ADVANTAGE STEEL

NO 50 SUMMER 2014

Steel rises to complex construction challenges for new LEED certified hotel

Economical truss design: A fabricator's perspective

Expanding the options: Structural fire engineering expands design and engineering choices

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NO 50 SUMMER 2014

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By Ed Whalen, P.Eng.
President

Take the time to dream and the future will be yours

From a business perspective we have a tendency to look in the past as a measure of our business performance and all too often set our future based on this alone. If it has worked for me over the past three years, it will work for me in the coming years.

This 20/20 vision shows us only what was, not what our potential holds. Having a complete picture allows us to fully analyze, study and make recommendations for change. We need this past reflection to improve our processes on what and how we do them today. The problem with looking only in the past, however, is that it may paralyze or prevent necessary change to address future needs, risks and opportunities.

History is full of companies that were once world leaders in their field only to have been blinded by their past successes. I am sure we all remember RIM or should I now say Blackberry? The company responsible for putting mobile data in the hands of every man, woman and even child these days, only to see the world pass them by because they were stuck looking in the rearview mirror. Great financial results led to complacency and the Canadian tech darling had their lunch eaten by Apple and Samsung (Google Android).

On the other end of the spectrum is Apple. Encouraging their employees to dream big, dream outside the box and look forward, they created a culture that today has made this company one of the largest in the world. They embraced innovation, creativity, change and strategic planning.

Sadly, the Canadian construction industry is great at looking back and not so good at looking forward. The rate of innovation, change and productivity is the lowest in our industry compared to any other Canadian industry sector. With changes in products, services and globalization consultants, general contractors, manufacturers and trades are or will soon feel the pressure like they have never before. Nowadays if you are only keeping up, you're already behind.

While many steel fabricators and detailers have added high tech and automation to their businesses to make structural steel the most accurate, safe and productive construction material, the bar is constantly moving and the level of adoption is quite broad. Recessions and other factors have created a culture of contracting expertise (connection design for example) preventing or reducing mentoring and the transfer of knowledge and skill.

Have you determined what your company will look like or better yet how will it thrive in five to 10 years? Is it reasonable to assume that your company can exist as it is now? Are you taking steps now to take advantage of opportunities while mitigating threats? Do you know what these threats would be? Will you be Blackberry, Apple or Samsung?

CISC started along this strategic path five years ago. Engaging all stakeholders, strategic planning resulted in a new vision and mission and a five-year strategic plan for the organization. After five years and with much accomplished, it is time to again engage in the process of setting our future course for the organization. This fall, our regions will engage in strategic planning and we will take this output to set regional and national strategic plans.

A little time spent daydreaming is time well spent. Please come out and help us create an exciting future.

ADVANTAGE STEEL

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Professional engineers, architects, structural steel fabricators and others interested in steel construction are invited to inquire about CISC membership. Readers are encouraged to submit their interesting steel construction projects for consideration for inclusion in this publication by contacting CISC.

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By Alfred F. Wong, P.Eng.
Director of Engineering

Question 1: I am designing a building structure consisting of a simple gravity frame and a perimeter rigid frame, which serves as the lateral-force resisting system. There is no braced bay or shear wall. Several gravity columns are subjected to significant bending about the strong axis due to a large connection eccentricity. Should these wide-flange columns be designed as beam-columns of an unbraced frame? How are the values for U_{1x} determined?

Answer: No, this is a braced-frame situation. Despite the absence of braced bent, the gravity columns are 'leaners', i.e. they do not participate as primary lateral-force resisting members. The U_{1x} values can be calculated in accordance with Clause 13.8.4 of CSA S16 but the U_{1x} values for determination of cross-sectional strength and lateral-torsional buckling strength must not be less than 1.0.

Question 2: I am involved with evaluation of an existing building structure for compliance with the current code. The structure is in a sound condition. It satisfies the building code and CSA S16-09 for the intended occupancy except that the column bases have 2 anchor rods instead of 4. In one area, a row of small wide-flange columns sit on a concrete wall. The x-axis of these columns, their anchor rods and the centre line of the wall all lie in the same plane. It is impossible to install 4 anchor rods in the normal configuration to this relatively thin wall. However, I can provide 4 collinear rods per column by adding 2 more. Is this collinear pattern acceptable?

Answer: I see two parts in your question:

a) Is collinear distribution of 4 anchor rods in compliance with Clause 25.2 of S16?

The requirement for a minimum of 4 anchor rods aims to ensure erection safety. The rods should be positioned to provide an adequate lever arm against overturning in more than one direction. Clause 25.2 of S16-14 specifies 4 non-collinear rods per column, unless special precautions are taken.

b) Does Clause 25.2 apply to a structure that has been completed and in service?

No, it is an erection safety requirement.

Question 3: Are bolted moment connections used in a canopy structure required to be slip critical? My question relates to a situation where slip critical connections are not required for deflection control. I have many years of connection design experience but seldom had to provide slip-critical connections for wind-load resisting braced bents or moment frames.

Answer: The key question here is whether fatigue is a consideration; will the structure be subjected to repetitive loading and stress reversal. A relatively light canopy type of structure subjected to gusty local wind load may experience stress reversal and significant number of load cycles to warrant such assessment. The judgment rests with the engineer responsible for the design of the structure. Fatigue design is covered in Clause 26 of S16.

Question 4: I find the design requirements for pin-connected tension members as stipulated in CSA S16-09 very difficult to satisfy. They appear to mandate the use of eyebars. Have I missed something?

Answer: The requirements in CSA S16-09 aim to ensure a gross-section yielding ultimate limit state and are therefore quite restrictive. New requirements have been introduced in CSA S16-14. This new provision improves design versatility considerably.

Questions on various aspects of design and construction of steel buildings and bridges are welcome. They may be submitted via email to faq@cisc-icca.ca. CISC receives and attends to a large volume of inquiries; only a selected few are published in this column.

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By Alfred F. Wong, P.Eng.
Director of Engineering

EBF – A system with unlimited potential

The eccentrically braced frame (EBF) was introduced to me as a new concept in the late 1970s while I was a graduate student. It intrigued me and I embraced it but did not envisage its potential and versatility.

A system developed for seismic resistance

Common lateral-load-resisting systems, such as rigid frames, concentrically braced frames and shear walls, have traditionally served to resist wind effects and provide stability in general. Since the trend towards more rigorous design for seismic loads began several decades ago, these systems have been adopted to resist seismic effects at various levels of ductility, through continuing modifications and improvement. The EBF is the only generic system that was developed specifically to be an effective seismic-force-resisting system.

In concentrically braced frames, the braces typically intersect a beam or a beam and a column at a common work point, avoiding connection eccentricity. In contrast, these members in The EBF configurations intersect eccentrically and create an eccentric link at each strategically selected location (see Figure 1). These links, typically quite short, attract large shear forces when subjected to ground motion; long links incur large bending moments also. Research studies conducted since the 1970s have demonstrated that well-proportioned EBFs behave very well. The concept was initially embraced by engineers in the U.S., followed by engineers in Japan, Canada and New Zealand, etc. The first EBF project in Canada was built in the 1980s and EBF design provisions were first introduced in CSA Standard S16.1-89 (as Appendix D). To date, EBFs appear in numerous projects around the world.

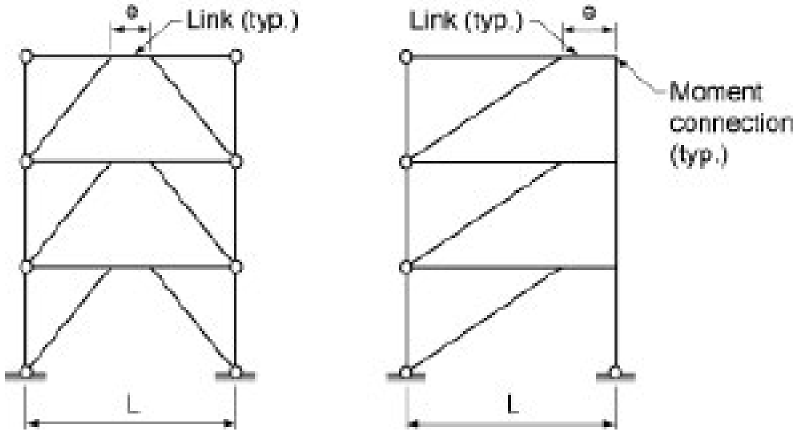
Capacity design

Professor Igor Popov initiated the first research project on EBFs in the 1970s when steel structures were designed in the U.S. using the allowable stress method. His vision guided his team to pursue not only an inelastic design solution but also a design philosophy ahead of their time – capacity design approach. In EBFs, the yielding elements, being the links, are precisely defined and contained while the rest of the system can be proportioned to remain elastic (capacity protected). The system presented itself to be a perfect example for capacity design, a term that did not exist in the steel design textbooks until many years later.

Displacement-based design

As the trend towards displacement-based design gains popularity, research papers and textbooks written for the topic are readily available. When will it find its way into the design codes? It depends on which jurisdiction

FIGURE 1
COMMON CONFIGURATIONS OF EBFS



or nation we practice in and what kind of structures we do. It is generally felt that displacement-based design for multi-storey building structures may be too complex to warrant its use in practice today. However, it is part of the performance-based design provisions proposed for CSA S6-14, Canadian Highway Bridge Design Code.

Displacement limits (for collapse prevention level), in the form of maximum link-end rotations, are already an essential design requirement for EBFs. Since the links are precisely defined and isolated while the rest of the system remains elastic, displacement-based design focuses on the links. The design procedure applies to multi-storey structures as well.

Multi-level performance-based design

As mentioned earlier, performance-based design provisions have been proposed for CSA S6-14. To meet multi-level performance, 'Other Bridges' (normal), for example, may have 'repairable damage' following a 475-year event, 'extensive damage' following a 975-year event and may need to be replaced following a 2475-year event. 'Lifeline Bridges' are permitted to suffer 'no damage', 'minimal damage' and 'repairable damage' at the 475-year, 975-year and 2475-year levels respectively and the designer must provide an explicit demonstration of compliance. Obviously, multi-level performance-based design can be significantly more onerous.

The seismic provisions in NBC 2010 focus on one level of structural design objective – collapse prevention at a 2475-year event. Future codes may include serviceability requirements for more frequent events.

Modular EBF link

Do we find an answer in EBFs for multi-level performance-based design?

Professor C. Christopoulos and his associates at the University of Toronto, in collaboration with Professor R. Tremblay at Ecole Polytechnique, are the pioneers of modular link research in Canada. In traditional EBFs, the links are part of a beam section that spans from one column to another. The modular link concept involves fabricating the link segment and the rest of the beam from separate sections then joined together using bolted connections. Yielded and damaged modular links can be conveniently replaced. Provisions for modular link beam design have been incorporated in S16-14.

The research conducted in Canada provided a timely solution for building repair and rebuilding following the 2012 quake that rocked Christchurch, New Zealand. Figure 2 shows part of an EBF with a modular link used in a recent project in Christchurch.

FIGURE 2
MODULAR-LINK EBF, CHRISTCHURCH, NZ
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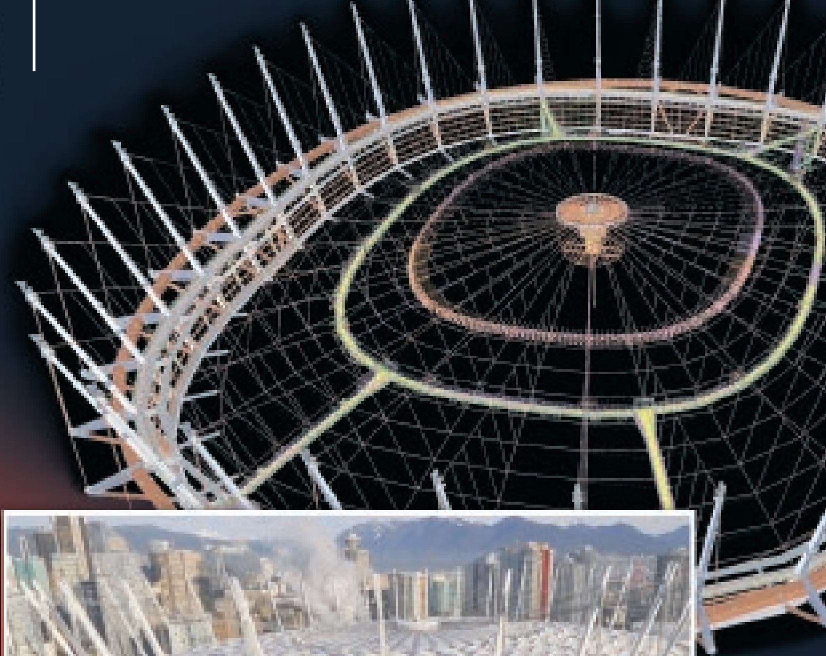

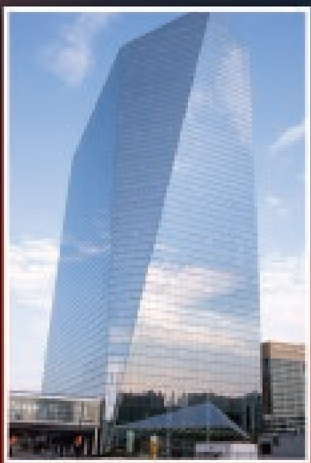

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By Tareq Ali, RPM
Director of Marketing

Resilient design and construction

Building sustainable communities for the future

With extreme weather events and natural disasters seemingly increasing in frequency and severity around the world, the global design community, building product manufacturers, governments and policy makers are all beginning to place an increased emphasis on incorporating "resilient" design practices in construction projects.

The U.S.-based Resilient Design Institute defines resilience as "the capacity to adapt to changing conditions and to maintain or regain functionality and vitality in the face of stress or disturbance." It is the capacity to bounce back after a disturbance or interruption of some sort.

While not a new concept, resiliency represents an important evolution in the "sustainability world," where the renewed focus is not just on building low carbon, energy efficient, and "healthy" green buildings and structures, but also on incorporating design, technology, systems and materials that will allow these structures to withstand and "bounce back" after a major disruption or natural disaster.

RESILIENT DESIGN CAN HELP BUILDINGS WITHSTAND THIS TYPE OF DESTRUCTION

EXTREME WEATHER EVENTS CAN CAUSE UNTOLD DAMAGE TO BUILDINGS

Resilient design is the intentional design of buildings, landscapes, communities and regions that are able to adapt to the impacts of climate change and ensuing erratic weather patterns and extreme events such as more intense storms, stronger flooding, longer and more severe droughts, melting permafrost and rising water levels, and disruption of power and other critical infrastructure systems.

There is a growing recognition that a holistic, integrated systems approach is needed in urban and community planning, design and construction that leverages all the interconnected relationships between the residents, the buildings and structures in which they live, work and play, and the major energy and infrastructure systems that power these communities.

Some of the key resilient design strategies that can be incorporated in urban design and planning include:

1. Reduce the carbon footprint: This is a proactive approach designed to reduce carbon and greenhouse gas emissions that can further stress our environment and accelerate climate change. A smaller carbon footprint can be achieved by using design and construction methodologies that promote a more energy efficient and environmentally responsive building envelope, use of sustainable building materials such as steel with its high recycled content, use of passive heating and cooling systems, and use of renewable energy sources in the operations of the building or structure.

2. Promote more diverse, mixed-use developments: Long recognized to be the more sustainable model for communities, a well-designed mixed use development has a diverse base of different business types, institutions, residential options, and sources of food and employment that allow the community to withstand and recover faster from environmental shocks and stresses such as natural calamities or other disasters.

3. Incorporate redundancies for key infrastructure systems: Communities that have built-in redundancies for key infrastructure systems such as electrical power, food and potable water supply, and fuel supply will be more resilient as they are well positioned to ensure at least partial continuity of services if one of the critical systems is compromised, and will have a quicker overall recovery from a disaster.

4. Use high-performance, ductile and durable construction materials: The increasing frequency and severity of natural disasters such as earthquakes, tsunamis and floods, and extreme weather events will drive demand for building materials and construction technologies that create more 'resilient' structures that can withstand strong winds and seismic shocks, severe storms, floods, wildfires and the like.

Architects, engineers and other structural designers should also focus on modelling design solutions based on probable future climatic conditions rather than relying on current or past data.

The extreme weather events and natural disasters that we are already experiencing and the threat of irreversible climate change have the potential to create a global environmental and humanitarian crisis on a scale that we have never seen. Resilient construction is an important proactive response to better prepare us to deal with these challenges. However, the long-term, sustainable solution can only be achieved with a commitment from governments and citizens alike to balance the "triple bottom line" of economic, social and environmental development.

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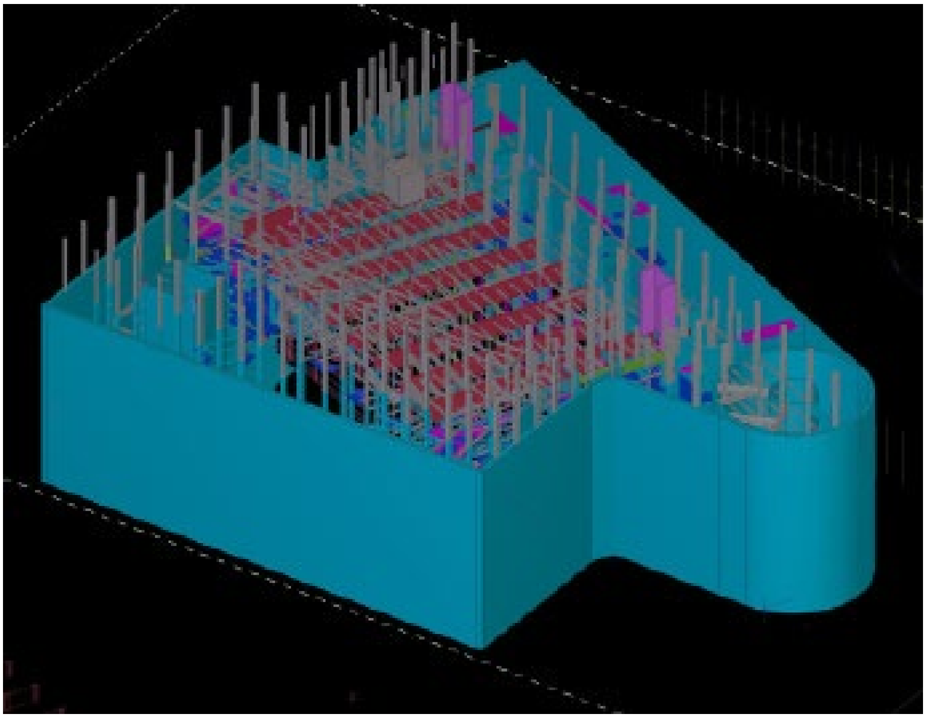
The Washington Marriott Marquis

A tall construction order with deep-seated challenges

Ginette Gélineau,
Senior Communications Advisor, CANAM Group

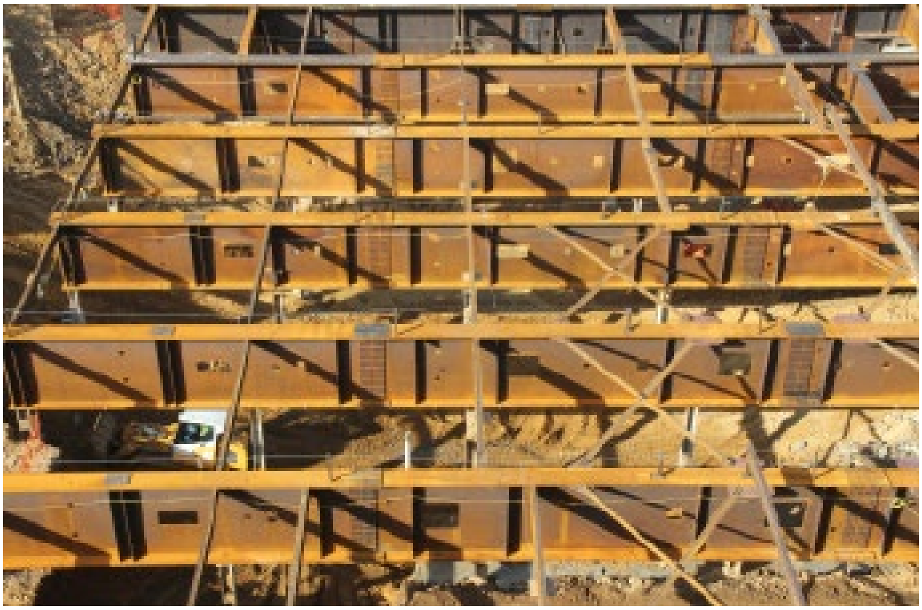
There's only one way to build a luxury hotel featuring over 1,100 rooms and over 105,000 sq. ft. (10,000 m^2) of ballroom and meeting space on a land area less than 86,000 sq. ft. (8,000 m^2) in size located in a city that bans the construction of buildings over 130 ft. (40 m): dig!



3D MODEL CREATED BY STRUCTAL-HEAVY STEEL CONSTRUCTION, A DIVISION OF CANAM GROUP INC. (STRUCTAL), ILLUSTRATING THE PERMANENT SLURRY RETAINING WALL AROUND THE BUILDING PERIMETER AND THE STEEL STRUCTURE OF THE SUBTERRANEAN LEVELS

This was the situation facing the Washington Marriott Marquis hotel in Washington, D.C. And while a top-down construction approach was the obvious natural choice, considerable factors would come into play making the project anything but conventional.

The hotel, which is designed to earn LEED Silver certification, opened on May 1, 2014. The 975,000-sq.-ft. (120,775 m^2) structure stands adjacent to the Walter E. Washington Convention Center and the two buildings connect underground via a subterranean walkway below 9th Street.



EIGHT MAIN GIRDERS SUPPORT THE 15 CONCRETE-FRAMED UPPER FLOORS

Given the hotel's design specifications and existing building codes, the top-down construction method was chosen. As a result, almost as much of the structure sits below grade (94 ft. [29 m]) as above ground (15 storeys). The upper floors are framed with concrete while steel was used for the seven underground levels that called for the fabrication of over 5,000 tons of steel components.

Insufficient soil bearing capacity

Since the bearing capacity of the soil was poor and could not support the projected weight of the building, the first step was to increase ground stability by drilling into bedrock. Steel columns were then placed in the shafts and encased in concrete in order to fully embed the components and protect them during the excavation of the lower levels.

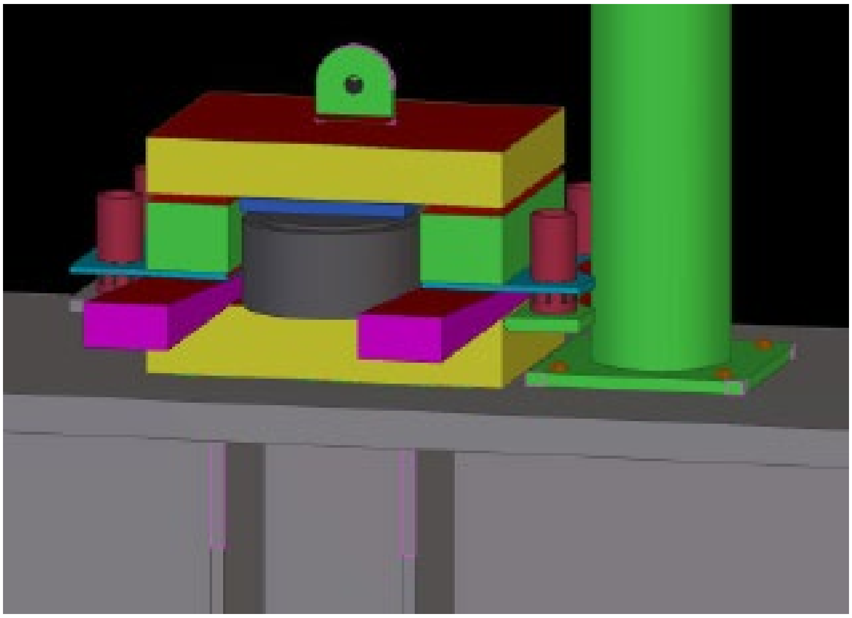
In addition, to resist the soil forces the underground perimeter of the building was encompassed with a slurry retaining wall that was left in place once the erection work was completed.

Traditionally, when a top-down approach is used, the upper floors are built simultaneously with the lower levels in order to save time and money. However, given the poor soil structure and the elevated risk of collapse, it was judged more prudent to wait until all the underground floors were almost fully completed before beginning construction above ground.

Structural steel framing at ground level

Composed primarily of eight main girders spanning 135 ft. (41 m) long by 11 ft. (3.4 m) deep, the ground floor, called level S7, is designed to support the multiple concrete-framed floors above it. The ground floor was initially intended to be built using immense trusses; however, during the design-build phase, Structal suggested that built-up steel three-plate girders be used instead. This value-added solution generated significant cost savings while also streamlining the connection of framing elements. Each girder was delivered in sections and bolted at the erection site. The girders are attached directly to the frame on one end and rest on top of a sliding bearing at the other end in order to withstand movements caused by temperature variations.

EIGHT HYDRAULIC JACKS WERE INSTALLED ON THE EXTERIOR GIRDERS TO ELIMINATE THEIR INITIAL CAMBER



A 3D MODEL OF A JACK INSTALLED ON ONE OF THE NINE INTERIOR COLUMNS. THE JACKING SYSTEM EXERTED PRESSURE ON THE COLUMN TO CONTROL TOWER SETTLEMENT DURING THE CONCRETE PLACEMENT

Once the girders were installed at ground level, excavation work could begin on the floor below, i.e., S6. After reaching the necessary depth, a concrete slab was poured in place to resist the lateral load caused by soil pressure at the perimeter.

Ballrooms

Level S5 features a vast 30,000-sq.-ft. (2,787 m2) grand ballroom that can accommodate 3,000 guests. The floor of this open, column-free ballroom stands at level S5 while the ceiling climbs two stories to level S7 for an overall height of 22 ft. (6.7 m).

Two junior ballrooms, also without columns and each featuring an area of 10,800 sq. ft. (1,003 m2), sit directly below the grand ballroom at level S3. The ceilings, which are equally 22 ft. (6.7 m) high, extend to level S5. The 82-ft. (25 m) trusses that support these two smaller ballrooms were shop-assembled and delivered to the erection site as single components.

Deflection control

The ground floor girders at level S7 were fabricated with a camber of three to 4.5 inches (7.6 to 10 cm) in anticipation of the load that would be placed on these components during the construction of the upper concrete-framed floors. However, it would be necessary to eliminate a large portion of this camber before pouring the concrete in order to avoid girder deflection, which would in turn caused cracks in the slab.

A system composed of eight hydraulic jacks installed on the two exterior girders and positioned at four equal intervals was used to pull the girders

Key players

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To complete the pre-deflection system, two jacks were mounted on nine interior plate girders below the concrete columns.

ONE END OF EACH GIRDER RESTS ON A SLIDING BEARING TO TRANSFER MOVEMENTS CAUSED BY TEMPERATURE VARIATIONS

downward, and preload them with 3,000 tons to eliminate the camber. As the upper concrete structure was built, the girder preload was progressively released.

To complete the pre-deflection system, two jacks were mounted on nine interior plate girders below the concrete columns. As each concrete floor was placed, the jacking system was used to push the column upward to eliminate the construction deflections. The jacks were removed once the concrete work was completed, but the supporting elements were permanently welded in place.

As the team in charge of the fabrication and erection of the structural steel frame of the seven underground levels, Structal was invited right from the pre-project phase to join the project management team in a design-assist capacity. According to Serge Dussault, Engineering Vice President at CANAM Group: "This was in no way a novel construction process. Though it's generally used only in exceptional circumstances, our teams possess the necessary expertise to overcome the various challenges that arose all along the way."


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Economical truss design from a fabricator's perspective

By Peter A. Boyle, Chief Designer, MBS Steel Ltd

Introduction

For statically loaded structures, trusses are a very economical alternative to wide flange beams and plate girders when an "Optimum Depth" can be achieved.

A good rule of thumb for an "Optimum Depth" is a depth to span ratio of 1:10.

This, however, may not always be achievable due to other factors such as architectural requirements (e.g., headroom, visual effects, etc.) and mechanical requirements (e.g., duct runs, etc.). Thus a premium cost will likely result.

For ratios of 1:18 and more, beams/ plate girders remain the economical solution.

It is also important to note that the web material should ideally constitute between 18% and 30% of the total weight, to stay within the economical zone.

Total cost per foot (metre) is the ultimate criteria to use, not weight per foot (metre) of material. In the discussion that follows we will attempt to illustrate that all materials are not alike in price per pound (kilogram) and that various material sections should be scrutinized at the design stage in order to optimize costs regarding material, fabrication, shipping and erection.

Material costs:

Unit Mass Costs

Structural members purchased directly from mills or warehouses are priced at Unit Mass Cost, expressed as dollars per pound or dollars per kilogram. The unit mass cost will vary with a number of changing factors, such as member type, member size, member mass, availability and demand. Hence the truss designer must constantly be alert to the pricing effect at any given time. In the following discussion, the terms price or cost per kilogram or per metre will be used for simplicity, consistent with the metric units.

Angles

Small and large angles are priced at a premium compared to the mid-range sizes. The most economical angle range is from 51x51x5 up to 127x127x16. These angle sizes are the most desirable for a mill to roll in angles and are priced accordingly, e.g., an L203x203x19 could be in the price per kilogram range of 75% more than an L51x51x6. Also, equal legged angles are priced much more aggressively than un-equal legged angles to the point where an L51x38x5 may cost more per metre than L51x51x5, in spite of the larger mass.

HSS

The cost of HSS members in the past has been subject to much more pricing volatility than any other structural member in the "Engineer's Toolbox" and unit mass costs are generally much higher than for angle sections.

Tees

Tees command a premium of approximately 15% more than the parent wide flange beam due to the splitting and straightening costs involved.

Wide flange beams

WFs are the most cost stable rolled sections with a cost at about 20% more per kilogram than the most economical equal leg angles.

Important notes for designers

There are several issues that the truss design engineer must keep in mind.

The first two items concern web design requirements of Clauses 15.2.5 and 15.2.6 of CSA S16-09

Standard of which many engineers may not be aware.

The other two items relate to load combination and are NBC 2010 Clause 4.1.6.3.2 and Clause 4.1.5.3

Clause 15.2.5.
Stipulates that the member and connection resistances for the 1st compression web member are to be reduced by 15%.

Clause 15.2.6.
Web members shall be designed and connected for a force equal to 2% of the axial force in the compression chord at the point they support, in addition to other forces.

All trusses are required to be designed for various load conditions, for whichever one produces the most critical effects for each member and its connections. The provisions below may produce more shear, and more shear reversal than just using a continuous UDL:

NBC Clause 4.1.6.3.2 – Roof Trusses subject to Snow Loads

Roof Trusses are to be designed for full and partial loadings, as follows:

- Full Loading – 100% of Dead Load + 100% of Snow Load, over full span.

- Partial Loadings – 100% of Dead Load over full span + 100% Snow Load over any portion of the Truss + 50% Live Load over the remainder.

NBC Clause 4.1.5.3 – Floor and Roof Trusses subject to Live Loads

Floor Trusses are to be designed for full and partial loadings, i.e., as follows:

- Full Loading – 100% of Dead Load + 100% of Live Load, over full span.

- Partial Loadings – 100% of Dead Load over full span + 100% Live Load over any portion of the Truss with no Live Load over the remainder.

These analyses are the truss design engineer's responsibility.

Chords
After the depth of the truss is determined, the choice of chord type is the next most important step in achieving an economical truss. From these decisions everything else follows. From a fabrication point of view the fewer number of members to be handled generally means the more economical the fabrication process. Eliminating gusset plates is certainly one possible step. This reduces the number of pieces (and mass) to be handled and also can reduce significantly the welding requirements (labour) sometimes by up to 50%. The ratio of material costs and labour costs vary from fabricator to fabricator; in eastern Canada, the rule of thumb is normally 50:50.

Double angles
For lightly loaded conditions, double angles in 380W material will prove most economical and will allow for a web system of alternating HSS (or channel sections) and angles, thus eliminating the extra work and cost of plates. Up to L127x127x16 it is fairly common to get in 380W grade.

Next the width thickness ratio for members in compression requires the designer's attention. All angles are Class 3, e.g., with an L152x152x13 in 380W, the effective leg width can only be 130.3 mm to attain the 380W yield range. Conversely the designer can use Clause 13.3.5(b) in CSA S16-09 to generate an equivalent yield, in the case of the 152x13 angle the yield will be 279 MPa. You will find that the equivalent yield concept will give a better result, because one can use the full section and all of its properties to determine the Cr. This clause is not economical when designing with unequal leg angles.

Tees

For medium loads the Tee section could prove to be an economical choice.

Note that the choice of Tee sizes is very important for maximum economy to be attained. The Tee stem must be capable of transferring the full shear with minimal if any, reinforcing. The stem must be deep enough so one does not need to add tab plates to the stem to attach the web members. In addition the stem thickness needs to be selected to accommodate the required size of fillet welds on both sides. One can see that the initial choice of a Tee section is very important, and probably input from the fabricator would be essential for an economical solution.

HSS

HSS chords should follow much the same rationale as Tee sections.

Wide flange beams

Wide flange sections for larger load range are usually cheaper per metre in the final analysis than Tees or HSS. These sections come in A992 or 350W grade material and the bending capacity for beams with webs in the plane of the truss is far superior to a T-section of equivalent mass. There is the downside that design depth will be less than with Tees that obviously increases axial forces and needs slightly more mass, but this is offset by the lower cost per kg, and in most cases the cost per metre will be less.

Unless the web system is HSS or WF there will be the requirement for gussets at every web joint. The gussets should be welded with simple fillet welds and will also take care of any additional web shear shortfall in the end two panel points. In long trusses where field splices are required (because of shipping concerns, etc.) wide flange chords are much easier to splice using high-strength bolts, which help reduce field costs even though shop costs will be increased. The designer may have the choice to rotate the wide flange sections through 90° and this has certain advantages if loads are applied only at the panel points. The choice of web members can be varied. Shear through the flanges should never be a problem. Assuming loads are transferred from intermediate members that sit on the top chord, support tees or plates have to be added at each of these framing members. Utilizing an Angle web system on the flanges could eliminate the need for gusset plates, so it is up to the designer to determine which configuration is the most economical.

Webs

The most economical web configuration is the Modified Warren System. For identical loading conditions, the Modified Warren uses only 80% of the members used for a Warren or Pratt configuration. Some members are larger but several are considerably smaller. The overall weight is not much different. In a best-case scenario, using the Warren system, with equal tension and compression member lengths, the weight may be 5% less than a Modified Warren, but the labour cost can be over 20% more, so the cost per metre would be greater.

For an economical truss one must design the members for the loads and not for esthetics 'i.e.' all members of similar sizes.

Web sections for various chord member types

Double angle chords: Alternating HSS (or channels) and double angles

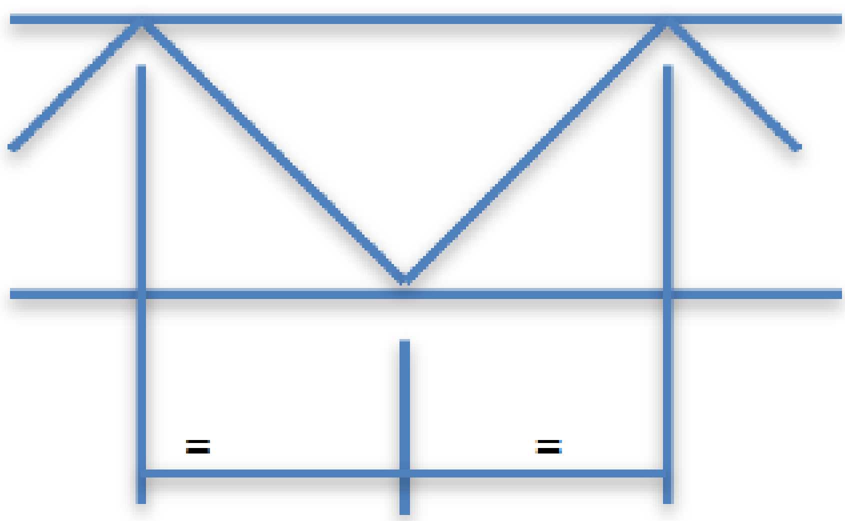

FIGURE 1 – WARREN SYSTEM WITH EQUAL-LENGTH TENSION AND COMPRESSION WEB MEMBERS

works well for web members for a Modified Warren System. The vertical works best as an HSS/channel section welded to the tension member. The tension member should also be an HSS/channel section as this can eliminate the shear lag concern of angles in tension when fully stressed. This is counterintuitive to the thinking that HSS is better in compression. This is true. The downside is that in the Modified Warren system, one would be welding the vertical to the compression member, and then welding the compression member to the chord to carry the additional load from the vertical, as the shear loads flow toward the support. Welding to the tension member eliminates this concern.

Double angles for both tension and compression pose their own problems. Shear must be transferred through the vertical legs of the angle chord members, as there is no overlap of web members. In addition shear lag in the tension members must be addressed.

Wide flange chords: A variety of web types can be used, including double angles, WF, channels or HSS sections. Depending upon the magnitude of the loads, the most economical are angles in 380W grade. If the wide flange chord web is vertical, then consideration should be given to utilizing star-shaped angles. This entails a slightly larger gusset plate and double fills in the star-shaped angle web but the added cost is offset easily by the reduction of the compression member size. Then r_y becomes the critical Radius of Gyration and being 25% larger than the r_x increases the C_r/A so that in most cases one needs one member weight (mass) size less for the same load. Using all star-shaped angles allows for nesting of connections and

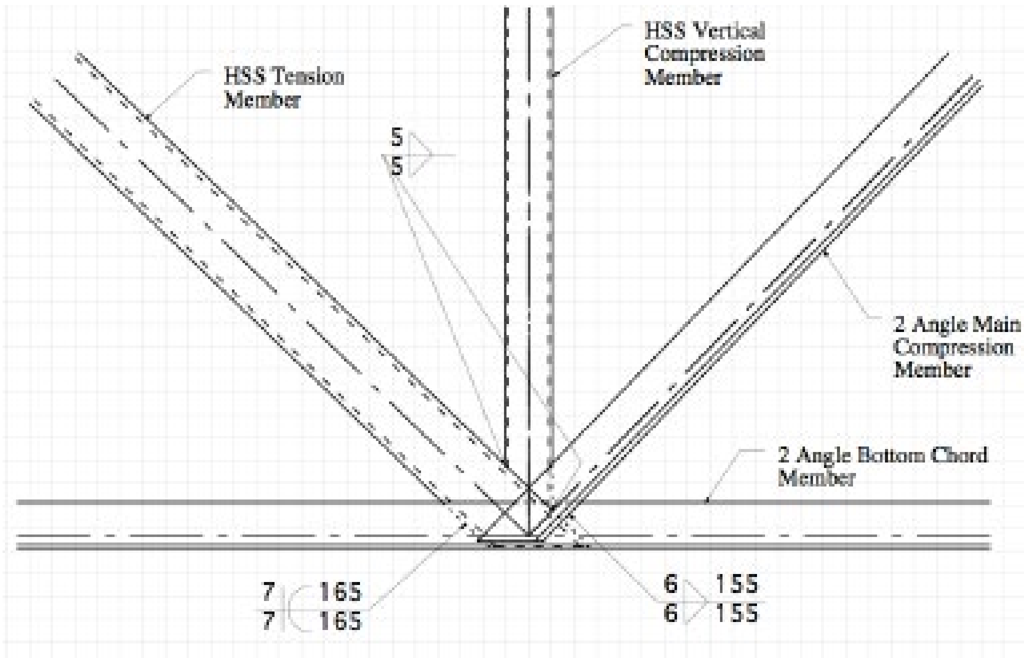

FIGURE 2 – TYPICAL LOWER JOINT FOR MODIFIED WARREN SYSTEM

overlapping of members on opposite sides of the common gusset plate.

If loads are such that the economical angle range cannot be used then a web consisting of wide flange sections should be considered. Points to consider when using a WF web system:

1) Shear capacity of the chord web (especially at the two end panel points). With WF webs in the plane of the truss, one can, with the right choice of chord size, eliminate the need for stiffeners and have a very clean look and economical truss.

2) The geometry is very important for the choice of weld types and sizes. A 45˚ slope is an ideal compromise for the flange welds of the web members. If the slope is too steep (shallow depth with large panel points) there usually is the requirement for additional weld preparation resulting in extra time and cost.

Tee section chords: The most common and economical web selection is the double angle sections. The Modified Warren system may not be the most economical in this case, because of limited space to place the vertical angles without adding tab plates. This, of course, depends on the magnitude of the loads and Tee section chosen.

HSS section chords: Angles or Channels welded to the vertical sides of the HSS section chords would appear to be the most economical solution if HSS section chords are used. HSS web sections would not have the same economy as simple angles or channels but architectural requirements may dictate the use of an all HSS Section truss. In this case the choice of member sizes is crucial to the best economy possible. In short, the lightest HSS sections will not give the most economical truss design. Heavier chord and/or web sections that eliminate reinforcing plates are the smart way to go.

Connections:

Whenever possible, keep the end connections as simple as possible and keep eccentricities as small as feasible. Much of the economic benefit of a well-designed truss can be lost when the end support connections are not also well planned and thought out.

Conclusion

From the foregoing we have shown that to produce an economical truss many factors must be assessed during the design stage. Remember that total weight (mass) is NOT the most critical factor in designing an economical truss.

Input from a steel fabricator at the design stage could be most beneficial to the material selection process and to the preliminary costing of the truss. It must be pointed out that each fabricator has slightly different fabrication procedures depending on equipment and methods available to them.

Therefore consideration, especially on large projects, should be given to an initial pre-bid budget consultation with two or three fabricators to determine which would be the most economical fit for the project.

Peter Boyle gives special thanks to Ted Aziz, P. Eng. and John Mark, P. Eng. for their valuable assistance in the writing of this article.


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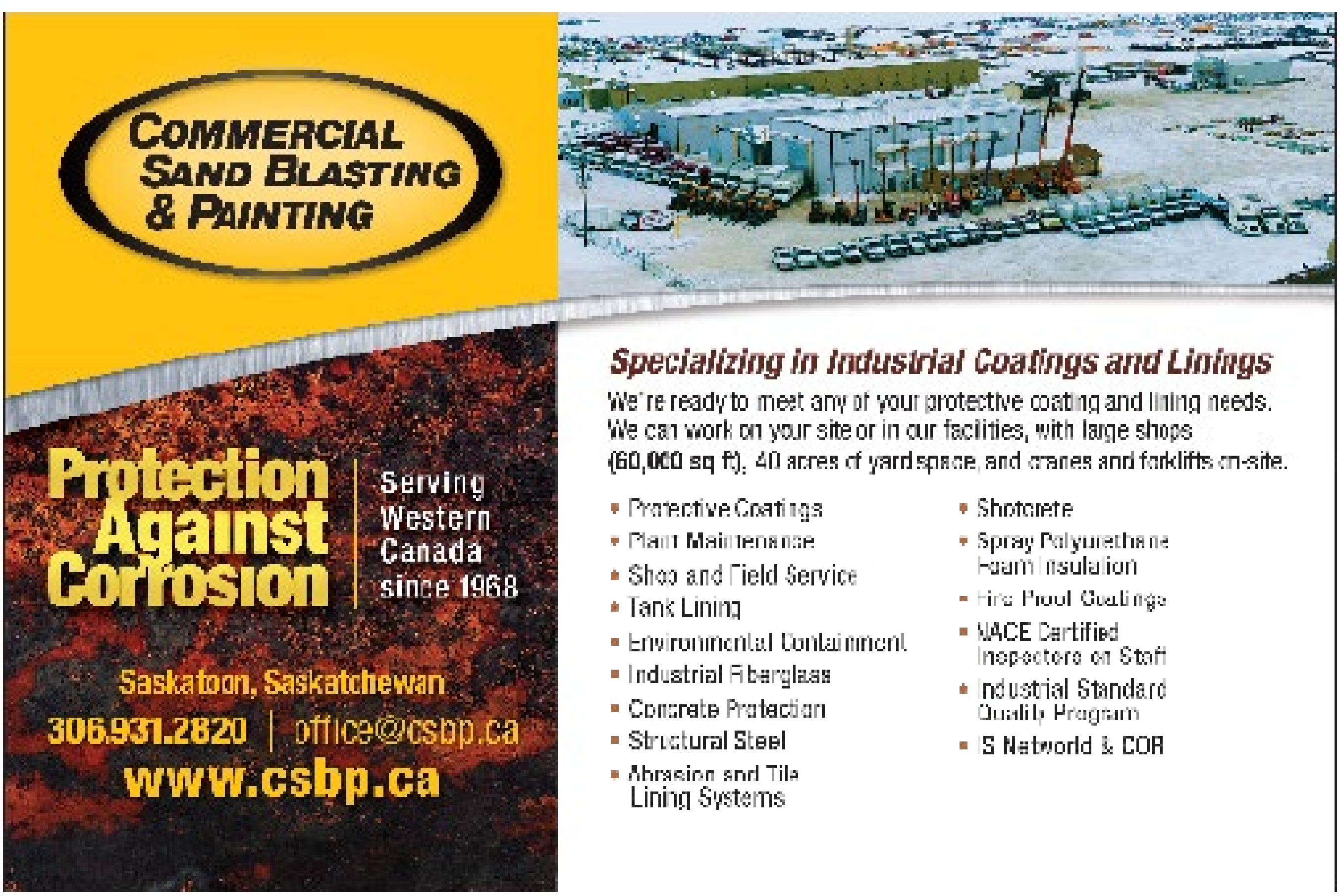
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Expanding the options

Structural fire engineering gives architects and owners a wider range of design and engineering choices

By Andrew Brooks

While the development of structural fire engineering (SFE) or performance-based design (PBD) dates back to at least 1927, when the American Institute of Steel Construction (AISC) issued a *Specification for Fireproofing Structural Steel Buildings,* its modern history begins in 2005. That's when provisions for structural design for fire conditions were introduced in Appendix 4 of the 2005 edition of the *ANSI/AISC 360-05, Specification for Structural Steel Buildings*. The Appendix, which bears the title "Structural Design for Fire Conditions," was revised and updated in 2010, and work is underway on another revision and update in 2016.

The development of structural fire engineering principles arises from the limitations of the prevailing standardized fire testing model, says Nestor Iwankiw, senior engineer with Hughes Associates in Chicago.

"In Canada and the U.S. – and across the world – passive fire protection has been done very prescriptively, based on standardized fire tests, furnace tests and fire ratings for one or two or three hours' endurance for different assemblies," Iwankiw says. Since 2010, Iwankiw has chaired the Design for Fire Conditions committee, a subcommittee of the AISC Committee on Specifications that maintains and updates Appendix 4.

"In construction the architects specify a certain assembly and then it's built the way it was tested or listed as a rated assembly," Iwankiw says. "So there's very little engineering or analysis involved. It's basically a matter of looking up a rated assembly that fits the type of building you're intending to design and then implementing that in the construction."

Following the terror attacks of 9/11, awareness grew that this prescriptive pre-rated approach limits what can be done in terms of fire safety and also limits the range of engineering and architectural design options. "It's very empirical and limiting to use a very finite set of listed standard assemblies," notes Iwankiw. "The primary intent of Appendix 4 was to enable fire engineered design – so part of it is about establishing the fire exposure in lieu of using a standard furnace curve, which is what is used for these prescriptive tests."

Structural fire engineering, simply put, allows for the development of alternative fire protection solutions when the particular structural assembly to be used is unique and does not appear on a list of assemblies that have been pretested for fire under laboratory conditions and then given a fire rating. The text in Appendix 4, as described in an article by Iwankiw and Bruce Ellingwood from the Georgia Institute of Technology in Atlanta, "permits project compliance to be accomplished either by performance-based structural fire engineering or by the traditional qualification testing."

As described by George Frater and Carol Kleinfeldt in the Spring 2011 edition of *Advantage Steel* ("Fire Protection of Steel Structures," pg. 16), "the method to arrive at an alternative solution is commonly referred to as 'Performance-based Design,' or PBD, or in other words is an engineered approach to fire protection and carried out by specialized 'fire protection engineers.'"

The provisions of AISC's Appendix 4 have been codified for application in Canada as part of the Canadian Standards Association (CSA) S16 standard, "Design of Steel Structures." The structural fire engineering provisions appear within Annex K of the S16 standard, which is titled "Structural Design for Fire Conditions."

Architectural impact

"One of the main things that's driving structural fire engineering, from what I see, is architecture," Iwankiw says. "In a lot of places architects want to reveal and express the structural form without obstructing it with gypsum board or covering it with spray-on coatings, so as to show off the structural framing." And when that framing has to be fire resistance qualified, typically analysis is required to establish that the framing will withstand fire exposure for the mandated period of time without collapsing.

Some of the typical applications for SFE are in large open spaces where greater heights and floor spans are required, as in atriums and monumental public buildings. And such cases, of course, generally call for structural steel to be used. "One type of construction I've been involved with is the use of unusual columns for some tall open spaces," Iwankiw says. "Frequently the preferred choice is to use hollow structural steel columns infilled with concrete, without using any kind of spray-on or intumescent coating. That's one way to provide fire resistance. But there's no direct assembly that can be used for that situation, plus often you have very large heights different from the usual 12- or 13-foot floor-to-floor heights." That means that in order to ensure the columns are safe, some additional analysis is required, to confirm that the columns will remain intact long enough to allow people to escape during a fire.

Structural fire engineering has been somewhat slow to catch on, for a number of reasons. Iwankiw notes that the decision to adopt SFE usually requires a firm insistence by the architect or building owner on precisely the kind of design that requires SFE. But perhaps more important is the fact that it simply requires more effort than a prescriptive solution that uses pre-rated assemblies.

UNPROTECTED STEEL FLOOR ASSEMBLY USING FIRE PROTECTION ENGINEERING

PROTECTED STEEL - SPRAY-APPLIED FIRE-RESISTIVE MATERIALS ON STEEL BEAMS AND STEEL DECK

"This isn't the same as an architect plucking a rated assembly out of a book," Iwankiw says. "It involves more engineering effort, meetings with the building jurisdiction, agreement on what's acceptable and what's not. And often for these situations, and particularly for more monumental buildings, an independent peer review is typically required by the building jurisdiction to confirm that what's being done with performance-based design is adequate and valid and accurate."

SFE advantages

But the added effort has a payoff, Iwankiw says. SFE yields better structural function, makes much better use of available space, and of course meets difficult and unique design objectives. The financial aspect can't be ignored either. SFE can reduce the cost of fire protection, for starters.

"On a multi-storey building, you can save some marginal amount on the fireproofing of the floor system, and that's multiplied over many floors," explains Iwankiw. "That can be one payout, and it will help defray some of the initial expenses of performance-based design, such as the added fees on the engineering piece and the analysis."

As reported in the Fall 2006 edition of *Advantage Steel* ("Citadel High School: A Performance-Based Solution for Unprotected Structural Steel," pg. 12), the designers of the new Citadel High School building in Halifax employed structural steel beams instead of floor joists. The design required fire engineering analysis as the architectural solution could not make use of pre-rated assemblies. By reducing floor heights, the performance-based design solution used for the building saved expanse on the exterior cladding of the structure. And as Iwankiw notes, the reduction of the size of a structure means smaller, lighter foundations can be provided – another cost saving, which of course multiplies over the number of storeys.

The Design for Fire Conditions Committee is currently at work on the next round of revisions and updates to AISC's Appendix 4, due for publication in 2016. With input

> "On a multi-storey building, you can save some marginal amount on the fireproofing of the floor system, and that's multiplied over many floors..."
>
> Nestor Iwankiw, Hughes Associates

from the U.S. National Institute of Standards and Technology (NIST), the committee's work is focusing on a number of areas, including material properties. "We're looking at work on the properties of steel at high temperatures," Iwankiw says. "This was generated by NIST as a result of their 9/11 investigations, where they did a lot of testing of steel at high temperatures." NIST's work and recommendations on strength curves and retention factors for steel differ slightly from the Eurocode model initially adopted in Appendix 4, Iwankiw says.

Other revisions will come in the areas of structural columns and the reliability of design formulas at high temperatures. And Iwankiw promises new work on connections, particularly bolts.

"Maybe the hardest part of structural fire engineering is determining what end connections are actually doing during a fire," says Iwankiw. "Member design, beams and columns, etc. are difficult enough, but it's at least a little simpler because you're dealing with a single member and one type of material. With connections you have plates, angles, bolts, maybe welds, some gaps, maybe oversize holes, slotted holes, etc. And when you compound that with potentially different materials and the thermal expansion and contraction during a fire, the connection becomes a very hard thing to analyze."

The committee is working to identify the properties of these connection elements – at least in the case of bolt connections. So far there isn't a critical mass of research information on welded connections available to work with, says Iwankiw.


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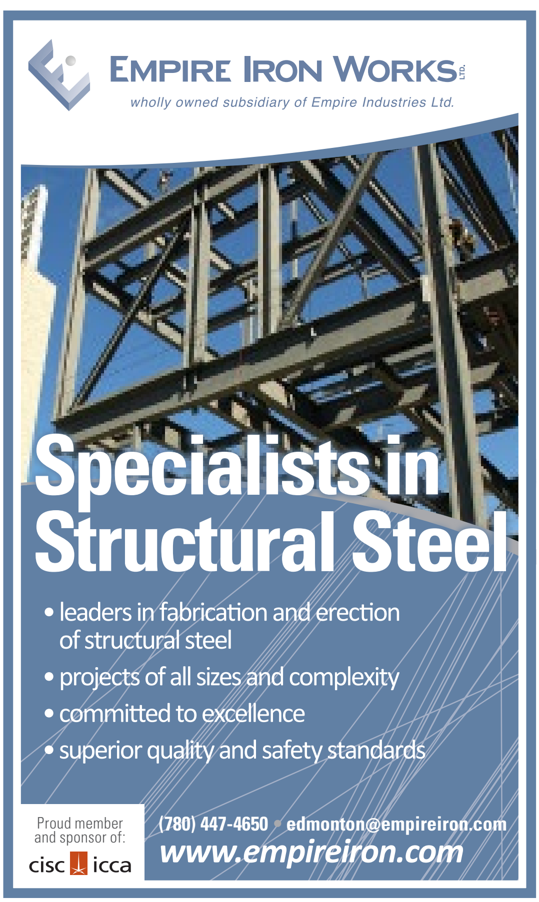
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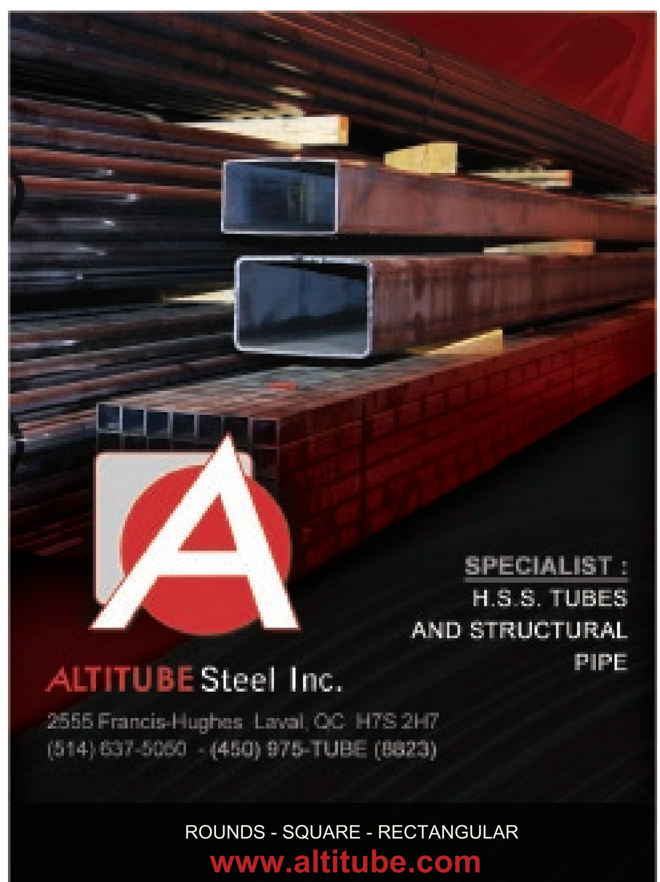
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News and Events

Continuing Education Courses

CISC is pleased to present a new English-language course in the fall of 2014 and a new French-language course in the winter of 2015 that leads to CISC Accreditation as a Steel Inspector – Buildings.

CISC has begun to offer self-paced, online education, using previously recorded material packaged with tutoring and examinations where applicable. Please watch the Education pages on the CISC website for the following courses and webinars: Handbook of Steel Construction, Statics and Strength of Materials, Connections I, Connections II, AESS Series and Fire Protection.

For the full course schedule, information, online registration and the latest updates, please visit our website at www.cisc-icca.ca/courses, or request a copy of our course calendar.

Building with Steel: Critical Stuff you Need to Know! *– New Course –*

This is a new course on steel building construction that will provide an overall, non-technical understanding of the structural design and engineering process, manufacturing approvals, applicable building codes and standards, roles and responsibilities, performance, critical aspects of drawings, specifications, design, details and the many manufacturing/fabrication engineering and construction documents, standard of practice, plan reviews, general review and inspection.

The course leader will review regulatory requirements in the building code and referenced standards related to steel construction, as well as the performance of duties by certified and licensed professionals, manufacturers, fabricators, erectors and building code officials. Experiences and lessons presented are based in part on an understanding of forensic engineering incorporating actual investigations (including many interviews with industry).

The course will also touch upon the steps involved in an investigation and evaluation in issues of design, manufacturing, fabrication, erection and inspection. As the industry and projects tend to be litigation orientated and are in constant dispute, the course will incorporate a roundtable discussion and summary of the above and assessment of responsibility, risk management, and recommendations on avoidance of issues.

Of interest to chief building officials, building inspectors, plan reviewers, architects, engineers, field inspectors, building envelope consultants, fabricators, erectors, project managers, general contractors and owners.

Course Leader:
Joseph (Joe) S. Vidican, P.Eng., Principal, Vidican Engineering (Building Science & Forensic Engineering)

Toronto, ON	Nov. 5	Vancouver, BC	Nov. 14
Winnipeg, MB	Nov. 6	Montreal, QC (E)	Dec. 1
Saskatoon, SK	Nov. 7	Halifax, NS	Dec. 2
Edmonton, AB	Nov. 12	Fredericton, NB	Dec. 3
Calgary, AB	Nov. 13		

Connections for Design Engineers

This course is intended to provide practical guidance to steel designers and clarify the complementary roles of the fabricator and the design engineer with respect to connection design. Emphasis is placed on connections and their impact on costs and economy.

The basic objective is to assist designers in their understanding of how connections influence member design and vice versa, and to emphasize the importance of considering both connections and member selection for optimum economy. The scope of the course is limited to connections normally encountered in common types of steel building structures.

The presenters will highlight major changes in S16-09 that influence the design of structural steel connections. Topics include high strength bolts, welds, bolts in tension and prying, slip-critical connections, welds and bolts in combination, eccentric connections, simple shear connections, seated beam connections, connection to concrete, column connections, moment connections (W & HSS Sections), bracing connections, gusset plates and truss connections.

Course Leaders:
John R. Mark, M.Sc., P.Eng., Past President, M&G Steel Ltd

Peter C. Birkemoe, Ph.D., P.Eng., Professor Emeritus, University of Toronto

Halifax, NS	Sept. 16	Calgary, AB	Sept. 18
Ottawa, ON	Sept. 17	Vancouver, BC	Sept. 19

Industrial Building Design

This course is intended to provide understanding on design theory and the rationale behind code provisions that are unique to steel-framed industrial buildings. It focuses on practical and economical solutions for framing a typical industrial building to the requirements of the 2010 National Building Code of Canada and the pertinent provisions of CSA Standard S16-09.

The learning goals for this course include the following: identify the unique environmental and mechanical loading conditions in industrial buildings, learn the applicability and limitations of current codes and standards in Canada, select the most cost-effective framing schemes, design crane-supporting girders, stepped columns, purlins and girts, explore lateral force resisting systems, roof trusses and efficient connections, understand serviceability considerations and limitations, design for high and low temperatures, learn the implications of seismic provisions, plus other topics such as fatigue, standing seam roofs, rehabilitation, tolerances and coatings.

Course Leaders:
Robert A. (Bob) MacCrimmon, P.Eng., Senior Civil/Structural Specialist, Hatch

Logan Callele, M.Sc., P.Eng., Engineering Manager, Waiward Steel Fabricators

Toronto, ON	September 23
Saskatoon, SK	September 24
Calgary, AB	September 25

Single Storey Building Design
– New Online Course –

This course focuses on practical and economical solutions for framing a single storey warehouse building with attached office area to the requirements of the 2010 National Building Code of Canada and the pertinent provisions of CSA Standard S16-09.

Practical steel framing concepts and integration with architectural and mechanical features will be discussed. The presenters will highlight major changes in NBCC 2010 and CSA S16-09.

Topics include ponding of rainwater, snow drifting, companion load combinations, wind and seismic loads, notional loads, P-delta effects, selection of deck and joist systems, design of Gerber girders, design of interior and exterior columns, girts, base plates and anchor rods, selection and design of braced frames and roof diaphragm, fire protection issues, steel fabrication considerations, material selection and economics.

Course Leader:
R. Mark Lasby, B.Sc., P.Eng., Principal Structural Engineer, Fluor Canada Ltd., Calgary

Webinar Format (4@2hrs)
Nov. 4-5, 12:00 - 2:00 p.m. and 3:00 - 5:00 p.m. ET

Inspection of Steel Building Structures
– Accreditation Program –

This 3½-day course will prepare inspectors, designers, building officials, fabricators, erectors and other specialists for the inspection of steel-framed buildings in the field. A course participant who achieves an 80% grade on the optional three-hour final exam on the final day will be designated by CISC as an Accredited Steel Inspector - Buildings.

Applicable sections of the National Building Code of Canada, CSA S16 plus referenced material, product and quality standards, CISC Code of Practice and CISC Certification guidelines will be addressed. Typical structural, erection and shop drawings for steel-framed buildings will be explained. Material identification, tolerances, bolting and welding processes and procedures will be reviewed. Included are OWSJ, floor and roof deck, shear studs, surface preparation and coatings.

Course Leader:
Robert E. Shaw, Jr., PE, President, Steel Structures Technology Center, Inc.

Toronto, ON - 1:00 p.m., Oct. 27 to 12:00 p.m., Oct. 31

Inspection des structures de bâtiments en acier
– Nouveau cours : Programme d'accréditation –

Ce cours de 3 ½ jours préparera inspecteurs, concepteurs, agents du bâtiment, fabricants, monteurs et autres spécialistes à inspecter sur le terrain des bâtiments à charpente métallique. Un participant qui obtient une note de 80% à l'exam final facultatif de 3 heures offert la dernière journée sera désigné par l'ICCA comme un inspecteur accrédité – bâtiments en acier.

Les sections applicables du Code national du bâtiment du Canada, la norme CSA S16 ainsi que le matériel référencé, les produits et normes de qualité, le Code pratique standard de l'ICCA et les lignes directrices de certification de l'ICCA seront abordés. Les dessins typiques de structure, de montage et d'atelier pour les bâtiments à charpente d'acier seront expliqués. Les processus et les procédures d'identification du matériel, des tolérances, du boulonnage et le soudage seront examinés. Les poutrelles, le tablier de plancher et de toit, les goujons de cisaillement, la préparation de la surface et des revêtements sont inclus.

Conférenciers: À confirmer

Montréal, QC	Février 2015
Québec, QC	Février 2015

Nouveautés CSA S16-09 et survol du Handbook
– Webinaire –

Ce cours traite des modifications apportées à la norme CSA S16-09 et au dimensionnement des charpentes métalliques à l'aide de la 10e Édition du « Handbook of Steel Construction ».

Ce cours est proposé en ligne, en quatre séances de deux heures, via le système GoToWebinarMC. Les personnes intéressées peuvent s'inscrire aux quatre séances (0,8 UFC/CEU seront accordés à la fin du cours), ou à la séance unique sur la norme CSA S16-09 (0,2 UFC/CEU seront accordés à la fin du cours). De plus, des offres de remise groupées avec le « Handbook » et l'adhésion à l'ICCA seront proposées auxparticipants lors de l'inscription.

Le cours de formation continue de l'ICCA, Nouveautés CSA S16-09 et survol du « Handbook », est présenté en ligne (webinaire) en quatre séances de deux heures comme suit.

Conférenciers:
Hellen Christodoulou, Ph.D., ing., B.C.L., LL.B., M.B.A., Directrice Régionale-Québec, ICCA

Charles Albert, M.Sc.E., P.Eng., Directeur des publications techniques, ICCA

12 novembre
12 h - 14 h et 15 h - 17 h (HAE)

13 novembre
12 h - 14 h et 15 h - 17 h (HAE)

Seismic Design of Steel-Framed Buildings

Held in tandem with the Seismic Connections for Steel-Framed Buildings (see following page, this course is intended to provide understanding on design theory and the rationale behind code provisions as well as the application of specific Code formulae and requirements. It will cover the design of seismic resisting systems for steel-framed buildings to the requirements of the 2010 National Building Code of Canada and the pertinent provisions of CSA Standard S16-09.

NBCC 2005 introduced very substantial technical changes, including a new subsection on Earthquake Load and Effects. NBCC 2010 introduces refinements to the way earthquake loads are calculated, some new SFRS and revised limitations on existing SFRS. In turn, S16 has modified many requirements in the seismic design clause, provided additional SFRS options, including buckling-restrained braces and plate walls with holes, and incorporated requirements for use of Conventional Construction for taller buildings in areas of moderate and high seismicity.

New topics include buckling-restrained braces and higher limits for conventional construction. Updated topics include tension only braced frames, concentrically braced frames, ductile eccentrically braced frames, Type LD moment resisting frames, ductile moment resisting frames, notional loads, P-delta effects and diaphragms.

Principal course leader: Alfred E. Wong, M.Eng., P.Eng., Director of Engineering, CISC

Seismic Connections for Steel-Framed Buildings
Held in tandem with the Seismic Design for Steel-Framed Buildings (see above), this course prepares consulting structural engineers and steel fabrication engineers for the design of connections in ductile Seismic Force Resisting Systems in steel-framed buildings to the requirements of the 2010 National Building Code of Canada and Clause 27 of CSA Standard S16-09. The critical connections in the design examples developed for the Seismic Design of Steel-Framed Buildings course are used.

Capacity design requirements, now well entrenched in Clause 27 of S16-09, have virtually revolutionized the design, detailing and construction of connections for seismic applications. These requirements make it almost impossible to design Seismic Force Resisting Systems in isolation since the overall behaviour of these frames is highly dependent on the configuration and proportioning of these connections. The course will take participants through the detailed design of connections for moment connections covered in the CISC publication "Moment Connections for Seismic Applications," links and brace connections in Eccentric Braced Frames, tension-compression brace connections, tension only brace connections, and more.

Principal course leader: Augustin Dukuze, Ph.D., P.Eng., Principal, AnalytiXal Designs Ltd.

	Seismic Design	Seismic Connections
Toronto, ON	December 1	December 2
Vancouver, BC	December 4	December 5

Conception, fabrication et construction de ponts en acier
Ce cours traite de la conception, de la fabrication et de la construction de ponts en acier selon la norme CAN/CSA-S6-06, Code canadien sur le calcul des ponts routiers, supplement no #1. Ce cours a pour but d'aider à mieux comprendre la théorie de conception et le raisonnement des dispositions du code ainsi que l'application de certaines formules et exigences du Code. Les aspects pratiques et économiques de la fabrication, du montage, du choix des matériaux et leurs conséquences sur la conception seront également mis en évidence.

Conférenciers:
Gilbert Grondin, Ph.D., P. Eng., Senior Bridge Engineer, AECOM & Adjunct Professor, University of Alberta

Jean de Gaspé Lizotte, M.Sc., ing., Directeur, Projets spéciaux, Dessau Soprin inc.

Richard B. Vincent, B.Eng., ing., Vice-président, recherche, Groupe Canam Inc.

Montréal & Québec, QC Mars, 2015

Common Codes and Standards for Design and Construction of Steel Structures Current Status and Future Publication Targets

CODE/STANDARD/ SUPPLEMENT/COMMENTARY/ REFERENCED DOCUMENT	CURRENT EDITION	NEXT EDITION/ REVISION	PUBLICATION TARGET
National Building Code of Canada (NBC)	NBC 2010	NBC 2015	Late 2015
NBC Structural Commentaries (Part 4 of Div. B)	NBC 2010 Str. Comm.	NBC 2015 Str. Comm.	2016
CSA S16 Design of Steel Structures	CSA S16-14	TBA	
CISC Commentary on CSA S16 (Part 2 of CISC Handbook of Steel Construction)	CISC Handbook 10th Edition[1]	CISC Handbook 11th Edition[2]	2015
CISC Moment Connections for Seismic Applications	2nd Edition	TBA	
CSA S6 Canadian Highway Bridge Design Code	CSA S6-06	S6-14	Fall 2014
- Supplements to CSA S6	CSA S6S3-13	None planned	
CSA S6.1 Commentary on Canadian Highway Bridge Design Code	CSA S6.1-06	S6.1-14	Fall 2014
- Supplements to CSA S6.1	CSA S6.1S3-13	None planned	
CSA G40.20/G40.21 General Requirements for Rolled or Welded Structural Quality Steel/Structural Quality Steel	G40.20-13 G40.21-13	TBA	
CSA W59 Welded Steel Construction (Metal Arc Welding)	CSA W59-13	TBA	
CSA W47.1 Certification of Companies for Fusion Welding of Steel	CSA W47.1-09	W47.1-15	2015

[1]CISC Handbook of Steel Construction - 10th Edition includes CSA S16-09, its Commentary, CISC Code of Standard Practice - 7th Edition, and design and detailing aids in accordance with CSA S16-09

[2]CISC Handbook of Steel Construction - 11th Edition includes CSA S16-14, its Commentary, CISC Code of Standard Practice - 7th Edition, and design and detailing aids in accordance with CSA S16-14

EVENTS

SAVE THE DATE: 6TH ANNUAL QUEBEC STEEL WORKSHOP

Quebec Steel Symposium & Trade Show and Quebec Steel Industry Awards of Excellence. The contribution of the steel industry to the world of transport and infrastructure, on October 30, 2014.

CISC ONTARIO'S REGIONAL FALL EVENT

CISC Ontario's Regional Fall Event will be held on Thursday, November 13, 2014. Details to follow.

ALBERTA STEEL DESIGN AWARDS OF EXCELLENCE

On April 16, 2015, in Hall D, Shaw Conference Centre in Edmonton, Alberta. This is the 10th anniversary of CISC Alberta region showcasing excellence in steel design and innovation through this gala awards event.

ALBERTA REGIONAL MEETING

All members are invited to attend the fall regional meeting. November 19, 2014, 12:00 - 4:00 p.m.
Hosted by Steel Plus, 201, 10171 Saskatchewan Dr. Edmonton, AB

SASKATCHEWAN REGIONAL FALL MEETING

All members are invited to attend the fall regional meeting. Join CISC Saskatchewan's Regional Fall Meeting on September 24, 2014 in Saskatoon at the Best Western Royal Inn, 2:30 - 4:00 p.m.

SASKATCHEWAN REGION FALL STEEL EVENT

On October 22, 2014, Benefits of Automation & Robotics in Steel Fabrication will be explored at the Boffins Club on the University of Saskatchewan campus. Reception begins at 5:30 p.m. The presentation will be held from 6:00 - 7:00 p.m.

New CISC Members and Associates

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Rempel Engineering & Management Ltd., Saskatoon, SK

Safe Roads Engineering, Gormley, ON

Steel Erectors

KWH Constructors Ltd., Burnaby, BC

Danco Steel & Fabrication Ltd., Edmonton, AB

Professional Individuals

Bernardin, Dave B., P.Eng., Project Engineer, Armtec Ltd., Calgary, AB.

Bevan Pritchard, Geoff L., P.Eng., President of Bevan-Pritchard Man Associates Ltd., Vancouver, B.C.

Boudreau, Marc, P.Eng., Roy Consultants Engineering Services, Bathurst, NB.

Delph, Marcel A., P.Eng., City of Ottawa, Greely, ON

Kelly, William J., P.Eng., Self Employed Engineer/Project Manager, Edmonton, AB

Mitrovich, Matthew L., P.Eng., Structural Engineer, Conestoga Rovers & Associates, Waterloo, ON

Ruggieri, Antonio, P.Eng., Alberta Engineering Ltd., Calgary, AB

Simpson, Ewart G., P.Eng., ECS Simpson Engineers Inc., Calgary, AB

Suppliers

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NUCAP Industries Inc., Toronto, ON

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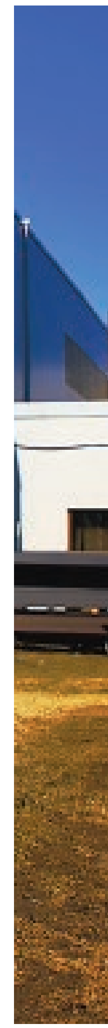

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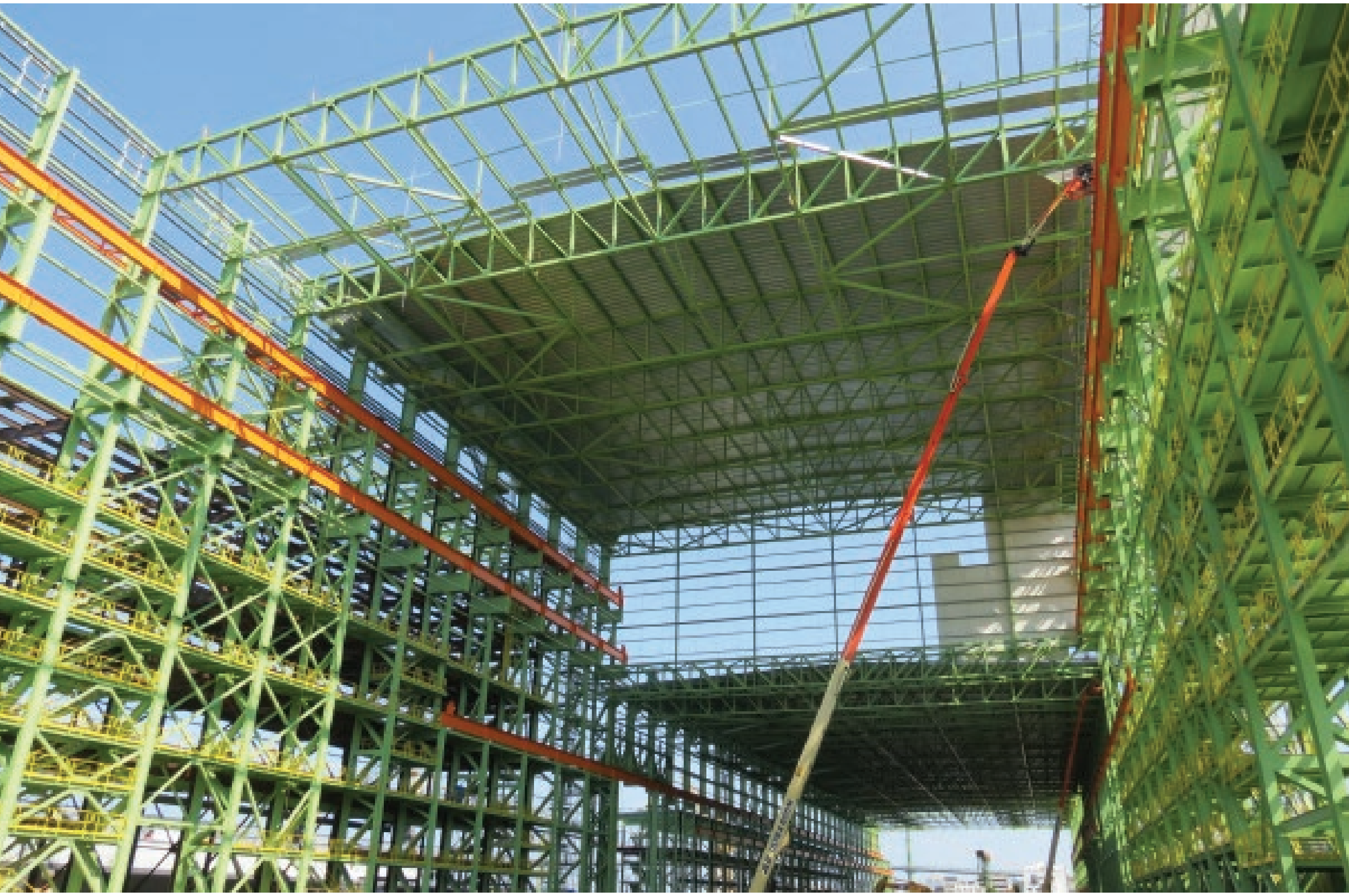

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