Ram Puthli*
Jeffrey A. Packer

DOI: 10.1002/stco.201310013

Structural design using cold-formed hollow sections

Dedicated to Prof. Dr. Ir. *Jaap Wardenier* on the occasion of his 70th birthday

This paper reviews the differences between the alternative types of structural hollow section products (cold-formed versus hot-finished) as they affect structural design in Europe, using the relevant product and design standards, with an emphasis on Rectangular Hollow Sections (RHS). Manufacturers of cold-formed structural hollow sections (CFSHS) are more numerous, so that their products are more widely available. Hot-finished structural hollow section (HFSHS) products are typically between 24 % and 54 % more expensive in Germany than their cold-formed counterparts, the lower differences being for large tonnages – a strong inducement in favour of CFSHS. The price difference may also vary within the European Union. The geometric and product properties which are distinctly unique to CFSHS are presented and shown to offer no restrictions in their use when in compliance with the appropriate clauses in the European standards. These are the influence of corner radii, welding in the corner area, material choice to avoid brittle fracture and suitability for welding CFSHS. A comparison of the structural performance of CFSHS and HFSHS shows equally efficient structural designs for both products. The points covered are the design of compression members – unfilled and concrete-filled, joint resistance – which typically governs selection of member sizes, as well as fatigue design, fire design and the resistance of braced steel frames to severe seismic loading. CFSHS are shown to be adequate under all these situations.

1 Introduction

The points discussed here are based upon long experience with structural hollow sections by the authors in research as well as design consultancy or as a checking engineer (Prüfingenieur in Germany) over several decades, covering their combined activities in Europe and North America, respectively.

This paper mainly discusses the aspects that would affect a structural designer using European standards, but could also apply internationally when making the correct product choice. As long as a structural steel designer and steel fabricator choose products in accordance with the product standards [1], [2] and comply with the relevant clauses in Eurocode 3 [3], [4], the designer should have no problem with the correct performance with either CFSHS or HFSHS. In this paper, commas are used as decimal points to follow the Eurocode practice. The reference to Eurocodes is based upon the British Standard and DIN Consolidated version of 2010, incorporating all Corrigenda up to 2010.

Full information on factors affecting the product properties and product performance are given by *Ritakallio* [5]. The next section mainly explains the clauses pertaining to CFSHS in Eurocode 3, Part 1-8 [4] referring to corner radii requirements and welding in this area as well as the chemical composition required to allow a smaller corner radius than generally permitted.

The subsequent sections compare the performance of CFSHS and HFSHS under different environmental actions and loading. These are accompanied by a sensible choice of section sizes and correct design approach, which would give a functional and economical solution. The points covered are: design of compression members (e.g. columns) over a range of effective lengths, considerations for joint resistance design, in addition to comparisons for fatigue design, fire design and resistance to earthquakes.

CFSHS have two advantages over HFSHS which are not directly related from the structural viewpoint. First, the unit costs per metre can be significantly lower. Written price quotations were obtained from three steel distributors in Germany for common sizes of RHS, in reasonable quantities, to grade S355J2H complying with EN 10210 [1] for HFSHS and EN 10219 [2] for CFSHS. The price premiums (in €/metre) for HFSHS over CFSHS ranged from 24 % to 54 % for the same RHS sizes, as shown in Table 1. For application as tension members, design is based on cross-sectional area. The cross-sectional areas for square and rectangular hollow sections are slightly different for HFSHS and CFSHS, depending on the wall slenderness ratio b/t or h/t, where b and h represent the outside width and height, and t represents the wall thickness, of an RHS. If (b or h)/t > 16 the difference is 1 % to 5 % (i.e. $1\,\% < (A_{HFSHS} - A_{CFSHS})/A_{CFSHS} < 5\,\%$). If (b or h)/t < 16 the difference is 5 % to 8 % (i.e. $5\,\% < (A_{HFSHS} - A_{CFSHS})/A_{CFSHS} < 8\,\%$), where A represents the cross-sectional area. These differences are sufficiently small that it usually results in the same member size selection. Thus, the price difference shown in Table 1 is very significant. For application as axially loaded columns or struts in compression (see Section 4 for details), for which one can compare the next higher CFSHS wall thicknesses that provide very similar buckling resistance, the price premiums are still positive and range from 5 % to 33 % (see

Received 19 January 2013, revised 16 February 2013, accepted 20 February 2013
*Corresponding author: puthli@kit.edu

Table 1. Cost premium (in €/metre) of hot-finished structural hollow sections (HFSHS) to EN 10210 [1] relative to cold-formed structural hollow sections (CFSHS) to EN 10219 [2], in Germany

RHS sizes	Distributor A	Distributor B	Distributor C
200 × 200 × 10 (HFSHS) / 200 × 200 × 10 (CFSHS)	+44 %	+54 %	+24 %
200 × 200 × 10 (HFSHS) / 200 × 200 × 12,5 (CFSHS)	+25 %	+33 %	+ 7 %
150 × 150 × 6,3 (HFSHS) / 150 × 150 × 6,3 (CFSHS)	+52 %	+30 %	+37 %
150 × 150 × 6,3 (HFSHS) / 150 × 150 × 8,0 (CFSHS)	+23 %	+ 5 %	+10 %

Table 1). In lattice girders (trusses) one should also bear in mind that about half the members are tension members and about half compression members.

The data above would be typical for orders covering the larger number of small industrial buildings in Germany. For large tonnages, the German price differentials for a dozen RHS sizes varying from 60 × 60 × 4 to 300 × 300 ×12,5 vary from 26 % to 45 %. However, these price differences could differ within the European Union.

Secondly, from the aesthetic viewpoint, cold-formed sections have a smooth surface, while hot-finished sections can be pitted, as shown in Figure 1, which can be an issue for architecturally exposed structural steel.

2 Material aspects of cold-formed structural hollow sections (CFSHS)

2.1 Welding in the cold-formed corners

Welding in the cold-formed corners of RHS produced to EN 10219 [2] is permissible providing the criteria of the July 2009 Corrigendum to EN 1993-1-8 [4] are fulfilled. Despite this, there have been misunderstandings in the interpretation of Clause 4.14 of EN 1993-1-8 [4] for welding in cold-formed zones. CIDECT (International Committee for the Development and Study of Tubular Construction) has therefore prepared a proposal for corrections to ECCS TC10, for submission to CEN (European Committee for Standardization).

Fig. 1. Hollow structural section bridge truss, with a close-up view of the surface finish on the hot-finished chord member

The text and note in EN 1993-1-8, Clause 4.14 should be corrected to make clear that, for the conditions specified, welding in the cold-formed corners and the adjacent zones is permitted.

According to CIDECT, the text should read:
Welding may be carried out in the corners and the adjacent cold-formed zones, provided that one of the following conditions is fulfilled:
- The cold-formed zones are normalized after cold-forming but before welding.
- The inside corner-to-thickness ratio r/t satisfies the relevant value obtained from Table 4.2 of EN 1993-1-8.

Further, the note should be presented more clearly:
Welding in the corners and within a distance of 5 t from the corners is also permitted for cold-formed hollow sections according to EN 10219, which do not satisfy the limits given in Table 4.2 of EN 1993-1-8, provided that the hollow sections satisfy the following additional requirements:
- The thickness ≤ 12,5 mm
- The steel is Aluminium-killed
- The quality is J2H, K2H, MH, MLH, NH or NLH
- The chemical analysis meets the following limits: C ≤ 0,18 %, P ≤ 0,020 % and S ≤ 0,012 %.

In other cases, welding in this area is only allowed if it can be shown by tests that welding can be permitted for that particular application.

Thus, if the conditions in Table 2 are met, which is the case for some EN 10219 [2] product, welding in the corners and adjacent cold-formed zones is automatically permitted. For other EN10219 [2] product which does not meet the geometric conditions in Table 2, but satisfies the chemical analysis given above, welding in the corners and adjacent cold-formed zones, such as shown in Figure 2, is permitted.

2.2 Brittle fracture

Eurocode 3, Part 1-10 [6] lacks clear rules for the toughness validation of structures with cold-formed hollow sections according to EN 10219 [2]. To allow the assessment of EN 10219 [2] cold-formed hollow section structures against brittle fracture, by using EN 1993-1-10, on the initiative of

Table 2. Conditions for welding cold-formed zones and adjacent material (EN 1993-1-8 Table 4.2 [3])

r/t	Strain due to cold forming (%)	Maximum thickness (mm)		
		Generally		Fully killed Aluminium-killed steel (Al ≥ 0,02 %)
		Predominantly static loading	Where fatigue predominates	
≥ 25	≤ 2	Any	Any	Any
≥ 10	≤ 5	Any	16	Any
≥ 3,0	≤ 14	24	12	24
≥ 2,0	≤ 20	12	10	12
≥ 1,5	≤ 25	8	8	10
≥ 1,0	≤ 33	4	4	6

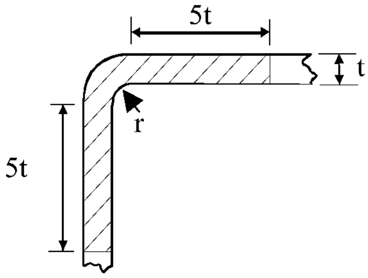

Table 3. Lowest permissible reference temperatures T_{Ed} and air (operating) temperatures T_{md}, for RHS CFSHS to EN 10219 [2], adapted from [8]

Square and Rectangular Structural Hollow Sections					
EN 10219 steel grade		Charpy V-notch impact test		Lowest design temperature[a)]	
		Test temperature (°C)	Impact energy (J)	T_{Ed} (°C)	T_{md} (°C)
S235	JRH	20	27	–85	–50
S355	J2H	–20	27	–95	–60
	MH	–20	40	–100	–65
	MLH	–50	27	–120	–85
S420	MH	–20	40	–90	–55
	MLH	–50	27	–110	–75
S460	MH	–20	40	–85	–50
	MLH	–50	27	–105	–70

a) $T_{md} = T_{Ed} - \Delta T\varepsilon_{cf}$ (this expression is valid if $\Delta T_r = \Delta T_\sigma = \Delta T_R = \Delta T\varepsilon = 0$: see EN 1993-1-10)
where T_{Ed} is the design reference temperature, as determined in EN 1993-1-10,
T_{md} is the design air temperature of the member, derived from T_{Ed} and corrected with the cold-forming factor $\Delta T\varepsilon_{cf}$,
$T\varepsilon_{cf}$ has been determined according to [7]

Notes: 1. The design temperature values are determined for wall thickness t = 12,5 mm. If the wall thickness is smaller, these values are on the safe side.
2. The design temperature values are determined for the serviceability limit state using $\sigma_{Ed} = 0{,}75f_y$. If the applied stress level is lower, these values are on the safe side.

Professors *Sedlacek* and *Wardenier* a European Commission Joint Research Centre Scientific and Policy Report for the evolution of Eurocode 3 was prepared [7]. That JRC document provides a conservative procedure, conforming to, and with proposals for amendments to, EN 1993-1-10 [6].

Eurocode 3, Part 1-10 Table 2.1 [6] gives allowable thicknesses for various steel grades and reference temperatures T_{Ed} , which at present only go down to –50 °C. In the new procedure a temperature shift due to cold forming, $\Delta T\varepsilon_{cf}$, to the reference temperature T_{Ed} is determined, and Table 2.1 [6] is extended down to –120 °C.

For cold-formed RHS, this temperature shift $\Delta T\varepsilon_{cf}$ is –35 °K for wall thicknesses t ≤ 16 mm and –45 °K for wall thicknesses 16 mm < t ≤ 40 mm. For cold-formed CHS, a temperature shift of –20 °K should be made in cases where the inside radius-to-thickness ratio is ≤ 15. In cases where the inside radius-to-thickness ratio is > 15, the value of $\Delta T\varepsilon_{cf}$ = 0 °K. More detailed information is given in [7]. *Ongelin* and *Valkonen* [8] have applied the procedure given in [7] and compiled a conservative example of the reference temperatures T_{Ed} and operating temperatures T_{md} for structures using cold-formed RHS to EN 10219 [2], see Table 3. As shown in this table (valid for wall thicknesses up to t = 12,5 mm), a large range of rectangular CFSHS satisfy the requirements for avoiding brittle fracture at very low operating temperatures T_{md} from –50 °C to –85 °C. If necessary, the applicability of rectangular CFSHS can be further extended by choosing lower Charpy V-notch testing temperatures. As an example, in Table 3 grade S355J2H has T_{Ed} = –95 °C whereas grade S355MLH has T_{Ed} = –120 °C. In principle the change in T_{Ed} here should be 30 °C (i. e. equal to the change in Charpy V-notch testing temperature) but the change in Table 3 is only 25 °C due to certain approximations [8]. Circular CFSHS to EN 10219 [2] can be applied at even lower temperatures since the temperature shift due to cold forming is smaller (or even zero) for them, as stated above.

2.3 Other considerations on product selection

The CE (Conformity European) mark is a mandatory European mark for specific products, including structural steel products, according to the 93/68/EEC marking directive of 22 July 1993 for the declaration of conformity. It is a symbol to show compliance with European legislation. Before a product is thus marked, the manufacturers have to satisfy the conformity assessment procedures according to the Construction Products Directive (CPD).

Stockholders have a duty of care that the CE marked products on offer have come from suppliers that have undergone the necessary attestation and been properly certified. This means that the CE mark for HFSHS and CFSHS should show compliance with EN 1090 [9], [10] as well as [1], [2], [3] and [4]. Therefore, CFSHS that are correctly specified, correctly produced and supplied with the appropriate test certification are suitable for all forms of construction.

Further information on product properties and selection can be gleaned from the paper by *Ritakallio* [5] in the August 2012 issue of this journal. Internationally, rectangular CFSHS from different sources can exhibit a diversity of quality regarding low temperature ductility in the corners [11], as well as potential corner cracking during hot-dip galvanizing [12]. However, these issues have been considered in detail by *Ritakallio* [5] who shows that well-manufactured rectangular CFSHS to EN 10219 [2] fulfil the required impact properties in all locations of the cross-section and in any orientation, and that corrosion protection can be safely performed by hot-dip galvanizing.

3 Influence of RHS corner radius

Cold-formed sections conforming to EN 10219 [2] have a smooth corner radius that increases with wall thickness. This has some advantages in comparison with the hot-finished sections, which have sharper and more irregular corners. In trusses (lattice girders) and welded connections to columns, corrosion protection is easier and it is reported by fabricators that welding around the corners of brace members when welding to a chord is easier with CFSHS, when welding on the flat face of the chord. In trusses, since the total weight of the braces is typically about 20 % of the total truss weight, brace-to-chord width ratios between 0,4 and 0,7 are common, which means that fillet welding of the brace to the "flat" of the chord is generally achievable. Brace-to-chord width ratios of 1,0, such as shown in Figure 2(b), are unusual in truss construction (but typical of Vierendeel frames with moment-resisting joints).

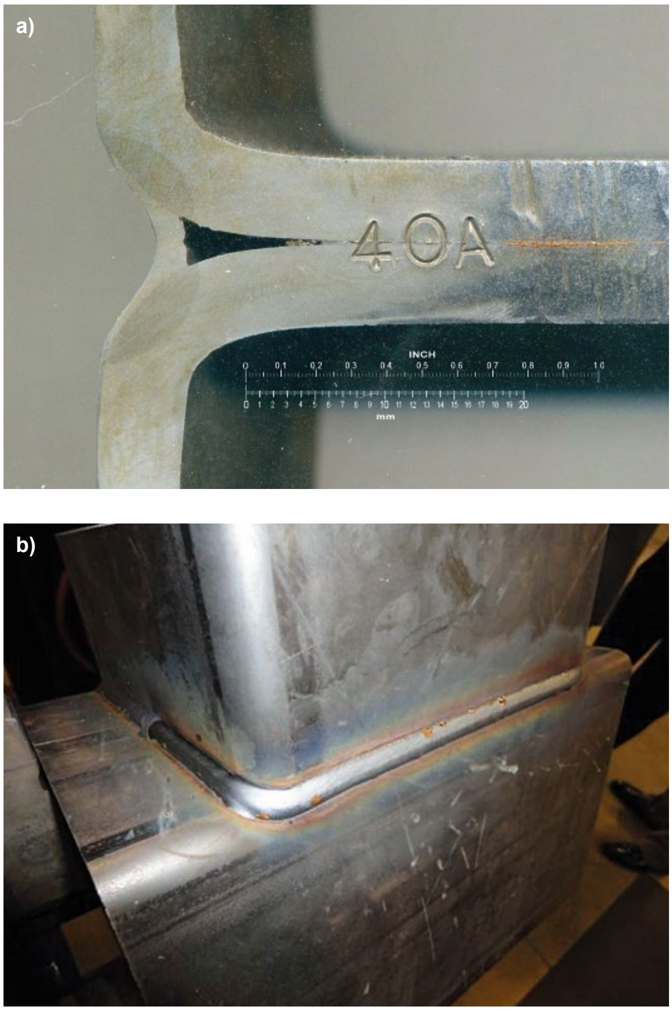

Fig. 2. Cold-formed RHS members welded in the corner regions: (a) cross-section of flare groove welding; (b) matched width T-joint

4 Design of compression members such as columns

CFSHS and HFSHS tension and flexural members are treated in an identical manner by EN 1993-1-1, but the lower level of residual stresses in HFSHS is recognized by assigning these sections to a more favourable buckling curve "a" when designing compression members. The buckling curves are shown in Figure 6.4 of Eurocode 3, Part 1.1 [3]. For a comparison of the performance under compression loading between CFSHS and HFSHS, the randomly selected popular sizes of 150 × 150 × 6,3 mm and 200 × 200 × 10 mm were chosen, for which price quotes were also obtained while writing this paper. Effective lengths of 3 m, 4 m, 5 m, 8 m and 10 m were chosen for the comparison. The section properties are taken from [1] and [2]. In Note 2B to Clause 6.1 (1) [3], the recommended value of the partial safety factor γ_{M1} of 1,0 is used for the buckling resistances in Table 4. The National Annex may define other numerical values for γ_{M1}, such as 1,1 in Germany, where the buckling resistances in these tables should be lowered proportionally. Table 4 shows that the hot-finished sections (using buckling curve "a") have 14 % to 28 % higher buckling resistances than the equivalent cold-formed sections (using buckling curve "c"), over the range considered.

Table 4. Buckling resistances according to EN 1993-1-1 for selected hot-finished [1] and cold-formed [2] square hollow sections to steel grade S355J2H, with cost comparison

CFSHS using buckling curve "c"				**HFSHS** using buckling curve "a"				**Relative performance**	
Size b=h	Nominal Thickness	Effective Length	Buckling resistance	Size b=h	Nominal Thickness	Effective Length	Buckling resistance	Buckling resistance	Cost (Lowest from Table 1)
[mm]	[mm]	[m]	[kN]	[mm]	[mm]	[m]	[kN]	HFSHS/CFSHS	HFSHS/CFSHS
150	6,3	3	**913**	150	6,3	3	**1094**	**1,20**	**≥ 1,30**
150	6,3	4	**739**	150	6,3	4	**937**	**1,27**	**≥ 1,30**
150	6,3	5	**580**	150	6,3	5	**741**	**1,28**	**≥ 1,30**
150	6,3	8	**288**	150	6,3	8	**347**	**1,20**	**≥ 1,30**
150	6,3	10	**196**	150	6,3	10	**230**	**1,17**	**≥ 1,30**
200	10	3	**2154**	200	10	3	**2450**	**1,14**	**≥ 1,24**
200	10	4	**1893**	200	10	4	**2281**	**1,20**	**≥ 1,24**
200	10	5	**1617**	200	10	5	**2040**	**1,26**	**≥ 1,24**
200	10	8	**931**	200	10	8	**1170**	**1,26**	**≥ 1,24**
200	10	10	**657**	200	10	10	**800**	**1,22**	**≥ 1,24**
150	8,0	3	**1123**	150	6,3	3	**1094**	**0,97**	**≥ 1,05**
150	8,0	4	**904**	150	6,3	4	**937**	**1,04**	**≥ 1,05**
150	8,0	5	**706**	150	6,3	5	**741**	**1,05**	**≥ 1,05**
150	8,0	8	**349**	150	6,3	8	**347**	**0,99**	**≥ 1,05**
150	8,0	10	**237**	150	6,3	10	**230**	**0,97**	**≥ 1,05**
200	12,5	3	**2559**	200	10	3	**2450**	**0,96**	**≥ 1,07**
200	12,5	4	**2237**	200	10	4	**2281**	**1,02**	**≥ 1,07**
200	12,5	5	**1898**	200	10	5	**2040**	**1,07**	**≥ 1,07**
200	12,5	8	**1077**	200	10	8	**1170**	**1,09**	**≥ 1,07**
200	12,5	10	**758**	200	10	10	**800**	**1,06**	**≥ 1,07**

Table 4 also presents the buckling resistances of the cold-formed sections with the next higher wall thicknesses (150 × 150 × 8 mm and 200 × 200 × 12,5 mm). These have up to about 20 % higher buckling capacities than their one-thickness-down CFSHS counterparts. Although the thinner hot-finished sections are between 14 % and 17 % lighter in weight, the use of HFSHS still involves 5 % to 33 % higher costs, as shown in Table 1.

5 Design of concrete filled hollow section columns

Concrete filled columns have the same buckling resistance, irrespective of the product (hot- or cold-formed), apart from the insignificant influence of different cross-sectional areas between the two hollow section products. The buckling curve to be chosen for both CHS and RHS in Eurocode 4 Part 1-1 [13] is „a" for reinforcement ratio ≤ 3 %, and „b" for reinforcement ratio between 3 % and 6 %. This is irrespective of whether it is a weak or strong axis for RHS. The reinforcement ratio is defined as the ratio between the cross-sectional area of the longitudinal steel reinforcement and the concrete cross-section.

6 Joint resistance in trusses and column connections

Joint resistance typically governs the selection of hollow section members, especially in trusses (lattice girders) and both cold- and hot-finished sections have the same joint resistance. This also applies to column connections. Eurocode 3, Part 1-8 [4] gives the design rules for hollow section joints in Clause 7.

7 Fatigue design

For fatigue design, hollow section joints have the same fatigue life in codes and standards, irrespective of the product (hot-finished or cold-formed), by both the classification method and the hot-spot stress method, despite well-known differences in residual stress levels. In both Eurocode 3, Part 1-9 [14] and CIDECT Design Guide No. 8 [15] there is no differentiation between the material types. All welded hollow section joints tend to be fatigue-critical, and major vehicular bridges nowadays favour CHS members with cast steel nodes in Europe, where weld detailing then becomes the critical issue [16].

8 Fire design

CIDECT Design Guide No. 4 [17] points out that any difference in the reduction in product properties between cold-formed and hot-rolled sections under fire loading is small, and hence there is no differentiation between the two product types in simple fire design, for both unfilled and concrete-filled hollow sections. Unprotected CFSHS and

Fig. 3. CFSHS in a column-testing furnace

Fig. 4. Welded through diaphragm connections, commonly used in Japan, to cold-formed RHS columns

HFSHS both have an identical fire resistance of 15 minutes for 8 mm wall thickness to 30 minutes for 25 mm wall thickness. Open sections have an even lower fire resistance. Additional measures are necessary to delay the rise in steel temperature when the building regulations require extended amounts of time in fire.

Under ambient temperature, the buckling resistance of all steel section compression members is based upon the imperfection constant α of 0,13, 0,21, 0,34, 0,49 and 0,76, corresponding to buckling curves a_0, a, b, c and d, respectively as given in Table 6.1 of Eurocode 3, Part 1-1 [3]. However, in Eurocode 3, Part 1-2 [18], the buckling resistance of all steel sections under compression loading (including unfilled hollow section columns) which are subjected to fire loading (Figure 3) is not dependent on imperfections. The value of α is based upon the nominal yield stress f_y, where $\alpha = 0{,}65 \cdot \sqrt{(235/f_y)}$.

In Eurocode 4, Part 1-2 [19], irrespective of the type of steel section (including hollow sections) or its material, all concrete filled hollow section columns subject to fire loading are assigned to the buckling curve "c" in Table 6.1 of Eurocode 3, Part 1-1 [3] for determination of their compressive resistance.

9 Earthquake design

Japanese building construction is governed by severe seismic loading criteria and proves an excellent case study for earthquake design of steel-framed buildings. In such structures, moment-resisting frames are normally used to resist the lateral loads and square hollow sections are the leading column shape. In Japan, as in nearly all the world, these are cold-formed. Extensive research and experience in Japan has justified this column choice and detailed design criteria are available for I-section beam-to-RHS column full-strength moment connections [20]. The most popular of these in Japan is the shop-welded through diaphragm connection, shown in Figure 4.

In North America, simple braced frames are more popular for the lateral load-resisting system and hollow sections are the typical choice for diagonal bracings. Such concentrically braced steel frames provide very good lateral strength and stiffness, with the bracings contributing to seismic energy dissipation by yielding in tension and buckling in compression during cyclic loading. CHS are the more popular shape for steel bracings but the deterioration of the brace properties/behaviour at mid-length during successive inelastic loading cycles has been a point of discussion, including the choice of product that one should specify for members in which high energy-dissipation is expected. Since the difference in performance between cold-formed, cold-formed + stress-relieved, and hot-finished CHS was a point of speculation, an experimental project was performed in Canada on long, 6,3-metre, full-scale, comparative braces, from these three different products, under seismic loading protocols (Figure 5). The hot-finished sections were imported from Europe and conformed to EN 10210 Grade S355J2H [1]. This study [21] found that the stockiness (d/t) of the tube cross-section was paramount. Here, d is the outside diameter and t the

Fig. 5. Cold-formed CHS brace during the compression cycle of a seismic loading protocol, in a 12 MN-capacity universal testing machine

wall thickness. Providing the tube wall slenderness was sufficiently low and met the requirements of the AISC seismic provisions [22] for highly ductile members (i. e. $d/t \leq 0{,}038\ E/f_y$, where E is the modulus of elasticity and f_y the nominal yield stress), then CHS braces of all product types met design requirements. All exhibited a similar hysteretic response and exceeded an inter-storey drift of 4 %, which is a value typically assumed for the "maximum considered earthquake" demand [23].

10 Concluding remarks

Cold-formed structural hollow sections (CFSHS) to EN 10219 are shown to be capable of meeting all structural design criteria, under a range of loading conditions, yet are significantly less expensive than their hot-finished (HFSHS) counterparts.

One structural advantage of HFSHS over CFSHS occurs in its use as a compression member, providing it is not concrete filled and also providing it is not subject to fire loading. In such situations, it is shown that the buckling strength of a HFSHS can be between 14 % and 28 % above that of its CFSHS counterpart, for the RHS size and column length range considered. The next higher wall thicknesses may need to be chosen for CFSHS to obtain comparable buckling strength. However, the large price premium paid for HFSHS over its CFSHS counterpart more than negates any advantage of HFSHS for compression members. In joint design, the joint resistance frequently dictates the section sizes – particularly in truss design – so any superior compressive resistance achievable by HFSHS is frequently not utilized anyway.

This paper lists all the arguments in discussion to show that, as long as the products are in accordance with European standards and a design is executed sensibly, cold-formed products perform efficiently in all structural applications. It is shown that CFSHS which are correctly specified, correctly produced and supplied with the appropriate European test certification are suitable for all forms of construction.

References

[1] EN 10210 (2006): Hot finished structural hollow sections of non-alloy and fine grain steels – Part 1: Technical delivery conditions; – Part 2: Tolerances, dimensions and sectional properties.

[2] EN 10219 (2006): Cold formed welded structural hollow sections of non-alloy and fine grain steels – Part 1: Technical delivery conditions; – Part 2: Tolerances, dimensions and sectional properties.

[3] EN 1993-1-1 (2010): Eurocode 3: Design of steel structures. Part 1-1: General rules and rules for buildings.

[4] EN 1993-1-8 (2010): Eurocode 3: Design of steel structures. Part 1-8: Design of joints; including Corrigendum dated July 2009.

[5] *Ritakallio, P. O.*: Cold-formed high-strength tubes for structural applications. Steel Construction, 5 (2012), No. 3, pp. 158-167.

[6] EN 1993-1-10 (2010): Eurocode 3: Design of steel structures. Part 1-10: Material toughness and through thickness properties.

[7] *Feldmann, M., Eichler, B., Kühn, B., Stranghöner, N., Dahl, W., Langenberg, P., Kouhi, J., Pope, R., Sedlacek, G., Ritakallio, P., Iglesias, G., Puthli, R. S., Packer, J. A., Krampen, J.*: Choice of steel material to avoid brittle fracture for hollow section structures. Report EUR 25400 EN, European Commission Joint Research Centre Scientific and Policy Report for the evolution of Eurocode 3, 2012.

[8] *Ongelin, P., Valkonen, I.*: Structural hollow sections. EN 1993 Handbook, Rautaruukki Oyj, Finland 2012.

[9] EN 1090-1 (2006): Steel and aluminium structural components. Part 1: General delivery conditions.

[10] EN 1090-2 (2007): Steel and aluminium structural components. Part 2: Technical requirements for steel structures.

[11] *Kosteski, N., Packer, J. A., Puthli, R. S.*: Notch toughness of internationally produced hollow structural sections. Journal of Structural Engineering, American Society of Civil Engineers 131 (2005), No. 2, pp. 279–286.

[12] *Packer, J. A., Chiew, S. P.*: Production standards for cold-formed hollow structural sections. Proceedings of the 13th International Symposium on Tubular Structures, Hong Kong 2010, China, pp. 413–421.

[13] EN 1994-1-1 (2010): Eurocode 4: Design of composite steel and concrete structures. Part 1-1: General rules and rules for buildings.

[14] EN 1993-1-9 (2010): Eurocode 3: Design of steel structures. Part 1-9: Fatigue.

[15] *Zhao, X. L., Herion, S., Packer, J. A., Puthli, R., Sedlacek, G., Wardenier, J., Weynand, K., van Wingerde, A. M., Yeomans, N.*: Design guide for circular and rectangular hollow section welded joints under fatigue loading. CIDECT Design Guide No. 8. Köln: TÜV-Verlag 2000.

[16] *Herion, S., de Oliveira, J. C., Packer, J. A., Christopoulos, C., Gray, M. G.*: Castings in tubular structures – the state of the art. Structures and Buildings, Proceedings of the Institution of Civil Engineers 163 (2010), No. SB6, pp. 403–415.

[17] *Twilt, L., Hass, R., Klingsch, W., Edwards, M., Dutta, D.*: Design guide for structural hollow section columns exposed to fire. CIDECT Design Guide No. 4. Köln: TÜV-Verlag 1994.
[18] EN 1993-1-2 (2010): Eurocode 3: Design of steel structures. Part 1-2: General rules – Structural fire design.
[19] EN 1994-1-2 (2010): Eurocode 4: Design of composite steel and concrete structures. Part 1-2: General rules – Structural fire design.
[20] *Kurobane, Y., Packer, J. A., Wardenier, J., Yeomans, N.*: Design guide for structural hollow section column connections. CIDECT Design Guide No. 9. Köln: TÜV-Verlag 2004.
[21] *Packer, J. A., Chiew, S. P., Tremblay, R., Martinez-Saucedo, G.*: Effect of material properties on hollow section performance. Structures and Buildings, Proceedings of the Institution of Civil Engineers 163 (2010), No. SB6, pp. 375–390.
[22] AISC 341-10 (2010): Seismic provisions for structural steel buildings. American Institute of Steel Construction, Chicago, USA.
[23] *Fell, B. V., Kanvinde, A. M., Deierlein, G. G., Myers, A. T.*: Experimental investigation of inelastic cyclic buckling and fracture of steel braces. Journal of Structural Engineering, American Society of Civil Engineers 135 (2009), No. 1, pp. 19–32.

Keywords: structural hollow sections; cold-formed; hot-finished; structural design; brittle fracture; welding; fire design; earthquake

Authors:
Prof. Dr. Ram Puthli, puthli@kit.edu
KIT Steel and Lightweight Structures,
Research Centre for Steel,
Timber and Masonry, Karlsruhe Institute of Technology,
Otto-Amman-Platz 1, 76131 Karlsruhe

Prof. Dr. Jeffrey A. Packer, jeffrey.packer@utoronto.ca
Department of Civil Engineering, University of Toronto,
35 St. George Street, Toronto, Ontario M5S 1A4, Canada