.1061/JSENDH.STENG-11065.
11. Murphy, T., and O. Bayrak. 2024. "Development of the *AASHTO Guide Specifications for Ultra-High-Performance Concrete*." *ASPIRE* 18 (1): 16–17. https://doi.org/10.15554/asp18.1.
12. Garber, D., R. Helou, and B. Graybeal. 2024. *Possible Framework for Using the Strut-and-Tie Method (STM) with Ultra-High Performance Concrete (UHPC)*. FHWA-RC-24-0004. Washington, DC: FHWA. https://www.fhwa.dot.gov/resourcecenter/teams/structures-geotechnical-hydraulics/FHWA-RC-24-0004.pdf.


Eriksson
technologies
PRECAST/PRESTRESSED
CONCRETE ENGINEERING
BIM Modeling
(Tekla & Revit)
Erection Drawings
& Piece Tickets
Design
& Detailing
Repair &
Restoration
www.ErikTech.com


Eriksson
software
BRIDGE ENGINEERING
SOFTWARE
Cv
CULVERT
Concrete culvert
design
Pp
PIPE
Reinforced concrete
pipe design
Gd
GIRDER
Precast/prestressed concrete
bridge beam design
Pr
PIER
Reinforced concrete
bridge pier design
Pl
PILE
Precast/prestressed
concrete pile design
SYNC
SYNC
Connects Eriksson
Bridge Suite to BIM models
www.ErikssonSoftware.com