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WINTER 2025

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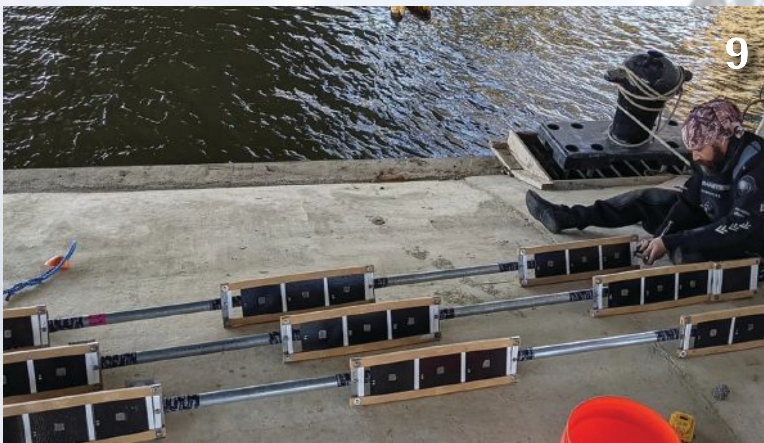

Photo: Siva Corrosion Services.

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CONTENTS

Features

Siva Corrosion Services 9

For almost 30 years, Siva Corrosion Services has investigated the corrosion conditions of bridges.

Sound Transit Light Rail 12

Departments

Editorial 2

Concrete Calendar 4

Perspective—Concrete Bridges: Market Share and Performance 6

Aesthetics Commentary 14

Concrete Bridge Technology—A Crack Is Not a Crack: Bridge Deck Cracking 18

Concrete Bridge Technology—Flexural Strength of Prestressed Concrete Bridge Beams with Composite Decks 22

Concrete Bridge Technology—Fatigue Behavior of Reinforced and Prestressed Concrete 25

Concrete Bridge Technology—Concrete Segmental Bridges-Preliminary Design Approximations for Span Layouts and Structure Depths 28

Creative Concrete Construction—Corrosion-Resistant Fiber-Reinforced Polymer Reinforcement for Concrete Structures 31

NCBC Spotlight—Celebrating 70 Years of PCI Accomplishments 34

Concrete Connections 37

CBEI Series—The Latest from the Concrete Bridge Engineering Institute 38

A Professor's Perspective—Reliability-Based Service III Evaluation for Prestressed Concrete Girder Bridges under Platoon Loads 42

LRFD—Shear-Friction Design Provisions of the *AASHTO LRFD Bridge Design Specifications* 46

FHWA—Concrete Bridge Shear Load Rating: Updates to AASHTO's *The Manual for Bridge Evaluation* and Further Guidance 50

Photo: Siva Corrosion Services.

Advertisers' Index

CBEI 45,49

Eriksson Technologies. . . . . Back Cover

Japan Life . . . . Inside Front Cover

Max USA 3

Moldtech 30

NCBC . . . . 4, Inside Back Cover

PCI 5

EDITORIAL


Photo: PCI.

Stay Vigilant and Innovate

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

I just returned from a fantastic session of the Concrete Materials for Bridge Training Program, presented by Dr. Kevin Folliard and Dr. Thanos Drimalas at the Concrete Bridge Engineering Institute (CBEI) in Austin, Texas (see Concrete Materials Course on pg. 40). During the course, I scribbled down several key points, but two really struck a chord. First, I was reminded of the importance of staying engaged with suppliers. This is certainly not a new concept, but it is one that we need to dust off and make a greater priority in our industry.

Second, the CBEI course drove home the point that we need to remain vigilant about the quality of concrete materials. The constituent materials that we use in our concrete mixtures can sometimes lead to "bad" concrete—concrete that may be prone to alkali-silica reaction, delayed ettringite formation, corrosion damage in marine environments or from deicing agents, and other types of damage. We can mitigate this risk by understanding the effects of cement type and supplementary cementitious materials (SCMs) on concrete quality, and avoiding the use of reactive aggregates and excessive admixtures.

While the CBEI training and course materials were up to date and absolutely relevant to today's industry, the topics we covered took me back some, nearly 40 years, to the start of my career. At that time, fly ash seemed to be the be-all and end-all with regard to the densification of high performance concrete. We were developing new concrete mixture designs that had fewer channels for moisture to penetrate and reach the reinforcement. "Holy sustainability, Batman!" We thought we had found the holy grail—with fly ash, we were recycling a product that otherwise would have ended up in a landfill (a fabulous new concept). The concrete industry also came to embrace other industrial byproducts and SCMs such as silica fume, slag, and metakaolin as options for densifying concrete. This had been cutting edge stuff for 40 years, and to say our industry was excited by the possibilities these SCMs offered is an understatement.

Fly ash did not turn out to be the be-all and end-all for the concrete industry. Coal-fired plants are going offline, which affects the supply of fly ash as an industrial byproduct, and we are also in search of sustainable, environmentally friendly, reliable alternatives to traditional SCMs. Chemists continue to play a critical role in developing ingredients for high-performing, durable concrete mixture designs.

We often talk about innovation and pushing new concepts—and I'm all in. The October-December 2024 issue of the Transportation Research Board's *TR News* is on the theme of transportation innovation and implementation, and several of the articles point out that we need to thoroughly test possible solutions and quantify the costs and benefits of their implementation. I think these lessons are critical in this new era of emerging blended cement technologies. Suppliers, designers, owners, and manufacturers alike need to be careful about how they implement new options. We need to fully understand the risks for long-term structural deterioration associated with those options so that we do not build bridges that are vulnerable to adverse mechanisms such as alkali-carbonate reaction. As we move back into the fundamentals of cement and concrete science, we cannot ignore the carbonization hazard. We need to deal with it. During my time at CBEI, I was heartened to learn that although we face a range of durability issues, there are sufficient guidance documents, test methods, and specifications available to ensure that we can deliver durable concrete bridges.

Collaboration strengthens our profession and community as a whole. We in the concrete industry are successful in exchanging information, engaging our peers, and sharing our research and innovative ideas with our colleagues. However, as we seek to understand our challenges and implement innovative solutions, we may need to look beyond the boundaries of our traditional community and its resources. Is there an idea, approach, or methodology outside of our industry that can help with the challenge of densifying concrete? Can we borrow it, perhaps for just a few years, while we rejigger our supply lines for silica fume and metakaolin?

It's important to admit that we don't know what we don't know. Nonetheless, we must keep focused on our quest to achieve long-lasting concrete bridge solutions. Simply put, that's our job.

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On the Sound Transit's Federal Way Link Extension project in Washington state, a cast-in-place concrete segmental structure is constructed from above using the balanced-cantilever method allowing for the long spans needed to avoid problematic soil conditions. Photo: Kiewit.

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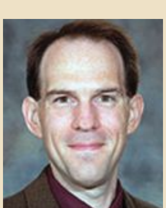

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CONCRETE CALENDAR

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

January 5–9, 2025
Transportation Research Board Annual Meeting
Walter E. Washington Convention Center
Washington, D.C.

January 20–23, 2025
World of Concrete
Las Vegas Convention Center
Las Vegas, Nev.

February 3–7, 2025
PCI Convention at The Precast Show
Indianapolis Marriott Downtown and Indiana Convention Center
Indianapolis, Ind.

March 4–7, 2025
NRMCA Annual Convention
JW Marriott Tucson Starr Pass Resort & Spa
Tucson, Ariz.

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
Columbus, Ohio

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.

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

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

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

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

Concrete Bridges: Market Share and Performance

by Angela Tremblay

Over the years, organizations such as the Portland Cement Association, the Precast/Prestressed Concrete Institute, and the American Segmental Bridge Institute (ASBI) have used data from the National Bridge Inventory (NBI) to determine trends in construction materials and bridge conditions.[1–4] The NBI database is maintained by the Federal Highway Administration, with data supplied annually by state departments of transportation, federal agencies, and tribal governments in accordance with the National Bridge Inspection Standards (*Code of Federal Regulations* Title 23, Part 650, Subpart C). The NBI data can be organized and analyzed to give us a snapshot of market share and durability for concrete bridges as compared with bridges of other materials. By evaluating these trends, we can gain insight into growth opportunities within the concrete bridge industry. Specifically, we can use the information to highlight the advantages of concrete bridges in terms of durability and sustainability, and we can identify potential areas for improvement.

Keith Ramsey, senior structural engineer and senior vice president at Strinteg, has analyzed the 2022 NBI data recorded through June 2023 on behalf of the National Concrete Bridge Council (NCBC). As a consultant working with PCI and NCBC, I used the data analysis provided by Ramsey to develop a report on the market share and advantages of concrete bridges. The goals of the data analysis and report are to inform the bridge community about the status of concrete bridges; to inspire us to build on our successes and seek areas for improvement; and to help us innovate, armed with the knowledge we have gained. This article provides a preview of the data and observations on the trends. The full 2023 report is available on the NCBC website (nationalconcretebridge.org).

NBI Data

The 2022 NBI included 474,844 bridges. For the purposes of the inventory, a bridge is defined as a structure that supports a public roadway with vehicular traffic and has a total structure length greater than 20 ft. Buried structures classified as culverts are excluded from the analysis in this article. The relevant data used in this summary article are coded according to the *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges.*[5] The data coding categories used in the data analysis include:

- State code (Item 1): The state code indicates the location of the structure.
- Year built (Item 27): The year built indicates the year that the original construction was completed. It is not affected by any rehabilitation projects, superstructure replacements, or bridge preservation activities. For example, if a new superstructure was placed in 2003 on existing substructure units that were built in 1953, the year built would be coded as 1953.
- Material and structure type of the main span (Item 43): This code represents the predominant material type (for example, concrete, concrete continuous, steel, steel continuous, prestressed concrete-pretensioned or post-tensioned) and the design or construction type (for example, slab, stringer/multibeam or girder, girder and floor-beam system, box beam, arch, segmental box girder). This item only indicates the predominant material and design of the main span, and it does not account for special cases in which multiple materials or designs are found within the same structure.
- Condition rating of the bridge deck, superstructure, and substructure (Items 58 to 60): A condition rating of the deck, superstructure, and substructure is assigned at each bridge inspection. The condition

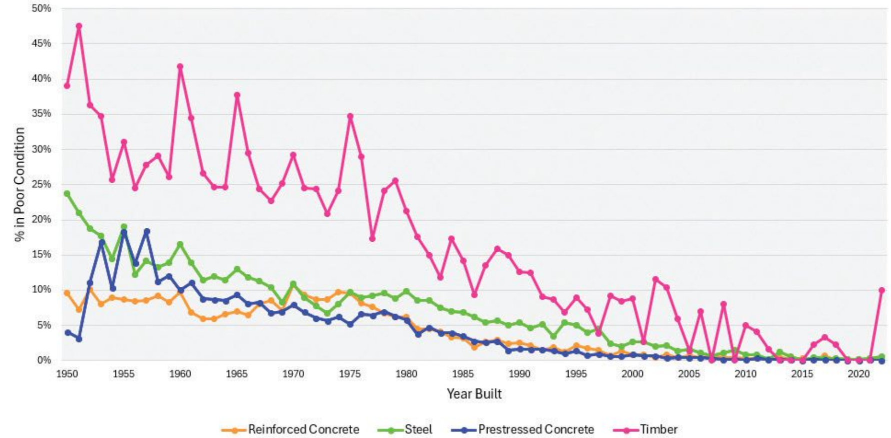

Figure 1. Comparison of the percentage of poor-condition bridges according to year built and construction material. All Figures: National Concrete Bridge Council.

ratings range from 0 (bridge closed) to 9 (excellent condition). In this data analysis, the overall condition of the bridge is taken as the lowest condition rating among these elements. A bridge that rates 4 or less on one or more of the bridge deck, superstructure, and substructure condition ratings is considered to be a "poor-condition" bridge.

There are some nuances and limitations to the NBI data. However, considering that special circumstances make up a relatively small portion of the data set, the trends based on the total number of bridges are still applicable.

Durability

The percentage of poor-condition bridges of a given structure type and material gives an indication of that structure type's durability and performance in safely carrying the design loads. **Figure 1** shows the percentage of bridges built in a given year (from 1950 to 2022) that are still in service and are in poor condition according to NBI data. For example, of the 3299 prestressed concrete bridges built in 1990 that are still in service in 2022, 55 bridges (1.7%) were in poor condition.

For bridges built in the past 30 years, the percentage of poor-condition reinforced or prestressed concrete bridges built in a given year remains consistently lower than the percentage of poor-condition steel or timber bridges. This trend speaks to the durability and longevity of concrete bridges. Concrete segmental bridges fall under the general concrete or prestressed concrete material codes and are therefore included in the overall trends. Specific analysis of the trends for concrete segmental construction are presented in ASBI's *Durability Survey of Concrete Segmental Bridges*[4] and in a summary article in the Winter 2023 issue of *ASPIRE*®.

For bridges older than 30 years, the results show greater variability from year to year. It is more difficult to draw conclusions from the trends in older bridges because after 30 years of service life, maintenance and preservation activities begin to have a greater impact on all bridge types.

Table 1, **Fig. 2**, and **Fig. 3** offer other insights into the data. For example, prestressed concrete bridges make up 40.6% of the total bridges in the NBI built from 1950 to 2022 that are still in service. However, of all the poor-condition bridges built within that time range, only 20.1% are prestressed concrete. Therefore, prestressed concrete bridges make up a smaller proportion of poor-condition bridges considering the percentage of such bridges that are in service. This comparison is interesting, but it does not account for the relative

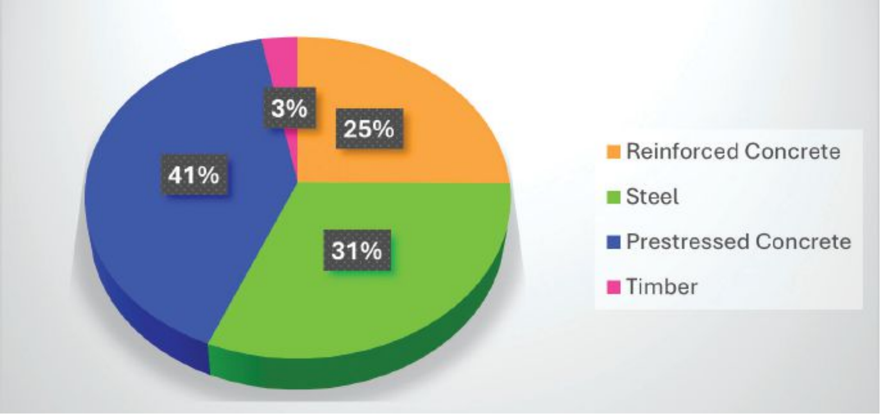

Figure 2. Percentage of in-service bridges built from 1950-2002 by material type.

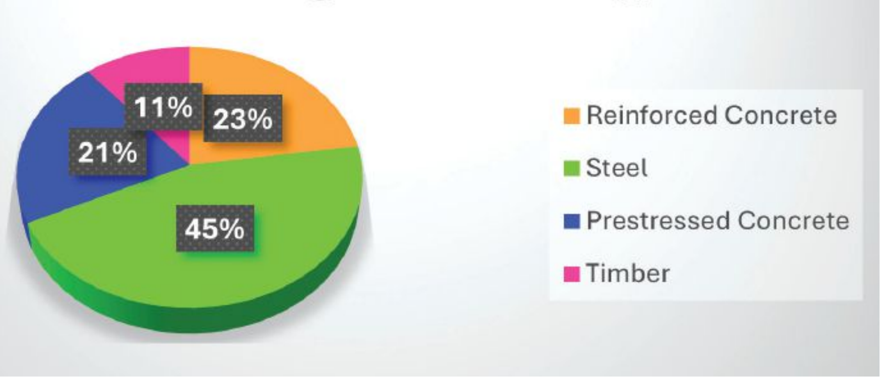

Figure 3. Percentage of poor-condition bridges built from 1950-2022 by material type.

Table 1. Material type comparison for all in-service bridges with a year-built date from 1950 to 2022

	Reinforced concrete	Steel	Prestressed concrete	Timber	All other	Total
Total bridges in service	102,349	127,298	165,880	12,331	273	408,131*
Percentage of bridges in service	25.08%	31.19%	40.64%	3.02%	0.07%	100.00%
Total bridges in poor condition	5396	10,762	5036	2603	22	23,819
Percentage of poor-condition bridges that are of the given material type	22.65%	45.18%	21.14%	10.93%	0.09%	100.00%
Percentage of bridges within the given material type that are in poor condition	5.27%	8.45%	3.04%	21.11%	8.06%	5.84%

* The total number of bridges in this data set represents those bridges with original construction dates from 1950 through 2022. The difference between this total (408,131) and the total number of bridges in the National Bridge Inventory (474,844) is the number of bridges coded with a year-built date before 1950.

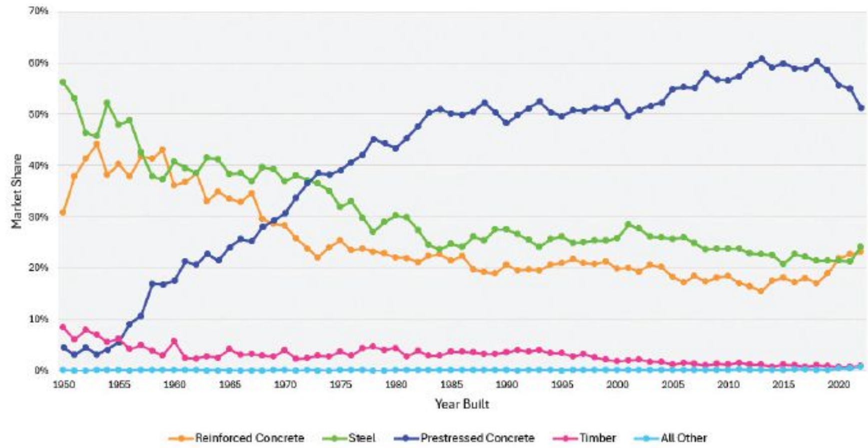

Figure 4. Comparison of the percentage of bridges built per year for different material types.

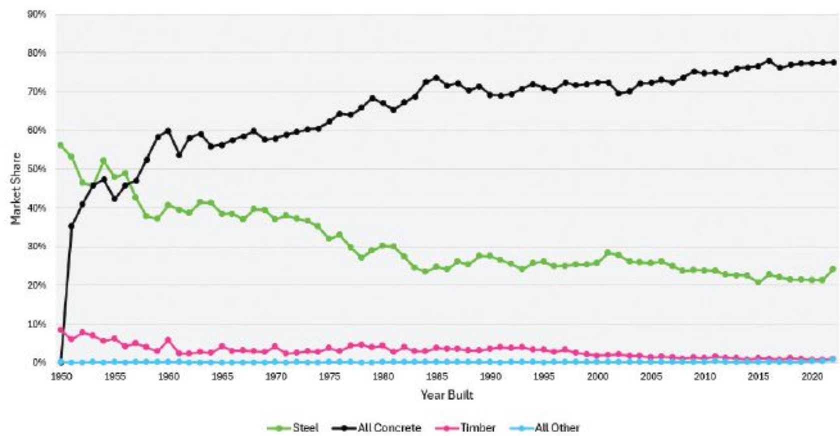

Figure 5. Comparison of the percentage of bridges built per year for different material types with reinforced concrete and prestressed concrete combined into one category.

age of the bridges, which can be seen more clearly in Fig. 1.

Market Share

"The Long Gray Line: Reflections on Concrete Bridge Progress,"[3] which appeared in the Winter 2019 issue of *ASPIRE*, considered how many states (including the District of Columbia and Puerto Rico) built at least 65% of their new bridges from 2007 through 2016 using concrete (either reinforced or prestressed) superstructure types. For that 10-year period, 31 states met the 65% threshold. When we extend this analysis to the most recent decade, we find that for the period from 2012 to 2022, 34 states met the 65% threshold. We can make a similar comparison for prestressed concrete superstructures. Between 2007 and 2016, 23 states built at least 65% of new bridges using prestressed concrete, whereas in the more recent 10-year period between 2012 and 2022, the number of states meeting the 65% threshold increased to 29. These comparisons show an increase in market share for concrete bridge construction across the United States.

Figures 4 and **5** illustrate this increase. Figure 4 shows significant growth in the prestressed concrete market share between 1950 and 1985, then a slower growth rate from 1985 to 2018. The slight decline in market share for prestressed concrete from 2018 to 2022 was accompanied by an increase in the market share for reinforced concrete bridges. The trends in market share considering all concrete bridges combined (Fig. 5) show a slowing of the growth rate and potential plateau in market share over the past 10 years.

Looking Forward

A deeper dive into the NBI data, analysis, and trends may deliver more opportunities to improve our concrete bridge industry and our infrastructure. Some states and owners historically prefer concrete superstructures. What drives that preference? How do material cost and availability affect the choices that owners make?

The most significant growth in the concrete bridge industry between 1950 and 1965 can be attributed to the introduction, acceptance, and widespread use of prestressed concrete beams and the initial construction of the interstate system. Aside from delivering what owners need in terms of cost-efficient and durable solutions, the best way to increase market share is to bring effective innovations to market. So, what is the next big thing for the concrete bridge industry?

In terms of technological advances, further development and acceptance of ultra-high-performance concrete applications and other advances could provide an opportunity for expanding the concrete bridge market share through the use of shallower and/or longer spans. Developing strategies for enhancing structural resilience and sustainability also offer an avenue for improvement and growth. Leveraging these opportunities could ultimately lead to engineering solutions that elevate the concrete bridge industry and have the potential to positively affect our communities.

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Siva Corrosion Services

For almost 30 years, Siva Corrosion Services has investigated the corrosion conditions of bridges as part of a mission to extend their service lives

by Monica Schultes

Siva Venugopalan founded Siva Corrosion Services (SCS) in 1997. Since then, the firm has become a leader in the field of materials and corrosion engineering, working primarily with state departments of transportation. SCS offers nondestructive testing (NDT); laboratory services; material testing; design, installation, and monitoring of wireless sensors for structural health monitoring; corrosion evaluation; and corrosion control design, among other services.

Siva Corrosion Services performs a corrosion evaluation of internal tendons in the trestle spans and external tendons in the concrete segmental box girders along the 18,460-ft-long Albemarle Sound Bridge. All Photos: Siva Corrosion Services.

History and Geography

"I started the company in 1997 with about five dollars and a dream of doing what I love," recalls Venugopalan. What sets SCS apart from other firms is its breadth of knowledge. "We understand materials and how they degrade in different environments; we understand corrosion and what it does to a structure. We understand NDT, which helps identify what is going on inside before it becomes visible," he explains.

Located in West Chester, Pa., in the suburbs of Philadelphia, much of SCS's work is in the mid-Atlantic region. They have worked on projects as far away as Alaska and Florida, but a significant percentage of its business is within driving distance along the Northeast Corridor. The small firm focuses on bridges, working primarily with state agencies evaluating and testing bridge components and materials to determine the cause and extent of corrosion. The SCS team can then select material and corrosion mitigation solutions based on their evaluations.

How Bad Is Bad?

Today's aging infrastructure and tight budgets can create a "worst-first" scenario. SCS is often called in to evaluate a problem and answer the question,"How bad is bad?" "Understanding the problem is the first step, and that is where we come in," says Venugopalan. "Using highly specialized testing, we thoroughly identify and quantify problems associated with internal deterioration. We then provide independent recommendations to cost effectively extend the service lives of structures."

"Understanding the problem is the first step, and that is where we come in."

Clients are understandably concerned when a problem arises. But it never helps to panic, and there are techniques to slow further corrosion. Results from SCS's evaluations help structural engineers pursue solutions that involve strengthening or rehabilitation. "Every day we go to a site and evaluate the problem and get a handle on the situation. Most of the time it is not as bad as you think, and it is not going to cost [the client] a large amount to ameliorate," says Venugopalan.

Corrosion Evaluation of Albemarle Sound Bridge

The Albemarle Sound Bridge carries North Carolina State Highway 32 over the Albemarle Sound. Constructed in 1990, it is a two-lane, 18,460 ft-long structure and consists of 224 concrete trestle superstructure spans at the south end of the bridge, 162 concrete trestle

The 1.7-mile, four-lane Harry Nice Middleton Bridge connecting Maryland and Virginia over the Potomac River has a 100-year service life due in part to a corrosion protection plan developed by Siva Corrosion Services.

spans at the north end, and 31 spans of post-tensioned (PT) concrete segmental box superstructure in between. The trestle spans are composed of PT precast concrete deck slabs supported by trestle bents. Each concrete segmental box span consists of multiple precast concrete box-girder segments supported by concrete piers. Substructures for the trestle sections are pile caps on prestressed concrete piles. Substructures for the concrete segmental box sections are concrete columns and caps on concrete pile footings supported by prestressed concrete piles.

SCS performed a corrosion evaluation of internal tendons in the trestle spans and external tendons in the box girders. In addition to PT evaluation, SCS also evaluated corrosion of the deck, barriers, columns, caps, piles, and footers. Testing included corrosion potentials, chloride profile analysis, reinforcing bar cover survey, compressive strength testing, and petrographic analysis. Based on the level of chloride contamination and corrosion activity, recommendations were developed to install corrosion-mitigation systems on each component, which would extend the life of the structure. Repairs were ranked and prioritized based on the severity of deterioration and the cost of preservation.

Technology

Historically, transportation agencies relied on visual inspections to evaluate the condition of bridge decks. But visual inspections do not tell the whole story. Automated ground-penetrating radar (GPR) and sounding, along with drones, assist SCS in their work every day. Venugopalan explains, "Unlike 30 years ago, we use a variety of methods to see what is happening inside the steel or concrete members." The firm's diagnostic arsenal includes magnetic particle inspection, GPR (at various frequencies), ultrasonic pulse velocity, and the basic rebound hammer methods. Infrared thermography, impact-echo testing, truck-mounted high-speed infrared cameras, automated sounding, and petrography can also be used.

SCS selects appropriate methods to determine the amount of concrete in the deck surface or substructure that is contaminated or affected by corrosion. The faster inspection systems minimize both the amount of time that field inspectors are on the structure and how their work affects traffic.

Nice Middleton Bridge

Working with the Maryland Transportation Authority, SCS consulted on the design and construction of the new Harry Nice Middleton Bridge, which carries U.S. Route 301 over the Potomac River. The design-build team also included Skanska and AECOM. SCS was tasked with developing a corrosion protection plan for the new 1.7-mile, four-lane bridge that connects Newburg, Md., and Dahlgren, Va. To achieve a 100-year service life, SCS developed a corrosion protection plan that identifies nonreplaceable elements and develops options to extend their lives to the 100-year mark. Corrosion mitigation strategies include materials selection, coatings, metalizing, and engineering design for reinforced concrete elements exposed to seawater and/or deicing salts. (Chloride-induced corrosion is the primary deterioration mechanism for the structure.) The design follows *fib* (International Federation for Structural Concrete) Bulletin 34, *Model Code for Service Life Design,*[1] which addresses service-life design for all types of concrete structures, with a focus on design provisions for managing the adverse effects of degradation. (For design and construction details of the Harry Nice Middleton Bridge, see the Project article in the Spring 2023 issue of *ASPIRE*®.)

Lab Work

SCS's tests are performed either on site or in-house at the firm's West Chester laboratory facilities. SCS currently outsources petrography but is planning to bring that work in-house to reduce the long lead time. Venugopalan says, "We like to have control over the testing so that our decisions are based on defendable results. This enables us to achieve our clients' goals at a lower cost with a higher confidence level."

The SCS team of professionals is cross-trained to complete different services, testing, and field work. Venugopalan describes a comprehensive training regimen for employees. They spend time in the office using premade specimens to learn the different types of equipment until they are proficient. Then engineers work in the field with other experienced professionals to perform evaluations efficiently and effectively. There is limited time to collect the data and get off the bridge; the team cannot go back and ask for another day. They need to be ready, and that requires both internal training in the office and hands-on training with all the equipment. According to Venugopalan, "Typically, we run through potential scenarios and practice sessions to make sure that the team can perform the work in the field in an expeditious manner."

"Typically, we run through potential scenarios and practice sessions to make sure that the team can perform the work in the field in an expeditious manner."

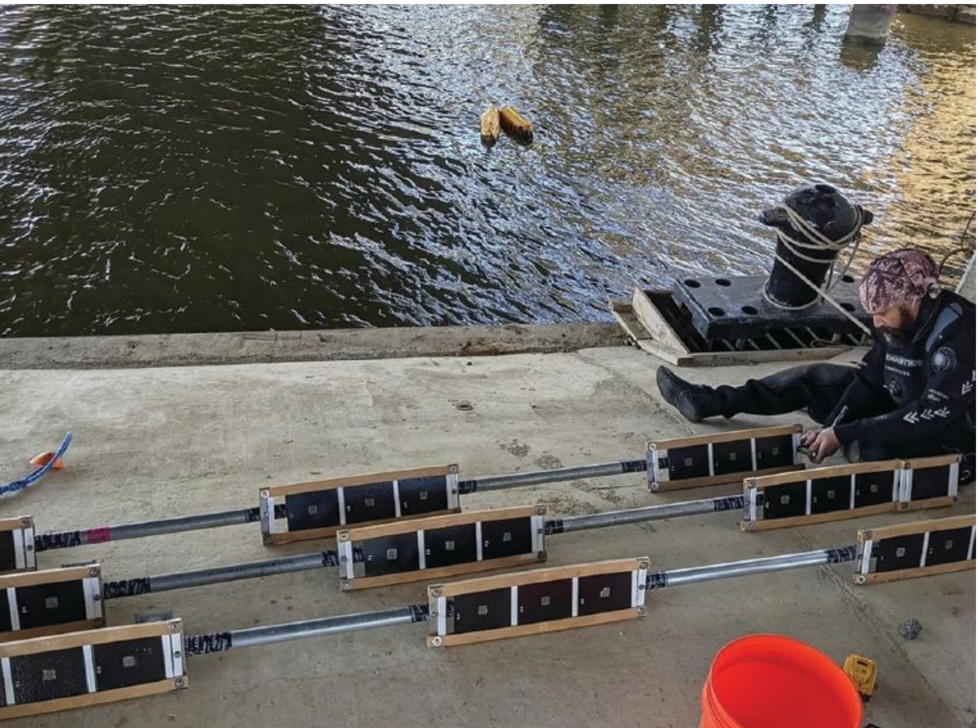
Venugopalan stays involved in data analysis, report writing, and quality control/quality assurance, and he reviews every project for lessons learned. "If the results or graphs are off, we need to determine why. With our years of experience, we can analyze the data, recognize anomalies, and identify underlying patterns and predict future outcomes."

Theodore Roosevelt Bridge Corrosion Evaluation and Rehabilitation

Partnered with TYLin and the District Department of Transportation, SCS evaluated the extent of corrosion activity on the concrete deck of the Theodore Roosevelt Bridge, which carries Interstate 66 traffic between Washington, D.C., and Arlington, Va. The bridge consists of two units: one over the Potomac River between the District of Columbia and Theodore Roosevelt Island, and the other over the Little River between Theodore Roosevelt Island and Arlington. The total length of the Theodore Roosevelt Bridge is approximately 3200 ft, and it serves as an important connection between Virginia and Washington, D.C. The bridge is categorized as "structurally deficient," with its deck in poor condition and in need of replacement.

SCS was tasked with identifying the causes of corrosion occurring in the deck, projecting future concrete deterioration through service-life analysis, and recommending repair and preservation options to extend the life of the deck by at least 50 years. Using data about the chloride profile and diffusion, reinforcing bar cover, and concrete damage, the team performed service-life modeling based on *Manual on Service Life of Corrosion-Damaged Reinforced Concrete Bridge Superstructure Elements*[2] to project future chloride-induced concrete deterioration. Given the current condition of the deck and projected future deterioration, viable repair and preservation options were identified to extend the life of the deck by at least 50 years.

A Siva Corrosion Services professional uses steel samples to measure the effects of damage from microbiological activity.



Industry Evolving

In addition to testing and evaluating, SCS engages in service-life design. This service has become more common in recent years for certain bridge types. SCS has been involved in design-build projects where they developed corrosion protection for 100-year service life using probabilistic modeling. "The industry is evolving, and our materials are evolving, but we need to understand what we did decades ago to really comprehend what is changing," say Venugopalan. "We constantly monitor FHWA [Federal Highway Administration] research and how it is progressing. It is critical to know what is going on in the concrete industry, what new materials or methods are being introduced, in order to determine how to effectively extend the service life of structures."

Impact of Corrosion Engineering

Corrosion engineering is crucial in the construction industry to help manage and prevent the degradation of materials, ensuring the longevity and safety of infrastructure. Reliable and safe infrastructure is needed for transporting goods and services, providing emergency services, and strengthening national defenses. Repairing structures instead of replacing them can reduce costs, lower carbon dioxide (CO_2) emissions, and minimize environmental risks. The efforts by SCS to develop plans for 100-year service life achieves all these benefits by incorporating material sustainability at the design stage. As Venugopalan notes, "When we incorporate material design at the design stage for new bridges or extend the service life of existing bridges, we significantly reduce CO_2 footprint by not replacing the structure prematurely."

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PROJECT

Sound Transit Light Rail

Site challenges provide an opportunity for concrete segmental construction solutions

by Angela Tremblay

Since its founding in 1993, the Central Puget Sound Regional Transit Authority, more commonly known as Sound Transit, has been committed to expanding transit options in the Washington state counties of Pierce, King, and Snohomish. The agency has expanded bus, commuter train, and light-rail service throughout the region with a focus on providing affordable access and environmentally friendly transit options. As part of Sound Transit's ongoing light-rail expansion, planning for the Federal Way Link Extension (FWLE) project began in 2012, followed by voter approval in 2016, and the final route selection in 2017. FWLE provides an 8-mile extension from Angle Lake to Federal Way. The project is Sound Transit's largest design-build contract to date, and, when it is completed in 2026, the extension will connect Seattle and Seattle-Tacoma International Airport to the communities of Des Moines, Kent, and Federal Way, attracting 18,000 to 23,000 passengers daily, up to 8000 of whom would be new to the system.[1]

The FWLE route has at-grade and elevated sections, as well as three new stations. Most structures along the route are simple-span precast, prestressed concrete girder bridges of varying span lengths (130 to 190 ft per span, up to 18 spans) supported by concrete columns and caps. The decks use various concrete solutions, including precast concrete deck panels, cast-in-place (CIP) decks, and CIP concrete rail plinths. The three stations are elevated platforms constructed using precast, prestressed concrete girders. This article describes the long-span CIP concrete segmental bridge, Structure C, which offered the best solution for a section of the route that passes through an extremely challenging site.

Difficult Site Conditions

The design-build team was selected in 2019, and design began based on the planning-level design and the draft environmental impact statement; subsurface exploration was limited at that point due to difficult access. As the design progressed for the elevated section in Kent, Wash., the team gathered more than 30 soil borings and discovered that site conditions for the Structure C bridges near South 259th Place and Interstate 5 (I-5) were considerably more challenging than the initial designs assumed. The planned route had unanticipated weak soil conditions, including undocumented landfill debris and wetland soils, as well as a large liquefiable layer. These findings ultimately led to the redesign of the bridges.

The Federal Way Link Extension Project provides an 8-mile extension of the light-rail system from Angle Lake to Federal Way, Wash. The project is Sound Transit's largest design-build contract to date. Figure: Sound Transit.

profile

FEDERAL WAY LINK EXTENSION STRUCTURE C / KENT, WASHINGTON

BRIDGE DESIGN ENGINEER: Kiewit Engineering Group, Denver, Colo.

OTHER CONSULTANTS: Construction engineer: McNary Bergeron & Johannesen, Denver, Colo.; independent reviewer and shop drawings: Systra/IBT, San Diego, Calif.

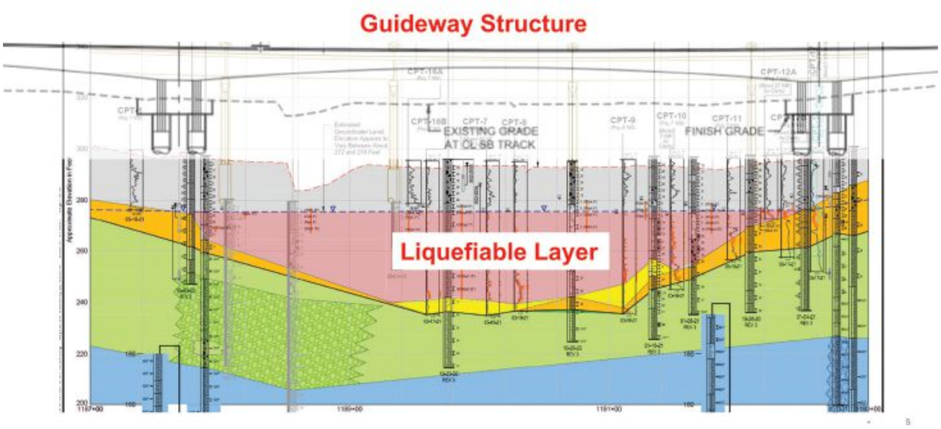
PRIME CONTRACTOR: Kiewit Infrastructure West Co., Federal Way, Wash.

CONCRETE SUPPLIER: Heidelberg Materials (formerly Corliss), Federal Way, Wash.

POST-TENSIONING CONTRACTOR: Schwager Davis Inc., San Jose, Calif.

The cast-in-place, concrete segmental Structure C allowed construction from above with the balanced-cantilever method and accommodated the long spans needed to avoid problematic soil conditions and mitigate impacts to wetlands. Photo: Kiewit.

The original design for Structure C used relatively short, 135-ft simple-span bridge units to support the light-rail extension. In November 2020, the design-build team notified Sound Transit that additional geotechnical investigations and analysis indicated that site conditions could not support the structure as originally designed. The team had analyzed a 2500-year design seismic event, as required by the *Sound Transit Design Criteria Manual*,[2] and determined that soil in the area would have the potential to liquefy, causing the adjacent slope to fail and slide into the FWLE guideway. These findings resulted in a substantial change order and redesign effort.



The main span of Structure C is located to clear a soil layer consisting of uncompacted fill. In an earthquake, this layer can liquefy and cause the embankment to slide downhill. Figure: Kiewit.

Other project-site concerns included McSorley Creek and wetlands, and the proximity and height of the adjacent I-5 roadway embankment. The design-build team proposed a rigid-pier bridge solution with a 190-ft-span precast concrete, post-tensioned girder superstructure designed to slide along seismic bearings, thereby keeping the system operable during the design seismic event, including a potential upslope landslide event. As the rigid-pier bridge design was being finalized, construction of a temporary access road commenced. According to the project history from Sound Transit, during the access road installation, the site experienced localized slope failures, and so the team designed and installed timber piles to stabilize the slope.[3] In July 2022, while the ground-improvement measure was being installed, a landslide briefly closed a southbound lane of I-5 and stopped all work at the site except for emergency slide repairs to stabilize the slope. While the design-build team reevaluated the bridge solutions, permanent slope repairs were performed in accordance with Washington State Department of Transportation (WSDOT) standards.

CENTRAL PUGET SOUND REGIONAL TRANSIT AUTHORITY, A PUBLIC CORPORATION ACTING UNDER THE SERVICE NAME OF SOUND TRANSIT, OWNER

OTHER MATERIAL SUPPLIERS: Reinforcing bar supply and installation: CMC, Auburn, Wash., and Nucor Rebar Fabrication, Tacoma, Wash.; disk bearings and expansion joints: RJ Watson, Alden, N.Y.; shock transmission units: Taylor Devices, North Tonawanda, N.Y.; drilled shafts: Michels Corp, Renton, Wash.

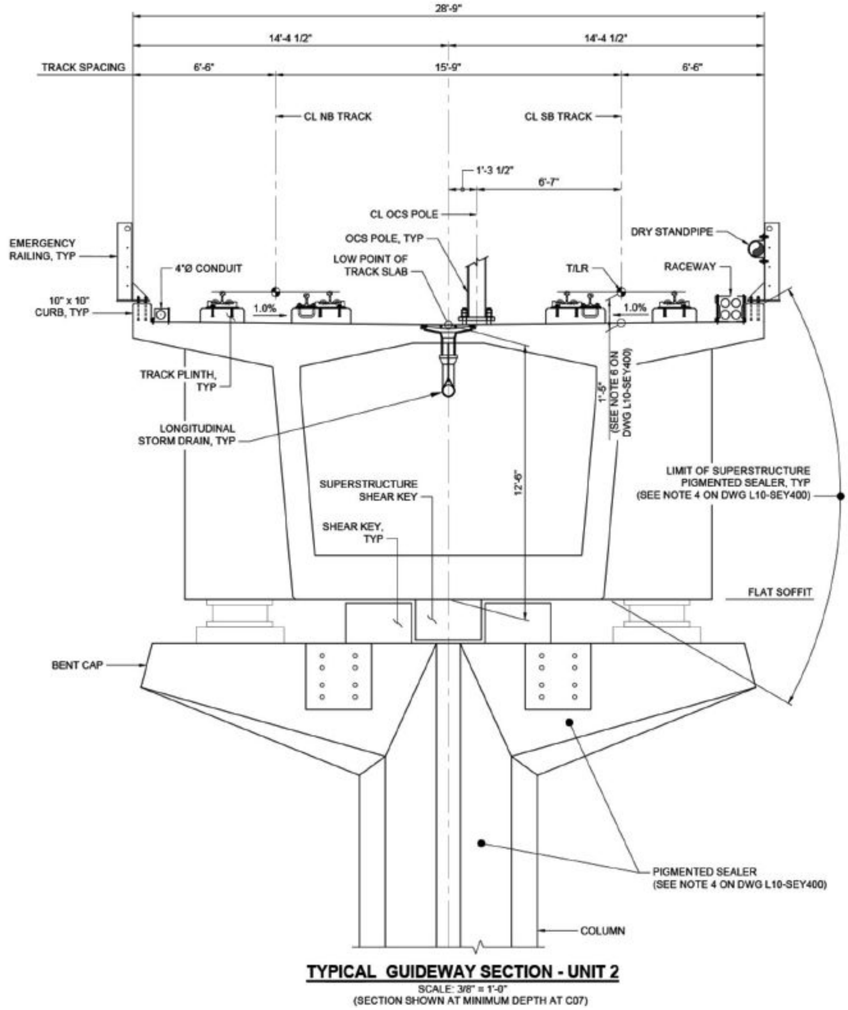
BRIDGE DESCRIPTION: 1097-ft 7-in.-long cast-in-place concrete segmental transit guideway bridge with 500-ft main span and 300-ft back spans

STRUCTURAL COMPONENTS: Cast-in-place concrete segmental superstructure with 62 segments. Two sections of the back spans were cast on falsework at either end of the structure for a length of about 50 ft. Cast-in-place concrete single-column piers supported by 12-ft-diameter, 110-ft-deep drilled shafts.

The 1100-ft-long cast-in-place concrete segmental box-girder Structure C is built adjacent to the heavily traveled Interstate 5, which is located on a 50-ft-high embankment. Photo: Kiewit.

Final Design

After some iterations and coordination with Sound Transit, the design of Structure C Unit 2 was finalized. This design featured an approximately 1100-ft-long CIP concrete segmental box-girder structure that would be built adjacent to the existing, heavily traveled I-5, which is located on a 50-ft-high embankment. The team chose to construct the CIP concrete segmental structure from above, using the balanced-cantilever method, to accommodate long spans, avoid problematic soil conditions, and mitigate the project's impact on the wetlands.



The constant-width, typical box-girder section of the concrete segmental bridge carries two light-rail tracks, emergency railings, and acoustic panels. Figure: Kiewit.

The Structure C bridge guideway design was performed in accordance with the *Sound Transit Design Criteria Manual*,[2] supplemented with the WSDOT *Bridge Design Manual*[4] and the seventh edition of the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*.[5] Sound Transit requires that structures be designed for a 2500-year maximum design earthquake to promote resiliency in their system and ensure that operations can resume quickly after lesser seismic events. In addition, the design criteria for this project included concrete durability, corrosion control, and sustainability requirements to provide a 100-year design life, which was investigated and then documented in a project-wide durability report.

Structure C consists of two units. Unit 1 is the approach spans that comprise three spans of precast, prestressed concrete wide-flange girders for a structure length of 370 ft 2 in. Unit 2 delivers the innovative solution required to span the weak soils and sensitive environment along the FWLE route. Unit 2 is a three-span 1097-ft 7-in.-long,

AESTHETICS **COMMENTARY**

by Frederick Gottemoeller

"'Improving aesthetics always adds cost!' How many times have you heard that one?" About a year ago, I began a commentary on the Honolulu Authority for Rapid Transportation's transit system with those sentences (see the Winter 2024 issue of *ASPIRE*®). Well, the Federal Way Link Extension is another example where improved aesthetics came along with reduced cost. This example is also a transit project, but this time, the impetus to investigate an innovative approach was a wretched foundation situation.

When an innovative solution competes against a conventional one, the innovative solution is often penalized because the design and construction team are not familiar with it and therefore feel the need to tack on unnecessarily large cost margins for uncertainties and "contingencies." On this project, the design-build team recognized that danger, so they brought aboard experts in segmental concrete design and construction early in the project's life. The results were money-saving innovations such as dual travelers and integrated shop drawings.

The solution derived for this project addresses the difficult soil conditions and costs less than competing solutions. Also, as a bonus, the design gives the guideway a sleeker and more streamlined shape, which is more interesting because it reflects the forces on it. As a result, the structure is a more welcome component of the communities through which it passes.

As I said a year ago, "Such 'twofers' are available more often than most designers imagine. They should be the goal of all engineering refinement."

Cast-in-place concrete box-girder segments vary in height from 12.5 at the midspan to 25 ft at the piers and typically have a width of 28 ft 9 in. The bottom-slab post-tensioning ducts and web shear keys are visible. Photo: Kiewit.

Two sections of the back spans were cast on falsework at either end of the structure for a length of about 50 ft, while the remainder of the segmental structure was constructed using form travelers and the balanced-cantilever construction method. Photo: Kiewit.

Integrated shop drawings and three-dimensional modeling allowed the design team to spot conflicts before they happened, which was critical to the successful installation of the many reinforcing elements required in the superstructure. Photo: Kiewit.

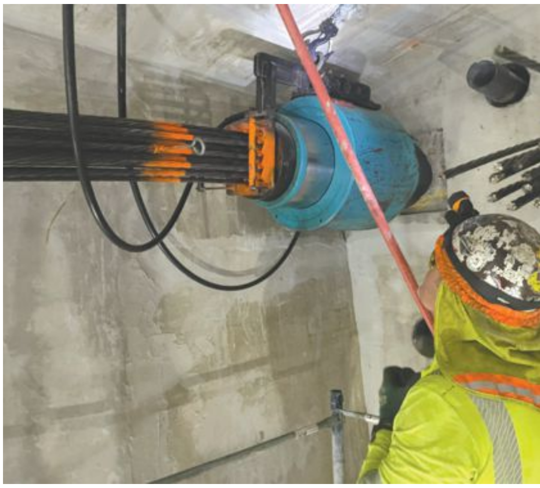
A view inside the box girder showing the internal continuity tendons before tensioning and the closure formwork in the background. The internal continuity tendons are ready to tension as soon as the closure pour (formwork in background) has reached the required strength. Photo: Kiewit.

CIP concrete segmental box-girder bridge consisting a main span of 500 ft and back spans of 300 ft and 297 ft 7 in. CIP concrete segments vary in height from 12.5 to 25 ft and have a constant width of 28 ft 9 in. The designers specified segment lengths of 11.5 ft for deeper sections and 15.5 ft for shallower sections based on the capacity of the contractor's form travelers. The typical section carries two light-rail tracks, emergency railings, and acoustic panels.

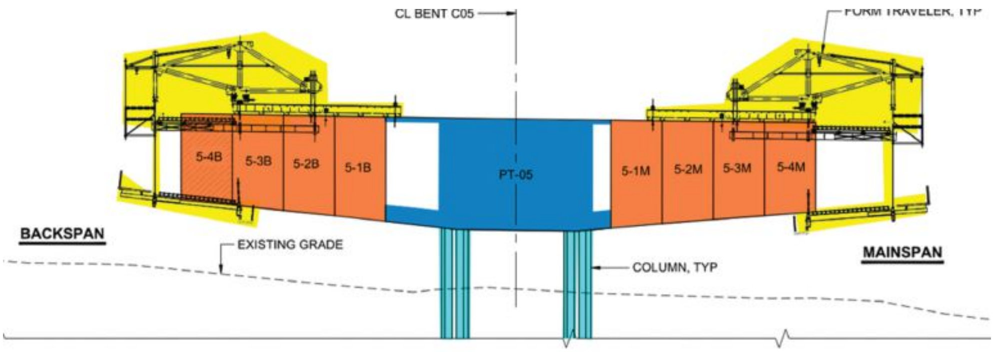
Lock-up devices (or shock transmission units) are provided to engage the two end columns of the Unit 2 structure during a seismic event. This type of system provides a temporary rigid link between the bridge superstructure and substructure under seismic or other fast-acting loads, while also allowing the bridge to expand or contract over time due to temperature changes, creep, or shrinkage.

Technical details of the bridge superstructure include concrete strengths for the CIP box girders of 6 ksi at 28 days and 8 ksi at 90 days. Mild reinforcing steel is uncoated ASTM A706,[6] Grade 60 or 80 ksi. Water-filled cooling tubes were used in the larger or thicker concrete sections, such as the thicker bottom slabs and diaphragms, to cool the concrete as it cured. During curing, the concrete temperatures could have reached well above 200°F; using the cooling tubes ensured that the temperatures stayed below 160°F, which is necessary to mitigate cracking.

Uncoated post-tensioning (PT) bars conforming to ASTM A722[7] Grade 150 standards were tensioned to a specified force and used to attach the shock transmission unit brackets to the bridge. The internal and external PT strands are 0.6-in.-diameter, seven-wire, low-relaxation steel conforming to ASTM A416[8] Grade 270 standards. The three-span structure has three types of PT, all running in the longitudinal direction: 128 bridge top-deck cantilever tendons with 10- or 11-strands per tendon, 26 bottom-slab tendons using 18- to 24-strands per tendon, and 12 internal (within the void of the box girder) continuity tendons that have 19 strands per tendon. The PT ducts—white plastic ducts for tendons encased in concrete or black high-density-polyethylene ducts for internal tendons—were grouted to provide a PL-2 tendon protection level, as described in the *Specification for Multistrand and Grouted Post-Tensioning* published by the Post-Tensioning Institute and American Segmental Bridge Institute.[9]



Tensioning the strands of a post-tensioning (PT) tendon. One advantage of the design-build delivery for this project was that the post-tensioning (PT) material supplier was chosen and engaged during the initial design phase. They worked with the design team as the bridge superstructure design was advanced. Photo: Kiewit.



Segments are being cast on both sides of one of the piers. Using four identical form travelers allowed segments to be cast on both sides of both cantilevers concurrently. Photo: Kiewit.

The Unit 1 superstructure is supported by single-column piers with tapered bent caps and a CIP concrete abutment—all with concrete design strengths of 4 ksi. The Unit 2 end supports also use single-column piers, while the interior/main-span supports use two columns with a pier table. For the two-column supports with the pier table, the design requires 6-ksi concrete, which can be achieved any time before the columns are loaded with the superstructure. The design team chose to list a 56-day time frame to reach 6-ksi strength in the plans as a matter of convenience. For the pier tables, the design team added the requirement of 7-ksi concrete strength at 90 days to accommodate stress checks on the structure that included the effects of live loads. This level of detail in the design took advantage of the predicted concrete strength at the time of operation. The foundation support for each pier column is provided by a single 110-ft-long, 12-ft-diameter drilled shaft for each column. The drilled shafts have a 5-ksi design concrete strength.

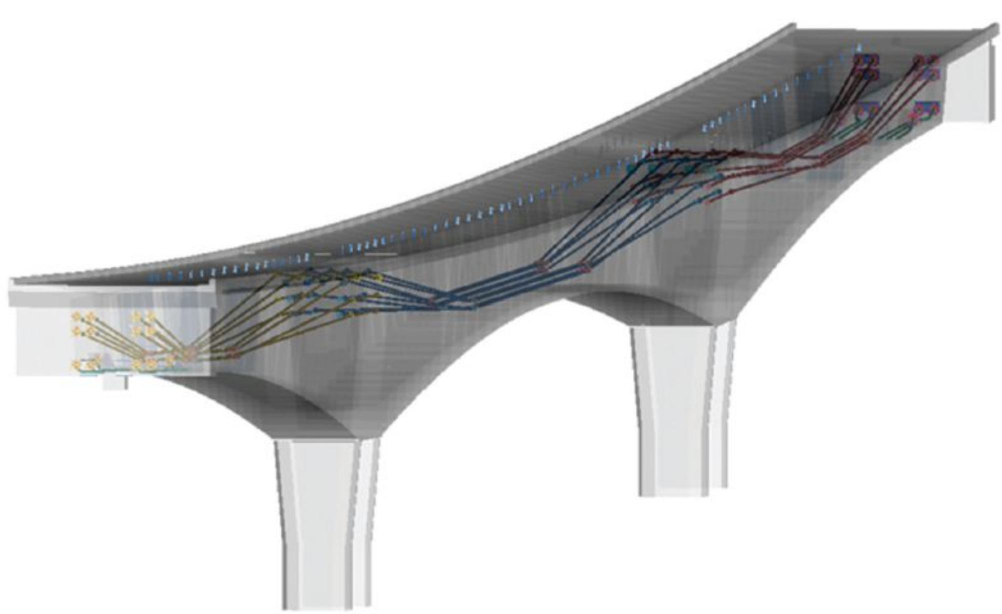
Construction Solutions

With the design-build delivery method, bridge engineers, construction engineers, suppliers, and contractors were able to work from the start as an integrated team to problem-solve and choose solutions that suited the project as well as the expertise of the team. For example, unlike many projects that perform the permanent concrete segmental bridge design with a generic PT system, the PT material supplier was chosen and consulted during the initial design phase and worked with the design team as the superstructure design was advanced. Similarly, the design and construction teams decided early in the project to design the pier table to be just long enough to fit two travelers back to back on a single pair of shared rails. This decision helped jumpstart the team's early designs for the pier table falsework and allowed the use of identical form travelers to cast all cantilevers concurrently.

One of the most significant efforts that involved both design and construction—producing integrated shop drawings for the concrete segmental bridge—started early in the project. This integration (inclusion of all embedded items) was not required by the project specifications, but the team elected to produce and submit the integrated shop drawings to limit interferences and thereby facilitate construction success and schedule certainty.

Using a previous project as a template, the design and construction team, including the reinforcing steel and PT subcontractors, brainstormed what needed to be included in the integrated shop drawings. Each segment was

A three-dimensional (3-D) model of the three-span concrete segmental structure showing the post-tensioning tendons. The 3-D model was used to create the two-dimensional integrated shop drawings for each segment. Figure: Kiewit.



modeled in three dimensions with a precision of ⅛ in. or better in size and location. All conflicts were noted and sent to the engineer of record for review and resolution. After the three-dimensional model was complete, a two-dimensional shop-drawing package that included reinforcing bar lists, PT data, and all other information for the specific segment was assembled. The 68 packages for the pier tables and box-girder segments were submitted to Sound Transit for approval before construction and will be used as the as-built drawings.

A key to the success of this endeavor was starting early—the team was assembled and started modeling, drafting, and reviewing before the bridge design was finalized. It took just over a year to complete the packages and stay ahead of construction.

Collaborative Success

This was a very challenging project, but selection of the long-span concrete segmental bridge and the integration of the experienced design and construction teams led to completion of the Structure C scope months early and within budget. The design-build delivery method for the FWLE helped the team make the creative adjustments needed to develop solutions for this project. Sound Transit and the design-build team collaborated in dealing with changing parameters and site conditions, and their collective problem-solving efforts led to a final design that meets the needs of Sound Transit and the communities that it serves. In recognition of the project's intentional focus on system resilience and sustainability, the project was awarded the Envision Platinum designation from the Institute for Sustainable Infrastructure.

Structure C was completed in October 2024, and Sound Transit will conduct systems integration testing along the extension next. The FWLE is scheduled to open to the public in 2026, and when it opens, it will provide the region with safe, reliable light-rail transit for years to come.

Structure C was completed in October 2024 and Sound Transit plans to begin systems integration testing along the extension as early as the end of 2024. Photo: Kiewit.

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A Crack Is Not a Crack: Bridge Deck Cracking

by Dr. Oguzhan Bayrak, University of Texas at Austin

Bridge deck cracking has been a problem for bridge owners for decades, and it continues to be a significant maintenance challenge. Deck replacement efforts are a significant expense for most bridge owners during the service lives of bridges. This article discusses key issues that affect bridge deck performance and is intended to demystify the art and science behind deck design and construction. Concrete bridge deck cracking can be influenced by the structural design of decks and the associated reinforcing bar detailing; deck behavior at service and ultimate limit states; the interplay between reinforcing bar details; concrete mixture design; and volume changes that the deck concrete experiences. These influences will be discussed in this article in an effort to tackle the variety of topics affecting bridge decks. Within that context, let us first focus our attention on the most common bridges and deck types. In the vast majority of cases, decks are made from reinforced concrete and supported on beams. To understand the root causes of cracking for this type of bridge deck, let us start with the design process and then discuss concrete mixture proportions, construction, curing, and maintenance processes.

Design of Bridge Decks

Section 9 of the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[1] is devoted to decks and deck systems. It provides guidance on empirical deck design techniques as well as deck design methods that approach the analysis of deck behavior and the design process in a more refined manner. When bridge owners have their own bridge design manuals, those manuals often reference this section of the AASHTO LRFD specifications but include some variations that reflect the local jurisdiction's preferences. Those preferences tend to be rooted in decades of observations of deck performance in the owner's region. For example, requirements for concrete cover to the top layer of reinforcement in the deck will vary depending on the exposure conditions and, in some cases, the use of corrosion-resistant reinforcement.

Lane loads and truck loads applied on the decks are primarily distributed to the supporting beams in the transverse direction relative to the direction of traffic. In this context, we can envision a design strip that runs perpendicular to the beam lines in a typical, traditional deck design method. This design strip has the greatest level of flexural stiffness, attracting the great majority of the design loads. The percentage of the reinforcement in this "primary" direction is typically greater than that in the longitudinal direction. As the spacing of the beams that support the deck increases, so do the bending moments created by the permanent and transient loads. With increasing bending moments, the need to use larger quantities of transverse deck reinforcement increases. How deck reinforcement is detailed relates to how service loads are carried and, ultimately, to the additive effects of service load–induced stresses to those stemming from restrained shrinkage effects (to be discussed in later sections of this article). In some jurisdictions, the quantity of transverse reinforcement in each deck design is varied accordingly. In other jurisdictions, given the high volume of construction and standardized girder spacings and superstructure geometries, the quantity of transverse reinforcement is kept constant to simplify both the design process and the inspection process during construction.

The deck reinforcement that is placed in the longitudinal direction is intended to control the widths of transverse cracks, if and when they form. In this sense, the design requirements are typically rooted in the crack-control reinforcement requirements. My first article in this series provides the technical basis for the crack-control requirements by providing the first principles–based explanation of the cracking phenomenon and the stresses that develop in reinforcing bars crossing those cracks (see the Summer 2024 issue of *ASPIRE*®). Spacing, bar size, and clear cover requirements all influence the efficacy of the crack-control reinforcement, as well as the widths of any cracks that may form.

Deck cracking is a complex topic. Transverse cracking of the decks is a phenomenon that has been observed in many bridges and in many jurisdictions. All Photos: Dr. Oguzhan Bayrak.

The deck reinforcement details ultimately influence the widths and lengths of the cracks that form under combined effects of restrained concrete shrinkage, as well as the quantity of cracks formed. While the primary reinforcement details are rooted in flexural design considerations, flexural effects rarely control the failure of decks (ultimate conditions). That is to say, findings from numerous bridge deck testing projects have led to the conclusion that punching shear is the most common failure mode observed in bridge decks loaded to ultimate in laboratory conditions. The primary variables that influence the strength of the punching shear cone that forms around patch-loaded areas (that is, a 10 × 20 in. truck tire area on the deck) include the deck thickness (commonly 8 to 9 in.), the percentage of transverse and longitudinal reinforcement, and the compressive strength of concrete (as a surrogate indicator for the tensile strength of concrete). The typical failure loads observed in testing are between 5 and 7.5 times the design truck or tandem loads of the AASHTO LRFD specifications. This type of safety margin over the design loads is very high. In fact, if we were to increase the live loads acting on a bridge 5 to 7.5 times, we would run into strength issues in our girder designs as well as in many aspects of bridge substructure designs. Such great levels of reserve capacities are unusual. Because we do not expect to see these types of loads on our bridges, we do not typically design for safety margins of that magnitude.

As an aside, the design of the overhang portion of the decks is typically governed by the loads transferred to the decks by the bridge rails under a vehicle- or truck-impact loading scenario; these loads deserve their own separate design considerations. All aspects of the aforementioned design and structural-behavior items point to the fact that loading and the need to safely support the HL-93 loads constitute only one aspect of deck design. The service performance of the decks and additional considerations that influence the service performance are at least as important, if not more important, as the structural aspect of deck design. Simply stated, we need to consider deck design, durability, and performance in a holistic manner. This type of comprehensive evaluation is particularly important in identifying the root cause of the deck cracks that form during the service life of a bridge deck.

An example of longitudinal cracking in a concrete bridge deck.

The design of complex bridges and post-tensioned decks involves additional considerations. With that said, and broadly speaking, the compressive stresses applied by the post-tensioning systems to the deck concrete are beneficial as a means to extend the service lives of bridge decks. National Bridge Inventory data and additional surveys conducted by the American Segmental Bridge Institute serve as testaments to the superior deck performance observed in post-tensioned bridges.[2,3]

Concrete Mixture Proportions

The quality of the concrete greatly influences bridge deck performance and can help control deck cracking. The leading cause of deck cracking is the volume changes experienced by the deck when it is restrained by the primary load-carrying elements that support the deck. (For in-depth discussion of restrained shrinkage cracking, see my second article in this series, in the Fall 2024 issue of *ASPIRE*.) To reduce shrinkage strains, it is important when designing the concrete mixture proportions to reduce the paste content for a given volume of aggregates. Well-graded aggregates help us achieve this goal and create optimal concrete mixtures. A good gradation of the aggregates helps minimize the volume that must be filled with a cementitious paste, thereby reducing the shrinkage potential.

We must do whatever we can do to minimize all forms of shrinkage (autogenous, drying, and plastic). To that end, an effective strategy may be to reduce the water content and carefully use admixtures to improve workability. When applying this strategy, we must give due consideration to all effects of those admixtures to avoid unintended consequences and adverse effects.

The type of coarse aggregate greatly influences the modulus of elasticity of concrete. The strongest and stiffest aggregates may not be the most effective to reduce deck cracking. The use of more-compliant aggregates (that is, aggregates with a lower modulus of elasticity) such as crushed limestone has been found to effectively reduce deck cracking. Because stiff aggregates will concentrate the deformations in the cement paste that binds the aggregates, it stands to reason that the use of stiff aggregates will challenge the paste/matrix to a greater extent and may lead to conditions that are conducive to early deck cracking.

Concrete mixtures with low coefficients of thermal expansion typically perform better than those that have higher coefficients of thermal expansion. The quantification of thermal expansion properties of concretes made with locally available aggregates, primarily coarse aggregates, can be burdensome; however, in some cases, testing may be necessary to identify these properties when developing best practices. After the placement of fresh concrete, heat is generated during the hydration process. Sometimes, the hydrating cement in concrete heats up too much (perhaps due to the environment in which concrete is placed, the type of cement that is used in the concrete mixture, and insulation provided during its curing). In such cases, as the concrete cools down, the restraint provided by the primary load-carrying system may lead to thermal cracking of the decks. Thermal strains experienced by the deck during early stages of its life (construction) and over the long run will be minimized if concrete mixtures are designed to have lower coefficients of thermal expansion.

Formwork

In addition to its primary function of shaping the deck concrete to a desired geometry, formwork plays an important role in maintaining or dissipating the heat that builds up during the hydration process. The use of wooden forms is the traditional option available to contractors for forming concrete. In many jurisdictions, common alternatives to wooden forms include permanent metal deck forms (PMDFs) and stay-in-place, partial-depth precast, pretensioned concrete deck panels. Whereas wooden forms and PMDFs can be used to construct full-depth cast-in-place (CIP) concrete decks, the use of partial-depth, pretensioned concrete deck panels can reduce the volume of CIP concrete considerably. The reduction may be 40% to 60%, depending on the deck thickness (commonly 8 to 9 in.) and panel thickness (commonly 3.5 to 4.5 in.).

The use of partial-depth deck panels as it relates to deck cracking deserves additional discussion. These panels are fabricated to industry standards in precast concrete plants and shipped to construction sites. There are guidelines and specifications for their production, storage, and shipping. (See the FHWA article in the Winter 2024 issue of *ASPIRE* for discussion of current practices for partial-depth precast concrete deck panels.) When considering deck cracking and overall deck performance, the age of the pretensioned concrete deck panels is important. Based on the considerable experience of the states that use these deck panels, it is recommended to allow time for creep and shrinkage to take place within the pretensioned panels before the panels are shipped to the jobsite and CIP concrete is placed. After the volume changes experienced by the partial-depth deck panels have stabilized, they can be transported and properly installed on the beams. At this point, the panels provide a safe construction environment for the installation of the top mat reinforcement and eventually CIP concrete placement. In the past, there were cases in which "green" panels were shipped to jobsites immediately after prestress transfer at the fabrication plant (that is, within the first few days after panel fabrication) and installed on the bridge beams. The use of such "green" panels led to the formation of longitudinal cracks that follow the beam lines on a bridge deck. The lessons learned from such practices have led to specifications that indicate the minimum age that deck panels must reach before they are shipped.

To establish ideal bonding between the panels and the CIP concrete, it is important to wet the precast concrete panels to saturated surface-dry condition before concrete placement. Additionally, it is necessary to provide sufficient room for the CIP concrete to flow underneath the panel edges so that the panels are evenly and firmly supported on the top flanges of the beams.

The experiences of states that employ this technology indicate that when state-of-the-art production, shipping, storage, and installation guidelines are followed, the incidence and severity of deck cracking are comparable to what is observed in full-depth CIP concrete decks. In other words, the use of partial-depth pretensioned concrete deck panels does not worsen deck cracking or compromise long-term deck performance. Therefore, decision-makers can focus on other issues such as structural performance, worker safety, project timelines, and project costs, where the use of partial-depth deck panels can offer significant benefits.

Deck Concrete Placement

The sequence of deck concrete placement can be important for longer-span bridges and, in particular, for continuous spans. Based primarily on their past experiences and structural analysis of these statically indeterminate systems, owners often define the placement sequence for simple and continuous spans with the ultimate objective of minimizing, if not eliminating, the tensile stresses caused by the deck weight—in addition to consideration of stability of the incomplete structure, beam deflections, and stresses experienced by the beams during construction. To minimize transverse deck cracking, it is important to detail construction joints and expansion joints correctly and select appropriate locations for them. If the deck has a cross slope or a significant elevation change between the two ends of a span, the deck geometry must also be taken into account when sequencing concrete placement. Starting at low points and "climbing up" during concrete placement will usually improve results and minimize deck cracking.

To minimize deck cracking, maximize the service lives of decks, and reduce maintenance expenditures, it is important to develop good practices for placing concrete in hot or cold weather and to take into account environmental exposure conditions at the jobsite. For example, in Texas, when decks are placed during summer months, the work may occur at night to take advantage of cooler nighttime temperatures. The ambient air temperature, the temperature of the fresh concrete at placement, the type of deck-forming system (wooden forms, PMDFs, concrete partial-depth deck panels), and the use of blankets, burlap, plastic sheets, or other materials that affect concrete moisture will all contribute to whether the heat of hydration dissipates or is trapped; these variables all affect the maximum concrete temperatures reached during curing. In addition, in windy conditions, fresh concrete dries faster during placement and curing. If we think about the massive exposed area of concrete in relation to the volume of deck concrete, we can better appreciate how quickly loss of moisture can occur in windy conditions. Rapid drying of such expansive concrete surfaces is detrimental for both short- and long-term deck performance because it will lead to significant increases in deck cracking and improper or deficient hydration of cement. Precipitation during concrete placement and curing will also affect the deck concrete's quality and curing temperatures, thereby impacting the durability of the deck.

Curing of Decks

Methods to keep concrete moist or wet during its early ages include the use of moisture barriers (for example, plastic sheets) or moisture retainers (for example, burlap and plastic sheets), wetting of the curing concrete (for example, by spraying), and the application of curing compounds and liquid membranes. The attention paid to all relevant details during the curing process promotes a favorable hydration environment for the concrete and minimizes shrinkage of concrete. That effort will pay dividends in minimizing cracking in decks and improving the quality of deck concrete.

Deck Maintenance

Like all things, bridge decks require routine maintenance. Installation of proper deck drains, grading the deck to eliminate ponding of water, and keeping the drains clean and functional are all important for deck durability and the safety of the traveling public. Vehicles traveling over bridges with clogged drains and ponding water may hydroplane, posing a safety concern. In addition, accumulated water and the contaminants it may contain can accelerate the degradation of decks. With regard to deck cracks and corrosion, it may make sense to routinely wash bridge decks, as is done in some northern states.

Conclusion

A crack-free deck seems to be a difficult, if not impossible, goal to achieve. Therefore, it is prudent to deploy effective strategies to minimize the number of deck cracks and limit the widths of those cracks when (not if) they form, and the designer and owner should understand the implications of deck cracking for the structure's integrity and service life. Undoubtedly, the presence of deck cracks can facilitate the delivery of contaminants such as chlorides from deicing salts to the deck reinforcement and accelerate the corrosion of the deck reinforcement. In regions where contamination from deicing salts is likely, corrosion-resistant reinforcement (metallic or nonmetallic) can be used to mitigate corrosion concerns and extend the service lives of bridge decks. In coastal regions, a greater concern than the corrosion of deck reinforcement may be corrosion of reinforcement in substructure components, which is a topic beyond the scope of this article. The implications of deck cracks can be quite different in different jurisdictions, depending on the environmental exposure conditions as well as the selected type of deck reinforcement.

Deck cracking is a complex topic. Transverse cracking of the decks is a phenomenon that has been observed in many bridges and in many jurisdictions. This type of cracking has been observed in full-depth CIP concrete decks as well as decks with precast, pretensioned concrete deck panels. Good concrete mixture designs, appropriate construction practices during concrete placement and curing, and ongoing deck maintenance are fundamental aspects of bridge deck stewardship that will extend the service lives of bridges. The different experiences of various jurisdictions translate into regional variations in best practices for deck design, construction, and maintenance. The good news is that the lessons learned by all bridge professionals contribute to better-performing decks with better details and longer service lives.

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Flexural Strength of Prestressed Concrete Bridge Beams with Composite Decks

by Richard Brice, Washington State Department of Transportation, and Dr. Maher Tadros, e.Construct USA LLC

This article, the second of a two-part series on strain compatibility analysis, compares the methods for estimating the flexural strength of prestressed concrete bridge beams with composite decks specified in the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*.[1] Part 1 of the series, published in the Fall 2024 issue of *ASPIRE*®, introduced strain compatibility concepts and presented an example of strain compatibility analysis.

The AASHTO LRFD specifications provide two methods for estimating the flexural strength of composite prestressed concrete beams: a simplified approach based on the well-known rectangular stress distribution and the strain compatibility approach. The simplified approach gives reasonably accurate estimates for reinforced and prestressed concrete structural components, and when the concrete strength of all compression elements is similar for composite components. However, cast-in-place slabs on precast, prestressed concrete beam sections typically have different concrete strengths for the slab and beam. Using the simplified approach for these types of sections, the estimated flexural capacity becomes progressively more conservative as the concrete strength of the beam increases relative to that of the slab concrete and with increasing levels of tension reinforcement. This strength differential can lead to a situation where the strength limit state governs the design over the service limit state, which typically controls, and can affect load ratings.

The conservatism of the simplified approach can be attributed to neglecting the contribution of the beam top flange and its higher concrete strength to the resultant compression force. With increasing levels of beam reinforcement, the depth of the compression block increases to balance the tension force in the reinforcement, and the internal moment arm between the compression and tension resultant force decreases, leading to a lower estimate of flexural capacity than that calculated by the strain compatibility method. This reduced estimate of flexural capacity can, in turn, lead to sections that are predicted to be in the compression-controlled or transition region with flexural resistance factors ϕ less than 1.0. In these cases, strain compatibility analysis can provide a more accurate estimate of flexural resistance.

Simplified Approach

The simplified approach, based on a rectangular stress distribution and shown on the idealized beam section in **Fig. 1**, is well known in the engineering community. The basic premise is that the compression force in concrete can be obtained by assuming a rectangular stress distribution of magnitude $\alpha_1 f'_c$ acting over a portion ($a = \beta_1 c$) of the distance c between the extreme compression fiber and the neutral axis. The equations for estimating flexural capacity using this method are given in Article 5.6.2.2 of the AASHTO LRFD specifications with design compressive strengths for normalweight concrete up to 15.0 ksi and for lightweight concrete up to 10.0 ksi.

There are some important limitations to the simplified approach. In the AASHTO LRFD specifications, Commentary C5.6.2.2 and Article 5.6.3.2.6 specify that when the compression stress field includes elements of the cross section with different strengths of concrete, the lower-strength concrete is to be used in the equations of Articles 5.6.2 and

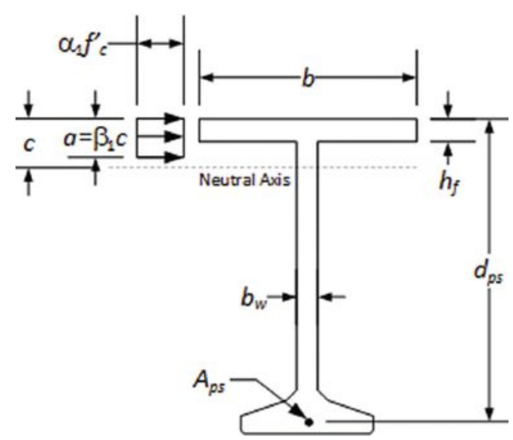

Figure 1. Idealized composite beam section for the AASHTO LRFD simplified, rectangular stress-block approach.
All Figures: Richard Brice.

5.6.3 for estimating flexural strength and other related parameters. A common misconception is that the lower-strength deck concrete can be transformed into an equivalent width of beam concrete using the modular ratio of the materials, as is done at the service limit state. This assumption is not valid at the strength limit state because strains are no longer in the linear-elastic range. Assumptions regarding the location and distribution of steel reinforcement are highlighted in Commentary C5.6.3.1.1. To use the equations, it must be reasonable to lump all the prestressing steel at a single location. However, it may not be reasonable to lump reinforcement spread vertically through the section, such as draped strands near end regions and post-tensioning tendons, into a single location. A similar assumption is made for nonprestressed tensile reinforcement. In sections where a significant number of prestressing elements are on the compression side of the neutral axis, C5.6.3.1.1 states that "it is more appropriate to use a method based on the conditions of equilibrium and strain compatibility as indicated in Article 5.6.2.1."

Strain Compatibility Approach

Article 5.6.3.2.5 of the AASHTO LRFD specifications recognizes the strain compatibility approach as an alternative to the simplified approach. Strain compatibility analysis involves subdividing the beam cross section into discrete parts or layers (as illustrated in Part 1 of this series), estimating the strain at the centroid of each layer based on the assumption of plane sections, evaluating the stress using stress-strain relationships, and integrating the stress field to determine the flexural strength of the section. This analysis is iterative until conditions of equilibrium are satisfied. The strain compatibility method can accurately analyze nonrectangular sections, concrete elements of varying strengths, and reinforcement at any position in the section.

Reinforcement strain limits can also be considered. The minimum properties of reinforcement are defined by their respective ASTM specifications. ASTM A416[2] defines the minimum properties for low-relaxation, seven-wire steel strand for prestressed concrete. The minimum elongation is specified as not less than 3.5% using a gauge length of not less than 24 in. Flexural capacity estimates should not rely on a strain greater than this value because the material is not required to provide a greater elongation before strand fracture occurs. In cases where concrete crushing at a strain of 0.003 is assumed and the equilibrium condition requires reinforcement strain greater than the minimum the material provides, the strain compatibility analysis should be repeated with strain in the reinforcement limited to 0.035. The resulting concrete compression strain will then be less than 0.003. This condition, which is easily accommodated with strain compatibility analysis, would negate a primary assumption of the simplified rectangular stress block approach.

The AASHTO LRFD specifications leave the details of the strain compatibility analysis, such as selection of appropriate stress-strain relationships for the concrete and reinforcement, to the engineering professional. Many stress-strain relationships for concrete have been developed, such as those by Hognestad[3] and Desayi and Kirshnan.[4] The bilinear stress-strain relationship used by El-Helou[5] and Tadros[6] is a simple and easy-to-use approximation. This bilinear model for concrete was used in the Part 1 example and is shown in **Fig. 2**. It is conservative, easily programmed into spreadsheet calculations, and amenable to hand calculations.

A bilinear elastic-plastic stress-strain relationship is often used for reinforcing bars and the well-known "power formula", presented in Part 1 of this article series, is typically used for prestressing strand.

Analysis Method Comparison

Consider a prestressed concrete beam with a composite cast-in-place deck slab with the dimensions, concrete strengths, and reinforcement presented in Part 1 of this series (**Fig. 3**). This section is analyzed with both the simplified and strain compatibility methods for increasing amounts of prestressing reinforcement, while keeping the center of gravity of the reinforcement constant for simplicity. The strain compatibility analysis uses the same bilinear model for the concrete stress-strain relationship (Fig. 2) and power formula for prestressing reinforcement from Part 1. **Figure 4** shows a comparison of the depth of the neutral axis and **Fig. 5** shows the comparison of factored flexural resistance for the two methods. **Figure 6** compares the stress in the prestressing reinforcement for each method.

The neutral axis depth is equal to the flange height when there is approximately 10 in.^{2} of prestressing reinforcement in the beam. Up to this amount of prestressing steel, only the slab is in compression and both methods result in essentially the same estimate of flexural capacity. Thereafter, the neutral axis depth, stress in the reinforcement, and flexural capacity diverge as the area of reinforcement increases.

At higher levels of reinforcement, the simplified method indicates that the section will enter the transition region between tension and compression-controlled behavior. At that point, the resistance factor for flexure becomes less than

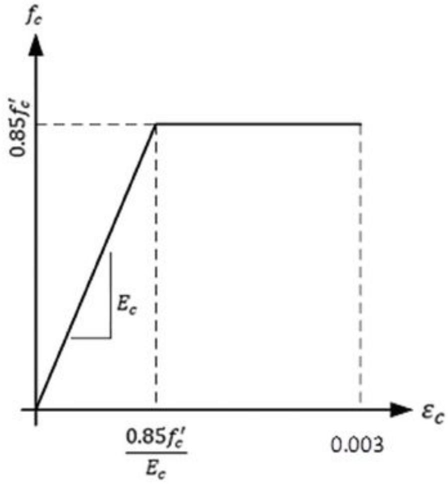

Figure 2. Bilinear stress-strain model for concrete.

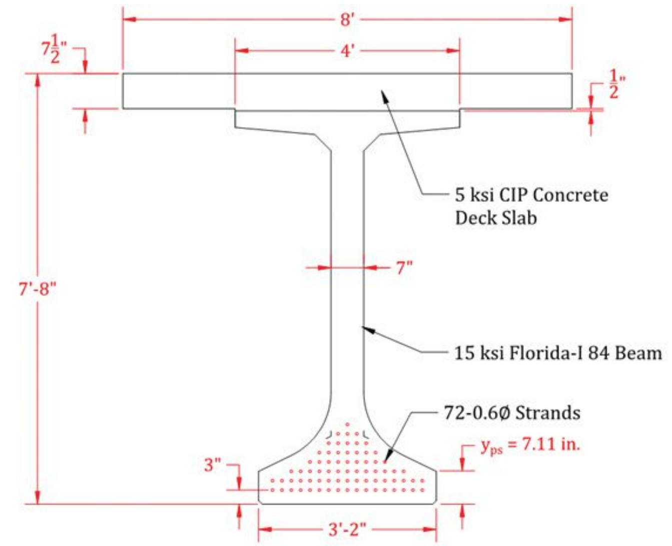

Figure 3. An 84-in.-deep Florida I-beam girder made composite with a 7.5-in.-thick, cast-in-place (CIP) structural deck slab used in the example analysis.

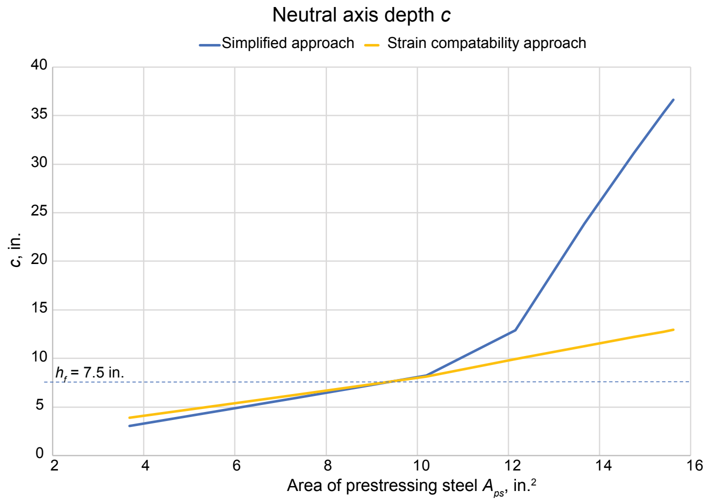

Figure 4. Depth of neutral axis comparison. Note: h_f = top flange thickness.

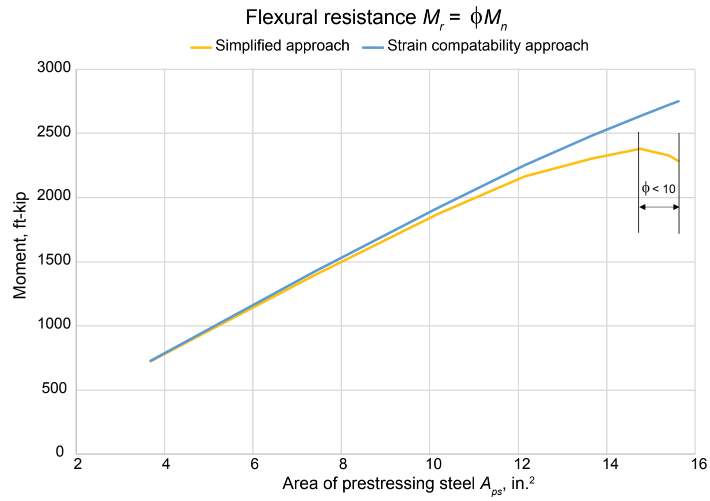

Figure 5. Comparison of flexural capacity.

1.0, increasing the conservatism of the estimated flexural capacity as compared with the strain compatibility analysis. The stress in the prestressing reinforcement decreases to less than the yield stress of 243 ksi for the simplified approach but remains significantly higher when the strain compatibility method is used.

The rectangular stress distribution–based simplified method has long been and remains an excellent method for estimating the flexural capacity of reinforced and prestressed concrete structural elements when the section analyzed conforms to the assumptions and limitations stated in the AASHTO LRFD specifications.

The simplified method becomes increasingly conservative because precast concrete has higher strength as compared with cast-in-place deck concrete, employs larger-diameter and higher-strength prestressing strands, and prestressed concrete beams with larger cross sections accommodate more strands, to the point where estimated flexural capacity at the strength limit state can control a design and limit load ratings.

The strain compatibility approach remedies this by providing more accurate estimates of flexural capacity that account for the size, shape, and properties of the prestressed concrete beam and the location and properties of the reinforcement.

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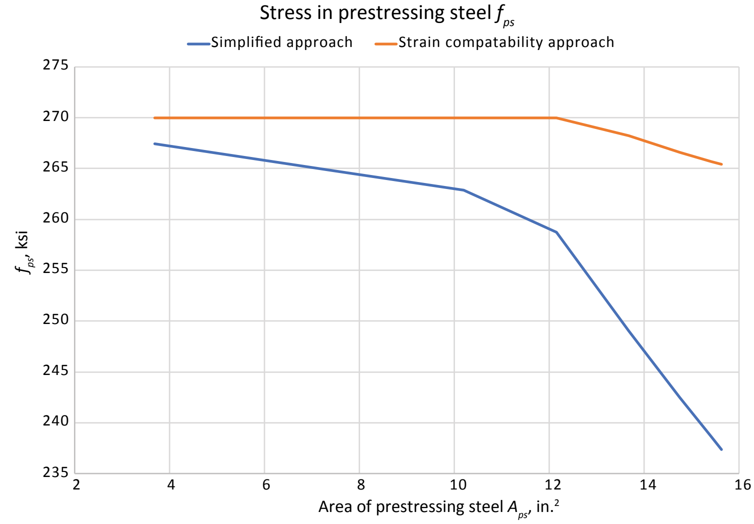

Figure 6. Comparison of stress in prestressing steel.

Fatigue Behavior of Reinforced and Prestressed Concrete

by Dr. Oguzhan Bayrak, University of Texas at Austin

This article explores fatigue behavior of reinforced and prestressed concrete components and explains the reasons behind a variety of different exemptions made in Article 5.5.3 of the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*.[1] Let's first explore the behavior of axially loaded components and then extend the discussion to flexural components.

Axially Loaded Prism

To understand the fatigue behavior of reinforced concrete, let us start with the behavior of a steel reinforcing bar cast in a concrete prism with a square cross section and then placed under tensile loads. To facilitate the discussion, let us focus on a typical reinforcing bar tensile test (that is, a "Bar" test) and a test of a reinforcing bar that is embedded at the centroid of a concrete prism with ends protruding (that is, a "Bar + Concrete" test). Refer to **Fig. 1** for illustrations of these tests.

Figure 1. Schematic of axial tensile tests. The specimen on the far left (Bar test) is a steel reinforcing bar, and the specimen at the center of the figure is a reinforcing bar that is embedded at the centroid of a concrete prism with ends protruding (Bar + Concrete test). The right side of the figure shows that upon cracking of the concrete, the stresses along the length of the bar encased in concrete vary. At the location of the cracks, the stress in the reinforcing bar is equal to applied load *P* divided by the area of the bar A_b. The stress between the cracks, in the concrete-encased portion, is lower. All Figures: University of Texas at Austin.

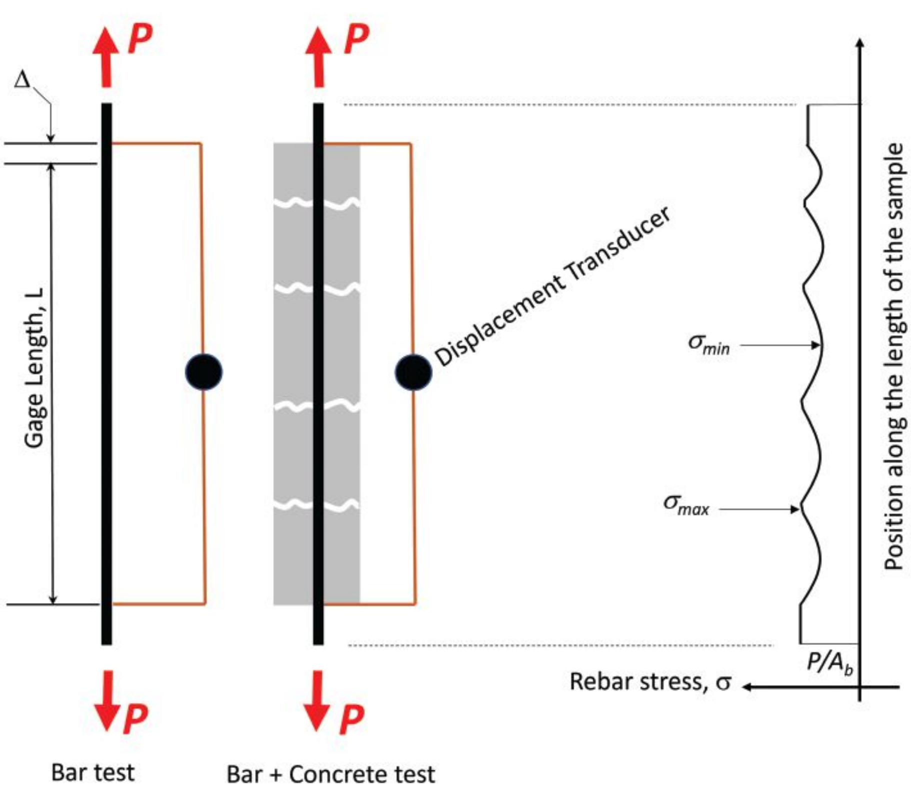

Figure 2 shows the stress-strain relationships from the two tensile tests for ASTM A615[2] reinforcing bars. The reinforcing bar will display a linear-elastic behavior with a modulus of elasticity of about 29,000 ksi up to the yield point, and the yield plateau of this mild reinforcing steel will take place at a stress that is typically slightly greater than 60 ksi. The behavior of the reinforcing bar after the yield plateau—within the strain hardening region—is noted, but that behavior is beyond the scope of our discussion herein.

The behavior of the same reinforcing bar encased in a concrete prism is quite different at low levels of axial strain, before the reinforcement yields. First, note the difference in axial stiffness. The presence of uncracked concrete substantially stiffens the overall response because the axial stiffness of concrete *AE*/*L* is significant even though the modulus of elasticity of typical concrete can be 5 to 7 times less than that of reinforcing bars. The significant increase in stiffness is because the axial prism under consideration has a cross-sectional area that may be 50 to 100 times more than the area of the bar encased in it (for example, a no. 9 bar in a 9 in. × 9 in. cross section). For our example, let us assume that the concrete cracks at 400 psi, as determined in a direct tension test (the cracking strength would be about 750 psi as determined by a modulus of rupture test), and its modulus of elasticity is 6000 ksi. This provides a modular ratio of steel to concrete (29,000/6000 = 4.83). At the point concrete cracks, the stress in the reinforcing bar would be about 2 ksi (1.93 ksi to be exact).

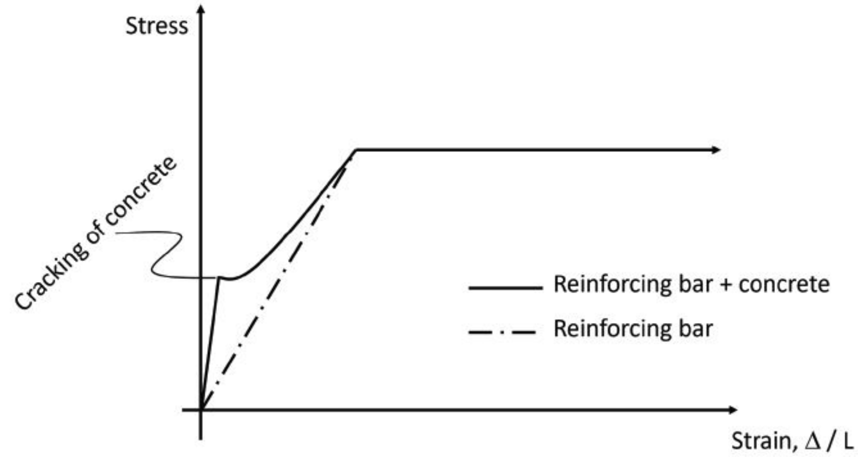

Figure 2. Stress-strain behavior for a tensile test of a reinforcing bar alone and for a reinforcing bar encased in a concrete prism.

Before discussing the post-cracking behavior of the bar encased in the concrete specimen, let us first discuss reinforcing bar fatigue. Fatigue in steel is a function of the mean stress and the cyclic stress applied to the bar. If the stress is low and below a stress level called the "endurance limit," the bar has an infinite fatigue life. As long as the concrete remains uncracked, it is difficult to envision tensile stresses developing a stress range that can lead to finite fatigue life under typical loading scenarios for typical reinforced concrete components. A good example of this type of tensile stress could be a reinforced concrete deck supported on several beams that experiences primary load effects due to "axial" effects that can be attributed to volume changes that the deck concrete experiences, as opposed to live loads supported on the decks.

Upon cracking, the stresses along the length of the bar encased in cracked concrete vary (Fig. 1). At the location of the cracks, the stress in the reinforcing bar is equal to applied load P divided by the area of the bar A_b because concrete does not contribute to the load-carrying capacity at those locations. The actual stress in the reinforcing bar between the cracks drops as the concrete "takes a bite" out of the stresses in the reinforcing bars. That is to say, the bond between the reinforcing bar and surrounding concrete will lead to load transfer from the bar to concrete. Conversely, the presence of concrete between the cracks will stiffen the response, an effect known as "tension stiffening." Importantly, the crack spacing and cross-sectional area of concrete that is stiffening the response of the bar both influence the drop in stress between the cracks. Those subtleties stated, at this stage of the cracked-concrete response, it is possible to envision theoretical scenarios in which the stress range can enter the territory of finite fatigue life and may surpass the endurance limit. At this point, I emphasize "theoretical" because fatigue failures are exceedingly uncommon in concrete bridges. To complete the discussion on the axially loaded component, the stress in both the bare reinforcing bar and the one encased in concrete will top out at the yield strength, and the capacity of the bar encased in concrete will be realized at the location of a crack.

Reinforced Concrete Beam

A natural extension of the preceding discussion is to consider, "How do we go from axially loaded components to flexural elements?" With that as the primary objective, let us look at the behavior of reinforced concrete beams before and after flexural cracking. For simplicity, consider the reinforced concrete beam depicted in **Fig. 3.** Before the flexural cracks form, the stress profile in the reinforcing bars on the flexural tension side of a typical Bernoulli beam follows the overall shape of the bending moment diagram. In our case, this is the bending moment diagram of a simply supported beam with uniform loads. Upon the formation of flexural cracks, and afterward, the stresses in the reinforcing bars will spike at the location of flexural cracks, as shown in Fig. 3. In this condition, the stress range experienced by the reinforcing bars under permanent loads and cyclical application of transient loads may produce stress ranges that may produce finite fatigue lives, at least hypothetically.

Prestressed Concrete Beams

Let us next discuss prestressed concrete girders, focusing our attention on pretensioned concrete beams. By precompressing the flexural tension side of pretensioned beams, we effectively control cracking. Under typical application of live loads, tension flange cracking is not expected in pretensioned components of bridges that are designed in accordance with the AASHTO LRFD specifications. In an effort to improve the expected service life of their bridges, some owners go even further in their bridge design manuals and use zero-tension designs. As a result, they do not expect to see flexural cracking under normal permanent and transient loads. However, under permit loads and superheavy loads, cracking is not only possible but anticipated in some cases. It is important to note that upon removal of such loads from pretensioned concrete girders, the flexural cracks that may form should close. The number of cycles associated with the application of such extreme loads (that is, very few load cycles) in the service life of any given pretensioned concrete bridge is in orders of magnitude less than any finite life (the number of cycles associated with fatigue concerns) we may calculate. We should also recognize that the stress ranges that can be supported by smooth wires in a seven-wire strand can be higher than those that can be tolerated by reinforcing bars, due to the stress concentrations at the locations where deformations (ribs) meet the shaft of the bar. That is to say, for infinite fatigue life, we have a higher stress range for strands than we do for reinforcing bars.

Compressive Stresses in Concrete

In reinforced or prestressed concrete designs, we rely on strands or reinforcing bars to carry tension while we count on concrete on the compression side. What about the fatigue life of concrete under cyclic compressive stresses? According to Article 5.5.3.1 of the AASHTO LRFD specifications, "for prestressed components in other than segmentally constructed bridges, the compressive stress due to the Fatigue I load combination and one-half the sum of the unfactored effective prestress and permanent loads shall not exceed $0.40f'_c$ after losses." This

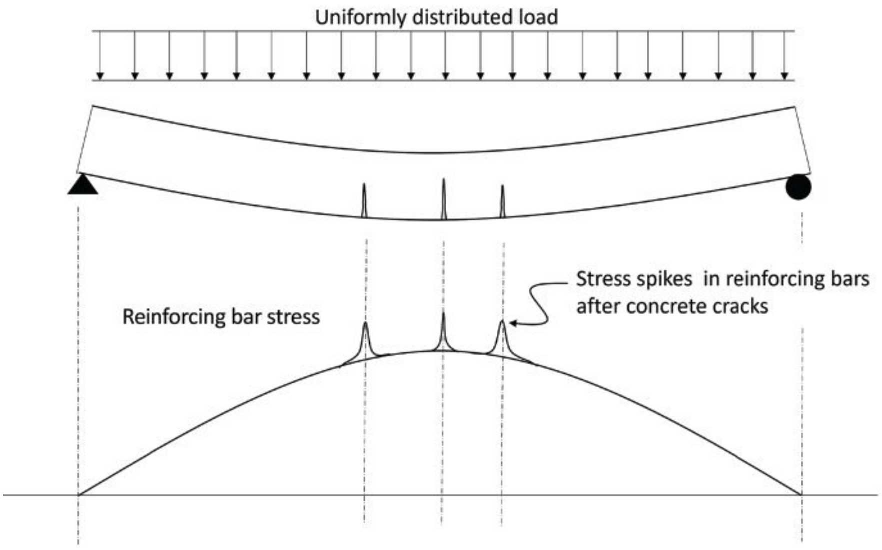

Figure 3. Stress-strain behavior for a tensile test of a reinforcing bar alone and for a reinforcing bar encased in a concrete prism.

stress limitation is intended to virtually eliminate the possibility of microcracking in concrete under repeated application of compressive stresses—a requirement that is easily met in a great majority of cases. This aspect of concrete behavior has not received wide-ranging attention in the research arena over the years, and there is a good reason for that: subjecting concrete to millions of cycles of compressive load with the maximum stress not to exceed 40% of its compressive strength has not produced any ill effects.

Because of the aforementioned material behavior, the AASHTO LRFD specifications offers many exemptions for explicit fatigue considerations, and, again, there is a good reason for these exemptions. Prestressed concrete components that meet Service III stress limit requirements, concrete decks supported on multiple beams, and box culverts are exempt from the checks. Why? We do not know of field problems or calculations that would raise a cause for concern. Even in cases where we check stress ranges in reinforced concrete components, typical stress ranges are very low. After all, fatigue-related failures in concrete components are exceedingly rare to the point of being almost nonexistent. Consideration of the behavioral aspects covered in this article may prove to be useful as context when one reviews the Article 5.5.3 requirements of AASHTO LRFD specifications, as discussed by Dr. Francesco Russo in a Concrete Bridge Technology article in the Winter 2024 issue of *ASPIRE*®.

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Concrete Segmental Bridges—Preliminary Design Approximations for Span Layouts and Structure Depths

by R. Kent Montgomery, GM2 Associates

This article discusses preliminary design approximations for span layouts and longitudinal structure depths for concrete segmental box-girder bridges. Strictly speaking, these approximations apply to concrete segmental bridges. However, they are useful approximations for all post-tensioned concrete bridges. Note that the recommendations presented here may not apply to some bridge designs. For example, designs outside of the recommended ranges can occur when span layouts must take into account obstructions below the bridge, or when structure depths are constrained by clearances and profile grade. However, these recommendations typically result in optimal designs.

Why are design approximations used? A starting point is necessary before proceeding to final design. In final design, the structure will be analyzed in detail and address the following matters:

- Time-dependent time-step analyses, both longitudinally and transversely
- Shear, torsion, and moment designs for the service and strength limit states
- Special element design for diaphragms, deviators, anchor blocks, and other elements

These detailed analyses and designs will vet the preliminary design and provide the exact amount of concrete, post-tensioning, and reinforcement required. Using final-design level of effort and analyses in preliminary design would be costly in both schedule and engineering hours. And while a final design without a high-quality preliminary design may meet all the required structural design specifications, it may not be the optimal solution for the project. Using proven preliminary design approximations, along with experience, provides a greater probability of reaching an optimal solution in terms of design and constructability.

The first step in preliminary design is determining an optimal span layout. Substructure units and foundations must avoid streets, navigation channels, buildings, and other obstacles. Sometimes, these obstructions play a large part in determining the span layouts and there is little room for variation. However, what spans should be used when there is more of a "blank canvas"?

The cost of the substructure units helps determine what span lengths to use. When substructures are relatively inexpensive, shorter spans are typically chosen. Examples of this situation include urban viaducts with short piers or shallow water crossings with good site access. For concrete segmental bridges, the span-by-span erection method is typically the least-expensive erection method for shorter spans. Span-by-span erection has been used in spans up to 180 ft. However, it is most efficient and economical for spans in the 100- to 150-ft range.

The cost of the substructure is greater when a bridge crosses deep water, deep canyons, rugged terrain, or spans obstacles such as navigable channels. For concrete segmental bridges, the longer span lengths associated with challenging site conditions or relatively expensive substructure units can push the erection method to the balanced-cantilever erection method. Precast concrete balanced-cantilever erection has typically been used for spans ranging from 180 to 450 ft, although longer spans have been used for some projects. Precast concrete segments can typically be transported on highway and road systems when their weight is 50 tons or less and there are adequate clearances for the segments along the proposed route. Beyond this weight, a detailed investigation of the route, including bridge capacities, must be performed to verify the feasibility of transportation options. Much larger segments can be transported by barges; in such cases, the casting yard and erection site equipment typically govern the segment weights.

Cast-in-place (CIP) concrete balanced-cantilever design is typically used for spans ranging from 300 to 600 ft, although, again, there are exceptions. CIP concrete segments may be chosen if the transportation route and/or equipment capacities indicate that precast concrete segments are not practical. Site constraints on crane placement can also make CIP balanced-cantilever construction the preferred alternative.

Span layouts for span-by-span erection strive to use uniform span lengths. This strategy aids in the design and operation of erection trusses or gantries. To simplify erection equipment needs and operations, it is optimal for the end spans to be approximately the same length as the interior spans (**Fig. 1**). The reduced post-tensioning secondary moments in end spans help offset the increased permanent and live-load moments. The secondary moments in end spans are less than those in interior spans because there is no rotation restraint at the expansion joint. The secondary moments are zero at the expansion joint and increase across the span to the first interior pier. Therefore, near midspan, the secondary moments are about half of the secondary moments for interior spans.

Bridges erected span by span are almost always constant depth with a span-to-depth ratio kept within the range of 16

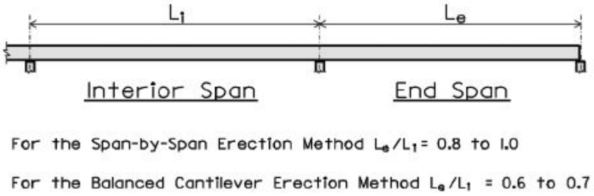


Figure 1. Ratios of end-span length to interior-span length for preliminary span layout. All Figures: R. Kent Montgomery.

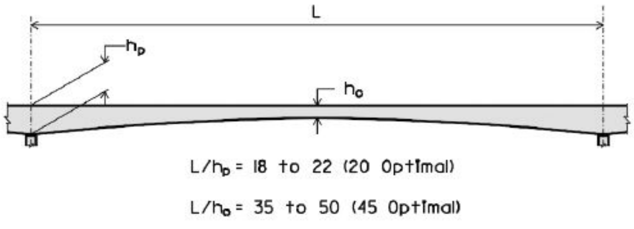
to 22, with 18 being optimal (**Fig. 2**). A minimum depth of approximately 7 ft should be used to provide reasonable access inside the box girder during construction operations as well as post-construction inspection and maintenance activities.

For balanced-cantilever erection, the end spans are typically set shorter than the structural optimum as a constructability consideration. A typical cantilever extends approximately one-half of the interior span length. For end spans, the portion of the span between the end of the cantilever and the abutment/pier must be erected by other means, typically on falsework. However, uplift at the end of a unit should be avoided. For the previously cited reasons, end spans are typically set at lengths of between 0.60 and 0.70 of an interior span length (Fig. 1).

Variable-depth structures add complexity and cost. Therefore, for spans erected by the balanced-cantilever method with span lengths less than approximately 250 ft, designers typically use constant-depth spans with a span-to-depth ratio of 18 to 22, with 20 being optimal (Fig. 2). The bottom slab will typically require thickening to resist the cantilever moments. This objective can be accomplished by thickening to the inside of the box girder, or with a small thickening to the outside over a short distance.

As spans increase to more than 250 ft, variable-depth spans become more economical. The span-to-depth ratio at piers is usually kept between 18 and 22, with 20 being optimal. The span-to-depth ratio at midspan is typically kept between 35 and 50, with 45 being optimal (**Fig. 3**). The guideline for midspan depth also applies for end spans near an abutment or expansion pier, with the same minimum depth of approximately 7 ft recommended for access. For variable-depth structures, the variation in depth along the span must be defined. This curve is typically referred to as the intrados. During preliminary design,

Figure 3. Structure depths for variable-depth concrete segmental bridges.



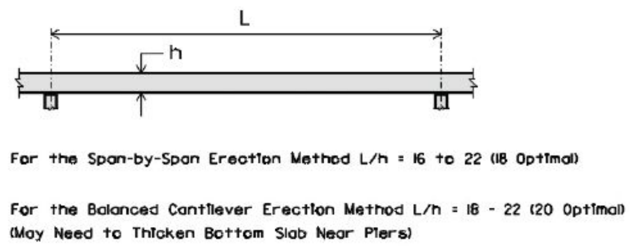


Figure 2. Optimal structure depths for constant-depth concrete segmental bridges.

a simple equation (**Fig. 4**) that provides flexibility can be used to define the depth variation along the span.

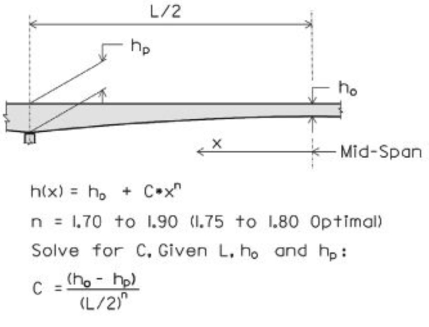
$$h = h_o + Cx^n$$

where

- C = constant depending on the cantilever length and depths at pier and midspan = $(h_p - h_o)/(L_c)^n$
- h = depth at distance x (typically measured from the edge of the midspan closure joint to the face of the pier diaphragm)
- h_o = depth at midspan
- h_p = depth at piers
- L_c = cantilever length (typically from midspan to the piers)
- n = exponent controlling the profile
- x = distance along the span (typically measured from the edge of the midspan closure joint to the face of the pier diaphragm)

The exponent n is typically set between 1.70 and 1.90, with 1.75 to 1.80 being optimal. Increasing the exponent toward the upper limit can aid in providing vertical clearances for navigational channels or other required clearances. However, increasing the exponent to more than 1.90 (approaching a parabola) results in a critical region near the quarter span (mid-cantilever) that will govern the design above other regions in the span/cantilever. This configuration can require added cantilever post-tensioning for the critical region, as well as an increased tendon size for all cantilever tendons whose anchorages are beyond the critical region. This configuration can also require increased web widths to control principal tension in the critical region, although the increased web widths would not be required elsewhere.

Figure 4. Intrados definition. Variation in depth along the span (intrados) for a variable-depth concrete segmental bridge.



If the exponent is set to less than 1.70 (a flatter curve), the self-weight of the cantilever increases, and the section near the pier will govern the design above other regions and will require an increased amount of cantilever post-tensioning. If the exponent is set optimally, each region (as represented by the model node at each segment joint) will be close to governing equally, both for cantilever and long-term moments along with shear.

The bottom slab will need to be thickened near the pier to control compression stresses and provide adequate negative-moment capacity. The thickness will depend on the span length and intrados selected. At the piers, a good rule of thumb is to provide 9 in. of thickness for every 100 ft of span length. Using this initial bottom-slab thickness at the pier, simple cantilever calculations can be performed by hand or using a spreadsheet to iterate to a final thickness, as follows:

1. Calculate the cantilever moment during construction and determine the amount of cantilever post-tensioning to limit the top tension stress to zero (precast concrete), or $0.0948\sqrt{f'_c}$ ksi (CIP concrete with continuous reinforcement).
2. Check the bottom compression stresses to determine whether they are within the limiting compressive stress. If the compressive stresses are outside the limit, increase the thickness of the bottom slab; if they are significantly less than the limit, decrease the thickness. If the thickness is adjusted, repeat steps 1 and 2.
3. Calculate the neutral axis location for the negative moment at the strength limit state to ensure that the axis is not located significantly above the bottom slab, which could result in a flanged section with less-ductile behavior. If required, thicken the bottom slab and repeat steps 1 through 3.

Note that the bottom thickness is sized for the cantilever state during construction, and calculations should account for any construction equipment load on the cantilever (such as beam and winch or form travelers), as well as 10 lb/ft^2 of general construction live load across the deck area. The 10 lb/ft^2 is required by Article 5.12.5.3 of the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[1] and accounts for miscellaneous construction loadings such as strand packs, reinforcing bars, portable toilets, or personnel. If there is significant construction equipment on the cantilever, the cantilever state during construction should govern over the in-service state of the structure with live load included. If the only load applied to the cantilever is the additional 10 lb/ft^2 general construction live load (that is, no significant construction equipment is considered), the cantilever state and in-service state should be close to governing equally. Minor adjustment to the bottom-slab thickness at the pier may be needed during final design. The bottom-slab thickness typically varies from maximum at the pier to the minimum thickness across a length, which is equal to approximately 30% of the span length. This variation can be linear or curved using the same exponent as for the structure depth variation (intrados).

Preliminary design involves still more effort to determine the remainder of the cross-sectional segment dimensions. The amount of continuity post-tensioning and anchorage locations must be determined, as well as the articulation (pier types, fixity, and location of expansion joints). However, good span layouts and structure depths provide the first steps in a quality preliminary design. Addressing these aspects of design in the early stage will result in minimal changes during final design, which is essential for an efficient final design process.

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Corrosion-Resistant Fiber-Reinforced Polymer Reinforcement for Concrete Structures

by Steven Nolan, Florida Department of Transportation, Matthew Chynoweth, RS&H, and Dr. Antonio Nanni and Dr. Francisco De Caso, University of Miami

An article in the July–August 1993 issue of *PCI Journal* opens with the observation, "By the 1980s, the technology that spawned the original AASHTO I-beams was 30 years old. The beams had more than met their intended goals, but times were changing: sophisticated structural analysis, improved materials and fabricating techniques, and advanced construction methods were being introduced at a rapid pace."[1] Readers today may note that this opening statement does not mention durability, resilience, or sustainability. Perhaps the author felt confident that the durability challenges for reinforced and prestressed concrete had been resolved, especially considering a 50-year nominal design life. It is not surprising that "resilience" and "sustainability" were not mentioned; these terms as currently used were absent from our lexicon in the early 1990s.

Increased Target Service Life

In 1994, the first edition of the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[2] increased the nominal design life from 50 to 75 years. This change was partially based on the performance of existing bridge stock, but it also reflected the reality of a growing bridge inventory, which exceeded 570,000 structures in 1990 and could not affordably be replaced every 50 years.[3] By this time, a longer design life was feasible because the industry had improved its understanding of the mechanisms involved in the corrosion of steel reinforcement and how they are exacerbated by long-term chloride diffusion and carbonation.[4]

Three decades later, the U.S. highway bridge inventory exceeds 623,000 bridges.[5] Another notable development is that bridge deck areas are often much greater than in the past, due to capacity and safety improvements such as additional travel lanes, wider roadway and bicycle shoulders, and additions of sidewalks on many non-limited-access bridges. Furthermore, increasing urbanization, managed lanes, and a generally more constrained roadway network have resulted in significant increase in the associated earth-retaining and water-conveyance structures, which are also predominantly reinforced concrete, including precast concrete. Additionally, there have been significant advancements in reinforced concrete structural materials technologies, such as fiber-reinforced polymers (FRP), high-strength stainless-steel strands and reinforcing bars, and ultra-high-performance, steel-fiber-reinforced concrete. Simultaneously, societal expectations for safety, maintainability, and reliability are increasing, while the intensity of both natural and human made shocks and stressors from the surrounding environment are also on the rise .[6,7]

With sufficient concrete cover, good detailing, and appropriate workmanship practices, uncracked high-performance concrete and carbon-steel reinforcement typically provide adequate durability to achieve a 75-year target service life. However, asset managers and bridge owners still face susbstantial durability challenges, including mitigating in-service concrete cracking (especially for bridge decks in colder regions), achieving corrosion resistance in low-level trestle bridges in coastal areas, and meeting target service-life expectations of 100 to 150 years.

One solution to such challenges is the use of extremely corrosion-resistant reinforcement. Of the available material classes that could meet an enhanced (100-year) target service life in an extremely corrosive environment, FRP reinforcing bar and prestressing strands are one practical solution and are the focus of this article.[8] A follow-up article presenting state-of-the-art stainless-steel reinforcing bar and strand reinforcement as another viable option is planned as part 2 of this series.

Developments in FRP Reinforcement

Many developments have occurred since our article, "Glass Fiber-Reinforced Polymer (GFRP) Reinforcement for Bridge Structures," in the Summer 2020 issue of *ASPIRE*®. The following are worth highlighting:

- The publication of the American Concrete Institute's *Building Code Requirements for Structural Concrete with Glass Fiber-Reinforced Polymer (GFRP) Bars—Code and Commentary* (ACI CODE-440.11-22)[9]

The 17th Street Bridge replacement over Indian River in Vero Beach, Fla., includes 168 prestressed concrete Florida slab beams with fiber-reinforced-polymer reinforcement. The 45-ft long beams use carbon-fiber-reinforced-polymer strands and glass-fiber-reinforced-polymer auxiliary reinforcement. All Photos: Florida Department of Transportation.

The Florida slab beams, piles, and bent caps of a low-level observation deck at the U.S. Route 1 bridge replacement in Jupiter, Fla., have carbon-fiber-reinforced-polymer prestressing strands and glass-fiber-reinforced-polymer reinforcement.

- The publication of ASTM D8505-23[10] for higher-modulus and higher-strength FRP reinforcing bars
- Forthcoming updates to the second edition of the *AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete*[11] and to Section 16, Fibre-Reinforced Structures, of the *Canadian Highway Bridge Design Code* (CSA S6:2019)[12]
- Other new, revised, and forthcoming design, construction, and material specifications and guidelines. For example, the National Cooperative Highway Research Program (NCHRP) project 12-121 is preparing recommendations for auxiliary (non-prestressed) FRP reinforcement for prestressed concrete to complement AASHTO's *Guide Specifications for the Design of Concrete Bridge Beams Prestressed with Carbon Fiber-Reinforced Polymer (CFRP) Systems*.[13]

Moreover, there has been significant research published in the last decade that can further advance design provisions related to axial columns and other components, seismic and fire design, durability of basalt and glass FRP reinforcing bars, and feasibility of using seawater or sea sand in concrete mixtures for marine applications.

It is hoped that construction operations can be streamlined by commercial applications of several innovations, including product development of post-production formable resin systems that will allow shop bending of FRP reinforcing bar at either regional distribution centers or remote facilities; mechanical connection systems for direct splicing and coupling of FRP reinforcement; and grid and mesh FRP systems for accelerated construction. ASTM Subcommittee D30.10 is currently standardizing grid and mesh FRP systems, and CFRP reinforcing and strand material specifications are also being completed.

In addition, nondestructive testing inspection technologies continue to advance; these changes were recently highlighted in the Federal Highway Administration–sponsored *A Framework for Field Inspection of In-service FRP Reinforced or Strengthened Concrete Bridge Elements*,[14] and a supporting TechBrief is anticipated soon. The AASHTO Committee on Bridges and Structures' Safety and Evaluation technical committee (formerly T-18) will ballot the associated bridge element updates for *The Manual for Bridge Evaluation*[15] at the committee's next annual meeting.

Finally, oversight and certification organizations for both product testing and evaluation have matured. Examples include the AASHTO Product Evaluation and Audit Solutions program for composite concrete reinforcing; NEx: An ACI Center of Excellence for Nonmetallic Building Materials; and the FRP Institute for Civil Construction auditing program.

Project Applications

Several state departments of transportation have made significant investments into the development and construction of FRP-reinforced precast concrete and reinforced concrete bridge components. More than two decades ago, the Michigan Department of Transportation (MDOT) began using CFRP prestressing strands for various concrete components. MDOT started this work in 2001 with the Bridge Street bridge in Southfield, Mich., and the agency continues to design and construct bridges with CFRP post-tensioned and prestressed components as part of the state's traditional bridge replacement and rehabilitation program. MDOT was also instrumental in securing a CFRP strand manufacturing facility in the United States.

The Virginia Department of Transportation was another early adopter of CFRP prestressing strand. Its largest project to date, the $3.9 billion Hampton Roads Bridge-Tunnel Expansion, began construction in 2020. (See the Project article in the Fall 2024 issue of *ASPIRE* for more information about the Hampton Roads marine trestles.) This significant project includes considerable quantities of FRP-reinforced precast, prestressed concrete cylinder piles and I-girders in the 5 miles of marine trestle spans.

The North Carolina Department of Transportation (NCDOT) recently completed the 3200-ft Harkers Island Bridge with prestressed concrete Florida I-beams and piles. The bridge deck, pile footings, and caps all contain GFRP reinforcement. NCDOT is also expected to begin construction on the 3.3-mile U.S. Route 64 over Alligator River bridge replacement in early 2025 and will use similar FRP-reinforced cast-in-place and precast concrete components.

The Florida Department of Transportation has completed or advanced design on more than 67 bridges with FRP-reinforced cast-in-place and precast concrete components. The 2017 Halls River Bridge demonstration project in Homosassa and the current 17th Street Bridge east end replacement in Vero Beach provide salient examples.

Design Refinements Based on Research

Currently, the most actively researched topic related to GFRP-reinforced concrete construction is seismic performance. ACI CODE-440.11-22 has very restrictive requirements related to GFRP-reinforced concrete under Section 1.4.13, including restrictions on its use in Seismic Design Categories (SDCs) D through F as well as restrictions in SDCs B and C for members that are part of the lateral-load-resisting system.

Recent refinement of several design parameters for various limit states should improve the efficiency of FRP-reinforced concrete for future designs, improving both sustainability and resilience. As clarifying examples, ACI CODE-440.11-22 increased the environment reduction factor C_E (accounting for degradation of GFRP under high-alkalinity) from 0.70 to 0.85 for outdoor exposure conditions. This can provide a 21% increase in the strength limit state capacities. Similarly, the standardization of high elastic modulus E_f and high tensile strength f_u basalt and glass FRP reinforcement under ASTM D8505-23, provides up to 33% increased component flexural resistance under both strength limit state and service limit state for crack control, sustained load, and transient load-induced fatigue. The higher modulus contributes a 33% increase to the reinforcing component strength for transverse shear capacity V_f, while ACI CODE-440.11-22 increased the associated maximum tensile strain of shear stirrups e_{ft} from 0.004 to 0.005, harmonizing with *Design and Construction of Building Structures with Fibre-Reinforced Polymers* (CSA S806)[16] for buildings and the forthcoming edition of the *Canadian Highway Bridge Design Code* (CSA S6:25)[17] for bridges. The increased strain limit provides a 25% increase to the shear reinforcement contribution V_f.

Both ACI and AASHTO technical committees are continuing work to refine interface shear and precast concrete beam shear provisions, with supporting research being done by NCHRP Project 12-121, NEx, and several Canadian research groups. The refinements are expected to yield further design economy and reduce the cost differential between traditional carbon-steel-reinforced and FRP-reinforced concrete/precast concrete. Opportunities exist for further refinement of resistance factors for axial, flexural, and shear design under the strength limit state, and there is a need for calibration and harmonization of ductility concepts and redundancy (both component and system). For example, regarding the flexural resistance factors, AASHTO now treats all CFRP precast concrete components essentially as compression-controlled designs, with a conservative resistance factor of 0.75 for both compression-controlled and tension-controlled designs.

Lastly, when envisioning the concrete and cement road maps for decarbonization by 2050, resilience (durability, robustness, and self-healing), and sustainability (global warming potential, adaptability, and reuse) will be important factors for design guidance and eventual codification will be critical. Here, the use of FRP reinforcement along with concrete that includes recycled aggregate and even seawater may result in a pivotal advancement.

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NCBC MEMBER SPOTLIGHT

Celebrating 70 Years of PCI Accomplishments

by Dr. Richard Miller, Managing Technical Editor of *ASPIRE*®

In 2024, the Precast/Prestressed Concrete Institute (PCI) celebrated its 70th anniversary. It is significant that the first prestressed concrete structure in the United States was a bridge. The original Walnut Lane Memorial Bridge opened in 1951, and PCI was formed just three years later. In those early years, there was a lot that was not known about reinforced precast concrete structures and prestressed concrete structures, so a group of dedicated engineers started to research this new engineered-to-order building solution. In 1956, just two years after PCI was formed, the first issue of the *PCI Journal* was published. (Every issue of the *PCI Journal* is available online at pci.org.)

Initial research focused on learning about the behavior of precast, prestressed concrete components. The first issue of the *PCI Journal* had two articles written by individuals who were later named PCI Titans of the Industry, Dr. T. Y. Lin and Dr. Paul Zia. Eventually, the basic behavior of precast, prestressed concrete components was reasonably well understood, and the research evolved to improving and refining our understanding of the behavior of prestressed (pretensioned and post-tensioned) concrete components and developing new materials, shapes, and fabrication processes. A quick glance at the table of contents of any issue of the *PCI Journal* will show articles that influenced all aspects of the industry.

To celebrate the 70th anniversary of PCI, the Journal Editorial Advisory Committee assembled and published a list of 70 influential articles in the November–December 2024 issue of the *PCI Journal*. The committee specifically did not call them the most influential or most important, as opinions about such rankings will vary. What is important or influential for one sector of the industry may be less important to another sector, and vice versa. Among the criteria that the committee members considered in their selection process are the following: items that are influential or provide a remarkable contribution to the industry's body of knowledge; articles that have been frequently cited by other researchers and engineers around the world; and articles that serve as a point of reference in codes or standards.

There are items on the list related directly to bridges and many on general subjects, such as materials, that have also influenced bridge design. The following are the *PCI Journal* articles that are among my favorites and have influenced my career.

"Plant Certification—A Program of Merit in the Prestressing Industry," by Charles W. Wilson (1968)[1]

When I taught, my students would ask why in the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications,*[2] the resistance factor for bending for tension-controlled prestressed concrete sections with bonded strands or tendons ($\phi = 1.0$) is more favorable than that for tension-controlled (regular) reinforced concrete sections ($\phi = 0.9$). I explained that difference is because prestressed concrete is made in a plant under carefully controlled conditions, which improves the quality and reliability of the product. The PCI Plant Certification Program started in 1967 with 35 volunteer plants. This paper, written by past PCI president and Titan of the Industry Charles W. Wilson, outlines

A feature from the November-December 2024 *PCI Journal*. All Figures and Photos: PCI.

EDITOR'S NOTE

Welcome to my first issue as managing technical editor of ASPIRE®. *After 36 years as a professor, I decided I had done all I could do at the university, so I retired. One of my favorite parts of being a professor was "disseminating knowledge," and taking over as managing technical editor of* ASPIRE *beginning with this Winter 2025 issue will allow me to continue to do that. I look forward to working with members of the National Concrete Bridge Council and the* ASPIRE *Editorial Advisory Board to bring you interesting and relevant articles about concrete bridges and related topics.*

I want to thank the previous managing technical editors and my good friends, Dr. Henry Russell, Dr. Reid Castrodale, and Dr. Kris Brown, for creating and maintaining the excellence of this publication. I especially want to thank Reid and Kris for helping me in this transition.

Fig. 2. Testing a 5-inch prestressed concrete slab, uniform load being applied by air pressure in plastic bags (University of California).

CAN WE DESIGN PRESTRESSED CONCRETE BY ALLOWABLE STRESSES

By Dr. T. Y. Lin

Associate Professor, Civil Engineering
University of California, Berkeley

(I) Introduction

Prestressed-concrete design has been made on the basis of the elastic theory, using working loads and allowable stresses. It has been also based on the ultimate strength theory, using factors of safety on the loads, often termed as load factors. For most countries in Europe, recommendations and codes for prestressed concrete permit design on either the ultimate or the elastic basis. In this country, our engineers, being used to the elastic theory for reinforced concrete, tend to prefer the elastic rather than the ultimate approach. While the elastic theory works beautifully for prestressed concrete before cracking, the strength after cracking can only be determined by the ultimate theory. Unless care is exercised in setting up variable allowable stresses to suit different conditions, design by the elastic theory will yield divergent degrees of safety.

Ultimate design is based on a set of given load factors. Hence, structures designed by that method will possess the desired degree of safety, although they may be overstressed or understressed if judged by the elastic theory. However, ultimate strength design gives no assurance of proper behavior under service conditions, unless the design is checked by the elastic theory or a suitable factor of safety is chosen for each case.

(II) What Should We Design Our Structures For?

Let us try to determine our basic requirements in design. First of all, it must be made clear that we should not design for fictitious or even real stresses unless we understand the significance of their values. Owing to the over-emphasis on stress calculations in our engineering training, our graduates do not know very much about real engineering although they do know how to compute stresses by theoretical and empirical formulas. They do not have sufficient idea of the actual behavior and strength of structures. The less they understand the structures, the more they tend to blindly follow certain codes of practice. It often ends up that we are simply designing our structures to satisfy certain code requirements which can be either too conservative or not sufficiently safe.

To experienced engineers, the real criteria for designing are:

1. We should design our structures so that they will perform satisfactorily under the usual service conditions. This means freedom from excessive camber, deflections, vibrations, cracks, etc.

May, 1956 3

An article by Dr. T. Y. Lin opened the first issue of the *PCI Journal*.

An early *Journal* cover from the newly formed Prestressed Concrete Institute.

the PCI Plant Certification Program. The Plant Certification Program is based on the continuous improvement of industry practices which advance the high quality of precast and precast, prestressed concrete components.

"Lateral Stability of Long Prestressed Concrete Beams—Parts 1 and 2," by Robert F. Mast (1989 and 1993)[3,4]

Several years ago, I participated in the drafting of PCI's *Recommended Practice for Lateral Stability of Precast, Prestressed Concrete Bridge Girders*[5] and the associated learning modules published on the PCI eLearning website (https://oasis.pci.org/Public/Catalog/Home.aspx). These resources made extensive use of Bob Mast's papers. Bob realized that precast, prestressed concrete girders did not fail in lateral torsional buckling like steel, but instead rolled over about a roll axis. His elegant mathematical solution forms the basis of our understanding of girder stability.

"The NU Precast/Prestressed Concrete Bridge I-Girder Series," by K. Lynn Geren and Maher K. Tadros (1994)[6]

Early in my career, I participated in the Federal Highway Administration's High-Performance Concrete Showcase. Several states were chosen to design and construct bridges using high-performance concrete and 0.6-in.-diameter strand. One of the goals was to extend girder spans. In Ohio, we designed a box girder but found we could not extend the span very much with that shape. The Nebraska University (NU) girder was optimized to take advantage of the larger strand area and higher-strength concrete to obtain longer spans with a favorable span-to-depth ratio. Another innovation was in the forming. AASHTO-type girders have a different shape for each depth of girder, so fabricators need a different set of forms for each shape. The NU girder uses a standard top and bottom flange, and the girder height is changed using inexpensive flat forms to vary the web height. The NU girder is used in some states, and many other states have created their own versions of this girder to fit their needs.

"Shear Transfer in Reinforced Concrete—Recent Research," by Alan H. Mattock and Neil M. Hawkins (1972)[7]

"Torsion Design of Prestressed Concrete," by Paul Zia and W. Denis McGee (1974)[8]

"Shear Transfer in Reinforced Concrete with Moment or Tension Acting Across the Shear Plane," by Alan H. Mattock, L. Johal, and H. C. Chow (1975)[9]

"Shear and Torsion Design of Prestressed and Non-Prestressed Concrete Beams," by Michael P. Collins and Denis Mitchell (1980)[10]

Whenever I taught bridge design or advanced concrete design, my favorite lectures were on shear behavior. Research dating back to the 1920s first proposed that shear failure was a diagonal tension failure and that a truss model could be used for shear. These four papers were instrumental in our understanding of shear and torsion in prestressed concrete and are among the most-cited papers from the *PCI Journal*. They form the basis of the modified compression field theory that is used in the AASHTO LRFD specifications.

I invite you to read "70 Significant Items from Seven Decades of *PCI Journal*," in the November–December 2024 issue of *PCI Journal* to see where many industry practices and design provisions started. Then go back and read or reread the papers. I then would ask you to look at the recently published second edition of the recommended practice for strand bond that is in the January–February 2025 issue of *PCI Journal*.[11]

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Dr. Richard Miller is professor emeritus and former head of the Department of Civil and Architectural Engineering and Construction Management at the University of Cincinnati, where he taught for 36 years. He has served on and chaired several PCI councils and committees and currently serves on the PCI Board of Directors as the chair of the Technical Activities Council. He is a PCI Fellow, and in 2024 he was named a PCI Titan of the Industry.

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

IN THIS ISSUE

https://www.fhwa.dot.gov/bridge/nbi.cfm
https://www.fhwa.dot.gov/bridge/mtguide.cfm
The Perspective on page 6 delves into National Bridge Inventory (NBI) data to review market share and performance of the concrete bridge inventory. Both links provide more information about the NBI; the second one also provides access to downloadable data sets from 1992 to 2024.

https://rosap.ntl.bts.gov/view/dot/68881
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 42 is the third in a series dedicated to investigating the statistical reliability indices that should be used to accommodate truck platoons.

https://highways.dot.gov/research/long-term-infrastructure-performance/ltbp/nondestructive-evaluation-structural-health-monitoring
Siva Corrosion Services (SCS) is the subject of the Focus article on page 9. SCS specializes in materials and corrosion engineering with the goal of extending the service lives of bridges. They use a variety of nondestructive testing techniques and offer structural health monitoring services and corrosion-control designs. This is a link to a Federal Highway Administration webpage that lists resources for nondestructive evaluation and structural health monitoring.

https://www.soundtransit.org/system-expansion/federal-way-link-extension
https://www.youtube.com/watch?v=2sBY0YVe_t8
Structure C, a long-span, cast-in-place concrete segmental bridge, that is part of Sound Transit's Federal Way Link Extension, is the subject of the Project article on page 12. Concrete segmental construction offered the best solution for a section of the route that passes through an extremely challenging site. The first link leads to the Sound Transit webpage for the Federal Way Link Extension, which has project maps, news, and updates. The second link is a flyover video of the project from 2023.

https://oasis.pci.org/Public/Catalog/Home.aspx?Criteria=177&Option=734
The Concrete Bridge Technology article on page 22 covers basic concepts of strain compatibility in concrete components. The PCI eLearning Course T130: Flexural Design of Precast, Prestressed Concrete—Strength Limit States can be accessed via this link. This free course covers the simplified method and strain-compatibility method for flexural design of precast, prestressed concrete beams.

https://asbi-assoc.org
Preliminary design and span layout considerations for concrete segmental box-girder bridges are the subject of the Concrete Bridge Technology article on page 28. The American Segmental Bridge Institute offers many design resources for concrete segmental bridges on their website.

https://www.fhwa.dot.gov/bridge/composite/cpdi.cfm
This is a link to a Federal Highway Administration (FHWA) webpage about fiber-reinforced polymer (FRP) composite technology, which provides additional links to various reports, articles, and design information for FRP components. Advances and development of corrosion-resistant FRP reinforcement for concrete structures are the subject of the Creative Concrete Construction article on page 31.

https://www.fhwa.dot.gov/resourcecenter/teams/structures-geotechnical-hydraulics/MCFT_for_Load_Rating_Examples.pdf
This is a link to the *Modified Compression Field Theory (MCFT) for Shear Load Rating—Pretensioned Example* published by the FHWA. Concrete bridge shear load rating is the subject of the FHWA article on page 50.

https://doi.org/10.15554/pcij69.6-03
The NCBC Spotlight on page 34 highlights the 70th Anniversary of the Precast/Prestressed Concrete Institute. The *PCI Journal* has published a list of 70 influential papers and other items that have contributed to the growth of precast, prestressed concrete industry, including many on bridge technology.

OTHER INFORMATION

https://static.tti.tamu.edu/tti.tamu.edu/documents/0-6958-R1.pdf
This is a link to a technical report developed by Texas A&M Transportation Institute titled *Developing Performance Specifications for High-Performance Concrete*. The report presents research results and tools to evaluate the expected durability of high-performance concrete mixtures in bridge decks.

https://store.transportation.org/Item/PublicationDetail?ID=5250
This is a link to the listing for the free download of the recently published *Resources for Concrete Bridge Design and Construction, 1st Edition*. The publication is the first product developed under an agreement between the American Association of State Highway and Transportation Officials and the National Concrete Bridge Council.

The Latest from the Concrete Bridge Engineering Institute

by Dr. Oguzhan Bayrak, Dr. Elias Saqan, Doug Beer, and Gregory Hunsicker, Concrete Bridge Engineering Institute

There has been a lot going on at the Concrete Bridge Engineering Institute (CBEI) in Austin, Tex., since the last report on the institute's activities (see the Summer 2024 issue of *ASPIRE*®). New components have been added to the concrete bridge component collection; the substructure for the bridge deck construction inspection training specimen has been constructed; CBEI has welcomed new team members; and new states have become members of the CBEI Transportation Pooled Fund. Additionally, the Concrete Materials for Bridges course, which was delivered four times in 2024, has been a big success and is now open to members of the public as well as the overall bridge community.

New Beams in the Concrete Bridge Component Collection

The Florida Department of Transportation (FDOT) recently donated three bridge beams to the CBEI concrete bridge component collection. Each of these beams is a unique addition to the collection, and FDOT has also shared valuable technical information based on their research as well as information about the components' historical significance. CBEI recognizes and is grateful for FDOT's extensive efforts to retain these specimens and relocate them to CBEI. CBEI is also thankful for the tremendous industry support from PCI, PCI's members, and the National Concrete Bridge Council (NCBC) that facilitated the effort to relocate the pieces. Brief descriptions of the specimens follow.

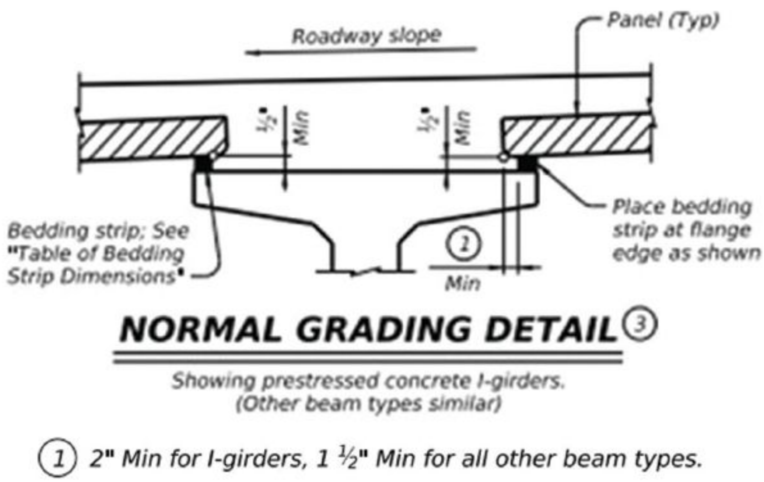
Early Florida Pretensioned Girder

One of the components donated by FDOT to CBEI is a bridge girder from State Road 72 (SR 72); it is an example of the earliest pretensioned concrete bridge girders used in the state. The SR 72 bridge was constructed in 1954 and was in service for nearly 55 years before being replaced. The bridge's original pretensioned girders with thin webs and limited vertical reinforcement provided an important opportunity to investigate shear capacity. In total, six girders were originally salvaged and tested. The results from load testing helped engineers determine the strength of similar existing girders in use on other structures. The FDOT project also evaluated the contribution to the shear capacity from the cast-in-place concrete bridge decks.[1] Results showed that despite the thin webs and minimal shear reinforcement, shear capacity exceeded the calculated nominal capacity.

Early Post-Tensioned Girder Replica

The second component donated by FDOT is a replica of an early (circa 1950s) post-tensioned concrete girder that was tested in an FDOT laboratory during research investigating shear

Bridge component specimens donated by the Florida Department of Transportation to the Concrete Bridge Engineering Institute component collection. Photos: Concrete Bridge Engineering Institute.



Current Texas Department of Transportation standard detail for a precast concrete deck panel.[5] Note the requirements for the bedding (bearing) strip.

capacity of different types of bridge girders.[2] The replica girders were constructed using the existing bridge plans and were tested in a three-point bending load setup. The I-girders were constructed with both parabolic and straight post-tensioning bars and minimal mild steel reinforcement in the end blocks. A three-point loading scheme was used to evaluate the girder shear behavior with a short shear span and no shear reinforcement.

Early Precast Concrete Deck Panel Application

The third FDOT specimen, which is the one most recently delivered to the CBEI component collection, was salvaged from the Interstate 75 overpass bridge on State Road 93 in Sarasota County, Fla. The component is an AASHTO Type III girder with a 3.5-in.-thick, partial-depth precast concrete deck panel and 3.5-in.-thick cast-in-place topping. The prestressed concrete beam and deck elements illustrate differences between the early use of precast concrete deck panels and current practices as specified by agencies such as the Texas Department of Transportation (TxDOT). The specimen will also be useful to help CBEI participants understand the importance of minimum requirements for bedding strip (the polystyrene between the deck panel and top flange of the girder) as discussed in the Bridge Deck Construction Inspection Program.

Bridge Deck Construction Inspection Program

The Bridge Deck Construction Inspection Program continues to make progress. Task group meetings have been conducted, and a final set of bridge construction plans has been completed. The drilled shafts have been constructed for the substructure, and CBEI is bringing a contractor on board to complete the remaining construction. Texas precast concrete fabricators organized through the Precast Concrete Manufacturers Association have agreed to donate the prefabricated elements for the structure, including the precast concrete panels, bent caps, and bridge girders. Valley Prestress, Bexar Concrete, Heldenfels Enterprises, Jobe Materials, Flexicore of Texas, and Legacy Precast have committed to supplying CBEI with the needed precast concrete elements, and CBEI is grateful for their contributions.

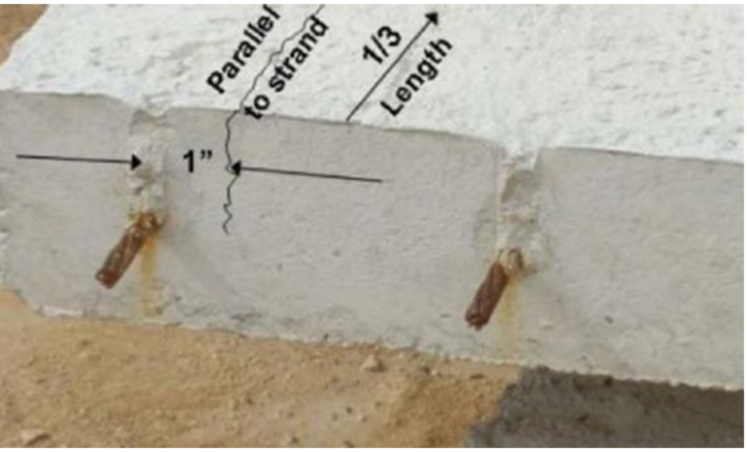
The first Bridge Deck Construction Inspection Program course is scheduled for March 2025. It will directly address training for bridge deck inspectors. Additionally, the course content will be of interest to engineers, inspectors, and contractors involved in bridge deck construction and inspection. The three days of training will include the following topics:

- Module 1: Minimum Bracing and Forming
- Module 2: Setting Forms and Grading
- Module 3: Reinforcement
- Module 4: Preplacement
- Module 5: Concrete Placement
- Module 6: Finish and Cure
- Module 7: Special Cases

Another anticipated topic in the Bridge Deck Construction Inspection course curriculum is on-site inspection of precast concrete deck panels for rejection or acceptance in accordance with TxDOT *Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges*[3] Item 424 Section 4.3.1, "Defects and Breakage," which states that prestressed concrete deck panels will be rejected for any of the following conditions:

- Cracks extending to the reinforcing plane and running parallel and within 1 in. of a strand for at least one-third of the embedded strand length
- Transverse or diagonal cracks, including corner cracks and breaks, intersecting at least two adjacent strands and extending to the reinforcing plane

Drilled shafts have been installed to support a full-sized specimen for the bridge deck construction inspection training course. Photo: Concrete Bridge Engineering Institute.



One of the anticipated topics in the CBEI Bridge Deck Construction Inspection course is the on-site inspection of precast concrete deck panels. Shown here are illustrations of types of cracks that may appear after the initial fabrication inspection. Photos: Texas Department of Transportation.

Initial inspection of precast concrete panels typically occurs at the precast concrete producer's facility through the use of industry guidance with the owner's quality assurance program. However, it is also important for inspectors and others on site to understand the criteria for acceptance with respect to the condition of a precast concrete panel and some of the defects that can occur during transport, storage, and erection.

The long-awaited training program for bridge deck construction inspection will enhance previous training classes with the CBEI full-scale bridge deck specimen. Training in both classroom and field environments will facilitate learning for field staff. In the past, inspectors and engineers were trained by a mentor in both the office and on site at multiple projects to gain the required knowledge and experience to perform their job duties. This traditional form of training can be efficient and can effectively achieve learning outcomes, but limited resources and volume of work have made it challenging to deliver. To address these challenges, TxDOT had a vision to construct a training facility displaying multiple phases of bridge construction, which would serve as the statewide training facility. Graham Bettis, former TxDOT Bridge Division director, and Reggie Holt of the Federal Highway Administration (FHWA) worked together to bring that vision to life. This training facility will allow learning in the classroom and from CBEI site visits. This combination will enhance the value of training in a controlled environment away from traffic hazards. When fully launched, the training program will be offered to pooled-fund members, departments of transportation (Texas, Georgia, Florida, Colorado, Tennessee, and Wisconsin), consultants, and contractors.

Concrete Materials for Bridges Course

The Concrete Materials for Bridges Course held its third and fourth training courses of its inaugural year on July 30–31 and October 29–30, 2024. Among the attendees were employees of the Texas, Georgia, Florida, Colorado, Tennessee, Michigan, Wisconsin, Minnesota, Iowa, Pennsylvania, Utah, and Nebraska departments of transportation. The course was recently opened to the public, and the participants in the October session included general contractors' employees. Many of the topics in the course are of value to individuals in a wide variety of professional roles and with varying types of experience. For example, materials engineers, inspectors, engineering consultants, and contractors may benefit from attending. Some of the topics such as mass concrete placement and use of ConcreteWorks software are important topics for contractors who need to develop and administer thermal control plans. The course content on the constituent materials currently available in the marketplace—such as ASTM C595, Standard *Specification for Blended Hydraulic Cements,*[4] Type IL, and other types of blended cements—is relevant and important to professionals who seek to understand current industry practices. The next training sessions are scheduled for the following dates:

- January 8–9, 2025
- April 29–30, 2025
- August 19–20, 2025

Feedback from course attendees has been tremendous, with participants sharing the following comments: "A full semester of a concrete materials course in just two days," "Concepts were clearly articulated," and "Very easy to understand, very relevant to the problems we see."

Research and New Technologies

TxDOT's NextGen Texas Bridge Deck research project (Project 0-7041) has been completed, and the final project report will be published soon. Project 0-7041 represents a significant endeavor aimed at developing the NextGen Texas Bridge Deck system. It is expected that the use of partial-depth panel pairs reinforced with wire trusses will eliminate the need for bracket-supported deck overhangs, an outcome that will reduce construction time and effort. Through a comprehensive research approach encompassing experimental testing and advanced analysis techniques, the project sought to evaluate the feasibility and performance of this innovative bridge construction method. Experimental tests conducted on wire truss–reinforced, partial-depth panels provided valuable insights into the components' structural behavior under various loading conditions, revealing a ductile load response and highlighting the influence of wire truss span on overhang deflections. Full-scale bridge deck tests demonstrated the feasibility of implementing the system and provided confidence about strength and serviceability. Furthermore, fatigue testing elucidated the system's endurance under cyclic loading, informing recommendations for reinforcement strategies and material selection. The culmination of these efforts resulted in a set of design recommendations and guidelines offering practical

The Concrete Bridge Deck Inspection Site will have a location for new bridge technologies, including the NextGen Deck Panels. Photo: Concrete Bridge Engineering Institute.

insights for implementing the NextGen Texas Bridge Deck system in accordance with TxDOT standards. These recommendations cover wire truss spacing, reinforcement details, panel dimensions, and construction considerations, and they provide engineers with valuable tools to streamline bridge construction processes and enhance overall project efficiency. Moving forward, the findings and recommendations from Project 0-7041 have the potential to transform bridge construction practices in Texas and beyond, offering a viable alternative to traditional methods and paving the way for safer, more efficient, and more resilient transportation networks.

The Concrete Bridge Deck Inspection Site will have a location for new bridge technologies that will include the NextGen Deck Panels. CBEI looks forward to featuring other research and innovative technologies to enhance the continuation of learning at the facility.

New CBEI Team Members

CBEI is excited to announce that Doug Beer, PE, recently joined the CBEI team. Beer retired from TxDOT in 2024 and brings tremendous experience to the CBEI team. In his most recent role as the Bridge Division Construction and Maintenance Branch manager, he oversaw the team of engineers and construction inspectors that provides support for the 25 TxDOT districts on matters related to bridge construction and maintenance. In his new CBEI role, Beer will continue to share his knowledge and passion for bridges with the bridge community in the upcoming courses. We expect Beer will be one of several key staff members in the deployment of the Bridge Deck Construction Inspection Training, which is set to launch in March 2025, and the Post-Tensioning Academy, which will be next held in the spring of 2026.

Thanks to all the current members for their participation, and welcome to the California Department of Transportation, which recently joined the CBEI Transportation Pooled Fund. Current members are the Iowa, Michigan, Minnesota, Georgia, Florida, Wisconsin, Nebraska, Pennsylvania, Utah, Colorado, Tennessee, California, and Texas departments of transportation, and FHWA. Many thanks to CBEI's industry partners American Segmental Bridge Institute, PCI, NCBC, and Post-Tensioning Institute for their continued support. Please visit www.cbei.engr.utexas.edu for more information about CBEI.

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Dr. Oguzhan Bayrak is a chaired professor at the University of Texas at Austin and director of the Concrete Bridge Engineering Institute. Dr. Elias Saqan is a senior engineering scientist and Doug Beer is a research engineer at the University of Texas at Austin. Gregory Hunsicker is a research engineer at the University of Texas at Austin and deputy director of the Concrete Bridge Engineering Institute.

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 third 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 underlying this strategy is that some trucks are—or will become—"smart," with the ability to drive long distances autonomously. With such intelligence, trucks will likely be able to 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 reinforced concrete T-beam bridge. The goal of that article was to provide an easy-to-follow example. We encourage readers to review the previous two articles for background relevant to this third article.

We need target reliability indices β to establish safe load limits that will not damage prestressed concrete girders. For example, for the strength limit state, $\beta = 3.5$ is often used for design, and $\beta = 2.5$ is used for load rating. However, there has not been much work on target reliability indices for Service Limit III. Wassef et al.[1,2] examined this issue in some detail with assumptions different from those for the 75-year design life. In the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*,[3] Service III was not formally calibrated, and load and resistance factors were based on judgment and experience.

In recent research,[4] we examined the estimated reliability index using the current AASHTO LRFD specifications and the 75-year design loads. Our goal was to determine whether the currently used index is acceptable for design as a benchmark for calibrating platoon permit load factors for service limits to be termed $\beta_{implicit}$. Once estimated, our calibration task was relatively straightforward. However, this task was also a significant one, and it produced information that could interest the design and rating communities. During the time that we were doing this work, there was also a robust debate—which continues to this day—about the computation of losses, the use of gross or transformed section, and live-load factors for design. Interim updates to the 9th edition of the AASHTO LRFD specifications (for example, Table 3.4.1-4) reflect this debate. The concepts here are also closely aligned with those issues. This article will focus on the reliability of different design options. The next article will address the determination of $\beta_{implicit}$ and the probability of cracking during a bridge's service life.

Background

Studies have investigated potential target β values for Service III, Wassef et al.[1,2] assumed that bridges designed based on the prestress loss method in the *AASHTO Standard Specifications for Highway Bridges*[5] performed well in service. To characterize live load means and coefficients of variation (CoVs), service evaluations were performed using weigh-in-motion (WIM) data for a site with an annual average daily truck traffic (ADTT) of 5000. Target $\beta_{implicit}$ indices were recommended for bridges according to various performance limit criteria (for example, decompression and maximum tensile stress). Wassef et al.[1] recommended a live load factor of 1.0 when the refined time-dependent loss method is used with elastic gains. Table 3.4.1-4 in the AASHTO LRFD specifications gives the Service III live load factor as either 0.8 or 1.0 depending on loss method and whether elastic gains are included.

Barker et al.[6] evaluated Service III for Wyoming bridges under large traffic volumes due to detours created by extended roadway closures that resulted in long strings of trucks. It had been assumed that bridges designed and evaluated under criteria from the current AASHTO LRFD specifications would perform satisfactorily. However, using Interstate 80 WIM vehicle load characteristics, roadway closures created load effects that resulted in negative Service III reliability indices. This research indicates that some reliability concerns may exist for heavy-load truck-train situations.

Reliability-based evaluation for Service III and its association with performance objectives, such as limiting cracking probability, are currently not clearly established in the AASHTO LRFD specifications, and that gap in the specifications hampers optimal truck platoon deployment. Furthermore, the flexibility afforded to bridge designers by language in the AASHTO LRFD specifications—such as selecting a prestress loss calculation method, using gross or transformed section properties, and determining whether elastic gains should be included—creates ambiguity concerning the level of performance intended by the specifications.

Our study focused on the following three objectives:

- Analyze the reliability of different design options.
- Determine $\beta_{implicit}$ for Service III.
- Investigate cracking probability under current design live load conditions during bridge service lives.

Sample results for the first objective are presented in this article.

General Reliability Analysis Procedure

Structural reliability analysis can determine whether the probability of exceeding a limiting criterion is acceptable. Equation (1) represents a general limit state function:

$$g(R,Q) = R - Q \quad (1)$$

where R and Q are random variables representing resistance and load effects, respectively. Monte Carlo simulation (MCS) is commonly used to conduct reliability analyses.[7] The index is routinely used in structural reliability frameworks to characterize structural safety (that is, strength limit states). **Figure 1** presents typical probability density functions (PDFs) for loads, resistance, and the strength limit state function. β represents the number of standard deviations that the mean value of $g(R,Q)$ is from zero (Fig. 1). At the strength level, design and inventory load ratings target $\beta = 3.5$, whereas operating load ratings relax the target β to 2.5. (See our article in the Summer 2024 issue of *ASPIRE* for details on the computation of β.)

Similar to Wassef et al.[1,2] and Barker et al.,[6] we assumed in this study that the $\beta_{implicit}$ based on current and past design criteria was adequate to provide satisfactory in-service performance. Nominal demands, live loads, and resistances were mapped onto probabilistic distributions with characteristic means and CoVs, and the probability of failure for each parametric combination was calculated using MCS with $N = 1$ million samples to determine β.

This process was repeated to conduct reliability analysis for different design scenarios, identify a $\beta_{implicit}$, and investigate $\beta_{cracking}$ for optimally designed prestressed concrete bridges for service. For this article, $\beta_{cracking}$ is defined based on a bridge designed using a tensile stress limit $0.948\sqrt{f'_c}$ (where f'_c is the design concrete compressive strength in ksi) and evaluated using a range of rupture moduli between $0.24\sqrt{f'_c}$ and $0.37\sqrt{f'_c}$.

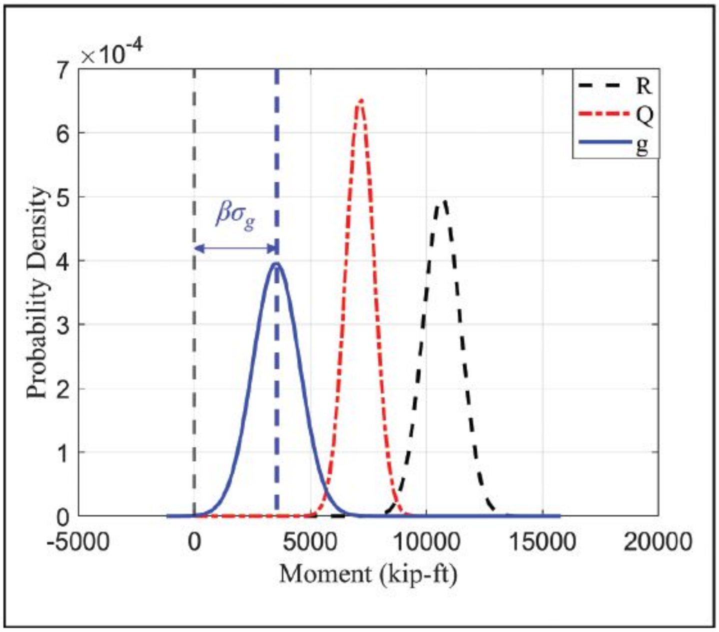

Figure 1. Example of probability density functions of load and resistance illustrating the reliability index for the strength limit state. All Figures: University of Nebraska.

Bridge Parameters

The AASHTO LRFD specifications, AASHTO standard specifications, and *Manual for Bridge Evaluation*[8] address tensile stress limits in positive moment regions. Accordingly, this study considered only positive moments at the middle of simple-span prestressed concrete girder composite bridges, and at 40% of the span length from supporting abutments for two equal-span, simple-made-continuous, prestressed concrete girder composite bridges. Bridges were assumed to carry two traffic lanes, and interior girders were designed using Nebraska University (NU) I-girders. The details are as follows:

- 60, 90, 120, and 150 ft spans
- NU 900 (60 ft), 1100 (90 ft), 1600 (120 ft), and 2000 (150 ft)
- 5 girders
- Girder spacings of 6, 8, 10, and 12 ft
- Composite deck thickness of 8.5 in. with 0.5 in. wear-off and 2 in. asphalt wearing surface
- 1 in. deck haunch
- 3.5 ft deck overhang
- 1-ft 5-in.-wide barrier with a weight of 0.124 kip/ft per girder
- Simple and continuous spans

Four bottom-fiber tension stress limit values f_t were considered for service designs. For simplicity, the coefficient before $\sqrt{f'_c}$ in the design f_t is referred to herein as κ. Preliminary analyses indicated that service reliability results for optimally designed bridges were not sensitive to final and initial concrete strength parameters. Initial concrete strength was used to calculate losses based on the refined time-dependent loss method in the AASHTO LRFD specifications. Parameters considered for κ, f_t, f'_c, and f'_{ci} included the following:

$f_t = \kappa\sqrt{f'_c}$ ksi, where κ = 0, 0.0948, 0.19, and 0.24

$f'_{c\_girder} = 8$ ksi

$f'_{ci\_girder} = 5$ ksi

$f'_{c\_deck} = 4$ ksi

Table 1 presents the design scenarios considered. Six different methods were used to design the girders optimally. Then, optimal designs were evaluated using MCSs for reliability analysis. In summary, hundreds of designs were evaluated with thousands of MCSs to yield the statistical data to compute the implicit reliability index $\beta_{implicit}$.

The design permutations totaled 96. However, the reliability index computations were observed to be insensitive to girder spacings; therefore, data presented here are limited to the 10 ft girder spacing.

The primary case considered was Post-1.0-Gains. Researchers also considered the Post-0.8-Gains scenario, representing how some bridges might have been designed in the period between the time when the post-2005 losses were included in the AASHTO LRFD specifications and the time when the required design γ_L was increased to 1.0. The Post-0.8-No-gains scenario is consistent with Table 3.4.1-4 in the AASHTO LRFD specifications. The use of approximate loss methods with or without elastic gains was interpreted by the investigators as an instance of ambiguity or subtlety in the AASHTO LRFD specifications, as Commentary C5.9.3.3 (Approximate Estimate

Table 1. Design scenarios for bridges

Scenario name	Loss computation	Design live load factor γ_L	Elastic gains considered?	Comment
Post-1.0-Gains	Post-2005 loss method (AASHTO LRFD specifications Article 5.9.3.4)	1.0	Yes	AASHTO LRFD specifications Table 3.4.1-4: Prestressed concrete components designed using refined estimates for time-dependent losses in Article 5.9.3.4 in conjunction with elastic gains
Post-0.8-Gains	Post-2005 loss method (AASHTO LRFD specifications Article 5.9.3.4)	0.8	Yes	Same as Scenario Post-1.0-Gains with a lower live-load factor
Post-0.8-No-gains	Post-2005 loss method (AASHTO LRFD specifications Article 5.9.3.4)	0.8	No	AASHTO LRFD specifications Table 3.4.1-4: All other prestressed concrete components
Approx-0.8-Gains	Approximate loss method (AASHTO LRFD specifications Article 5.9.3.3)	0.8	Yes	AASHTO LRFD specifications Table 3.4.1-4: All other prestressed concrete components
Approx-0.8-No-gains	Approximate loss method (AASHTO LRFD specifications Article 5.9.3.3)	0.8	No	AASHTO LRFD specifications Table 3.4.1-4: All other prestressed concrete components
Pre-1.0-No-gains	Pre-2005 loss method	1.0	No	AASHTO standard specifications Article 9.16.2.1

of Time-Dependent Losses) notes that "the losses or gains due to elastic deformations at the time of transfer or load application should be added to the time-dependent losses to determine total losses. However, these elastic losses (or gains) must be taken equal to zero if transformed section properties are used in stress analysis." After this work,[4] the *PCI Bridge Design Manual* clarified γ_L in Table 8.2.2.6-1.[9] Due to higher prestress loss predictions and a design live-load factor of 1.0, bridges designed using the Pre-1.0-No-gains scenario typically required the most strands.

Limit State Function for Service III

Reliability analyses were implemented by using nominal values and statistical parameters. HL-93 loading, girder distribution factors GDF_m, and dynamic load allowance IM from the AASHTO LRFD specifications were applied.

Equation (1) represents the general Service III limit state function. The present study expanded the equation as follows:

$$g = R - Q = \left(f_t \frac{P_{eval}}{A_g} + \frac{P_{eval} e_{nc}}{S_{ncb}} \right) - \left(\frac{\left(D_{gw} + D_{nc}\right)}{S_{ch}} + \frac{\left(D_c + D_w\right)}{S_{ch}} + \frac{LL_{HL\text{-}93}\left(1+IM\right)GDF_m}{S_{ch}} \right) \quad (2)$$

$$P_{eval} = A_{ps}\left(f_{pi} - \Delta f_{S_{eval}}\right) = A_{ps}\left(f_{pi} - \left(\Delta f_{pES} + \Delta f_{pLT} - \Delta f_{GainDL} - \Delta f_{GainLL}\right)\right) \quad (3)$$

$$f_{Reval} = f_t + \frac{P_{eval}}{A_g} + \frac{P_{eval} e_{nc}}{S_{ncb}} \quad (4)$$

where

P_{eval} = effective prestressing force for the evaluation of reliability index β

Δf_{Seval} = prestress loss for the evaluation of reliability index β

Δf_{pES} = elastic shortening loss

Δf_{pLT} = long-term prestress loss

Δf_{GainDL} = elastic gain from dead loads

Δf_{GainLL} = elastic gain from live loads

f_{Reval} = available resistance to tension stress for evaluation of reliability index β, which can be determined using Eq. (4).

Optimal Designs

The AASHTO LRFD specifications were used to determine the optimal numbers of strands for each prestress loss method, increased with span, with the design tensile stress parameter increased from $\kappa = 0$ to $\kappa = 0.24$. Fractional strands were used to avoid rounding issues. This article presents example summaries of these designs. The designs were then evaluated to determine the reliability indices for the Service III and cracking. Here, we focus on the design comparisons using the different assumptions. Different design assumptions lead to different reliabilities, which will be presented in the next article.

As one example, f_t with $\kappa = 0.0948$ is used to show intermediate calculation results for design and evaluation.

Figure 2 presents the optimal area of prestressing steel A_{ps} for f_t with $\kappa = 0.0948$, using an initial tensioning stress of $0.75f_u$. For 60 ft bridges, A_{ps} was close for different loss methods, but A_{ps} varied more across design cases when span lengths increased. A_{ps} for the Approx-0.8-Gains and Post-0.8-Gains scenarios were generally close and smaller than A_{ps} for the

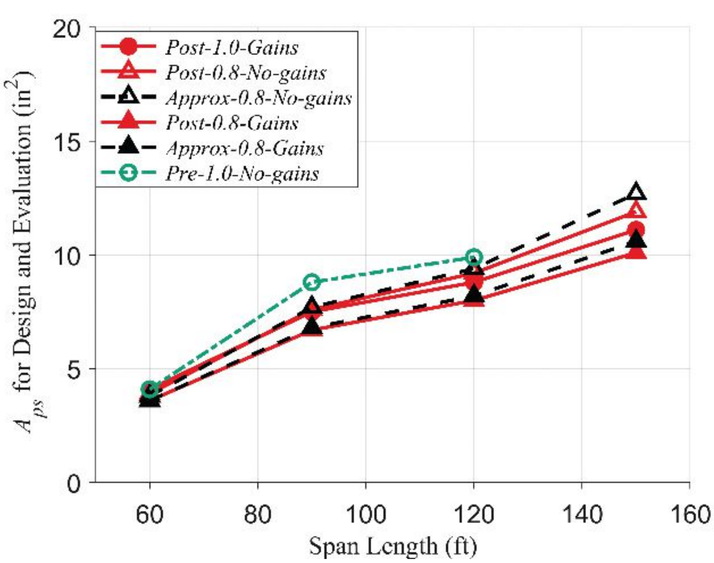

Figure 2. Optimal area of prestressing steel A_{ps} for various design scenarios.

Post-1.0-Gains option. Without considering elastic gains, A_{ps} for 150-ft-span bridges designed with approximate and post-2005 loss methods (γ_L = 0.8) was slightly larger than that for the Post-1.0-Gains method. Generally, the Pre-1.0-No-gains scenario produced larger A_{ps} than other methods. The Post-1.0-Gains scenario predicted the lowest losses, and the Pre-1.0-No-gains scenario gave the highest. The approximate and post-2005 methods predicted similar losses for cases using the same design live load factor and the same consideration of elastic gains.

Other design cases for f_t with κ ranging from 0.0 to 0.24 and as provided in Table 1 are found in Steelman et al.[4]

Summary

In this article, we presented the optimal designs used to evaluate platoon permits for Service III. The various design options with the AASHTO standard specifications and AASHTO LRFD specifications were used. This article summarizes the effects of a variety of typical design assumptions on the required A_{ps} for one span length; other spans are presented in Steelman et al. and Yang et al.[4,10] The next article will extend this work to evaluate the reliability indices for Service III and cracking limits.

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Shear-Friction Design Provisions of the *AASHTO LRFD Bridge Design Specifications*

by Dr. Oguzhan Bayrak, University of Texas at Austin

Among the recent questions received by our *ASPIRE®* team are several related to shear-friction design. More specifically, we are hearing from members of the concrete bridge community about current challenges in bridge-widening projects, the design and detailing of precast concrete spliced-girder bridges, and a variety of complex bridge applications. With these issues serving as a backdrop, this article discusses the shear-friction design provisions in the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[1] and the performance of those provisions in light of some recent test data for spliced-girder bridges.

What Do the AASHTO LRFD Specifications Say?

Article 5.7.4 of the AASHTO LRFD specifications states the following:

Interface shear transfer shall be considered across a given plane at:

- *An existing or potential crack;*
- *An interface between dissimilar materials;*
- *An interface between two concretes cast at different times; or*
- *The interface between different elements of the cross-section.*

Article 5.7.4.3 of the specifications is devoted to interface shear resistance, as follows:

The factored interface shear resistance, V_{ri}, shall be taken as:

$$V_{ri} = \phi V_{ni} \qquad (5.7.4.3\text{-}1)$$

and the design shall satisfy:

$$V_{ri} \geq V_{ui} \qquad (5.7.4.3\text{-}2)$$

where:

V_{ni} = *nominal interface shear resistance (kip)*

V_{ui} = *factored interface shear force due to total load based on the applicable strength and extreme event load combinations in Table 3.4.1-1 (kip)*

ϕ = *resistance factor for shear specified in Article 5.5.4.2. For the extreme limit state event ϕ may be taken as 1.0.*

The nominal shear resistance of the interface plane shall be taken as:

$$V_{ni} = cA_{cv} + \mu(A_{vf}f_y + P_c) \qquad (5.7.4.3\text{-}3)$$

The nominal shear resistance, V_{ni}, used in the design shall not exceed either of the following:

$$V_{ni} \leq K_1 f'_c A_{cv} \qquad (5.7.4.3\text{-}4)$$

$$V_{ni} \leq K_2 A_{cv} \qquad (5.7.4.3\text{-}5)$$

in which:

$$A_{cv} = b_{vi}L_{vi} \qquad (5.7.4.3\text{-}6)$$

where:

b_{vi} = *interface width considered to be engaged in shear transfer (in.)*

L_{vi} = *interface length considered to be engaged in shear transfer (in.)*

c = *cohesion factor specified in Article 5.7.4.4 (ksi)*

μ = *friction factor specified in Article 5.7.4.4*

P_c = *permanent net compressive force normal to the shear plane; if force is tensile, P_c = 0.0 (kip)*

f'_c = *design concrete compressive strength of the weaker concrete on either side of the interface (ksi)*

K_1 = *fraction of concrete strength available to resist interface shear, as specified in Article 5.7.4.4.*

K_2 = *limiting interface shear resistance specified in Article 5.7.4.4 (ksi)*

If a member has transverse reinforcement with a specified minimum yield strength greater than 60.0 ksi for flexural shear resistance, interface reinforcement may be provided by extending the transverse reinforcement across the interface zone. In this case, the value of f_y in Eq. 5.7.4.3-3 shall not be taken as greater than 60.0 ksi.

Design codes around the globe have design provisions similar to those in the AASHTO LRFD specifications. To

Table 1. Shear-friction design provisions

Specification	Nominal Shear Resistance, V_{ni}
ACI 318-19	$\mu(A_{vf}f_y + P_c)$
AASHTO LRFD	$cA_{cv} + \mu(A_{vf}f_y + P_c)$
Eurocode 2	$cf_{ctd}A_{cv} + \mu(A_{vf}f_y + P_c)$

c = Cohesion factor
μ = Coefficient of friction
Specified in Individual Codes

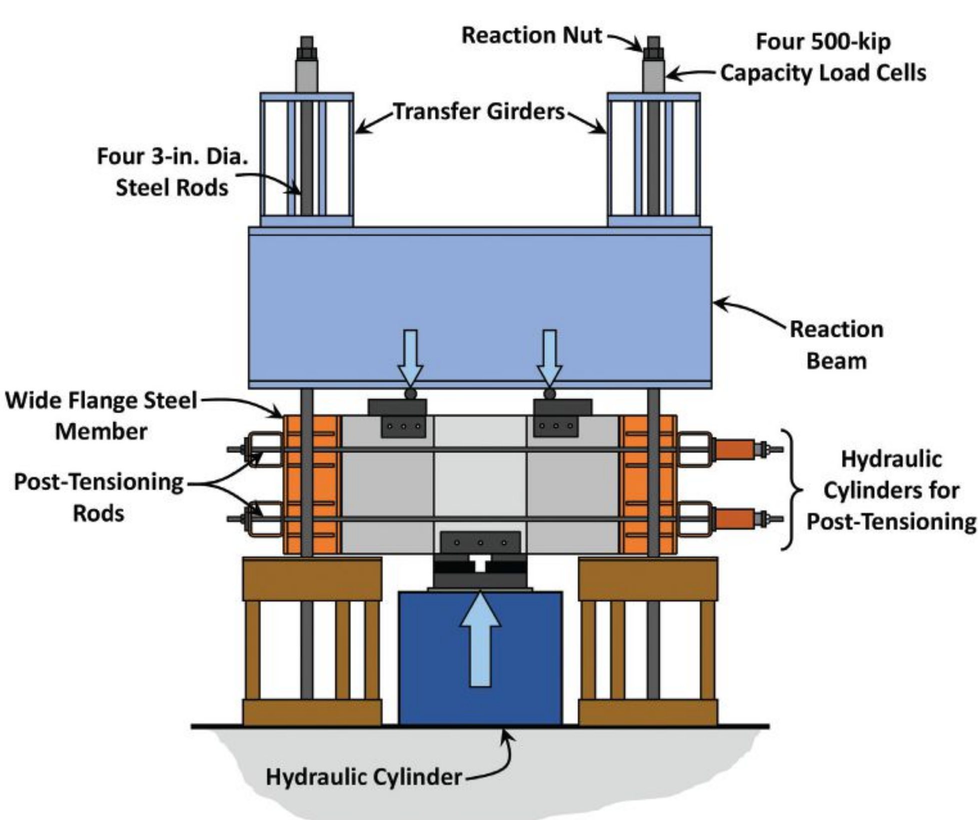
Surface Detail	c			μ		
	ACI 318	AASHTO LRFD	Eurocode 2	ACI 318	AASHTO LRFD	Eurocode 2
Not Intentionally Roughened or Smooth	-	0.075 ksi	0.10	0.6	0.6	0.5
Int. Roughened or Indented	-	0.24 ksi	0.50	1.0	1.0	0.9

provide a basis of comparison in light of some recent experimental data, we will focus our attention in this article on the relevant provisions of the American Concrete Institute's *Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary (ACI 318R-19)*[2] and *Eurocode 2: Design of Concrete Structures*,[3] in addition to the AASHTO LRFD specifications. **Table 1** succinctly summarizes the applicable design expressions for nominal interface shear resistance. For brevity, additional design provisions are not repeated herein, and readers are directed to the original design references. Note that some of the terminology in Table 1 has been modified from the original references to match the terms used in the AASHTO LRFD specifications.

As is the case in most structural engineering applications, the devil is in the details. That is to say, the presence or absence of cohesion terms, the actual value of the terms and coefficients, and the assumptions that go into calculating the net compression acting on the shear-friction interface are all important. In this context, I will reference a recent project we completed at the Phil M. Ferguson Structural Engineering Laboratory at the University of Texas and some shear-friction tests we conducted in an effort to define the best practices that can be employed in detailing the cast-in-place closure pours in precast concrete spliced-girder bridges.[4–7] As part of our comprehensive research effort, we conducted several tests to study the behavior of the splice region in spliced-girder bridges. **Figure 1** illustrates the test setup used to determine the structural capacity of a variety of cold-joint details and clamping forces through active (applied force) and passive (reinforcement) means. In all, we tested 11 assemblies in 2 series (or sets). **Figure 2** shows the details of the joints examined in series 1 test specimens that were used to examine the geometry of the interface details. Series 2 was devoted to testing the performance of the design expressions for cases in which the active clamping force (applied by using hydraulic rams) across the boundary and passive clamping force provided by mild reinforcement were employed in various combinations. All series 2 test specimens had a single shear key and a sum of active and passive clamping forces between 61 and 251 kips acting across the boundary (**Fig. 3**). Figure 3 shows that, in some cases, the clamping forces across the boundary were provided by different percentages of active and passive components. The passive clamping forces were adjusted by varying the quantity of reinforcement that crossed the interface, and active clamping forces were adjusted by varying the active post-tensioning force, where applicable. Williams[6] provides complete details of the test specimens, which are explained concisely in Williams et al.[7] The summary provided herein serves to establish the backdrop for the discussion on various design expressions in the AASHTO LRFD specifications, ACI 318 and Eurocode 2.

As can be observed in **Fig. 4**, the shear-friction design expressions in the AASHTO LRFD specifications provide the most accurate strength estimates with the least scatter for the 11 test specimens considered in our study. Eurocode 2 expressions display a similar performance to those in the AASHTO LRFD specifications but are more scattered. Interestingly, both the AASHTO LRFD specifications and Eurocode 2 have a "cohesion" term, cA_{cv} and $cf_{ctd}A_{cv}$, respectively, which I personally like to call "concrete contribution to shear friction." Adhesion of two concretes to each other in the boundary (commonly understood to be cohesion) must be overcome to activate the contribution of the reinforcing steel crossing that boundary. At this point the "cohesion" term references aggregate interlock. In my view, the phrase "concrete contribution" seems more appropriate than the term "cohesion" since that "contribution" remains active after the cohesion is overcome and "sliding" becomes activated—that is, both the reinforcement crossing the boundary and the aggregate interlock across a rough surface concurrently exist. In Eurocode 2, f_{ctd} refers to the design value of the direct tensile strength of concrete. With semantics and terminology left aside, and unlike the provisions in the AASHTO LRFD specifications and Eurocode 2, ACI 318 provides a more conservative estimate, likely due to the absence of the cohesion term and perhaps because additional considerations likely went into

Figure 1. Test setup. All figures: C. S. Williams and Phil M. Ferguson Structural Engineering Laboratory researchers at the University of Texas at Austin.[4–7]



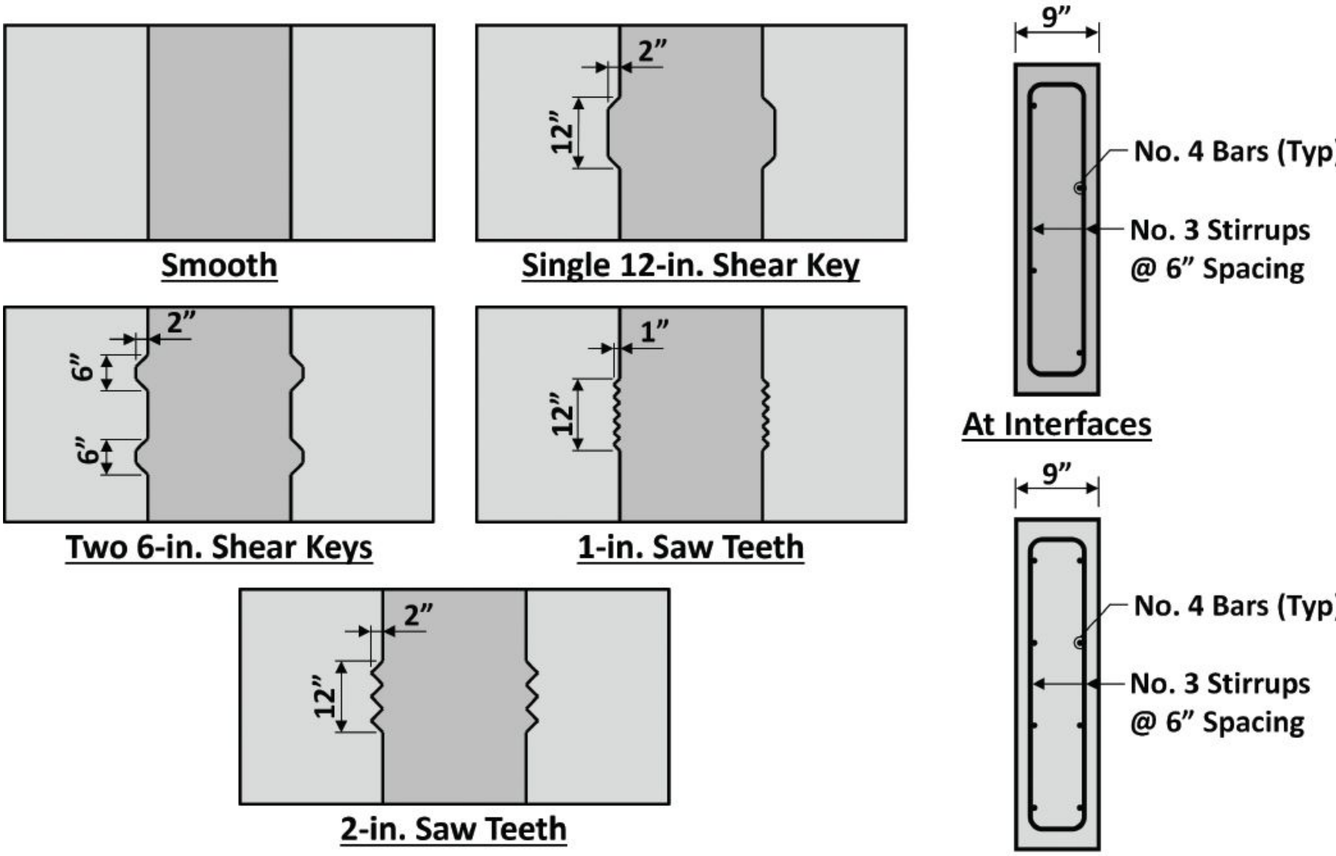
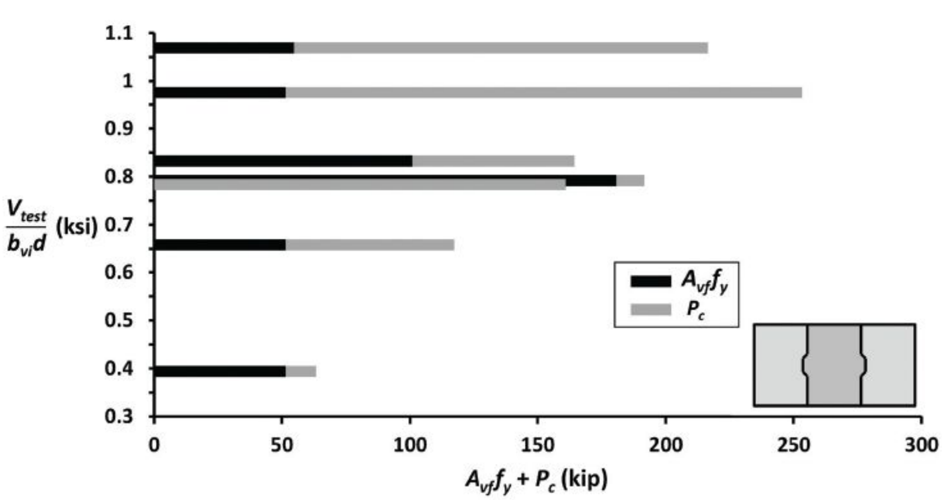


Figure 2. Joint details of series 1 test specimens.

calibrating ACI 318 provisions, such as simplicity of the equations, calibration of load factors, strength reduction factors, and so on.

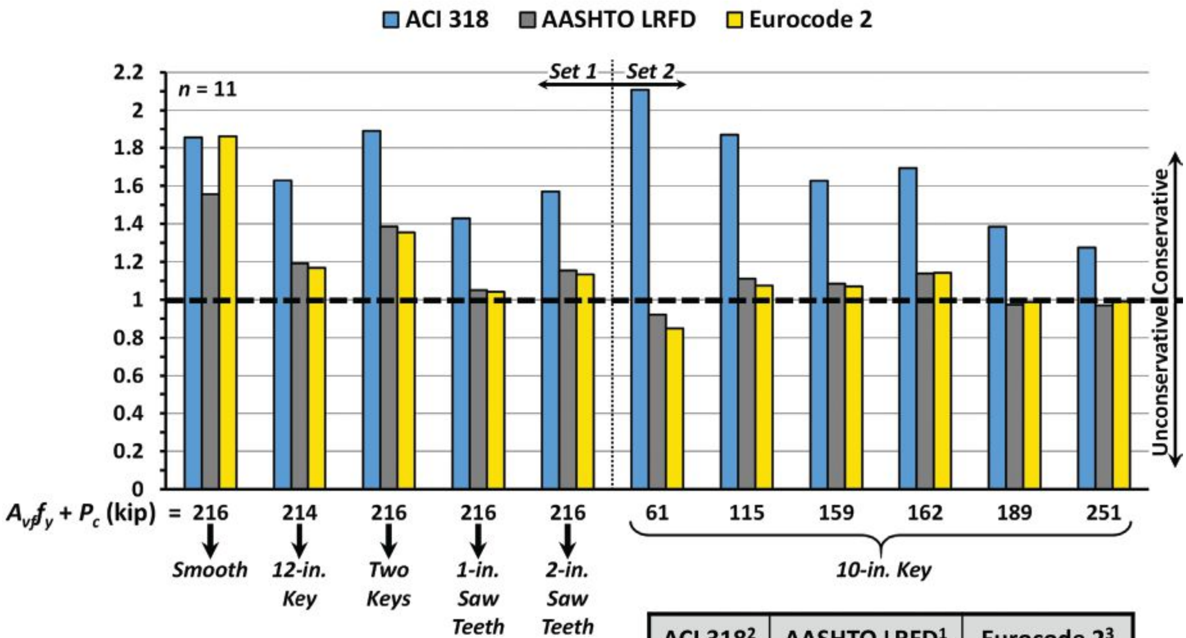
It is important to appreciate that the 11 test specimens considered in our study[4–7] were not intended to be exhaustive of all cases. Instead, they were representative of the spliced-girder details employed in various jurisdictions. With that disclaimer, it is comforting to note the superior performance of the design expressions from the AASHTO LRFD specifications. The overall calibration of design expressions would require more than just the 11 tests and was considered beyond the scope of our efforts. However, overall examination (or reexamination) of shear-friction expressions in the AASHTO LRFD specifications must consider tests that are representative of bridge applications.

Figure 3. Active (force) and passive (reinforcement) contributions to the "clamping" force of each series 2 test specimen.



As we go from experimental observations to a design setting, it is important to appreciate that the calibration of each design code differs. For the AASHTO LRFD specifications, we should keep in mind that if the factored loads are applying both a "clamping effect" (capacity side) and a "factored shear load" (demand side) on the boundary (for example, a cold joint), we must use the applicable load factors. Permanent loads acting on the boundary deserve a different load factor when they appear on the capacity (less than 1) or demand (greater than 1) side of the expressions from the AASHTO LRFD specifications.

This article is not intended to cover all of the articles and commentary that apply to shear-friction (Article 5.7.4 of the AASHTO LRFD specifications). Instead, the focus is placed on the conservativeness and accuracy of the relevant design provisions as



	ACI 318[2]	AASHTO LRFD[1]	Eurocode 2[3]
Mean	1.67	1.14	1.15
Standard Deviation	0.24	0.18	0.25

Figure 4. Comparison of shear-friction design expressions and test results for the 11 test specimens.

they apply to typical joints used in precast concrete spliced-girder bridges. Overall, our recent testing validates both the theoretical basis and the calibration of relevant design expressions in the AASHTO LRFD specifications.

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Concrete Bridge Shear Load Rating: Updates to the AASHTO *Manual for Bridge Evaluation* and Further Guidance

by Dr. David Garber and Dr. Lubin Gao, Federal Highway Administration

Shear design procedures have evolved since the American Association of State Highway and Transportation Officials (AASHTO) first released its *Standard Specifications* in 1931. This evolution of shear design provisions has led to reinforced and prestressed concrete bridges with different section shapes and types as well as different quantities of transverse shear reinforcement. It has been challenging to load rate these structures consistently and accurately for shear.

Several recent publications have investigated the history of shear load rating and developed recommended procedures for load rating concrete structures for shear.[1–4] The recommended procedure for shear load rating uses the modified compression field theory (MCFT), which was first introduced into the *AASHTO LRFD Bridge Design Specifications* in 1994.[5]

The recent Federal Highway Adminstration (FHWA) report *Modified Compression Field Theory (MCFT) for Shear Load Rating – Pretensioned Example*[6] provides guidance on using MCFT for shear load rating of pretensioned concrete components. The report includes a summary of MCFT theory and application, possible shear failure mechanisms that may occur in pretensioned components, and a shear load rating example based on Example 9.4 from the *PCI Bridge Design Manual*.[7]

Shear Failure Mechanisms

There are three general shear failure mechanisms that may control the calculated resistance for pretensioned concrete components:

- Sectional shear resistance
- Longitudinal tension resistance
- Horizontal shear resistance

Figure 1 illustrates these three failure mechanisms. Sectional shear resistance (Fig. 1 [a]) is typically associated with a diagonal shear crack leading to either crushing of the web concrete or yielding of the transverse reinforcement and sliding along the shear crack.

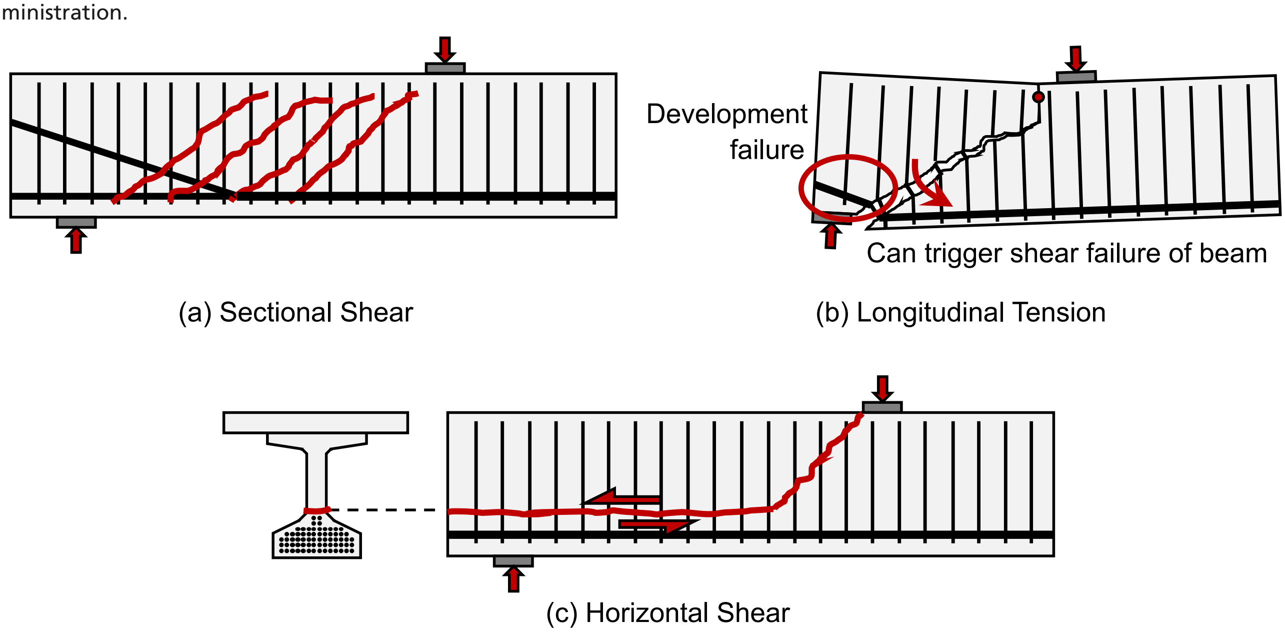
If properly developed or anchored, the longitudinal tension reinforcement will work with the transverse reinforcement to hold a diagonal shear crack closed and allow the concrete shear resistance to engage. If the amount of developed longitudinal tension reinforcement is insufficient, diagonal shear cracks can open and lead to a shear failure (Fig. 1 [b]).

Article 5.7.4.1 of the AASHTO LRFD specifications[8] states that interface shear transfer (that is, shear friction) shall be considered across a given plane at:

- *An existing or potential crack;*
- *An interface between dissimilar materials;*
- *An interface between two concretes cast at different times; or*
- *The interface between different elements of the cross section.*

Interface shear transfer in bridge design is

Figure 1. Three possible shear failure mechanisms related to modified compression field theory. All Figures: Federal Highway Administration.



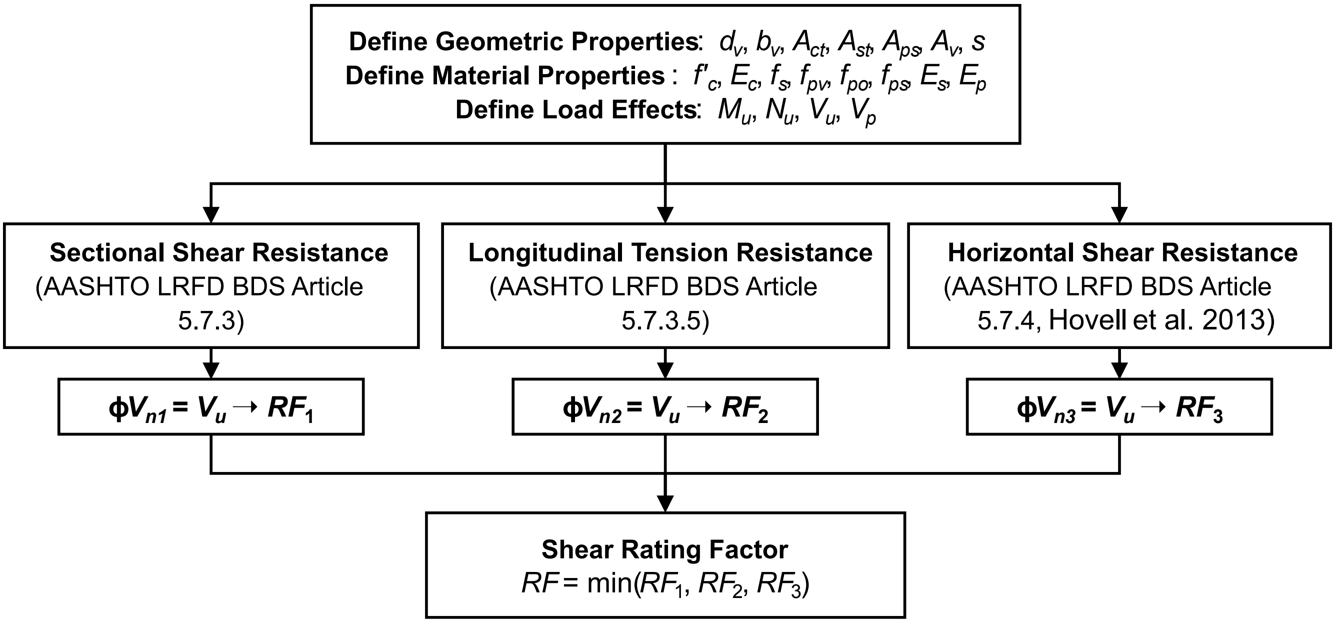


Figure 2. Flowchart for shear load rating using modified compression field theory and the procedure proposed in *Concrete Bridge Shear Load Rating Guide and Examples: Using the Modified Compression Field Theory*.[3]

most frequently checked in relation to the interface between a cast-in-place composite concrete deck and the top flange of a precast concrete beam, but it also needs to be considered at any other sections where it may be critical. For a prestressed concrete beam, one interface that should be considered is the interface between its thin web and the heavily prestressed bottom flange (Fig. 1 [c]). This type of shear failure, called a horizontal shear failure, has been shown to control the Strength I limit state design for some U-beams and bulb-tee beams.[9,10]

While lack of sectional shear resistance may be the most common cause of shear distress observed in in-service bridges, there have also been cases of distress due to inadequate anchorage or bond of longitudinal tension reinforcement.[11,12] All three possible failure mechanisms should be considered when load rating for shear.

Shear Resistance Corresponding to Each Failure Mechanism

The overall shear resistance and associated shear rating factor *RF* are calculated considering each of the three aforementioned failure mechanisms (**Fig. 2**). The procedures for calculating the sectional shear resistance and longitudinal tension resistance are iterative as the resistance in each case is a function of the demand. A higher shear and moment demand will result in a smaller shear resistance. The shear and moment demand consist of those caused by dead loads, which will remain constant, and those caused by live loads, which can be increased. For the iterative procedures, the shear and associated moment caused by live load are increased or decreased until the calculated shear resistance is equal to the assumed shear demand. The horizontal shear resistance can be calculated directly using Article 5.7.4 of the AASHTO LRFD specifications with the modifications recommended by Hovell et al. (2013).[9] The rating factor associated with each of these failure mechanisms can be determined using the calculated shear resistance, dead-load shear, and live-load shear caused by either operating- or inventory-level loading. The minimum rating factor for the three failure mechanisms is the shear rating factor at the section of interest. This process must be repeated at multiple sections along the length of a component. More details on the procedure and a pretensioned concrete example are provided in *Modified Compression Field Theory (MCFT) for Shear Load Rating – Pretensioned Example*.[6]

Approved Revisions to the *Manual for Bridge Evaluation*

This new procedure will be reflected in the next release of AASHTO's *Manual for Bridge Evaluation* (MBE).[13] The evaluation for shear in load rating is specified in Article 6A.5.8 of the MBE and references Article 5.7.3.6.3 of the AASHTO LRFD specifications. Revisions to the MBE based on the FHWA report *Concrete Bridge Shear Load Rating Guide and Examples: Using the Modified Compression Field Theory*[3] were approved in June 2022. The following are the primary updates to the MBE shear load rating:

- Article C6A.4.2.1 describes the iterative procedure needed for shear load rating using MCFT.
- Articles 6A.5.8 and C6A.5.8 provide additional details on how to determine the shear load rating considering the longitudinal reinforcement requirement of Article 5.7.3.5 of the AASHTO LRFD specifications. An equation is provided in the MBE commentary to calculate the rating factor based on this check.
- MBE allows two modifications to Article 5.7.3.4.2 of the AASHTO LRFD specifications, as follows:
  - ε_s may be taken as zero if $M_u \leq M_{cr}$.
  - The β factor for prestressed concrete components (where $f_{pc} / f'_c \geq 0.02$) may be calculated using Eq. 5.7.3.4.2-1 of the AASHTO LRFD specifications, regardless of whether there is minimum transverse reinforcement provided.
- MBE adds a provision specifying that concurrent load effects should be used in shear load rating analyses. This means that the moment and shear used in the shear load rating should be from the same placement of the live load.

Conclusion

By considering the primary failure and resistance mechanisms, the procedure developed in *Concrete Bridge Shear Load Rating Guide and Examples: Using the Modified Compression Field Theory* improves the consistency and accuracy of shear load rating of concrete structures. Additional guidance and a pretensioned concrete example problem are provided in *Modified Com-*

pression Field Theory (MCFT) for Shear Load Rating – Pretensioned Example.

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Richard Miller, PhD, PE, FPCI, is Professor Emeritus and former head of the Department of Civil and Architectural Engineering and Construction Management at the University of Cincinnati, where he taught for 36 years. Dr. Miller's research focuses on concrete materials and prestressed concrete bridges. He has been principal or co-principal investigator on seven projects for the National Cooperative Highway Research Program. Work performed by Dr. Miller and his colleagues has resulted in numerous changes to the *AASHTO LRFD Bridge Design Specifications*, including incorporation of high-strength reinforcing bar and provisions on debonded strands and continuous for live-load bridges. Dr. Miller has also completed numerous projects for the Ohio Department of Transportation and the Federal Highway Administration related to concrete bridges. He has served on and chaired several PCI councils and committees and currently serves on the PCI Board of Directors as the chair of the Technical Activities Council. He is a Fellow of PCI, and in 2024 he was named a PCI Titan of the Industry.

Clay Naito, PhD, PE, FPCI, is a professor of structural engineering at Lehigh University in Bethlehem, Pa., where he has taught for 22 years. Dr. Naito's research focuses on experimental and analytical evaluation of reinforced and prestressed concrete structures subjected to extreme events, including earthquakes, tsunamis, and intentional blasts. He has also conducted research studies for the Pennsylvania Department of Transportation, the Federal Highway Administration, and the Precast/Prestressed Concrete Institute on the performance of concrete bridge structures. Research topics include the performance of adjacent box-beam bridges, integration of electrically isolated tendons, use of self-consolidating concrete and ultra-high-performance concrete in bridges, and strand bond. He received the Distinguished Educator Award from PCI in 2015 and was elected Fellow of PCI in 2019.

Resources

PTI/ASBI Specification for Multistrand and Grouted Post-Tensioning

PTI Specification for Grouting of Post-Tensioned Structures, 4th Edition

FHWA Replaceable Grouted External Post-Tensioned Tendons

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