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Concentrically loaded slender high strength box columns

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ABSTRACT

This paper presents an experimental investigation of welded slender high strength composite box sections. The average yield strength of steel is 760 MPa and average compressive strength of infill concrete is 111 MPa. A total of 15 test specimens, having width-to-thickness ratios (b/t) ranged from 15 to 40, were tested to failure. The axial load-deformation behaviour and failure mode of the tested specimens were presented herein. In addition, calculated ultimate axial strengths using the current design provisions are compared with the experimental ultimate axial strengths to verify the adequacy of the current design provisions concerning very high strength composite box columns.

1. INTRODUCTION

Slender composite columns have been used in the past to support roofs of oil storage tanks, decks of motorways and railways and floors of high-rise buildings (Rangan and Joyce 1992). The advent of high strength materials, such as high strength steel (HSS) and high strength concrete (HSC), provided a significant demand in constructions due to its achievement in further enhancement of the axial load capability of such tubular columns. Recently, sufficient amount of research has been undertaken on short high strength box sections by using HSS and HSC (Thai et al 2015, Khan et al 2013 and Uy 2001).

However, a limited amount of research has been undertaken by experiments on very high strength slender composite columns. Kilpatrick et al. (1999) tested 41 circular composite columns under axial loading incorporating 400 MPa HSS sections filled with 96 MPa HSC. Mursi and Uy (2004) conducted tests on axially loaded slender columns

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made of 690 MPa HSS box section filled with normal strength concrete.

The current design standards restrict the use of very high strength materials to design composite columns. For instance, Part 6 of the Australian Bridge design - steel and composite construction (AS5100.6), Eurocode 4, American Institute of Steel Construction (AISC/LRFD) and Canadian Standard Association (CSA S16-09) restrict steel sections to 350, 460, 525 and 550 MPa steel respectively, and also restrict the infill concrete to 65, 60, 70 and 80 MPa concrete respectively. Due to this current restrictions as being highlighted, this paper aims to investigate the design provisions by utilising test specimens from the experimental programme on composite box sections fabricated from nominal 690 MPa HSS and 100 MPa HSC.

2. EXPERIMETAL PROGRAMME

Test specimens described herein are part of an experimental programme on slender high strength columns. The experimental programme comprised a total of 65 hollow and composite square box sections having width to thickness ratios (b/t) of 15 to 40. The programme also comprises three identical tests for each composite test specimen. The box columns were designed as slender in order to achieve member capacities or failure modes by flexural buckling. However, only 15 composite sections (CB) are reported herein, as listed in Table 1.

Table 1 Ultimate load, measured dimensions (see Fig.1) and properties of infill concrete of 15 composite test specimens.

Specimens	b mm	t mm	b/t	L mm	f_c MPa	f_{ck} MPa	N_{exp} kN
CB15-SL1(A)	73.93	4.89	15	1060	111	100	1732
CB20-SL1(A)	98.87	4.93	20	1060	111	100	2087
CB25-SL1(A)	124.12	4.93	25	1060	111	100	3748
CB30-SL1(A)	149.77	4.93	30	1060	111	100	5164
CB40-SL1(A)	198.92	4.91	40	1060	111	100	7478
CB15-SL2(A)	74.36	4.90	15	2060	111	100	583
CB20-SL2(A)	99.95	4.90	20	2060	111	100	1377
CB25-SL2(A)	124.60	4.93	25	2060	111	100	2823
CB30-SL2(A)	149.72	4.94	30	2060	111	100	3938
CB40-SL2(A)	199.62	4.95	40	2060	111	100	6141
CB15-SL3(A)	74.41	4.95	15	3060	111	100	469
CB20-SL3(A)	98.99	4.95	20	3060	111	100	810
CB25-SL3(A)	124.41	4.92	25	3060	111	100	1535
CB30-SL3(A)	149.33	4.93	30	3060	111	100	4142
CB40-SL3(A)	199.84	4.93	40	3060	111	100	5002

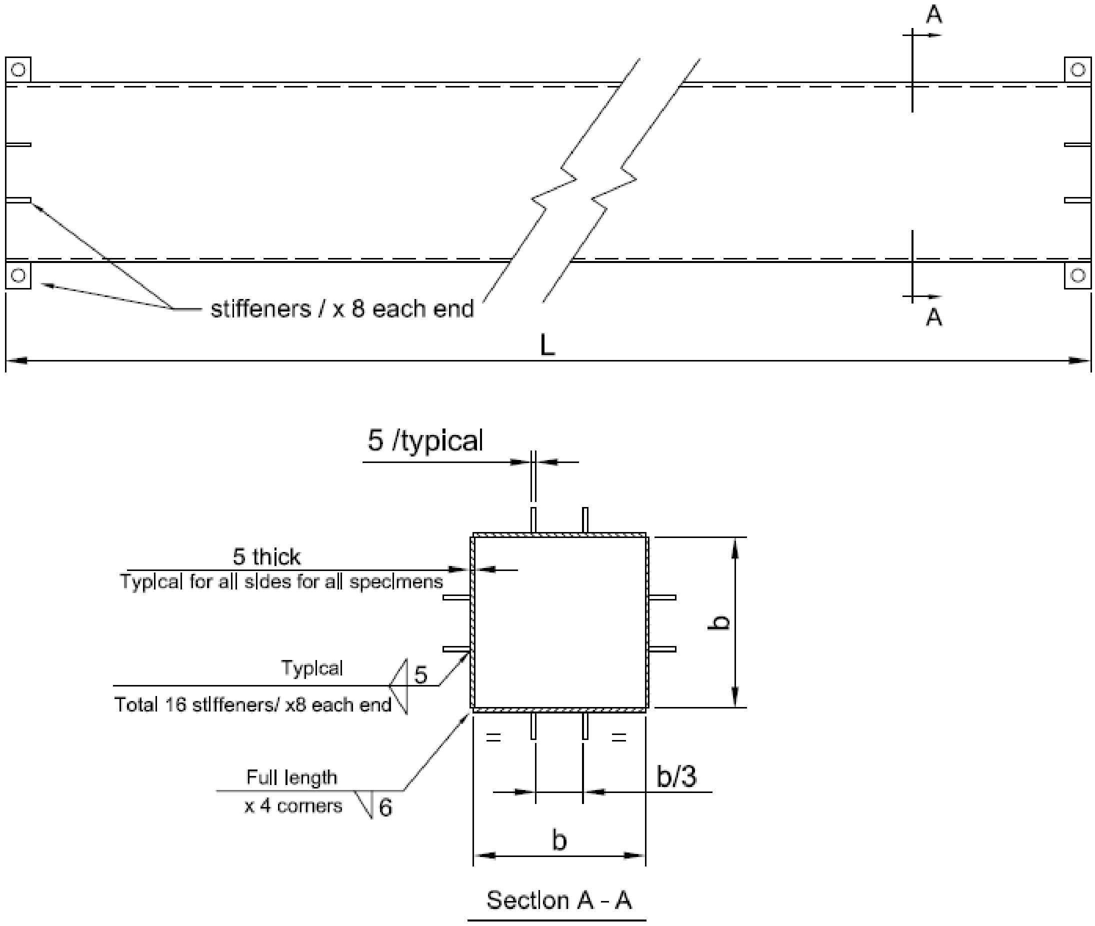


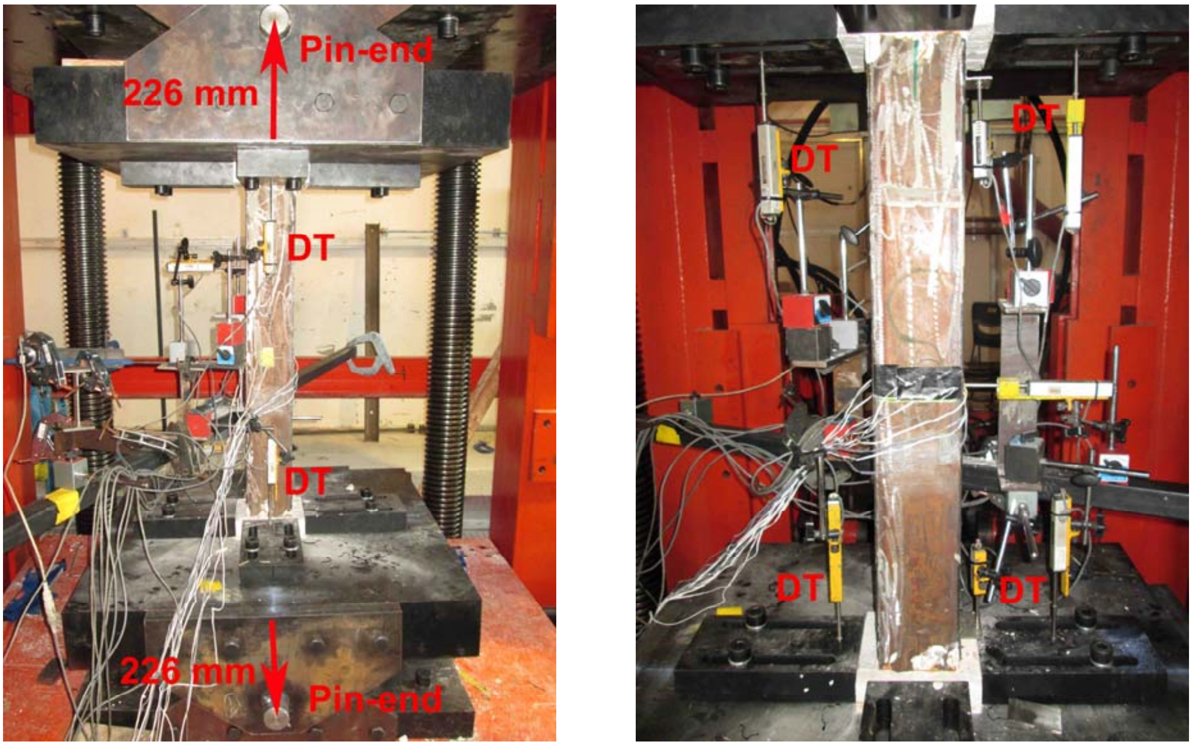
Fig. 1 Dimensions of the test specimens.

2.1 Physical properties and fabrication

Box sections were fabricated from 5 mm thick plates of BISALLOY 80 by using single pass welds. Stiffeners or lugs were fabricated additionally to each end of columns to avoid localised bearing failures at the ends, as shown in Fig 1. The standard tensile coupon tests provided an average yield stress (f_y) of 762 MPa. The infilled concrete of composite columns consisted of commercially available 100 MPa HSC. The standard cylinder tests provided an average compressive strength (f_c) of 111 MPa and characteristic compressive strength (f_{ck}) of 100 MPa, as listed in Table 1.

2.2 Test Procedures

Test specimens were concentrically loaded by using the 10,000 kN capacity testing machine as shown in Fig 2. To achieve flexural bending, both ends of the test specimens were designed as pin ends as shown in Fig 2a. Displacement transducers were installed at the top and bottom of test specimens and end plates to record the axial shortening of columns. In addition, longitudinal and bi-directional strain gauges were also installed in the middle section of all sides of the test specimens to monitor strain of steel sections.



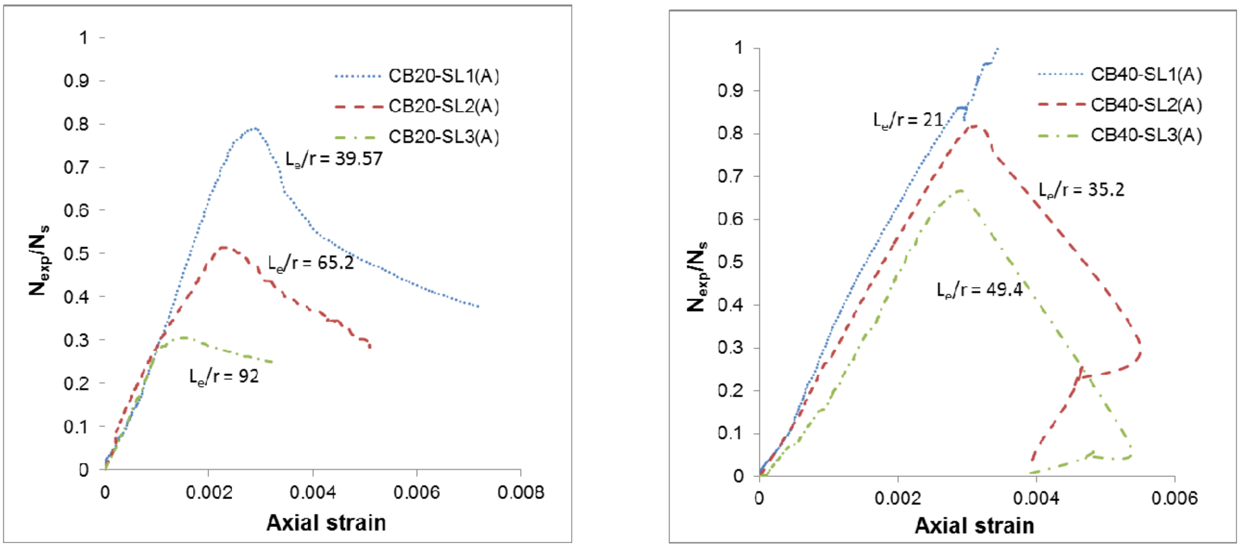
(a) (b)

Fig. 2 Test procedures and locations of displacement transducers (DTs): (a) Front view of the test setup and (b) Side view of the test setup.

3. TEST RESULTS AND OBSERVATIONS

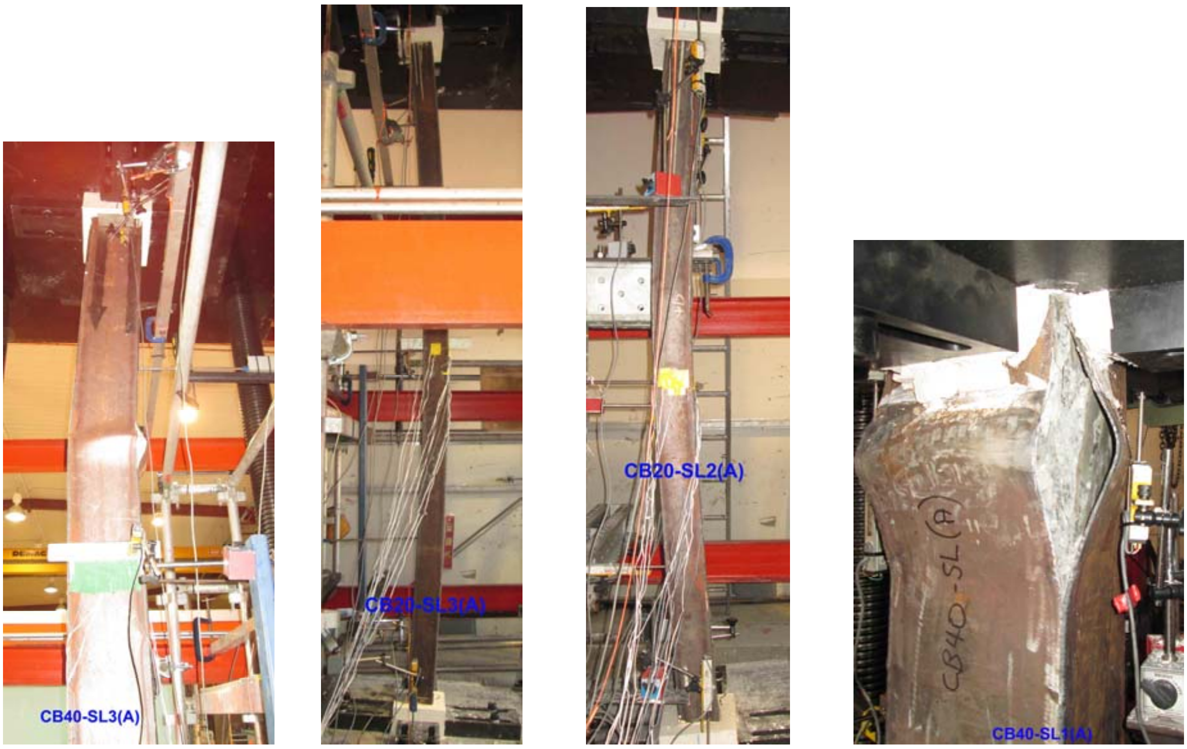
The ultimate test strengths (N_{exp}) and measured dimensions of composite test specimens are listed in Table 1. Fig. 3 demonstrates examples of axial deformation behaviour of test specimens, which have b/t values of 20 and 40 with three different geometrical slenderness ratios (L_e/r). L_e is the effective length of the test specimens from the centre of pin-end connections as shown in Fig. 2a, and r is the radius of gyration of the composite section. The axial shortening of the test specimens suggests that as the slenderness of the columns increases, the ultimate strength of the columns decreases as shown in Fig. 3, where N_s is the section capacity calculated from Eq.(1).

Fig. 4a to 4c demonstrate flexural buckling of the test specimens having L_e/r values of 49.4, 65.2 and 92, when subjected to concentric loading to failure. This flexural buckling behaviour applies to other slender test specimens having L_e/r values greater than 40 from the experimental programme.



(a) (b)

Fig. 3 Axial shortening of test specimens: (a) Test specimens having a b/t value of 20 and (b) Test specimens having a b/t value of 40.



(a) (b) (c) (d)

Fig. 4 Failure modes of composite test specimens after the peak loads: (a) CB40-SL1 (A), (b) CB20-SL3 (A), (c) CB20-SL2 (A) and (d) CB40-SL1 (A).

The test specimen, CB40-SL1 (A) having a lower L_e/r value of 21, exhibited local buckling failure as shown in Fig. 4d. Hence, the specimen, CB40-SL1(A), achieved the ultimate section capacity, i.e. $N_{exp}/N_s > 1$ as shown in Fig. 3b.

4. COMPARISION OF TEST STRENGTHS AND DESIGN STRENGTHS

The main focus of this paper is to investigate whether the current design procedures can conservatively predict very high strength slender box composite sections. Herein, four design provisions, AS5100.6, Eurocode 4, AISC/LRFD and CSA S16-09, are used to investigate the ultimate strength of the test specimens. These specifications are also briefly summarised below. Furthermore, both nominal and actual materials strength are used in the comparison between test strengths and design strengths. The physical properties of steel and concrete are described in section 2.1 and listed in Table 1.

4.1 Comparison of test strengths *and AS5100.6*

In accordance with AS5100.6, Eurocode 4, AISC/LRFD and CSA S16-09, the ultimate section capacity (N_s) of a composite section can be calculated by applying Eq. (1).

$$N_s = \varphi_s A_s f_y + \varphi_c A_c f_c \tag{1}$$

where A_s is the cross-sectional area of steel, A_c the cross-sectional area of infill-concrete, f_y the yield stress of steel and f_c the compressive strength of infill concrete. The value of φ_c and φ_s is 1.

To take into account the effect of flexural buckling in accordance with AS5100.6, the ultimate member capacity (N_c) of a composite member is calculated by multiplying the slenderness or flexural reduction factor (α_c) to Eq. (1), i.e. $N_c = N_s\alpha_c$. Eq. (2) to (6) show the procedures or steps to calculate α_c.

$$\alpha_c = \xi\left[1-\sqrt{1-\left(\frac{90}{\xi\lambda}\right)^2}\right] \tag{2}$$

$$\xi = \frac{\left(\frac{\lambda}{90}\right)^2 + 1 + \eta}{2\left(\frac{\lambda}{90}\right)^2} \tag{3}$$

$$\lambda = \lambda_\eta + \alpha_a\alpha_b \tag{4}$$

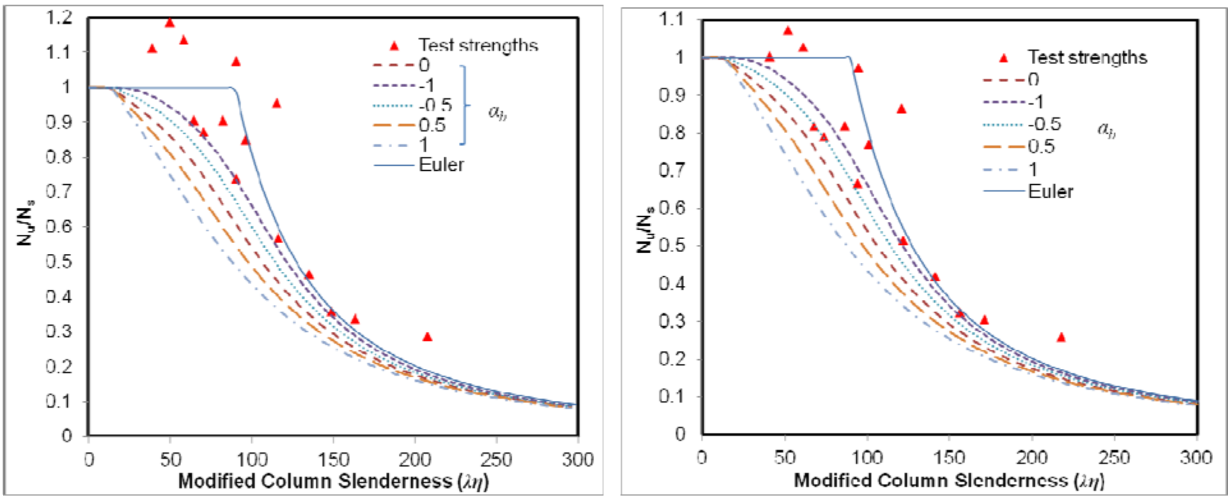
$$\lambda_\eta = 90\lambda_r \tag{5}$$

$$\alpha_a = \frac{2100(\lambda_\eta - 13.5)}{\lambda_\eta^{\ 2} - 15.3\lambda_\eta + 2050} \tag{6}$$

$$\lambda_r = \sqrt{\frac{N_s}{N_{cr}}} \tag{7}$$

where α_b = -0.5, 1, 0, 0.5,1 and N_{cr} = elastic buckling load (Composite).

The α_b value determines the shape of column buckling curves, and accounts for induced residual stresses and various types of sections. AS5100.6 recommends α_b value of 0 in the case of welded box sections.



(a) Nominal material properites (b) Actual material properties

Fig. 5 Comparison of column curves of AS5100.6 and non dimensional test strengths.

Non-dimensional test strengths, N_{exp}/N_s, were compared against five bucking curves of AS5100.6 in Fig 5. The buckling curves comprise nominal and actual material strengths of the test specimens, as shown in Fig. 5a and 5b respectively. It should be Noted that N_u/N_s denotes ratio of ultimate load to section capacity.

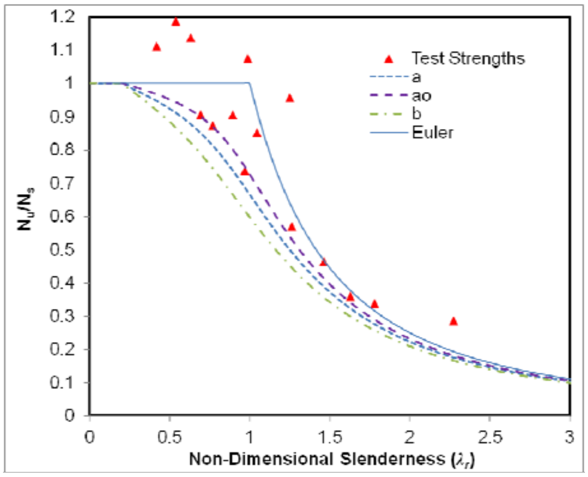
4.2 Comparison of Test Strengths and Eurocode 4

The N_s in accordance with Eurocode 4 can be calculated from Eq. (1). To calculate member capacity due to flexural buckling, the slenderness or flexural reduction factor (χ), similar to α_c, is calculated from Eq. (8)

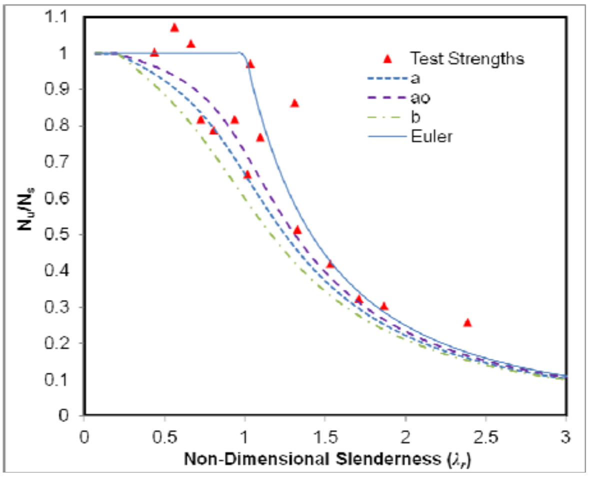
$$\chi = \frac{1}{\phi + \sqrt{\phi^2 + \lambda_r^{\ 2}}} \tag{8}$$

where $\phi = 0.5\left[1+\alpha(\lambda_r - 0.2)+\lambda_r^2\right]$, and α is the imperfection factor that defines the buckling curves a_0, a, b, c and d.

The column buckling curves are based on section 6.3.1.2 of EN1993-1-1. Eurocode 4 recommends α values of 0.21 (curve a) and 0.34 (curve b) for rectangular composite sections.



(a) Nominal material properites



(b) Actual material properties

Fig. 6 Comparison of column curves of Eurocode 4 and non-dimensional test strengths

Non-dimensional test strengths, N_{exp}/N_s, are compared against three bucking curves of Eurocode 4 using nominal and actual material properties, as shown in Fig. 6.

4.3 Comparison of Test Strengths against AISC/LRFD and CSA S16-09.

To calculate section capacity (N_s) in accordance with AISC/LRFD and CSA S16-09, Eq. (1) can be applied as mentioned earlier. However, the φ_c value of Eq. (1) is 0.85 and $0.85\text{-}0.0015f_c \geq 0.67$ for AISC/LRFD and CSA S16-09, respectively.

When $\dfrac{N_s}{N_{cr}} \leq 2.25$

$$N_c = N_s\left[0.658^{N_s/N_{cr}}\right] \tag{9}$$

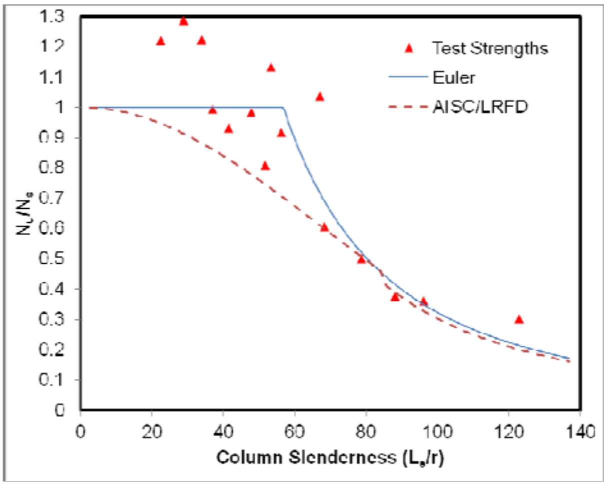
When $\dfrac{N_s}{N_{cr}} > 2.25$

$$N_c = N_{cr}0.877 \tag{10}$$

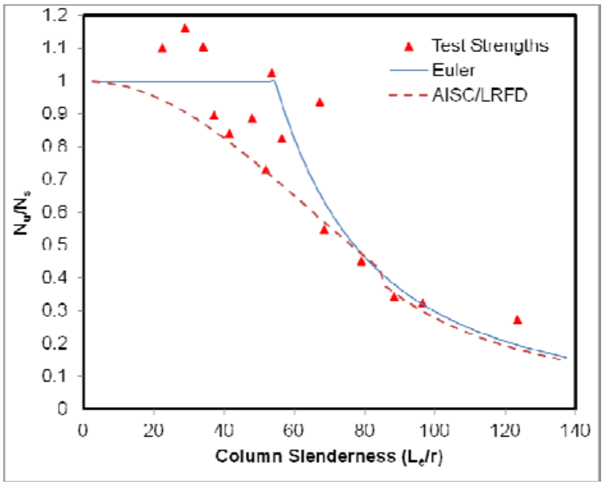
$$N_c = N_s\left(1+\lambda_r^{2n}\right)^{\frac{-1}{n}} \tag{11}$$

where the value of n is 1.8.

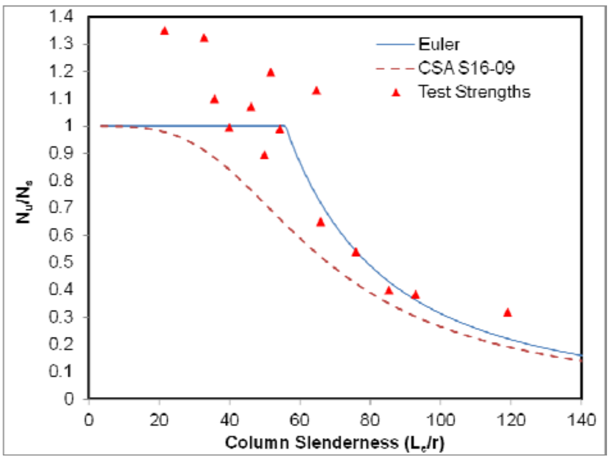
The design provisions, AISC/LRFD and CSA S16-09, can generate one buckling strength curve to calculate the member capacity (N_c) of composite columns. Eq. (9) and (10) calculate member capacity in accordance with AISC/LRFD, and Eq. (11) calculates member capacity in accordance with CSA S16-09.



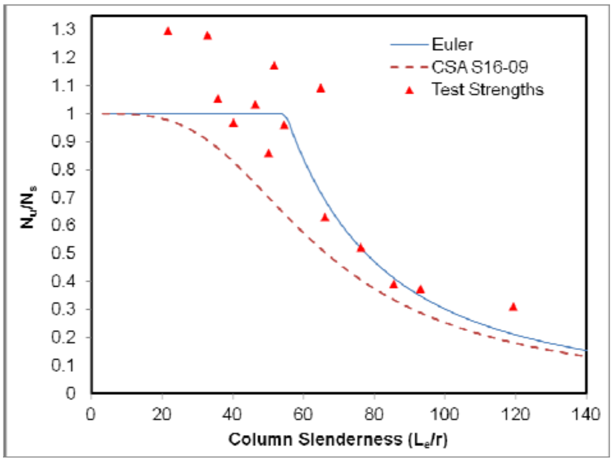
(a). Nominal material properites (AISC)



(b). Actual material properties(AISC)



(c). Nominal material properites (CSA)



(d). Actual material properties(CSA)

Fig. 7 Comparison between design strengths and non dimensional test strengths.

Since there is no equation for the buckling curve of AISC/LRFD and CSA S16-09, the buckling curves were generated by using normalised member capacities (N_c) of a section from Eq. (9) to Eq. (11), as shown in Fig. 7. The non-dimensional test strengths, N_{exp}/N_s, are compared against the generated bucking curves of AISC/LRFD (as shown in Fig. 7a and 7b) and CSA S16-09 (as shown in Fig. 7c to 7d).

5. DISCUSSION

When test strengths are compared with the buckling curves of AS5100.6 pertaining to nominal and actual material strengths as shown in Fig. 5, the design provision predicts ultimate strength conservatively. All ultimate test strengths are higher than buckling curve of $\alpha_b = 0$, which is the designated buckling curve for welded box sections, as shown in Fig. 5. As can be seen in Fig. 5a and 5b, a less conservative buckling curve ($\alpha_b = -0.5$) can predict ultimate test strengths conservatively in the case of both nominal and actual material strengths of the test specimens.

As shown in Fig. 6a, the buckling curves (a and b) recommended by Eurocode 4 have predicted test strengths conservatively, when nominal material strengths are considered in the calculation. As can be seen in Fig. 6a, a less conservative buckling curve pertaining to nominal strength materials, a_0, can also predict test strengths safely. However, only the buckling curve b seems to predict ultimate strengths conservatively, when actual material strengths are considered as shown in Fig. 6b.

Considering the AISC/LRFD provision, ultimate design strengths calculated by using actual material properties exhibit slightly less conservative results, as can be seen in Fig. 7b between L_e/r values of 70 and 88. Nonetheless, as shown in Fig. 7a and 7b), overall strength predictions by AISC/ LRFD are conservative in the case of both nominal and actual material properties.

As has been the case with the other above design specifications, CSA S16-09 predicts the ultimate test strengths conservatively, as shown in Fig. 7c and 7d. As can also be seen in Fig. 7, the gap between the Euler buckling curve and the generated CSA S16-09 buckling curve is larger and more conservative, when compared with generated AISC/LRFD buckling curve. As a result, CSA S16-09 can predict ultimate strength conservatively than AISC/LRDF.

6. CONCLUSIONS

Limited test results presented herein have been compared with four design provisions (AS5100.6, Eurocode 4, AISC/LRFD and CSA S16-09) for very high strength composite columns fabricated from 762 MPa HSS and 111 MPa HSC. The comparison between ultimate test strengths and design strengths has revealed conservative predictions from the design provisions. Hence, AS5100.6, Eurocode 4, AISC/LRFD and CSA S16-09 can be used to design such high strength slender composite members under compression loads.

7. REFERENCES

American Institute of Steel Construction (AISC). (2010).*Specification for Structural Steel Buildings*. ANSI/AISC360-10, Chicago (IL, USA).

AS5100.6-2004, *Bridge design, Part 6: Steel and Composite Construction*, Standards Australia, Australia.

AS4100-1998*, Steel Structure*, Standards Australia, Australia.

Canadian Standards Association (CSA). (2009). *Design of Steel Structures*. S16-09, Mississauga (Ontario, Canada).

Eurocode 3, pr EN 1993-1-1:2004. *Design of Steel structures. Part1-1: General structural rules*. European committee for standardisation, Brussels

Eurocode 4, I.S EN 1994-1-1:2005. *Design of Composite Steel and Concrete Structures. Part1.1: General Rules and Rules for Buildings*. Dublin, Ireland.

Kilpatrick, A.E., and Rangan, B.V. (1999), "Tests on high-strength concrete-filled steel tubular columns", *ACI Struct. J.*, **96**(2), 268–275.

Khan, M., Uy, B., Tao, Z., and Mashiri, F. R. (2013). "Concentrically loaded short high strength composite columns" *Advancements and Achievements in Structural Steel: Proceedings of the 10th Pacific Structural Steel Conference (PSSC 2013), 8-11 October 2013, Singapore.*

Mursi, M., and Uy, B. (2004), "Strength of slender concrete filled high strength steel box columns", *J. Constr. Steel Res.*, **60**(12), 1825-1848.

Rangan, B.V., and Joyce, M (1992), "Strength of Eccentrically Loaded Slender Steel Tubular Columns Filled With High-Strength", *ACI Struct. J.*, **89**(6), 676-681.

Thai, H.T., Uy, B., and Khan, M (2015), "A modified stress-strain model accounting for the local buckling of thin-walled stub columns under axial compression", *J. Constr. Steel Res.,* **111**, 57-69.

Uy, B. (2001), "Strength of short concrete filled high strength steel box columns", *J. Constr. Steel Res.,* **57**, 113-134.