Canadian Journal of Civil Engineering

Concrete-Filled Hollow Structural Sections. I: Materials, Cross-Section Classification, and Concentrically Loaded Columns

Journal:	<i>Canadian Journal of Civil Engineering</i>
Manuscript ID	cjce-2020-0440.R3
Manuscript Type:	Article
Date Submitted by the Author:	31-Mar-2021
Complete List of Authors:	Tousignant, Kyle; Dalhousie University, Civil Engineering Packer, Jeffrey; University of Toronto,
Keyword:	Composite structures, hollow structural sections, concrete filling, columns, materials
Is the invited manuscript for consideration in a Special Issue? :	Not applicable (regular submission)

SCHOLARONE™
Manuscripts

Concrete-Filled Hollow Structural Sections. I: Materials, Cross-Section Classification, and Concentrically Loaded Columns

by

Kyle Tousignant*

Department of Civil and Resource Engineering, Dalhousie University, 1360 Barrington Street, Halifax, NS, B3H 4R2, Canada

and

Jeffrey A. Packer

Department of Civil & Mineral Engineering, University of Toronto, 35 St. George Street, Toronto, ON, M5S 1A4, Canada

Abstract

This paper reviews contemporary rules in CSA S16:19, AISC 360-16 and EN 1994-1-1 for concrete-filled hollow structural sections (HSS), covering materials, cross-section classification, and concentrically loaded columns. Results from 453 tests on axially compressed concrete-filled HSS members are compared to predicted strengths using current (CSA S16:19, AISC 360-16 and EN 1994-1-1) and recommended CSA S16 design equations (with limits of validity), and a first-order reliability analysis is performed in accordance with CSA S408-11. The recommendations herein are shown to maintain the current CSA S16:19 Clause 18.2 level of reliability for concrete-filled HSS compression members. Design examples are provided, and results are compared to the counterpart American code (AISC 360-16). This paper is Part I of a two-part series. Part II covers flexural members, beam-columns, tension and shear.

Keywords

Composite structures, hollow structural sections, concrete filling, columns, materials.

* Corresponding author, email: kyle.tousignant@dal.ca

1. Introduction

Since the advent of hollow structural sections (HSS) in Canada in the 1960s, in the form of both circular hollow sections (CHS) and rectangular hollow sections (RHS), there has been interest in filling the HSS void with concrete. Concrete-filled HSS, in relation to unfilled HSS, offer a number of advantages (BSC 1984; Wardenier et al. 2010; Zhao et al. 2019):

- An increase in the load-bearing capacity, in both compression and flexure (which is the topic of the current paper)
- A reduction in the quantity of steel required (hence a lighter HSS), for the same applied load
- A smaller diameter or outside dimension, for the same applied load, resulting in a reduction of the floor space occupied by a column as well as a lower surface area for coatings
- An increase in the fire resistance (either with or without external protection), and especially with the addition of steel reinforcing bars or steel fibres to the concrete
- An increase in the stiffness of columns in building frames
- A potential increase in the strength of connections. This was demonstrated for HSS-to-HSS truss connections by concrete filling of the chord (or “through”) member (Packer and Henderson 1997).

Relative to reinforced concrete (RC) columns, concrete-filled HSS columns also present advantages (BSC 1984):

- The elimination of formwork
- An appreciable decrease in the occupied floor space
- The omission, in many cases, of reinforcing bars
- The introduction of off-site fabrication, and faster on-site construction
- The introduction of tighter construction tolerances, typical of steel construction.

In Canada, concrete-filled HSS column design was covered in the allowable strength version of CSA S16 (CSA 1969). Therein, the axial compressive strength was taken, simply, as the addition of the allowable axial load on the steel section (A_s) plus the allowable axial load on the concrete area (A_c), with the latter computed using the slenderness ratio (KL/r) of the steel member.

Following its publication in the late 1970s, CIDECT Monograph No. 5 (CIDECT 1979) was used as the basis for a design manual on concrete-filled HSS columns in Canada, by Stelco (Stelco 1981). The design method, attributed to Guiaux and Janss (1970), represented a considerable advance in concrete-filled HSS column design by taking account of the interaction that develops between the steel and the concrete. As it involved an iterative process, graphical design aids were presented (Stelco 1981).

Other design methods followed in Europe, such as by Roik et al. (1975), Viridi and Dowling (1976), finally resulting in versions of Eurocode 4 (Bergmann et al. 1995; CEN 2004a). The current Canadian design approach for concrete-filled HSS columns (CSA 2019a), described more fully in Section 4, is similar to that of Eurocode 4. By taking advantage of the longitudinal bond between steel and concrete, and hence the sectional properties of the composite member, efficient designs have been achieved – particularly for concrete-filled CHS columns (Fig. 1) where the concrete confinement action can often be utilized.

Fig. 1. Toronto Pearson Airport, Terminal 1: (a) CHS columns filled with concrete; (b) concrete pumped from the bottom of the tubes through a spigot

2. Materials

2.1. HSS

The principal options for HSS in Canada are product manufactured to “CSA G40” in Class C or Class H (CSA 2018) or ASTM A500 (ASTM 2021). The relative availability of these alternatives varies around the country, as described in Table 6-8 of the CISC Handbook (CISC 2016). CSA G40 HSS is produced as cold-formed CHS and

RHS from round by an electric resistance welding (ERW) process, to a single grade of 350W with a specified minimum yield strength (F_y) of 350 MPa. A tolerance of -5% is permitted on wall thickness, with a further tolerance on mass (and hence effectively cross-sectional area) of -3.5%. These stringent tolerances ensure structural reliability comparable to competing hot-formed open sections, hence a design wall thickness (t) equal to the nominal thickness (t_0) is applied. Section properties are provided in the CISC Handbook (CISC 2016), where the outside corner radius of RHS (r_o) is taken to be $2t$. The cold-formed HSS end product (Class C) can be heat-treated to form an alternative Class H product by heating to 450° C or higher, followed by cooling in air. While Class H enables a higher compressive resistance for unfilled HSS columns, the higher cost associated with Class H is unwarranted for HSS once it is filled with concrete (as is shown in Section 4).

ASTM A500 (ASTM 2021) HSS is cold-formed from round by ERW in Canada, to CHS and RHS shapes. In theory, grades B and C are available for each of CHS and RHS, but the product is now routinely dual-certified to both grades B and C and hence only the higher grade (C) need be specified. This is reflected in the CISC Handbook Part 4, where compressive resistance tables are provided for unfilled HSS for only ASTM A500 grade C. The specified minimum yield strength, F_y , of RHS grade C is 345 MPa (50 ksi), but is only 317 MPa (46 ksi) for CHS grade C. A500 (ASTM 2018) permits a tolerance of -10% on wall thickness, with no tolerance on mass, weight, or cross-sectional area, resulting in HSS being produced routinely undersize since manufacturers interpret tolerances as allowances. CISC (2016) hence stipulates a design wall thickness of $t = 0.90t_0$ for structural design with A500 in Canada. This design wall thickness is incorporated already in the A500 section properties provided in the CISC Handbook, along with an assumed outside corner radius of $2t$ for RHS (CISC 2016).

One further HSS product, ASTM A1085 (ASTM 2015), is now heavily promoted in North America as a desirable substitute for ASTM A500. A1085 is directly equivalent to the G40 Class C (CSA 2018) product except that $F_y = 345$ MPa (50 ksi) and there is a minimum Charpy impact toughness provided (whereas a Charpy notch toughness rating needs to be called up with G40 Class C by an additional toughness Category number). Section properties for ASTM A1085 are identical to those for CSA G40. As a consequence, if ASTM A1085 is ordered in Canada one is directed to substitution with CSA G40 Class C.

CSA S16:19 Clause 18.2 (CSA 2019a) does not impose any limitations on HSS material. For comparison, AISC 360-16 Section I1.3 (AISC 2016) imposes a maximum specified yield strength of 525 MPa (for strength-

calculation purposes), and the corresponding limit in EN 1994-1-1 Clause 3.3(2) is 460 MPa (CEN 2004a). It is worth noting that these limits on HSS strength for concrete-filled HSS columns well-exceed the specified material strengths available and used in Canada ($F_y = 350$ MPa or less). AISC 360-16 Section I2.2a has a requirement that the HSS cross-sectional area is at least 1% of the total composite cross section. All of the HSS listed in the CISC Handbook (CISC 2016) meet this criterion.

If a concrete-filled HSS building column is not completely protected by external insulating materials in case of a fire, the HSS should be provided with vent holes in the walls. These holes serve to prevent the column from bursting, under the steam pressure generated by vaporization of the water locked in the concrete fill, during a fire. Pairs of vent holes should be placed near the top and bottom of each column, at every floor level, at a distance of 100 to 200 mm from the floor surfaces. Intermediate holes are required along the column length for storey heights in excess of 5 m, and the holes themselves should be at least 20 mm in diameter (Dutta et al. 1998).

2.2. Concrete

The concrete mix must have adequate plasticity, thus it is recommended that a higher sand and cement content be used and the maximum aggregate size be reduced, especially if reinforcing bars are placed inside the HSS. Recommendations for maximum aggregate diameter are available elsewhere (Dutta et al. 1998). Cementitious grouts, plasticizers and self-consolidating concretes are used, but additives which may cause corrosion of steel (e.g. calcium chloride) must be excluded (Dutta et al. 1998). Both normal- and light-weight concretes are options, the latter having a density (ρ_c) of 1840 kg/m³ or less (AISC 2016). To ensure mechanical bond between concrete and steel a non-shrink concrete mix is advisable.

Limitations do exist on the permissible range of concrete compressive strength, f'_c , however, due to the extent of experimental testing that has been performed. At high concrete strengths there is scant data on concrete-filled HSS columns and changes in behaviour have been observed. The scope of CSA S16:19 Clause 18.2.1.1(c) (CSA 2019a) covers $20 \text{ MPa} \leq f'_c \leq 80 \text{ MPa}$ for concentrically loaded columns, and $20 \text{ MPa} \leq f'_c \leq 40 \text{ MPa}$ for columns subjected to combined axial compression and bending. For normal-weight concrete, the design methods of EN 1994-1-1 (CEN 2004a) are applicable for all load actions with concrete having $20 \text{ MPa} \leq f'_c \leq 60 \text{ MPa}$. AISC 360-16 Section I1.3 (AISC 2016) limits the applicability, for all load actions, to: $21 \text{ MPa (3 ksi)} \leq f'_c \leq 69 \text{ MPa (10$

ksi) for normal-weight concrete, and $21 \text{ MPa (3 ksi)} \leq f'_c \leq 41 \text{ MPa (6 ksi)}$ for light-weight concrete, for strength calculations, in the absence of appropriate testing and analyses. Higher-strength concrete material properties may be used for stiffness calculations. The lower limit of 21 MPa for all concrete densities, and the upper limit of 41 MPa for light-weight concrete, are set to encourage good quality, readily available, grades of structural concrete (Commentary to AISC 360-16 Section I.3). Given the consideration of different concrete densities by AISC, it would seem prudent to adopt rounded ranges of $20 \text{ MPa} \leq f'_c \leq 70 \text{ MPa}$ for normal-weight concrete, and $20 \text{ MPa} \leq f'_c \leq 40 \text{ MPa}$ for light-weight concrete, in CSA S16. For normal-weight concrete, this is slightly more restrictive than the present CSA S16:19 (CSA 2019a) range of validity for concentrically loaded columns but, considering that nearly all columns in practice will be subject to combined loading, a validated wider range of application for beam-columns is valuable. The upper limit of 70 MPa, for normal-weight concrete, is also in accord with CSA S16 Clause 18.3 for partially encased concrete columns (CSA 2019a).

2.3. Reinforcing Bars

The presence of reinforcing bars will enhance the load-bearing capacity and fire resistance of concrete-filled HSS. However, CSA S16 Clause 18.2 (CSA 2019a) is intended to apply to the size range of manufactured HSS (not to very large tubular members) for which it has been shown that no mechanical shear connectors are required between the HSS and the concrete. The size of an HSS member chosen will very much dictate the potential for inclusion of longitudinal reinforcement in the concrete. It is recommended not to attempt to provide reinforcement in HSS smaller than 200 mm (diameter or width) for concrete-filling on site, and 160 mm for filling in the shop. The gap between reinforcing bars/stirrups and the HSS inside wall should be at least 1.5 to 2 times the maximum aggregate size, with a maximum concrete cover between 25 and 50 mm. A maximum longitudinal reinforcing bar area (A_{sr}) of $0.04A_c$ has been recommended for strength calculations (Dutta et al. 1998), and $0.06A_c$ in a more-recent Eurocode (CEN 2004a). AISC 360 (AISC 2016) notes that if longitudinal reinforcement is provided, internal transverse reinforcement is not required for strength.

CSA S16:19 Clause 18.2 (CSA 2019a) does not currently cater to the use of reinforcing bars in the concrete. EN 1994-1-1 (CEN 2004a) imposes no restrictions on reinforcing bar strength and only advises that the elastic modulus of the bars may be taken equal to the value for structural steel (E_s). AISC 360-16 (AISC 2016) sets an

upper limit of 550 MPa on the specified minimum yield strength (F_{yr}) of reinforcing bars, to be in accord with ACI 318 (2014). The Canadian concrete standard (CSA A23.3:19) (CSA 2019b) has an upper limit of 500 MPa on F_{yr} , although the most common strength used in Canada is still 400 MPa.

3. Classification of Cross Sections

In concrete-filled HSS, the concrete is well-known to change the local buckling mode of the HSS wall (Bradford et al. 1998, 2002) by only allowing it to buckle outwards, for both the cross section (Fig. 2b) and along the length of the HSS. This is reflected in the design provisions for concrete-filled HSS in Canada, via CSA S16:19 (CSA 2019a), which cover HSS with Class 1 or 2 cross-sections that can undergo complete plastification. In Clause 18.2.1.1 (CSA 2019a), such sections are identified as those having width- (or height-) to-thickness ratios (b_{el}/t and h_{el}/t , for RHS) or a diameter-to-thickness ratio (d_0/t , for CHS) (herein referred to, generally, as “width-to-thickness ratios”) less than or equal to the corresponding “maximum permitted” width-to-thickness ratio in Table 1 or Table 2. As shown in Fig. 2a, b_{el} and h_{el} (for RHS) are the clear width and height, respectively, of the RHS cross-sectional element, calculated by assuming an outside corner radius (r_0) = $2t$ (i.e. $b_{el} = b_0 - 4t$ and $h_{el} = h_0 - 4t$) (Clause 11.3.2) (CSA 2019a).

Table 1. Maximum width-to-thickness ratios for elements in concrete-filled HSS under axial compression

Shape	Specification	λ_p Compact	λ_r Noncompact	Maximum permitted
RHS	CSA S16 ¹	-	-	$1350 / \sqrt{F_y}$
	AISC 360 ²	$1010 / \sqrt{F_y}$	$1340 / \sqrt{F_y}$	$2240 / \sqrt{F_y}$
	EN 1994-1-1 ³	-	-	$800 / \sqrt{F_y}$
CHS	CSA S16 ¹	-	-	$28,000 / F_y$
	AISC 360 ²	$30,000 / F_y$	$38,000 / F_y$	$62,000 / F_y$
	EN 1994-1-1 ³	-	-	$21,200 / F_y$

Note: The above values assume a modulus of elasticity of steel (E_s) = 200,000 MPa for CSA S16:19 and AISC 360-16, and E_s = 210,000 MPa for EN 1994-1-1.

¹ CSA S16:19 Clause 18.2.1.1.

² AISC 360-16 Table I1.1a.

³ EN 1994-1-1 Table 6.3, but for RHS the overall height is used. For CHS, the limit is the same as for an unfilled CHS Class 3 (CEN 2004b).

Table 2. Maximum width-to-thickness ratios for elements in concrete-filled HSS under flexural compression

Shape	Specification	Elements	λ_p Compact/Class 2	λ_r Noncompact/Class 3	Maximum permitted
RHS	CSA S16 ¹	Flanges	$700 / \sqrt{F_y}$	-	$1350 / \sqrt{F_y}$
	AISC 360 ²	Flanges	$1010 / \sqrt{F_y}$	$1340 / \sqrt{F_y}$	$2240 / \sqrt{F_y}$
		Webs	$1340 / \sqrt{F_y}$	$2550 / \sqrt{F_y}$	$2550 / \sqrt{F_y}$
	EN 1994-1-1 ³	Flanges	$580 / \sqrt{F_y}$	$640 / \sqrt{F_y}$	-
		Webs	$1270 / \sqrt{F_y}$	$1900 / \sqrt{F_y}$	-
CHS	CSA S16 ¹		-	-	$28,000 / F_y$
	AISC 360 ²		$18,000 / F_y$	$62,000 / F_y$	$62,000 / F_y$
	EN 1994-1-1 ³		$16,500 / F_y$	$21,200 / F_y$	

Note: The above values assume $E_s = 200,000$ MPa for CSA S16:19 and AISC 360-16, and $E_s = 210,000$ MPa for EN 1994-1-1; AISC 360-16 uses the designations “compact” and “non-compact”; EN 1994-1-1 uses the designations “Class 2” and “Class 3”.

¹ CSA S16:19 Clause 18.2.1.1 (maximum permitted) and Commentary to Clause 18.2.3 (for RHS flanges).

² AISC 360-16 Table I1.1b.

³ EN 1994-1-1 refers to the limits for unfilled HSS in EN 1993-1-1 Table 5.2.

Fig. 2. Concrete-filled HSS (without reinforcing bars): (a) nomenclature; (b) cross-sectional buckling modes

The CSA S16:19 (CSA 2019a) maximum width-to-thickness ratios in Tables 1 and 2, and those of AISC 360-16 (AISC 2016), are greater than the corresponding limits for unfilled HSS. In AISC 360-16, concrete-filled HSS cross-sections are classified as “compact”, “noncompact” or “slender”. “Compact” sections can undergo complete plastification and possess a modest rotation capacity before local buckling (akin to CSA’s Class 1 and Class 2 sections), “noncompact” sections will yield but not undergo full plastification (akin to CSA’s Class 3 sections), and “slender” sections are expected to locally buckle before reaching yield (akin to CSA’s Class 4 sections). The maximum width-to-thickness ratios (or “limits”) for these classifications are summarized in Tables 1 and 2, where

λ_p = limiting width-to-thickness ratio for “compact” elements and λ_r = limiting width-to-thickness ratio for “non-compact” elements. The “maximum permitted” width-to-thickness ratios (in Tables 1 and 2) represent the bounds of existing, reliable experimental data (Zhao et al. 2019; AISC 2016). Unlike CSA S16:19, for RHS AISC 360-16 assumes an outside corner radius of $r_0 = 1.5t$ for calculating b_{el} and h_{el} (i.e. $b_{el} = b_0 - 3t$ and $h_{el} = h_0 - 3t$) [Section B4.1(b)]. This is a slightly more conservative calculation of the RHS wall slenderness than CSA’s approach.

In Table 1, for axial compression, there is very close correspondence between AISC’s RHS “noncompact” limit ($1340/\sqrt{F_y}$) and the maximum permitted by CSA ($1350/\sqrt{F_y}$), both of which are intended to avoid local buckling of the HSS wall. The reason for AISC 360 including a “compact” limit (a flexural categorization) on the cross section for the case of axial compression (Table 1), which is not done for unfilled HSS (AISC 360 Table B4.1a), relates to the ability of the HSS to provide enhanced concrete confinement. For concrete-filled CHS in axial compression (Table 1), the AISC “noncompact” limit is considerably more liberal than CSA’s ($38,000/F_y$ versus $28,000/F_y$), while the EN 1994-1-1 value ($21,150/F_y$) can be considered excessively conservative as it repeats the Class 3 limit for unfilled CHS in axial compression.

In Table 2, for flexural compression, EN 1994-1-1 can again be considered excessively conservative as it adopts the same classification limits for concrete-filled HSS as for unfilled HSS, from EN 1993-1-1 (CEN 2004b). AISC 360 has, unlike CSA S16, logically considered the different local buckling cases for RHS flanges and webs in flexural compression, with more liberal values for the latter. The CSA RHS compression flange “compact” (or Class 2) limit, described in the Commentary to CSA S16 Clause 18.2.3 (namely $700/\sqrt{F_y}$) is, as expected, greater than its CSA S16 Table 2 counterpart for unfilled RHS ($525/\sqrt{F_y}$), but is still significantly lower than the AISC 360 limit ($1010/\sqrt{F_y}$). The CSA S16 limit merely represents the scope of experimental validation performed by Lu and Kennedy (1994).

3.1. Proposed New Limits, Rationale, and Analysis

For axial compression of concrete-filled HSS, CSA S16 Table 1 can be extended by two further cases:

(i) For concrete-filled RHS, $b_{el}/t \leq 1350/\sqrt{F_y}$. This represents the current limit in CSA S16 Clause 18.2.1.1(a) and is almost identical to the AISC 360 limit for “noncompact” sections.

(ii) For concrete-filled CHS, $d_o/t \leq 38,000/F_y$. This is more generous than the existing CSA S16 Clause 18.2.1.1(b) but is supported by the work of Bradford et al. (2002).

For flexure of concrete-filled HSS, CSA S16 Table 2 can be extended by addition of the AISC 360 wall slenderness limits for “compact” (or Class 2) classification. This is because the concrete-filled HSS design method in CSA S16:19 requires HSS members to be Class 1 or Class 2 in flexure, in order that fully plastic stress blocks be developed in the steel and concrete at ultimate moment capacity. These limits below are again supported by Bradford et al. (1998, 2002). Thus:

(i) For flanges of concrete-filled RHS, $b_{el}/t \leq 1010/\sqrt{F_y}$.

(ii) For webs of concrete-filled RHS, $h_{el}/t \leq 1340/\sqrt{F_y}$.

(iii) For concrete-filled CHS, $d_o/t \leq 18,000/F_y$.

It can be shown that almost all of the CSA G40 (CSA 2018) Grade 350W Class C/H sections in the CISC Handbook (CISC 2016) (with $F_y = 350$ MPa), when concrete-filled, meet the above-prescribed cross-section slenderness limits for both axial compression and flexure. This is also true for ASTM A1085 (ASTM 2015) sections (with $F_y = 345$ MPa) when evaluated using the CSA G40 sections in the CISC Handbook, and for ASTM A500 Grades B and C (ASTM 2021) sections. Exceptions to this, for flexure, are listed in Table 3. All sections therein (in Table 3), except one, are Class 3 or 4 for flexure about either axis. Thus, Table 3 illustrates that the plastic method of design used in Clause 18.2 of CSA S16:19 can be applied to nearly all HSS in flexure. For axial compression, only two of the sections considered are Class 3 or 4 (i.e. HSS 254×254×4.8 and HSS 254×152×4.8), and only when made to ASTM A500 Grade C.

Table 3. Concrete-filled HSS in the CISC Handbook (CISC 2016) not meeting the cross-sectional requirements of Section 3.1 for flexure

Shape	Specification	Grade	Yield Strength (MPa)	Non-compliant sections
RHS	CSA G40.21 ¹	350W	350	-
	ASTM A1085 ²	50W	345	-
	ASTM A500 ³	B	317	-
	ASTM A500 ³	C	345	HSS 254×254×4.8 and HSS 254×152×4.8 ⁴
CHS	CSA G40.21 ¹	350W	350	HSS 406×6.4, HSS 356×6.4, and HSS 273×4.8
	ASTM A1085 ²	50W	345	HSS 406×6.4, HSS 356×6.4, and HSS 273×4.8
	ASTM A500 ³	B	290	HSS 508×6.4, HSS 457×6.4, HSS 406×6.4, HSS 356×6.4, and HSS 273×4.8
	ASTM A500 ³	C	317	HSS 508×9.5, HSS 508×6.4, HSS 457×6.4, HSS 406×6.4, HSS 356×6.4, and HSS 273×4.8

¹ CSA (2018).² ASTM (2015).³ ASTM (2021). Width-to-thickness and diameter-to-thickness ratios computed by using $t = 0.9t_0$.⁴ Class 3 or 4 for flexure about the minor axis; Class 1 or 2 for flexure about the major axis.

4. Axial Compression

The axial compressive resistance of a non-slender composite member (C_{rc}) in CSA S16:19 (CSA 2019a) is given as (Clause 18.2.2):

$$(1) \quad C_{rc} = \frac{(\tau\phi A_s F_y + \tau'\alpha_l\phi_c A_c f'_c)}{(1 + \lambda^{2n})^{1/n}}$$

where ϕ and ϕ_c = resistance factor for steel and concrete (= 0.9 and 0.65, respectively); τ = confinement reduction factor for steel (due to tensile hoop stresses that develop to confine the concrete); τ' = confinement enhancement factor for concrete; α_l = ratio of average stress in rectangular stress block to f'_c (= $0.85 - 0.0015f'_c \geq 0.73$); λ = non-dimensional slenderness parameter; and n = column curve parameter (= 1.80 for concrete-filled HSS).

Eq. (1) contains a summation of the HSS resistance and concrete resistance, with an adjustment for the potential increase in concrete resistance due to confinement. With concrete-filled RHS columns, the shape is not ideal for restraining the lateral deformation of the concrete, with the effect mainly limited to the corner areas [Fig. 2(b)], hence concrete confinement is discounted. On the other hand, for relatively stocky concrete-filled CHS columns, the lateral deformation of the concrete infill is restrained by the steel shell. There is a hoop effect around the concrete and the core is subjected to a triaxial state of stress, whereas the CHS is subjected to a bi-axial state of stress (CIDECT 1979). Thus, in Eq. (1), the factor $\tau = \tau' = 1.0$, except for CHS with $L/d_0 < 25$, for which:

$$(2) \quad \tau = \frac{1}{\sqrt{1 + \rho + \rho^2}}$$

$$(3) \quad \tau' = 1 + \left(\frac{25\rho^2\tau}{d_0/t} \right) \left(\frac{F_y}{\alpha_1 f'_c} \right)$$

where ρ = slenderness factor [= $0.02(25 - L/d_0)$], and:

$$(4) \quad \lambda = \sqrt{\frac{C_p}{C_{ec}}}$$

where C_p = nominal compressive resistance [i.e. Eq. (1) with $\phi = \phi_c = 1.0$] calculated with $\lambda = 0$, and C_{ec} = Euler buckling strength; i.e.:

$$(5) \quad C_{ec} = \frac{\pi^2 EI_e}{(KL)^2}$$

where EI_e = effective column stiffness, given in Table 4. The parameters A_s and A_c are shown in Fig. 1a.

Table 4. Effective column stiffness (EI_e)

Specification	EI_e	C_l
CSA S16 ¹	$E_s I_s + C_1 E_c I_c$	$\frac{0.60}{1 + C_{fs} / C_f}$
AISC 360 ²	$E_s I_s + E_s I_{sr} + C_1 E_c I_c$	$0.45 + 3 \left(\frac{A_s + A_{sr}}{A_g} \right) \leq 0.9$
EN1994-1-1 ³	$E_s I_s + E_s I_{sr} + C_1 E_c I_c$	$\frac{0.60}{1 + (C_{fs} / C_f) \varphi}$

Note: I_s , I_c and I_{sr} = moments of inertia of the steel section, uncracked concrete, and reinforcing bars; E_c = elastic modulus of concrete; C_l = stiffness modification factor for concrete; C_{fs} = factored sustained axial load; C_f = factored axial load; and A_g = gross cross-sectional area of composite section (= $A_s + A_c + A_{sr}$).

¹ CSA S16:19 Clause 18.2; $E_s = 200,000$ MPa and $E_c = (3300\sqrt{f'_c + 6900})(\rho_c/2300)^{1.5}$ (CSA S16:19 Clause 3.1).

² AISC 360-16 Clause I2.2b; $E_s = 200,000$ MPa and $E_c = 0.043 \rho_c^{1.5} \sqrt{f'_c}$ (AISC 360-16 Clause I2.1b).

³ EN1994-1-1 Clause 6.7.3.3(3) & (4); φ = creep coefficient (found in EN 1992-1-1); $E_s = 210,000$ MPa and $E_c = 22,000(f'_c/10)^{0.3}$ (EN1992-1-1 Table 3.1) (CEN 2004c).

The value of $n = 1.80$ in Eq. (1) represents an average between values of $n = 2.24$ and 1.34 for the “upper” and “lower” column curves in CSA S16:19 Clause 13.3.1, which are applicable to unfilled Class H and Class C HSS, respectively. The value of $n = 1.80$ also provides a good fit to experimental results (CSA S16:19 Commentary to Clause 18.2.2). Accordingly, C_{rc} for composite sections will be computed as the same value for HSS produced

to Class H and Class C, so there is no benefit in terms of resistance when using the former as there would be if the HSS was unfilled.

In AISC 360-16 (AISC 2016), λ is calculated by using Eq. (4) with C_p determined by using either Eq. (6) or Eq. (7) (with $\phi_a = 1.0$ and $\lambda = 0$), and C_{ec} [Eq. (5)] determined by using the value of EL_e for AISC 360 in Table 4. The expression for C_{rc} for “compact” cross sections (according to Table 1) is then given as [Clause I2.2b(a)]:

(a) For $\lambda^2 \leq 2.25$:

$$(6) \quad C_{rc} = \phi_a \left(A_s F_y + C_2 A_c f'_c + C_2 A_{sr} \frac{E_s}{E_c} f'_c \right) \cdot (0.658^{\lambda^2})$$

(b) For $\lambda^2 > 2.25$:

$$(7) \quad C_{rc} = \phi_a \left(A_s F_y + C_2 A_c f'_c + C_2 A_{sr} \frac{E_s}{E_c} f'_c \right) \cdot \left(\frac{0.877}{\lambda^2} \right)$$

where ϕ_a = resistance factor for compression (= 0.75) and C_2 = ratio of average concrete stress to f'_c accounting for confinement effects (= 0.85 for RHS and 0.95 for CHS). The parameters A_s , A_c and A_{sr} are shown in Fig. 3. It should be noted that the term $(E_s/E_c)f'_c$ in Eqs. (6) and (7) is contradictory to AISC’s plastic design approach.

Fig. 3. Concrete-filled HSS (with reinforcing bars): (a) RHS; (b) CHS

The AISC 360 approach differs philosophically from both the CSA S16 and EN 1994-1-1 design methods by:

- (a) having just one resistance factor for the composite column, rather than individual ones for separate materials;
- and (b) having two separate column resistance expressions, corresponding to “compact” and “noncompact” cross

sections in Table 1, for the case of axial compression. The compression resistances expressed by Eqs. (6) and (7), which apply to “compact” cross-sections per AISC 360, are applicable to 99.5% of the HSS in the CISC Handbook (CISC 2016) when concrete-filled. The exceptions to this are HSS 254×254×4.8 and HSS 254×152×4.8 when produced to ASTM A500 Gr. C, as noted in Section 3.1.

The EN 1994-1-1 (CEN 2004a) equation for C_{rc} according to the “simplified method of design” (Clause 6.7.3) is presented in Eq. (8):

$$(8) \quad C_{rc} = \left(\frac{A_s F_y}{\gamma} \eta_s + \frac{A_{sr} F_{yr}}{\gamma_r} + \frac{A_c f'_c}{\gamma_c} \left(1 + \eta_c \frac{t}{d_0} \frac{F_y}{f'_c} \right) \right) \cdot \chi$$

where γ , γ_r and γ_c = partial safety factor for the steel section, longitudinal reinforcing bars and concrete (= 1.0, 1.15 and 1.5, respectively) (which are approximately equal to the inverse of the resistance factor); η_s = confinement factor for steel; η_c = confinement factor for concrete; and χ = stability factor. For concrete-filled RHS columns, and concrete-filled CHS columns with $\lambda > 0.5$, $\eta_s = 1.0$ and $\eta_c = 0$; however, for concentrically loaded concrete-filled CHS columns with $\lambda \leq 0.5$:

$$(9) \quad \eta_s = 0.25(3 + 2\lambda) \leq 1.0$$

$$(10) \quad \eta_c = 4.9 - 18.5\lambda + 17\lambda^2 \geq 0$$

where λ is calculated by using Eq. (4) with C_p determined by using Eq. (8) (with $\gamma = \gamma_r = \gamma_c = 1.0$ and $\lambda = 0$, plus $\eta_s = 1.0$ and $\eta_c = 0$ for RHS and CHS) and C_{ec} [Eq. (5)] determined by using the value of EI_e for EN 1994-1-1 from Table 4. Eqs. (9) and (10) imply that, as λ increases, there is a decrease in confinement effects that are beneficial to column strength.

The stability factor χ in Eq. (8) is given in EN 1993-1-1 (CEN 2004b) Clause 6.3.1.2 as:

$$(11) \quad \chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} \leq 1.0$$

$$(12) \quad \Phi = 0.5 \left[1 + \alpha (\lambda - 0.2) + \lambda^2 \right]$$

where Φ = stability parameter; α = column curve parameter (= 0.21 when $A_{sr}/A_c \leq 3\%$, and 0.34 when $3\% < A_{sr}/A_c \leq 6\%$, corresponding to the European “column curves a and b”, respectively). The EN 1994-1-1 “simplified method of design” is only valid for concrete-filled HSS columns with $\lambda \leq 2.0$ (Clause 6.7.3.1) (CEN 2004a).

4.1. Proposed New Equation and Rationale

As shown in Section 4:

- AISC 360-16 (AISC 2016) and EN 1994-1-1 (CEN 2004a) consider the contribution of longitudinal reinforcing bars when calculating C_{rc} , whereas CSA S16:19 (CSA 2019a) does not. It is noteworthy that the capacity of the reinforcing bars *is* included in the formulae for C_{rc} that cover partially encased and encased composite columns in CSA S16:19 Clauses 18.3.2 and 18.4.2, respectively (CSA 2019a).
- In AISC 360-16, the average intensity of the concrete stress block is taken as $0.85f'_c$ (when confinement effects are omitted). In previous editions of CSA S16 (e.g. CSA 2001), that value was also taken as $0.85f'_c$ with a resistance factor of $\phi_c = 0.60$ for the concrete. However, in CSA S16-09 (CSA 2009), it was modified to $\alpha_l f'_c$, with $\phi_c = 0.65$. This change was hence penalizing (relative to CSA 2001) for concretes with $f'_c > 43.5$ MPa. However, the value used today in CSA S16:19 ($\alpha_l f'_c$), when compared with that in AISC 360-16 ($0.85 f'_c$) and EN 1994-1-1 ($1.0f'_c$), needs to be assessed as a contributor to the overall composite column resistance, C_{rc} (discussed later).
- CSA S16:19 (CEN 2019a) and EN 1994-1-1 (CEN 2004a) use unique resistance factors for HSS steel, concrete and reinforcing bars, rather than a single resistance factor for the conversion of nominal strength to resistance. This approach, though debatable, is commensurate with equations for encased columns (in CSA S16:19 Clauses 18.3.2 and 18.4.2) and RC columns (in CSA A23.3:19).
- Fig. 4 shows a comparison of the “stability factors” [i.e. the term in the denominator of Eq. (1) and to the right of the dot operator in Eqs. (6), (7) and (8)] in all three codes. However, it should be noted that composite column stability factors are founded on those for unfilled columns, which in turn use different methodologies for fitting “column curves” to available data (Bjorhovde and Birkemoe 1979). Furthermore, the subsequent choice and application of resistance (or partial safety) factors affects the final correlation of design curves to test data.
- Based on the forgoing discussion, it is proposed to include a “rebar term” for calculating C_{rc} ; i.e.:

$$(13) \quad C_{rc} = \frac{(\tau\phi A_s F_y + \tau'\alpha_l\phi_c A_c f'_c + \phi_r A_{sr} F_{yr})}{(1 + \lambda^{2n})^{1/n}}$$

where ϕ_r = resistance factor for reinforcing bars (= 0.85) (CSA S16:19 Clause 13.1) and $\alpha_l = 0.85 - 0.0015f'_c \geq 0.75$. The lower limit of 0.75 is a consequence of adopting an upper limit of 70 MPa for f'_c , for normal-weight concrete. Addition of the term $E_s I_{sr}$ to the CSA S16:19 EI_e equation in Table 4 would also be required. The proposed nominal reinforcing bar stress term in Eq. (13) [i.e. F_{yr}] is based on the assumption of a fully plastic cross-section. It differs from the corresponding term in AISC 360-16 [i.e. $(E_s/E_c)f'_c$ in Eqs. (6) and (7)], which is based on compatibility. For normal-density concrete ($\rho_c = 2400 \text{ kg/m}^3$), with $f'_c = 35 \text{ MPa}$, it can be shown that $(E_s/E_c)f'_c = 234 \text{ MPa}$ (using values of E_s and E_c in footnote 2 of Table 4). This is much less than $F_{yr} = 400 \text{ MPa}$ (see Section 2) and is, hence, conservative.

Fig. 4. Comparison of stability factors in CSA S16:19, AISC 360-16 and EN 1994-1-1

A graphical demonstration of the effect of concrete filling an HSS, according to Eq. (13) but omitting reinforcing bars, is given in Fig. 5. An HSS 435×16 (CHS) parent tube (considered in both CSA G40 Class C and Class H) is flattened into an HSS 356×356×16 (RHS), and thus has the same steel area, A_s , and the same specified yield stress, F_y . The difference in the compressive resistance unfilled, C_r , and filled with normal-density concrete having $f'_c = 35 \text{ MPa}$, C_{rc} , is quite evident. For short- and intermediate-length columns there is a considerable gain by using CHS over RHS, because of the advantageous concrete-confinement effect and the additional area of enclosed concrete. For long columns, the concrete infill provides little benefit, for both CHS and RHS. In fact, because the column curve parameter is reduced from $n = 2.24$ for Class H, to $n = 1.8$ for concrete-filled HSS, it is

theoretically possible to obtain a lower axial compressive resistance for a very long, concrete-filled Class H column than a very long, unfilled Class H column. However, this will not be the case in practice.

Fig. 5. Axial compression resistance of unfilled and concrete-filled HSS with the same steel area

4.2. Evaluation of Current and Proposed Equations

The current (CSA S16:19, AISC 360-16 and EN 1994-1-1) and proposed C_{rc} equation(s) are evaluated herein by comparison to 453 tests on concrete-filled HSS. These tests were extracted from a larger database of tests on concentrically and eccentrically loaded concrete-filled HSS members compiled by Thai et al. (2019).

The database (Thai et al. 2019) was reduced by the Authors to only include tests meeting the parameter limits of validity for local slenderness [d_o/t , b_{el}/t (and h_{el}/t)] and concrete strength (f'_c) outlined, for each code/equation, in Sections 3 and 3.1. Concrete cube strength (when reported) was converted to cylinder strength (i.e. f'_c) by using the relationship outlined in EN 1992-1-1 (CEN 2004c). Tests with missing or incomplete information (often, E_s) were removed and, for EN 1994-1-1, tests with $\lambda > 2$ were also omitted (see Section 4) (CEN 2004a). Finally, F_y was limited to $300 \text{ MPa} \leq F_y \leq 450 \text{ MPa}$ to reflect the range of typical measured yield strengths for CSA G40, ASTM A500 and ASTM A1085 HSS.

Predicted values for the nominal compressive strength (C_{nc}) and design resistance (C_{rc}) were hence computed for valid tests by using Eqs. (1), (6)/(7) and (8), for CSA S16:19, AISC 360-16 and EN 1994-1-1, and the proposed

equation [Eq. (13)], respectively. For C_{nc} , resistance factors in Eqs. (1), (6)/(7), and (13), and the partial safety factors in Eq. (8), were taken as unity (i.e. $\phi = \phi_c = \phi_a = 1.0$, and $\gamma = \gamma_c = 1.0$) and, for C_{rc} , the equations were used as written. All calculations assumed an outside corner radius for RHS of $2t$ and a concrete density, $\rho_c = 2400$ kg/m³. Additionally, for the CSA (existing and proposed) and EN equations, C_{fs}/C_f was taken as 0 in calculating EI_e for all tests, and φ (the creep coefficient in EN 1994-1-1) was taken as 1.0.

4.2.1. Test-to-Predicted Strength Statistics

Plots comparing the actual (experimental) column strengths (A) to C_{nc} (on the left) and C_{rc} (on the right) are presented in Figs. 6 – 13. Note that Figs. 6 – 9 cover concrete-filled RHS, while Figs. 10 – 13 cover concrete-filled CHS. The number of available tests (“no. tests”) plus mean values and coefficients of variation (CVs) of A/C_{nc} (termed δ_P and V_P , respectively) and A/C_{rc} are presented in each figure. The parameters δ_P and V_P are further summarized in Table 5.

Draft

Fig. 6. Correlation of existing CSA S16:19 design method [Eq. (1)] with tests on concrete-filled RHS: (a) actual strength vs. nominal predicted strength; (b) actual strength vs. predicted design strength

Fig. 7. Correlation of AISC 360-16 design method [Eqs. (6) and (7)] with tests on concrete-filled RHS: (a) actual strength vs. nominal predicted strength; (b) actual strength vs. predicted design strength

Fig. 8. Correlation of EN 1994-1-1 design method [Eq. (8)] with tests on concrete-filled RHS: (a) actual strength vs. nominal predicted strength; (b) actual strength vs. predicted design strength

Fig. 9. Correlation of proposed design method for CSA S16 [Eq. (13)] with tests on concrete-filled RHS: (a) actual strength vs. nominal predicted strength; (b) actual strength vs. predicted design strength

Fig. 10. Correlation of existing CSA S16:19 design method [Eq. (1)] with tests on concrete-filled CHS: (a) actual strength vs. nominal predicted strength; (b) actual strength vs. predicted design strength

Fig. 11. Correlation of AISC 360-16 design method [Eqs. (6) and (7)] with tests on concrete-filled CHS: (a) actual strength vs. nominal predicted strength; (b) actual strength vs. predicted design strength

Fig. 12. Correlation of EN 1994-1-1 design method [Eq. (8)] with tests on concrete-filled CHS: (a) actual strength vs. nominal predicted strength; (b) actual strength vs. predicted design strength

Fig. 13. Correlation of proposed design method for CSA S16 [Eq. (13)] with tests on concrete-filled CHS: (a) actual strength vs. nominal predicted strength; (b) actual strength vs. predicted design strength

Table 5. Test-to-nominal predicted strength statistics and reliability analysis results for concentrically loaded concrete-filled RHS and CHS columns for load combination $\alpha_D D + \alpha_L L$ over $0.1 \leq L/D \leq 3$ considering available tests within each respective range of applicability

Shape	Parameter	CSA S16:19 Eq. (1)	AISC 360-16 Eq. (6) or (7)	EN 1994-1-1 Eq. (8)	Proposed CSA S16 Eq. (13)
RHS	no. tests	190	149	130	182
	δ_G	0.97	0.97	0.97	0.97
	δ_M	1.24	1.24	1.24	1.24
	δ_P	1.26	1.24	1.17	1.27
	δ_P/ϕ_{req}	1.60	1.65	1.35	1.60
	V_G	0.02	0.02	0.02	0.02
	V_M	0.10	0.10	0.10	0.10
	V_P	0.16	0.15	0.15	0.16
	δ_R/ϕ_{req}	1.92	1.98	1.62	1.92
	V_R	0.19	0.18	0.18	0.19
	α_D	1.25	1.20	1.35	1.25
	α_L	1.50	1.60	1.50	1.50
	δ_D	1.05	1.05	1.05	1.05
	δ_L	0.78	0.78	0.78	0.78
	V_D	0.10	0.10	0.10	0.10
	V_L	0.32	0.32	0.32	0.32
	V_S	0.10 – 0.25	0.10 – 0.25	0.10 – 0.25	0.10 – 0.25
	β^+	3.76 – 4.24	4.03 – 4.43	3.36 – 3.88	3.76 – 4.24
CHS	no. tests	263	238	228	241
	δ_G	0.97	0.97	0.97	0.97
	δ_M	1.24	1.24	1.24	1.24
	δ_P	1.10	1.22	1.13	1.09
	δ_P/ϕ_{req}	1.43	1.62	1.34	1.41
	V_G	0.02	0.02	0.02	0.02
	V_M	0.10	0.10	0.10	0.10
	V_P	0.17	0.13	0.15	0.17
	δ_R/ϕ_{req}	1.72	1.95	1.61	1.70
	V_R	0.20	0.17	0.18	0.20
	α_D	1.25	1.20	1.35	1.25
	α_L	1.50	1.60	1.50	1.50
	δ_D	1.05	1.05	1.05	1.05
	δ_L	0.78	0.78	0.78	0.78
	V_D	0.10	0.10	0.10	0.10
	V_L	0.32	0.32	0.32	0.32
	V_S	0.10 – 0.25	0.10 – 0.25	0.10 – 0.25	0.10 – 0.25
	β^+	3.34 – 3.63	4.09 – 4.64	3.34 – 3.85	3.30 – 3.57

By comparing the values of δ_P and V_P in Table 5, for concrete-filled RHS columns, the proposed C_{nc} equation [based on Eq. (13)] can be seen to have almost the same accuracy and precision as AISC 360-16 [based on Eq. (6) or (7)], but marginally less favourable results than EN 1994-1-1 [based on Eq. (8)] in that regard. For concrete-filled CHS columns, the proposed C_{nc} equation is markedly more accurate than AISC 360's; however, the inherent scatter (V_P) is somewhat greater. These statistics, for the proposed equation, are similar to the results for EN 1994-

1-1 [based on Eq. (8)]. For both RHS and CHS, the proposed C_{rc} equation provides test-to-predicted strength statistics that are nearly identical to the current CSA S16:19 equation.

4.2.2. Reliability Analysis by North American Methods

To assess whether adequate safety margins exist in CSA S16:19, AISC 360-16, EN1994-1-1 and the proposed method [using Eq. (13)] for C_{rc} , one can check that a minimum reliability/safety index of $\beta^+ = 3.0$ [per Annex B.4 of CSA S16:19 (CSA 2019) and Chapter B of the AISC 360-16 Commentary (AISC 2016)] is achieved across a range of live-to-dead load (L/D) ratios by using an approximate first-order reliability method (FORM) analysis. Because researchers (Ellingwood et al. 1980; Kariyawasam et al. 1997) have found β^+ to be relatively insensitive to the variable loads on a structure, the FORM analysis herein is undertaken by considering the basic load combination $\alpha_D D + \alpha_L L$ only, in which (Schmidt and Bartlett 2002a; MacPhedran and Grondin 2011):

$$(14) \quad \beta^+ = \frac{1}{\sqrt{V_R^2 + V_S^2}} \ln \left[\frac{\delta_R \left(\alpha_D + \alpha_L (L/D) \right)}{\phi_{req} \left(\delta_D + \delta_L (L/D) \right)} \right]$$

where δ_R = bias coefficient for the resistance; V_R = coefficient of variation (CV) of δ_R ; V_S = CV of the total load effect; ϕ_{req} = overall resistance factor; α_D and α_L = load factors for dead and live loads, respectively [e.g. per NRC (2015) or ASCE (2016)]; and δ_D and δ_L = bias coefficients for dead and live loads, respectively. Eq. (14) is expressed in “non-dimensional L/D form”, for simplicity.

Additionally, since the CSA S16:19, EN 1994-1-1 and proposed formulae for C_{rc} in Section 4 of this paper have multiple resistance factors (for steel and concrete), rather than one overall resistance factor (like AISC 360-16), Eq. (14) is implemented by using the bias coefficient for the resistance, i.e.:

$$(15a) \quad \delta_R = \delta_G \delta_M \delta_P$$

expressed as:

$$(15b) \quad \frac{\delta_R}{\phi_{req}} = \delta_G \delta_M \text{mean}(A/C_{rc})$$

with:

$$(15c) \quad V_R = \sqrt{V_G^2 + V_M^2 + V_P^2}$$

where δ_G , δ_M , and δ_P , are the bias coefficients of the geometric, material, and professional factors, respectively, and V_G , V_M , and V_P , are the associated CVs. In other words, Eq. (14) is implemented by using the mean value of $A/C_{rc} = \delta_P/\phi_{req}$ to calculate the δ_R/ϕ_{req} .

Eq. (14) is in accord with the Annex B.2.5 Approximate Method of CSA S408-11 (CSA 2011), and considers variations in material and geometric parameters, as well as the so-called professional factor.

In this paper, δ_M and V_M consider the material overstrength of both the HSS member and the concrete infill. Liu (2016) reported $\delta_M = 1.28$ and 1.37 , and $V_M = 0.09$ and 0.08 , for ASTM A500 dual-certified Grade B/C RHS and CHS, respectively. The values of $\delta_M = 1.24$ and $V_M = 0.10$ can be adopted from Bartlett (2007) for cast-in-place concretes. Given that the combined values of δ_M and V_M depend on the relative contributions of the HSS and the concrete to the resistance of the member, values of $\delta_M = 1.24$ and $V_M = 0.10$ (which represent a lower and upper bound, respectively) are adopted herein.

Typical distribution parameters for HSS geometric properties (including A_s) are given in Table 5 of Schmidt and Bartlett (2002b). Since the geometric variation of the concrete infill directly depends on that of the HSS, values of $\delta_G = 0.97$ and $V_G = 0.02$ for HSS from that study are adopted herein. These variables again represent a lower bound for the mean and an upper bound for the CV, but are very similar to the statistical parameters reported much earlier by Kennedy and Gad Aly (1980).

The dead load effect was assigned $\delta_D = 1.05$ and $V_D = 0.10$ and the live-load effect was assigned $\delta_L = 0.78$ and $V_L = 0.32$ (where V_D and V_L = CV of δ_D and δ_L , respectively) (Schmidt and Bartlett 2002a). To fully incorporate the effects of these load variances (considering the load combination $\alpha_D D + \alpha_L L$), the following equation is adopted (per Clause 14.15.2.3 of CSA 2019c):

$$(16) \quad V_s = \frac{\sqrt{(\alpha_D V_D)^2 + (\alpha_L V_L (L/D))^2}}{\alpha_D + \alpha_L (L/D)}$$

Eqs. (14) and (16) are applied to determine the range of the reliability index, β^+ , for a typical range of L/D ratios. For CSA S16:19 and the proposed method [using Eq. (13)], values of $\alpha_D = 1.25$ and $\alpha_L = 1.50$ are adopted [per the National Building Code of Canada (NBCC)] (NRC 2015). For AISC 360-16, values of $\alpha_D = 1.20$ and $\alpha_L = 1.60$ [per ASCE 7-16] (ASCE 2016) are adopted. Results are shown in Figs. 14a and 14b for concrete-filled

RHS and CHS, respectively, for $0 \leq L/D \leq 3$; however, the discussion below (and the comparisons in Table 5) consider $0.1 \leq L/D \leq 3$ (where it is conservative to assume that the basic load combination of $\alpha_D D + \alpha_L L$ governs).

Fig. 14. Reliability index vs. L/D ratio for CSA S16:19 [Eq. (13)], AISC 360-16 [Eqs. (6) and (7)], and the proposed design method [Eq. (13)] for: (a) concrete-filled RHS; (b) concrete-filled CHS

For design according to CSA S16:19 [Eq. (1)], β^+ ranges from 3.76 to 4.24 for concrete-filled RHS. The current CSA S16:19 provisions, using $\phi_s = 0.90$ and $\phi_c = 0.65$, can hence be deemed conservative (i.e. to provide $\beta^+ > 3.0$). For concrete-filled CHS, β^+ ranges from 3.34 to 3.63, and the same conclusion stands.

For design according to AISC 360-16 [Eq. (6) or (7)], β^+ ranges from 4.03 to 4.43 for concrete-filled RHS and, for concrete-filled CHS, β^+ ranges from 4.09 to 4.64. Because $\beta^+ > 3.0$ for both cases, it can be concluded that the AISC 360-16 C_{rc} equation is conservative.

For design according to the proposed method [Eq. (13)], β^+ ranges from 3.76 to 4.24 and 3.30 to 3.57 for concrete-filled RHS and CHS, respectively. These values are nearly identical to those of CSA S16:19, since the two methods are identical (without reinforcing bars) except for small differences in parameter ranges.

For design according to EN 1994-1-1 (CEN 2004a), with $\alpha_D = 1.35$ and $\alpha_L = 1.50$, β^+ ranges from 3.36 to 3.88 and 3.34 to 3.85 for concrete-filled RHS and CHS, respectively.

To ensure that including only test results that fall within the range of applicability of each approach does not unjustly skew the data, an alternative analysis was performed by using a single data set to compute $A/C_{rc} = \delta_P/\phi_{req}$, and V_P (and hence ρ_R and V_R). For concrete-filled RHS, the single data set contained 130 tests that fell within the range of applicability of all four approaches considered. For concrete-filled CHS, the single data set contained 228 tests. The reliability analysis results are summarized in Table 6. By comparing the β^+ values in Tables 5 and 6, there is virtually no impact of using the different data sets.

Table 6. Test-to-nominal predicted strength statistics and reliability analysis results for concentrically loaded concrete-filled RHS and CHS columns for load combination $\alpha_D D + \alpha_L L$ over $0.1 \leq L/D \leq 3$ considering available tests within all ranges of applicability

Shape	Parameter	CSA S16:19 Eq. (1)	AISC 360-16 Eq. (6) or (7)	EN 1994-1-1 Eq. (8)	Proposed CSA S16 Eq. (13)
RHS	no. tests	130	130	130	130
	δ_G	0.97	0.97	0.97	0.97
	δ_M	1.24	1.24	1.24	1.24
	δ_P	1.26	1.25	1.17	1.26
	δ_P/ϕ_{req}	1.56	1.66	1.35	1.56
	V_G	0.02	0.02	0.02	0.02
	V_M	0.10	0.10	0.10	0.10
	V_P	0.15	0.15	0.15	0.15
	δ_R	1.88	2.00	1.62	1.88
	V_R	0.18	0.18	0.18	0.18
	α_D	1.25	1.20	1.35	1.25
	α_L	1.50	1.60	1.50	1.50
	δ_D	1.05	1.05	1.05	1.05
	δ_L	0.78	0.78	0.78	0.78
	V_D	0.10	0.10	0.10	0.10
	V_L	0.32	0.32	0.32	0.32
	V_S	0.10 – 0.25	0.10 – 0.25	0.10 – 0.25	0.10 – 0.25
	β^+	3.73 – 4.27	4.05 – 4.46	3.36 – 3.88	3.73 – 4.27
CHS	no. tests	228	228	228	228
	δ_G	0.97	0.97	0.97	0.97
	δ_M	1.24	1.24	1.24	1.24
	δ_P	1.08	1.22	1.13	1.08
	δ_P/ϕ_{req}	1.40	1.62	1.34	1.40
	V_G	0.02	0.02	0.02	0.02
	V_M	0.10	0.10	0.10	0.10
	V_P	0.16	0.12	0.15	0.16
	δ_R	1.68	1.95	1.62	1.68
	V_R	0.19	0.16	0.18	0.19
	α_D	1.25	1.20	1.35	1.25
	α_L	1.50	1.60	1.50	1.50
	δ_D	1.05	1.05	1.05	1.05
	δ_L	0.78	0.78	0.78	0.78
	V_D	0.10	0.10	0.10	0.10
	V_L	0.32	0.32	0.32	0.32
	V_S	0.10 – 0.25	0.10 – 0.25	0.10 – 0.25	0.10 – 0.25
	β^+	3.33 – 3.65	4.15 – 4.78	3.36 – 3.88	3.33 – 3.65

It is worth restating that, although there is little difference in the β^+ range (for $0.1 \leq L/D \leq 3$) using CSA S16:19 and the proposed method, as shown in both Tables 5 and 6, the proposed method: (i) has revised limits of validity; and (ii) accounts for the (sometimes significant) contribution of steel reinforcing bars to the resistance of concrete-filled HSS (see Section 4.1).

5. Recommendations

Based on the forgoing evaluation of design provisions for composite concrete-filled HSS according to CSA S16:19, AISC 360-16 and EN 1994-1-1:2004, the following changes are recommended for the next scheduled revision of CSA S16 (concerning materials, cross-section classification and concentrically loaded columns):

- Extension of Tables 1 and 2 to cover new width-(or diameter-)to-thickness ratios for concrete-filled RHS and CHS under axial compression or flexure. Proposed new limits are given in Section 3.1 of this paper.
- Revision of Clause 18.2.1.1 (Scope), such that Clause 18.2 covers:
 - concrete-filled HSS meeting the proposed new width-(or diameter-)to thickness limits (see Section 3.1); with
 - $20 \text{ MPa} \leq f'_c \leq 70 \text{ MPa}$ for normal-weight concrete, and $20 \text{ MPa} \leq f'_c \leq 40 \text{ MPa}$ for light-weight concrete.
- Revision of Clause 18.2.2 (Compressive resistance) to give C_r as Eq. (13), which “adds” the beneficial effect of longitudinal reinforcing bars to the existing CSA S16:19 C_{rc} equation based on plastic analysis.
- Addition of the term $E_s I_{sr}$ to the CSA S16 EI_e equation [see Table 4 and Design Examples 1(i) and 2(i)].

The following section provides two design examples which compare the results obtained by using the proposed methods for the design of concentrically loaded CHS and RHS columns to those obtained by using AISC 360-16.

Design Examples

Example 1

Determine the axial compressive resistance (C_{rc}) of an HSS 324×9.5 made to CSA G40.20/21 filled with normal-density concrete ($f'_c = 40$ MPa and $\rho_c = 2400$ kg/m³), which has 8×20M steel reinforcing bars located as shown in Fig. 15. The yield stress of the reinforcing bars is $F_{yr} = 400$ MPa and $KL = 4$ m.

Fig. 15. Concrete-filled CHS with steel reinforcing bars

Solution 1

Properties of steel section (CISC 2016):

$$A_s = 9410 \text{ mm}^2$$

$$I_s = 116 \times 10^6 \text{ mm}^4$$

$$E_s = 200,000 \text{ MPa}$$

$$F_y = 350 \text{ MPa}$$

Properties of reinforcing bars:

$$A_{sr} = 8 \times 300 = 2400 \text{ mm}^2 \quad \text{CAC (2014) Table 1.5}$$

$$I_{sr} \approx 2(300 \times 112^2) + 4(300 \times 80^2) = 15.2 \times 10^6 \text{ mm}^4$$

$$F_{yr} = 400 \text{ MPa}$$

Properties of concrete:

$$A_c = \frac{\pi}{4} (324 - 2(9.53))^2 - 2400 = 70.6 \times 10^3 \text{ mm}^2$$

$$I_c = \frac{\pi}{64} (324 - 2(9.53))^4 - 15.2 \times 10^6 = 409 \times 10^6 \text{ mm}^4$$

$$f'_c = 40 \text{ MPa}$$

(i) According to the proposed method

$$C_{rc} = \frac{(\tau \phi A_s F_y + \tau' \alpha_1 \phi_c A_c f'_c + \phi_r A_{sr} F_{yr})}{(1 + \lambda^{2n})^{1/n}} \quad \text{Eq. (13)}$$

where $\alpha_1 = 0.85 - 0.0015(40) = 0.79$. For CHS with $L/d_0 < 25$:

$$\tau = \frac{1}{\sqrt{1 + \rho + \rho^2}} \quad \text{Eq. (2)}$$

$$\tau' = 1 + \left(\frac{25 \rho^2 \tau}{d_0 / t} \right) \left(\frac{F_y}{\alpha_1 f'_c} \right) \quad \text{Eq. (3)}$$

where $\rho = 0.02(25 - L/d_0)$. In this example, $L/d_0 = 4000/324 = 12.4 < 25$; hence $\rho = 0.253$ and:

$$\begin{aligned} \tau &= \frac{1}{\sqrt{1 + 0.253 + 0.253^2}} \\ &= 0.871 \end{aligned}$$

and

$$\begin{aligned} \tau' &= 1 + \left(\frac{25(0.253)^2(0.871)}{(324/9.53)} \right) \left(\frac{350}{0.79(40)} \right) \\ &= 1.45 \end{aligned}$$

Calculating λ :

$$\lambda = \sqrt{\frac{C_p}{C_{ec}}} \quad \text{Eq. (4)}$$

where C_p is calculated by using Eq. (13) with $\phi = \phi_c = \phi_r = 1.0$ and $\lambda = 0$; i.e.:

$$C_p = \tau A_s F_y + \tau' \alpha_1 A_c f'_c + A_{sr} F_{yr}$$

$$= 0.871(9410)(350) + 1.45(0.79)(70.6 \times 10^3)(40) + 2400(400)$$

$$= 7060 \times 10^3 \text{ N}$$

From Table 4, with the addition of the implied “rebar term” ($E_s I_{sr}$):

$$EI_e = E_s I_s + E_s I_{sr} + \frac{0.60}{1 + C_{fs} / C_f} E_c I_c \quad (\text{Table 4})$$

The above equation for EI_e could, alternatively, be written with the “rebar term” as $(E_s - E_c)I_{sr}$ to account for the reduction of I_c due to the area of concrete displaced by the reinforcing bars.

Assuming $C_{fs}/C_f = 0.5$, and taking $E_c = (3300\sqrt{40} + 6900)(2400 / 2300)^{1.5} = 29,600 \text{ MPa}$ (CSA S16:19 Clause 3.1):

$$EI_e = 200,000(116 \times 10^6 + 15.2 \times 10^6) + \frac{0.60}{1.5}(29,600)(409 \times 10^6)$$

$$= 31,200 \times 10^9 \text{ Nmm}^2$$

Hence:

$$C_{ec} = \frac{\pi^2 (31,200 \times 10^9)}{4000^2} \quad \text{Eq. (5)}$$

$$= 19,200 \times 10^3 \text{ N}$$

and:

$$\lambda = \sqrt{\frac{7060 \times 10^3}{19,200 \times 10^3}}$$

$$= 0.606$$

Solving Eq. (13) with the above variable inputs gives $C_{rc} = 5050 \text{ kN}$.

(ii) According to AISC 360-16

From AISC 360-16 Clause I2.1b (and Table 4), $E_c = 0.043(2400)^{1.5} \sqrt{40} = 32,000 \text{ MPa}$. Calculating C_p :

$$C_p = A_s F_y + C_2 A_c f'_c + C_2 A_{sr} \frac{E_s}{E_c} f'_c$$

$$\begin{aligned}
 &= 9410(350) + 0.95(70,600)(40) + 0.95(2400)\frac{200,000}{32,000}(40) \\
 &= 6550 \times 10^3 \text{ N}
 \end{aligned}$$

From Table 4:

$$\begin{aligned}
 C_1 &= 0.45 + 3 \left(\frac{9410 + 2400}{9410 + 2400 + 70,600} \right) \leq 0.9 \\
 &= 0.88 \leq 0.9 \\
 &= 0.88
 \end{aligned}$$

Hence:

$$\begin{aligned}
 EI_e &= 200,000(116 \times 10^6 + 15.2 \times 10^6) + 0.88(32,000)(409 \times 10^6) \quad (\text{Table 4}) \\
 &= 37,900 \times 10^9 \text{ Nmm}^2
 \end{aligned}$$

Therefore:

$$\begin{aligned}
 C_{ec} &= \frac{\pi^2(37,900 \times 10^9)}{4000^2} \quad \text{Eq. (5)} \\
 &= 23,400 \times 10^3 \text{ N}
 \end{aligned}$$

$$\lambda = 0.529 \quad \text{Eq. (4)}$$

and

$$\lambda^2 = 0.280$$

For $\lambda^2 \leq 2.25$:

$$\begin{aligned}
 C_{rc} &= \phi_a \left(A_s F_y + C_2 A_c f'_c + C_2 A_{sr} \frac{E_s}{E_c} f'_c \right) \cdot (0.658^{\lambda^2}) \quad \text{Eq. (6)} \\
 &= 4370 \text{ kN}
 \end{aligned}$$

Example 2

Determine the axial compressive resistance of an HSS 254×254×9.5 made to CSA G40.20/21 filled with normal-density concrete ($f'_c = 60$ MPa and $\rho_c = 2400$ kg/m³), which has 8×20M steel reinforcing bars as shown in Fig. 16.

The yield stress of the reinforcing bars is $F_{yr} = 400$ MPa and $KL = 4$ m.

Fig. 16. Concrete-filled square RHS with steel reinforcing bars

Solution 2

Properties of steel section (CISC 2016):

$$A_s = 9090 \text{ mm}^2$$

$$I_s = 89.3 \times 10^6 \text{ mm}^4$$

$$E_s = 200,000 \text{ MPa}$$

$$F_y = 350 \text{ MPa}$$

Properties of reinforcing bars:

$$A_{sr} = 8 \times 300 = 2400 \text{ mm}^2 \quad \text{CAC (2014) Table 1.5}$$

$$I_{sr} \approx 6(300 \times 82^2) = 12.1 \times 10^6 \text{ mm}^4$$

$$F_{yr} = 400 \text{ MPa}$$

Properties of concrete:

$$\begin{aligned}
 A_c &= b_i h_i - (4 - \pi) r_i^2 - A_{sr} \\
 &= (254 - 2(9.53))^2 - (4 - \pi)(9.53)^2 - 2400 \text{ mm}^2 \\
 &= 52,700 \text{ mm}^2
 \end{aligned}$$

$$\begin{aligned}
 I_c &= \frac{b_i h_i^3}{12} + 4 \left[\left(\frac{1}{3} - \frac{\pi}{16} - \frac{1}{3(12 - 3\pi)} \right) r_i^4 + \left(1 - \frac{\pi}{4} \right) r_i^2 \left(\frac{h_i}{2} - \left(\frac{10 - 3\pi}{12 - 3\pi} \right) r_i \right)^2 \right] - I_{sr} \\
 &= 243 \times 10^6 \text{ mm}^4
 \end{aligned}$$

$$f'_c = 60 \text{ MPa}$$

(i) According to the proposed method

$$C_{rc} = \frac{(\tau \phi A_s F_y + \tau' \alpha_1 \phi_c A_c f'_c + \phi_r A_{sr} F_{yr})}{(1 + \lambda^{2n})^{1/n}} \quad \text{Eq. (13)}$$

where $\alpha_1 = 0.85 - 0.0015(60) = 0.760$. For RHS, $\tau = \tau' = 1.0$.

$$\begin{aligned}
 C_p &= \tau A_s F_y + \tau' \alpha_1 A_c f'_c + A_{sr} F_{yr} \\
 &= 1.0(9090)(350) + 1.0(0.76)(52,700)(60) + 2400(400) \\
 &= 6540 \times 10^3 \text{ N}
 \end{aligned}$$

Assuming $C_{fs}/C_f = 0.5$, and taking $E_c = (3300\sqrt{60} + 6900)(2400/2300)^{1.5} = 34,600 \text{ MPa}$ (CSA S16:19 Clause

3.1):

$$\begin{aligned}
 EI_e &= 200,000(89.3 \times 10^6) + 200,000(12.1 \times 10^6) + \frac{0.60}{1.5}(34,600)(243 \times 10^6) \\
 &= 23,600 \times 10^9 \text{ Nmm}^2
 \end{aligned}$$

Hence:

$$\begin{aligned}
 C_{ec} &= \frac{\pi^2 (23,600 \times 10^9)}{4000^2} \\
 &= 14,600 \times 10^3 \text{ N}
 \end{aligned} \quad \text{Eq. (5)}$$

and:

$$\lambda = 0.669 \quad \text{Eq. (4)}$$

Solving Eq. (13) with the above variable inputs gives $C_{rc} = 4660 \text{ kN}$.

(ii) According to AISC 360-16

From AISC 360-16 Clause I2.1b (and Table 4): $E_c = 0.043(2400)^{1.5} \sqrt{60} = 39,200 \text{ MPa}$.

$$\begin{aligned} C_p &= A_s F_y + C_2 A_c f'_c + C_2 A_{sr} \frac{E_s}{E_c} f'_c \\ &= 9090(350) + 0.85(52,700)(60) + 0.85(2400) \frac{200,000}{39,200}(60) \\ &= 6500 \times 10^3 \text{ N} \end{aligned}$$

and

$$\begin{aligned} C_1 &= 0.45 + 3 \left(\frac{9090 + 2400}{9090 + 2400 + 52,700} \right) \leq 0.9 \\ &= 0.99 \leq 0.9 \\ &= 0.9 \end{aligned}$$

Hence:

$$\begin{aligned} EI_e &= 200000(89.3 \times 10^6 + 12.1 \times 10^6) + 0.9(39,200)(243 \times 10^6) \quad (\text{Table 4}) \\ &= 28,900 \times 10^9 \text{ Nmm}^2 \end{aligned}$$

$$\begin{aligned} C_{ec} &= \frac{\pi^2 (28,900 \times 10^9)}{4000^2} \quad \text{Eq. (5)} \\ &= 17,800 \times 10^3 \text{ N} \end{aligned}$$

$$\lambda = 0.604 \quad \text{Eq. (4)}$$

and

$$\lambda^2 = 0.365$$

For $\lambda^2 \leq 2.25$:

$$\begin{aligned} C_{rc} &= \phi_a \left(A_s F_y + C_2 A_c f'_c + C_2 A_{sr} \frac{E_s}{E_c} f'_c \right) \cdot (0.658^{\lambda^2}) \quad \text{Eq. (6)} \\ &= 4180 \text{ kN} \end{aligned}$$

Acknowledgements

Financial support for this research was provided by the Natural Sciences and Engineering Research Council of Canada (NSERC).

Draft

References

- ACI. 2014. Building code requirements for structural concrete and commentary. ACI 318-14, American Concrete Institute, Farmington Hills, MI, USA.
- AISC. 2016. Specification for structural steel buildings. AISC 360-16, American Institute of Steel Construction, Chicago, IL, USA.
- ASCE. 2016. Minimum design loads and associated criteria for buildings and other structures. ASCE/SEI 7-16, American Society of Civil Engineers, Reston, VA, USA.
- ASTM. 2015. Standard specification for cold-formed welded carbon steel hollow structural sections (HSS). ASTM A1085/A1085M-15, American Society for Testing and Materials International, West Conshohocken, PA, USA.
- ASTM. 2021. Standard specification for cold-formed welded and seamless carbon steel structural tubing in rounds and shapes. ASTM A500/A500M-21, American Society for Testing and Materials International, West Conshohocken, PA, USA.
- Bartlett, F.M. 2007. Canadian Standards Association standard A23.3-04 resistance factor for concrete in compression. *Canadian Journal of Civil Engineering*, **34**: 1029-1037.
- Bergmann, R., Matsui, C., Meinsma, C. and Dutta, D. 1995. Design guide for concrete filled hollow section columns under static and seismic loading. CIDECT Design Guide No. 5, Comité International pour le Développement et l'Étude de la Construction Tubulaire and Verlag TÜV Rheinland, Köln, Germany.
- Bjorhovde, R. and Birkemoe, P.C. 1979. Limit state design of HSS columns. *Canadian Journal of Civil Engineering*, **6**: 276-291.
- Bradford, M.A., Wright, H.D. and Uy, B. 1998. Local buckling of the steel skin in lightweight composites induced by creep and shrinkage. *Advances in Structural Engineering*, **2**(1): 25-34.
- Bradford, M.A., Loh, H.Y. and Uy, B. 2002. Slenderness limits for filled circular steel tubes. *Journal of Constructional Steel Research*, **58**(2): 243-252.
- BSC. 1984. Construction with hollow steel sections, 1st ed. British Steel Corporation Tubes Division, Corby, UK.
- CAC. 2014. Concrete design handbook, 4th ed. Cement Association of Canada, Ottawa, Canada.

- CEN. 2004a. Design of composite steel and concrete structures – Part 1-1: General rules and rules for buildings. EN 1994-1-1, European Committee for Standardization, Brussels, Belgium.
- CEN. 2004b. Design of steel structures – Part 1-1: General rules and rules for buildings. EN 1993-1-1, European Committee for Standardization, Brussels, Belgium.
- CEN. 2004c. Design of concrete structures – Part 1-1: General rules and rules for buildings. EN 1992-1-1, European Committee for Standardization, Brussels, Belgium.
- CIDECT. 1979. Calcul des poteaux en profils creux remplis de béton. Fascicule 1: Méthode de calcul et technologie de mise en oeuvre; Fascicule 2: Abaques de calcul. CIDECT Monograph No. 5, Comité International pour le Développement et l'Étude de la Construction Tubulaire, Geneva, Switzerland.
- CISC. 2016. Handbook of steel construction, 11th ed. Canadian Institute of Steel Construction, Toronto, Canada.
- CSA. 1969. Steel structures for buildings. CSA S16-1969, Canadian Standards Association, Toronto, Canada.
- CSA. 2001. Limit states design of steel structures. CAN/CSA S16-01, Canadian Standards Association, Toronto, Canada.
- CSA. 2009. Design of steel structures. CAN/CSA S16-09, Canadian Standards Association, Toronto, Canada.
- CSA. 2011. Guidelines for the development of limit states design standards. CSA S408-11, Canadian Standards Association, Toronto, Canada.
- CSA. 2018. General requirements for rolled or welded structural quality steel / Structural quality steel. CSA G40.20-13/G40.21-13 (R2018), Canadian Standards Association, Toronto, Canada.
- CSA. 2019a. Design of steel structures. CSA S16:19, Canadian Standards Association, Toronto, Canada.
- CSA. 2019b. Design of concrete structures. CSA A23.3:19, Canadian Standards Association, Toronto, Canada.
- CSA. 2019c. Canadian highway bridge design code. CSA S6:19, Canadian Standards Association, Toronto, Canada.
- Dutta, D., Wardenier, J., Yeomans, N., Sakae, K., Bucak, Ö. and Packer, J.A. 1998. Design guide for fabrication, assembly and erection of hollow section structures. CIDECT Design Guide No. 7, Comité International pour le Développement et l'Étude de la Construction Tubulaire and TÜV-Verlag GmbH, Köln, Germany.

- Ellingwood, B., Galambos, T.V., MacGregor, J.G. and Cornell, C.A. 1980. Development of a probability based load criterion for American National Standard A58. Special Publication 577, National Bureau of Standards, Gaithersburg, MD, USA.
- Guiaux, P. and Janss, J. 1970. Comportement au flambement de colonnes constituées de tubes en acier remplis de béton. Centre de Recherches Scientifiques et Techniques de l'Industrie des Fabrications Métalliques, MT65, Brussels, Belgium.
- Kariyawasam, S.N., Rogowsky, D.M. and MacGregor, J.G. 1997. Resistance factors and companion-action load factors for reinforced concrete building design in Canada. *Canadian Journal of Civil Engineering*, **24**: 593-602.
- Kennedy, D.J.L. and Gad Aly, M. 1980. Limit states design of steel structures – performance factors. *Canadian Journal of Civil Engineering*, **7**: 45-77.
- Liu, J. 2016. Updates to expected yield stress and tensile strength ratios for determination of expected member capacity in the 2016 AISC Seismic Provisions. *Engineering Journal*, **2016**(4): 215-228.
- Lu, Y.Q. and Kennedy, D.J.L. 1994. The flexural behaviour of concrete-filled hollow structural sections. *Canadian Journal of Civil Engineering*, **21**: 111-130.
- MacPhedran, I. and Grondin, G.Y. 2011. A simple steel beam design curve. *Canadian Journal of Civil Engineering*, **38**: 141-153.
- NRC. 2015. National building code of Canada, 14th ed. NR20-2/2-2015E. National Research Council of Canada, Ottawa, Canada.
- Packer, J.A. and Henderson, J.E. 1997. Hollow structural section connections and trusses – A design guide, 2nd ed. Canadian Institute of Steel Construction, Toronto, Canada.
- Roik, K., Bergmann, R., Bode, H. and Wagenknecht, G. 1975. Tragfähigkeit von ausbetonierten hohlprofilstützen aus baustahl. TWM-Heft No. 75-4, Institut für Konstruktiven Ingenieurbau, Ruhr Universität Bochum, Bochum, Germany.
- Schmidt, B.J. and Bartlett, F.M. 2002a. Review of resistance factor for steel: resistance distributions and resistance factor calibration. *Canadian Journal of Civil Engineering*, **29**: 109-118.

- Schmidt, B.J. and Bartlett, F.M. 2002b. Review of resistance factor for steel: data collection. *Canadian Journal of Civil Engineering*, **29**: 98-108.
- Stelco. 1981. Hollow structural sections: Design manual for concrete-filled HSS columns. Stelco Inc., Toronto, Canada.
- Thai, S., Thai, H.T., Uy, B. and Ngo, T. 2019. Concrete-filled steel tubular columns: Test database, design, and calibration. *Journal of Constructional Steel Research* **157**: 161-181.
- Virdi, K.S. and Dowling, P.J. 1976. A unified design method for composite columns. International Association for Bridge and Structural Engineering (IABSE), Zürich, Switzerland, Mémoires Vol. 36-II, pp. 165-184.
- Wardenier, J., Packer, J.A., Zhao, X.L. and van der Vegte, G.J. 2010. Hollow sections in structural applications, 2nd ed. Comité International pour le Développement et l'Étude de la Construction Tubulaire, Geneva, Switzerland, ISBN 978-90-72830-86-9.
- Zhao, X.L., Packer, J.A., Wang, Y.C. and McCormick, J.P. 2019. Design guide for concrete-filled hollow section columns under static, impact, blast, seismic and fire loading. CIDECT Final Report 16H-4/19, Comité International pour le Développement et l'Étude de la Construction Tubulaire, Geneva, Switzerland.

List of Symbols

A	actual (experimental) column strength
A_c	cross-sectional area of concrete (after allowing for any space occupied by reinforcing bars)
A_g	gross cross-sectional area of composite section
A_s	cross-sectional area of steel section (HSS)
A_{sr}	cross-sectional area of reinforcing bars (total)
C_l	stiffness modification factor for concrete
C_2	ratio of average concrete stress to f'_c accounting for confinement effects in AISC 360-16
C_{ec}	Euler buckling strength of composite column
C_f	factored axial load
C_{fs}	factored sustained axial load
C_p	nominal compressive resistance calculated with $\lambda = 0$
C_r	compressive resistance of unfilled column
C_{rc}	compressive resistance of composite column
C_{nc}	nominal compressive strength of composite column
E_c	elastic modulus of concrete
E_s	elastic modulus of steel
EI_e	effective column stiffness
F_y	specified minimum yield strength of steel section (HSS)
F_{yr}	specified minimum yield strength of reinforcing bars
I_c	moment of inertia of concrete
I_s	moment of inertia of steel section (HSS)
I_{sr}	moment of inertia of reinforcing bars
K	effective length factor
L	length of member
V_D	coefficient of variation of δ_D
V_G	coefficient of variation of δ_G
V_L	coefficient of variation of δ_L
V_M	coefficient of variation of δ_M
V_P	coefficient of variation of δ_P
V_R	coefficient of variation of δ_R
V_S	coefficient of variation of the total load effect
b_{el}	clear width of stiffened compression element (assumed to be equal to $b_0 - 4t$ for all test specimens)
b_0	outside width of RHS
d_0	outside diameter of CHS

f'_c	specified cylinder compressive strength of concrete at 28 days
h_0	outside height of RHS
h_{el}	clear height of web between flanges (assumed to be equal to $h_0 - 4t$ for all test specimens)
n	column curve parameter in CSA S16:19; total number of reinforcing bars
r	radius of gyration
r_0	outside corner radius of RHS
t	design thickness of HSS member
t_0	nominal thickness of HSS member
Φ	stability parameter in EN 1994-1-1:2004
α	column curve parameter in EN 1994-1-1:2004
α_l	ratio of average concrete stress to f'_c in CSA S16:19
α_D	load factor for dead load
α_L	load factor for live load
β^+	safety/reliability index
δ_D	bias coefficient for dead load
δ_G	bias coefficient for the geometry
δ_L	bias coefficient for live load
δ_M	bias coefficient for the material
δ_P	bias coefficient for the professional factor
δ_R	bias coefficient for the resistance
λ	non-dimensional slenderness parameter
λ_p	limiting width-to-thickness ratio for compact/Class 2 elements
λ_r	limiting width-to-thickness ratio for noncompact/Class 3 elements
γ	partial safety factor for steel section
γ_r	partial safety factor for reinforcing bars
γ_c	partial safety factor for concrete
η_c	confinement factor for concrete in EN 1994-1-1:2004
η_s	confinement factor for steel in EN 1994-1-1:2004
ρ	slenderness factor in CSA S16:19
ρ_c	density of concrete (in kg/m ³)
τ	confinement reduction factor for steel in CSA S16:19
τ'	confinement enhancement factor for concrete in CSA S16:19
ϕ	resistance factor for steel in CSA S16:19
ϕ_a	resistance factor for compression in AISC 360-16
ϕ_c	resistance factor for concrete in CSA S16:19

ϕ_r	resistance factor for reinforcing bars in CSA S16:19
ϕ_{req}	overall resistance factor
φ	creep coefficient in EN 1994-1-1:2004
χ	stability factor in EN 1994-1-1:2004

Draft

Figure Legend(s)

Fig. 1. Toronto Pearson Airport, Terminal 1: (a) CHS columns filled with concrete; (b) concrete pumped from the bottom of the tubes through a spigot

Fig. 2. Concrete-filled HSS (without reinforcing bars): (a) nomenclature; (b) cross-sectional buckling modes

Fig. 3. Concrete-filled HSS (with reinforcing bars): (a) RHS; (b) CHS

Fig. 4. Comparison of stability factors in CSA S16:19, AISC 360-16 and EN 1994-1-1

Fig. 5. Axial compression resistance of unfilled and concrete-filled HSS with the same steel area

Fig. 6. Correlation of existing CSA S16:19 design method [Eq. (1)] with tests on concrete-filled RHS: (a) actual strength vs. nominal predicted strength; (b) actual strength vs. predicted design strength

Fig. 7. Correlation of AISC 360-16 design method [Eqs. (6) and (7)] with tests on concrete-filled RHS: (a) actual strength vs. nominal predicted strength; (b) actual strength vs. predicted design strength

Fig. 8. Correlation of EN 1994-1-1 design method [Eq. (8)] with tests on concrete-filled RHS: (a) actual strength vs. nominal predicted strength; (b) actual strength vs. predicted design strength

Fig. 9. Correlation of proposed design method for CSA S16 [Eq. (13)] with tests on concrete-filled RHS: (a) actual strength vs. nominal predicted strength; (b) actual strength vs. predicted design strength

Fig. 10. Correlation of existing CSA S16:19 design method [Eq. (1)] with tests on concrete-filled CHS: (a) actual strength vs. nominal predicted strength; (b) actual strength vs. predicted design strength

Fig. 11. Correlation of AISC 360-16 design method [Eqs. (6) and (7)] with tests on concrete-filled CHS: (a) actual strength vs. nominal predicted strength; (b) actual strength vs. predicted design strength

Fig. 12. Correlation of EN 1994-1-1 design method [Eq. (8)] with tests on concrete-filled CHS: (a) actual strength vs. nominal predicted strength; (b) actual strength vs. predicted design strength

Fig. 13. Correlation of proposed design method for CSA S16 [Eq. (13)] with tests on concrete-filled CHS: (a) actual strength vs. nominal predicted strength; (b) actual strength vs. predicted design strength

Fig. 14. Reliability index vs. L/D ratio for CSA S16:19 [Eq. (13)], AISC 360-16 [Eqs. (6) and (7)], and the proposed design method [Eq. (13)] for: (a) concrete-filled RHS; (b) concrete-filled CHS

Fig. 15. Concrete-filled CHS with steel reinforcing bars

Fig. 16. Concrete-filled square RHS with steel reinforcing bars