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## STANDARDIZED CAST STEEL CONNECTORS FOR TUBULAR HOLLOW STRUCTURAL SECTIONS

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**ABSTRACT** : Since castings were used in the main stadium at the 1972 Olympic Games in Munich, Germany, cast steel structural components have become increasingly popular in Europe. This popularity has recently made its way to Canada in the form of structural castings at the University of Guelph Science Complex in Guelph, Ontario and the CDP Capital Centre in Montréal, Québec. Previously, structural castings were only considered for connections with complex geometry or strict aesthetic requirements. Research at the University of Toronto has given rise to several mass customizable cast steel connectors that offer a simple alternative to common yet complex fabricated steel connections. This paper identifies two cases where the use of mass produced cast steel components could provide significant advantages over standard fabrication practices: seismic-resistant and aesthetic pin connections for hollow structural sections. This paper presents a detailed discussion on the design and fabrication of seismic-resistant bracing connections for hollow structural sections and introduces an innovative alternative joint detail which utilizes a standardized cast steel connector developed at the University of Toronto. Also presented are cast connectors for round hollow structural section members that are suitable for architecturally exposed steel connections. These connectors are intended to be used when an aesthetic true-pin type structural connection is required. A case study on a Canadian construction project that used fabricated pin connections is presented. The difficulties associated with the fabricated solution are contrasted with the simplicity offered by the standardized aesthetic cast steel connector.

### 1. INTRODUCTION

The use of steel castings for structural applications is becoming evermore prevalent. In Europe, castings are commonly used in bridge applications, where the use of cast steel nodes significantly improves the fatigue life of otherwise highly complex welded connections. In contrast, castings used in building construction applications have been predominantly used for architecturally exposed connections, particularly in structures made up of hollow structural sections. Canadian examples of the architectural use of castings are the roof supporting tree structure at the University of Guelph Science Complex in Guelph, Ontario and the eight-storey tall façade supporting trusses of the CDP Capital Centre in Montréal, Québec.

In many cases, castings have been used creatively to solve particularly difficult structural engineering or fabrication challenges. One example of this use of castings is in the new Beijing Poly Plaza (Figure 1). In this structure, cast steel rocker mechanisms support a suspended eight-storey lantern-like museum that protrudes through the structure's cable-net façade (the façade is believed to be the world's largest cable-net glass wall). The cast steel rockers transmit significant tensile loads while decoupling the supporting cables, which also stiffen the cable-net wall, from the larger structure's lateral system. Indeed, casting

manufacturing offers structural engineers the opportunity to demonstrate the true flexibility and strength of designing and constructing with structural steel.



Figure 1. Casting use in Beijing Poly Plaza (photographs courtesy of Skidmore, Owings & Merrill LLP)

With the use of steel castings gaining in popularity, research at the University of Toronto focused on exploiting the most significant advantage that casting manufacturing provides over standard fabrication – mass production. The result has been the development of innovative cast steel connectors for two common yet complex steel fabrication issues for hollow structural sections: seismic bracing connections and true-pin connections suitable for architectural exposure. This technology has since been licensed to the Ontario-based Cast ConneX Corporation whose goal is to provide the steel fabrication industry with the economies of scale that are possible through the use of standardized structural steel components.

## 2. CAST STEEL CONNECTORS FOR SEISMICALLY LOADED TUBULAR HOLLOW SECTIONS

Concentrically braced frames are amongst the most popular lateral force resisting systems for medium- to low-rise steel structures. The diagonal brace members of braced frames are subject to predominantly axial forces, and in the event of an earthquake, seismic energy is dissipated through the cyclic yielding in tension and inelastic buckling in compression of the brace members. This cyclic yielding and buckling imparts arduous loading on the brace end connections; consequently North American design codes (CSA, 2001; AISC, 2005a; AISC, 2005b) require that seismic bracing connections be detailed such that the connections are significantly stronger than the nominal cross-sectional capacity of the brace member. Designing connections to accommodate this strength is quite difficult, particularly for hollow structural sections (HSS), which are the preferred bracing element due to their efficiency in carrying compressive loads, their improved aesthetic appearance, and the wide range of sections sizes that are readily available in North America. To further justify the use of HSS bracing in seismic applications, research has shown that HSS members suffer less degradation of their hysteretic response than other structural sections (Lee and Bruneau, 2005).

End connections for hollow section brace members are typically achieved through a gusset connection between the brace end and the beam-column intersection. In statically loaded connections, a shear-lag inducing slotted HSS-to-gusset connection can be accommodated since axial loads are typically well below the cross-sectional capacity of the brace. However, both in the laboratory and in the field as witnessed during post-earthquake reconnaissance, conventional slotted HSS-to-gusset connections have been shown to be prone to failure when subjected to cyclic inelastic loading. These premature failures, which may lead to structural collapse, occur due to a concentration of inelastic strain at the reduced section of the HSS connection. Thus, current seismic design provisions recommend the use of net-section reinforcement whenever slotted HSS-to-gusset connections are specified in seismic-resistant frames. As reinforcement is more readily fabricated for flat surfaces, the industry has tended toward the specification of square or rectangular HSS rather than round HSS for seismic resistant braces. This is unfortunate as the energy absorbing capability of North American cold-formed rectangular HSS under cyclic inelastic loading is now thought to be less than that implied by the current design provisions, as evidenced by field reconnaissance and laboratory testing (AIJ, 1995; Bonneville and Bartoletti, 1996; Shaback and Brown, 2003). In addition, North American rectangular HSS have been shown to have inherently poor notch toughness (Kosteski et al. 2005), which severely limits their ability to resist dynamic loadings. Thus round HSS is a more appropriate choice for seismic-resistant bracing, however designing, detailing, and fabricating reinforced slotted connections to round HSS is still both challenging and costly.

Recognizing the need for a simple solution to the seismic brace connection dilemma, the research team at the University of Toronto developed standardized cast steel connectors that could be shaped to eliminate the need for connection reinforcement (Figure 2). The geometric freedom that casting manufacturing provides allowed for a design that accommodates bolted connection to a single gusset plate and complete joint penetration (CJP) welded connection to a round HSS brace member. Thus, in practice, the cast connectors can be welded to the round hollow section braces in the shop, with the brace-connector assembly being bolted to the frame in the field. Further, as shown in Figure 2, the nose of the connector accommodates round HSS members of any wall thickness for a given outer diameter, thereby promoting standardization of the cast components. The use of a double-shear bolted connection halves the number of bolts that would otherwise be required in a spliced connection. Spliced connections are commonly specified to eliminate the need for field welding in conventional seismic-resistant reinforced brace connections. The specification of a pre-qualified CJP shop weld between the casting and the round HSS eliminates the need for field welding of the demand-critical welds between the gusset plate and the brace member.

Figures 3 and 4 below compare three seismic-resistant brace connections designed according to the current Canadian standard (CSA, 2001). The figures illustrate the significant reduction in design, detailing, and fabrication complexity that the use of the standardized connectors developed by the authors provides. Each of the connections shown below requires a resistance greater than 3,200 kN (ASTM A500 brace members;  $R_y A_g F_y \approx 3,200$  kN, with  $R_y = 1.1$ ).

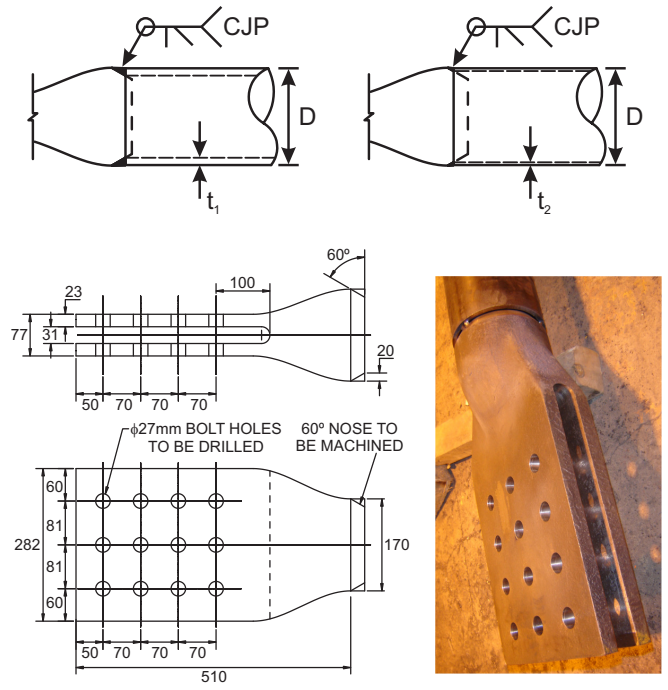
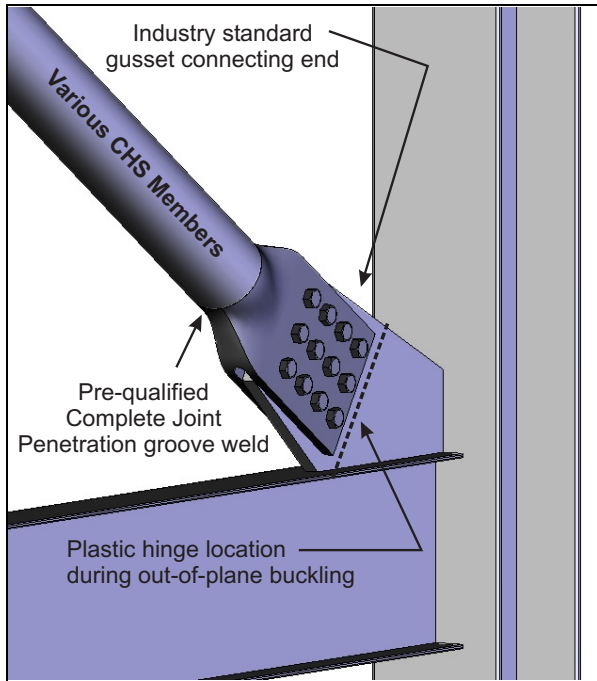


Figure 2. Patent-pending cast steel seismic-resistant hollow structural section brace connector

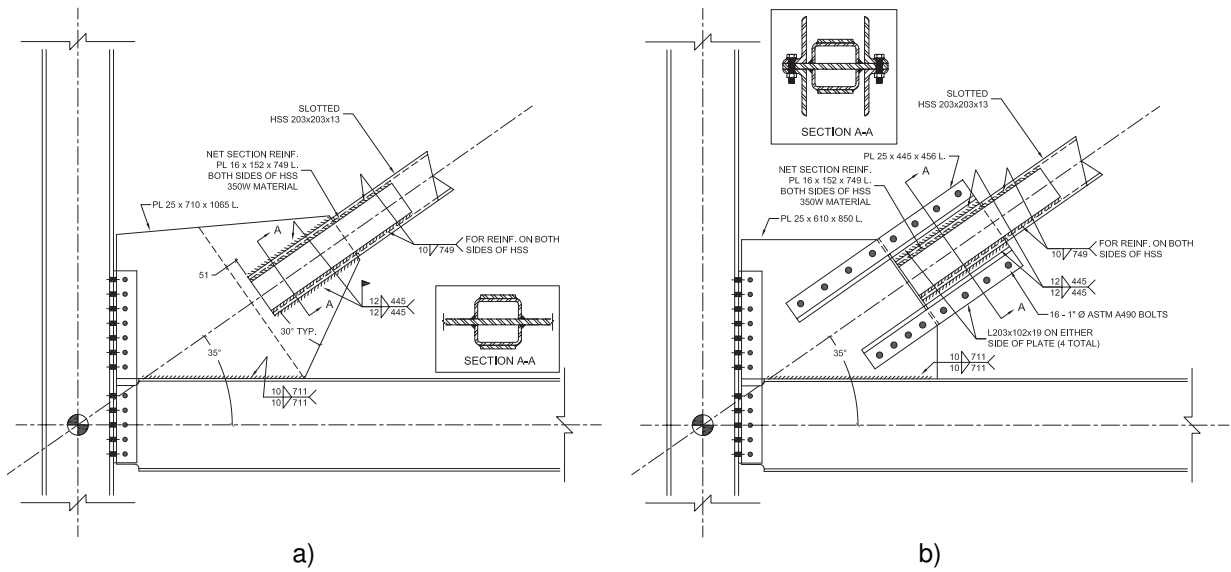


Figure 3. Seismic-resistant brace connection details: a) Field-welded, b) field-bolted

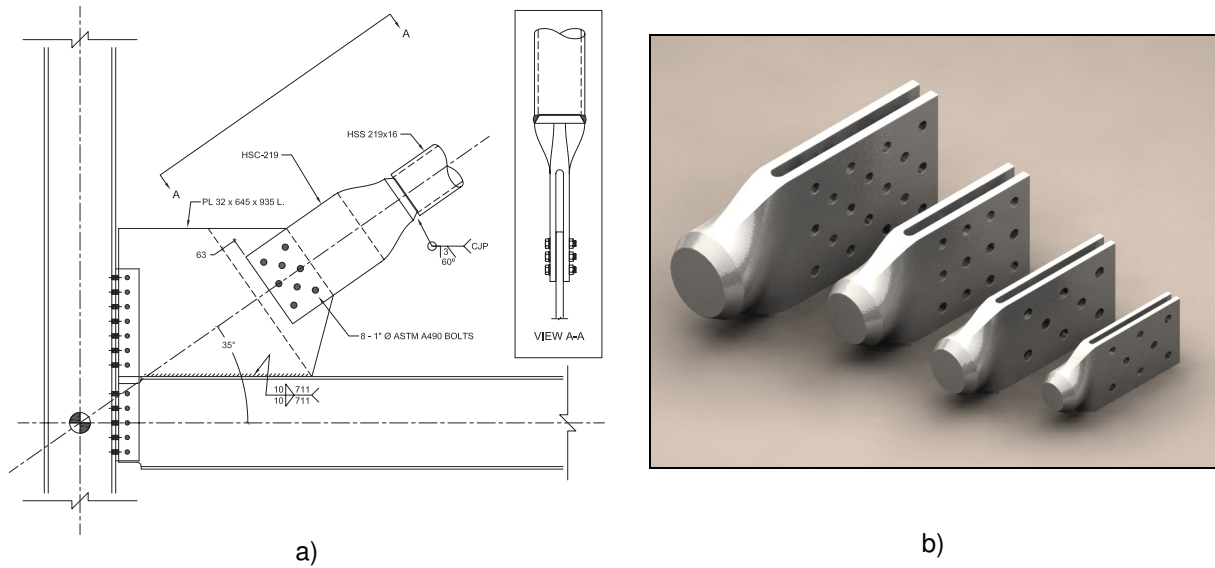


Figure 4. a) Seismic-resistant brace connection detail using a standardized cast steel connector, b) Patent pending Cast ConneX™ High-Strength Connector Series of products

The connection detail shown in Figure 3 a) requires shop welding of net-section reinforcement plates to the brace followed by field-welding of the reinforced brace to the gusset plates during erection. As these field welds are part of the critical load path, often in-situ non-destructive examination is required to confirm their quality. A gap of twice the gusset plate thickness (“2·tp”) is provided between the end of the brace and any stiffening element connected to the gusset. This detail accommodates the out-of-plane rotation that is expected during a seismic event and negates the need to provide flexural strength in the connection in excess of the brace member’s probable flexural capacity.

Alternatively, the connection detail shown in Figure 3 b) removes the requirement for field welding of the brace assembly. The geometry of this detail is aimed at providing a compact spliced connection with a relatively small gusset plate, hence the “2·tp” gap is not provided. As a result, the Canadian design standard requires that the connection must have an out-of-plane flexural strength that exceeds the brace member’s probable flexural capacity ( $1.1ZR_yF_y$ ). This requirement is met using angles to both splice the connection and provide the required flexural strength over the connection region.

The connection detail shown in Figure 4 a) makes use of the standardized seismic-resistant connector developed by the authors. As in the connection detail in Figure 3 a), the detail shown in Figure 4 a) makes use of the “2·tp” gap to eliminate the need for an increased flexural strength through the connection region, however, the ability to shop weld the connector to the HSS brace and field bolt the assembly (using half the number of bolts) while eliminating the need for connection reinforcement and slotting of the HSS section, vastly simplifies the design, detailing, and fabrication requirements for the connection. Another key advantage of the use of the standardized connector is that increases in connection capacity requirements do not require significant redesign of the connection detail. For example, steel design standards in the United States require the use of overstrength factors,  $R_y$ , of 1.4 for ASTM A500 bracing members and 1.6 for ASTM A53 bracing members. In Canada,  $R_y$  is currently set at 1.1. Thus, if the details shown in Figures 3 a) and b) were applied for the same brace members in the United States, a complete redesign of the connection details would be required. Conversely, the only required changes to the detail shown in Figure 4 a) would be the addition of more bolts at the casting-gusset connection and a slight lengthening of the gusset plate to accommodate these bolts.

The development of the standardized connector at the University of Toronto included full-scale laboratory testing of brace-connector assemblies under the action of inelastic cyclic loading. These tests confirmed

the validity of the cast connector concept. Figure 5 shows the axial load-displacement results from cyclic loading according to a protocol developed to test the connection strength of a HSS 168x9.5 brace-connector assembly. A complete summary of the conceptual development, prototype design and manufacturing, and destructive laboratory testing for the cast steel seismic-resistant connector can be found in de Oliveira et al., 2008. A full range of brace connector components are now commercially available, as shown in Figure 4 b).

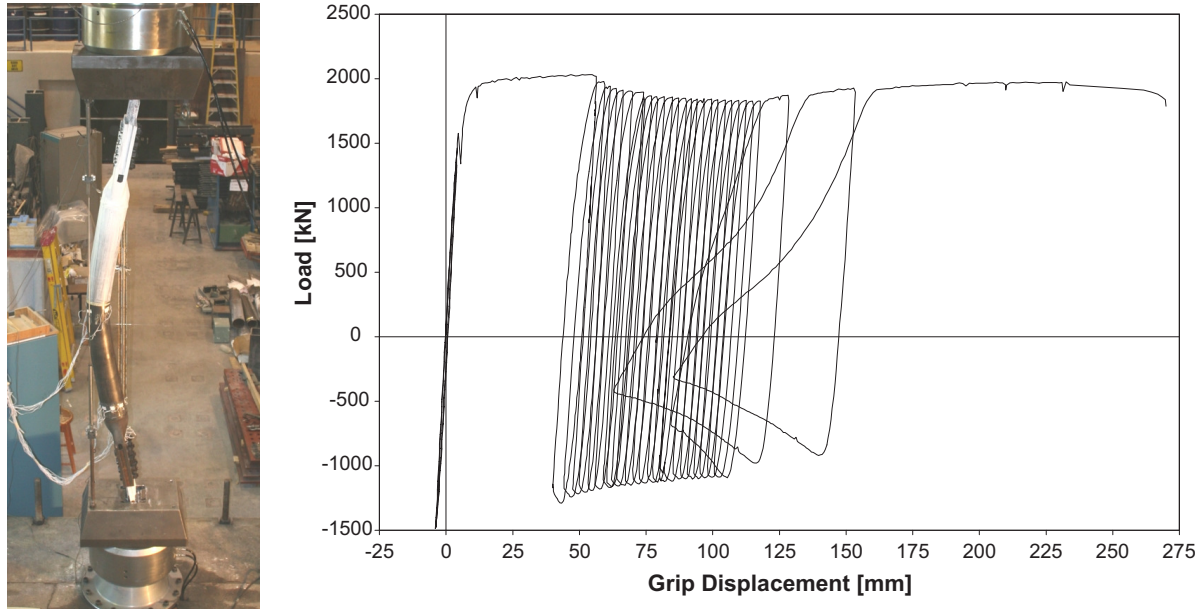


Figure 5. Hysteretic brace response of cast connector-HSS brace assembly (HSS 168x9.5)

### 3. CAST STEEL CONNECTORS FOR ARCHITECTURALLY EXPOSED TUBULAR HOLLOW SECTIONS

An emerging trend in steel construction is the increasing popularity of architecturally exposed structural steel connections. The increasing demand for architecturally exposed structural steel (AESS) prompted the Canadian Institute of Steel Construction to develop a guideline for AESS engineering. There are numerous structures around the world that feature exposed steel and connections and AESS is becoming evermore commonplace in new construction.

Architecturally exposed connections are more complex and generally more costly to fabricate than typical steel connections. An example of a relatively expensive architectural detail is a true-pin end connection for a HSS member. The fabrication complexity of a true-pin connection increases significantly as the required aesthetic increases. Several examples of typical fabricated true-pins, henceforth referred to as pin connections, for HSS members are shown in Figure 6.

Figure 6 a) illustrates what is likely the most simple fabricated pin detail for HSS members. This detail requires that a single slot is created in the HSS into which a plate is inserted; the plate is subsequently fillet welded to the HSS member. This slotted HSS-to-gusset detail is a typical end connection for statically loaded round HSS braces. The complexity arises in the clevis-type connection at the base of the pin that receives the single plate that is fixed to the HSS member. Here, two plates are necessary since pin connections must maintain a concentric load path. Due to the relatively small gap between the two plates, both plates cannot be fillet welded to the base plate; one of the two plates must be fastened to the base with a groove weld. Further, alignment of these two plates is critical as is discussed in regards to the subsequent detail.



the nose of the cast pin was hollowed in an effort to reduce the weight, and thus the cost, of each part. Computer generated renderings of the final cast pin design are shown in Figure 7. Unfortunately, to the great disappointment of the fabricator and the research team, the cast solution could not be selected for use in the project due to manufacturing timing issues.

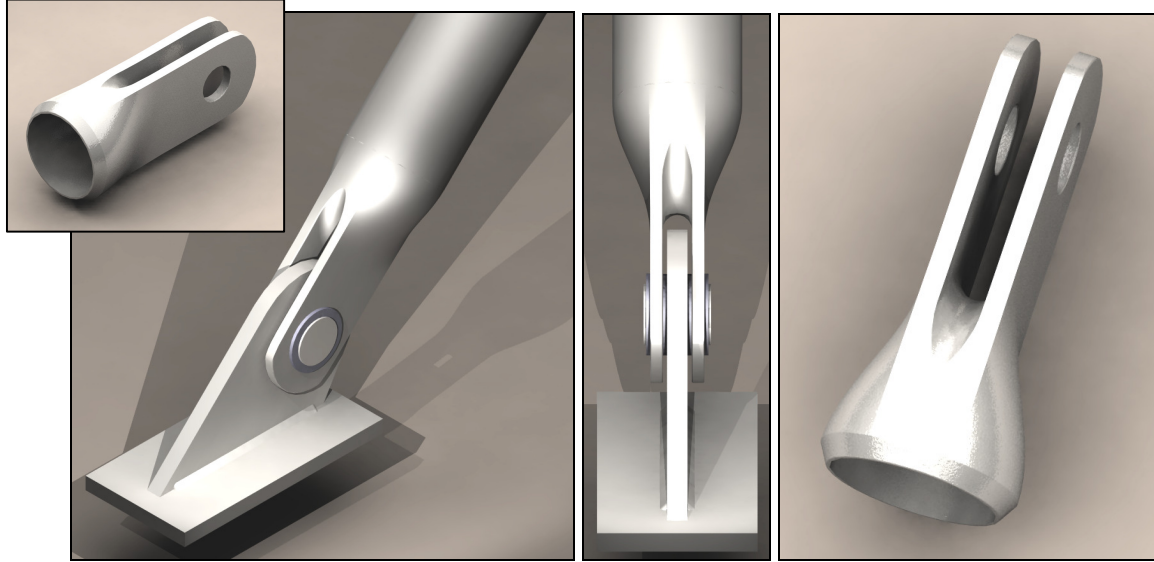


Figure 7. Cast steel “true pin” type connector suitable for aesthetic CHS connections

This experience revealed an industry need for a simple solution for the aesthetic pin connection problem. It has been well established that castings become economically viable as the fabricated alternative increases in complexity and with repetition. Given the complexity of aesthetic fabricated pin connections and the opportunity for mass production, it is clear that a series of standardized cast steel pin connectors would be a welcome addition in the structural steel industry.

The design methodology previously established during the development of the seismic-resistant cast connector was applied for a proposed mass-produced pin connector series. However, unlike the seismic-resistant connector that must be designed to accommodate the expected cross-sectional yield capacity of the heaviest walled round HSS, there was no obvious design force for each cast pin connector for a given tube diameter. It was decided that an appropriate design capacity for each pin connector in both tension and compression would correspond to 60% of the nominal factored cross-sectional yield strength of the thickest walled round HSS that the connector could accommodate. This capacity was selected since the most common pin ended members are compression members, like struts and arches, which are usually sized for their buckling resistance, not their cross-sectional tensile capacity. Also, to be widely accepted in the industry, a mass produced solution should satisfy both the Canadian (CAN/CSA S16-01) and US (ANSI/AISC 360-05) steel design codes.

Upon examination of the design provisions for pin connected members loaded in tension, several empirical geometric limitations were found to govern the design of the cast steel pin connectors. These provisions, which can be found in CAN/CSA S16-01 Clause 12, were derived from a paper published in 1939 (Johnston, 1939). This paper describes the testing of 106 pin connections constructed with a range of plate thicknesses and edge distances. The resulting geometric recommendations that are stipulated in the North American codes address three failure modes that may occur in pin connections.

The first failure mode is referred to as “dishing”. This behaviour was observed in connections fabricated from thin plates with large edge distances and is a stability failure in which the part of the plate that extends beyond the pin hole buckles laterally, taking on a concave shape. Dishing failure occurs after



the plate begins to yield in the cross section at the pin. Johnston's work showed that in pins fabricated with thin plates, general yield is promptly followed by dishing and ultimate failure, whereas pins fabricated with thicker plates have a large plastic reserve between yielding and ultimate failure. This failure mode is addressed in Clause 12.4.2 in CAN/CSA S16-01 which requires that the distance from the edge of the pin hole to the edge of the plate, measured normal to the axis of the member, must be no more than 4 times the thickness of the plate material.

The second of Johnston's recommendations that was adopted in the Canadian design code is found in Clause 12.4.1 and is intended to account for the non-uniform stress distributions that are found in the vicinity of the pin hole. This clause requires that the net area of the cross-section of the plate(s) at the pin hole, measured normal to the direction of the member, must be greater than or equal to 1.33 times the cross-sectional area of the connected member.

The final requirement for pin connections that is codified is necessary to avoid tensile end splitting of the plate(s) beyond the pin. As suggested by Johnston, this failure mode can be avoided by proportioning the net area of any section on either side of the axis of the member, measured at an angle of 45° or less to the axis of the member, to be greater than 0.9 times the cross-sectional area of the connected member.

Johnston's recommendations to avoid failure due to the non-uniform stress distributions in the region near the pin hole and to avoid end splitting were based on the assumption that the design force of the connected member was very close to the gross yield capacity of that section. In 1939 this may have been the case for many tension-loaded pin type connections; however, in modern AESS HSS connections, the member utilization ratio may be far less than unity, particularly for compression members.

In the design of the series of standardized pin connectors, the research team followed the width to thickness limit but elected to apply a modification to the net section clauses. Rather than base the minimum net area of the cast pin's plates on the cross section of the connected member, i.e. the round HSS section, the net area of the plates is set to the equivalent area of cast steel that would be required to resist  $\alpha$  times the design force for the connector. The factor  $\alpha$  is set to 1.33 for the perpendicular net section check and to 0.9 for the end-splitting net end section requirement. This modification is expressed in Equation 1. Non-linear finite element stress analysis confirmed the validity of this design methodology.

$$[1] A_n \geq \frac{\alpha \cdot T_{\text{design}}}{F_{y\text{Casting}}}$$

The final designs for the series of connectors, as shown in Figure 8, provide attractive, smooth, compact and robust castings that can be easily integrated into a structure and can save steel fabricators and designers hours of complex engineering, detailing, and fabricating.

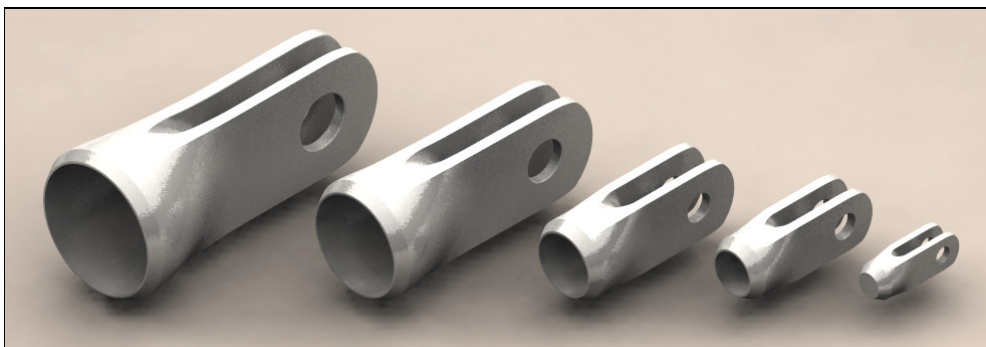


Figure 8. Patent pending Cast ConneX™ Universal Pin Connector Series of products

## 4. CONCLUSIONS

To date, the use of steel castings for structural applications has been primarily based on one-off design and manufacturing on a project-by-project basis, which does not fully exploit casting manufacturing's predisposition to mass production. The authors have identified two cases where the use of mass produced cast steel components could provide significant advantages over standard fabrication practices: seismic-resistant and aesthetic pin connections to hollow structural sections. These innovative connection concepts, which have been shown to provide significant economies over standard fabrication, are now commercially available and may herald a new era of pre-engineered connection design and fabrication for hollow structural section connections.

## ACKNOWLEDGEMENTS

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## REFERENCES

- AIJ., 1995. Reconnaissance report on damage to steel building structures observed from the 1995 Hyogoken-Nanbu (Hanshin/Awaji) earthquake, Architectural Institute of Japan, Steel Committee of Kinki Branch, Tokyo, Japan.
- American Institute of Steel Construction (AISC), 2005a. Specification for structural steel buildings. ANSI/AISC 360-05, Chicago, Illinois.
- American Institute of Steel Construction (AISC), 2005b. Seismic provisions for structural steel buildings. ANSI/AISC 341-05, ANSI/AISC 341s1-05, Chicago, Illinois.
- Bonneville, D., and Bartoletti, S., 1996. Case Study 2.3: Concentric Braced Frame, Lankershim Boulevard, North Hollywood. 1994 Northridge Earthquake; Building Case Studies Project; Proposition 122: Product 3.2, SSC 94-06, Seismic Safety Commission State of California, pages 305-324.
- Canadian Standards Association (CSA), 2001. Limit states design of steel structures. CAN/CSA-S16-01, Toronto, Ontario.
- de Oliveira, J.-C., Packer, J. A., Christopoulos, C., 2008. Cast Steel Connectors for Circular Hollow Section Braces Under Inelastic Cyclic Loading. *ASCE Journal of Structural Engineering*, 134(3), 374-383.
- Johnston, B. G., 1939. Pin-Connected Plate Links. *Transactions, American Society of Civil Engineers*, p. 314-339.
- Kosteski, N., Packer, J.A., and Puthli, R.S., 2005. Notch toughness of internationally produced Hollow Structural Sections. *ASCE Journal of Structural Engineering*, 131(2), 279-286.
- Lee, K., and Bruneau, M., 2005. Energy dissipation of compression members in concentrically braced frames: Review of experimental data. *ASCE Journal of Structural Engineering*, 131(4), 552-559.
- Shaback, B., and Brown, T., 2003. Behavior of square hollow structural steel braces with end connections under reversed cyclic axial loading. *Canadian Journal of Civil Engineering*, 30, 745-753.