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Ductility and Strength of Thin-Walled Concrete Filled Box Columns

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Summary

This paper is concerned with the ultimate strength and ductility of thin-walled concrete filled steel box columns. An extensive set of experiments on concrete filled thin-walled steel box columns will be presented and comparisons with a numerical model will be made. Included in this paper are experimental results and comparisons of box columns filled with normal strength and high strength concrete. Ultimate strength and ductility obtained from test results is used to calibrate the numerical model and to monitor the effects of the use of the increased concrete strengths.

1. Introduction

Concrete filled steel box columns have undergone a renaissance in Australian tall building construction over the last decade due to the increased economies established by reduced construction times and structural costs (Watson and O'Brien[1], Bridge and Webb[2]). The use of very thin walled steel boxes reduces the capital outlay of steel which is a relatively expensive construction material. The concrete pumping operation which takes place after many levels of construction have been completed then speeds the rate of construction which further reduces the overall structural cost. The development of high strength concrete has made the use of concrete filled steel columns extremely attractive as the relatively inexpensive concrete is utilised to resist compressive forces more efficiently than the steel plate. The increased use of high strength concrete needs to include a study of the effects of the concrete strength on the ductility and strength of the structural member. This is particularly important in the design of frames for lateral loading which are designed to fail with strong columns and weak beams. This failure mode needs to be assisted with sufficient rotation capacity from the structural member and it is therefore necessary to consider the ductility in relation to strength for concrete filled steel box columns.

This paper presents an experimental and theoretical study of the ductility and strength of concrete filled steel columns using very thin steel plate. A set of experiments is presented, using normal and high strength concrete and a numerical model developed elsewhere is used to compare the moment curvature response of the model to the experiments for varying thrusts. A method for the comparison of the strength interaction diagram for bending and compression is also presented for the theoretical and experimental results. With further development and experimental data this model can be used to develop design rules and relationships for structural engineers using these members in the design of buildings.

2. Experimental Programme

The aim of the experiments was to determine the cross-sectional strength of concrete filled thin-walled steel box columns subjected to combined axial compression and bending moment. This was achieved by testing stubby column specimens in combined compression and bending and testing beam specimens in pure bending. Furthermore the experiments were undertaken to compare the strength and ductility behaviour of concrete filled steel columns using normal and high strength concrete.

The normal strength concrete study was undertaken on columns which had an overall cross-section size of 180 mm with a steel plate thickness of 3 mm. The properties and characteristics of the specimens are outlined in Table 1. A further set of experiments which utilised high strength concrete having a cross-section size of 120 mm were constructed and tested. Full details of each of these specimens are shown in Table 2 where (C) and (B) represents the column and beam specimens respectively. Specimens NS5 and HS5 denoted as (LB) were tested by loading the steel only to determine the local buckling capacity and this is further dealt with by Uy[3]. The columns were tested under varying eccentricity until the failure load N_u was reached. The ultimate moment was calculated as $M_u=N_u.e$ for the columns and $M_u=N_uL/2$ for the beam specimens tested under two point loading. Figure 1 shows the normal strength concrete filled series after failure.

Specimen No.	b (mm)	t (mm)	f_y (MPa)	f_c (MPa)	e (mm)	N_u (kN)	M_u (kNm)
NS1 (C)	186	3	300	32	0	1555	0
NS2 (C)	186	3	300	32	37	1069	39.6
NS3 (C)	186	3	300	32	56	1133	63.4
NS4 (C)	186	3	300	32	84	895	75.2
NS5 (LB)	186	3	300	32	0	517	0
NS6 (B)	186	3	300	32	L=950	131	62.6

Table 1. Series 1 - Normal Strength Concrete Filled Box Columns

Specimen. No.	b (mm)	t (mm)	f_y (MPa)	f_c (MPa)	e (mm)	N_u (kN)	M_u (kNm)
HS1 (C)	126	3	300	50	0	1114	0
HS2 (C)	126	3	300	50	20	996	19.9
HS3 (C)	126	3	300	50	40	739	29.6
HS4 (C)	126	3	300	50	50	619	31.0
HS5 (LB)	126	3	300	50	0	454	0
HS6 (B)	126	3	300	50	L=600	93	27.9

Table 2. Series 2 - High Strength Concrete Filled Box Columns

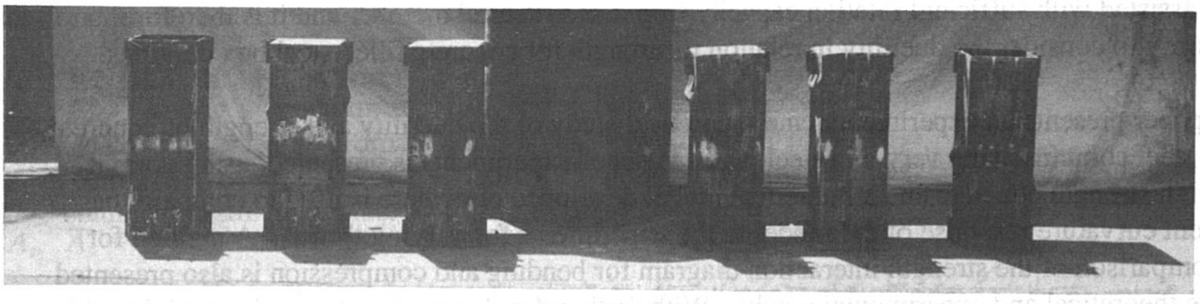

Fig. 1 Normal Strength Concrete Test Specimens after Failure

3. Numerical Model

A numerical model for the thrust moment-curvature behaviour developed by Uy[4] has been used to predict the behaviour of these columns. Pertinent points of this model are briefly described here.

3.1 Cross-Sectional Analysis Technique

A typical strain distribution over the cross-section is established in Fig. 2, and it is characterised by the strain ε_{top} at the top fibre and the curvature ρ. For a given curvature ρ, a position d_n for the concrete neutral axis depth was assumed, so that a strain distribution was obtained. The curvature ρ is the same in the steel and concrete core, so that if there is another neutral axis in the steel caused through slip, then it is uniquely defined. Slip was ignored in this analysis as the slip and slip strain measured in the experiments was negligible. The axial force N in the section was determined from a summation of the forces in each slice. The stress in each slice was expressed as a function of strain at the centroid of each slice, obtained using the relative material constitutive relationship. The concrete neutral axis depth d_n was incremented successively from $d_n=0$ by steps of D/20 until the value of N changed sign for pure bending or until the value of applied axial force was reached unity for the case of bending and compression. The method of bisections was then used to converge on the value of d_n for greater accuracy. The moments of the forces in each slice were then summed to produce the section moment M.

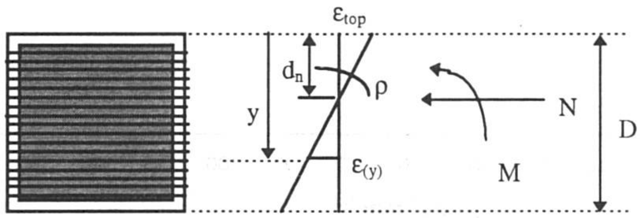

Fig. 2 Concrete Filled Box Section Typical Strain Distribution

This procedure was followed for increasing steps of curvature and steps of thrust to obtain the thrust moment curvature response, and so to observe either ductile or brittle behaviour. The steel was modelled as mild structural steel which is initially elastic linear then goes through a plastic range until strain hardening occurs upon which the steel increases in stress. The CEB-FIP model for the stress-strain relationship of concrete was utilised for the ensuing analyses, (CEB-FIP[5]). It should be noted that this stress-strain relationship is for unconfined concrete. For a steel box column local buckling may occur prior to the concrete crushing and thus the confinement effect can not occur and is thus conservatively ignored in the analysis.

3.2 Moment-Curvature Results

Using the method developed a moment curvature analysis was carried out on the cross-sections of the experiments for varying thrust. It was decided to use a thrust of 0, 25, 50 and 75 % of the maximum axial thrust N_u. The results of the analysis are shown in Figs 3 and 4 for normal and high strength concrete respectively where it is illustrated that the presence of axial force initially causes an increase in moment carrying capacity. However for higher levels of axial force the primary compression failure means that a reduced bending moment carrying capacity is caused. A reduction in the ultimate curvature is experienced for an increasing value of thrust.

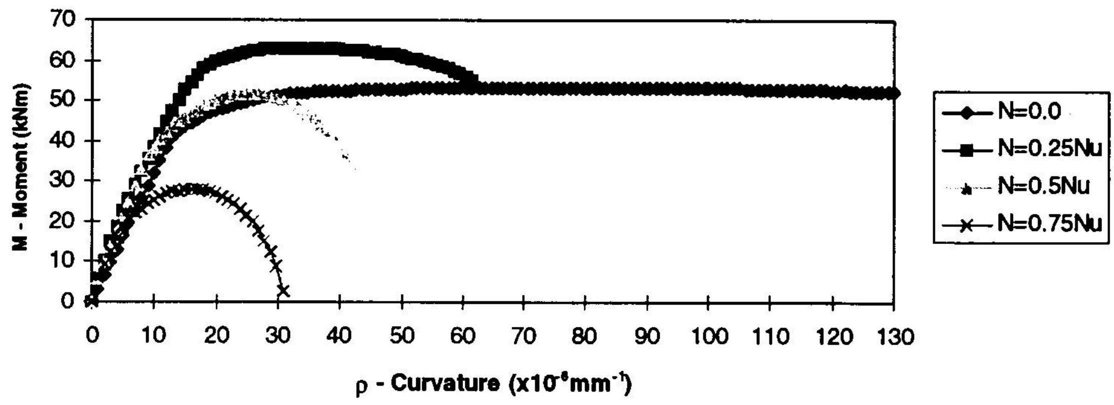

Fig. 3 Thrust-Moment-Curvature Response for Normal Strength Concrete Filled Columns

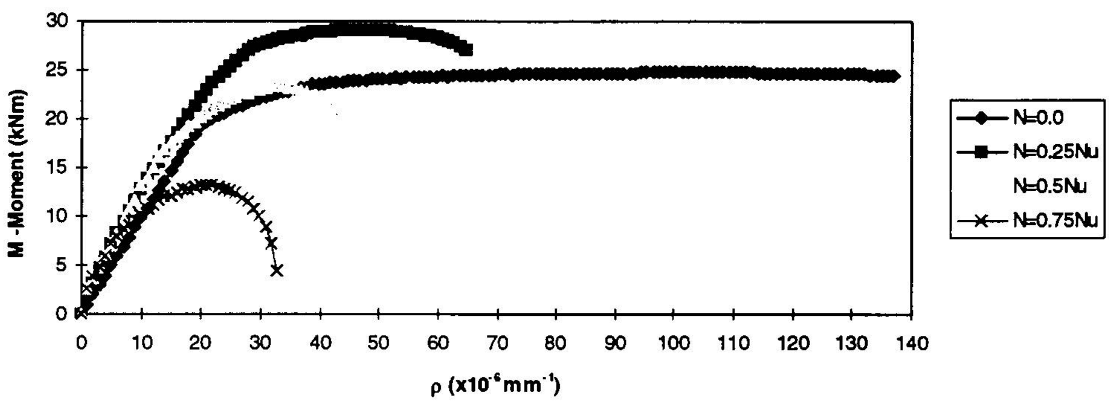

Fig. 4 Thrust-Moment-Curvature Response for High Strength Concrete Filled Columns

4. Comparisons

4.1 Ultimate Strength

The peaks of the moment curvature plots for varying thrusts are used to construct the strength interaction diagrams between axial force and bending moment. From Figs 3 and 4 the peaks of each curve are used to construct the strength interaction diagrams shown in Figs 5 and 6 for normal and high strength concrete respectively. The strength interaction diagrams are compared with the experimental results of Tables 1 and 2 in Figs 5 and 6. Firstly it should be noted that there is extremely good agreement for the case of pure axial force in both the normal and high strength concrete column cross-sections. Furthermore it is shown that the numerical model is conservative in it's estimate of the cross-sectional strength for both the normal and high strength concrete specimens. Thus the model presented should therefore be appropriate for design with the use of appropriate capacity reduction factors.

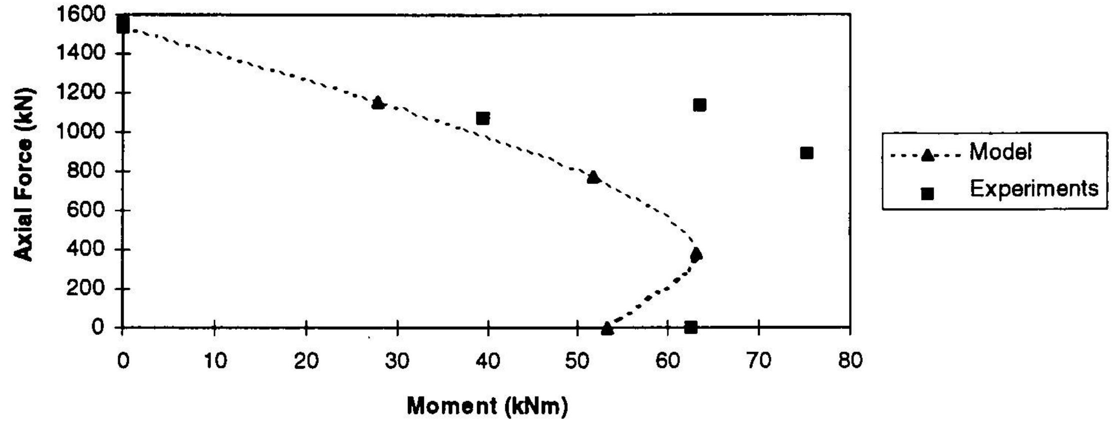

Fig. 5 Strength Interaction Diagram for Normal Strength Concrete Filled Columns

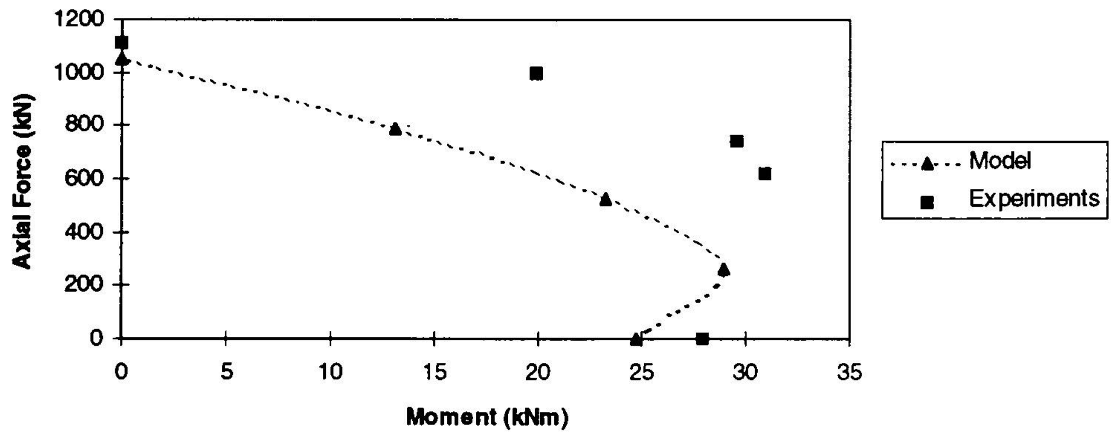

Fig. 6 Strength Interaction Diagram for High Strength Concrete Filled Columns

4.2 Ductility

The ductility ratio is calculated as the ratio of the ultimate curvature ρ_u to the yield curvature ρ_y where $\delta = \frac{\rho_u}{\rho_y}$. These ratios are calculated and compared for each of the specimens in Tables 3 and 4 for the experiments and theoretical model.

$\frac{N}{N_u}$	$\left(\frac{M}{M_u}\right)_{test}$	$\left(\frac{M}{M_u}\right)_{theory}$	$\delta_{theory} = \left(\frac{\rho_u}{\rho_y}\right)_{theory}$
0.0	1.0	1.0	9.56
0.25	1.20	1.18	4.42
0.5	1.01	0.97	2.80
0.75	0.63	0.52	1.62

Table 3. Normal Strength Ductility Ratios

$\frac{N}{N_u}$	$\left(\frac{M}{M_u}\right)_{test}$	$\left(\frac{M}{M_u}\right)_{theory}$	$\delta_{theory} = \left(\frac{\rho_u}{\rho_y}\right)_{theory}$
0.0	1.0	1.0	6.75
0.25	1.11	1.17	3.25
0.5	1.06	0.94	2.25
0.75	0.71	0.53	1.50

Table 4. High Strength Ductility Ratios

It is shown that the use of a higher strength concrete significantly reduces the ductility of a concrete filled steel column subjected to pure bending. However as the ratio of axial force is increased the apparent loss of ductility is less pronounced. As most columns under gravity load will have at least 40-50 % of the ultimate load as service loads it is therefore not as significant.

6. Conclusions

This paper has discussed the behaviour of concrete filled steel columns using both normal and high strength concrete. Experiments have been conducted to determine the cross-sectional strength and ductility of these two forms of columns. A numerical model developed elsewhere has been calibrated with these results with good agreement and has been found to be conservative in it's prediction thus rendering it appropriate for design. It is necessary to undertake more tests to adequately calibrate results which can therefore suggest design procedures particularly in relation to capacity reduction factors required for limits states or load resistance factor design.

7. Acknowledgements

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