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Behaviour of High-strength Composite Columns

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Summary

Twenty-five slender high strength concrete-filled steel tubular columns were tested in order to study the influence of eccentricity of force upon their strength and load-deflection response. The eccentricity was varied from that producing single curvature bending through to double curvature bending whilst the length of the column was kept constant. Predictions of column strength and force-deflection response were obtained using a deformation control method of analysis and conclusions are drawn on the basis of comparisons between measured and predicted data.

1. Introduction

Concrete-filled steel tubular (CFST) columns offer a number of advantages in both design and construction. The steel tube (i) acts as permanent formwork for the plastic concrete, (ii) provides well distributed reinforcement in the most efficient position to resist applied bending moments, (iii) confines the hardened concrete which increases its strain capacity and strength, and (iv) protects the surface of the concrete from physical damage and deleterious environmental effects such as carbonation. In turn, the concrete increases the critical buckling stress of the steel tube by changing its buckling mode. Overall, composite column construction enables (i) speed of construction to be improved, (ii) smaller cross-sectional dimensions for a given column strength, (iii) simple connections to steel floor beams which reduces the design time, and (iv) higher impact and seismic resistance.

Test data on the behaviour of CFST columns which incorporate low or medium strength concrete, together with guidelines for their design, are available in the literature, but very little is known about such columns when they are made of high-strength materials. This paper describes the results of 25 tests on high strength circular CFST columns (Kilpatrick and Rangan 1997) and compares these with the predictions of a deformation-control method of analysis.

2. Materials, Specimens and Equipment

All specimens were constructed using commercially available circular hollow steel tube which was manufactured by cold-forming and high frequency electric resistance welding. Tests showed that the 101.5 mm × 2.4 mm ($D_o \times t$) tube had a 0.2% offset tensile strength of 410 MPa and an ultimate tensile strength of 475 MPa.

The 10 mm aggregate high-strength concrete was commercially supplied. Standard tests conducted on ten 100 mm diameter cylinders showed the average unconfined compressive strength to be 96 MPa. In addition, two 150 mm diameter cylinders were tested to determine the modulus of elasticity which was found to be an average of 40 500 MPa.

After facing both ends of the empty steel tube in a lathe, each column was secured in a vertical position and filled with concrete in layers which was in turn compacted by an immersion vibrator. The length of each column was kept constant at 2175 mm centre-to-centre of male knife edges.

Testing of the columns was carried out using a 2500 kN capacity Avery-Denison universal testing machine. Both ends of each column were clamped to specially made hardened knife-edge assemblages which had been previously located on the top and bottom platens of the testing machine. The eccentricity of the applied compressive force was obtained by displacing the end of the column laterally from the axis of the testing machine. During the test simultaneous recordings were made of the applied force and lateral deflections of the columns at mid-height and quarter-height. After the recording system was initialised, the testing machine ram was moved upwards at a steady rate of approximately 1 mm/min until it became apparent that the limit of either the knife-edge assemblages or the LVDT at mid-height was approached.

3. Results

A summary of the test programme and the measured column strengths is given in Table 1.

Col. no.	Force eccentricity top, btm (mm)	Measured strength (kN)	Col. no.	Force eccentricity top, btm (mm)	Measured strength (kN)	Col. no.	Force eccentricity top, btm (mm)	Measured strength (kN)
SC-16	+50,+50	157	SC-25	+40,+10	243	SC-34	+20,0	367
SC-17	+50,+30	183	SC-26	+40,0	260	SC-35	+20,-10	411
SC-18	+50,+20	196	SC-27	+40,-10	281	SC-36	+40,-30	344
SC-19	+50,+10	215	SC-28	+40,-20	331	SC-37	+40,-40	385
SC-20	+50,0	237	SC-29	+30,+20	244	SC-38	0,0	523
SC-21	+50,-10	256	SC-30	+30,0	318	SC-39	+50,-30	303
SC-22	+50,-20	266	SC-31	+30,-10	340	SC-40	+50,-50	344
SC-23	+50,-20	266	SC-32	+30,-20	384			
SC-24	+40,+30	197	SC-33	+20,+20	282			

Column SC-23 was a repeat of column SC-22.

Table 1 Slender column test programme

4. Method of Analysis

A simple iterative displacement-controlled method of analysis, fully described elsewhere (Kilpatrick 1994, 1996), was used to predict the complete short-term force-deformation response of the tested columns. The method requires prior knowledge of the moment-thrust-curvature relationships of the cross-section of the column which can be obtained using the technique described by Warner et al (1989). For eccentrically loaded columns subjected to combined thrust and bending, the analytical procedure is as follows:

1. Discretise the column into n rigid segments of length ΔL with n+1 nodes.
2. Choose a value of the control slope θ_0 applied at node 1 at the bottom of the column
3. Select a trial value of the force N.
4. For the current value of N generate the corresponding moment-curvature relation up to the point at which the maximum moment capacity $M_{i\,max}$ is realised.
5. For all nodes along the column, calculate the deflection v_j of node j from the initial (unloaded) position of the column using Eq. (1), (2) and (3).

$$v_j = v_{j-1} + \Delta v_j + \Delta\theta_{j-1}\,\Delta L \quad (1)$$

$$\Delta\theta_{j-1} = \kappa_{j-1}\,\Delta L \quad (2)$$

$$\Delta v_j = v_{j-1} - v_{j-2} \quad (3)$$

In using Eq. (2) the curvature κ_{j-1} at the previous node j-1 is obtained from the moment-thrust-curvature relationship by satisfying the equilibrium requirement that the (internal) moment capacity of the cross-section must equal the (external) moment $M_{e\,j-1}$ given by

$$M_{e\,j-1} = N(e_{j-1} - v^{i}_{j-1} - v_{j-1}) \quad (4)$$

The moment-thrust-curvature relationship which must be used depends upon whether the bending moment at the node has increased or decreased compared with the previous successful solution. If the bending moment has increased then the moment-curvature relation generated previously in step 4 is used. However, if the bending moment has decreased then it is assumed that the section follows an unloading line whose slope is equal to that of the initial (zero load) tangent stiffness of the moment-force-curvature relationship of the previous successful solution for the node.

6. Check the misclose at node n+1 at the top of the column. If the magnitude of the force N is precisely correct then the displacement v_{n+1} will be zero, but rounding off and approximation errors will make this event unlikely. It is therefore usual to prescribe a tolerance to the error which is permissible and a value of ±0.5% of the largest deflection along the column was adopted in the present study. Data on the deflected shape and distributions of moments and curvatures may be printed. A new increased value of the control slope θ_0 is selected and the process returns to step 2. If v_{n+1} is not within the prescribed tolerance limit, a new value of N must be selected and the process returns to step 4. If, for the current value of the control slope, the process will not converge after a specified number of trials then it should be stopped because the solution lies on or very close to the section capacity line. Plots of the section capacity line and the force N versus its total eccentricity at the critical section may then be constructed.

5. Analysis of Test Data

A modified version of the stress-strain relationship for concrete confined by a steel tube proposed by Tomii (1991) was used for the concrete in compression: the tensile strength of the concrete was neglected. The curvilinear stress-strain relationship for the steel in both tension and compression was approximated by seven linear segments. Complete details of the material properties are provided elsewhere (Kilpatrick 1996, Kilpatrick and Rangan 1997). Each column was discretised into segments whose length was approximately equal to $D_0/2$. To simulate the (stiffening) effect of the knife edge/clamp assemblages at each end of the column, the nodes in these portions were assumed to possess an infinitely stiff moment-curvature relationship. Predictions of the strength of the eccentrically loaded columns are given in Table 2.

Col. no.	Column strength (kN) Measured	Predicted	Measured/ Predicted	Col. no.	Column strength (kN) Measured	Predicted	Measured/ Predicted
SC-16	157	154	1.019	SC-29	244	247	0.988
SC-17	183	180	1.017	SC-30	318	312	1.019
SC-18	196	196	1.000	SC-31	340	354	0.961
SC-19	215	213	1.009	SC-32	384	406	0.946
SC-20	237	232	1.022	SC-33	282	280	1.001
SC-21	256	252	1.016	SC-34	367	371	0.989
SC-22	266	274	0.971	SC-35	411	436	0.943
SC-23	266	274	0.971	SC-36	344	366	0.940
SC-24	197	199	0.990	SC-37	385	437	0.881
SC-25	243	241	1.008	SC-38	523	-	-
SC-26	260	267	0.974	SC-39	303	298	1.017
SC-27	281	296	0.949	SC-40	344	369	0.932
SC-28	331	328	1.009		*Col. SC-38 concentrically loaded*		

Table 2 Predictions of column strength

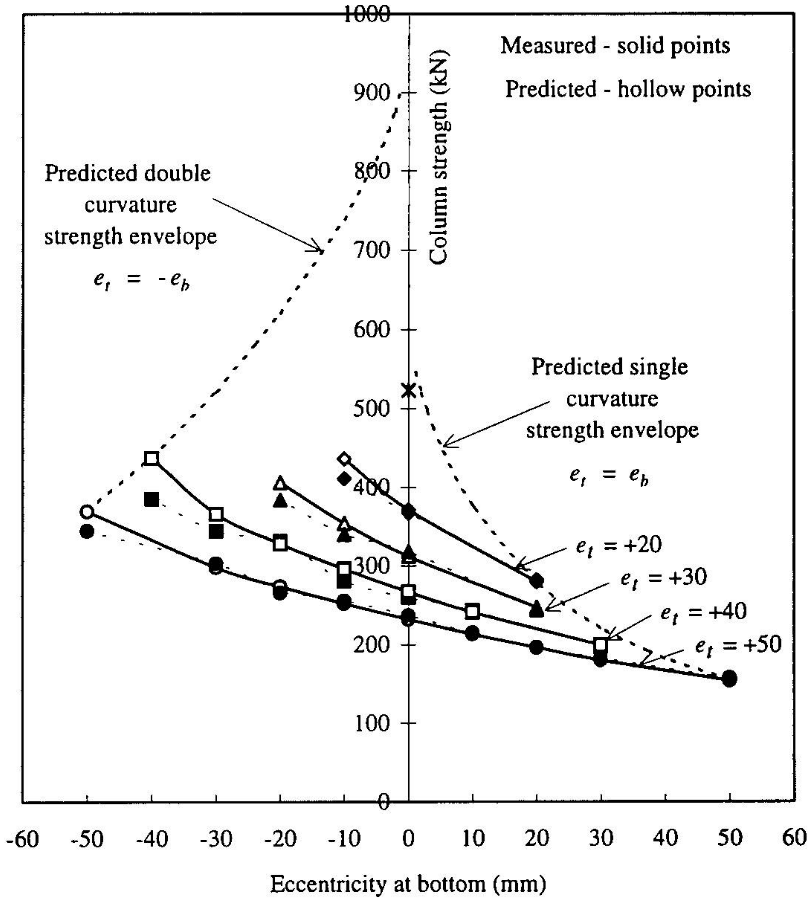

Figure 1 Measured and predicted column strengths

6. Discussion and Conclusions

6.1 Column strength

The mean value of the ratio of measured/predicted column strengths given in Table 2 of 0.982 and standard deviation of 0.036 indicate excellent correlation. Figure 1 presents an interesting pictorial comparison between measured and predicted column strengths. Predicted strengths are shown for contours of constant eccentricity at the top of the column, with values of eccentricity at the bottom varying between equal to the top eccentricity and equal and opposite to the top eccentricity. Measured and predicted strengths generally compare better for cases of single curvature bending than for double curvature bending. Also shown in Fig.1 are the predicted strength envelopes which join the limiting values of eccentricity. The envelope of equal eccentricity top and bottom shows a rapid increase in column strength towards an estimated 580 kN under axial compression. A similar trend is evident in the envelope of strength for equal and opposite eccentricity of force. Although the type of the curve is similar to that of the other envelope, the rate of increase is greater. Moreover, the envelope approaches a column strength under concentric loading of about 930 kN. This limit is appreciably higher than that of the single curvature envelope because of the reduced theoretical effective length.

6.2 Force-deflection responses

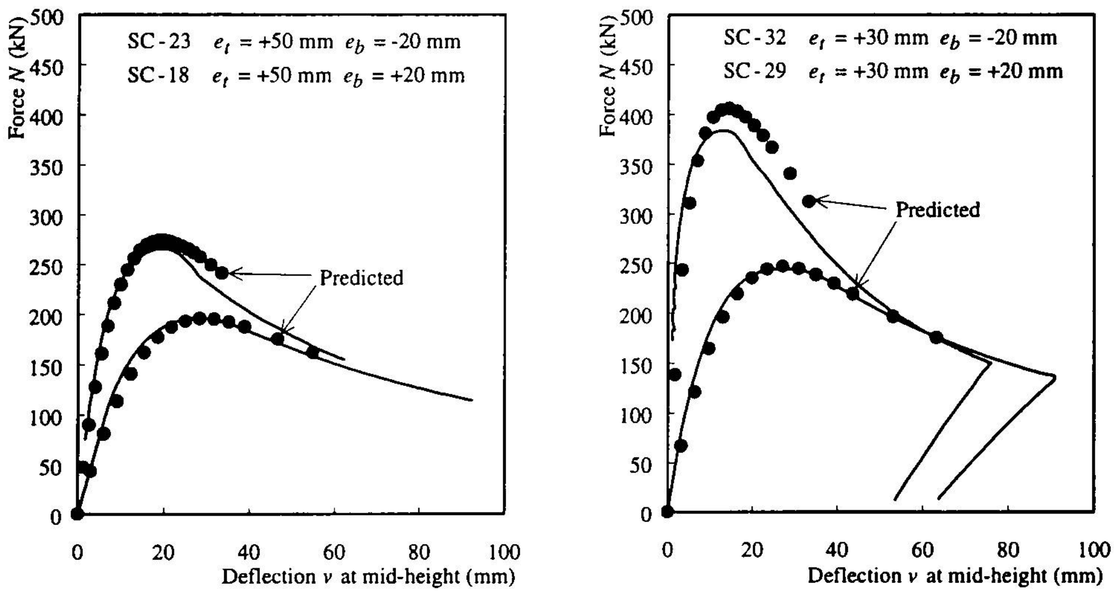

Figure 2 Measured and predicted force-deflection responses

Comparisons between measured and predicted responses for selected columns are given in Fig.2. Generally, the best correlations were obtained for the columns bent into single curvature, particularly those whose eccentricities were large.

6.3 Error analysis

The response of a column is influenced primarily by its length (slenderness) and the (primary) eccentricities of force at its ends. Given that it is a comparatively easy matter to accurately determine the length of a column the most likely source of (significant) error in the tests is the magnitude(s) of the primary eccentricity of force. A study was undertaken (Kilpatrick 1996) of the sensitivity in the strength of the columns to variations in the magnitudes of these eccentricities. A realistic absolute error of 1 mm (1% of the column diameter) in each eccentricity was chosen for the purpose of the exercise. Thus, column SC-40 which had nominal eccentricities of +50 mm and -50 mm was reanalysed firstly for eccentricities of +51 mm and -51 mm, and secondly for +49 mm and -51 mm. Other columns were similarly analysed. As expected, it was found that the strength of the column under the second set of modified eccentricities was severely reduced because the columns could no longer exhibit a perfect anti-symmetrical deflected shape, and began to unwrap as loading advanced.

Col. no.	Eccentricity top, btm (mm)	Column strength (kN)	Eccentricity top, btm (mm)	Column strength (kN)	Eccentricity top,btm (mm)	Column strength (kN)
SC-40	+50,-50	369	+51,-51	363	+49,-51	347
SC-37	+40,-40	437	+41,-41	429	+39,-41	401
-	+30,-30	520	+31,-31	511	+29,-31	458
-	+20,-20	619	+21,-21	608	+19,-21	512
-	+10,-10	737	+11,-11	724	+9,-11	551

Table 3 Effect of errors in eccentricity upon predicted column strength

6.4 Conclusions

The excellent correlation between measured and predicted data suggests that the deformation control method of analysis can reliably predict both the strength and entire force-deflection relationship of eccentrically loaded circular CFST columns. Small variations in eccentricity have little effect upon the predicted response of a column bent into single curvature but a pronounced effect for columns bent into double curvature.

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