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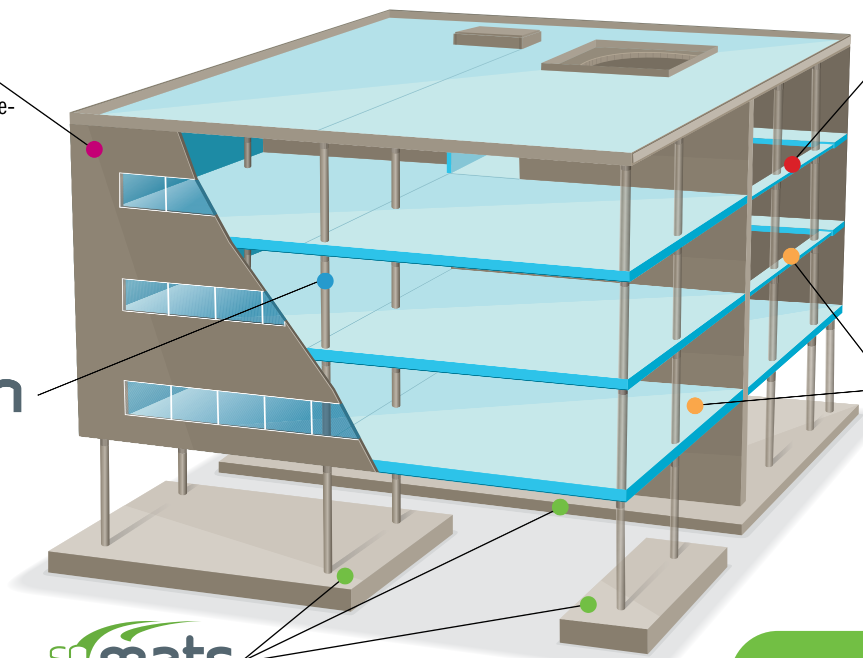

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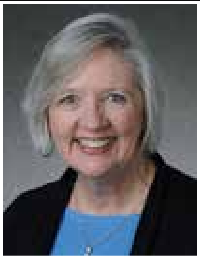
November 2016

On the cover The three-story connector lobby of the Cottonwood Cornerstone Center houses a curved glass walkway. Read about the structural challenges in the article on page 38.

Editorial | *Structural Engineering Licensure Coalition*

By Susan Jorgensen, P.E., NCSEA Treasurer



The pursuit of structural licensure in every jurisdiction is a challenge. It means getting all the stakeholders behind it, convincing the licensing board that it is in the best interest of the public, understanding the legislative process, gaining support from legislators to get the bill sponsored and passed, and making sure that the state governor will sign it. In addition to these daunting tasks, there are always those who do not see the benefits of structural licensure and some can be vocal about their opinion.

One of the recurring arguments is the fact that there are four separate structural engineering professional organizations – NCSEA, CASE, SEI, and SECB – with varying opinions on the details of structural licensure. It is not a valid argument, but it was hard to convince our opponents of that. In June 2012, individuals from these organizations, adamant about pursuing structural licensure in every state, came together to form the Structural Engineering Licensure Coalition (SELC). The mission of this initiative is to "serve as a unified voice for the structural engineering profession for the promotion of structural engineering licensure."

The coalition is led by a steering committee made up of two representatives, and an optional alternate, from each of the four organizations. The committee meets in person at the spring SEI Structures Congress and the fall NCSEA Structural Engineering Summit with the goal to further the efforts of developing structural licensure in each jurisdiction.

I have been helping to lead such efforts on behalf of NCSEA for almost ten years. The concern is that there are still many in our profession who do not understand the importance of what we are trying to accomplish, and are not providing support for the local Member Organizations (MOs). With an eye toward rectifying the situation, SELC has adopted the following positions:

1) SELC endorses the Model Law Structural Engineer (MLSE) standard developed by the National Council of Examiners for Engineering and Surveying (NCEES) as establishing the minimum set of qualifications for a licensed Structural Engineer (S.E.).
2) SELC advocates that jurisdictions require S.E. licensure for anyone who provides structural engineering services for designated structures. SELC recommends that each licensing board adopt rules to define appropriate thresholds for these structures.
3) SELC recognizes that, when S.E. licensure is enacted in each jurisdiction, it is important to ensure that an equitable transition process, as defined by the licensing board, is available for any individual who has been practicing structural engineering as a licensed Professional Engineer (P.E.).
4) SELC encourages all jurisdictions to incorporate these provisions into their current engineering licensure laws, adapting them to their unique individual situations. SELC supports the modification of existing P.E. statutes and regulations to implement S.E. licensure as a post-P.E. credential.

In simple terms, we want to establish licensure of structural engineers that recognizes qualifications already established, is required only for the design of structures designated by each licensing board, provides for an equitable transition process for professionals already licensed and practicing structural engineering, and works within the current engineering licensing laws. It is our sincere belief that the best way to protect the health, safety, and welfare of the public is to restrict the design of certain structures – perhaps defined by height, area, or other criteria that reflect increased risk – to those who are focused *entirely* on designing structures.

The structural engineering community has long realized the complexity of our profession, working with NCEES to develop the 16-hour Structural Engineering Principles & Practice Examination. The former SE I and SE II exams, as well as a few state-specific exams, were attempts to measure structural engineering proficiency beyond the four-hour structural module of the Civil Engineering exam. The two-day exam, first offered in 2011, tests the practitioner's experience and understanding of both lateral and vertical force-resisting systems – essential to the design of safe structures.

Tim Gilbert, the chair of the Structural Engineers Association of Ohio (SEAoO) Licensure Committee, has written a number of excellent articles for their newsletter in support of structural licensure. In his article from February 2015, entitled *Second Order Effects and Structural Licensure*, he writes, "In the instances where a structural design has the potential for significant impact on the public, we favor a requirement that the engineer has demonstrated sufficient proficiency in structural engineering. Structural licensure would provide a means for engineers to demonstrate proficiency in the subject to the public."

SELC has developed a website (**www.selicensure.org**) intended to be a gathering place for information related to structural licensure that may be used as a resource for MOs working on making changes within their states. The site is a work in progress, and we are always looking for more positive and supportive information to include. Should you be looking for any specific information and cannot find it, please contact us.

Although SELC was formed primarily for the pursuit of structural licensure, as we go forward, bringing together the four professional organizations that represent us could serve as an avenue for advocating any number of issues that affect our profession. With that in mind, it is my personal hope that this group will eventually become the Structural Engineering Leadership Coalition.▪

*Susan Jorgensen (**susiejorg315@comcast.net**) is the Quality Control Manager for Studio NYL, a structural engineering and façade design firm in Boulder, CO. She is currently the Treasurer on the NCSEA Board of Directors.*

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A Division of Copper Creek Companies, Inc.
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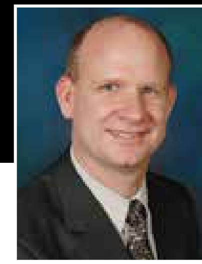
November 2016, Volume 23, Number 11
ISSN 1536-4283. Publications Agreement No. 40675118. Owned by the National Council of Structural Engineers Associations and published in cooperation with CASE and SEI monthly by C³ Ink. The publication is distributed free of charge to members of NCSEA, CASE and SEI; the non-member subscription rate is $75/yr domestic; $40/yr student; $90/yr Canada; $60/yr Canadian student; $135/yr foreign; $90/yr foreign student. For change of address or duplicate copies, contact your member organization(s) or email **subscriptions@STRUCTUREmag.org**. Note that if you do not notify your member organization, your address will revert back with their next database submittal. Any opinions expressed in STRUCTURE magazine are those of the author(s) and do not necessarily reflect the views of NCSEA, CASE, SEI, C³ Ink, or the STRUCTURE Editorial Board.

InFocus

Why is Counting Engineers So Difficult?

By Barry Arnold, P.E., S.E., SECB



Fundamental to every engineering degree is the requirement for advanced mathematics courses. Far beyond basic arithmetic, algebra, and trigonometry, the engineering curriculum requires, at a minimum, courses in differential, integral, and multivariable calculus, linear algebra, and differential equations. With such stringent requirements with respect to mathematical principles and problem-solving tools, it is interesting that counting structural and professional engineers is difficult for so many. It is hard to know if the source of the problem is the result of an unintentional error by well-meaning individuals, a result of an over-zealous marketing department, or indifference to state laws governing licensure.

Regardless of the origins of the problem, the fact is that each state has licensing laws which must be followed. Each state has established criteria a person must meet for that person to be able to use certain titles. Those requirements are listed in the Licensing Act.

Violations of the Act are too common and cause unnecessary confusion. Recent examples of misuse I have witnessed include:

- A registered architect announced during an interview that "I am also an engineer. I completed my education at XYZ University." He had completed an undergraduate university degree in engineering, but never obtained experience and did not take the required examinations which would have enabled him to use the title engineer.
- A website announcing a company employs over 1000 engineers. In fact, the company only had around 20 licensed engineers, and the remaining employees, although engaged in engineering-type work, were not licensed and therefore should not be counted as engineers.
- A government employee who is exempt from licensure and legally allowed to practice without being licensed insisting that she "Is a professional and does engineering work; and, therefore she is a professional engineer." Being exempt from licensure by state law and being allowed to practice engineering does not give a person the right to use titles that are protected and reserved for use by those who have met the requirements stipulated in the Licensing Act.
- An employee of a construction firm using the title "Project Engineer" and the abbreviation PE on correspondence. The individual did not meet any of the education, experience, or examination requirements for licensure and should not use the title Engineer.

Each state defines acts that constitute unlawful conduct concerning licensure. Usually included in the definition of wrongful conduct are acts such as using the title professional engineer, professional structural engineer, structural engineer, or any other words, letters, abbreviations, or designations which represent recognized professional engineering disciplines indicating that a person using them is a professional engineer or professional structural engineer, if the individual has not been licensed. Also, using terms like engineering, or structural engineering, or any similar words, letters, or abbreviations in marketing material, to describe the type of activity performed or offered to be performed, is considered unlawful conduct if the person has not been licensed under the Act.

Protected titles are a way of protecting the public. Protected titles may include:

- Professional Engineering Intern (EI): EI means a person who has graduated and received a bachelor or graduate degree from an engineering program, has passed the fundamentals of engineering examination and is engaged in obtaining the four years of qualifying experience for licensure under the direct supervision of a licensed professional engineer.
- Professional Engineer (PE): PE means a person licensed as a professional engineer. Beyond submitting an application, providing evidence of good moral character, and paying the required fees, to be licensed as a PE means that the person has graduated and received a bachelor or graduate degree, has successfully completed a program of qualifying experience, and has successfully passed the 8-hour NCEES Principles and Practice of Engineering (PE) examination.
- Professional Structural Engineer (SE): SE means a person licensed as a professional structural engineer. Beyond meeting the requirements of licensing as a PE, to be licensed as an SE means that the individual has completed an additional program of qualifying experience and successfully passed the 16-hour NCEES Structural Engineering examination or been granted equivalency due to grandfathering.

Each state has laws in place regarding the use of these (or similar) titles. These titles are PROTECTED by law and can only be employed by a person meeting all of the established requirements.

Please note that the above definitions are generic. The requirements and wording may vary from state to state. It is incumbent on each person performing, or offering to perform, engineering work to know the laws in the state where the project is located.

Penalties for violating state law may include receiving a citation, formal notice of non-compliance, fines ($500 to $10,000 depending on the severity of the offense), probation, postponed licensure, or suspension or revocation of existing licenses. Egregious violations may be prosecuted in civil court as a fraud. Consequences as a result of a violation of the code of ethics for misrepresentation may also be applicable.

Before you pursue work in any state, take the time to familiarize yourself with, and understand, the laws governing the practice of engineering. Because each state may interpret wording in their Licensing Act differently, contact the licensing board for clarifications.

To avoid problems, follow the established laws regarding protected titles with exactness. Be willing to report violations. I believe that protecting and defending our titles is essential to maintain the foundation of the engineering profession and, more importantly, the health, safety, and welfare of the public. What are your thoughts? Would you like to share your ideas? The discussion continues at **www.STRUCTUREmag.org**.▪

*Barry Arnold (**barrya@arwengineers.com**) is a Vice President at ARW Engineers in Ogden, Utah. He chairs the STRUCTURE magazine Editorial Board and is the Past President of NCSEA and a member of the NCSEA Structural Licensure Committee.*

A similar article was published in NCSEA's August, 2016 Structural Connection. Content is reprinted with permission.

Structural Design

design issues for structural engineers

AISC and Damage Tolerance Approaches

Part 2

By Paul W. McMullin, S.E., Ph.D.

Continuing on the foundation established in the last article (STRUCTURE, August 2016), let's now look at two fatigue design methodologies: AISC and Damage Tolerance. AISC is based on the safe life philosophy – if the engineer keeps the stresses low enough, the structure will perform adequately. It also assumes cracking occurs at the end of the structure's life. Damage Tolerance approaches the problem from the opposite perspective. It assumes the structure inherently has discontinuities in critical locations from the first day it is in use. These discontinuities are below the inspection threshold, but will grow as time goes on. The engineer designs toughness, redundancy, and inspection into the structure. This is done in a closed loop system, receiving feedback at critical stages in the structure's life.

AISC Fatigue Design

General Concepts

AISC fatigue design methodology is very similar to that found in AASHTO and AREMA. Key concepts of AISC fatigue design include:

- Fatigue design is not required if the structure will see less than 20,000 cycles, or when the stress range is below the threshold F_{TH}.
- Use service loads (allowable stress load combinations).
- The AISC provisions assume suitable corrosion protection.
- Calculating the number of cycles can at best be a guess. Talk to the operator and be conservative.

Stress Calculation

When calculating stresses, the following need to be considered:

- Use an elastic stress analysis.
- Include prying effects in bolts.
- Include the effect of eccentricities.

AASHTO – American Association of State Highway Transportation Officials
AISC – American Institute of Steel Construction
AREMA – American Railway Engineering and Maintenance-of-Way Association

- Ignore the stress concentration (the table values take this into account).

Stress range is calculated considering only the fluctuating stresses, not total stresses. Permanent stresses, such as dead loads, do not contribute to the fatigue stress range.

For example, if there is a 5 ksi cyclic load in combination with a 15 ksi dead load (*Figure 1a*), the stress range is only 5 ksi. It is possible to make the mistake that the stress range is 20 ksi, which would lead to a substantially heavier design.

Looking at another condition, if a 10 ksi fully-reversing stress exists but no permanent loads are present (*Figure 1b*), the stress range is 20 ksi. This is because we are adding peak-to-peak stresses. If we took the stress from zero to peak, we would underpredict our stress range by a factor of two.

Allowable Stress Range

Once the engineer has accurately calculated the stress range, they need to compare it to the allowable stress range. There are two ways to do this: calculate the stress range based on the number of cycles, or limit the stress to the threshold. A description of both methods follows.

Using an estimate of the number of cycles, the allowable stress range, F_{SR}, can be calculate based on the following equation:

$$F_{SR} = \left(\frac{C_f}{n_{SR}}\right)^{0.333} \geq F_{TH}$$

Where

C_f = factor from AISC tables
n_{SR} = number of cycles in design life
F_{TH} = fatigue threshold stress range

If the design of the structure is based on the fatigue threshold stress – which may be prudent for structures that may be in service well beyond their service

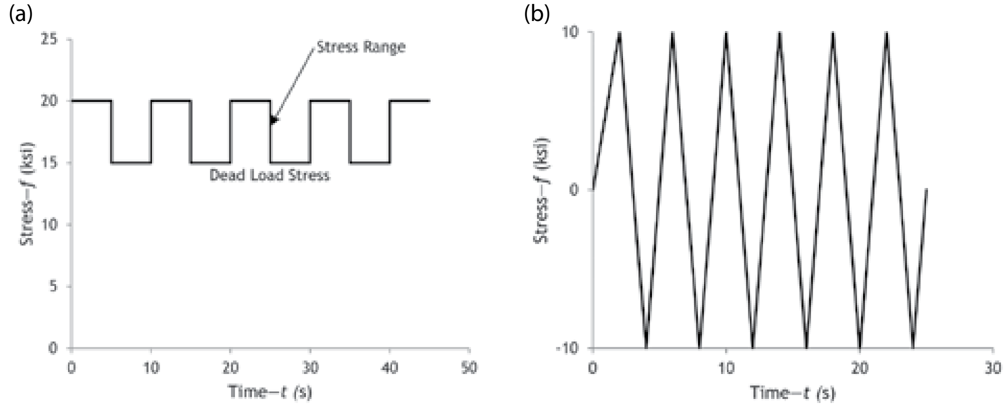


Figure 1. Stress range examples for (a) high permanent stress, and (b) fully reversing stresses.

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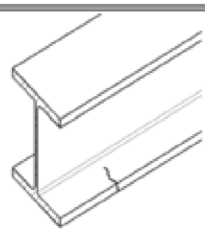
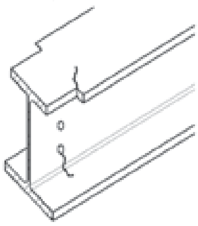
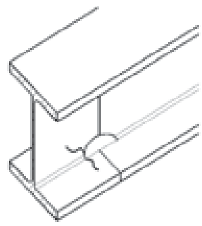
Sketch		Description	Stress Category	Constant C_f	Threshold F_{TH}
1		Base metal	A	$250x10^8$	24
2		Holes and re-entrant corners	B	$120x10^8$	16
3		Rolled sections with weld access holes	C	$44x10^8$	10

Figure 2. Representative AISC, AASHTO, or AREMA fatigue design data (after AISC).

life – the engineer simply sizes the component, so the stress range is below the threshold value from data similar to that shown in *Figure 2*. While this doesn't ensure an absence of cracking for the life of the structure, it is a place to start and can be combined with a robust inspection plan to ensure safe performance.

Damage Tolerance Approach

Damage tolerance flips the traditional design approach on its head. Rather than saying everything is great if the stresses are small enough, it assumes there is already a problem, and we need to design for it. The engineer must assume there is a discontinuity in the most critical point in the structure, and design for it. Below is an outline of how this is accomplished.

1) Inspect the critical locations in the structure after construction
2) Assume an inherent discontinuity at least the size of the threshold of detection
3) Use fracture mechanics to predict the critical crack size
4) Use fracture mechanics correlations to predict how long it will take the crack to reach its critical size
5) Inspect at intervals that can catch the crack before it reaches its critical size
6) Repair cracks or retire the structure/element from service

Fracture Mechanics

Before A.A. Griffith proposed his theory on crack propagation in glass, and Irwin made it useable and extended it to other materials in 1948, design techniques could not explicitly consider cracks. No one could analytically predict at what size a crack would propagate unstably.

Fracture mechanics received its start while Griffith was trying to understand the effect of surface treatment on the strength of cyclically loaded metal parts. To reduce the potential confusion plastic deformation might cause, he began testing glass because of its "brittle" behavior at room temperature. From his investigations from 1918 to 1920, Griffith proposed that a crack would propagate when the change in elastic energy with respect to crack length equaled the energy required for that increment of growth. From this concept, for a linear elastic material, Griffith derived the following relationship.

$$\underbrace{\sigma\sqrt{\pi c}}_{K} = \underbrace{\sqrt{2E\gamma}}_{K_c}$$

Where
σ = far-field stress
E = elastic modulus
γ = surface tension
c = half crack length of a center cracked specimen

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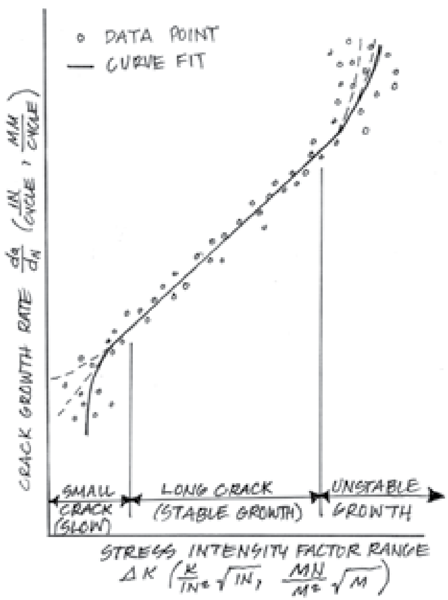

Figure 3. da/dN versus ΔK fatigue correlation curve.

When the left side equals the right side, fracture will occur. The only challenge with solving the equation is that gamma, γ, is difficult to obtain. So challenging, in fact, that nobody used the Griffith expression until George Irwin modified it while at the Naval Research Lab decades later. Irwin proposed that the right-hand side of the Griffith Equation could be experimentally determined, and called it fracture toughness. When the left-hand side of the Griffith Equation, known as the stress intensity factor K, equals the toughness K_c, the crack will propagate unstably (approximately ⅓ the speed of sound in the material). From this concept, many analysts have developed stress intensity solutions for a wide variety of geometry and loading conditions. These are available in a multitude of handbooks.

These developments opened a new world in predicting fracture behavior. It was no longer based solely on experience, and engineers could predict the behaviors of structures that hadn't been built yet. Regarding the functional application of fracture mechanics, Irwin stated:

> *The practical importance of fracture mechanics appears when one asks how much of each remedy is needed in quantitative terms, or when one attempts to link together prior estimates of stresses, crack sizes, and material toughness so as to calculate in advance a service load which will be safe relative to fracture propagation.* (Irwin 1958, p. 557)

The power of fracture mechanics is that it tells the designer the size of a crack-like discontinuity that a structure can withstand before final instability. One can then predict how long it will take for a fatigue crack to reach the critical size. The safe life philosophy cannot do this.

Fatigue Correlations

Extending fracture mechanics to fatigue, the engineer can relate the change in crack length to stress intensity factor range per cycle. This is accomplished through a *da/dN* versus *ΔK* curve, like the one in *Figure 3*. The curve is based on test data and because it is related to change in stress intensity factor, can be extended to different component and crack geometries.

By curve fitting the data to an equation, rearranging so *da* and *dN* are on opposite sides of the equation, and integrating with respect to crack size *a*, we determine the total life. The distinct advantage of presenting fatigue data in this manner is it explicitly considers initial discontinuity size.

Inspection

Inspection is to damage tolerance as energy methods are to statics. It allows the engineer to know what a structure's initial discontinuity state is due to fabrication and evaluate changes as the structure's ages. Inspection is the feedback in a closed loop system. It is, therefore, critical that we have a rational and robust inspection plan.

The key components of any inspection plan are:

1) what to look for
2) when to look
3) how to look
4) where to look
5) how often to look
6) the threshold of detection
7) the probability of detection

Let's briefly review how to look, or inspection methods. Non-destructive test methods can be broken into two groups: surface and internal. Each group has a unique place and ability to find discontinuity.

1) surface
   a) magnetic particle
   b) eddy current
   c) liquid penetrant
2) internal
   a) ultrasonic
   b) radiographic

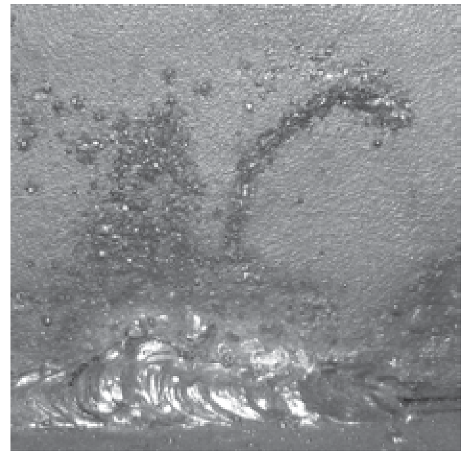

Figure 5. Arc strike on a structural steel member.

Magnetic particle and ultrasonic testing are the most common in civil structures to detect surface and internal cracks, respectively.

Coupling inspection technique with a threshold of detection, we can know what our initial crack size is for design. *Figure 4* shows the minimum and maximum crack sizes each inspection method can find.

Pulling damage tolerance together, we begin with design, which is based on an initial crack size, crack growth rate, and fracture toughness. We couple this closely to inspection, gaining feedback at key points in the structures life. This provides a clearer picture of what is going on, than just keeping our stresses low and hoping for the best.

Fabrication Considerations

Regardless of what design methodology we choose, prudent fabrication practice is key to well-performing structures. Let's review some key requirements from AISC and AWS D1.5 *Bridge Welding Code*.

AISC general fatigue requirements include:

- Remove transverse backing bars on full penetration welds. The author recommends removing all backing bars

Test Method	Discontinuity Sizes			
	Minimum		Maximum	
	(in)	(mm)	(in)	(mm)
Surface				
Liquid Penetrant	0.017	0.43	0.700	17.78
Magnetic Particle	0.039	0.99	0.237	6.02
Eddy Current	0.022	0.56	0.750	19.05
Internal				
Ultrasonic	0.014	0.36	0.265	6.73
Radiographic	0.024	0.61	0.729	18.52

Figure 4. Nondestructive testing crack detection thresholds.

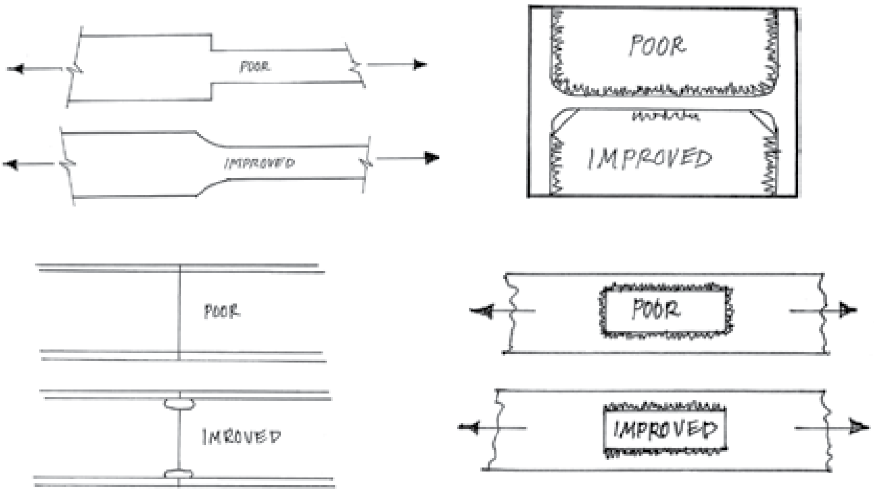

Figure 6. Poor and improved fatigue detailing examples.

– which can easily be accomplished by using copper or ceramic backing.

- Grind thermally cut edges to 1,000μin.
- Place a ⅜-inch radius on thermally cut edges.
- Pretension bolts.

AWS D1.5 requires the clear definition and requirements for the following:

- design
- workmanship
- technique
- procedure qualification
- inspection
- repair
- Fracture Control Plan for fabrication
  - contract documents
  - base metal
  - weld processes
  - consumables
  - procedures
  - certification & qualification
  - cutting
  - repair
  - straightening
  - tack welds
  - preheat & interpass temp
  - heat treatment
  - inspection

Remember, these are all fabrication requirements and do nothing to address in-service maintenance or inspection.

Two fabrication considerations are illustrative of the care the engineer needs to exercise in steel fabrication, hole punching and arc strikes.

When the fabricator punches holes, little cracks are left behind around the edge. Normally, this is not a problem. However, in fatigue sensitive structures, these cracks can grow. To address this, a fabricator can punch a hole smaller than the finished size, and ream to the final size or simply drill the holes.

When a welder accidentally drags the welding electrode across a steel part, it arcs and creates a trail of little puddles, like those in *Figure 5*. These leave behind a martensitic steel phase that is very hard and prone to cracking. Many great fatigue failures have started from such strikes. To correct them, we simply need to grind them out to sound metal and use magnetic particle testing to check for surface cracks.

Detailing

Let's end with a look at some detailing considerations. A notch in commercial construction often is not a problem, but in a fatigue sensitive structure it could be catastrophic. Let's look at four details, shown in *Figure 6* that with simple modifications can provide substantially longer fatigue life.

Notice how the changes center on smoothing out notches, reducing constraint, and lowering weld residual stresses.

Conclusion

This article has introduced fundamental concepts of traditional fatigue design and an alternate, more robust methodology, Damage Tolerance. When we couple initial crack sizes, toughness, fracture mechanics, and inspection, we are far better prepared to design for and evaluate cracks in our structures. We go from hoping for the best, to rationally predicting, monitoring, and repairing cracks in our structures – giving us more confidence in our engineering decisions. How nice is that?▪

For Reference

Irwin, G.R., (1958). "Fracture Mechanics." *Proc. Symposium on Naval Structural Mechanics*, VI, 557-594.

Building Blocks

updates and information on structural materials

Three Benefits of the Composite Steel Joist

By Angelo Nieves, P.E.

*Angelo Nieves is the Assistant Engineering Manager at the New Millennium Building Systems, Lake City, Florida Plant. He can be reached at **angelo.nieves@newmill.com**.*

Lighter, shallower, and cheaper: composite steel joist floor systems can save costs, save materials, and optimize space. These cost-saving reasons for designing a composite steel joist floor system are described in depth along with how these proven systems are best coordinated between the specifier and the joist manufacturer.

Lighter Materials Reduce Costs

When joists are supporting a concrete slab, the specification of composite joists – as opposed to standard Steel Joist Institute (SJI) non-composite joists – should be investigated, because they are lighter. Potential weight savings can be determined by using the project-specific design loads in conjunction with Load Resistance Factor Design (LRFD), composite steel joist weight tables, and the joist manufacturer's economic joist design table. Used in floor construction applications, composite joists improve the efficiency and advantages of non-composite joists. For example, accounting for the concrete slab adds extra strength, allowing the joist top chord to be smaller and therefore, reducing material, because the joist alone does not support full design loads.

Composite joists are typically designed using the LRFD design method to be consistent with the concrete slab design. For example, consider a joist spanning 40 feet, spaced at 4 feet on center, subject to 25-psf non-composite/construction dead load, 50-psf composite superimposed dead load and 100-psf composite live load. Using the appropriate LRFD factors, the total load to be carried by the joist is 1000 plf. A non-composite joist design shows a 32LH10 is the most economical at 18.1-plf joist weight. Conversely, a composite joist design shows the same 32-inch deep joist would weigh only 11.2 plf, which is a 38% savings in the material per joist (*Figure 1*).

Shear Studs Are Key Difference

Composite joists are manufactured open-web steel trusses used to support floors and roofs with a structural concrete slab. The main difference between composite and non-composite joists is that shear studs are attached through the composite steel deck to the joist top chord (*Figure 3, page 16*). A composite section is created between the slab and the joist top chord after the concrete is poured and cured. This composite action allows for the use of a lighter joist. The reduction in weight is due to designing the joist to withstand only construction loading with no strength contribution from the slab and then designing the joist top chord as a

Joist Span (ft.)		Total Load (plf) Factored	Service	Live Load (plf)		Joist Designation	Joist Wgt. (plf)
		LRFD	ASD	1/240	1/360		
40	F	190	127	96	64	20K3	5.5
	F	253	169	136	91	22K4	6.4
	F	277	185	163	109	24K4	6.5
	F	285	190	153	102	22K5	7.0
	F	312	208	183	122	24K5	7.2
	F	340	227	217	145	26K5	7.2
	F	370	247	235	157	26K6	7.8
	F	399	266	266	183	28K6	7.9
	F	412	275	261	174	26K7	8.3
	F	445	297	297	203	28K7	8.4
	F	478	319	319	234	30K7	8.6
	F	492	328	328	222	28K8	9.2
	F	529	353	353	256	30K8	9.3
	F	535	357	357	241	28K9	9.9
	F	576	384	384	278	30K9	10.2
	F	589	393	364	243	26K10	11.4
	F	636	424	424	284	28K10	11.6
	F	657	438	438	315	30K10	11.6
	F	711	474	474	368	32LH07	12.8
	F	771	514	514	400	32LH08	14.2
	F	793	529	529	360	28LH07	15.4
	F	967	645	645	500	32LH09	16.8
	F	1069	713	713	552	32LH10	18.1
	F	1146	764	764	514	28LH10	20.2
	F	1171	781	781	604	32LH11	19.9
	F	1228	819	819	549	28LH11	22.8
	F	1377	918	918	705	32LH12	22.8
	F	1407	938	938	628	28LH13	24.6
	F	1534	1023	1023	784	32LH13	25.8
	F	1581	1054	1054	807	32LH14	26.8
	F	1633	1089	1089	834	32LH15	27.0

Joist Span (ft.)	Joist Depth (in.)	Total Safe Factored Uniformly Distributed Joist Load in Pounds Per Linear Foot									
		TL	300	400	500	600	700	800	900	1000	1200
40	26	Wt(plf)	6.2	6.7	7.3	8.0	9.1	10.7	11.6	12.0	14.8
		W360(plf)	223	281	328	369	424	487	529	568	694
		N-ds	20-3/8"	22-3/8"	26-3/8"	30-3/8"	36-3/8"	24-1/2"	28-1/2"	30-1/2"	36-1/2"
		Ieff(in4)	333	418	488	550	631	725	788	845	1030
		Bridging	(1)X+(2)H	(1)X+(2)H	(1)X+(2)H	(1)X+(2)H	(1)X+(2)H	(2)H	(2)H	(2)H	(2)H
	28	Wt(plf)	6.1	6.5	7.2	8.0	8.9	10.4	11.4	11.6	14.3
		W360(plf)	228	293	349	420	457	521	580	615	748
		N-ds	20-3/8"	20-3/8"	24-3/8"	30-3/8"	32-3/8"	22-1/2"	26-1/2"	28-1/2"	34-1/2"
		Ieff(in4)	339	436	519	625	681	776	864	916	1110
		Bridging	(1)X+(2)H	(1)X+(2)H	(1)X+(2)H	(1)X+(2)H	(1)X+(2)H	(2)H	(2)H	(2)H	(2)H
	30	Wt(plf)	6.3	6.7	7.3	7.9	9.0	10.1	11.1	11.4	14.0
		W360(plf)	256	329	392	441	514	562	624	665	797
		N-ds	20-3/8"	20-3/8"	24-3/8"	28-3/8"	32-3/8"	22-1/2"	24-1/2"	26-1/2"	30-1/2"
		Ieff(in4)	382	490	584	657	765	837	930	991	1190
		Bridging	(1)X+(2)H	(1)X+(2)H	(1)X+(2)H	(1)X+(2)H	(1)X+(2)H	(2)H	(2)H	(2)H	(2)H
	32	Wt(plf)	6.2	6.4	7.0	7.6	8.2	9.0	10.9	11.2	13.6
		W360(plf)	286	333	400	465	523	571	652	707	843
		N-ds	20-3/8"	20-3/8"	22-3/8"	26-3/8"	30-3/8"	32-3/8"	22-1/2"	24-1/2"	28-1/2"
		Ieff(in4)	425	496	595	693	780	850	971	1050	1260
		Bridging	(1)X+(2)H	(1)X+(2)H	(1)X+(2)H	(1)X+(2)H	(1)X+(2)H	(1)X+(2)H	(2)H	(2)H	(2)H

Figure 1. An example of joist economic design tables and CJ weight tables.

24	CJ	2000	1000	400
Depth (in.)	Composite Joist Series	Total Factored Composite Design Load (plf)	Total Factored Composite Live Load (plf)	Total Factored Composite Dead Load (plf)

Figure 2. Definition of composite joist designation.

composite section with the slab to withstand the full-design loading after the concrete has cured. Conversely, non-composite joists are designed to support the full design loading, assuming there is no strength contribution from the concrete slab because there are no shear studs present. Using this design methodology allows for composite joists to build on the efficiency and benefits inherent in using open-web steel joist construction.

Optimize Space Using Shallower Floors

Using a composite joist narrows the floor, creating more headroom, often with adequate MEP routing options through the joist. Also, shear studs increase joist strength, thereby expanding the span-to-depth ratio limit to 30 times the joist depth, compared to 24 times the joist depth for non-composite joists. The increase in headroom can be achieved with no loss in the length of the joist span. For example, a non-composite 24-inch deep joist can span up to 48 feet; however, a 20-inch deep composite joist can also span 48 feet. The shallower composite joist floor may still accommodate MEP integration. If larger duct penetrations are required, contact the joist manufacturer to determine the feasibility and options available for special web layouts to accommodate duct penetrations. For the same example above, if the building design required limiting the joist depth to 28-inch deep, a non-composite joist would weigh approximately 20.2 plf, while a composite joist would weigh approximately 11.6 plf. Therefore, using a composite design in this situation would allow for an additional 4 inches more of headroom and would only slightly increase the joist weight as compared to a non-composite design.

Reduce Steel and Related Costs

Another composite joist benefit is the ability to space joists farther apart than non-composite joist framing. Fewer joists can often translate to reductions in steel cost, manufacturing cost, shipping costs, and erection costs. Joists in a floor application are generally spaced at 2 feet on center. It is not uncommon for composite joists to be spaced 4 or 5 feet on center and beyond. Minimum chord width and thickness requirements must be met depending on shear stud diameter. That said, the objective is to space joists farther apart than typical floor joist framing to assure the joist top chord is fully utilized for maximum cost savings. Any cost savings expected by using composite joists is lost if they are spaced too close together.

Responsibilities of the Specifying Professional

Understanding what is expected of the specifying professional and the joist manufacturer when electing to use a composite joist system guarantees a smooth project life cycle. Not only will this minimize questions during the approval process, but complete design information allows the joist manufacturer to provide an accurate price during the quoting phase. Most importantly, clear communication reduces the likelihood of change orders due to missing information during the drawing approval process.

All Design Loading

The construction requirements between the specifier and the joist manufacturer are similar to those on a conventional joist project. However, due to the special nature of composite joists, additional information is required as outlined in the SJI *Code of Standard Practice for Composite Steel Joists*. A breakdown of all loading to be considered in the joist design is critical information for the joist manufacturer. Since the joist is designed for two different loading situations – during construction and after construction – a breakdown of the dead and live loading for each phase is required.

The loading that the joist is subjected to consists of the non-composite construction dead load and the construction live load. Each of these non-composite loads should include any loading the joist is expected to carry prior to the concrete slab curing. This includes, but is not limited to, the joist self-weight, the wet-concrete weight, and any equipment and personnel loads expected during the pouring of the slab. The second phase of loading required is the

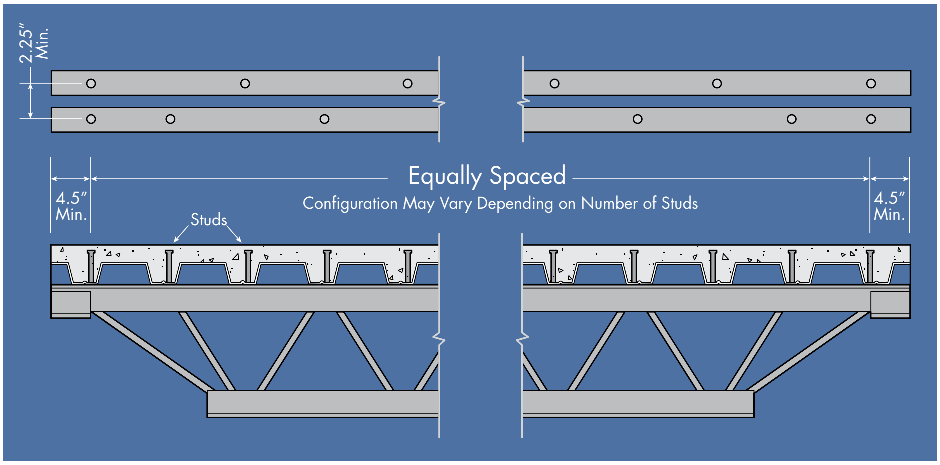

Figure 3. Cross-section of a composite joist top chord with shear stud layout.

composite dead load and composite live load. These loads would typically include the building code prescribed live loads and the total of the non-composite dead load plus any expected superimposed dead loads not accounted for in the construction phase. Additionally, if there are to be any considerations for concentrated loads, axial loads, or end moments, these must be shown in the drawings, in the notes, or on a load diagram. Explicitly displaying this loading information on the structural plans facilitates accurate and economical joist design.

Knowing the best way to represent composite joist design criteria on structural drawings is just as important as knowing what information must be provided. The most direct way to convey design criteria is to display it all in one table including slab, deck, and joist design requirements. Also, showing joist designations on the plan, in the form required for composite joists, avoids confusion for the joist manufacturer. The SJI format for a composite joist designation is 24CJ2000/1000/400. See *Figure 2* for the definition of joist designation.

It is permissible to supply service loads for Allowable Stress Design (ASD) joist design. However, it must be clearly indicated on the drawings that service level loads are provided in place of factored LRFD loads. In the SJI *Code of Standard Practice for Composite Steel Joists*, there is a composite joist design parameter checklist to assist specifying professionals in identifying information required by the joist manufacturer. Using this checklist as a guide and representing this information on structural drawings ensures that the joist manufacturer has the design criteria necessary to quote and design a composite joist framing system.

Manage Camber and Vibration Challenges

Camber specification is often overlooked on composite joists. SJI recommends the joist be cambered for a minimum of 100% of the non-composite dead load. The inclusion of a camber results in the joist being flat after the concrete slab has cured. If a flat joist under a different loading condition is desired, the joist manufacturer must be advised on which loading to use in determining joist camber. For example, a joist is cambered for 100% non-composite dead load + 50% composite dead load + 10% composite live load. Although the camber is a function of many variables, it is typically in line with the SJI standard camber published in the SJI catalog.

Vibration analysis is also an important design factor when using composite joists in a floor application. The specifying professional is responsible for conducting the required vibration design analysis of the floor system to verify the floor framing meets the project requirements. See the sidebar for additional resources on joist vibration.

For more information on vibration considerations, contact the joist manufacturer. They are able to provide direction on conducting vibration analysis. However, final confirmation of the framing adequacy is the responsibility of the specifying professional.

Responsibilities of the Joist Manufacturer

Similar to a conventional project, the joist manufacturer provides joist placement plans containing all relevant information required for erection. Due to the nature of composite joist design, all design criteria is represented on the joist plans. This includes, but is not limited to, the design loading, camber, shear stud information, and slab information provided by the specifying professional. Shear stud criteria includes the diameter, quantity, and a simple shear stud layout (*Figure 3*). Shear stud sizes vary from ⅜-inch ø to ¾-inch ø. All of this information is typically provided in tables displaying the total quantity of studs required for each joist. Note, shear studs are not furnished by the joist manufacturer.

In Conclusion: Communicate

Providing clear expectations and open lines of communication between the specifying professional and the joist manufacturer leads to a seamless design process, from beginning to end. Additionally, there are significant cost savings due to the three benefits of using composite joists: materials reduction, shallower structural floor framing, and the savings realized in the erection phase. Consider composite steel joists when designing your next floor system to improve floor strength and reduce costs.▪

Steel Joist Institute (SJI) Additional Information

- The SJI Standard Specifications for Composite Steel Joists **https://steeljoist.org/ansi** provides guidance on allowable duct sizes based on joist depth.
- Also available through SJI are design tools to assist in floor vibration analysis. Visit **www.steeljoist.org/design_tools** for more information on available design tools.
- SJI provides direction on conducting vibration analysis of floor systems in Technical Digest number 5. **https://steeljoist.org/product-category/publications**
- Introduced by the SJI in 2007, composite steel joists or CJ-Series joists take the efficiency of steel joists and improve upon it by accounting for the composite action between the concrete slab and joist top chord.

Thickness for Passive Fire Protection Coatings

Why Extrapolated Data Won't Work on Small Structural Steel Sections

By Bob Glendenning

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In a commercial building fire, the fate of the structure – and the safety of people in, on, or around the structure – may all come down to a layer of protective intumescent coating. However, how can structural engineers be sure they have used the right amount of coating material to provide the necessary fire protection? It all comes down to complying with available specification data, and not making assumptions when data for a specific steel section profile is unavailable.

Structural steel under load can quickly lose strength in a fire, eventually reaching a critical failure temperature at which it could collapse and possibly bring down large building sections or potentially the entire structure. To delay, and hopefully avoid such catastrophic losses, engineers and architects need to specify some form of passive fire protection, which usually includes cementitious fire-resistive materials, intumescent coatings, or a combination of both. These passive materials provide fire resistance, up to a certain number of minutes, while a fire is contained.

Cementitious materials provide a physical barrier of gypsum or cement to slow down the transfer of heat to the steel underneath.

Intumescent coatings work differently. They react chemically in fire, swelling to approximately 50 times their dry film thickness (DFT). The coatings form a char that expands with heat exposure and acts as insulation to reduce the rate of heat transfer to the structural steel. This extends the time for a given steel section to reach its critical failure temperature.

The required intumescent coating DFT to achieve a specified fire-resistance rating varies with the size of each structural steel member. Small, lightweight sections need a higher DFT to achieve the desired protection compared to larger, heavier sections. Underwriters Laboratories (UL) lists the required coating DFTs for structural steel members in its *Fire Tests of Building Construction and Materials* specification (ANSI/UL 263 (ASTM E-119)). However, the specification does not include every possible size, leaving data gaps for very small and very large sections.

To overcome the lack of sufficient UL 263-listed data for a particular steel section, some coatings suppliers engage in the potentially hazardous practice of extrapolating data to recommend an intumescent coating DFT. However, extrapolated DFT data is outside the UL certification program's scope, and using this noncompliant data could lead to two similarly dangerous scenarios:

1) The steel section may not have a sufficient coating DFT, which means it will not achieve its desired fire resistance.
2) The steel section may have a coating DFT that's too high, in which case the weight of the expanded char could cause the coating to delaminate and fall off, exposing the steel directly to fire with no protection in place.

Given these hazards, UL has stated that it is not safe to make assumptions about intumescent coating thicknesses. Therefore, when structural engineers encounter steel section sizes outside of UL's listing, they need to work with a coatings supplier to find a safe, workable alternative.

What Leads to Data Extrapolation

Building fire resistance requirements vary based on several considerations, including building codes, the structure's design, insurance regulations, and other factors. Common fire-resistance ratings include half-hour increments from 60 to 180 minutes.

While attempting to meet these ratings, engineers are also trying to minimize building material costs by creating sound designs with lightweight materials. In doing so, they may not realize that particular steel sections are smaller than those tested and listed in UL 263. No data exists to confirm whether those sections can meet the defined fire-resistance rating with a certain intumescent coating DFT applied. In such cases, some coatings suppliers may look at the closest size listed and assume that a correlative percentage of added coating provides sufficient protection. However, UL has determined that this extrapolated data is noncompliant, stating in its *The Fire & Security Authority* publication (2014, Issue 2) and its BXUV/CDWX guide for UL 263-compliant fire-resistance ratings that the average intumescent coating DFT "should not exceed the maximum thickness published in the individual [steel section] designs."

When faced with unavailable data, an engineer's best solution is to either consider an alternate steel size or profile or use more advanced fire engineering principles that consider how much of the steel strength supports the structure and how much reserve strength is available to resist fire. The latter option is deserving of a stand-alone article, so we will focus on specifying different steel sections.

UL 263 Steel Section Data Explained

It is important to remember two key points when determining intumescent coating DFT requirements: 1) Different UL listing categories have different test and pass criteria, and 2) different steel member shapes and orientations have different coating requirements. Therefore, it could be unsafe to use a maximum thickness from one UL category listing on another listing.

Each steel section has a "section factor" that helps to determine the intumescent coating DFT

Table 1. Fire-resistance ratings for Column Y UL 263 listing (fully exposed I-beam column).

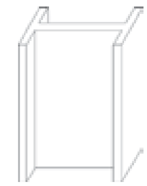
	**Column Size**	**W/D**	**Required DFT (inches) for Fire Rating Duration**		
			60 Minutes	**90 Minutes**	**120 Minutes**
	W6x12	0.45	0.129	0.247	N/A
	W10x39	0.78	0.072	0.134	0.198
	W10x49	0.84	0.067	0.126	0.185

Table 2. Fire-resistance ratings for Beam N UL 263 listing (I-beam covered by concrete on top flange).

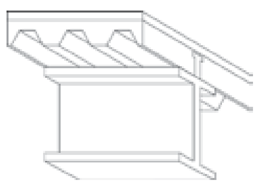
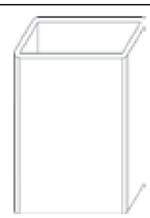
	**Beam Size**	**W/D**	**Required DFT (inches) for Fire Rating Duration**		
			60 Minutes	**90 Minutes**	**120 Minutes**
	W6x12	0.53	0.093	0.118	0.192
	W10x39	0.93	0.076	0.099	0.161
	W10x49	1.01	0.072	0.095	0.154

Table 3. Fire-resistance ratings for HSS Column Y UL 263 listing (fully exposed hollow column).

	**HSS Tube Size**	**A/P**	**W/D Equivalent**	**Required DFT (inches) for Fire Rating Duration**		
				60 Minutes	**90 Minutes**	**120 Minutes**
	3 x 2 x ¼	0.21	0.72	0.149	0.223	0.331
	10 x 10 x ¼	0.23	0.78	0.137	0.209	0.309
	10 x 4 x ⅜	0.34	1.157	0.099	0.162	0.239

required to meet various fire-resistance ratings. The section factor is a ratio that differs based on the style of the steel section and its exposure to fire. An I-beam (or H-beam) section uses the ratio "W/D," while a hollow structural section (HSS) uses "A/P." "W" is the weight of the section (in pounds/foot). "A" is the cross-sectional area of all sides of the HSS (in inches). "D" and "P" both represent the heated perimeter of the section (in inches), or the total square area that would be in contact with fire.

For a fully exposed I-beam (*Table 1*), D is the entire surface area of the section. For a similar section that's in contact with or partially encased by another material (e.g., a steel beam supporting a concrete ceiling/slab above (*Table 2*)), D only includes the surface area of the steel that is not in contact with the other material. The other material serves as a heat sink, which offers some fire resistance itself. For an HSS (*Table 3*), A is the entire surface area of the section, less any areas in contact with heat sinks.

Dividing the weight (W) or area (A) by the heated perimeter (D or P) provides a ratio (W/D or A/P) that represents how quickly the steel heats up in a fire. Converting A/P to W/D enables a direct comparison of I-beam and HSS sections. A larger ratio indicates that the steel section requires less fire protection (or mils of DFT). A smaller ratio means it needs more fire protection. *Tables 1*, *2*, and *3* include a few examples that show how the smaller W/D and A/P ratios at the top require a greater coating DFT, as well as how the DFT requirement increases with longer fire rating durations.

Let's make some direct comparisons to demonstrate why you cannot use W/D and DFT data from one category listing to the next, even when the steel section is the same size. Looking at the W10x39 Beam N size in *Table 2*, the required intumescent coating thickness for a 120-minute fire rating is 161 mils DFT. The same W10x39 Column Y size in *Table 1* has a smaller W/D ratio, which equates to higher DFT requirements. For the same 120-minute rating, the coating must have a DFT of 198 mils, which is 23% greater than the Beam N requirement. While the two sections are the same size, their heated perimeter is much different because Beam N is in contact with concrete on one face.

Moving to a similarly sized HSS column (*Table 3*), the requirements are drastically different, as HSS members usually require significantly higher intumescent coating DFTs due to their structural profile. The A/P ratio has been converted to W/D for comparison. Here, a 10.0 x 10.0 x ¼ HSS Column Y with the same column section factor and a similar size to a W10x39 Column Y (both are 10 inches deep and have a similar weight per foot) requires a 309-mil DFT for a 120-minute fire rating. This is 92% and 56% greater than the W10x39 Beam N and Column Y DFT requirements, respectively.

Why Data Extrapolation Doesn't Work

The available UL 263 data has limitations on the lower and upper ends of steel member sizes because UL has either not tested those sections or has determined they are not able to be protected using intumescent coatings. If a coatings supplier extrapolates data beyond those limits, it runs the risk of recommending a DFT that is either too low or too high. Both scenarios can result in having insufficient fire protection. Still, the UL 263 specification offers some flexibility in specifying coating thicknesses for any size steel sections in between the lower and upper limits.

In its revised fire-resistance rating guidance documents, UL notes that the following scenarios are acceptable:

- Using the minimum listed coating DFT for a specific beam size (specific W/D) on a larger steel section (greater W/D) that has a greater heat sink than the listed steel section
- Substituting a steel member for a heavier weight (greater W/D) section using the same specified coating thickness

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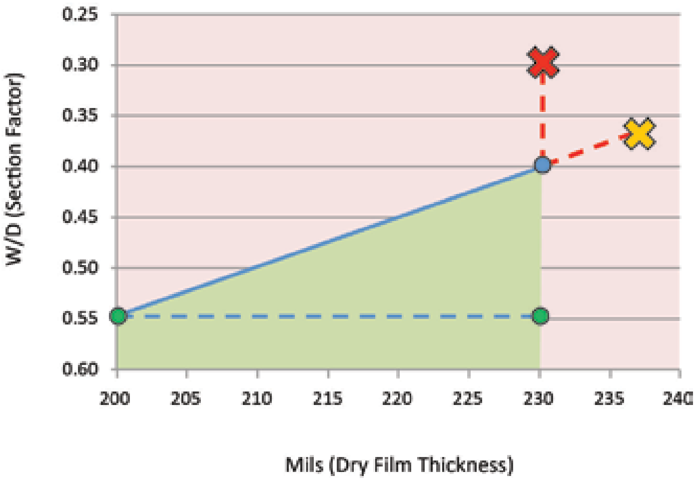

Figure 1. Intumescent Coating DFTs for 120-Minute Fire Rating. The lightest steel section listed in UL 263 has a W/D ratio of 0.40 W/D (blue dot). Use any point on the blue line or within the green area to determine the appropriate DFT for this and larger-sized steel members. Don't extrapolate data in the direction of the red or orange Xs.

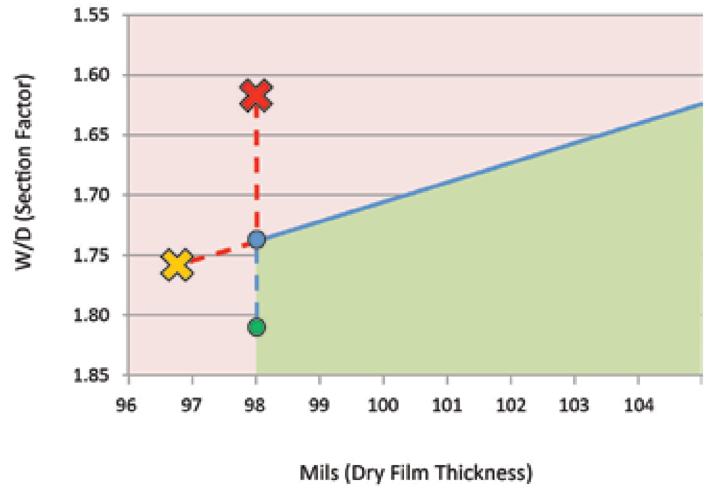

Figure 2. Intumescent Coating DFTs for 120-Minute Fire Rating. The heaviest steel section listed in UL 263 has a W/D ratio of 1.74 (blue dot). Any DFT on the blue line or within the green area is acceptable. It is not acceptable to extrapolate in the direction of the red or orange Xs.

UL also notes that the following scenarios are not acceptable:

- Using a coating DFT specified for a larger steel section to cover a smaller steel section that has a lower W/D than is listed
- Substituting a steel member for a lighter weight (lower W/D) section using the same specified coating DFT

Figures 1 and *2* demonstrate these points. In both diagrams, you need to stay within the green areas and keep out of the red areas when specifying the intumescent coating DFT for a given steel section W/D ratio. Any point on or below the blue lines is OK.

The blue lines represent the maximum allowed data points for steel sections listed in UL 263, based on what UL has tested. In *Figure 1*, the blue line terminates at a W/D ratio of 0.40, as that is the lightest steel listed in UL 263. The graph plots the required coating DFT on the X-axis against the steel section's W/D ratio on the Y-axis. A steel section with a W/D ratio of 0.40 requires a 230-mil DFT (see the blue dot) for a 120-minute fire resistance. A section with a 0.55 W/D ratio requires a minimum DFT of 200 mils (left-hand green dot), but it could be coated up to a 230-mil DFT without worry (right-hand green dot). For either of these sections, the DFT cannot exceed 230 mils because that data is not included in UL's listing. For this reason, specifiers cannot extrapolate the data to a lower W/D ratio (red X) or a higher DFT (orange X). Per UL guidelines, such extrapolation is not acceptable.

The same principle holds true when looking at stronger/heavier steel sections. In *Figure 2*, the W/D ratio of 1.74 (blue dot) is the lowest listed in UL 263. The 120-minute DFT requirement at this ratio is 98 mils. UL permits specifiers to coat sections with a greater W/D ratio – for example, 1.8 (green dot) – with the same minimum 98-mil DFT. However, because UL has not tested sections beyond the 1.74 W/D ratio, it does not permit specifiers to extrapolate a reduced DFT for stronger steel sections (orange X). Also, UL does not allow specifiers to extrapolate data for lighter steel sections (red X). Instead, specifiers must move up the blue line to match a lower W/D ratio with the correct minimum DFT.

Overcoming Data Extrapolation

UL does not condone the practice of extrapolating UL 263 data, and the organization is reminding the industry to avoid the practice. UL published an updated position on the issue in 2014 and added language to the UL 263 specification. Also, UL is scheduled to publish a new best practice guide in 2017 that will include even stronger language against using extrapolated data.

When designing a commercial building, structural engineers and architects may be unaware that they need to avoid using certain steel section sizes that are not tested for passive fire protection using intumescent coatings. The scenario is likely to happen, as building designers often choose steel beams and columns that are as lightweight as possible to save costs, yet UL's published data does not include a range of smaller, as well as larger, steel member sizes. When encountering this situation, engineers and architects need to examine the available UL 263 steel section sizes to come up with a solution. A credible coatings supplier can also assist in finding a safe, proven alternative solution.▪

A Point about Topcoats

After finalizing intumescent coating specifications for a structure, all structural steel members need to be coated with the appropriate material thickness. The coating process can take place off-site in a controlled facility or in the field. Applicators use a wet film gauge – a comb-like gauge with different depth prongs – to confirm the applied wet film thickness (WFT) per the coating manufacturer's guidelines. After curing, applicators can use an electronic gauge to determine the resulting DFT. If it is not sufficient, applicators apply more coating material to reach the specified DFT.

For exposed structural steel, architects often choose to apply a topcoat to intumescent coatings for a more aesthetically pleasing finish. They may also need to cover non-exposed steel sections with a protective coating in areas where durability is a concern, such as areas exposed to weathering or wet/dry cycling. Any cured topcoat added on top of an intumescent coating adds more DFT to the structure. However, because the intumescent coating material has already been built to the proper DFT, this added material does not push a steel section out of UL's specification. Still, it is important to note that a topcoat may eventually need to be recoated. Adding too many layers of topcoat material can create a situation in which the topcoat thickness is too much for the intumescent coating underneath to activate in a fire. The parties involved need to plan carefully to mitigate this situation.

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PROFESSIONAL ISSUES

issues affecting the structural engineering profession

Forward Thinking Engineers Are Automating Calculations, Are You?

By Stuart Broome, P.E.

*Stuart Broome (**stuart.broome@trimble.com**) is Business Manager for Engineering at Trimble.*

Increasingly, the design and construction industry is relying on Building Information Modeling (BIM) to conceptualize, plan, detail, create, and guide the building of structures. As a result, the industry as a whole is moving toward more efficient processes. Surprisingly, structural calculations have not evolved in the same way. Over the years, almost without planning, engineers have settled into the practice of using a combination of handwritten calculations and spreadsheets in design. However, why do engineers continue to rely on time-consuming hand calculations and cumbersome spreadsheets?

How Did We Get Here?

The use of spreadsheets was born out of a desire for greater efficiency. Spreadsheets are often created on a very ad-hoc basis with little or no planning from higher management and tend to start as small "scratchpad" applications that develop into much larger design tools over time. While spreadsheets might work just fine for the finance side of business, they were not built for the complex calculations that engineers must perform and communicate.

In some parts of the world, all calculations are required to be submitted to the local government for checking. There's a greater emphasis on calculations. In many countries, such as the UK, this has driven the adoption of technology and the desire to move away from scribbled hand calculations and spreadsheets. Some might say that the lack of a similar requirement to backup designs with calculations in many parts of the U.S. has stifled progress.

The Problems with Spreadsheets

The ease with which even an inexperienced user can enter data and calculations into a spreadsheet, and very quickly produce results, can lead to the belief that spreadsheets are inherently easy to use. However, as the requirements of a spreadsheet increase so does the complexity.

Well-Intentioned but Erroneous

While spreadsheets can be useful when implemented and used correctly, a study by Professor Ray Panko of the University of Hawaii (**www.marketwatch.com/story/88-of-spreadsheets-have-errors-2013-04-17**) showed that close to 90 percent of spreadsheets contained errors.

Quality Assurance

Completed spreadsheets often include many pages containing hundreds or even thousands of calculations. The problem with spreadsheets is that all those calculations are hidden in the formulae and cross-referenced within the cells of the spreadsheet. While it is possible to examine the formula in each cell one at a time, understanding how an entire spreadsheet works is often a monumental task. This makes full quality assurance tough to achieve – something that is essential in the design environment.

Collaboration and Knowledge Sharing

As spreadsheets are written by different staff members for their personal use, they are often difficult for others to use. It would seem reasonable that if someone has written a useful spreadsheet that there are likely to be others within the same organization who would benefit from it too, but that is not typically the case.

The Solution

Fortunately, technology is available today that can automate repetitive structural and civil calculations to increase productivity and minimize errors. Engineers in other countries and some in the U.S. are already utilizing technology to drive efficiency either by developing their own small scale software to assist with repetitive calculations or by using programs developed by others. This is revolutionizing their processes for creating easy to use, high quality and accurate calculations that they can share with others.

Civil engineering firms can take advantage of automated calculations to improve workflow, increase business efficiency and expand the scale of projects, making a company more competitive. Replacing a combination of hand calculations, spreadsheets and various computer software programs with an all-inclusive, commercially available software package places all data in a consistent editable format. With these programs, firms can spend far less time making revisions and engineers can write their own unique calculations. Calculations are saved for multiple uses, on regular projects, and by any team member.

Because design codes are constantly changing, using a comprehensive software solution also ensures engineers can leverage the detailed output to learn how to use the most recent edition of a particular design code.

Taking the leap

Engineers looking to move into the 21st century with calculation production should look for a single solution that provides the capabilities to automate component design, electronically create calculations, and produce output in a way that is very transparent. With this kind of solution, anyone checking those calculations can understand and follow them.

There has never been a better time to consider adopting technology to automate structural calculations. The initial investment is likely negligible compared to the potential return so the business case is usually transparent. Why would you want to delay improving processes to save time and reduce costs?■

How Familiar are You with the IEBC?

By Chris Kimball, S.E., P.E., MCP, CBO

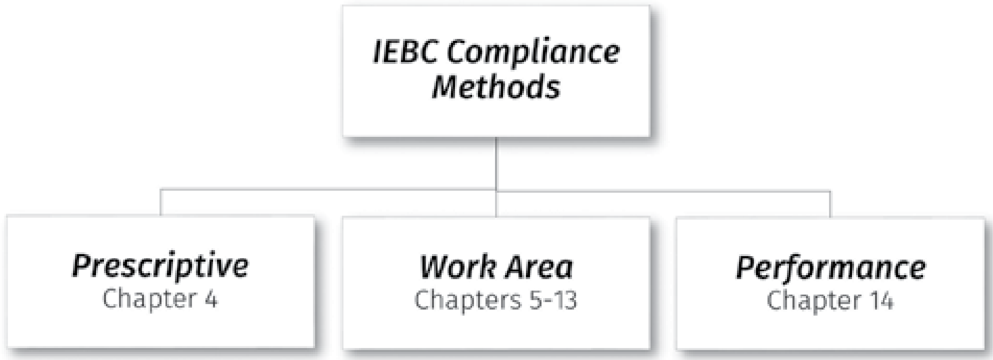

Figure 1. Compliance methods.

Throughout the United States, many jurisdictions are now adopting the 2015 International Codes. Perhaps the greatest impact to structural engineers is the fact that Chapter 34 of the 2015 *International Building Code* (IBC) has been removed. This chapter provided minimum design requirements for existing buildings. When a mandatory trigger, such as a change in occupancy, requires a seismic evaluation of an existing structure, the design professional now has the option of either showing compliance with the IBC as if it were new construction or conforming to the provisions of the *International Existing Building Code* (IEBC). How familiar are you with the IEBC?

IEBC Background

The IEBC was first introduced in 2003 after an exhaustive effort that began in 2000. The original intent was to create a comprehensive set of regulations for existing buildings based on the requirements previously included in the codes developed by BOCA, ICBO, and SBCCI. The purpose of the IEBC is to encourage the use and reuse of existing buildings while also maintaining minimum life safety requirements.

While the IEBC was first introduced in 2003, it has taken quite some time for it to be accepted and actually adopted for use throughout the United States. Several associations were not happy with portions of the initial code requirements and, as a result, quite a few changes have been made since its introduction. The International Code Council's website currently shows that more than half of States throughout the U.S. have currently adopted the IEBC either statewide or approved it for local adoption. Because Chapter 34 of the IBC has now been removed, even more states and local jurisdictions will be turning to the IEBC.

The initial thought might be, "Just what we need, another code!" While it may require designers to learn something new, it also provides much more flexibility for the reuse of existing buildings. Section 101 of the IBC now lists the IEBC as a companion code similar to the mechanical, plumbing, fire, and energy codes. Like it or not, the IEBC appears to be here to stay.

Compliance Methods

The beauty of the IEBC is that it allows the owner and design professional to select one of several paths for code compliance. As shown in *Figure 1*, there are three compliance options: (1) Prescriptive, (2) Work Area, and (3) Performance. It is important to know that mixing-and-matching is not allowed and once a compliance path has been chosen all members of the design team must follow that same path. If the architect desires to follow the performance method, then all members of the design team will also need to comply with the performance method requirements.

General Requirements

Chapters 1 through 3 of the IEBC cover general provisions that are applicable to all three compliance methods. As with all of the International Codes, Chapter 1 covers scope and administrative provisions while Chapter 2 provides definitions of key terms used throughout the code. It is very important that both the design professional and local authorities having jurisdiction (AHJ) have an understanding of Chapter 3. The following are some key items defined in Chapter 3:

1) **Seismic Forces Levels:** Throughout the code, specific triggers will require a seismic evaluation of the entire structure or of an individual component. Each time a trigger is specified, it will state whether the evaluation needs to meet the full IBC code-level or the "reduced" code-level seismic forces. Section 301.1.4.2 of the IEBC clarifies that the "reduced" code-level forces are equivalent to evaluations (1) considering 75% of the IBC prescribed

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forces, (2) using Appendix A of the IEBC, or (3) complying with Table 301.1.4.2 of the IEBC when using ASCE 41.

2) **Performance Objectives:** Structural engineers must be intimately familiar with Tables 301.1.4.1 and 301.1.4.2 of the IEBC prior to commencing a seismic evaluation of an existing structure. These tables clarify the minimum performance levels expected for existing buildings based upon risk category. Many structural engineers have been accustomed to evaluating existing buildings to meet the Basic Safety Objective (BSO) described in ASCE 41 but, per the IEBC, that would not be adequate for a Risk Category III or IV structure.
3) **New and Replacement Materials:** All new construction and new materials used as part of the work must comply with the requirements of the current IBC unless specifically noted otherwise in the IEBC.

In addition, there are two other key items to be aware of regardless of which compliance method is chosen. First, all new structural elements and their connections are to comply with the requirements of the IBC. Second, if the existing structure is located within a flood hazard area and the repairs, alterations or additions performed constitute a "substantial improvement," the entire structure will be required to comply with Section 1612 of the IBC or Section R322 of the International Residential Code.

Prescriptive Method

The prescriptive method is essentially a duplicate of the provisions previously provided under Chapter 34 of the IBC. Structural triggers are specified for additions, alterations, repairs, change of occupancy, historic buildings, and moved structures. The prescriptive method will not be discussed in detail in this article due to its previous incorporation in the IBC.

Work Area Method

The work area method is the most flexible of the three compliance options and comprises Chapters 6 through 13 of the IEBC. It provides many benefits to building owners and design professionals, building on the premise that specific code provisions are only triggered if the scale and level of work warrant.

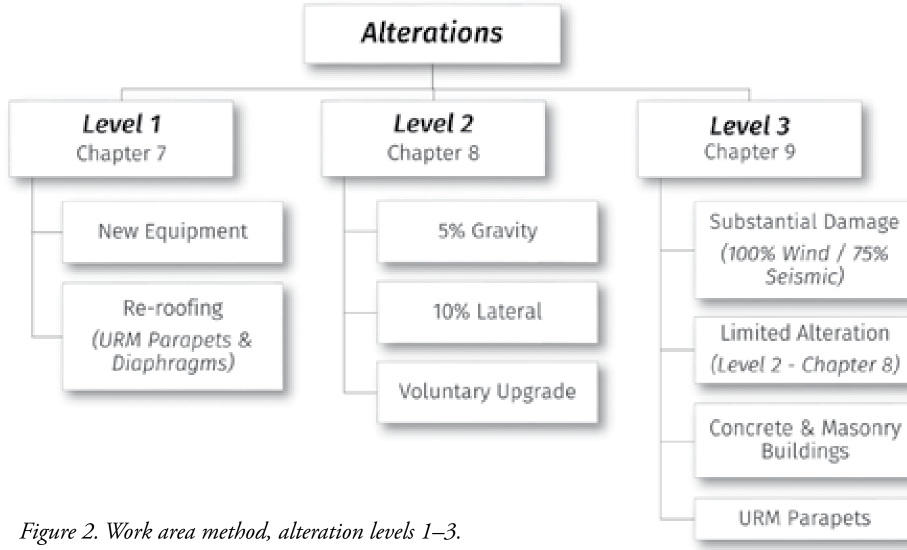

Figure 2. Work area method, alteration levels 1–3.

Repairs

When it comes to repairs, the key term to consider is *Substantial Structural Damage*. When the required repairs are less than *substantial*, as defined by Chapter 2 of the IEBC, the building can be repaired to its pre-damaged state. Those repairs that are considered substantial require an evaluation using the full IBC wind loads and "reduced" seismic.

Alterations

The work area method actually divides alterations into three separate categories, Level 1 through Level 3 (*Figure 2*). Level 1 alterations are considered very minor, such as the addition of new mechanical equipment or re-roofing. Re-roofing can also trigger the requirement to brace unreinforced masonry parapets in Seismic Design Categories D-F or to upgrade diaphragm connections in high wind regions.

Level 2 is a "catch-all" for alteration work that does not fall into the Level 1 or Level 3 categories. Most alteration projects will likely fall under this category. Level 2 is very similar to the requirements provided in the prescriptive method and is based on the 5% gravity and 10% lateral rules. That is, if existing gravity members have been decreased in capacity by more than 5%, or more than 5% additional load has been placed on them due to the alteration, those members must be analyzed to show compliance with the current IBC requirements. Similar to the gravity requirements, if lateral load carrying members have been decreased in capacity by more than 10%, or more than 10% additional loads will be applied to them, an analysis must be completed.

The difference between lateral and gravity members is that the 10% rule now triggers an analysis of the entire structure while the 5% gravity rule only requires an analysis of that one member. A key benefit to analyzing alterations using the work area method rather than the prescriptive or performance methods is that the evaluation can be performed using the "reduced" seismic forces.

Level 2 alterations also include voluntary seismic upgrades. Voluntary upgrades simply require an engineering analysis showing that the building will be no less compliant and that new components comply with the current IBC provisions.

Level 3 alterations only apply to projects considered as Substantial Structural Alterations. These are defined as projects that will undergo alterations within a 5-year period of time that affect more than 30% of the total floor and roof areas. When this occurs, the IEBC requires an analysis of the lateral systems for the full IBC wind loads and "reduced" seismic forces. Additional provisions are also required for ensuring that a proper roof-to-wall attachment is provided for existing masonry and concrete buildings, and for the bracing of unreinforced masonry parapets.

Additions

The requirements for additions are very similar to Level 2 alterations. Both the 5% gravity and the 10% lateral rules still apply, but the major difference is that if the 10% lateral trigger has been met, the evaluation provided must consider the full IBC wind and full seismic forces. The provisions for alterations allow the use of "reduced" seismic forces in the evaluation.

Change of Use

Similar to past IBC requirements, buildings undergoing a change of use are only required to provide a seismic evaluation if the new use causes the building to be assigned to a higher Risk Category per Table 1604.5 of the IBC. In addition, all gravity members should be checked to ensure that they can support higher live loads, if applicable, per Table 1607.1 of the IBC.

Historic Buildings

For buildings located in Seismic Design Categories D-F, a structural evaluation must be provided describing the strengths and weaknesses of the vertical and horizontal elements of the lateral force resisting system. All deficiencies should be noted in a report and discussed in a meeting with the owner and building official. While a complete seismic upgrade is not required, all dangerous conditions must be remedied.

Moved Buildings

Under Chapter 34 of the IBC, the evaluation for a moved structure was required to show that it met the requirements for a new building. Per the IEBC, this requirement is only triggered when the building is moved to a location such that the new snow, wind, and seismic loads trigger the 5% gravity or 10% lateral rules. In truth, the building would still need to be analyzed in order to properly design the foundation system and anchorage of the building to the foundation.

Performance Method

The performance method is likely the least understood and the least used. It provides the building owner and design team with a method to score the existing fire and life-safety conditions of a building. If the score is below the minimum accepted level, the building owner and the building official should determine what improvements need to be made to raise the score to an acceptable level. While fire and life-safety items receive a score, a detailed structural analysis must also be provided considering the full IBC wind and seismic loads. The structural analysis report listing any noncompliant items must be presented to the building official along with documentation for updating any noncompliant items.

Conclusion

As Chapter 34 of the IBC has been removed, design professionals will have an increased need to learn and rely upon the provisions included in the IEBC. The purpose of the IEBC is to encourage the use and re-use of existing buildings while requiring reasonable upgrades and improvements. It offers several paths for compliance which provide flexibility to both building owners and to design professionals. In many cases, buildings may only need to meet the "reduced" seismic requirements whereas they previously were required to comply with the full seismic loads under the IBC. In addition, the IEBC allows the building official to use discretion in determining what minimum code requirements need to be met and to ensure that all dangerous conditions are alleviated. The building officials will likely rely upon the recommendations of the structural engineer when making such a determination. All design professionals should take some time to become familiar with the IEBC provisions, as they will likely be referring to it often in the future.▪

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Design of Bolted Connections per the 2015 NDS

By John "Buddy" Showalter, P.E.

This article provides an overview of a bolt design example utilizing the *American Wood Council's (AWC) 2015 National Design Specification® (NDS®) for Wood Construction*. Topics include connection design philosophies and behavior, an overview of 2015 NDS provisions related to bolt design including local stresses in fastener groups, and a detailed design example.

Connection Design Philosophies

Discussion of several important design philosophies should help designers better understand connection design for wood members. First, wood is anisotropic, meaning it has different strength properties in various directions: longitudinal, tangential, and radial. Wood is composed of elongated, round, or rectangular tube-like cells. A simple analogy is to imagine the cellular nature of wood as a bundle of drinking straws. When axial compression is applied, the "bundle" is strong longitudinally and connecting the ends, primarily for bearing, is very simple. The "bundle" can also develop considerable tensile strength. Therefore, aligning connections so that loads are transferred concentrically along the length of the wood member is the most efficient design philosophy. However, this is not always practical or possible.

Continuing the analogy, if the load is applied perpendicular to the longitudinal axis of the "bundle" in compression, the straws tend to crush because of the weaker cell walls relative to the axial direction. While capacities are more limited when wood is loaded in compression perpendicular to grain (versus parallel to grain), the limits for bearing conditions on the surface of wood members are deformation-based, not strength-based, and published design values can be increased for smaller bearing areas. Accordingly, dowel bearing strengths are higher relative to compression parallel or compression perpendicular to grain design values. However, dowel bearing strengths perpendicular to grain are lower relative to dowel bearing strengths parallel to grain for larger diameter (>¼ inch) fasteners (see *Table*).

When tension is applied perpendicular to grain, the "bundles" tend to separate. Low strength values for this property can be encountered in commercial grades of lumber. For this reason, no sawn lumber tension design values perpendicular to grain have been published in the NDS. Cautionary provisions have been provided to alert designers to avoid design configurations that induce tension perpendicular to grain stresses wherever possible. Connections where moderate to heavy loads are acting through the tension side of a bending member (see NDS Table 12.5.1C, footnote 2) should be avoided. These connections should be designed to ensure that perpendicular-to-grain loads are applied through the compression side of the bending member, either through direct connections or top-bearing connectors.

Second, wood connections are stronger when the load is spread out over a number of fasteners. Large concentrated loads should be avoided unless designed not to exceed wood's strength capabilities (e.g., net tension and shear). Spreading the load also builds in a degree of redundancy, which is useful in high wind or seismic events. To accomplish this, designers are advised to:

- Use small fasteners;
- Use multiple fasteners when possible; and
- Keep the scale of fasteners relative to the size of wood members being connected.

Third, as with other building materials, wood moves in response to environmental conditions. The main driver for this movement in wood is moisture. Allowances must be made to accommodate potential shrinkage and swelling, particularly in connections.

Dowel-Type Fasteners

Wood members connected with dowel-type fasteners are probably the most common mechanical connection type because they are effective at transferring loads while also being relatively straightforward and efficient to

Design values from 2015 NDS and 2015 NDS Supplement *(psi).*

No. 2 Southern Pine 2x12	**Parallel to Grain**	**Perpendicular to Grain**
Compression Design Value	1,250	565
Tension Design Value	450	No published values
Shear Design Value	175	n/a
Modulus of Elasticity	1,400,000	n/a
Dowel Bearing Strength for 1-inch diameter fastener	6,150	2,550

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The online version of this article includes a detailed Design Bolt Example. Please visit **www.STRUCTUREmag.org**.

install. They come in many forms, and their strength properties can be calculated using the NDS. Dowelled connections transfer the force between members through a combination of dowel bearing and bending of the dowel fastener. Bolts must be structural quality bolts, equal to or better than ANSI/ASME Standard B18.2.1. Bolt holes must be a minimum of 1⁄32 inch and a maximum of 1⁄16 inch larger than the bolt diameter.

Yield Limit Equations

The NDS yield equations are mechanics-based and are valid for a broad range of theoretical connection possibilities for dowel-type lateral connections (see *Figure*). These equations account for variables, such as dowel diameter, side-member size, main-member size and strength of the components. NDS yield equations are not limited to wood-to-wood connections. For example, wood-to-steel and wood-to-concrete design values are tabulated in the NDS. In keeping with the dual-format of the NDS, both allowable stress design (ASD) and load and resistance factor design (LRFD) provisions are included for connections.

Yield equations have been developed for each mode relating the joint load to the maximum stresses in the wood members and the fastener. The capacity of the connection under each yield mode is keyed to the bearing strength of the wood under the fastener and the bending yield strength of the fastener, with the lowest capacity calculated for the various modes taken as the reference design value for the connection. Four limiting yield modes characterized by the NDS equations include:

- MODE I: bearing-dominated yield of wood fibers
- MODE II: pivoting of fastener with localized crushing of wood fibers
- MODE III: fastener yield in bending at one plastic hinge and bearing-dominated yield of wood fibers
- MODE IV: fastener yield in bending at two plastic hinges and bearing-dominated yield of wood fibers

Subscripts “m” and “s” denote main member and side member, respectively. Accordingly, Mode I_m represents the case where the main member controls by bearing-dominated yield of wood fibers, whereas Mode I_s is side member controlled. Mode III is similarly characterized.

The dowel bending yield strengths, F_{yb}, of bolts are given in NDS Appendix I. For A36 and stronger steels, F_{yb} equal to 45,000 psi is a conservative value and is equivalent to the bolt strength reported in the original bolt test research used to develop the methodology. For detailed technical information on lateral design equations, see AWC’s *Technical Report 12: General Dowel Equations for Calculating Lateral Connection Values* available at **www.awc.org**.

Spacing, End, and Edge Distance

For dowel-type fasteners with diameters equal to or greater than ¼ inch, the geometry factor, C_Δ, provides a proportionate reduction of reference lateral design values for less than full end distance or less than full spacing distance. The lowest geometry factor for any fastener applies to all other fasteners in that same connection, not just to the end fastener or a pair of fasteners in a row. It should be noted that further reductions may be necessary when checking stresses in members at connections

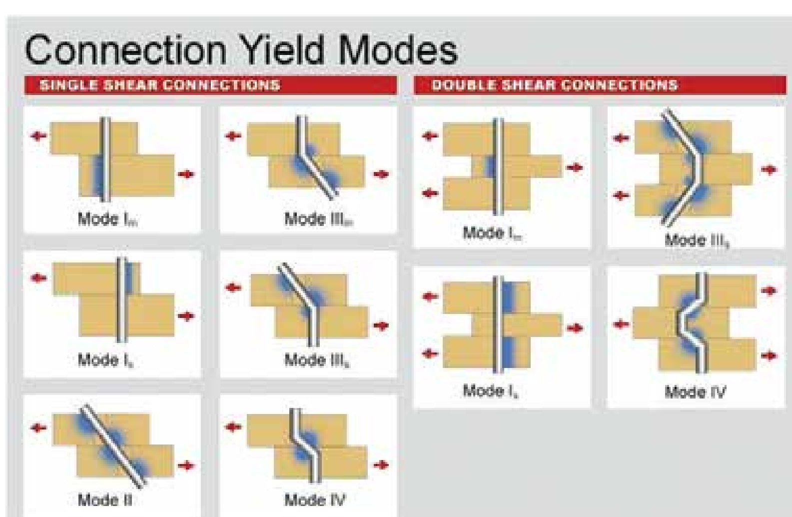
For parallel or perpendicular to grain loading, limiting the maximum distance between outer rows of fasteners on the same splice plate to 5 inches avoids splitting that could occur in members at connections as a result of restraint of shrinkage associated with drying in service (structural glued laminated timber has different limits). Special detailing can be utilized in cases where distances between outer rows of bolts exceed the limits in 2015 NDS 12.5.1.3, such as the use of multiple splice plates or a single splice plate with slotted holes to allow shrinkage.

Local Stresses in Fastener Groups

Where a fastener group is composed of closely spaced fasteners loaded parallel to grain, the capacity of the fastener group may be limited by wood failure at the net section or tear-out around the fasteners caused by local stresses. The capacity of connections with closely spaced, large diameter bolts has been shown to be limited by the capacity of the wood surrounding the connection. Connections with groups of smaller diameter fasteners, such as typical nailed connections in wood-frame construction, may not be limited by wood capacity. The 2015 NDS Section 11.1.2 states that “Local stresses in connections using multiple fasteners shall be checked in accordance with principles of engineering mechanics. One method for determining these stresses is provided in Appendix E.” NDS Appendix E includes provisions for calculating net section tension capacity, row tear-out capacity, and group tear-out capacity.

Conclusion

Connection design is an iterative process that sometimes requires trade-offs. Bolt design per the 2015 NDS is not just a matter of selecting a design value from a table. In addition to the yield limit equations for dowel-type connections, application of spacing, end, and edge distance requirements for connections and provisions related to bolt design including local stresses in fastener groups must be considered. Due to the anisotropic nature of wood, consideration for load direction relative to grain orientation and moisture effects are essential. Good connection design for wood considers all of these factors.▪



Dowel-type connection yield modes.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Robert F. Kennedy Bridge

AERODYNAMIC EVALUATION AND RETROFIT DESIGN

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

A comprehensive wind study of the Robert F. Kennedy Bridge suspended span was performed to determine if the bridge meets current aerodynamic criteria and ensure that it responds to wind events in a predictable manner. The suspended structure, an important facility in the New York Metropolitan area, features a 1,380-foot long main span, two 670-foot wide side spans and a minimum navigational vertical clearance of 150 feet (*Figure 1*). The bridge carries eight lanes of traffic in an eighty-seven-foot curb-to-curb width. The suspended structure is composed of two, 20-foot deep stiffening trusses connected to the main support cables and suspenders, and transverse floor beam trusses spaced at approximately twenty-eight feet on centers. In 2000, the original concrete decks and crossbeams were replaced with steel orthotropic decks.

Long-span bridges, such as the suspended spans of the RFK Bridge, need to be aerodynamically stable. The wind study tasks included analysis of wind climate, the establishment of equivalent static wind loads, sectional model testing, aerodynamic stability analysis, analysis of suspended spans for wind load, and retrofit design. The investigation also included the safety of wind-sensitive vehicles, such as trucks and buses, on the bridge during strong winds. These studies were performed for the RFK bridge by the Boundary Layer Wind Tunnel Laboratory (BLWTL) at the University of Western Ontario.

Design Criteria

Long span bridges need to be evaluated under wind loads for three limit states: serviceability, strength, and stability.

The serviceability limit relates to the usage of the bridge by passengers and can be expressed as deflection or accelerations. Normally, vertical accelerations in the range of 5%g to 10%g are considered acceptable for pedestrians. The maximum annual (1-year return period) wind speed is used for this evaluation.

The criteria for the strength limit are the same as those outlined in AASHTO's *LRFD Bridge Design Specification* which is intended to ensure that the bridge has sufficient strength to resist the maximum wind loads during its design life. Load factors are used on the wind loads for checking of structural member capacities against yielding, buckling or shear failures. The maximum wind event for this limit state has a return period of 100 years.

Figure 2. Wind section model of the RFK Bridge. Courtesy of Boundary Layer Wind Tunnel Laboratory.

Table 1. Wind climate study evaluation criteria.

	Limit State	Return Period	Mean Hourly Wind Speed (mph)	10-Minute Mean Wind Speed (mph)
Completed Bridge	Serviceability	1-Year	47.6	—
	Strength	100-Years	75	—
	Stability	10,000-Years	—	105.4

The stability limit is a wind speed limit (flutter wind speed) above which the bridge will become unstable. Flutter is a self-excited instability caused by the interaction of the wind and the bridge structure involving either pure torsional motion or coupled vertical and torsional motion of a bridge deck. The instability can grow to very large amplitudes and lead to the collapse of the structure. The maximum wind speed for this limit state has a return period of 10,000 years.

Based on the wind climate study performed at BLWTL the criteria was recommended in *Table 1* for the evaluation.

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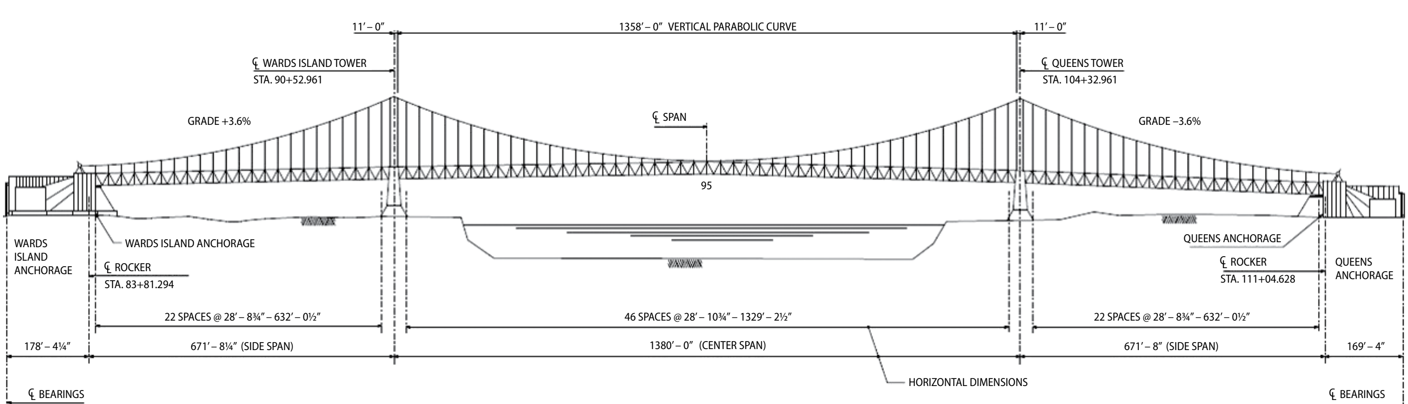

Figure 1. Suspended span. Courtesy of MTA Bridges and Tunnels.

Section Model

A physical model of a typical cross section of the bridge was constructed at a geometric scale of 1:60 (*Figure 2, page 33*). The model was ballasted to the scaled mass properties and mounted on a dynamic test rig in which the fundamental vertical and torsional modal frequencies of the bridge deck were simulated. These properties were obtained from field vibration measurement and finite element modeling simulation. During testing, the bridge behaved satisfactorily under strong winds with regard to the Service and Strength limits. However, its flutter wind speed was only 88 mph, which does not meet the wind speed criteria for aerodynamic stability of 105.4 mph. The return period of 88 mph winds at the RFK Bridge site was approximately 1,000 years.

Conceptual Retrofit Design

Given the low flutter wind speed for the main suspended span, it was apparent that making the bridge elevation more open to air flow was necessary to improve its aerodynamic performance. Conceptual design options for retrofit alternatives were developed that would improve the aerodynamic stability of the suspended spans. The effects of protective fencing were also included in this study.

The three retrofit alternatives developed were:

- Option 1: Replace existing solid roadway barriers with new open barriers (*Figure 3*)
- Option 2: Introduce perforations in the solid walkway fascia girders (*Figure 4*)
- Option 3: Replace existing solid walkway fascia girders with new shallower girder (*Figure 5*)

Phase 1 Verification Testing

Phase 1 involved testing the wind retrofit alternatives and combinations of alternatives with protective fencing and selecting the alternative with best aerodynamic stabilities. Results are shown in *Table 2*.

On examination of the wind retrofits alternatives, the open barrier alone (Option 1) with no walkway or fencing modifications, the flutter criteria is satisfied. The combination of shallow fascia girder and open roadway barrier (Option 1+3) offers very significant improvements and increases the flutter speed to 132 mph, well above the 105.4 mph criterion. Protective fence components on both the roadway and sidewalk did not significantly reduce flutter wind speeds

A combination of Options 1 and 3 was recommended, together with protective fences on both walkway and roadway, as the final wind retrofit alternative (*Figure 6*).

Phase 2 Verification Testing

In this phase of the testing program, the selected combination scheme from Phase 1 was tested for the following conditions:

- Turbulence
- Wind attack angles of +/- 1 degree
- Snow and ice accumulation in protective fences

Turbulent flow tests offer a more realistic indication of a bridge's response in strong winds since the natural wind tends to be turbulent. In these tests, the effect of turbulence shows a comparatively small benefit towards an improvement in flutter wind speed of between 3 and 4 mph (*Figure 7, page 36*).

continued on page 36

Table 2. Phase 1 test results.

TEST	Option 1	Option 2	Option 3	Option 1+3	Option 1 + Fence	Option 1+3 + Fence
Flutter Speed (mph)	110	95	97	132	106	130

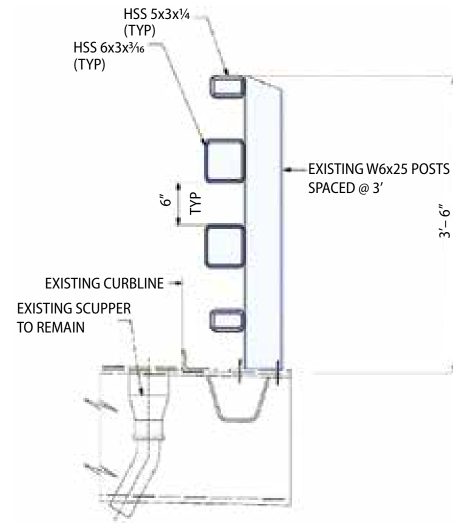

Figure 3. Retrofit open barrier.

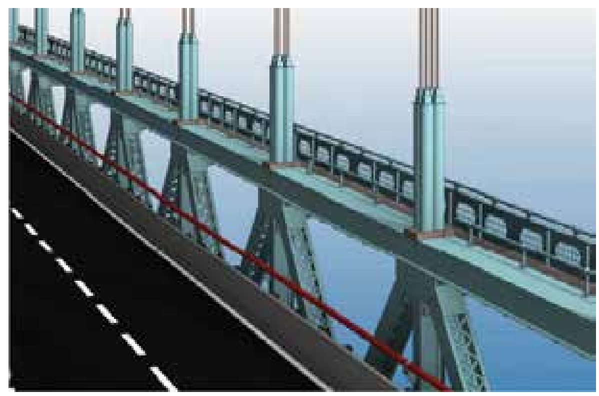

Figure 4. Perforations in sidewalk fascia girder.

Figure 5. Shallow Fascia girder.

Figure 6. Combination retrofit.

The angle of attack is the inclination of the wind to the horizontal plane of the deck, being positive when the windward leading edge moves upwards ("nose up"). Torsional flutter instability was observed for angles of attack of +1° and -1°; however, the wind speeds far exceed the flutter wind speed criterion of 105.4 mph (*Figure 8*).

The effects of icing and snow accumulation were also explored in the Phase 2 sensitivity tests; the concern was that snow and icing would increase the solidity of fencing elements, thereby reducing the flutter wind speed. Various icing conditions were represented by different porosities and tape was used over fence and barrier components to simulate these conditions. The tests were conducted to provide insight into the sensitivities of bridge vibrations to icing conditions. Results showed that flutter wind speeds were reduced significantly when there is ice in fences on walkways and roadways. However, this condition should not be of major concern due to the low probability of the combination of extreme ice and wind conditions. Also, the Triborough Bridge and Tunnel Authority (TBTA) facility have the option of removing ice on fences if necessary.

Sidewalk Extension Study

The suspended span has two sidewalks located at the top of the stiffening trusses. Sidewalks are about 6 feet wide. One side is currently used for pedestrians, and the other side is used only for maintenance and inspection. Given the possibility that sidewalk modification may be part of future capital programs and the possibility that it may enhance future bridge aeroelastic behavior, additional studies of sidewalk extensions were performed with the recommended retrofits from the Phase 1 and 2 tests.

Critical wind speeds for flutter instability and wind effects based on geometry and dynamic properties of the following configurations of the bridge were studied:

- Two 10-foot wide new sidewalks – one situated on each side of the bridge
- One 12-foot wide new sidewalk on one side, with the other sidewalk at existing width (i.e. 6-foot wide)

For all tested sidewalk widening cases, all the flutter wind speeds in smooth flows meet the criterion of the 10,000-year return period wind speed, as shown in *Table 3*. It was observed that increasing walkway width improves aerodynamic stability by increasing flutter wind speed.

Wind on Vehicle Study

High wind conditions are the frequent cause of vehicle rollovers or skidding, often forcing the shutdown of major roadways and halting the movement of traffic.

Wind tunnel tests were conducted to determine the wind forces that act on an array of wind sensitive vehicles. The forces were used to evaluate the behavior of vehicles and susceptibility to overturning and skidding on the suspended spans with proposed wind retrofits developed in the previous phases. The results of the tests were used to assess the relative wind forces that users of the bridge may experience and to assess the relative differences in the forces which act on high-sided vehicles. The tests were performed using force balance models of the following vehicles: a) a truck/semi-trailer, b) a truck/double trailer, c) a highway bus and d) delivery van.

For testing vehicles on the bridge, the existing section model from previous wind tunnel testing was used, with the retrofits as included. The vehicle models were attached to the bridge deck in each of the four northbound traffic lanes and mounted on a rig on the turntable

Figure 7. Section model test with turbulence. Courtesy of Boundary Layer Wind Tunnel Laboratory.

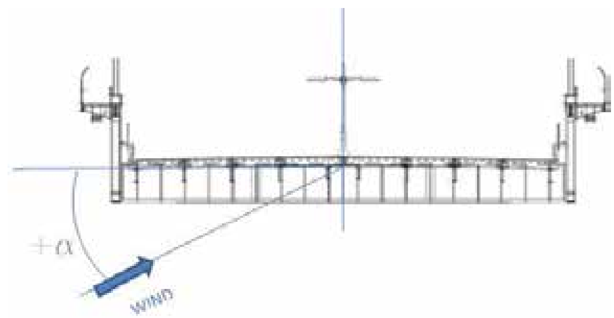

Figure 8. Attack angle on bridge section.

Figure 9. Wind on vehicle test. Courtesy of Boundary Layer Wind Tunnel Laboratory.

Figure 10. Shielding effects of bridge stiffening truss.

which allowed multiple wind angles to be examined (*Figure 9*). The instrumented vehicles were tested without traffic present in the remaining lanes to develop the critical loads for vehicle overturning. The measured forces on all vehicle types were integrated into an analytical vehicle overturning model which assessed the sensitivity to vehicle overturning and potential handling difficulty experienced by drivers in high winds.

Test results showed that stiffening trusses and protective fences in the suspended spans had a shielding effect on vehicles and provided enhanced resistance to blow over. Compared to the results from the ground tests, there is an average 24 mph increase in critical wind speeds when the vehicles are placed in suspended spans (*Figure 10*).

Conclusions

As a major long-span bridge in the New York City Metro area, the Robert F. Kennedy Bridge needed to be evaluated for strength, serviceability, and stability. Wind tunnel testing demonstrated that the suspended span did not meet the 10,000-year stability requirement of 105.4 mph. The innovative solution proposed was replacing solid roadway barriers with open barriers and replacing the deep fascia girder on the walkway with a shallower girder railing, increasing flutter wind speed to 130 mph including protective fences. An extensive wind on vehicle testing program was also performed. Lane by lane tests were conducted on wind sensitive vehicles for overturning and skidding for multiple wind azimuths. Test findings demonstrated that such vehicles are very safe, thanks to the shielding effects of the stiffening trusses.▪

Table 3. Sidewalk extension study.

Test	Flutter Wind Speed (mph)
Two 10-foot Sidewalks	142
6-foot sidewalk leeward 12-foot sidewalk windward	144
6-foot sidewalk windward 12-foot sidewalk leeward	136

Project Team

Owner: MTA Bridges and Tunnels – TBTA
Joint Venture: Thornton Tomasetti – Weidlinger Transportation, T.Y. Lin International
Wind Consultant: Boundary Layer Wind Tunnel Laboratory at the University of Western Ontario.

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Cottonwood Cornerstone Center Office Building

Synergy of Art and Function

By Troy M. Dye, S.E. and Ryan E. Smith, S.E.

Figure 1. Office building complex.

The Cottonwood Cornerstone Center office building complex has risen from the depths of an ancient lake, through the gravel shores, and sits on the remnants of a reclaimed gravel pit. Land with panoramic views is valuable even if the site is unusually difficult to build on. Undeterred, the client brought together a design team that would explore and evaluate options so that a signature structure, which complements the beautiful natural surroundings and community, could be built. The result of those efforts was an aesthetically pleasing work environment for the tenants, and an efficient and cost-effect structural solution to a very challenging site and strict design parameters.

The Cottonwood Cornerstone Center office building complex is located in the shadows of the Wasatch Mountains and less than 1 mile from the Wasatch Fault. The office building consists of a six-story steel structure, a three-story lobby connector steel structure (Figure 1), and a two-story parking garage with conventionally reinforced concrete slabs and post-tensioned concrete beams. The engineering design of this elegant Class A office building complex provided some unique challenges due to local soil conditions, high seismic forces, the owner's desire for a shallow structural depth, a curved cantilevered lobby walkway, and a light gauge steel stud façade framing, supporting expensive imported cut sandstone with horizontal seismic slip joints at each floor.

Varying Soil Conditions

About 25,000 years ago, Lake Bonneville covered the Salt Lake Valley with its shores nestled up against the Wasatch Mountains. Over time, sediment and gravel layers were deposited along the shoreline. The geotechnical report provided the following background of the site: "For many decades, the Cottonwood Corporate Center site was utilized as a working gravel pit. During the early 1990s, it was decided to terminate the gravel pit operations, reclaim the area, and ultimately develop the reclaimed areas as a part of an extensive office park."

Numerous borings on the site revealed that non-engineered fill was used as part of the reclamation of the site. The fill varies in depth from 3.5 feet to 25 feet. All of the non-engineered fills exhibited variable and, in most cases, poor to very poor engineering characteristics. The geotechnical engineer provided three soil preparation options that use conventional spread footings and continuous wall foundations.

- Bear the foundations on suitable natural soils
- Replace granular fill extending to suitable natural soils
- Improve the granular fill through the installation of Geopiers®/ stone columns

The basement space under the six-story building was utilized for parking and mechanical equipment. The space was divided into two areas; a 19-foot 6-inch deep mechanical basement and an 11-foot 4-inch deep parking garage. At the deeper basement, the excavation extended down into native soils allowing the use of conventional spread and spot footings. However, in the parking garage, the native soil was at least 10 feet below the bottom of the footings. Rather than over excavate down to the native soil, the design team decided to use Geopiers/stone columns to improve the bearing capacity of the non-engineered fills.

Some of the soil could not be improved enough with Geopiers/stone columns to support the 910-kip column loads due to the presence of moderately sized cobbles and small boulders. It was decided that at four column locations, steel "H" piles would be used to penetrate through the inadequate soil and into the strong native soils below. The design team became concerned that differential settlement might be an issue due to the use of shallow footings, Geopiers, and deep foundations. The geotechnical engineer indicated that the settlements for foundations on native soil and foundations on improved soil would essentially be the same. After receiving the "H" pile depths and loads, the geotechnical engineer was able to calculate a differential settlement of less than ½ inch between the "H" pile system and the spread footings. The footings and foundations were designed and reinforcement detailed to accommodate this small differential settlement without damaging the foundations or architectural finishes.

High Seismic

This building complex is located in a high seismic region and was designed using an S_s value equal to 1.572g and S_1 value equal to

Figure 2. SidePlate connection.

Figure 3. Steel girder web openings.

0.634g, with a soil site classification of D. The design team faced the challenge of meeting the owner's desire for a "brace-free space" to obtain unobstructed views while avoiding the premium in cost associated with a conventional steel moment frame system. What made matters more challenging was the client-imposed beam depth limit of 24 inches. To solve these problems and do so in an economical way, the design team evaluated numerous structural systems and determined the most cost effective steel solution was special steel moment frames utilizing the increased stiffness of the SidePlate Connection Technology (*Figure 2*).

When comparing the various lateral systems, the design team determined that, to meet the allowable 0.02h code-mandated drift requirement, a conventional moment frame system such as a Reduced Beam Section (RBS) would have required the use of W24x131 beams and W24x370 columns. Utilizing the SidePlate system, the final beam sizes were W24x94s, and the final column sizes were W24x192s. As required per the seismic provisions, each joint was designed to resist the probable maximum moment of the beam taking into account specific hardening ratios. For these particular joints, the design moment was equal to 1455 k-ft.

The estimated cost savings associated with the decision to use SidePlate was substantial. Based on the steel tonnage reduction of 154 tons (1.47 psf) and the simplified shop and field fillet welding used in the SidePlate system, the net cost savings to the project, after the SidePlate licensing fee was paid through the steel fabricator, was approximately $375,000. In addition to the steel package savings, the owner benefited from savings in ultrasonic testing due to the elimination of full joint penetration welds. When added together, the savings brought to the project by the design team's decision to use the SidePlate Connection Technology was over $400,000 or $2 per square foot.

By working closely with the designers at SidePlate, the design team was able to provide the open layout the owner desired without the cost premium associated with conventional moment frame systems.

Minimal Beam Depths

The owner wanted to provide tenants with high ceilings while limiting the floor-to-floor height to 13 feet 6 inches. Limiting the floor-to-floor height also had the added benefit of minimizing the overall building height and reducing the amount of costly architectural finishes. Typically plumbing loops and mechanical ducts run below the structural framing, but the design team chose to penetrate the main steel girders with these elements. For this approach, the W30x108 composite steel girders were designed using the American Institute of Steel Construction's (AISC) *Design Guide #2* with a 12-inch by 12-inch web opening for the plumbing loop and a 12-inch by 34-inch web opening for the main duct loop (*Figure 3*). This coordinated effort produced potential ceiling heights of 11 feet without affecting the structural performance of the floor system or resulting in problematic vibrations.

The size of the web openings were well coordinated with the size of the pipes and ducts, but during construction, it was revealed that the pipes required an insulation wrap which had to be compressed at the openings in the beam web. Although the pipes and duct sizes were discussed and coordinated during design, only the rough duct and pipe sizes were provided, and no mention was made regarding insulation thickness or the need to have larger openings. When penetrating beam webs with ducts and pipes, the design team learned that it is essential to know the total required opening size and not just the duct and pipe size.

continued on next page

Figure 4. Curved lobby walkway.

Figure 5. Two-story lobby.

Curved Cantilevered Lobby Walkway

The two-story lobby is a beautiful synergy of interior art and majestic mountains viewed through the second story curved glass walkway (*Figures 4, page 39* and *5*). The curved walkway was created using 5⁄16-inch thick bent plate edge angles supported by L3x3x¼ kicker braces aligned with the W12x19 floor beams. At other locations, the walkway floor framing was hung from the third floor to create an open lobby area below. Because the connector experiences high levels of human traffic, vibration control and floor stiffness were a primary focus of the design of the cantilevered floor systems.

Design details showed the mullions of the curved window system bearing on top of the curved light gauge track. Typically this is not an issue with a straight wall and a stiff track, but, in this case, the flanges and webs of the tracks were cut to produce the curve. Because the track was cut, the stiffness of the track was significantly reduced causing the window system to sag. Additional studs aligned with the window mullions were installed and attached to the structure to provide a stiff support system which prevents the framing from sagging and provides adequate support for the windows.

Figure 6. Cut sandstone façade and full height windows.

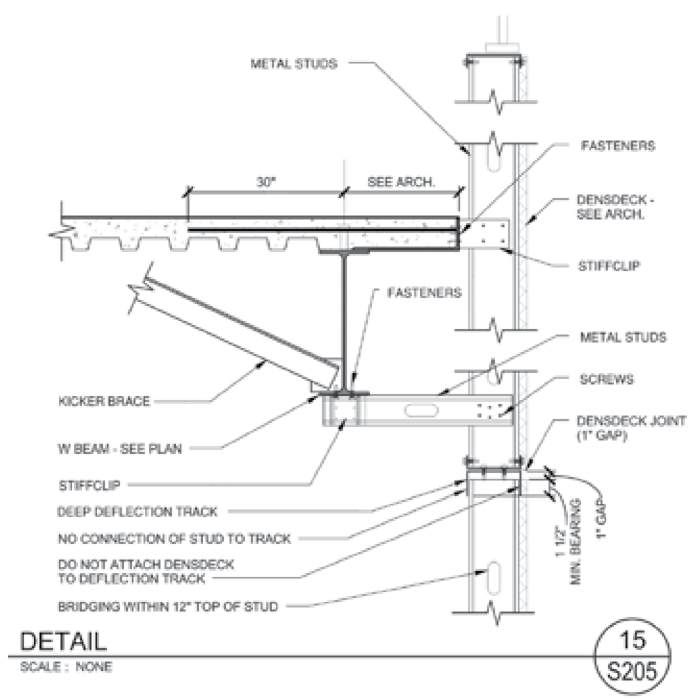

Figure 7. Vertical and horizontal slip joint detail.

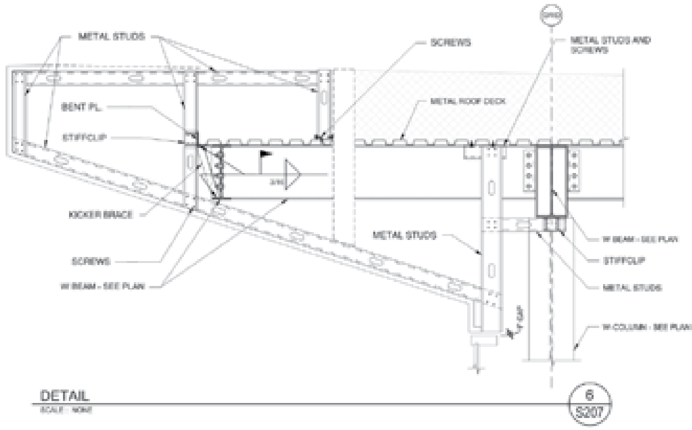

Figure 8. Sombrero light gauge framing detail.

Façade Framing

The façade of the building is a balance of floor to ceiling windows and punched openings surrounded by cut sandstone (*Figure 6*). There were many challenges in the design of the façade framing which included allowances for vertical deflection between floors due to gravity loads, allowances for horizontal movement between floors due to seismic and wind loads, and supporting the roof overhang framing)often referred to as "The Sombrero" by the design team).

The cut sandstone was carefully selected and quarried from India. The sandstone was a large lead item that had to be carefully coordinated so that material could be harvested and shipped to avoid the monsoon season. Needless to say, great care was taken to design a stiff framing system to protect and support this beautiful stone. A vertical deflection criterion equal to ¾ of an inch was used to design the perimeter beams supporting the stone clad walls. A vertical gap was designed into the façade framing just below each floor at the tops of the floor-to-ceiling window systems. This gap in the façade framing was accomplished using a 2½-inch deep 16 gauge deflection track with a 1-inch gap between the track and top of the stud. The track and wall framing above the joint were attached to the floor slab edge angle and braced back to the steel structure. To allow for vertical movement, it was critical to clarify that the Densdeck must not be attached to the track, but could be attached to the stud below the joint (*Figure 7*).

A horizontal slip joint was designed into this same detail to allow each floor to move independently during a wind or seismic event. The typical façade framing consisted of 600S162-54 studs spaced at 16 inches on center. The 2½-inch deep track above the joint provides the out-of-plane resistance at the top of each stud. The stud wall with Densdeck below the joint is not attached to the deep track and, therefore, can slide in the plane of the wall inside the deep track.

The geometry of the "Sombrero" was designed to cantilever 5 feet past the steel beam structure and maintain a 21-inch fascia depth. 362S162-54 studs spaced at 16 inches on center were utilized to form the roof and soffit framing. Both the roof light-gauge framing members and the soffit light-gauge framing members cantilever over the steel beam to provide a redundant cantilever support system (*Figure 8*).

Conclusion

The Cottonwood Cornerstone Center office building was challenging for many reasons, but the success of the project reinforced the importance of clear communication between the owner, design team, geotechnical engineers, contractor, subcontractors, and suppliers. The key to the success of this project was that open communication was encouraged throughout the design and construction project, thus providing an avenue to exchange and explore new ideas which ultimately resulted in a quality and cost-effective project.▪

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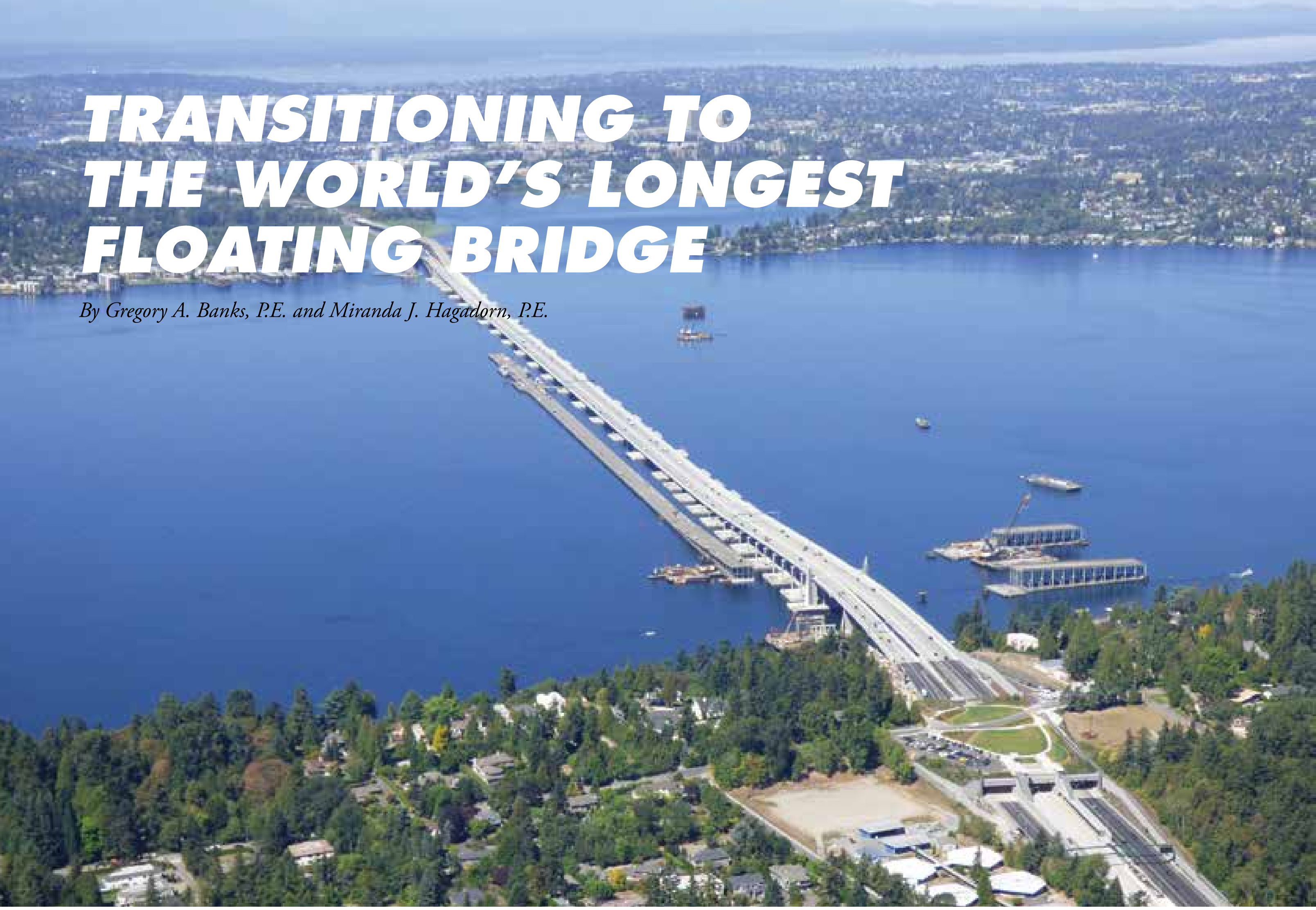

TRANSITIONING TO THE WORLD'S LONGEST FLOATING BRIDGE

By Gregory A. Banks, P.E. and Miranda J. Hagadorn, P.E.

As one of the major East-West traffic corridors into downtown Seattle, WA, the SR520 Floating Bridge and Landings (SR520 FB&L) spans Lake Washington. The pontoon design for the floating span was discussed in the October 2016 issue of STRUCTURE magazine. The floating span is dynamic, moving with fluctuations in the water level of the lake as well as wind and wave action. As a result, each end of the bridge requires a transition span from the floating bridge portion to the fixed, land-based approaches.

Figure 1 illustrates the eastern transition span; the west end is supported by Pontoon W of the floating bridge, and the east end is supported by a cantilever of the eastern approach. Similarly, the west transition span is supported by Pontoon A on one end and by Pier 36, a land based fixed pier, at its other. Westbound and eastbound traffic lanes are located on separate superstructures; thus, 4 transition spans in total were needed: two each on the east and west ends of the floating span. Each transition span is approximately 190 feet long and consists of a series of 8-foot deep, I-shaped steel plate girders.

Design Motions

The design motions for the pontoons were based on the wind-wave analyses conducted by the Washington Department of Transportation's (WSDOT) naval architect. Design motions, in addition to standard bridge movements, include:

- Fluctuations in the lake water elevation: The water elevation is regulated by the Ballard Locks and results in a plus/minus 2-foot fluctuation in lake water elevation annually. In extreme cases, the design considered mooring failures in a 20 and 100-year storm, and also a failure of the Ballard Locks, which might result in a 20-foot drop in the lake water level.
- Lateral sway of the bridge: In a storm event, the transition span needed to accommodate a 2-foot lateral sway in the floating bridge position at each supported end.
- Roll of the pontoons: Due to wind and wave action, the floating bridge rolls about its longitudinal axis, for which the transition span needed to twist to accommodate the rolling motions and prevent the steel plate girders from lifting off of the bearing supports.

As for the standard bridge movements, the transition span needed to accommodate over 6 feet of total longitudinal motion at the end.

Transition Span Concept

Common practices by designers of mechanical equipment include improvements to existing successful concepts rather than developing new ones, the reason being that it is often difficult to recognize all the potential problems that may be inherent in a new concept. The analogy of machine design is relevant to the transition spans of the SR520 FB&L project because the transition spans must articulate movement in many directions without adversely affecting the serviceability of the transition spans or the supporting structures.

The design storm, seismic loads, and expected motions on the project significantly exceed the demands of other floating bridges

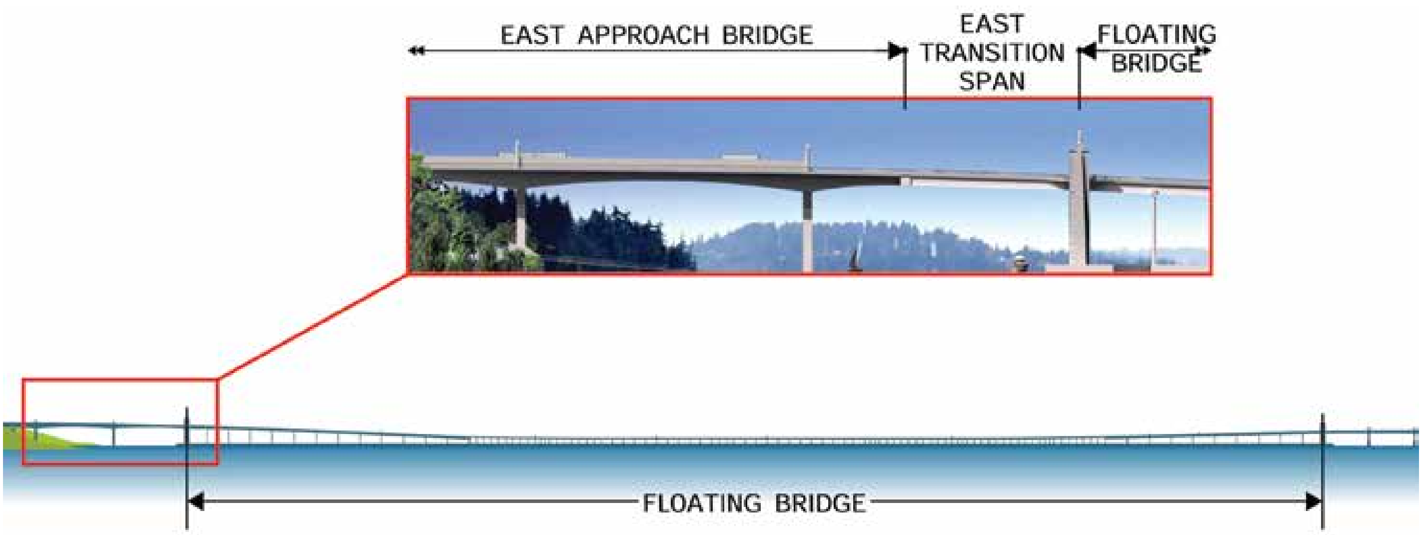

Figure 1. Plan/elevation schematic of transition spans.

currently in service in the State of Washington. Applying transition span concepts utilized for other floating bridge designs could not work on this project due to their high torsional stiffness. With these motions, a torsionally stiff deck results in deck liftoff at one corner of the transition spans under the design pontoon roll. Consequently, a new bearing concept, ultimately termed the "trailer hitch," was developed, which was more robust, economical, and provided true six degrees of freedom movement capability. One of these degrees of freedom is provided by the torsional flexibility of the span, which had to be adequate to prevent corner liftoff.

As illustrated in *Figure 2*, the east transition-span utilized an open soffit design, consisting of torsionally flexible steel I-beams pinned-supported at the approach by the trailer hitch, and roller-supported on the pontoon end; that is, girder stops restrain transverse movement but are free to slide longitudinally.

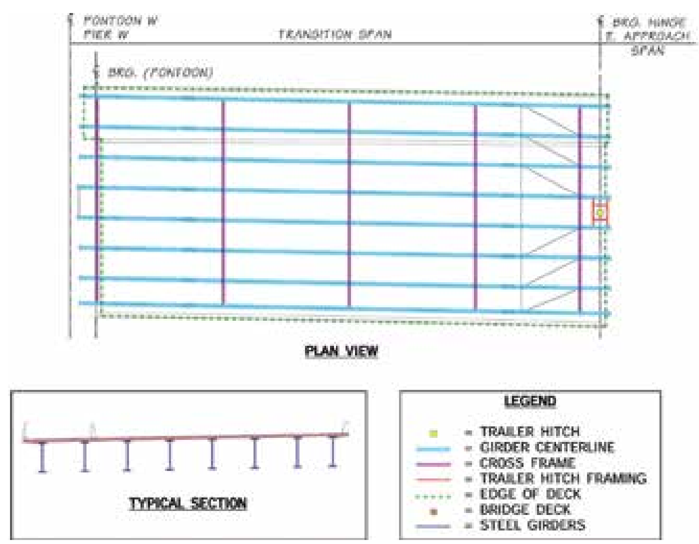

Figure2. Transition span framing plan (East North; others similar).

Trailer Hitch

The trailer hitch is a built-up steel weldment shown schematically in *Figure 3* and pictured in *Figure 4* (*page 44*). As shown in *Figure 3*, the embedded portion of the trailer hitch has flanges which house high strength post-tensioning rods that anchor the trailer hitch to the approach span. The exposed portion of the trailer hitch is box-shaped with spherical bearings on each face. The spherical bearings share a common radius, essentially forming a sphere, and act similar to an automobile trailer hitch. *Figure 4* shows the steel plate girders being set over the trailer hitch.

Kinematics of the Transition Span

The kinematics of the West and East transition spans are mirror opposites in behavior while the South and North spans have nearly identical movement, differing mainly in deck width. As a result, discussion of kinematics focuses only on the East span.

Teflon-lined spherical bearings provide vertical support at each girder end. Since the pontoon is free to slide longitudinally, long Teflon sliders were mounted to the underside of the girder bottom flanges at the pontoon end. This configuration was needed to accommodate the design motions, including rotations about the vertical axis due to flexural deformations of the floating span. HDPE-lined girder

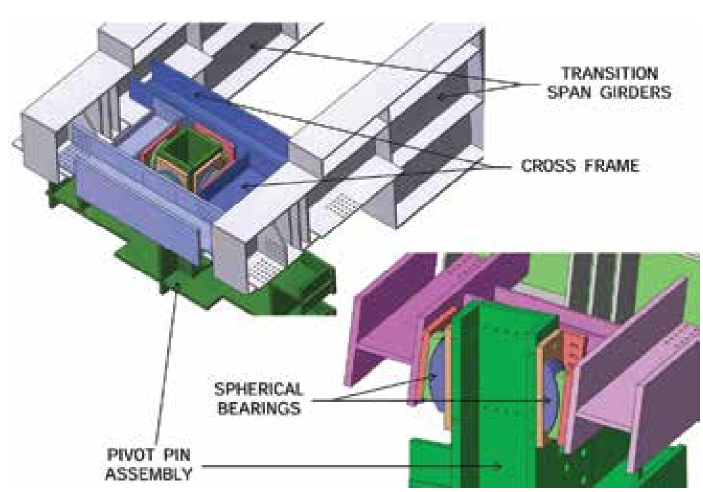

Figure 3. Trailer hitch schematic.

Figure 4. Girders being erected over the trailer hitch.

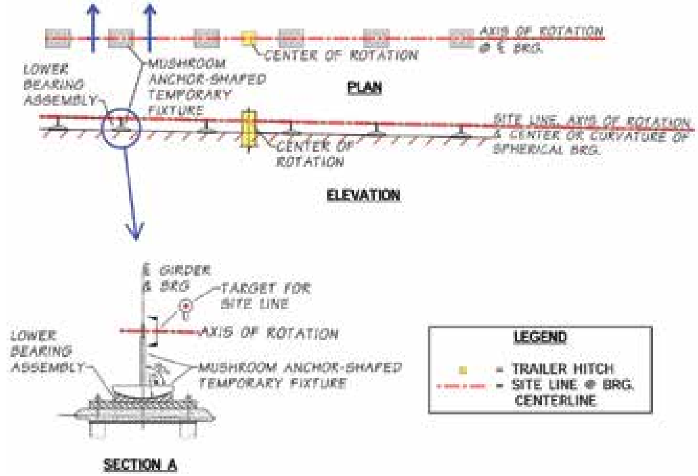

Figure 5. Temporary target for aligning girder bearings with trailer hitch bearings.

stops provide transverse lateral restraint to the pontoon. The girder stops were configured to allow free rotation about a central vertical axis at the pontoon end. At the East end of the span, longitudinal and transverse restraint is provided by the trailer hitch post which is mounted on the cantilevered end of the approach span. The side mounted spherical bearings on the trailer hitch post allow free rotation about the central vertical axis. Short sliders at each girder's spherical bearing accommodate vertical axis rotation at the east end of the span.

The center-of-curvatures of the four spherical bearings on the trailer hitch post align with an axis of rotation of the spherical bearings on the girder ends located on the approach span. Similarly, the center-of-rotation of the spherical bearings on the girder ends located at Pontoon W, at the baseline neutral pontoon elevation position of the pontoons, were located on a line parallel to the bearing center-of-rotation line at the approach span end, so that the span can accommodate pontoon heave without binding (*Figure 5*).

The lower half (concave plates) of the spherical bearings for each girder end were set plumb on individual pads following the transverse slope of the roadway. The trailer hitch post was set plumb. The centers-of-curvature of all four convex plates on the trailer hitch post were adjusted so as to be coincident with each other and the line through the axis-of-rotation of the girder bearings.

Torsional Flexibility of the Transition Span

So as to not disrupt the driving surface with deck lifting at the expansion joint during pontoon motion, the deck of the transition span had to be sufficiently flexible in torsion. Flexibility was achieved by using steel I-section girders with no bracing along the girder bottom flanges and transverse diaphragms only at the ends, quarter points, and mid-span.

The transition spans were analyzed using a finite element model (FEM). The FEM model with solid end diaphragms comprised an open-bottom and closed-top box. The transition span was analyzed for the worst-case service load condition associated with pontoon storm roll and zero live load. In the FEM model, a comparison was made between the uncracked and cracked (a concrete modulus reduced by 50%) bridge deck stiffness. It was determined that the deck had only a nominal influence on the torsional stiffness. The most significant factors contributing to the torsional deck stiffness were the inclusion/omission of the end and intermediate diaphragms. With the selected transition span configuration, the compression reaction was approximately 50 kips under the worst-case service load condition.

Construction

The spherical girder bearings were aligned by sighting a laser along the axis of rotation. As shown in *Figure 5*, a mushroom-shaped temporary fixture, machined to fit the lower concave bearing plates with a target on the end of a shank to locate the center-of-curvature, was plumbed in place. Screw jacks on the lower bearing plate were used to adjust the bearing center-of-rotation to match the laser line.

A hole through the trailer hitch and bearings allowed the same laser line to define the location of its center-of-rotation. Once the girder lower bearing elements were positioned relative to the trailer hitch bearings, the girder grout pads were cast and cured, and then the girders were set transversely plumb with their individual convex plates and sliders on the previously erected lower halves of the girder bearings (*Figure 5*).

The girders were erected in pairs, at a minimum, with the end diaphragm and trailer hitch framing beam installed. The uphill trailer hitch convex spherical bearing assembly was loose fit to the framing beam. The uphill bearing on the trailer hitch post keeps each span from sliding down onto the pontoon. The bearings could be adjusted up and down, and sideways, to locate its center-of-curvature coincident with the site line. Once the girders were set on their bearings, the uphill post spherical bearings were locked into position, allowing the approach bridge end of the span to rotate freely about the sloped line of the girder bearings.

The downhill trailer hitch spherical bearings could be adjusted the same way; however, they were only snug fit to avoid jamming when the span rotates. The trailer hitch bearings aligned with the cross slope were adjusted the same way as the longitudinal bearings.

By accommodating these transition span movements, the SR520 Floating Bridge spanning Lake Washington can remain serviceable even under extreme weather events.▪

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Guam Regional Medical City

By Frank K. Humay, Ph.D., S.E. and Yuriy Mikhaylov, S.E.

Exterior view of the completed hospital.

Imagine reading the front page of your daily newspaper to find out the current bed availability or the number of people on the waiting list for the only hospital in town. Now imagine it is a necessary announcement because this hospital is almost always at full capacity and that the next closest U.S. hospital is an eight-hour plane ride away. Fluctuating hospital availability has been the reality on the remote U.S. island territory of Guam. Fortunately, Guam recently received a much-needed influx of health care professionals and specialists from a newly completed hospital.

Guam Regional Medical City (GRMC) is a new, private, $240 million, 130-bed acute care facility in Dededo, Guam. Located approximately 4,000 miles west of the Hawaiian Islands in the Micronesia region of the Western Pacific Ocean, Guam has a population of approximately 165,000 and a land area of 212 square miles (about three times the size of Washington D.C.). Guam is home to strategic U.S. military installations for both the U.S. Navy and U.S. Air Force. Furthermore, the U.S. Navy is in the early stages of planning for the relocation of 5,000 Marines from Okinawa, Japan.

Completion of the five-story state-of-the-art medical facility overcame several major design and construction challenges: First, Guam is subjected to some of nature's most destructive forces, including damaging earthquakes, major typhoons, and a corrosive tropical environment. Second, Guam is extremely remote and locally available labor, materials and construction techniques were a major concern in the final selection of the structural systems. Finally, the unique criteria relevant to hospital function, such as stringent vibration control of the floor system, were incorporated through meticulous coordination with the first two challenges.

View from tower crane during construction. Courtesy of dck pacific guam, LLC.

Typhoon Winds and High Seismic Requirements

GRMC is designed in accordance with the *International Building Code* (IBC), 2009 edition, and ASCE 7-05 for a three-second gust wind speed of 170 mph (the highest value included in IBC 2009), seismic accelerations of $S_s = 1.50g$ and $S_1 = 0.60g$ and a corrosive tropical environment. Design wind pressures were extremely high as illustrated by the maximum component and cladding wall and roof uplift pressures of approximately 180 psf and 260 psf, respectively. Seismic demands were similarly high and controlled the design of the lateral force-resisting system. The structure was assigned to seismic design category D and had a seismic response coefficient, C_s, equal to 0.25.

The GRMC project reveals that design approaches for dealing with high winds are often at odds with those for handling high seismic demands. One clear example is in the design of the roof structure. Whereas it is advantageous to add mass to the roof in order to counteract the extreme wind uplift pressures, added mass increases the inertial seismic forces that the lateral force-resisting system must resist. Such considerations were carefully weighed and fine-tuned throughout the design process.

Constructability and Primary Structural System

Another key consideration from the onset of the project was constructability in Guam. The selected delivery method for the project, construction manager (CM) with a guaranteed maximum price, allowed for direct collaboration between the design team and the general contractor (GC) from the onset of design. For the GRMC project, the CM also acted as the GC. Various structural schemes were evaluated including structural steel, cast-in-place (CIP) concrete and precast concrete. Ultimately, the decision was made to employ a combination of CIP concrete for the vertical framing and precast concrete for the horizontal framing.

Structural steel was considered but eliminated as an option for several reasons, primarily because of shipping costs, the cost of fire protection and the long lead time needed for ordering the material. Another potential risk was the absence of local steel fabricators. Addressing steel related field issues or fabrication errors might create significant schedule impacts if repairs could only be done off-island. CIP and precast concrete, on the other hand, is produced locally. Furthermore, the CM was experienced in CIP concrete and was capable of self-performing much of the CIP concrete work.

The final structural framing for GRMC consists of 24-inch deep precast prestressed concrete double tees, precast prestressed beams, CIP columns and walls, and a foundation consisting of spread and mat footings. Although rarely used for hospital construction, precast double tees were selected as the most desirable option for the project. Precast double tees are locally produced in Guam, allowing for shorter lead times and ensuring field issues can be readily addressed. Precast floors also eliminated the majority of horizontal formwork and shoring, increased the speed of construction, and cost less compared to CIP floors. Coordination between the design team, CM/GC, and precaster was critical because prestressed double tee stems allow for less flexibility than other types of floor framing, particularly when considering the layout of MEP systems. To help address this concern, a BIM model was shared between the design team and the CM/GC.

The lateral load-resisting system consists of a building frame with special reinforced concrete shear walls. To provide optimum flexibility, shear walls are primarily located at elevator and stair cores and the short ends of the building. Due to the concentrated high lateral forces, shear walls are 12 inches to 26 inches thick and heavily reinforced. The majority of the concrete shear walls require boundary elements and the end walls contain coupling beams reinforced with diagonal

Detail at precast double-tee support to provide continuity.

Boundary element reinforcement at the special shear wall.

Diagonal reinforcement at the special shear wall.

bars. A minimum 3.5-inch thick composite CIP concrete topping over the precast double tees acts as a horizontal diaphragm at each floor/roof level. The reinforcement in the topping slab is designed to transfer all the diaphragm shear forces. Also, chord and drag strut reinforcements are provided to ensure an adequate load path for the inertial floor loads into the shear walls.

Vibration Design

Vibration control of the floor structures for both patient comfort and specialized equipment was a critical consideration in the structural system selection. The final selected criteria is summarized as follows:

- Patient rooms (fourth and fifth floors) – maximum peak acceleration = 0.5% of g; maximum vibration velocity = 8,000 µin/second.
- Surgery, operating rooms, etc. (first, second and third floors) – maximum peak accelerations = 0.2% of g; maximum vibration velocity = 4,000 µin/second.
- Vibration limits were based on a "moderate" walking speed.

Vibration criteria were fine-tuned in collaboration with the hospital planner based on proposed usage, future flexibility, cost constraints, and other use issues. In assessing the vibration of the precast floors, "slow," "moderate" and "fast" walking scenarios were investigated. These separate cases were first evaluated using the structural properties determined, assuming the precaster's typical details. Next, different aspects of the typical precast details were slightly modified to enhance the vibration control of areas not meeting the selected criteria. Among the detail changes were providing rotational restraint at the ends of the double tees, providing rotational restraint at the ends of the girders, and reducing the widths of the typical 10-foot wide double tee member.

One modification incorporated into the project was to provide rotational restraint at the ends of the double tees, which are usually designed and detailed as simple span members. Instead of typical precast inverted tee beams, the double tees are supported on precast prestressed soffit beams. The double tee prestressing strands extend past the ends of the stems and into the web of the soffit beams that are poured monolithically with the concrete topping. Furthermore, the top of each adjacent double tee stem is tied together across the soffit beam by reinforcement steel welded to embedded plates. This detail was used at the first through third floors to decrease the maximum vibration velocity under a moderate walking pace excitation. Providing rotational restraint at the ends of the double tees decreased the maximum velocity for a typical 37-foot span from approximately 5,440 µin/second to 1,920 µin/second.

For functional reasons, the primary mechanical rooms were located on the third floor which is also the main laboratory level. To isolate noise, shock, and vibration from the primary structure, the large air handling units are supported on a floating concrete slab, which in turn bears on a layer of insulation and fiberglass isolators. A challenge was incorporating the appropriate isolation characteristics and ensuring appropriate stability from seismic overturning forces.

As one of the most remote places in the U.S., Guam is both a strategic hub for U.S. military operations and an exotic island getaway destination. The island is inundated with frequent typhoons and large earthquakes, which its self-sufficient and resilient residents typically take in stride. Much like its inhabitants, buildings in Guam must possess these same traits. Construction of GRMC provides a new world-class hospital designed to withstand the extreme forces of nature and provide much needed alternative health care options for the residents of Guam.▪

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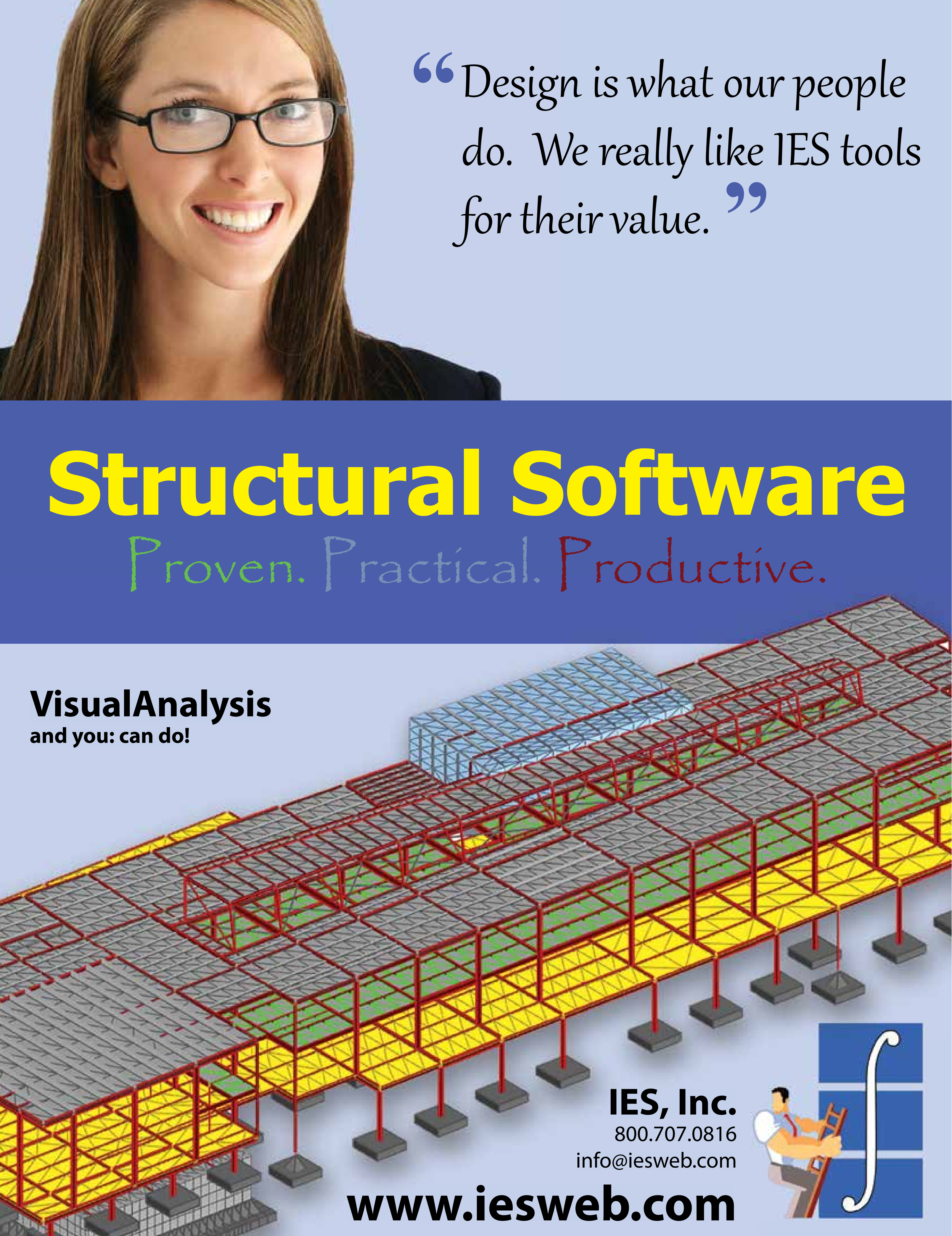
Project Team

Owner: Guam Healthcare Development, Inc.
Structural Engineer of Record: BASE Micronesia, Inc.
Special Inspector of Record: BASE Micronesia, Inc.
Architect of Record: Setiadi Architects, LLC
Hospital Planner: Flad Architects
CM/GC: dck pacific guam, LLC
Precast Concrete Supplier: Rocky Mountain Precast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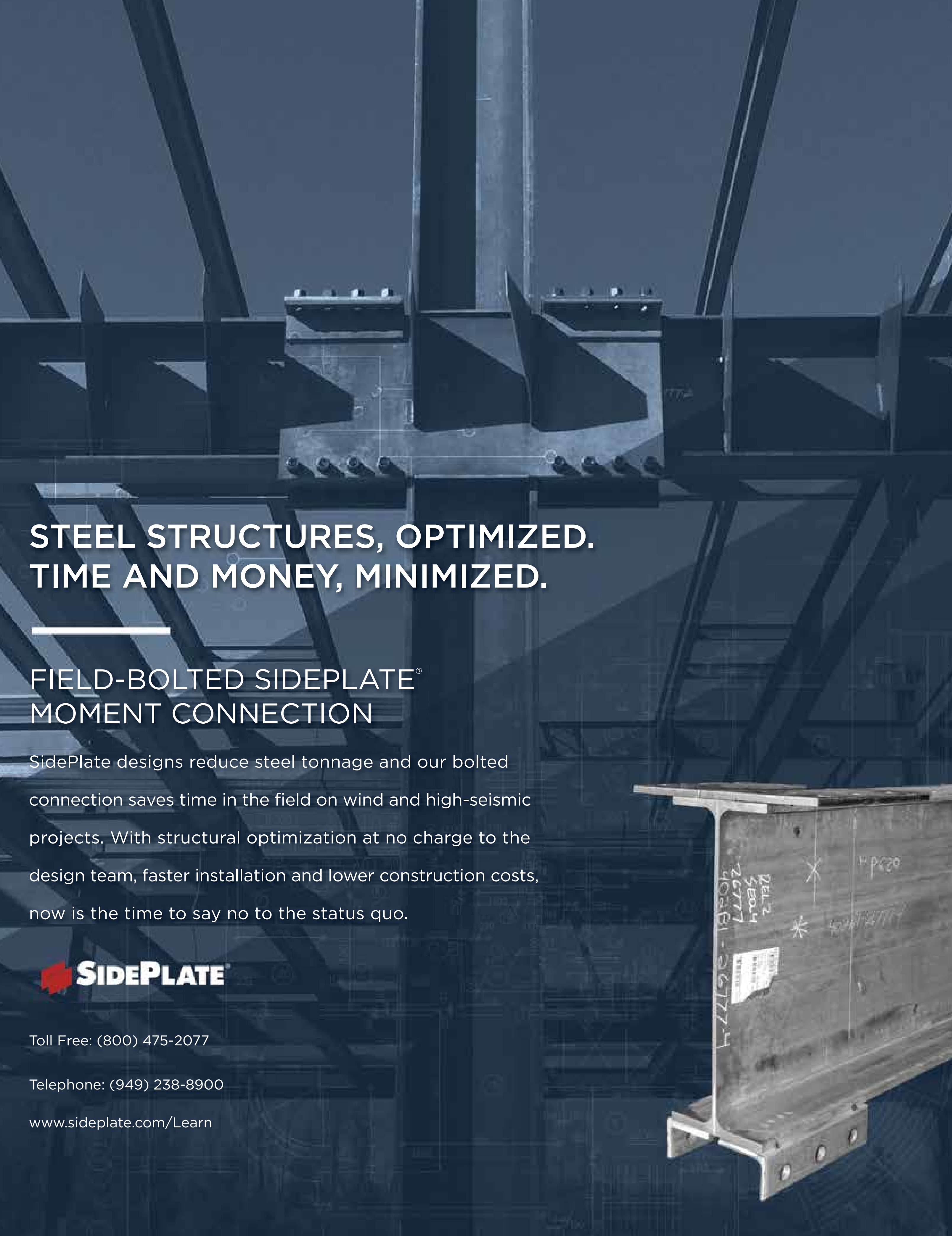
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Brooklyn Bridge

Part 2

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

Washington A. Roebling assisted his father in the design of the bridge from March 1868, after he returned from his combination honeymoon and fact-finding trip to Europe. While in Europe, he visited Telford's Menai Straits suspension bridge, Brunel's Clifton suspension bridge, and many others. Washington toured the steel mills of England and Germany and learned all he could about pneumatic caissons. He wrote lengthy letters to his father about his trip, containing information that would be helpful in the bridge design. He spent most of the summer working on plans and the equipment necessary to place the huge pneumatic caissons for the Brooklyn and Manhattan Piers. Based on borings, rock was 106 feet deep on the Manhattan side and 98 feet on the Brooklyn side. No one had ever reached those depths with pneumatic caissons – even though James Eads was about to in St. Louis – as the pressure of 50 psi and greater made it necessary to keep the water out. By the end of 1868, the father and son team had worked out tentative plans for the caissons. As mentioned in Part 1 (STRUCTURE, August 2016), the team of consulting engineers in the spring of 1869 made their tour of Roebling's other suspension bridges at Niagara, Pittsburg, and Cincinnati, and submitted a favorable report.

John A. Roebling suffered his accident near the Fulton Ferry on June 28, 1869, and died on July 22^{nd}, less than a month later. At this time, very few detailed plans for the bridge and caissons had been prepared other than preliminary plans submitted with John's 1867 report. He had indicated to William Kingsley and others that Washington was more than qualified to take over for him. In early August, the Board appointed Washington Roebling Chief Engineer with the support of the *Brooklyn Eagle*, one of the main supporters of the bridge. He was only 32 years old and found himself in charge of the longest suspension bridge in the world.

The first thing Washington did was recruit a team of associates to assist him in the design and construction of the bridge. His father had used two young German engineers in 1867 to help him in the design. Washington, however, went to his alma mater, Rensselaer Polytechnic Institute, and hired Francis Collingwood (class of 1855) and C. C. Martin (class of 1856). He added Wilhelm Hildenbrand, who had assisted his father in 1867, and Col. William Paine, whom he may have met during the Civil War. Sam Probasco and George McNulty completed the seven-man team. These men would stay on the project for the next 14 years.

The first task was to prepare the final design for the Brooklyn caisson. Roebling knew that this was the most difficult part of the project given the depth to a firm foundation. To support the huge masonry towers and twin Gothic arch passageways his father had designed, a caisson 168 feet by 102 feet was required. He put Collingwood, Hildenbrand, and Paine to work on the design. A contract was awarded to Webb & Bell, a local shipbuilder, on October 25, 1869, to build the caisson in a nearby shipyard and launch it like a ship when completed. The caisson had two dredging (water shafts) wells, two supply shafts, and two airlocks for worker's access. The pressurized chamber was 9½ feet high and had 5 layers of timbers for a roof and tapered walls with an iron cutting edge at their base. The outside was covered with sheet metal. Also, he had five cross walls included supporting the roof during the launch and the placing of the masonry. The 6,000,000-pound caisson was launched on March 19, 1870. It was the heaviest structure ever launched in the United States at the time. His father's early plans had a much thicker roof that Washington changed when he became Chief Engineer. It was towed into a prepared enclosure and positioned on May 3^{rd}. He then added 10 more tiers of timber to the top of the caisson, making it 15 feet thick.

Washington A. Roebling.

As masonry was added to the top of the caisson, it sank into the softer surface soils. When it was sealed all around, compressed air was pumped into the chamber, expelling the water. Men then accessed the chamber through the air locks and started digging out under the cutting edges and cross walls. They loaded the dirt into wheelbarrows and dumped it under the water shafts, where it would be removed by clamshell buckets, lifted through a column of water, and dumped into scows. Progress was slowed when a fire broke out, burning many timbers in the roof. At a depth of 44½ feet below the river surface, Roebling determined the dense, rocky soil was firm enough to rest his tower on. The chamber was filled with concrete and brick piers and was completed in March 1871.

The New York Caisson was a little larger, 172 feet by 102 feet, and weighed 3,250 tons. Its timber roof was also 12 feet thick. Like the Brooklyn caisson, additional timbers, 10 layers, were added due

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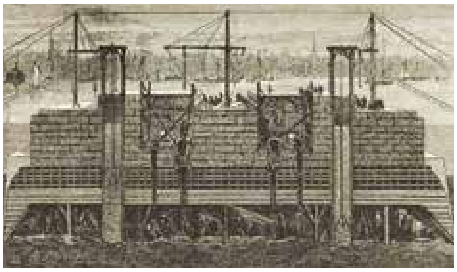


Brooklyn Caisson.

to the extra weight of masonry planned to be placed on it when it was floated into position. It was launched and moved into position in October 1871. To guard against fire, Roebling lined the caisson with sheet iron. The soil was mainly sand, and he used a sand pump, similar to that employed by James Eads at his St. Louis Bridge, as well as a piping system that allowed the compressed air in the caisson to eject sand to the surface. Roebling had visited Eads to observe his pneumatic caissons and his use of sand pumps.

As he went deeper, past 45 feet, men started suffering from what became known as caisson disease, the Bends, with some fatal attacks (estimated at five deaths). Eads, at St. Louis, determined he had to regulate the number of hours men worked under pressure and how long they should decompress before exiting the air locks. Roebling believed it was a matter of how long men worked under pressure and cut the workday to 4 hours under pressure, 2 hours out of the caisson, and another four hours under pressure. He later reduced the shift time to 5 hours. His doctor on staff prepared a list of precautions the men should take and eventually increased the time it would take to decompress in the air locks. Despite this, his men continued to suffer. On May 18, 1872, at a depth of 78 feet 6 inches, Roebling decided the dense sand and gravel would support his tower.

The chamber was filled with only concrete this time. While it was being placed, Roebling suffered another case of the "bends," and many thought he might die. He recovered but had recurring bouts of pain. The caisson was filled with concrete on July 12, 1872. Roebling started missing many days at work due to the lingering effects of the disease.

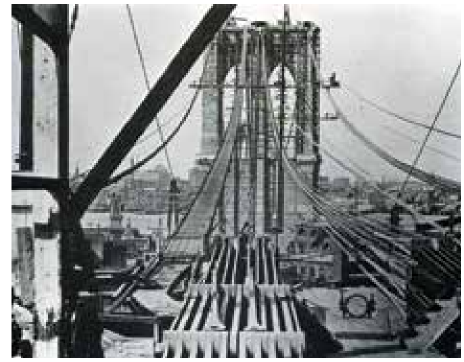
Masonry continued to be added to both towers. By late November, work stopped on the Brooklyn Tower at an elevation of 145 feet, well above the deck level. The Company had widened the bridge to 85 feet, Roebling had modified his father's plan, and now the three legs of the tower were taking shape. In late December, work stopped on the New York Tower at a height of 60 feet. Roebling's health deteriorated rapidly. During the winter, when able, he wrote detailed instructions for his assistants to follow when work began again in the Spring.

Roebling asked for a leave of absence in the Spring of 1873 and took a trip to Germany with his wife. In his absence, work on the towers continued. The Brooklyn Tower was completed in June 1875 and the New York Tower in July 1876. They were the second tallest structures in New York at the time.

Work on the anchorages started in February 1873, with McNulty in charge of the Brooklyn anchorage. It was completed in 1875. The New York anchorage began in May 1875 and was completed by Collingwood in July 1876. They were massive structures, 119 feet by 129 feet at the base and 104 feet by 117 feet at the top, and were temporarily stopped at a height of 80 feet. Each weighed 60,000 tons. Massive anchor castings were built with a double row of chain with 3-inch by 7-inch wrought iron eyebars near the anchor casting, increasing to 3- by 9-inch approximately. In the end, there were 38 links for each cable to pick up the 19 strands in the cable.

Roebling returned from Germany in early 1874. His doctors told him to stay away from the bridge, so he and Emily lived in Trenton for the next two years. Washington kept in touch with his assistant engineers, providing detailed written instructions on how to place the masonry in the arches of the towers and anchorages. While the masonry was finished, he began planning for the spinning of the massive cables. Aiding him in this effort was his Master Mechanic, E. F. Farrington, who had worked with him on spinning the cables for the Covington and Cincinnati Bridge.

He completed the specifications for the cables in 1876. They would be 15½ inches (later 15¾ inches) in diameter, consisting of 5,282 wires about ⅛-inch in diameter (Number 7 Birmingham) and for the first time would be steel. He would spin the cables in 19 strands of 278 wires each, and pack them so they could be squeezed into a cylinder that would be wrapped with iron wires. The wires were to be "superior quality steel" and galvanized. He needed 3,400 tons of material with a tensile strength of 160,000 psi. Farrington set up the spinning equipment. On August 14, 1876, the first two cables were carried across the river and connected to form a continuous loop mounted on drive wheels. Farrington, on a boatswain's chair, went across the river on August 25. The Board, on a motion by Abram Hewitt, voted that no one associated with the Bridge Company would be allowed to bid on the wire contract, including John Roebling's Sons wire factory nor his company. Roebling threatened to resign over what he considered to be a direct slam on his integrity



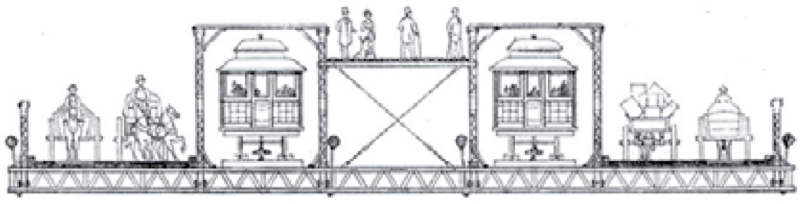
Cable spinning, showing anchor chains, foot walk, adjusting cross walkways, and tower.

but was convinced to carry on. He sold his stock in the Company, so John Roebling's Sons could bid on the wire. He did write a letter to Hewitt stating, "If you receive a bid from a Mr. Haigh of South Brooklyn, it will be well for you to investigate a little."

In October 1876, Roebling returned to Brooklyn from Trenton by boat and saw the bridge for the first time in three years. He stayed with his brother-in-law, General G. K. Warren, for a while before moving back to Brooklyn Heights. As his health was still poor, he communicated with his assistant engineers through detailed written instructions frequently picked up at the apartment, but sometimes carried to them by his wife Emily, who also wrote some of them from dictated notes by Roebling.

On December 4, 1876, nine bids were received. One of John Roebling's Sons bids was for Bessemer steel, and it was the lowest submitted, but their other bid, for crucible steel, was higher than J. Lloyd Haigh's. The Board was ready to award the contract to Roebling for Bessemer steel but many, including the Editor of the *Brooklyn Eagle*, claimed that just because it was cheaper, it should not be used in a bridge of this size and importance. In late December, the Board yielded and awarded the contract to Haigh for crucible steel.

Roebling, not trusting Haigh, instructed his men to check every "ring" of wire at Haigh's factory before delivery. By June 1878, he determined that Haigh unsuccessfully attempted to bribe the inspectors and that he had been delivering previously rejected wire to the bridge as having passed inspection. What Haigh did was to replace accepted wire with rejected wire overnight and then delivered it as good wire, after which he submitted the previously accepted wire for testing. At other times, the wagons carrying good wire were rerouted to another location where the good wire was replaced with the rejected wire. It and the acceptance paperwork were taken to the bridge. The inspectors noticed the rejected



SECTION OF BRIDGE, SHOWING FOOT, RAIL, AND CARRIAGE WAYS.

Deck Layout with elevated promenade.

Brooklyn Bridge, Currier & Ives 1883. The Great East River Suspension Bridge.

pile of wire never grew. Roebling and his men found out about this fraud and notified Murphy on July 29.

Murphy and the Board, who had selected Haigh over Roebling, chose not to make this fraud public nor to switch suppliers as Roebling suggested. Roebling estimated that as much as 221 tons of rejected wire had been spun into the cables, which would leave a safety factor of five in the cables. He did, however, add 150 wires to each cable as an additional safety factor and required Haigh to supply them free of charge. Cable spinning of the four big cables began on June 11, 1877, and was completed on October 5, 1878. Washington made sure that the point where the wire cables linked to the anchor chain was open to the air and visible. John had, erroneously it turned out, always covered this connection with concrete, thinking it would protect the wires.

With the cables complete and wrapped, the next step was to install the hangers, deck trusses, and stay cables. The process was the same as Washington used at Cincinnati but on a much larger scale. Roebling decided in the spring of 1879 to use steel for his deck structure for the first time. His father had specified wrought iron. So with steel wire cables and steel trussing, it was, except for the anchor chains and cast iron saddles on top of the towers, an all steel bridge. Eads was using some steel at St. Louis, but the first all steel truss bridge was at Glasgow, Missouri across the Missouri River in 1879. Roebling had Paine visit several steel manufacturers to determine their ability to supply the amount he needed. The Edge Moor Iron Company was awarded the contract. The floor trusses were the first to go in, starting in the spring of 1880. They were hung from the Bessemer Steel suspenders, supplied by the Roebling Company, which were spaced 7½ feet on center. The trusses were placed symmetrically working outward from both towers to balance the load on the cables towards the land and mid-span. Each truss hung from two cables, and they were connected at mid span. Edge Moor started falling behind in deliveries, which delayed finishing of the deck trusses until December 1881.

Washington modified his father's layout of the deck and promenade but kept the cable driven cars and, as noted, increased the deck width to 85 feet. Due to slow deliveries from Edge Moor, who blamed it on slow deliveries by the Cambria Iron Company, the longitudinal trusses, four high trusses close to the centerline of the bridge and two low trusses at the edges of the bridge, were not completed until April 1883. Washington made the trusses continuous except for an expansion joint midspan. At Cincinnati, his father built the trusses in 30-foot sections connected only by slip joints.

The last structural elements to be added were the stay cables, which were a feature of all John A. Roebling's bridges. Some of them rested on the saddles on top of the tower, and some were connected to eyebars built into the masonry. Washington did not like stays and reported that it was very difficult to adjust them so that they and the cables worked in tandem with each other. There were 25 stays on each side of the tower per cable or a total of 400. They were Bessemer steel cables supplied by the Roebling Company. The stays were connected with the suspenders at each crossing giving a spider web appearance that was one of the most aesthetic features of the bridge. Stay cables were not used to any significant degree after the Brooklyn Bridge.

Southern yellow pine wooden decking completed the project. The endless cable car system was the last feature installed in the bridge. It was patterned after the famous San Francisco cable cars, where the cable is constantly moving, and the cars hook onto or release the cable when desired. Col. Paine was the main assistant engineer working on this feature of the bridge. The first passengers were carried across the bridge in September 1883 when C. C. Martin was Chief Engineer, replacing Roebling who resigned in July.

Roebling never went to the bridge to observe what was going on from 1873 to 1883, but still maintained control of the design and construction through his dedicated assistant engineers. Near the end of construction in 1882, an attempt was made to remove him as Chief Engineer by Mayor Seth Low of Brooklyn and others, including two of the leading engineers of the period, Charles Macdonald and T. C. Clarke. Roebling was ordered to attend a meeting of the Board on June 26 to discuss the delay in finishing the bridge. He refused to attend and sent a short telegram giving his reasons. Low later visited him at his apartment in July and told him he should resign. Roebling refused and said if he wanted him out they would have to fire him. A motion was made to appoint Roebling as a consulting engineer and to name C. C. Martin as Chief. The motion failed by a vote of 10 to 7. By the Spring of 1883, Washington's health had improved. He was able to move around his apartment but was not well enough to visit the bridge. Work was quickly finished on the deck and promenade in April 1883. On May 23, the bridge was opened with a grand celebration of fireworks, parades, and speeches. The bridge took 14 years to build. Roebling had estimated 5 years, and its cost rose from Roebling's $7,000,000 estimate to $15,500,000.

The bridge is a monument to the genius and perseverance of John A. and Washington A. Roebling. Its conception was John A. Roebling, but the thousand and one decisions that made the bridge a success were from Washington A. Roebling. Numerous contemporary accounts of the bridge indicate that Washington carried the details of the bridge around in his head. He had to fight not only caisson's disease but the political infighting of politicians and many times a lack of funding. His battle with James Eads over the credit of the design and sinking of pneumatic caissons was also a distraction. His assistant engineers and master mechanic also deserve credit as they, in Washington's prolonged absence from the bridge, were able to take his written instructions and working in stone, steel and wood build *The Great East River Bridge*.

It remained the longest suspension bridge in the world, with its central span of 1,595 feet 6 inches, until Leffert L. Buck's Williamsburg Bridge opened in 1903 with a span of 1,600 feet. It is a National Historic Civil Engineering Landmark (1972) and is on the Register of Historic Places (1966) of the National Park Service.▪

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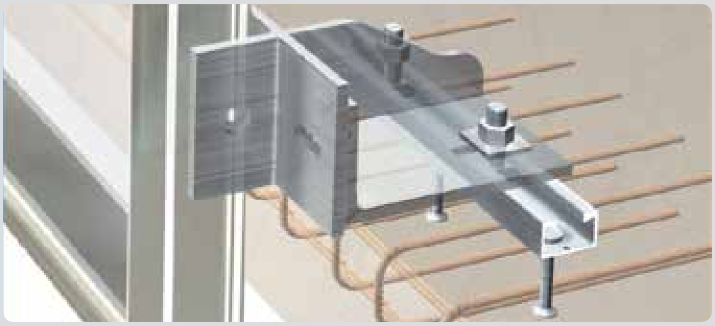

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Drones and the Forensic Engineering Industry

By Bruce A. Barnes, M.S., P.E.

On January 10, 2015, a refinery in Lima, Ohio suffered a catastrophic failure resulting in an explosion. This explosion reportedly rattled windows as far away as five miles. Following suppression of the fire, the investigation into what caused the explosion commenced. However, investigators probing the debris field were faced with the challenge of documenting the layers of damage relative to each other in three dimensions. Furthermore, as the investigation progressed, much of the debris that was not related to the fundamental cause had to be removed. Removal of the debris can be problematic if later in the investigation these materials are found to have contributed to the causation and proper debris documentation has not been secured. This scenario is common in massive explosions and structural failure cases. Investigators must approach debris fields carefully, all the while preserving the location and condition of the evidence as it is encountered in the field. Unmanned Aircraft Systems (UASs or drones) can map and document these complex debris fields without disturbing the physical evidence. As a result, drones are becoming a go-to tool for the forensic engineering industry.

Increasing Popularity

The usage of UASs in the United States has increased significantly, varying from recreational pursuits to commercial endeavors. The technology associated with drones is rapidly evolving allowing for more autonomous operation of these vehicles. Because the vehicles themselves operate independently, the Federal Aviation Administration (FAA) has created specialized certifications for operators of UASs (FAA Section 333 Exemption) with 5,537 granted as of July 19, 2016. The Association of Unmanned Vehicle Systems International (AUVSI) reported that over 180,000 drones have been registered with the FAA as of June 1, 2016. Recently, the FAA has relaxed the regulations regarding drone usage; operators no longer need to be licensed pilots, resulting in increased opportunities for the use of drones in the forensic market.

Events occurring in and around airports where drones have nearly collided with aircraft highlight the dangers associated with the use of drones in terminal control areas. Despite the risks, drones offer several advantages enticing the utilization of these technologies in numerous industries. The growth in the consumer market alone is estimated to more than quadruple over the next five years. Given the rapid expansion in the availability of individual platforms and the associated technological growths, the use of drones is certain to spread throughout the forensic engineering industry.

Drone Technology

Drones are available in several configurations, from hand-launched aircraft configurations to rotary-wing type vehicles. For obvious reasons, the rotary wing configuration (Vertical Take Off and Landing, or VTOL) offers significant benefits when inspecting buildings and confined spaces, as this platform allows stationary observation of individual items of interest. These VTOL platforms are often characterized by the number of rotors attached to the frame, such as quad-copter or hex-copter.

Currently, drones are often utilized to capture aerial images and measure distances, areas, and volumes in the mining industry. Commercially available photogrammetry software (a process that is fundamentally based on triangulation between points in numerous images of the same subject, taken from differing directions), coupled with the aerial platform of a drone, can produce horizontal accuracies of plus or minus 1.2 inches and vertical accuracies of plus or minus 2.0 inches. Higher resolutions are reported, but hard documentation of these higher resolutions was unavailable. Significant placement of ground reference targets is required for higher resolutions. Additional cameras such as infrared devices are also mounted on drones, producing relevant documentation of fire scenes and critical water intrusion/roofing inspections.

As technology has evolved, terrestrial laser scanning systems have been miniaturized and mounted on UASs. This technology is commonly referred to as LiDAR (Light Detecting and Ranging) and can produce resolutions approaching 0.2 inches or less of XYZ coordinates in a point cloud.

The LiDAR technology is based on laser scanning and is highly useful when paired with drones in a forensic investigation. Frequently, terrestrial-based laser scanning is utilized in the design and construction market to document structures for Building Information Modeling (BIM). LiDAR, as applied to early forms of BIM and infrastructure, was pioneered by Departments of Transportation through the usage of bulky laser scanning instruments mounted on tripods and paired with cameras.

The laser scanning technology has since advanced, and modern instruments are significantly smaller and more lightweight offering up to a one million point-per-second scan rate. However, terrestrial-based laser scanning does not capture detail from above. Hence, shadows, where the laser beam is obstructed by target items, exist. Because complex debris fields demand scans from above to maximize the scan coverage, multiple

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instrument setups are required to overcome these shadows. Furthermore, to utilize terrestrial-based scanning, one must physically access the area surrounding the items of interest to set up and level the instrument. This physical access becomes problematic when structures are damaged, the items of interest are within a collapse zone, and access is unsafe. Therefore, the pairing of LiDAR systems with drone technology becomes attractive.

Research into fully autonomous drones has rapidly advanced when combined with onboard artificial intelligence. While this technology is still under development, piloted drones are currently being implemented to scan large debris fields, damaged structures, and remote infrastructural elements such as powerline systems for inventory management as illustrated in *Figure 1*.

Major limiting factors applicable to a drone system's flight time are weather, payload, and battery power. The time to map an individual site at a low resolution is significantly less than that required at a high resolution; a shorter time-on-station is required. High definition scans require multiple passes over the same area of interest. Of note, navigation of a drone into a partially collapsed structure requires a significant amount of time to accomplish high-resolution scans, and the power demands associated with this longer flight time are high. The ingress and egress and the energy demands associated with the lighting and instrumentation package can severely limit overall flight durations and tasking. As battery technology advances and the power density in a given battery volume correspondingly increases, time-on-station and payload capabilities will certainly increase.

Figure 1. Aerial scan of transmission line using photogrammetry.

Figure 2. LiDAR scan of damaged school following a tornado. Courtesy of Richard Wood from the University of Nebraska-Lincoln.

Industry Usage

Numerous insurance carriers are developing drone systems for deployment. As of April 2015, three insurance carriers have been granted FAA approval to operate drones in the United States. Moreover, forensic engineering firms have also obtained FAA clearance for the use of drones. Given the intensely competitive nature of the insurance and forensic engineering markets, the intended use of the drone systems is generally not disclosed. However, drone and scanning technologies are ideally suited for activities like aerial surveying and mapping. Specifically, identified usages include:

- Roof Investigations
- Building Envelope Investigations
- Fire and Explosion Investigations
- Catastrophe Assessment
- Underwriting Surveys

Drones are ideally suited for providing useful information in a disaster such as wide-area surveys of the extent of damage from windstorms, mudslides, tornadoes, and hurricanes. Reports of drone use for forensic hail and wind investigations are often suggested as a means to reduce costs and minimize site expenses. However, a drone cannot probe a suspected hail impact site for bruising, nor can a drone lift an individual shingle tab to determine whether or not the tab is disengaged from the substrate. As such, drones have limited applicability to wind and hail surveys other than standard documentation of visible damage. Causation relative to visible damage still requires access and inspection by a human hand. While drones can measure roof surfaces, commercially available sources provide this service for a reasonable fee. In contrast, drones excel when tasked for wide-area surveys and surveys of inaccessible sites on individual buildings.

Probably the most readily identifiable use of drones in the forensic engineering and claim adjusting market involves the documentation of damage in inaccessible areas. For instance, the failure of a rigging system in New York City while lifting new HVAC components into a high-rise structure produced damage in highly inaccessible areas. Inspection of the damage in these areas required special equipment and specially trained individuals. Installation of this specialty equipment, such as swing-stage scaffolding, can take months to schedule, deploy, and install. The usage of a drone platform in this instance, while not employed, could have accelerated the damage evaluation. However, the complexities of deploying a drone in the New York City airspace precluded the use of this valuable tool at that time. In similar cases, drones equipped with high-resolution cameras and LiDAR could potentially inspect the damage and provide useful documentation of quantities for the adjustment of the claim within

Figure 3. Complex debris field.

days. Furthermore, the high-resolution images could be utilized in a forensic engineering investigation to determine the extent of damages and the required potential repair methodologies. Notwithstanding the possible advantages, the deployment of drones in dense urban areas and secure sensitive installations such as harbors is problematic and requires at least the approval of all involved parties, and police and fire departments. Also, navigating drones capable of flying in gusty wind environments surrounding buildings is troublesome, given the lightweight nature of the vehicles.

A less complicated application of photogrammetry and laser scanning utilizing drones is dramatically illustrated by research performed by the University of Nebraska, Lincoln following a tornado. In this research, faculty members used a drone to capture point cloud data of a damaged school building as shown in *Figure 2 (page 57)*.

This three-dimensional digital model exists in cyberspace and can be the basis for accurately acquiring quantities and measurements. The model can be viewed through a secure, online digital portal by anyone associated with the job. Furthermore, the high-resolution nature of the digital model lends itself to excellent communications with clients regarding the scope of damage and resulting cost of repair. Without drones, this level of data collection would require extensive effort and would lead to a much lower quality level of documentation for the client.

While extensive work is still required within the interior of the structure, the gross characterization of the damage from the exterior of the structure was obtained rapidly (significant post-processing is necessary to render a digital model). The use of drone technology clearly presents advantages in illustrating dangerous areas surrounding the structure and documenting the limits of damage for cost estimation purposes. The image provided by the University was based on the use of high-definition cameras, and a LiDAR 3D scanner mounted on a drone platform.

Not only is the creation of a virtual model of a structure following a disaster critical for cost estimating purposes, but it is also important for the preservation of evidence. Often, a failure results in large debris fields that are challenging to reconstruct and document. The position, orientation, and condition of each piece of debris can be critical to understanding the failure scenario. Each piece of debris must be thoroughly documented before it is moved to preserve the evidentiary value of the components. Drones are uniquely suited to mapping the debris field with a high level of accuracy from an aerial perspective, without disturbing the scene, and in a rapid fashion. Additionally, the aerial perspective provided by drones allows for a rapid assessment of the debris so that any evidence is not overlooked and the limits of the scene can be secured. Subsequent flights are made as the investigation moves forward and debris is moved to access critical failure locations. Refinery losses and structural failures with large debris fields, such as those illustrated in *Figures 3* and *4*, illustrate the unique applicability of drones in these complex and scattered debris field failure scenarios.

Finally, one further potential usage of drones is related to the regular façade inspections that modern cities require. These inspections identify loose and damaged materials, most often in masonry claddings. Loose cladding material may fall on pedestrians and represent a growing hazard in the as-built urban environment. For example, on April 13, 2015, a section of façade collapsed in downtown Cleveland resulting in property damage. Currently, these types of inspections are performed by engineers and technicians rappelling down the face of the building. Drones, while facing limitations discussed previously relative to wind and hail inspections, could provide a faster, more economical initial inspection with less intrusion, and therefore represent an improvement on the current practice.

Additional opportunities to use drones include pier and wharf inspections, property condition assessments, environmental site evaluations, vehicle accident reconstruction, and pre-loss documentation. The sky is the limit when it comes to applications for drones in the forensic engineering industry.▪

Figure 4. Localized debris field following wind turbine over-speed, due to mechanical failure within the nacelle. Remnants and evidence were found 650 feet (198 meters) from the tower base.

Concrete Trends

Not Your Father's Concrete

By Jeremy Chilton, P.E., S.E., LEED AP

Concrete sculpture: Stealth-Atlanta.

Concrete is the most widely used material in the world. With U.S. cement consumption at 3.4M metric tons through May and 9.4% growth over the same period in 2015, it is safe to say that concrete plays a significant role in nearly all types of construction projects (PCA July Monitor, 2016).

With such widespread use, it is easy to recognize that concrete is also a very versatile material. A wide-ranging rheology, the ability to develop strengths in excess of 10,000 psi and withstand harsh climates and corrosion make it clear why concrete is used in everything from our national monuments to art. A key point to remember is that concrete is a composite material; it is only as good as its weakest component. Additionally, concrete, as good as it seemingly is, still has some inherent flaws. The development of many admixture technologies was founded on the fact that the quality of materials and their respective properties vary widely and to improve upon some of the known gaps in concrete as a composite material. Concrete chemical admixtures help account for this variability by altering or improving various aspects of the mix and ultimately bring out the full potential of concrete as we know it today. However, many engineers know very little about concrete mix design and available products on the market that can unlock the full potential of concrete as a building material.

As engineers and specifiers, it is in the best interest of engineers and specifiers to know and understand the concrete admixture technologies available, how they can improve various concrete properties and the respective impacts they can yield on projects such as labor/material cost savings and improved construction schedules. ACI 212.3R-16 is the newly released Report on Chemical Admixtures for Concrete by the American Concrete Institute. This document provides a comprehensive review of all current admixture technologies, respective material properties, applications, the effect on plastic and hardened concrete, quality assurance and concrete production.

Among the leading trends is concrete durability enhancement. As the cost of structures increases so does the need to keep these structures in service for as long as possible. This effectively reduces the life-cycle costs and aids in the investment decision from a time-value of money perspective. The design community answered this trend with many prescriptive code based requirements all with durability in mind. Several admixture technologies at the forefront of this trend are:

- Shrinkage Reducing/Compensating Admixtures,
- Permeability Reducing Admixtures

Limiting and preventing cracks protects the high alkaline environment that the concrete inherently creates. This, in turn, protects the reinforcement from corrosion. This is one of the most important attributes of concrete durability and one of the hardest to achieve consistently, but that is all about to change. A new shrinkage reducing/compensating admixture will be coming to market soon that has the potential to generate a shrinkage neutral concrete, effectively, non-shrink concrete.

Permeability reducing admixtures (PRA) address another known characteristic of concrete, its porosity. PRA can be used to effectively reduce concrete's ability to absorb water either through capillary absorption or through application under hydrostatic pressure. Moisture is a leading contributor to corrosion within the concrete. Keeping moisture away from the reinforced zones within concrete can improve overall concrete durability.

Lincoln Memorial reflecting pool-Washington D.C.

Structure geometry is becoming more complex and more congested with reinforcement as design forces increase and the design envelope is pushed. Self-consolidating concrete (SCC) is an underutilized resource available to engineers to better ensure concrete placement quality and reduce structure cost. Other technologies, like hardening accelerators, help concrete achieve near design strength in 24 hours. Time-saving features allow forms to be pulled more quickly and improve construction sequencing, which can represent major cost savings to contractors and owners.

Most importantly, concrete should not be considered an unchanging material within standard specifications that work, without modification, for everything. Each project, each location represents a new set of variables to consider. So review your specifications while considering your project objectives. Utilize prescriptive specifications as required by design codes to establish certain minimums/maximums. Outside of what is prescribed by design codes, keep specifications more performance based. Performance-based specifications allow the ready mix producer flexibility to optimize mixes while still meeting design and durability requirements. In the end, there is a unique and optimized solution that exists, with the right combination of chemical admixtures.▪

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The Logic of Ingenuity

Part 3: Engineering Reasoning

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

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

The thesis of this series is that *engineering* reasoning is a practical implementation of what Charles Sanders Peirce described as *diagrammatic* reasoning. Most people associate the word "diagram" with a picture of some sort, but he viewed it primarily as "a concrete, but possibly changing, mental image of such a thing as it represents. A drawing or model may be employed to aid the imagination; but the essential thing to be performed is the act of imagining" (NEM 4.219n1; 1906). Here is his technical definition:

> *A* diagram *is a representamen which is predominantly an icon of relations and is aided to be so by conventions. Indices are also more or less used. It should be carried out upon a perfectly consistent system of representation, founded upon a simple and easily intelligible basic idea (CP 4.418; 1903).*

Peirce's terminology here may require some explanation. A *representamen* (pronounced "rep-re-sen-TAY-men") is what he alternatively called a *sign*: "something which stands to somebody for something in some respect or capacity" (CP 2.228; 1897). "All thought being performed by means of signs, logic may be regarded as the science of the general laws of signs" (CP 1.191, EP 2.260; 1903) – i.e., *semeiotic* – which classifies them by, among other things, how they represent their objects: *icons* (e.g., statues) do so "only in so far as they resemble them in themselves"; *indices* (e.g., weathervanes) do so "only by virtue of real connections with them"; and *symbols* (e.g., sentences) do so "because dispositions or factitious habits of their interpreters [i.e., conventions] insure their being so understood" (EP 2.461; 1911).

Peirce further subdivided icons into *images*, "which partake the simple qualities"; *diagrams*, "which represent the relations … of the parts of one thing by analogous relations in their own parts"; and *metaphors*, "which represent the representative character of a representamen by representing a parallelism in something else" (CP 2.277, EP 2.274; 1903). Because diagrams embody formal *relations*, they need not always do so visually; although geometric figures are obvious examples, algebraic expressions also qualify. A free-body sketch and the associated equations of static equilibrium *both* reflect the relations among the forces that are acting upon and within a structural element.

Indices in a diagram point to its *reference*, the actual relations that it represents. Conventions help convey its *signification*, the new information that emerges from manipulation of it in a manner that complies with the explicit or implicit rules of "a particular system of symbols – a perfectly regular and very limited kind of language" (CP 2.599; 1902) – such as a collection of postulates and axioms, or a stipulated notation. The principles of mechanics serve this function for *deriving* the proper equilibrium equations from a free-body sketch – which typically includes depictions such as lines for members, vector arrows for forces, and triangles for supports – and *solving* them subsequently reveals what are designated as the reactions, shears, and moments due to an applied load.

This is what makes diagrammatic (and engineering) reasoning so powerful. Although it constitutes deductive inference – there is nothing in the conclusion that was not already embedded somehow in the premises – it still brings to light something that was not initially evident:

> *… deduction consists in constructing an icon or diagram the relations of whose parts shall present a complete analogy with those of the parts of the object of reasoning, of experimenting upon this image in the imagination, and of observing the result so as to discover unnoticed and hidden relations among the parts (CP 3.363, EP 1.227; 1885).*

It is important to keep in mind that the diagram itself and the representational system that governs it are each *provisional*. They inevitably include abstractions and idealizations, which are *selected* by the person who engages in this type of reasoning – which thus involves *creativity*, because it is active, not purely passive: "Thinking in general terms is not enough. It is necessary that something should be DONE. In geometry, subsidiary lines are drawn. In algebra permissible transformations are made" (CP 4.233; 1902).

Note again that not just *any* modifications are allowed; rather than being completely arbitrary, they must conform to the precepts of the chosen representational system, which also then dictate their outcomes. As Peirce wrote elsewhere: "… all reasonings turn upon the idea that if one exerts certain kinds of volition, one will undergo, in return, certain compulsory perceptions … certain lines of conduct will entail certain kinds of inevitable experiences" (CP 5.9; 1905). Nature corroborates or falsifies a theory through such encounters in the actual world, but how does a *hypothetical* one possess a similarly normative aspect?

> *Now, sometimes in one way, sometimes in another… certain modes of transformation of Diagrams… have become recognized as permissible. Very likely the recognition descends from some former Induction, remarkably strong owing to the cheapness of mere mental experimentation. Some circumstance connected with the purpose which first prompted the construction of the diagram contributes to the determination of the permissible transformation that actually gets performed (NEM 4.318; 1906).*

In other words, which moves are legitimate becomes apparent mainly through the persistent activity of the intellect – which is far less costly or time-consuming than a genuinely inductive investigation, because "it does not deal with a course of experience, but with whether or not a certain state of things can be imagined" (CP 2.778; 1902). How one proceeds in an individual case, subject to such constraints, depends on one's *intention*; the entire train of thought – i.e., the sequence of signs – has to incorporate the features that are relevant to achieving that end, while other considerations are largely ignored. This exercise of judgment guides the configuration of not only the diagram itself but also the representational system.

For modeling the behavior of something material, an acceptable degree of approximation is more likely when these have been developed, tested, and refined through rigorous inquiry. This is what ultimately enables the simulation of *contingent* events with *necessary* reasoning: "Such operations upon diagrams, whether external or imaginary, take the place of the experiments upon real things that one performs in chemical and physical research" (CP 4.530, 1905). There is no such thing as a frictionless pin, but engineering *science* has demonstrated that treating standard shear connections at the supports of a steel beam *as if* they provide no rotational restraint whatsoever facilitates a valid assessment of the member's strength and serviceability.

The bottom line is that diagrams and representational systems are *artifacts* that people *design*, so it should not be surprising that engineers routinely employ them. As summarized by Michael H. G. Hoffmann, a philosophy professor at Georgia Tech, in a working paper entitled "Seeing Problems, Seeing Solutions: Abduction and Diagrammatic Reasoning in a Theory of Scientific Discovery" (**https://smartech.gatech.edu/bitstream/handle/1853/24031/wp15.pdf**, emphasis in original): "… *seeing a solution* presupposes *seeing a problem* … The central idea of this kind of reasoning is that we *see* problems when we try to represent what we know about something … We have to *represent* what we know – or think to know – in order to *see*, first, its limitations and, second, new possibilities."

This hints at broader applications, even outside the realm of engineering, which will be the subject of my concluding article.▪

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The online version of this article contains detailed references. Part 2 of this series appeared in the October 2016 issue. Visit **www.STRUCTUREmag.org**.

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Product: S-CONCRETE
Description: For designing and detailing reinforced-concrete columns, beams and walls. Optimize a single section or evaluate thousands of concrete sections at once. No Black-Box solution: generates comprehensive reports that include clause references, equations employed, intermediate results and diagrams. Release 11.3.7 includes Eurocode 2 updates, ADAPT Builder Wall Design integration.

Product: S-FOUNDATION 2017
Description: Design, analyze and detail foundations with the most customizable and automated foundation management solution available. Use as standalone application or integrated within S-FRAME Analysis. Easily import support data from any 3rd party analysis program. Automatically generates and manages the underlying foundation model while optimizing.

Simpson Strong-Tie®

Phone: 800-925-5099
Email: **web@strongtie.com**
Web: **www.strongtie.com**
Product: Anchor Designer™ Software for ACI 318, ETAG and CSA
Description: The latest anchorage design tool for structural engineers to satisfy the strength design provisions of multiple design methodologies. Quickly and accurately analyze an existing design or suggest anchorage solutions based upon user-defined design elements in cracked and uncracked concrete conditions.

Product: CFS Designer™ Software
Description: Gives designers the ability to design cold-formed steel beam-column members according to AISI specifications, and to analyze and design complex span and loading configurations, including system design for framed openings, shearwalls, x-braces, floor joists, and roof rafters.

Standards Design Group, Inc.

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

StructurePoint

Phone: 847-966-4357
Email: **info@StructurePoint.org**
Web: **www.structurepoint.org**
Product: spColumn and spMats
Description: spColumn is used for design of shear walls, bridge piers as well as typical framing elements in buildings and structures. spMats is used for analysis, design and investigation of commercial building foundations and industrial mats and slabs on grade.

Product: spSlab and SpWall
Description: spSlab is used for analysis, design and investigation of reinforced concrete floor systems. spWall is used for design and analysis of cast-in-place reinforced concrete walls, deep beams, coupling beams, tilt-up walls, ICF walls, and precast architectural and load-bearing panels.

StruMIS LLC

Phone: 610-280-9840
Email: **sales@strumis.com**
Web: **www.strumis.com**
Product: StruMIS Steel Fabrication Software
Description: A complete management information and production system for every steel fabrication company; minimize overheads and costs, maximize productivity and profitability; in every step.

Trimble

Phone: 770-426-5105
Email: **kristine.plemmons@Trimble.com**
Web: **www.tekla.com**
Product: Tedds
Description: Perform 2D frame analysis, access a large range of automated structural and civil calculations to U.S. codes, and speed up daily structural calculations.

Product: Tekla Structural Designer
Description: Fully automated and packed with many unique features for optimized concrete and steel design. Helps engineering businesses win more work and maximize profits. From the quick comparison of alternative design schemes to cost-effective change management and seamless BIM collaboration, Tekla Structural Designer can transform your business.

Product: Tekla Structures
Description: Create and transfer constructible models throughout the design lifecycle. From concept to completion. Allows you to create accurate and information-rich models that reduce RFIs and enable structural engineers proven additional services. Models are used for drawing production, material take offs and collaboration with disciplines like architects, consultants, fabricators and contractors.

Veit Christoph GmbH

Phone: +49 711 518573-30
Email: **melanie.engel@vcmaster.com**
Web: **www.vcmaster.com**
Product: VCmaster
Description: Comprehensive software application for digital technical documentation in the field of structural engineering. The dynamically calculating and reusable documents offer an excellent opportunity to increase efficiency for structural analysis.

WoodWorks® Software

Phone: 800-844-1275
Email: **sales@woodworks-software.com**
Web: **www.woodworks-software.com**
Product: WoodWorks Software
Description: Version 11 now available – NDS 2015, IBC 2015, SDPWS 2015 & ASCE 7-10 compliant. SHEARWALLS: designs perforated and segmented shearwalls; generates loads; rigid and flexible diaphragm distribution methods. SIZER: designs beams, columns, studs, joists; up to 6 stories. CONNECTIONS: Wood-to-wood, -steel, -concrete.

All Resource Guide forms for 2017 are now available on the website, **www.STRUCTUREmag.org**.

2040 Market Street

Vacant Building Transformed Thanks to Prefab Construction

By Jason A. Squitiere, P.E.

The Harman Group, Inc. was an Outstanding Award Winner for its 2040 Market Street project in the 2015 NCSEA Annual Excellence in Structural Engineering Awards Program in the Category – Forensic/Renovation/Retrofit/Rehabilitation Structure over $20M.

Originally built in the 1960s, 2040 Market Street was developed as a bustling office building in the heart of Center City, Philadelphia. By the early 2000s, however, the former headquarters of the America Automobile Association (AAA) was felled by the Great Recession, eventually sitting vacant after the travel company relocated in 2005. The once coveted piece of real estate sat empty for several years, a symbol of the downtrodden economy. In 2011, PMC Property Group of Philadelphia acquired and transformed the existing five-story concrete-framed building into luxurious mixed-use residential apartments by expanding both vertically and horizontally.

Structural engineering firm The Harman Group (THG) was tasked with determining how many additional floors could be added to the existing structure. Using a system of load-bearing steel wall panels and an Ecospan floor system, THG added eight residential floors vertically and expanded the building's horizontal footprint by 68,000 square feet, reaching the same top floor elevation as the overbuild. The vertical and horizontal expansion revamped a 120,000 square- foot vacant office building into more than 300,000 square feet of residential units and ground-level retail space. In total, the new addition used 256 tons of structural steel.

Integrity Max is a simple and effective solution to wall panels. The 11-foot 2½-inch tall panel is constructed with 4-x 4-inch cold-formed tubes supplemented with 4-inch hollow structural section (HSS) members, where required. The interior tube can carry greater loads, without any horizontal bridging, making an open vertical space between studs for mechanical and other systems. These panels also helped minimize the weight of the overbuild portions, while reducing the erection time as well.

Thanks to its lightweight construction, the Ecospan floor system from Nucor-Vulcraft helped maximize the number of floors that could be added to 2040 Market. Ecospan is a composite floor system comprised of a 3½-inch slab on metal deck supported by 12-inch deep open web steel bar joists. The light-weight construction of this system allowed for three more overbuild levels than a traditional steel-framed system would have allowed. This expansion was critical for the Owner, as it enabled them to develop the maximum number of leasable apartments. The floor joists also have a special flush joist seat which allows for uninterrupted bearing through the floor slab from level to level, a critical factor for load transfer.

The existing building is concrete waffle-slab construction with columns spaced 27 feet apart. To minimize cost and depth for the transfer structure between the concrete building and the new steel-framed floors of the overbuild, Integrity Wall prefabricated structural steel in-wall trusses designed to transfer the loads from above. With the combined structure serving as an architectural wall and providing load support, the transfer level structure was reduced to only 10 to 15 pounds per square foot, compared to the 25 pounds per square foot offered by a conventional stick-built structural steel transfer floor. When construction was complete, the transfer trusses "disappeared" within the demising walls.

Securing the overbuild to the existing structure posed some unique challenges. Traditional baseplate and anchor rod tie-down connections to the existing roof slab were not robust enough to resist the high net uplift forces at the interface with the overbuild. Tie-down connections to the existing roof slab proved inadequate. The team decided to bypass the existing roof slab to combat this issue. 1.5-inch diameter anchor rods were installed through holes drilled in the existing roof slab and welded to steel plates with post-installed anchors on all faces of the existing concrete column.

The location of the 13-story horizontal expansion needed to be on top of the single-story basement at the back of the existing building. The existing concrete columns were adequate for the overbuild, but the existing foundation had to be reinforced. Working together with the contractor, THG developed an innovative but simple solution to maximize the height of the horizontal expansion by shoring the existing basement, removing the existing footings and replacing them with larger footings.

Integrity Wall and South Shore Iron Works prefabricated Viernedeel trusses for 2040 Market to act as in-floor girders where the layout required large web openings. The 50 percent open structure provided the ability to pass utilities through congested areas easily. Vierendeel trusses with HSS top and bottom chords and vertical tubes were also used for the exterior. The trusses were installed at each floor below the glazing and erected concurrently with the building to support a Trespa façade.

Today's construction industry is driven by two elements: time and money. Projects need to be built on a tight schedule and delivered within budget. Prefabricated elements are an effective way to meet these goals without compromising quality. The overbuild of 2040 Market used every imaginable way of prefabricating the steel structure to enable a lightweight overbuild structure that could be constructed quickly at an efficient cost. The result was the transformation of an abandoned eyesore into a glittering new mixed-use residential building in the heart of Philadelphia.▪

*Jason A. Squitiere is a Project Manager at The Harman Group. Jason is a member of the American Institute of Steel Construction (AISC), the Delaware Valley Association of Structural Engineers (DVASE), and the Structural Engineers Association of Pennsylvania. He can be reached at **jsquitiere@harmangroup.com**.*

YMGSC Celebrates Five Year Anniversary

The first committee members gathered five years ago to form the NCSEA Young Member Group Support Committee (YMGSC) to respond to the growing gap between engineering education at the university and professional practice. As engineers contend with rapidly changing codes, technological advances, and innovations in materials, communication, design, and construction methods, the need for a group to address both technical and professional competency in young engineers was evident. The group also recognized that young engineers within SEA member organizations (MOs) are the future of NCSEA and the structural engineering profession as a whole. From this beginning, a mission statement was developed that continues to be the center of the committee's focus and efforts:

> *The YMGSC facilitates the formation, growth, and success of NCSEA Member Organization Young Member Groups (YMGs) through collaboration, support, and outreach in an effort to transition students and young engineers into successful, professional engineers and future leaders of the Structural Engineering Profession.*

Since the group's formation, this mission has expanded into multiple objectives and resources that support the growth of YMGs and young engineers.

In the five years since the committee was established, the number of active YMGs has increased from 15 to 24, with 8 more in their first year or under formation. The number of young member attendees at the NCSEA Summit has increased every year. Eight young members were awarded scholarships to the 2016 Summit, and, for the second year, Young Member Chapters of the Year were honored.

Representatives from the Young Member Chapters of the Year finalists Arizona, Oklahoma, Illinois and Minnesota. Idaho is not pictured. The SEAOI (Illinois) group was named Young Member Chapter of the Year.

Upcoming This Year

Each year, the Summit marks a period of transition in leadership within the committee. The successes of the YMGSC and young member groups around the country are celebrated. The YMGSC then refocuses to establish the objectives and goals for the coming year. This year, the YMGSC aims to expand on ALL of the resources, programs, events, and opportunities that have been made available in the past while also focusing on:

- Enhanced communication between young member groups across the country;
- More support for new and recently formed young member groups;
- Regular webinars specifically designed for YMG development and young engineer education;
- Additional opportunities for young engineers to get involved with the NCSEA YMGSC;
- Special highlights of YMG accomplishments throughout the year.

Why Start and Support a Young Member Group?

Young engineers within a member organization or young member group are the future of NCSEA and the profession. A YMG is intended to complement the SEA MO by providing a blended community for young structural engineers in the first levels of their career development as they transition into the MO. Supporting the growth of YMGs will facilitate the transition of leadership and ensure those future leaders are prepared and well acquainted with the professional society.

As an employer or business leader, supporting and encouraging your young engineers to be involved with their young member group ensures they are continually educated on evolutions in structural engineering, construction, and business methods that may not be learned in school or on the job. This knowledge will make your business more competitive and your engineers more competent.

For more information or to get involved, please visit the YMGSC webpage at **www.ncsea.com**, or contact committee members, Jera Schlotthauer (NCSEA YMGSC Past-Chair), **jschlotthauer@mmwyo.com** or Seth Thomas (NCSEA YMGSC Chair), **seth.thomas@kpff.com**.

2016 Structural Engineering Summit Shines in Orlando

Retiring NCSEA Executive Director Jeanne Vogelzang, center, with Bill Bast, left, and Sarv Nayaar.

Hilti was one of the many exhibitors at the Summit trade show.

The NCSEA Code Advisory Subcommittee Special Inspections/Quality Assurance Code hard at work.

Ashraf Habibullah and Computers & Structures Inc. (CSI) sponsored Wednesday's festive event at the Orlando Museum of Art.

Service Award Recipient Carrie Johnson, center, with, from left, Ken Basden, Jim Malley, Joe Shepard, and Tom DiBlasi.

NCSEA Delegates share information at the Delegate Collaboration Session.

The Delegate Breakfast featured some fun with a shake table and Legos.

Mark your Calendars! 2017 Structural Engineering Summit:
October 11–14, Washington Hilton, Washington, DC

Thank You to the 2016 Summit Trade Show Exhibitors:

AISC
Alpine TrusSteel
Amer. Concrete Inst.
Armatherm
Atlas Tube
AZZ Galvanizing
BASF Corporation
Blind Bolt
Cast Connex
CLP Systems
DeWalt/Powers
Dlubal Software
Euclid Chemical
Fabreeka Intl.
Geopier Foundation
Hayward Baker
Headed Reinforcement
Hilti
Hubbell Power
ICC-ES
ITW Red Head
LafargeHolcim
Lindapter
Meadow Burke
Menard USA
MiTek USA
New Millennium
Nucor
Peikko USA
RISA Technologies
SCIA/Nemetschek
SidePlate Systems
Simpson Strong-Tie
Steel Deck Institute
Steel Joist Institute
Steel Tube Institute
Strand7
SECB
Trimble Solutions
USG
Vector Corrosion

Thank You to the Structural Engineering Summit Sponsors:

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Registration Now Open

The Premiere Event for Structural Engineering

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

STRUCTURES CONGRESS 2017
Denver, Colorado | April 6-8

Register as early as possible to take advantage of early-bird rates and book your accommodations through our room block to maximize savings and enhance your networking experience.

Convention Hotel: **Hyatt Regency Denver**
650 15th Street
Denver, CO 80202

Top Reasons to Attend

- Network with researchers, designers, project/construction managers, and contractors from around the world to discuss the current and future challenges for structures
- Gain knowledge by attending outstanding technical sessions – over 120 from which to choose
- Visit a broad range of exhibitors in one location and find the latest tools to help your organization
- Earn Professional Development Hours (PDHs) in technical sessions to maintain your professional licensure
- Attend the opening and closing plenary sessions to hear compelling presentations by innovative top leaders in the field
- Interface with students and young professionals
- Enjoy learning & earning PDHs from the Council of American Structural Engineers (CASE) at their Spring Risk Management Convocation

Visit the congress website at **www.structurescongress.org** for more information and to register.

SEI Opportunities for Students

Student Competitions at Structures Congress
April 6 – 8, 2017 in Denver – Showcase your talent and compete for prizes including conference registration. Submission deadlines in December

SEI Student Career Networking Event
April 7 in Denver – Apply by March 15

Affiliate/Establish a SEI Grad Student Chapter
SEI Graduate Student Chapters broaden students' horizons as structural engineering professionals, and prepare them for a successful transition from college to career. Benefits for SEI Grad Student Chapters include affiliation with SEI and logo branding, one free group webinar per year, best practices networking with members and other local chairs on quarterly conference calls, funding for Chair to attend the annual fall SEI Local Leaders Conference, and receive outreach materials and more.

For information on these programs and more, see the SEI website at **www.asce.org/structural-engineering/sei-students**.

Education and Careers

Manage your career with ASCE's help. Take charge of your training to achieve the body of knowledge needed, obtain the right experience, and obtain your PE license. New guidelines on experience that will help you advance your career are available on the ASCE website at **www.asce.org/licensure**.

SEI Young Professional Scholarship

Apply by December 1, 2016

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

Advance to SEI Fellow

Apply by December 1, 2016

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

ASCE 7 Supplement 1 Public Comment Period Now Open

ASCE is conducting a public comment period on the Supplement 1 provisions to ASCE/SEI 7-16 *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* on Chapters 4, 5, 7, 11, 12, 13, 14,15, 21, 23, 26, 29, and 30. The sections eligible for public comment are highlighted. Commentaries will be available for information only. The public comment runs from September 23, 2016, through November 7, 2016. Accessing the Public Comment System requires using or creating an ASCE web user account, **https://secure.asce.org/ASCEWebSite/SECURE/SignIn/SignIn.aspx?ASCEUrl=~/StandardsBalloting/BallotInfo.aspx**. For additional questions contact James Neckel, ASCE's Codes and Standards Coordinator (**jneckel@asce.org**, or 703-295-6176).

2016 O.H. Ammann Fellowship Winners

Stamatina Chasioti

Amal Elawady

Maha Kenawy

Parisa Khodabakhshi

Seyedsina Yousefianmoghadam

SEI is proud to announce the winners of the 2016 O. H. Ammann Fellowships in Structural Engineering:

- Stamatina Chasioti – University of Toronto
- Amal Elawady – The University of Western Ontario
- Maha Kenawy – University of California, Davis
- Parisa Khodabakhshi – Texas A&M University
- Seyedsina Yousefianmoghadam – University at Buffalo

The Ammann Fellowship is awarded annually to a member or members of ASCE or SEI for the purpose of encouraging the creation of new knowledge in the field of structural design and construction. Learn more about the winners on the SEI website **www.asce.org/all/news/20160921-2016-oh-ammann-fellowship-winners.**

SEI Local Activities

GEORGIA TECH GRADUATE STUDENT CHAPTER

The SEI Georgia Tech Graduate Student Chapter hosted a seminar presentation by former SEI Georgia Chapter President Richard Morales, P.E., F.ASCE. Mr. Morales talked about the complexities, challenges, and innovative onsite solutions using cellular structure cofferdams during the expansion of the Panama Canal. The seminar was well attended by 40-50 students who engaged in an active interaction with the presenter. After the presentation, Mr. Morales had an informal discussion on how the SEI Georgia Chapter could address the needs of the graduate student chapter.

GET INVOLVED IN LOCAL SEI ACTIVITIES

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

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

SEI Welcomes New Sustaining Organization Members

Michael Baker International and Alfred Benesch & Company

Michael Baker International and Alfred Benesch & Company are SEI's newest Sustaining Organization Member. We hope you will join them, Geopier Foundations, Inc., Hayward Baker, International Code Council, MiTek, Schnabel Foundation company, and Simpson Strong-Tie in support of SEI. Being a Sustaining Organization Member will raise recognition for your organization with decision makers in the structural engineering community year-round, and show your leadership and support for SEI in their goal to advance and serve the structural engineering profession. Demonstrate your commitment and increase your organization's visibility with more than 30,000 SEI members and at SEI conferences through **www.asce.org/SEI**, the monthly SEI Update e-newsletter, and STRUCTURE magazine. Learn more at **www.asce.org/SEI-Sustaining-Org-Membership**.

Questions? Contact Suzanne Fisher **sfisher@asce.org**.

International Cold-Formed Steel Building Student Design Competition

The SEI Cold-Formed Members Committee is sponsoring the CFSEI International Cold-Formed Steel Building Student Design Competition. The goal of this competition is to push the creative bounds of structural design with light-steel framed buildings. SEI Fellow Cristopher D. Moen is chairman of this year's program and encourages undergraduate and graduate students to enter as teams or individuals. Each team can request an engineer mentor and submissions are due March 3, 2017. For more information, or to enter, visit the CFSEI website at **https://cfsei.memberclicks.net/student-competition**.

Errata

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

CASE Practice Guidelines Currently Available

CASE 962-E — Self-Study Guide for the Performance of Site Visits During Construction (Updated 2016)

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

CASE 962-F — A Guideline Addressing the Bidding and Construction Administration Phases for the Structural Engineer

This document, *A Guideline Addressing the Bidding and Construction Administration Phases for the Structural Engineer*, has been developed to assist all the parties associated with the bidding and construction administration phases of a project with the primary emphasis on those issues related to the structural engineer (SER). It is important that the design team remains proactive in communicating with the contractor and the owner after the construction documents have been issued. This communication during the construction phase, as well as during the pricing and bidding process, should have as its primary goal assistance, interpretation, and documentation for the improvement of the constructed project.

This is a guide to the SER's roles after the construction documents have been issued for construction. It provides guidance on pre-bid and pre-construction activities through to the completion of the project. The appendices contain tools and forms to assist the SER in applying this guide to their practice. This guideline includes suggested approaches to the various components that can make up the bidding and construction administration phases.

CASE 962-G — Guidelines for Performing Project Specific Peer Reviews on Structural Projects

Increasing complexity of structural design and code requirements, compressed schedules, and financial pressures are among many factors that have prompted the greater frequency of peer review of structural engineering projects. The peer review of a project by a qualified third party is intended to result in an improved project with less risk to all parties involved, including the engineer, owner, and contractor.

Many aspects of the peer review process are important to establish before the start of the review, to ensure that the desired outcome is achieved. These items include the specific goals, scope and effort, the required documentation, the qualifications and independence of the peer reviewer, the process for the resolution of differences, the schedule, and the fee. The intention of these guidelines is to increase awareness of such issues, assist in establishing a framework for the review, and improve the process for all interested parties.

CASE 962-H — National Practice Guideline on Project and Business Risk Management

This guideline is intended to assist structural engineering companies in the management of risk associated with projects and to provide commentary regarding the management of risk associated with business practices. The guideline is organized in two sections that correspond with these two areas of risk, namely Project Risk Management and Business Practices Risk Management. The goal of the guideline is to educate and inform structural engineers about risk issues so that the risks they face in their practices can be effectively mitigated, thus making structural engineering firms more successful.

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

CASE Risk Management Convocation in Denver, CO

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

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

8:00 AM – 9:30 AM	*Contractual Risk Transfers for Professionals: Mastering Indemnity, Insurance and the Standard of Care* Moderator/Speaker: Ryan J. Kohler, Collins, Collins, Muir + Stewart, LLP
10:00 AM – 11:30 AM	*Construction Administration as a Risk Management Tool* Moderator / Speaker: Daniel T. Buelow, Willis Towers Watson
2:00 PM – 3:30 PM	*Projects with the Largest Losses and Claim Frequency* Moderator: Mr. Timothy J. Corbett, SmartRisk Speaker: Brian Stewart, Esq., Collins, Collins, Muir + Stewart, LLP
4:00 PM – 5:30 PM	*Tackling Today's Business Practice Challenges – A Structural Engineering Roundtable* Moderator: David W. Mykins, P.E., Stroud Pence & Associates

Follow ACEC Coalitions on Twitter – @ACECCoalitions.

CASE Winter Planning Meeting – SAVE THE DATE

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

2017 Small Firm Council Winter Seminar: Defining HR for Your Firm

February 17 – 18, 2017; San Diego, CA

Why is human resources management (HRM) important to your small A/E firm?

At its heart, it is all about managing people, your most vital asset. A strong HRM focus helps you find and retain new talent, helping them perform better and stay motivated so you can focus on profitable growth and a strong bottom line.

Presented by Barbara Irwin, Principal and Founder of HR Advisors Groups, this 1½ -day seminar will focus on how firms can create programs, processes, and procedures that meet the needs of the workforce while continuing to focus on the bottom line.

This seminar is for any employee in a small firm tasked with making human resources decisions, such as owners, principals, HR professionals, CEOs, CFOs.

Registration

ACEC Coalition Members – $399
ACEC Members – $499
Non-members – $599

Location

DoubleTree by Hilton Hotel San Diego – Mission Valley
7450 Hazard Center Drive
San Diego, California, 92108
Phone: 619-297-5466
Special Rate – $139/night until January 15, 2017

To register for the seminar:
www.acec.org/calendar/calendar-seminar/2017-small-firm-council-winter-seminar-defining-hr-for-your-firm.

Questions? Call 202-682-4377 or email at **htalbert@acec.org**.

Learning from Disasters

By Jessica Mandrick, P.E., S.E., LEED AP

Natural disasters devastate communities, destroy structures, halt livelihoods, and take lives. With each event, engineers aim to improve our practices to lessen the impact of future incidents. Reconnaissance trips following natural or manmade disasters can provide a valuable education. As a young engineer, I have had the opportunity to work in three areas following natural disasters, exposing me to collaboration among disciplines, foreign codes and practices, new research, damage to structures at full scale, and the consequences of our designs.

As an undergraduate in 2005, I met researchers from Louisiana State University. These scientists presented on the erosion of coastal Louisiana due in part to the extensive levees historically placed along the length of the Mississippi River in response to river flooding. The levees reduce the amount of sediment entering the Mississippi River and channel what sediment is in the river off of the continental shelf into deep water, rather than onto the delta where it could build land. The loss of this land is the loss of a significant storm buffer between New Orleans and the Gulf of Mexico. The researchers stressed an urgent warning that just months later became a reality with Hurricane Katrina.

Nine months after the event, I returned to work with this same group of scientists. Houses and neighborhoods still lay abandoned, while the team worked on modeling strategies for river diversions (opening up areas of levees) to build land in the Mississippi River Delta. More than ten years later, the conversation about abandoning the bird's foot delta and allowing the release of sediment is ongoing. The experience highlighted the risks of interfering with nature on a large scale and the need for the involvement of the whole community and those downstream in decision making. Measures taken to mitigate present concerns need to be properly vetted against future concerns.

The morning following the landfall of Hurricane Sandy in 2012 in New York City, I made an emergency visit to a construction site where the ensuing flood had undermined several neighboring buildings, resulting in partial collapse into the site. It was necessary to communicate where it was *not* safe to access, strategies for shoring up the site, and the importance of contacting the Department of Buildings. The structures were in an evacuation zone, so there were no occupants at the time of the storm. I spent the following two weeks involved in the surveying and tagging of buildings. Houses and decks that were insufficiently anchored shifted off their foundations. High rises experienced flooding of multiple cellar levels due to below grade seepage. Each type of failure stressed the importance of well thought out engineering designs from concept to details to construction. New York City incorporated many of the lessons learned into code provisions, adding additional requirements for flood zone special inspections, coastal construction, hospitals, utilities, and retroactive requirements.

This February, I participated in a trip to study the damage in Tainan, Taiwan due to the 2016 M6.4 Earthquake. While many of the types of damage observed have been categorized, studied, and incorporated into the language in the building code, it was the first time *I* was able to see captive columns, soft stories, poor seismic detailing, and liquefaction. The performance of numerous structures in a natural disaster can be observed and compared, and on a scale not available in laboratories or textbooks. Observing success is equally valuable to observing a failure, as we can learn what to promote in our designs. For the damaged structures, owners had temporary shoring in place, at times ineffectively, and were beginning to make repairs, often without engineering guidance. It was clear that the engineering community needs to better prepare the public for expectations of structural performance during earthquakes and reconstruction/reoccupation afterward.

The National Center for Research on Earthquake Engineering (NCREE) in Taiwan shared their research on the retrofit of schools and street houses. We discussed the societal and financial value of retrofitting structures and the data needed to communicate this value to politicians and the general public. The need for special inspection requirements in Taiwan was highlighted in some of the failures observed, such as inconsistently spaced rebar ties and embedded architectural items in columns. Often codes and practices are developed in parallel, with each country focusing on its concerns and needs. In an increasingly global society, we need to learn from our neighbors.

There are many ways to become involved after a disaster, including humanitarian and recovery efforts or the participation in committees that set performance levels in engineering design. It is important that engineers see hurricanes, floods, and earthquakes as more than just loads, and consider their societal impact. Engineers are well educated to take a seat at the table in the larger conversation on disaster preparedness, risk tolerance, and infrastructure investment. If you are a leader at a firm or university and have the opportunity to visit a disaster area or participate in a resiliency committee or conference, consider taking a junior engineer with you. It is eye opening. The experience will not only benefit those who directly participate, but it will benefit your firm as well as society in general.▪

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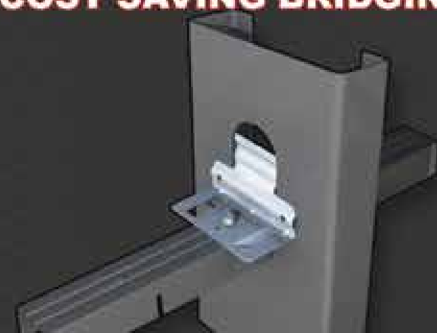

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