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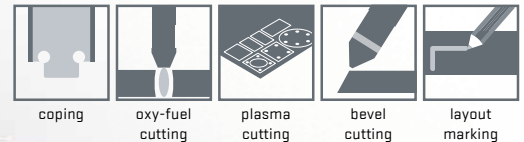
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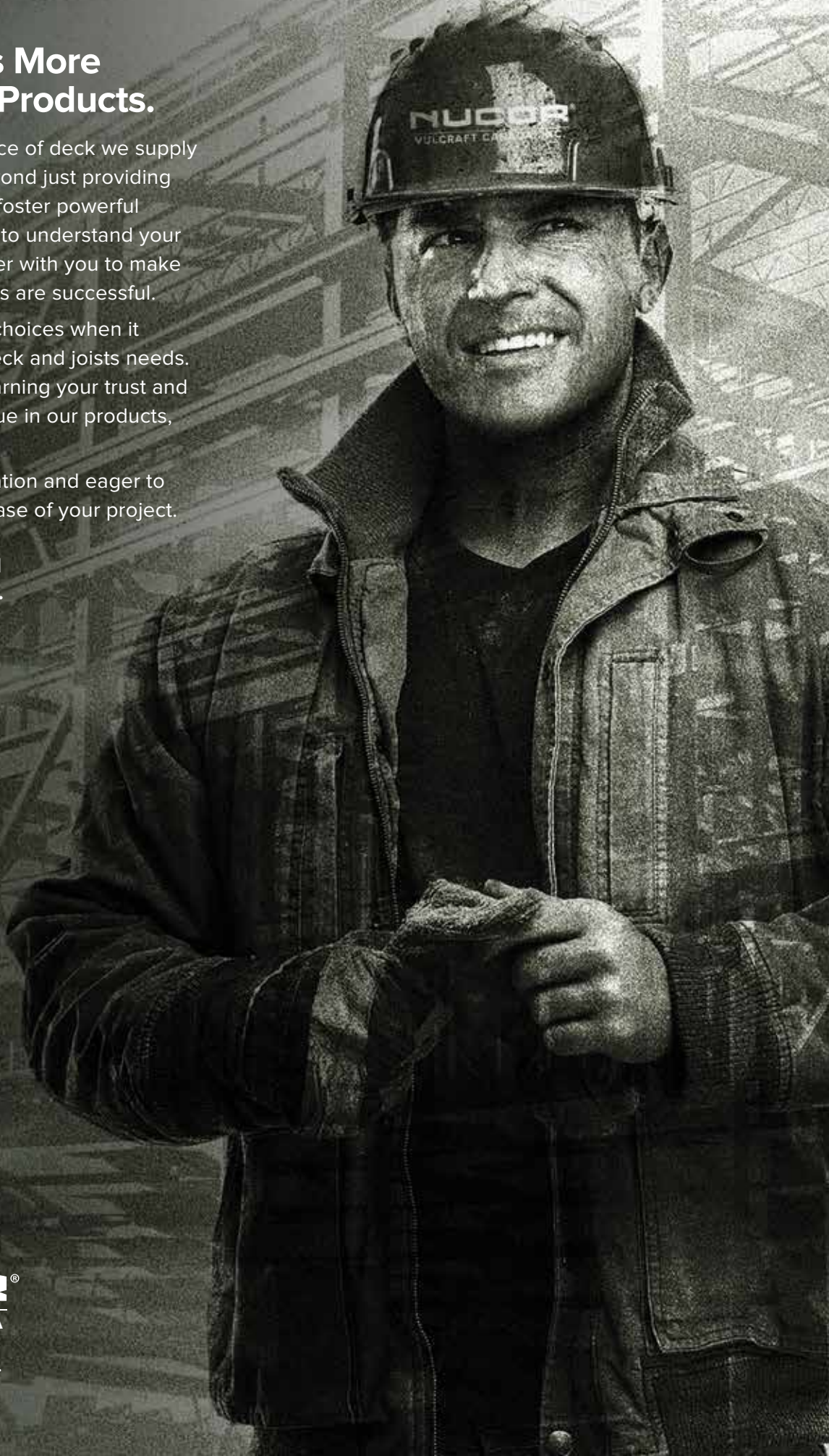
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Ed Whalen, P.Eng.
President & CEO, CISC

Uncertain and Unsettling Times

IF THERE IS ANY term I have learned to hate more than all others, it's "uncertain and unsettling times". It seems to be used on a daily basis, and I am sick of it. Sick of it because if we just followed a few simple rules, life would be a whole lot more certain and less unsettling. For example, our initial reaction in February to wear masks to protect ourselves was correct. We were smarter than we give ourselves credit for. Now 6 months later, after being told masks were of no use to the regular population, governments have flipped and are now just beginning to mandate their use in some locations. In some locations? Things would be a lot less uncertain and less unsettling if all people would simply wear a mask. And yes, that means all you folks that are having parties and visiting your extended family.

Listen, I'm not a doctor, but anyone can read the tea leaves and see what will be coming. That's right, the so called second wave. Did we even finish the first one yet? The stores are full, people are out and about and the number of people wearing masks is or was less than 50 per cent. I'm not sure what defines a wave, but unless Canadians all begin to wear masks, there will be many more surges and waves worse than the first. Just look south of the border for a glimpse of our possible future.

This is where the governments failed, in my opinion, but there is still time. All governments across Canada should pass legislation for mandatory mask usage. This should not be restricted to just hot zones, but everywhere. By doing so we will be able to continuously open all economies across Canada well past Stage 2, allowing businesses to open and remain open. Isn't this what it's all about – how to get business operating again and people back to work?

When would the masks come off? When there is a vaccine and it has been provided to everyone. This might seem harsh, but the alternates are the bumpy ride of COVID-19 waves and economic shutdowns. Shutdowns negatively impact the economy and construction in more ways than one. The federal government has racked up \$350 billion in debt from the first COVID-19 wave (as of July 2020) and they haven't even started any financial, stimulus funding on infrastructure yet. Can we assume that every wave will cost the Canadian economy \$350 billion? If that's the case, we can't afford a second wave, let alone the thought of hearing about a third one. Someone will have to pay for all this, and don't think your RRSPs and CPP will be a safe. Now that's unsettling!

So, here's my plan for governments:

1. All governments mandate the use of Canadian masks until all Canadians have received the vaccine. For you anti-vaxxers, no complaints and no health care funding after rejecting the vaccine. You should have to pay for your beliefs.
2. The governments encourage local Canadian fashion designers and textile companies to manufacture "near" N95-grade masks and make more fashionable than 3Ms. This would also stop the selfish people that won't wear masks because they believe it won't help them personally or fashionably.
3. The government sign contracts for manufacturing capacity of COVID-19 vaccine. Only Canadian! Nothing from the rest of the world. This is my gift to the anti-vaxxers.
4. All governments to allow all businesses to open using COVID-19 safety measures and, of course, Canadian masks.



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By doing these things, the governments in Canada could make our lives a whole lot more certain and keep our country above water financially.

5. Governments cease the financial aid for people sitting at home. This summer there are a lot of people that would rather sit at home and receive their government money than work. Been to a store lately? The shelves are half empty. I'm sorry, but put businesses back to work and get workers off the government payroll. Companies need workers and we don't need higher taxes.
6. Governments provide stimulus to Canadian companies to build and make things. No infrastructure money to foreign companies, period.
7. Governments put their money where their mouths are and buy everything they need from their local and Canadian businesses. Made in Canada for Canada.
8. Governments put Canadian construction and manufacturing to work to stimulate the economy by rebuilding Canada's ageing infrastructure. Offshore companies need not apply.
9. The government keep the borders open for the supply chains but closed to everyone else (yes, especially the U.S.).

By doing these things, the governments in Canada could make our lives a whole lot more certain and keep our country above water financially. I am not in the mood to spend all my retirement savings (what little I have) to pay for a mismanaged crisis. We all know what is needed, though some don't want to hear it, and governments and the population need to suck it up, stop complaining and do what is required – for certain times! **AS**





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Charles Albert, P.Eng.
Manager of Technical
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Question 1: What are the different grades of steel covered in ASTM A709?

Answer: ASTM A709 steel is used mainly for bridges and has been referenced in CSA S6 since the 2014 edition. It is an “umbrella” standard that includes the grades listed below. For each designation, the first number indicates the minimum specified yield stress, F_y , in ksi, and the second number [in brackets] in MPa.

- 36 [250] – Corresponds to ASTM A36/A36M.
- 50 [345] – Corresponds to ASTM A572/ASTM A572M grade 50/345.
- 50S [345S] – Has a maximum yield stress and a maximum yield-to-tensile ratio, useful for seismic applications. It corresponds to ASTM A992/A992M and is generally available in shapes.
- 50W [345W] – Has enhanced atmospheric corrosion resistance (weathering steel). This grade corresponds to ASTM A588/A588M.
- HPS 50W [HPS 345W], HPS 70W [HPS 485W], HPS 100W [HPS 690W] – High-performance steels with low carbon content, resulting in improved weldability, toughness and corrosion resistance. These grades are available in plates only.
- 50CR [345CR] – Stainless steel with improved corrosion resistance. It corresponds to ASTM A1010/A1010M.

When notch-tough material is required, ASTM A709 grades can be ordered with the addition of a suffix to specify the type of tension component (“T” for non-fracture critical or “F” for fracture-critical). And for each type of component, the impact testing temperature zone (1, 2 or 3) is specified.

Question 2: When loading is applied on the top flange of an unbraced beam segment, CSA S16:19 Clause 13.6.1(a) states that the destabilizing effect can be taken into account using a rational analysis. Could you provide a reference?

Answer: When loading is applied above the shear centre, a destabilizing moment ($P \times a$) is induced as the cross-section rotates during lateral-torsional buckling, decreasing the beam's moment capacity. A typical situation involving gravity loading on a floor beam is illustrated

in Figure 1(a). This generally occurs during construction while the top compression flange is not yet braced by the deck.

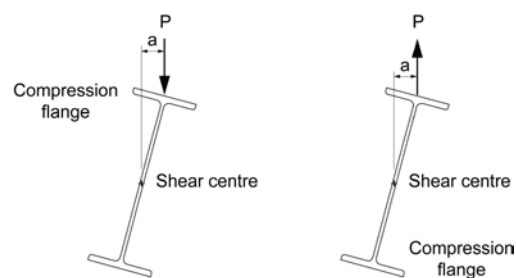


FIGURE 1
(a) Gravity Loading (b) Wind Uplift

The elastic buckling moment may be determined according to “Lateral Buckling of Beams,” J.W. Clark and H.N. Hill, ASCE Journal of the Structural Division, Vol. 86, July 1960:

$$M_u = \frac{\omega_2 \pi}{L} \left[\frac{\pi E I_y}{L} C_2 g + \sqrt{\left(\frac{\pi E I_y}{L} C_2 g \right)^2 + E I_y G J + \left(\frac{\pi E}{L} \right)^2 I_y C_w} \right]$$

EQUATION 1

where $\omega_2 = 1.13$ and $C_2 = 0.45$ for simply supported beams with uniformly distributed loading. The magnitude of “g” is the vertical distance between the shear centre and the point of application of the load, or $d/2$ where “d” is the beam depth. For gravity loading, the sign of “g” is negative when loading is applied above the shear centre.

Another common situation involves wind uplift on a roof beam, as shown in Figure 1(b). In this case, the unbraced bottom flange is in compression and the upward load on the top flange exerts a stabilizing effect. The sign of “g” in Equation 1 would then be positive, increasing the moment resistance.

Alternatively, S16:19 provides a simple (although conservative) method to account for the destabilizing effect by using $\omega_2 = 1.0$ and $C_2 = 0$ and an effective unbraced length of $1.2L$ for flexurally simply supported ends or $1.4L$ for other end restraint conditions. **AS**

Questions on various aspects of design and construction of steel buildings and bridges are welcome. They may be submitted via email to info@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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Bob Shaw

CSA S16's New Annex P on Specifying Third-Party Inspections



THE NEW CSA S16:19 Design of steel structures introduces “Guidance for specifying third party inspection of steel structures” as an informative annex. The challenge of writing such a document is one that was well-stated in the Foreword of the 1946 American Welding Society D1.0 Standard Code for Arc and Gas Welding in Building Construction, as follows,

“The importance of adequate inspection of welded work must never be overlooked. Yet, in drafting a code equally applicable to large structures supporting substantial loads and minor installations or alterations, explicit, elaborate provisions which might appear desirable in the first case become a useless burden upon the latter.

Unless a logical criterion can be devised – and none has been – to mark the dividing line between important and unimportant work, so that separate regulations governing inspection of the two can be formulated, it must rest with the sound judgment and common sense of the administrative authority whether any distinction is to be made.”

The Technical Committee on Steel Structures for Buildings is to be commended for bringing this topic forward and attempting to address inspection of fabricated and erected structural steel (materials, welding, bolting, workmanship and details) and nondestructive examination (NDE) of welds for the structural steel industry, in a manner that provides for public safety without becoming a “useless burden.”

The new CSA S16:19 Design of steel structures introduces “Guidance for specifying third party inspection of steel structures” as an informative annex.

There are several points that must be understood. The Annex is informative, not normative, and provides the engineer recommendations for consideration, the engineer being ultimately responsible for including any third-party inspection requirements into the project specification. The Annex is written using normative “shall” language in clause P2, to ease the specification-writing process for the engineers who choose to require third-party inspections and NDE. However, there are also

numerous “should” statements throughout the remaining sections that require careful consideration by the engineer, with a few “shall” requirements included.

Clause P.2.1 addresses responsibility for the third-party inspection. It mentions the engineer, the fabricator and/or erector. As this Annex addresses third-party inspection, by definition, those responsible for the work (fabricator and/or erector) should be excluded from selecting and compensating the third-party inspection agency. In addition to the

engineer, the owner of the project, whether public or private, would be an appropriate choice to select and compensate the third-party inspection agency.

Clause P.2.1 also mentions that the project specification should identify who pays for initial testing, presumed to include inspection, and who pays for follow-up or additional testing to rectify deficiencies. To address this item, the engineer should reference the CISC Code of Standard Practice for Structural Steel, clause 6.7 “Inspection of Steelwork,” that

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The importance of adequate inspection of welded work must never be overlooked.

states "The cost of this inspection and testing is the responsibility of the Client. Deficiencies in the Work of the Fabricator and/or Erector requiring re-inspection or re-testing shall have costs borne by the Fabricator and/or Erector."

The engineer selects the Inspection Class or Classes (IC1 through IC4), as addressed in clause P3. This should not be confused with Importance Categories as used in building codes. The engineer is to select the Inspection Class for the structure, and perhaps for specific elements of the structure, based on "the required reliability, the type of structure, and the type of loading for which the structure is designed." Table P.1 "Inspection classes" provides guidance to the engineer but leaves questions as to areas of high-rise buildings (over 15 storeys), grandstands, arenas, stadia and other high-occupancy assembly buildings that are not deemed "critical." For more

detailed guidance, the engineer may want to look to other globally used standards, such as EN 1993-1-1:2005/A1:2014, Annex C, which replaced the system described in EN 1090-2:2008. Another resource addressing only risk for various building types is the International Building Code (2018), Table 1604.5 "Risk Category of Buildings and Other Structures."

Table P.2 "Frequency of third-party inspections" provides recommended rates of inspections for numerous aspects of steel fabrication performed in the shop, in terms of percentage of welds, bolted connections, headed stud anchors and general conformance in terms of workmanship. It also addresses field inspections of the same items and adds inspection of steel decks, braced frames, seismic braces and seismic frames. The rate of inspection varies according to Inspection Class, ranging from "Optional" to

100 per cent. Some items, such as reviewing mill certificates, company welding certification, weld procedures and welder qualifications are simply designated "Optional" or "Yes."

Three elements of Table P.2 warrant a more detailed consideration by the engineer:

1. Under field inspection, there are recommendations for visual inspections of shop-bolted and shop-welded connections. These visual inspections are also called out under shop inspection. Fabricator preference is always for shop inspection, as the cost and time required for field corrections is substantially higher. Repeat visual field inspection may not be warranted and may not be effective.

2. For bolted connections, Table P.2 also recommends witnessing installation for a percentage of pretensioned and slip-critical connections, whether pretensioned (tightened) in the shop or field. A "witness" activity is not called for in clause 23.8 for bolting inspection procedures. Also, there is no requirement for witnessing of production welding in CSA W59, only specific items such as qualification tests. Although bolting does not require bolting supervisors and qualified bolting installers, as does welding, such programs should be considered by the industry to increase confidence in the quality of bolt installations and changing the recommended witnessing to periodic routine observation of installation techniques and use of visual inspection upon completion.

3. Note 5 of Table P.2 suggests increasing rates of inspection when the steel is "fabricated in regions where the degree of compliance with Canadian requirements is less established." This may be difficult to write into a project specification if the engineer does not have control of the source of fabrication, but a separate table of inspections tasks and rates of inspection should be considered by the engineer if the fabrication is to be performed offshore.

Table P.3 "Extent of NDE of welds" addresses specific weld types (fillet, full penetration [CJP] groove or partial penetration [PJP] groove) and joint types (butt, T or cruciform), with consideration for whether the welded joint is part of the seismic system, whether the weld is longitudinally or transversely loaded or if



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The Annex is informative, not normative, and as such, the engineer should not invoke it without clarification using clear and concise language.

the joint is in compression. The tables apply equally to shop and field welds. A percentage of groove welds are recommended to receive either Ultrasonic Testing (UT) or Radiographic Testing (RT) and to receive either Penetrant Testing (PT) or Magnetic Particle Testing (MT). A percentage of fillet welds are recommended to receive either PT or MT.

Clause P.5.4 provides a project failure rate above which the rates of NDE should be increased. The project specification should state whether this failure rate is determined by using the number of welds that fail the

NDE or using the length of the welds that fail the NDE. Also, this clause makes it clear that the contractor responsible for the welding is also responsible for any increased rates of NDE. This particular clause is within the NDE provisions, yet also uses the term "inspection" in addition to "testing." The engineer should also consider implementation of this clause to the visual inspection tasks described in Table P.2 "Frequency of third-party inspections."

Any new provision, such as Annex P, that is added to a standard is subject to a variety of interpretations by the parties involved. The

engineer should work with all parties involved in the work to ensure that there is a common understanding and should use the project specification for this purpose. As stated, the Annex is informative, not normative, and as such, the engineer should not invoke it without clarification using clear and concise language.

A CISC Commentary to this Annex should be expected to clarify the recommendations and further assist the engineer in writing the project specifications, as well as helping those who manage or perform the inspections and NDE tasks described. **AS**

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EXCELLENCE IN STEEL construction is a shared goal for our industry. We know that steel is the obvious choice for an economic, flexible, creative and sustainable infrastructure. In order to see steel continue to be the material of choice for construction – and to let others know as well – we must pay attention to how we support the next generation of steel professionals and drive innovation in our industry.

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- competitions and scholarships for architecture students to drive creative thinking in steel design,
- research grants to support innovation in steel construction,
- support for Steel Centres of Excellence, building communities of steel professionals within academia and industry, and
- development of training programs for existing steel professionals, helping to keep knowledge current and promote excellence.

As you can see, the ERC's work is focused not just on structural engineers but on multiple areas that will help drive the industry forward. Having skilled trades, architects and engineers with a solid understanding of steel construction is of critical importance – as is connecting them to our

industry and the CISC. Research and innovation are also critical as we look to make our industry more competitive and creative in the face of ever-evolving construction methods and global pressures. The ERC is also pleased to support the development of new and expanded training courses to help both novice and seasoned steel professionals upgrade and maintain their knowledge in the areas of codes & standards, design methodology, estimating, inspection and other key areas.

I would encourage you to visit the CISC's website and click on the #SupportSteel tab for a full overview of all the ERC's programs and initiatives. You can also hear – in their own words – the stories of individuals that have benefitted from the work of the ERC.

The ERC is grateful for the continued support of the CISC and our funding partners. Much has been accomplished, but we need to continue to expand the breadth of support by our industry to see our programs expand and address evolving needs. If you have a passion for supporting the next generation of steel professionals and for the future of the Canadian steel construction industry, I encourage you to consider becoming an ERC financial supporter.

I would like to acknowledge the support of my fellow council members as I move into the role of Chair of the ERC and appreciate their continued commitment to the work that we do. I would also like to recognize the work of our outgoing Chair, Mike Holleran. Mike effectively led the ERC as it worked to expand and build on our programs, and his contributions have been key in the ERC's continued success. Thanks Mike!

Please contact the CISC for more information on how you can support the work of the ERC. Working together, we can help steel be the material of choice for construction now and in the future. **AS**

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Steel Construction Modelling (Steel Detailing Technician) has been a standout program in VCC's CAD and BIM department for over 50 years. We are proud to be the only steel-detailing training program in Canada, and we have produced thousands of proficient steel detailers, many of whom are now top personnel in our industry.

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We consult with our Program Advisory Committee, made of successful steel detailing companies and fabricators in the Metro Vancouver area. With their help, we are able to keep our course content relevant and current to present industry methodology.

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UNDERSTANDING DRIVES **SOLUTIONS**

TORONTO'S NEW CIBC SQUARE HAS

Nothing is standard about this project meant to change everything

By Derek Howchin, Senior Project Manager, Walters Group and Jessica Ranalli, Project Manager,



CHALLENGES ONLY STEEL CAN SOLVE

about office buildings

Walters Group



HEAD DOWN THE Gardiner Expressway in Toronto and you can't miss the towering structure next to Scotiabank Arena: CIBC SQUARE.

It's the most advanced tower ever constructed in Canada and likely all of North America. It has been planned for more than seven years, with construction challenges that rival anything previously built in Canada. Yet the tower is just one part of this extremely complex project, which also includes a pedestrian bridge to the neighbouring arena and a one-acre park being built over the second-busiest railway station in North America. Added to all this is a second tower, which will begin construction this summer.

The first tower, 81 Bay Street, is currently under construction and will have 49 occupied stories upon completion this fall. Its twin will be 141 Bay Street (site of the current GO bus terminal), which will consist of 50 occupied stories, scheduled for occupation in 2024. Together, the towers will add three million square feet of space to downtown Toronto.

GETTING WITH THE PLAN

Real estate firms Ivanhoé Cambridge and Hines began work on the project over 10 years ago with architect Adamson Associates, resulting in a stunning design built to meet several criteria: efficient use of space, speed of construction, design-forward features, connections to transportation and innovations in environmental sustainability.

"We chose a hybrid structure with a concrete core and structural steel, primarily for speed," says John Frank, Senior Vice President (Construction) for Hines. "With steel, a building can be completed six to eight months faster."

It was in April of 2017 that CIBC chose the building to be the home of their new Canadian head office. In that same timeframe, Canadian steel icon Walters Group Inc. was brought on to work with EllisDon, the general contractor. "Walters was able to provide the engineering expertise and best practices and assist us with the design, as we knew it would evolve over the course of this project," says Andrea Quadrini, Project Manager for EllisDon.

The project's many challenges resulted in solutions that only steel could solve. With a location in one of the busiest parts of Canada, construction needed to be fast and design changes would be coming quickly, so steel was chosen to be more easily reconfigured to accommodate the changes.

The building design included floor plates that were designed with access to natural daylight and column-free floor plates, so steel was needed to span large, open spaces. Steel was also found to be the best support for glass and retain its durability over time. RJC Engineers understood early on that steel would be best for this project. "It made economic sense to go with steel," says Andrew Voth, RJC Engineers' Engineer on Record for the pedestrian bridge and rail corridor overbuild.



It's the most advanced tower ever constructed in Canada and likely all of North America.

"Steel also comes in ready for erection," adds his colleague Benoit Boulanger, the Engineer of Record for the tower, who started work as a designer for the project seven years ago. "There's more to be done beforehand, but if it were concrete, columns like those in the tower lobby would need to be at least twice as large."

Many efficiencies were developed in the planning stages that ended up saving countless days of work, one of which was the use of a custom-made formwork system that was able to climb more than six metres per cycle, which has never been done on any project in Canada.

"We worked closely with our formwork, rebar and structural steel sub-trades to develop a program that would be based on a five-day cycle," says Quadrini. "This enabled us to form, pour and strip concrete, erect steel and curtain wall based on this cycle."

AS YOU BUILD IT, CHALLENGES WILL COME

After years of planning, the project's groundbreaking was held in June 2017. Since then, even with constant communication and coordination with every possible organization involved, challenges have still happened – even when they're foreseen well in advance.

A frequent challenge is difficulty in stopping traffic to accommodate construction. The increase in road and pedestrian traffic from one of Canada's most popular entertainment venues immediately next door is also an ongoing challenge as construction finishes up.

"Every time you do one step forward, you do two steps backward, trying to please everyone," says Boulanger. "Every change affects everyone, so we need to discuss them constantly with the City, neighbours, crews, architects and tenants."

CHALLENGE HIGHLIGHT: RAIL CORRIDOR OVERBUILD

Spanning 16 rail tracks and connecting the towers will be a one-acre park – a concept that has never been done to this scale before in Toronto.

"There's no way the overbuild could have been built with anything except steel," says Frank. "If any column is taken out, the overbuild will still stay up. Enormous steel cantilevers will even be installed to support a restaurant."

"We looked at concrete and precast construction, but for the construction logistics and the economics and space, steel was really the only option," adds Voth. "In one evening, Walters' crew swung into place a 45.5-metre long-span truss, the weight of 53 mid-sized cars, over the corridor tracks. There was a very short window to stop all train traffic, and they did it perfectly."

Any time work along the tracks was required, a work plan needed to be submitted, reviewed and approved for every single operation along the long spans. "Sometimes Walters only had four hours a night to work," says Boulanger. "And other times there would be a full work crew ready to go, then a train would be delayed, so the entire evening would be cancelled."



CHALLENGE HIGHLIGHT: THE SCOTIABANK ARENA BRIDGE

The 36-metre bridge is unlike any other, with a glass wall on the south side, spanning across Bay Street to the arena.

"The Scotiabank Arena bridge has a very challenging structural system to accommodate a full-height glass wall overlooking Lake Ontario," says Tim Verhey, Executive Vice President, Engineering & Operations, Walters Group. "Our crews preassembled it at the Cherry Street docks and delivered it to the site in two sections." The bridge was installed over Labour Day weekend last year, one of the few occasions that this section of Bay Street could be shut down with minimal disruption.

THE NEXT GENERATION OF TOWER CONSTRUCTION

In addition to the stunning steel structures that require years of planning and unparalleled communication and logistics, CIBC SQUARE also represents the leading edge of technology and environmental innovation.

"This is a next-generation building," says Frank. "It's the most technologically advanced project I've ever worked on, and I've been with Hines for 35 years. It's certainly the best office building in Canada. Maybe even North America."



STEEL FACTS:

15,360 tonnes of structural steel are being used

The heaviest truss on the overbuild is 45.5 metres long, 4 metres deep and weighs **85,000** kilograms

The pedestrian bridge connecting the tower to Scotiabank Arena is just over 36 metres long and weighs **210,000** kilograms

The Bay Street canopy is **69** metres long, 14 metres wide and stands 25 metres above ground level

If you stacked all **100** trusses from the rail corridor overbuild end-to-end, they would be 2.5 times the height of the CN Tower

There are **208,000** bolts being used. If you laid them out end-to-end, they would stretch as long as the Gardiner Expressway

Highlights include:

- Engineered to LEED® platinum standards (Walters' fourth LEED®-certified office tower)
- Pursuing WELL Building Standard™ building certification
- First building in Toronto to have earned Wired® platinum certification for best-in-class technology infrastructure
- Energy costs will be reduced by 40 per cent as outlined by the National Energy Code of Canada for buildings
- Tenants will receive feedback on their energy consumption
- Each tower will include more than 500 bicycle storage racks and tenant-shower facilities

A BUILDING OF SUCCESSFUL STORIES

This project will have a positive, lasting impact on Toronto and will change the way commuters and residents enjoy downtown.

"This has been a very challenging, complex project for all involved. It truly has been an honour to be a part of the team and work amongst some of the most respected and talented individuals within the industry," shares Verhey. "It will stand out for all of those who worked on the project, as well as amongst Toronto's skyline, for many years to come."

"Once both towers have been completed, CIBC SQUARE will be one of the most iconic landmarks in the City of Toronto," concludes Quadrini. **AS**

STEEL COLUMN BASES

Under Combined Axial Load and Bi-Axial Bending

By Dr. Muntasir Billah, P.Eng. Assistant Professor, Department of Civil Engineering, Lakehead University

COLUMN BASE PLATE connections, typically consisting of a steel member welded to a steel base plate connected to the concrete base via anchor rods and grout, are commonly found in building and non-building structures. This base connection is one of the most important structural components, which acts as a transfer medium for all the forces and moments from the entire building into the foundation. Failure of these base plate connections can result in the collapse of the entire frame as it affects the ductility demand and force distribution in the structure (Grauvilardell et al., 2005). After the 1994 Northridge Earthquake, design of steel connections has been significantly revamped across North America. However, a significant amount of effort has been dedicated to the steel beam-column connections under seismic loading. In comparison, limited experimental and numerical investigations were devoted to study the behaviour of column base connections. As a result, the design of a column base connection presents several issues, which are not limited to characterization of force and deformation demands, characterization of deformation capacities of the connection components and failure mechanisms and development of desirable hierarchies of failure modes (Gomez et al., 2010).

Most of the previous studies and design guidelines are focused on base plate design under axial load and uniaxial bending moment (typically major axis). However, very often these base plates are subjected to bidirectional bending moment from lateral loads such as wind and earthquake. Current design codes and guidelines do not address the design of these exposed base plate connections under axial load and bi-axial bending. Although the columns are designed and checked under combined axial load and bi-axial bending, when it comes

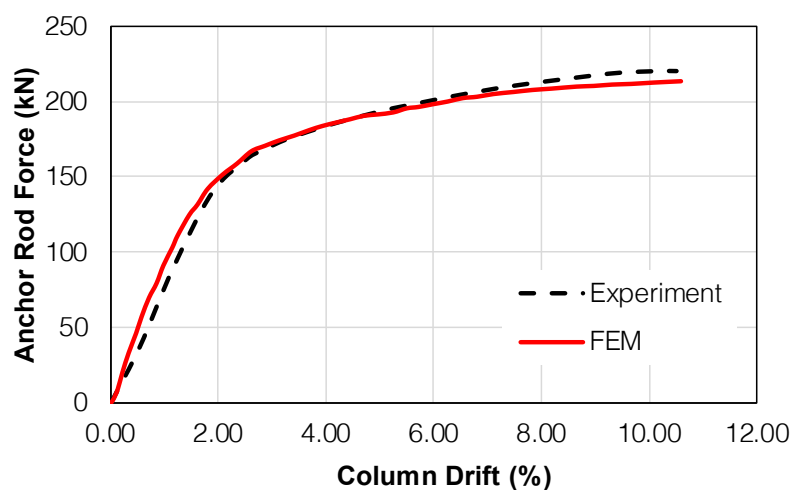


FIG. 1: Comparison of experimental and numerical results

to the base plate connection, only the axial load and major axis bending are considered. Practising engineers often adopt complex finite element methods or design them in the two directions separately, which often results in overly conservative design.

Currently, there exists a gap in the research to develop a design guideline for exposed column base plate connection under combined axial load and bi-axial bending. The overarching objective of this research is to develop a simplified design guideline for exposed column base plates under combined axial load and bi-axial bending. This will be achieved through the pursuit of the following objectives: (i) finite element simulation of base plates under combined axial load and bi-axial bending, (ii) experimental investigation of column base plates under combined axial load and bi-axial bending, (iii) comprehensive parametric study to identify the parameters that influence the

behaviour of exposed column base plates and (iv) developing interaction equations, design methods and simplified tools for practising engineers.

PRE-TEST ANALYTICAL STUDY AND FINITE ELEMENT ANALYSIS

This pre-test analytical study is conducted to investigate the performance of column base connections under axial load and uniaxial bending. The purpose of this analytical study is to select and validate a suitable modelling strategy that can mimic the experimental response of column base connections with reasonable accuracy. For pre-test analytical study, a 3D finite element (FE) model is developed using ABAQUS simulation platform. In order to validate the accuracy of the adopted modelling techniques and material models, the results from the FE model is compared with the experimental

results from Gomez et al. (2010, large scale experiment test no. 1). This specimen was tested monotonically without any axial (gravity) load and having 10.6 per cent peak column drift. Fig. 1 shows the comparison of the experimental and numerical results in terms of anchor rod force and column drift. From Fig. 1, it can be concluded that the developed numerical model can predict the experimental results with reasonable accuracy. The maximum anchor rod force is found 220.12 kN and 213.24 kN for the experimental and numerical results, respectively having a 3 per cent difference between them.

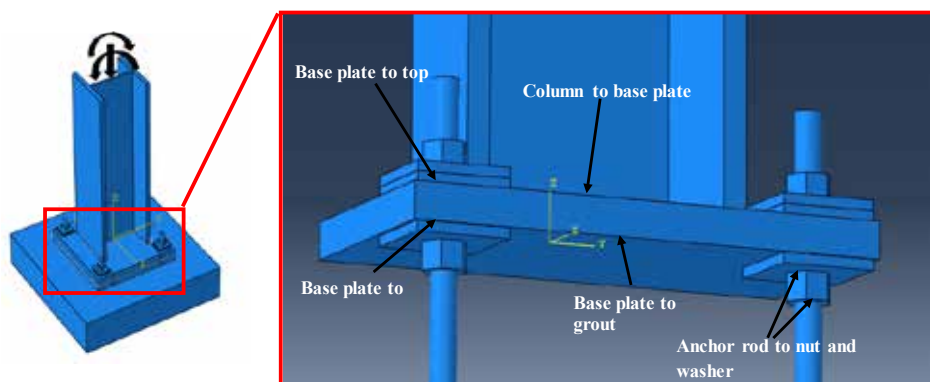


FIGURE 2: Overview of the finite element model

BASE CONNECTION RESPONSE UNDER UNIAXIAL AND BI-AXIAL BENDING

As the first phase of this research, column base connections are numerically investigated under combined axial load and bi-axial bending. Fig. 2 schematically represents the FE model and the applied loadings. Tie constraints are provided

between the column and base plate, anchor rod, nut and washer and grout and concrete, since they have monolithic properties. Surface-to-surface contact interactions

are defined between the interface of base plate-grout, base plate and both the top and bottom washer and anchor rod-base plate with finite sliding formulation. Two different

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interaction properties are defined for these surface-to-surface interactions. Contact details between the elements are also shown in Fig. 2.

Column behaviour: To represent the gravity load, an axial load equal to 30 per cent of the column capacity was applied as an axial compression. For uniaxial monotonic loading, a lateral drift of 10.6 per cent was applied along the strong axis direction of the column, whereas for bi-axial loading, additional 4.9-per-cent drift was introduced towards its weak axis direction in addition to the axial compression. The column subjected to bi-axial loading suffered considerable out-of-plane local buckling near its base, compared to uniaxial loading. This occurred due to localized compression, as well as not having sufficient thickness of the web and flanges to resist weak axis bending. In Fig. 3, it can be seen that the stresses are concentrated near the column base when subjected to uniaxial loading. However, for bi-axial loading (Fig. 4), the stresses are spread along the column's height up to half of its total length due to an angular resultant displacement caused by simultaneous bidirectional loading. Maximum stress of the column is found to be three per cent higher for bi-axial loading compared to the uniaxial loading.

Base plate behaviour: Identifying yield lines in the base plate of deformed base connections is important and can be a difficult task. The current AISC design guideline assumes the yield line forms in parallel with the flange of the column. A detailed experimental program conducted by Gomez et al. (2010) revealed that yield lines develop in inclined patterns under uniaxial bending. Similarly, the FE model showed development of inclined yield lines on the tension side of the base plate (Fig. 5a) under uniaxial loading. This can be attributed to the resistance to the tensile force by the anchor rods causing a curve-shaped deflection at the tension end. A straight yield line is formed beneath the column flange on the compression side of the plate

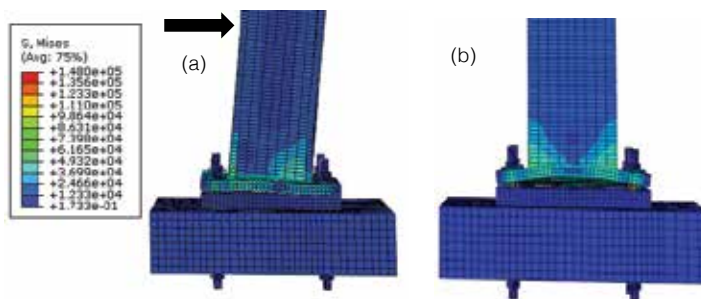


FIGURE 3: Column stress (Von mises) along (a) weak axis and (b) strong axis under uniaxial loading

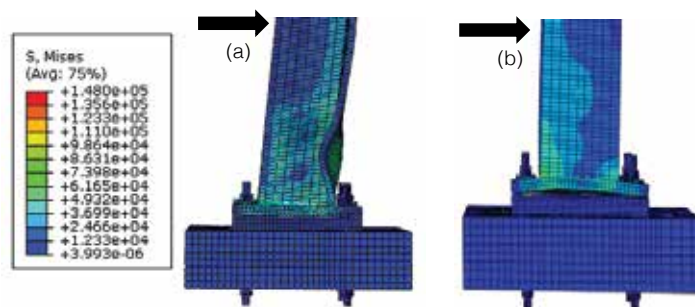


FIGURE 4: Column stress (Von mises) along (a) weak axis and (b) strong axis under bi-axial loading

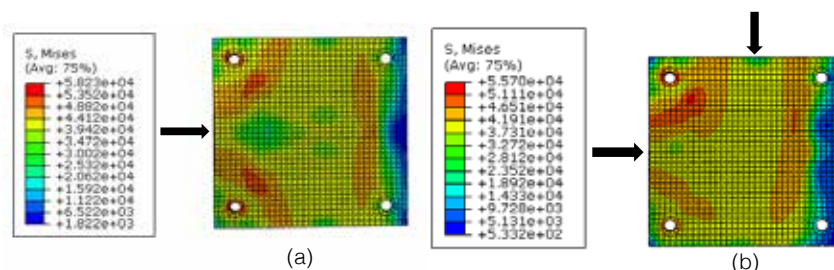


FIGURE 5: Base plate behaviour under (a) uniaxial and (b) bi-axial loading

“After the 1994 Northridge Earthquake, design of steel connections has been significantly revamped across North America.”

(Fig. 5a). Under bi-axial loading, yield line is critical near the anchor rod which is in tension during both loading direction (Fig. 5b). There is also a formation of yield lines under the column flange across the width on the other side of the base plate extending to the edge of the base plate (Fig. 5b). Maximum stress is found to be four per cent lower in case of bi-axial loading compared to uniaxial loading since there is alternative tension-compression behaviour under bi-axial loading.

PLANNED EXPERIMENTAL PROGRAM

With the financial support of the CISC, an extensive experimental program will be conducted on six reduced-scale exposed-type steel column base plate connections in summer 2020 to establish the design methods and interaction equations for column base connections under combined axial load and bi-axial bending. Two steel HSS columns and four steel wide flange columns (W-section) will be welded to the centre of the base plate. The column size, as well as width-thickness ratio, will be selected to prevent yielding and local buckling of the column before failure of the base connection. In the experimental study, the thickness of the baseplate for the HSS columns will be kept constant, while two different base plate thicknesses will be considered for wide flange columns. The proposed test setup is shown in Fig. 6.

NEXT STEPS

The next step in this ongoing research is to perform experimental investigations on column base connections under combined axial load and bi-axial bending. This research will be critical to potentially improve base plate and anchor rod design with accuracy and economy under multi-axial loading. All the specimens have been designed and are currently being fabricated. Detailed experimental testing will be done in summer 2020.

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Gomez, I.R., Karvinde, A.M., and Deierlein, G.G. (2010), "Exposed Column Base Connections Subjected to Axial Compression and Flexure," Technical Report submitted to the American Institute of Steel Construction, AISC, Chicago, Illinois. **AS**

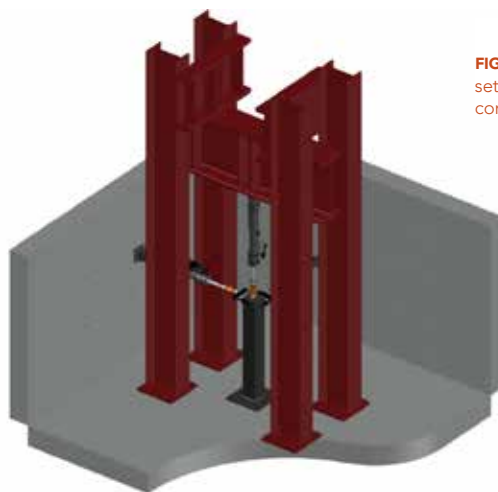


FIGURE 6: Proposed experimental setup for column base plate connection testing



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STEEL BRIDGES AND B

Evaluation Methods for Fatigue Life and Toughness Assessment of

By Michelle Y.-X. Fan, BAsC, University of Waterloo; Prof. Bertram Kuehn, PhD, Technische



RITTLE FRACTURE

Steel Bridges

Hochschule Mittelhessen, Germany ; Prof. Scott Walbridge, PhD, University of Waterloo



AMONG THE COMMON FAILURE MODES for steel bridges, brittle fracture is a major concern to structural engineers as it has significant consequences in terms of safety and cost. Although occurrences are rare in the present day, it is well known that they occur without warning and may lead to the sudden closure of a bridge, loss of service, expensive repairs and/or loss of property or life.

One special case of brittle fracture is known as constraint-induced fracture (CIF), which may occur under a tri-axial state of stress, when there are multiple intersecting welds, such as a web-flange-stiffener connection without a sufficiently wide web gap. In recent years, several cases of CIF in bridges, notably the US 422 Bridge (built 1965) and Hoan Bridge (built 1972), have demonstrated the need for a review of current practices in identifying CIF-prone details.

In Canada, steel bridge fracture is a major concern due to our harsh climate, which, if the toughness properties are improperly specified, could put many steels on the lower shelf of the toughness-temperature curve. Based on a review of recent research conducted in Europe and the United States, it is observed that more sophisticated approaches have been developed in terms of modelling and understanding brittle fracture in existing and new bridges than the ones currently in use in Canada.

This project aims to increase the Canadian state of knowledge surrounding brittle fracture and CIF by assessing the reliability level of the current toughness provisions in the Canadian Highway Bridge Design Code (CSA S6)¹ and developing improved assessment tools to evaluate the brittle fracture risk in bridges.

The brittle fracture provisions within Eurocode 3: Design of steel structures (EN 1993-1-10)² consist of two methods of analysis, a simplified method using design tables and a fracture-mechanics-based method. Both the Eurocode simplified method and CSA S6 (Section 10.23) design for brittle fracture use the minimum service temperature of the location of interest to determine the appropriate steel grade, subgrade and CVN test requirements: test temperature [°C] and energy absorbed [J]. However, aside from the service temperature, the similarities end there. The Eurocode takes into account numerous other factors such as plate thickness, yield strength of the material, stresses on the component, radiation losses, member shape and dimension, safety allowances, strain rate and cold forming (if applicable). A very simplified summary of the Eurocode steps are as follows:

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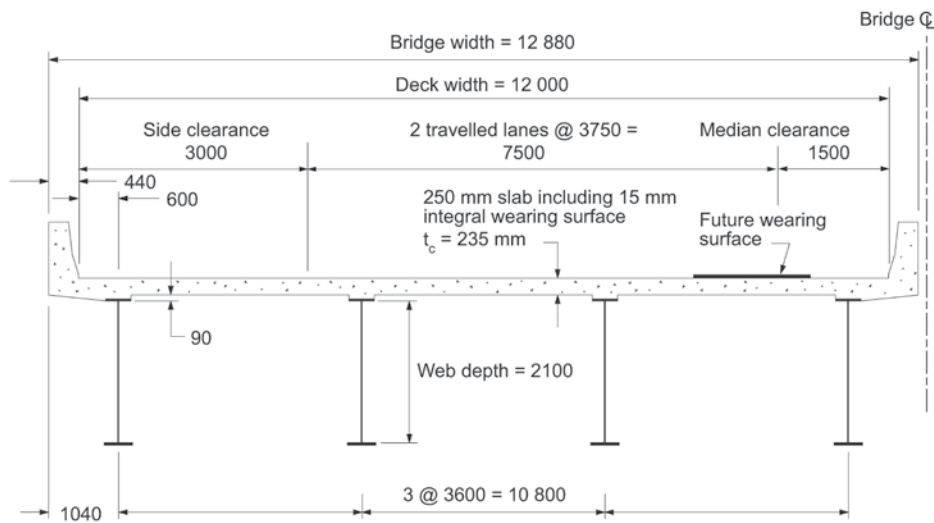


FIGURE 1: CISC Steel Bridge Design Example 1.

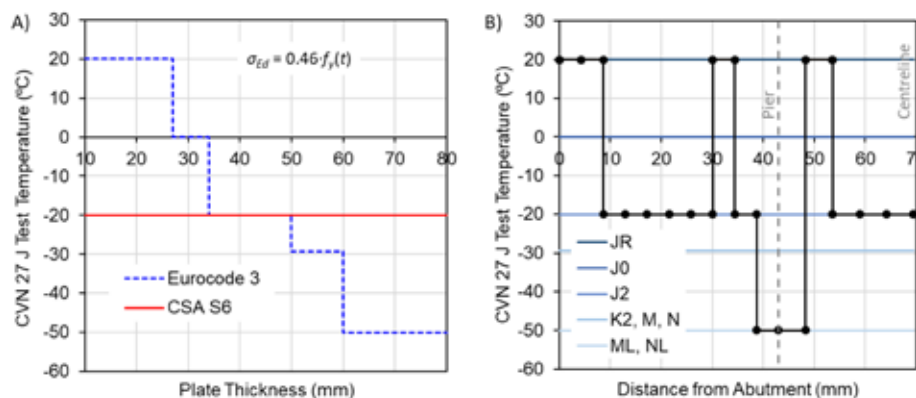


FIGURE 2: Comparison of CSA S6 [1] and Eurocode [2] brittle fracture provisions.

- A thickness-adjusted yield strength $f_y(t)$ is calculated from the nominal strength.
- The maximum stress on the component is calculated using an SLS load combination.
- The maximum stress level is expressed as a proportion of adjusted yield strength (e.g. $0.75 \cdot f_y(t)$)
- The reference temperature is determined from the minimum extreme temperature with a 50-year return period and radiation losses considered, along with temperature adjustments for member shape and dimension, safety allowance, strain rate and degree of cold forming.

- A steel subgrade is then selected from the Eurocode design table, based on the reference temperature, stress level and plate thickness. Conversely, if the subgrade is known, the maximum allowable plate thickness for the component can be determined.

When the stress level and/or reference temperature fall in between two columns in the design table, interpolation may be used. Typically for rapid assessments, it is acceptable to take a more conservative approach by applying the next-highest stress level or the next-lowest temperature on the chart and determining the toughness requirements based on this information.

Although the Eurocode is more complex and takes more factors into consideration, a question of interest is whether the complexity takes a more or less conservative approach than the current Canadian provisions and, even if so, whether this increase complexity is justified.

To answer these questions, a basic analysis was performed to compare the Eurocode with the CSA S6 brittle fracture provisions, using the CISC Straight Plate Girder Bridge Design Example 1 as a common “base case” scenario (see Figure 1). This bridge has four x three span continuous girders (43 m / 53 m / 43 m) with flange and web plate thicknesses varying along the span. For comparison purposes, the steel grades chosen are European S355 and Canadian W350, which have similar yield strengths of 355 MPa and 350 MPa, respectively.

The first part of the analysis involved varying plate thickness at the location of maximum positive stress along the girder (Span Reference 5), while holding all other parameters constant to study the effects of different thickness on the toughness requirements between the two codes. Note that instead of applying a minimum mean daily temperature like CSA S6, the Eurocode requires the input of a minimum extreme temperature with a 50-year return period. Historical weather data

References:

1. CSA (Canadian Standards Association). (2019). Canadian highway bridge design code. CAN/CSA-S6-19.
2. European Commission. (2006). Eurocode 3: Design of steel structures. EN 1993.
3. Kuehn, B. (2005). Beitrag zur Vereinheitlichung der europäischen Regelungen zur Vermeidung von Sprödbruch (Contribution to the standardization of the European regulations to avoid brittle fracture), PhD Dissertation, TU Aachen, Germany.

for Waterloo, ON, was collected accordingly, and a Gumbel distribution is fitted to the data.

While the Canadian W350's CVN toughness is 27 J at -20 C, the different subgrades of the European S355 do not all correspond to a 27 J CVN requirement, rendering a direct toughness comparison difficult. The non-27 J subgrades are converted to an "equivalent 27 J," with an adjustment made to their test temperature using an equation developed in [3] to obtain a more direct comparison between the steels. Figure 2(a) shows the variation in the required CVN test temperature corresponding to the required steel grade/subgrade in the two design codes for varying plate thickness.

The toughness requirements in CSA S6 are independent of plate thickness, as represented by the horizontal line, and are more conservative than the Eurocode for thinner plates (below 34 mm), resulting in a lower CVN test temperature requirement for 27 J. Both codes have the same toughness requirement in the middle range of plate thicknesses, and the Eurocode becomes more conservative for thicker plates (above 50 mm). Although this analysis is specific to a particular loading scenario for the CISC Straight Plate Girder Bridge Design Example 1 and the specific climatic data for Waterloo, ON, the step function trend is similar for other European steel grades, such that toughness requirements for thicker plates are more rigorous.

A second comparison is done for the entire span of the CISC Design Example 1. While the first analysis is performed for a single location on the bridge (location of maximum positive stress), this second analysis (see Figure 2(b)) looks at the changes in steel grade/subgrade requirements using the Eurocode where the stress level and plate thickness are both varying along the length of the bridge.

Depending on the stresses and plate thickness, the toughness requirement along the length of the bridge can vary significantly, based on this analysis using the Eurocode. This type of graph can help designers determine the toughness requirement to properly specify steel grade. Although the toughness requirement varies along the length, for ease of construction, it is generally preferable to use the same grade along the entire bridge. In this example case study, the pier is the location of maximum negative stress and governs the fracture toughness requirement for the entire bridge. It is observed that the locations of lower toughness requirement (JR) are generally near the inflection points.

One possible use for this type of output might be to permit a lower toughness level at positions along the span other than the critical one for assessment purposes, such as assessing a point along the bridge span where a fatigue crack has been detected or the structure has been damaged – e.g. by vehicle impact – or has a fatigue or CIF-prone detail present. Further work would be needed, however, to explore the value and potential pitfalls of using such an approach for assessment purposes. Other future work planned for this CISC-supported research project will include such things as comparing the results obtained so far with results obtained using the Eurocode fracture mechanics approach, as well as incorporating reliability considerations in the comparison of these various design approaches. It is expected that this work will lead to proposals for possible improvements to the brittle fracture approach currently specified in CSA S6 or additional provisions for use in bridge assessment. **AS**



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FEATURE

THINKING INSIDE THE BOX

Steel curing structure at Calgary's Composting Facility

By James Peters





WHEN THE CITY of Calgary was looking for expertise on pre-engineered buildings for its new compost facility, the project's general contractor (GC) included Behlen Industries, a manufacturer of steel building systems, on the tendering list. Behlen's buildings are expandable, energy efficient and straightforward to construct – and they also retain architectural flexibility. Everything, in fact, the city facility was looking for.

The company was first approached by the joint venture consortium of Nason Contracting Group (a subsidiary of Bird Construction) and Maple Reinders in June 2015. The two companies together became the general contractor for the composting facility, and Behlen and the GC solidified the plans over the next several months. Steel was first shipped to the site from the Behlen plant in Brandon, Manitoba in late 2015, and the building was virtually completed by August 2017. As it now stands, the Calgary Composting Facility remains the largest of its kind in Canada.

Behlen's Vice-President of Engineering and Innovation, Pat Versavel, says, "I'm sure Behlen was originally included in the tendering process because of our previous involvement in the City of Hamilton's compost facility. These kinds of plants don't typically use pre-engineered steel buildings, but we were involved in a successful project in Hamilton, and that was obviously to our favour."

Behlen manufactures two distinct building systems – rigid frame and frameless – and the two have their separate advantages. Which system is going to be the most suitable and cost-effective for the client depends on a host





of criteria, such as size, purpose, function and the site itself. Rigid frame systems excel in energy efficiency, condensation control and noise reduction, and the construction process is engineered to create virtually no waste, keeping the building both cost-effective and environmentally sustainable. The technique was used for the Calgary project.

The Calgary Composting Facility produces high quality compost from both food and yard waste collected from the city's green cart

program and dewatered solids, a nutrient-rich by-product from wastewater treatment. There are three principal buildings that make up the composting facility: the main building, curing building and storage building. Taken together, the entire compound is 521,000 square feet and processes upwards of 145,500 metric tons of residential food and yard waste and dewatered biosolids every year. Behlen's component for the compound was the curing building, which is where the composted

material enters and remains for 21 days. Pipes in the floor draw air in and through the material to further enhance break down. After curing, the material is moved to the storage facility. Through a series of defined stages, compost is ultimately sold back into the market in bulk, with proceeds helping to reduce the processing cost and lower the Green Cart program fee.

Versavel says, "At the beginning of this project, the GC only had some rough outlines in terms of what they wanted to do. So, we came up with some preliminary drawings and presented a design solution. In the beginning, we were only given the basics, such as that the roof would need a slope, but there weren't any specifics as to pitch and degree. So, we gave the GC a concept they could work with, and there was a lot of back and forth throughout before any steel was manufactured. But that's not typically how we work. Behlen usually receives specific instructions from a GC on what's required for a project, and that's what we provide."

In spite of the atypical start on the Calgary project, the final steel structure ultimately worked. Versavel adds, "Our company designs and fabricates steel shells but we always work from the foundation up. We provide the materials and ship them in parts to the site. We don't do any electrical or mechanical or actually put up the building. So yes, the compost plant was challenging and occasionally frustrating to work on but



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RECIPIENT: BEHLEN INDUSTRIES **OWNER:** CITY
OF CALGARY **ARCHITECT:** STANTEC **STRUCTURAL**
ENGINEER: STANTEC **GENERAL CONTRACTOR:** MAPLE/
NASON CHINOOK DBJV **STEEL ERECTOR:** TRU-STEEL INC.

"Looking back at the project, I know everyone on the team feels the same way. It was a challenging and unusual project for us, but we're all glad we did it."

Pat Versavel, Behlen Industries

ultimately satisfying when it all came together."

Uncertainties in the building were less concrete. Versavel says, "The structural framing on this project was very long and not very tall, which are both unusual conditions. And because of the building's overall size and width, you have a higher number of structural members. That becomes a fabrication challenge, because your tolerances have to be more precise, obviously. As you add up all the pieces to form the assembly, you can't be out with your measurements as you progress down the line."

The effect of creep was also a challenge. The curing building was large in square footage but short in height. So, if tolerances weren't as precise as they need to be, a millimetre at the front would show up as a metre at the back end. A taller structure, for example, allows for more wiggle room, and that meant the team at Behlen's fabrication plant had to maintain tighter tolerances.

Versavel says, "Remember, we're not exactly building a Swiss watch with our structures – they are steel buildings. But with the compost facility, everything had to be seal-welded and the steel was hot-dip galvanized. Those were all extra layers of complexity. What was also different about this job was to introduce a concept as a solution. That's not typical for us. The GC may have had other solutions, but they liked ours and made some changes to it as we progressed, but they didn't have much developed on their side. So, the two things were going on in tandem."

In April 2019, Behlen Industries received the award in the Pre-Fabrication category at the CISC Manitoba/NW Ontario Steel Design Awards Gala in Winnipeg. Versavel concludes, "Looking back at the project, I know everyone on the team feels the same way. It was a challenging and unusual project for us, but we're all glad we did it. This kind of a build only broadens your horizons as a company, and you learn from it. Ultimately it was very satisfying, and the award was just the icing on the cake." **AS**



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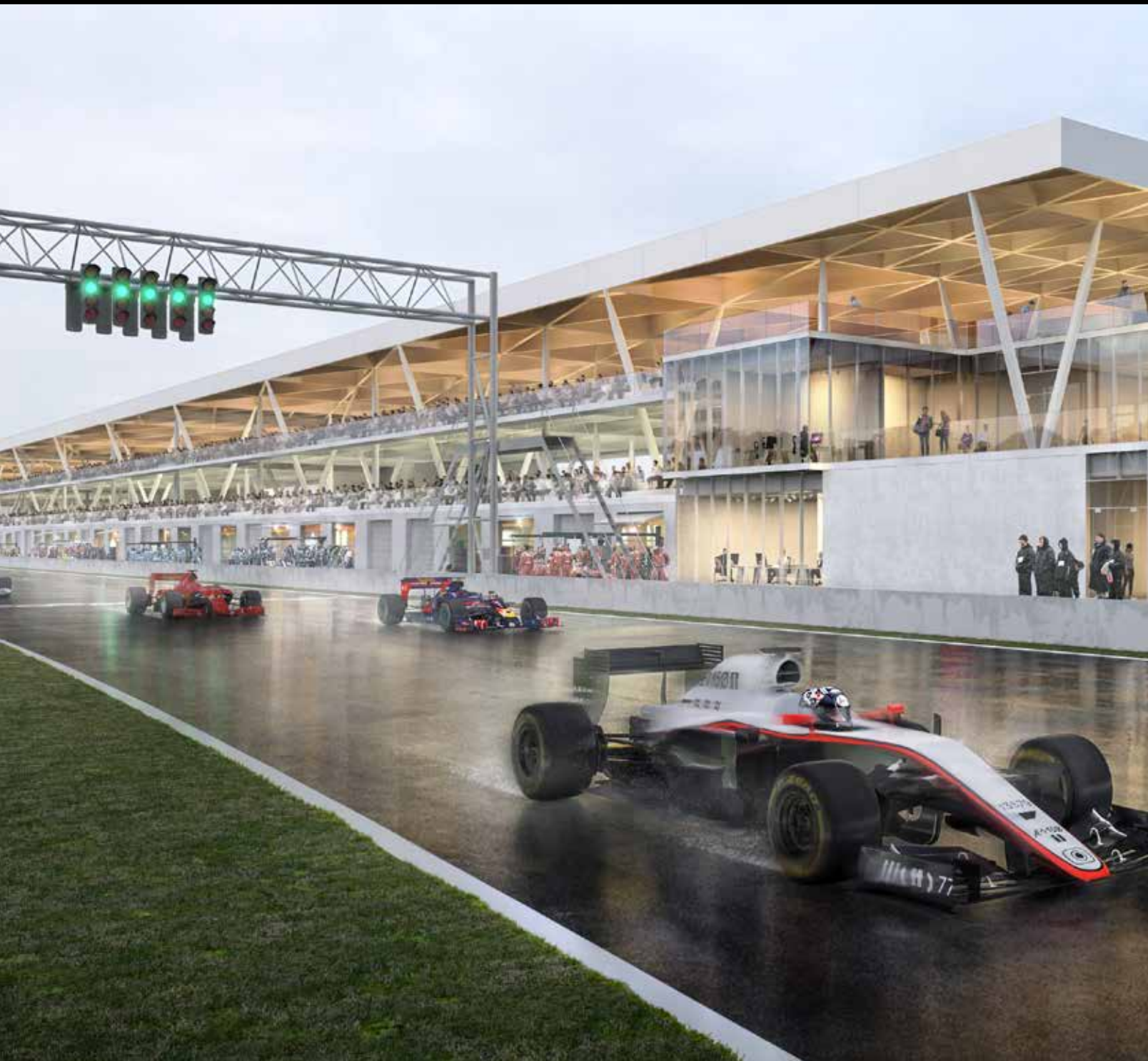
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FEATURE

CANADIAN FORMULA

Infrastructure refurbishment

By Hellen Christodoulou, Ph.D. Eng., B.C.L., LL.B., M.B.A., Regional Director, Quebec, Canadian



1 GRAND PRIX

Institute of Steel Construction (CISC-CISC)



THE CIRCUIT GILLES-Villeneuve covers most of Montréal's Notre Dame Island and is an integral part of Parc Jean-Drapeau and its annual activities. The Canadian Formula 1 Grand Prix has been held there every year since 1978. This major event garners international visibility for both Montréal and Parc Jean-Drapeau.

NEW INFRASTRUCTURE AT THE CIRCUIT GILLES-VILLENEUVE
As part of the 2015 to 2029 renewal of the Canadian Grand Prix Agreement, the Société du Parc Jean-Drapeau (SPJD) committed to contributing to the refurbishment and expansion of the Circuit Gilles-Villeneuve infrastructure.

The objective of the project was to upgrade the equipment in the paddocks, which was built in 1988, to the requirements of the Fédération Internationale de l'Automobile (FIA) and the Formula One World Championship (FOWC) while increasing the capacity of the loges above the garages to 5,000 people, compared to 1,800 in the former building. Several structures had exceeded their useful lifespan and required upgrading.

The new construction inevitably required the demolition of the existing tower and garages, as well as the reconstruction of the building that houses, among other things, the team spaces, the control tower and the Paddock Club. In addition to meeting budget requirements, the mandate's main challenge was absolute adherence to the schedule prioritizing planned events.

Jonathan Binette, Eng., mentored by Alexandre Poulin, Eng., and Associate Partner (CIMA) acted as structural designer in addition to

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PROJECT TEAM

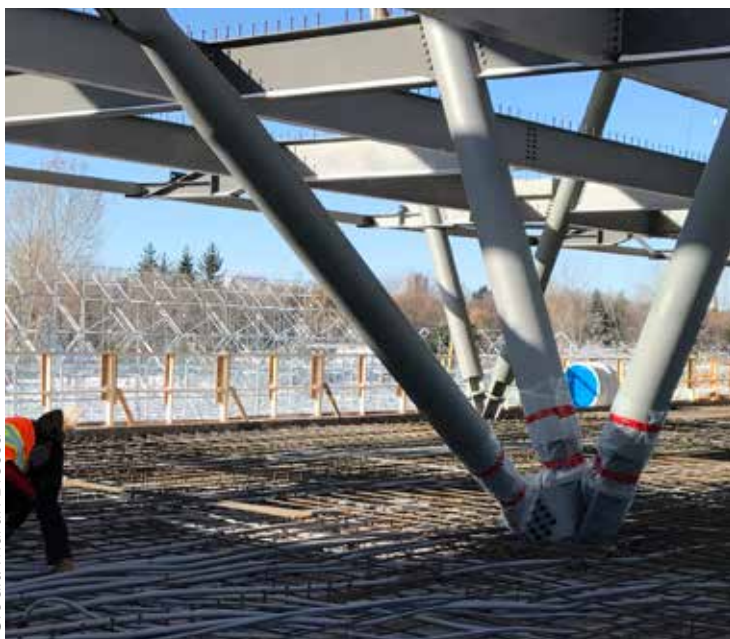
CLIENT: SOCIÉTÉ DU PARC JEAN-DRAPEAU (SPJD) **STRUCTURAL ENGINEERS:**

CIMA+ **ARCHITECT:** ARCHITECTES FABG **GENERAL CONTRACTOR:** GEYSER GROUP

GIRDERS: CANAM GROUP **QUALITY CONTROL:** FNX-INNOV



Two Ironworkers completing bolting operations at a splice location of two track elements.



Credit: Martin Brossu

Major projects: Espace Paddock – An international-scale experience, just steps from downtown Montréal

providing heightened oversight of the Canadian Formula 1 Grand Prix infrastructure refurbishment. Very specific constructability challenges had to be considered in the design, such as absolute adherence to the schedule to limit any impact on event activities, a small, enclosed workspace between the Olympic Basin and the racetrack and structural work primarily undertaken in winter conditions. Binette opted for a steel structure design, which allowed the team to meet both the technical challenges and the project's very tight deadlines. As compared to other construction materials, this choice afforded the Société du Parc Jean-Drapeau a more cost-effective solution.

The new paddocks at the Circuit Gilles-Villeneuve houses the garages of the various F1 teams, control tower, staff premises, corporate loges, media spaces and podium. The three-storey building, with a surface area of more than 21,500 square metres, comprises a ground-floor structural slab on screw piles, steel framing covered primarily by composite cement slabs on the upper floors and glued-laminated timber roofing. A design using concrete slab on metal deck and girders was favoured in the office area due to smaller spans. The trusses were supplied by CANAM Group Inc.

With a length of more than 300 metres, the building was separated into four structurally independent sections with connections allowing for longitudinal movement. Sliding steel beams allow the expansion joints separating these sections to move.

BUILDING SEPARATED INTO FOUR SECTIONS

- Longitudinal expansion joints to mitigate the effects of thermal expansion
- Fabrication of exposed steel meets AESS requirements and specifications
- Composite beams on upper floors to maximize the capacity of steel beams
- Exposed columns in HSS with tapered ends
- Plan design: Final plans and specifications within six months
- Construction of the paddocks: Nine-month construction timeline

A building that aims to be multi-purpose, conventional in its choice of materials and modern in its envelope. While blending almost seamlessly into the Parc's landscape, the refurbishment of the Circuit Gilles-Villeneuve paddocks includes:

- An accelerated project completed in record time – ten months between two Canadian Grand Prix events,
- A concern for the surrounding ecosystem and users of nearby facilities, including the Olympic Basin and Jean-Doré Beach, as well as cyclists on the circuit, and
- Carefully selected construction methods, including factory-prefabricated structures requiring little assembly on site.

ENTERPRISING DESIGN BY ARCHITECTES FABG

The bold design of Architectes FABG presented structural engineers with several significant challenges. Among them are very long spans (more than 12 metres), numerous overhangs at each end of the building, leaning tubular steel columns with conical tops slanted on two floors, precast concrete slabs on tiered beams, giant



“We are very proud to have been able to showcase the expertise of Canadian steel artisans on a project that has incredible international visibility.”

ALEXANDRE POULIN, Eng. Associate Partner / Project Engineer / Building-Structure, CIMA

screen mounts on the roof, solar panel mounts and virtually no margin for error with respect to completely masking the steel connections in the wooden beams.

Composite steel beams were used in the design for their extremely broad span. Adding Nelson studs on the upper cap makes composite action between the steel beams and the concrete slab feasible. This optimizes the structure and minimizes the weight, thus cutting costs.

Designing the connections between the triangulated timber frame and the tubular steel columns was also a puzzle. The engineering and modelling of details required a high degree of optimization and precision. To meet both aesthetic and fire-resistance requirements, the steel joining plates had to withstand significant stress while remaining hidden inside the wood. Differences in the fabrication and installation tolerances of the two materials also had to be taken into account in the design of the connections.

An efficient and rigorous coordination process had to be implemented rapidly during the execution phase to allow the suppliers of wood and steel structures to quickly begin production.

A bright, modern and private space of astonishing magnitude in a unique location only five minutes from downtown Montréal. Facilities specially intended for the Formula 1 racing world, offering visitors from near and far a spectacular show and providing competitors and their teams a flawless experience.

Now occupying three floors, the building's layout has been completely reimaged to meet everyone's needs: team staff, media and commentators, but especially crowds eager for thrills in a state-of-the-art space!

On the first floor are more spacious garages for sports teams, plus a hospitality section in the back completely redesigned for added comfort.

The new paddocks will be able to accommodate technical garages and up to 13 teams, each of which will have two front entrances

for single-seaters, drivers and technical teams, as well as a service entrance at the back of the building for equipment or quick access to the hospitality area.

Designed with no fixed dividing wall, the garage space is modular to best meet the team's needs during each Grand Prix du Canada event. Temporary partitions will be used to break up the space according to the technical needs of this ever-evolving sport.

When designing and drafting the plans, it was essential for the building's architecture to seamlessly blend into the natural surroundings of the Parc, with an emphasis on local know-how.

The elegant structure and choice of materials of the new paddocks enhance the Parc's unique environment: its insular nature, proximity to the city, the legacy of Expo 67 and, of course, the mythical Circuit Gilles-Villeneuve.

During the demolition of the old paddocks and construction of the new paddocks, the FNX-INNOV team of soil and material engineers and building specialists was contracted to monitor and control the quality of materials. Throughout the project, specialists provided ongoing lab-based quality-control services for all materials, including cement concrete structures, embankments, steel and timber structures and monitored the building's waterproofing and roofing work for optimum performance.

“This modern, bright and versatile building, a new iconic addition to the City of Montréal, is a major project that positions FNX-INNOV as an expert and beacon in the field.” FNX-INNOV

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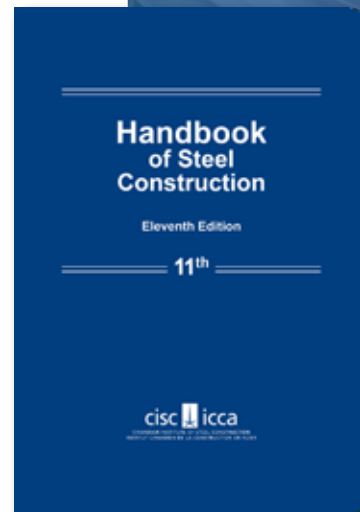
The professionals who worked on the project therefore had a rich source of inspiration and judiciously opted for characteristics putting Quebec Engineering front and centre. **AS**

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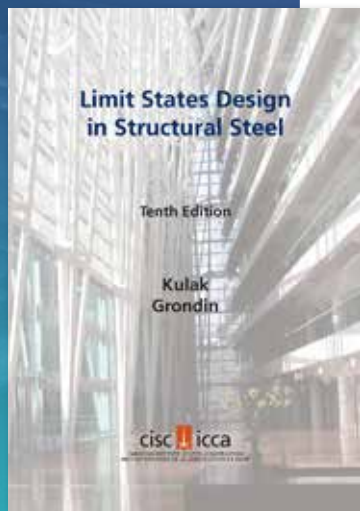
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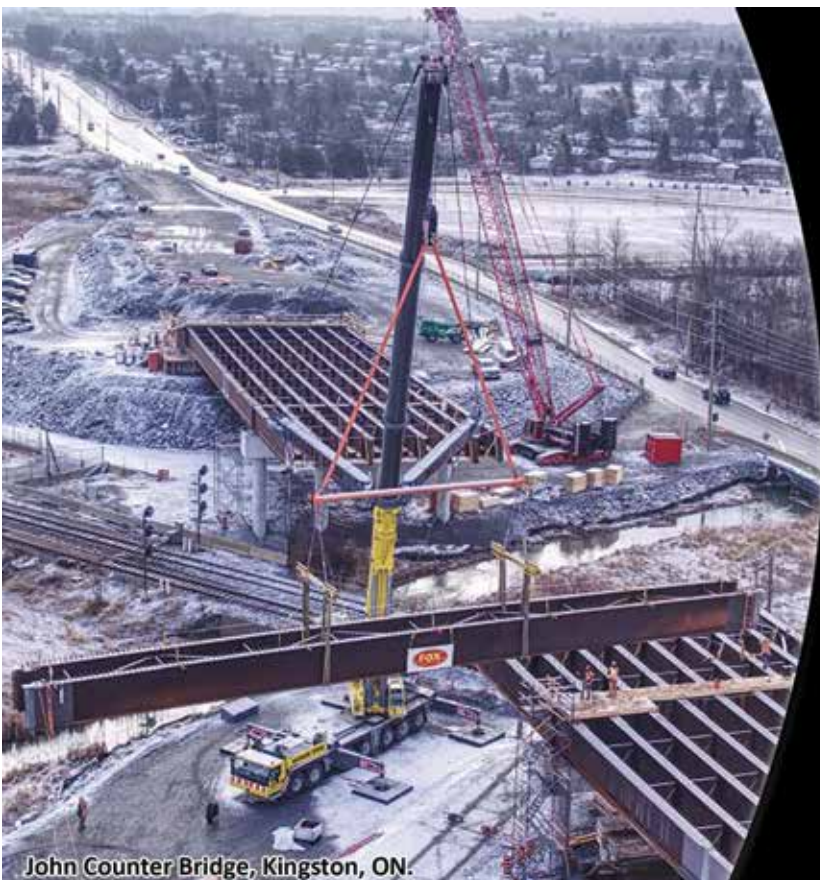


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 www.tecnometal.qc.ca

ONTARIO

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ACL Steel Ltd. S
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 www.adsteel.ca

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 www.akalsteel.ca

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 www.algonquinbridge.com

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 www.bensonsteel.com

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 Concord, ON 905-761-6155
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 Oakville, ON 905-829-8588
 www.coremetal.com

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 www.canam-construction.com

Central Welding & Iron Works B, Br, P, S
 North Bay, ON 705-474-0350
 www.centralwelding.ca

Cooksville Steel Limited - Kitchener Plant S
 Kitchener, ON 519-893-7646
 www.cooksvillesteel.com

Cooksville Steel Limited - Mississauga Plant S
 Mississauga, ON 905-277-9538
 www.cooksvillesteel.com

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 Newmarket, ON 905-836-6612

Fortran Steel Contracting Ltd. S
 Ottawa, ON 613-821-4014
 www.fortransteel.com

G & P Welding and Iron Works P, S
 North Bay, ON 705-472-5454
 www.gpwelding.com

Gensteel
Division of Austin Steel Group Inc. S
 Brampton, ON 905-799-3324
 www.gensteel.ca

Hans Steel Canada B, Br, P
 Stouffville, ON 905-640-1000

IBL Structural Steel Limited B
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 Strathroy, ON 519-518-0246
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 Sarnia, ON 519-344-3939
 www.lambtonmetalservice.ca

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 Brampton, ON 905-458-8850
 www.lorvinsteel.com

M&G Steel Ltd. S
 Oakville, ON 905-469-6442
 www.mgsteel.ca

M.I.G. Structural Steel
(Div. of 3526674 Canada Inc.) S
 St-Isidore, ON 613-524-5537
 www.migsteel.com

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 www.marianimetal.com

Mirage Steel Limited S
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 www.miragesteel.com

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 www.quadsteel.ca

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 www.tradetech.ca

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www.iwlsteel.com 306-242-4077

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www.iwlsteel.com 306-242-4077

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Russel Metals Inc. [Winnipeg]
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www.varsteel.ca 780-955-1953

VARSTEEL Ltd. [Saskatoon]
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www.dtechenterprises.com 604-536-6572

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| Husky Detailing Inc. Zurich, ON www.huskydetailing.com | B 226-219-6293 | | |
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| JITECH ASSOCIATES, INC. Montreal, QC http://jitech.ca | B, Br, P, S 514-697-8999 | | |
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| JP Drafting Ltd. Maple Ridge, BC www.jpdrafting.com | B, Br, J, P 604-465-8933 | | |
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| Service Technique Asimut inc Charny, QC www.asimut.ca | 418-988-0719 | | |
| Summyx inc. Ste-Marie, Beauce, QC www.summyx.com | Br, S 418-386-5484 | | |
| TDS Industrial Services Ltd. Prince George, BC www.tdsindustrial.com | B, P 250-561-1646 | | |
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| | | Arcweld Industries Inc. Winnipeg, MB www.arcweld.ca | B, Br, J, P, S 204-661-3867 |
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| | | Daam Galvanizing Edmonton Ltd. Edmonton, AB www.daamgalvanizing.com Hot dip galvanizing | 780-468-6868 |
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| La Compagnie Américaine de Fer et Métaux Inc. / American Iron & Metal Inc. East Montréal, QC www.scrapmetal.net | 514-494-2000 | Rapid Check Solution Delson, QC http://rapidchecksolution.com | 514-434-8778 | Vicwest Building Products [Moncton] Memramcook, NB www.vicwest.com Steel metal floor/roof deck, wall and roof cladding | 506-758-8181 | Blackwell, Toronto, ON | 416-593-5300 |
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| Les Produits Métalliques Bailey Limitée Dorval, QC www.bmp-group.com | 514-735-3455 | Selectone Paints Inc. Weston, ON www.selectonepaints.ca Paint primers, fast dry enamels, coatings | 416-742-8881 | Vicwest Building Products [Winnipeg] Winnipeg, MB Steel metal floor/roof deck, wall and roof cladding | | Brenik Engineering Inc., Concord, ON | 905-660-7732 |
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NRE takes to the water for the completion of the swing bridge on the Severn River



Final touches before completion of one side of the swing bridge



The timber was the last step for inspection before the opening of this new bridge



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