Design resistance of helical seam pipe columns with limited tensile test data

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ABSTRACT

This study conducts a capacity factor calibration for steel and steel-concrete composite columns with helical seam pipe (also known as spiral welded tube) sections considering the following three member types: (i) steel columns under axial compression, (ii) concrete-filled steel tubes (CFSTs) under axial compression, and (iii) CFSTs under combined axial compression and uniaxial bending due to load eccentricity. The calibration has been conducted for both forward and inverse reliability analyses. The forward analysis calibrates the capacity factor of steel contribution in the design models given in NZS 3404.1, AS 4100, AS/NZS 5100.6 and AS/NZS 2327 to meet the target reliability level provided in both ISO 2394 and AS 5104 when using API 5L products in non-composite and composite column applications. Whilst the inverse analysis estimates the required minimum number of steel tensile tests when the target reliability level and the capacity factors are all provided. In these analyses, a total of 68 experimental data collected from the literature are utilised to estimate the modelling uncertainty in terms of bias and scatter. For all member types, the design models achieve similar or higher reliability than the target reliability, and the corresponding required minimum amount of steel material tests are calculated and provided.

Keywords: API 5L; steel-concrete composite columns; steel columns; design resistance; capacity factors; steel reuse.

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1. Introduction

1.1 Background

Steel and concrete composite construction has achieved a high market share in multi-storey buildings and bridges. Composite columns, or concrete-filled steel tubes (CFSTs), using circular hollow sections have become popular because: greater resistance than rectangular or square hollow sections is achieved through confinement of the concrete core [1,2,3]; the rules in current design standards such as AS/NZS 2327 [4], EN 1994-1-1 [5], AISC 360-16 [6] imply that more structurally efficient cross-sections can be achieved, due to reduced local bucking from the presence of the concrete infill (however, it has recently been shown that this enhancement may be limited [7]); and improved ductility and damping characteristics [8]. The presence of the steel section also eliminates the need for formwork and, as the erection schedule is not dependent on concrete curing time in these situations, CFSTs improve the speed of construction [9].

Whilst there has been a focus on the performance of CFSTs using standard section sizes, tailor-made circular sections are becoming popular with designers as larger diameter (or non-standard diameter) tubes can be specified in order that more structurally efficient solutions can be achieved. One of the most popular types of tailor-made circular section are helical seam pipe, which are also known as spiral welded tubes (SWTs). SWT’s have historically been used in the conveyance of liquids or gases, and are fabricated by helically bending a continuous length of steel plate and welding the abutting edges [10]. The benefit of manufacturing SWTs compared to longitudinally welded tubes (LWTs) arise from the efficiencies generated from the continuous spiral welding process, the lower cost of SWT manufacturers, and the fact that SWTs of different diameters can be readily manufactured using the same forming tools. One of the most widely used product standards internationally for SWTs is API 5L [11]. This is recognized by the International Organization for Standardization (ISO) where continued harmonisation of ISO 3183 [12] with API 5L has led to the 2019 version of this standard only presenting supplementary rules to complement API 5L, thereby resulting in a document that is around six-times shorter than previous versions of ISO 3183.

In the construction sector, API 5L products have been successfully used for columns in multi-storey buildings in the Asia-Pacific region for several years. Based on this long experience, the Hong Kong Buildings Department [13] and the Building Construction Authority in Singapore [14] permit the use of API 5L products in the design of steel structures. Whilst full-scale tests on both non-composite and composite columns using SWTs have been conducted, and the results compared with the predictions given by different design standards, the present authors are unaware of any structural reliability analyses that have been undertaken to evaluate the capacity reduction factors required for design. Moreover, given that API 5L SWTs are normally used for conveyance of liquids or gasses, the yield strength of the material is measured in the circumferential direction. As a consequence of this, there is a need to define the required minimum yield strength of the steel in the longitudinal direction when SWTs are designed as columns.

The magnitude of the capacity factors given in steel and composite design standards are often based on production control measurements of geometry and yield strength from steel producers that are servicing a particular market at that time [15]. However, if different steel producers subsequently enter the market, this could potentially invalidate the magnitude of the capacity factors previously calculated because the required statistical input values are not normally given in product standards [16]. This issue was recently identified in the RFCS SAFEBRICTILE project, where it was intended that the statistical properties evaluated for design could be included within future versions of the European product standards [17]. Moreover, there is also a growing need to reuse reclaimed steel in structural applications due to its environmental advantages. In these circumstances, not all of the material and geometrical characteristics are known. Recent design guidance has recommended that more onerous capacity factors than currently given in design standards should be used [18], but these are not based on structural reliability analyses. Conversely, when tensile coupons are taken from the reclaimed steel products and tests are undertaken, a calculation procedure is given to evaluate the characteristic mechanical properties from the results [18], but what impact this has on design resistance of the composite member remains unknown.

In the present paper, the sensitivity of the capacity reduction factors for the design models given in NZS 3404.1 [19], AS 4100 [20], AS/NZS 5100.6 [21] and AS/NZS 2327 [4] are investigated when using API 5L [11] products in non-composite and composite column applications. Whilst the focus of this paper is to support SWTs in Australian/New Zealand standards, given that the design models are similar to those used in other international standards, coupled with the fact that the reliability analyses are conducted according to ISO 2394 [22] (in Australia and New Zealand AS 5104 [23] is an identical adoption of ISO 2394), the results from this paper have a wider international relevance. In a similar way as an earlier study that considered the reliability of beams and columns using standard section sizes [24], it is proposed to investigate the sensitivity of using API 5L products by undertaking inverse reliability analyses for each of the design models to identify the minimum number of mechanical tests that would be required to deliver a particular capacity reduction factor. This latter information will be of particular importance to designers when the nominal yield strength of a product is unknown.

1.2 Research objectives

In this study, the following research objectives are identified: (i) experimental databases for steel and steel-concrete composite columns with SWTs are established by collecting data from the literature. Then, reliability analyses are conducted to evaluate the capacity reduction factors for the API 5L [11] steel grades identified below using the design models given in NZS 3404.1 [19], AS 4100 [20], AS/NZS 5100.6 [21] and AS/NZS 2327 [4]. (ii) Inverse reliability analyses are undertaken for each of the design models to identify the minimum number of mechanical tests that will be required to deliver the target capacity reduction factors.

Whilst API 5L [11] provides a wide range of sections, from consultation with the industry it was proposed to narrow the scope to include only welded steel (seamed) pipe, considering the cost premium for seamless pipes. On this basis, the study was confined to product specification level 2 (PSL 2) pipes which have a maximum f_y/f_u ratio of 0.93, using the following steel grades: X42 (290), X46 (320), X52 (360), X56 (390), X60 (415), X65 (450), X70 (485), and X80 (555) [N.B. The values in the parenthesis indicate the nominal yield strengths in MPa].

2. Reliability based capacity factor calibration

2.1 Target reliability level

Capacity factors are required to be calibrated to meet the acceptable level of consequences of structural design failure. This acceptable level is described by a target failure probability or a reliability index. The failure probability P_f and the reliability index β has the following relationship:

$$P_f = \Phi(-\beta) \tag{1}$$

where Φ is the cumulative distribution function (CDF) of the standardised normal distribution.

Both ISO 2394 [22] and AS 5104 [23] suggest that, at the ULS when the failure costs are large and the relative costs of safety measures are normal, the target reliability index related to a one-year reference period is β_1 = 4.4. However, in most design standards around the world the evaluation of the capacity reduction factors have been based on a 50-year reference period where the previous version of ISO 2394 [25] and EN 1990 [26] give β_{50} = 3.8. According to EN 1990, the values of β for a different reference period can be calculated using the following equation:

$$\Phi(\beta_n) = \left[\Phi(\beta_1)\right]^n \tag{2}$$

where β_n is the reliability index for a reference period of n years and β_1 is the reliability index for one-year.

From using Eq. (2) it can be seen that for β_{50} = 3.8 this is equivalent to β_1 = 4.7. However, considering that full mutual independence of failure events in subsequent years and expressed as the multiplications of the terms. This assumption is unrealistic, and it is suggested that β_{50} = 3.8 can be more realistically interpreted to be β_1=4.4 [27]. From this finding, and for consistency with the basis of international design standards for steel and composite construction, the target reliability index used in the present paper is based on a 50-year reference period with $\beta_t = \beta_{50}$ = 3.8.

When we consider a case that the resistance has an unfavourable value without considering the load effect, this probability has the following relationship with the corresponding reliability index:

$$P(R \le R_d) = \Phi(-\alpha_d \beta_t) = \Phi(-\beta_R) \tag{3}$$

where α_d is the design value for the first-order reliability method (FORM) [28] influence coefficient which has the value of 0.8 according to ISO 2394 [22]/AS 5104 [23], R is the resistance, and R_d is the design resistance.

2.2 Capacity factor calibration procedure considering the effect of statistical uncertainty in material tests

The capacity factor calibration procedure proposed in Kang *et al.* [24] based on EN 1990 Annex D.8 [26] is used to carry out the following two types of analyses by considering the effect of statistical uncertainty in material tests: (i) forward analysis for calibrating a capacity factor for steel contribution in a structural member; and (ii) inverse analysis to estimate the minimum number of tensile tests for the steel yield strength. This method is selected because it considers that the resistance value always has a non-negative value, and thus it follows a lognormal distribution. This method calibrates capacity factors separately from load factors using the concept of the first-order reliability method (FORM) [27], where sensitivity factors separate the calibration procedures for resistance and load. In this method, the modelling error of the prediction models is statistically estimated from the comparison between the model predictions and the experimental observations for those predictions.

In the forward analysis, the capacity factor (ϕ) for a resistance prediction model is defined as follows:

$$\phi = \frac{R_d}{R_n} \tag{3}$$

where R_d = the design resistance to meet the target reliability for resistance, and R_n = the nominal resistance.

The calculation of these resistance parameters are carried out as follows: let us assume that $g_R(\mathbf{x})$ is a resistance prediction model and $\mathbf{x}$ is a vector of mean-measured values for design parameters. The constant bias of this model can be statistically represented as follows:

$$\bar{b} = \frac{1}{N}\sum_{i=1}^{N}\left(\frac{R_{ei}}{R_{ti}}\right) \tag{4}$$

where N = the number of experimental data, R_{ei} = the resistance observed from the i^{th} specimen in the experiment, and R_{ti} = the resistance predicted by the resistance prediction model $g_R(\mathbf{x}_\mathrm{i})$ for the i^{th} specimen. The unbiased resistance prediction R using this bias correction term is calculated as follows:

$$R = \bar{b} g_R(\mathbf{x})\delta \tag{5}$$

where δ = the modelling error of the unbiased resistance prediction. The modelling error for the i^{th} experiment, δ_i, can be estimated as follows:

$$\delta_i = \frac{R_{ei}}{\bar{b} R_{ti}} \tag{6}$$

As it is assumed that R in Eq. (5) follows a lognormal distribution with non-negative values, the coefficient of variation (COV) of R is estimated as follows:

$$V_R \cong \sqrt{\left(V_\delta^2 + V_{Rt,inf}^2 + V_{Rt,finite}^2\right)} \tag{7}$$

where V_δ = the COV of the modelling error estimated using Eq. (6), and $V_{Rt,inf}$ = the COV of the parametric uncertainty for the parameters with an infinite number of experiments, and $V_{Rt,finite}$ = the COV of the parametric uncertainty for the parameters with a finite number of experiments. This equation assumes that the modelling error and the parametric uncertainty are statistically independent. V_{Rt} can be calculated by using the Monte Carlo simulations or the first-order approximations. The standard deviation of ln$R$ ($\sigma_{\ln R}$) is estimated as follows:

$$\sigma_{\ln R} = \sqrt{\ln\left(1+V_R^2\right)} \tag{8}$$

which is used to calculate the design resistance (R_d) in Eq. (3) as follows:

$$R_d = \bar{b} g_R(\mathbf{x}) \exp\left(-k\sigma_{\ln R} - 0.5\sigma_{\ln R}^2\right) \tag{9}$$

where

$$k = \frac{\left(k_{d,m}V_\delta^2 + \beta_R V_{Rt,inf}^2 + k_{d,p}V_{Rt,finite}^2\right)}{V_R^2} \tag{10}$$

where $k_{d,m}$ = the fractile factor determined for a finite number of structural member tests, and $k_{d,p}$ = the fractile factor determined for a finite number of structural material tests, to represent the statistical uncertainty due to the finite number of test data. These factors are for the target reliability index for resistance (β_R) at the 75% confidence level and can be calculated for unknown $\sigma_{\ln R}$ as follows:

$$k_d = t_\beta(n-1) \times (1+1/n)^{0.5} \tag{11}$$

where $t_\beta(n-1)$ is the fractile of the *t*-distribution for the probability corresponding to β_R. This fractile factor is used to consider the statistical uncertainty caused by a finite number of tests that is far from infinity.

R_n in Eq. (3) can be calculated using the resistance prediction model $g_R(\mathbf{x}_\mathrm{n})$ by inserting the nominal input parameters $\mathbf{x}_\mathrm{n}$. When nominal input parameters are not available, the characteristic values based on the 5% fractile value can alternatively be used [29], or the nominal parameters can be inferred from the product standard tolerance ranges.

The inverse analysis also uses the above-mentioned procedures, and the purpose is not to calibrate the capacity factor for the given target reliability level, but to estimate the minimum required number of material tests to meet the target reliability level for a fixed/given capacity factor [24]. In particular, in the inverse analysis, the number of test samples (n) in Eq. (11) for material tests for the steel tensile strength is estimated. When the capacity factor (ϕ) is fixed or given, Eq. (3) takes the following form:

$$\phi R_n = R_d(n,\mathbf{x}) \tag{12}$$

where n is the only unknown term, and it can be numerically solved using optimisation algorithms such as Active-Set Optimisation algorithm [30] and pattern search [31].

3. Forward and inverse reliability analyses for steel and composite columns

The forward and inverse analysis methods presented in the previous section are conducted to estimate the capacity factors and the required minimum number of steel material tests for steel and composite columns, respectively. All the considered columns utilised helical seam pipe, or spirally welded steel tubes (SWTs). The following three types of sections are considered in these analyses: (i) steel hollow sections under pure compression, (ii) concrete-filled steel tubes (CFSTs) under pure compression, and (iii) CFSTs under eccentric loading (combined axial compression and uniaxial bending).

One concern of using SWTs in structural applications is whether anisotropy of the steel properties might exist, owing to the fact that the plate rolling direction and the longitudinal axis of the tube are no longer parallel. However, this concern has recently been addressed through two independent investigations on SWTs [32,33], where the results from tensile tests suggest that the material properties may be treated as isotropic. Due to this finding, in the remainder of this paper it is assumed that the yield strength of the steel in SWTs is independent of orientation.

3.1 Parameter uncertainties

All the parameters in the resistance prediction equation have aleatoric uncertainties, and in this study, they are estimated from the manufacturing tolerances or the literature. The parametric uncertainties are represented in terms of the bias and scatter, and the manufacturing tolerances for the tube diameter, thickness, and the tensile strength of steel presented in Tables 1 and 2 are considered to estimate these. We assume that these parameters are uniformly distributed within the tolerance ranges. It is assumed that the mean of each parameter is the mid-point of the range, and the standard deviation of each parameter that is uniformly distributed within the range of the minimum value of r_{min} and the maximum value of r_{max} is calculated as $(r_{max} - r_{min})/\sqrt{12}$. For all the other parameters that are not specified in these manufacturing tolerances, the COV values have been taken from the literature such that the COV of the compressive strength of concrete (f'_c) is taken as 0.10 [34] and the COVs of all linear dimensions unspecified in the manufacturing tolerances are taken as 0.01 [34].

Table 1. Manufacturing tolerances for hollow sections in API specification 5L [11]

Parameter	**API 5L**	
Outside dimension for CHS (d_o) (mm)	$d_o \leq 168.3$ $168.3 < d_o \leq 610$ $d_o > 610$	$-0.0075\ d_o$ $-0.0075\ d_o$ (> -3.2) $-0.005\ d_o$ (> -4.0)
Thickness for CHS (t) (mm)	$t \leq 5.0$ $5.0 < t \leq 15.0$ $t > 15.0$	-0.5 $-0.1\ t$ -1.5

Table 2. Requirements for the results of tensile tests for PSL 2 pipes in API specification 5L [11]

Pipe grade	**Yield strength (MPa)**	
	Minimum value	**Maximum value**
X42 (290)	290	495
X46 (320)	320	525
X52 (360)	360	530
X56 (390)	390	545
X60 (415)	415	565
X65 (450)	450	600
X70 (485)	485	635

X80 (555)	555	705

3.2 Steel hollow sections

3.2.1 Experimental database

Tests on 11 specimens have been collected for SWT columns to estimate the modelling uncertainty statistically, and their input parameters together with their load bearing capacities are provided in Table 3. In the table, D = the diameter of the SWT, t = the thickness of the tube, f_y = the steel yield strength, L_e = the effective length of the column, and P_{max} = the axial load bearing capacity of the column. The experimental data were collected from the following four publications: Akiyama *et al.* [35], Bao *et al.* [36], Gardner [37], and Aslani *et al.* [38]. These data are compared with resistance model predictions, and the modelling error δ_{i} is statistically estimated using Eq. (6). It should be noted that, in the tests by Aslani *et al.*, the SWTs were manufactured with only single-sided MIG welding on the outside of the pipe, as opposed to the requirement given in API 5L that the welding should be both on the inside and outside. However, given the paucity of experimental data available on SWTs, coupled with the fact that it is likely that any geometrical imperfections introduced from single-sided welding would lead to more conservative resistances, the experimental data presented in Table 3 were first assumed to belong to the same population within the reliability analyses. To investigate whether this assumption was reasonable, the data was subsequently split into subsets corresponding to the welding method and the reliability analyses repeated.

Table 3. Experimental database for spirally welded steel tube columns under axial compression

Article	**Specimen**	D **(mm)**	t **(mm)**	f_y **(MPa)**	L_e **(mm)**	P_{max} **(kN)**
Akiyama *et al.* [35]	SA-7	400	7	409	1600	3360
	SA-9	400	9	470	1600	5350
Bao *et al.* [36]	c-st-1	323	6	344.35	970	1980
	c-st-2	323	6	344.35	970	1990
Gardner [37]	9	168.8	2.64	298.12	1730	412.66

	10	169.3	2.64	317.74	1730	365.79
Aslani *et al.* [38]	H0SWT102-S	103.05	1.9	288	300	183.6
	H0SWT152-S	152.75	1.9	288	450	221.93
	H0SWT203-S	204.25	1.9	288	600	274.5
	H0SWT254-S	251.75	1.9	288	750	320.4
	H0SWT203-L	203	1.9	288	1400	296.8

3.2.2 Resistance prediction models

In NZS 3404.1 [19], AS 4100 [20], and AS/NZS 5100.6 [21], the nominal section capacity (N_s) of a steel column under axial compression is predicted as follows:

$$N_s = \phi k_f A_n f_y \tag{13}$$

where ϕ = the capacity factor for steel with a value of 0.9; k_f = the form factor defined as a ratio of the effective to the gross areas of the section; A_n = the nominal area of the steel sections; and f_y = the nominal yield strength of the structural steel.

The ultimate member capacity (N_c) of a steel column under pure axial compression is predicted as follows:

$$N_c = \alpha_c N_s \le N_s \tag{14}$$

where α_c = the member slenderness reduction factor calculated according to NZS 3404.1, AS 4100, and AS/NZS 5100.6.

3.2.3 Analysis results

The calibrated capacity factors for steel in the circular hollow sections under axial loading are plotted in Figure 1. The forward analysis procedure introduced in the previous section is used. The capacity factors are calculated for the target reliability index β_R for resistance values, for the range of 2.5-4.2.

The analyses are repeated for different COV values of steel yield strength. As expected, the capacity factor values decrease as the target reliability level increases and the COV of steel yield strength increases. For the COV of steel yield strength of 7% [39] and β_R = 3.04 (see Eq. (3)), the estimated capacity factor has a value of 0.893 (see Table 4), which is close to the value of 0.90 provided in NZS 3404.1, AS 4100, and AS/NZS 5100.6. This means that the capacity factor provided in the current design standards just meets the desired target reliability level, but it does not have extra safety or redundancy over the target reliability level. This is different from the design of other member types or failure modes that usually have additional safety or conservatism by using capacity factors. This is observed especially in the design of hollow section columns under axial compression, because the effect of the uncertainty in the section thickness directly and significantly affects the overall parametric uncertainty of a member. This observation is consistent with the findings presented by Kang *et al.* [24] for rectangular hollow sections under axial compression.

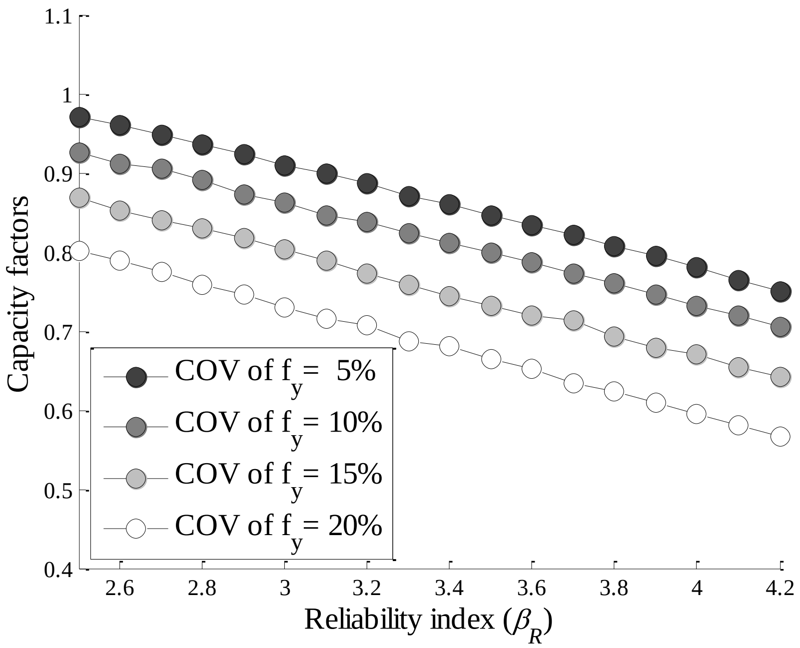

Figure 1. Calibrated capacity factor for steel in spirally welded steel tube columns under axial compression for various COV of the steel yield strength

To investigate whether it was reasonable to include the experimental data with single-sided welding, the data was split into subsets corresponding to the welding method, and the reliability analyses were repeated with a COV for steel yield strength of 7% and β_R = 3.04 (see Table 4). To ensure a fair comparison, the capacity factor ϕ values are calculated by assuming that the fractile $k_{d,m}$ evaluated for the combined data (see Eq. (3)), remains constant in order not to introduce any additional statistical uncertainty. As can be seen from Table 4, the values of bias and overall error are relatively insensitive to the welding method. From this simple comparison, it appears that the initial assumption made in Section 3.2.1 that the inclusion of the single-sided welding data would lead to more conservative design resistances is reasonable, which is reflected in the lower capacity factor value in Table 4.

Table 4. Comparison of uncertainties and capacity factor for steel hollow sections under axial compression

	n (number of tests)	$\bar{b}$ (bias)	V_δ (Model error)	V_{Rt} (Parametric error)	V_R (overall error)	ϕ
All	11	0.993	0.064	0.083	0.105	0.893
Double-sided welding	6	0.984	0.068	0.083	0.108	0.906
Single-sided welding	5	1.004	0.064	0.084	0.105	0.861

Next, the inverse analysis according to the procedure provided in the previous section has been carried out, and the minimum number of steel tensile tests required to meet the target value of β_R = 3.04 have been calculated. The analyses have been repeated for the COV of f_y values with the range of 2-12%. The results are plotted in Figure 2, and it shows an increasing trend of the required number of material tests according to the increasing COV of f_y, as the uncertainty of the resistance prediction increases, the statistical uncertainty due to the insufficient amount of material tests should be reduced to keep the target reliability level. The proposed number of required steel material tests are also tabulated in Table 5 according to this analysis for practical purposes. As can be seen, more than 30 tests are required when the observed COV of the material tests is greater than 8%. This finding is because the resistance model has a very small conservatism and just meets the target reliability level even with infinite material tests.

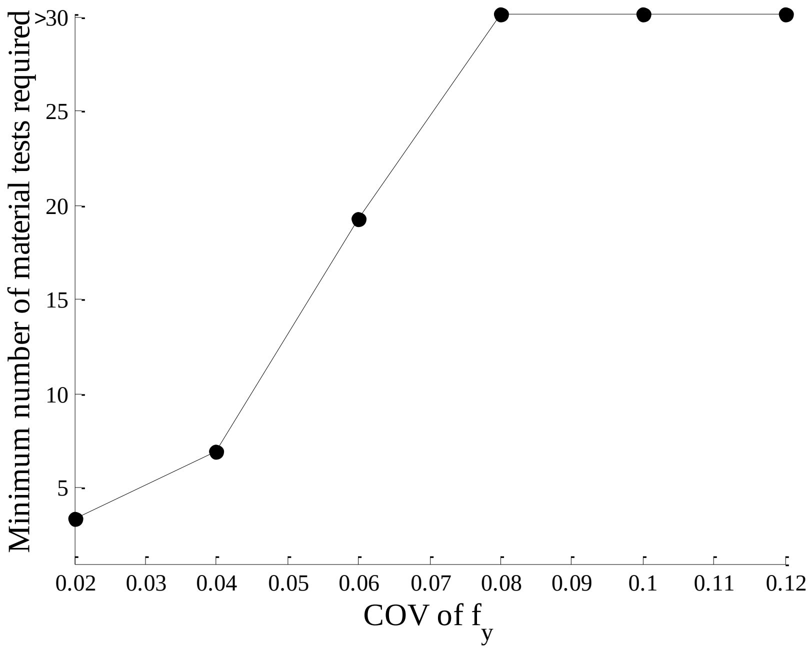

Figure 2. The minimum number of required steel material tests to meet the target reliability level for spirally welded steel hollow tubes under axial compression

Table 5. Recommended number of steel yield strength tests for spirally welded hollows sections under axial compression (based on f_{ym}/f_{yn} =1.35)

COV of f_y	**2%**	**4%**	**6%**	**≥ 8%**
# of tests	3	7	19	>30

3.3 CFST sections under axial compression

3.3.1 Experimental database

A total of 41 specimens have been collected for CFSTs with SWTs under axial compression. The input parameters and load bearing capacities are provided in Table 6, where f_{cm} is the mean measured concrete compressive strength. The experimental data were collected from the following six publications: Wang *et al.* [40], Aslani *et al.* [38], Akiyama *et al.* [35], Gardner [37], Gunawardena *et* *al.* [41], and Gunawardena and Aslani [42]. However, in the tests by Aslani *et al.* together with Gunawardena and Aslani, the SWTs were manufactured with only a single-sided MIG welding on the outside of the pipe (as opposed to the requirement given in API 5L that the welding should be both on the inside and outside of the pipe). For the same reasons given in Section 3.2.1, the experimental data presented in Table 6 were initially assumed to belong to the same population within the reliability analyses. However, to investigate whether this assumption was reasonable, the data was again split into subsets corresponding to the welding method and the reliability analyses subsequently repeated.

For the cases when the mean measured cylinder compressive strength of concrete (f_{cm}) values are not provided in the experimental data, but only the cube strength (f_{cu}) are provided instead, the conversion table provided by Yu *et al.* [43] was used, which represents the approximate relationship between the cylinder strength (f_{cm}) and the cube strength (f_{cu}).

Table 6. Experimental database for CFSTs under axial compression

Article	**Specimen**	**D (mm)**	**t (mm)**	**f_y (MPa)**	**f_{cm} (MPa)**	**L_e (mm)**	**P_{max} (kN)**
Wang *et al.* [40]	CD4-1	425.8	5.2	259.8	42.51	1278	10523
	CD4-2	427.1	5.1	259.8	42.51	1278	10784
	CD6-1	628.5	6.9	276	42.51	1890	21207
	CD6-2	628	7.1	276	42.51	1890	21582
	CD8-1	817.4	9	278.8	42.51	2460	36933
	CD8-2	820.8	9.3	278.8	42.51	2460	37221
Aslani *et al.* [38]	C-SWT102-S	103	1.9	288	38.78	300	609.81
	C-SWT152-S	152.25	1.9	288	38.78	450	1044.92
	C-SWT203-S	204	1.9	288	38.78	600	1788
	C-SWT254-S	252	1.9	288	38.78	750	2525
	C-SWT203-L	203.25	1.9	288	38.78	1400	1698
Akiyama *et al.* [35]	SB20-7	400	7	409	21.7	1600	6740
	SB20-9	400	9	470	25.6	1600	9510
	SB60-9	400	9	470	40.5	1600	12494
Gardner [37]	1a	168.8	2.64	298.12	17.95	305	1323.9
	2a	168.8	2.64	298.12	34.13	305	1220.93
	3a	169.3	2.62	317.74	36.58	305	1304.28

	4a	169.3	2.62	317.74	33.54	305	1328.8
	5a	168.3	3.6	221.63	26.58	305	1559.26
	6a	168.3	3.6	221.63	32.75	305	1431.77
	6b	168.3	3.6	221.63	32.95	305	1461.19
	7a	168.8	5	260.86	32.95	305	1961.33
	7b	168.8	5	260.86	32.95	305	1966.23
	8a	168.8	5	260.86	27.46	305	2010.36
	8b	168.8	5	260.86	27.46	305	2010.36
	1	168.8	2.64	298.12	17.95	1830	823.76
	2	168.8	2.64	298.12	34.13	1830	916.92
	3	169.3	2.62	317.74	36.58	1830	757.07
	4	169.3	2.62	317.74	33.54	1830	690.39
	5	168.3	3.6	221.63	26.58	1830	947.32
	6	168.3	3.6	221.63	32.75	1830	1049.31
	7	168.8	5	260.86	32.95	1830	1132.67
	8	168.8	5	260.86	27.46	1830	1166.99
Gunawardena *et al.* [41]	LD1E0	102.67	1.83	234.9	24.20	1226	304
	LD2E0	152.74	1.76	234.9	23.05	1676	570
	LD3E0	203.04	1.93	234.9	24.34	2135	996
	LD4E0	229.81	1.96	234.9	24.22	2594	1228
Gunawardena and Aslani [42]	D1E0	102.9	1.83	234.9	29.9	614	363
	D2E0	152.7	1.95	234.9	30.4	764	740
	D3E0	202.9	1.92	234.9	31.1	917	1249
	D4E0	230.0	1.94	234.9	29.3	1070	1495

3.3.2 Resistance prediction models

In AS/NZS 5100.6 [21] and AS/NZS 2327 [4], the design ultimate section capacity (N_{us}) for a circular CFST column under axial compression is given as:

$$N_{us} = \phi A_s \eta_2 f_y + \phi A_c f_c^{'} \left(1 + \frac{\eta_1 t f_y}{d_0 f_c^{'}}\right) \tag{15}$$

where A_s and A_c = the areas of the steel and concrete sections, respectively; f_y = the nominal yield strength of the steel; f'_c = the characteristic compressive strength of the concrete; ϕ and ϕ_c = the capacity reduction factors for steel and concrete, respectively (with the existing target values given as 0.9 and 0.65); and η_1 and η_2 = the coefficients accounting for the confinement effect (where η_1 represents the concrete strength increase, and η_2 represents the steel strength reduction).

The design ultimate member capacity (N_{uc}) of CFST is given as follows:

$$N_{uc} = \alpha_c N_{us} \leq N_{us} \quad (16)$$

For a stub column defined by L_e/d_o or $L_e/b \leq 4$, in which L_e = the effective length of a column and b = the section width of a rectangular tube, $\alpha_c = 1$.

3.3.3 Analysis results

The forward analysis results for CFSTs with SWTs under axial compression are presented in Figure 3. The analyses are repeated for the target reliability index β for resistance values, for the range of 2.5-4.2 and for different COV values of steel yield strength. The results are more sensitive to the target reliability level as shown by the steeper slopes because there are two materials in CFSTs, and two types of capacity factors are included in the resistance model (i.e. one for steel and one for concrete). When the concrete capacity factor is fixed, it is more difficult to change the reliability level by changing the steel capacity factor alone compared to the case when there is only one capacity factor in the whole design equation. Due to this issue, the increase of the COV of f_y does not significantly reduce the capacity factor values because this uncertainty in f_y affects the steel contribution only. The capacity factor value for steel with a COV of f_y = 7% [39] is estimated as 1.194 when the capacity factor for concrete is fixed at 0.65 (see Table 7), which is greater than 0.90 provided in AS/NZS 5100.6 [21] and AS/NZS 2327 [4]. This shows that the design model has extra safety above the target reliability level, thereby providing a safe design.

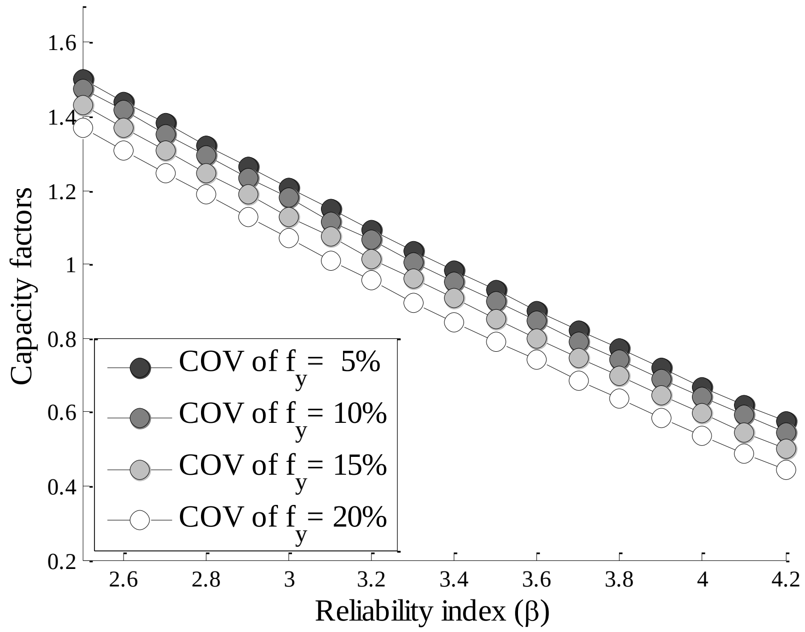

Figure 3. Calibrated capacity factor for steel in CFSTs with spirally welded steel tubes for various COV of the steel yield strength

To investigate whether it was reasonable to include the experimental data with single-sided welding, the data was split into subsets corresponding to the welding method, and the reliability analyses were repeated with a COV for steel yield strength of 7% and β_R = 3.04 (see Table 7). Again, to ensure a fair comparison, the capacity factor ϕ values are calculated by assuming that the fractile $k_{d,m}$ evaluated for the combined data (see Eq. (3)), remains constant in order not to introduce any additional statistical uncertainty. As can be seen from Table 7, the biases are overall similar for all data, double-sided welding, and single-sided welding. However, the modelling error of the single-sided welded specimens is lower than the other specimens, which is considered to be caused by the random nature of the data from the smaller sample size, together with better controlled experiments that were undertaken within the same laboratory. Notwithstanding this, the overestimation of the capacity factor does not much affect the overall capacity factor and, the calculated capacity factor is already well above the target value of 0.90 given in AS/NZS 5100.6 and AS/NZS 2327. It is concluded that the addition of the single-sided welding data reduces the overall statistical error and the associated extra safety margin.

Table 7. Comparison of uncertainties and capacity factor for CFST sections under axial compression

	n (number of tests	$\bar{b}$ (bias)	V_δ (Model error)	V_{Rt} (Parametric error)	V_R (overall error)	ϕ (without extra statistical error)
All	41	1.041	0.142	0.067	0.158	1.194
Double-sided welding	28	1.058	0.164	0.064	0.176	1.046
Single-sided welding	13	1.003	0.121	0.072	0.141	1.310

Next, the inverse analyses have been carried out for CFST columns with SWTs. The results are shown in Figure 4. As the design model for CFST columns has extra safety due to the conservatism that exists within the equation and the capacity factors, the required material test numbers are quite low to achieve the target reliability level for resistance with β_R = 3.04. The shallow slope of the graph shows that the statistical uncertainty due to the number of material tests does not significantly affect the overall reliability of the design, compared to other types of uncertainties such as parametric and modelling uncertainties. The proposed required material test numbers are also tabulated in Table 8 for practical purposes.

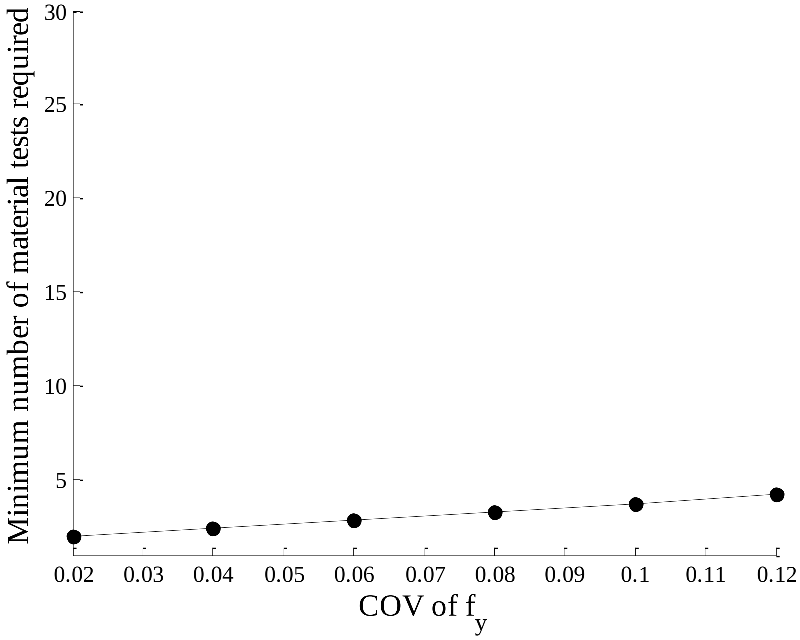

Figure 4. The minimum number of required steel material tests to meet the target reliability level for CFSTs under axial compression

Table 8. Recommended number of steel yield strength tests for CFST with spirally welded tubes under axial compression (based on f_{ym}/f_{yn} =1.35)

COV of f_y	**2%**	**4%**	**6%**	**8%**	**10%**	**12%**
# of tests	2	2	3	3	4	4

3.4 CFST sections under axial compression and uniaxial bending

3.4.1 Experimental database

A total of 16 specimens have been collected for CFSTs with SWTs under axial compression and uniaxial bending due to eccentric loading (see Table 9). The experimental data were collected from the following two publications: Gunawardena *et al.* [41] and Gunawardena and Aslani [42]. The uniaxial bending occurs due to the loads applied with the eccentricity of either at 0.15*D* or 0.40*D*. The SWTs for these tests were manufactured with only a single-sided MIG welding on the outside of the pipe (as opposed to the requirement given in API 5L that the welding should be both on the inside and outside of the pipe). However, from the comparisons of the uncertainties and capacity factors for the different welding methods presented in Section 3.2.3 and 3.3.2, coupled with the lack of experimental data available, it is assumed that the results are insensitive to the welding method.

Table 9. Experimental database for CFSTs under axial compression and uniaxial bending

Article	**Specimen**	***D*** **(mm)**	***t*** **(mm)**	f_y **(MPa)**	f_{cm} **(MPa)**	L_e **(mm)**	M_{max} **(kNm)**	P_{max} **(kN)**	***e*** **(mm)**
Gunawardena *et al.* [41]	LD1E1	102.77	1.77	234.9	24.2	1226	5.8	254	0.15*D*
	LD1E2	102.72	1.83	234.9	24.18	1226	8.1	162	0.40*D*
	LD2E1	152.76	1.78	234.9	23.15	1676	13.7	421	0.15*D*
	LD2E2	152.72	1.81	234.9	23.15	1676	20	269	0.40*D*
	LD3E1	202.99	1.95	234.9	24.27	2135	25.4	601	0.15*D*
	LD3E2	202.15	1.93	234.9	24.34	2135	33.6	351	0.40*D*
	LD4E1	229.69	1.98	234.9	24.25	2594	33.9	723	0.15*D*
	LD4E2	230.14	1.95	234.9	24.25	2594	43.7	395	0.40*D*
Gunawardena and Aslani [42]	D1E1	102.9	1.88	234.9	29	614	4.8	298	0.15*D*
	D1E2	102.9	1.86	234.9	29.9	614	8.9	195	0.40*D*
	D2E1	152.7	2.1	234.9	30.4	764	15.1	557	0.15*D*
	D2E2	153	1.99	234.9	30.4	764	21.7	327	0.40*D*
	D3E1	202.8	1.86	234.9	31.1	917	27.1	778	0.15*D*
	D3E2	202.7	1.85	234.9	31.1	917	38.2	445	0.40*D*
	D4E1	229.5	1.86	234.9	29.7	1070	37.9	939	0.15*D*
	D4E2	229.7	1.93	234.9	29.3	1070	51.1	522	0.40*D*

3.4.2 Resistance prediction models

The axial and moment resistance of a CFST column under axial compression and uniaxial bending is derived from the moment axial load (M-N) interaction curves where $M = N{\cdot}e_{eff}$, considering the effective increase of eccentricity because of the 2nd order effects, and e_{eff} = effective eccentricity. The interaction curve is derived from the stress distributions along the section as shown in Figure 5.

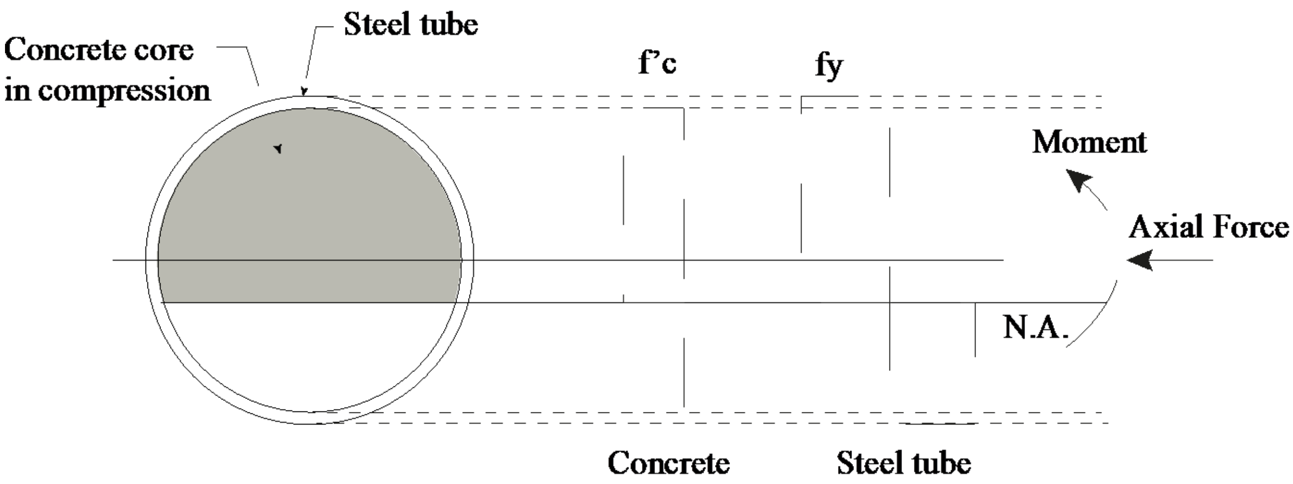

Figure 5 Stress blocks for calculation of a moment axial load capacity interaction curve

3.4.3 Analysis results

The forward analysis results for CFSTs with SWTs under axial compression and uniaxial bending are presented in Figure 6. The forward analysis results show the reliability level similar to the CFSTs under axial compression. The slope is steep, similar to the case of CFSTs under axial compression, which is due to the fixed concrete capacity factor that gives a constant amount of safety. A greater change in a steel capacity factor is needed to change the overall reliability level. The capacity factor value for steel for the COV of f_y = 7% [39] is estimated as 1.081 when that for concrete is fixed at 0.65, which is greater than 0.90 provided in NZS 3404.1 [19], AS 4100 [20], AS/NZS 5100.6 [21] and AS/NZS 2327 [4]. This means that the design model provides extra safety against the target reliability level.

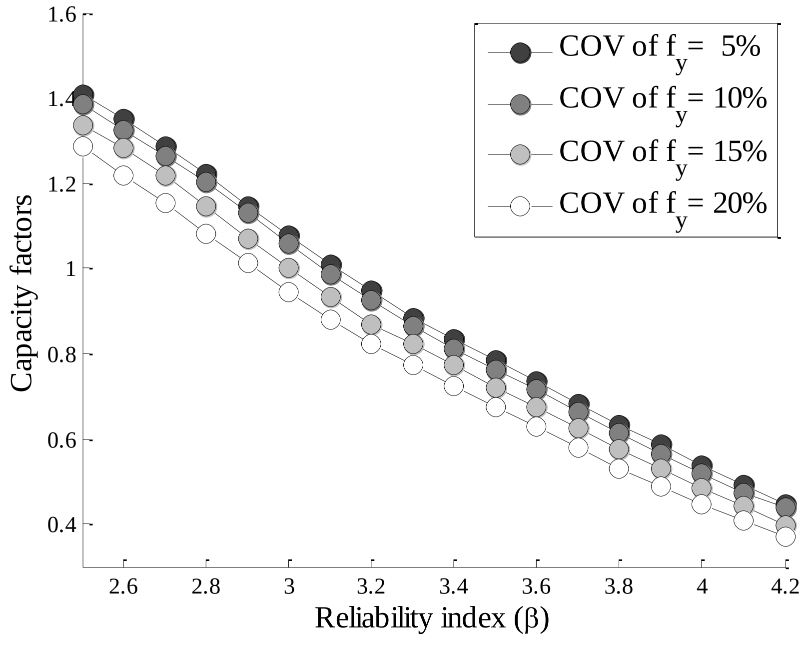

Figure 6. Calibrated capacity factor for steel in CFSTs with spirally welded steel tubes under axial compression and uniaxial bending for various COV of the steel yield strength

The inverse analysis results are shown in Figure 7. The range of COVs of steel yield strength from 4% and 12 % have been considered. The trend is similar to CFST under axial compression, and it requires a slightly greater number of material tests compared to that for CFST under axial compression. It is also shown that the effect of COV of f_y is similar to that of CFST under axial compression. The proposed numbers of required material tests are also tabulated in Table 10 for practical purposes.

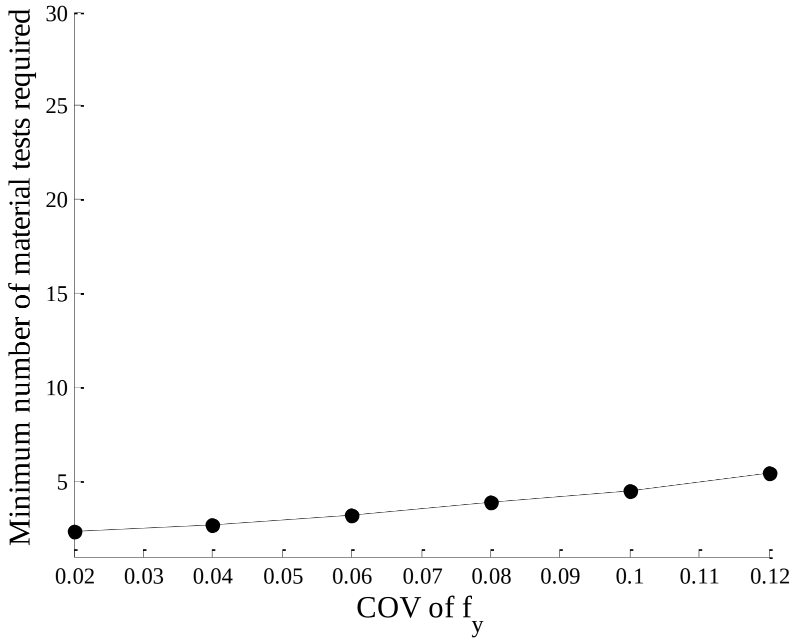

Figure 7. The minimum number of required steel material tests to meet the target reliability level for CFSTs under axial compression and uniaxial bending

Table 10. Recommended number of steel yield strength tests for CFST with spirally welded tubes under eccentric loading (based on f_{ym}/f_{yn} =1.35)

COV of f_y	**2%**	**4%**	**6%**	**8%**	**10%**	**12%**
# of tests	2	3	3	4	4	5

4. Conclusion

This study carried out the capacity factor calibration for the design of steel and steel-concrete composite columns with helical seam pipe, which are also known as spirally welded steel tubes (SWTs). The analysis included the forward analysis to calibrate the capacity factor for steel, and the inverse reliability analysis to estimate the required number of material tests to meet the target reliability level for given capacity factors. This latter information will be of particular importance to designers when the nominal yield strength of a product is unknown, as may occur when reclaimed steel is reused. The following three types of members with circular cross-sections were considered: (i) columns under axial compression, (ii) CFST columns under axial compression, and (iii) CFST columns under combined axial compression and uniaxial bending due to eccentric loading.

For the SWT columns, 11 data were collected and used to estimate the modelling error. The design equation gave the reliability just close to the target reliability index for resistance. This was because of the effect of the uncertainty in thickness that directly affected the reliability and also the statistical error due to the number of tests far from infinity. The inverse analysis results showed that the required number of steel tensile tests quickly increased as the measured COV of f_y from the limited number of material tests increased. For the CFST columns with SWTs under axial compression, 41 data were collected and used to estimate the modelling error. The reliability achieved by the resistance model was greater than the target reliability level showing the conservatism of the model. The inverse analysis results showed that the required number of material tests did not quickly increase according to the measured COV of f_y from the limited number of material tests, as the high reliability level was already achieved by the design model itself. For the CFST columns with SWTs under axial compression and uniaxial bending, 16 data were collected and used to estimate the modelling error. The reliability achieved by the resistance model was similar to that of CFSTs under pure axial compression and still higher than the target reliability level, showing the conservatism of the model. The inverse analysis trend was similar to that of CFSTs under pure axial compression and it required a slightly increased number of material tests to meet the target reliability level.

Some of the test specimens considered in this study consisted of SWTs manufactured with only single-sided MIG welding on the outside of the pipe (as opposed to the requirement given in API 5L that the welding should be both on the inside and outside of the pipe). From splitting the data into subsets corresponding to the welding method, reliability analyses were repeated for SWTs and CFSTs under axial compression. Whilst the test data is limited, the results suggest that the inclusion of test specimens with single-sided welding leads to more conservative design resistances. However, the calculated capacity factors generally still achieve similar or higher target values than provided by the current Australian and New Zealand design standards. Notwithstanding this, the authors were unable to find any test data for double-sided welded CFSTs subjected to combined axial compression and uniaxial bending. As a consequence of this, further tests on members with different fabrication methods are encouraged.

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