

International Codes in the Prediction of Load-Bearing Capacity of Slender Columns



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Abstract The bearing-buckling capacity of slender columns is carefully addressed in modern codes by different analytical methods and non-linear advanced numerical methods for structural analysis. This work summarizes an experimental campaign and the safety formats of international codes available for the prediction of slender columns load-bearing capacity. This work is motivated on a round-robin investigation considering numerical simulations supported on non-linear material models and second-order effects (one of the methods suggested in the Eurocode), that proved an expressive overestimation of the load capacity of single slender columns when comparing the numerical results with the experimental ones. Consequently, a high interest in the safety formats of international codes arises, aiming the comparison of the predictions provided by different methods described in design codes from Canada, China, Japan and USA with the European one (namely, the nominal curvature method), as well as the identification of strengths and weaknesses of such methods.

Keywords Slender columns · International safety formats · Load-bearing capacity · Buckling capacity · Round-robin tests

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1 Introduction

The Eurocode is a semi-probabilistic based design code that takes into account geometric, material, model and load uncertainties, by means of partial safety factors. Thus, the reliability of the structures designed accordingly is assured by partial safety factors determined based on reliability classes, characterized by a target probability of failure or reliability index and expected levels of consequences given the violation of a certain limit states [1]. The partial safety factors are usually separated into two main groups, one for the material strength and the other one for load effects. The material strength is reduced by a partial safety factor γ_M and load effects are amplified by a partial safety factor γ_F . The design of slender columns according to Eurocode can be achieved by three different methods, aiming the quantification of the buckling load, namely, the general method non-linear second-order analysis, the nominal stiffness and the nominal curvature method [2]. The buckling load of slender compressed elements must be determined since the failure of such an element can occur before the material strength is reached in its critical cross-section. Hence, a partial safety factor for buckling failure is a must for the assurance of adequate reliability levels [3].

The design of slender columns is still nowadays a matter of controversy because of known contradictions of the design concepts. Concerning the non-linear method suggested in EN 1992-1-1 [2], there is still a need for research given the overestimation of the buckling load by such method as reported in the preceding paper [4] of the paper series that this paper is part of. The work developed in Strauss [4] is focussed in a collaborative round-robin investigation on numerical simulation aiming the prediction of the bulking load of a single slender element with different finite element software and engineers from several countries of Europe. The accuracy of the predictions performed according to non-linear methods is afterwards assessed according to the laboratory experimental results.

The IABSE Commission 1—Task Group TG1.4 was inspired to participate in the investigation due to the following reasons:

- Recently engineers have shown some interest in the non-linear analysis given the method ability to better duplicate real structural behaviour and enhance the efficiency of material used for the overall safety of the structure. In times that sustainability is significantly addressed such method can prove very useful.
- Nevertheless, the non-linear method demands some extra efforts from the users since it requires careful selection of the appropriate incremental and iterative solution procedure and more complex material and geometrical models.
- The assessment of existing structures and design of new structures can be further improved through probabilistic methods by direct consideration of uncertainties (e.g. material, model, geometric and modeling uncertainties) into the structural non-linear analysis and response prediction, addressing also failure modes sensitivity to such uncertainties.

Committed to addressing the previously enumerated concerns, the IABSE Commission 1 has documented some findings in [4] concerning different approaches available i.e. analytical methods and non-linear finite element methods. Furthermore, high interest in international safety formats and their differences arise. Hence, safety formats from Canada, China, Europe, Japan and the USA, are addressed in this second paper of the IABSE paper series concerning carrying capacity of isolated slender columns.

2 Experimental Investigation

To validate and assess the accuracy of the analytical and non-linear finite element models (NLFEM) outputs, an experimental campaign was conducted aiming the quantification of the slender column true load-carrying capacity. The slender column geometry (cross-section and height) as well as its reinforcement layout (see Fig. 1),

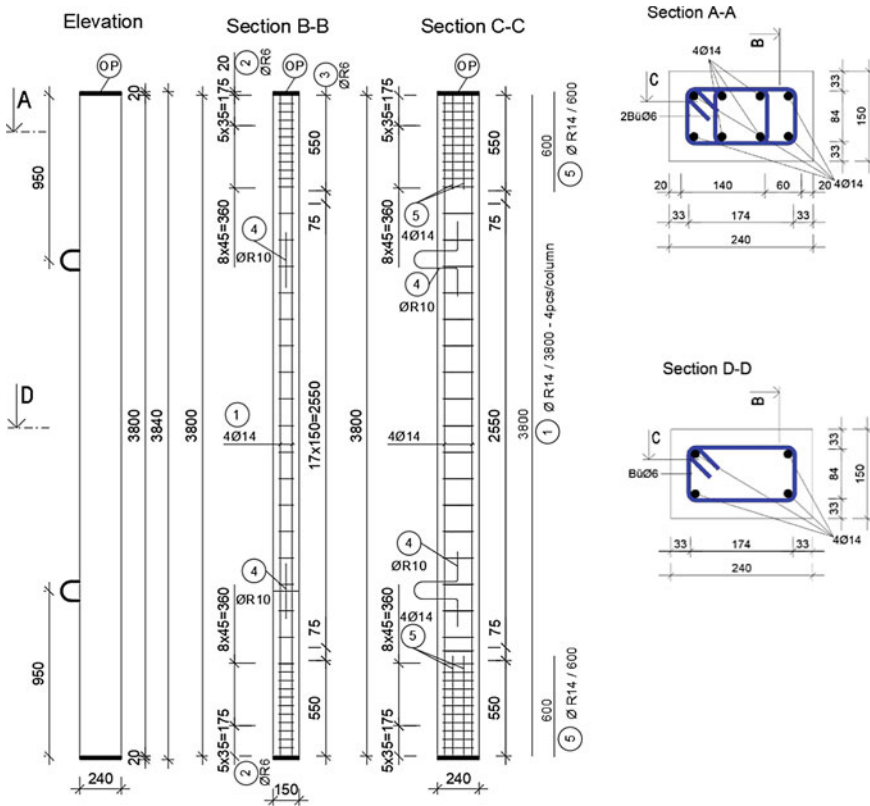


Fig. 1 Investigated slender column [4]

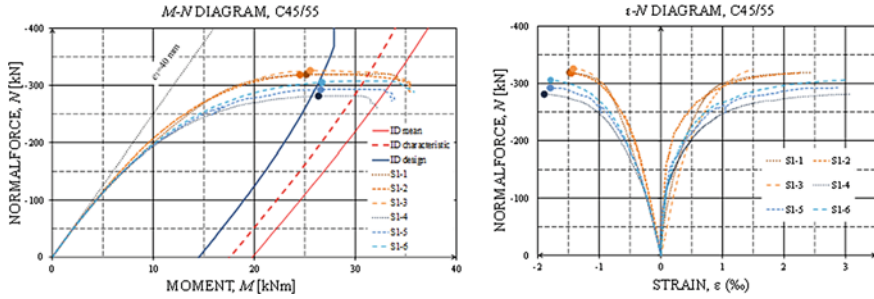


Fig. 2 Experimental results of the specimens S1-1 to S1-6 in the fracture-prone cross-section at the middle height of the column [4]

was carefully designed so it could fail due the loss of stability considering an initial eccentricity of 40 mm for the axial force applied. The column was designed considering a C45/55 concrete and B 500B steel reinforcement. A total number of 6 columns (see [4] for more details on NLFEM prediction) were tested in the laboratory. The column geometry and further information are carefully detailed in Fig. 1.

The experimental results are here displayed in Fig. 2 through two curves. The figure on the left side displays the axial force variation versus the moment generated by the force due to the initial eccentricity and second-order effects or, in other words, geometric non-linearity; and the other shows how the axial force plus the bending moment generated by second other deformation is increasing the compressive and tensile strain in the fracture-prone cross-section at the middle height of the column. The blue and orange dots in Fig. 2 features the verified maximum axial force of each column i.e. the column load-carrying capacity N_{max} . The failure of the column by stability loss is evident since the failure occurs before the N-M interaction threshold curve given by the EN1992-1-1. Three N-M interaction curve as displayed in Fig. 2 based on design, characteristic and mean values. Given the nature of the experimental campaign, the N-M interaction curve given by mean values should be considered as the reference to determine if a stability failure or not have taken place. Additionally, the documented compressive strain was somewhere between 1.4 and 1.8‰, below the maximum compressive strain allowed by the Eurocode i.e. 3.5‰. The results are also summarized in Table 1 for each of the specimens of the experimental campaign, namely, the maximum axial force (N_{max}) as well as the corresponding second-order eccentricity in the critical cross-section (e_2), and the associated bending moment (M_{max}). Some statistics regarding the results are also summarized.

Employing the method of estimation of a coefficient of variation of resistance, also known as the ECOV method, according to [5], the slender column resistance uncertainty can be summarized by a global resistance factor γ_R given by Eq. 1. Here, the resistance of the column is modelled by a lognormal probabilistic distribution. The ECOV method is summarized in Eqs. 1, 2 and 3:

$$\gamma_R = \exp(\alpha_R \beta V_R) \tag{1}$$

Table 1 Statistic of the experiments

Test	N_{max} (kN)	e_2 (mm)	M_{max} (kNm)
S1-1	324.4	57.6	31.7
S1-2	323.4	42.7	26.8
S1-3	332.6	38.3	26.0
S1-4	271.2	58.4	26.7
S1-5	296.0	59.4	29.4
S1-6	311.4	55.0	29.6
S1-1 bis S1-3	326.8 (0.02)	46.2(0.22)	28.2 (0.11)
S1-4 bis S1-6	292.9 (0.07)	57.6 (0.04)	28.6 (0.06)
S1-1 bis S1-6	309.8 (0.07)	51.9 (0.17)	28.4 (0.08)

where:

α_R is a sensitivity factor for the reliability of resistance.

β is the target reliability index.

V_R is the coefficient of variation of resistance determined according to Eq. 2:

$$V_R = \frac{1}{1.65} \ln \frac{R_m}{R_k} \tag{2}$$

where:

R_m is the mean resistance value.

R_k is the characteristic value of resistance correspond to 95% exceedance probability (or 5% fractile).

Considering a target reliability index of 3.8 ($P_f \approx 10^{-4}$) for a 50-year reference period and a sensitivity factor of 0.8 the global resistance factor of the column maximum axial force should be 1.24 and 1.28 for the corresponding bending moment (M_{Nd}). If a well-validated mathematical model (Analytical or numerical) is used for the design, a partial factor of 1.06 targeting the model uncertainty in resistance (a generous value) can be used, aiming to cover the gap between the model (not the geometric and material uncertainty) and the real slender column performance. Thus, the design buckling axial force can be computed according to Eq. 3. In summary, for a reliable design of the slender column, the maximum axial load that the column should be exposed to is 227.98 kN, in order to comply with a 3.8 target reliability index.

$$N_d = \frac{N_m}{\gamma_{Rd}\gamma_R} = 227.98 \text{ kN and } M_d = \frac{M_m}{\gamma_{Rd}\gamma_R} = 20.89 \text{ kNm} \tag{3}$$

3 European Design Format

The European standardized rules for the design of engineering structures are summarized in the Eurocode. The Eurocode was developed by the European Committee for Standardization (CEN) which is constituted by several researchers, engineers, and users from different member states. In Germany, the Eurocode was introduced by the authorities since July 2012 as applicable law, as well as in Austria with their national annexe since July 2009.

As mentioned in the introduction, the Eurocode provides three methods for the design of slender columns, that is [2]: (i) a general method based on non-linear analysis considering geometric nonlinearity of the structure i.e. second-order effects and material nonlinearity; (ii) the method based on nominal stiffness where the second-order analysis is based on stiffness, nominal values of the flexural stiffness, considering cracking, creep and material non-linearity on the performance of the column. This method also allows the consideration of the stiffness of adjacent members such as beams, slabs or foundations. Soil structure interaction should be considered when relevant. Here the total design considering the second-order effect is obtained by a moment magnification factor applied to the moment obtained from the first-order analysis. (iii) the method based on nominal curvature which is mostly suitable for isolated elements with constant axial force and a defined effective length l_0 conditioned by the column supports and other factors. Here the nominal second-order moment is dependent on the column deflection which turns to be dependent on the effective length and an estimated maximum curvature.

In Fig. 3 is summarized the basics of the Eurocode’s nominal stiffness and nominal curvature method based on the M–N interaction threshold of the cross-section capacity considering the second-order effects by means of a magnification factor or rotation. Aiming the assurance of failure by material damage and not stability loss, the buckling load (N_B) must be greater than the cross-section resistance (N_{Rd}).

According to the nominal stiffness method, the eccentrically compressed column ultimate bearing capacity is $N_d = 205$ kN, below the design value (227.98 kN) computed according to the global resistance factor obtained from the ECOV method

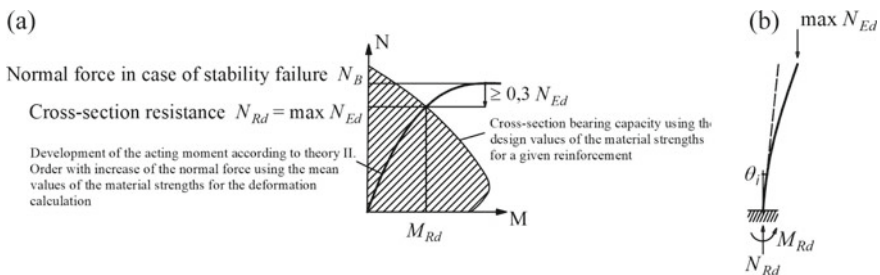


Fig. 3 The Eurocode’s ‘nominal stiffness’ and the ‘nominal curvature’ approach basics: **a** based on M–N interaction threshold diagram of the cross-section capacity, **b** including second-order moment effects, using magnification factor ψ_l or (θ_i) , [6]

applied to the experimental results in Eq. 3 (see Sect. 3). The nominal curvature method, however, provided an ultimate bearing capacity of $N_d = 222$ kN ($M_{Nd} = 24.28$ kNm), closer to the design value given by the ECOV method. For practical applications, the nominal stiffness method may be used for both isolated members and whole structures if the nominal stiffness of the adjacent elements are estimated properly since it takes into account the stiffness contribution of other elements connected to the main element. On the other hand, the nominal curvature method does not consider such information and is considered mainly suitable for isolated elements. Thus, here the nominal curvature proves again to be most suited for isolated elements. However, when efficiency is concerned, which by the way is the aim of engineering activities, the nominal curvature method is the approach one should consider for isolated elements.

4 American Design Format

4.1 USA Design Format

The American Concrete Institute (ACI) in ACI 318-14 [7] describes three methods to assess or predict the carrying capacity of slender columns taking into account second orders deformations, namely: (a) nonlinear second-order analysis: the method should take into account material a geometric non-linearity, lateral drifting, creep and shrinkage. Considered as the optimum approach, but the computational complexity turns it not very attractive; (b) elastic second-order analysis: this method considers the cracks down the length of the column and creep effects, and; (c) moment magnification procedure: here the first-order elastic effects are quantified and then scaled to consider second-order effects. The member stiffness and cracking effects are quantified through a set of equations based on studies of Macgregor et al. [8].

To maintain reliable design levels ACI employee's resistance and load factor accounting for uncertainties of simplification assumptions, material properties variation and demanded reliability levels. An N-M interaction diagram is usually generated and divided into two regions where two different safety factors are considered. For potential brittle failure (above the balance point) a 0.65 factor is used and for ductile failure mechanism (below the balance point) a 0.90 factor is considered.

Considering the moment magnification factor, the design capacity of the slender column was estimated to be $N_d = 280.00$ kN for a correspondent moment of $M_{Nd} = 17.5$ kNm.

4.2 *Canada Design Format*

The Canadian standards approach for the design of reinforced concrete slender columns as several resemblances to ACI 318-14, considering column slenderness ratio, interaction diagrams, second-order effects through a moment magnification factor. Thus, the obtained design axial force is $N_d = 280.00$ kN for a correspondent moment of $M_{Nd} = 16.6$ kNm. The Canadian design format can be found detailed reported in a handbook from the Cement Association of Canada [9].

5 *Asia Design Format*

5.1 *China Design Format*

Concerning design and prediction of carrying capacity of eccentric compressed components, an investigation was initiated in 1980 by the Ministry of Housing and Urban-Rural Development of the People's Republic of China. The research was supported by experience and theoretical formulations on eccentric compressed elements from Germany, USA and the former Soviet Union [10]. Several experimental studies were carried out, nevertheless, the investigation was only restricted to material damage considering second-order effects, disregarding true instability problems. Furthermore, the investigation considered beam and columns of ordinary houses, generating several uncertainties or unexpected behaviour with regarding columns of big size. Therefore, later on in Specifications for Design of Highway Reinforced Concrete and Prestressed Concrete Bridges and Culverts [11] a method considering the P- Δ effect is provided, introducing the instability of slender columns. The concept described in such specification is similar to the one implemented in ACI318-08 [12], but the limit curvature expression still adopts the Chinese customary. Here the eccentricity of unfavourable direction is considered.

The Chinese approach can be summarized into the following steps: initially, the compression bearing capacity of the column is quantified, followed by the consideration of eccentricity effects. The height of the pressure zone is computed to determine if the section is under small eccentricity or large eccentricity followed by the quantification of a bending moment increasing coefficient to take into account secondary effects and stiffness of the element. Finally, the column ultimate bearing capacity is obtained, and the column's performance is identified in an N-M interaction diagram to highlight its theoretical performance. According to china design format the column capacity is summarized in $N_d = 282.32$ kN and $M_{Nd} = 13.09$ kNm.

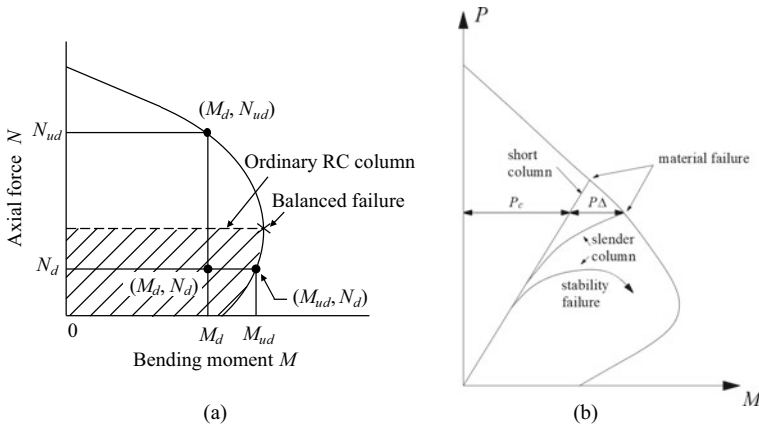


Fig. 4 **a** N - M interaction diagram (adopted from JRA [13]. **b** N - M interaction diagram of short and slender columns

5.2 Japan Design Format

In Japan, the research on eccentrically compressed columns in civil engineering structures focused mainly on the bending performance of short column (see Fig. 4b), in opposition to the extensive studies on slender columns carried out by countries where strong earthquakes are not so frequent as in Japan. The effects of slenderness ratio on the carrying capacity of reinforced concrete columns are not considered in japan since it is demanded that every highway bridge in japan must be designed taking into consideration earthquake forces of high intensity. Thus, the shear span ratio of reinforced concrete columns allowed is lesser than the acceptable in non-susceptible earthquake regions. In other words, the slender columns analysed here in this paper is not in agreement with safety formats of the Japan Road Association (JRA). Nevertheless, the carrying capacity of a short-column can be computed based on the cross-sectional resistance design procedure given in JRA [13]. Here, the N - M interaction threshold displayed in Fig. 4a is used as the flexural and axial capacity of the column cross-section. Accordingly, the required flexural (M_d) and axial (N_d) capacity due to the loading conditions must be beneath the balanced failure point and inside the hatched area so the failure can be ductile.

6 Conclusions and Final Discussion

The design formats from each country were further investigated considering geometric and material uncertainties, disregarding safety factors, to produce the resistance probabilistic distribution given a random consideration of such uncertainties. The column resistance is later on modelled by a lognormal probabilistic distribution.

The ECOV method (as described in Sect. 2) is also employed to compute the global resistance factor that is used as a measure of the resistance deviation due to material and geometric uncertainties. The safety formats outputs are also plotted against the experimental result to provide a clear view of the agreement and/or disagreement of the safety formats and the real performance of the slender column (experiment). The results are summarized in Fig. 5.

The European design method produced a histogram of the axial resistance of column close to the experimental results with a mean value of 300 kN and standard deviation of 16.4 kN in opposition to 309.8 and 21.68 kN produced by the experimental results, respectively. According to the ECOV method, one produced a global resistance factor equal to 1.18 (Eurocode) and the other 1.24 (Experiment). In summary, the nominal curvature method exhibits minor influence from material and geometric uncertainties, when compared to experimental results. Concerning the mean axial resistance, a deviation of $[-3.1\%]$ is observed with respect to the experimental results. On the other hand, the mean bending moment caused by the maximum mean normal force given geometric imperfections and second-order deformations exhibits a deviation of $[+15.5\%]$ with respect to the experimental results.

The European design format considered here (i.e. nominal curvature), according to Fig. 5, match more or less the experimental results and provides the lowest global safety factor according to ECOV method, while the others safety formats from other continents clearly overestimate the column axial force required for stability failure. Nevertheless, the general method based on non-linear analysis also suggested in the Eurocode [2] leads to the overestimation of the load-bearing capacity of slender columns, as demonstrated in [4].

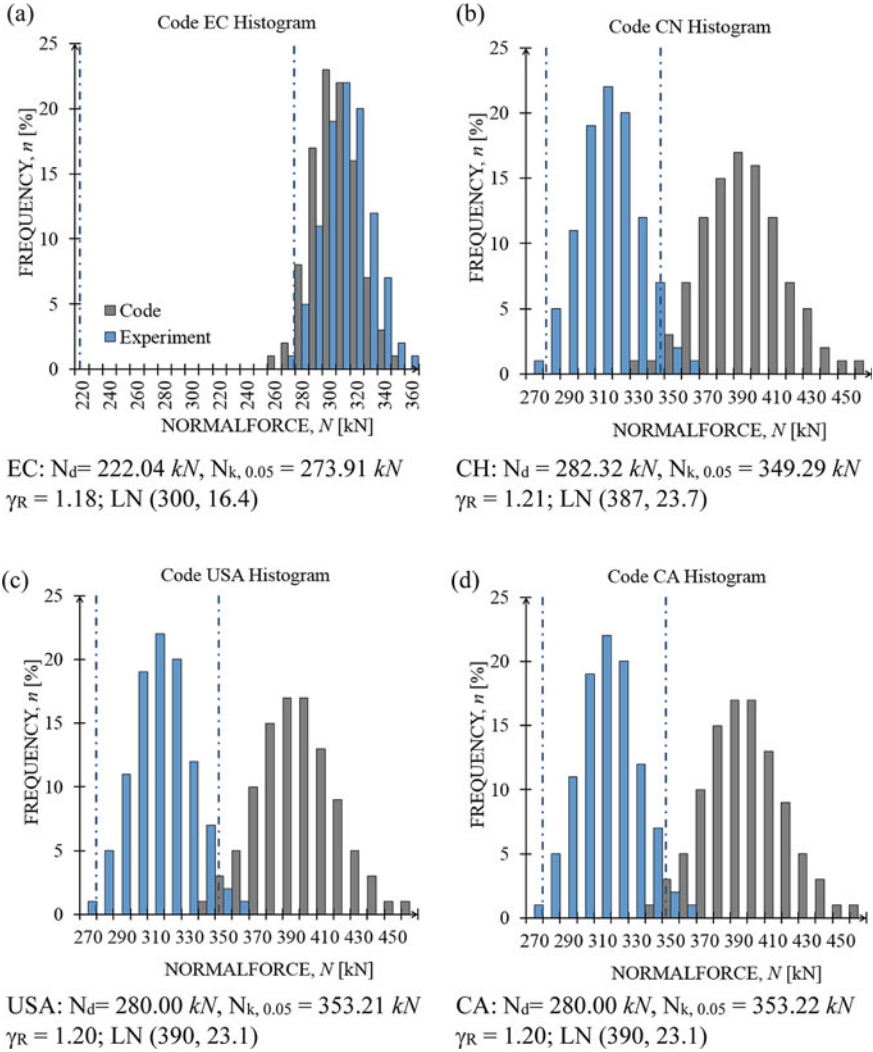


Fig. 5 International design format sensitivity to geometric and material parameters uncertainties

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