ABSTRACT: Diagonal bracings are extremely popular elements for lateral load resistance in steel-framed buildings. In turn, the most common shape used for bracing members is the hollow structural section. While the design of such members is straightforward, the design of gusset-plate connections at the member ends is controversial. This paper reviews the current ‘state-of-the-art’ for the design of such connections, under both static and seismic loading conditions, and for fabricated and cast connections.

1. INTRODUCTION

The total global output of welded tubes, which represent the manufacturing process used for most of the world’s structural tubing, has been approximately constant – despite some fluctuations – over the last 10 years: 40.1 million metric tons in 1995 and 41.1 million metric tons in 2004 (IISI 2005). In this same period, however, the world production of crude steel has increased by 41%, from 752 million metric tons in 1995 to 1 058 million metric tons in 2004. Thus, in 2004 welded tubes represent about 4% of the total steel market, but a very important component of the structural steel sector. While some countries have decreased welded tube output in the last decade (e.g. U.S.A.), there has been a huge increase in production in China (by 245% over the period 1995 - 2004). National production statistics, for the 10 leading countries, are shown in Figure 1 (IISI 2005). These figures do not include other (less-common) types of hollow sections (e.g. seamless tubes and fabricated sections). While not all of these tonnages will be used for structural purposes, the data is indicative of local consumption and export levels.

In steel structures the most common applications for welded tubes are as columns, in trusses and as lateral bracing members, where the structural engineer can take advantage of excellent properties in compression and the architect can utilise aesthetic qualities in exposed steelwork. Simply-connected steel frames are typically laterally-braced with diagonal members as shown in Figure 2. The ends of the Hollow Structural Section (HSS) bracings are then usually connected to the steel frame via gusset plates, as shown in Figure 3. The design of the bracings, as compression or tension members, is performed in accordance with applicable national or regional structural steel specifications. For low-rise structures with lateral loads governed by static (wind) loading, bracing member selection will often be controlled by maximum permitted member slenderness limits. (For example, in Canada (KL/\eta)_{max} = 200 in compression and, generally, 300 in tension (CSA 2001)). In structures with lateral load design governed by seismic actions, bracing member selection will be further restricted by limits on the slen-
derness of the member cross-section. For example, for moderately ductile concentrically braced frames in Canada, where moderate amounts of energy are dissipated through yielding of bracing members with \((KL/r) \leq 100\), the flat width-to-thickness ratio of square and rectangular HSS must be \(\leq 330/\sqrt{F_y}\), and the diameter-to-thickness ratio of circular HSS must be \(\leq 10000/F_y\). These cross-section slenderness limits, in which the yield stress \(F_y\) is expressed in MPa or N/mm², are considerably lower than the normal Class 1 limits (CSA 2001). In current U.S. provisions for ‘special’ and ‘ordinary’ concentrically braced frames, these cross-section slenderness limits are even more restrictive: \(286/\sqrt{F_y}\) for square/rectangular HSS and \(8000/F_y\) for circular HSS (AISC 2005a).

2. GUSSET PLATE CONNECTIONS TO THE ENDS OF HOLLOW SECTIONS – STATIC LOADING

Single plates are often inserted into the slotted ends of a round or square HSS, concentric to the axis of the HSS member, both in roof trusses (typically to avoid round-to-round HSS tube profiling associated with directly-welded members) and in diagonal bracing members in braced frames. This inserted plate is frequently then connected to a single gusset plate, usually by bolting. In such situations a bending moment is induced in the joint by the eccentricity between the plates which must be considered. Under compression loads the plates need to be proportioned as beam-columns, and assuming that both ends of the connection can sway laterally relative to each other. This is frequently overlooked, leading to periodic structural failures, but the American HSS Connections Manual (AISC 1997, Chapter 6) is not guilty of this omission and gives a reasonable and simple design method. Alternatively, the single gusset plate attached to the building frame can be stiffened, typically by adding another transverse plate along one edge of the gusset, thereby giving the gusset attached to the building frame a T-shape in cross-section.

With regard to the performance of the HSS in such connections, load is only transmitted initially to a portion of the HSS cross-section, thereby creating a shear lag effect which may result in a lower HSS capacity in both compression and tension. For tension loading on the HSS member, the effective area \((A_e)\) is determined by the net area \((A_n)\) multiplied by a shear lag factor, \(U\). For the latter, the most recent specification version is given by AISC (2005b). These \(U\) factors have been revised by AISC from the previous specification (AISC 2000), where \(U\) had an upper limit of 0.9. Based on the work of Cheng & Kulak (2000) the \(U\) factor can now be taken as 1.0 for connections to circular HSS with a sufficiently-long inserted plate and weld length \((L_w)\). Table 1 shows the current AISC \(U\) factors for circular HSS compared to those from other Canadian codes/guides, and Figure 4 illustrates the geometric parameters used. For the shear lag effect, Eurocode 3 (CEN 2005) only addresses bolted connections for angles connected by one leg and other unsymmetrically connected tension members.

North American specifications have gone through many revisions (Geschwindner 2004) concerning the design methods for the limit state of tensile fracture affected by shear lag. Table 1 illustrates the two main prevailing methods: based on the connection eccentricity (AISC) or based on the distance between the welds (CSA). In this table it can be seen that the Packer and Henderson (1997) approach is just a modification of the CSA (1994) method. Note that the resistance factor of \(\phi = 0.75\) for AISC (2005b) is approximately the same as \((0.9)(0.85) = 0.765\) for CSA (2001). The other tensile limit state for these connections is ‘block shear’ (or tear-out) and the current North American and European design provisions are given in Table 2. As can be seen, all use a design model based on the summation of the resistance of the part in tension (where all use the net area in tension multiplied by the ultimate tensile stress) and the resistance of the part in shear. The latter can be calculated based on the net/gross area in shear multiplied by the shear yield stress/shear ultimate stress, depending on the specification. At present the American and Canadian specifications use a common design model but quite different resistance factors. (The Canadian resistance factor is currently under review).

A study of both concentric gusset plate-to-slotted tube and slotted gusset plate-to-tube connections, under both static tensile and compression member loadings, using both round and elliptical HSS, has been underway at the University of Toronto since 2002. The connection

Figure 3: Statically-loaded steel frame, braced with diagonal hollow sections.

Figure 4: Important geometric parameters influencing connection design.
fabrication details investigated, which include both end return welds and connections leaving the slot end unwelded, are shown in Figure 5. Complete details of the experimental testing programme can be found elsewhere (Willibald et al. 2006) but examples of the two classic failure modes are shown in Figure 6.

The experimental program by Willibald et al. (2006) concluded that the block shear design model (Table 2), although based on limited correlations, was suitable, particularly if predictions were calculated using a theoretical fracture path excluding the welds. Yet another proposal has been recently made to improve the general block shear model in Table 2 (Franchuk et al. 2004) for slotted rectangular HSS connections, whereby they propose an ‘exact’ term calculated by using a distance from the edge of the gusset plate to the wall size. Interestingly, a very similar conclusion has just been reached by Dowswell & Barber (2005) for slotted rectangular HSS connections, whereby they propose an ‘exact’ term calculated by using a distance from the edge of the gusset plate to the wall size. Dowswell & Barber (2005) verify their proposal by showing improved accuracy of the gusset plate, which is often substantial relative to the tube size. Following experimental research on the connection types shown in Figure 5, an extensive detailed numerical study followed on the same connections using non-linear Finite Element (FE) Analysis (Martinez-Saucedo et al. 2005). A full parameter study expanded the total experimental and numerical database to over 700 connections (Martinez-Saucedo et al. 2005). The FE models revealed a gradual transition between the failure modes of block shear/tear-out (TO) and circumferential tension fracture (CF), with the latter sometimes influenced by the shear lag phenomenon (see Figure 7). A continual monotonic increase in the connection capacity was achieved as the weld length increased. The transition point between these failure modes depended on factors such as: the connection type, the weld length, the tube diameter-to-thickness ratio and the connection eccentricity, \( \gamma \) (the latter having a strong influence for elliptical HSS). This gradual transition between the failure modes is in contrast to the behaviour given by that by AISC, but Willibald et al. (2006) suggested that the existing formulation could be much improved by reducing the connection eccentricity term – used to calculate \( U – \gamma \), as shown in Figure 4. This essentially accounts for the thickness of the gusset plate, which is often substantial relative to the tube size.

### Table 2: Block shear (tear-out) design provisions.

<table>
<thead>
<tr>
<th>Specification or design guide</th>
<th>Effective area</th>
<th>Shear lag coefficients</th>
<th>Range of validity</th>
</tr>
</thead>
<tbody>
<tr>
<td>AISC (2005b): Specification for Structural Steel Buildings</td>
<td>( A_e = A_n \cdot U )</td>
<td>( U = 1 - \frac{\pi}{w} ) for ( 1.3D &gt; L_w \geq D )</td>
<td>( L_w \geq D )</td>
</tr>
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<td>Packer and Henderson (1997): Hollow Structural Section Connections and Trusses – A Design Guide</td>
<td>( A_e = A_n \cdot U )</td>
<td>( U = 1.0 ) for ( L_w/w \geq 2.0 )</td>
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</table>

\( T_r = \phi A_e F_u \) (AISC (2005b) Specification, \( \phi = 0.75 \)) or \( T_r = 0.85 \phi A_e F_u \) (CSA (2001) Specification, \( \phi = 0.9 \)), where \( T_r \) = factored tensile resistance, \( F_u \) = ultimate tensile stress and \( \phi \) = resistance factor.

### Table 1: Shear lag design provisions for circular and elliptical hollow sections.

<table>
<thead>
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<th>Effective area</th>
<th>Shear lag coefficients</th>
<th>Range of validity</th>
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design models in current specifications, since these specifications do not consider a gradual change between these limit states. Thus, a more unified and less conservative design model for slotted gusset plate HSS connections can be expected in the near future. Figure 7 also confirms that a value of $U = 1.0$ (hence $100\%$ of $A_n F_y$) for circular HSS with $L_w/D \geq 1.3$ (AISC 2005b) is indeed correct, and for all practical tube diameter-to-thickness $(D/t)$ ratios. However, the conservative connection capacity predictions by over-estimating the severity of the shear lag effect at $L_w/D \leq 1.3$ are very apparent.

3 GUSSET PLATE CONNECTIONS TO THE ENDS OF HOLLOW SECTIONS – SEISMIC LOADING

If the results in Figure 7 are re-plotted in terms of $N_{ud}/A_y F_y$, where $A_y$ is the tube gross area, then it can be shown that long plate insertion lengths can achieve tension capacities very close to $A_y F_y$, even for this connection type with an open slot end. However, in tension-loaded energy-dissipating braces the connection will be required to resist an even greater load of $A_y R_y F_y$, where $R_y$ is a material over-strength factor to account for the probable yield stress in the HSS bracing. This value of $R_y$ is specified as 1.1 in Canada (CSA 2001), and 1.4 (for A500 Grades B and C (ASTM 2003)) or 1.6 (for A53 (ASTM 2002)) in the U.S. (AISC 2005a). The Canadian value is too low, based on personal laboratory testing experience, and a realistic value for the mean expected yield strength-to-specified minimum yield strength ratio is around 1.3, for CSA-grade HSS (CSA 2004). Tremblay (2002) reported a mean over-strength yield value of 1.29 for rectangular HSS surveyed, and Goggins et al. (2005) have reported a mean over-strength yield value of 1.49 for rectangular HSS (Europe), but the latter was the result of specifying low grade 235 MPa steel. The high U.S. values were determined by a survey of mill test reports by Liu (2003) and are not surprising because, in a market like North America with several different steel grades and production standards, manufacturers will produce to the highest standard and work to a ‘one product fits all’ approach. (For example, in her survey Liu’s ASTM A500 data all pertained to Grade B tubing, whereas manufacturers will knowingly produce to meet the higher Grade C strengths). AISC, however, has now introduced another material over-strength factor, $R_t$, to account for the expected tensile ultimate strength relative to the specified minimum tensile strength (AISC 2005a) with these values being 1.3 for ASTM A500 Grades B and C and 1.2 for ASTM A53. This $R_t$ factor is applied to fracture limit states in designated yielding members – such as bracings in concentrically braced frames where circumferential fracture (CF) is a design criterion. Thus, applying capacity design principles to preclude non-ductile modes of failure within a designated yielding member (bracing) and setting the resistance factor $\phi = 1.0$, one obtains the following, to avoid circumferential fracture of the HSS at the gusset plate (refer to the equations below Table 1):

AISC (2005a):

$$R_t F_u A_e \geq R_y F_y A_y,$$

hence for ASTM A500 HSS and setting $F_y \leq 0.85 F_u$, $A_e \geq 0.92 A_y$

CSA (2001):

$$(0.85 F_u A_y) R_y \geq R_y F_y A_y,$$

hence for CSA HSS and setting $F_y \leq 0.85 F_u$, $A_e \geq 1.0 A_y$

From the above, one can see that the required minimum effective net area – after consideration of shear lag and application of the $U$ factor – is near the gross area of the HSS bracing.

In compression, type A connections (see Figures 5 and 8) can be shown to achieve capacities that also approach $A_y F_y$, provided the length of the open slot is kept short (in the order of the plate thickness) and the tube is relatively stocky (see Fig. 8). However, despite the achievement of high compression load capacity this is accompanied by considerable plastic deformation in the tube at the connection, which is likely to also render the connection unsuitable for use in energy-dissipating brace members.

Fabricated end connections to tubular braces, in concentrically braced frames, hence have great difficulty meeting connection design requirements under typical seismic loading situations. Reinforcement of the connection is then the usual route. It is difficult to plate
round HSS members so square HSS with flat sides have become the preferred section, resulting in costly reinforced connections as shown in Figure 9. Moreover, recent research on the performance of HSS bracings under seismic loading still concentrates on square/rectangular hollow sections (Goggins et al. 2005; Elghazouli et al. 2005; Tremblay 2002). A drawback of using cold-formed, North American square/rectangular HSS is that they have low ductility in the corners and are prone to fracture in the corners after local buckling during long-cycle fatigue.

A clear improvement is to use cold-formed circular hollow sections, which do not have corners, and to attempt to avoid reinforcement. Yang & Mahin (2005) recently performed six tests on slotted square HSS and slotted circular HSS under seismic loading and highlighted the improved performance of the circular member, which was “much more resistant to local buckling”. Additionally, the use of ASTM A53 Grade B (ASTM 2002) pipe, which is readily available in the U.S. but not Canada, provides a suitably low nominal $F_{\text{y}}/F_{\text{u}}$ ratio of 0.58, which makes the connection much more resistant to fracture at the critical net section and a real design option without reinforcement. ASTM A53 Grade B can be compared to the popular ASTM A500 square HSS Grade C which has a nominal $F_{\text{y}}/F_{\text{u}}$ ratio of 0.81. North American-produced square/rectangular HSS are also known to have poor impact resistance properties since, unlike their European cold-formed counterparts, they are normally produced with no impact rating (Kosteski et al. 2005). Regardless of the section shape and steel grade chosen for energy-dissipative bracings, it is clearly necessary to specify a maximum permissible material strength on engineering drawings, as per Eurocode 8 (CEN 2004).

The use of fabricated, slotted circular HSS gusset plate connections, without reinforcement, is hence being further explored at the University of Toronto. Fabrication with the slot end un-welded (i.e. without an end return weld) is a very popular practice in North America, so special details are being investigated which still permit this concept yet provide a net area ($A_{\text{n}}$) equal to the gross area ($A_{\text{g}}$) at the critical cross-section, such as shown in Figure 10. As can be seen in Figure 10, a small gap is still provided at the tube end for fit-up, but the weld terminates at the end of the gusset plate which corresponds to a tube cross-section where the gross area applies.

## 4. CAST STEEL CONNECTIONS – SEISMIC APPLICATIONS

Cast steel joints have enjoyed a renaissance in Europe in conjunction with tubular steel construction, mainly as truss-type nodes in dynamically-loaded pedestrian, highway and railway bridges where fabricated nodes would have been fatigue-critical. Another popular application has been in tree-like tubular roof structures where the smooth lines of a cast node have great architectural appeal. Cast steel connectors to tubular braces under severe seismic load conditions have not been used to date, but cast steel connections represent a solution to the design dilemma of fabricated bracing member connections and these can be specially shaped to provide material where it is particularly needed. Types currently under investigation at the University of Toronto, which are designed to remain elastic under the full seismic loading regime, are shown in Figure 11. By mass-producing cast end connectors, to suit popular circular HSS bracing member sizes, an economic and aesthetic solution can be reached that still allows the use of regular HSS members and avoids the use of alternatives like buckling-restrained braces, which require pre-qualification by testing and a high level of quality assurance (AISC...
Cast end connectors thus represent another exciting development in the evolution of tubular steel construction. Current work in Canada on cast connectors to tubular members is summarised elsewhere by De Oliveira et al. (2006).

Further research on cast steel nodes, oriented to wide flange beam-to-column moment connections and primarily for seismic applications, is also underway at present at the University of Arizona. Another innovative connection solution for wide flange beam-to-HSS columns has been launched by California-based ConXtech Inc., termed the SMRSF. With this, a pre-engineered collar connection is fitted around 4” or 8” square HSS columns and bolted together on site, resulting in very fast construction times. Although it uses machined components that are shop-welded in place, rather than cast components, this connection is also pre-qualified for use as a fully-restrained, Special Moment Resistant Frame connection under the latest FEMA and AISC seismic provisions. Novel connection solutions such as these herald a potential paradigm shift in HSS construction technology.

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References


Figure 10: Fabricated connection detail using an over-slotted circular HSS but with A_n = A_g at the weld termination.

Figure 11: Cast steel connections to tubular braces for seismic load applications.