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October 2016 Bridges

Inside:

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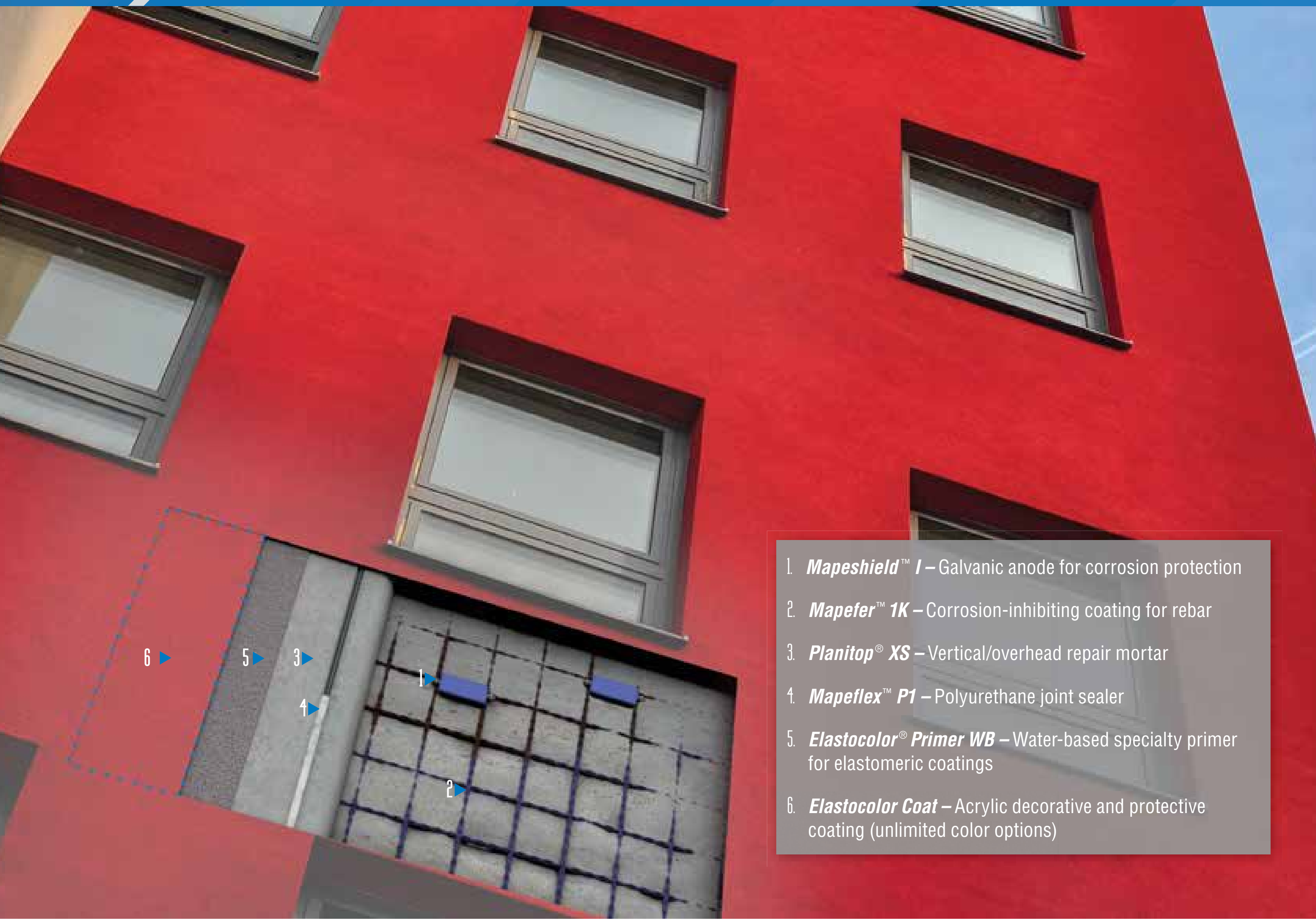

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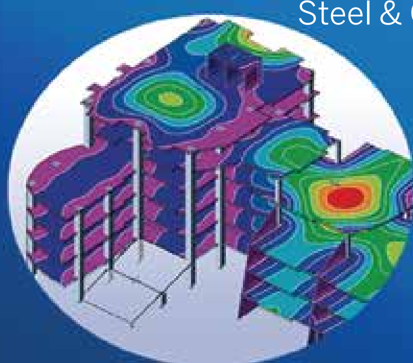

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STRUCTURE®

October 2016

On the cover An aerial view of the new Evergreen Point Floating Bridge across Lake Washington in Seattle. Alongside is the old crossing, currently decommissioned and being removed from the lake. See page 30 for details.

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NEW! What do you think of the current issue?

The STRUCTURE magazine homepage has a new link for a four question survey about the current month's issue. Please visit STRUCTURE's website and answer the questions. It should not take any more than 4 or 5 minutes. This information will assist us in providing the content you want. Thank you for your feedback!

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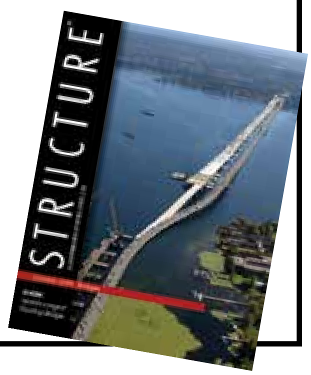

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Editorial | *United We (and our structures) Stand*

By David W. Mykins, P.E., Chair CASE Executive Committee

In early August of this year, hot on the heels of both major political parties' national conventions, a meeting took place in a secluded hotel conference room in downtown Chicago that, despite including many high-ranking officials from national organizations, has received distressingly little coverage in the mass media. I'm referring of course to the summer meeting of the leaders of CASE (Council of American Structural Engineers), NCSEA (National Council of Structural Engineers Associations) and SEI (Structural Engineering Institute).

Unlike our major political parties, these three organizations share a single platform: supporting and elevating the profession of structural engineering, in practice, research and education. Each of these groups provides services and resources that are unique and complementary in support of that goal. As a reader of STRUCTURE magazine, you are probably aware that while NCSEA takes the lead, it is published through the combined efforts of all three groups. In fact, we take turns giving you our two cents on the latest topics in structural engineering in this very column every month.

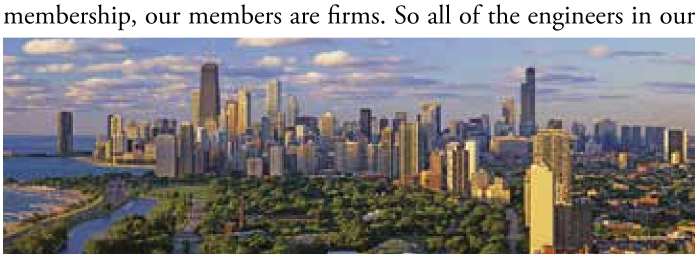
This cooperative effort is in large part what makes STRUCTURE magazine one of the most informative, educational and relevant publications for structural engineers. At a recent meeting at the Structures Congress, we began to wonder whether there were other ways in which we could collaborate and combine our respective strengths to enhance the profession. As a first step to explore this, we decided to get together during the CASE Summer Risk Management Seminar.

Like most meetings, we started with introductions and a statement of purpose. We then talked about the accomplishments and goals of our groups. Now, I consider myself pretty involved but, as I sat and listened to the leaders of SEI and NCSEA, I had to ask myself "How do I not know this?" For example, how did I not know about SEI's initiative to become a more global organization, and NCSEA's plans to build a database of engineers who would be available to help in disasters?

Even though the three organizations have existed and worked together for more than two decades, we seemed to have existed in silos. We are very good at what we do, but sometimes you have to look around at what others are doing to help guide your direction. Or at least to make sure you aren't duplicating efforts.

Since we were not fully aware of some of the details of the other groups, it follows that others might not be. In that spirit, I'd like to share some basic information about CASE with you.

CASE is one of six practice-based coalitions within the American Council of Engineering Companies (ACEC). Rather than individual membership, our members are firms. So all of the engineers in our member firms are members too. We represent over 160 firms employing more than ten thousand structural engineers nationwide. Our member firms range from sole proprietors to some of the largest firms in the nation.



We are organized into five committees who are always working on new products to help our profession. They are:

- Contracts
- Guidelines
- Programs and Communications
- Toolkit
- Membership

So what does CASE do? Our mission is to improve the practice of structural engineering. We give you practical tools to help you reduce risk and make you more efficient and more profitable. These include national practice guidelines that outline best practices, contracts designed specifically for structural engineers, and an extensive list of risk management tools and educational sessions designed to keep liability in check. All of our products and publications are designed to be "plug and play". We have, for example, a project kick-off meeting agenda and site inspection checklists that you could start using today. To learn more about CASE, visit our website at **www.acec.org/CASE**.

Our meeting in Chicago was eye-opening for all of us and, in the end, we resolved to have more frequent discussions and explore our respective strengths to develop ways we can support each other and avoid duplication of effort. We discovered that we have a unique opportunity to reach the structural engineering community worldwide and influence the practice in a big way. The first step toward meeting this goal is the refreshing of our 20-year-old cooperative agreements. What are now three separate agreements will be combined into one joint agreement.

Maybe structural engineers won't ever get the mass media coverage we deserve for the work we do. But in the spirit of this election season, let's take inspiration from one of our great historic political activists, Thomas Paine, who said, "It's not in numbers, but in unity that our great strength lies."▪

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Aesthetics in Bridge Structures

The Role of Engineers, Architects, and Builders

By Roumen V. Mladjov, S.E., P.E.

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The online version of this article contains detailed references. Please visit **www.STRUCTUREmag.org**.

Figure 1.

Bridges are among the oldest structures used by mankind. From meeting purely utilitarian necessities, bridges have evolved with time to become symbols of human progress, of cities and entire countries. Among the thousands of bridges around us are the bridges that we all admire, the bridges that are the symbols of the eternal human aspiration for building longer and taller, stronger and faster.

When discussing bridges, important issues to consider are aesthetics and the respective roles of engineers, architects, and builders in designing a bridge. What makes a bridge structure elegant and appealing? Do we need to involve architects in the bridge design?

Engineers, Architects, and Builders

The Caravan Bridge, a single arch stone structure in Turkey built around 850 BC, is considered the oldest still functioning bridge. In his *Histories*, Herodotus reports a bridge on stone piers built in Babylon over a channel of the Euphrates River around 550 BC. He also describes temporary military pontoon bridges constructed by the Persian armies: one on the Ister River (Danube) and two long ones used for crossing the Straits of Hellespont (Dardanelles) during their invasion of ancient Greece around 500 BC and 480 BC, respectively. This is about 2,500 years before *Bosphorus Bridge I*, the first permanent bridge between two continents, was completed in 1973.

Until the early 19th Century there were no structural bridge engineers and architects. These professions and "titles" simply did not exist at that time. Writers and scholars often refer to architects when describing ancient constructions; however, these "*chief builders*" practiced the combined tasks of present-day engineers, architects, artists and craftsmen. The tasks were performed by single practitioners who were learning their skills by apprenticeship, following the experience of their predecessors and the "trial-and-error" method. Later, during the Middle Ages, with the building of Gothic cathedrals, the leaders of larger projects were called "master builders."

Master builders produced remarkable structures over the centuries before the first engineers and architects started receiving a formal education. Later, during the Industrial Revolution, with a need for more construction (bridges and buildings) and the development of engineering knowledge, the functions and duties of a single master builder were separated among architects, engineers, and builders. It was only during the 19th Century that civil engineering and architecture took off as technical professions.

During the "Heroic Age" of bridge engineering, Thomas Telford, Isambard K. Brunel, Robert Stevenson, John, Washington and Emily Roebling, and Alexander Gustave Eiffel built astonishing bridge structures. To this day, we still admire and consider these bridges as part of the highest achievements in engineering. Among these bridge builders, only Washington Roebling and Eiffel had formal engineering education (Eiffel being a chemical engineer). The others were self-taught without a formal education in engineering; the bridge-engineering genius John Roebling had completed his academic training but did not take the final exam. At the time, being able to provide a safe and efficient bridge was valued more than having formal degrees.

The general term "engineer" is used in this article for all structural, bridge, civil or self-educated engineers and builders. All remarkable bridges of the past were designed and built by engineers. There are no reports of architectural involvement (in the current meaning of the term) in their design. Only a few bridges in the past required the skills and knowledge of present day architects: for example, the Venetian Rialto Bridge, the Ponte Vecchio in Florence, and the London

Tower Bridge, for the buildings integral to these bridges, or others like Pont Alexandre III in Paris due to their rich ornamentation.

Today many engineers working on bridges believe that, due to their education, experience, and skills, they are able to work alone and do not need architectural involvement in bridge design except for secondary elements like vehicle/pedestrian barriers and light poles. Other engineers do work with architects, or at least consult an architect for their bridge design. Since their structures are mostly exposed, bridges are the most "sincere" constructions. Therefore, the bridge designer must consider aesthetics; he is the one that best knows how to resolve the challenge of balancing the contradicting requirements for robustness and slenderness to obtain security and elegance at the same time. So what is the answer to the initial question "Do we need an architect to design a bridge?"

To respond to this question, it is necessary to look at the specifics of bridge aesthetics. Most bridge professionals agree on the fundamental principle that a bridge has to be robust (strong, stiff, and resilient), functional, efficient and economical, but also that it should be elegant – slender with simple forms and well proportioned. It has to be in harmony with its surrounding environment, and if possible, to embellish its natural site (*Figure 1* and *2*). Even as early as the Roman period, Vitruvius had formulated three important structural qualities – *firmitas, utilitas, venustas* – meaning that a structure should be solid/robust, useful and beautiful. In modern times, David Billington has set the core principles of good structural design as *efficiency* (of materials), *economy* (of cost and time) and *elegance* (slenderness, elegance, and good proportions). Since then, some of the most prominent bridge engineers, Fritz Leonhardt, Michel Virlogeux, and Christian Menn, have expressed the same understanding of the essential qualities of bridge design.

Regarding the robustness and functionality of bridges, all bridges shall be designed and built to satisfy their primary function – to transfer pedestrians, vehicles and/or trains from one side of an obstacle to the other side. This means that all bridges must have the necessary strength and stiffness to safely carry the prescribed loads per bridge codes. For this reason, robustness is a must for all bridges and need not be discussed further in this article.

The engineer's role is to provide an efficient and economic structure, while also trying to make it elegant. The engineer's task is to select the most appropriate bridge type and the correct parameters for a particular project; the builder can provide significant assistance in this process if involved on time. The architect, while advising the engineer on aesthetics, should avoid recommendations that may significantly increase the cost. While aesthetics is more or less subjective, efficiency and economy can be measured objectively by the cost, main structural materials and construction time with respect to bridge span lengths.

Figure 2.

Designing Strong and Elegant Bridges

How can an engineer, with or without help from an architect, deliver an elegant and appealing bridge structure?

There is a consensus among professionals that a well-designed bridge in conformance with the structural "basic principles" usually results in an elegant, well-proportioned and appealing structure without the need for additional ornamentation. A bridge design should also take into consideration the visual exposure and appearance of the structure in relation to its site environment. Based on their exposure and location, bridges are:

- Non-visible structures, usually short span bridges on roads without underpasses, *not requiring specific* attention to aesthetics;
- Typical short or medium span bridges, *requiring regular* attention to aesthetics;
- Long-span bridges or bridges with significant exposure, *requiring special* attention to aesthetics;
- Complex bridges with exposure and long approaches, *requiring special* attention to aesthetics;
- Some bridges span a river, strait, gorge or ravine directly, often without any approaches. Such locations increase the bridge visibility exposure and are more aesthetically demanding.

The most important part of bridge design is the overall concept for the structure and its elements – the selection of the appropriate structural system for the bridge considering its specific function, site location, and required spans. This concept is always the most important, challenging and creative part of engineering. Economy depends mainly on an efficient design concept. Good design concepts minimize future difficulties both in the design office and on the construction site. While experienced engineers can deliver excellent projects even without an architect, it would be preferable for engineers to work in collaboration with an architect with knowledge and understanding of bridge design and aesthetics. Few engineers have the advantage of both engineering and architectural training, with skills like Santiago Calatrava. Perhaps it is a good idea for engineers to work with an architect on their next bridge project. It is also important for engineers and architects to work closely with the builders, starting in the early phases of design.

An ideal solution for most bridge professionals would be a simple straight-line structure with constant depth and without any supports within the span. However, such a solution is possible only for relatively short spans, as it is limited by the strength of the structural material. The required depth for simple or continuous supported girders, even at medium spans, will make the depth-to-span proportion either inefficient or unattractive, therefore, requiring the use of other static systems with slender superstructures.

Renowned bridge engineers have provided the following recommendations for designing more attractive bridges:

- The bridge should be appealing by itself, in harmony with its environment, scale, and character of the site. When complementing its site environment, a bridge is regarded as a high achievement (*Figure 1* and *2*);
- A bridge system should be selected considering its surroundings – over a

Figure 3.

river, over a sea, over a deep canyon, within urban areas, etc.;
- Transition from approaches to main spans should be smooth with appropriate relation between neighboring spans;
- Simple forms expressing the flow of forces should be used, maintaining precise order and unity for the entire structure;
- Slender bridge elements and minimum types of elements should be utilized;
- Fewer and lighter pier-supports should be used for more transparency of the substructure;
- Light and shadow effects may be used to visually enforce the slenderness of the bridge components;
- The design should permit appropriate maintenance during the lifetime of the bridge.

Useful recommendations for bridge design are published in *Bridge Aesthetics Sourcebook. Practical Ideas for Short and Medium Span Bridges* (TRB, 2009).

The Role of Architects in Bridge Design

An architect can provide essential assistance to engineers on the following parts of a bridge project:

- Selection of the most appropriate structural system and the overall concept for the project;
- Improvement of the proportions of the main bridge components – ratio of depth to span lengths, ratio of central to side spans, providing good order and proportions;
- Form shaping of the main components – piers and superstructure elements; design of the towers and pylons for suspension and cable-stayed bridges – their composition (single, two, three or more columns at one support), shape of the frames, configuration of cable-stays (fan, harp, etc.), general aesthetic advice for the best visual design in cable-stayed bridges;
- Pedestrian bridges, where the relatively shorter spans, combined with higher visibility and lighter loads, provide more opportunities for design creativity;
- Bridges with above-deck structures, regardless of whether they are arch, suspension, or cable-stayed structures, have a significant visual effect on everyone on the bridge deck.
- Secondary elements like vehicle barriers, pedestrian railings, and light poles. These are usually considered the architect's domain; however, even the best-designed secondary elements cannot save a mediocre bridge design.

Long-Span Suspension and Cable-Stayed Bridges

The long-span bridge design is mainly governed by structural efficiency. These bridges benefit from the natural elegance of their structural systems; as stated by Michel Virlogeux, "The scale of long-span bridges alone gives them an inherent majesty" (*Figure 3*).

In suspension bridges, the towers with their imposing size and shape, combined with the natural elegance of the catenary main cables, have predominance on the projected image. This "natural" inherited quality is usually enhanced with the appropriate articulation of the tower legs and cross-girder ties. Here again, the role of an architect is helpful for achieving the maximum aesthetic effect. Well-designed towers provide a feeling of elegance and strength at the same time.

Architect Irving Morrow provided one of the best known aesthetic improvements to the Golden Gate Bridge in San Francisco. The Art Deco style of the bridge towers and the selected "International Orange" color significantly contributed to the fame of this amazing 1937 structure, considered even today as one of the greatest bridges ever built (*Figure 1*). The Golden Gate Bridge is always on the list of the "Greatest" and "Most Famous" bridges in the world. It is one of the few structures that have enhanced their site environment.

Figure 4 and *5* show the towers of several bridges considered as some of the best; while the architects involved in their design have not received similar credit as Irving Morrow, it is evident that the shape of these

Figure 4.

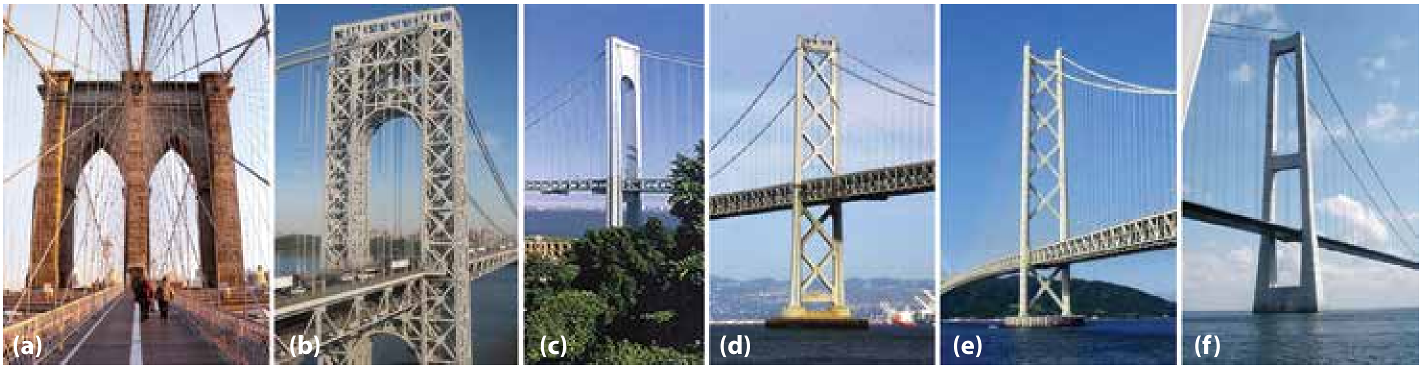

Figure 5.

majestic and elegant towers have benefitted from the contribution of architectural expertise. For example, the suspension towers of the San Francisco – Oakland Bay Bridge, 1936 (*Figure 5d*), with stylization by architect Timothy Pflueger, have inspired the designers of the towers of Akashi-Kaikyo Bridge (*Figure 5e*), the longest span in the world.

For cable-stayed bridges, the towers also have a major significance in creating an attractive overall image. The relatively shorter spans for "middle long-range" structures allow more "free" treatment of the tower's design; therefore, more creativity and originality – more possibilities for innovative work between engineers and architects. With the multitude of options in suspension and cable-stayed bridges, a bridge does not need to be extravagant. Indeed, as mentioned above, bridges are the most "honest" constructions as their structure is exposed. Ludwig Mies van der Rohe's principle "Less is More" is valid as much for bridges as for buildings.

The Role of Builders in Bridges

In creating a successful bridge project, the role of the builder is much larger than just providing the construction. It is important to involve an experienced builder in the team as early as possible in the selection of the bridge system. The builder can supplement the effort with his knowledge of efficient construction methods, and can prevent the designers from selecting a system that may lead to significant problems during design and construction. In addition to organizing and managing the construction, the builder often has to design detailed phases of the structural assembly with all necessary temporary structures. This process requires close collaboration with the (design) engineer. Once the project construction starts, the builder's primary task is to provide high quality and to deliver the project on time and within budget.

Aesthetics, Efficiency, and Economy

It is important to keep a good balance between the aesthetics and the efficiency and economy in bridge design. Any deviation to extremes in either direction has adverse effects. For example, one of the most talented bridge designers, Santiago Calatrava, created Alamillo Bridge at Seville, Spain for Expo'1992 (*Figure 6a*). It is a beautiful, while controversial, bridge design due to its deviation from basic cable-stayed systems, resulting in high inefficiency. The unusual omission of back span cable-stays creates a dramatic view and contributes to the attractiveness of the bridge, but such a concept should be discouraged for any bridge that is not built as a monument. Similar comments are valid for the Erasmus Bridge in Rotterdam, Netherlands (*Figure 6b*), credited to Ben van Berkel, the architect of the bridge. While the Erasmus Bridge became the symbol of Rotterdam, no one can pretend that its structure complies with the basic rules of statics, another deviation resulting in a much higher cost.

Figure 6.

Some pedestrian bridges with spans 100 – 150 meters (330 – 490 feet), designed with an eye to the extraordinary, have significantly higher costs per unit area than most suspension and cable-stayed bridges with much longer spans of 500 – 1000 meters (1650 – 3300 feet).

According to Christian Menn and Michel Virlogeux, the art of engineering, with its optimization of elegance, requires creativity and fantasy; and engineers should avoid multiplied, repetitive structures and illogical shapes. Creativity in good design is essential, but "excessive originality" should only be found in justified exceptions.

A bridge does not need to be expensive or extravagant – the simplest bridge with sincere structure is often the best. The elegance of bridge structures as discussed in this article can be obtained following a few basic rules, using simple forms and proportions in compliance with the statics of structures. The right balance of the leading role of engineers, combined with the important contribution of architects and builders, is essential for creating a successful bridge project.▪

Structural Forensics

investigating structures and their components

Fracture Critical Bridge Inspection

By Peyman Kaviani, Ph.D., P.E., Anousheh Rouzbehani, C.Eng and Carlos Villalobos, P.E.

In 1967, there was a sudden collapse of the Silver Bridge, a pin-connected link suspension bridge over the Ohio River at Point Pleasant, West Virginia, that resulted in a loss of 46 lives. As a result, a 1968 federal act initiated a national bridge inspection program that recognized the need for periodic and consistent bridge inspections. The first National Bridge Inspection Standards (NBIS) were developed in 1971.

The 1983 failure of the Mianus River Bridge in Connecticut caused more concern related to fatigue and fracture critical bridges. This failure and further research resulted in mandated fracture critical bridge inspections.

Much has been learned in the field of bridge inspection, and a national Bridge Inspection Training program is now fully implemented. State and federal inspection efforts are more organized, better managed and much broader in scopes of work. The technology used to inspect and evaluate bridge members has significantly improved.

Fracture Critical Bridges

As specified in the *National Bridge Inspection Standards* (NBIS), Code of Federal Regulations (CFR) Title 23, PART 650, a fracture critical member (FCM) is "a steel member in tension, or with a tension element, whose failure would probably cause a portion of or the entire bridge to collapse." Bridges that contain FCMs are defined as fracture critical (FC) bridges, and the Federal Highway Administration (FHWA) Bridge Inspector's Reference Manual classifies FCMs as:

1) Steel girders in structural systems with up to two-girder (in California, three-girder) configuration
2) Tension members of steel trusses in structural systems with up to two-truss line (in California, three-truss line) configuration
3) Steel box girders with up to two-cell (in California, three-cell) configuration
4) Main suspension cables of suspension bridges
5) Steel hangers of suspension or arch bridges
6) Steel ties of tied arches or trusses
7) Pin-and-hanger assemblies (*Figure 1*) in structural systems with up to two girders or truss lines (in California, three girders/lines and pins shall also be tested ultrasonically)
8) Steel floor beams or cross girders
9) Steel bent subjected to tensile stress due to flexure

Also, moveable bridges and floating bridges are classified as FC bridges. California Department of Transportation (Caltrans) defines bridges with special fatigue prone details as special feature (SF) bridges.

Figure 1. Pin-and-hanger assembly.

Inspection Procedures

A fracture critical bridge inspection is defined by the NBIS as a "hands-on" (i.e. within arm's length of the component) inspection of fracture critical members. This type of inspection uses visual methods that may be supplemented by non-destructive testing (NDT). A detailed, visual, hands-on inspection is the primary technique of detecting cracks on steel tension members. Therefore, the inspection may require the bridge inspector to thoroughly clean critical areas before the inspection and use additional lighting and magnification. Other non-destructive testing methods (e.g. ultrasonic test, liquid dye penetrant test) may also be used to inspect the areas if the hands-on visual testing method is not sufficient to detect defects.

Figure 2. Under bridge inspection truck (UBIT).

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According to the NBIS, fracture critical bridges are required to be inspected at regular intervals not to exceed 24 months. Also, Caltrans mandates that special feature bridges be inspected at intervals not to exceed 48 months.

Because of the hands-on access requirement and the focus on specific bridge components, fracture critical bridge inspections are highly detailed and complex to plan and execute. The procedure can be described in three main steps: 1) preparing for the FC bridge inspection; 2) performing the FC bridge inspection; and 3) writing the FC bridge inspection report.

Preparing for the FC Bridge Inspection

The first step of FC bridge inspection preparation is to either develop the inspection plan (for an initial inspection) or to review the existing inspection plan (for subsequent inspections). The inspection plan should include:

- The bridge description
- FCMs and details that require inspection
- Location of the FCM's on the bridge structure
- Inspection frequency
- Description of each inspection method (e.g. visual inspection, ultrasonic testing) applicable to the inspection
- Traffic management plan (e.g. lane closures, railroad flagging)

- Personal inspection equipment needed (e.g. flashlight, wire brush, nondestructive testing equipment)
- Access vehicles required for the inspection (e.g. aerial lift, under bridge inspection truck [UBIT], see *Figure 2*)

Before the inspection, the inspector should review previous FC bridge inspection reports to familiarize himself with the structural element types, condition states, and recommendations related to FC steel members. Per inspection plan, the inspector submits a written request for flagging to the railroad company, and for traffic control and lane closures to the bridge maintenance crew. The final step of the FC bridge inspection preparation is the testing of all inspection equipment the day before the inspection.

Performing the FC Bridge Inspection

First, all participants of the bridge inspection meet at a time and location assigned by the FC bridge inspection team leader. Safety and the responsibilities of each member should be discussed during the meeting. In addition to the FC inspector, participants in an FC bridge inspection may include: 1) the UBIT operator or any other FC bridge inspector who drives the aerial truck during the inspection, 2) the traffic control team, and 3) the railroad flaggers if the bridge crosses railroad tracks.

Only after the traffic control team provides the required lane closures and the railroad flaggers inform the FC bridge inspector that entry into the railroad right of way is safe, the FC inspector can begin the inspection. Keep in mind that the FC bridge inspector should maintain continuous communication with the traffic control manager and the railroad flaggers to maintain a safe workplace throughout the duration of the bridge inspection.

The FC bridge inspector meticulously implements the inspection methods defined in the inspection plan. It is required that all existing cracks and deficiencies, which had been recorded in previous FC inspection reports, be carefully monitored. If an existing crack has propagated since the last inspection, then the new tip of the crack is punched to monitor any new growth during the next scheduled FC bridge inspection. If a new crack is found, the FC bridge inspector documents the crack details (i.e. length, location, and type of crack) on the bridge component where the crack is located, as well as in the FC bridge inspection report, supplemented by pictures of the crack.

continued on page 14

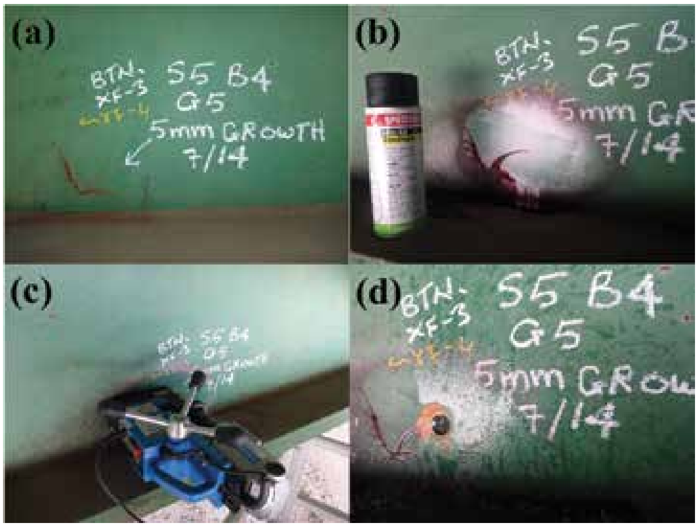

Figure 3. Fatigue crack arrest.

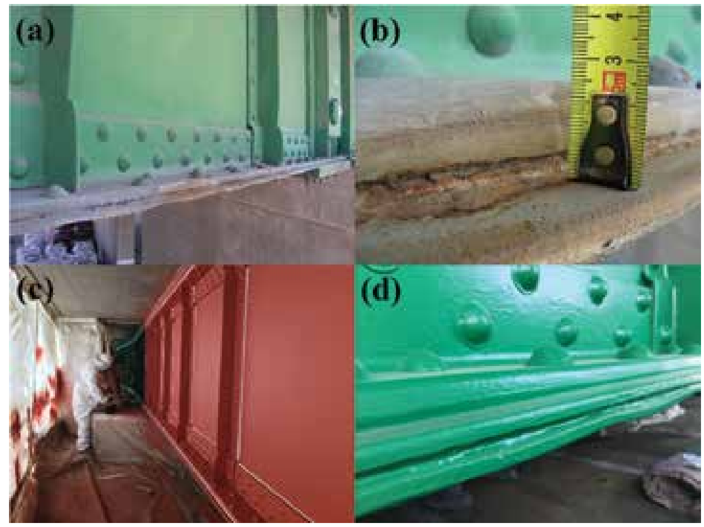

Figure 4. Pack rust corrosion.

Writing the FC Bridge Inspection Report

All FC bridge inspection findings are documented within the FC bridge inspection report. The report follows the standard format specified by the inspection agency. For example, based on the Caltrans standard, the main items included in an FC bridge inspection report are:

- Bridge identification information
- Name of FC bridge inspectors
- Date of inspection
- Access equipment used
- Traffic control team information
- Condition of existing cracks
- Description of new cracks
- Non-destructive testing (e.g. ultrasonic testing) results
- Supplementary pictures of the FC bridge inspection

Also, the FC bridge inspector may provide recommendations regarding critical findings (i.e. a structural or safety related deficiencies that require immediate follow-up inspection or action). If the critical findings need immediate attention, the FC bridge inspector should inform upper managers without delay.

Deficiencies

The most common deficiencies are:

- Fatigue cracks
- Fractures/dents due to impact loading
- Loss of cross section due to corrosion (e.g. pack rust)
- Misalignment of tension members
- Flaws in pin-and-hanger assemblies

Fatigue Crack Propagation

A fatigue crack occurs at a stress level below the yield stress and is due to repeated loading. This type of cracking can cause sudden and catastrophic failure of FC bridges. Fatigue cracks should be monitored during each FC bridge inspection cycle and, if a crack is showing continuous growth, the propagation should be stopped by drilling a hole at the crack tip. As shown in *Figure 3a*, a 5 mm fatigue crack growth on Girder 5 in Span 5 was found during an FC bridge inspection. To prevent further propagation of the crack, an arrest hole was drilled at the crack end. Before drilling, liquid dye penetrant testing was performed to locate the crack tip (*Figure 3b*). As shown in *Figure 3c* and *3d*, a drilling machine was used to drill an arrest hole at the crack tip. The arrest hole stops the crack growth by releasing stresses at the crack tip. The bridge inspector will monitor the situation to see if the crack growth continues beyond the arrest hole during the next scheduled FC bridge inspections.

Pack Rust

Cross section loss of a steel girder due to pack rust corrosion is another defect that can be found in an FC bridge inspection. The pack rust occurs between two mating surfaces and is a volume of rust formed over the original steel. The pack rust may create localized distortion, and possibly cracking and loss of cross section.

Figure 4a and *4b* shows 10 mm of pack rust that occurred between the bottom flange of the exterior steel girder and the bottom steel cover plate. As shown in *Figure 4c*, the pack rust between the bottom flange and the cover plate was removed during a painting project. Typically, caulking material is inserted into the cleaned areas to avoid further corrosion. *Figure 4d* shows the completed repair. These areas will be monitored for corrosion during the next FC bridge inspection cycles.

Future Bridge Inspection Trends

In the future, bridge inspection may focus on the quantitative assessments of bridge performance and conditions. Certainly, bridge engineers will use an array of increasingly more sophisticated instruments, procedures, and systems to inspect the structures.

Using present technology, a variety of permanent sensors on bridges may collect critical performance data. These sensors will likely be powered by, and will report to, wireless networks. Data may be analyzed, and any flaw/deterioration will be detected automatically. Extensive use of sensors will also become possible as advances in the miniaturization of electronic devices, increased availability of wireless communications, and lower costs for devices and communication combine to provide an array of compact, permanent, inexpensive data acquisition systems.

Another advanced system that may assist bridge inspectors when performing an inspection of hard to reach locations and parts of any complex bridge will be the use of Unmanned Aerial Vehicles (UAVs). Using UAVs could help minimize risks associated with current bridge inspection methods, which include – but are not limited to – rope systems and special inspection vehicles. Extensive projects are underway to study the effectiveness of using UAVs to aid in bridge inspection work, typically in gathering images without the use of a UBIT and in areas where access is difficult or not safe for bridge inspectors. The increasing costs of bridge inspections are a concern for the Departments of Transportation in all states. The use of UAVs may help alleviate these costs and improve the quality of bridge inspections.▪

BUILDING BLOCKS

updates and information on structural materials

Composite Deck System

A Novel Lightweight Solution for Long-Span Bridges

By Xudong Shao, Ph.D., Lu Deng, Ph.D. and Anil Agrawal, P.E., Ph.D.

Orthotropic steel decks (OSDs) have been used commonly in long-span bridges to reduce self-weight, and therefore improve the spanning ability of these bridges. The OSDs are usually covered with a 2- to 3-inch-thick asphalt wearing course. Under cyclic heavy traffic loads, these steel decks are susceptible to fatigue cracks, while asphalt overlays can suffer from cracking and shoving problems. Both issues compromise the serviceability and durability of the bridge deck.

Over time, some countermeasures have been proposed to address these problems, including increasing the thickness of deck plates, refining the configuration of fatigue-prone details, and enhancing the welding quality. However, none of these approaches have proved to be very effective since none of them provide many benefits for increasing the stiffness of the deck plate. Recently, Buitelaar et al. (2004), Murakoshi et al. (2007), and Dieng et al. (2013) have proposed to use a reinforced high performance concrete (RHPC), a steel-fiber-reinforced concrete (SFRC) overlay and a fiber-reinforced UHPC (UHPFRC) layer, respectively, to strengthen the stiffness of the steel deck. However, these attempts did not achieve satisfactory results. Cracks developed in the RHPC and SFRC while sliding occurred between the steel deck and UHPFRC layer. The reason was either that the concrete did not have sufficient cracking strength, or the concrete layers did not develop sufficient composite action with the steel deck.

Proposed Steel-UHPC Lightweight Composite Deck (LWCD) System

To systematically address the issues above, Prof. Xudong Shao's research group at the Hunan University introduced a novel lightweight composite deck (LWCD) system. The LWCD is composed of a conventional OSD covered by a 1.38- to 2.36-inch-thick (35-60 mm) UHPC layer (*Figure 1*). The OSD and UHPC are connected through headed studs to ensure that the desired bonding performance of full composite action could be achieved between the two structural components. In the LWCD, the UHPC layer functions as a structural component and is designed to have the same service life as that of the OSD. To ensure the desired cracking strength and fatigue performance, the UHPC is compactly reinforced with a steel mesh, as shown in *Figure 1*.

A substantial amount of research in the past six years has explored the fundamental behaviors of the LWCD, including the material property of the UHPC, shear performance of the shear studs, static and fatigue performance of the LWCD. Some of these studies are briefly introduced in the following sections.

Behaviors of LWCD

The Humen Bridge, a suspension bridge that has a main span of 2,913 feet (888 m) and was opened to traffic in 1997 in Guangdong, China, was selected as the test bed for evaluating the performance of the proposed LWCD. Bridge deck segments and longitudinal deck strips were fabricated and tested in the laboratory. Finite element (FE) analysis was also performed to develop the field testing plan.

Static Performance

The performance of the LWCD under the design vehicle loads specified in the General Code for Design of Highway Bridges and Culverts in China (MTC 2004) was investigated based on the FE analysis using the ANSYS program. The performance of a standard OSD without UPHC layer was also studied for the purpose of comparison. The main dimensions of the cross section are

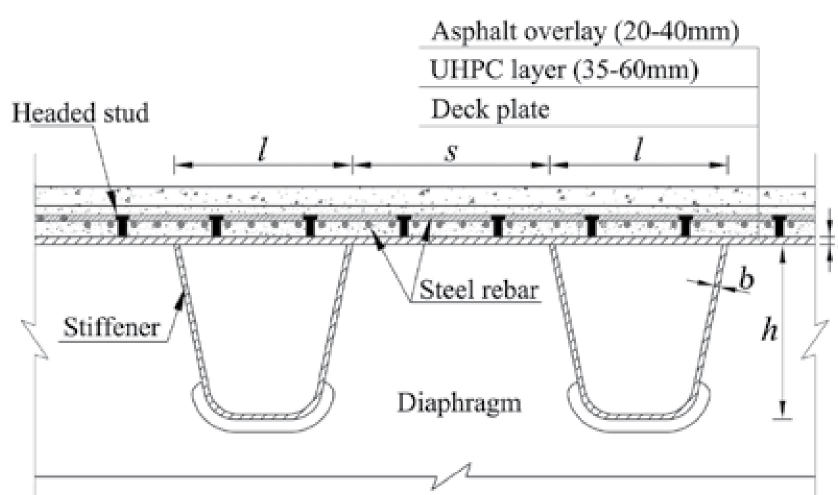

Figure 1. Schematic of the LWCD.

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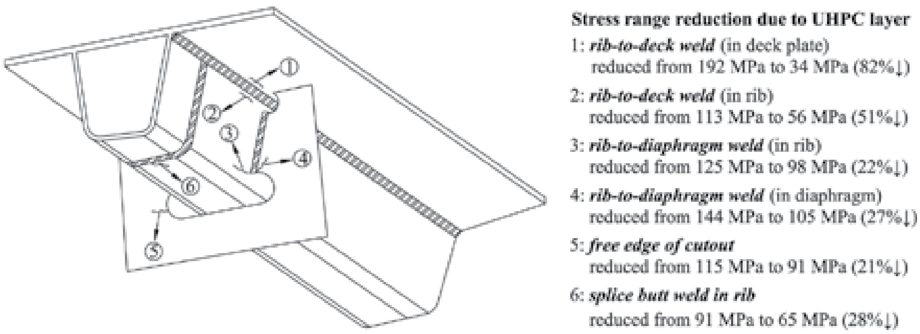

Figure 2. Comparison of stress ranges in fatigue-prone details.

as follows: t = 0.47 inch, b = 0.31 inch, h = 10.31 inches, s = 12.05 inches, l = 12.36 inches (t = 12 mm, b = 8 mm, h=262 mm, s = 306 mm, l = 314 mm) (refer to *Figure 1*). The UHPC layer was 1.77-inch (45 mm) thick. Steel rebars with 0.39-inch (10 mm) diameter were arranged in both directions with a center-to-center spacing of 1.48 inches (37.5 mm). The stress levels at the six typical fatigue-prone details in the steel deck were examined and compared. The analysis results are shown in *Figure 2*.

With the addition of UHPC on the steel deck, the stress ranges in all six details of the OSD have been reduced significantly, especially in the rib-to-deck welds where the stresses are reduced by 82% and 51% in the deck plate and rib, respectively. The stress ranges are below the corresponding constant-amplitude fatigue limits (CAFLs) specified in the bridge design codes (European Committee for Standardization 2005), indicating that these details would theoretically not have fatigue problems during their service life.

Performance of Headed Studs

The headed studs used in the LWCD have a height of 1.38 inches (35 mm) and diameter of 0.51 inch (13 mm), resulting in a height-to-diameter ratio of 2.7. Push-out tests were performed to study the behavior of the short-headed studs embedded in the UHPC. The test results show that when the load was increased to a certain value, the headed studs were sheared off from the steel plates while the UHPC layer was intact with no observable cracks developed, indicating that even with a low height-to-diameter ratio of 2.7, the studs could still develop full shear strength in the LWCD.

Performance at Negative Bending Moment Zone

When exposed to traffic loads, tensile stresses develop at the negative bending moment zones on the UHPC layer, e.g. at the diaphragm sections. To reveal the behavior of the UHPC layer under such negative bending moments, a static load test was performed on a steel-UHPC composite beam specimen (*Figure 3, page 18*), which consisted of an OSD strip and a 1.77-inch-thick UHPC layer. In the test, the load was incrementally increased until the sample failed.

The test results show that when the bottom flange of the OSD began to yield due to excessive compression, no visible cracks were observed on the UHPC surface. When local buckling developed at the bottom flange of the OSD at the peak load, cracks with a maximum width of 0.01 inch (0.3 mm) were observed. These observations clearly indicate that the OSD failed before the UHPC layer.

Fatigue Performance

Fatigue tests were also performed on the LWCD specimen. With the compact reinforcement inside, the cracking strength of the UHPC used in this study can reach 6.19 ksi (42.7 MPa) (Shao et al., 2013), as compared to 1.16-1.45 ksi (8-10 MPa) without reinforcement. The fatigue load was set to produce a stress range of 3.09 ksi (21.3 MPa), which is half of the cracking strength at the most critical location of the UHPC layer. The test results showed that the UHPC layer developed no fatigue cracks after 3.1 million cycles at this stress level. Based on FE analysis, the design load only causes a maximum stress range of about 1.45 ksi (10 MPa) in the UHPC layer, indicating that the UHPC layer can meet the design requirements regarding fatigue safety.

Application to Field Bridges

To date, the LWCD has been applied to four bridges in China (Shao et al., 2015), among which the first pilot project was the Mafang Bridge constructed in 1984. This bridge consists of fourteen 210-foot-long (64 m) simply supported spans. Due to the heavy traffic, the pavement suffered from severe deterioration, and cracks were also observed in the OSD. In 2011, a major retrofit was undertaken for the asphalt overlay

Figure 3. Set-up of the static load test.

(Cao et al., 2016). Five different retrofitting schemes utilizing different wearing courses were adopted for the various spans, including a 3.15-inch-thick (8 cm) stone asphalt concrete layer, 3.15-inch-thick (8 cm) epoxy asphalt layer, 3.15-inch-thick (8 cm) sandwich plate, 2.76-inch-thick (7 cm) polymer asphalt concrete layer, and the proposed compactly reinforced UHPC layer (on the 11th span). To examine crack development in the UHPC layer, the first 177 feet (54 m) of the 11th span was covered by a 1.97-inch-thick (5 cm) UHPC layer with a 1.18-inch-thick (30 mm) asphalt overlay on top. The remaining 33 feet (10 m) was covered by a 3.15-inch-thick (8 cm) UHPC layer without an asphalt overlay.

Three routine checks have been performed during the past four years, and no fatigue degradation has been observed in the LWCD. No further crack propagation on the OSD and noticeable deterioration in the asphalt pavement were observed. No cracks were found on the top surface of the 3.15-inch-thick (8 cm) UHPC layer. On the other hand, crack propagations have been observed on the steel decks and severe degradation of the pavement has been seen in decks retrofitted using four other retrofitting schemes approximately 4 years after the installation of decks (*Figure 4*). *Figure 4e* is the deck using LWCD, which is entirely damage-free after 4 years. It should be noted that all 5 decks shown in *Figure 4* have been subjected to the same traffic loading during the last 4 years.

Advantages and Potential Use of the LWCD

Field verification of LWCD, compared to other retrofit schemes of the deck shown in *Figure 4*, indicates excellent potential for the utilization of the LWCD. In summary, the LWCD has the following advantages over the conventional "OSD + asphalt overlay" system:

1) The UHPC layer improves the stiffness of the bridge deck significantly, leading to a considerable reduction in vehicle-induced stresses in the steel deck and therefore a pronounced extension of the fatigue life of the steel deck;
2) The UHPC layer needs no major retrofits or replacement during the service life of the bridge. Therefore, although the LWCD scheme has a slightly higher initial cost compared to conventional schemes that adopt an epoxy asphalt overlay on top of the OSD, its life-long total cost, including costs related to the maintenance and retrofitting of the asphalt overlay, is much (estimated at 85%) lower since the cost of the asphalt overlay in the LWCD scheme is much lower;
3) The weight of the LWCD is comparable to that of the conventional "OSD + asphalt overlay" system. Also, field applications have demonstrated that it is convenient and feasible to construct the LWCD on either a newly-built bridge or an older bridge, making it a very promising deck system for long-span bridges.

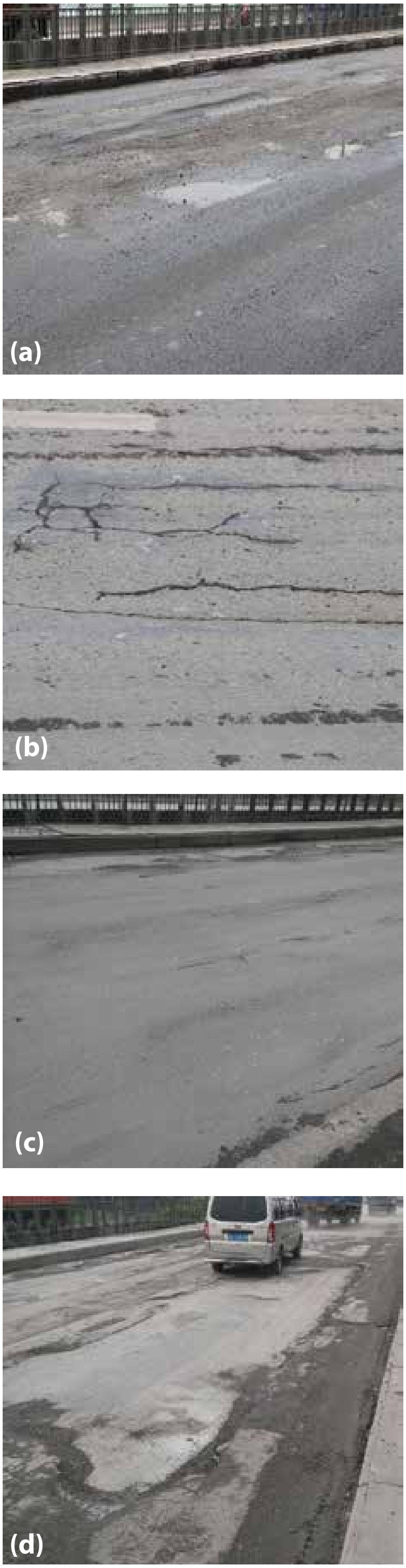

Figure 4. Service state of five retrofitting schemes on the Mafang Bridge after nearly 4 years of service (photos taken in Sep. 2015). (a) Stone asphalt concrete; (b) Epoxy asphalt; (c) Sandwich plate system; (d) Polymer modified asphalt concrete; (e) Proposed LWCD.

Conclusions

In conclusion, the LWCD has shown excellent static and fatigue performance and significant potential for application in long-span bridges. High cracking strength and low permeability of the UHPC layer along with excellent bonding between UHPC layer and steel deck are the keys to ensuring desired performance and durability of the LWCD. Further research should focus on the effects of the following parameters: (1) the ingredients and material ratios, (2) type, shape and volume ratio of the steel fibers, (3) reinforcement ratio of the UHPC, (4) layout of the shear studs, and (5) thickness and size effect of the LWCD specimen on the performance of the LWCD. Also, structural optimization should be pursued to further reduce the cost and to ensure that the stress range levels of the key details are below their CAFLs.▪

Acknowledgements

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The online version of this article contains detailed references. Please visit **www.STRUCTUREmag.org**.

When Structural Blast Design Doesn't Really Include Blast Resistant Design

Damage-Limiting Construction and Explosion Protection by Deflagration Venting

By Kirk A. Marchand, P.E. and Eric L. Sammarco, Ph.D., P.E.

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Many chemical, pharmaceutical, laboratory and general industrial facilities have requirements for storage of chemicals, gasses, fuels, lubricants, and other hazardous materials used in everyday operations. When stored appropriately and not subject to puncture, spillage, and exposure to flame or other ignition sources, these materials are benign and safe. However, accidents and events can combine to cause the unintentional release of these materials and their exposure to flame, electrical arc or other ignition sources. In those scenarios, these materials can, in the best case, burn with significant temperature. In the worst case, as the flame front accelerates in the released combustible materials, the transition from burning to deflagration can occur; deflagration being defined as the propagation of a combustion zone or flame front at a velocity that is less than the speed of sound in the unreacted medium (typically air). Further acceleration of this deflagration could reach supersonic velocity, or could cause an explosion, resulting in significant pressure rise and accompanying damage to the structures in which they are stored.

Because the calculation of 1) the release mechanisms (puncture, rupture, spill), 2) dispersion (entrainment in the air or pools) and 3) ignition and flame spread of and in these materials can be very difficult to quantify, industry methods have been developed to limit the effects of "worst case" releases and ignition through what is called "deflagration venting". This approach is essentially equivalent to the installation of a relief valve on a containment structure, where this "valve" limits the pressure buildup inside the structure to a predetermined and safe level. The "venting" eliminates the need to design the containment structure for a maximum credible event or release through the employment of vent panels or explosive vents, areas of the (typically exterior) wall that are designed to open or "fail" at a predetermined opening pressure.

Similar approaches are very often used in industry for mechanical equipment (hoppers, ducts, etc.) when dusts are a byproduct of manufacturing or processing. Dust can be defined as combustible when they constitute a finely divided particulate solid that presents a flash fire hazard or explosion hazard when suspended in air or a process-specific oxidizing medium. Typical combustible dust can occur where processes produce metal dust, such as aluminum and magnesium; wood dust; plastic or rubber dust; biosolids; coal dust; organic dust, such as flour, sugar, paper, soap, and dried blood; and dusts from certain textiles.

Two industry approaches that can be used to determine the venting required for safe storage of hazardous chemicals or dusts for a particular combination of structure type, stored chemical or potential dusts are the National Fire Protection Association's NFPA 68, *Explosion Protection by Deflagration Venting*, and Factory Mutual's FM 1-44, *Approval Standard and Data Sheet for Storage Buildings and Lockers for Damage-Limiting Construction*. Both of these documents provide approaches and guidelines for venting and construction utilizing venting such that structural damage is mitigated by limiting the pressure rise in a material containment or storage room, or facility.

For flammable gasses, dusts or hybrid mixtures, NFPA 68 provides guidance that has been developed over many decades, starting in 1945. Then titled NFPA 68T, *Explosion Venting Standard*, the document was subsequently improved to bring together all the best available information on the fundamentals and parameters of explosions, test data supporting design approaches, and guidance for the use of vents and vent closures for mitigation of those explosion effects. NFPA 68 is presented with both performance-based and prescriptive procedures and contains extensive explanatory material including further descriptions of deflagration fundamentals, measurement and estimation procedures for reactivity of dust and burning velocity of chemicals, and details regarding vent panel configuration and parametric limitations.

To determine required "safe" vent area, the NFPA and FM approaches provide and define methods to quantify and relate critical chemical, geometric, and structural parameters. Critical chemical and combustion parameters include K_{st}, the deflagration index of a dust cloud, S_u, the fundamental burning velocity of a gas-air mixture, ρ_u, the mass density of an unburned gas-air mixture, λ, the ratio of gas-air burning velocity accounting for turbulence and instabilities, and P_{max}, an optimum maximum pressure expected for a given material in a deflagration in a contained volume. Strictly speaking, the volume of the stored chemical would be an important parameter since a stoichiometric mix (combustible mix of fuel and air) must be achieved for combustion to occur. However, in most instances, sufficient material is available in the stored volume to reach this concentration. Thus critical parameters are based on the chemical with the highest combination of S_u and P_{max}. Although, an adjustment to λ and A_v can be determined through a partial volume determination.

Critical geometric parameters include internal volume (V), A_s, the internal surface area of exterior (non-partition) walls, floor, roof and potential venting surfaces, internal volumes segregated by partitions, and A_{obs}, the surface area of internal obstructions including tanks, drums, pipes, and machinery. A_{obs} is critical, as it directly effects λ and the acceleration of the flame front.

The critical structural and venting parameters are P_{es}, the enclosure strength, P_{stat}, the static activation pressure of the vent, and A_v, the vent area required. The enclosure strength, P_{es}, is defined as the maximum or ultimate internal static pressure that the structure can resist. In the parlance of the structural engineer, this would be an ultimate resistance or capacity of the structural wall, roof, and doors/windows (if included in the resisting portion of the calculation) using expected strengths, but without applying increase factors associated with load rate or inertia (dynamic load factors). P_{es} is further defined as the limiting (typically flexural) capacity of all walls, roofs, doors or windows, or a limiting capacity of any connections between those elements.

The critical chemical/material, geometric, and structural parameters described above are used to determine derivative parameters for vent size determination. One such parameter is P_{red} or the maximum expected pressure inside the containment or storage structure. P_{red} is essentially the pressure for which the vent area and orifices are designed. For relatively ductile structures that can accommodate moderate deformations (as in most reinforced concrete, masonry or structural steel and cladding type systems), P_{red} is defined as follows:

$$P_{red} = \frac{P_{es}}{DLF}$$

Where DLF is the dynamic load factor or the dynamic effect of the rate of rise of the pressure. DLF is further defined as:

$$DLF = \frac{X_m}{X_s}$$

Where X_m is the maximum dynamic displacement and X_s is the displacement produced in the system when the peak load is applied statically. In the absence of detailed analysis using expected pressure rise rate, a DLF of 1.5 can conservatively be used. Similarly, if the structure possesses limited deformation capacity (i.e., where a lack of ductility prevents sufficient deformation before failure), P_{red} is limited to ⅔ of the ultimate strength of the vented enclosure (essentially the same as applying the DLF of 1.5). Other derivative structural calculations might include vent panel reactions and appropriate design for those reactions. If $P_{stat} > = 0.1$ bar, reaction calculations and design are required.

Additional derivative parameters and adjustments might include a further reduction in P_{red} (and a corresponding increase in vent area) if the vents are ducted; i.e., there is a restricted pathway from the vents to the exterior. Minimum distance to air intakes or adjacent structures based on vented fireball diameter must be calculated per the given equations. Acoustic wall linings can reduce λ and a subatmospheric internal pressure can reduce P_{max} and A_v. Vent mass exceeding an upper threshold based on P_{red}, n (the number of vents), V, S_u and λ increase A_v.

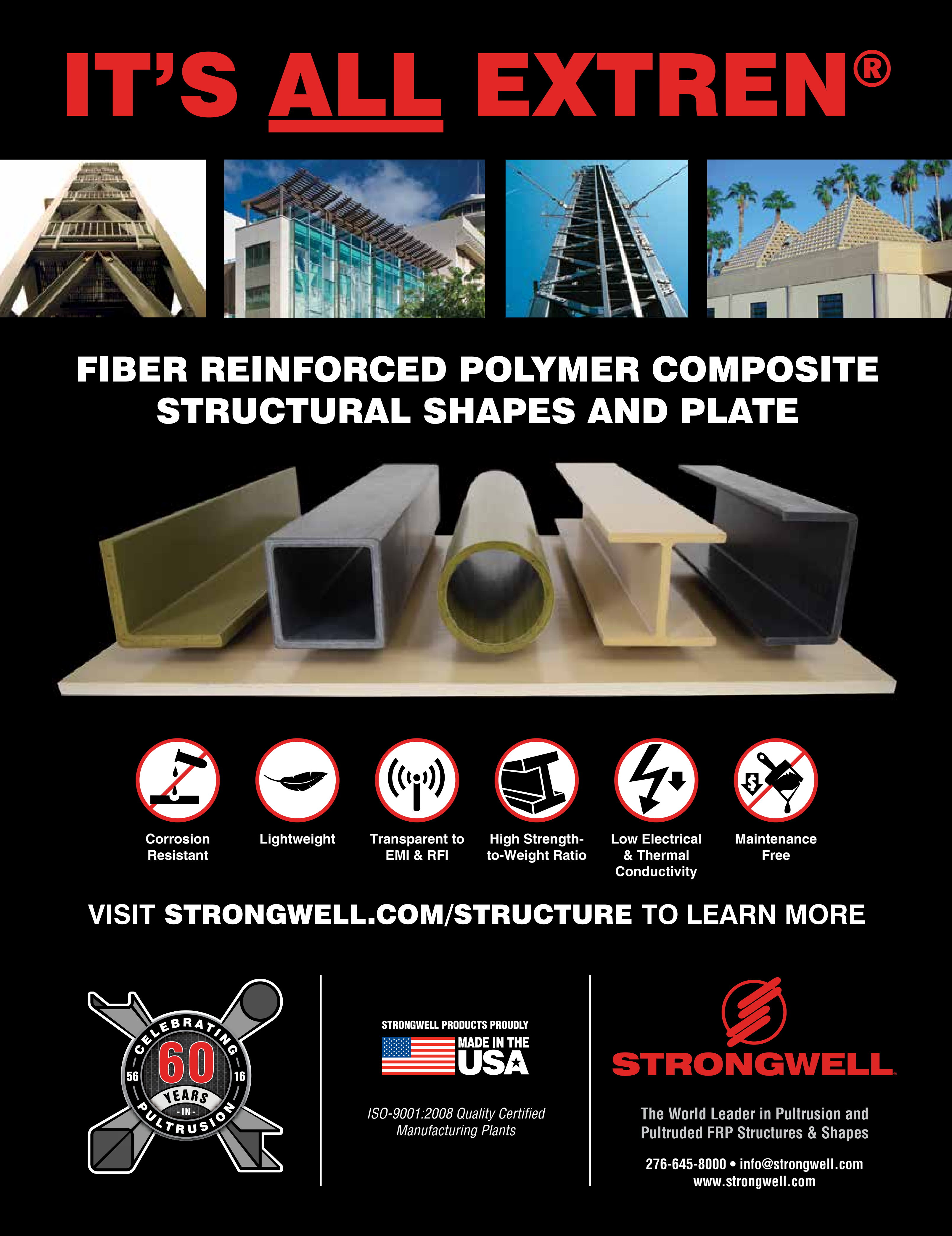
Mechanical vents or simple openings can be used to satisfy the venting requirements. Normally open (louvered or hangar type) vents, as well as normally closed panels with pull-through fasteners, shear pins/bolts, spring, magnetic or friction latches, and closed rupture diaphragms can be used. Tethering of vents may be required to protect adjacent equipment or personnel. Hinged devices must be tested to assure the vents do not deform significantly or become detached during operation. P_{stat} and vent area and weight are provided by the manufacturer. P_{red} is also commonly used to specify the vents. Except for pressures below 0.1 bar, P_{stat} must be no greater than 75% of P_{red}.

Two project examples can be used to illustrate the procedure and parameters described above. The first is a relatively large (40-foot x 20-foot x 12-foot high) enclosure at a manufacturing facility that produces a fuel cell hydrogen carrier, where methanol is used as a key component in the development of fuel cells. Methanol is an ideal hydrogen carrier with more hydrogen atoms in each gallon than any other liquid that is stable under normal conditions. This storage room also has propane tanks for heating operations. The proposed storage room walls consist of 8-inch reinforced CMU with #5 bars every other cell, and the CMU is fully grouted. The storage room roof consists of a corrugated steel deck and 6-inch concrete with #5 bars at 6 inches on center each way. The total area of internal obstructions is 250-foot-square, and the vents are not ducted (open to the exterior).

The first step for a quick determination of required vent area is the selection of fuels. In this case, methanol has a higher burning velocity at 56 cm/sec (propane has an S_u of 46 cm/sec); thus methanol controls. Second, P_{es} should be determined for the structure and its subcomponents. The R_u (in this case the ultimate flexural capacity of the CMU walls was determined to be 2.1-psi, while the ultimate capacity of the concrete-over-steel-deck roof was determined to be 2.4-psi; thus the wall capacity controls. Calculating P_{red} and the derivative parameters and stepping through the NFPA 68 procedure yields a required vent area (A_v) of 198-foot-square, or 48% of the wall area, an undesirably large area. As a first revision to reduce that required area, the wall strength is increased by grouting and reinforcing every cell to yield a new wall capacity at 3.6-psi. However, the roof capacity now controls at 2.4-psi. This somewhat higher P_{es} results in a new and slightly lower required A_v of 186-foot-square; still 39% of the wall area. A final iteration simply reduces the storage area (splits the storage into more than one space) to a 15-foot x 20-foot x 12-foot high space. Because of the reduced roof span (and its increased capacity), the wall capacity (P_{es}) of 3.6-psi now controls, and a new P_{red} results in a required A_v of 79-foot-square, or 33% of the new wall area; both architecturally and structurally acceptable.

A second project example concerns ethyl acetate stored at a pharmaceutical plant. Ethyl acetate is used in the pharmaceutical industry as an extraction solvent. In this case, a low-cost exterior "shed" was desired for drum storage. A 30-foot x 10-foot x 8-foot high rectangular building with 8-inch CMU walls (cells grouted with #5 bars at 32-inch on center) and a 3-inch lightweight concrete-on-steel-deck roof supported by open-web steel joists (OWSJ) at 5-foot spacing was desired. The stored drum surface area was 200-foot-square (A_{obs}), and unducted vents were proposed. Based on a S_u for ethyl acetate of 38 cm/sec, a wall and roof capacity (P_{es}) of 2.5-psi and 1.6-psi respectively (roof controls), a vented area (A_v) of 41-foot-square was determined to be required. While acceptable, a second design iteration was performed to see if structural costs could be reduced by replacing the lightweight concrete with a built-up roof over the same OWSJ system, now at a 4-foot spacing. This reduced joist spacing resulted in an increased roof capacity of 2.0-psi, allowing the wall capacity to control the design. The new required vent area was 47-foot-square, still acceptable and with a reduced structure cost.

These examples illustrate the tradeoffs in volume, structural capacity and even combinations of hazardous materials that can be made to generate efficient designs for deflagration venting in damage-limiting construction. While not used directly, concepts and approaches for determination of ultimate capacity used routinely in blast resistant design can support the optimization of vent sizing. The NFPA 68 and FM procedures are tools structural engineers should be aware of when asked to support the industry with safe and efficient structural systems.■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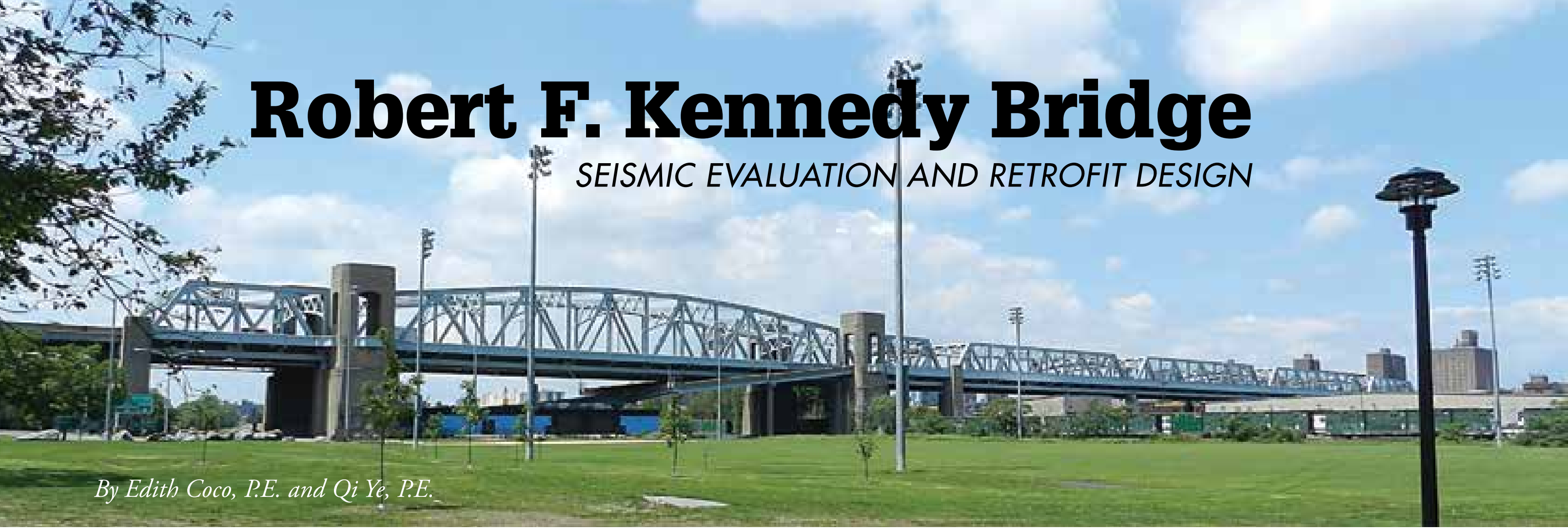
Robert F. Kennedy Bridge

SEISMIC EVALUATION AND RETROFIT DESIGN



By Edith Coco, P.E. and Qi Ye, P.E.

Figure 1. Bronx Kills Truss. Courtesy of T.Y. Lin International.

Othmar Ammann's Robert F. Kennedy Bridge (formerly the Triborough Bridge) is a complex of nearly two dozen bridge structures and approach viaducts and ramps, including the Harlem River Lift, the Bronx Kills Truss, and a suspended span over the East River (*Figures 1, 2, 3* and *4*). The historic bridge complex, one of the largest in the United States, connects the New York boroughs of Manhattan, Queens, and the Bronx, and carries 200,000 vehicles per day. As part of MTA Bridges and Tunnels capital program to ensure the safety of the bridge during earthquakes and strong winds, Thornton Tomasetti Weidlinger Transportation, in a joint venture with T.Y. Lin International, performed seismic and wind evaluations of these structures. This article discusses the seismic evaluation and conceptual retrofit.

During the past 70+ years, the bridge has been reconstructed many times, significantly altering the bridge complex (*Figure 5*). Starting in the early 1990s, Triborough Bridge and Tunnel Authority (TBTA) initiated a long-term $1 billion program to rehabilitate the bridge. The majority of these contracts were deck and bearing replacement. A thorough understanding of the history, geology and structural details of the bridge was vital for proper finite element modeling and analysis of the superstructure and foundations. The project team conducted extensive research to obtain all relevant drawings of the original construction of the bridge and subsequent modifications.

Design Criteria

New York City's bridges are categorized as either "Critical", "Essential", or "Other" based on a bridge's importance and seismic performance objectives. Because the RFK Bridge is a vital link between the New York City boroughs of Queens, Manhattan, and the Bronx, it is categorized as Critical. In accordance with the 2014 New York City DOT *Seismic Criteria Design Guidelines*, the RFK Bridge must be analyzed and designed for two earthquake levels (*Figures 6* and *7, page 27*).

Performance Criteria

As part of the project scope, the team was required to develop project-specific performance criteria for the seismic analysis. The criteria defined the allowable damage for every element of the bridge such as bearings, piers, and foundations for both earthquake levels. These damage levels were expressed as demand/capacity ratios.

It was crucial to develop a set of project-specific design criteria that was consistent with the performance-based design approach to ensure the vulnerability evaluation and retrofit recommendations were rational. Due to the lack of comprehensive guidelines for developing such design criteria for bridges, the engineers most often assume the responsibility for recommending these design criteria based on research and their experience. Thornton Tomasetti Weidlinger used its experience with seismic investigation projects on the Manhattan Bridge and Verrazano Bridge Approaches.

The following standards and specifications were used as reference:

- AASHTO *Guide Specifications for LRFD Seismic Bridge Design, 2011*
- AASHTO *Guide Specifications for Seismic Isolation Design, 2010*
- FHWA *Seismic Retrofitting Manual for Highway Structures: Part 1 – Bridges*
- FHWA *Seismic Retrofitting Guidelines for Complex Steel Truss Highway Bridges, 2006*

Geotechnical Investigation

At the start of the project, geotechnical information from the original construction soil borings was gathered. Also, a geotechnical investigation was completed including test borings, seismic cross-hole testing, and cone penetration testing. In-situ and laboratory testing such as water content, Atterberg Limits, grain size distribution, specific gravity, tri-axial shear, consolidation test and rock unconfined compression tests were also performed on representative soil and rock samples. Subsurface cross sections were developed, as well as soil parameters

New York City seismic hazard levels.

Seismic Hazard	Return Period	Event	Performance Criteria
Upper Level- Safety Evaluation Earthquake (SEE)	2500 Years	2% in 50 years Probability of Exceedance	No collapse. Repairable damage, limited access for emergency traffic within 48 hours, full service within month(s).
Lower Level- Functional Evaluation Earthquake (FEE)	1000 Years	7% in 75 years Probability of Exceedance	No collapse. No damage to primary structural elements, minimal damage to other components, full access to normal traffic available immediately (allow few hours for inspection).

Figure 2. Suspended span. Courtesy of Thornton Tomasetti Weidlinger.

Figure 3. Harlem River Lift Bridge. Courtesy of T.Y. Lin International.

for each stratum. These tests were essential to obtaining the dynamic soil properties which were necessary for the site response analysis and soil structure interaction analysis, where the finite element model inputs are defined, and foundation stiffness matrices are developed.

Vibration Measurement

A field vibration monitoring of the suspended spans, Harlem River Lift Span and Bronx Kills Truss was conducted to collect field measurements. Tri-axial accelerometers were deployed at different locations on the bridge to measure vibration responses in the longitudinal, transverse and vertical directions. They were mounted along the truss chords, cables, and towers (*Figures 8* and *9, page 27*). Measurements were used to identify the dynamic characteristics of the bridge including natural frequencies of vibration and associated mode shapes and damping ratios. These measurements were then used to help calibrate the finite element models of the bridge to reflect the actual behavior better.

Figure 4. Ward's Island viaduct structure. Courtesy of Thornton Tomasetti Weidlinger.

Modeling

Seismic evaluation of large bridge complexes with a large number of components, like the RFK, is rare. Due to the large size of this bridge and its various structures, it was not feasible to analyze it as a single model. The bridge was divided into several parts and individually modeled for analysis. The interaction between adjacent structures was captured by repeating the first two spans of each adjacent structure in the individual models as a boundary condition. This ensured that the stiffness and mass effects of adjacent structures were captured in the overall global behavior.

Most of the structures comprising the RFK Bridge should be considered "complex" structures based on irregular configuration or span length, or high curvature. Structures such as the steel girder bridges or ramps required multi-mode response spectrum analysis and

Figure 5. Robert F. Kennedy Complex. Courtesy of Thornton Tomasetti Weidlinger.

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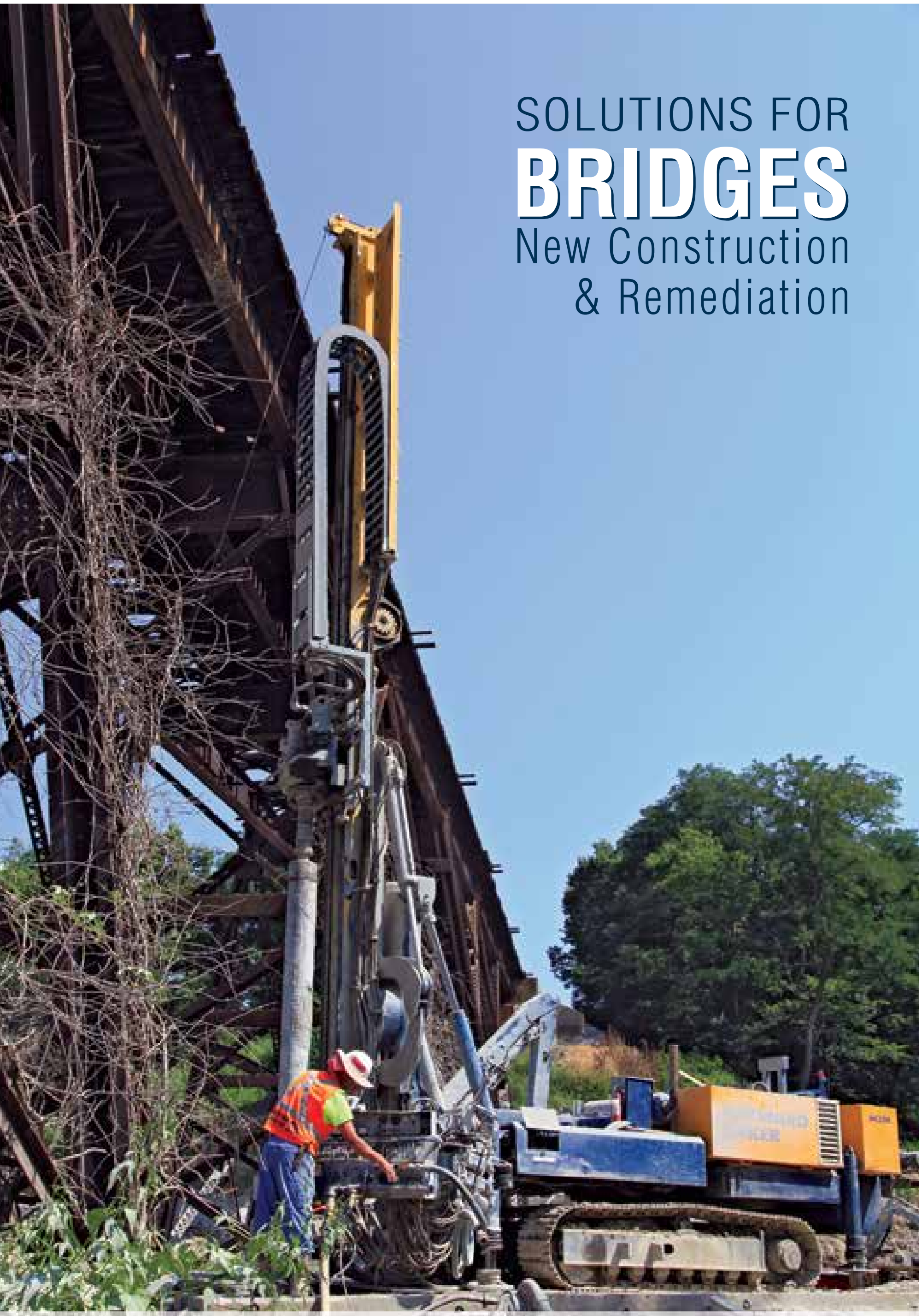
Micropiles for new bridge foundations beneath an existing bridge.


Drilled shafts provided deep foundations for bridges along a freeway.

Vibro replacement for ground improvement beneath bridge retaining walls.


Soils nails to support an elevated freeway.

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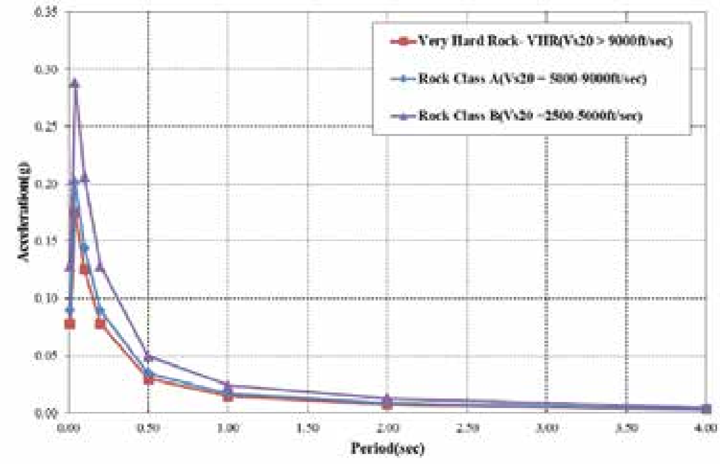

Figure 6. Rock-horizontal design spectra for New York Downstate Zone 1000-year return period. Courtesy of New York State Department of Transportation.

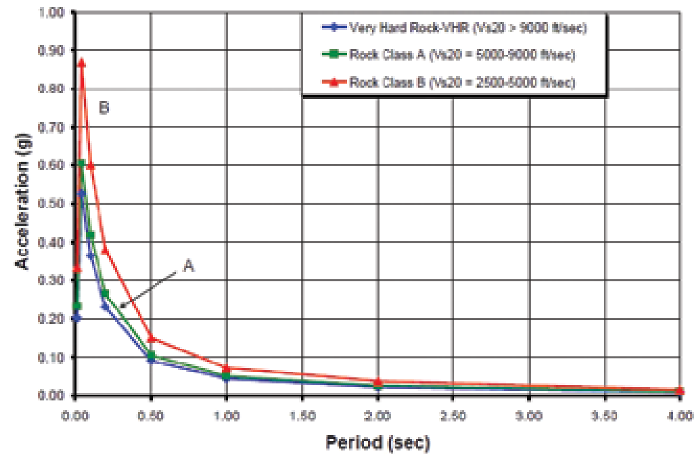

Figure 7. Rock-horizontal design spectra for New York Downstate Zone 2500-year return period. Courtesy of New York State Department of Transportation.

were analyzed using CSI Bridge and SAP2000. However, many of the bridge structures, such as the suspended spans, truss spans, and lift span exhibit highly nonlinear behavior during earthquakes due to the existence of nonlinear elements, such as isolation bearings and cables, and required multiple support time history analysis of seismic effects. These spans were analyzed using the ANSYS and ADINA software programs.

Vulnerability and Conceptual Retrofit Design

Evaluation of the vulnerability of the bridge was performance-based, with the objective of identifying the damage that will occur during both the Lower- and Upper-Level Events. Based on the performance criteria, all structural members in the bridge were checked for potential seismic vulnerabilities. Cost effective conceptual retrofit was then recommended to mitigate those vulnerabilities.

Suspended Span and Anchorages

Overall, the suspended span performed well for both Lower and Upper earthquake events and evaluation revealed minor allowable yielding to secondary members such as laterals.

Seismic vulnerability in the anchorages was found primarily in the steel bent structure that supports the deck. Since the Queens Anchorage was rehabilitated in a previous contract, it was concluded that there was less seismic vulnerability than the Ward's Island Anchorage. Yielding above the acceptable criteria was found in steel columns and secondary members such as diagonals and struts. Retrofit schemes for these elements include strengthening of columns, replacement or strengthening of diagonal bracing, and replacing vulnerable rivet connections with high strength bolts.

Some anchor bolts of elastomeric bearings in anchorages were found to be vulnerable to concrete breakout from the pedestal in the transverse direction of the bridge, due to inadequate concrete cover. A recommended retrofit was to add a concrete overlay to be dowelled to the existing pedestal to provide more confinement and edge distance for the anchor bolts.

It was recommended that vulnerable fixed steel column connections in the Queens and Ward's Anchorage be replaced with new guided elastomeric bearings. Proper installation of elastomeric bearings will reduce the amount of inertial force transmitted to the base connection.

Harlem River Lift Span

Vulnerabilities in the Harlem River Lift Bridge were primarily found in the existing steel structure and bearings.

It was recommended that some of the tower diagonals, counterweight guides, diaphragms, and rails be strengthened with steel plates or replaced. It was also recommended that existing riveted connections be replaced with high strength bolts. Some of the elastomeric bearings on the tower span were found to be vulnerable to lateral capacity and required replacement with higher strength bearings.

Bronx Kills Truss

On the Bronx Kills Truss, a few of the pedestals supporting the bearings that carry the truss spans were overloaded, either in shear or bending. The pedestals may be effectively strengthened by the addition of a layer of reinforced concrete, doweled into the existing pedestal and to the pier cap below.

Some of the disc bearings on this bridge were found to be overloaded laterally. Replacement with similar bearings was recommended, having the same vertical capacity but larger lateral capacity.

continued on next page

Figure 8 and 9. Tri-axial accelerometers attached to the bridge. Courtesy of Thornton Tomasetti Weidlinger.

The expansion joint between the Bronx Kills Truss and the adjacent concrete Junction Structure has insufficient displacement capacity for the FEE earthquake. The existing joint consists of a strip seal anchored to armor angles. It was recommended that this joint be either replaced with a wider joint with greater displacement capacity or to install seismic restrainers to reduce the demands on the joint.

Concrete Viaduct and Ramp Structures

The approach viaducts and ramps on this bridge consist of a steel girder/stringer superstructure supported on reinforced concrete piers on spread footings or pile supported foundations.

Most of the concrete elements were detailed before modern seismic requirements and were found to have inadequate development lengths and confinement at joint locations. Some vulnerabilities in the concrete structure include vulnerable cap beams and columns at the footing interface. It was recommended that these locations be strengthened with concrete jackets with reinforcement doweled into the existing structure.

For the superstructure, it was found that some bearings should be replaced since they are old NYSDOT standard elastomeric bearing details. Other elastomeric bearings can be retrofitted with the addition of transverse restrainers and longitudinal dampers.

A few locations had vulnerable concrete footings and pile caps. These locations can be strengthened with a concrete overlay and doweled shear reinforcement. At locations with vulnerable piles, pile caps should be extended, and new piles added. On the Queens approach structure, the number of foundation vulnerabilities would be reduced with the addition of transverse and longitudinal bracing in between pier columns.

Some expansion joints were found to be vulnerable. It was recommended that they be replaced to accommodate larger superstructure displacements.

Conclusions

A comprehensive study of the entire Robert F. Kennedy bridge complex was performed to determine if the bridge meets current seismic criteria and standards. The study also ensured that the bridge responds to a seismic event in a predictable manner, to protect the safety of the public, and to identify the vulnerabilities to be retrofitted to prolong the lifespan of this bridge in a practical manner.

This study serves as a prototype for seismic assessment of other older bridge complexes like the RFK that consist of various structures and bridge types.■

Project Team

Owner: MTA Bridges and Tunnels – TBTA
Joint Venture: Thornton Tomasetti – Weidlinger Transportation Practice, T.Y. Lin International
Geotechnical Engineers: Mueser Rutledge

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Construction of the WORLD'S LONGEST FLOATING BRIDGE

By W. Gregory Hess, P.E. S.E.,
Jason B. K. Pang, P.E. and Ben Nelson, P.E.

Aerial view of State Route (SR) 520 Evergreen Point Floating Bridge high-rise superstructure under construction.

The new Evergreen Point Floating Bridge, which carries State Route 520 (SR 520), is the world's longest floating bridge, stretching 7,708.5 feet across Lake Washington in Seattle, Washington. Opened to traffic in April 2016, the bridge replaces the previous SR 520 floating bridge, which was completed in 1963 and had reached the end of its useful service life. The new bridge was constructed in place, adjacent to and just north of the old bridge. Construction of the bridge required a highly-coordinated process to ensure that pontoon freeboard and concrete stresses in the pontoons were maintained within acceptable limits throughout construction.

Floating Bridges of Washington State

Floating bridges have been a major part of Washington State's infrastructure since 1940 when the first floating bridge was constructed across Lake Washington. Washington is currently host to four of the five longest floating bridges in the world; the William A. Bugge Bridge which crosses Hood Canal (6,521 feet), and three bridges that cross Lake Washington: the Lacey V. Murrow I-90 bridge (6,620 feet), the Homer M. Hadley I-90 bridge (5,811 feet) and the new SR 520 floating bridge (7,708.5 feet). The fourth longest floating bridge is the Demerara Harbor Bridge (6,074 feet) in Georgetown, Guyana.

The original floating bridge over Lake Washington was the brainchild of Homer M. Hadley, an early 20th century Seattle engineer and namesake of one of the two I-90 bridges. His grand scheme of a concrete pontoon floating bridge across Lake Washington is said to have originated from his experience designing barges during World War I. Because of the depth of Lake Washington, over 200 feet in some locations, and very soft soils consisting of thick volcanic ash deposits, traditional bridges have proven too costly to construct.

Bridge Configuration

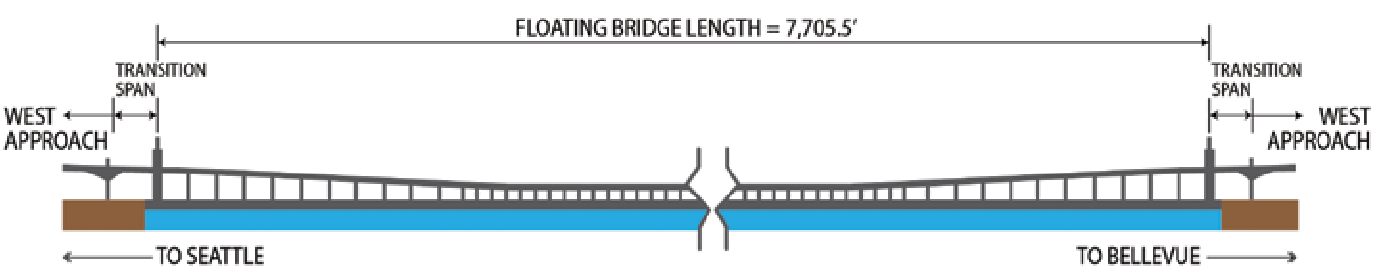
The new SR520 floating bridge is unique in that the entire roadway is elevated above the pontoons over the full length of the bridge. Other Lake Washington floating bridges utilize the top deck of the pontoon as the driving surface for a majority of the length. The benefit of the elevated roadway is two-fold. First, it keeps vehicles above the lake's wave spray and splash that occur during large storm events. Second, it provides a maintenance corridor below the elevated roadway that allows maintenance staff access to the pontoons.

The bridge currently has six vehicular lanes and a 14-foot wide bicycle and pedestrian path on the north side. It was also pre-designed for future widening that would allow for the addition of two light rail train lines down the center.

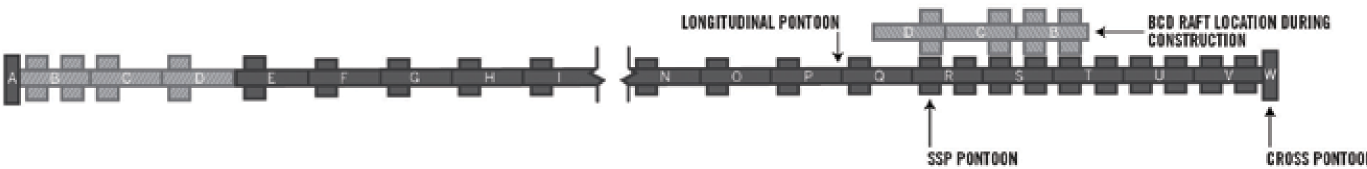
A precast concrete, segmental, ribbed-superstructure slab post-tensioned in two directions, referred to as the "low-rise", makes up the center 5,580 feet of the elevated structure. At the east and west ends of the floating bridge, the elevated structure transitions upward to provide for acceptable navigation clearances at the approach structures. This portion of the elevated structure is referred to as the "high-rise" and is comprised of prestressed precast girders with a cast-in-place deck supported on cast-in-place crossbeams and columns. The floating bridge is flanked by 190-foot-long steel I-girder transition spans that connect the floating structure to the fixed land structures. The transition spans and their connections are designed to accommodate all six-degrees of differential movement that can occur between the floating bridge and the fixed approaches.

Pontoons and Ballast

The backbone and floating portion of the bridge are the pontoons themselves, cellular concrete box structures. They were constructed off-lake in Tacoma, WA and Aberdeen, WA and towed to Lake Washington for assembly. A total of 77 concrete pontoons are joined together to complete the floating bridge. There are three types of pontoons: two cross-pontoons, which are added to the ends of the bridge for additional stability and buoyancy; 21 longitudinal pontoons, which make up the spine of the bridge; and 44 supplementary stability pontoons (SSPs), which are post-tensioned to the longitudinal pontoons to provide additional stability and buoyancy. Each longitudinal pontoon is 360 feet long by 75 feet wide and about 28 feet deep. Once ballasted down to their design 'freeboard' of 7 feet, the pontoons draft 21 feet of water. The width of the pontoons was



Elevation of the floating bridge.



Plan view of pontoons.

limited by the 80-foot clear opening of the Hiram M. Chittenden Locks in Ballard, WA, through which the pontoons must pass at the end of their ocean voyage up the Washington coast from Aberdeen.

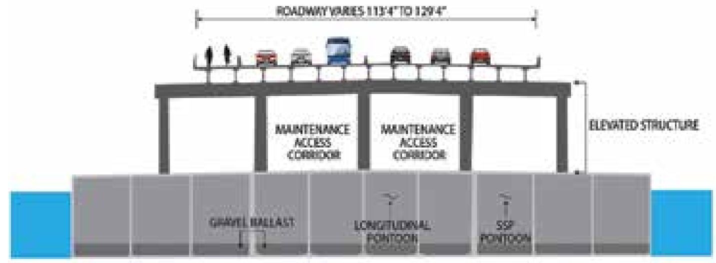
The floating bridge is essentially a permanently moored floating structure that is laterally supported in the longitudinal direction and transverse directions by 3.125-inch diameter anchor cables. There are a total of 50 transverse anchor cables spaced approximately every 360 feet and eight longitudinal anchor cables which are connected near both ends of the bridge. Anchor cables extend as much as 800 feet and are affixed to specially constructed anchor structures at the bottom of the lake. The anchor cables have 60 tons of pre-tension to enhance their stiffness.

Ballast is a critical component of the floating bridge that allowed the contractor to keep the pontoons trim, or raise and lower the pontoons as required during various stages of construction. Water ballast, which can be readily pumped in and out of the pontoons, was used as temporary ballast during construction. When the pontoons are brought together for joining, adjacent pontoons are ballasted to within one half-inch of each other. A series of rams and winches are used to pull the pontoons together and align shear keys so that large, 20-foot long by 3.5-inch diameter post-tensioning rods, also known as bolts, can be installed. There are a total of 80 bolts evenly distributed around the perimeter at each longitudinal joint. Also, ballast must be continuously removed during the elevated structure construction to keep the pontoons trimmed within allowable tolerances while avoiding locked-in stresses in either the pontoons or the elevated structure. At the end of the floating bridge construction, all water ballast was removed and replaced with permanent gravel ballast. It should be noted that the bridge was designed so that a sufficient amount of 'reserve' permanent ballast is available for a future widening to accommodate light rail. The widened configuration would utilize the buoyancy from reserve ballast and require 26 additional SSPs to be added along the length of the bridge to offset the extra weight.

Floating Bridge Design Philosophy

Unlike traditional land-based bridges in Washington State, which are usually controlled by seismic loads, floating bridges are governed by wind and wave forces. The floating bridge was designed to withstand a 100-year storm, defined as a storm having 98 mph winds and 6-foot waves. The pontoons, which are fully post-tensioned structures, are designed for zero tension stress in the pontoons under service conditions with rigorous crack control criteria for extreme loading combinations. At the extreme 100-year event,

Aerial view of mainline bridge and BCD raft pontoon.



Cross-section of floating bridge.

the pontoon hull reinforcing is designed to stay well within the elastic range. The elevated structure was designed to accommodate the imposed deflections from the pontoons and accelerations associated with the 100-year storm event.

Construction Staging and Analysis

Assembly of the floating bridge required a highly planned, coordinated and choreographed effort to allow for multiple construction activities along the length of the bridge. For example, while Pontoons O and P were being joined, cross-beams and columns were being cast on Pontoons S and T and girders were also being set on Pontoons U and V. This orchestration of construction activities required careful coordination between the contractor, the project naval architect, and the project structural engineer.

The project freeboard criteria required that the difference in freeboard between opposing sides of the pontoons, and along the length of the pontoons over a distance of 360 feet, be less than 2 inches at all times during construction. Over 1,500 unique construction steps were analyzed as part of the construction staging process to ensure that freeboard and stress criteria were maintained throughout construction. At times, analyzed loadings exceeded contract freeboard requirements, and pontoons were pre-ballasted for 50 percent of the out-of-balance construction load to stay within project freeboard tolerances.

An analytical model of the floating bridge using commercially available structural engineering software was developed to perform the construction staging analysis. For vertical and torsional loading, the floating bridge was analyzed as a continuous beam on an elastic foundation using roll, pitch and vertical support springs located along the bridge's longitudinal axis to represent the foundation stiffness, in this case based on the density of water. As the bridge is constructed, the center of mass tends to grow in height as ballast is removed from inside the pontoons and the elevated structure constructed above. Since a

Placing concrete from the mainline to BCD raft. Courtesy of KGM.

pontoon's roll stiffness is a function of both the water plane area and the mass center, the foundation springs were updated throughout the staged construction process. The construction staging model stress output was used to check floating bridge stresses for each of the 1500 construction steps analyzed.

The input for the construction staging analysis required detailed loading data from the contractor and naval architect. First, work activities for each construction step were developed and defined by the contractor. These steps were then analyzed by the naval architect who developed a ballast adjustment plan to balance the construction loads within contract freeboard requirements. For example, when a series of columns or crossbeam were poured, a corresponding activity to remove a proportionate amount of water ballast was required to keep the bridge trim. Once the construction activities and ballasting requirements were defined, the structural engineer used the structural model to check stresses in the pontoons and the elevated structure.

Pontoons B-D Elevated Structure Construction

Pontoon assembly and elevated structure construction progressed from both the east and west ends of the bridge. The last three pontoons, Pontoons B through D, were set in July 2015, which completed the assembly of the entire pontoon string. One of the biggest challenges the floating bridge project faced was finding a way to provide and maintain vehicular access for materials from land to the bridge. The contractor's goal was to turn a 'marine project into a land project'. For example, the ability to drive concrete trucks to a pour location versus having to tug a barge loaded with concrete trucks is a much more efficient and economical means of delivering concrete.

The east end of the floating bridge was more accessible than the west end due to its proximity to shore. A temporary bridge and several barges were linked to form an access trestle in the early stages to connect the easternmost pontoon, Pontoon W, to land. Once the east fixed approach structures and transition spans were connected, access shifted from the trestle to the finished roadway at the east end. At the west end of the bridge, Pontoon A, which is located farther

Land to water access trestle.

away from shore than Pontoon W, was not readily accessible from land. To achieve 'land access' for the west portion of the bridge, a three pontoon string, consisting of Pontoons B, C, and D (known as the "BCD raft"), was constructed and then moored alongside Pontoons R, S, and T at the east end of the bridge. The superstructure was constructed on Pontoons B, C, and D while it was temporarily moored at this location. In addition to improved access, this allowed all construction activities to take place simultaneously.

A series of 8-foot-diameter Yokohama-type marine fenders, existing bollards, and mooring lines were used to secure the BCD raft to the main string. A small ramp between the two structures was constructed for access. Once the raft was moored, construction vehicles and crews were able to drive directly from land onto the raft. Concrete pours for the westerly high-rise elevated structure on BCD raft were done by staging pump and concrete trucks on the easterly high-rise portion located on Pontoons R, S and T. Columns and crossbeams were poured by driving pump trucks and concrete trucks from shoreline directly onto the BCD raft. Decks were poured with a 61-meter pump truck staged on the mainline superstructure. A 100-foot-long tremie pipe extension was connected to the end of the pump truck discharge and supported in the air by a 400-ton crane. Construction of the BCD raft elevated structure was completed in July of 2015. The entire raft was pushed as a single unit, like a large 1,000-foot-long ship, to the west end of the bridge where it was joined to Pontoon E and eventually the westernmost Pontoon A.

The grand opening of the new SR 520 Evergreen Point Floating Bridge was a historical event attended by over 50,000 people on April 2 and 3, 2016. The bridge officially opened to vehicle traffic on April 11, 2016. Currently, the old SR520 floating bridge has been decommissioned and is being removed from the lake. The old pontoons have been sold and will be repurposed and reused globally for port expansion, marine offloading facilities, marinas, an off-shore floating stage, and breakwater construction projects.▪

Project Team

Owner and Pontoon Designer: Washington Department of Transportation
Prime Designer and Civil/Structural Engineers: KPFF Consulting Engineers, Seattle, WA, and BergerABAM, Federal Way, WA
Designer/Builder: KGM, a joint venture comprising Kiewit, Omaha, Nebraska; General Construction Company, Federal Way, WA, and Manson Construction Co., Seattle, WA
Naval Architect: Elliot Bay Design Group

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*Ben Nelson, P.E., is the 520 floating bridge construction manager for KGM, a joint venture comprised of Kiewit, Omaha, Nebraska; General Construction Company, Federal Way, Washington; and Manson Construction Co., Seattle, Washington. Ben can be reached at **ben.nelson@kiewit.com**.*


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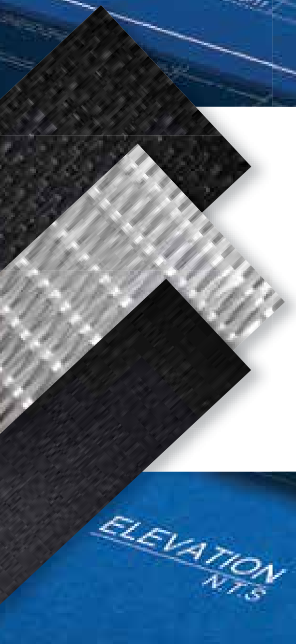

Cracking Moment and Safety of Post-Tensioned Members

By Bijan O. Aalami, Ph.D., S.E., C.Eng

When major building codes are not in agreement on specific structural members, confusion and uncertainty are propagated among design engineers. In this case, the industry may need to start a conversation with code authorities for clarification.

Three issues specific to the design of post-tensioned members are currently treated differently by the American Concrete Institute's *Building Code Requirements for Structural Concrete and Commentary*, ACI-318 (2014), and Europe's *Design of Concrete Structures – Part 1-1 General rules and rules for buildings*, European Code EC2 (2004). For any designer working globally, these two code documents define the design of post-tensioned members. Also, the go-to literature for the design and analysis of PT members is contradictory. The three sources considered here are the Post-Tensioning Institute's *Design Manual* (2000) and two design treatise, one by E. G. Nawy and one by the author of this article. (*See the references included in the online version of this article for specifics.*)

The discrepancies are found with regard to:

1) The value of the bending moment at which a section must be reinforced to be safe at the initiation of cracking. This is typically referred to as the "cracking moment" of the section, M_{cr}.
2) Whether the provision for safety at the initiation of cracking due to the bending moment applies to members reinforced with bonded, unbonded, or both bonded and unbonded post-tensioning tendons.
3) Which structural systems the provision applies to – beams, one-way slabs, two-way slabs, or all structural systems.

Given the increasing popularity of post-tensioned buildings worldwide, it seems logical that an effort should be made to standardize these design criteria.

Value of the Cracking Moment

The cracking moment of a post-tensioned member is of interest to structural engineers for two reasons. First, depending on the application, cracking may need to be limited for serviceability concerns. In these cases, the objective is to avoid the "initiation" of cracking. The second reason is the "safety" of the member; in these cases, the aim is to ensure that the section at the location where the crack forms has adequate strength to avoid failure, irrespective of the cause of the cracking. The serviceability objective is achieved by controlling the conditions that lead to cracking. The focus of the two major building codes, ACI 318 and the European Code (EC2), is about the safety of the member if cracking occurs.

The following sections explain both the safety and serviceability objectives and include applicable expressions for the calculation of the cracking moment.

Service Condition

Cracking at a section begins when the extreme fiber tensile stress of the member reaches the modulus of rupture, f_r, of the section's material. ACI 318 recommends that f_r for concrete be calculated as: $f_r = 7.5\sqrt{f'_c}$ (*Equation 1*)
When considering the condition that leads to cracking at a given section in a post-tensioned member, such as section X-X in the member shown in *Figure 1a*, it is recognized that the post-tensioning tendons at the section provide a stress distribution as shown in *Figure 1b*. *P/A* is the precompression from the post-tensioning force *P*, acting on the section with cross-sectional area *A*, and f_b is the extreme fiber stress from the post-tensioning moment.

Where, $f_b = M_{pt}/S$ (*Equation 2*)
M_{pt} is the moment from post-tensioning and *S* is the section modulus. M_{pt} is also referred to as the balanced moment since the post-tensioning is typically designed to balance a certain percentage of the dead load. The magnitude of M_{pt} depends on the geometry and support conditions of the member, as well as the tendon profile and post-tensioning force. It includes both the primary moment (*Pe*) and the hyperstatic (secondary) moment from the reactions at the supports as a result of the post-tensioning.

For cracks to form at section X-X, the applied forces on the member must exceed the combined extreme fiber compressive stresses from the prestressing, f_b and *P/A*, to the extent there is a tensile stress f_r. Hence the required moment at the section at the initiation of cracking, calculated from principles of static equilibrium will be:
$M = (f_r + \frac{P}{A})\,S + M_{pt}$ (*Equation 3*)

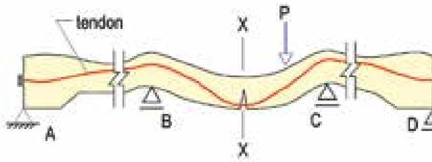

Figure 1a. Partial elevation; continuous member reinforced with post-tensioning forces leading to cracking at section X-X depend on the overall geometry of the entire structure, features of its post-tensioning tendons and the applied load (PTS798).

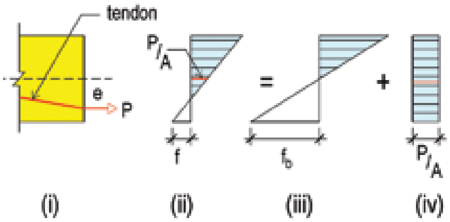

Figure 1b. Distribution of stress from post-tensioning on a section. P/A is the average precompression; f_b is the extreme fiber stress from post-tensioning moment, M_{pt}. P is the post-tensioning force and e is the eccentricity of the force with respect to the section's centroid.

Safety Condition

The second area of interest relative to the cracking moment is the "safety" of the member. Unlike serviceability which is concerned with the member before cracking, the safety objective requires consideration of the relevant section after cracking. In the case of cracking having occurred, the objective is to ensure that the section at the location where the crack forms has adequate strength to avoid failure, irrespective of the cause of the cracking.

Although the focus of ACI 318 and European Code EC2 is on the serviceability factors, both codes include text on the safety requirement after cracking; however, neither provides details on the required calculations. Also, the topic is addressed differently in the publications above.

The safety of a section after the initiation of cracking is achieved by providing adequate reinforcement. The reinforcement should develop a moment not less than the moment that initiated cracking. At cracking, the distribution of stresses over the section is as shown in *Figure 1b* (*ii*), irrespective of the overall characteristics of the member, such as the number of spans or the loads on the member.

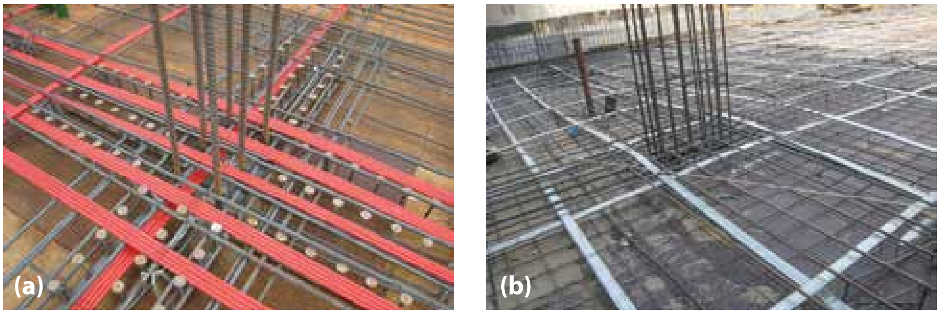

Figure 2. Two-way slab construction with unbonded and bonded post-tensioning tendons. (a) Slab construction with unbonded tendons. Neither ACI nor EC2 requires a strength check at the initiation of cracking (P998). (b) Slab construction with bonded tendons ACI requires a strength check at the initiation of cracking. EC2 does not (P999).

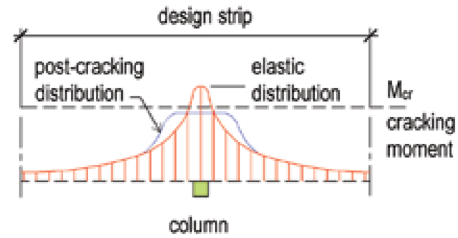

Figure 3. Plan – Tributary (design strip) of a column support in two-way slab construction. Distribution of moment at the face of column before and after cracking. Loss of capacity due to the cracking spreads the cracking and mobilizes the slab's strength beyond the column (PTS761).

The moment for this distribution, M_{cr} is:

$M_{cr} = (f_r + P/A)\ S$ (*Equation 4*)

The difference between the numerical values of the two conditions, *service*, and *safety*, is that the service condition includes the post-tensioning moment M_{pt}, with the hyperstatic (also referred to as secondary) moments as an integral part of it. The applicable moment M_{cr} for the safety condition includes only the precompression (P/A) from prestressing.

In summary, to avoid failure of a section at the initiation of cracking, the reinforcement in the section must develop a moment capacity M_n not less than the moment M_{cr} that initiated the crack. Factors of safety must be applied both to reduce M_n and increase M_{cr}. K, the factor of safety for M_{cr} depends on the building code; ACI 318 recommends a safety factor of 1.2.

$\phi\ M_n \geq KM_{cr}$ (*Equation 5*)

Where, ϕ is the strength reduction factor.

A possible source of discrepancy between the calculations for the two conditions could be the interpretation of the wording used in ACI 318 -14, where the requirement is expressed in terms of "cracking load," as opposed to cracking moment. The European Code EC2 states the requirement in terms of the "cracking moment."

Evaluation of the Cracking Moment

ACI 318-14 requires the evaluation of the cracking moment for slabs reinforced with *bonded* tendons (*Figure 2b*) – both one-way and two-way systems, using a factor of safety K=1.2. Evaluation of the cracking moment is not required for slabs reinforced with unbonded tendons based on the argument that, because there is no bond between the strands and the concrete, a sudden local increase in strain at the crack location will be distributed over a length of strand crossing the crack, thus avoiding a local rise in strand strain and the strand's rupture (*Figure 2a*).

The European Code EC2 also requires an evaluation of the cracking moment, but only for beams reinforced with *unbonded* tendons. Per EC2, a safety factor of 1.15 is required. ACI 318-14 requires the same with a factor of 1.2 for beams reinforced with bonded tendons. Clearly, there is a disagreement as to whether to apply a safety factor to unbonded or bonded tendons or do designers assume that the safety factor applies to both conditions?

Note that the distribution of moment in beam stems and one-way slabs is essentially uniform in the transverse direction to the span. Initiation of cracking at one point will result in the extension of the crack over the entire width of the beam stem or slab width. For two-way, column-supported slabs, however, due to the non-uniform distribution of stress (*Figure 3*), loss of capacity due to cracking at the location of maximum moment demand will result in a redistribution of the moment to regions where capacity is available, thus providing greater resilience for two-way slab systems. The moment redistribution property of the two-way system is the likely reason why in the European code it is not required to check its safety at the initiation of cracking.

Literature Discrepancies

ACI 318-14 applies only to *two-way slabs* reinforced with *bonded* post-tensioning:

> *8.6.2.2.2 – For slabs with bonded prestressed reinforcement, total quantity of A_s and A_{ps} shall be adequate to develop a factored load at least 1.2 times the cracking load calculated on the basis of f_r defined in 19.2.3.*

(Note: the value of f_r as defined in section 19.2.3, is given in Equation 1 above.)

EC2 (European Code) applies only to *beams*. EN 1992-1-1:2004, Section 9.2.1-1(4) states:

> *For members prestressed with permanently unbonded tendons or with external prestressing cables, it should be verified that the ultimate bending capacity is larger than the flexural cracking moment. A capacity 1.15 times the cracking moment is sufficient.*

The PTI *Design Manual* Fifth Edition, Section 5.3.3.5, explains the application of the ACI 318's statement for strength calculation of members after cracking as follows:

> *… the requirement is intended to prevent abrupt flexural failure immediately after cracking… This requirement is commonly satisfied, when required, at each section as follows:*

$$\phi M_n = 1.2(7.5\sqrt{f'_c}S + \frac{SF}{A} + M_{bal}) \qquad (3\text{-}1)$$

This is the same as expression *Equation 3* above, with the safety factor (1.2) recommended by ACI 318 added, and M_{bal} used to represent the post-tensioning moment M_{pt}, and F is the post-tensioning force.

PTI's expression (3-1) is also used frequently in the literature for the safety of section after cracking.

Conclusions

In addition to the differences in the literature on the calculation of the demand forces for the safety of a section at the initiation of cracking, the requirements as to when the cracking moment needs to be evaluated to verify the safety are different in ACI 318 and EC2. Observation of thousands of buildings designed according to each of these codes, and the apparent lack of reported failure resulting from these provisions, could imply that the requirement may not be critical in the design of post-tensioned members. But, if they are not of critical concern, why are they detailed in the codes? However, in light of the increasing popularity of buildings constructed with post-tensioning, the safety requirements of post-tensioned members at the initiation of cracking should be revisited and clarified.▪

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The online version of this article contains detailed references. Please visit **www.STRUCTUREmag.org**.

STRUCTURAL REHABILITATION

renovation and restoration of existing structures

Repair of Log Cabins

By Paul A. Gossen, P.E., F.ASCE

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Log cabins have been around for a long time. The first evidence of these structures dates back to the Bronze Age in northern Europe.

They were used as temporary shelters because they were easy to assemble and disassemble. Fabrication and shaping of the logs were minimal and required very little work.

In 1600, immigrants from northern Europe introduced these log cabins to North America. At that time, they were very basic but quickly gained popularity because they incorporated many architectural features that made them suitable for living quarters. Horizontal logs, stacked one above the other, served as the exterior cladding with a good thermal insulation, and as the interior wall finish. In the old designs, gaps between the round logs were filled with clay reinforced with hay. However, round logs stacked one above the other to form a wall are inherently unstable. Only the interlocking of the logs at corners provided the stability. When cutting the logs for openings, such as doors and windows, only one corner would provide stability, which was insufficient. Vertical timbers, i.e. vertical girts, were then added to connect the loose ends of the cut logs and tie them to continuous logs above and below the opening, or to grade. The size of the buildings was dictated by the length of the walls, which could only be as long as the tree trunks used. Splicing of the round logs within the length of a wall was not possible without tie irons or very long spikes. Those were seldom used for round logs. In later designs, the logs were shaved flat at their mating surfaces to achieve a better seal and to make the walls more stable when fastening the logs together with spikes. It even allowed logs to be spliced within the length of the wall, which overcame the restrictions set by the tree trunk length. This flexibility enabled smaller log sections to be used. Interlocking logs at the corners were and are still used today as a hallmark of the cabins.

Log cabin structures are still popular today. They symbolize a return to environmentally friendly living. The log cabins that are manufactured today use router shaped logs for a tighter fit, as well as splines that are inserted into a groove in the top and bottom of the log. The spline seals the horizontal joint between the logs. Logs are fastened to each other by long spikes or log screws.

The shaping of sections for a tighter seal in some designs, however, creates some unintended consequences that jeopardize the life of the logs. These horizontal mating surfaces result in a tighter seal but do not drain rainwater that runs down the facade. Water can enter the joint and finds its way into the groove of the spline, causing dry rot of the log.

Treatment of Dry Rot of Log Cabin Walls

Dry rot in log cabin walls is generally treated by three methods. The choice of treatment depends on the extent of the damage:

1) If only a few logs are affected, they may be replaced by cutting out the logs and inserting new ones. This procedure, however, is not as straightforward as it seems. The logs are interconnected through matching contour surfaces or with splines

Figure 1. Elevation of deteriorated log wall.

Figure 2. Close-up of rotten logs.

in reglets that are destroyed during the replacement operation. It is impossible to reinsert the spines or match up the reglets unless the log is at a corner where the timber can be slid into place horizontally.

2) Another repair procedure is to treat the decayed logs with an epoxy or petrifying compound. The timbers are injected with a compound that stops the rot and solidifies the decayed wood sections. This procedure is very expensive and has drawbacks in terms of fire safety and insulation value. Though the code allows a one-hour fire rating for heavy timbers (most logs qualify for this provision), a code enforcement officer may not label the treated logs as one hour rated, depending on the quantity of resin that has been injected into the timber. Epoxy and petrifying compounds are more flammable than heavy timbers. In most cases, however, the one-hour rated construction is not required because the building may be a one family residence with not more than two stories, and may not abut a garage (which could require a fire rating).
3) The most radical treatment would be to shore up the roof and rebuild the walls. This alternative is not cost effective.

The Log Cabin in Roxbury, New York

A log cabin in Roxbury had a dry rot problem. The logs on three walls had lost 50% of their section due to dry rot (*Figure 1*). The decay was on the exterior face of the wall, through half the width of the logs. The logs were seven inches wide and six inches high (*Figure 2*).

The log surface was highly irregular and had horizontal splits. An attempt had been made to seal the gaps with silicone sealant. The extent of the damage was such that the structural integrity of the building was in question. This structure, located in the Catskill Mountains, must support code loads based upon a ground snow load of 70 pounds per square foot (psf). As a result, the roof, as well as the log walls, must support the roof dead load and snow load, a total of 60 psf. The interior was not affected by the decay and exhibited the beautiful texture and wood finish that makes these structures so appealing (*Figure 3*).

The logs were shaved flat on the top and bottom with a groove in the center that received a continuous spline (*Figure 4, page 38*). This spline has no structural significance and is used solely to provide an air seal between the logs. Also, 10-inch vertical spikes placed alongside the groove at approximately 2 feet on center, tied the logs together. A close examination showed that the sill plate, on which the logs rested, and the top plate that supported the rafters, were in good condition.

The challenge was to find a solution that would retain the interior logs as finished walls but would at the same time render the walls structurally sound.

The Solution

The design was straightforward; however, the solution was a challenging one. Ultimately, this was only possible through the use of modern construction technology. Laser equipment and stud finders were the essential equipment in making this repair procedure feasible.

The solution consisted of removing the decayed wood and reinforcing the walls, which are 8 feet high, with pressure treated 4x4 posts and then covering the wall with a new exterior facing (*Figure 5, page 38*).

The posts were designed to carry the roof load. They also served as stiffeners for the remainder of the log wall section, since approximately 50% of the 7-inch log width was rotted, leaving only 3½ inches of sound log material. The remaining 3½ inches stacked log sections formed a slender wall that required bracing. The posts then would be covered with a new exterior sheathing and log siding.

Since the work was done during the late fall and early winter, the area was enclosed and heated. A large tarp covered the area to protect the workplace and to enable the laser beams to be more visible (see below).

The procedure was as follows: vertical slots were cut into the outside of the log walls. The slots were 3.5 inches deep and 4 inches wide, extending from the bottom plate to the top plate. Slots and posts were spaced at approximately 2 feet on center, depending upon the location of windows. A stud finder was used to locate the spikes so they could be avoided when cutting the slots.

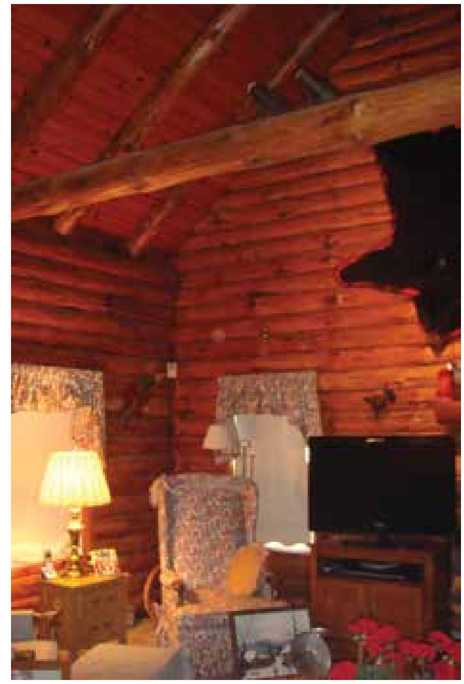

Figure 3. Log cabin interior worth saving.

The design called for a ½-inch thick AdvanTech sheathing, an engineered moisture resistant high-performance product, on the outside to cover the studs. The studs needed to align to a common vertical and horizontal plane to fasten the sheathing. The log wall was very uneven, and there was no location on the structure that could provide a plane for such a reference. The contractor solved this issue by creating a virtual vertical plane parallel to the face of the existing wall using a laser as a guide. The tarp enclosure reduced the light in the work area so that the laser beam was easier to track. A constant horizontal distance from this virtual vertical plane was measured, establishing the cutting depth for the slots. Slots were cut with a small chainsaw and the wood (or what was left of it) chiseled out to a predetermined depth.

After cutting the slots, all loose and decayed material was removed. A preservative was then applied to the remaining exposed wood on the outside to discourage further decay. With the logs at 7 inches on center, the logs are 7 inches high; the posts were screwed to every other remaining log section vertically at fourteen inches on centers. At adjacent posts, the screws were offset vertically by seven inches so that every other post was screwed to the same log section (*Figure 6, page 38*). The posts, connected to the remaining log sections, provided the intended stability for the remaining log wall. The cavity left from the removed log material was filled with a closed cell insulating foam. The placement of the foam replaced and even enhanced the insulation lost by removing the rotten log sections.

continued on next page

Figure 4. Log profile; note reveal for the spline.

The sheathing was screwed to the stud, and Tyvek protective barrier house wrap was stapled over the sheathing. New log siding was fastened to the wall as an exterior finish. The siding matched the old log appearance and was stained to match its color (*Figure 7*). The repaired wall ended up being 2½ inches wider than the original, and thus, new windows and doors had to be fitted with extension jams to match this new wall width.

This repair procedure turned out to be very cost effective, considering the alternatives. It not only restored the walls but also retained the interior appearance of the aged logs. Also, it resulted in a house envelope that is now superior to the old log wall with its increased tightness, to control air infiltration, and its enhanced insulation properties.

What Can We Learn?

There are log cabins in existence that are centuries old and are still standing in good condition. The secret to their longevity lies not only in the species of the wood used but also in the detailing of the mating of the logs. When water is trapped between wood surfaces decay initiates. Thus, a good detail that assures drainage of the joint is the first step to the longevity of the structure. The details used in this log cabin were derived from the desire to achieve a tight joint that controlled air infiltration into the building. Thus, the builder used a spline and groove design. This design in itself would not have caused a problem, but in combination with a flat non-draining exterior top surface of the cut log, rain water running down the facade can find its way into the groove and get trapped. Moist wood, when not vented, causes dry rot.

Very rarely do dry rot problems occur on log walls where the outside mating surface of the log is slanted downward. Round logs, stacked one above the other, provide good drainage on the exterior curved surface, a feature of the centuries- old cabins that still stand today.

Another caution is in the use of sealants. No sealant is 100% waterproof over the long run. Sealants deteriorate over time and, when they fail, they can do more harm than good by trapping water. A sealant joint should always be configured so that it can drain once the sealant fails. Proper drainage is something to keep in mind when using sealants not only on log cabin walls but in general construction.▪

Figure 6. Staging of repair in progress.

Figure 7. Log cabin after wall repair.

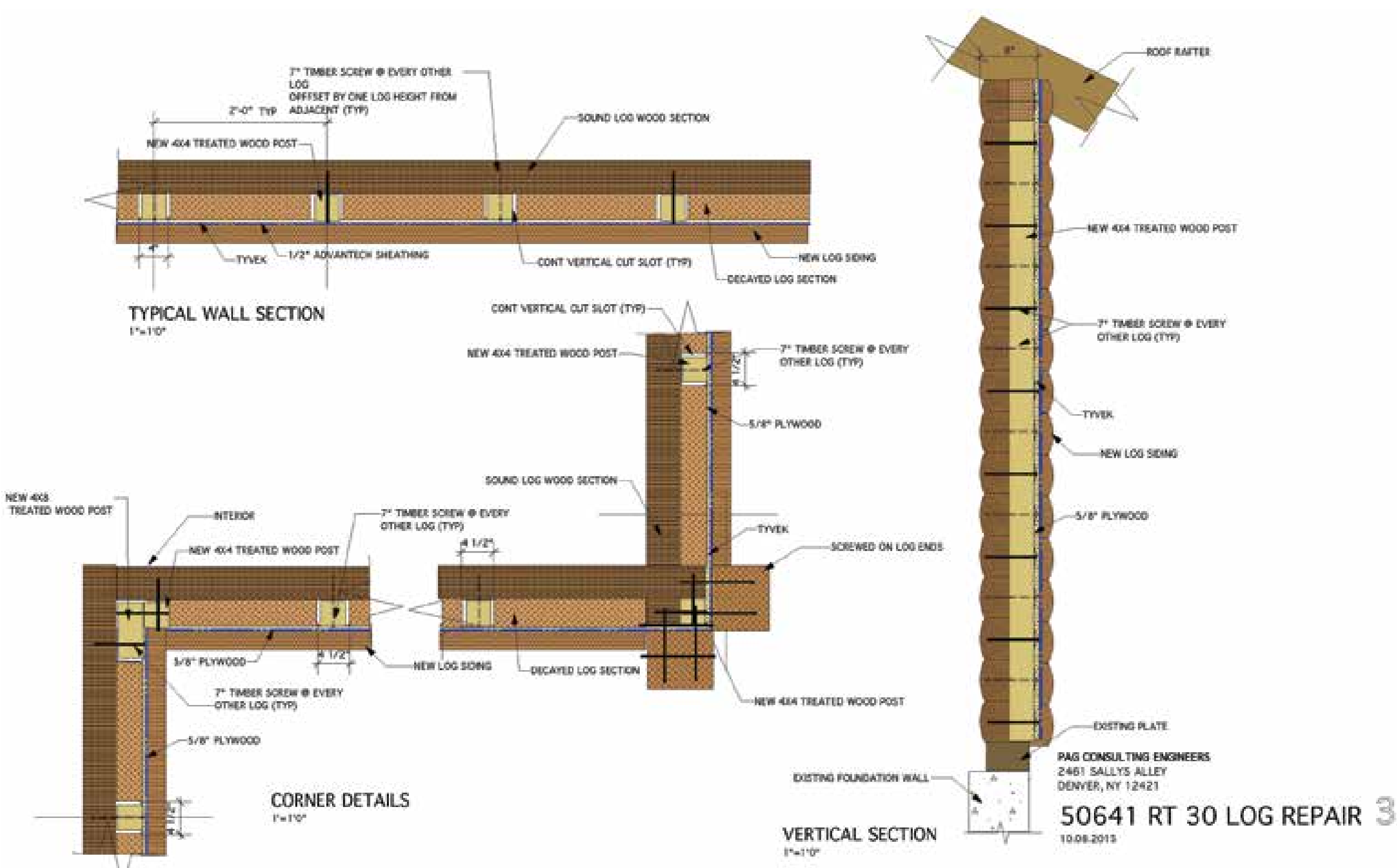

Figure 5. Contract drawing for repair.


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Life-Cycle Performance of Civil Structure and Infrastructure Systems

An International Workshop Organized by ASCE

By Fabio Biondini, Ph.D., P.E., M.ASCE and Dan M. Frangopol, Ph.D., P.E., Dist.M.ASCE

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Structural engineering is undergoing a profound change towards a life-cycle-oriented design philosophy to fulfill the continuously increasing demand from societal, political, economic and environmental needs. In this approach, the classical point-in-time design criteria are extended to account for more comprehensive time-variant performance indicators over the entire service life. Considering this need, the American Society of Civil Engineers (ASCE) proposed the use of Life-Cycle Cost Analysis in conjunction with the Grand Challenge of reducing life-cycle costs of civil infrastructure projects by 50% by 2025.

The recent advances accomplished in the fields of modeling, analysis, design, maintenance and rehabilitation of deteriorating civil structure and infrastructure systems are hence perceived to be at the heart of a modern approach to structural engineering. These advances are of crucial importance to establish guiding policies and support decision-making processes for reliable design of durable structures and rational planning of maintenance, repair, or replacement of existing structures. Furthermore, the availability of quantitative life-cycle performance metrics provides for effectively incorporating emerging issues in structural design, such as the effects of global warming and climate change. Societal issues in adopting life-cycle concepts in the decision-making process may also play a major role within the political system to comply with the different methods, metrics, needs, and priorities addressed by public officials, civil infrastructure users, and owners.

Despite significant advances and accomplishments, life-cycle concepts are not yet explicitly dealt with in structural design codes, and the checking of system performance requirements is referred to the initial time of construction when the system is intact. In this approach, design for durability with respect to chemical-physical damage phenomena is based on simplified criteria associated with classes of environmental conditions. As an example, for concrete structures, such criteria introduce threshold values for concrete cover, water-cement ratio, and amount and type of cement to limit the effects of local damage due to carbonation of concrete and corrosion of reinforcement. However, a durable design cannot be based only on such indirect evaluations of the effects of structural damage, but also needs to take into account the global effects of the local damage phenomena on the overall performance of the structure. These considerations indicate that there is still a strong need to promote further research in the field of life-cycle performance of structural systems under uncertainty, and to fill the gap between theory and practice by incorporating life-cycle concepts in structural design codes.

The research and applications in the field of life-cycle assessment, prediction, and optimal management of structures and infrastructure systems under uncertainty are promoted within SEI/ASCE by the Technical Council (TC) on Life-Cycle Performance, Safety, Reliability, and Risk of Structural Systems (authorized November 7, 2008, and chaired by the second author). The Technical Council and its three Task Groups provide a forum for reviewing, developing, and promoting the principles and methods of life-cycle performance, safety, reliability, and risk of structural systems in the analysis, design, construction, assessment, inspection, maintenance, operation, monitoring, repair, rehabilitation, and optimal management of civil infrastructure systems under uncertainty. In particular, the purpose of Task Group 1 (TG1) on Life-Cycle Performance of Structural

Group picture of participants (ASCE Headquarters, 10th November 2015). In alphabetical order: Mitsuyoshi Akiyama, Japan; Alfredo Ang, USA; Fabio Biondini, Italy; Paolo Bocchini, USA; Sofia Diniz, Brazil; Bruce Ellingwood, USA; Dan Frangopol, USA; Michel Ghosn, USA; Emad Iskander, USA; Zoubir Lounis, Canada; Adam Matteo, USA; Ehsan Minaie, USA; Terry Neimeyer, USA; Kostas Papakonstantinou, USA; Pariya Pourazarm, USA; Arturo Rodriguez Tsouroukdissian, USA; Samantha Sabatino, USA; Mauricio Sánchez-Silva, Colombia; Mark Sarkisian, USA; Behrouz Shafei, USA; Sarbjeet Singh, USA; Ian Smith, Switzerland; Iris Tien, USA; Lucia Tirca, Canada; Andrea Titi, Italy; George Tsiatas, USA; Naiyu Wang, USA; Arnold Yuan, Canada; Wei Zhang, USA; Robert Zobel, USA.

Systems under Uncertainty is to promote the study, research, and application of scientific principles of safety and reliability in the assessment, prediction, and optimal management of life-cycle performance of structural systems under uncertainty.

The ongoing activities of SEI/ASCE TC TG1 include a Special Project approved by the SEI Technical Activities Division Executive Committee for the development of a state of the art report outlining the current status and research needs in the fields of life-cycle of civil structure and infrastructure systems. The task of the Special Project was to conduct a survey and organize an International Workshop on Life-Cycle Performance of Civil Structure and Infrastructure Systems. The objectives were to overview the advances accomplished in the field of life-cycle civil engineering, to promote a better understanding of life-cycle concepts in the structural engineering community, and to discuss methodologies and tools to incorporate life-cycle concepts into structural design codes and standards.

This International Workshop, organized and chaired by the writers of this article, was held on November 10th, 2015, at the ASCE Headquarters in Reston, VA, USA. The Workshop program included invited plenary lectures addressing the current state of research and practice, as well as breakout working sessions and group reports. Over 30 invited participants from several countries attended the Workshop (see *Figure*). The workshop was very successful in assembling information on the development and implementation of criteria, methods and tools for life-cycle design and assessment of civil structure and infrastructure systems.

The final results of both the survey and workshop complement information on the state-of-research and -practice. In particular, there is an awareness that a robust prediction of the time-variant structural performance must rely on a reliable and efficient probabilistic deterioration modeling of materials and structural components. Advanced models are well established for some of the most detrimental damage processes, such as corrosion and fatigue, and are rapidly becoming available for a wider range of deterioration mechanisms. However, deterioration models are very sensitive to the change of the probabilistic parameters of the input random variables, and their robust validation and accurate calibration are difficult tasks to be performed due to the limited availability of data. Further efforts in this direction, aimed at gathering new data from both existing structures and experimental tests, are crucial for a successful implementation in practice of life-cycle methods. In this context, inspection and monitoring activities could provide a powerful aid to reduce the level of epistemic uncertainty and to improve the accuracy of predictive probabilistic models.

Civil infrastructure systems are the backbone of modern society and among the major drivers of the economic growth and sustainable development of countries. It is hence a strategic priority to consolidate and enhance criteria, methods, and procedures to protect, maintain, and improve the safety, durability, efficiency, and resilience of critical structure and infrastructure systems under uncertainty. We sincerely hope that the effort ongoing within the SEI/ASCE TC and TG1 contributes to promoting the application of life-cycle concepts in design practice, influence the development of structural design codes and standards, and enhance the state of the civil structures and infrastructures to protect the public safety and improve the quality of life.▪

The Future of Bridge Design

By Barbara H. Day, P.E.

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Design, modeling, and analysis are keys to the success of today's bridge projects. The next generation of bridge modeling software requires that the model is *purpose-built* for bridge designers and contractors who need to create, construct, maintain, and document a wide variety of bridge information throughout the lifecycle of the bridge. Sharing information in an information-rich 3D model increases data quality, collaboration, constructability, and operational aspects including asset management. Reduction in the project's overall costs for the entire system are important for all stakeholders, and the availability of intelligent 3D models are an essential component in providing accelerated project delivery and information mobility.

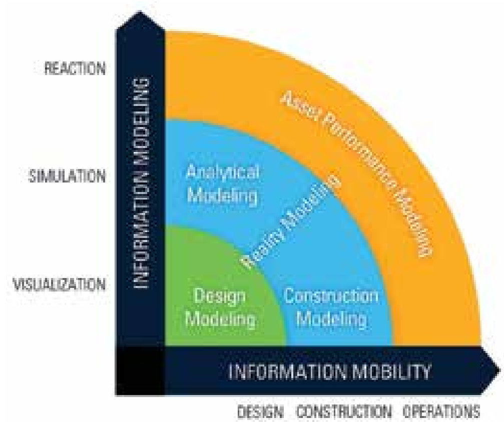


Figure 1. The availability of intelligent 3D models is a key component in providing accelerated project delivery and information mobility.

Information Modeling

In just a matter of a few years, the bridge industry has shifted focus to hone the definition of intelligent 3-D models. What does it mean to provide an intelligent and data-rich 3D model that connects design, construction, inspection, operations, and maintenance? How much is too much information and what is our goal? Can we shorten the construction schedule? Are we designing for construction and can we meet the expectations of the traveling public during construction with the proposed design? A true data model addresses these questions and increases the integrity of our engineering profession. For example, the owner/designer and contractor could be involved in the development of a model in the earliest stages, and they can address constructability issues before construction begins to ensure that the project stays on course, on schedule, and on budget. Evaluating construction phasing ahead of time eliminates conflicts during construction. An intelligent, data-rich model is more than a visualization of a conceptual design. It provides valuable insight into the future of a project, making the outcome more predictable. Vision meets reality with the creation of an information-rich data model that can be used throughout the life of the bridge.

Build It Better Trends – Driven to Innovate

Engineers, detailers, and contractors are under increased pressure to find the solution that promotes better designs resulting in smarter and more reliable construction methods. A disconnect between project and stakeholders is no longer acceptable in our industry and is driving the need for data interoperability throughout the lifecycle of the bridge.

Pretty picture models do not lead to reality when constructability is at stake. The days of bidding on infrastructure projects have evolved into the owner-operator setting higher expectations for minimizing allowances, omissions, and errors as added risks on the project.

This has led to a situation where the market demands better methods for reducing construction costs and minimizing economic impacts. The MAP-21 compliance requirements, Every Day Counts (EDC) legislation, and the growing popularity of design-build and Public Private Partnerships (P3) has set the stage for bridge project delivery expectations.

These initiatives on design/construct/rehabilitate contracts are driving the owner demand for faster, more cost-effective and constructible means of building our transportation assets.

Traditional Bridge Design Process

Unfortunately, the bridge design and construction workflow is often a fragmented and linear process with very little automation or exchange of data in a useful or integrated manner. The ability to reuse data across disciplines is challenging and creates an environment prone to data re-input among multiple programs and spreadsheets as a byproduct – resulting in the introduction of error-prone results.

These processes traditionally involve centralized automation (roadway does roadway, bridge does bridge, inspectors do inspection) and there is a minimal exchange of critical project and engineering data between the primary disciplines.

Imagine a typical scenario of designing a facility over another new or existing facility and all of the conflicts and challenges this presents. The instantaneous access and ability to tweak pier placement as you evaluate existing conditions is critical in these situations and common in design-build projects. Typically, geometric information transferred from the roadway design team is an inefficient manual and repetitive data entry process. There is no time to waste when alignments require shifting to keep a project moving. With errors, re-dos, model translations, multiple spreadsheets, and shortened delivery timeframes, communication often breaks down while working under tight deadlines. This causes data to be dropped and, in most cases, it is not reused.

Dealing with broken lines of communication among teams and error-prone workflows, or just dealing with multiple software products from different vendors, can exacerbate the disjointed design process. Lastly, but certainly one of the more important in traditional project delivery, is plan production. Often, this is not an automated process or at least not one that is efficient or automated in a manner to which stakeholders are accustom.

There are tremendous advantages in connecting project team members with a 3D approach and 3D technology.

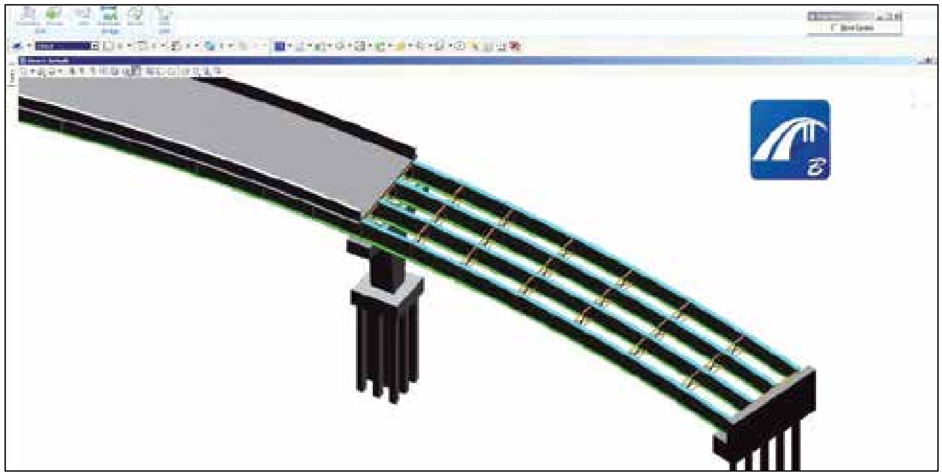


Figure 2. OpenBridge Modeler is purpose-built for bridge designers and contractors who need to create, visualize, analyze, construct, maintain, and document a wide variety of bridge information throughout the lifecycle of the asset.

An Intelligent Data Process

The benefits of having a geometry that is relevant and current will tie roadway and bridge engineers together from the onset of the project, and throughout design revisions, in a bi-directional manner. Not only are they working in a connected manner, but they are also working geospatially for improved accuracy.

Bridges can be developed and modeled in a real-world environment. Referencing existing conditions becomes easy and meaningful. Models become the immediate mechanism for design and analytics. Imagine the time and cost savings of efficiently developing an intelligent model in the preliminary stages of a project – and carrying this through to design and analysis without requiring the time or expense of re-engineering. Currently, most modeling technology does not allow for a direct link to analytics without some re-entry of data; nor do these models contain the level of detail required for today's projects.

The ability to link the physical model directly to the analytics allows for alternate design options to be realized initially, in the office, as you are saving time and effort by reviewing alternatives, constructability issues, and conflicts in the earliest development of the bridge. Much of the design is for construction of course, but at what point do we begin to insert intelligence into the design so that we can predict and plan for construction? When does the design take into account construction steps? Today, that is usually a post-design process, where the design is passed to a construction engineering team, who dissects the design and reassembles it into construction plans.

With the massive amount of intelligence available between disciplines, it is only logical to collaborate and leverage it in a seamless workflow. Tools are available to make intelligent bridge design and analysis a reality. These tools will speed construction with little effort on the front end.

An efficient bridge design process allows you to directly connect and reference existing and proposed conditions, as well as civil data to perform constructability analysis – key to the maintenance of traffic – and for the facility over facility situations. By allowing the designer to visualize, render, perform clash detection, generate quantities, and evaluate clearances with the information-rich model, you can be assured of reliable construction methods from the onset.

Bridge modeling technology should provide engineers with the ability to create a workflow that promotes accurate information modeling and mobility. There needs to be an interoperability component that allows all project disciplines to evaluate and share critical data from the planning/bidding phases all the way through to commissioning, operations, and maintenance. The real value of a model doesn't solely reside in its aesthetic appeal, but also in the usability and life of the data associated with it.

OpenBridge Modeler by Bentley is one software package that addresses the challenges that design teams and contractors face with complex geometry needs, parametric updating of changes, and evaluating constructability early in the process, including conflicts not seen in a 2D workflow. Software like OpenBridge Modeler enables the user to work on a bridge project, share engineering-rich data, and make more informed decisions within a 3D model, thus accelerating construction.

All disciplines (roadway, utilities, bridges, existing conditions, and so on) operating in a single modeling environment with no re-creation of critical project data is vital in meeting the challenges of a 3D deliverable by industry standards. Such models facilitate collaboration and integration with other disciplines to ensure everyone has the data they need when they need it.

3D bridge models provide the ability to reference related designs that connect or affect the project. Subsurface utilities, rebar detailing, bridge element placement, and traffic maintenance are all key construction issues that, in an integrated and interoperable workflow, can be detected and resolved in the office rather than in the field. This ability enables you to meet the owner's expectations for minimizing omissions and errors.

Summary

With a focus on operational excellence, sustainability, and the economic impacts of a bridge not being available, it has never been more important to evaluate our processes. Bridge design and construction processes are evolving, and 3D deliverables are imminent.

Interoperability and collaboration are keys to the success of bridge projects of all sizes and construction methods. Leveraging complex geometry from the beginning to generate physical bridge models, and preparing the design and analytical requirements, is essential to moving to a more fluid and seamless reality modeling workflow. With intelligent as-designed models and as-built data, engineers can provide operations and maintenance value for the entire life of the bridge.▪

The Lake Wobegon Effect and Structural Licensure

By Timothy M. Gilbert, P.E. S.E., SECB

Lake Wobegon is a fictional Minnesota Town from *A Prairie Home Companion*, a popular public radio show by Garrison Keillor. One of the show's famous lines about the town is:

> *"...the little town that time forgot and the decades cannot improve ... where all the women are strong, all the men are good-looking, and all the children are above average."*

Part of the humor is the plainly impossible circumstance that every member of each group excels. The very definition of an average requires that some members of a population are above the mean, and some are below. Unfortunately, people are not good at making an accurate self-assessment to determine where their performance ranks in relation to the group as a whole.

In 1999, David Dunning and Justin Kruger of Cornell University examined our abilities to judge our proficiency at certain skills. Their finding that the least capable were generally the most likely to significantly overestimate their proficiency became known as the Dunning–Kruger effect. More recently, Professor D. G. Myers termed this type of illusory superiority the Lake Wobegon effect. Last year, Prof. Myers showed that more than three-quarters of those polled thought themselves to be safer than the average driver, and nearly two-thirds said they are better than average at parallel parking.

The research indicates that we fail at making accurate self-assessments. And while we can find humor in an overestimation of our parking skills, when it comes to driver safety, there's a little more at stake. Is there any amount of added caution that the truly above-average drivers can take to mitigate the risks posed by those who falsely think they also fit into that category? I doubt the risk can be completely mitigated when it comes to driving cars; however, those who drive others or operate large vehicles can be required to pass tests indicating a higher level of mastery of critical driving skills.

Based on the research, is it irrational to contend that engineers may be affected by the Lake Wobegon effect? Does our engineering education and training make us immune to tendencies of the human brain toward cognitive bias?

It is rational to conclude that some engineers overestimate their ability and take on projects beyond their capability. A system of checks and balances can stop the abuse of the professional seal – sometimes. However, that system is not foolproof.

The potential risk to the public as a result of an engineer's inability to adequately recognize their limits is a strong reason for advocating structural licensure. Just as we require those who chauffeur passengers or transport heavy loads to undergo more rigorous certification than most drivers, we should also expect engineers who design significant structures to demonstrate a higher degree of proficiency. This is a natural conclusion from the growing body of evidence that shows we can and do misjudge our capabilities.

If you are not swayed yet, consider the same logical stance applied to driving. Would it be rational to let someone who just passed a driving test to decide what limits should be placed on their driving? Let's also mix in financial incentives in this hypothetical situation and assume their financial well being and the well being of their family depend on being able to drive. Under those conditions, how well will that person assess their own skills when offered a chance to earn a good income for driving a truck that looks to be just a little more vehicle than they are accustomed to driving? This scenario parallels circumstances faced by engineers regularly.

A recently registered PE may be confronted with some very weighty choices. Take Pat, a hypothetical structural engineer who recently became registered by passing the civil-structural PE exam. Pat works for a small consulting firm and is their only registered engineer with structural experience. Pat's manager, the owner, has a great opportunity for the firm. The project involves the design of a five-story healthcare facility that will also serve as an emergency shelter. Working on this project could mean significant growth opportunities as well as better financial stability for Pat. While this design is greater in magnitude than any of Pat's prior efforts, Pat is familiar with its elements: foundations, concrete design, and steel design.

During construction, everything appeared to be in order and progressed according to schedule. Unfortunately, there is an undiscovered design flaw – the anchorage details for the shear walls to the foundation are not adequate. The inadequacy is not so egregious as to cause failure during construction, but the anchorage might fail in a design level storm or seismic event. Sadly, nobody involved in the project is aware of this flaw, and all involved view the project as a success. The error stemmed partly from Pat's overconfidence, and partly from pressure, possibly self-induced, to help the firm obtain a significant project.

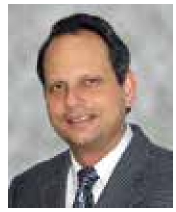
Confirmation bias is a further complication that arises from this situation. The apparent success of the healthcare facility project is likely to give Pat even more confidence to tackle a similar project, and very possibly repeat the same error. Like Bill Gates said, "Success is a lousy teacher. It seduces smart people into thinking they can't lose."

Circumstances like Pat's are one reason structural engineers should support structural licensure. Having a process in place to assure that structural engineers, for significant structures, are vetted by testing and experience helps to guard against human fallibility. Admittedly, the dilemmas in these situations are hypothetical and written to demonstrate the advantages of structural licensure. But they are entirely within the realm of reasonable possibility.▪

Timothy M. Gilbert is a Project Specialist for TimkenSteel in Canton, OH. He is also the current President for SEAoO and chairs its Structural Licensure Committee. He may be reached at ***tgilbert.pe@gmail.com***.

This article was initially published in the April 2016 SEAoO Newsletter (**www.seaoo.org**). It is reprinted with permission.

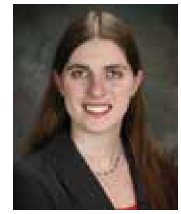
Brian Leshko Steps Down from the STRUCTURE Editorial Board



Brian J. Leshko, P.E., F.SEI, F.ASCE, is stepping down as a member of the STRUCTURE® magazine Editorial Board. Brian joined the Board in 2005 as an SEI representative. He currently serves as Vice President, Principal Professional Associate and HDR's National Bridges & Structures Inspection Program Leader based in Pittsburgh, Pennsylvania. Brian has held numerous leadership roles with ASCE/SEI, including 2013 Structures Congress co-chairman; treasurer for the National SEI Board of Governors; Pittsburgh Section of ASCE Board of Directors; and current chair of the SEI Bridge Inspection, Management, and Rehabilitation Committee.

Barry Arnold, P.E., S.E., SECB, Chair of the STRUCTURE magazine Editorial Board, had this to say about Brian's departure: "Brian has served diligently and faithfully on the Editorial Board for 11 years. His continuous support for the magazine and particular interest in developing articles related to bridges contributed to its success and relevance. His professionalism and commitment to the profession and STRUCTURE magazine are commendable. He will be missed."

Regarding his tenure on the Board, Brian commented, "I have thoroughly enjoyed my tenure on the Editorial Board, serving as the lead reviewer for bridge-related articles and championing the annual bridge-themed October issue. During the previous 11 years, I have been fortunate to work with the most professional structural engineers, editors and publishers engaged in producing the seminal magazine for structural engineers. I look forward to reading future issues, as I know the magazine is definitely in good hands."



Linda M. Kaplan, P.E., will replace Mr. Leshko as an SEI representative. Ms. Kaplan is a Senior Structural Engineer and Project Manager with TRC in Pittsburgh, PA. Her professional experience includes both steel and concrete bridge design for highway, rail, and pedestrian structures, with expertise in 3D Finite Element Modeling and full structure vibration analysis. She is the current chair of the SEI Young Professionals Committee, a member of the SEI Steel Bridge Committee and Aesthetics in Design Committee, and a member of the SEI Task Force for Digital Presence. Linda is a co-author of the recently published book *Bridges... Pittsburgh at the Point… a Journey Through History*.

Barry Arnold said this about Linda Kaplan's appointment to the Editorial Board: "It is a great pleasure to welcome Linda to the Editorial Board. She has extensive experience in writing and editing and is passionate about supporting and improving the profession. Linda came highly recommended by her colleagues and will be a great addition to the STRUCTURE magazine team."

Please join STRUCTURE magazine in congratulating Brian Leshko on his many years of service to the magazine and welcoming Linda Kaplan to the team.

The Logic of Ingenuity

Part 2: Engineering Analysis

By Jon A. Schmidt, P.E., SECB

The Logic of Ingenuity

The process of (abductively) creating a diagrammatic representation of a problem and its proposed solution, and then (deductively) working out the necessary consequences, such that this serves as an adequate substitute for (inductively) evaluating the actual situation.

Charles Sanders Peirce wrote many thousands of pages during his lifetime on a wide variety of topics, but evidently had little to say about engineering. One place where he did address it was in an 1898 article, "The Logic of Mathematics in Relation to Education." It appeared initially in a journal called *Educational Review*, and subsequently as CP 3.553-562 and PMSW 15-21.

Peirce briefly discussed and rejected several characterizations of mathematics throughout history; most notably, the still-common misunderstanding of it as merely "the science of quantity." He then favorably quoted the definition advocated by his father Benjamin, one of the foremost 19th century practitioners in that field: "the science which draws necessary conclusions." He also cited the ninth edition of the *Encyclopaedia Brittanica* to support his contention that "it is only about hypotheses that necessary reasoning has any application," where a hypothesis is "a proposition imagined to be strictly true of an ideal state of things."

Next came the key passage (CP 3.559), which I will quote throughout the rest of this article. Peirce sought to describe what a mathematician *does*, rather than what mathematics *is* or what sort of objects it studies:

> *A simple way of arriving at a true conception of the mathematician's business is to consider what service it is which he is called in to render in the course of any scientific or other inquiry. Mathematics has always been more or less a trade. An engineer … finds it suits his purpose to ascertain what the necessary consequences of possible facts would be; but the facts are so complicated that he cannot deal with them in his usual way. He calls upon a mathematician and states the question.*

In Peirce's day, this is what literally occurred on many occasions – engineers would retain mathematicians to perform a lot of their calculations. It is worth noting that Peirce's rare mention of engineering here may not be coincidental. At about the same time, he was providing precisely this type of assistance to George S. Morison in support of the latter's preliminary design for a span over the Hudson River, near the eventual site of Othmar Amman's George Washington Bridge. Portions of the resulting report survive in Peirce's manuscripts that Richard S. Robin numbered 1357-1360 in his 1967 catalog.

> *Now the mathematician does not conceive it to be any part of his duty to verify the facts stated. He accepts them absolutely without question. He does not in the least care whether they are correct or not.*

Today the engineer normally serves as the mathematician, as well – typically aided by a computer, which likewise "does not in the least care whether [the facts] are correct or not." A machine is perfectly capable of drawing necessary conclusions by executing a deterministic algorithm, but it is up to the engineer to formulate the initial hypothesis – i.e., the *model* – in a way that adequately *represents* the circumstances of interest.

> *He finds, however, in almost every case that the statement has one inconvenience, and in many cases that it has a second. The first inconvenience is that, though the statement may not at first sound very complicated, yet, when it is accurately analyzed, it is found to imply so intricate a condition of things that it far surpasses the power of the mathematician to say with exactitude what its consequences would be. At the same time, it frequently happens that the facts, as stated, are insufficient to answer the question that is put.*

In other words, it is rarely feasible to incorporate *all* aspects of the "condition of things" into an engineering model; and a *complex* system is one for which it is not even feasible to incorporate all of the *relevant* aspects. Also, there are inevitable uncertainties that require the engineer to make various assumptions. The upshot is that, despite being the creator of the model and presumably familiar with it in all of its details, the engineer will probably not be able to anticipate all of its results in advance.

> *Accordingly, the first business of the mathematician, often a most difficult task, is to frame another simpler but quite fictitious problem (supplemented, perhaps, by some supposition), which shall be within his powers, while at the same time it is sufficiently like the problem set before him to answer, well or ill, as a substitute for it.*

Here Peirce calls attention to something that engineers would do well to keep in mind: We routinely develop a viable solution to a real problem by solving a "quite fictitious" one in its place. Indeed, even the most fundamental phenomena of engineering science – for structural engineers, concepts like force, moment, shear, and stress – do not strictly *exist*, except as convenient tools for mental and mathematical manipulation of idealized scenarios.

> *This substituted problem differs also from that which was first set before the mathematician in another respect: namely, that it is highly abstract. All features that have no bearing upon the relations of the premises to the conclusion are effaced and obliterated. The skeletonization or diagrammatization of the problem serves more purposes than one; but its principal purpose is to strip the significant relations of all disguise. Only one kind of concrete clothing is permitted – namely, such as, whether from habit or from the constitution of the mind, has become so familiar that it decidedly aids in tracing the consequences of the hypothesis.*

This is where judgment comes into play. When translating an artifact into an abstract representation thereof, it is up to the engineer to ascertain which features "have no bearing" and which relations are "significant" enough to warrant making them explicit. The only people who can do this successfully are those who have cultivated the appropriate instincts and sentiments – habits of feeling, action, and thought – by virtue of gaining the requisite experience.

> *Thus, the mathematician does two very different things: namely, he first frames a pure hypothesis stripped of all features which do not concern the drawing of consequences from it, and this he does without inquiring or caring whether it agrees with the actual facts or not; and, secondly, he proceeds to draw necessary consequences from that hypothesis.*

This is engineering analysis in a nutshell; and in my next installment, I will further explore the nature of the reasoning that is involved.■

*Jon A. Schmidt (**jschmid@burnsmcd.com**) is an associate structural engineer in the Aviation & Federal Group at Burns & McDonnell in Kansas City, Missouri. He serves as Secretary on the NCSEA Board of Directors, chairs the SEI Engineering Philosophy Committee, and shares occasional thoughts at **twitter.com/JonAlanSchmidt**.*

The online version of this article contains detailed references. Part 1 of this series appeared in the September 2016 issue.

American Wood Council

Phone: 202-463-2766
Email: **info@awc.org**
Web: **www.awc.org**
Product: 2015 Special Design Provisions for Wind and Seismic
Description: Criteria for proportioning, designing, and detailing engineered wood systems, members, and connections in lateral force resisting systems. Design of wood structures to resist wind or seismic forces is by allowable stress design (ASD) or load and resistance factor design (LRFD). Nominal shear capacities of diaphragms and shear walls are provided.

Applied Science International, LLC

Phone: 919-645-4090
Email: **support@appliedscienceint.com**
Web: **www.appliedscienceint.com**
Product: Extreme Loading for Structures 4.0 (ELS)
Description: An advanced non-linear structural analysis software tool designed specifically for structural engineers; allows structural engineers to study the 3D behavior of structures through both the continuum and discrete stages of loading. Includes static and dynamic loads such as those generated by blast, seismic events, impact, progressive collapse, and wind.

Product: SteelSmart System 7.3 (SSS)
Description: For light steel framing analysis and design; integrates new and additional functionality and accessibility. The Lateral Load Generator/Distributor tool uses dimensions and load specification for a building and calculates the total lateral wind and seismic loads according to ASCE 7 Standard *Minimum Design Loads for Buildings and Other Structures*.

Bentley Systems, Inc.

Phone: 800-BENTLEY
Email: **Samantha.Langdeau@bentley.com**
Web: **www.bentley.com**
Product: RAM Structural System
Description: Tackle projects with confidence and produce high quality and economical designs, using various concrete, steel and joist building materials. Quickly design, analyze and create documentation for your building projects, saving time and money. Design anything from individual components to large scale building and foundations.

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CADRE Analytic

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Web: **www.cadreanalytic.com**
Product: CADRE Pro
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Concrete Masonry Association of California and Nevada

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Web: **www.cmacn.org**
Product: CMD12 Design Tool for Masonry
Description: Structural design of reinforced concrete and clay hollow unit masonry elements for design of masonry elements in accordance with provisions of Ch. 21-1997 UBC, 2001 through 2013 CBC or 2003 through 2012 IBC and 1999 through 2011 Bldg. Code Requirements for Masonry Structures (TMS 402/ACI 530/ASCE 5).

Product: 2012 Design of Reinforced Masonry Structures, 7th Edition
Description: Useful text in classroom, or as a reference for practicing engineers. Based on 2011 "Building Code Requirements for Masonry Structures" (TMS 402-11/ACI 530-11/ASCE 5-11) as developed by MSJC and the 2012 IBC. Design load calculations referenced to ASCE 7-10.

DEICON

Phone: 937-901-6449
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Product: Tuned Mass Dampers
Description: DEICON designs, fabricates, and implements passive, semi-active, and active tuned mass dampers for a variety of structures including floor systems, pedestrian bridges, tall buildings, observation towers, and piping systems. Tuned mass dampers need to be connected to the vibrating structure at one end only.

Dlubal Software, Inc.

Phone: 267-702-2815
Email: **info-us@dlubal.com**
Web: **www.dlubal.com**
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Description: Automatic wind and snow load generators according to ASCE 7 are included in the non-linear FEA program, RFEM. The dynamic add-on modules allow for a response spectra analysis according to USA/CAN/International Standards or create a user-defined response spectra generated from accelerogram input data.

Hardy Frames/Mitek Builder Products

Phone: 800-754-3030
Email: **dlopp@mii.com**
Web: **www.hardyframe.com**
Product: Hardy Frame Shear Wall System
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Web: **www.iesweb.com**
Product: VisualAnalysis
Description: Simulate the sway and shake, as easy models you will make, reduce your stress, increase your load, and satisfy the building code.

JFE Techno Manila, Inc.

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Web: **www.jfetechnomanila.com**
Product: Engineering Design Services
Description: Engineering Design Services for JFE Engineering Corporation.

The Masonry Society

Phone: 303-939-9700
Email: **info@masonrysociety.org**
Web: **www.masonrysociety.org**
Product: Masonry Codes and Design Guides
Description: The Masonry Society is a non-profit, professional organization of volunteer Members, dedicated to the advancement of masonry knowledge. Through our Members, all aspects of masonry are discussed. The results are disseminated to provide guidance to the masonry and technical community on various aspects of masonry design, construction, evaluation, and repair.

Pile Dynamics, Inc.

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Email: **info@pile.com**
Web: **www.pile.com/pdi**
Product: Pile Driving Analyzer (PDA)
Description: A high strain dynamic load testing and pile riving monitoring system for most types of deep foundations. The PDA calculates bearing capacity and assesses structural integrity, driving stresses and hammer performance.

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Phone: 800-925-5099
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Web: **www.strongtie.com**
Product: Strong-Rod™ Systems
Description: Simpson Strong-Tie introduces the Strong-Rod continuous rod tiedown system for light-frame, multi-story wood construction. The Strong-Rod Anchor Tiedown System for shearwall overturning restraint and Strong-Rod Uplift Restraint System for roofs address many of the design challenges specifically associated with multi-story buildings that must withstand seismic activity or wind events.

Product: Strong Frame® Special Moment Frame
Description: Features Yield-Link™ structural fuses that eliminate lateral-beam bracing and are replaceable after a seismic event, making it easier to specify and saving building owners significant cost. There is no welding, only bolted connections, and it is designed for wood and steel construction.

Standards Design Group, Inc.

Phone: 800-366-5585
Email: **info@standardsdesgin.com**
Web: **www.standardsdesign.com**
Product: Wind Loads on Structures 4
Description: Performs computations in ASCE 7-10, Chapters 26-31 and ASCE 7-98, 02 or 05, Section 6; computes wind loads by analytical method rather than the simplified method, provides basic wind speeds from a built-in version of the wind speed, allows the user to enter wind speed. Numerous specialty calculators.

StructurePoint

Phone: 847-966-4357
Email: **info@structurepoint.org**
Web: **www.StructurePoint.org**
Product: Reinforced Concrete Design Software
Description: spColumn is widely used for design of shear walls, bridge piers as well as typical framing elements in buildings and structures. spWall is a program for design and analysis of cast-in-place reinforced concrete walls, deep beams, coupling beams, tilt-up walls, ICF walls, and precast architectural and load-bearing panels.

Struware, LLC

Phone: 904-302-6724
Email: **email@struware.com**
Web: **www.struware.com**
Product: Struware Code Search
Description: Provides all pertinent wind, seismic, snow, live and dead loads for your building in just minutes. The program simplifies ASCE 7 & IBC (and codes based on these) by catching the buts, ifs, insteads, footnotes and hidden items that most people miss. Demo available at the website.

Trimble Solutions USA, Inc.

Phone: 770-426-5105
Email: **kristine.plemmons@trimble.com**
Web: **www.tekla.com**
Product: Tekla Structural Designer
Description: Built-in loading wizards will automatically calculate all wind and seismic forces, generate design cases and optimize the design of steel and concrete members to the latest AISC, ACI and ASCE 7 design codes. Review detailed calculations with code clauses and print complete reports for review submittals.

Product: Tedds
Description: Built-in library of calculations allows you to quickly calculate the ASCE 7 wind and seismic forces for your structure. Then use one of the component design modules to design beams, columns and foundations. Link the modules together to create a professional report for review submittals.

WoodWorks® Software

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Web: **www.woodworks.org**
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Description: WoodWorks provides free project support as well as education and resources related to the code-compliant design of non-residential and multi-family wood buildings – including wind/seismic design. WoodWorks field teams have expertise in a wide range of building types, from schools and mid-rise/multi-family, to commercial, office, retail, public, institutional and more.

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South Park Bascule Bridge Replacement

By Tim Lane

HNTB Corporation was an Outstanding Award Winner for its South Park Bascule Bridge Replacement in the 2015 NCSEA Annual Excellence in Structural Engineering Awards Program in the Category – New Bridges and Transportation Structures.

When HNTB designed the new first-of-its-kind South Park Bridge in Seattle, the goal was to make the structure something that would provide excellent service to area residents and make them proud.

HNTB was awarded the final design contract in 2008. Final funding was secured in 2010 and construction got underway in 2011. Completed and opened to traffic on June 30, 2014, it is the first bascule bridge designed to meet strict post-seismic operational requirements in an AASHTO Seismic Zone 4 with 70- to 105-foot-deep soft soils.

Among numerous project challenges, the most significant were:

- Designing a movable span in a seismically volatile region.
- Designing for enhanced maintainability and reliability.
- Keeping the community involved during high-impact periods of the project.
- Preserving the historic features of the original bridge.
- Designing and building an environmentally responsible project.

The original South Park Bridge, constructed in 1931, was a double-leaf bascule bridge listed on the National Register of Historic Places and designated a King County Landmark. The 1,045-foot-long bridge spanned the Duwamish Waterway and connected the industrial South Park area and downtown Seattle.

The closure of the old bridge in June 2010 occurred earlier than scheduled because of safety issues. The main piers were settling and tilting in an unpredictable manner due to the original foundation piles not penetrating through a deep layer of liquefiable soil, as well as the concrete structure having been weakened by several earthquakes. In its last years of operation, the bridge required extensive maintenance operations to maintain alignment of the movable spans and machinery.

Designing the new bridge to be fully functional after an Operational Level (108-year return) earthquake and experience only moderate, repairable damage during a Design Level (975-year return) earthquake was unprecedented. HNTB designed several innovative solutions to meet stringent seismic performance requirements, including sunken caisson foundations, isolated trunnion frames, and collapsible center joints on draw spans.

Compared to drilled shaft foundations, the sunken caissons provide enhanced stiffness and resilience, reducing seismically induced displacements.

Each bascule leaf and its machinery are supported on a free-standing steel trunnion frame inside each pier that is designed to respond elastically at the higher level event. During an earthquake, the machinery and bascule leaf move together as one. Relative displacements between components after an earthquake are small, preventing machinery damage.

At mid-span, where the two tips of the movable spans come together, a gap measuring 18 inches just under the expansion joint plates prevents contact between the steel framework of the leaves during the higher-level event. This minimizes the transfer of loads to the trunnion frames in each pier and economizes the design of the trunnion frames and bearings. Above, on the driving surface, large joint plates which are only a few inches apart are minimally secured to the bridge so that they become sacrificial if the leaves come into contact with each other.

The main girders, comprising one of the most prominent and unique elements of the bridge, are believed to be the first known use of a "trussed" web girder. The continuous welded plate construction eliminates gusset plates and thousands of fasteners, and will significantly improve future inspection, bridge maintenance, and safety.

Key cost-effective aspects of the $167 million project include the repurposing of materials and elements of the original bridge and a decorative rain garden that serves as landscape art while collecting. Also, the rain garden naturally treats storm water runoff from the bridge before discharging it into the waterway. The inclusion of a rain garden eliminated the need to install an enormous and expensive underground detention vault.

Besides promoting cost effectiveness, repurposing materials from the old bridge and creating a rain garden enhanced the project from the standpoint of environmental responsibility. The rain garden's park-like setting was also a benefit to the community.

At the same time, incorporating elements of the old bridge preserved historic aspects of that structure that the community was fond of. As one the few working examples of an original Scherzer Rolling Lift Bridge, the old bridge had been listed on the National Register of Historic Places and had received historic landmark designation from the King County Landmarks Commission.

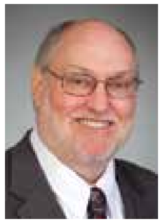
Public outreach was prevalent throughout the entire project with a series of electronic updates, regular public meetings and door-to-door visits to key businesses along the closed corridor. Extensive outreach continued during construction and played a significant part in the acceptance of the project despite four years of disruption.

From the innovation and expertise of HNTB's design at the beginning to the conscientious effort to ensure construction was of the highest quality, this bridge exceeded expectations. Everyone involved put their hearts into this project, and that care is reflected in what stands out there now and for the next 100 years.▪

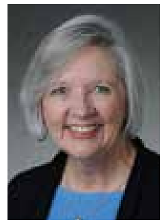
Tim Lane is the Bridge and Tunnel Department Manager in the Seattle office of HNTB Corporation and has been an NBIS certified bridge inspector for over 30 years.

New President, Board Members Take Office

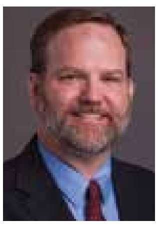
Thomas Grogan, Jr.
The Haskell Company
NCSEA President



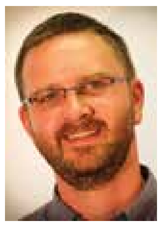
Williston "Bill" Warren IV
SESOL, Inc.
NCSEA Vice President/ President Elect



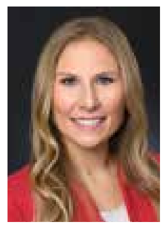
Susan Jorgensen
Studio NYL
NCSEA Treasurer



Jon Schmidt
Burns & McDonnell
NCSEA Secretary



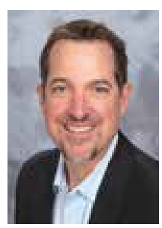
Brian Dekker
Sound Structures
NCSEA Past President



Emily Guglielmo
Martin/Martin, Inc.
NCSEA Director

Jonathan Hernandez
Gilsanz Murray Steficek, LLP
NCSEA Director

Chun Lau
Brown and Caldwell
NCSEA Director



Ed Quesenberry
Equilibrium Engineers, LLC
NCSEA Director

The 2016-17 NCSEA Board of Directors took office at the Structural Engineering Summit last month in Orlando. Leading the Board as President is Thomas Grogan, Jr., P.E., S.E., F.ASCE, Director of Quality Assurance/Chief Structural Engineer, The Haskell Company. He is a member of FSEA, and has served on the Board as Director, Treasurer and Vice President.

Williston "Bill" Warren IV, S.E., SECB, Principal SE, SESOL, Inc., now serves as Vice President/President Elect. He has served on the Board as Director, Secretary and Treasurer, and is a member of SEAOC.

The position of Treasurer has been filled by Susan Jorgensen, P.E., SECB, LEED Quality Control Manager, Studio NYL. She has served on the Board as Director, and is a member of SEAC.

Joining the Board as a Director is Chun Lau, P.E., S.E., P.Eng, Supervising Structural Engineer, Brown and Caldwell. He has served on the Board of the SEAW Seattle Chapter and SEAW State Board.

Ed Quesenberry, S.E., Principal/Owner, Equilibrium Engineers, LLC, also joins the Board as a Director. He was a member of the NCSEA Board from 2013-2015. He is a member of SEAO.

Brian Dekker, P.E., S.E., LEED AP, President, Sound Structures, assumes the role of Past President on the Board. He is a member of SEAOI.

Remaining on the Board are: Secretary Jon Schmidt, P.E., SECB, Associate Structural Engineer, Burns & McDonnell, a member of SEAKM; Director Emily Guglielmo, S.E., P.E., Principal, Martin/Martin, a member of SEAOC; and Director Jonathan Hernandez, P.E., SECB, Partner, Gilsanz Murray Steficek, a member of SEAoNY.

Retiring from the Board of Directors are Past President Barry Arnold, P.E., S.E., SECB, Vice President of ARW Engineers, Ogden, Utah, and a member of SEAU, and Director Chad O'Donnell, P.E., S.E., LEED AP BD+C, Associate Vice President and Director of Structural Engineering for HGA Architects & Engineers, Milwaukee, Wisconsin, and a member of SEAWI.

Save the Date!

NCSEA Structural Engineering Summit

October 11 – 14, 2017

Washington Hilton • Washington, D.C.

NCSEA Webinars

October 20, 2016

Design of Advanced Composite Rehabilitation Systems – Avoiding Pitfalls and Confidently Detailing Designs

Scott Arnold, P.E., Director of Engineering and Research & Development, Fyfe Company

November 3, 2016

Deferred Submittals – Who is Responsible and When?

Dean Brown, P.E., Lauren Engineering & Constructors

November 15, 2016

Structural Engineering Ethics – Black & White or 50 Shades of Grey

Marc S. Barter, S.E., SECB, President, Barter & Associates

Detailed information on the webinars and a registration link can be found at **www.ncsea.com**. Subscriptions that include both live and recorded webinars are available for NCSEA members! A library of over 150+ Recorded Webinars is now available online 24/7/365. *Webinars provide 1.5 hours of continuing education, approved for CE credit in all 50 states.*

Grant Program Recipients Announced

The recipients of the 2016 NCSEA Grant Program were announced at last month's NCSEA Structural Engineering Summit in Orlando. A total of $20,670 will be disbursed in grants to NCSEA Member Organizations.

The Grant Program was instituted in 2014 to assist NCSEA Member Organizations in growing and promoting their organizations and the structural engineering field.

Any NCSEA Member Organization or member(s) of a Member Organization are eligible to apply. Requests can be submitted for any program or endeavor that is consistent with, and supportive of, NCSEA's Mission Statement. All applications must be approved by the appropriate Member Organization.

"The NCSEA Grant Program is a great way for us to advance our mission," stated NCSEA President Brian Dekker. "We're excited to help strengthen our MOs through this program."

The projects receiving the 2016 Grants are:

- **Structural Engineers Association of Kansas & Missouri (SEAKM)** – Awarded grant funding for the Kansas State University chapter of SEAKM to send students to structural conferences throughout the U.S.
- **Structural Engineers Association of Illinois (SEAOI)** – Awarded grant funding for a technology improvement package to improve the quality and quantity of online programming.
- **Structural Engineers of New Hampshire (SEANH)** – Awarded grant funding for the Young Members Group to host an S.E. Exam Review, participate in student outreach programs, and work with the local Habitat for Humanity.
- **Minnesota Structural Engineers Association (MNSEA)** – Awarded grant funding for a Structural Engineering Breakfast Forum with presentations and panel discussions for University of Minnesota engineering students.
- **Structural Engineers Association of Metropolitan Washington (SEAMW)** – Awarded grant funding for a Pecha Kucha Dinner Event and Presentation developed and organized by the SEAMW Young Members Group.
- **Structural Engineers Association of Colorado (SEAC)** – Awarded grant funding for an expansion of educational programming, such as project presentations, job site tours, and Plant Tours.
- **Structural Engineers Association of Texas (SEAOT)** – Awarded grant funding for setting up an Engineers Alliance for the Arts and Student Impact Project at a Houston high school.
- **Structural Engineers Association of Utah (SEAU)** – Awarded grant funding for an update to the 25-year-old Snow Load Study, utilizing new data and new nationally recognized statistical modeling techniques. This project is a grant pledge based on Member Organization fundraising.

Information and the application for the 2017 Grant Program is available on the Member Organization page of **www.ncsea.com**. The 2017 grant recipients will be announced at the 2017 Structural Engineering Summit in Washington, DC.

2017 NCSEA
EXCELLENCE IN
STRUCTURAL ENGINEERING AWARDS

Call for Entries

The NCSEA Excellence in Structural Engineering Awards annually highlights some of the best examples of structural engineering ingenuity throughout the world. Awards will be presented in eight project categories:

- New Buildings under $20 Million
- New Buildings $20 Million to $100 Million
- New Buildings over $100 Million
- New Bridge and Transportation Structures
- Forensic / Renovation / Retrofit / Rehabilitation Structures up to $20 Million
- Forensic / Renovation / Retrofit / Rehabilitation Structures over $20 Million
- Other Structures

Eligible projects must be substantially complete between January 1, 2014 and June 30, 2017. **Entries are due Tuesday, July 18, 2017.**

Awards will be presented in October at the NCSEA Structural Engineering Summit in Washington, D.C. Winning projects will be featured in future issues of STRUCTURE magazine. For award program rules, project eligibility and entry forms, see the Call for Entries on the NCSEA website at **www.ncsea.com**.

2017 Call for Abstracts Open

Abstract submission for the 2017 NCSEA Structural Engineering Summit is now open.

The 2017 NCSEA Structural Engineering Summit Committee is seeking presentations that deliver pertinent and useful information that the attendees can apply in their structural engineering practices.

Submissions on best-design practices, new codes and standards, recent projects, advanced analysis techniques and other topics that would be of interest to practicing structural engineers are desired.

The 2017 Summit will feature education specific to the practicing structural engineer, in both technical and non-technical tracks. The 2017 NCSEA Structural Engineering Summit will take place at the Washington Hilton in Washington, D.C., October 11–14.

The Abstract submittal form can be found on the Summit page of **www.ncsea.com**, and must be returned by February 24, 2017. Speakers will be notified of abstract acceptance by March 22, 2017.

SAVE THE DATE

THE PREMIERE EVENT FOR STRUCTURAL ENGINEERING

Come for the innovative solutions and cutting-edge knowledge, leave with connections and resources to advance your career.

Technical Program Released

The Structures Congress 2017 will feature 12 tracks of sessions covering current topics in structural engineering. A wide variety of sessions will be presented, including ASCE 7-16, Seismic, Tall Buildings, Bridge Practice, Wood and Timber, Professional Practice, Extreme Loads, Blast, Steel, Sustainability, and much more. View the Structures Congress technical program at **http://submissions.mirasmart.com/ASCE/Structures2017/Itinerary/ConferenceMatrix.asp**.

Structures Congress 2017 Keynotes

SEI has announced the keynote speakers for Structures Congress 2017, April 6 – 8, 2017 in Denver, CO. Thursday morning will begin with a talk by Greg MacGillivray, director of the ASCE sponsored IMAX film "Dream Big." Greg will present some behind-the-scenes moments in the creation of this awe-inspiring film. Friday's luncheon will feature former Denver mayor, Guillermo "Bill" Vidal, P.E. Born in Cuba, trained as a civil engineer, and a lifetime civil servant, Mayor Vidal will talk about leadership as a personal journey.

Visit the Structures Congress website at **www.structurescongress.org** for the latest information.

Advance to SEI Fellow

Apply by December 1, 2016

The SEI Fellow grade of membership recognizes accomplished SEI members as leaders and mentors in the structural engineering profession. The benefits of becoming an SEI Fellow include recognition via SEI communications and at the annual Structures Congress along with a distinctive SEI Fellow wall plaque and pin, and use of the F.SEI designation. SEI members who meet the SEI Fellow criteria are encouraged to submit application packages online by December 1, 2016, to advance to the SEI Fellow grade of membership and be recognized at Structures Congress, April 6 – 8, 2017 in Denver, CO. Visit the SEI website at **www.asce.org/structural-engineering/sei-fellows** for more information.

SEI Young Professional Scholarship

Apply by December 1, 2016

Apply for the SEI Young Professional Scholarship (for age 35 and younger) to attend Structures Congress 2017, April 6 – 8, 2017, in Denver, CO. SEI is committed to the future of structural engineering and offers a scholarship for Young Professionals to participate and get involved at the annual Congress. Many find this event to be a career-changing and energizing experience, opening up networking opportunities and expanding horizons to new and emerging trends. Visit the SEI website at **www.asce.org/structural-engineering/sei-young-professionals** for more information.

Errata

SEI posts up-to-date errata information for our publications at **www.asce.org/SEI**. Click on "Publications" on our menu, and select "Errata." If you have any errata that you would like to submit, please email it to Jon Esslinger at **jesslinger@asce.org**.

SEI Student Video Competition

Enter by December 19, 2016

Enter the 2017 SEI Student Video Competition for a chance to win a complimentary registration to Structures Congress 2017 in Denver, CO. Your student team is invited to create a video on the theme "Physical Connections in the World Around You." The top 6 videos will be shown at the Congress and used to promote structural engineering on various social media platforms. The winning team will receive complimentary registration for up to five students and their faculty advisor to participate at 2017 Structures Congress. Visit the SEI Student page at **www.asce.org/structural-engineering/sei-students** for more information.

SEI Student Career Networking Event

Employers:

The Student Career Networking Event at the 2017 Structures Congress is a new opportunity for companies to connect with the best and brightest structural engineering students. Participating employers can send up to 4 of their representatives. Your organization will also be listed in event promotions, and will receive student profiles and contact info (including resumes) in advance. SEI Elite Sustaining Organization Members enjoy complimentary participation; other organizations can elect to participate in this event for a nominal fee. Visit the SEI website at **www.asce.org/structural-engineering/sustaining-organization-members** for more information about Sustaining Organizational Membership.

Students:

Full-time and graduating students can apply to attend and have the opportunity to network one-on-one with employers for structural engineering positions and internships. Apply by March 15, 2017. Visit the SEI Student page at **www.asce.org/structural-engineering/sei-students** for more information.

NEW SEI Futures Fund Initiatives

The SEI Futures Fund Board recently approved $64,000 funding for the following FY17 strategic initiatives:

- SEI Global Activities initiatives
- Stakeholder workshop for Structural Engineering Continuing Education
- Creating a new SEI Student Struct. Engineering Competition
- Scholarships for Young Professionals to engage at Structures Congress

Your gift of support provides critical funding to realize our Vision the Future of Structural Engineering. Learn more about efforts made possible with your support at **www.asce.org/structural-engineering/sei-futures-fund**. Gifts are fully deductible for income tax purposes.

Interview with Journal of Structural Engineering Editor

The *Journal of Structural Engineering* is one of the oldest and most respected professional journals in its field. Key topics include the art and science of structural modeling and design; develop, apply and interpret the results of novel analytical, computational and experimental simulation techniques; propose new structural systems and study the merits of existing ones; pioneer methods for maintenance, rehabilitation and monitoring of existing structures; and investigate the properties of engineering materials as related to structural behavior. Recently, Editor of the journal Sherif El-Tawil, Ph.D., P.E., F.SEI, F.ASCE, sat down with ASCE Publications to talk about the journal and his passion for structural engineering. Read the complete interview on the ASCE Library website at **http://ascelibrary.org/page/jsendh/editorjse**.

How You Can Help ICC Adoption of ASCE 7-16

It is not too late to educate your building officials about the importance of supporting adoption of the 2016 Edition of ASCE 7-16, *Minimum Design Loads & Associated Criteria for Buildings and Other Structures*.

It is especially important for the ICC to adopt ASCE 7-16 because of the following significant changes to the standards;

- New wind speed maps
- New regional snow data
- New chapter on tsunami design provisions

SEI is asking building officials and other ICC Governmental Member voting representatives to support ASCE 7-16 during the Group B Public Comment Hearings at the 2016 ICC Annual Conference, Kansas City, MO, on October 19 – 25, 2016. If you plan to attend this hearing, voice your support.

For more information and a list of organizations supporting ICC adoption of ASCE 7-16, visit the SEI website at **www.asce.org/structural-engineering/asce-7-and-sei-standards**.

SEI Local Activities

Georgia Chapter

The SEI Georgia Chapter recently provided technical assistance and some tools to the ASCE Georgia Section Younger Members Group. The group was working, with other groups, on a project to build a bridge in Nicaragua. This bridge will be a vital link for the villagers to access schools, healthcare, and markets. See the SEI news web page for complete details.

San Francisco Chapter

The SEI San Francisco Chapter is serving their members by conducting tours and providing technical presentations. In the next few weeks, the chapter will be visiting the Transbay project in downtown San Francisco and hosting a presentation on accelerated bridge construction. See the SEI news web page for complete details.

Get Involved in Local SEI Activities

Join your local SEI Chapter, Graduate Student Chapter (GSC), or Structural Technical Groups (STG) to connect with colleagues, take advantage of local opportunities for lifelong learning, and advance structural engineering in your area. If there is not an SEI Chapter, GSC, or STG in your area, review the simple steps to form an SEI Chapter at **www.asce.org/structural-engineering/sei-local-groups**.

Local SEI Chapters and Structural Technical Groups of the ASCE Sections/Branches serve local member structural technical and professional needs through a variety of innovative programs. SEI supports local SEI Chapters with opportunities for local Chairs to learn about new initiatives and best practices with other local SEI Professional Chapter and Grad Student Chapter leaders (quarterly conference call and annual funded SEI Local Leader Conference including technical tour and training). Those local structural groups that affiliate with SEI and establish local Chapters receive SEI Chapter logo/branding, complimentary webinar and banner, and more.

JUST RELEASED
Updated Guideline and Tool for Performance of Site Visits

For the first time since 2009, CASE Guidelines and Toolkit Committees have updated the Site Visit Guidelines and companion Site Visit Cards tool to reflect updated industry standards and practices.

Guidelines for the Performance of Site Visits is a guide intended for the younger engineer but is useful for engineers of all experience levels. Structural engineers know that site visits are crucial construction phase services that help clarify and interpret the design for the contractor. Site visits are also opportunities to identify construction errors, defects and design oversights that might otherwise go undetected. Engineers should include adequate construction phase services as a part of their scope of services to ensure the design intent is properly implemented.

The committee did a complete overhaul of this document which included adding key points to summarize each section, updated references and definitions, and discussions of current tools of the trade.

CASE Tool 10-1: Site Visit Cards provides sample cards for the people in your firm who make construction site visits. These cards provide a brief list of tasks to perform as a part of making a site visit: What to do before the site visit; What to take to the construction site; What to observe while at the site; What to do after completing the site visit. The sample cards include several types of structural construction, plus a general guide for all site visits.

To go along with the updated practice guideline document, the tool has consolidated general information to one section or card, with the remaining cards dealing with specific types or materials of construction. Added sections include Drilled Piers, Driven Piles, Auger Cast Piles, Earthwork Beneath Building, Post-Tensioned Concrete, Tilt-Up Concrete, and Cold-Formed Steel Framing.

To view the updated practice guideline, go to **www.acec.org/case/getting-involved/guidelines-committee**.

To view the updated tool, go to **www.acec.org/case/getting-involved/toolkit-committee**.

CASE Practice Guidelines Currently Available

CASE 976-A — *Commentary on Value-Based Compensation for Structural Engineers*

The importance of receiving adequate fees for structural services is vital for the engineering practice to thrive. If fees are not adequate, the structural engineering professional becomes a commodity; libraries are not maintained, computer software and equipment becomes out-dated, and the quality of our product declines significantly.

Value Based Compensation relies on the concept that there are specific services, which may vary from project to project, that provide valuable information to the client and whose impact on the success of the project is far in excess of the prevailing hourly rates. Value Based Compensation is based on the increased value or savings these innovative structural services contribute to the project. As a result, the primary beneficiary of an innovative design or a concept is the owner, but the innovative engineer is adequately compensated for his knowledge and expertise in lieu of his time

CASE 976-C — *Commentary on Code of Standard Practice for Steel Buildings and Bridges*

The 2010 COSP addresses many recent changes in the practice of designing, purchasing, fabricating and erecting structural steel and is, therefore, a continuation of the trend of past improvements and developments of this standard. It is important to note that the Structural Engineer can change any of the requirements of the Code of Standard Practice by specifying an alternative in the Contract Documents.

This document discusses the list of changes published in the preface to the 2010 Edition and provides some commentary on these changes. This document also addresses areas of the COSP that may not be well understood by some SERs but will likely have an impact on the structural engineer's practice of designing and specifying structural steel.

CASE 976-D — *Commentary on 2010 & 2015 Code of Standard Practice for Steel Joists and Joist Girders*

The specification of Joists and Joist Girders can provide an economical structural solution, but there are very specific requirements that must be understood by all parties. The updated 2010 SJI COSP provides a more practical approach to specifying joists, to introduce new design terms for use by the structural engineer, and to identify and clarify topics that may have been subject to varying interpretation in the past. The more recently released 2015 SJI COSP provides additional clarifications and minor revisions.

This commentary provides observations and analysis of the revisions and additions in both documents and discusses specific aspects of the COSP that have a direct impact on the structural engineer's practice of specifying steel joists. A familiarity and understanding of the entire SJI COSP are necessary to ensure the proper design and documentation of Steel Joists and Joist Girders. However, the discussion highlights sections of particular interest to the specifying structural engineer.

You can purchase these and the other Risk Management Tools at ***www.acec.org/coalitions/coalition-publications****.*

CASE Risk Management Convocation in Denver, CO

The CASE Risk Management Convocation will be held in conjunction with the Structures Congress at the Hyatt Regency Denver and Colorado Convention Center in Denver, CO April 6 – 8, 2017. For more information and updates go to **www.structurescongress.org**.

The following CASE Convocation sessions are scheduled to take place on Friday, April 7:

8:00 AM – 9:30 AM *Contractual Risk Transfers for Professionals: Mastering Indemnity, Insurance and the Standard of Care*
Moderator/Speaker: Ryan J. Kohler, Collins, Collins, Muir + Stewart, LLP

10:00 AM – 11:30 AM *Construction Administration as a Risk Management Tool*
Moderator / Speaker: Daniel T. Buelow, Willis Towers Watson

2:00 PM – 3:30 PM *Projects with the Largest Losses and Claim Frequency*
Moderator: Mr. Timothy J. Corbett, SmartRisk
Speaker: Brian Stewart, Esq., Collins, Collins, Muir + Stewart, LLP

4:00 PM – 5:30 PM *Tackling Today's Business Practice Challenges – A Structural Engineering Roundtable*
Moderator: David W. Mykins, P.E., Stroud Pence & Associates

Follow ACEC Coalitions on Twitter – @ACECCoalitions.

CASE Winter Planning Meeting – SAVE THE DATE

The 2017 CASE Winter Planning Meeting is scheduled for February 17 – 18 in San Diego, CA. If you are interested in attending the meeting or have any suggested topics/ideas from a firm perspective for the committees to pursue, please contact Heather Talbert at **htalbert@acec.org**. Agenda will be published in early December!

Engineers Joint Contract Documents Committee (EJCDC) NEW SERIES

2016 Design-Build "D-Series" Documents

This significant modernization, revision, and expansion of the 2009 edition brings the D-Series to the leading edge of 21st Century alternative project delivery business practices. The 2016 D-Series accommodates the latest practices in design-build contracting, is scalable for use on large and small projects, tailorable to project requirements, and is the most complete family of standard design-build contract documents available in North America.

What's NEW in the 2016 EJCDC's D-Series:

- D-110, Guide to Request for Qualifications – Design-Build Project
- D-111, Guide to Request for Proposals – Design-Build Project
- D-425, Price Proposal Form – Design-Build Project
- D-512, Agreement between Owner and Design-Builder for Progressive Design-Build
- D-523, Construction Subcontract for Design-Build Project
- D-620 Design-Builders Application for Payment
- D-800, Guide to the Preparation of Supplementary Conditions (Design-Build)
- D-940, Work Change Directive – Design-Build Project
- D-941, Change Order – Design-Build Project

What's REVISED from 2009:

- D-500, Agreement between Owner and Owner's Consultant
- D-505, Agreement between Design-Builder and Engineer
- D-520, Agreement between Owner and Design-Builder (Stipulated Price)
- D-525, Agreement between Owner and Design-Builder (Cost-Plus)
- D-610, Design-Build Performance Bond
- D-615, Design-Build Payment Bond
- D-700, Standard General Conditions of the Contract between Owner and Design-Builder.

EJCDC Contract Documents offer many benefits:

- Created and peer-reviewed by experienced industry experts
- Reduced conflicts and litigation
- Balanced and fair provisions
- User-friendly and easily customizable (provided in MS Word)

Notice: EJCDC encourages the use of its standard contract documents for the benefit of all parties involved in engineering design and construction projects. Any change or modification to the language of a document should be reviewed by legal counsel before using. Users should also be aware that a change in one document may affect related documents and should be coordinated to avoid confusing or conflicting language.

To purchase these documents, please visit the ACEC Bookstore at **www.acec.org/bookstore**.

Millennials in the Structural Engineering Workforce

By Greg McCool, P.E.

As sure as the Earth keeps turning, each generation finds ways to stereotype the ones that preceded it. Generation "Y" or the "Millennial" generation is no different, and it has no shortage of opinions on preceding generations. *Those in the Greatest Generation are curmudgeons who are out of touch with technology. Baby boomers define their lives by their work and are overly competitive. Generation Xers are greedy, cynical, and blunt.* The youngest entrants to the structural engineering profession (who happen to be Millennials at this point in time) must ascertain to what extent these stereotypes are true and how best to navigate them. Let's consider how the differing viewpoints of generations lead to struggles and miscommunication by following the career path of a hypothetical young structural engineer.

Trouble starts as the engineer, fresh out of college, begins the search for employment. Increasingly, young job seekers value company culture and various "perks" over direct compensation. Millennials hear stories about the flexible hours, amenities, and collegial atmosphere at tech companies like Google and Facebook. Why wouldn't you want to work at a place where you can bond with your coworkers over Ping Pong and organic food buffets? These whimsical but often unrealistic expectations of work environments may result in head-scratching from baby boomers and their older peers, who tend to have a more utilitarian approach to setting up an office.

While perhaps more nebulous and harder to define, the millennial generation is also marked by its desire for work/life balance and future leadership opportunities. In a recent survey conducted by Deloitte on those born after 1982, respondents listed "good work/life balance" and "opportunities to progress/be leaders" as the two most important criteria by which they evaluate job opportunities after salary and benefits. Given that structural engineering firms do not desire and cannot afford to engage in bidding wars to hire young engineers coming out of college, it is important for hiring managers to consider what else is valued by millennial job-seekers. Most firm leaders are currently baby boomers or Generation Xers who may tend to overlook the non-salary criteria.

Assuming the bright-eyed engineer has found a meaningful job at a casual, perk-filled workplace, the struggles continue as he or she dives into the day-to-day. Young structural engineers consider themselves masters of technology. They are fluent in Microsoft Excel and can operate analysis software as quickly as tying their shoes. Unfortunately, these proficiencies, coupled with a lack of experience, can lead the young engineer to create inaccurate models, rely solely on computer output, and disregard the first principles and limitations behind a software program's operation. Often, the young engineer's work is located entirely within the computer model, with no written record of modeling decisions made, input, analysis results, and member design. An older engineer utilizing the Moment Distribution Method and various rules of thumb would rightly consider this lack of "engineering common sense" to be quite alarming.

On the soft skills side of the profession, some young engineers are frustrated by older colleagues who do not respond to electronic communication with urgency. This lack of immediacy can be blamed on Millennials coming of age with texting and social media, in which responses are expected to be instantaneous. Similarly, regarding feedback from management, millennials were showered with near-instant praise from parents, teachers, and coaches throughout their youth, and they expect the same through college and into the workforce. Needless to say, the more stoic baby boomers tend not to provide this sort of feedback immediately unless something has gone seriously wrong, and they probably wonder why their younger colleagues need to be coddled so much.

Suppose the young engineer, having spent some time with a particular company, begins to look for a new job opportunity. Increasingly, this conversation happens sooner for young engineers than any generation preceding them. Results of a survey reported in *Forbes* show only 13 percent of millennials believe that workers should stay with the same employer for at least five years, compared to almost half of baby boomers. Millennials are a generation full of people content in swinging from vine to vine. However, such ambition can be construed as flightiness and disloyalty to older engineers who tend to be more devoted to their employers.

In addition to switching jobs in order to find better opportunities, an alarming number of young engineers feel compelled to drop out of the profession entirely. According to a study conducted by the SEI Young Professionals Committee and reported in STRUCTURE magazine (April 2015), almost 30% of structural engineers who leave the profession do so because they felt discriminated against (most within the first six years of employment). Because society has changed considerably in recent decades on issues of race, gender, and sexual orientation, younger engineers may have more progressive views and expectations in these areas. Encountering overt or covert discrimination is jarring for those affected and remains a problem that the structural engineering community needs to address.

Young structural engineers entering the workforce today have many strengths. However, their weaknesses, including perceived disloyalty and lack of common sense, should not be overlooked. What is the way forward? Young engineers must reframe their perception of the older generations so that negative stereotypes become positive traits to emulate. Revisiting the opening paragraph, *those in the Greatest Generation are solidly grounded in the fundamentals of engineering and have unsurpassed common sense due to years of "pencil and paper" practice. Baby boomers are loyal to their companies and go to great lengths to win new work. Generation Xers display excellent business acumen and are not afraid to speak up if something is not right.* Conversely, these generations must recognize the positive change that a generation of open-minded, ambitious and tech-savvy structural engineers brings to the table. At the end of the day, everyone is in this industry together. Young or old, we must all make the most of the opportunity to learn from other generations.▪

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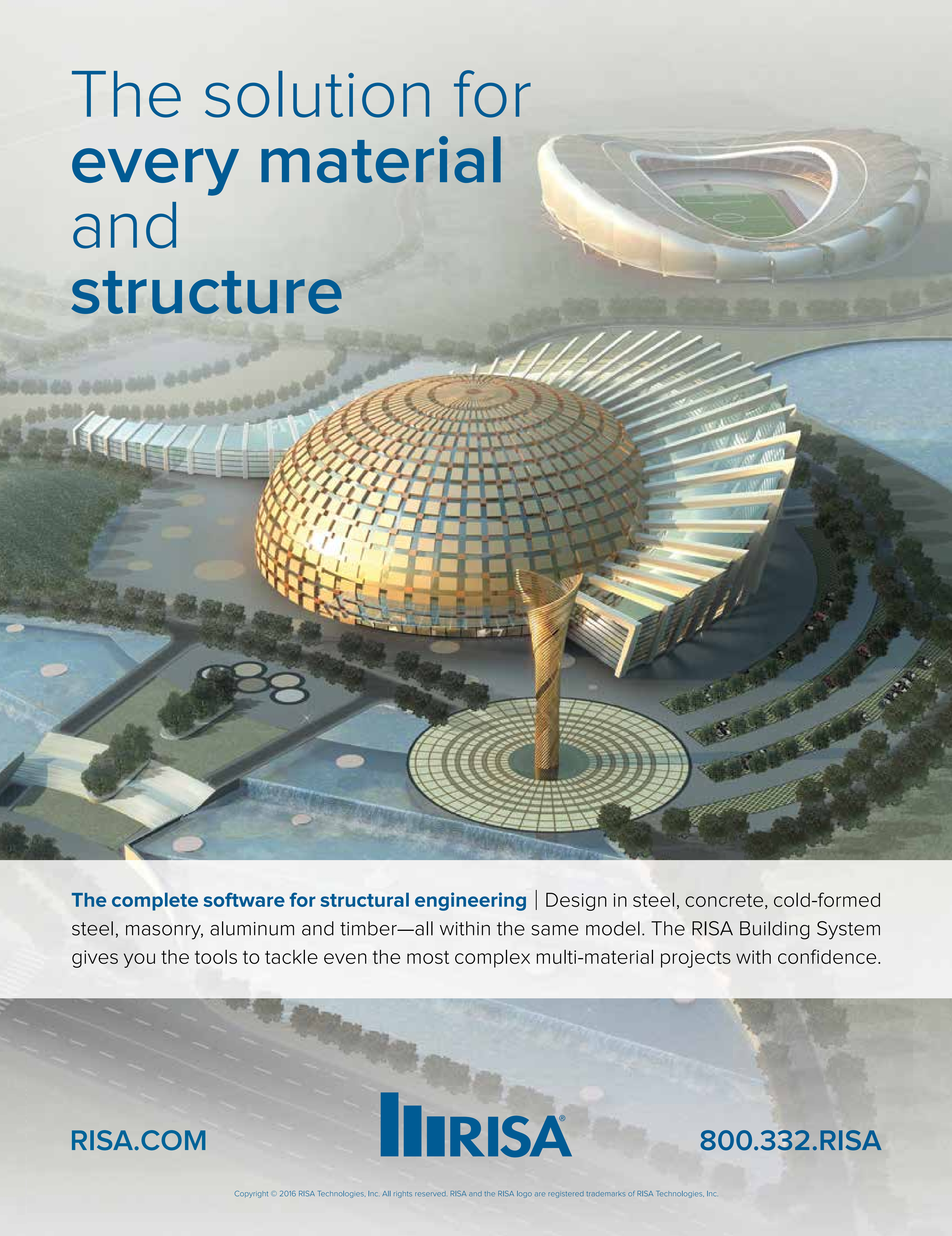
The online version of this article contains detailed references. Please visit **www.STRUCTUREmag.org**.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