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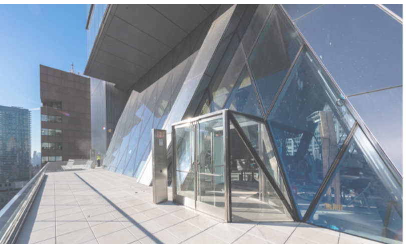

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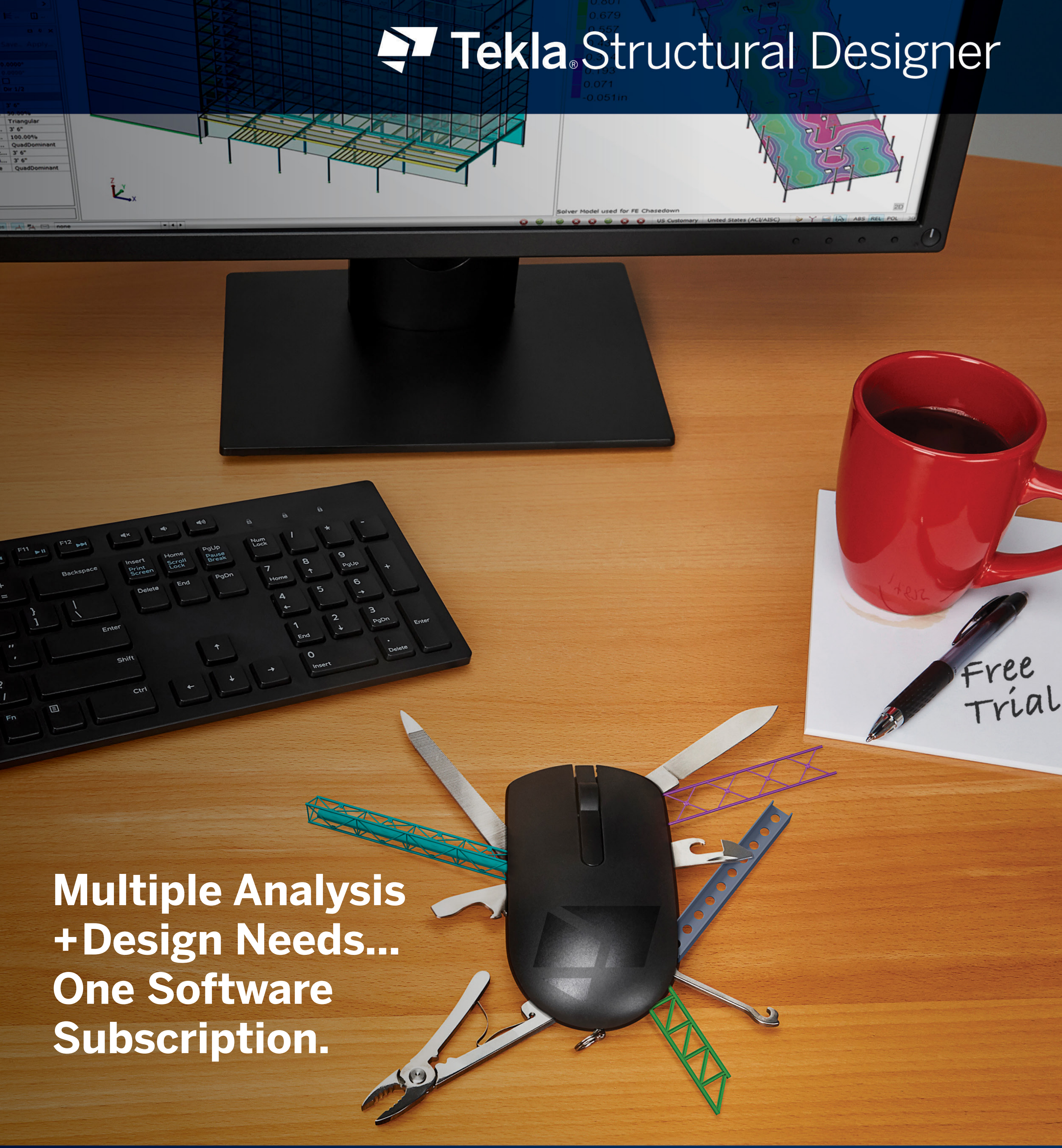
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## FEATURES, CON'T



## COLUMNS AND DEPARTMENTS

# Exciting News for SEI Standards

*By Donald R. Scott, P.E., S.E., F.SEI, F.ASCE*

For many in the structural engineering industry, when we hear of the SEI standards, we automatically think of the ASCE/SEI 7, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, and that is where our knowledge stops. However, the Codes and Standards Activities Division (CSAD) of SEI is responsible for developing and updating twenty-five standards.

## Recently Published and New Standards

Recently, SEI published the 2022 edition of ASCE/SEI 7, which includes new provisions for the design of buildings for tornado effects and updates for all hazards. New in ASCE/SEI 7-22, all hazard data is available digitally from the ASCE 7 Hazard Tool at **asce7hazardtool.online**, which is free to access by everyone and used to determine the required design criteria for any project site. Other widely used industry standards recently published include ASCE/SEI 43-19 *Seismic Design Criteria for Structures, Systems and Components in Nuclear Facilities*, ASCE/SEI 48-19 *Design of Steel Transmission Pole Structures*, and ASCE/SEI 49-21 *Wind Tunnel Testing for Buildings and Other Structures*. In addition to the well-known standards, SEI recently published a new standard to aid the profession in designing athletic field lighting in the ASCE/SEI 72-21 *Design of Steel Lighting System Support Pole Structures*. Each of these standards is used every day by segments within our profession to complete designs for their clients and provide the safety to the public that we are obligated to provide.

In partnership with the Charles Pankow Foundation, two SEI publications advance performance-based design: ASCE/SEI *Prestandard for Performance-Based Wind Design* and the *Performance-Based Structural Fire Design*, available for free from **asce.org/sei**.

## ASCE 7 Wind Research

It has been more than 40 years since any concerted and focused effort has been made to perform wind tunnel research in support of the ASCE 7 wind load provisions. Thousands of wind tunnel studies have been performed for select buildings and bridges during the past several decades. Those studies have enabled the design and construction of many landmark structures and iconic tall and supertall buildings around the globe. Unfortunately, despite our growing knowledge, the needed funding to bring our wind provisions to an achievable level with today's technology was not available until recently.

In 2020, SEI members of the ASCE 7 Wind Load Subcommittee appealed to many firms, organizations, and industry partners to support the development of a three-year wind research effort. The goal is to study the possible combination of the ASCE 7 Chapter 27 *All-Heights Directional Method* and the Chapter 28 *Low-Rise Envelope Method* into a single procedure. Not only will this bring current technology to the provisions, but it is also intended to reduce the confusion associated with having two methods for determining the wind loads on buildings. The response to these fund-raising efforts has been overwhelming. Several firms, individuals, and industry partners generously pledged this research (*see the sidebar*).

The research effort includes two components: wind tunnel testing and the development of a single methodology. Data for 66 models ranging from low- to high-rise buildings have been analyzed in previous research efforts, including 30 wind tunnel models funded previously with support from the Charles Pankow Foundation and SEI. Greg Kopp, Ph.D., and Jin Wang, Ph.D., have analyzed this data to identify how the overall wind loads for uplift and shear depend on the geometric parameters, with results published in the ASCE/SEI *Journal of Structural Engineering*. Building on these previous studies, the researchers have defined the path needed to develop a full set of geometries to analyze in the wind tunnel to create a complete dataset. Further, the previous analyses results suggest that once they have a full set of data, it will be reasonable to develop a single, new method for wind load analysis. These new provisions will capture the important advantages of each of the current Chapter 27 and 28 methodologies while eliminating the current disadvantages and confusion of using two.

Currently, wind tunnel studies are underway to provide data for the additional building and roof configurations needed. A video explaining these efforts and procedures can be viewed at **https://youtu.be/eYOn1qmDDwE**. After completing the wind tunnel testing, the researchers, an advisory panel, a peer review team, and many ASCE 7 Wind Loads Subcommittee members will collaborate to develop the new methodology. Results of these research efforts will include proposals to the ASCE 7-28 Wind Load provisions to change the way wind loads are evaluated by unifying the various methods into a single procedure. The comprehensive effort and partnership across many experts is only possible because of the generous support of many in the profession. We are all indebted to them for advancing the profession.

Although SEI is known primarily for developing the ASCE/SEI 7 Standard, there is much more happening within SEI to support and advance our profession. If you would like to join a committee effort, apply at **asce.org/SEI**.▪

*Donald R. Scott is Senior Principal at PCS Structural Solutions, SEI President-elect, and chairs the SEI Codes and Standards Executive Committee.*

### ASCE 7 Research Supporters

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# Automation and the Future of Structural Engineering – Installment 2

By Eytan Solomon, P.E., LEED AP

*Continuing our series on automation, I sat down (virtually) in September 2021 with two more industry experts in digital design:* **Michael Bolduc, P.E.**, *a Senior Project Manager at Simpson Gumpertz & Heger, and* **Dr. Kristopher Dane**, *Vice President and Director of Digital Design at Thornton Tomasetti. Both serve on the SEI Digital Design Committee. Below are highlights from our discussion.*

**The two of you wrote an excellent article in STRUCTURE magazine called *Communicating in a BIM World* (March 2021). The article alludes to the increasing pressure on structural designers to respond to major architectural changes or coordination issues late in the process. Do you foresee this pressure resulting in more "studio integration," i.e., engineers and architects working for the same company, or perhaps under a design-build/integrated project delivery (IPD) arrangement?**

**Dane:** I doubt that an acquisition that makes a multidiscipline shop will resolve the kinds of issues that we outlined in the article. The challenges are due to the nature of the design process; how the different disciplines approach their work. For example, structural engineers will still need to build lateral and gravity models separate from the BIM. We need time to do the work and bring it back to our partners. Being under one roof may ease communications because it is easier to wander over to your architectural colleague's desk and have a chat. But as a consulting engineer, especially with the prevalence of virtual meetings, I think those conversations are just as easy for us as in a multidiscipline shop.

Regardless of the firm organization, the technology puts us under pressure to emphasize our communication skills. We need to articulate what is driving our design, what will be important early on, and communicate that to our clients in a way that resonates with them. Technology may also be part of the solution. If we can automate some bits of our design and figure out where artificial intelligence (AI) fits in, we will have more time to communicate with design partners and clients. If we're leveraging automation and AI well, we will assess options quickly and will not need as much time between conversation and structural resolution. Those answers will also be given with more confidence because they are based on previous work.

On the second part of your question about design-build or IPD type arrangements, I think any contractual arrangement that helps bring the design team closer to the builders and helps incentivize collective and collaborative behavior is the right answer. This is not a technical problem, but getting the incentives and risk management correct is important to enable the technology.

**Bolduc:** I think the solution really is in the technology. The tech allows us to automate responses, automate some of those preliminary studies and get a sense of things. It helps provide that feel that engineers have had for years, i.e., a senior engineer can look at a beam size and know it is too small. How do you know? I just know because I've seen a lot before.

You can start to teach your staff, whether it is AI or conventional software, by checking those things with immediate feedback. The software is coming along to the point where it does not take three days to run a whole building anymore. They can run almost instantaneously, so you have this instant feedback, and we can almost start to offer shared structural models where the architect can ask, "what happens when such and such happens?" They can start doing some push-and-pull stuff and get feedback from the software.

We can then incorporate and guide them on what that feedback really means and whether it is viable. But you know, if you stretch something and use the color coding – red, yellow, green – and they start stretching, start seeing red? Hey, that is not good. I'm going too far, or I need to thicken something up. I think that technology can simplify some of these structural engineering tasks, especially in the early stage, so that architects understand the ramifications of their concepts.

*Regardless of the firm organization, the technology puts us under pressure to emphasize our communication skills.*

Dr. Kristopher Dane

**Another resonating passage from your article goes: "…as daily work processes become more model-driven and less time is spent looking at the drawings, the potential to miss critical details that are not modeled is introduced." Do you worry that it will lead to structural failures?**

**Dane:** Look at the classic examples like the Hyatt Regency Kansas City; mistakes are often in the details. I think it is important that we leverage technology to reduce the cognitive load on our designers so that they can spend more time thinking about the detailing and about macro issues like the load path and client intent. We should strive to spend less time shuffling paperwork.

**Bolduc:** Tech can help solve the mundane tedious tasks. If you can take away those mundane tasks, you've just freed up 20% of young engineers' time to focus on the details. Focus on how it actually is going to get built, focusing on its complexities that might have gotten pushed too late in the game, like that "easy" button in the Staples commercials. Take away the easy stuff, and now they can focus on the things that require an actual brain to think through. Engineers can then get back to engineering, not just redlining.

**Dane:** If we were sitting here with somebody nearing the end of their career, they might say, "I'm nervous about the technology. The kids these days spend too much time with the models, and I am not sure AI can replace engineering judgment." But we don't think twice about letting Excel do the math for us; we trust SAP and RAM. So as we start to create interoperability tools and design with AI, we *all* must understand the value of the tools and how they work. In addition, for AI tools, we will need to understand the limits of the underlying data sets and the risks involved. Eventually, though, we'll get to a point where we are comfortable having AI as part of our toolset.

*I go into a litany with every architect I work with, even if I worked with them before. Remind them: Here's my workflow. Here's how I operate as an engineer.*

Michael Bolduc

**Bolduc:** It's the "trust but verify." I'm not saying we go and check individual members by hand anymore because that has been vetted and trusted for long enough. Did we say, "OK, we can now trust this?" We do not need to verify anymore. We did that process of verifying to prove the tool works. And until you have used that tool, tens, dozens, hundreds of times, you are constantly worried because, as engineers, we are responsible for public safety. This is what we did for the last 100-foot span. Now it is 150 feet, so double-check and make sure it's right.

**In the article, you had a great phrase recommending that we "proactively open lines of communication within the design team when sharing models and reviewing changes." Could you illustrate how you've seen that work or not work?**

**Bolduc:** I go into a litany with every architect I work with, even if I worked with them before. Remind them: Here's my workflow. Here's how I operate as an engineer. I understand your workflow; you jump in right away, start designing, and then the drawings come out of it. Next, I explain how our checks happen outside of that model that you see.

I explain to them: there will be times when I go silent on the model, or the model stays stagnant for a few weeks at a time. But, you know what I'm doing, I am engineering, and I am verifying what is shown in those drawings, or I'm about to update all those placeholder sizes that I gave you early on, and I explain the process. So, I think that is the big thing – really explaining our workflow early in the project. And then, later in the process, you can remind them that we talked about this.

I explain that "I will keep you apprised as we go, but you will see me just come and go, and you will see things change and not change for periods of time."

**Dane:** Establishing norms and expectations is important. The structural engineering workflow is different enough that it's worth explaining every time. As Mike mentioned, we do our design outside Revit, but we can put placeholders in the model to keep the design coordination moving and support model coordination. So, for example, on a typical project where we are modeling elements to Level of Development 300, we're not going to model steel connections. Still, if we have a structural frame that is being featured or some tight coordination issue, we can model gusset plates in those areas.

In other words, if there are ways we can do a little extra modeling to help ease design or constructability concerns, then, of course, we are going to do that. I want to be a trusted partner. For projects that have a lot of constructability or speed of construction concerns, services like Thornton Tomasetti's Advanced Project Delivery, where a full fabrication model is developed, may address those concerns.

Finally, discuss communication norms for the team at the kickoff. Align on primary communication platforms (email, Microsoft Teams, phone calls, text messages), expected response time, model and drawing exchange frequency, design lockdown dates, and critical path items. Those can be slightly awkward conversations, but in a kickoff with the other disciplines in the room, I find that everybody has those concerns once the conversation is started. If we don't sort it out at the start, issues may snowball as the project progresses. So establishing group norms is never wasted time.▪

*The author would like to thank Michael and Kristopher for sharing their experiences and insights. It is fascinating to see the common themes on how we as engineers might more effectively communicate with clients and collaborators in this era of evolving technology.*

Eytan Solomon is a Senior Associate at Silman and a member of STRUCTURE's Editorial Board. (solomon@silman.com)

# 2021 IBC Significant Structural Changes

## *Part 5: Wood (Chapter 23)*

*By Sandra Hyde, P.E., and John "Buddy" Showalter, P.E.*

This five-part series discusses significant structural changes to the 2021 *International Building Code* (IBC) by the International Code Council (ICC). Part 5 includes an overview of changes to Chapter 23 on wood. Only a portion of the total number of code changes to this chapter is discussed in this article. More information on the code changes can be found in the 2021 *Significant Changes to the International Building Code*, available from ICC (*Figure 1*).

IBC Chapter 23 provides minimum accepted practices for the design and construction of buildings and structural components using wood. The following modifications were approved for the 2021 IBC. Changes are shown in strikethrough/underline format with a brief description of the change's significance.

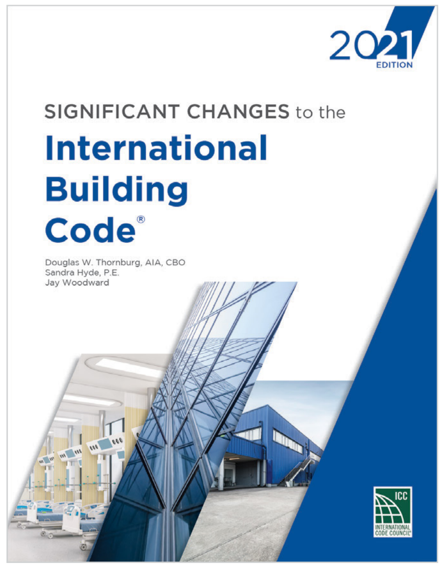



Figure 1. 2021 Significant Changes to the IBC.

### *Wood Truss Bracing*

Revised IBC Section 2303.4.1 now clarifies the installation of permanent truss member restraint and permanent diagonal bracing of individual wood truss members (*Figure 2*).

**2303.4.1.2 Permanent individual truss member restraint (PITMR) and permanent individual truss member diagonal bracing (PITMDB).** Where the truss design drawings designate the need for permanent individual truss member restraint, it shall be accomplished by one of the following methods:

1. PITMR and PITMDB installed using standard industry lateral restraint and diagonal bracing details in accordance with TPI 1 Section 2.3.3.1.1, accepted engineering practice, or Figures 2303.4.1.2 (1), (3), and (5).
2. Individual truss member reinforcement in place of the specified lateral restraints (i.e., buckling reinforcement such as T-reinforcement, L-reinforcement, proprietary reinforcement, etc.) such that the buckling of any individual truss member is resisted internally by the individual truss. The buckling reinforcement of individual truss members shall be installed as shown on the truss design drawing or on supplemental truss member buckling reinforcement details provided by the truss designer or in accordance with Figures 2303.4.1.2 (2) and (4).
3. A project-specific PITMR and PITMDB design provided by any registered design professional.

**2303.4.1.2.1 Trusses installed without a diaphragm.** Trusses installed without a diaphragm on the top or bottom chord shall require a project specific PITMR and PITMDB design prepared by a registered design professional.

**Exception:** Group U occupancies.

**2303.4.1.3 Trusses spanning 60 feet or greater.** The owner or the owner's authorized agent shall contract with any qualified registered design professional for the design of the temporary installation restraint and diagonal bracing and the PITMR and PITMDB for all trusses with clear spans 60 feet or greater.

*(Deleted text not shown for clarity)*

**Permanent individual truss member restraint (PITMR)** Restraint that is used to prevent local buckling of an individual truss chord or web member because of the axial forces in the individual truss member.

**Permanent individual truss member diagonal bracing (PITMDB)** Structural member or assembly intended to permanently stabilize the PITMR's.

**Individual truss member** A truss chord or truss web.

**Change Significance:** The current industry standard of care for installing permanent truss member restraint and diagonal bracing requires that a truss installer (framer) rely on standard industry details. Such details are found in the document B*uilding Component Safety Information (BCSI) – B3: Permanent Restraint/*

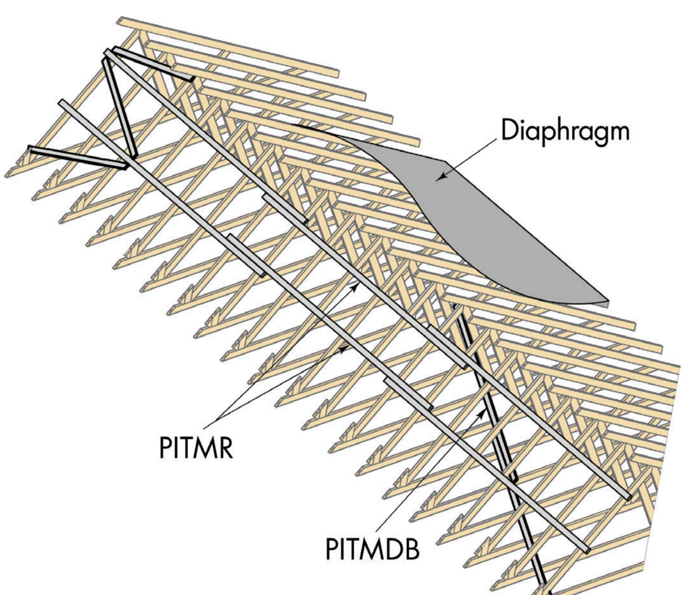



Figure 2. Permanent lateral and diagonal truss web bracing. (Only IBC Figure 2303.4.1.23 is shown for brevity).

*Bracing of Chords & Web Members* as referenced in ANSI/TPI 1 *National Design Standard for Metal Plate Connected Wood Truss Construction* Section 2.3.3.1.1. However, the reality in the field is that those framers are often not familiar with BCSI-B3 and not provided a copy of that document with the trusses. Owners, building designers, truss designers, truss manufacturers, and building officials typically rely on framers to accurately and completely interpret when, where, and how to install required restraint and diagonal bracing for pre-engineered wood trusses.

The new IBC Section 2303.4.1.2 requirements are intended to clarify truss bracing needs. Definitions for an Individual Truss Member, a Permanent Individual Truss Member Restraint (PITMR), and Permanent Individual Truss Member Diagonal Bracing (PITMDB) have been added to IBC Section 202. These definitions should eliminate some confusion in the design community and on the job site regarding what specific bracing members are required and their intended purpose. Terms such as bracing, bridging, continuous lateral brace, and x-bracing are often used but do not necessarily mean the same thing to everyone.

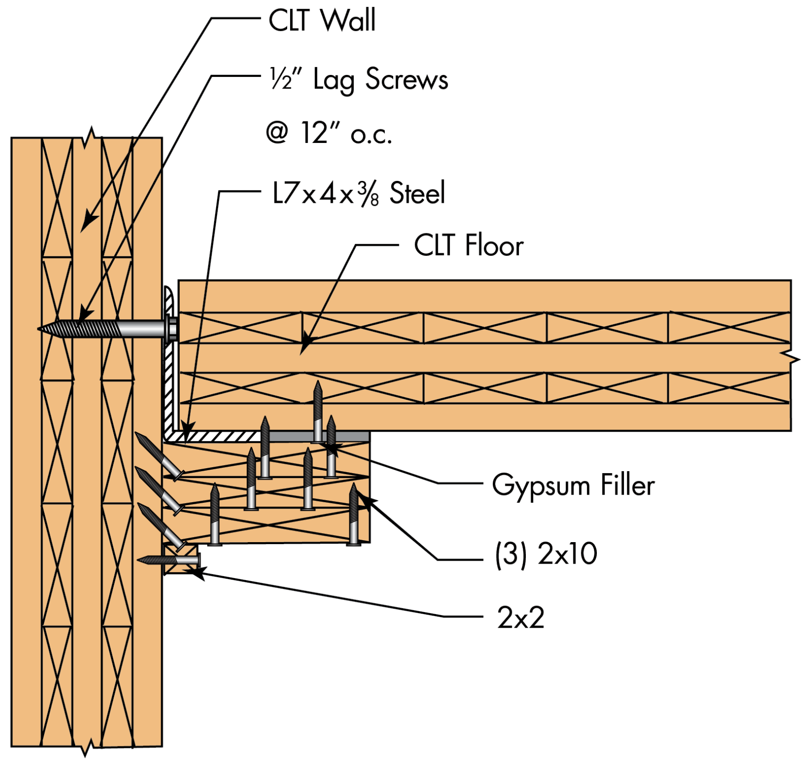


Figure 3. CLT floor-to-wall example from AWC TR10. Courtesy of the American Wood Council.

### *Type IV-A, IV-B, and IV-C Connection Protection*

In new Type IV-A, IV-B, and IV-C construction, a testing option for connections that are part of a fire-resistance-rated assembly is provided. As a second option, a calculation approach for connections required to be protected for the fire-resistance rating of the connected elements is also available.

**2304.10.1 Connection fire resistance rating.** Fire resistance ratings for connections in Type IV-A, IV-B, or IV-C construction shall be determined by one of the following:

1. Testing in accordance with Section 703.2 where the connection is part of the fire resistance test.
2. Engineering analysis that demonstrates that the temperature rise at any portion of the connection is limited to an average temperature rise of 250°F (139°C), and a maximum temperature rise of 325°F (181°C), for a time corresponding to the required fire resistance rating of the structural element being connected. For the purposes of this analysis, the connection includes connectors, fasteners, and portions of wood members included in the structural design of the connection.

**Change Significance:** IBC Sections 704.2 and 704.3 require connections of columns and other primary structural members to be protected with materials that have the required fire-resistance rating. The new Section 2304.10.1 provides two options for demonstrating such compliance for connections in Types IV-A, IV-B, and IV-C construction: a testing option and a calculation option. The provisions do not apply to heavy timber (IV-HT) construction connections because heavy timber structural members do not have a prescribed fire-resistance rating.

IBC Sections 704.2 and 704.3 do not require connections that join elements of the structural frame to be tested in accordance with ASTM E119. The sections require the connections to be protected with a material having a fire-resistance rating greater or equal to the rating of the structural members to which they connect. It is neither practical nor possible to test connections in a standard fire test furnace since there is no capability to test the large connections used to transfer gravity loads. In addition, ASTM E119 does not include any provisions on how to test connections and assess their performance.

Option 1, described in Section 2304.10.1, Item 1, is consistent with ASTM E119 because the connection is included as part of the overall assembly being tested. In other words, the connection itself is not being tested; instead, the assembly is being tested with the connection included within it and is, therefore, subject to the ASTM E119 pass/fail criteria applicable to that assembly.

Some connections used in Types IV-A, IV-B, and IV-C construction are not part of the mass timber element or assembly testing. Option #2, an engineering analysis, is required for those connections by Section 2304.10.1 Item 2. IBC Section 722 permits structural fire-resistance ratings of wood members to be determined using Chapter 16 of the American Wood Council's (AWC) *National Design Specification® (NDS®) for Wood Construction*.

Where a wood connection is required to be fire-resistance-rated, NDS Section 16.3 requires all components of the wood connection, including the steel connector, the connection fasteners, and the wood needed in the connection's structural design, to be protected for the minimum required fire-resistance time. The connection is permitted to be protected by wood, gypsum board, or other approved materials.

Analysis procedures have been developed that allow protection of these connections to be designed based on test results of ASTM E119 fire tests from protection configurations using the exterior thickness of the wood structural member, additional wood cover, and/or gypsum board. The AWC's *Technical Report 10* (TR10): *Calculating the Fire Resistance of Wood Members and Assemblies*, referenced in the NDS Chapter 16 Commentary, provides examples of connection designs meeting the requirements of IBC Section 704 and NDS Section 16.3 (*Figure 3, page 13*).

## General Design Requirements for Lateral Force-Resisting Systems

The 2021 edition of AWC's *Special Design Provisions for Wind and Seismic* (SDPWS) is referenced in the 2021 IBC (*Figure 4*).

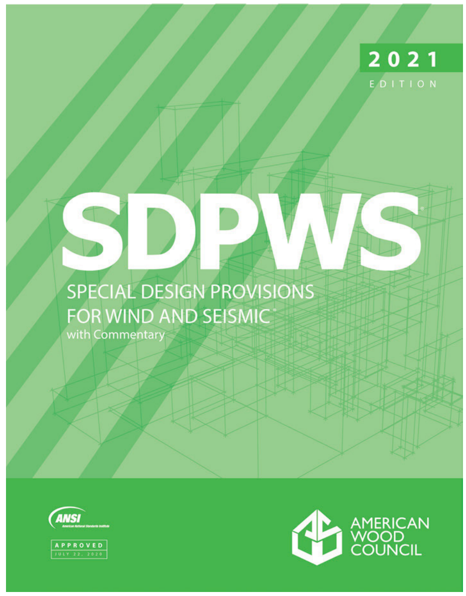


Figure 4. 2021 SDPWS is referenced in the 2021 IBC.

**2305.1 General.** Structures using wood-frame shear walls or wood-frame diaphragms to resist wind, seismic or other lateral loads shall be designed and constructed in accordance with AWC SDPWS and the applicable provisions of Sections 2305, 2306 and 2307.

### Chapter 35

ANSI/AWC SDPWS – 2021~~2015~~: *Special Design Provisions for Wind and Seismic*

**Change Significance:** SDPWS provides criteria for proportioning, designing, and detailing engineered wood systems, members, and connections in lateral force resisting systems. Engineered design of wood structures to resist wind or seismic forces is either by allowable stress design (ASD) or load and resistance factor design (LRFD). Nominal shear capacities of diaphragms and shear walls are provided for reference assemblies. See the article (STRUCTURE, *July 2021*) outlining changes to the 2021 SDPWS.

## Cripple Walls

Cripple wall requirements have been clarified to emphasize that if only interior wood-framed cripple walls exist in a design, no sheathing or solid blocking is required.

**2308.5.6 Cripple walls.** Foundation cripple walls shall be framed of studs that are not less than the size of the studding above. Exterior cripple wall studs shall be not less than 14 inches (356 mm) in length; or shall be framed of solid blocking. Where exceeding 4 feet (1,219 mm) in height, such walls shall be framed of studs having the size required for an additional story. See Section 2308.6.6 for cripple wall bracing.

**2308.6.6.2 Cripple wall bracing in Seismic Design Categories D and E**. For the purposes of this section, cripple walls in Seismic Design Categories D and E ~~having~~ shall not have a stud height exceeding 14 inches (356 mm) ~~shall be considered to be a story and~~, and studs shall be ~~braced~~ solid blocked in accordance with Section 2308.5.6 for the full dwelling perimeter and for the full length of interior braced wall lines supported on foundations, excepting ventilation and access openings. ~~Table 2308.6.1. Where interior braced wall lines occur without a continuous foundation below, the length of parallel exterior cripple wall bracing shall be one and one-half times the lengths required by Table 2308.6.1. Where the cripple wall sheathing type used is Method WSP or DWB and this additional length of bracing cannot be provided, the capacity of WSP or DWB sheathing shall be increased by reducing~~


Figure 5. Interior cripple walls.

~~the spacing of fasteners along the perimeter of each piece of sheathing to 4 inches (102 mm) on center.~~

**Change Significance:** Section 2308.5.6 has been clarified by adding the term *exterior* to the requirements. Also, contradictory text has been deleted from Section 2308.6.6.2. Where cripple walls are exterior walls supporting one or more stories, the wall must now be braced with either solid blocking or sheathing based on wall bracing requirements. These walls have been found to rack sideways and fail in moderate and large earthquakes. Adding sheathing or blocking stiffens the wall.

For buildings where exterior walls are supported by concrete or CMU foundation walls, and a cripple wall is part of an interior wall line, whether below an interior braced wall line or simply supporting the floor above, there is no requirement for bracing the wall line by blocking or sheathing. These walls are inside a much stiffer exterior foundation wall of concrete or CMU block and will not move independently of the floor and exterior walls during an earthquake (*Figure 5*).

Cripple wall bracing in Seismic Design Categories (SDC) D and E is now limited to 14 inches in height and must be solidly blocked along both interior and exterior braced wall lines. Therefore, buildings may only be one-story with a slab on grade foundation, concrete foundation walls, or a crawlspace consisting of studs 14 inches or less in height with solid-blocked cripple walls per Table 2308.2.1. Because cripple walls over 14 inches in height are considered an additional story, a one-story building over taller cripple walls is considered a two-story building and prohibited in SDC D and E. The extent of solid blocking of cripple wall studs to allow for ventilation and access openings has also been clarified.

## Conclusion

Structural engineers responsible for wood design should be aware of significant structural changes in the 2021 IBC. New provisions include clarity on the installation of permanent truss web member lateral and diagonal bracing, both a testing and a calculation option for connections that are required to have a fire-resistance rating for new Type IV-A, IV-B, and IV-C construction, reference to the 2021 SDPWS, and cripple wall requirements emphasizing that, if only interior wood-framed cripple walls exist in a design, no sheathing or solid blocking is required.▪

Parts 1 through 4 of this series ran in STRUCTURE November and December 2021, and January and February 2022, respectively.

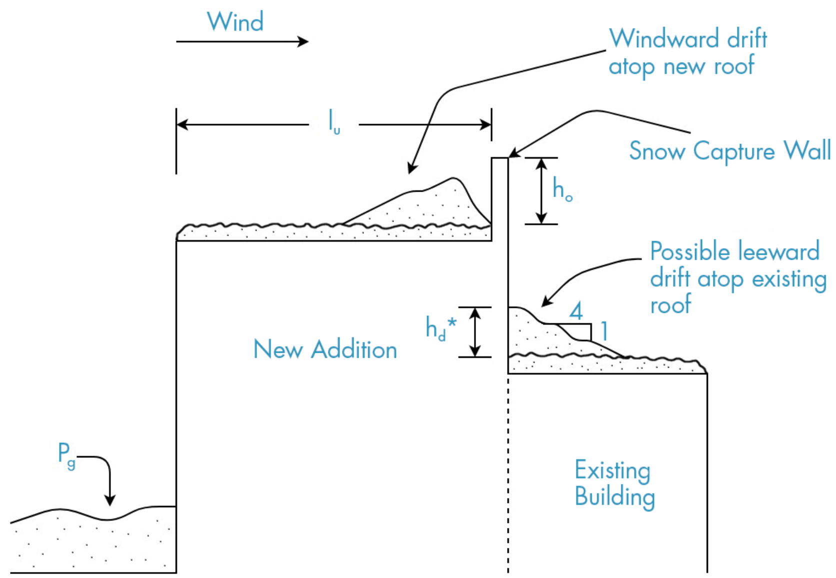
*Sandra Hyde (**shyde@iccsafe.org**) is Managing Director, and John "Buddy" Showalter (**bshowalter@iccsafe.org**) is Senior Staff Engineer, both with ICC's Product Development Group.*

# Snow and Rain Loads in ASCE 7-22

*Part 2*

*By Michael O'Rourke, Ph.D., P.E., and John F. Duntemann, P.E., S.E.*

The American Society of Civil Engineers' ASCE 7-22 load standard, *Minimum Design Loads for Buildings and Other Structures*, is now available, and substantive changes have been made to both the snow and rain provisions. This article is the second in a two-part series regarding these changes. Part 1 (STRUCTURE, *January 2022*) reviewed changes to the ground snow loads, which represents a shift away from uniform hazard to uniform risk, and the addition of a winter wind parameter to account for the variability in winter wind speeds on drift loads. This article reviews other revisions to the snow loads, including a more accurate estimation of the horizontal extent of windward drifts, revised thermal factors $C_t$ to account for current trends in roof insulation and venting, and guidance on design loads for snow capture walls added to this edition. Also, changes were made to Chapter 8 to include a ponding head to the rain load, which provides a consistent approach to assess ponding.




*Capture wall atop new addition.*

## Windward Drifts

The *leeward* roof step drift formation process is straightforward and reasonably well understood. Wind causes upper-level roof snow to be transported (i.e., blown) to the edge of the upper-level roof. A percentage of that snow transport (typically taken to be 50%) remains in the region of aerodynamic shade until the wind stops, the upwind snow source area is depleted, or the leeward drift becomes full. The situation for *windward* roof step drifts is more complex. Based upon measurements by Potac and Thiis in Norway, the initial trapping efficiency is nominally 100%. That is, all the transported snow initially stays upwind of the wall. If the windward drift grows large enough, wind streamlines along the snowdrift surface (snow ramp) move high enough up the wall to carry some windblown snow over the wall, dropping the windward trapping efficiency to less than 20%. The windward drift's slope has a rise-to-run of 1:8 compared to 1:4 at the non-full leeward drift. For the same upwind fetch, ground snow load, and winter wind parameter, the cross-sectional area of the windward roof step drift could be larger or smaller than that for the leeward roof step drift. For roof steps with large differences in elevations, the windward drift's trapping efficiency approaches 100%, producing a larger cross-sectional area than the leeward drift with its roughly 50% trapping efficiency. The reverse is true for small steps for which the net windward trapping efficiency can be closer to 20% (i.e., less than the 50% for leeward drifts).

For simplicity, the ASCE 7-22 Snow and Rain Load Subcommittee changed the windward drift rise-to-run to 1:8 but kept the windward drift height as 75% of the corresponding leeward drift height and kept the current right triangular shape for both. Hence, for the same conditions (i.e., same $P_g$, $l_u$, and $W_2$), the non-full leeward drift has a height of $h_d$ and width of $4h_d$, while the windward drift has a height of $0.75h_d$ and width of $6h_d$ (.75 x $8h_d$).

The advantage of these new provisions is a more accurate estimate of the horizontal extent of windward drifts. The disadvantage is in the determination of the governing drift at a step. Consider the typical case where the upwind fetch parameters for the leeward and windward drifts are different. Using the 7-16 provisions, one only needed to compare the windward and leeward drift heights to determine the governing drift since both had a rise-to-run of 1:4. Using the 7-22 provisions, one must determine the induced bending moments and shear forces in the individual structural components. It is possible that some structural components on the same roof would be governed by the leeward drift, while others would be governed by the windward drift.

## Thermal Factor $C_t$

The thermal factor $C_t$ in ASCE 7 is intended to account for the expected reduction in roof snow loads due to heat flow upward through the roof. There are, of course, other thermal effects, such as solar radiation and above freezing ambient temperatures. However, the other thermal effects result in a similar reduction in both roof and ground snow loads and hence do not influence the ground-to-roof conversion factor.

In ASCE 7-16, the $C_t$ factors ranged from 0.85 for certain greenhouses to 1.3 for freezer buildings. For structures with human wintertime occupancy, the thermal factor is 1.1 for ventilated roofs and 1.0 for all others (unventilated roofs). Historically, these factors have generated minimal comment by practicing structural engineers.

Table 1. Thermal factor $C_t$ for heated structures with unventilated roofs.

| Roof R-value (h ft$^2$°F/BTU) | Ground Snow Load $P_g$ (psf) | | | | | | |
|---|---|---|---|---|---|---|---|
| | 15 | 30 | 45 | 60 | 75 | 90 | >105 |
| 20 | 1.20 | 1.12 | 1.06 | 1.02 | 1.00 | 1.00 | 1.00 |
| 30 | 1.20 | 1.17 | 1.15 | 1.13 | 1.12 | 1.11 | 1.10 |
| 40 | 1.20 | 1.19 | 1.17 | 1.16 | 1.16 | 1.15 | 1.15 |
| 50 | 1.20 | 1.20 | 1.19 | 1.19 | 1.19 | 1.18 | 1.18 |

However, starting in about 1995, ASHRAE and governmental authorities have been requiring (or *recommending*) increased levels of roof insulation. As a result, structural engineers involved in retrofit work started wondering if these insulation increases might result in more roof snow than envisioned by the *old* thermal factors, mainly based upon observations of structures with *old* levels of roof insulation.

The ASCE 7-22 provisions for $C_t$ address these recent changes to roof insulation practice. Specifically, the current required insulation for *modern* ventilated roofs results in essentially no heat flow through the snow layer atop a ventilated roof "meeting the minimum requirements of the applicable energy code." As such, in relation to the potential melting of roof snow, modern ventilated roofs act thermally the same as unheated roofs. Hence, the thermal factor for the modern ventilated roof is now $C_t = 1.2$.

Similarly, increases in insulation levels for unventilated roofs resulted in revised $C_t$ values in ASCE 7-22. In this case, the expected melting of roof snow due to heat flow upward through a simple thermal model was used. As described in more detail in O'Rourke and Russell, the model consisted of a snow layer atop an insulated roof layer. The critical parameter was the location of the 32° F isotherm. Melting of roof snow only occurs in below-freezing outdoor temperatures if the 32° F isotherm is at the bottom of the snow layer. If the 32° F isotherm is within the roof insulation layer, there is no melting of roof snow due to heat flow through the roof insulation/snow layers. Simulation using the simple roof thermal model with outdoor temperatures for several locations across the U.S. resulted in the $C_t$ values for unventilated roofs shown in *Table 1*.

Notice, as one would expect, increasing roof insulation for any given ground snow load value results in less melting of roof snow and hence large $C_t$ values. Also, note that $C_t = 1.2$ for ground snow loads of 15 pounds per square foot (psf) or less for all roof R-values. In such cases, the snow layer is so thin that the 32° F isotherm is always within the

> “
> *...drainage systems for new construction are no longer allowed to discharge water onto existing roofs unless the existing roof is evaluated.*

insulation layer; hence, there is no reduction in roof snow load due to heat flow up through the roof, the same as for an unheated structure.

Finally, note that the $C_t$ values for a roof insulation value of R = 50 are close to or equal to 1.2, the unheated structure value. In these cases, the roof insulation layer is so thick that the 32° F isotherm is most often within the roof insulation layer.

## Full Capture Walls

The addition of a new higher roof structure next to an existing lower roof structure leads to leeward snow drift loads atop the existing roof. These new drift loads were likely not considered in the original design of the existing low roof structure.

To avoid the often costly and challenging retrofit of the existing roof, structural engineers frequently envision a taller than usual parapet wall atop the new roof. The purpose of the new parapet wall would be to capture the expected drift snow atop the addition before it leaves the new addition, thereby eliminating the “unexpected” drift atop the existing roof.

The ASCE 7-22 *Commentary* will provide guidance for both a *full capture* and a *partial capture wall*. By its nature, the captured drift atop the addition will be windward, as shown in the *Figure (page 16)*. As mentioned above, the initial trapping efficiency at a windward drift is 100%. To achieve full capture, the parapet wall height above the new addition, $h_o$, needs to be larger than 1.86 $h_d$, where $h_d$ is the expected drift height for the leeward drift atop the existing roof for the case of no capture wall.

The ASCE 7-22 *Commentary* will also provide relations for the expected leeward drift height for a partial capture wall with $h_o < 1.86\, h_d$. For a comparatively tall partial capture wall with $0.51 < h_o/h_d < 1.86$, the expected drift height atop the existing roof $h_d^*$ is

$$h_d^* = \sqrt{0.80\, h_d^2 - 0.23\, h_o^2} \qquad \text{(C7.7-9)}$$

using the equation number in the ASCE 7-22 *Commentary*. For a comparatively small partial capture wall with $h_o/h_d \leq 0.51$

$$h_d^* = \sqrt{h_d^2 - h_o^2} \qquad \text{(C7.7-10)}$$

As one would expect, for $h_o = 0$, equation C7.7-10 applies and yields $h_d^* = h_d$. Also, note that at $h_o = 0.51\, h_d$, both equations yield the same result of $h_d^* = 0.86\, h_d$.

## Rain Loads

In ASCE 7-16 and previous editions, there is a requirement to perform a ponding analysis, yet there was limited guidance on performing that analysis. The commentary referenced the methods in Appendix 2 of the AISC Specification (AISC 360, *Specification for Structural Steel Buildings*). However, these provisions are of limited scope, and they are currently under ballot to be removed from the AISC *Specification*. A significant change to Chapter 8 of ASCE 7-22 applies a ponding head ($d_p$) to the rain load, which provides a more consistent approach to assessing ponding. The new rain loads are based on the summation of the static head, $d_s$, hydraulic head, $d_h$, and ponding head, $d_p$, using Eqn. 8.2-1, reproduced below.

$$R = 5.2(d_s + d_h + d_p) \qquad \text{(8.2-1)}$$

The static head is equal to the depth of water on the undeflected roof up to the inlet of the secondary drainage system for structural loading (SDSL). The hydraulic head is based on hydraulic test data or calculations assuming a flow rate corresponding to a rainfall intensity equal to or greater than the 15-minute duration storm with a return period and risk category given in Table 8.2-1. The ponding head is based on structural analysis using the depth of water due to deflections of the roof subjected to unfactored rain load and the unfactored dead load.

Table 8.2-1 Design Storm Return Period by Risk Category*

| Risk Category | Design Storm Return Period |
|---|---|
| I & II | 100 years |
| III | 200 years |
| IV | 500 years |

*ASCE 7-22

Other changes to Chapter 8 include adding a requirement that the inlet to the SDSL be vertically separated from the inlet to the primary drainage system by not less than 2 inches. This allows activation of the SDSL to serve as a warning that the primary drainage system is blocked or not working. Also, drainage systems for new construction are no longer allowed to discharge water onto existing roofs unless the existing roof is evaluated. Either the existing roof can support the loads determined by Chapter 8 or be upgraded to support the new rain loads.

## Summary

This article is Part 2 of a two-part series summarizing some of the more substantive changes to the Snow and Rain provisions of ASCE 7-22. The changes to ASCE 7-22 include a more accurate estimation of the horizontal extent of windward drifts, revised thermal factors $C_t$ to account for current roof insulation and venting trends, and guidance on the design loads for snow capture walls added to this edition. A significant change to Chapter 8 is the addition of a ponding head to the rain load, which provides a more consistent approach to assess ponding.▪

References are included in the PDF version of the online article at **STRUCTUREmag.org**.

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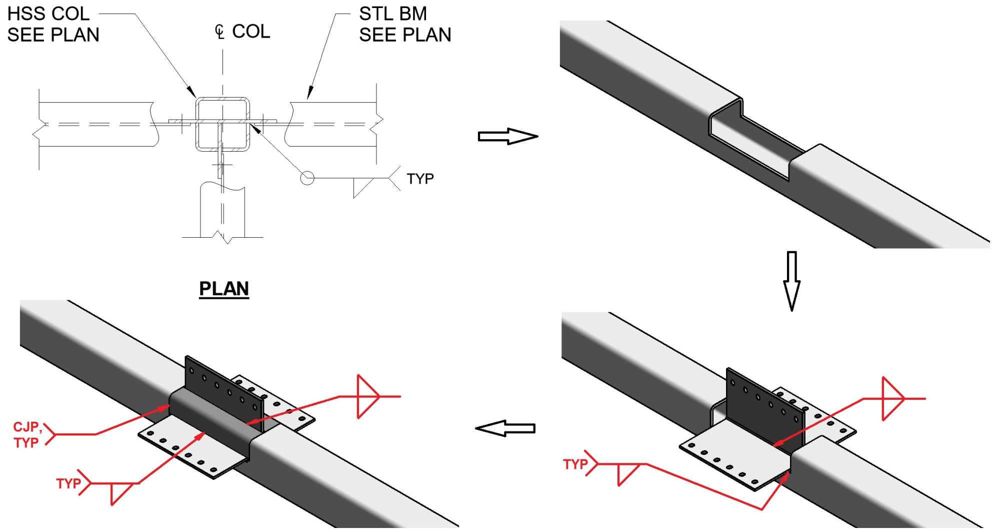
# The Hidden Cost of Copy and Paste

## Part 3

By Jason McCool, P.E.

In the previous two installments (STRUCTURE *December 2021, January 2022*), I reviewed seven bad habits I have seen from structural engineers around the country over the years while performing delegated connection design for the fabricators of their projects. Here, I offer my final three observations.

**Reuse of details that should not have been used the first time.** There is a detail I wish I could erase from every structural engineering firm's templates. AISC has been discouraging this detail for almost 20 years now, but I still see it a lot. Maybe you are one of those who have been reusing the old through-plate shear connection for simple gravity beam connections to HSS columns for most of your career, but it is time to bury it once and for all. That is an expensive detail with no real benefit unless you have beam axial loads such that engaging the opposite column wall actually helps increase the connection capacity. A shear tab will always be cheaper, simpler, and just as strong for typical shear reactions. Even in the case of beam axial loads, a through-plate is still not the best choice. The expense of this all-too-common detail gets magnified horribly when intersecting through-plates in the middle of a multi-story column are mandated. The *Figure* is what one fabricator told me they had to do to fabricate a column for this condition. I call that scenario *how-to-make-the-fabricator-hate-you*. Now compare that to simply fillet-welding shear tabs cut from flat bar onto each face of the HSS column: 6 simple fillet welds with no cutting of the column versus 8 fillet welds, 2 CJP groove welds on the HSS, and all the prep work of cutting and beveling the HSS. This is just one of many examples where small changes in our typical details can have significant impacts.




*Fabrication steps to realize the common detail shown.*

**Uncoordinated details.** When your typical detail says to "SEE ARCH" or "SEE MEP," verify that the other design team members are actually addressing what you are referencing. I have seen quite a few sets of contract drawings over the years that are circular references; the structural drawings refer to the architectural drawings for something, and the architectural drawings conveniently refer back to the structural drawings for the same item. We all feel the pressure to get our part of a project finished on time and within budget, and we rightly push back against scope creep increasing our workload with little chance of fair compensation for the added work. However, there still needs to be coordination to keep things from falling through the cracks.

**Unworkable details.** Lastly, I highly recommend that engineers do some of the design they are delegating in order to understand what problems their directions might cause for others downstream. You cannot be an expert at everything, and you may not have all the tools of the specialist, but even a basic attempt will likely change how you design your structures. For example, one school project I worked on in a low-seismic area had moment connections from W21×93 beams to W8×40 columns that were supposed to develop the full capacity of the beams. The W21 has a plastic bending moment of 921 kip-feet, while the W8 column only has a plastic bending moment of 166 kip-feet. For those curious, 1.75-inch-thick doubler plates on each side of the column web combined with 2.75-inch-thick stiffener plates could technically make the numbers work out according to one connection design program. However, that is a disagreeable prospect for most fabricators (especially since there was also a braced frame connection on the column weak axis). Another project had HSS columns with only ⅛-inch-thick walls with connections that were specified to develop the full tensile strength of the wind brace connection. If the EOR of either of those projects had actually worked through even a basic connection design on those joints at any point, they would have changed their framing sizes to work with the connection instead of against it.

To be clear, I do not think any of this is done out of the desire of any fellow engineer to cut corners on the quality of design documents. Still, we must be aware of the danger of rationalizing practices that can have unintended consequences. As professional engineers, we enjoy a great deal of respect in the building industry and overall society. People assume we have a lot of knowledge, technical skill, and judgment. But the rampant reuse of details and notes without adequate care reflects poorly on our profession, particularly among those who have to deal with the effects of us "just getting something on paper."▪

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# A NEW VISION FOR A MODERN HOSPITAL

## Seismic Excellence Through Base Isolation

*By Anthony Giammona, S.E., Michael Gemmill, S.E., and Sudharshan Navalpakkam, S.E.*

*Figure 1. The New Stanford Hospital showing patient towers cantilevering. Courtesy of Bruce Damonte.*

The New Stanford Hospital (NSH), designed by Rafael Viñoly Architects, has replaced an existing aging facility with a modern base-isolated structure at the Stanford University Campus. The new seven-story state-of-the-art Level 1 trauma center includes 368 patient rooms, 20 operating suites, imaging and radiology suites, and an emergency department. Following the latest healthcare trend, the hospital was designed to be patient-centric, with patient rooms located around the perimeter of four towers. All rooms are private, with large floor-to-ceiling windows overlooking the surrounding campus and foothills.

A three-story common podium joins the towers and provides a 4-acre green roof terrace accessible for patients and visitors to enjoy. A large column-free atrium with a 120-foot-diameter glazed dome roof is the centerpiece of the public spaces at the podium. A sky bridge and below-grade tunnel connect the new hospital to the adjacent existing hospital, and a helipad is located atop one of the patient towers. The project has several unique aspects, including a new base isolation system, a daring dome structure, extensively cantilevered floor plates, and a novel base-isolated pedestrian sky bridge.

## Structural System and Resiliency

The structural system consists of a base-isolated steel bi-directional moment frame utilizing steel box columns and Welded Unreinforced Flange (WUF-W) moment frame connections with concrete slab on metal deck diaphragms. The system was designed with seismic performance and resiliency in mind. The incorporation of base isolation substantially reduces the amount of earthquake energy imparted on the building during an earthquake. Additionally, the moment frame system was designed to be essentially elastic (R = 1) in a Maximum Credible Event (MCE). This provided the dual benefits of eliminating expected damage during a major earthquake and allowing more architectural freedom where prescriptive code requirements could be relaxed.

Additionally, the resiliency-based design enhanced protection for critical non-structural elements like building cladding, medical equipment, and Mechanical, Electrical, and Plumbing (MEP) systems needed for a functional hospital following a major seismic event. The *California Building Code* (CBC) requires hospitals to be designed to a higher standard than a typical building, and the New Stanford Hospital was designed to an even higher standard with the goals of protecting the Client's investment and ensuring that the hospital was fully functional to the community after a major earthquake.

A nonlinear response history analysis was utilized to design the structure at the Design Earthquake (DE) and MCE levels. A building model was created in ETABS to evaluate the superstructure, and a separate detailed finite element sub-model was created in SAP2000 to evaluate the base isolator connection to the superstructure. The detailed sub-model proved to be more complicated as it had to consider multiple scenarios, including the P-Delta forces resulting from earthquake displacements in the base isolator and a post-earthquake scenario where the building would need to be jacked up should an isolator bearing require replacement.

## Base Isolation

The project was one of the first in the world to use Triple Friction Pendulum isolation bearings, and extensive work was done to validate how to model and test these new bearings. The project also relied on recent research performed on similar bearings tested in a full-scale building at the E-Defense shake table near Kobe, Japan, to provide valuable modeling input and verify the real-world performance.

The bearing manufacturer, Earthquake Protection Systems, performed real-time bearing prototype testing to determine the bearing properties used in the design. Upper and lower bound isolator properties were developed to be used in the analysis to capture potential performance variability of the isolators under different loading conditions. Performing prototype testing early allowed the actual isolator properties to be used during design rather than using an assumed range which would have been more conservative and added cost. All bearings installed in the building also went through production testing to ensure their performance was within the specified limits and the production results matched the prototype results.

This testing and analysis program produced a base isolation system that allows the building to move up to 37 inches horizontally in any direction during an earthquake. A void space known as a base isolation *moat* completely surrounds the basement allowing the building to move unimpeded horizontally. Additionally, because of the geometry of the Triple Friction Pendulum bearings, the building will rise up to 3 inches vertically as it travels horizontally.

A detailed system of moat covers was provided to bridge the moat around the building to interface with the adjacent sitescape. The moat


*Figure 2. A view of the gravity-defying 120-foot-span dome skylight over the central atrium. Courtesy of Bruce Damonte.*

The project was one of the first in the world to use Triple Friction Pendulum isolation bearings and extensive work was done to validate how to model and test these new bearings.

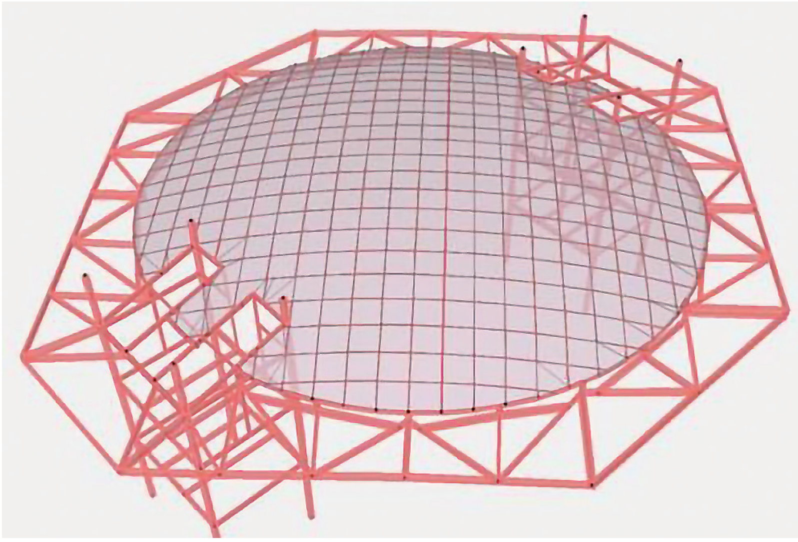


Figure 3. SAP2000 Structural Analysis model of the 120-foot-diameter dome.

covers were designed to remain operational over the range of base-isolated structure movements and support a variety of site finishes and pedestrian and vehicular traffic loads – a challenging and creative aspect of the project. The moat covers articulate to keep the moats covered before, during, and after an earthquake while still allowing the abovementioned movements. The covers also needed to match the architecture and preferably be invisible and watertight. The typical moat cover consists of a *pop-up* assembly provided by Construction Specialties that will articulate and pop up during an earthquake and return to its original at-rest position when shaking stops. The typical moat cover consists of a hinged metal pan filled with concrete and topped with other finishes to blend in with the surrounding site.

The extensive set of utilities required to serve a modern hospital connect to the building within the moat and use a system of flexible couplings and ball joints to accommodate the large differential movements between the base-isolated structure and the adjoining fixed-site during an earthquake, without interruption or damage. One of the project's successes was the early involvement of Mechanical, Electrical, and Plumbing subcontractors and their active participation in detailing and coordinating the large clearances required around MEP utilities crossing the isolation plane.

## Cantilevered Floors

An immediately noticeable aspect of the hospital architecture is the large number of cantilevers throughout the building (*Figure 1, page 22*). Over one-third of the overall building floor is cantilevered to achieve this architectural vision. In addition, the perimeter bay of all four patient towers and most of the second-floor perimeter bays are cantilevered 29 feet beyond the last column line. Each cantilever condition was different, requiring a unique structural solution to achieve the cantilever.

At the patient towers, where four stories of the perimeter framing bays are cantilevered 29 feet beyond the last column line, vertical Vierendeel frames are used at each gridline for support. This Vierendeel system, combined with the bi-axial moment frames at the tower cores, resulted in essentially every beam-column connection of the towers being moment-connected. This interconnection required the entire patient tower to be considered in an ETABS lateral analysis model when designing for gravity and seismic loading.

A mix of diagonal tension hangers at the corners and custom plate girders at the typical bays were used at the second floor to achieve the similarly long cantilevers. In particular, the double cantilevered corner conditions proved to be complicated and challenging to design. They required a second system of perpendicular cantilevers at the corner supported from the typical floor cantilever systems.

These extensive building cantilevers were temporarily shored as part of the initial steel erection. The release of shoring and overall construction sequencing, including the timing and build-out of finishes, interior construction, and exterior cladding systems, was carefully coordinated with the contractor to protect drift and movement-sensitive non-structural systems.

## Atrium Dome Structure

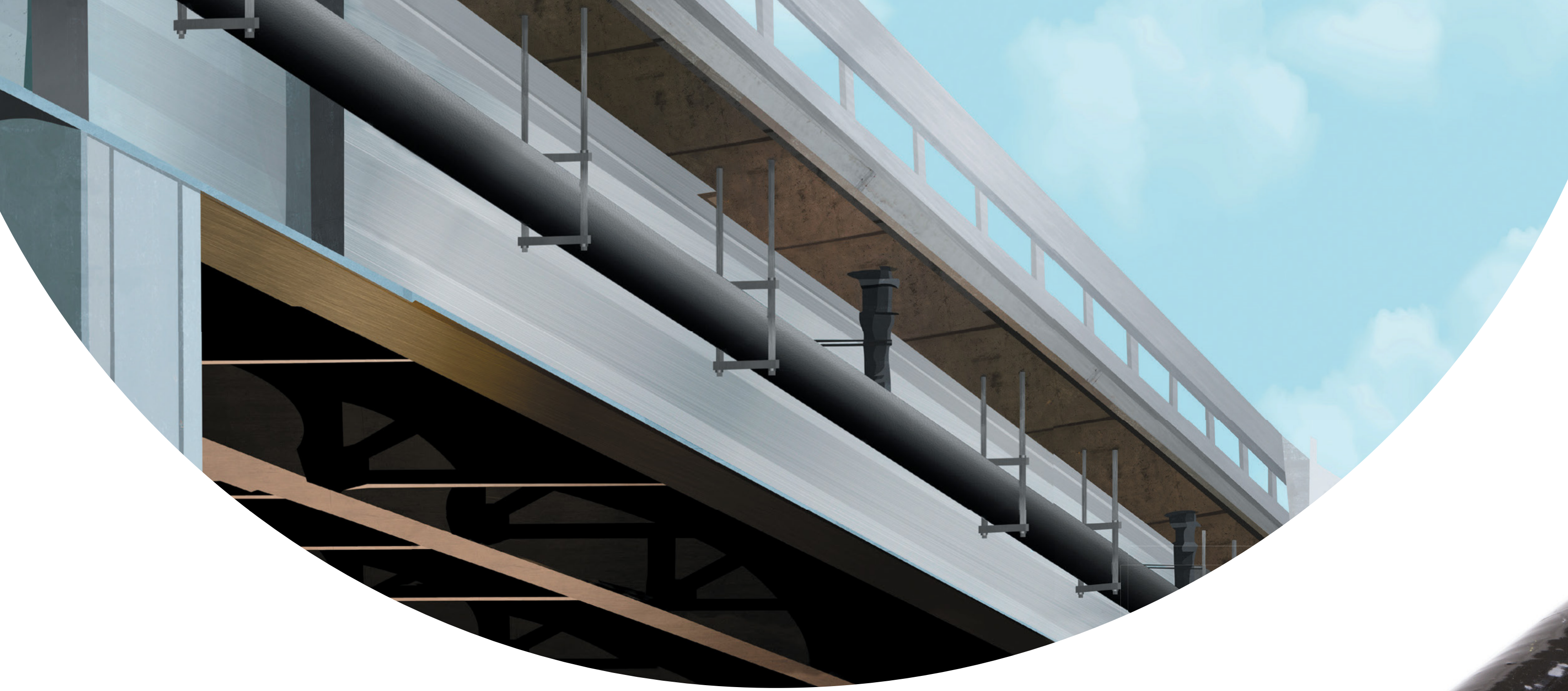
The main entrance to the NSH hospital opens into a striking 120-foot-diameter column-free atrium filled with natural light (*Figure 2, page 23*). This is a centerpiece of the hospital and is covered by a glass dome skylight to allow light to reach the occupants below. The structure of the dome is composed of a grid of 5-inch-diameter curved steel ribs surrounded by a circular tension ring at the perimeter of the dome. The ribs are located on an orthogonal grid in plan and are curved vertically to follow the dome's profile. At the center of the dome, the ribs rise a maximum of 12 feet, creating a gravity-defying thin lens assembly. A series of articulated and repeating glazing panels sit on top of the structure, forming the skylight. The analysis of the dome used the program SAP2000 (*Figure 3*) and considered construction loading, construction sequencing, and penetrations for elevator towers and tower crane leave-out panels. Designing around these discontinuities to the symmetry and regular geometry of the dome proved to be one of the significant challenges of the dome design.

The steel fabricator chose to sub-contract the dome steel fabrication to a specialized fabricator with experience in curved HSS fabrication and construction in the roller coaster and amusement theme park industry to meet the tighter construction tolerances required for the dome construction. The dome was test fit and coordinated with the glazing sub-contractor in the steel shop prior to the eventual successful erection and installation on site.

## Pedestrian Skybridge

An innovative long-span pedestrian sky bridge connects the new hospital to the existing hospital at the second floor (*Figure 4, page 26*). A first-of-its-kind structural system was developed to allow the sky bridge to be laterally supported by the new isolated hospital at one end and by the existing fixed-base hospital at the other end while accommodating up to 4 feet of differential seismic movement between the two structures without a large seismic joint. A traditional approach would have used a very large (4 feet in any direction) seismic joint between the isolated and non-isolated structures, which would have been challenging to implement architecturally and taken up valuable space. Instead, the bridge is base-isolated and uses an articulated three-pin *dogleg* configuration in plan to convert the large seismic movements of the hospital into small rotations at each of the three joints.

This approach effectively transformed the irregular shape of the bridge into a structural benefit to the project. The resulting joint size is only 3 inches at each of the three articulating joints, yet the bridge can accommodate the full displacement capacity of the hospital isolation bearings. In addition, the base of the bridge is supported by custom and first-of-this-size Tension-Capable cross linear slider isolation





bearings, also by Earthquake Protection Systems. These bearings simultaneously resist overturning in the bridge while allowing the bridge to move with the hospital.

## Vibration Analysis

The project was required to meet stringent vibration criteria for occupant comfort and imaging performance. Extensive vibration analysis was performed to evaluate nearly all floor areas, including the significant cantilevers described above, and ensure the floor systems met the criteria. Typical framing bays were evaluated using the program FloorVibe, but irregular and cantilever conditions required a more advanced approach. In those cases, a SAP2000 model of the structure was developed, and custom vertical time history forcing functions were applied to the model to emulate the footfall of one or more walkers on the floor. Careful post-processing of the velocity and acceleration data allowed comparison to the project vibration criteria.

The MRI suites posed a unique challenge as they are typically located on a slab on grade to take advantage of the inherent vibration performance of an at-grade condition. However, this was not an option due to the use of base isolation and the presence of the crawlspace below the isolation system. Therefore, the imaging suites had to be located on elevated floors. A stiff 3-foot-deep steel girder system was utilized to provide a steady platform for the MRIs. This system was ultimately performance verified to provide performance up to 500 m-in/sec to allow for future ultra-high-performance MRIs.

## Summary

After 12 years of design, review by the California Office of Statewide Health Planning and Development (OSPHD), Peer Review, and construction, the state-of-the-art hospital opened its doors in October 2019. The base-isolated hospital represents the latest in hospital seismic performance.▪

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*Sudharshan Navalpakkam is a Vice President (snavalpakkam@nyase.com).*

## Project Team

**Owner**: Stanford Health Care

**Structural Engineer**: Nabih Youssef Associates

**Architect**: Rafael Viñoly Architects

**Contractor**: Clark Construction and McCarthy Building Companies Joint Venture

**Base Isolator Supplier**: Earthquake Protection Systems

**Moat Cover Supplier**: Construction Specialties

*Figure 4. Skybridge connecting the new base-isolated hospital to an existing building. Courtesy of Bruce Damonte.*

# Improved Seismic Bracing for Steel Buildings

*By Leo Panian, S.E., Gina Beretta, S.E., and Isaac Williams, C.E.*

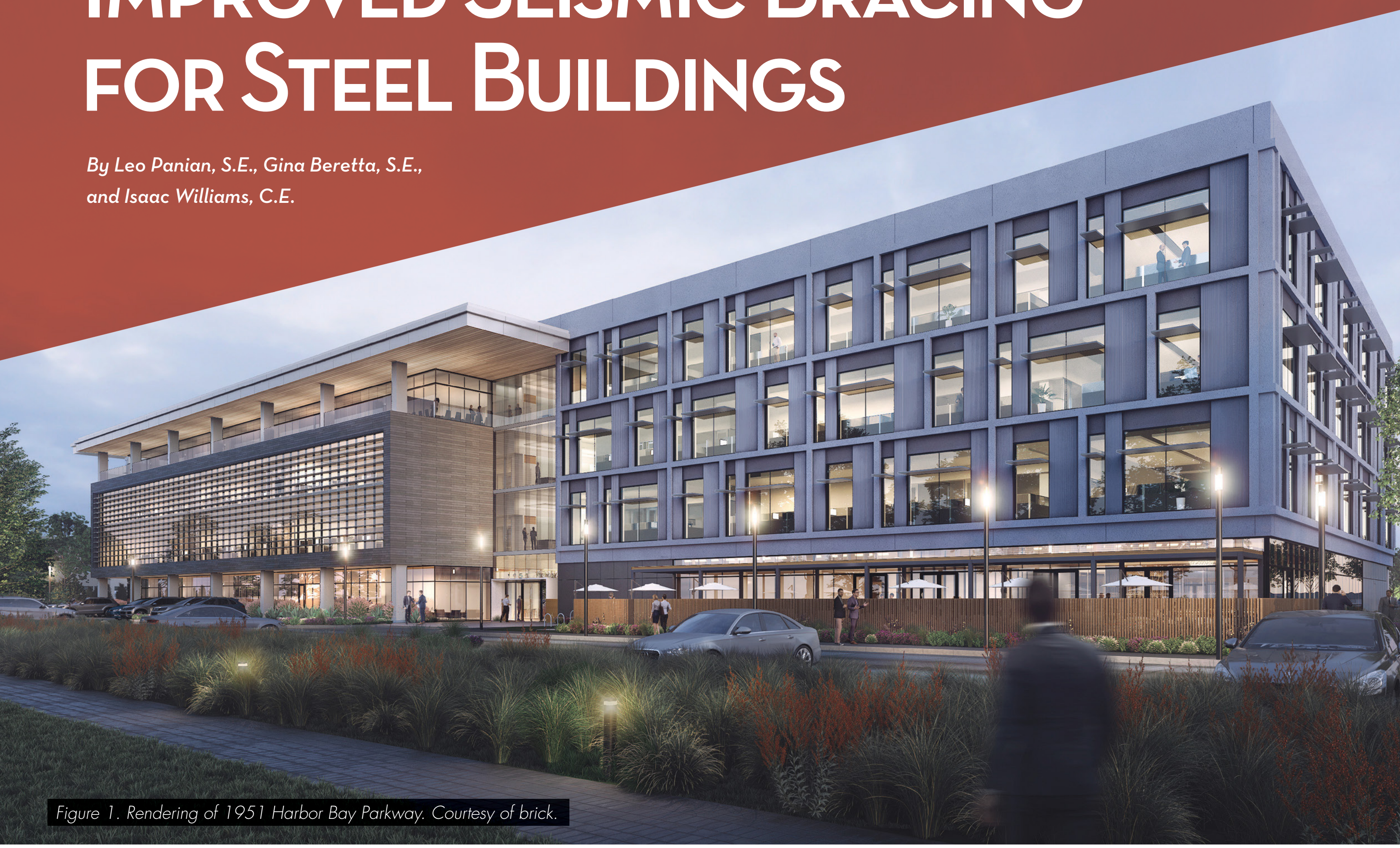

*Figure 1. Rendering of 1951 Harbor Bay Parkway. Courtesy of brick.*

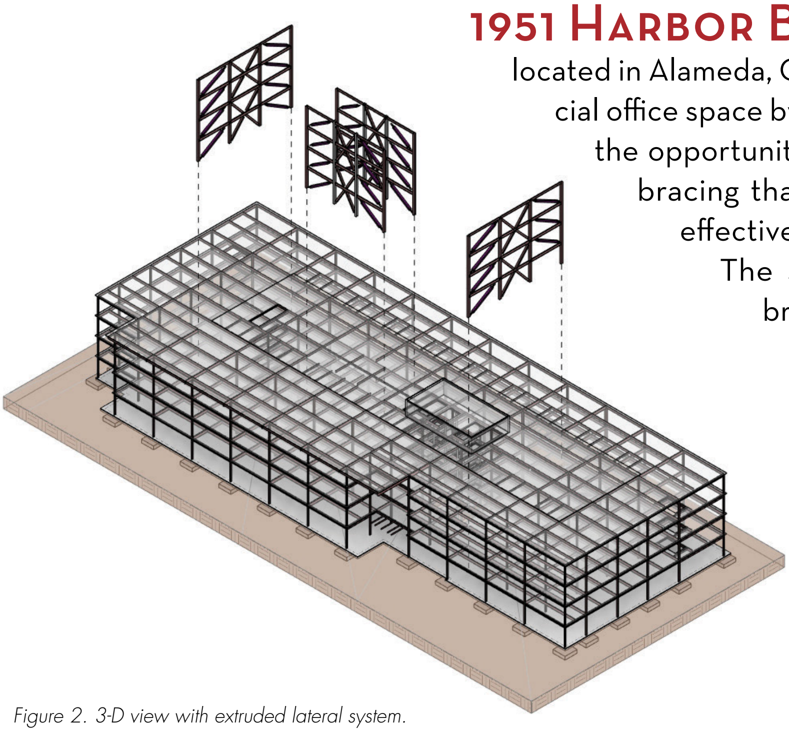

*Figure 2. 3-D view with extruded lateral system.*

**1951 Harbor Bay Parkway,** a new building (*Figure 1*) located in Alameda, CA, was developed privately for use as commercial office space by a life-sciences company. The project provided the opportunity to utilize an innovative approach for seismic bracing that provides improved performance and cost-effectiveness over conventional braced-frame systems. The system uses concentric buckling-restrained braces (BRBs) in conjunction with a vertical mast or strong-back to reduce drift, eliminate weak stories, and increase redundancy. The yielding BRBs work in tandem with an elastic mast frame to create controlled rocking behavior that provides more resiliency and improved protection for the building frame, cladding, and interior construction.

*continued on next page*

The four-story structure measures 384 by 141 feet in plan and provides approximately 220,000 square feet of total floor area. The ground story measures 16 feet tall, while the remaining stories are 14½ feet tall. A central core area provides accommodation for vertical circulation, restrooms, and utility spaces, allowing for flexible programming for the office areas. The regular column grid, measuring 32 by 25 feet, was laid out around the central core area to provide efficient span arrangements and floor assemblies. Floor framing consists of wide-flange beams supporting concrete slab on metal decking. *Figure 2 (page 29)* depicts the structural layout and highlights the lateral support system.

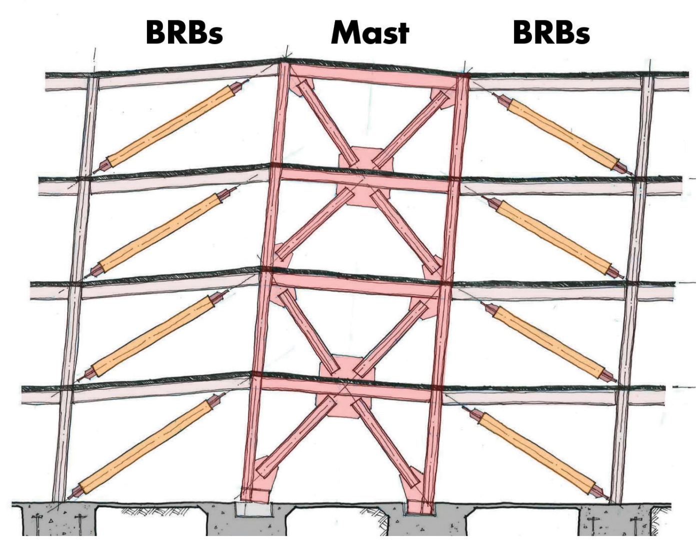


Figure 3. Mast frame rocking.

## System Overview

Special concentric braced frames (SCBFs) are typically among the most efficient solutions for resisting lateral loads for midrise steel structures in highly seismic areas. Incorporating BRBs is an effective way to improve the performance and reliability of these systems. The ductility and controlled response of the BRBs allows the structure to be designed for reduced seismic loads, making them a cost-effective alternative to conventional braces. In typical applications, braces would be arranged in a stacked chevron configuration to provide flexibility for locating window and door openings. The braces would also be located at numerous locations in each frame line at each story to provide redundancy in the system.

There are, however, shortcomings to this approach for both performance and economy. Under high seismic loads, these systems can experience large drifts that are concentrated at certain floors. This characteristic weak-story response increases the likelihood of localized damage needing repair and limits the ability of the building to function following a large earthquake. Ultimately, the full benefit of the BRBs is not effectively utilized since the ductility of the frame is limited by just a few critically loaded members.

Critical building elements, such as the structural frame, exterior cladding, interior construction, and elevators, are susceptible to damage resulting from concentrated story drift, which can hamper functional recovery following an earthquake. Improving resilience is about limiting the overall drift of the system and distributing that movement uniformly over the height of the structure. For SCBFs, this means counteracting the tendency for weak-story response and limiting the concentration of damage.

The key is to design the frame for rocking, rather than racking, under inelastic response. The rocking mechanism is achieved by introducing a stiff elastic spine into the frame capable of distributing forces between stories to create a more uniform drift profile.

The spine, sometimes referred to as a *mast* or *strong-back*, is essentially a vertical truss extending up the structure's height and interconnecting the BRBs to form an integral framework. The vertical truss forming the mast is made of conventional steel members and is designed to pivot or rock at its base. The mast frame occupies the same footprint as a conventional frame but uses far fewer BRB members. The mast effectively forces all of the BRBs in the system to work together to resist movement at any story, which fully mobilizes the BRB elements' deformation capacity and increases the system's inherent redundancy.

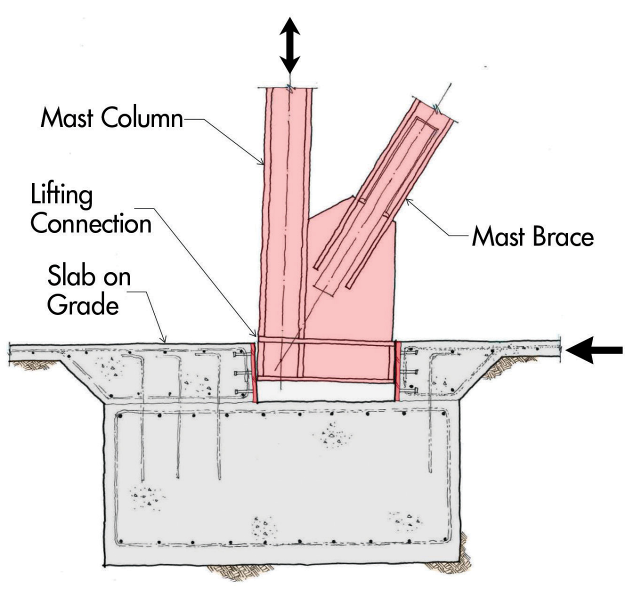


Figure 4. Mast frame column base connection detail.

## Case Study

At 1951 Harbor Bay Parkway, several lateral-load resisting systems were evaluated for cost and performance. Moment-resisting frames provided maximum flexibility for space planning but were more costly and offered less seismic protection than braced-frame alternatives. Conventional SCBFs designed and proportioned according to ASCE 7, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, and NIST guidelines were considered. Still, they were discarded in favor of an SCBF system using BRBs, along with a mast frame. The mast frame system offered an innovative approach to provide improved performance without additional cost.

The lateral frames comprised three bays of varying width, with the central bay being utilized as the mast frame when applicable. The outer bays varied between 23 and 26 feet wide, while the corresponding mast bays varied between 18 and 25 feet wide. Ultimately, the mast frame was comprised of 650 kip BRBs, W14x233 mast frame braces, and W14X283 columns.

Preliminary member sizing of the mast frame during schematic design was based on the assumption that the BRBs would resist the entire design lateral force, and the mast would be sized relative to the BRBs and their yield strengths. The vertical trusses that form the masts were designed to remain elastic and were proportioned using overstrength factors, similar to the design approach for columns. This design approach resulted in a costly and unnecessarily stiff structure. During initial

Figure 5. In-progress construction.

analyses, the mast's stiffness was identified as the primary influence on the structural response, so softening measures that allowed the structure to drift more were further investigated. The mast stiffness was predominately controlled by the footprint of the central bay relative to the outer bays of the lateral frames. As such, the stiffness was tied to the architectural programming to some degree, and softening had to be achieved without changing the column layout.

The mast frame's rocking behavior (*Figure 3*) was key to the lateral system's expected response, as it engaged the BRBs and provided the desired mode shaping. The mast frame base connections to the foundation elements are detailed to facilitate this rocking by permitting the base of the columns to uplift within a recessed pocket in the spread footing (*Figure 4*). As noted, the BRBs were initially sized to resist the entire design lateral force. Eventually, the mast braces were explicitly assumed to resist a portion of the design lateral force, allowing an approximate 30% reduction to the typical BRB yield strength. Even with the rocking mast in place, the mast frame still carried upwards of 65% of the design lateral force. As the yielding elements in the system, the BRBs still defined the seismic design criteria.

The final mast frame design was established through iterative equivalent lateral force (ELF) analyses with the member demands scaled by the appropriate overstrength factor, when applicable. The design was further validated through non-linear studies.

## Results

The performance of the system scheme was evaluated using different analytical approaches. In addition to the standard equivalent lateral force (ELF) and response spectrum analysis (RSA) methods, nonlinear static analyses and dynamic shaking simulations were used to gauge performance and validate design assumptions. Cost estimates were made by tracking steel tonnage and BRB quantities.

The suite of results studied indicated that the mast frame produces lower maximum story drifts and displacements than a conventional SCBF by up to 50%. Perhaps most importantly, the mast frame provided an essentially uniform drift profile without any major drift concentrations that might be typically observed in a more conventional frame. The dynamic analyses of the system showed that the mast frame was far less sensitive to variation in ground motions, with coefficients of variation at approximately 50% of that of conventional SCBF across results of interest for the suite of ground motions considered. In addition, the mast frame results showed improved utilization of the adjacent BRBs, while also providing more control of the BRB strain. Further studies of the redundancy of the frames were undertaken, where BRBs were removed from the analyses, and the mast frame results continued to display the previously listed advantages. In most cases, the advantages displayed were amplified when investigating the redundancy, with the mast performing as intended and redistributing lateral forces throughout the system.

These results were consistent with a previous case study, published in a March 2017 STRUCTURE article authored by Leo Panian, S.E., for a similar BRB mast system used in a four-story commercial building in nearby Berkeley, CA.

## Key Takeaways

For the 1951 Harbor Bay Parkway project, the mast frame system required additional steel tonnage to achieve the design intent. The added cost was offset mainly by the overall reduction in the quantity of BRBs. The mast braces, beams, and columns were the main driver of the additional tonnage, as they had to be designed to transmit the forces that resulted from the yielding BRBs. The total tonnage of steel framing for the entire structure was approximately 12 psf, further validating the choice in lateral system as an economical one.

Constructability was a concern throughout the design process for a variety of reasons. The heavy steel elements and the atypical system were primary causes of concern. Still, collaboration with trade partners regarding the erection sequence and the use of typical gusset plates helped alleviate these issues (*Figure 5*). In addition, the heavy steel element connections were designed and detailed with small construction tolerances that could have been preemptively adjusted to allow more flexibility throughout the erection process.

While other jurisdictions may vary, the approval process for this lateral system was relatively seamless and did not require peer review, despite stepping outside conventional BRB frame design methods. Through more industry and academic research, prescriptive approaches may be developed to facilitate the approval process further and make the future implementation of similar systems more commonplace.▪

References are included in the PDF version of the online article at **STRUCTUREmag.org**.

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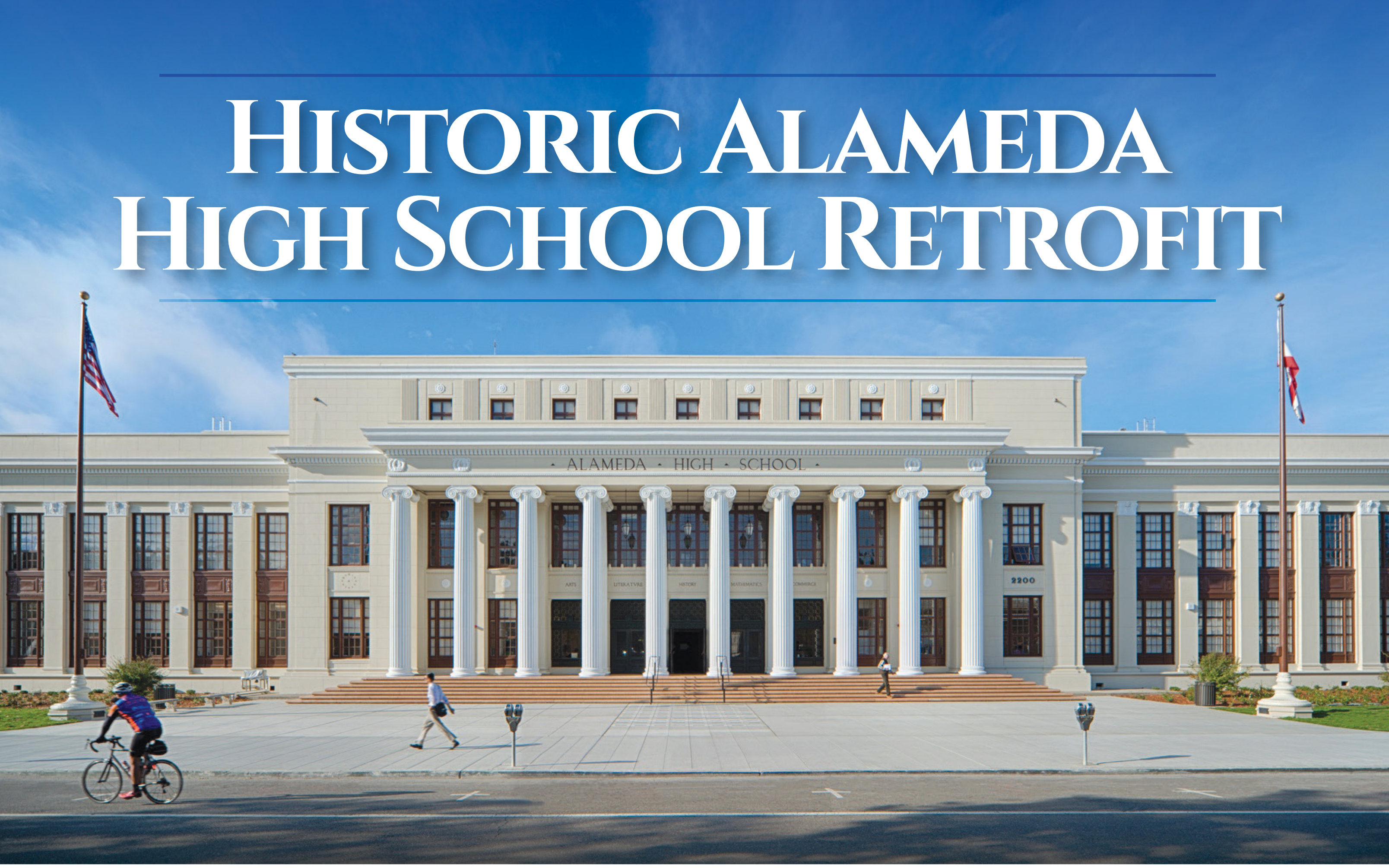
### Project Team

**Structural Engineer**: Tipping Structural Engineers, Berkeley, CA
**Architect**: brick., Oakland, CA
**General Contractor**: Pankow Builders, Oakland, CA

# Historic Alameda High School Retrofit




## Part 2: Preserving Historic Value, Providing Modern Seismic Safety

*By Nik Blanchette, P.E., Steve Heyne, S.E., and Chris Warner, S.E.*

A California public school campus constructed in 1924, partially retro fitted in 1936, recognized as a historic place in 1977, vacated shortly after that in 1978, partially retrofitted again in 1989, shuttered in 2012, was brought back to life in 2018. For a brief history of the Historic Alameda High School campus and the state government regulations setting seismic safety standards for public school buildings in California, see Part 1 of this article series in the January 2022 issue of STRUCTURE.

Reductions in seismic demands are permissible to reduce the impact on the aesthetics and recognize the shorter remaining useful life for older buildings.

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The 1920s neoclassical-style buildings with ornate design elements copied from ancient Rome are truly beautiful and benefit the community of Alameda. However, it is not of great surprise that such historic concrete buildings were not designed and detailed with seismic performance in mind. Maintaining the regal aesthetics while upgrading to current code structural performance requirements proved challenging. Structural challenges included soil liquefaction; lightly reinforced, nonductile concrete walls; the absence of collectors; and inadequate out-of-plane concrete wall anchorage load paths. Any one of these deficiencies could lead to significant damage or possible collapse during a seismic event.

## Analysis Options

Selecting a retrofit scheme starts with evaluating options for code-compliant analysis. The least onerous analysis approach is a voluntary retrofit using Sections 317.11 and 319.12 of the *California Existing Building Code* (CBC) (based on the *International Existing Building Code*). This option provides the Structural Engineer flexibility to choose which elements warrant retrofit and offer the best improvement for a given budget. However, a potentially overlooked requirement of this option is that a *more dangerous* condition must not be created as a result. For example, adding a shear wall is typically a significant improvement for most buildings. Still, the resulting change to the collector demand could lead to overstressing a gravity beam that also serves as a collector.

Another approach, since the Alameda High School buildings are historic, could be the *California Historic Building Code* (CHBC). This code is intended to allow the structural engineer to improve seismic safety while not compromising architectural heritage. Reductions in seismic demands are permissible to reduce the impact on the aesthetics and recognize the shorter remaining useful life for older buildings. Seismic design force reductions are on the order of 25% to 50%, and redundancy and overstrength factors used in the design of new buildings can also be ignored. The CHBC permits the structural engineer to exercise judgment in deciding which elements should

*Looking down Central Avenue.*


THINK SAFE

Cast Connex HSC brace connection.

be upgraded to improve seismic performance. However, this option is not permitted for California public school projects, as the seismic performance needs to be equivalent to the current building code for new structures.

This project was largely made possible by partial financial reimbursement from the State through the Seismic Mitigation Program (SMP; see Part 1 of this article series). To qualify for SMP funding, analysis options are limited to a full retrofit at current building code seismic levels in accordance with ASCE 41 *Seismic Evaluation and Retrofit of Existing Structures* or ASCE 7 *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*. This involves analyzing all the building components of the seismic force-resisting system and retrofitting them as required to conform with one of these two current codes. Typically, ASCE 7 is more difficult to employ because it is often impossible to comply with the prescriptive detailing requirements and choose a code-defined lateral system. Therefore, ASCE 41 is often used with existing buildings that do not comply with current code prescriptive seismic detailing requirements since the document was written primarily for existing buildings. However, an ASCE 7 analysis is typically simpler compared to ASCE 41. In the end, ASCE 7 was chosen for this project because the existing components of the lateral system were too deficient to be of use (or absent entirely), and a full new lateral system was required.

## Ground Improvement

In addition to structural issues above grade, the soil under the building posed a seismic hazard. The uppermost ten feet of soil was loose, granular, and located below the near-surface water table, creating favorable conditions for liquefaction to occur. Liquefaction is the sudden loss of soil strength during an earthquake resulting from pore water pressure increase; seismic waves turn the soil particles and surrounding water into a liquid solution, so foundation support is significantly reduced or lost. A mat slab or deep pier and grade beam foundation are good candidates for new construction at a liquefaction site; however, these are not feasible for large existing buildings.

Instead of designing a foundation to accommodate liquefaction, the soil properties were improved using compaction grouting. This process densifies loose sandy soil by injecting high-pressure low-viscosity

> "Helical piles are modular elements composed of relatively slender steel shafts with larger diameter helical plates on the lead end.

Top of battered helical piles at pile cap.

> It was challenging to integrate the new collectors with the existing gravity framing because they needed to occupy the same space to hide them from view and maintain historic visuals as much as possible.

cement grout columns below grade on a regular grid. The pressure-injected grout displaces and compacts the adjacent soil. Grouting was performed under and around footings in a triangular grid spaced at three feet on-center in each direction to a depth of ten feet and over a width extending five feet beyond the footprint of all footings. Careful monitoring was required to avoid ground heave, which would inadvertently lift the footings and damage the buildings. This ground improvement technique is expected to make the soil supporting the footings perform as if there were no liquefaction potential at all.

## Braced Frames

By default, the existing seismic force-resisting system (SFRS) was concrete shear walls. The walls did not appear to be designed and detailed for in-plane seismic forces and, of course, lacked prescriptive detailing requirements. Retrofitting the walls by adding more rebar and more concrete was a non-starter since window openings would need to be infilled, affecting the historic architecture.

A new steel SFRS was another option to evaluate. Steel offers an efficient, compact, and modular solution. Relatively lightweight elements can be fabricated offsite then installed through small openings in the building. Comparing braced frames and moment frames, braced frames were selected due to their greater stiffness, which was required to limit in-plane drift of the existing concrete walls. Drift was limited to prevent yielding of the existing reinforcing steel in the concrete walls supporting floor and roof gravity loads. Braced frames also integrate better with wood-framed floors since the beam bracing requirements for moment frames are difficult to resolve into wood framing. Architecturally, it was decided not to encase the frames in finishes, so Cast Connex High-Strength Connectors were utilized at brace to gusset plate connections instead of traditional welded or bolted connections. The connectors are more visually appealing and allow for field bolted connections, which was more cost-effective to install.

## Helical Piles

Three-story-tall braced frames supporting concrete wall construction result in sizeable seismic overturning demands. High demands combined with the need to fit new foundations within the footprint of an existing building, with pad footings throughout, required deep foundations. Due to limited access and vertical clearances within the existing building, helical piles were chosen for the new foundations that were installed under the braced frames. Helical piles are modular elements composed of relatively slender steel shafts with larger diameter helical plates on the lead end; essentially, a large screw torqued into the soil. These piles come in short lengths and can be installed with a hydraulic head mounted on a Bobcat, perfect for working inside an existing building. After one shaft is driven flush with the ground or slab, another shaft is added using a bolted coupler. Helical piles are stiff along their axis but weak perpendicular to the shaft. Therefore, vertical piles were used to support vertical forces but, to resolve base shear at the braced frames, battered piles at an angle of thirty degrees to vertical were used.

*HSS collector recessed into the top of floor framing.*

The pile manufacturer was unsure if the piles could be driven through the grout injection columns without damage and/or refusal. Therefore, the piles were installed first. After grout injection was performed, the foundations were excavated, taking care not to damage the embedded pile shafts. This was an unusual sequence but turned out to be successful.

## Collectors

Existing concrete beam reinforcing was already fully utilized for gravity loads, so new collectors had to be introduced. It was challenging to integrate the new collectors with the existing gravity framing because they needed to occupy the same space to hide them from view and maintain historic visuals as much as possible. Braced HSS beams offered a small profile, which was hidden in the top of notched floor joist ends. Roof collectors were hidden in the attic. This resulted in only braced-frame elements being exposed in a limited number of rooms.

New steel collectors had to pass through perpendicular concrete beams in many locations. An elaborate set of condition-specific details was developed, using steel rods to limit the extent of concrete removal. Field welding was avoided in most cases, favoring a threaded rod and nut solution.

## Diaphragms and Wall Anchorage

Upgrades to floor and roof diaphragms were required for both in-plane structure shears and out-of-plane wall forces. In-plane structure shears were quite high due to the heavy concrete walls and the desire to maximize the spacing of braced frames to limit the quantity and cost of new material. Wood structural sheathing panels were added on top of the existing diagonally or straight sheathed diaphragm to provide strength and ductility. The existing nominal one-inch-thick sheathing was utilized as blocking for the new panels; however, in a few locations, the existing sheathing had to be removed entirely to allow for the installation of a new high-load wood diaphragm.

A common deficiency of buildings with heavy walls and flexible diaphragms is out-of-plane wall anchorage strength, and these buildings were no exception. Light diaphragms brace heavy walls, and the result is typically catastrophic in a large seismic event. Insufficient wall anchorage can lead to loss of gravity support of floor/roof framing, as documented in the 1971 San Fernando and 1994 Northridge earthquakes. Horizontal hold-downs were added at approximately five feet on-center throughout the buildings to resist out-of-plane forces. Out-of-plane anchorage forces must be developed into the diaphragm, typically via attachment to perpendicular joist framing. Where the hold-downs did not attach to existing joists, blocking and light gauge steel straps were added to connect to a sub-diaphragm.

## Success

Through the efforts of all stakeholders, the regal beauty of the Historic Alameda High School was preserved and has been extended to serve a new generation of students. Through careful coordination with the architect, neither aesthetics nor structural performance was sacrificed to reinvigorate the nearly century-old campus that sits just a few miles from the Hayward fault. Some (engineers) might even say it looks better now with the subtly exposed braced frames peeking through the historic framework.▪

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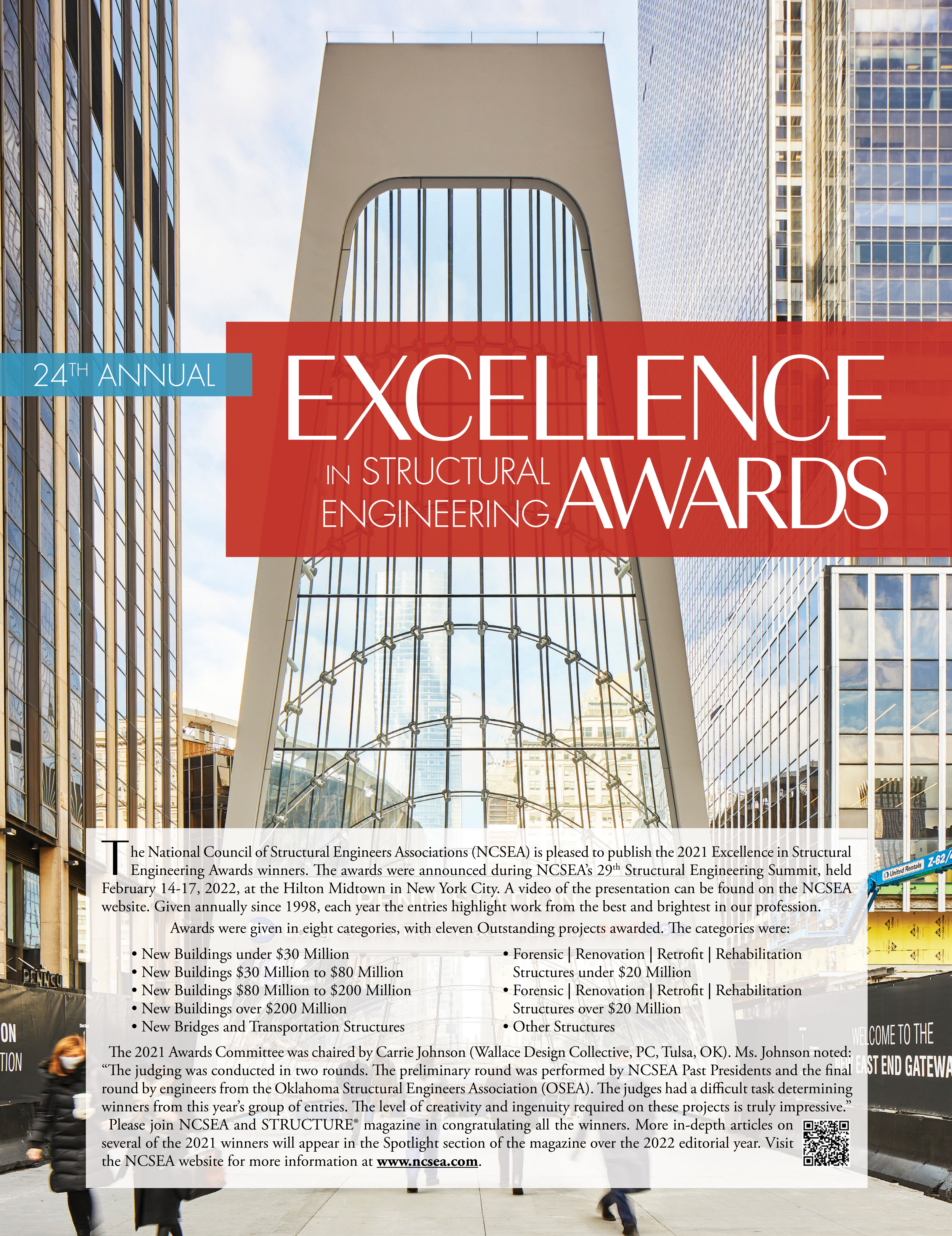


24TH ANNUAL

# EXCELLENCE IN STRUCTURAL ENGINEERING AWARDS

The National Council of Structural Engineers Associations (NCSEA) is pleased to publish the 2021 Excellence in Structural Engineering Awards winners. The awards were announced during NCSEA's 29th Structural Engineering Summit, held February 14-17, 2022, at the Hilton Midtown in New York City. A video of the presentation can be found on the NCSEA website. Given annually since 1998, each year the entries highlight work from the best and brightest in our profession.

Awards were given in eight categories, with eleven Outstanding projects awarded. The categories were:

- New Buildings under $30 Million
- New Buildings $30 Million to $80 Million
- New Buildings $80 Million to $200 Million
- New Buildings over $200 Million
- New Bridges and Transportation Structures
- Forensic | Renovation | Retrofit | Rehabilitation Structures under $20 Million
- Forensic | Renovation | Retrofit | Rehabilitation Structures over $20 Million
- Other Structures

The 2021 Awards Committee was chaired by Carrie Johnson (Wallace Design Collective, PC, Tulsa, OK). Ms. Johnson noted: "The judging was conducted in two rounds. The preliminary round was performed by NCSEA Past Presidents and the final round by engineers from the Oklahoma Structural Engineers Association (OSEA). The judges had a difficult task determining winners from this year's group of entries. The level of creativity and ingenuity required on these projects is truly impressive."

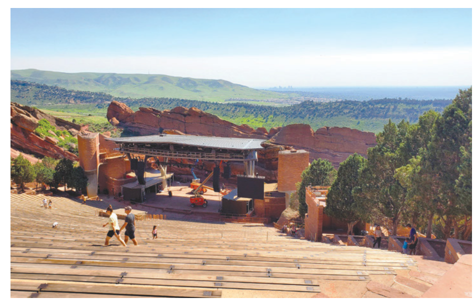
Please join NCSEA and STRUCTURE® magazine in congratulating all the winners. More in-depth articles on several of the 2021 winners will appear in the Spotlight section of the magazine over the 2022 editorial year. Visit the NCSEA website for more information at **www.ncsea.com**.

## Category 1: New Buildings under $30 Million

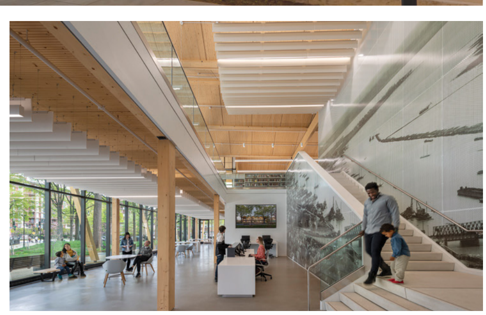

# RED ROCKS AMPHITHEATER STAGE ROOF REPLACEMENT

*Morrison, CO | Martin/Martin, Inc.*

Red Rocks Amphitheater, one of the world's premier concert venues, replaced its 30-year-old stage roof with a new cantilevered structure, incorporating increased rigging capacity, enhanced safety features, and a design more complimentary to its surroundings. Widening the field of view for both performers and audience members meant reducing column count from 12 to 4. The new roof utilizes moment frames in each direction, with primary and secondary trusses participating in the lateral system for both strength and stiffness. The newly enhanced design includes movable rail-supported rigging beams for configuration options and a walking surface that allows riggers to work unencumbered by fall restraint harnesses.



## Category 1: New Buildings under $30 Million

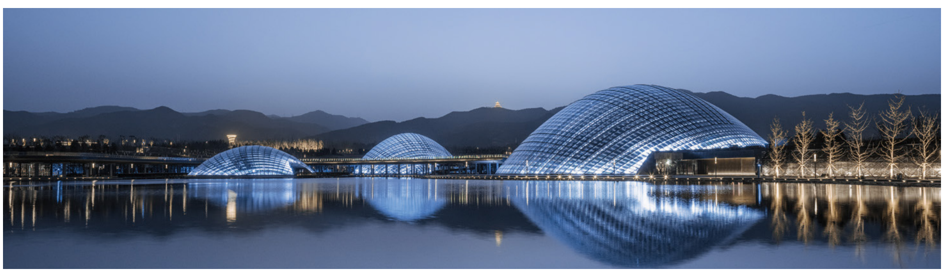



# DC SOUTHWEST LIBRARY

*Washington, D.C. | StructureCraft*

The new DC Southwest Library brings a sustainable and unique design to replace the previous outdated library. Needed by the community was a Library that could provide State-of-the-Art Technology, a place for training education and workforce development, and vitality. Comprised of a mass timber structure, the Library displays a world-first timber folded plate roof using Dowel Laminated Timber. Glulam beams and columns with detailed timber-to-steel connections support the steel and timber lateral system. The Library achieved LEED Platinum Status, implementing sustainable strategies such as local material, solar panels on the green roof, and timber throughout.

OUTSTANDING PROJECT



## TAIYUAN BOTANICAL GARDEN DOMES

*Taiyuan, Shanxi Province, China | StructureCraft*

The Taiyuan Botanical Garden complex in Taiyuan, China, features 3 paraboloid domes ranging from 43 to 88 meters in diameter and 12 to 30 meters in height. To the designer's knowledge, the largest of the three domes is the world's longest clear-span timber gridshell (non-triangulated). All three gridshells comprise light, doubly-curved glulam beams arranged in two or three crossing layers. The project pushes the boundaries of structural engineering, materiality, and construction. A diagrid of almost invisible cables was inserted below the gridshell surface, which stabilized and organized the buckling modes, to solve the inherent issue of local buckling instability resulting from the non-triangulated surface.

CATEGORY 3: NEW BUILDINGS $80 MILLION TO $200 MILLION

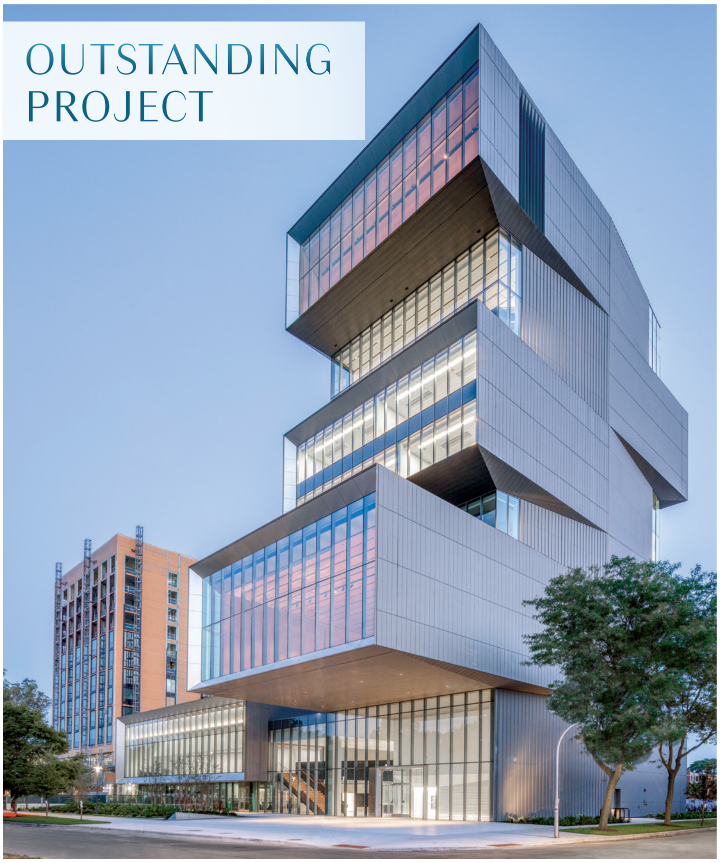
## STANFORD CENTER FOR ACADEMIC MEDICINE

*Stanford, CA | HOK*

Stanford School of Medicine's new Center for Academic Medicine is a U-shaped, four-story building encompassing 170,000 square feet above a three-level subterranean parking structure. Sitting adjacent to the Stanford University Arboretum, the site presented unique architectural opportunities, such as the building's chosen U-shape to maximize daylight, views, and access to the arboretum itself. Structural features include using the latest design advances to meet seismic demands surpassing code-required performance, carefully coordinated buckling restrained braced frames, cantilevered roof trusses, two pedestrian bridges, and a heavily landscaped at-grade level. The structural design supports the architectural vision and meets Standford's rigorous Seismic Safety performance objectives.

OUTSTANDING PROJECT

## Category 3: New Buildings $80 Million to $200 Million




# DAVID RUBENSTEIN FORUM, UNIVERSITY OF CHICAGO

*Chicago, IL | LERA Consulting Structural Engineers*

The David Rubenstein Forum at the University of Chicago is a new center for intellectual exchange, scholarly collaboration, and special events. The 97,000-square-foot facility consists of a 2-story podium and a 10-story tower of stacked "neighborhoods" with a zinc and glass exterior. These stacked neighborhoods posed complex structural challenges. Incorporating post-tensioned concrete proved crucial to achieving long spans, cantilevers, and column-free spaces. A 285-seat auditorium sits above the podium. A large multipurpose space on the 2nd floor can accommodate groups of up to 600 people. The top of the tower features a flat-floor multipurpose space that can accommodate over 100 people.

## Category 4: New Buildings over $200 Million

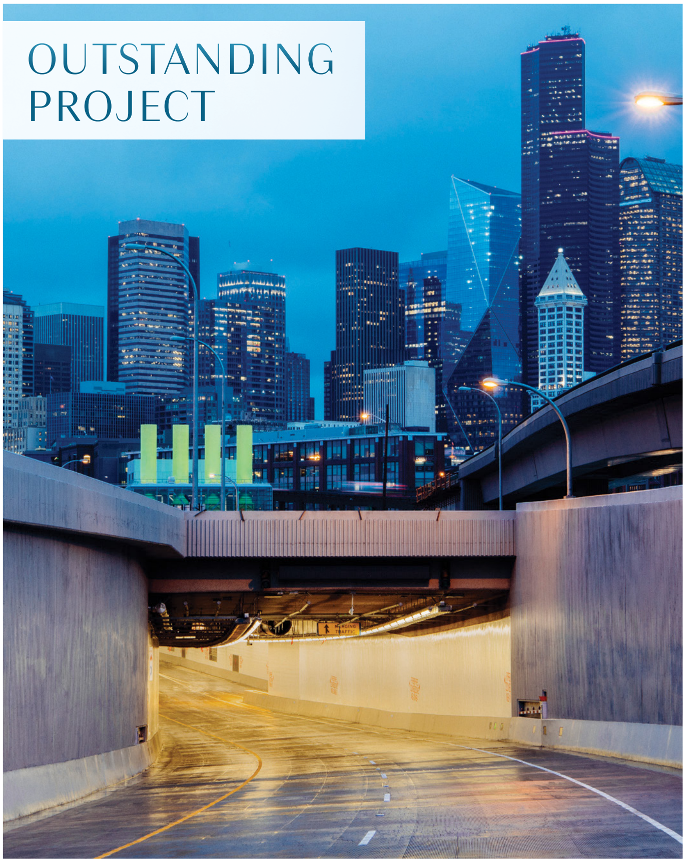

# RAINIER SQUARE

*Seattle, WA | Magnusson Klemencic Associates*

An industry-transforming, non-proprietary structural system was the key to realizing Rainier Square (RSQ), Seattle's second-tallest building. The first-ever use of SpeedCore allowed construction to proceed 43% faster than traditional methods, shaving an astonishing 10 months off the original 32-month schedule. SpeedCore is a modular, prefabricated, composite shear wall system developed through 10 years of collaborative research. In addition, RSQ includes other extraordinary engineering feats, like a jaw-dropping, 100-foot-deep earth retention system supporting the adjacent 40-story Rainier Tower; a Bi-directional, Tuned Liquid Mass Damper (one of only two in the world); and the world's longest telescoping Building Maintenance Unit.

## Category 5: New Bridges or Transportation Structures

OUTSTANDING PROJECT



# STATE ROUTE 99 ALASKAN WAY VIADUCT REPLACEMENT PROGRAM

*Seattle, WA | HNTB Corporation*

The largest soft-ground bored tunnel in North America, the 2-mile SR 99 Tunnel is one of the most significant infrastructure projects in the U.S. Stacked, 32-foot-wide roadways carry two southbound lanes atop two northbound lanes, with shoulders. The tunnel features state-of-the-art structural design, fire detection, fire suppression and ventilation systems, and an intelligent transportation system, all contained within a tunnel lining designed to withstand a 2,500-year return period earthquake. The tunnel is designed as one of the safest tunnels worldwide. It has transformed Seattle's waterfront, setting a new bar for creative tunneling solutions under densely populated cities.

## Category 6: Forensic/Renovation/Retrofit/Rehabilitation Structures under $20 Million

OUTSTANDING PROJECT


# ALBERTA BAIR THEATER

*Billings, MT | Cushing Terrell*

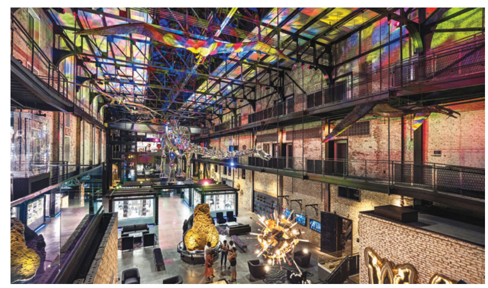
The historic Alberta Bair Theater renovation was an exercise in forensic investigation, creative detailing and problem-solving, and construction coordination. Lack of original drawings, combined with a previous remodel, required extensive field research to document existing construction and load paths. Close coordination during construction was necessary to validate designs and deliver final details. Creative structural solutions were necessary to triple the technical rigging capacity of the theater. An original concrete roof of nearly 100,000 pounds was removed to allow for the flyloft to be reframed and the rigging capacity to be increased, all with a reduction in loads and stresses on the existing members.

## Category 7: Forensic/Renovation/Retrofit/Rehabilitation Structures over $20 Million

# SAVANNAH PLANT RIVERSIDE PROJECT

*Savannah, GA | Browder + LeGuizamon and Associates, Inc.*

The JW Marriott project was a complex renovation of a 1912 Savannah Power Plant Station into a luxury hotel. The original open boiler room was replaced with five levels of new framing, transforming the skeleton of the building. The use of composite steel joists topped with four inches of lightweight concrete on composite metal deck eliminated the need for foundation retrofit at gravity columns. Analysis of the lateral system was triggered by replacing and adding entire floor levels and large new openings in the exterior masonry walls. This resulted in new steel braces through new and existing beams and complete replacement of existing braces.

## Category 7: Forensic/Renovation/Retrofit/Rehabilitation Structures over $20 Million


# SYRACUSE UNIVERSITY STADIUM NEW ROOF PROJECT

*Syracuse, NY | Geiger Lynch MacBain Campbell Engineers, PC, d.b.a. Geiger Engineers*

The 1980 Carrier Dome at Syracuse University is a 50,000 seat multipurpose domed stadium. The stadium's new roof is a first-of-its-kind cable truss, employing tensioned membrane and rigid panels to cover 250,000 square feet. The ingenious design uniquely addresses the challenges of replacing the original air-supported roof. One of the structure's most effective and iconic features is the configuration of the external crown truss, which was found by optimization techniques to most efficiently withstand the significant snow loads of the region while minimizing the demand on the existing structure. In addition, creative solutions repurposed the original compression ring as a key component of the new structure.

# LITTLE ISLAND

*New York, NY | Arup*

The 2.4-acre urban oasis is part park, part performance venue sitting atop 132 precast concrete "pots" soaring high above the Hudson River. The utilization of parametric modeling, electronic information transfer, digital fabrication, and offsite construction was critical to the project. Complex geometries were developed by the architect using parametric scripts and further refined by the structural engineering team to make them structural. 3-D geometry files were sent to the fabricator for CNC-milled foam formwork, automatic rebar bending, and virtual fit-up with a 3-D scan. Full assembly was completed offsite, and pots were delivered to the site for erection onto the precast cylinder piles.


Courtesy of Timothy Schenck

# 2021 Panel of Judges

## Preliminary Round – NCSEA Past Presidents

Ben Nelson, P.E. – *Martin/Martin Consulting Engineers*
Bill Bast, S.E. – *LPI, Inc.*
Carrie Johnson, P.E., S.E. – *Wallace Design Collective, PC*
Jim Cagley, P.E. – *Cagley & Associates*
Jim Malley, S.E., P.E. – *Degenkolb Engineers*
John Joyce, P.E. – *Engineering Solutions, LLC*
Marc Barter, P.E., S.E., SECB – *Barter & Associates, Inc.*
Ron Hamburger, S.E. – *Simpson Gumpertz & Heger, Inc.*
Tom Grogan, P.E. – *Retired*
Vicki Arbitrio, P.E. – *Gilsanz Murray Steficek LLP*

## Final Round – Oklahoma Structural Engineering Association – OSEA

Aaron Landrum, P.E., S.E. – *360 Engineering Group*
Alan Kirkpatrick – *KFC Engineering*
Andrew Stuart, P.E., S.E., – *US Army Corp of Engineers*
Ben Nelson, P.E. – *Martin/Martin Consulting Engineers*
Carisa Ramming – *Oklahoma State University*
Carrie Johnson, P.E., S.E. – *Wallace Design Collective, PC*
Chris Snider – *CEC Corporation*
Greg Poston, P.E., S.E. – *Wallace Design Collective, PC*
Isabella Horton – *Frankfort Short Bruza*
Katie Faulkner, P.E., S.E. – *Wallace Design Collective, PC*
Kyle Haskett, P.E., S.E. – *Wallace Design Collective, PC*
Mike Thompson – *ZFI Engineering*
Nick Chapman, P.E. – *Frankfort Short Bruza*
Orin Johnston – *JAG Engineering LLC*
Shannon Koeninger – *Benham, a Haskell Company*
Vinay J. Thottunkal, P.E., S.E. – *Star Building Systems*

AWARD WINNER | CATEGORY 2: NEW BUILDINGS $30 MILLION TO $80 MILLION

# THE GEORGE W. PEAVY FOREST SCIENCE CENTER

*Corvallis, OR | Equilibrium Consulting Inc*

The George W. Peavy Forest Science Center provides Oregon State University with various classrooms, laboratories, and gathering places within their Forest Science Complex. The building sets a new bar for modern timber construction with large timber-concrete composite spans, the first self-centering post-tensioned CLT shear walls in North America, active structural monitoring, and elegantly integrated design. The building is living proof that mass timber has earned its rightful place among high-tech construction materials, offering a beautiful, low-carbon alternative to traditional high-performance construction systems.


CATEGORY 3: NEW BUILDINGS $80 MILLION TO $200 MILLION

# 2461 BROADWAY

*New York, NY | WSP USA*

2461 Broadway is a 20-story, 210-foot-high residential building in Manhattan with cantilevers totaling 50 feet over 3 levels. A central concrete core acts as the primary lateral load resisting system. Vierendeel trusses on 3 sides of the building support the large cantilever without impacting the architectural design. An innovative temporary embedded-steel frame platform was implemented for the construction of the cantilevers, which then absorbed into the permanent structure when construction was completed. This unconventional method was designed for a faster, safer, and cost-effective method to efficiently support the cantilevers throughout different stages of construction.

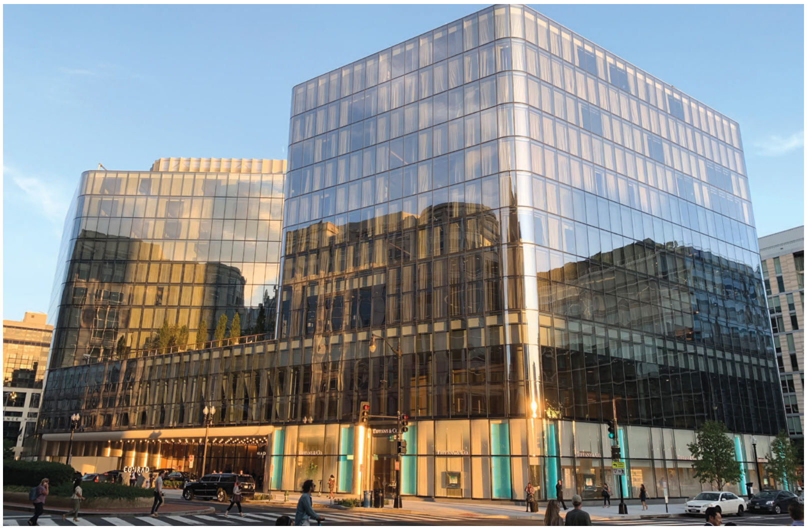

CATEGORY 4: NEW BUILDINGS OVER $200 MILLION

# 425 PARK AVENUE

*New York, NY | WSP USA*

425 Park Avenue, a modern 21st-century iconic office building, is the first full-block new construction on Park Avenue in half a century. LEED Gold and WELL certified, the building is designed around the wellness and well-being of its occupants. The tower's structure is conceived around the vision of providing a well-ventilated open working environment filled with natural light and column-free spaces. Tenants will enjoy unique amenities, including a world-renowned restaurant and exterior sky gardens at the building setbacks. Its unique architecture, featuring large glass sloped surfaces, aspires to bestow a new "crown jewel" upon the NYC skyline.

AWARD WINNER | Category 4: New Buildings over $200 Million



## CONRAD WASHINGTON, DC

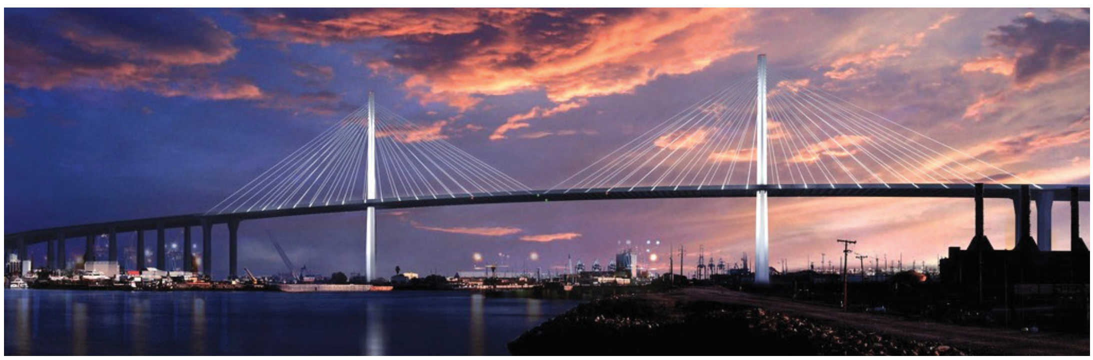
*Washington, D.C. | Thornton Tomasetti, Inc.*

The five-star, 360-room Conrad Washington, DC spans 509,000 square feet and includes meeting facilities, retail, restaurants, outdoor terraces, and parking. The 10-story building is composed of concrete flat slabs supported by concrete columns, a concrete transfer mat slab on the third floor, and steel and concrete transfer beams at ground level to transfer columns over the large ballrooms. A unique project challenge was the tall column-free grand ballroom. The columns and shear walls were transferred using heavy steel plate girders supported on elastomeric bearing pads on top of concrete columns to maximize ceiling height within the ballrooms and amenity levels.

AWARD WINNER | Category 5: New Bridges or Transportation Structures

## GERALD DESMOND BRIDGE REPLACEMENT

*Long Beach, CA | ARUP*



The Gerald Desmond Bridge Replacement Project replaced one of the most highly trafficked bridges in North America with a unique and iconic signature bridge. Though this cable-stayed suspension bridge is the first of its kind in California, it also pushed the envelope in seismic protection, architectural design, constructability, and traffic engineering. To achieve the extreme seismic requirements of the site with a cable-stayed structure, the bridge towers and end bents feature a unique design to remain essentially elastic during seismic events. The unique design reduces maintenance requirements for the Port, ensuring performance during the design seismic event and improved lifecycle resilience.

AWARD WINNER | Category 5: New Bridges or Transportation Structures

## SEA-TAC INTERNATIONAL ARRIVALS FACILITY PEDESTRIAN WALKWAY

*SeaTac, WA | KPFF Consulting Engineers*

The new International Arrivals Facility Pedestrian Walkway signature structure spans 600 feet and nearly 800 feet in total length. It soars 85 feet above the active Seattle-Tacoma International Airport aircraft taxi lanes. The walkway is comprised of three spans of steel box girders and cables combined in a king post truss configuration. Innovative wind-tunnel testing, material use, and adjustable structural components improved performance and decreased material use. In addition, close collaboration between the design team and contractor allowed large portions of the structure to be built offsite and transported into place with minimal disruption of airport operations.

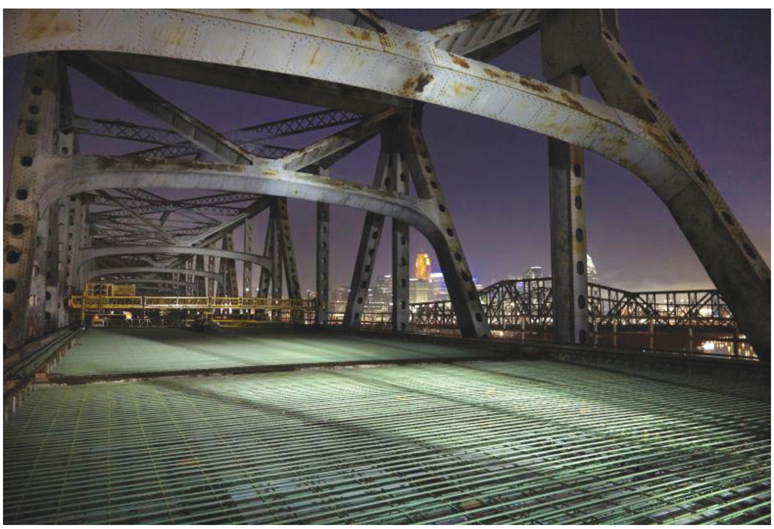
AWARD WINNER | Category 6: Forensic/Renovation/Retrofit/Rehabilitation Structures under $20 Million

## APPLE PARK

*Greenville, NC | Collins Structural Consulting, PLLC*

This project retrofitted an existing building to create a 39,400-square-foot open-floor production facility. Fifteen interior columns were replaced by eight super-trusses (up to 206 feet long) with perimeter supports. A paired super-truss design balances the loads on each side of a column line, preventing torsional displacement while allowing the existing columns to remain in place during installation. Construction simulations were used to predict the camber of the trusses and deflection at each stage of the erection. Restricted by the interior construction space, an innovative method was developed to assemble the trusses in-place and then transfer loads to the new structure.

AWARD WINNER | Category 6: Forensic/Renovation/Retrofit/Rehabilitation Structures under $20 Million



## BRENT SPENCE BRIDGE EMERGENCY REPAIR

*Covington, KY to Cincinnati, OH | Michael Baker International*

A 1500°F fire resulting from the collision of two semi-tractor trailers on the Brent Spence Bridge, a double-decker cantilevered truss structure, shut down vehicular and maritime traffic. A massive industry effort had over 30 inspectors rapidly deployed to the site to provide hands-on inspection of the fracture-critical truss within the heat-affected zone. The inspection of the bridge included determining global stability and performing a structural appraisal, a condition assessment, and material testing. Based on recommendations for replacing steel bridge components, the team chose to replace in-kind wherever possible or design new components to be equal or better.

AWARD WINNER | Category 6: Forensic/Renovation/Retrofit/Rehabilitation Structures under $20 Million

## SPERRY CHALET RECONSTRUCTION

*Glacier National Park, MT | JVA, Inc.*

In 2017, the Sprague Fire destroyed the historic Sperry Chalet sited within Glacier National Park's backcountry. Exposed log framing was rebuilt, having internal steel reinforcement capable of resisting extreme snow loads while maintaining the original proportionality. New internal wood-framed shear walls and cantilevered diaphragms were designed for compatibility with the perimeter masonry walls. The stone masonry was salvaged by re-mortaring, pinning, reconstructing the most fire-damaged areas, and anchoring to the diaphragms to meet performance objectives for seismic resistance. Summer access to the site was via a 6.7-mile, 3360-foot elevation-gain hike, and materials were transported by helicopter.

AWARD WINNER Category 8: Other Structures

# EAST END GATEWAY – ENTRANCE CANOPY, MTA C&D

*New York, NY | Skidmore, Owings & Merrill in association with AECOM*

The East End Gateway Entrance Canopy is designed to bring the grandeur of Moynihan Train Hall to the eastern side of Penn Station, the busiest train station in the Western Hemisphere and a hub for the MTA's LIRR and NYC Transit's subway lines, NJ Transit, and Amtrak. The monumental glass and steel canopy marks the entrance to the LIRR concourses. The structure rises 40 feet and gently curves to the ground. Pre-tensioned steel cables, spanning two ways, support the smoothly curved, high-performance glass enclosure. The cables vary in length and are tensioned in opposing directions, creating a doubly curved, anticlastic form.

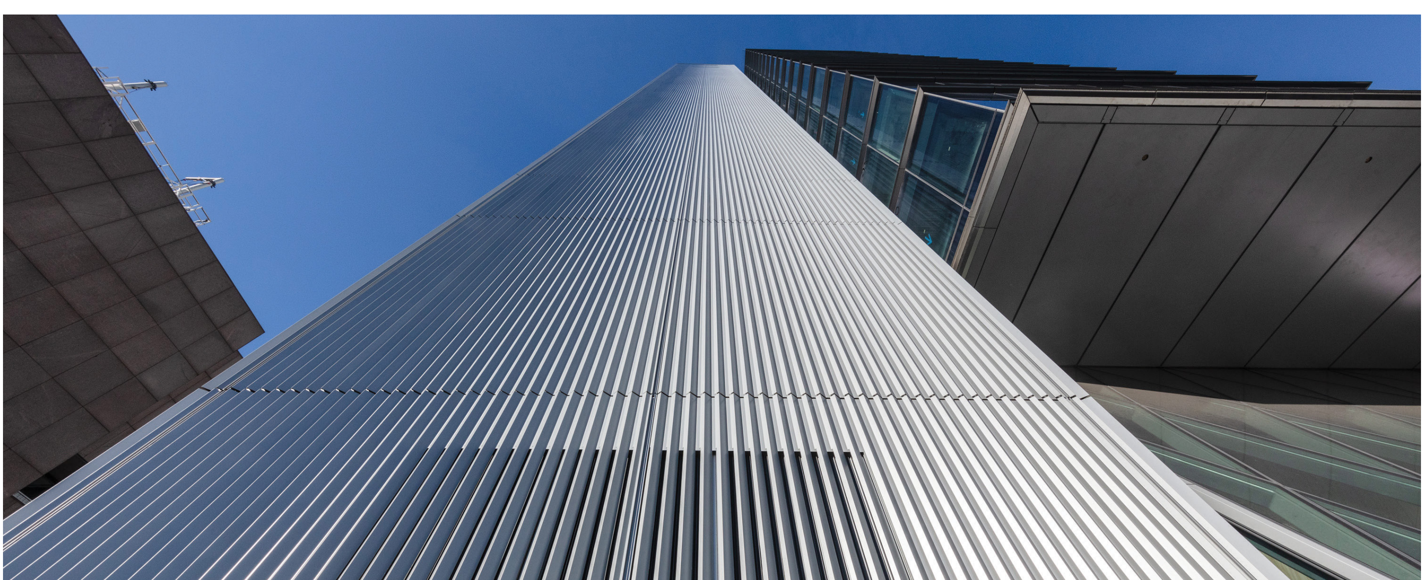
*Courtesy of Lucas Bliar Simpson*

AWARD WINNER Category 8: Other Structures

# MOYNIHAN TRAIN HALL SKYLIGHTS

*New York, NY | schlaich bergermann partner (skylight structural engineer)*

The James A. Farley Post Office Building is transformed with four gridshell skylights in the Moynihan Train Hall and a gridshell skylight in the Midblock space. These lightweight skylights are designed to rest minimally on the building and historic steel trusses. The design arranged larger panels with decreasing steel member depth toward the middle of the shells. Steel elements are built-up T-Sections with diagonal x-cables to brace the surface and establish the load-bearing behavior of a true shell. The x-cables run continuously at the top of the plates, underneath the glass, and at each node are fixed to the steel structure with clamping disks.

# The Renovation and Retrofit of 100 Stockton Street

Updated exterior, completion slated for early 2022.

## CONVERTING AN INTROVERT INTO AN EXTROVERT

By David Rossi, S.E.

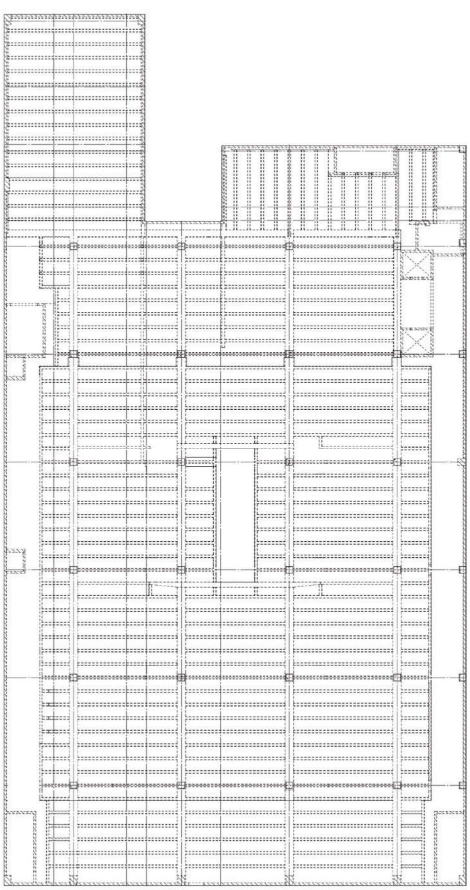


*Figure 1. Typical original floor framing plan.*

In 2016, Macy's announced they were shuttering their Men's Store in San Francisco's Union Square, and Southern California-based developer Blatteis and Schnur partnered with Morgan Stanley with the intent of winning the development rights. Gensler signed on as the architect for the pursuit and, hopefully, the project, and the KPFF San Francisco office joined the team as both the civil and structural engineers. Although the structural work was initially viewed as renovation and a seismic retrofit, it soon became apparent that the intent would be a complete transformation. The Blatteis and Schnur/Morgan Stanley team submitted the winning bid, Plant Construction joined the team, and the adventure began.

### Building Description

Originally designed in 1973 by Seattle engineers Hadley Properties, the single-tenant retail building has a floor plate of 31,000 square feet on each of eight levels, plus a one-story basement. The typical floor system shown in *Figure 1* consists of 4½-inch-thick reinforced concrete slabs spanning east-west between 18-inch-deep post-tension pan joists. The pan joists span between 24-inch-wide by 18-inch-deep post-tensioned girders spaced at 32 feet on-center, and 7-foot-wide, 4½-inch-thick closure pour slabs were placed on the north and south sides to allow the pan joists to be stressed. 24-inch-square reinforced concrete columns supported the girders. The joists were turned approximately 30 degrees from square at the roof, effectively increasing the center-to-center span to 34 feet. The street-level slab was stepped in one location due to the sloping streets on the west and south sides. Reinforcement for the framing elements was typical for that era: 60 ksi yield strength, but light on shear reinforcement and confinement steel. Column foundations were spread footings, and wall foundations were grade beams with piers.

The lateral system consisted of perimeter reinforced concrete shear walls. On the north and east sides, the 12-inch-thick concrete walls had no openings to speak of and served as the fire separation walls for the adjacent properties. The building was held off the property lines on the north and east sides by 4 inches. *Figure 2* shows the west and south elevations of the original structure. Adjacent to the storefront windows at the street level, the 22-foot-long west walls were 3 feet thick from the basement to the underside of the fourth floor and 12 inches thick above the fourth floor. Above the street-level storefront windows, the exterior elevation was solid 12-inch-thick reinforced concrete walls at the second through fifth floors. Similar to the west elevation, the south elevation consists of 30-foot-long, 2-foot-thick walls adjacent to the storefront windows, from the basement to the underside of the fourth floor, and 12 inches thick for the remaining levels.

## Conceptual Design

The original building was designed for a single retail tenant when solid walls were acceptable, and natural light was sometimes an afterthought. The development team's vision was to convert the lower three floors plus basement to retail (potentially multiple tenants at each level), two floors of space that could be office or retail (flex space), three upper floors of office space, and a rooftop restaurant. The team knew that the solid walls had to go, with corner spaces being a premium. The exact concept was yet to be determined, but Gensler recognized that the structural system would play a significant role in the vision, and KPFF's team was brought into the conceptual design.

KPFF assembled a team of four engineers, a project manager, and a principal in charge and began analyzing the existing structure to assess the expected performance relative to a new structure. The existing lateral system turned out to be quite stiff, and the 4-inch seismic joint was adequate even by modern code standards. The gravity system was also generally adequate for the new intended use, except for the rooftop joists and girders, which appeared to be designed for 20 psf roof live loads. KPFF also evaluated the typical floor joists and found that the post-tensioning could be removed and still achieve the required capacity by adding reinforcing plates and carbon fiber wrap. Knowing the joists could be modified opened the architectural possibilities.

"The original building was designed for a single retail tenant when solid walls were acceptable, and natural light was sometimes an afterthought.

For the lateral system, one thing was clear: the west and south walls were going to be demolished. Gensler described this as "changing an introverted building into an extrovert." The KPFF team rapidly assessed over forty possible schemes, including interior cores and exoskeletons of various configurations. Rapid lateral analyses were performed to verify the feasibility of each scheme to meet the current *California Building Code* (CBC), tuned it as required, prepared graphics files that Gensler imported into their SketchUp model, and prepared weekly presentations to the development team to accept or reject the proposed lateral bracing concept.

In the end, the ownership group decided that some form of core system would best suit their needs and allow for the most significant future flexibility. As the conceptual design progressed, they also expressed a desire for at least six feet of column-free space on the two street sides to allow flexibility in window displays and provide a terrace at the third floor. Structurally speaking, the request for column-free perimeter space was a relatively simple concept on the south edge due to the closure pour. However, removing columns on the west face meant modifying the existing post-tensioned girder, which would increase the cost significantly. Finally, ownership wanted the ability to demise the west side of the building into either three or four tenants to allow for the maximum future leasing options. This request meant columns would be moving.

KPFF is often asked if it would be simpler to demolish the building and design a steel structure that satisfies every request

*Figure 2. Exterior of the former Macy's Men's Store.*

> Shortening the girders and removing columns involved shoring the entire building at every level.

and allows maximum future flexibility. KPFF asked this question of the architectural and ownership teams every week until they were told to stop asking. The building is in a historic district in one of the busiest areas of San Francisco, and it was estimated that the Environmental Impact Review would take up to four years to compete, which put construction permits about five years into the future.

## Selected Scheme

*Figure 3* is an upper-level floor plan illustrating the selected scheme. The existing post-tensioned girders are highlighted yellow and originally extended from the west face of the building to a grid line 32 feet into the blue highlighted area. The magenta highlights are new 3-foot-deep concrete transfer girders at every floor. The blue highlighted region is a zone where the existing concrete floor system was replaced with steel framing to accommodate new elevator banks, exit stairs, and mechanical shafts. The existing floor framing was removed and replaced with cast-in-place concrete slabs in the gray highlighted zones for architectural reasons.

Shortening the girders and removing columns involved shoring the entire building at every level, as was described in the article by Robert Graff of Degenkolb in his recent article (STRUCTURE, *January 2022*). Once shored, the ends were cut to release the cables, and the concrete was removed to the face of the nearest support, either an existing column or new transfer girder. Next, the cables were retained, reprofiled, and the girder ends were repoured as cantilevers, as shown in *Figure 4*. The bottom

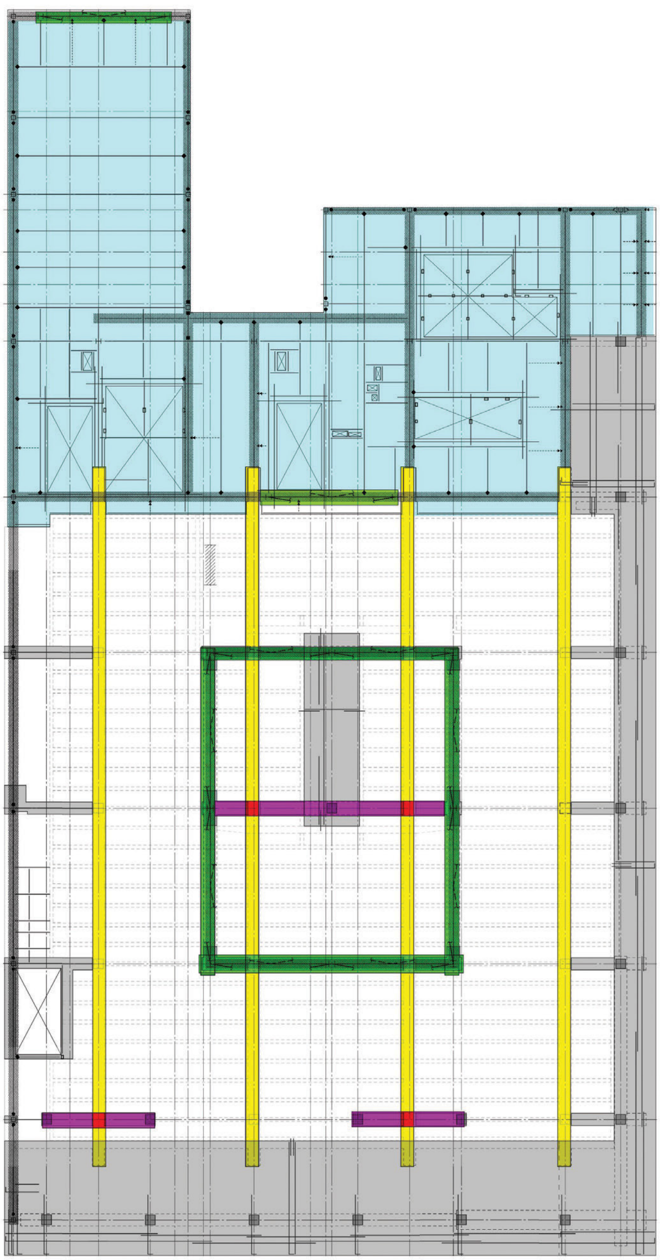


*Figure 3. Typical upper-level floor plan.*

*Figure 4. New cantilevered girder ends at various stages of construction, with continuous shoring installed.*

girder end in *Figure 4* shows the retained cables and longitudinal reinforcing, the middle girder has been poured and stressed, and the top girder has been poured and stressed while the column has been removed and the slab prepared for the transfer girder.

In addition to the work described for the typical floors and shown in *Figure 3*, two levels were removed entirely and replaced with steel framing: the ground floor and the roof. Due to the sloping streets adjacent to the building and the desire to allow for multiple retail tenants, the original ground floor framing was removed and replaced with steel framing stepped as established by Gensler. The roof was similarly replaced with steel framing because the original framing could not accommodate the dead and live loads of a restaurant and accessible outdoor landscaped space.

An outdoor terrace at the third floor wraps the building on the west and south sides, and its sloping soffit provides a dramatic focal point from the street level. The structural design included a horizontal steel truss cantilevering off the newly poured columns. The façade is clad with white terra cotta tiles to match the surrounding buildings, and KPFF's Portland office was the structural engineer for the curtain wall.

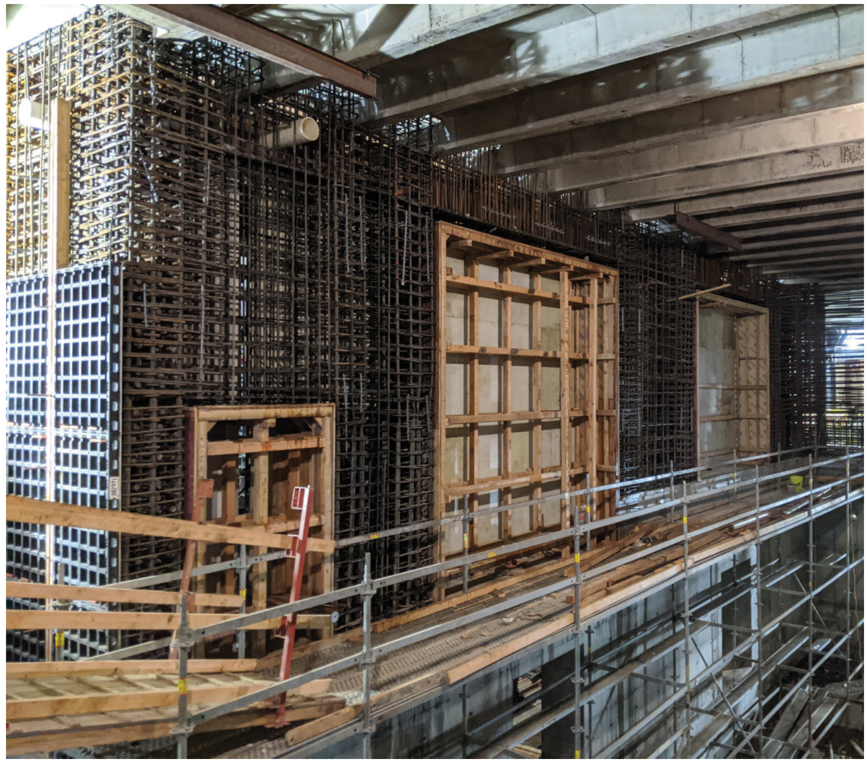


*Figure 5. North-South core wall reinforcing.*

The selected lateral scheme is highlighted green in *Figure 3* and is designed to substantially meet the provisions and performance objectives of the 2016 CBC. The heavily reinforced core (*Figure 5* ) covers two bays in each direction and is perforated with large openings to allow open retail spaces. Two additional lines of north-south bracing were required both for loads and for reducing torsion. From the basement to the underside of the fourth floor, the lateral system is special reinforced concrete shear walls which vary in thickness from 3 feet 6 inches at the west edge of the core to 2 feet elsewhere. From the fourth floor to the roof, the lateral system changes to buckling-restrained braces (BRB's) to allow for clear views on the office floors.

The transitions from steel bracing to concrete cores were particularly challenging to detail and build. Steel beams were buried in the concrete beams, allowing for load transfer between the two materials and providing locations for attaching gusset plates (*Figure 6* ). Uplift on the frame columns was too high to rely on anchor bolts, so full-height steel columns were buried in the shear wall boundaries from the third to fourth floors. The confinement reinforcing maintained continuity by passing through predrilled holes in the buried columns.

The remaining significant revisions to the lateral system included widening the seismic joint and adding collectors and ties. The new lateral system was more flexible than the 1973 design, notably at the braced frame levels, so KPFF called for the installation of perimeter beams and columns on the north and east sides, removing the original 12-inch-thick perimeter concrete bearing walls, and cutting the floor slab back. Existing gravity columns were wrapped with carbon fiber to ensure ductile behavior when the building deflects during a major earthquake. Carbon fiber collectors resolved the diaphragm discontinuity issues created when merging new and existing concrete slabs.

*Figure 6. Braced frame column buried in the core wall for load transfer.*

Above the roof, the restaurant lateral system transitions to steel special moment resisting frames, while the back-of-house spaces utilize SureBoard sheathing as bracing. The KPFF San Francisco office currently provides structural engineering services for the new restaurant slated to open in 2022.

A project as complicated as 100 Stockton Street has a list of unknown conditions and lessons learned too numerous to mention in an article. However, the key to the entire project was that the structural team adopted a solutions-based mindset and abandoned the "problem-finding" mentality structural engineers can sometimes exhibit. Creating a rapid response team during conceptual design encouraged creative solutions. It ensured that the path taken throughout the design delivered a renovation and retrofit consistent with the development team's expectations.▪

*David Rossi is a Principal in KPFF's San Francisco office (david.rossi@kpff.com).*

# Bioinspiration and Structural Engineering

*By Austin Dada, P.E., M.ASCE, Frederick R. Rutz, Ph.D., P.E., S.E., F.SEI, M.ASCE, and Wil V. Srubar III, Ph.D., M.ASCE*

Humans have been building with biology for thousands of years. Dimensional lumber and mass timber are mature technologies based on materials produced using biological processes. Today, there is fertile ground for additional research and development of materials based on biological processes or biomimetic principles finding application in structural engineering. Advancements in biotechnology are giving rise to new biomaterials for construction that go beyond wood; bioinspired structures and biomimicry are potential tools to augment existing and entirely new structures. The term *biomimicry* is derived from the words *bios* and *mimicry* coming from the Greek language meaning *life* and *imitation*, respectively. The fundamental tenet of biomimicry is to mimic nature and leverage the biological principles that have resulted from evolutionary processes.

*Biomimetic Antifreeze Polymer-Modified Concrete sample that passed ASTM C666, the standard test method for rapid freezing and thawing of concrete. Courtesy of the University of Colorado Boulder, College of Engineering and Applied Science.*

In addition to trees, microorganisms play a key role in inspiring many new materials and structures of the future. For example, biomineralization is a process by which bacteria precipitate calcite, a hard mineral that is the most stable polymorph of calcium carbonate. Certain strains of microorganisms precipitate calcite when exposed to solutions that include calcium. These strains have been added to sand to create concrete-like alternatives. Compressive strengths of these bioinspired, biocemented bricks rival those of other cementitious materials. The major advantage to these biologically cemented sandstone bricks is their lower embodied energy and embodied carbon, as well as their autonomous nature of fabrication.

Microorganisms have also inspired new approaches to engineering durable infrastructure. For example, a common issue encountered by engineers and construction professionals is the harmful impact of freezing and thawing on concrete durability. A bioinspired solution to this issue is currently being researched at the University of Colorado Boulder. Synthetic polymers that mimic the ice recrystallization inhibition behavior of antifreeze proteins in certain plant and animal species are being added to concrete in place of air entrainment. Antifreeze proteins exist in plant and animal species that must survive in cold climates. Although beneficial to the durability of concrete structures, air entrainment includes some drawbacks, such as reductions in compressive strength, increases in chloride permeability, and

> *"Synthetic polymers that mimic the ice recrystallization inhibition behavior of antifreeze proteins in certain plant and animal species are being added to concrete in place of air entrainment.*

*Advances in biomimetic design will include concepts such as designing biological systems using computational algorithms to model biological processes and optimize the topology of structural systems.*

inconsistencies in achieving sufficient air void systems in the field. As with many burgeoning technologies, the long-term behavior of biomimetic antifreeze polymers has not yet been investigated. Further research into biomimetic antifreeze polymers may yield a commercially viable bioinspired alternative to air entrainment.

Applications of bioinspired design can span multiple scales. For instance, subterranean tunnels that ant colonies produce are currently being studied at the California Institute of Technology. Their relative complexity and longevity make them interesting and particularly suitable for advanced geotechnical engineering applications. These ant tunnels can extend up to 25 feet below ground and last for decades. This research aims to determine how to emulate and scale these ant tunnels to a degree suitable for human applications in construction and determine the social mechanisms behind how the individual ants communicate to build their structures. This bioinspired research has implications in foundation engineering, including new possibilities ranging from deep foundation types to mechanically stabilized soil retaining wall structures.

Bioinspired engineering design of built infrastructure is an emerging field with a wide variety of future applications. Advances in biomimetic design will include concepts such as designing biological systems using computational algorithms to model biological processes and optimize the topology of structural systems. For example, structural materials researchers are already working with synthetic biologists to specifically design engineered strains of bacteria that are more efficient at producing calcite or engineered proteins. These products could serve a wide range of functions in augmenting or replacing conventional civil engineering materials. However, this type of biological design will require structural engineers to embrace interdisciplinary concepts, strategies, and methodologies – something that current civil and structural engineering curricula within conventional university degree programs do not easily enable.

While technical and curricular challenges still need to be overcome, possibilities abound within the emerging field of bioinspired structural engineering. Interested students and practitioners should seek out opportunities for bioinspired innovation and connect with the ASCE Bioinspired Structures committee, which is devoted to advancing innovations at the intersection of biomimicry and structural engineering. The chairperson of this committee is Dr. Hongyu (Nick) Zhou, an Assistant Professor in the Department of Civil and Environmental Engineering at the University of Tennessee in Knoxville. He can be reached by email at **hzhou8@utk.edu**.▪

References are included in the PDF version of the online article at **STRUCTUREmag.org**.

*Austin Dada is a Bridge Structures Engineer at RockSol Consulting Group.*

*Frederick R. Rutz is an Associate Professor at the University of Colorado and a structural engineer at J.R. Harris & Company in Denver, CO.*

*Wil V. Srubar III is an Associate Professor at the University of Colorado in Boulder who specializes in the research of bioinspired infrastructure.*

STRUCTURAL ENGINEERING FIRM *Profile*

# Structural Engineering at the Table

*PCS Structural Solutions Elevates Opportunities to Lead with Impact-Driven Results*

The time is now for impact-driven structural engineers. Sustainability and resilience are the imperative. Challenges with climate change, shifts in supply chains, and availability of resources demand innovation and new skill sets. It is a new frontier, and PCS is poised to meet the challenge.

PCS engineers thrive in integrated team environments, which translates well to the AEC landscape of progressive delivery models and positions structural engineers to have an enormous impact on developing solutions to these highly complex challenges.

## *Resilience and Adaptable Design*

Resilience issues have been topping recent news cycles and will continue to do so. PCS provides engineering for about 800 projects a year, 99% of which include seismic analysis to meet resilience targets. In addition, performance-based design is often required for hospitals and high rises, for example, and more accurately informs how a structure will respond under extreme stress.

PCS structural engineers take a key role in identifying opportunities for structural resilience and adaptable design. PCS engineers directly impact community safety and well-being through a commitment to full structural participation.

## *Tackling Climate Change*

PCS signed on as a coalition partner to the SEI SE 2050 Commitment Program, developed to rally structural engineers around the climate imperative to limit the increase of global temperatures below 2°C, a target set by the Paris Climate Agreement. Getting critical metrics and recommendations in front of the design team early on allows them to make intentional project choices for maximum benefit.

Mass timber is gaining momentum as another option in the sustainable design toolbox. From committee leadership to research projects, PCS's growing mass timber portfolio is an exciting opportunity to boost structural engineers' impact on climate change.

## *Building Passion and Growth*

PCS provides a supportive environment for the personal passions of its staff, helping to sustain professional fulfillment and satisfaction. In addition, PCS-U provides in-house continuous learning opportunities, while project manager-led expertise focus groups allow PCS engineers to be at the forefront of industry advancements.

PCS Structural Solutions continues to build on a legacy of decades of success. Our collaborative culture has been key in finding solutions to supply chain and Covid-related challenges. Our agility in engineering across many markets has allowed us enormous stability through periods of market disruption. Best of all, our passion for our work and its impact on our communities drives us to continually create innovative solutions for structures in an increasingly complex world.

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# Tacoma Narrows Bridge Failure 1940

*Galloping Gertie, Part 2*

*By Frank Griggs, Jr., Dist. M.ASCE, D.Eng, P.E., P.L.S.*

The design evolution for the Tacoma Narrows Bridge (aka Galloping Gertie) was presented in the February 2022 issue of STRUCTURE. Soon after opening, the Tacoma Narrows Bridge exhibited significant vertical movements under various wind conditions. To address the issue, the owners and designers tried various modifications and were investigating additional steps to control or at least minimize the motions.

*Tacoma Narrows Bridge twisting prior to collapse.*

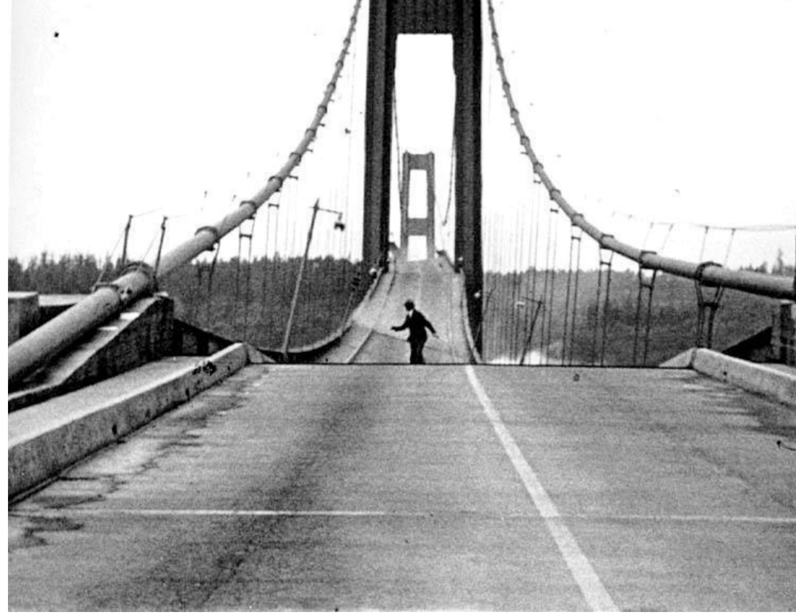
Early in the morning on November 7, 1940, sustained winds of 38 miles per hour were recorded, rising to 42 miles per hour by 10:00 AM. Farquharson was at the bridge and noted the deck was raising and falling in sine waves at 38 cycles per minute with a 3-foot double amplitude that was abnormal. He recorded the motion with motion picture and still cameras. Then something happened that had not happened before; the oscillation slowed to 12 cycles per minute and turned from a vertical motion to a two-wave torsional movement. The deck began to twist and roll violently, with the roadway twisting 45 degrees from the horizontal one way, then 45 degrees the other way. He continued filming until the twisting finally collapsed the deck. This film is well known to every civil engineering student.

Kenneth Arkin, the chairman of the Washington State Toll Bridge Authority, arrived at the bridge just before the collapse. After talking to Farquharson, he shut it down shortly after 10:00 AM due to the increased twisting of the deck. The bridge failure ultimately started as the north center stay broke, and the bridge began twisting even more violently in two parts. "Two cars were on the bridge when this wild movement began: one with Leonard Coatsworth, a newspaper reporter, and his cocker spaniel and the other with Arthur Hagen and Judy Jacox. All three people crawled to safety." Shortly after 11:00 AM, the other stay broke, and the stiffening girders buckled in the middle, followed by the breaking of several suspenders. Most of the main span then dropped into Puget Sound. With a large portion of the center span deck gone, the towers tilted 12 feet towards the anchorages causing significant deflections in the side spans. The failure was complete.

> *"With a large portion of the center span deck gone, the towers tilted 12 feet towards the anchorages causing significant deflections in the side spans. The failure was complete."*

The State of Washington and the United States government both appointed boards of experts to investigate the bridge's collapse. The insurance companies also established a *Narrows Bridge Loss Committee*. The Federal Works Administration (FWA) appointed a 3-member panel of top-ranking engineers: Othmar H. Amman, Dr. Theodore Von Karmen, and Glen B. Woodruff. Their report to the Administrator of the FWA, John Carmody, became known as the *Carmody Board* report. On March 28, 1941, the panel announced its findings; 139 pages plus 8 appendices accessible online through Hathi Trust. They wrote,

> "As a result of the investigations which are described in detail in this report, we have reached the following conclusions:
>
> 1. The Tacoma Narrows Bridge was well designed and built to resist safely all static forces, including wind, usually considered in the design of similar structures. Its failure resulted from excessive oscillations caused by wind action.



Photographer Howard Clifford, escapes the Tacoma Narrows Bridge during collapse.

2. *The excessive vertical and torsional oscillations were made possible by the extraordinary degree of flexibility of the structure and of its relatively small capacity to absorb dynamic forces. (emphasis added)* It was not realized that the aerodynamic forces, which had proven disastrous in the past to much lighter and shorter flexible suspension bridges, would affect a structure of such magnitude as the Tacoma Narrows Bridge, *although its flexibility was greatly in excess of that of any other long-span suspension bridge. (emphasis added)*
3. The vertical oscillations of the Tacoma Narrows Bridge were probably induced by the turbulent character of wind action. Their amplitudes may have been influenced by the aerodynamic characteristics of the suspended structure. There is, however, no convincing evidence that the vertical oscillations were caused by so-called aerodynamic instability. *At the higher wind velocities, torsional oscillations, when once induced, have the tendency to increase their amplitudes. (emphasis added)*
4. Vertical oscillations of considerable amplitudes were first observed during the erection of the suspended floor and continued, at intervals, until the day of failure. While, at times, the resulting stresses in the stiffening girders were high, there is no evidence that any structural damage resulted. Under certain observed conditions, very high stresses were caused in the ties which connected the suspended floor structure to the cables at mid-span.
5. *It appears reasonably certain that the first failure was the slipping of the cable band on the north side of the bridge to which the center ties were connected. This slipping probably initiated the torsional oscillations. (emphasis added)* These torsional movements caused breaking stresses at various points of the suspended structure, and further structural damage followed almost immediately. The dropping of the greater part of the suspended structure of the center span was made possible by the failure of the suspenders. This was followed by the sudden sagging of the side spans with resulting bending and overstressing of the towers and of the side spans.
6. The suspension type is the most suitable and the most economical that could have been selected for the Tacoma Narrows Bridge. No more satisfactory bridge type could have been chosen.
7. Both the Public Works Administration and the Reconstruction Finance Corporation were entirely justified in assuming that, because of the experience and reputation of the consultants employed by the Washington Toll Bridge Authority, there could be no possible question as to the adequacy of the design. Both agencies exercised thorough and competent supervision during the construction of the bridge.
8. There can be no question that the quality of the materials in the structure, and the workmanship, were of a high order.
9. Certain parts of the towers were severely overstressed and permanently deformed during the failure. While there is no visual evidence of damage to the cables, except at the center of the north cable, it is probable that they were overstressed during the torsional oscillations and as a result of the sagging of the side spans. The main piers were not damaged, except locally, during the failure and could possibly withstand considerably heavier tower reactions than they received from the bridge as it existed. The anchorages were not damaged and could safely resist forces greater than those imposed by the original construction.
10. *The criteria usually considered for rigidity against static forces do not necessarily apply to dynamic forces. (emphasis added)*
11. The remedial installations in the bridge represented a rational effort to control the amplitudes of the oscillations. Further installations, including diagonal stay ropes from the top of the towers to the floor, were being investigated when the failure occurred, and these would have increased the rigidity. *It is doubtful that any measures of this nature would have been sufficient to compensate for the extreme flexibility of the structure. (emphasis added)*
12. The evidence as to whether the vertical oscillations of the bridge would have been affected by fairing (streamlining) is inconclusive. There is certain evidence that fairing would have had an unfavorable influence on the torsional stability.

*continued on next page*

*Collapse complete.*

> *What Moisseiff had done was to extend the deflection theory to an extent far beyond what other engineers, including himself, had done in the past.*

13. Further experiments and analytical studies are desirable to investigate the action of aerodynamic forces on suspension bridges.
14. Pending the results of further investigations, there is no doubt that sufficient knowledge and experience exist to permit the safe design of a suspension bridge of any practicable span. The results of further research should furnish knowledge that will permit more economical design.
15. This report has been restricted to the Tacoma Narrows Bridge, except that available information from other bridges has been considered."

The report that followed these conclusions was lengthy and comprehensive. It is suggested that the reader looks at the entire Carmody report to learn the state-of-the-art in suspension bridges just prior to WWII and what happens when lessons from the past are either ignored or forgotten.

Clark Eldridge was very vocal about the design stating at different times,

"We were assured that the solid girders would be practical for the Narrows Bridge and besides would be cheaper than the truss work. With this assurance, we adopted the design. I want it to be clear that the bridge collapse was due solely to design. No blame can be attached to the P. W. A. or R.F. C. The blame belongs on the designers. It is extremely unfortunate that the plans they prepared failed."

From the resonance theory to Van Karman's vortex shedding hypothesis and most recently torsional flutter, many theories have been advanced over the years as to the actual cause of the failure. Still, all of them agree on one point – that the long length, shallow depth, narrow width, and light weight of the span were factors in the failure. A discussion of all of these theories is beyond the scope of this article.

A new bridge was built ten years later and opened on October 13, 1950. It still serves today, along with a sister parallel bridge constructed in 2007. They have 33-foot-deep open trusses, a deck width of 46 feet, and two 3-foot sidewalks. On the deck, 19-inch open grates along the outside of the outer lanes and 33-inch-wide grates between each lane were installed to permit passage of the winds. In other words, they, except for the grates, resemble the suspension bridges built before the advent of the deflection theory.

What Moisseiff had done was to extend the deflection theory to an extent far beyond what other engineers, including himself, had done in the past. He accomplished this by building a longer, narrower, lighter, and thinner bridge which brought into play aerodynamic forces that had not been encountered with wider, deeper, and heavier bridges. These unforeseen forces resulted in the failure of the bridge. Moisseiff participated in the investigation, but at one time, he said he was "completely at a loss to explain the collapse." He died three years later without designing any additional bridges. The reader may recall that Theodore Cooper (STRUCTURE, *November 2021*) recommended lengthening his Quebec middle span by 200 feet, just as Moisseiff had in the Tacoma Narrows Bridge. Just as the Quebec failure ruined Cooper's reputation, the Tacoma Narrows Bridge failure ruined Moisseiff's reputation, even though the Board did not explicitly blame him. Unlike the Quebec Bridge that seemed to be safe almost up to the moment of failure, the Tacoma Narrows Bridge underwent severe "galloping" during and after construction. No one believed, despite the galloping, that it was not safe. The same thing could be said of Moisseiff as it was of Cooper and Szlapka at Quebec, "The failure cannot be attributed directly to any cause other than errors in judgment on the part of both engineers."

O. H. Amman, who was on the Panel of Engineers and had worked extensively with Moisseiff, wrote, "The Tacoma Narrows bridge failure has given us invaluable information…It has shown [that] every new structure [that] projects into new fields of magnitude involves new problems for the solution of which neither theory nor practical experience furnish an adequate guide. It is then that we must rely largely on judgment and if, as a result, errors or failures occur, we must accept them as a price for human progress."▪

*Dr. Frank Griggs, Jr. specializes in the restoration of historic bridges, having restored many 19th Century cast and wrought iron bridges. He is now an Independent Consulting Engineer (fgriggsjr@twc.com).*

# An Overview of Design Professional Contracts

*By Bruce Burt, P.E.*

If you are a practicing structural engineer, you have no doubt come across various forms of contracts outlining the terms of your agreements. These contracts may be with parties that engaged your services or from whom you sought services. You may have your own contract composed by your legal counsel. More likely, your contract is based on a template created by an organization that develops and periodically updates standard contracts. This article provides a brief overview of the types of contracts you may encounter and contract offerings available from several respected sources.

## Types of Contracts

The most common contract type is between the structural engineer and the owner or architect. This may take a short form when the project is small with an easily defined scope of service or a longer form for larger projects of greater complexity. There are other contract types for use in special situations. There are agreements for Special Inspection Services when they are included in the Structural Engineer of Record's (SER's) services. Contracts cover Peer Review Services and when the structural engineer is not the SER but is responsible for designing a portion of the project, such as a building façade, connections, or other engineered elements. There are also contracts for forensic engineering services, generally related to construction claims or building failures, and the client is usually an attorney.

For the above contracts, the structural engineer is retained by someone else. However, there are occasions when the structural engineer requires the services of another party. The other party may be a geotechnical engineer or a testing lab. The structural engineer may also act as the Prime Design Professional and must retain other consultants. Specific contracts deal with each of these scenarios.

> *Teaming agreements are usually associated with design-build projects but can be used on any project pursued jointly by a contractor and design professional.*

## Sources of Contracts

Organizations whose contracts a structural engineer will most likely encounter are the American Institute of Architects (AIA), the Engineers Joint Contract Documents Committee (EJCDC), the Council of American Structural Engineers (CASE), and Consensus Docs.

AIA has developed an extensive family of documents for design and construction projects, intended for use when the prime owner-design professional is an architect. For design projects when the SER's client is the architect, AIA Document C401 is often used as the agreement for providing structural engineering services.

EJCDC is a joint venture between The American Council of Engineering Companies (ACEC), The National Society of Professional Engineers (NSPE), and The American Society of Civil Engineers (ASCE). EJCDC offers contracts in five principal areas. Its E-Series documents pertain to professional service agreements and related contracts. Document E-001 is available for free download on the EJCDC website and provides a commentary on the entire series of EJCDC engineering services documents.

CASE "represents more than 200 structural engineering firms dedicated to making structural engineering a fair, profitable, and robust industry." CASE offers twelve contracts expressly for use by structural engineers and three commentaries that guide the use of specific AIA documents.

Consensus Docs "is a coalition of associations representing diverse interests in the construction industry that collaboratively develops and promotes standard form construction contract documents…." Consensus Docs' 200-series contract numbers 240 and 245 are standard long- and short-form agreements between owners and design professionals, and number 250 is a standard agreement between design professionals and consultants.

## Design-Build Contracts

Design-build project delivery requires a different form of agreement. In traditional project delivery, the owner retains the prime design professional. In design-build, the owner contracts with an entity comprised of construction trades and consultants known as the Design-Builder. The contractual relationship between the design professional(s), including structural engineers, can vary based on the project and the composition of the design-build team.

EJCDC document D-001 (available for free download) provides an excellent commentary on the contractual issues faced by the design professional on a design-build project. D-001 also provides commentary on the eighteen documents that comprise EJCDC's D-Series. EJCDC D-505 encompasses the standard agreement between the design-builder and engineer.

Other organizations also offer standard agreements covering design-build projects. AIA offers C441-2014 for agreements between architect and consultant on design-build work. Consensus Docs offers contract numbers 400 and 422, and the Design Build Institute of America (DBIA) provides contract numbers 501 and 540.

## Teaming Agreements

An essential element of the contractual arrangement when an engineer and contractor pursue a project together is the teaming agreement. Teaming agreements are usually associated with design-build projects but can be used on any project pursued jointly by a contractor and design professional. They are critical when the design-builder requires the assistance of an engineer in developing sufficient project scope to prepare its bid. EJCDC document D-580 provides contractual language on teaming agreements. Per EJCDC's D-001 commentary, document D-580 "enumerates the respective duties of each team member in the pursuit of the award of contract; specifies the contractual relationship that the Design-Builder and Engineer will enter into if the contract is awarded to the Design Builder…; addresses the responsibility of costs incurred in pursuit of the work; requires confidentiality and assigns ownership rights with respect to documents prepared during the teaming agreement; and specifies rules for exiting the team."

DBIA (DBIA-580), AIA (C102-2015), and Consensus Docs also offer standard teaming agreements.

> *Without appropriate contract language, design-build projects can elevate the standard of care, potentially reducing or eliminating your professional liability insurance coverage in the event of a claim.*

Consensus Docs number 296 pertains to the pursuit of traditional projects, and number 498 covers design-build projects.

Your firm should exercise additional care when entering into design-build agreements. Without appropriate contract language, design-build projects can elevate the standard of care, potentially reducing or eliminating your professional liability insurance coverage in the event of a claim. Likewise, entering into a joint venture can have ramifications for your professional liability insurance coverage if the arrangement is not structured correctly.

Other project delivery methods, such as Integrated Project Delivery (IPD) and Public-Private Partnerships (PPP), also require special contractual treatment. Though AIA and Consensus Docs offer documents covering IPD projects, a custom contract is more common with these types of endeavors.▪

*Bruce Burt is Vice President of Engineering at Ruby+Associates, Inc., and Chair of CASE Contracts Committee.*

## Adhesives Technology Corp.
Phone: 754-399-1057
Email: **atcinfo@atcepoxy.com**
Web: **atcepoxy.com/software**
**Product:** Pro Anchor Design Software
**Description:** This adhesive anchor-focused design tool aids in meeting the design strength requirements of ACI 318. For use with any of ATC's IBC compliant anchoring products. Single pane interface minimizes data input time. Rapid 3-D modeling and real-time optimization of loading conditions, embedment depths, anchor sizes, and more. FREE download!

## Altair® Engineering
Phone: 203-421-4800
Email: **sframe-sales@altair.com**
Web: **s-frame.com**
**Product:** Altair S-CONCRETE®
**Description:** The right concrete design solution is critical to reduce design time while ensuring code compliance. S-CONCRETE provides advanced design capabilities for reinforced concrete beams, columns, and walls for code compliance checks according to regional design codes. All design results are included in comprehensive, customizable engineering design reports.

**Product:** Altair S-FRAME
**Description:** Analyze and design with confidence using Altair S-FRAME. Perform advanced 3-D analysis of structures regardless of geometric complexity, material type, loading conditions, or nonlinear effects. Quickly design and produce code compliance reports with integrated concrete, steel, and foundation design. Advanced DXF and BIM data transfer links ensure optimum efficiency.

## ASDIP Structural Software
Phone: 407-284-9202
Email: **support@asdipsoft.com**
Web: **www.asdipsoft.com**
**Product:** ASDIP Suite
**Description:** Consists of 4 intuitive software packages and over 18 modules to help you with all your daily engineering design tasks. For the past 29 years, ASDIP has been developing powerful yet simple-to-use tools to easily analyze, design, optimize, and check your structural members in significantly less time.

## Computers and Structures, Inc.
Phone: 510-220-5310
Email: **sales@csiamerica.com**
Web: **www.csiamerica.com**
**Product:** SAP2000, CSiBridge, ETABS, SAFE, PERFORM 3D
**Description:** CSI is recognized globally as the leading developer of software for structural and earthquake engineering. Backed by four decades of R&D, SAP2000, CSiBridge, ETABS, SAFE, and PERFORM-3D each offer unique capabilities that are tailored to different types of structures, allowing users to find just the right solution for their work.

## DEWALT Anchors & Fasteners
Phone: 800-524-3244
Email: **anchors@dewalt.com**
Web: **http://anchors.dewalt.com**
**Product:** DEWALT DESIGN ASSIST™
**Description:** State-of-the-art structural design software for concrete anchorages. Facilitates design efforts in base plate, equipment, and deck member anchorages, and post-installed rebar designs. Utilize an extensive library of mechanical, adhesive, and cast-in-place anchors with the Anchor Comparison Tool to easily see differences across anchor types, sizes, and brands. Download at website.

## ENERCALC, Inc.
Phone: 800-424-2252
Email: **info@enercalc.com**
Web: **https://enercalc.com**
**Product:** ENERCALC SEL/Structural Engineering Library/ENERCALC For Revit
**Description:** ENERCALC for Autodesk Revit simplifies structural design by bridging the gap between calculation and documentation. It allows engineers to access the familiar power of ENERCALC SEL as a seamless real-time extension of your Revit environment. ENERCALC's use of the Revit API results in fast-paced, intuitive design with no import/export process.

## National Council of Examiners for Engineering and Surveying
Phone: 800-250-3196
Email: **jbarker@ncees.org**
Web: **ncees.org**
**Product:** Professional Engineering License
**Description:** The National Council of Examiners for Engineering and Surveying (NCEES) is a nonprofit organization dedicated to advancing professional licensure for engineers and surveyors.

## RISA
Phone: 949-951-5815
Email: **benf@risa.com**
Web: **risa.com**
**Product:** RISA-3D
**Description:** The newly released RISA-3D Version 20 is the next step in the evolution of the completely redesigned RISA-3D. With new features including customizable toolbars, masonry seismic detailing, Canadian concrete wall panel design, and other usability improvements, engineers can effortlessly complete complex projects.

**Product:** RISAFloor
**Description:** The newly released RISAFloor Version 16 includes hanger columns, Canadian concrete wall panel design, steel joist updates, and the design of back-to-back hot rolled channels and WT/LL members, giving engineers enhanced functionality for the design and optimization of multi-story building systems.

## The Masonry Society
Phone: 303-939-9700
Email: **info@masonrysociety.org**
Web: **masonrysociety.org**
**Product:** Masonry Codes and Standards
**Description:** TMS402/602 *Building Code Requirements and Specification for Masonry Structures* contains two standards and their commentaries: TMS *Building Code Requirements for Masonry Structures* as TMS402 and *Specification for Masonry Structures* as TMS602. Also, *Standards for Architectural Cast Stone* contains three standards: TMS404, 504, 604. Print and online versions are available.

## Trimble
Phone: 678-737-7379
Email: **jodi.hendrixson@trimble.com**
Web: **www.tekla.com/us**
**Product:** Tekla Structures
**Description:** Create and transfer constructible models throughout the design lifecycle, from concept to completion. With Tekla Structures, accurate and information-rich models reduce RFIs, leverage models for drawing production, material take-offs, and collaboration with architects, consultants, fabricators, and contractors.

**Product:** Tekla Structural Designer
**Description:** Engineers have the power to analyze and design multi-material buildings efficiently and cost-effectively with Tekla Structural Designer. Fully automated and packed with unique features for optimized concrete and steel design, Tekla Structural Designer helps engineering businesses win more projects and maximize profits.

**Product:** Tekla Tedds
**Description:** Automates repetitive and error-prone structural and civil calculations, allowing engineers to perform 2-D frame analysis, access a large range of automated structural and civil calculations to US codes, and speed up daily structural calculations.

## xsec
Phone: 760-984-4327
Email: **xsec13@gmail.com**
Web: **xsecweb.com**
**Product:** xsec
**Description:** Reinforced concrete cross-section analysis. Any shape, input X, Y for each corner and each bar. Any materials, input a sequence of stress and strain for each. Most codes can be accommodated. iOS, Android, MacOS, or Windows apps. No in-app advertisements.

# Not listed?
All 2022 Resource Guide forms are now available on our website.
**STRUCTUREmag.org**.


Arbor Hills Wind, Stuart, IA
Triangle Apartments, Omaha, NE
LSC Storage Facility, Long Island, NY
Union University Library – Jackson, TN
Engineered Aggregate Pier Solutions for All Types of Structures
Marina Lofts, Toledo, OH
PennDOT Engineering Bldg., Clearfield, PA
22 Sussex Mixed-Use, Hackensack, NJ
HEADQUARTERS
St. Louis: 866-421-2460
Northeast Region: 609-577-2724
Great Lakes Region: 412-420-8383
North Central Region: 612-280-8940
SUBSURFACE CONSTRUCTORS INCORPORATED
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PHILADELPHIA
PITTSBURGH
ST. LOUIS

# NCSEA News

## Announcing the 2022-2023 NCSEA Board of Directors

NCSEA is pleased to announce the 2022-2023 Board of Directors. Their term runs from April 1, 2022 to March 31, 2023.

**President**
***David Horos***
P.E., S.E., LEED AP; Skidmore, Owings & Merrill; SEAOI (Illinois)

**Vice President**
***Ryan Kersting***
S.E.; Buehler Engineering, Inc.; SEAOC (California)

**Treasurer**
***Christopher Cerino***
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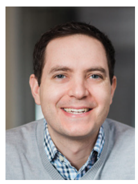
**Secretary**
***Jami Lorenz***
P.E.; DCI Engineers; SEAMT (Montana)

**Director**
***Sarah Appleton***
P.E., S.E.; Wallace Engineering; SEAOG (Georgia)



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**Director**
***Jeannette Torrents***
P.E., S.E.; JVA, Inc.; SEAC (Colorado)

**Director**
***Ken O'Dell***
S.E.; MHP, Inc Structural Engineers; SEAOC (California)

**Past President**
***Ed Quesenberry***
P.E., S.E.; Equilibrium Engineers LLC; SEAO (Oregon)

Thank you to those concluding their service on the NCSEA Board!

- ***Emily Guglielmo***, P.E., S.E.; Martin/Martin, Inc; SEAOC (California)
- ***Eli Gottlieb***, P.E.; Thornton Tomasetti; SEAoNY (New York)

## Congratulations to the 2021 Young Member Group of the Year – SEAoNY

The Young Member Group of the Year Award was presented to the Structural Engineers Association of New York (SEAoNY) at the Structural Engineering Summit in New York City during the Young Member Reception on Monday, February 14. This award recognizes Young Member Groups that are providing an outstanding benefit to their young members, member organizations, and communities. SEAoNY received an additional $2,500 for their Young Member Group to use for future activities. A round of applause for SEAoNY, and also this year's finalists, Minnesota Structural Engineers Association (MNSEA) and Structural Engineers Association of Northern California (SEAONC).

## Call for Abstracts for the Next Structural Engineering Summit

*November 2-4, 2022 in Chicago, IL*

NCSEA is seeking abstracts for the 2022 Structural Engineering Summit. Sessions will be 45-60 minutes total and should deliver pertinent and useful information that is specific to the practicing structural engineer, in both technical and non-technical tracks.

For more information and to submit your abstract, visit **https://bit.ly/2022SummitAbstracts**. The deadline to submit is April 1, 2022.

**FOLLOW @NCSEA ON SOCIAL MEDIA FOR THE LATEST NEWS & EVENTS!**

# 2021 SEA Grant Award Recipients

The SEA Grant Program awards state structural engineers associations (SEAs) funding for projects that advance their SEA and the structural engineering profession in accordance with the NCSEA Mission Statement. Supported by the NCSEA Foundation, the SEA Grant Program has delivered more than $70,000 in grants since its inception.

This year, NCSEA awarded four SEA Grants:

- The Structural Engineers Association of New Mexico (SEANM) received a grant to start a new Young Member Group for college students, recent graduates, and young engineers to connect, network, and learn.
- The Structural Engineers Association of Northern California (SEAONC) was awarded grants to fund an outreach program to build STEM awareness and to support a local Structural Engineering Engagement and Equity (SE3) Symposium featuring research and data on racial demographics of the local structural engineering field and conversation about what can be done to promote broader attraction and retention of all structural engineers.
- The Structural Engineers Association of South Carolina (SEA of SC) will use their grant money to launch a chapter of the Engineers Alliance for the Arts high school student bridge design project, a ten-week curriculum where volunteer structural engineers will visit participating high school classrooms weekly to lead student groups through conceptualization, design, construction, testing, and presentation of foamboard bridges.
- The Structural Engineers Association of Southern California (SEAOSC) will put their grants to use implementing a public speaking workshop for their Young Member Group to kick off a year-long learning program, and a Diversity Equity Inclusion roundtable event to increase participation and engagement in diversity and inclusion efforts in the local structural engineering community.

# Diversity in Structural Engineering Scholarship Program

The NCSEA Diversity in Structural Engineering Scholarship was established by the NCSEA Foundation to award students who have been traditionally underrepresented in structural engineering (including but not limited to Black/African Americans, Native/Indigenous Americans, Hispanics/Latinos, and other people of color). In 2021, four amazing students with bright futures in structural engineering received scholarships, and yet, there were many more deserving students that did not receive support.

The NCSEA Foundation believes it is time to increase available financial resources to work toward a goal of delivering scholarships to all deserving students. A new partner program has been launched to support this effort. The program allows you, your firm, and/or your SEA to partner with the NCSEA Foundation to create an endowed scholarship or an annual named scholarship, or to provide a one-time donation to support the Diversity in Structural Engineering scholarship program.

Applications are being accepted now through April 30 for the 2022 Diversity in Structural Engineering Scholarship program. Multiple scholarships will be awarded to junior college, undergraduate, and/or graduate students actively pursuing structural engineering degrees and careers.

Please visit **www.ncsea.com/awards/scholarship** to learn more about the partner program and/or to submit a scholarship application.

## Learning / Networking

# Structures Congress 2022

April 20 - 23 | Atlanta, Georgia

**Don't miss out on learning with us!**

We have these fantastic special sessions just for you.

- *Innovation Workshop*
- *The Kendeda Building*
- *Collaborative Reporting for Safer Structures*
- *Structural Education of 2040*
- *Embodied Carbon: What You Need to Know*
- *Innovation within the AEC Sector*

www.structurescongress.org

## 2022 Fazlur Rahman Khan Distinguished Lecture Series

*Friday, March 25, 2022 – 4:30 pm EST*

*Supertall Towers + Green Cities* by Adrian D. Smith, Partner, Adrian Smith + Gordon Gill Architecture, Chicago, IL
Lectures are in-person at Lehigh University and live-streamed. Register at **www.lehigh.edu/~infrk** and view other presentations. The SEI Lehigh Valley Chapter will award 1 PDH credit for each lecture to eligible attendees.

## Membership

### Welcome to Computers & Structures, Inc. as an SEI Elite Sustaining Organization Member

Thank you for your support, along with other SEI Sustaining Organization Members! Learn how to reach SEI members year-round, and show your support for SEI to advance and serve the structural engineering profession **www.asce.org/SEIMembership**.


## Errata

**SEI Standards Supplements and Errata including ASCE 7.** See **www.asce.org/SEI**.
To submit errata, contact **sei@asce.org**.

## Students and Young Professionals

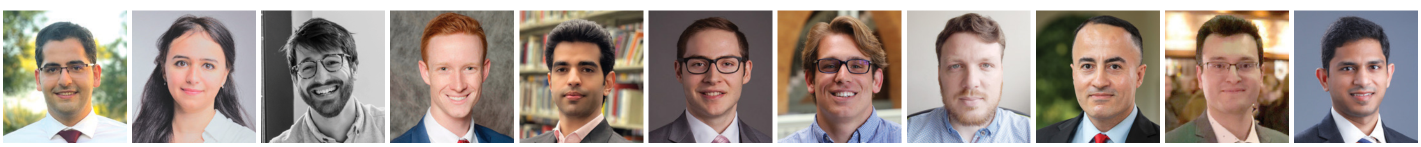
# Congratulations to the Recipients of SEI Futures Fund Scholarships to the Structures Congress in Atlanta

*We look forward to welcoming you!*

### Students

**Adetona Adediran** *S.M.ASCE, University of South Alabama*
**Rawan Al Naabi** *A.M.ASCE, Oregon State University*
**Wael Aloqaily** *S.M.ASCE, University of Delaware*
**Santiago Bertero** *Ing., S.M.ASCE, Virginia Tech*
**Jonathan Broyles** *A.M.ASCE, Pennsylvania State University*
**Edvard Bruun** *P.Eng, S.M.ASCE, Princeton University*
**Ariana Cabral Felix** *S.M.ASCE, University of Cincinnati*
**Giovanna Fusco** *S.M.ASCE, University of Connecticut*
**Fiz Hassan** *S.M.ASCE, University of Illinois at Urbana Champaign*
**Nafiseh Kiani** *S.M.ASCE, University of Miami*
**Brian Lassy** *S.M.ASCE, University of Connecticut*
**James Aaron Redus** *EIT, S.M.ASCE, Oregon State University*
**Sajjad Safari** *S.M.ASCE, University of Delaware*
**Babak Salarieh** *Ph.D., A.M.ASCE, University of Alabama*
**Naveen Senthil** *S.M.ASCE, Texas Tech University*
**Caleb Stevenson** *S.M.ASCE, Iowa State University*
**Margaret Sullivan-Miller** *S.M.ASCE, University of Cincinnati*
**Mohammad Syed** *S.M.ASCE, University at Buffalo*
**Michael Vaccaro** *S.M.ASCE, University of Connecticut*
**Thomas Vitalis** *S.M.ASCE, University of Massachusetts*

### Young Professionals



**Mohammad Abedin** *Ph.D., S.M.ASCE, Miami, FL*
**Reza Ameri** *Ph.D., S.M.ASCE, Seattle, WA*
**Rana Ayman** *Aff.M.ASCE, Cairo, Egypt*
**Daniel Bergsagel** *C.Eng, M.ASCE, Brooklyn, NY*
**Christopher Bird** *EIT, A.M.ASCE, Washington, DC*
**Mohamed Elhassan** *Aff.M.ASCE, Alexandria, VA*
**Saman Farhangdoust** *Ph.D., S.M.ASCE, Miami, FL*
**Jeremy Feist** *A.M.ASCE, Seattle, WA*
**Daniele Malomo** *Ph.D., P.E., M.ASCE, Montreal, Canada*
**Sohil Paudel** *A.M.ASCE, Boise, ID*
**Matthew Powell** *C.Eng, M.ASCE, Leeds, UK*
**Mehrdad Shafei Dizaji** *Ph.D., Aff.M.ASCE, Charlottesville, VA*
**Ali Shokrgozar** *S.M.ASCE, Pocatello, ID*
**Sanjeev Mohan Sri Balasu** *P.E., M.ASCE, Rocky Hill, CT*
**Ruoyang Wu** *Ph.D., P.E., S.M.ASCE, Herriman, UT*

# New Scholarship for Students to Electrical Transmission and Substation Structures Conference

*October 2-6 in Orlando*

Expand your career opportunities and connect with leaders. Apply by April 25 to participate in this important industry and conference that innovates for critical global infrastructure. The scholarship was made possible by the SEI Futures Fund in collaboration with the ASCE Foundation. **www.etsconference.org/program/student-scholarship**

## SEI Online

# NEW: Bridge Asset Management Collection

With 42% of bridges in the United States being over 50 years old and 7.5% of them labeled structurally deficient, it is time that scholars and practitioners take stock of the most recent developments in the fields of bridge asset management and maintenance, as a first step to improving America's bridges. ASCE's Special Collection brings together recent research on a number of key issues in these fields – such as bridge testing and inspection, climate change, and life cycle management – to help engineers and decision-makers make bridges safer. This collection is curated by Dan M. Frangopol, Dist.M.ASCE, Lehigh University, and Sriram Narasimhan, Ph.D., P.Eng (Ontario), M.ASCE, University of California, Los Angeles. Access at **https://ascelibrary.org/bridge_asset_management**.

Follow SEI on Social Media:

# Join a Coalition

The Coalition of American Structural Engineers (CASE) is a dedicated community in ACEC committed to advancing the business practices of structural engineering through education, networking, and the development of critical business resources. CASE is open to all ACEC members. Coalition membership is firm based and all firm employees are invited to take advantage of the membership.

ACEC COALITIONS

**For more information contact Michelle Kroeger at coalitions@acec.org**

## WHY CASE?

**Peer To Peer Information-Sharing**

Meet with other SE coalition members to share ideas on trending industry topics, including today's supply chain challenges, recruitment and retention plans, and updates on codes and standards, structural design and performance, legal specifications, and more. CASE has established relationships with NCSEA, SEI, and AISC.

**Networking with Firms in Your Discipline**

Coalition membership offers a variety of networking opportunities. Meet with other structural firm members to build relationships and share best practices – including free Coalition events, committee meetings, roundtables, and an online community.

**Educational Opportunities and Resources**

CASE membership gives your firm FREE access to practical resources and tools that will help you manage risk, draft smarter contracts, and learn sound business practices. Want to see the latest CASE Publications?
Visit **acec.org/case/news/publications**

**Advocacy**

CASE member firms have an active voice in ACEC's advocacy efforts. ACEC has an active role in today's key issues including the FAR Credit Clause, the Infrastructure Investment and Jobs Act (IIJA), Risk Management legislative priorities.

**Looking to make a further impact in your industry?**
**Join one of the CASE Committees:**

National Guidelines | Contracts | Toolkit | Programs and Communications

ACEC
www.acec.org/coalitions

Members of the ACEC Business Insurance Trust (BIT) have a portion of their coalition dues paid by the Trust.

***Your firm must be a member of ACEC to be eligible for coalition membership.**
**Not sure if your firm is a member of ACEC?**
**Contact coalitions@acec.org**

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# Invest in the future of our workforce. Donate to the CASE scholarship fund!

Your monetary support is vital in helping CASE and ACEC increase scholarships to those students who are the future of our industry. All donations toward the program may be eligible for tax deduction and you don't have to be an ACEC member to donate! Donate today: www.acecresearchinstitute.org/scholarships

Follow ACEC Coalitions on Twitter @ACECCoalitions.

# Event Reminders

## 2022 NASCC: The Steel Conference

*March 23-25, 2022, Denver, CO*

CASE will sponsor a presentation on delegated design at the upcoming Steel Conference in Denver. The presentation, *Delegated Design and the Engineer of Record*, with speaker Bruce F. Brothersen, Research Engineer at Vulcraft-Nucor, reviews the roles and responsibilities between the Engineer of Record and specialty engineer or specific product engineer. Included is a review of the IBC for direction and key aspects to follow.

For registration information, go to **www.aisc.org/aisc-events/2022-nascc-the-steel-conference**.


## ACEC Annual Convention and Legislative Summit

*May 22-25, 2022, Grand Hyatt, Washington, D.C.*

ACEC sponsors two major national meetings each year: the Annual Convention and the Fall Conference. National meetings provide attendees an opportunity to obtain information about issues that affect the industry through informative education, networking, and exhibits.

To register visit **https://www.acec.org/conferences/annual-convention**.

## Wanted: Engineers to Lead, Direct, Engage with CASE Committees!

If you are looking for ways to expand and strengthen your business skill set, look no further than serving on one (or more!) CASE Committees. Join us to sharpen your leadership skills and promote your talent and expertise to help guide CASE programs, services, and publications.

We currently have openings on all CASE Committees:

**Contracts** – The Committee is responsible for developing and maintaining contracts to assist practicing engineers with risk management.

**Guidelines** – The Committee is responsible for developing and maintaining national practice guidelines for structural engineers.

**Programs** – The Committee is responsible for developing program themes for conferences and sessions that enhance and highlight the structural engineering profession.

**Toolkit** – The Committee is responsible for developing and maintaining the tools related to CASE's Ten Foundations of Risk Management program.

To apply, your firm should:

- Be a current member of ACEC
- Be a member of the Coalition of American Structural Engineers (CASE); or be willing to join the Coalition
- Be able to attend the groups' usual face-to-face meetings each year: August, February (***hotel, travel partially reimbursable***)
- Be available to engage with the committees via email and video/conference call
- Have some specific experience and/or expertise to contribute to the group

Please submit the following information to (**mkroeger@acec.org**), subject line CASE Committee:

- Letter of interest indicating which committee
- Brief bio (no more than a page)

Thank you for your interest in contributing to advancing the structural engineering profession!

# Tips for Working Under an NDA

*By Stephen Murray*


Occasions may arise where parties collaborate on a project involving proprietary or otherwise sensitive information. To prevent that information from being misused, shared, and/or publicized, the parties often enter into a non-disclosure agreement (NDA). However, breaching an NDA can have serious consequences, even in cases of accidental misuse or disclosure. Below are some basic steps you can take to protect yourself and your company if you are working within the confines of an NDA.

## *Ask to See and Review the NDA*

Executives or counsel typically negotiate NDAs, but the terms most directly apply to the engineers or others working with the received confidential information. Nonetheless, those engineers often never actually see the agreement. If you anticipate receiving or using confidential information, ask to review a copy of the NDA. This practice can substantially reduce the possibility of inadvertent disclosure or accidental misuse of the information – it is easier to comply with obligations when you know what they are. Significant provisions to review include the definition of confidential information, the duties attached to receiving that information, and the length of time those duties will last. If there is something in the agreement you do not understand, ask for an explanation. This may come from an attorney for the company or someone involved in negotiating the agreement that hopefully better explains the contract's terms. For your own sake, be fully informed of your responsibilities for handling someone else's confidential information.

## *Set Automatic Calendar Reminders*

NDAs often include multiple dates. For example, although an NDA's basic terms could expire tomorrow, the parties may still have to avoid disclosing or using confidential information for another two years (or more). An NDA can also have a deadline for returning or destroying confidential information in the receiving party's possession (e.g., ten days after expiration). Setting automated reminders can prevent you from overlooking actions that need to be performed. Many agreements expire well after work has ended, so some actions might otherwise slip through the cracks because the NDA is no longer front of mind. For example, forgetting to return confidential material to the disclosing party can create adverse inferences in a potential dispute, either by implying that you used the information still in your possession or by providing an opportunity to mistakenly use it contrary to the agreement. An automated reminder can help you remember that the confidential information you used six months ago needs to go back to its owner or be destroyed. Of course, many companies have an attorney or some other designated individual collect that information when the time comes. Still, it does not hurt to protect yourself in case your company does not have the resources or they forget you had relevant information.

## *Keep Relevant Information Centralized and Safe*

Accidental disclosure or misuse is more likely to occur when access to confidential material is unfettered, such as when documents are lying precariously on a desk. Suppose you are responsible for handling confidential information. In that case, physical embodiments, e.g., printed documents, prototypes, samples, etc., should be kept in a single, preferably securable location, such as a lockable cabinet. If others require access to the information in your care, it may also be advisable to have them sign in and out when removing and returning the materials. That way, it can be easier to track who has what and where.

Digital information is trickier, but comparable procedures can be implemented. Many companies (particularly after Covid accelerated the transformation to remote work) now have centralized servers or cloud-based document storage options. If the disclosing party allows, it is preferable to set up a folder or other similar structure in the server or cloud and store (and have others store) documents referencing or relating to the confidential information in that location, rather than on your local hard drive. This reduces unnecessary and uncontrolled proliferation, but appropriate security safeguards should be implemented to prevent unauthorized outside access. Password protection of the folder or other types of restrictions should also be put in place to prevent access by particular internal users. Many document management software programs can limit folder access to select individuals or groups and wall off others. In some programs, the very existence of the file or folder may be invisible to those without proper credentials. It is much easier to maintain control over confidential information when it resides in one or two known and secured locations, as opposed to being scattered between offices or individual personal computers.

## *If You are Confused – Ask for Help*

Many NDAs require that confidential information be marked as such – for example, by placing a "Confidential" or similar label on appropriate documents. This allows you to clearly distinguish between information that needs protection and information that does not. But NDAs are usually very forgiving when the disclosing party either forgets to label or over-designates. It is in your own best interest to make sure a disclosing party follows the rules. If information is received that may only arguably be confidential and is not marked, or if non-confidential information appears to be improperly designated, bring it to the disclosing party's attention and get the issue resolved right away. Avoid a dispute that a jury must decide and make the disclosing party be clear upfront. ▪

*Stephen Murray is a Partner at the intellectual property firm of Panitch Schwarze Belisario & Nadel. He focuses on protecting intellectual property, particularly patents, for clients of all sizes ranging from individuals to multi-national corporations (**smurray@panitchlaw.com**).*

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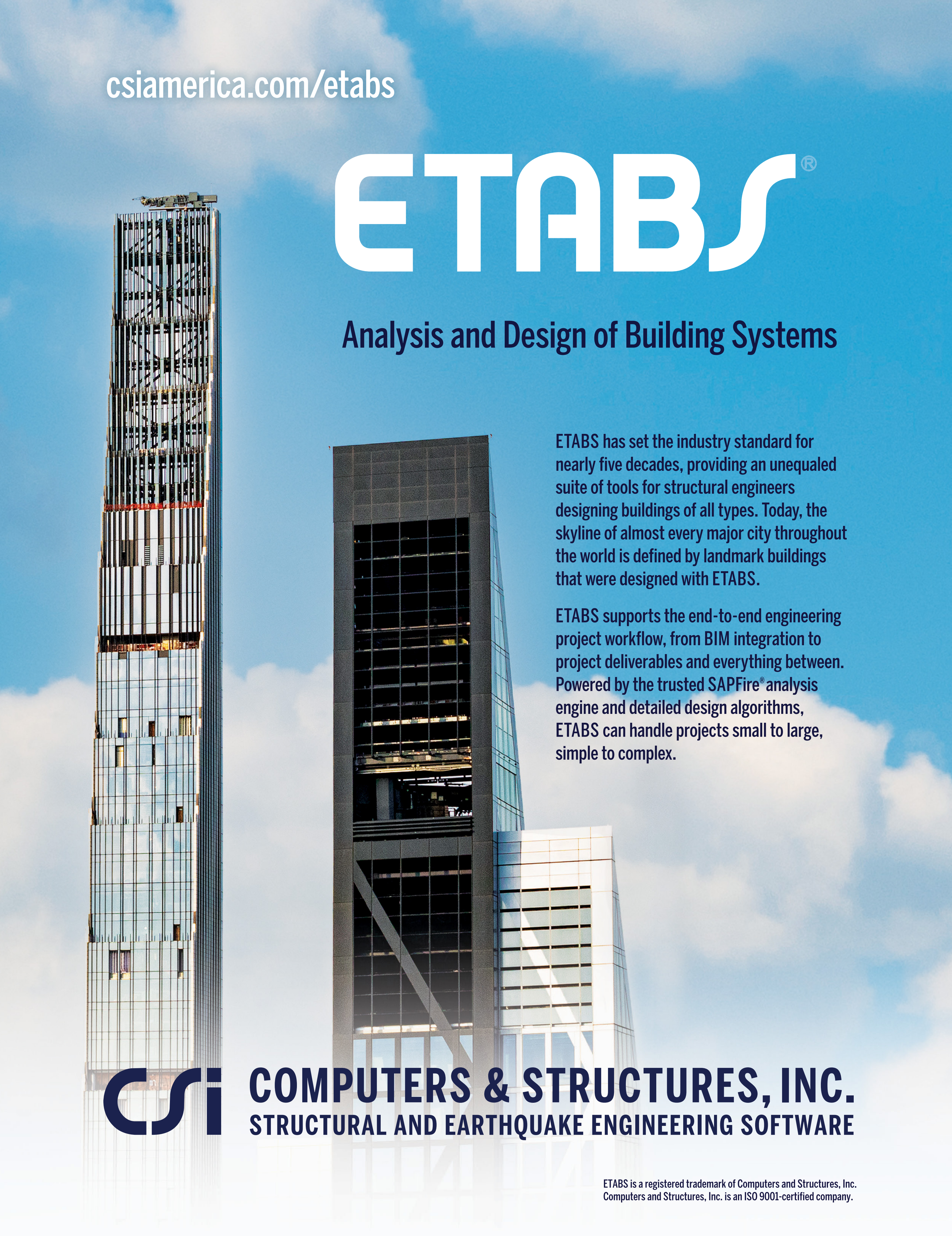
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