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Application of Eurocode 4 Design Provisions to High Strength Composite Columns

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Summary

Concrete-filled steel tubular (CFST) columns offer significant structural and economic benefits in a wide variety of applications in the construction industry. This paper examines the applicability of the EC4 simplified method of design to CFST columns which use high strength concrete. Measured column strengths reported in the literature are compared with those predicted by EC4 and, on the basis of the 146 columns analysed, conclusions are drawn on the suitability of the present EC4 provisions to high strength CFST columns.

1. Introduction

Concrete-filled steel tubular (CFST) columns have increased in popularity as the significant number of advantages they offer in both design and construction have come to be appreciated. In recent years high strength concrete, in excess of 60 MPa, has become commercially available for use in construction. If the economic benefits of this material are to be fully realised, then the provisions of design codes must be extended to include high strength concrete. Eurocode 4 (EC4) is, arguably, the pre-eminent code in the world for the design of CFST columns and the aim of this paper is to examine the applicability of the EC4 simplified method of design to CFST columns which use high strength concrete.

2. EC4 Strength Criteria for CFST Columns

In the usual design situation for a member subjected to combined compression and uniaxial bending, the design action effects at the ends of the column N_{Sd} and M_{Sd} are generally known from a structural analysis, and it is then required to find a cross-section size which satisfies the strength criteria given in Eq. (1) and (2).

$$N_{Sd} \leq \chi N_{pl.Rd} \tag{1}$$

$$M_{max.Sd} \leq 0.9 \mu M_{pl.Rd} \tag{2}$$

The maximum moment that the column must support - $M_{max.Sd}$ - may be either at the end of the column or somewhere along its length, and provisions are available (EC4 1992; Bergmann et al 1995) to estimate this moment.

In experiments, however, columns of known length and section size are tested to determine the maximum force $N_{max.meas}$ that they can support. This force is applied either concentrically or eccentrically to the longitudinal axis of the column. Consequently, the analysis of this data requires an approach opposite to that of the usual design situation. Two variations of this approach are possible. Firstly, use the maximum measured force $N_{max.meas}$ to generate a predicted value of the maximum moment along the column and compare this with the measured value. Secondly, use some analytical method to calculate the maximum moment along the column (as a function of the applied force) and predict the corresponding maximum force that the column can sustain. The first method requires a knowledge of the position of the critical section where the bending moment will be largest and measurements of deflection taken at this point. This is possible only in simple cases such as equal eccentricities of force but even so, in many cases, this data is either not measured or reported. This effectively precludes the first method. Because of this, the second method was adopted here, and this has the added benefit of more closely reflecting the process used in design. In the context of the EC4 design procedure and notation, the load N_{Sd} effectively equates to the predicted maximum eccentric force $N_{max.pred}$ under the action of a co-existing moment at the end of the column $M_{Sd} = N_{Sd} \times e$. Because of the complicated inter-relationships between applied force and moment and the interaction curve of the cross-section, the process becomes iterative, eventually converging upon the predicted column strength $N_{max.pred}$.

3. Method of Analysis

In order to allow the analysis to proceed, it is necessary to know the following data:

(i) dimensions of the steel tube, b and h or D, and wall thickness t

(ii) yield stress f_y and modulus of elasticity E_a of the steel tube

(iii) concrete cylinder strength f_{cyl}

(iv) column length L, and

(v) eccentricities of the applied load at the top e_t and bottom e_b of the column

Steps in the analytical procedure are as follows:

1. Check the cross-section for local buckling.

   For a rectangular section: $h / t \leq 52\,\varepsilon$

   For a circular section: $D / t \leq 90\,\varepsilon^2$

   where $\varepsilon = \sqrt{235 / f_y}$ with f_y in units of MPa

2. Set the material partial safety factors to unity.

   Hence $f_{yd} = f_y / \gamma_{Ma} = f_y / 1.0$

   and $f_{cd} = f_{ck} / \gamma_c = f_{cyl} / 1.0$

3. Calculate the Euler buckling load N_{cr} for the column.

$$N_{cr} = \frac{\pi^2 (EI)_e}{L^2} \tag{3}$$

where $(EI)_e = E_a I_a + 0.8 \frac{E_{cm}}{1.35} I_c$

$E_{cm} = 9\,500\,(f_{cyl} + 8)^{1/3}$ MPa, the secant modulus of the concrete from Eurocode 2, for f_{cyl} in MPa

I_a and I_c are the second moments of area of the steel tube and uncracked concrete respectively

and L is the buckling length of the column. In the present study, this was taken to be the column length quoted in the literature.

4. Compute the relative column slenderness $\bar{\lambda}$.

$$\bar{\lambda} = \sqrt{\frac{N_{pl.R}}{N_{cr}}} \tag{4}$$

where $N_{pl.R} = A_a f_{yd} + A_c f_{cd}$

and A_a and A_c are the cross-sectional areas of the steel tube and concrete respectively.

5. Calculate the plastic resistance of the cross-section $N_{pl.Rd}$ to axially applied force.

For a rectangular cross-section:

$$N_{pl.Rd} = A_a f_{yd} + A_c f_{cd} \tag{5}$$

For a circular cross-section in which the effect of confinement may be included:

$$N_{pl.Rd} = A_a f_{yd} \eta_2 + A_c f_{cd} \left(1 + \eta_1 \frac{t}{D} \frac{f_y}{f_{cyl}}\right) \tag{6}$$

where, if both the relative slenderness $\bar{\lambda} \leq 0.5$ and the eccentricity ratio $e / D \leq 0.1$,

$$\eta_1 = \eta_{10}\left(1 - \frac{10e}{D}\right)$$

$$\eta_2 = \eta_{20} + (1 - \eta_{20})\frac{10e}{D}$$

$$\eta_{10} = 4.9 - 18.5\bar{\lambda} + 17\bar{\lambda}^2 \quad \text{but} \not< 0.0$$

$$\eta_{20} = 0.25(3 + 2\bar{\lambda}) \quad \text{but} \not> 1.0$$

otherwise, $\eta_1 = 0$ and $\eta_2 = 1$

6. Check whether the column is defined to be a composite column.

The ratio δ must lie within the range $0.2 \leq \delta \leq 0.9$

where $\delta = \frac{A_a f_{yd}}{N_{pl.Rd}}$

7. Calculate the reduction factor under axial load using the buckling curve "a".

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \tag{7}$$

where $\Phi = 0.5\left[1 + 0.21(\bar{\lambda} - 0.2) + \bar{\lambda}^2\right]$

8. Select a trial value of $N_{max.pred}$ ($\equiv N_{Sd}$) taken to be the predicted strength of the column.

9. Calculate the associated (predicted) maximum bending moment along the column $M_{max.Sd}$ using the proposal of Bergmann et al (1995).

$$\text{If} \quad N_{max.pred}/N_{cr} \leq \left(\frac{\cos^{-1} r}{\pi}\right)^2 \quad \text{then} \quad M_{max.Sd} = M_{R.Sd} \tag{8}$$

$$\text{otherwise} \quad M_{max.Sd} = \frac{M_{R.Sd}}{\sin \varepsilon}\sqrt{r^2 - 2r\cos\varepsilon + 1} \tag{9}$$

where $M_{R.Sd}$ is the larger of the moments at the end of the column and is equal to $N_{max.pred} \times$ (the larger of the end eccentricities e_t or e_b)

$$\varepsilon = \pi\sqrt{\frac{N_{max.pred}}{N_{cr}}}$$

r = ratio of smaller to larger moments (eccentricities) at the end of the column, positive for single curvature bending $(-1 \leq r \leq +1)$

10. Evaluate the resistance of the section to pure bending, $M_{pl.Rd}$

The resistance of the section under pure bending $M_{pl.Rd}$ was evaluated using a computer programme (Goode 1996) designed to determine the force-moment interaction diagram for the cross-section. (Comparisons between the output from this programme and the interaction curves provided by Bergmann et al (1995) were almost identical.) This analysis assumes a full plastic distribution of stresses in the concrete (f_{cd} in compression, zero in tension) and the steel (f_{yd} in compression and tension). (Alternatively, eq.(27) and (28) given by Bergmann et al (1995) may be used.)

11. Calculate the moment ratio μ required to satisfy the moment criterion.

$$M_{max.Sd} = 0.9\,\mu\,M_{pl.Rd} \tag{10}$$

For eccentrically loaded columns where the bending moment at the end is due solely to the action of the eccentricity of the force, the value of μ may exceed 1.0.

12. Determine the imperfection moment ratio μ_k.

Using the computer programme (Goode 1996), the value of μ_k corresponding to χ on the cross-section interaction curve was determined - Figure 1. This was done by selecting different neutral axis depths until the load ratio was equal to χ; the resulting moment ratio was the required value of μ_k.

13. Evaluate the moment ratio μ_d and force ratio χ_d.

After setting $\chi_n = \chi(1-r)/4$ such that $\chi_d \geq \chi_n \geq 0$, values of μ_d and χ_d were evaluated by an iterative process using the cross-section interaction curve until the following was satisfied:

$$\mu = \mu_d - \mu_k\frac{\chi_d - \chi_n}{\chi - \chi_n} \tag{11}$$

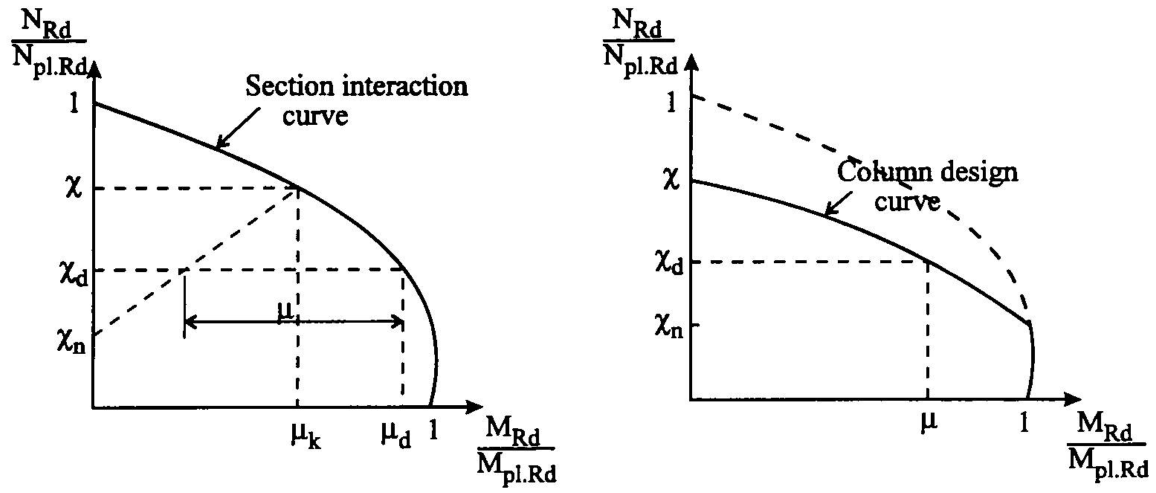

Figure 1 Design for compression and uniaxial bending

14. Compute the new value of predicted column strength $N^*_{max.pred}$.

$$N^*_{max.pred} = \chi_d \times N_{pl.Rd} \tag{12}$$

15. Identify the final predicted column strength $N_{max.pred}$.

Compare the trial value of the force $N_{max.pred}$ initially assumed in step 8 with the value $N^*_{max.pred}$ calculated in step 14. If the two agree to within a prescribed tolerance - a value of 1 kN was selected in the present investigation - the predicted column strength is taken as $N_{max.pred}$. Otherwise select a new trial column strength, by interval halving or some other technique, and return to step 8.

Column strengths predicted by this analysis were then compared with measured data reported in the literature.

4. Analysis of Laboratory Test Data

A total of 146 columns from 6 different investigations were analysed. The concrete cylinder strength ranged from 23 MPa to 103 MPa. Predictions of column strength versus concrete cylinder strength are presented in Fig. 2 which suggests no evidence of any significant effect of the strength of the concrete upon the quality of the column strength prediction. Similarly, no relationship could be detected with the other major parameters of slenderness *L/D*, which varied from 4.0 to 31.6, or eccentricity ratio *r* which varied from +1 to -1 (Kilpatrick and Taylor 1997).

5. Discussion and Conclusions

For the 146 columns analysed, the mean ratio of measured/predicted column strength was 1.10 with a standard deviation of 0.13. EC4 safely predicted the failure load in 73% of the columns analysed. There was no obvious relationship between concrete cylinder strength and the accuracy of the prediction of column strength. Based on this, it is suggested that EC4 can reliably predict the short-term strength of eccentrically loaded slender CFST columns, including those using concrete strengths in the range of 50 to 100 MPa.

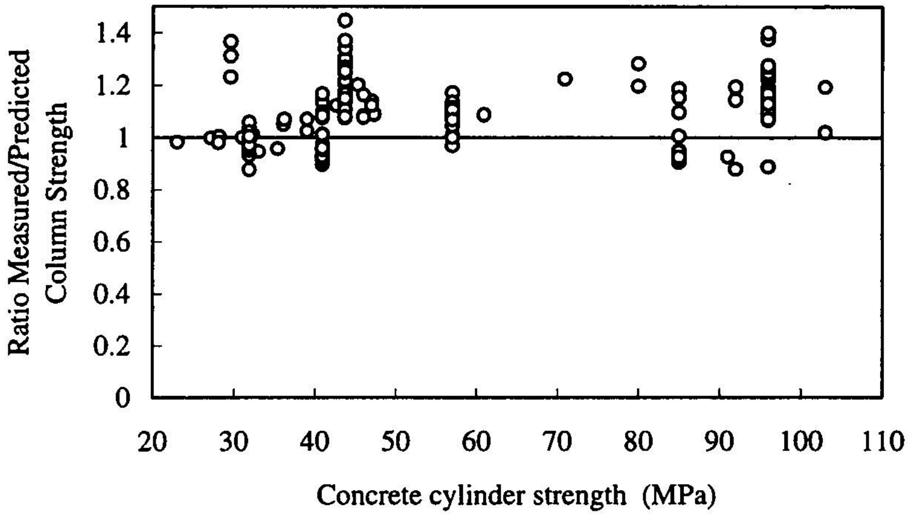

Figure 2 Column strength versus concrete strength

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References

BERGMANN R., MATSUI C., MEINSMA C. and DUTTA D. (1995), 'Design guide for concrete filled hollow section columns under static and seismic loading', CIDECT, Verlag TÜV Rheinland, pp. 68.

EUROCODE 4 (1992), 'Design of composite steel and concrete structures - Part 1.1 General rules and rules for buildings', ENV 1994-1-1:1992, European Committee for Standardization, Brussels.

GOODE C.D. (1996), Computer programme to determine the interaction diagram for a CFST cross-section, Private communication.

KILPATRICK A.E. and TAYLOR T.L. (1997), 'Comparisons Between Measured and Predicted CFST Column Strengths using EC4', Research Report No. CE 97/1, Faculty of Engineering and Surveying, University of Southern Queensland, Australia, Feb.