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# Statistical calibration of safety factors for flexural stiffness of composite columns

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**Abstract.** Composite column design is strongly influenced by the computation of the critical buckling load, which is very sensitive to the effective flexural stiffness (EI) of the column. Because of this, the behaviour of a composite column under lateral loading and its response to deflection is largely determined by the EI of the member. Thus, prediction models used for composite member design should accurately mirror this behaviour. However, EI varies due to several design parameters, and the implementation of high-strength materials, which are not considered by the current composite design codes of practice. The reliability of the design methods from six codes of practice (i.e., AS 5100, AS/NZS 2327, Eurocode 4, AISC 2010, ACI 318, and AIJ) for composite columns is studied in this paper. Also, the reliability of these codes of practice against a serviceability limit state criterion are estimated based on the combined use of the test-based statistical procedure proposed by Johnson and Huang (1997) and Monte Carlo simulations. The composite columns database includes 100 tests of circular concrete-filled tubes, rectangular concrete-filled tubes, and concrete-encased steel composite columns. A summary of the reliability analysis procedure and the evaluated reliability indices are provided. The reasons for the reliability analysis results are discussed to provide useful insight and supporting information for a possible revision of available codes of practice.

**Keywords:** composite columns; flexural stiffness; reliability analysis

## 1. Introduction

In recent decades, the implementation of composite-column members has become increasingly popular in the construction sector. Composite construction combines the advantages of both structural steel and concrete, namely in structure strength, the speed of construction, and economy. Both concrete-filled steel tube columns (CFSTCs) and concrete-encased steel columns (CESCs)

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show increased strength and ductility performance compared to conventional reinforced concrete columns (Gho and Liu 2004, Han *et al*. 2005).

The use of CFSTCs permits rapid on-site construction, with the steel tube serving as formwork and reinforcement to the structure. The concrete may then be placed into the steel tube later, increasing the member's stiffness, and the load bearing capacity of the column. CFSTCs are generally circular or rectangular in shape with the steel placed in the most effective position, increasing the deformation capacity of the column. Added benefits of CFSTCs include increased fire resistance compared to non-composite columns, and high durability. CESCs consist of a steel member encased by concrete. These members offer the rigidity and strength of reinforced concrete members with a rapid construction rate, with steel members reportedly being erected approximately 10 stories in advance of concrete placement (Ricles and Paboojian 1994). The concrete used for encasing the steel increases the columns strength and serves as fire protection to the steel section inside.

Practical applications for composite members can be found in low- and high-rise structures, piers, and deep foundations (Aslani *et al*. 2015a, b). However, despite the advantages of composite construction, there is relatively little research reported on the flexural behaviour of composite columns (Chitawadagi and Narasimhan 2009), and design codes do not consider all parameters involved in effective flexural stiffness (*EI*) calculation. (Ellobody and Young 2010, Tikka and Mirza 2006a, b). International codes of practice for *EI* prediction generally differ in their respective capacity reduction factors. Previous investigations have been conducted in order to determine appropriate reduction factors, as well as determine the influence of a range of parameters on *EI* prediction. Han (2003) presented a comparison of *EI* equations from 4 international codes of practice to the flexural behaviour of 16 composite members. It was found that the codes of practice were generally un-conservative when predicting *EI*.

This paper aims to estimate the reliability of the AS/NZS 2327 (2015 Draft) prediction model for *EI* against a deflection-based serviceability limit state, using a statistical method based on the combined use of the test-based statistical procedure proposed by Johnson and Huang (1994) and Monte Carlo simulations. The reliability analysis is conducted on an experimental database of 100 composite column specimens tested under flexure, composed from available literature. The capacity reduction factors for six international codes of practice will be assessed against the experimental results, to determine which codes are accurate for composite members.

## 2. Past research

Previous past research is categorized into three categories: (a) Square and circular concrete-filled tubes; (b) Fully- and partially-encased composite columns; and (c) Reliability analyses of column design codes.

### *2.1 Square and circular concrete-filled tubes*

Past research has shown varying correlation between theoretical and code predicted values for the *EI* of composite members, with many authors proposing improved *EI* models. Prion and Boehme (1994) carried out an experimental investigation into the behaviour of thin-walled steel tubes with high-strength concrete infill. Results of 26 tests on specimens were reported. Significant slippage occurred between the steel and concrete, however this did not seem to lower the moment capacity of the specimens; an observation mirrored by Han (2003), Chitawadagi and

Narasimhan (2009), Wheeler and Bridge (2006).

Elchalakani *et al*. (2001) presented an experimental investigation into the flexural behaviour of circular concrete-filled steel tubular columns (CCFSTCs) subject to pure bending. The tests on compact specimens showed a similar behaviour between the concrete-filled tubes and the unfilled specimens until a certain point (also noted in Wheeler and Bridge 2006). It was found that the ultimate moment capacity of cold-formed circular sections by Eurocode 4 (1994) was in good agreement with experimental results.

Varma *et al*. (2002) investigated the experimental flexural force-deformation behaviour of high-strength rectangular concrete-filled steel tubular columns (RCFSTCs). The parameters in the study included width-to-thickness ratio, the yield stress of the steel tube, and axial load level. A comparison of experimental results with current design codes showed that ACI (1999) provisions were more conservative than AIJ (1987) provisions, except at low levels of axial load. The AISC (1999) provisions significantly underestimated the moment capacity of the RCFSTC specimens, as they did not appropriately account for the contribution of the concrete infill. The AIJ (1987) predicted moment capacities were reasonably accurate but tend to be un-conservative. Han (2003) performed a series of flexural tests on RCFSTCs with a range of variables. A total of 16 RCFSTC specimens, 1100 mm in length were tested in the experiment. The experimental stiffness values were compared to international *EI* codes: AIJ (1997), BS5400 (1979), Eurocode 4 (1994), and AISC (1999). The AIJ (1997) method proved the best predictor of the codes. The BS5400 (1979), Eurocode 4 (1994), and AISC (1999) methods were deemed un-conservative in predicting effective flexural stiffness values.

Wheeler and Bridge (2006) undertook an extensive research program on the flexural behaviour of CCFSTC, with particular emphasis on thin-walled steel tubes. A series of flexural tests were carried out on full-scale tube specimens. The results allowed the influence of the concrete infill on the flexural stiffness and strength of the tubes to be established. It was concluded that for CCFSTCs under typical service loads, the stiffness of the cross-section is close to the theoretical stiffness of the bare steel tube, with little or no contribution to the stiffness coming from the concrete. However, the size effect for small tubes would have a bearing on the flexural stiffness of the member. The test demonstrated that the flexural strength and ductility of a circular hollow section is increased when filled with concrete.

Han *et al*. (2005) presented a further study on the flexural behaviour of concrete filled steel tubes. A further 36 composite beam specimens were tested. The specimens were tested under two types of lateral loading. The one point method induced a bending moment from a single point load on the mid-span of the section. The two-point method used four points of applied loading. Experimental stiffness values were compared with predicted values from AIJ (1997), BS5400 (1979), Eurocode 4 (1994), and AISC (1999). A model was proposed, shown in Eq. (1). For beams with circular cross-sections, both BS5400 (1979) and AISC (1999) were deemed non-conservative. For beams with square and rectangular cross-sections, BS5400 (1979), Eurocode 4 (1994), AISC (1999) and the proposed model all predict an initial flexural stiffness 10-15% higher than experimental results. The AIJ (1997) was the best predictor with a mean value of 0.918, and a coefficient of variation (COV) of 0.142.

$$K_i = 0.2 M_u / \phi_e \tag{1}$$

where $K_i$ is the initial section flexural stiffness, $M_u$ is the moment capacity of the composite member, and $\phi_e$ is the curvature of the member.

### 2.2 Fully- and partially-encased composite columns

Mirza and Lacroix (2004) provided a comprehensive comparison of 150 physical tests of rectangular CESC columns from available published literature to ACI 318-02 (2002) and Eurocode 4 (1994) procedures. The comparative study provided a critical review of the reliability of the computational methods examined. The report compared tested columns strengths against strengths computed from AISC (1999), ACI 318-02 (2002) and Eurocode 4 (1994) methods. It was found that Eurocode 4 (1994) predicted column strength most accurately with an average strength ratio of 1.04 and a coefficient of variation of 0.15.

Tikka and Mirza (2006a, b) simulated approximately 12,000 tests on fully encased CESC columns to determine the influence of a full range of variables on the short-term *EI*. The study examined the existing *EI* model used by the ACI 318-02 (2002), and proposed a new expression that considered prominent parameters in composite column behaviour shown in Eq. (2). The proposed design equation illustrates the importance of slenderness, and eccentricity upon composite column behaviour. These variables were not considered in the ACI 318-02 (2002) design equations and, accordingly, resulted in a much higher variability for *EI* calculations. Tikka and Mirza (2006a, b) included a graphical design aid to ensure efficient computations.

$$EI = \left[ 0.47 - 3.5\frac{e}{h}\left( \frac{1}{1+9.5\dfrac{e}{h}} \right) + 0.003\frac{l}{h} \right] E_c \left( I_g - I_{ss} \right) + 0.8 E_s \left( I_{ss} + I_{rs} \right) \tag{2}$$

where $e$ is the end eccentricity of the member, $h$ is the overall thickness of cross section perpendicular to the axis of bending, $l$ is the unsupported height of the member (column), $E_c$ is the modulus of elasticity of concrete, $E_s$ is the modulus of elasticity of steel, $I_g$ is moment of inertia of gross concrete section, $I_{rs}$ is moment of inertia of longitudinal reinforcing steel bars, and $I_{ss}$ is moment of inertia of structural steel section taken about centroidal axis of composite cross section.

Elghazouli and Treadway (2008) presented an account of experimental testing of partially encased composite concrete/steel beam-columns under combined bending and axial loading. Three tests were carried out under pure bending conditions. Characteristic wide flexural cracks were observed on the specimens, extending through most of the member depth. The estimated experimental values of stiffness were found to be largely in agreement with the assumption based on 50% of the concrete contribution in both major and minor-axis tests.

Tokgoz and Dundar (2008) reported on experimental investigations on the behaviour of concrete-encased composite columns subject to short-term axial load and bi-axial bending. Six square, and four L-shaped cross sections were constructed and tested to examine load-deflection behaviour. Comparative results showed a good agreement between theoretical and experimental results. A flexural rigidity parameter taken from previous studies was employed, which played a significant role on the computation of slender composite columns. A comparison study was made on Virdi and Dowling (1973) who performed a similar experimental investigation. A good agreement between the two studies test results was accomplished. An iterative theoretical method including slenderness effects was suggested to determine the complete load-deflection behaviour of composite columns.

### 2.3 Reliability analyses of column design codes

The procedure developed by Johnson and Huang (1994) is a statistical determination for partial safety factors required for composite beams in limit state design, which is applicable to any structural member composed of more than one material. The procedure can be used when the resistance and the load models are separately considered in the target limit state function. Kang *et al*. (2015) provided a calibration based on the statistical method proposed by Johnson and Huang (1997) for safety factors in AS 5100.6 (2004) used for short concrete-filled steel tubular columns. The method was applied to an extensive database of 929 stub column tests developed by Tao *et al*. (2008). The calibration shows more interaction between steel and concrete than the values provided by AS 5100.6 (2004), and the authors suggest an improvement for the capacity reduction factors and the capacity prediction models.

Monte Carlo Simulation (MCS) for statistical evaluation has been utilized by Mirza and Skrabek (1991), who investigated the reliability of composite column strength interaction, and Lundberg and Galambos (1996), who examined the reliability indices inherent in the 'Load and Resistance Factor Specification' AISC design code. MCS is widely used in the reliability analysis and design code calibration as it is easy to implement when dealing with nonlinear limit state functions.

## 3. Experimental database

The experimental results described in section 2 are collected and categorized into four categories: (a) RCFSTCs; (b) CCFSTCs; (c) Fully-encased CESCs; and (d) Partially-encased CESCs. Each category contains specimens with high- and normal-strength concrete, and high- and normal strength steel. The parameters recorded for each test are: Outer diameter of circular cross-

Table 1 Experimental results database properties

| Properties | CCFSTCs | RCFSTCs | Full-encased | Part-encased |
|---|---|---|---|---|
| $D$ or $B$ (mm) | 100 – 456 | 100 – 250.1 | 125 – 150 | 140 – 240 |
| $h$ (mm) | – | 100 – 200 | 125 – 150 | 133 – 224 |
| $t$ (mm) | 1.9 – 6.4 | 1.9 – 5.8 | – | – |
| $L$ (mm) | 840 – 2000 | 840 – 4135 | 850 – 1300 | 2440 |
| $f'_c$ (MPa) | 40 – 102 | 27.3 – 96.4 | 25.7 – 45.4 | 38.72 |
| $f_y$ (MPa) | 235 – 350 | 235 – 495 | 235 | 460 |
| $\lambda$ | 34.0 – 118.7 | 22.1 – 111.7 | – | – |
| $D/t$ | 31.4 – 105.2 | 20.4 – 105.2 | – | – |
| No. of specimens | 26 | 63 | 2 | 8 |

Table 2 Compressive concrete strength conversion factors (Yi *et al*. 2006, Aslani 2013)

| For high strength concrete | $f'_{cy(150\times300)}$ | $f'_{cy(100)}$ | $f'_{cy(150)}$ | $f'_{c,pr(150)}$ |
|---|---|---|---|---|
| $f'_{cy(100\times200)}$ | 1.04 | 0.96 | 1.02 | 1.11 |
| $f'_{cy(150\times300)}$ | 1.00 | 0.92 | 0.98 | 0.94 |
| **For normal strength concrete** | $f'_{cy(150\times300)}$ | $f'_{cu(100)}$ | $f'_{cu(150)}$ | $f'_{c,pr(150)}$ |
| $f'_{cy(100\times200)}$ | 1.03 | 0.85 | 0.91 | 1.07 |
| $f'_{cy(150\times300)}$ | 1.00 | 0.82 | 0.88 | 1.05 |

section ($D$), width ($B$) and height ($h$) of square or rectangular cross-sections, steel tube thickness ($t$), compressive strength of concrete ($f'_c$), length of the specimen ($L$), yield strength of the steel tube ($f_y$), and slenderness ($\lambda$), and are listed in Table 1.

Only specimens tested in flexure, and those with sufficient experimental information were considered for the database. If the elastic modulus of steel ($E_s$) was not provided in the study, it was assumed to be 200,000 MPa. Furthermore, compressive concrete strength ($f'_c$) was defined as the strength obtained from 150 × 300 mm cylinder tests. Other concrete types were converted using factors proposed by Yi *et al*. (2006) and Aslani (2013) listed in Table 2.

Table 3 Codes of practice models for *EI*

| Ref. | *EI* specification |
|---|---|
| AS 5100.6 (2004) | $(EI)_e = \phi EI_s + \phi EI_r + \phi_c E_c I_c$<br>$E_c = 5050\sqrt{f'_c}$<br>$E_s = 200{,}000$ MPa<br>$\phi = 1.0$ for composite members<br>$\phi_c = 1.0$ for composite members |
| AS/NZS 2327 (Draft 2015) | $(EI)_e = E_s I_s + E_s I_{sr} + 0.6 E_c I_c$<br>$E_c = w^{1.5}(0.024\sqrt{f_{cmi}} + 0.12)$<br>$E_s = 200{,}000$ MPa |
| Eurocode 4 (2004) | $(EI)_e = E_a I_a + E_s I_s + 0.6 E_{cm} I_c$<br>$E_{cm} = 22{,}000[(f_{cm}/10)]^{0.3}$<br>$E_s = 210{,}000$ MPa |
| AISC (2010) | $EI = E_s I_s + E_s I_{sr} + C_3 E_c I_c$ for CFSTCs<br>$E_c = 0.043 w_c^{1.5}\sqrt{f'_c}$<br>$E_s = 200{,}000$ MPa<br>$C_3 = 0.6 + 2\left(\frac{A_s}{A_c + A_s}\right) \le 0.9$<br>$EI = E_s I_s + 0.5 E_s I_{sr} + C_1 E_c I_c$ for CESC*s*<br>$C_1 = 0.1 + 2\left(\frac{A_s}{A_c + A_s}\right) \le 0.3$ |
| ACI 318 (2010) | $EI = \frac{0.2 E_c I_g + E_s I_{se}}{1 + \beta_d}$<br>$E_c = 4734\sqrt{f'_c}$<br>$E_s = 199{,}948 MPa$<br>$\beta_d = 0$ for short-term loads |
| AIJ (1997) | $K_e = E_s I_s + 0.2 E_c I_c$<br>$E_c = 2100\sqrt{f'_c/19.6}$<br>$E_s = 205{,}800 MPa$ |

## 4. Design code models

This study considers several international design codes for *EI* prediction including: Australian Standard AS 5100.6 (2004), AS/NZS 2327 (Draft 2015), Eurocode 4 (2004), AISC (2010), Japanese code AIJ (1997), and ACI 318 (2010). Each code provides respective $E_c$ and $E_s$ specifications for calculation.

Continual research is performed to update design models and specifications for international codes of practice, as well as independently proposed models. Therefore, this research considers the most recent specifications available. Of the design models listed, only AISC (2010) provides an alternative design model for CFSTC and CESC members. Table 3 provides a complete list of the codes of practice and their respective specifications.

## 5. Reliability analysis

In this paper, MCS is conducted in combination with the test-based statistical procedure proposed by Johnson and Huang (1997) to check the reliability of the composite columns included in the experimental database used in this study when they are designed using the current international design codes. This study focuses on the reliability analysis of a deflection based serviceability limit state, unlike the conventional reliability analyses against an ultimate failure based limit state. The following MCS procedure is proposed and used:

I. Select a design equation that estimates the deflection of composite columns. Multiply a bias-correction constant to the design equation to make the equation unbiased; the bias-correction constant is estimated as the average of the ratio of the test results and the design equation estimations for all columns. In the design equation, all the input parameters are taken as mean-measured values instead of nominal values, and the capacity factors are all omitted.

II. Estimate the COV of the prediction error of the unbiased equation obtained in the previous step; the prediction error is calculated as the ratio of the test results and the estimations from the un-biased design equation.

III. Calculate the design load given to each column inversely, using the design equation where input parameters are nominal values and all capacity factors are considered. In other words, we calculate the design load given to each column using this inverse calculation. Here, the design equations include all capacity factors.

IV. From the calculated design load, calculate the nominal dead load for an assumed ratio of the dead load and the other load types in the chosen load combination. Here, the load combination model includes all load factors. In this study, we use the following load combination: Design load = 1.0 DL + 0.4 LL and assume the ratio of DL and LL (DD/LL) = 1. In addition, we assume that the serviceability failure occurs at approximately half the ultimate load.

V. Using the distributions of input parameters in Table 4 and the modelling error obtained from step II, generate random resistance and load for each column.

VI. Check if the randomly generated deflections exceed the threshold value. In this study, the threshold value is taken as L/500.

VII. Calculate failure probability and reliability index $\beta$ by repeating steps I–VI for $10^6$ samples.

Table 4 Distributions of design parameters and design equations

| Parameters | Mean | COV | Distribution type |
|---|---|---|---|
| Dead load (DL) | 1.00 × nominal | 0.10 | Normal |
| Live load (LL) | 0.6 × nominal | 0.35 | Normal |
| All geometries | 1.00 × nominal | 0.01 | Lognormal |
| Concrete compressive strength ($f_{cm}$) | 1.00 × nominal | 0.10 | Lognormal |

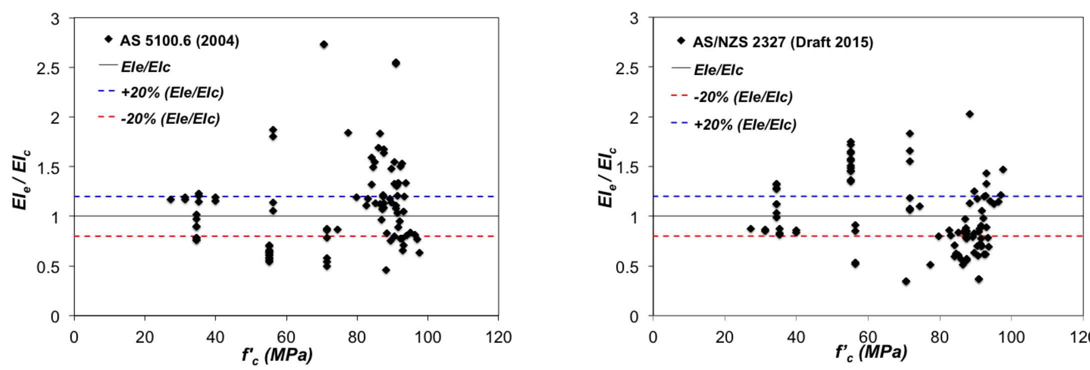
In step V, the uncertainties in the input parameters of the design equations and the load effects are considered as shown in Table 4. It is assumed that all design parameters in the resistance prediction models follow a lognormal distribution with the lower limit at zero as this corresponds to reality (Gulvanesian and Holicky 2005), but the parameters in the load models follow a normal distribution.

## 6. Discussion

### *6.1 Effects of parameters on effective flexural stiffness*

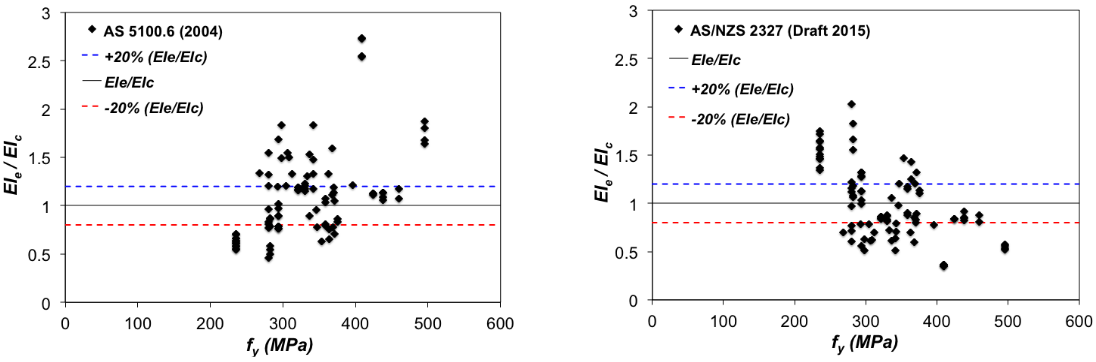
Test specimens taken from literature were examined under a range of varying parameters. The main parameters considered by this study are: Outer diameter of circular cross-section ($D$), width ($B$) and height ($h$) of square or rectangular cross-sections, steel tube thickness ($t$), compressive strength of concrete ($f'_c$), length of the specimen ($L$), yield strength of the steel tube ($f_y$), and slenderness ($\lambda$).

High-strength concrete and high-strength steel are used more frequently in construction, and an increase in $EI$ was seen with increasing $f'_c$ and $f_y$ values for test specimens. Fig. 1 shows plots of $EI_{exp}/EI_{calc}$ versus $f'_c$ for RCFSTCs with codes AS 5100.6 (2004) and AS/NZS 2327 (Draft 2015) respectively. Fig. 2 shows plots of $EI_{exp}/EI_{calc}$ versus $f_y$ for RCFSTCs with codes AS 5100.6 (2004) and AS/NZS (Draft 2015). Fig. 3 shows plots of $EI_{exp}/EI_{calc}$ versus $f'_c$ for CCFSTCs with codes AS



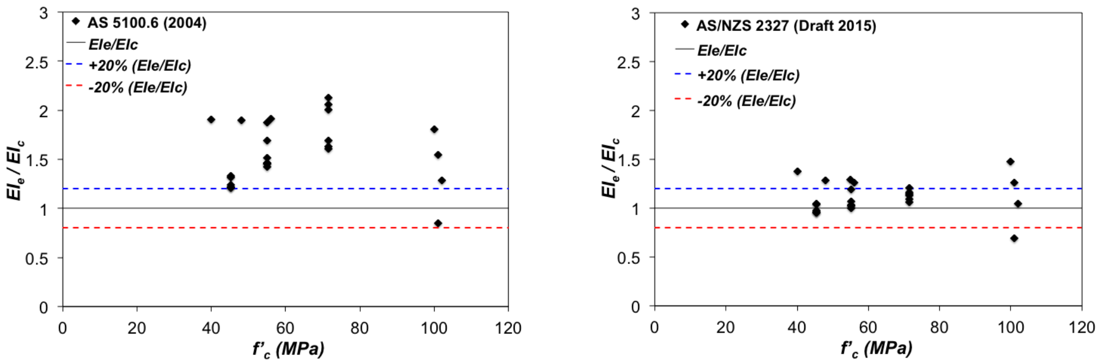

(a) (b)

Fig. 1 The influence of $f'_c$ on RCFSTCs using: (a) AS 5100.6 (2004); and (b) AS/NZS 2327 (Draft 2015) prediction models



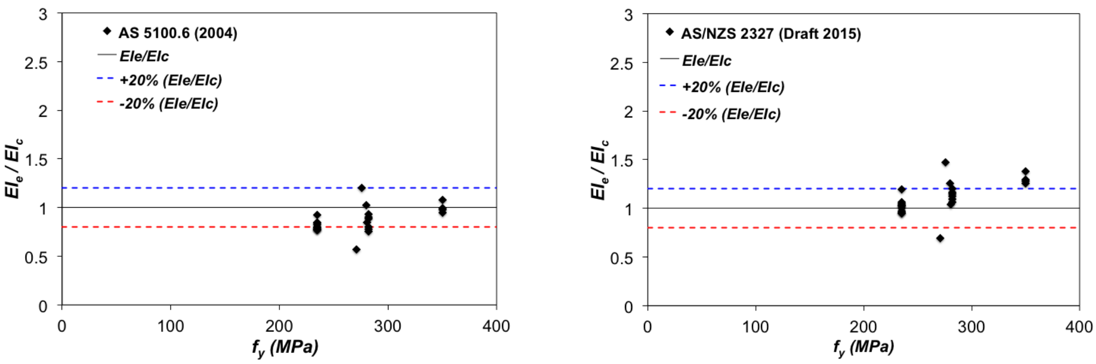

(a) (b)

Fig. 2 The influence of $f_y$ on RCFSTCs using: (a) AS 5100.6 (2004); and (b) AS/NZS 2327 (Draft 2015) prediction models




(a) (b)

Fig. 3 The influence of $f'_c$ on CCFSTCs using: (a) AS 5100.6 (2004); and (b) AS/NZS 2327 (Draft 2015) prediction models




(a) (b)

Fig. 4 The influence of $f_y$ on CCFSTCs using: (a) AS 5100.6 (2004); and (b) AS/NZS 2327 (Draft 2015) prediction models

Table 5 $EI_{exp}/EI_{calc}$ values for CFSTCs

| $EI_{exp}/EI_{calc}$ | RCFSTCs | | CCFSTCs | |
|---|---|---|---|---|
| | Mean | SD | Mean | SD |
| AS 5100.6 (2004) | 1.10995 | 0.4699 | 1.092506 | 0.3766 |
| AS/NZS 2327 (Draft 2015) | 0.86749 | 0.2985 | 1.330350 | 0.4040 |
| Eurocode 4 (2004) | 1.30324 | 0.5435 | 1.294915 | 0.3700 |
| AIJ (1997) | 1.59924 | 0.6301 | 1.698908 | 0.4318 |
| AISC (2010) | 1.55706 | 0.6089 | 1.690100 | 0.4326 |
| ACI 318 (2010) | 1.64064 | 0.6468 | 1.741433 | 0.4434 |

5100.6 (2004) and AS/NZS 2327 (Draft 2015) respectively. Fig. 4 shows plots of $EI_{exp}/EI_{calc}$ versus $f_y$ for CCFSTCs with codes AS 5100.6 (2004) and AS/NZS 2327 (Draft 2015) respectively. Cross-sectional dimensions *D*, *B* and *h*, and *t* were found to have a high influence on *EI* prediction. The effects of confinement are noted to increase strength in a composite member (Roeder *et al*. 2010), and *EI* prediction relies heavily on a moment of inertia (*I*) of the section, which uses cross sectional dimensions for calculating, and will therefore reflect these parameters.

### 6.2 Results for effective flexural stiffness

Existing codes of practice were examined using a database of 100 composite columns. All codes provide limitations and specifications, which are considered in this study. Respective *EI* predictions were compared with experimental results to determine whether the code was conservative or un-conservative. It was found that codes of practice for CFSTCs were generally conservative. Table 5 shows codes of practice AS 5100.6 (2004) had better *EI* prediction for both RCFSTCs and CCFSTCs, with respective average $EI_{exp}/EI_{calc}$ values of 1.109 for RCFSTCs and 1.092 for CCFSTCs.

### 6.3 Results for deflection

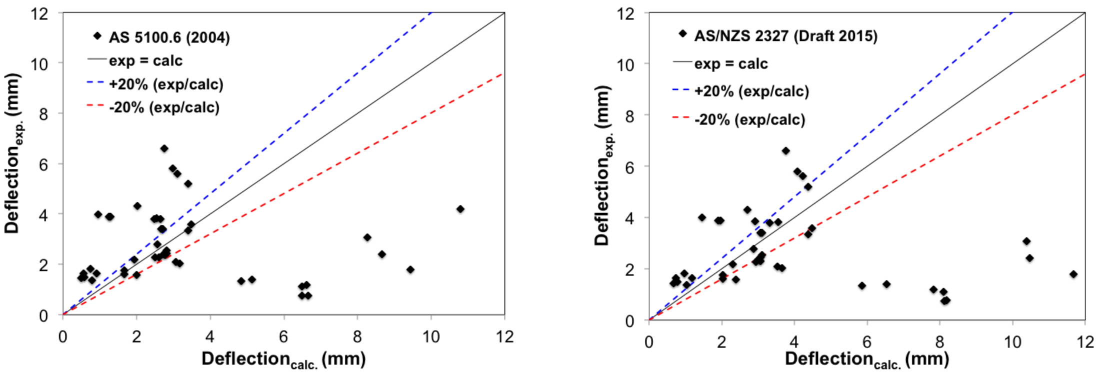
This study also considers the deflection of the specimens under lateral loading. Deflection of a composite member can be determined using *EI* prediction, and is used in calculating serviceability and ultimate limit states for a structure. A deflection value using *EI* models was determined and

Table 6 $\delta_{exp}/\delta_{calc}$ values for composite columns

| $\delta_{exp}/\delta_{calc}$ | RCFSTC | | CCFSTC | | Full. Encased | | Par. Encased | |
|---|---|---|---|---|---|---|---|---|
| | Mean | SD | Mean | SD | Mean | SD | Mean | SD |
| AS 5100.6 (2004) | 1.012 | 0.930 | 1.483 | 1.011 | 1.138 | 0.270 | 0.519 | 0.172 |
| AS/NZS 2327 (Draft 2015) | 0.782 | 0.653 | 1.077 | 0.645 | 0.704 | 0.095 | 0.462 | 0.146 |
| Eurocode4 (2004) | 0.801 | 0.674 | 1.108 | 0.672 | 0.742 | 0.111 | 0.467 | 0.149 |
| AISC (2010) | 0.891 | 0.750 | 1.236 | 0.754 | 0.221 | 0.014 | 0.426 | 0.127 |
| AIJ (1997) | 0.616 | 0.463 | 0.784 | 0.401 | 0.245 | 0.036 | 0.415 | 0.125 |
| ACI 318 (2010) | 0.601 | 0.453 | 0.766 | 0.393 | 0.244 | 0.036 | 0.404 | 0.122 |

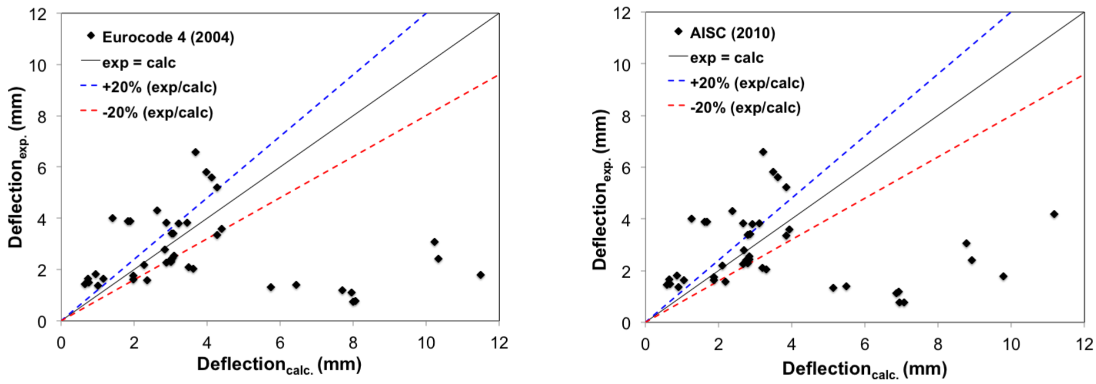
compared to experimental deflections recorded in the literature ($\delta_{exp}/\delta_{calc}$). Table 6 shows for RCFSTCs, *EI* predictions taken from AS 5100.6 (2004), Eurocode 4 (2004), and AISC (2010) produced the closest deflection value to experimental results with an average of 1.012, 0.801 and 0.891, respectively. AS/NZS 2327 (Draft 2015) and Eurocode 4 (2004) were found to have a better prediction for CCFSTCs with an average $\delta_{exp}/\delta_{calc}$ value of 1.077.

Table 6 shows for CESCs, deflections calculated using *EI* models from AS/NZS 2327 (Draft 2015) and Eurocode 4 (2004) showed a better prediction for fully encased members with average $\delta_{exp}/\delta_{calc}$ values of 0.704 and 0.742 respectively. Deflections calculated using *EI* models from AS 5100.6 showed a better prediction for partially encased members, with an average value of 0.519. Table 6 lists the complete comparisons between deflections calculated using respective code *EI* values and experimental deflections. Figs. 5-16 show a comparison of experimental deflections with predicted deflections using AS 5100.6 (2004), AS/NZS 2327 (Draft 2015), Eurocode 4 (2004), AISC (2010), AIJ (1997), and ACI 318 (2010) for RCFSTCs, CCFSTCs, and CESCs.



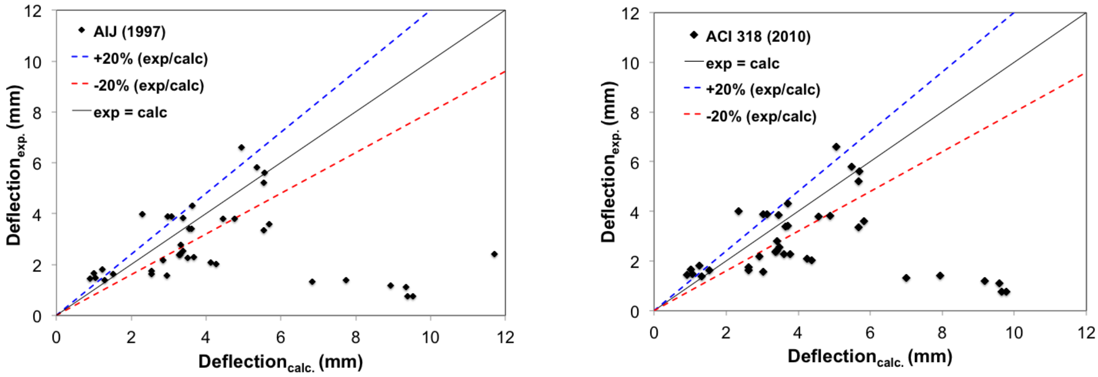

(a) (b)

Fig. 5 Comparison of experimental deflections of with predicted deflections for RCFSTCs using: (a) AS 5100.6 (2004); and (b) AS/NZS 2327 (Draft 2015)



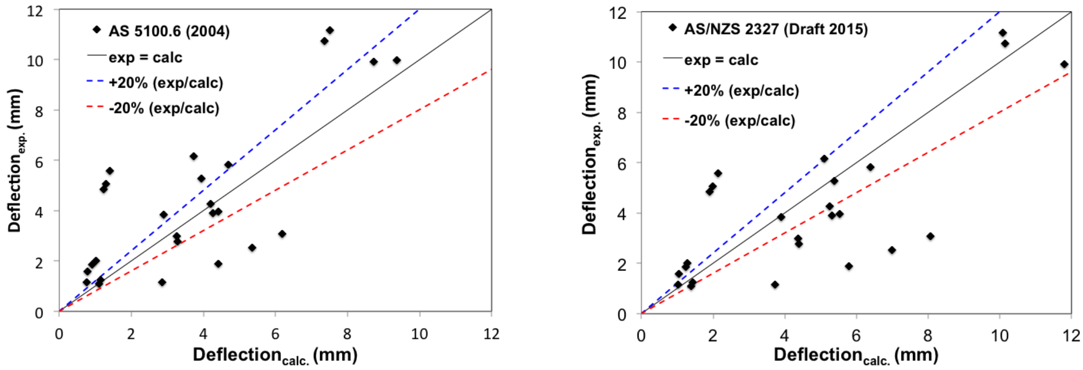

(a) (b)

Fig. 6 Comparison of experimental deflections of with predicted deflections for RCFSTCs using: (a) Eurocode 4 (2004); and (b) AISC (2010)



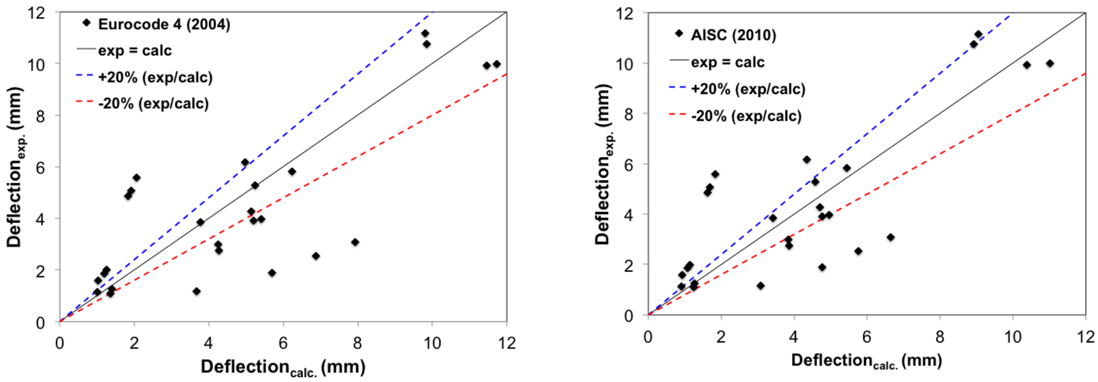

(a) (b)

Fig. 7 Comparison of experimental deflections of with predicted deflections for RCFSTCs using: (a) AIJ (1997); and (b) ACI 318 (2010)



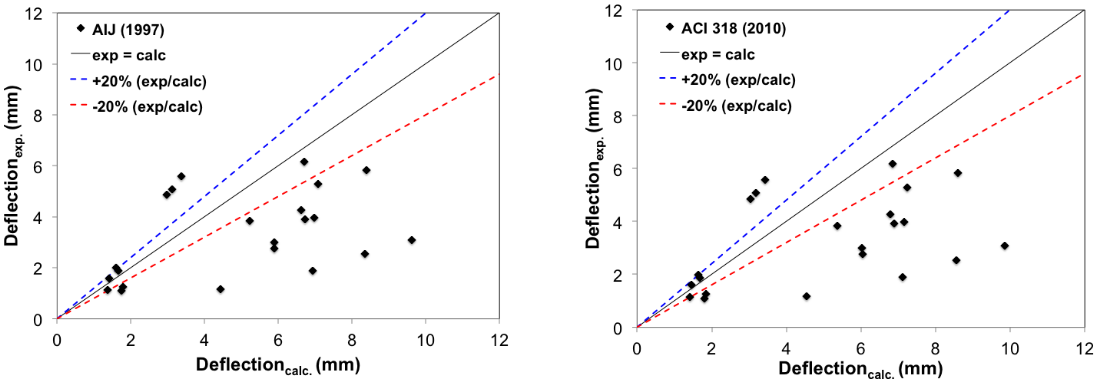

(a) (b)

Fig. 8 Comparison of experimental deflections of with predicted deflections for CCFSTCs using: (a) AS 5100.6 (2004); and (b) AS/NZS 2327 (Draft 2015)



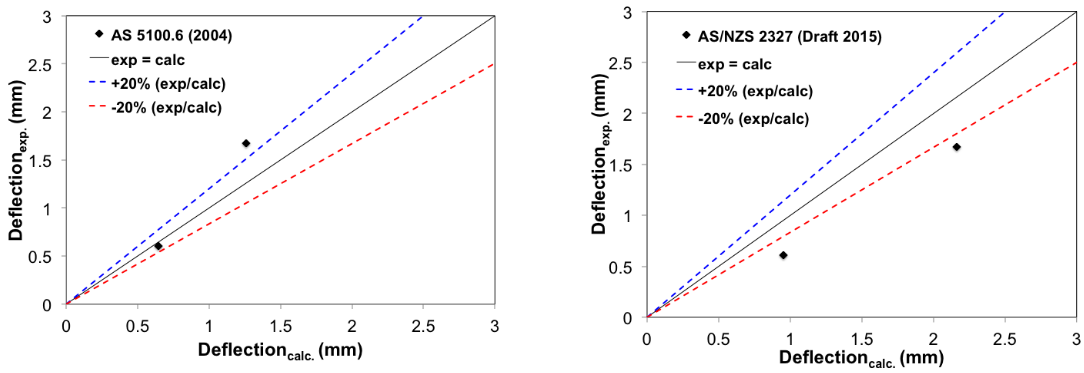

(a) (b)

Fig. 9 Comparison of experimental deflections of with predicted deflections for CCFSTCs using: (a) Eurocode 4 (2004); and (b) AISC (2010)



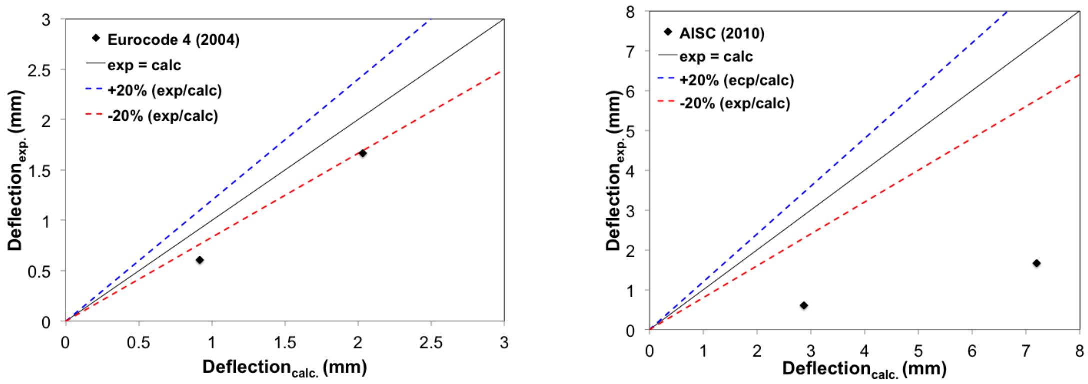

(a) (b)

Fig. 10 Comparison of experimental deflections of with predicted deflections for CCFSTCs using: (a) AIJ (1997); and (b) ACI 318 (2010)



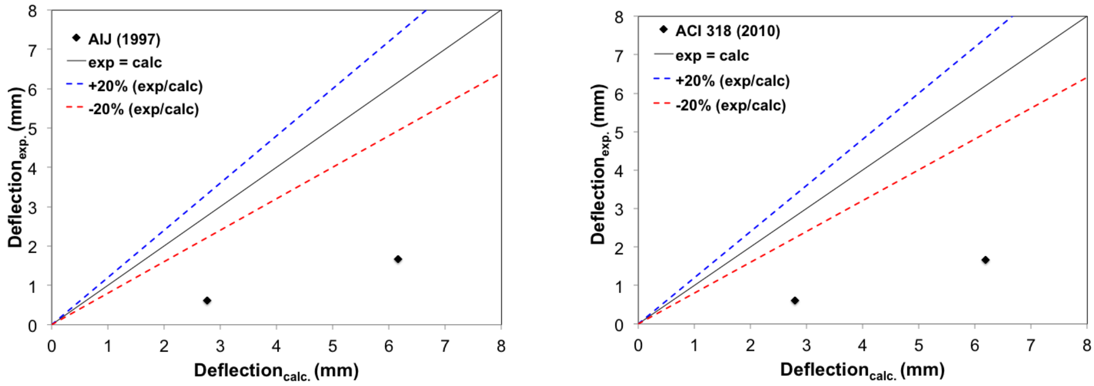

(a) (b)

Fig. 11 Comparison of experimental deflections of with predicted deflections for fully- encased CESCs using: (a) AS 5100.6 (2004); and (b) AS/NZS 2327 (Draft 2015)



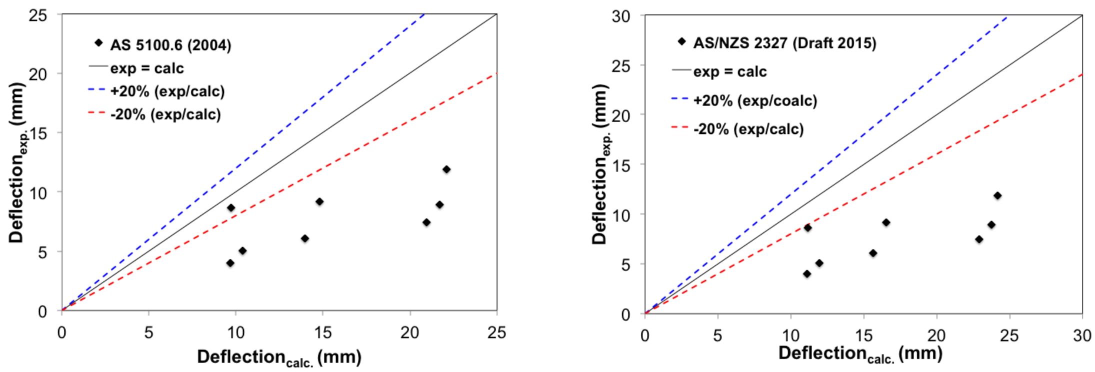

(a) (b)

Fig. 12 Comparison of experimental deflections of with predicted deflections for fully-encased CESCs using: (a) Eurocode 4 (2004); and (b) AISC (2010)



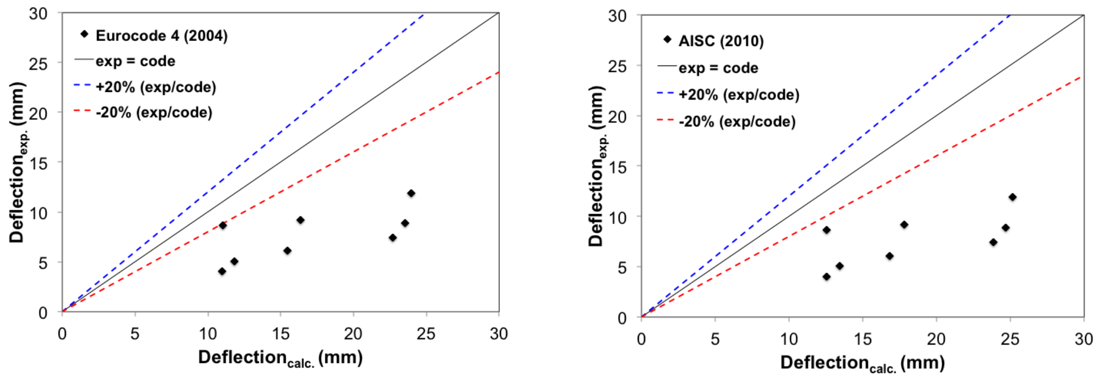

(a) (b)

Fig. 13 Comparison of experimental deflections of with predicted deflections for fully- encased CESCs using: (a) AIJ (1997); and (b) ACI 318 (2010)



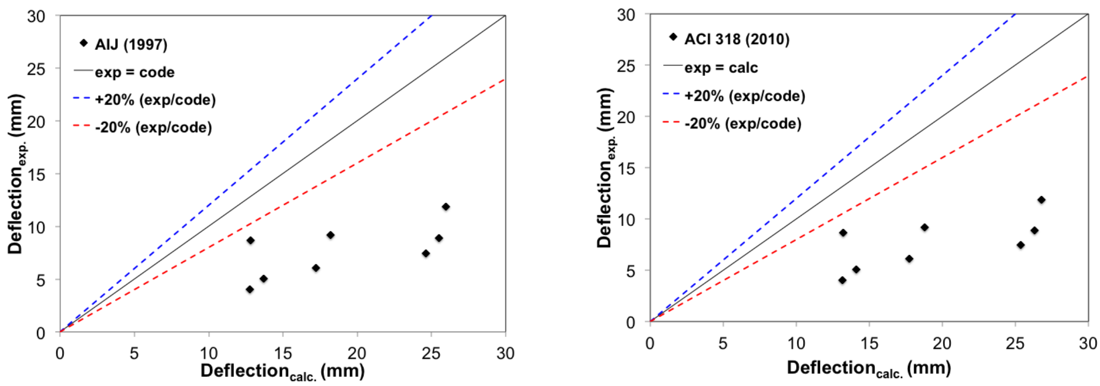

(a) (b)

Fig. 14 Comparison of experimental deflections of with predicted deflections for partially-encased CESCs using: (a) AS 5100.6 (2004); and (b) AS/NZS 2327 (Draft 2015)




(a) (b)

Fig. 15 Comparison of experimental deflections of with predicted deflections for partially-encased CESCs using: (a) Eurocode 4 (2004); and (b) AISC (2010)




(a) (b)

Fig. 16 Comparison of experimental deflections of with predicted deflections for partially-encased CESCs using: (a) AIJ (1997); and (b) ACI 318 (2010)

### *6.4 Results of reliability indices for serviceability limit state*

Table 7 shows the reliability indices of composite columns for a serviceability limit state defined by a threshold deformation limit. The threshold value is taken as 1/500 of the length of a composite column. The MCS procedure described in Section 5 is used to estimate the reliability indices of the composite columns included in the database. The results are reported for different column types and design codes in an average manner. These values can be compared with the typical target reliability index $\beta$ used for capacity factor calibration, which is taken as $\beta = 1.5$ for a serviceability limit state as recommended in AS 5104 (2005)/ISO 2394 (1998). This value is determined based on relatively smaller failure consequences compared to the ultimate limit state failure, which requires the target reliability index $\beta = 3.8$.

For Rectangular CFSTCs, AIJ (1997) and ACI 318 (2010) show a greater reliability index than the target $\beta = 1.5$, while the other codes including AS 5100.6 (2004), AS/NZS 2327 (Draft 2015) and Eurocode 4 (2004) show reliability indices smaller than the target reliability index. The values of the reliability indices are mostly affected by the modelling error (the accuracy of the design equations) usually estimated as the COV of the ratio between the test results and predictions, together with the embedded safety in the prediction model represented by the constant bias of the design equations. Therefore, as the Table 7 shows, the order of the reliability indices follows that of the modelling error. Likewise, in circular CFSTCs, the order of the reliability indices follows that of the modelling error. Most of the reliability indices for circular CFSTCs are smaller than the target reliability index $\beta = 1.5$ and only AIJ (1997) and ACI 318 (2010) are close to the target reliability index. Fully- and partially- encased columns mostly show very high reliability indices even greater than 3.0, but this is not directly comparable to the target reliability index $\beta = 1.5$ because they are estimated based on only a very limited number of test specimens (2 specimens for fully encased columns and 8 specimens for partially encased columns). In most international design codes for composite columns, a serviceability limit state function usually adopts capacity factor 1.0 for both steel and concrete, but it was developed analogously from those for ultimate limit state or from experts' opinion, and thus the results from this study show variation from the target reliability index $\beta = 1.5$. For the design codes that do not meet the target reliability index $\beta =$

Table 7 Reliability indices of composite columns against serviceability limit state

| $\beta_{seviceability}$ | RCFSTC | CCFSTC | Full. Encased | Par. Encased |
|---|---|---|---|---|
| AS 5100.6 (2004) | 1.05 | 0.54 | 1.54 | > 3.00 |
| AS/NZS 2327 (Draft 2015) | 1.36 | 0.95 | > 3.00 | > 3.00 |
| Eurocode 4 (2004) | 1.33 | 0.91 | > 3.00 | > 3.00 |
| AISC (2010) | 1.22 | 0.79 | > 3.00 | > 3.00 |
| AIJ (1997) | 1.69 | 1.33 | > 3.00 | > 3.00 |
| ACI 318 (2010) | 1.72 | 1.36 | > 3.00 | > 3.00 |

1.5, increased safety can be achieved by conducting more experiments or by developing better-performing design equations based on statistical fitting or further mechanical investigation.

## 7. Conclusions

The following conclusions can be made with the present scope of investigation:

- The results show for RCFSTCs and CCFSTCs, AS 5100.6 (2004) and AS/NZS 2327 (Draft 2015) codes of practice had better *EI* predictions with high standard deviations.
- The results show for RCFSTCs, deflections calculated using *EI* models from AS 5100.6 (2004), Eurocode 4 (2004), and AISC (2010) showed better prediction with high standard deviations compared to the other codes of practice.
- The results show for CCFSTCs and fully-encased CESCs, deflections calculated using *EI* models from AS/NZS 2327 (Draft 2015) and Eurocode 4 (2004) showed better prediction with high standard deviations compared to the other codes of practice.
- Deflections calculated using *EI* models from AS 5100.6 showed better prediction with high standard deviations for partially-encased CESCs.
- The results show for RCFSTCs and CCFSTCs, AIJ (1997) and ACI 318 (2010) show a greater reliability index than the target reliability index $\beta = 1.5$, while the other codes including AS 5100.6 (2004), AS/NZS 2327 (Draft 2015) and Eurocode4 (2004) show reliability indices smaller than the target reliability index.
- In most of the codes of practice for composite columns, a serviceability limit state function usually accepts capacity factor 1.0 for both steel and concrete, but it was established analogously from those for ultimate limit state or from professionals' view, and therefore the results from this study display variation from the target reliability index 1.5.
- For RCFSTCs, CCFSTCs, and CESCs, the AIJ (1997) and ACI 318 (2010) show the lowest standard deviations, making them more reliable predictors for deflection. Both AIJ (1997) and ACI (2010) use a capacity factor of 0.2 in the *EI* equations, which is far lower than the other codes. The AS5100.6 (2004) uses a factor of 1.0 and AS/NZS 2327 (Draft 2015) together with Eurocode 4 (2004) uses a factor of 0.6. From these results, a capacity factor of 0.2 provides the most reliable deflection results.

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