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Design resistance evaluation for steel and steel-concrete composite members

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ABSTRACT

This study evaluates the performance of the design equations given in the Australian/New Zealand bridge and steel structures design standards AS 5100.6, AS 4100 and NZS 3404.1 based on reliability analysis. For this evaluation, the following two methods were utilised: (i) a capacity factor calibration method to meet the target reliability level when there are a limited number of steel yield strength tests; and (ii) an inverse reliability analysis method to calculate the required minimum number of steel yield strength tests to achieve the target reliability level when using capacity factors provided in the design standards. The methods were applied to steel and composite members including I-beams, hollow section columns, CFST columns, and composite beams. To ensure the adoptability of imported steel for these members, structural steel that conforms to European, Korean, Japanese, American, Chinese and Australasian manufacturing standards were considered in the analyses. The results showed that, for an infinite range of manufacturing data, the capacity factors were insensitive to the different manufacturing tolerances. Furthermore, when a limited number of mechanical tests were available, a much larger number of results were needed to achieve the target capacity factor for composite members in comparison with non-composite members. Finally, when considering hollow sections used as columns, the current design equations were unable to deliver the target reliability levels for any of the manufacturing standards used internationally.

List of symbols

α_R	the First Order Reliability Method (FORM) sensitivity factor for resistance
β	the reliability index
$\sigma_{\ln f_y}$	the standard deviation of the steel yield strength with the lognormal distribution
σ_r	the sample standard deviation of resistance
β_t	the target reliability index
γ_M	the partial safety factor
δ	the error of the unbiased resistance prediction
δ_i	the prediction error for each test result
ν	the degree of freedom
Φ	the cumulative distribution function of the standardised normal distribution
ϕ	the capacity factor
b	the section width of a rectangular section
d_o	the diameter of a circular section
f_{cm}	the mean measured compressive cylinder strength of concrete
f_{cu}	the mean measured compressive cube strength of concrete
f_y	the yield strength of steel
f_{yk}	the characteristic yield strength of steel
f_{ym}	the mean measured value of the yield strength
k_d	the fractile factor of the *t*-distribution corresponding to the number of test data and the target reliability index β at the 75% confidence level
$k_{d,\,Rt}$	the fractile factor corresponding to the target reliability index at the 75% confidence level, determined for a number of finite observations from a *t*-distribution
k_n	the design fractile factor for a specified probability
L_e	the effective length of a column
N	the number of experimental data
n	the size of the population
P_f	the probability of failure
R	the resistance
R_d	the design resistance
R_{ei}	the *i*-th experimental result
R_k	the lower characteristic resistance

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R_m the sample mean value
R_n the nominal resistance
R_{ti} the theoretical mean resistance prediction for the i-th specimen
$t_\beta(\nu)$ The fractile of the t-distribution for the probability corresponding to the target reliability index and the number of degrees of freedom
t_p the p fractile of the known t-distribution
u_p the p fractile of the standardised normal distribution
V_R the coefficient of variation of resistance
V_r the sample coefficient of variation of resistance
V_{Rt} the COV of parametric uncertainty
$V_{Rt,\ finite}$ the COV of parametric uncertainty for the parameters with a finite number of observations
$V_{Rt,\ inf}$ the COV of parametric uncertainty for the parameters with an infinite number of observations
V_δ the COV of modelling uncertainty
$\mathbf{x}$ input parameters
$\mathbf{x}_i$ parameters used in the i-th specimen

1. Introduction

1.1. Background

Structural steel is an international commodity that is commonly shipped thousands of miles from where it is produced to wherever there is a market. The members of the industry association worldsteel represent around 85% of world crude steel production. Fig. 1(a) presents the annual crude steel production data from worldsteel members in Australia, China, Japan, UK and USA between 1980 and 2016 [1]. As can be seen from Fig. 1(a), whilst Australia, Japan, UK and USA have broadly maintained their output, steel production in China has increased remarkably over this 36-year period. As can be seen from Fig. 1(b), China accounted for 50% of world steel production in 2016, amounting to an output of 808.4 Mt. It is therefore important for designers in the Asia-Pacific region to be able to gain access to the vast supply of Chinese made steel.

For an Asia-Pacific country who wishes to adopt the Eurocodes as their national design standard, an immediate problem is that the normative references in Eurocode 3 [2] and 4 [3] list harmonised European product and execution standards (hENs). Two options exist for designers in these countries: source steel products from mills that manufacture to hENs; or deem steel products manufactured to other standards to be equivalent in performance to hENs. Whilst the former option may be considered attractive, sourcing can be problematical and CE Marking is not mandatory in countries outside the European Economic Area where the Construction Products Regulation [4] is enforced. As a consequence of this, the latter option of accepting equivalent steel products is commonly used.

In Singapore and Hong Kong, two guides have been developed to enable designers to use alternative steel products that are deemed to have equivalent performance to hENs [5,6]. Provided that an alternative steel product is manufactured to a national standard recognized by these two guides, the steel mill is required to supply: a factory production control (FPC) certificate issued by a notified body; and a test certificate for each batch of steel product delivered to the project issued by an independent third-party inspection agency (the latter is consistent with the level of traceability required by EN 1090-2 [7] for grade S355JR and S355J0 steel in EXC2, EXC3 and EXC4 structures). Depending on the alternative steel product satisfying certain requirements [8], three product classes are defined with different partial factor values, viz. Class 1 with $\gamma_{M0} = 1.0$ (i.e. deemed to be directly equivalent to hENs, so the recommended value in Eurocode 3 and 4 is used); Class 2 with $\gamma_{M0} = 1.1$; and Class 3 with $f_{yd} = 170$ MPa for steel thicknesses not >16 mm (an identical value is given for unidentified steel in Australasia).

Whilst there are no immediate plans to adopt the Eurocodes in Australia and New Zealand, there is beginning to be greater harmonization through joint Australian/New Zealand (AS/NZS) design standards. However, in a similar way to Singapore and Hong Kong, due to a limited range of AS/NZS steel products, steel produced to British (BS and BS EN) and Japanese (JIS) standards have been used in New Zealand design for the last 35-years [9]:

Following the decision to revise the Australian steel and composite bridge design standard AS 5100.6 [10] as a joint AS/NZS standard, concerns were raised by the Committee responsible that the different cross-sectional tolerances of the structural steel products recognized in the New Zealand steel structures design standard NZS 3404.1: 1997 [11] may cause an erosion of safety margins. In response to these concerns, reliability analyses were undertaken by Kang et al. [9] for non-composite beams in bending which, unlike the Singapore [5] and Hong Kong [6] guide, directly evaluated the required capacity reduction factor ϕ (N.B. $\phi \equiv 1/\gamma_{M0}$). This work demonstrated that, for a coefficient of variation of the yield strength $V_{fy} = 10\%$ (which is consistent with the value used in the original Australian standard calibration [12]), the calculated capacity factors were insensitive to cross-sectional geometrical tolerances. More recently, the reliability analyses were extended by Uy et al. [13] to include structural steel complying with GB/T 11263 [14]; again, it was found that the capacity factors were insensitive to different tolerances. However, it was shown that there was a direct relationship between the coefficient of variation and the capacity factors where for $V_{fy} = 5\%$, 10%, 15% and 20% resulted in capacity factors of $\phi =$ 1.00, 0.94, 0.87 and 0.78, respectively, for a reliability index $\beta = 3.04$.

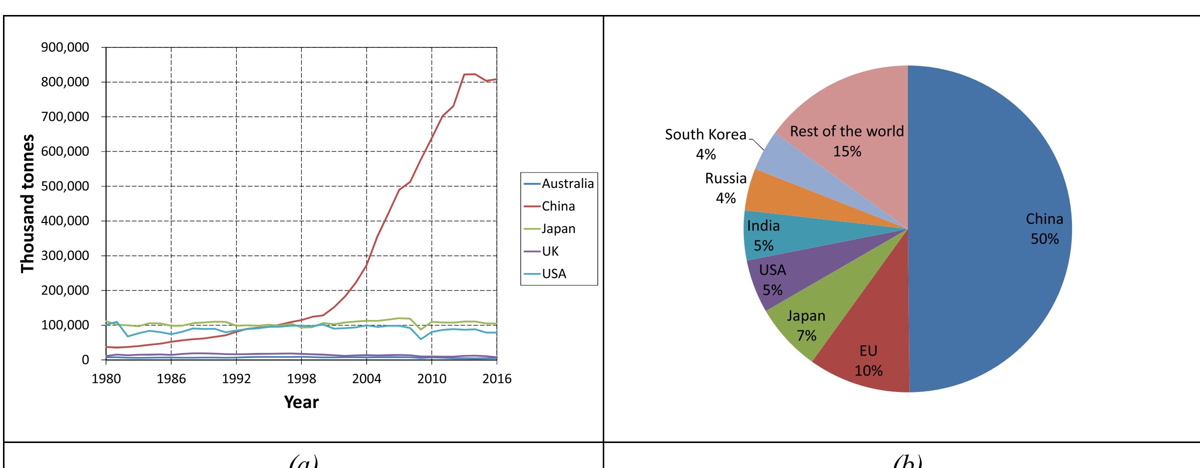


Fig. 1. (a) annual crude steel production for Australia, China, Japan, UK and USA between 1980 and 2016 (b) percentage of world steel production by country for 2016.

In practice, the coefficient of variation for the yield strength is established from a particular manufacturer when they decide to produce a particular grade of steel and section size. This is established through what is known as Initial Type Testing (ITT), where tests are undertaken for each grade designation on the highest strength that the manufacturer places on the market; for AS/NZS steel sections, the number of tests consists of 6 tests for each heat for a minimum of 5 heats (i.e. 30 tests in total). To ensure that the performance characteristics established during the ITT is maintained, FPC is undertaken, where a smaller number of tests are undertaken at a particular frequency (for AS/NZS steel sections, this consists of one test for each batch of steel not >50 t, or 2 tests for batches > 50 t). The coefficient of variation of the yield strength that is used as a basis for the design standards is evaluated from ITT or, when a particular grade of steel has been available for a length of time, tests from FPC undertaken over a particular period (12-months). Either the number of tests undertaken for ITT requirements or tests over a production period for FPC requirements are often deemed to represent an infinite amount of data and have been published through, for example, the JCSS Model Code [15].

In New Zealand, structural steel products that are not listed in NZS 3404.1: 1997 [11] are more frequently being encountered in design. In this case, the coefficient of variation for the yield strength from a particular manufacturer is unknown due to ITT or FPC data being unavailable, and there are only a small number of test results reported through mill certificates (typically, 1 to 2-tests depending on the size of the batch of steel, for FPC purposes). It is therefore required that the performance of the design equations given in such steel and composite standards are evaluated when there is a smaller amount of test data available, which creates some uncertainty in the results. Guidance on estimating properties of construction materials from a limited number of tests is given in ISO 12491 [16].

1.2. Research objectives

In this study, the following two-step analysis was performed for steel and composite members including I-beams, hollow section columns, CFST columns, and composite beams: (i) capacity factor ϕ calibration to meet the target reliability level for infinite material tests; and (ii) required number evaluation for material strength tests to meet the target reliability level when using the capacity factors provided in the design standards. For this analysis, the adoptability of imported steel for these members was also checked by considering various international steel manufacturing tolerances. Although these analyses were carried out mainly for the Australian/New Zealand standards, the results have important ramifications for international codes of practice in constructional steel.

2. Target reliability index and design resistance

To calibrate capacity factors, the acceptable level of life-cycle consequences of structural failure needs to be determined. This acceptable level is often formulated in terms of a target reliability index. In this section, the definition of the target reliability index is provided together with the suggested value in AS 5104: 2005 [17]/ISO 2394:1998 [18].

According to AS/NZS 1170.0 [19], the provisions given are based on the philosophy and principles set out in AS 5104:2005 [17]/ISO 2394:1998 [18] entitled *General principles on reliability for structures*. AS 5104:2005 [17]/ISO 2394:1998 [18] provide a common basis for defining design rules relevant to the construction and use of a wide majority of buildings, bridges and civil engineering works, whatever the nature or combination of the materials used; they include methods for establishing and calibrating reliability based limit states design standards. In probability-based design, the probability of failure P_f is the basic reliability measure that is used in AS 5104:2005 [17]/ ISO 2394:1998 [18]. An alternative measure is the reliability index and is related to the probability of failure P_f through the following equation:

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

where Φ is the cumulative distribution function of the standardised normal distribution and β is the reliability index.

The target reliability index is related to the expected social and economic consequences of a design failure. According to AS 5104:2005 [17]/ISO 2394:1998 [18], the suggested reliability index for ultimate limit state design is $\beta = 3.8$, which corresponds to the case when the consequence of failure is great (the highest level) and the relative costs of safety measures are moderate.

Design values of resistances are defined such that the probability of having a more unfavourable value is as follows:

$$P(R \le R_d) = \Phi(-\alpha_R\beta) \tag{2}$$

where α_R is the First Order Reliability Method (FORM) sensitivity factor for resistance, R is the resistance, and R_d is the design resistance.

For a dominating resistance parameter, AS 5104:2005 [17]/ISO 2394:1998 [18] recommend $\alpha_R = 0.8$. Therefore, the design value for resistance corresponds to the product $\alpha_R\beta = 0.8 \times 3.8 = 3.04$ (equivalent to a probability of the actual resistance falling below the design resistance of 1 in 845 = 0.0012). The remaining safety is achieved in the specification of actions.

For design assisted by testing, the design or characteristic value is based on a prediction model, which is a procedure for estimating a population's fractile from an available sample of limited size n. If the standard deviation of the population is known, the fractile factor is calculated from the following equation:

$$k_n = -u_p(1/n+1)^{1/2} \quad \text{(when the standard deviation of the population is known)} \tag{3}$$

where u_p is the p fractile of the standardised normal distribution and n is the size of the population. Alternatively, if the standard deviation of the population is unknown, the fractile factor is calculated from the following equation:

$$k_n = -t_p(1/n+1)^{1/2} \quad \text{(when the standard deviation of the population is unknown)} \tag{4}$$

where t_p is the p fractile of the known Student t-distribution (with $v = n - 1$ degrees of freedom) and n is the size of the population.

The design resistance R_d can be derived in the following two ways: the first way is by the direct determination from the following equation:

$$R_d = (R_m - k_n\sigma_r) = R_m(1 - k_nV_r) \tag{5}$$

where R_m is the sample mean value, k_n is the design fractile factor from Eq. (3) or (4) (for a probability of 0.0012, $k_n = 3.04$ when $n = \infty$), σ_r is the sample standard deviation and V_r is the sample coefficient of variation [N.B. coefficient of variation = (standard deviation) / (mean value)]. The second way is by assessing a characteristic value, which is then divided by a partial factor γ_M (or multiplied by the capacity factor ϕ) as follows:

$$R_d = \frac{R_k}{\gamma_M} = \frac{1}{\gamma_M}(R_m - k_n\sigma_r) = \frac{R_m}{\gamma_M}(1 - k_nV_r) = R_m\phi(1 - k_nV_r) \tag{6}$$

where R_k is the lower characteristic resistance, k_n is the characteristic fractile factor from Eq. (3) or (4) (for a probability of 0.05, $k_n = 1{,}64$ when $n = \infty$), γ_M is the partial factor which accounts for uncertainties of the basic variables contained within the equation for the design model, i.e. material and geometrical uncertainties, as well as uncertainties in the theoretical resistance function when compared with experimental values from tests ($\gamma_M = R_k / R_d$) and ϕ is the capacity factor ($\phi = R_d / R_k$).

Table 1
Manufacturing tolerances used for reliability analysis for I beams.

Parameter	EN10034: 1993 KS D 3502: 2007		JIS G 3192: 2005 JIS A 5526: 2005	ASTM A 6/A 6M - 07		AS/NZS 3679.1 AS/NZS 3679.2	GB/T 11263:2005	
Depth (h) (mm)	−2		-2	−3		−3	−2	
Width (b) (mm)	$b \le 110$	−1	−2.5	−5		−5	$100 < b \le 200$	−2.5
	$110 < b \le 210$	−2					$200 < b$	−3
Web thickness (t_w) (mm)	$t_w < 7$	−0.7	−0.7	$t_w < 7$	−0.7	−0.7	−0.7	
	$7 \le t_w < 10$	−1		$7 \le t_w < 10$	−1			
Flange thickness (t_f) (mm)	−1		−1	−1		−1	−1	

3. Reliability analysis

3.1. Capacity factor calibration

In this study, the capacity factor calibration was conducted by modifying the statistical method given in EN 1990 Annex D.8 [20], which provides procedures for calibrating a single capacity reduction factor. The modification was to consider a finite number of material test data. This method calibrates a capacity reduction factor as the ratio of the design resistance to the nominal resistance. The calculation steps of this method are provided in EN 1990 Annex D.8 [20].

Our rationales for selecting this method compared to other methods are as follows [21]: (i) this method assumes that the resistance model follows the lognormal distribution, which always has a positive value and thus corresponds to reality [22]. The target reliability index should be selected under this assumption. (ii) It calibrates resistance factors separately from load factors. (iii) The modelling error is rigorously estimated based on experimental data.

In the proposed method, the capacity factor (ϕ) for a resistance model is estimated as follows:

$$\phi = \frac{R_d}{R_n} \tag{7}$$

where R_d denotes the design resistance that meets the target reliability level, and R_n denotes the nominal resistance. The estimation of R_d requires the following calculation procedures. Let $g_R(\mathbf{x})$ be a theoretical model and $\mathbf{x}$ is a vector of mean-measured input parameters. The constant bias correction-term for this function can be obtained as follows:

$$\bar{b} = \frac{1}{N}\sum_{i=1}^{N}\left(\frac{R_{ei}}{R_{ti}}\right) \tag{8}$$

where N is the number of experimental data, R_{ei} is the i-th experimental result, and R_{ti} is the theoretical mean resistance prediction for the i-th specimen using $g_R(\mathbf{x}_i)$ where $\mathbf{x}_i$ are parameters used in the i-th specimen. Using this bias correction term, the unbiased resistance prediction R is calculated as follows:

$$R = \bar{b} g_R(\mathbf{x})\delta \tag{9}$$

where δ is the error of the unbiased resistance prediction. The prediction error for each test result, δ_i, can be statistically estimated as follows:

$$\delta_i = \frac{R_{ei}}{\bar{b} R_{ti}} \tag{10}$$

Assuming that R in Eq. (9) follows a lognormal distribution, the COV of R can be estimated as follows:

$$V_R \cong \sqrt{\left(V_\delta^2 + V_{Rt}^2\right)} \tag{11}$$

where V_δ denotes the COV of modelling uncertainty estimated from Eq. (10), and V_{Rt} denotes the COV of parametric uncertainty that can be estimated by using the Monte Carlo simulations or the first-order approximation of moments [23] putting perturbation into the parameters in the theoretical prediction. Under the lognormal assumption, the standard deviation of ln$R$ ($\sigma_{\ln R}$) is estimated as follows:

$$\sigma_{\ln R} = \sqrt{\ln\left(1 + V_R^2\right)} \tag{12}$$

This standard deviation is used to estimate the design resistance (R_d) as follows:

$$R_d = \bar{b} g_R(\mathbf{x}) \exp\left(-k\sigma_{\ln R} - 0.5\sigma^2_{\ln R}\right) \tag{13}$$

where

$$k = \frac{\left(k_d V_\delta^2 + \beta V_{Rt}^2\right)}{V_R^2} \tag{14}$$

and k_d denotes the fractile factor of the t-distribution corresponding to the number of test data and the target reliability index β at the 75% confidence level. It is calculated as follows when $\sigma_{\ln R}$ is unknown:

$$k_d = t_\beta(\nu) \times (1 + 1/n)^{0.5} \tag{15}$$

where $t_\beta(\nu)$ = fractile of the t-distribution for the probability corresponding to the target reliability index β and the number of degrees of freedom $\nu = n - 1$. This fractile factor is used to consider the uncertainty caused by the number of test data that is usually far from the ideal value, infinity. According to ISO 2394 [18], β can be empirically estimated as $\alpha_R \times \beta_t$, where β_t = the target reliability index considering both resistance and load effects and α_R = the FORM sensitivity factor for resistance, which is taken to be 0.8. Note that β is multiplied to V_{Rt} instead of k_d or an equivalent value in Eq. (14) because it is assumed that the distribution and the partial descriptors of random parameters are fully known. However, if there are any parameters that are defined by statistical distributions based on a finite number of observations, Eqs. (11) and (14) are respectively modified as follows:

$$V_R \cong \sqrt{\left(V_\delta^2 + V_{Rt,\,\mathrm{inf}}^2 + V_{Rt,finite}^2\right)} \tag{16}$$

and

$$k = \frac{\left(k_d V_\delta^2 + \beta V_{Rt,\,\mathrm{inf}}^2 + k_d V_{Rt,finite}^2\right)}{V_R^2} \tag{17}$$

where $V_{Rt,\,inf}$ = the COV of parametric uncertainty for the parameters with an infinite number of observations, and $V_{Rt,\,finite}$ = the COV of parametric uncertainty for the parameters with a finite number of observations. $k_{d,\,Rt}$ = the fractile factor corresponding to the target reliability index β at the 75% confidence level, determined for a number of finite observations from a t-distribution using Eq. (15).

Next, R_n is calculated by plugging in the nominal values of the parameters $\mathbf{x}_n$ into the resistance prediction model, i.e. $g_R(\mathbf{x}_n)$. When the nominal values of the parameters are not available from the test database, the characteristic values based on the 5% fractile value of 1.64 can be alternatively used to replace the nominal input parameters [18]. For example, the characteristic yield strength of steel (f_{yk}) is defined as

$$f_{yk} = f_{ym} \exp\left(-1.64\sigma_{\ln f_y} - 0.5\sigma^2_{\ln f_y}\right) \tag{18}$$

where f_{ym} is the mean measured value of the yield strength and $\sigma_{\ln f_y}$ is the standard deviation of f_y with the lognormal distribution. The characteristic values for the concrete compressive strength can also be calculated in the same way. In this study, not-reported nominal values for the yield strength of steel and the compressive strength of concrete were estimated in this way, and the nominal values of geometric parameters were assumed to be equal to the mean measured values.

3.2. An inverse analysis framework to find the required minimum number of material test results

This study proposes an approach to find the required minimum number of material test results by taking the inverse of the capacity factor calibration procedure introduced in the previous section. When the capacity factor is fixed or given in a structural design code, for a certain

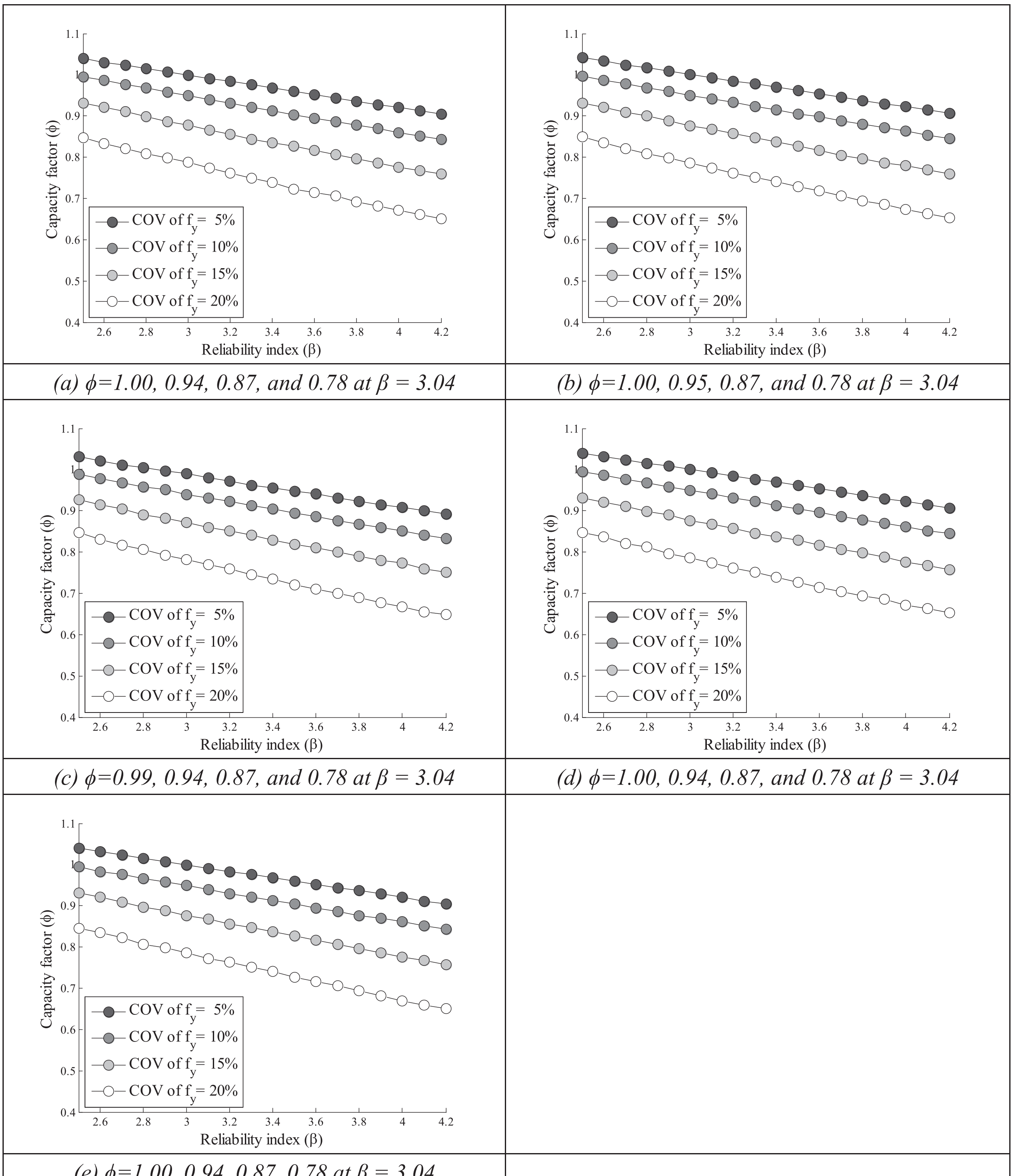


Fig. 2. Capacity factors versus reliability index for compact I sections using products complying with manufacturing tolerances given in (a) EN 10034/KS D 3502 (b) JIS G 3192/JIS A 5526 (c) ASTM A 6/A6M (d) ASNZS 3679.1-ASNZS 3679.2 (e) GB/T 11263:2005.

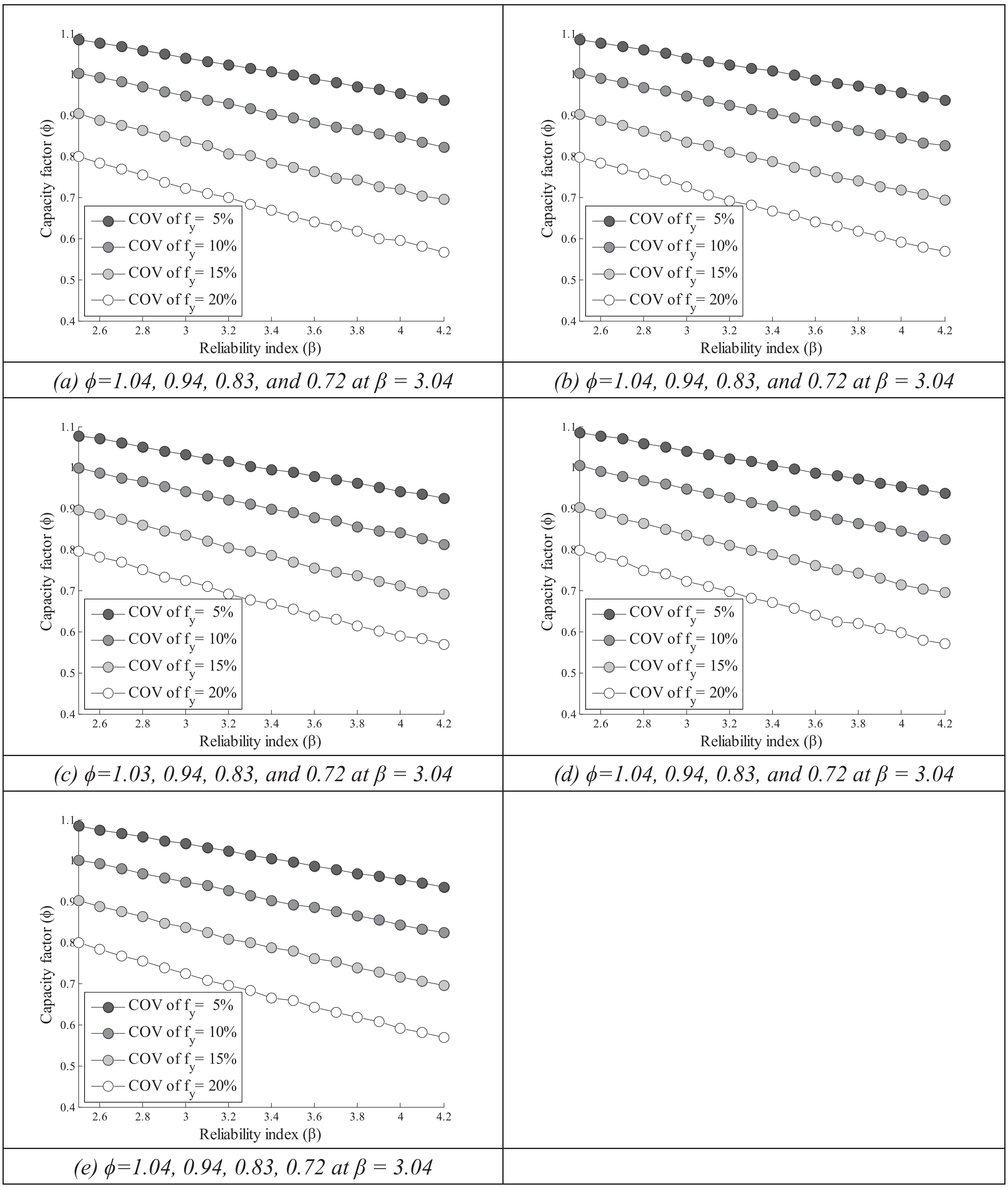
target reliability index, we can inversely calculate the required number of material tests represented in terms of the degree of freedom ν in Eq. (15). The degree of freedom (ν) and the number of test samples (n) have the relation $\nu = n - 1$. This inverse approach is useful when we do not have knowledge of the material property of one of the input parameters such as the strength of a material and therefore need to use a statistical value from new material tests. When the capacity factor (ϕ) is fixed or given, Eq. (7) becomes the following form:

$$\phi R_n = R_d(\nu, \mathbf{x}) \tag{19}$$

where $\nu =$ the degree of freedom of the parameter whose value is obtained from new tests and included in Eq. (15). $\mathbf{x} =$ input parameters including the parameter whose value is obtained from the new tests. In this equation, ν is the only unknown term in this equation, and it can be solved numerically. In this study, the Active-Set Optimisation algorithm [24] and pattern search [25] were adaptively used as solvers.

4. Applications

The proposed methods were applied to calibrate the capacity factors in design equations for various structural members and to estimate the



(a) ϕ=1.04, 0.94, 0.83, and 0.72 at $\beta = 3.04$

(b) ϕ=1.04, 0.94, 0.83, and 0.72 at $\beta = 3.04$

(c) ϕ=1.03, 0.94, 0.83, and 0.72 at $\beta = 3.04$

(d) ϕ=1.04, 0.94, 0.83, and 0.72 at $\beta = 3.04$

(e) ϕ=1.04, 0.94, 0.83, 0.72 at $\beta = 3.04$

Fig. 3. Capacity factors versus reliability index for not-compact I sections using products complying with manufacturing tolerances given in (a) EN 10034/KS D 3502 (b) JIS G 3192/JIS A 5526 (c) ASTM A 6/A6M (d) ASNZS 3679.1-ASNZS 3679.2 (e) GB/T 11263:2005.

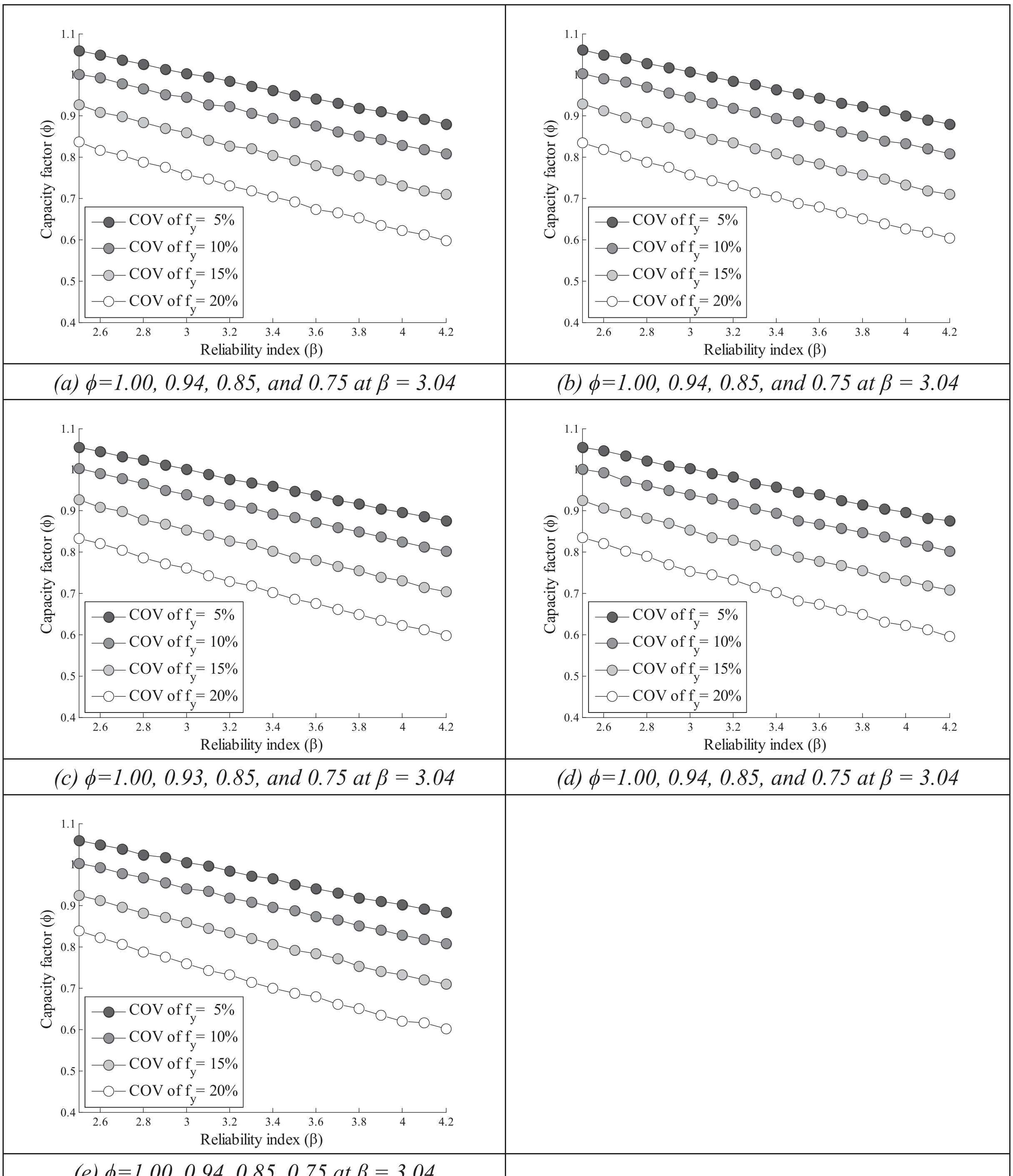
minimum number of strength tests required to meet the safety level made by current structural design standards. The analyses were conducted for the following four structural member types: I section steel beams, hollow section columns, concrete-filled steel tube composite columns and composite beams.

4.1. I section beams

4.1.1. Manufacturing tolerance

The calibration of capacity factors (forward analysis) and the estimation of the required number of material tests (inverse analysis) proposed in Section 3 were carried out for I section beams based on the experimental data provided by Byfield and Nethercot [26] together with Bureau [27]; the full details of the experimental data can be found from [26,27] or [9].

The analysis was repeatedly carried out for five different types of manufacturing tolerances given in EN 10034 [28]/KS D 3502 [29], JIS G 3192 [30]/JIS A 5526 [31], ASTM A 6/A6M [32], AS/NZS 3679.1 [33], AS/NZS 3679.2 [34], and GB/T 11263 [14], which resulted in different variation in geometric parameters. It was assumed that the tolerance values are equal to the standard deviations of the design parameters. The tolerance values used in this analysis are provided in Table 1. It is



(a) ϕ=1.00, 0.94, 0.85, and 0.75 at β = 3.04

(b) ϕ=1.00, 0.94, 0.85, and 0.75 at β = 3.04

(c) ϕ=1.00, 0.93, 0.85, and 0.75 at β = 3.04

(d) ϕ=1.00, 0.94, 0.85, and 0.75 at β = 3.04

(e) ϕ=1.00, 0.94, 0.85, 0.75 at β = 3.04

Fig. 4. Capacity factors versus reliability index for non-compact I sections using products complying with manufacturing tolerances given in (a) EN 10034/KS D 3502 (b) JIS G 3192/JIS A 5526 (c) ASTM A 6/A6M (d) ASNZS 3679.1ASNZS 3679.2 (e) GB/T 11263:2005.

also conservatively assumed that all the negative geometrical tolerances can occur together and are not limited by the mass tolerance requirements given by the different manufacturing standards.

4.1.2. Capacity factor calibration for compact, not-compact, and non-compact sections

The capacity factor for the design of I-beams was calibrated for (i) compact, (ii) not-compact, and (iii) non-compact sections. For the modelling uncertainty estimation, experimental data collected from literature were used as follows: for compact sections, experimental data provided by Byfield and Nethercot [26] were used, which include 32 member failure test results on laterally restrained I-beams, which are all compact based on the plate element slenderness limits given in AS 5100.6 [10], AS 4100 [35] and NZS 3404.1 [11]. Two section types 203 × 102 × 23UB and 152 × 152 × 30UC with a steel grade FE430A are included in the data. For not-compact sections, the experimental data provided by Bureau [27] were used, which contains 24 member failure test results of I-beams with two section types HEA 200 and HEA 200 A. All the sections in this database are not compact as their plate element slenderness lies between the plastic and elastic slenderness limits based on AS 5100.6 [10]. In the calibration, the yield moment capacity of these sections was calculated based on AS 5100.6 [10] and compared to the experimental elastic moment capacity. For non-compact sections, the same test data provided by Bureau [27] were

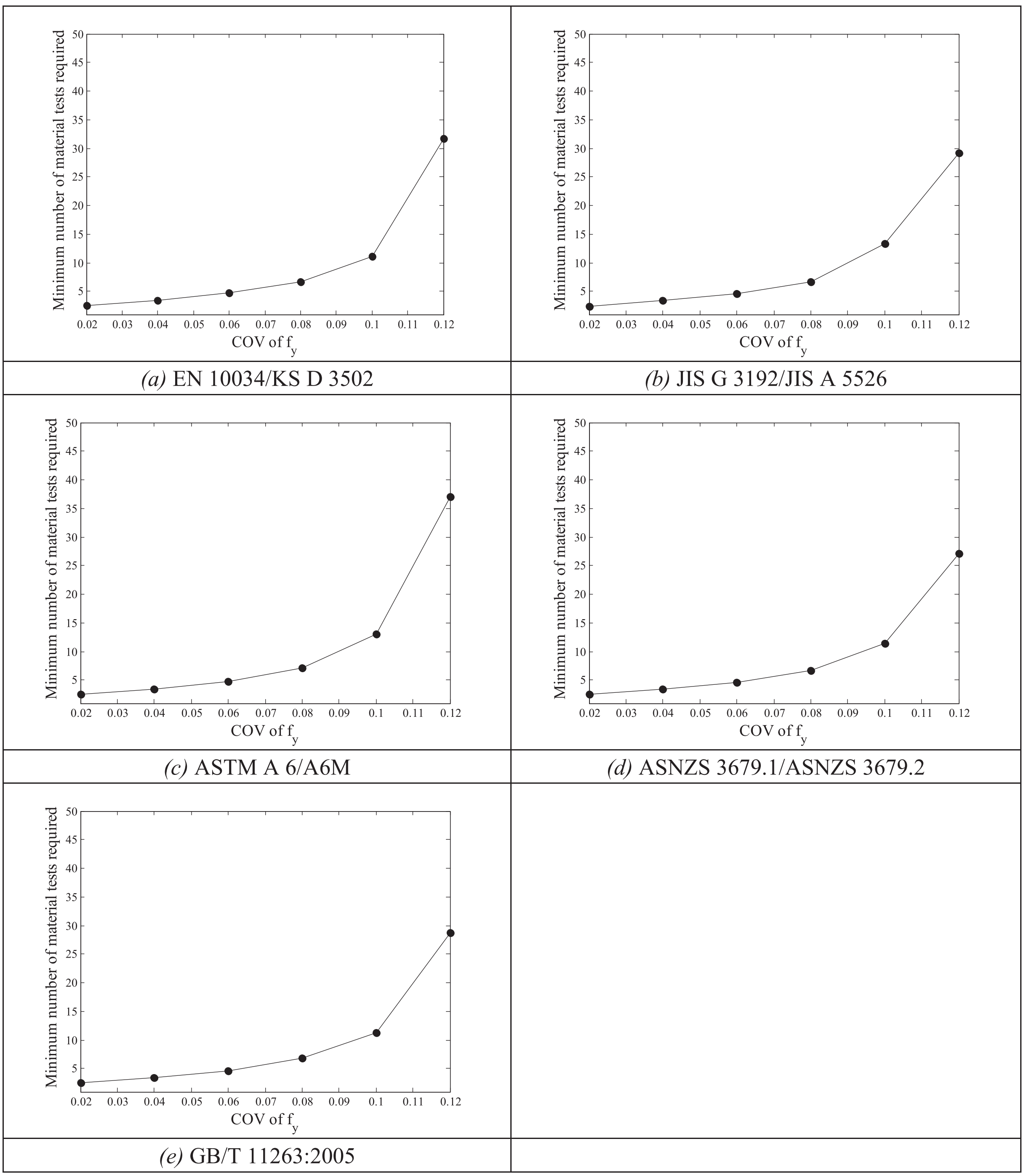


Fig. 5. The minimum number of material tests required to achieve $\phi = 0.90$ for compact I sections using products complying with manufacturing tolerances given in international design codes.

used, but the section moment capacity of non-compact sections was calculated based on AS 4100 [35] and NZS 3404.1 [11], which suggest a linear interpolation between the plastic and elastic section capacities. In the calibration, the calculated section capacity is compared to the experimental ultimate moment capacity.

For the calibration, the following assumptions were made: (1) the mean measured values of fillet radius were not reported in the database, and the nominal values were used instead; (2) member capacity was not considered. This calibration is an extension of the work done by Kang et al. [9].

The calibration results are provided in Figs. 2, 3, and 4, for compact, not-compact, and non-compact sections, respectively, where each sub figure corresponds to each of the manufacturing tolerances in EN 10034 [28]/KS D 3502 [29], JIS G 3192 [30]/JIS A 5526 [31], ASTM A 6/A6M [32], AS/NZS 3679.1 [33], AS/NZS 3679.2 [34], and GB/T 11263 [14]. The results are plotted for a range of target reliability values between 2.5 and 4.2. In the caption of each figure, four capacity factor values for the target reliability index ($\beta = 3.04$) are provided, each of which corresponds to 5%, 10%, 15%, and 20% COV of f_y, respectively. These results are part of this study, and they were also reported in [12] with a brief description as a preliminary study. The full details of these results are provided in this study. It is seen from all figures that, for the target reliability 3.04, the calibration results are almost identical within round-off error regardless of different manufacturing tolerances.

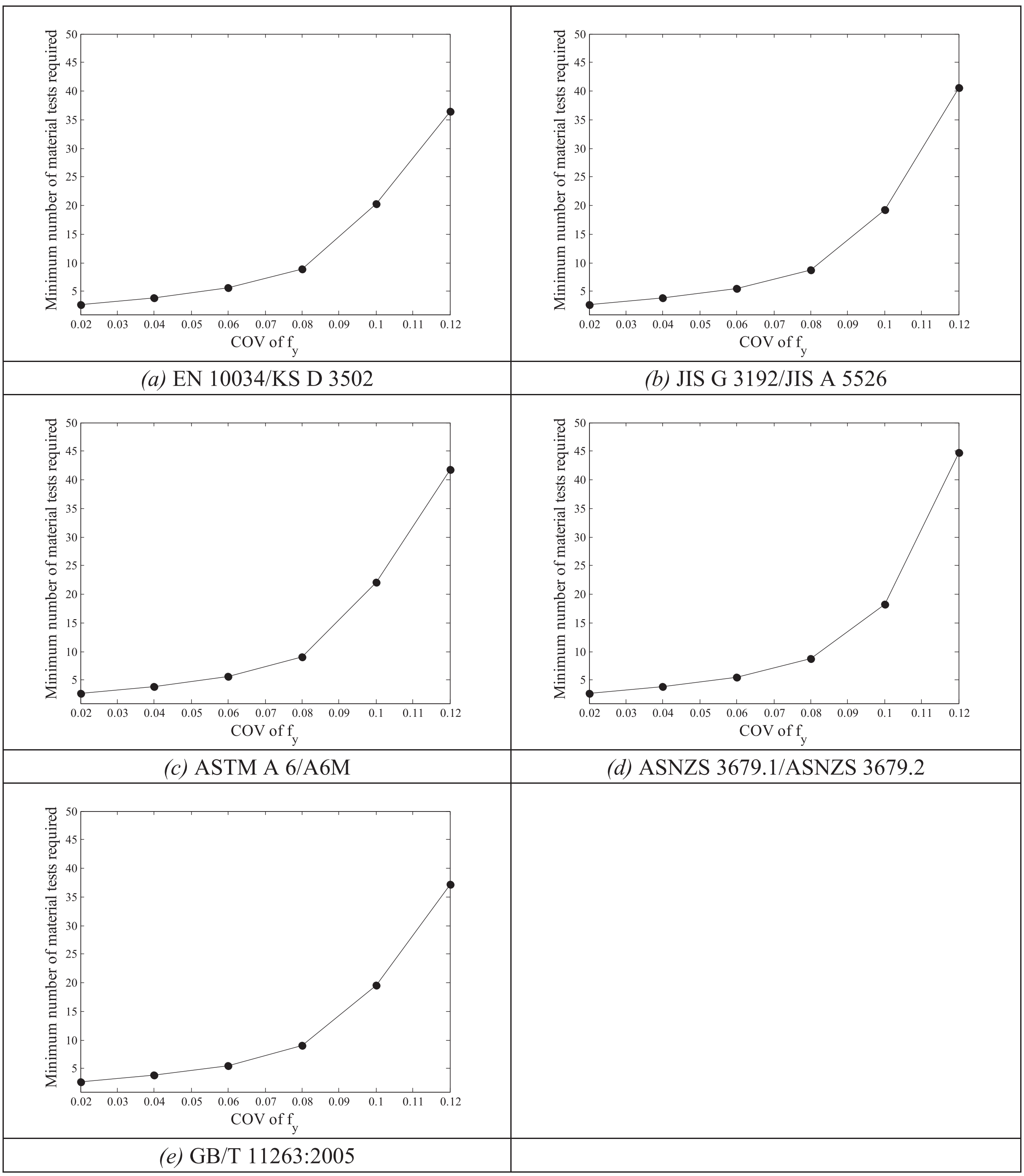


Fig. 6. The minimum number of material tests required to achieve $\phi = 0.90$ not-compact I sections using products complying with manufacturing tolerances given in international design codes.

It is also seen that the calibrated capacity reduction factor values are greater than the proposed value in AS 5100.6, AS 4100 [35] and NZS 3404.1 [11], i.e. $\phi = 0.9$, for the target reliability values smaller than 3.5 when the COV of f_y is smaller than or equal to 10%.

4.1.3. Inverse reliability analysis for compact, not-compact, and non-compact sections

In the capacity factor calibration, it was assumed that the number of material tests for the steel yield strength is infinite. However, if the number of tests is finite, the capacity factor should be reduced to compensate for the additional uncertainty created due to the insufficient number of test data. Therefore, in this study, the minimum number of test data for steel yield strength that achieves the capacity factor of 0.9 given in AS 5100.6 [10], NZS 3404.1 and AS 4100 was evaluated using the inverse calculation procedure proposed in Section 3.2. Again, the calculation was repeated for five different types of manufacturing tolerances given in EN 10034 [28]/KS D 3502 [29], JIS G 3192 [30]/JIS A 5526 [31], ASTM A 6/A6M [32], AS/NZS 3679.1 [33], AS/NZS 3679.2 [34], and GB/T 11263 [14], and all the assumptions remained the same.

The calculation results are provided in Figs. 5, 6, and 7 for compact, not-compact, and non-compact I sections, respectively, where each sub figure corresponds to each of the manufacturing tolerances in EN 10034 [28]/KS D 3502 [29], JIS G 3192 [30]/JIS A 5526 [31], ASTM A 6/A6M [32], AS/NZS 3679.1 [33], AS/NZS 3679.2 [34], and GB/T 11263

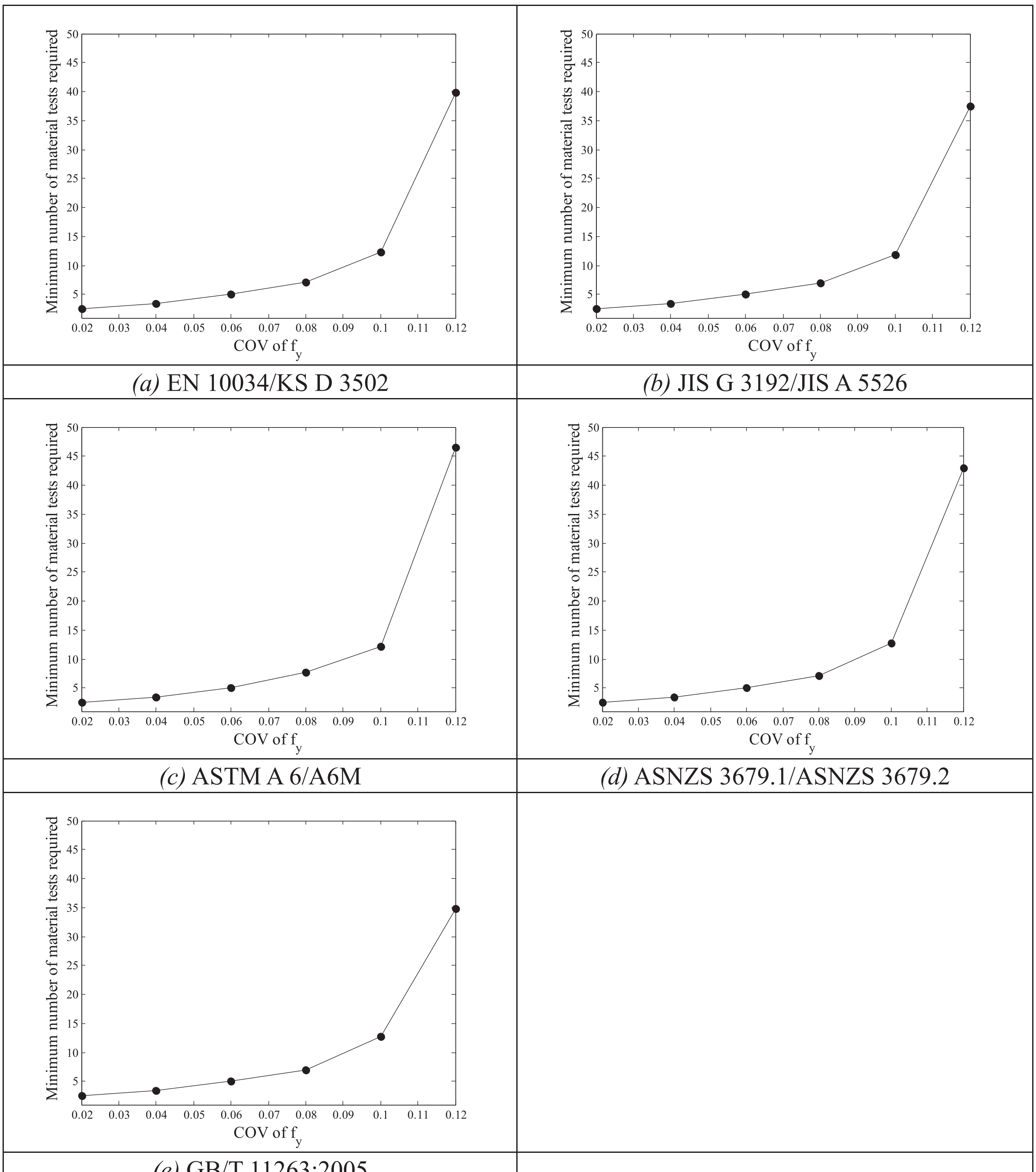


Fig. 7. The minimum number of material tests required to achieve $\phi = 0.90$ non-compact I sections using products complying with manufacturing tolerances given in international design codes.

[14]. The results show the required minimum number of steel yield tests against the sample COV of f_y obtained from a finite number of samples. The COV of f_y has been presented from 0.02 to 0.12 to cover the range of the values around 7–10% [11,14]. As expected, the minimum number of required tests increases as the COV of f_y increases because the increased uncertainty of f_y also increases the uncertainty of the overall beam. The results mean that if the number of material tests is less than the values, the capacity factor of 0.9 cannot achieve the intended target reliability level. As seen in the figures, the required numbers for compact sections are 7–15 for the COV of steel yield strength of 7–10%. The results for not-compact sections are similar to those for compact sections, but the numbers are slightly increased overall; the required numbers are 7–20 for the COV of steel yield strength of 7–10%. The results for non-compact sections are almost identical with the not-compact section results as the same database is used and the equation is just slightly different. For all three types of sections, the results are similar for all five different international manufacturing tolerances considered in this study with very small variations.

4.2. Hollow section columns

The forward and inverse reliability analyses proposed in Section 3 were carried out for hollow section columns based on the experimental member failure test data provided by Key et al. [36] with 11 specimens; full details of the experimental data can be found from [36] or [37].

The calibration was repeated for five different types of manufacturing tolerances given in EN 10219 [38], JIS G 3444 [39]/JIS G 3466 [40]/KS D 3566 [41]/KS D 3568 [42], ASTM A 500 [43], AS/NZS 1163 [44], and GB/T 6728 [45]. It was assumed that the tolerance values are equal to the standard deviations of the design parameters. The tolerance values used in this analysis are provided in Table 2. The same as for the I beams, it was also conservatively assumed that all the negative geometrical tolerances could occur together, except for the column length that considered the positive tolerances to represent an extreme case, and were not limited by the mass tolerance requirements given by the different manufacturing standards.

4.2.1. Capacity factor calibration

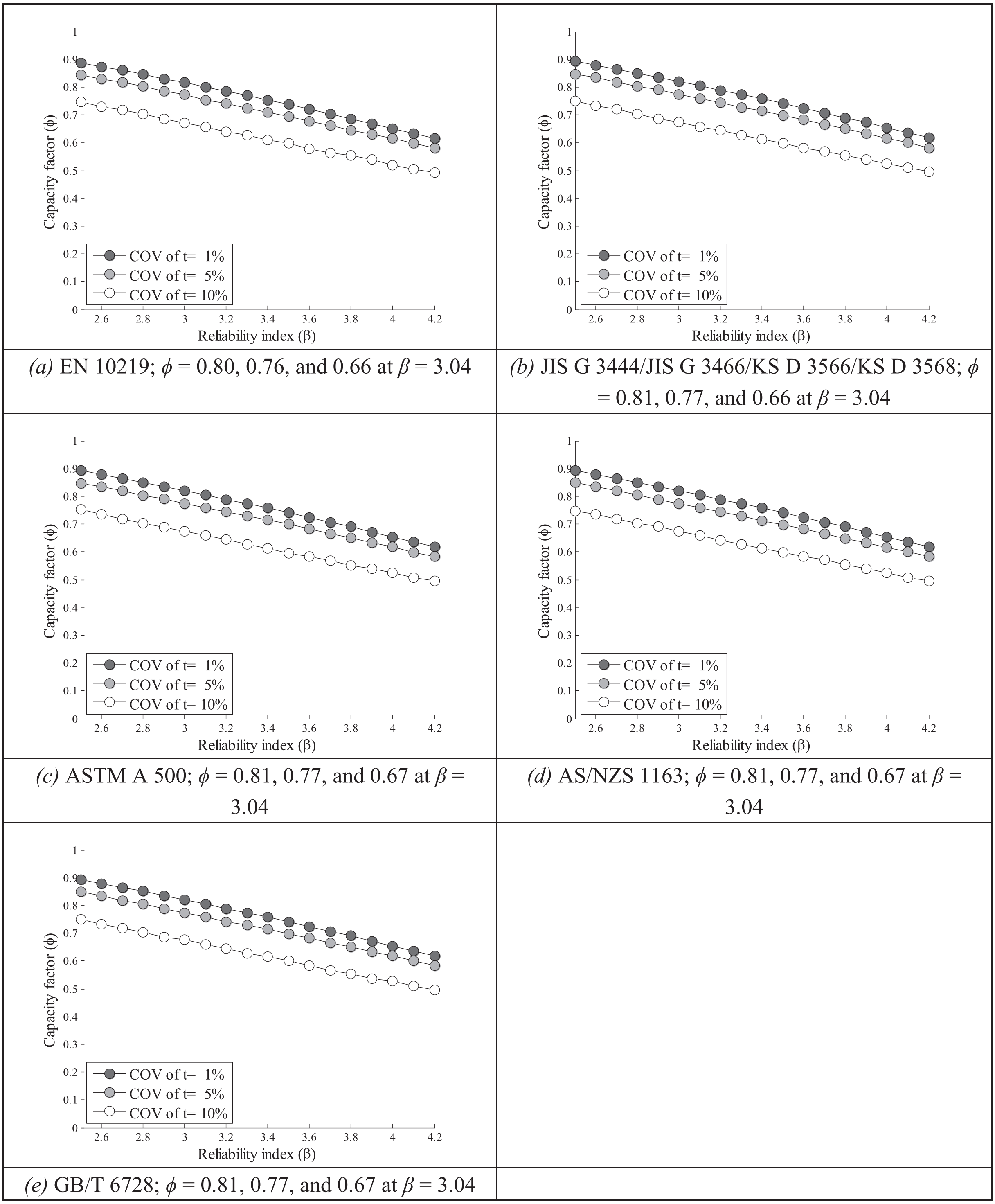
First, the capacity reduction factor for the design of hollow section columns was calibrated based on the experimental data provided by Key et al. [36]. These data include 11 test results on purely axially loaded cold-formed hollow sections with the three section types, 76 mm × 76 mm × 2.0 mm, 152 mm × 152 mm × 4.9 mm, and 203 mm × 203 mm × 6.3 mm, with yield steel strengths 425 N/mm, 416 N/mm, and 395 N/mm, respectively. For the calibration, the method provided in Section 3.1 was used where the number of yield strength tests is assumed to be infinite. The theoretical predictions for the resistance of the columns were made based on AS 5100.6 [10], AS 4100 [35] and NZS 3404.1 [11] where Euler buckling was considered for slender columns using the slenderness factor provided in these codes.

The calibration results are provided in Fig. 8 where each sub figure corresponds to each of the manufacturing tolerances in EN 10219 [38], JIS G 3444 [39]/JIS G 3466 [40]/KS D 3566 [41]/KS D 3568 [42], ASTM A 500 [43], AS/NZS 1163 [44], and GB/T 6728 [45]. For this analysis, first, the coefficient of variation for the yield strength of steel is conservatively taken to be 10%, which is consistent with the value used in the original Australian standard calibration conducted by Pham et al. [12].

The results are plotted for a range of target reliability values between 2.5 and 4.2, and the results for the target reliability $\alpha_R\beta = 3.04$ are reported as well. To investigate the effect of tightening the manufacturing tolerance for thickness in Table 2, the analysis was repeated with the tolerance represented as a COV of section thickness corresponding to 1%, 5%, and 10%. These three types of variations were chosen to see the effect of the gradual reduction of the uncertainties in the thickness where the 10% COV is a conservative assumption corresponding to the tolerance given in Table 2. The overall results show that the capacity

Table 2
Manufacturing tolerances used for reliability analysis for hollow sections.

Parameter	EN10219: 2006		JIS G3444: 1994 JIS G3466: 1988 KS D 3566: 1999 KS D 3568: 2009		ASTM A500		AS/NZS 1163: 2016		GB/T 6728: 2002	
Outside dimension for CHS (d_o) (mm)	$-0.01\ d_o$		$d_o \le 50$	-0.25	$d_o \le 48$	$-0.005d_o$	$d_o \le 50$	-0.8	$-(0.0098d_o + 0.09)$	
			$d_o > 50$	$-0.005d_o$	$d_o > 51$	$-0.0075d_o$	$d_o > 50$	$-0.01d_o$		
Outside dimension for RHS and SHS (b) (mm)	$b \le 100$	$-0.01b$	$b \le 100$	-1.5	$b \le 64$	-0.5	$b \le 50$	-0.5	$-(0.0076b + 0.12)$	
	$100 < b \le 200$	$-0.008b$	$b > 100$	$-0.015b$	$64 < b \le 89$	-0.64	$b > 50$	$-0.01b$		
	$200 < b$	$-0.006b$			$89 < b \le 140$	-0.76				
					$140 < b$	$-0.01b$				
Thickness for CHS (t) (mm)	$d_o \le 406.4$ and $t \le 5$	$-0.1\ t$	$t \le 3$	-0.3	$-0.1\ t$		$-0.1\ t$		$t \le 10$	$-0.1\ t$
	$d_o \le 406.4$ and $t > 5$	-0.5	$3 < t \le 12$	$-0.1\ t$					$t > 10$	-0.08 t
	$d_o > 406.4$	$-0.1\ t \le 2$	$t > 12$	-1.2						
Thickness for RHS and SHS (t) (mm)	$t \le 5$	$-0.1\ t$	$t \le 3$	-0.3	$-0.1\ t$		$-0.1\ t$		$t \le 10$	$-0.1\ t$
	$t > 5$	-0.5	$t > 3$	$-0.1\ t$					$t > 10$	-0.08 t
Length (L) (mm)	$L \le 6\ m$	$+5$	+nominal size		Not specified. This study uses the tolerance in AS/NZS 1163		$L \le 14\ m$	$+6$	$L \le 6\ m$	$+5$
	$6\ m < L \le 10\ m$	$+15$					$14\ m < L \le 18\ m$	$+10$	$6\ m < L \le 12\ m$	$+10$
	$10\ m < L$	$+5 + 1\ mm/m$								



(a) EN 10219; ϕ = 0.80, 0.76, and 0.66 at β = 3.04

(b) JIS G 3444/JIS G 3466/KS D 3566/KS D 3568; ϕ = 0.81, 0.77, and 0.66 at β = 3.04

(c) ASTM A 500; ϕ = 0.81, 0.77, and 0.67 at β = 3.04

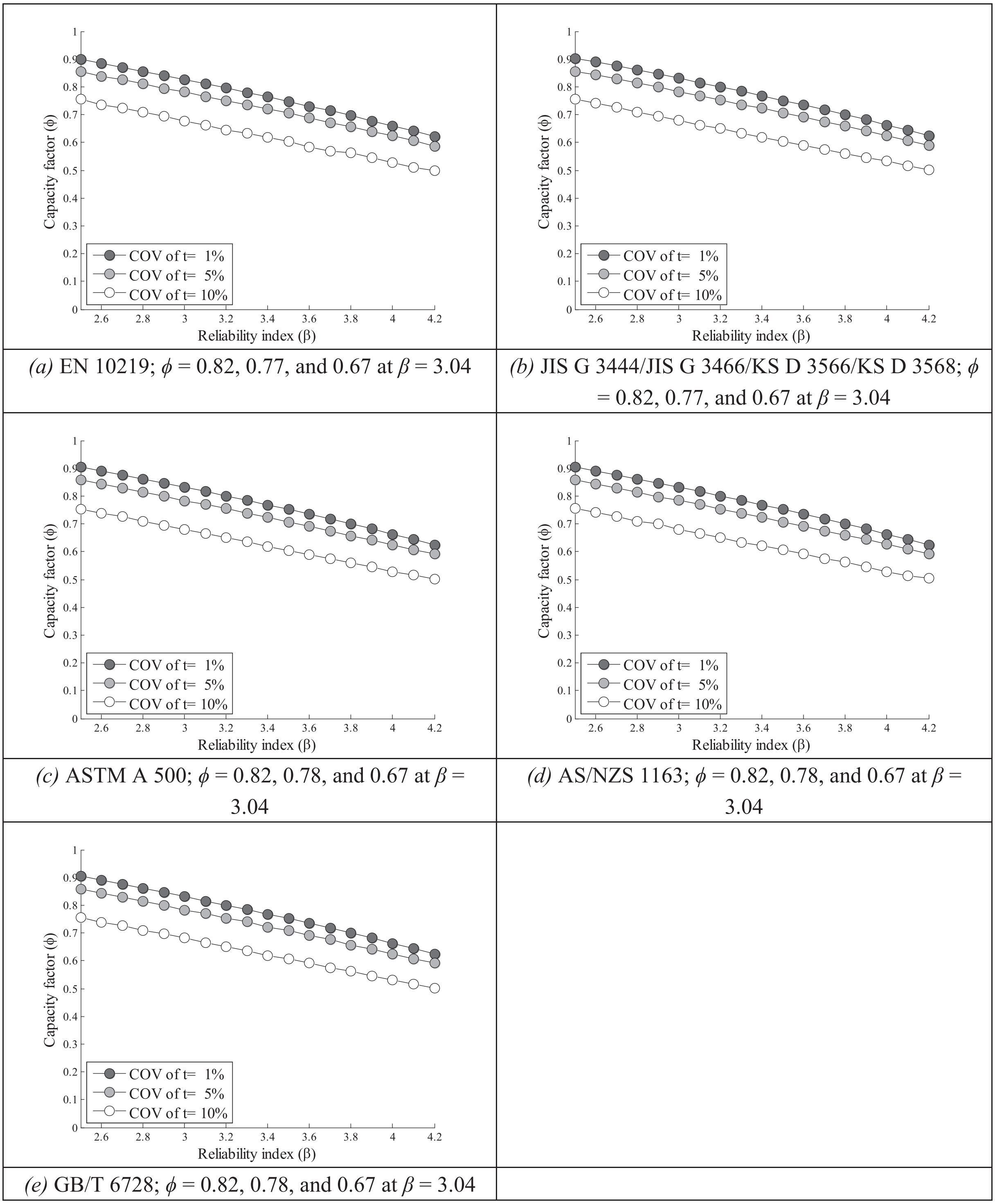
(d) AS/NZS 1163; ϕ = 0.81, 0.77, and 0.67 at β = 3.04

(e) GB/T 6728; ϕ = 0.81, 0.77, and 0.67 at β = 3.04

Fig. 8. The capacity factors for hollow section columns using products complying with manufacturing tolerances given in international design codes when the coefficient of variation for the yield strength of steel is 10%.

factors vary from 0.80 to 0.66 according to the increasing COV of thickness, but all of them are <0.90 that is provided in AS 5100.6 [10], NZS 3404.1 [11] and AS 4100 [35], which suggests that the current design equations for hollow section columns are unconservative. Two reasons for these low capacity factors are as follows: (i) the hollow section thickness is one of the main parameters to determine the sectional area, and the COV of the thickness directly affects the capacity of hollow section columns. (ii) The number of hollow section column tests considered in this study is 11, which is relatively far from infinity that is the ideal case, and it creates an additional error. The results are almost identical for all the international manufacturing tolerances considered in this study within round-off error, meaning that the results are not sensitive to the variations in the geometric parameters.

Second, the analysis was repeated for the coefficient of variation for the yield strength of steel is taken to be 7% that is often used internationally [15] to see if this change improves the capacity factors. In



(a) EN 10219; $\phi = 0.82$, 0.77, and 0.67 at $\beta = 3.04$

(b) JIS G 3444/JIS G 3466/KS D 3566/KS D 3568; $\phi = 0.82$, 0.77, and 0.67 at $\beta = 3.04$

(c) ASTM A 500; $\phi = 0.82$, 0.78, and 0.67 at $\beta = 3.04$

(d) AS/NZS 1163; $\phi = 0.82$, 0.78, and 0.67 at $\beta = 3.04$

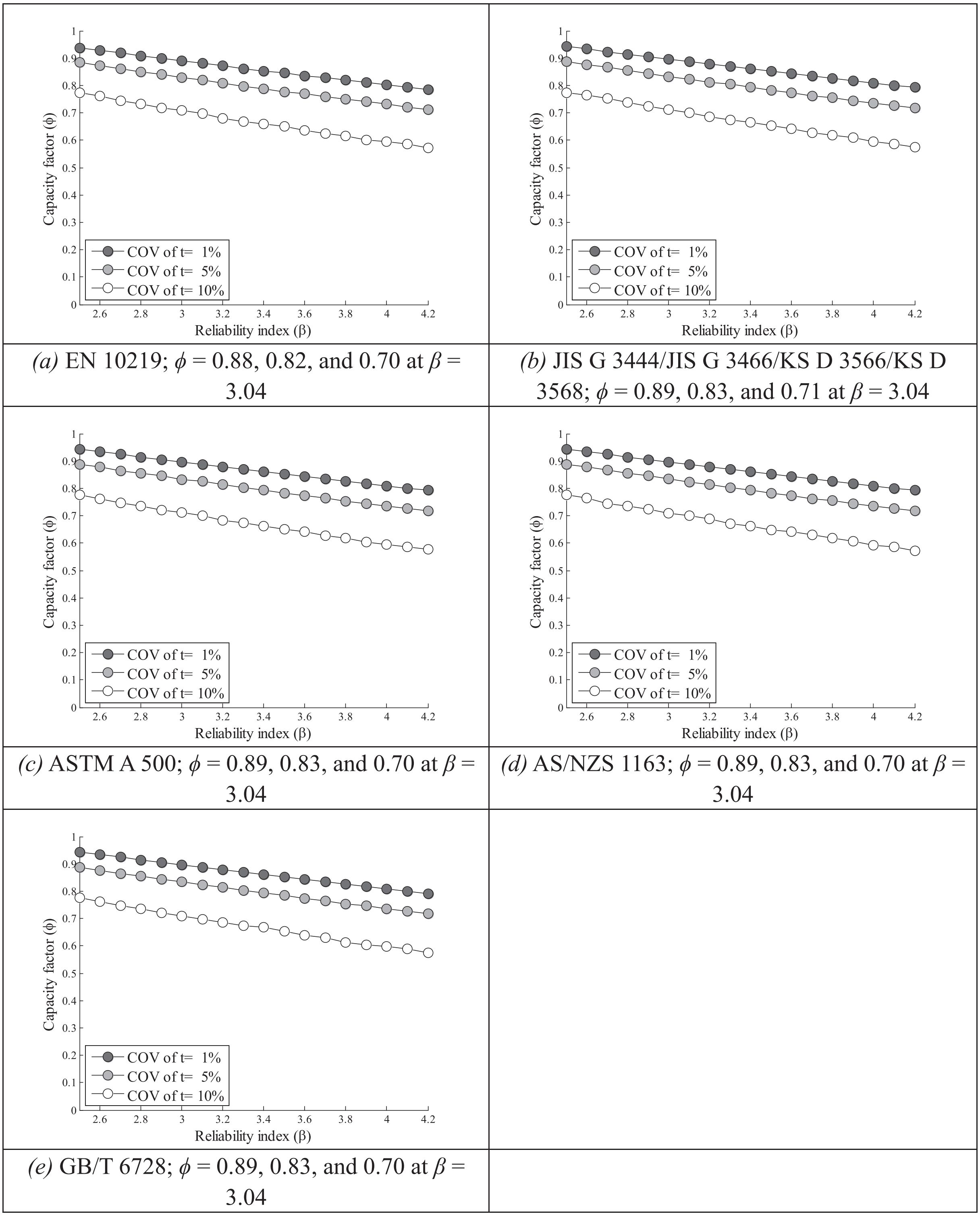
(e) GB/T 6728; $\phi = 0.82$, 0.78, and 0.67 at $\beta = 3.04$

Fig. 9. The capacity factors for hollow section columns using products complying with manufacturing tolerances given in international design codes when the coefficient of variation for the yield strength of steel is 7%.

Fig. 9, the results are plotted for a range of target reliability values between 2.5 and 4.2, and the results for the target reliability $\alpha_R\beta = 3.04$ are reported as well. The analysis was again repeated for three different coefficient of variation of the section thickness, 1%, 5%, and 10%. The values vary from 0.82 to 0.67 according to the varying COV of section thickness, and we can see that the capacity factor is slightly improved by around 0.02 from the case with 10% COV in the steel yield strength but the effect is not significant and the design equations still remain unconservative. Again, these capacity factors are not sensitive to the different international manufacturing tolerances in geometric parameters.

Third, to see the effect of the uncertainty due to the insufficient number of test data to the capacity factors, the uncertainty was intentionally neglected, and the coefficient of variation for the yield strength of steel is kept being taken to be 7%.

In Fig. 10, the results are plotted for a range of target reliability values between 2.5 and 4.2, and the results for the target reliability $\alpha_R\beta = 3.04$ are reported as well. The analysis was again repeated for three different



(a) EN 10219; ϕ = 0.88, 0.82, and 0.70 at β = 3.04

(b) JIS G 3444/JIS G 3466/KS D 3566/KS D 3568; ϕ = 0.89, 0.83, and 0.71 at β = 3.04

(c) ASTM A 500; ϕ = 0.89, 0.83, and 0.70 at β = 3.04

(d) AS/NZS 1163; ϕ = 0.89, 0.83, and 0.70 at β = 3.04

(e) GB/T 6728; ϕ = 0.89, 0.83, and 0.70 at β = 3.04

Fig. 10. The capacity factors for hollow section columns using products complying with manufacturing tolerances given in international design codes when the coefficient of variation for the yield strength of steel is 7% with the uncertainty due to the insufficient data neglected.

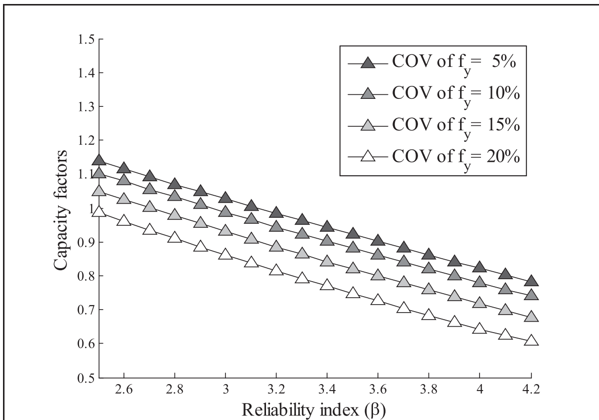
coefficient of variation of the section thickness, 1%, 5%, and 10%. The results were improved, and the capacity factors were varying from 0.89 to 0.70. This shows that by collecting more data, the uncertainty due to the number of data can be improved. However, the capacity factor of 0.90 provided in AS 5100.6 [10], NZS 3404.1 [11] and AS 4100 [35] can only be achieved if the manufacturing tolerances specified in the international standards considered in this study are tightened from 10% to around 1% of the section thickness.

4.2.2. The required number of steel yield tests

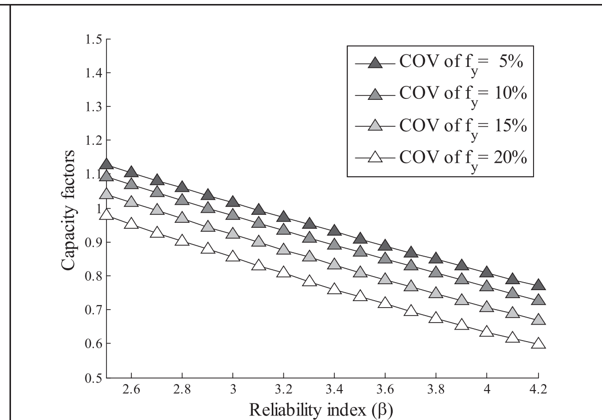
Even if the number of steel yield strength test results is infinite, the performance of the design equation does not meet the target reliability level due to the large COV of section thickness. Thus, the inverse analysis for hollow sections has not been performed. Stricter quality control in section thickness is needed to achieve the sufficient reliability. Given that the Australian and New Zealand design provisions are similar to other international design standards, such as Eurocode 3, this finding should be considered by the appropriate national standards committees to ensure that uniform margins of safety are being delivered.

4.3. Concrete-filled steel tube (CFST) columns

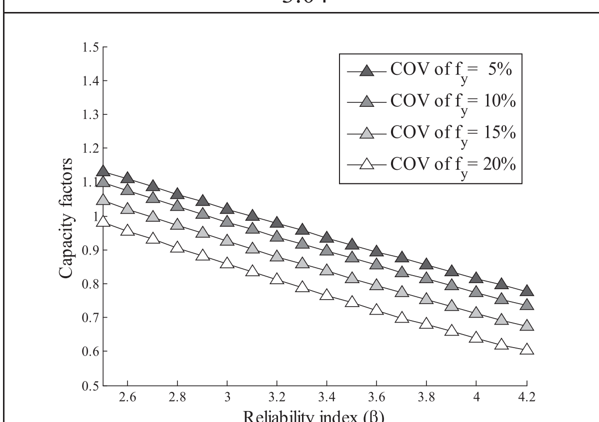
The forward and inverse reliability analyses proposed in Section 3 was carried out for concrete-filled steel tube (CFST) columns. This study utilised the extensive database developed by Tao et al. [46] to estimate the modelling error, and the database includes experimental results for CFST members over the last few decades by merging the



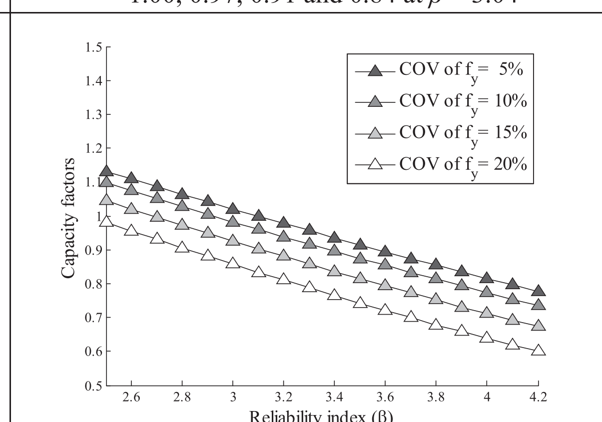
(a) EN 10219; ϕ = 1.02, 0.98, 0.92 and 0.85 at β = 3.04



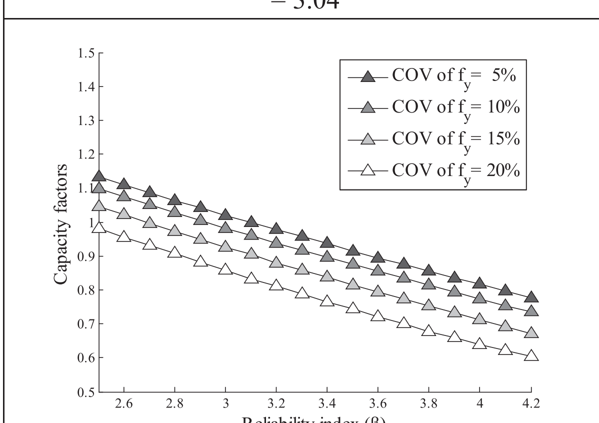
(b) JIS G 3444/JIS G 3466/KS D 3566/KS D 3568; ϕ = 1.00, 0.97, 0.91 and 0.84 at β = 3.04



(c) ASTM A 500; ϕ = 1.01, 0.97, 0.91 and 0.84 at β = 3.04



(d) AS/NZS 1163; ϕ = 1.00, 0.97, 0.91 and 0.84 at β = 3.04

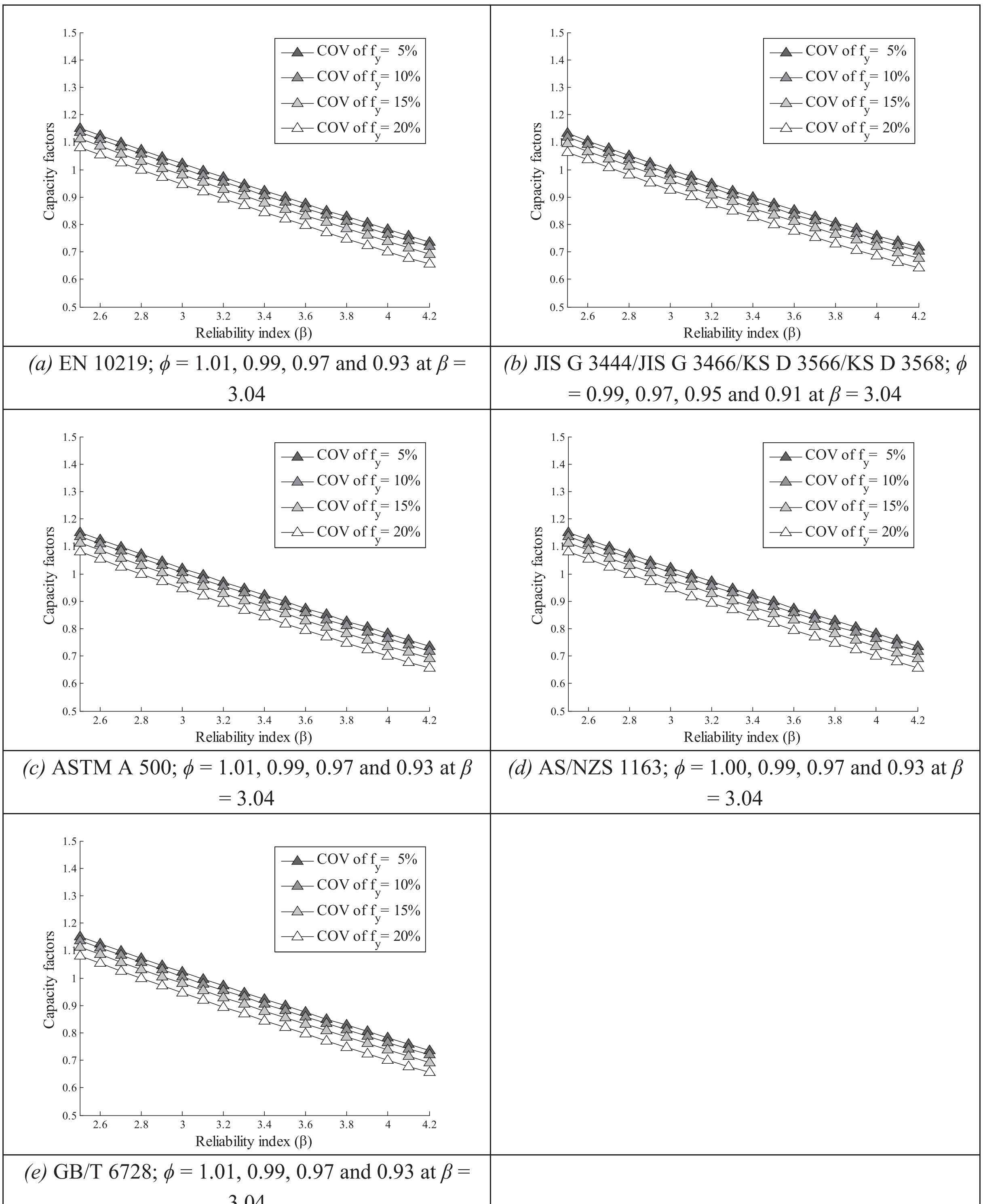


(e) GB/T 6728; ϕ = 1.01, 0.97, 0.92 and 0.84 at β = 3.04

Fig. 11. The capacity factors for steel in rectangular stub CFST columns for the various coefficient of variation for the yield strength of steel.

databases established by Goode [47] and Wu [48]. It has a total of 2194 test results from 130 references. In this study, a subset of the database for column members was considered with 1583 test results, which include 445 results for rectangular stub columns, 234 results for long rectangular columns, 484 results for circular stub columns, and 420 results for circular long columns. When $L_e/d_o \le 4$ for circular members or $L_e/b \le 4$ for rectangular members, the members are defined as stub columns with no slenderness effect, where L_e is the effective length of a column, d_o is the diameter of a circular section, and b is the section width of a rectangular section.

In the Tao et al.'s database [46], some references do not provide the mean measured compressive strength of concrete (f_{cm}) values but report the values of 150 mm cubes (f_{cu}) instead. For these cases, the equivalent compressive strengths were obtained using the conversion table given by Yu et al. [49] representing the approximate relationship between cylinder strength (f_{cm}) and cube strength (f_{cu}). This table was



(a) EN 10219; ϕ = 1.01, 0.99, 0.97 and 0.93 at β = 3.04

(b) JIS G 3444/JIS G 3466/KS D 3566/KS D 3568; ϕ = 0.99, 0.97, 0.95 and 0.91 at β = 3.04

(c) ASTM A 500; ϕ = 1.01, 0.99, 0.97 and 0.93 at β = 3.04

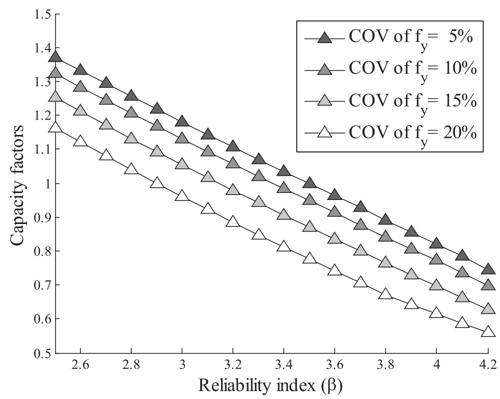
(d) AS/NZS 1163; ϕ = 1.00, 0.99, 0.97 and 0.93 at β = 3.04

(e) GB/T 6728; ϕ = 1.01, 0.99, 0.97 and 0.93 at β = 3.04

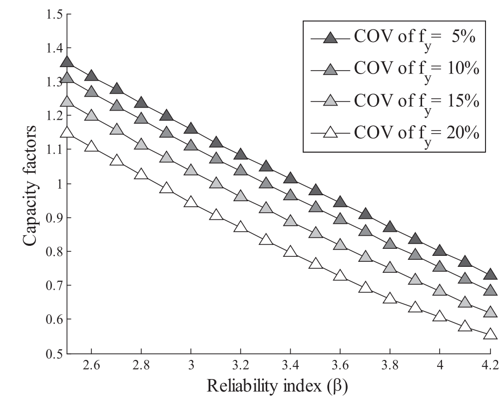
Fig. 12. The capacity factors for steel in rectangular long CFST columns for the various coefficient of variation for the yield strength of steel.

developed based on Chen et al.'s work [50], which determined the equivalent compressive strength of high-strength concrete. The uncertainty in this conversion table was not evaluated separately but included in the modelling uncertainty of the resistance prediction equations.

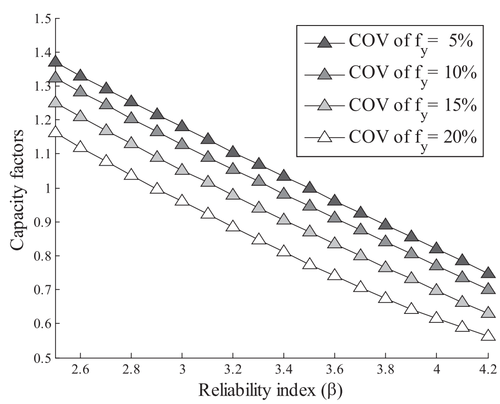
In this study, the same as the hollow section columns, calibration was repeated for five different types of manufacturing tolerances given in EN 10219 [38], JIS G 3444 [39]/JIS G 3466 [40]/KS D 3566 [41]/KS D 3568 [42], ASTM A 500 [43], AS/NZS 1163 [44], and GB/T 6728 [45]. It was assumed that the tolerance values were equal to the standard deviations of the design parameters. The tolerance values in Table 2 were used again in this analysis. It was also conservatively assumed that all the negative geometrical tolerances can occur together, except for the column length that considered the positive tolerances to represent an extreme case, and were not limited by the mass tolerance requirements given by the different manufacturing standards.



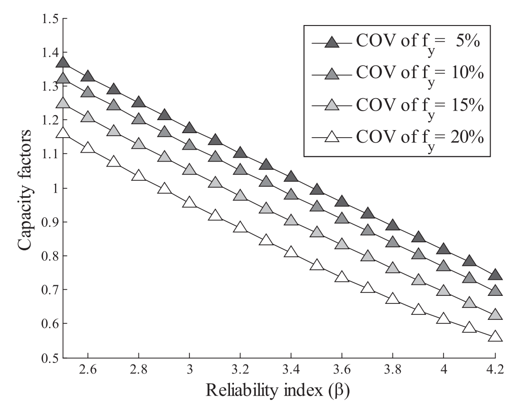
(a) EN 10219; ϕ = 1.16, 1.11, 1.04 and 0.94 at β = 3.04



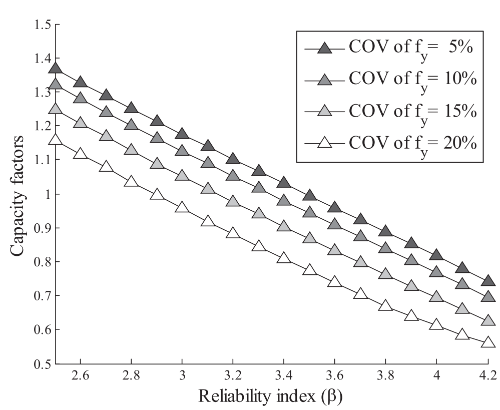
(b) JIS G 3444/JIS G 3466/KS D 3566/KS D 3568; ϕ = 1.14, 1.09, 1.01 and 0.92 at β = 3.04



(c) ASTM A 500; ϕ = 1.16, 1.11, 1.03 and 0.94 at β = 3.04

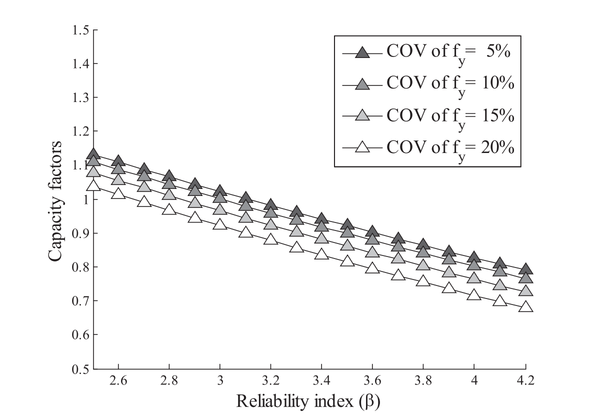


(d) AS/NZS 1163; ϕ = 1.16, 1.10, 1.03 and 0.94 at β = 3.04

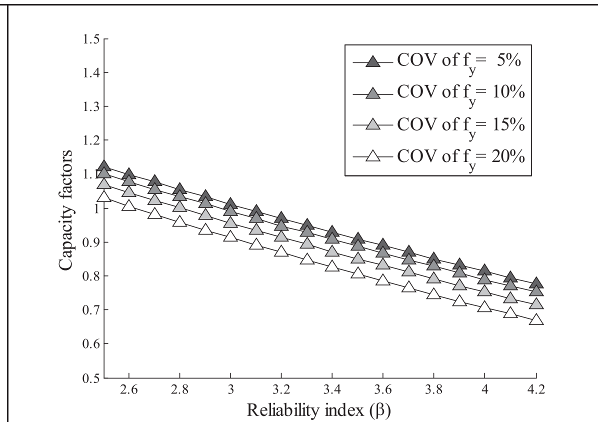


(e) GB/T 6728; ϕ = 1.16, 1.11, 1.03 and 0.94 at β = 3.04

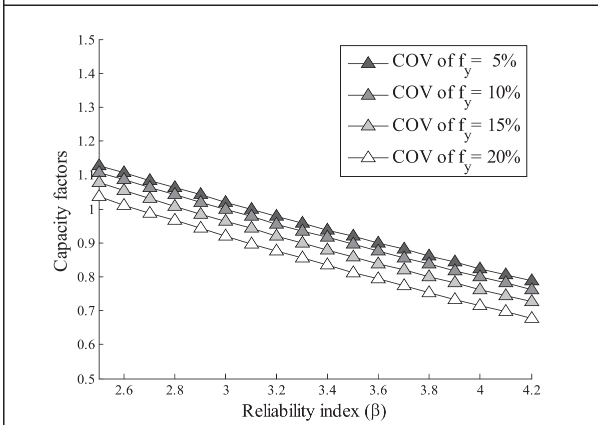
Fig. 13. The capacity factors for steel in circular stub CFST columns for the various coefficient of variation for the yield strength of steel.



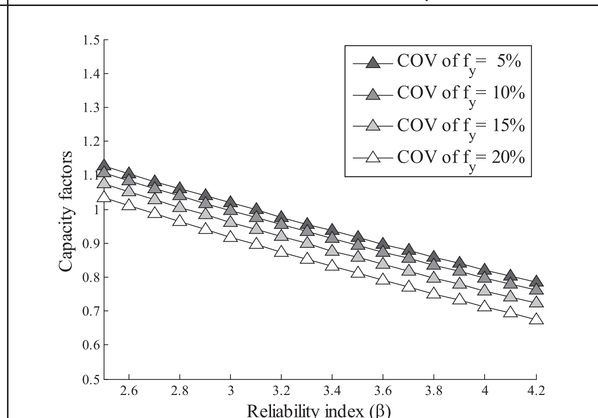
(a) EN 10219; ϕ = 1.01, 0.99, 0.95 and 0.91 at β = 3.04



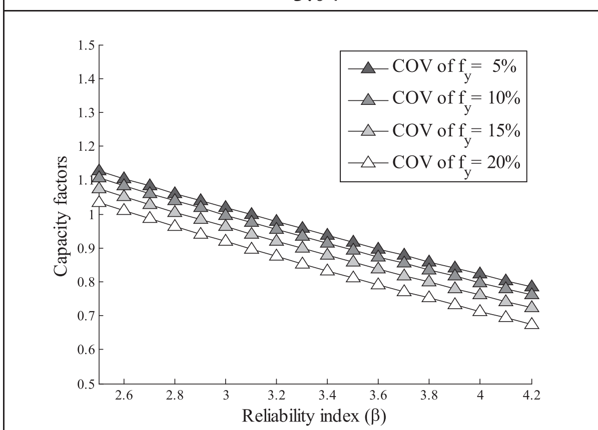
(b) JIS G 3444/JIS G 3466/KS D 3566/KS D 3568; ϕ = 1.00, 0.98, 0.94 and 0.90 at β = 3.04



(c) ASTM A 500; ϕ = 1.01, 0.99, 0.95 and 0.91 at β = 3.04



(d) AS/NZS 1163; ϕ = 1.00, 0.98, 0.95 and 0.91 at β = 3.04



(e) GB/T 6728; ϕ = 1.01, 0.99, 0.95 and 0.91 at β = 3.04

Fig. 14. The capacity factors for steel in circular long CFST columns for the various coefficient of variation for the yield strength of steel.

4.3.1. Capacity factor calibration

The capacity factor was calibrated using the method in Section 3.1 by taking the number of steel yield strength tests to be infinity. The capacity prediction equation provided in AS 5100.6 [10] includes two capacity factors for steel and concrete, respectively. However, the calibration method in Section 3.1. is only for a single capacity factor, and in this study, the value for the capacity factor for concrete was fixed to be 0.60 as provided in AS 5100.6 [10], and the other capacity factor for steel was calibrated and compared with the value provided in AS 5100.6 [10] that is 0.90.

The calibration results are provided in Figs. 11–14 where the four figures show the results for rectangular stub columns, rectangular long columns, circular stub columns, and circular long columns, respectively. In each figure, each sub figure corresponds to each of the manufacturing tolerances for steel in EN 10219 [38], JIS G 3444 [39]/JIS G 3466 [40]/KS D 3566 [41]/KS D 3568 [42], ASTM A 500 [43], AS/

NZS 1163 [44], and GB/T 6728 [45]. The results are plotted for a range of target reliability values between 2.5 and 4.2, and the results for the target reliability $\alpha_R\beta = 3.04$ are reported as well.

In all the figures, it is consistently seen that the capacity factor for steel decreases as the COV of steel yield strength increases. This is because the increase in the COV of steel yield strength also increases the parametric uncertainty of the whole capacity prediction model, and therefore, a smaller capacity factor is required to provide extra safety to offset the increased uncertainty. Considering that the COV of steel is usually taken to be 7%–10% [12,15], the results for the capacity factor for steel are consistently far >0.90. It also confirms that the value of 0.90 provided in AS 5100.6 [10] achieves safety requirement corresponding to the target reliability index 3.8. The calibrated capacity factors are almost identical for different international manufacturing tolerances, because the tolerance values specified in the international standards considered in this study are very similar to each other, and the uncertainties in geometric parameters have a relatively indirect effect to the prediction model compared to the uncertainties in models and strength parameters, which has a direct effect.

4.3.2. Inverse reliability analysis

The minimum number of test data for steel yield strength was calculated when it achieved the capacity factor of 0.90 for steel and the factor for concrete was fixed. The inverse calculation procedure proposed in

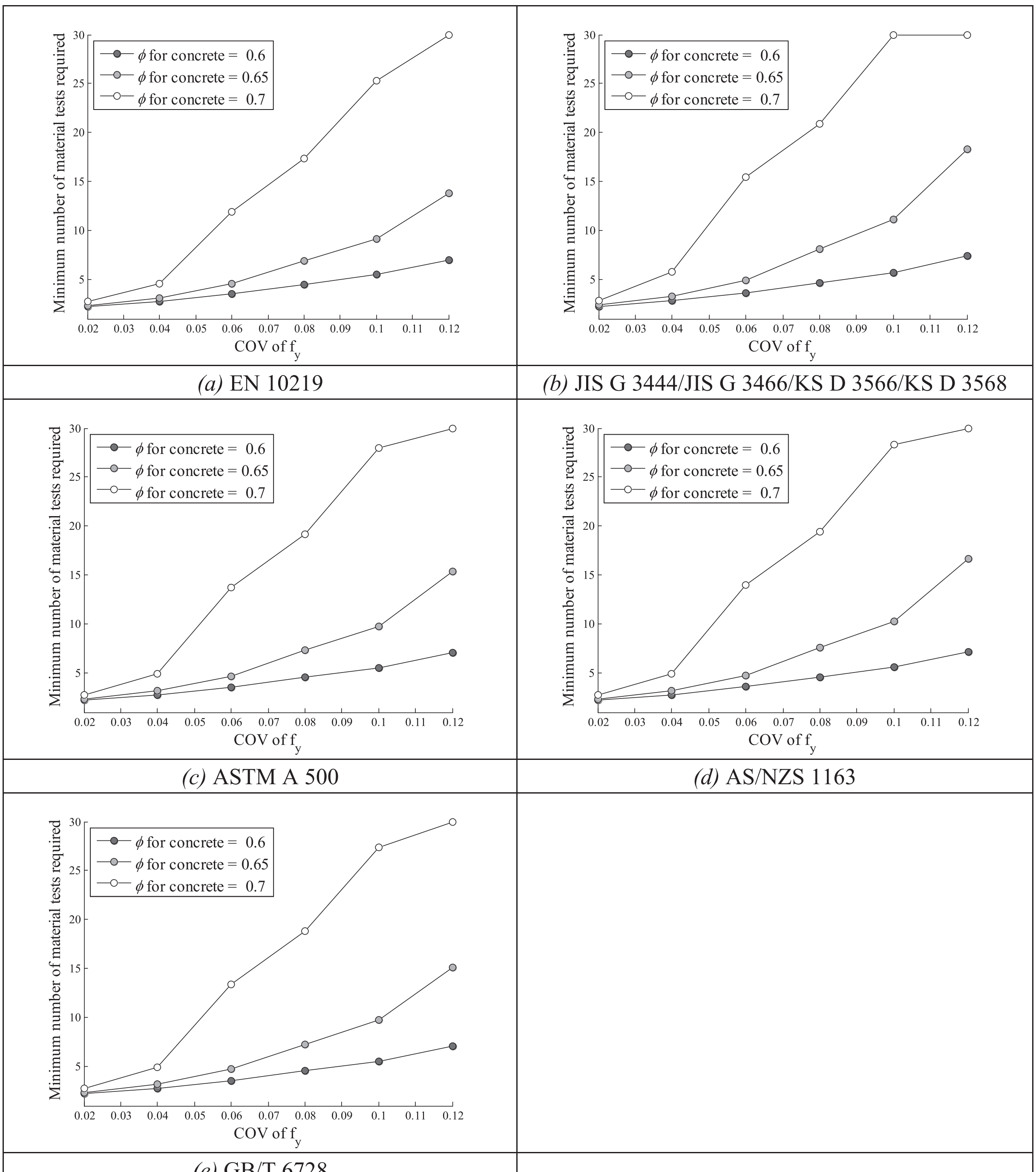


Fig. 15. The required minimum number of yield strength tests for rectangular stub CFST columns.

Section 3.2 was used. The calculation was repeated for five different types of manufacturing tolerances for steel given in EN 10219 [38], JIS G 3444 [39]/JIS G 3466 [40]/KS D 3566 [41]/KS D 3568 [42], ASTM A 500 [43], AS/NZS 1163 [44], and GB/T 6728 [45].

The calculation results are provided in Figs. 15–18 where the four figures show the results for rectangular stub columns, rectangular long columns, circular stub columns, and circular long columns, respectively. Each sub figure corresponds to each of the manufacturing tolerances in EN 10219 [38], JIS G 3444 [39]/JIS G 3466 [40]/KS D 3566 [41]/KS D 3568 [42], ASTM A 500 [43], AS/NZS 1163 [44], and GB/T 6728 [45]. The results show the required minimum number of steel yield tests against the COV of the steel yield strength obtained from the finite number of samples when the capacity factor for steel is fixed at 0.90 and that for concrete is 0.60, 0.65 or 0.70. The three values show the effect of the target capacity factor for concrete. These values are chosen because the current Australian standard such as AS 3600 [51] adopts the value 0.60 but the recent reliability analysis such as Kang et al. [9] suggest that the capacity factor for steel can be increased above 0.65, which has now been adopted in both AS/NZS 2327 [52] and AS/NZS 5100.6 [53]. For further analysis, a capacity factor for concrete 0.7 has been also considered, and the corresponding results are also presented together.

In Fig. 15 for rectangular stub columns, the number of required tests for steel yield strength is estimated as values between 2 and 7 for the overall range of the COV of steel yield strength between 2% and 12% when the capacity factor for concrete is fixed to be 0.60 according to

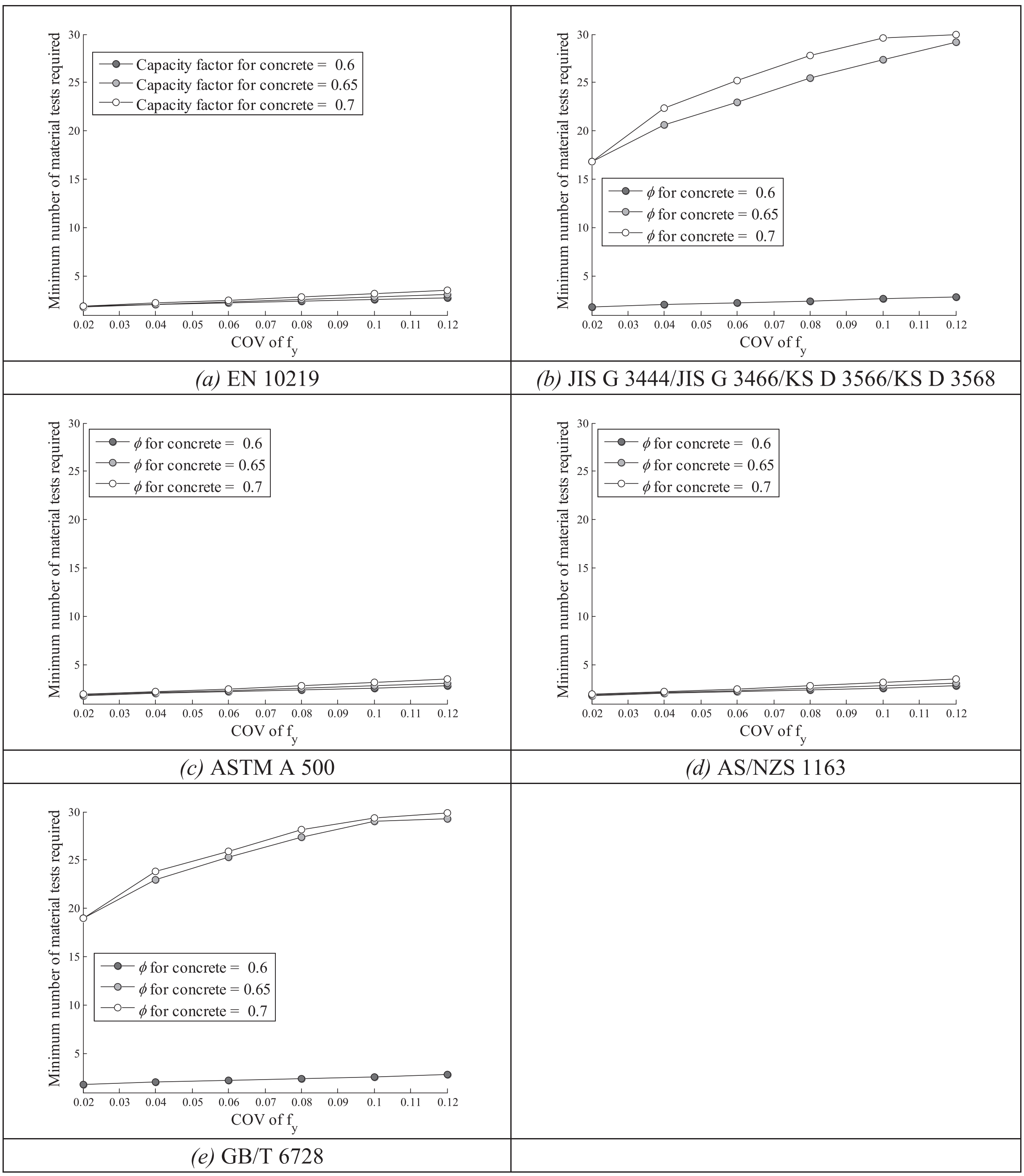


Fig. 16. The required minimum number of yield strength tests for rectangular long CFST columns.

the current Australian standard. A significant increase in the required number of material test is observed when the capacity factor for concrete is increased to 0.65 or 0.70 to compensate the reduced safety in the concrete by reducing the material test uncertainty. The required number of material tests is relatively small compared to the I section beams when the capacity factor for concrete is 0.60 because CFST columns have greater safety in their design equations. Similar to the forward analysis and the results for the other member types, the variation in the results according to different international manufacturing tolerances is not clearly observed, which means that the international manufacturing tolerances are similar to each other, and the uncertainties for geometric parameters negligibly affect the design safety.

For the other CFST member types including rectangular long columns, and circular stub and long columns, the results generally show the same trend when the capacity factor for concrete is fixed to be 0.60, although the prediction equations have all different forms. Circular stub columns and rectangular stub columns show a relatively larger increase in the required number of material tests because their safety is more sensitive to the COV of steel yield strength as shown in the forward analysis results. When the capacity factor for concrete is 0.65 or 0.70, rectangular stub columns, circular stub columns, and some cases of rectangular long columns requires an increase in the number of material tests because of the reduced safety that goes beyond the target level. Also, variations are found in Fig. 16 over different international

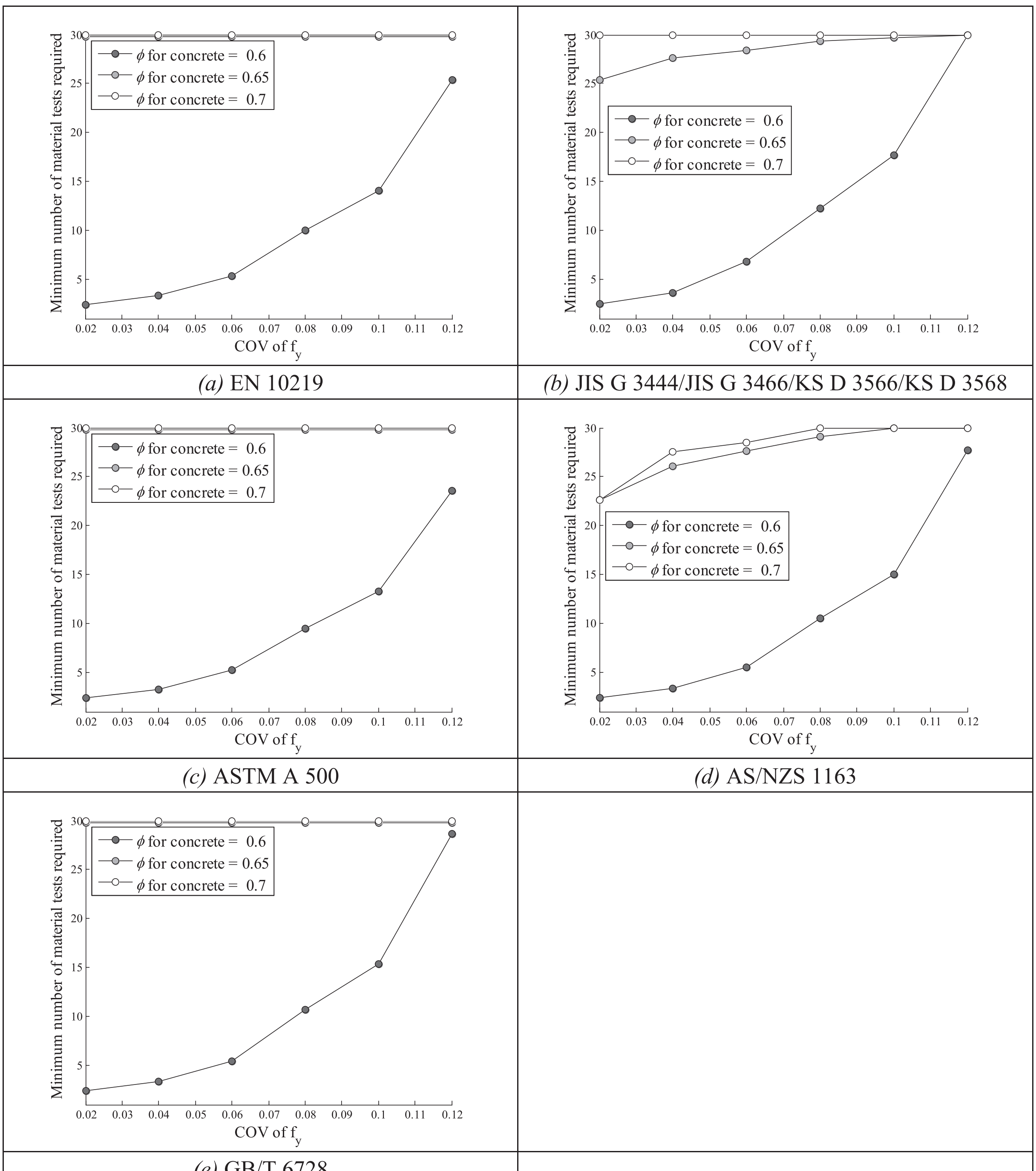


Fig. 17. The required minimum number of yield strength tests for circular stub CFST columns.

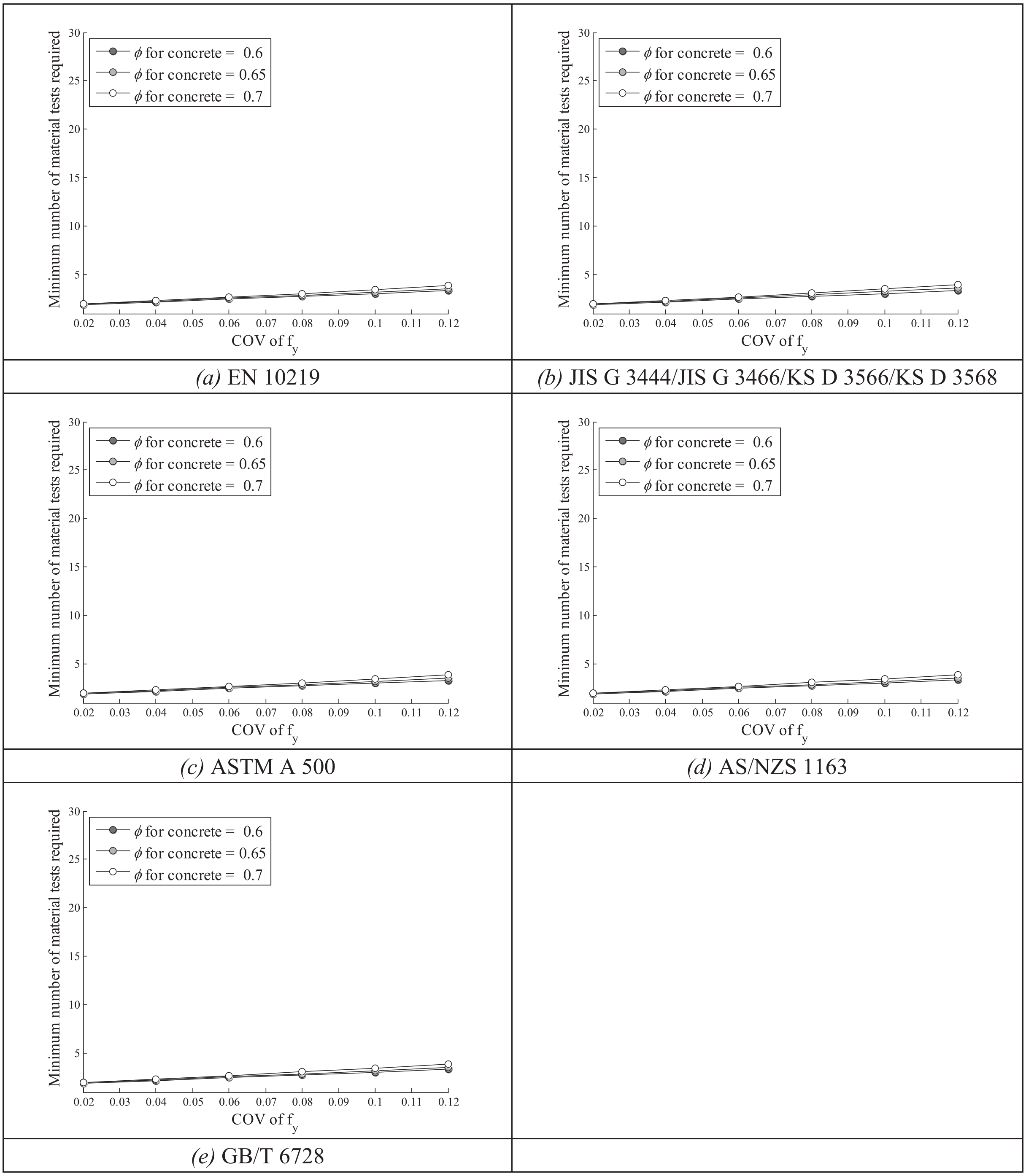


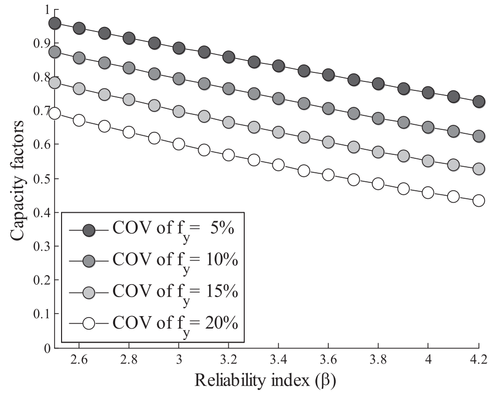
Fig. 18. The required minimum number of yield strength tests for circular long CFST columns.

manufacturing tolerances when the capacity factor for concrete is either 0.65 or 0.7. This is because the number of the required material tests does not significantly change the reliability level, and the overall uncertainty depends more on the uncertainties of other parameters. Unlike the capacity factor 0.6, which provides conservative design against the target reliability level ($\beta = 3.04$), the capacity factors 0.65 and 0.7 meet the target reliability level, and other factors including geometric parameters become relatively more influential to the reliability of the CFST design. The relatively small number of material tests also means that the reliability level is insensitive to the number of material tests and more sensitive to the variation of other parameters.

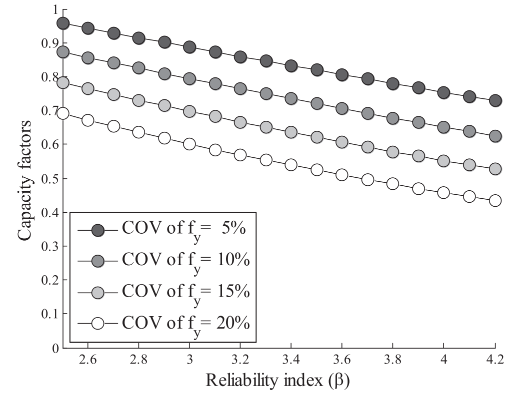
In this study, we limited the scope on the variability of the COV of steel yield strength and the required number of material test data for the steel yield strength. However, the proposed framework can be extensively applied to other types of parameters such as the compressive strength of concrete and the yield strength of shear studs.

4.4. Composite beams

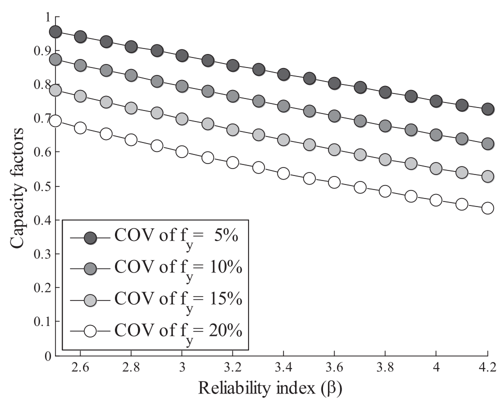
The forward and inverse reliability analyses proposed in Section 3 was carried out for composite beams. For the analysis, the database provided in Hicks & Pennington [54] was used, which has in total 164 tests



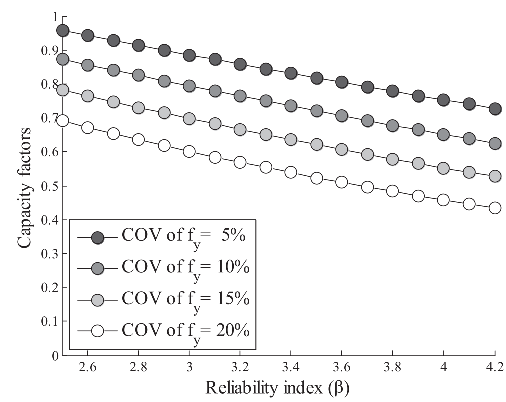
(a) EN 10034/KS D 3502; $\phi = 0.88, 0.78, 0.69$ and 0.59 at $\beta = 3.04$



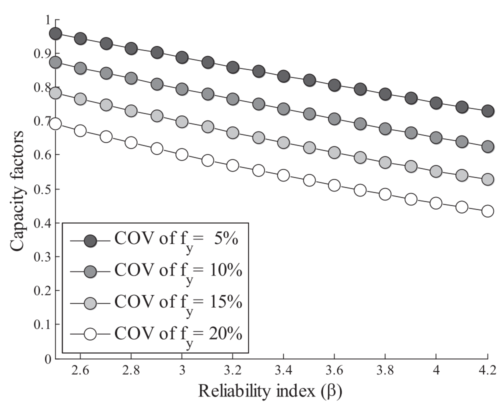
(b) JIS G 3192/JIS A 5526; $\phi = 0.88, 0.79, 0.69$ and 0.59 at $\beta = 3.04$



(c) ASTM A 6/A6M; $\phi = 0.88, 0.79, 0.69$ and 0.59 at $\beta = 3.04$



(d) ASNZS 3679.1/ASNZS 3679.2; $\phi = 0.88, 0.79, 0.69$ and 0.59 at $\beta = 3.04$



(e) GB/T 11263; $\phi = 0.88, 0.79, 0.69$ and 0.59 at $\beta = 3.04$

Fig. 19. The capacity factors for steel in composite beams for the various coefficient of variation for the yield strength of steel.

on composite beams that were subjected to sagging bending. In this study, 83 test data were selected, which satisfy that they have partial shear connection and ductile shear connectors with 235 MPa ≤ f_y ≤ 355 MPa.

In this study, similar to the I section beams, calibration was repeated for five different types of manufacturing tolerances given in EN 10034 [28]/KS D 3502 [29], JIS G 3192 [30]/JIS A 5526 [31], ASTM A 6/A6M [32], AS/NZS 3679.1 [33], AS/NZS 3679.2 [34], and GB/T 11263 [14].

4.4.1. Capacity factor calibration

The capacity factor was calibrated using the method in Section 3.1 by taking the number of steel yield strength tests to be infinity. The capacity prediction equation provided in AS/NZS 2327 includes three capacity factors for steel, concrete, and shear connectors, respectively. To use the calibration method in Section 3.1. that is only for a single capacity factor, in this study the values for the capacity factors for both concrete and shear connectors were equally fixed to be 0.80 as provided in AS

5100.6 [10], and the other capacity factor for steel was calibrated and compared with the value provided in AS 5100.6 [10] that is 0.90.

The calibration results are provided in Fig. 19. In each figure, each sub figure corresponds to each of the manufacturing tolerances in EN 10034 [28]/KS D 3502 [29], JIS G 3192 [30]/JIS A 5526 [31], ASTM A 6/A6M [32], AS/NZS 3679.1 [33], AS/NZS 3679.2 [34], and GB/T 11263 [14]. The results are plotted for a range of target reliability values between 2.5 and 4.2, and the results for the target reliability $\alpha_R\beta = 3.04$ are reported as well.

In all the figures, it is consistently seen that the capacity factor for steel decreases as the COV of steel yield strength increases. The calibrated capacity factors are identical for different international manufacturing tolerances because the tolerance values specified in the international standards considered in this study are very similar to each other like the case of I section beams. The parametric uncertainty has a relatively indirect effect to the prediction model compared to the modelling uncertainty, which has a direct effect. The variations in international tolerances do not significantly change the safety level in structural design. It is also seen that for the target reliability index $\beta = 3.04$, the calculated capacity factor for steel is consistently smaller than 0.90 that is specified in AS5100.6 [10].

4.4.2. Inverse reliability analysis

The minimum number of test data for steel yield strength was calculated such that it achieves the capacity factor of 0.90 for steel when the factors for both concrete and shear connectors are all fixed to be 0.80.

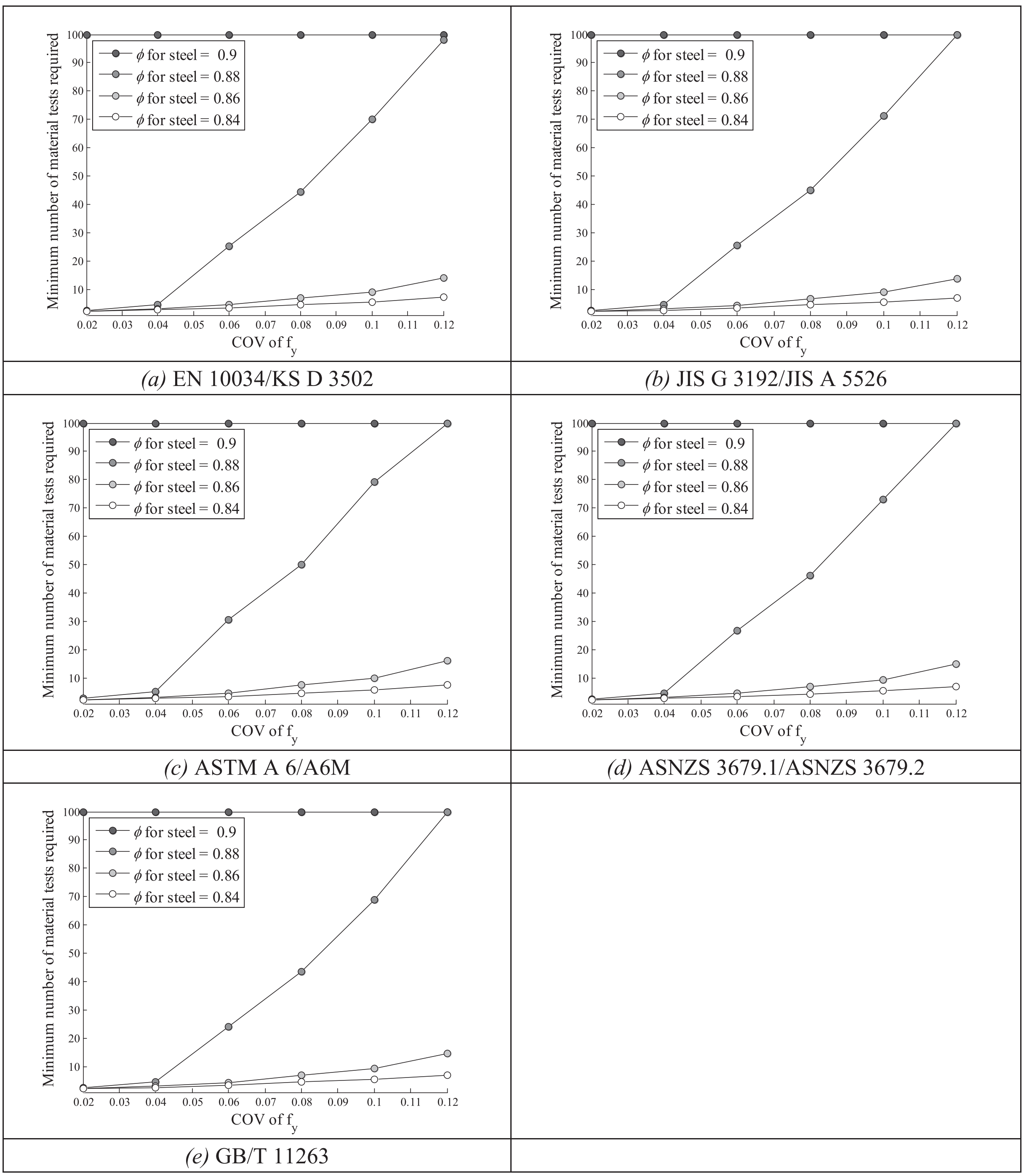


Fig. 20. The required minimum number of yield strength tests for composite beams.

The inverse calculation procedure proposed in Section 3.2 was used. The calculation is repeated for five different types of manufacturing tolerances given in EN 10034 [28]/KS D 3502 [29], JIS G 3192 [30]/JIS A 5526 [31], ASTM A 6/A6M [32], AS/NZS 3679.1 [33], AS/NZS 3679.2 [34], and GB/T 11263 [14].

The calculation results are provided in Fig. 20 where each sub figure corresponds to each of the manufacturing tolerances in EN 10034 [28]/KS D 3502 [29], JIS G 3192 [30]/JIS A 5526 [31], ASTM A 6/A6M [32], AS/NZS 3679.1 [33], AS/NZS 3679.2 [34], and GB/T 11263 [14]. The results show the required minimum number of steel yield tests against the COV of the steel yield strength obtained from the finite number of samples when the capacity factors for concrete and shear connectors are both 0.80. Since the forward analysis in the previous section showed that the capacity factor for steel is 0.90 does not meet the target reliability level even with an infinite number of material tests. Therefore, the capacity factor for steel is varied from 0.90 to 0.84, and the required number of material tests is calculated according to such variations. The results show that the required number of material tests is significantly reduced as the capacity factor for steel decreases. When the capacity factor for steel is 0.90, an infinite number of material tests cannot meet the target reliability level, but when the factor for steel is 0.84, from 2 to 8 material tests are required according to the sample COV of steel yield strength. It is also seen that the manufacturing tolerance requirements have almost no impact on the results.

5. Conclusions

This study evaluated the performance of the design equations given in the Australasian steel and composite design standards AS/NZS 5100.6 [53], AS 4100 [35], NZS 3404.1 [11] and AS/NZS 2327 [52], when the coefficient of variation of steel yield strength was unknown, and there was only a small amount of test data. A capacity factor calibration method was developed when there were infinite or a limited number of material property test data, and an inverse procedure was also developed to calculate the minimum number of material property test data required to achieve the target reliability level for a given capacity factor.

The proposed methods were applied to structural members including I-beams, hollow section columns, CFST columns, and composite beams, the following results were obtained: (i) capacity factors for various COV values of steel yield strength; and (ii) The minimum number of material tests required to achieve the target reliability.

From the reliability analysis, it was found that the design equations for I beams in AS/NZS 5100.6 [53], AS 4100 [35], and NZS 3404.1 [11] provided safety for a various range of the COV of steel yield strength, and the number of 7–20 material tests were required for the COV of steel yield strength of 7–10%. However, the reliability of hollow section columns was highly depended on the COV of section thickness, and the current design standards are unconservative by not delivering the targeted reliability level. In this case two options are available to remedy this situation: the capacity factors need to be reduced (for partial factor design standards, such as the Eurocodes, it is difficult to see how $\gamma_{M0} = 1.0$ is justified); or the manufacturing tolerances for hollow section product standards need to be significantly tightened (the results from this study, suggest that the thickness tolerance should reduce from 10% to 1% of the section thickness). CFST columns achieved the targeted safety level when the number of 2–15 material tests were available for the COV of steel yield strength of 7–10%. Composite beams could not achieve the targeted safety level when the capacity factor for steel is 0.90, and to meet the safety level that can be achieved by the capacity factor for steel = 0.84–0.86, the number of 5–10 material tests were required for the COV of steel yield strength of 7–10%.

For all the structural member types, it was consistently observed that, for the different manufacturing tolerances specified in different countries' standards, the reliability analysis results were almost identical, suggesting that the capacity factors were insensitive to cross-sectional geometrical tolerances. It should be noted that, although these studies have been undertaken for the purposes of the Australian/New Zealand standards, the results have important ramifications for international codes of practice in constructional steel.

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