THE STRUCTURAL DESIGN OF TALL BUILDINGS
Struct. Design Tall Build. **7**, 113–133 (1998)

DUCTILITY, STRENGTH AND STABILITY OF CONCRETE-FILLED FABRICATED STEEL BOX COLUMNS FOR TALL BUILDINGS

BRIAN UY*,†
Department of Civil, Mining and Environmental Engineering, University of Wollongong, Wollongong, NSW 2522, Australia

SUMMARY

This paper is concerned with the ductility, strength and stability behaviour of concrete filled steel box columns. A cross-sectional analysis procedure which considers the nonlinear material properties of steel and concrete has been developed. The steel behaviour includes the presence of residual strains and stresses. A parametric study is undertaken to monitor the influence of residual stresses, concrete strength and steel strength on the thrust–moment–curvature response. A parameter known as the ductility ratio is calculated and this gives an indication of the ability of a structural member to deform. From the results of the thrust–moment–curvature response, strength interaction diagrams are also developed. Slender column behaviour for these structural members is described and methods by which to determine the strength are discussed. A design example is presented which shows the use of both the strength interaction diagrams and the slender column buckling procedure developed by other researchers. Further research areas are then outlined and discussed in order to elucidate the behaviour and design of these highly efficient column members in tall buildings. © 1998 John Wiley & Sons, Ltd.

1. INTRODUCTION

Concrete filled steel box columns (CFSBs) have been developed during the 1960s and their use in tall buildings since then has been fairly limited. The main advantage of their previous use was that the steel tube or box column acted as the permanent formwork and eliminated the need for reinforcing steel to be tied and placed. The overall cost and construction efficiency of the CFSB column members was not as competitive in comparison with conventional reinforced concrete or structural steel columns and has been mainly used as an architectural finish to date. Recently, with the development of high-strength concrete and with the establishment of a method to consider the local buckling behaviour of the steel, concrete-filled steel tube and box columns have seen a resurgence in their use.

The 57-storey Two Union Square in Seattle is testimony to the large variety of advantages that this type of column provides during construction. Very-high-strength concrete of 130 MPa was poured into steel pipes on this project and provides substantial benefits.[1] The main benefit is that the high-strength concrete resists the majority of the axial force and thereby minimizes the amount of steel required to be

* Correspondence to: Brian Uy, Department of Civil, Mining and Environmental Engineering, University of Wollongong, Wollongong, NSW 2522, Australia. E-mail: brianu@uow.edu.au
† Senior Lecturer in Civil Engineering

Contract/grant sponsor: Australian Research Council Small Grants Scheme

CCC 1062-8002/98/020113–21 $17.50

Received November 1996
Revised May 1997

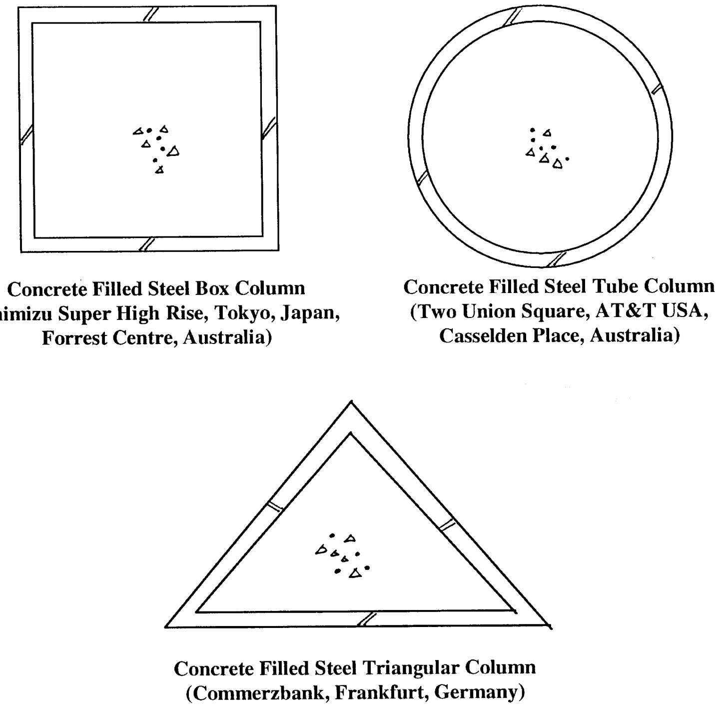


Figure 1. Concrete-filled steel columns

used. Furthermore, the concrete may be pumped inside after the steel has been constructed up to several levels. This has also found application in Australia with projects such as the 43-storey Casselden Place, Melbourne, using 80 MPa concrete strength in steel tubes.[2] Recently in Frankfurt, Germany, the Commerzbank, which is the tallest building in Europe at 300 m, used triangular concrete-filled columns for a 63-storey building.[3,4]

Currently in the proposal stage, the Shimizu Super High Rise in Japan will represent the tallest building in the world and stand at 550 m on the Tokyo skyline when construction is complete. It will be constructed using the concept of steel columns filled with high strength concrete.[1] Typical cross-sections of concrete-filled steel columns are shown for recent projects in Figure 1.

Other projects in Australia, such as the Forrest Centre and Westralia Square,[1,5] have utilized steel box columns filled with concrete. The increase in the use of this method in Australia has been due to the competitive costs between concrete filled steel columns and reinforced concrete columns. The labour cost for construction in Australia is high and since this method can reduce costs and increase construction speed it has found increased applicability. In light of this increased useage it is necessary to identify the ductility, strength and stability characteristics.

This paper will consider the thrust–moment–curvature response of very thin walled concrete filled steel box columns and will include a parametric study to consider the various material properties and inclusion of residual stresses. From these results, a set of strength interaction diagrams is developed. Oehlers and Bradford[6] have also outlined methods for slender column behaviour and this paper discusses how these may be used to determine the slender column strength and how columns in tall buildings may be designed so that they are stubby, which allows for greater structural efficiency and the use of high-strength materials.

2. CONCRETE-FILLED THIN-WALLED STEEL BOX COLUMNS

When designing concrete-filled steel box columns in a tall building, careful consideration of all stages of loading needs to be borne in mind. The three stages of loading to consider for CFSB are

(i) construction loading;
(ii) long-term service loading; and
(iii) ultimate loading.

2.1. Construction loading

This is a very important stage, as hollow steel boxes are subjected to both axial stress and hydrostatic pressure from wet concrete. This has been considered by Uy and Das,[7] and slenderness limits have been developed to enable structural engineers to design the columns to be suitable for strength and serviceability during this stage. Adequate propping was necessary on the Commerzbank,[3] to eliminate these deformations, and this has been the subject of much concern in the design of Australian tall buildings using this form of column construction.[8] A typical diagram of this construction stage is shown in Figure 2. The diagram shows a floor plan and elevation of a typical tall building with concrete-filled steel columns and reinforced concrete core for lateral load resistance. The pumping of concrete can be undertaken independently of floor construction operations and up to six levels have been pumped in previous projects such as Casselden Place.[2] Adequate bracing was the subject of an investigation by Uy and Das,[9] which provides guidance on the spacing and number of restraints required to ensure that deflections are limited to construction tolerance limits between floor levels.

2.2. Long term service loading

During the construction of a tall building, the concrete inside the steel boxes suffers creep and shrinkage deformations and it is during this stage that the concrete may shed its stress to the steel and thereby reduce the stress in the concrete. Experiments have been conducted by Terrey *et al.*,[10] Morino *et al.*[11] and Nakai *et al.*[12] to determine the shrinkage and creep behaviour of concrete inside steel tubes. These results have been used to identify the aspect of stress redistribution by Uy and Das,[13] who determined percentage reductions in concrete stress and increase in steel stress, and showed them to be quite substantial. The effects of creep and shrinkage on shortening of columns and differential shortening of columns with respect to internal core walls in a tall building have also been considered and shown to be of particular importance.

2.3. Ultimate loading

The third stage of loading is when the material properties of steel and concrete are within the nonlinear range of behaviour and the steel may buckle locally. Uy[14] has developed a cross-sectional analysis procedure incorporating residual stresses and this paper will consider the behaviour of concrete-filled steel box columns considering the nonlinear properties of steel and concrete and incorporating residual stresses which are general to both normal strength and high-strength materials.

Each of these stages of loading is influenced by

(i) local buckling; and
(ii) residual stresses.

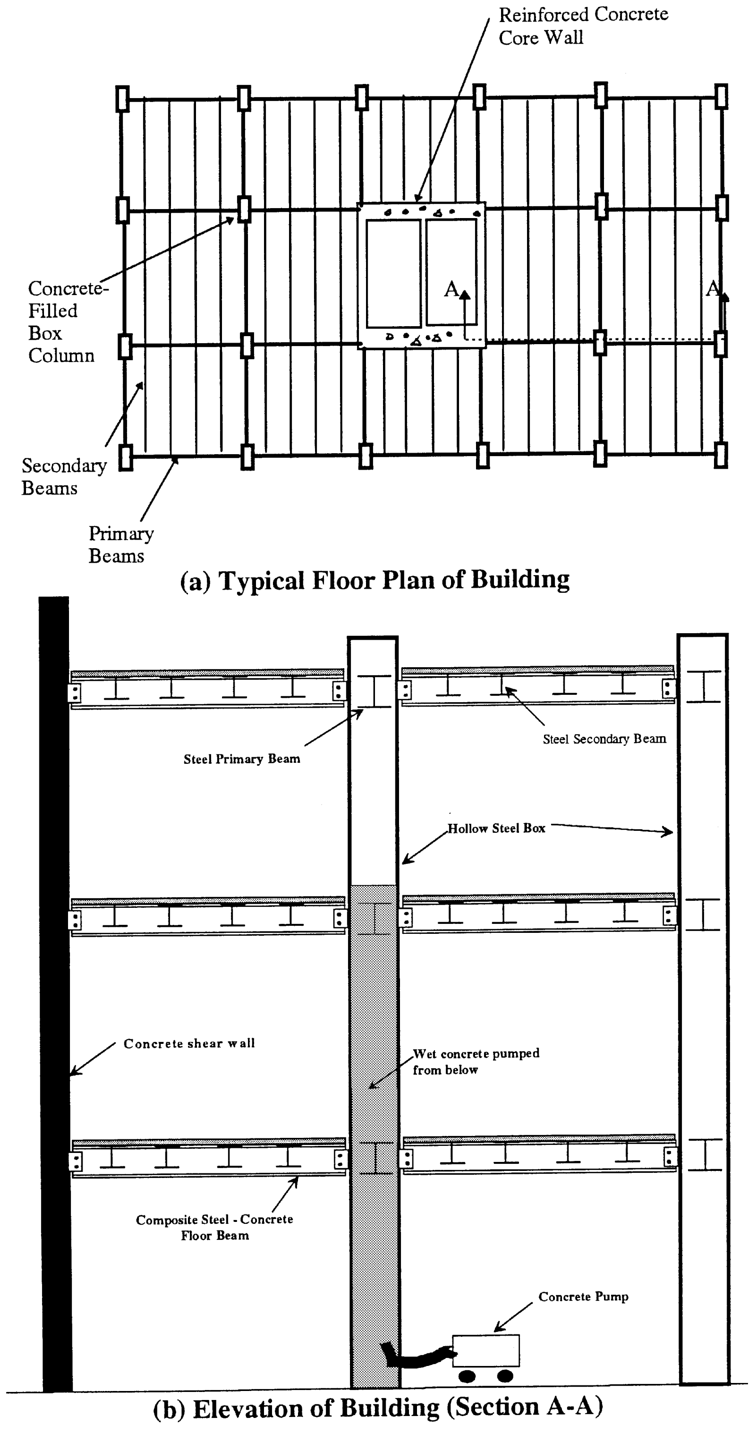


Figure 2. Construction procedure for concrete-filled steel box column in a tall building

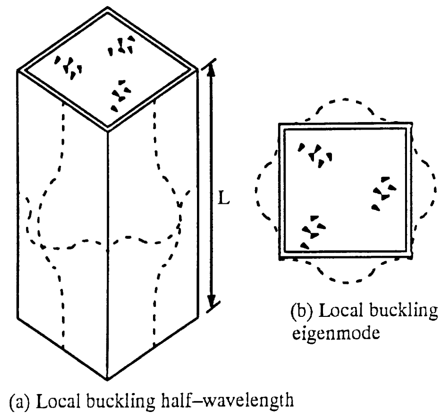


Figure 3. Local buckling of concrete-filled box column

2.4. Local buckling

Concrete-filled steel box columns are an efficient manner in which to use structural steel plate. Steel box columns are expensive, as the amount of steel required to resist large compressive forces is high. The use of steel plate to act as permanent formwork needs to be optimized in order for the columns to become cost effective. The concrete is used to resist the majority of the axial force, as this is more cost effective. The steel can be optimized by the use of a rational local buckling model that considers the effect of the concrete in restraining the steel component plates from local buckling as shown in Figure 3. This has been considered by Uy and Bradford,[15,16] and augmented recently by Uy[17] to incorporate residual stresses due to welding and fabrication. Extensive experimental results have also been obtained by Bridge *et al.*[18] This model is used in this analysis for the design of the steel component plates and is considered to be necessary in the economic use of concrete-filled steel box and tube columns.

2.5. Residual stresses

Fabricated steel box columns are subjected to welding procedures that cause shrinkage of the weld metal and thereby result in the development of tensile yield stresses at the plate junctions which are

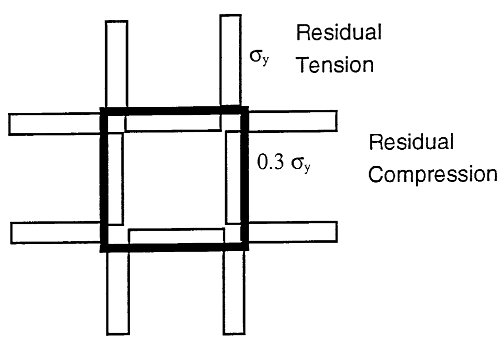


Figure 4. Residual stress distributions

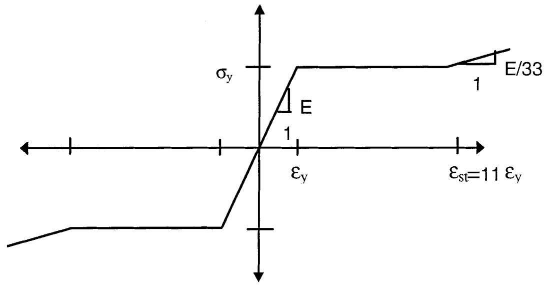


Figure 5. Stress-strain relationship for mild structural steel

equilibrated by residual compressive stresses in the unwelded regions of the box. A typical residual stress distribution is shown in Figure 4 where a maximum of 30% of yield in compression is caused by the tensile yield stresses at the weld locations.

3. CROSS-SECTIONAL ANALYSIS

A cross-sectional analysis method developed and calibrated against experimental results for profiled composite beams by Uy and Bradford[19] has been augmented to study the influence of material and geometric properties on the thrust–moment–curvature relationships of concrete-filled steel box columns and to thereby consider the ductility of concrete filled steel box columns. The results are useful in determining design relationships which may then be used in the determination of strength and stability criteria.

3.1. Steel

Two types of steel were used in the ensuing analyses. Firstly, mild structural steel which was idealized having linear elastic-plastic-strain hardening ranges until fracture occurs as shown in Figure 5. High-strength steel was modelled using a bilinear relationship of elastic perfectly plastic behaviour until fracture occurs as shown in Figure 6. The yield stress in tension and compression was determined to be approximately 690 MPa by Rasmussen and Hancock[20,21] on tests carried out on hollow-steel-box and I-sections using Australian manufactured quenched and tempered steel.

3.2. Concrete

Normal and high-strength concrete were used in this study and the constitutive relationships were

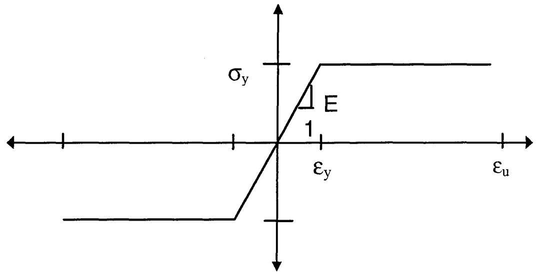


Figure 6. Stress–strain relationship for high strength structural steel

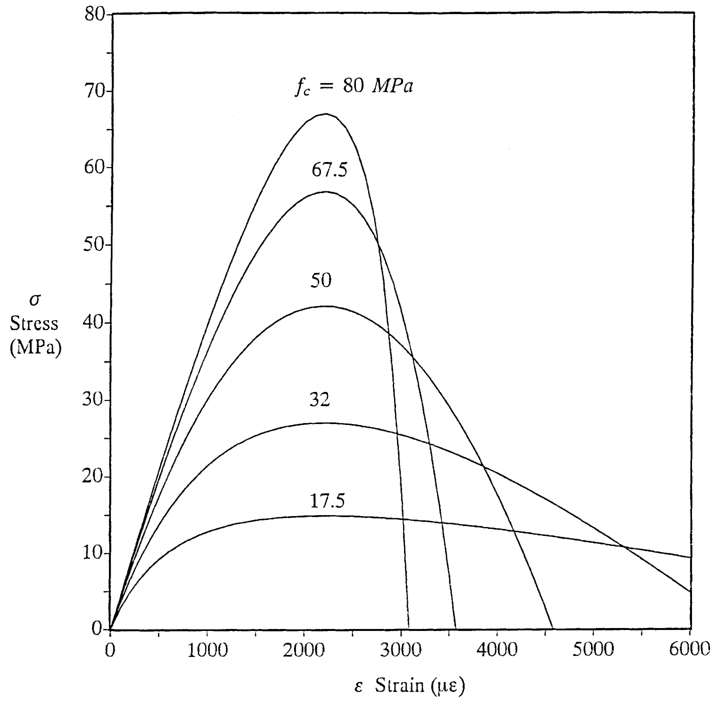


Figure 7. Stress–strain relationships for unconfined concrete

modelled using the CEB-FIP model code[22] for unconfined normal and high-strength concrete as shown in Figure 7. Unconfined concrete stress–strain behaviour was adopted, as confinement is generally only considered to occur in circular columns and test data has shown that this does not occur in rectangular columns. [23,24] This stress–strain relationship of concrete allows the softening branch to be modelled. With this approach, the stress is represented as a function of strain by

$$\sigma = \frac{\sigma_0 \varepsilon (a - 206\,600\varepsilon)}{1 + b\varepsilon} \tag{1}$$

where σ is in megapascals,

$$\sigma_0 = 0.85 f_c \tag{2}$$

represents the peak stress from the curves in Figure 7 and a and b are calculated using

$$a = 39\,000(\sigma_0 + 7.0)^{-0.953} \tag{3}$$

$$b = 65\,600(\sigma_0 + 10.0)^{-1.085} - 850.0 \tag{4}$$

3.3. Analysis method

A typical strain distribution over the cross-section is shown in Figure 8 and it is characterized by the strain ε_{top} at the top fibre and the curvature ρ. To obtain the moment–curvature relationship for a given cross-section, the following procedure is adopted. 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

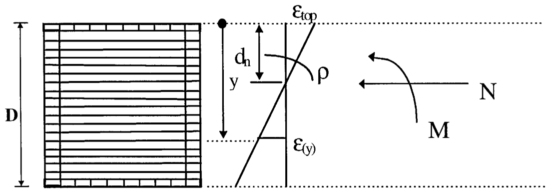
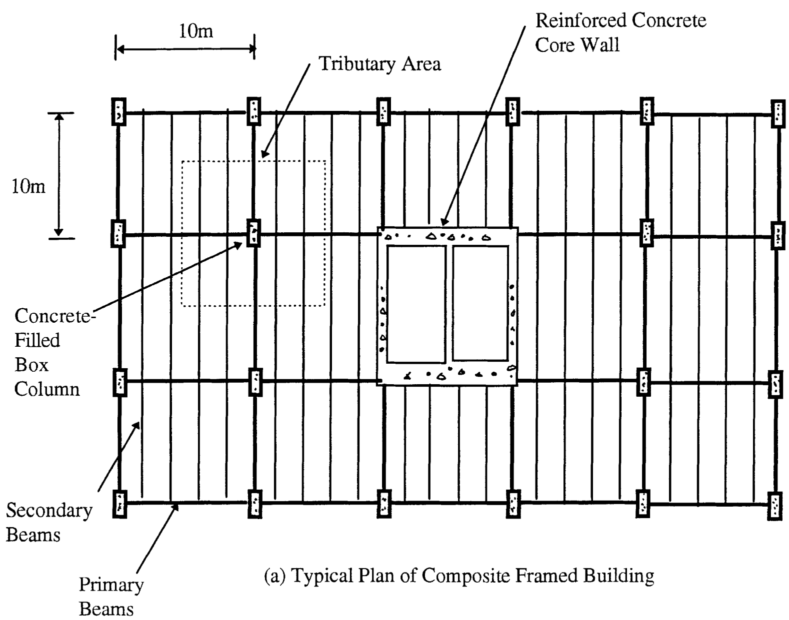


Figure 8. Cross-sectional analysis method

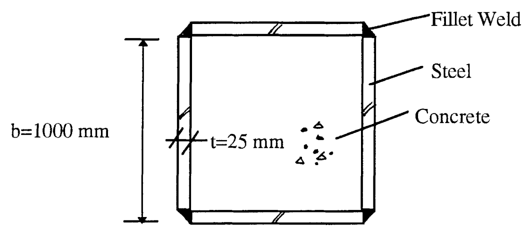
the same in the steel and concrete core, so that if there is another neutral axis in the steel, then it is uniquely defined. The axial force N in the section was determined from

$$N = \int_{y=0}^{y=D} \sigma(y)\ \mathrm{d}A \tag{5}$$

where $\sigma(y)$ is the stress as a function of strain $\varepsilon(y)$ at the centroid of each slice, obtained using the



(a) Typical Plan of Composite Framed Building



(b) Concrete-Filled Steel Box Section

Figure 9. Typical column section for parametric study

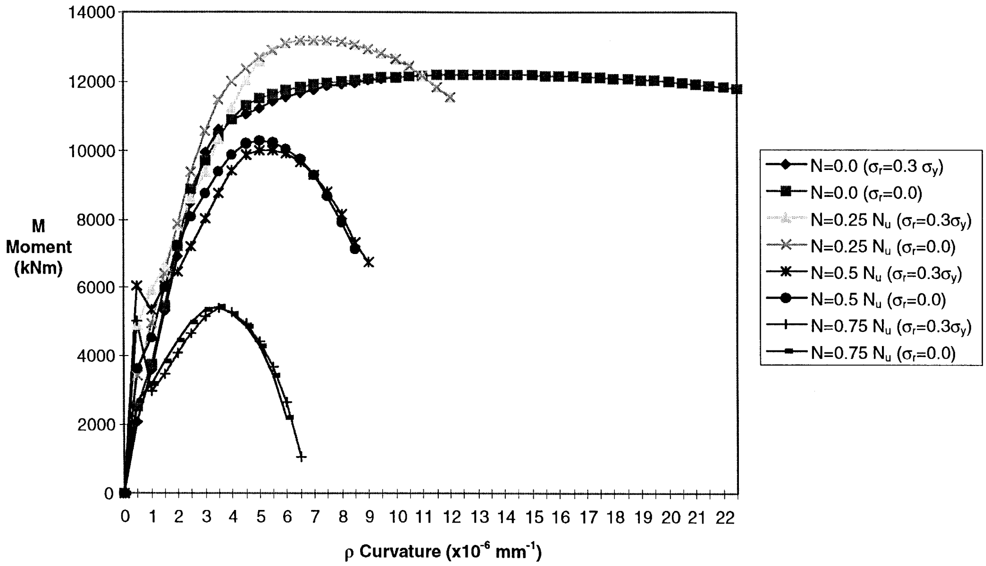


Figure 10. Effect of residual stresses

relative material constitutive relationship for concrete or steel. 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 axial force reached the applied axial force for the case of combined bending and compression. The method of bisections is then used to converge on the value of d_n for which the axial force N holds true. The moments of the forces in each slice were then summed to produce the section moment M from

$$M = \int_{d_n}^{D} y'\sigma(y)\ \mathrm{d}A + N\left(\frac{D}{2} - d_n\right) \tag{6}$$

where

$$y' = y - d_n \tag{7}$$

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.

4. PARAMETRIC STUDY

A parametric study has been undertaken to study the influence of various material properties on the behaviour of concrete-filled steel box columns. The parameters considered include the presence of residual stresses, the yield strength of steel and the concrete compressive strength and stiffness.

For the parametric study, a cross-section size which represents a typical column in a tall building has been used. A cross-section size of 1000 mm × 1000 mm was used and this is typical of the size of column required to resist axial force and bending moments in the base columns of a 25-storey building given a regular 10 m column grid and 10 kPa service load acting over the entire area as shown in Figure 9. The plate thickness has been determined so as to eliminate local buckling using the model developed by Uy.[17]

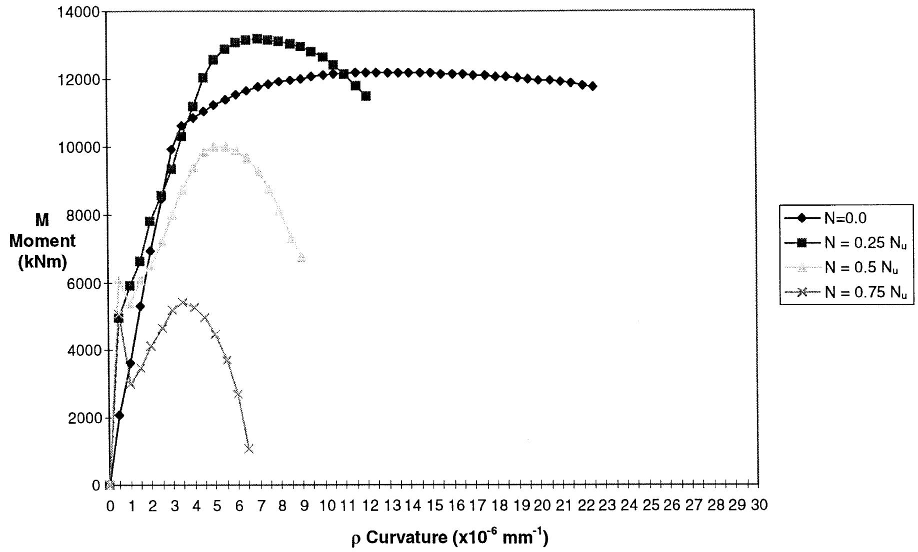


Figure 11. Thrust–moment–curvature diagrams for mild structural steel

4.1. Residual stresses

The residual tensile and compressive stresses in the steel box were varied in order to determine the effect on the stiffness and strength of the section. The analysis was undertaken for a section without residual stresses and with residual stresses as shown in Figure 4. The influence of residual stresses shown in Figure 10 is such that the initial stiffness of the section is slightly higher until the compressive stresses reach yield and a slight loss of stiffness is encountered since the presence of residual stresses are only effective in the elastic range. There is no reduction in the ultimate strength caused through the presence of residual stresses. However, a slight increase in the ultimate curvature is noted.

4.2. Steel yield strength, σ_y

The steel yield strengths which were studied in this analysis included mild structural steel plate of 300 MPa and high-strength steel plate of 690 MPa. The mild structural steel plate is the most commonly used material for fabricated steel box columns in Australia. Results of mild structural steel and high strength steel are shown in Figures 11 and 12, respectively. These show that the use of high-strength steel results in doubling the ultimate bending capacity for a column with the same section size. The ultimate curvature is only slightly reduced, but this will be shown to result in a reduction in overall ductility.

In Australia, high-strength steel in columns of tall buildings has been used to advantage for the columns of a 32-storey building in Latrobe Street, Melbourne.[25] These were designed as stubby columns, which eliminate slenderness effects and utilize the full potential of high-strength steel. The use of high-strength steel in CFSB has been previously considered by Uy and Patil,[26] who considered both the behaviour and design and identified the cost savings and reduction in weight of a typical 50-storey building.

The Shimizu Super High Rise, which is currently in the evaluation stage, has been designed with

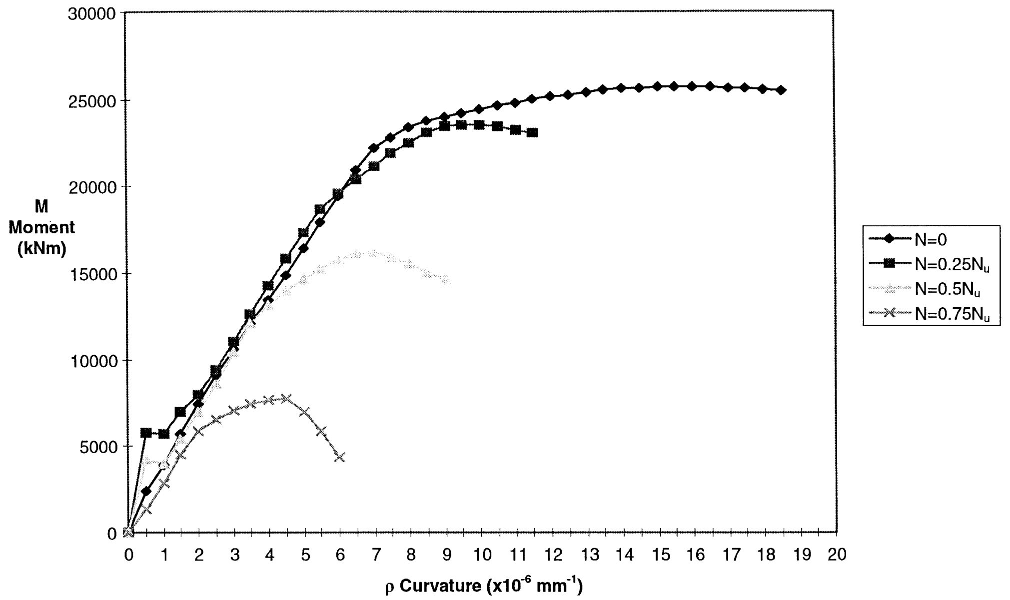


Figure 12. Thrust–Moment–Curvature diagrams for high-strength structural steel

concrete-filled steel columns using high-strength steel plate of 600 MPa.[1] Other researchers in Japan working with the Japanese Architecture Institute are considering very tall buildings of up to 1000 m in height for Tokyo and this requires the use of high-strength steel of 780 MPa yield strength.[27,28]

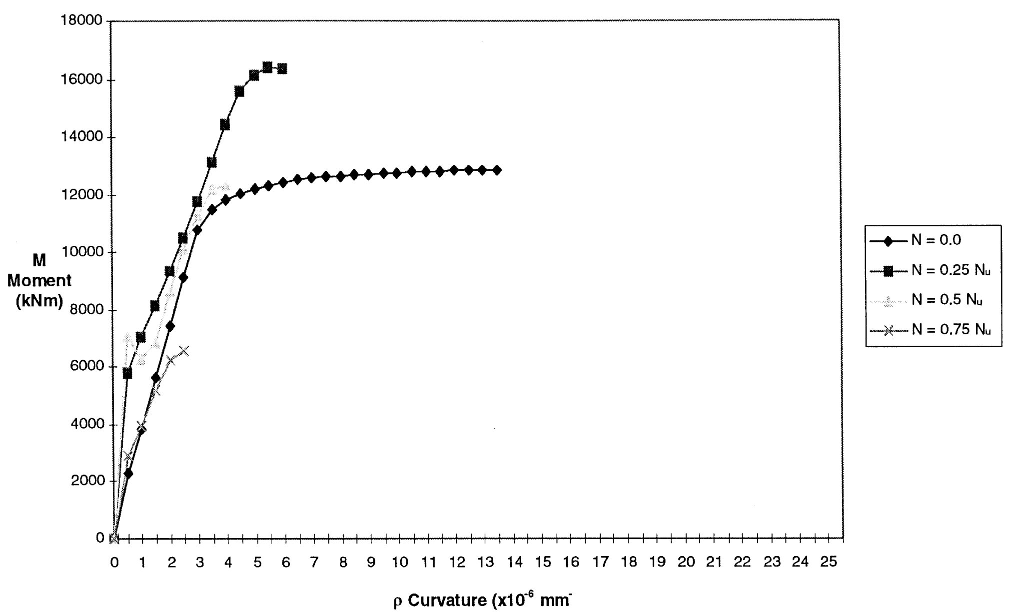


Figure 13. Thrust–Moment–Curvature diagram for high-strength concrete

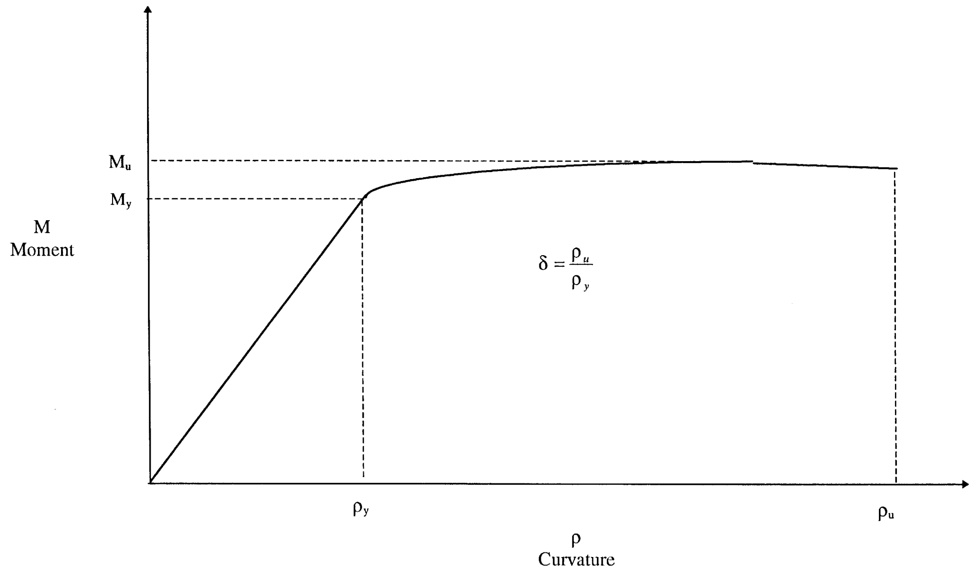


Figure 14. Calculation of ductility ratio

4.3. Concrete compressive strength, f_c

The extensive use of high strength concrete warrants the study here of various concrete strengths. Normal strength concrete of 32 MPa and high-strength concrete of 80 MPa were studied and the results are shown in Figures 11 and 13, respectively. The use of high-strength concrete results in an increase in the ultimate bending capacity with a reduction in the ductility. However, since yielding of the steel takes place fairly early the ductility is still fairly large and therefore this type of section is still desirable.

The use of concrete-filled columns throughout the world has seen a proliferation in the application of high-strength concrete. By using high-strength concrete to resist the majority of compressive stresses, the steel can be minimized. This has substantial benefits for the cost of the members. Concrete strengths up to 130 MPa have been used on projects such as Two Union Square, concrete of 80 MPa compressive strength was used in steel tubes for Casselden Place in Melbourne, Australia, and the Shimizu Super High Rise has been currently designed with a concrete strength of 60 MPa.[1]

5. DUCTILITY RATIOS OF SHORT COLUMNS

In order to quantify the ductility of a particular section, a measure of its deformation capabilities is required. A parameter known as the ductility ratio is calculated herein. The ductility ratios, δ, have been determined as

$$\delta = \frac{\rho_{\mathrm{u}}}{\rho_{\mathrm{y}(N=0)}} \tag{8}$$

The ductility ratio is determined from the moment–curvature response of a section as shown in Figure 14 where ρ_{u} is the ultimate curvature and $\rho_{\mathrm{y(N=0)}}$ is the yield curvature for the pure bending case. These ductilities are a measure of the ability of a particular member to deform and provide a parameter in which to compare the behaviour of the various materials used in the paper. The results in

Table I· Ductility ratios

$\frac{N}{N_u}$	f_c (Mpa)	σ_y (Mpa)	$\delta = \frac{\rho_u}{\rho_{y(N=0)}}$	$\alpha = \frac{M_u}{M_{u(N=0)}}$
0	32	300	7·5	1·0
0·25	32	300	4·0	1·08
0·50	32	300	3·0	0·81
0·75	32	300	2·2	0·44
0	32	690	2·9	1·0
0·25	32	690	1·8	0·92
0·50	32	690	1·4	0·63
0·75	32	690	0·9	0·30
0	80	300	4·5	1·0
0·25	80	300	2·0	1·27
0·50	80	300	1·3	0·96
0·75	80	300	0·8	0·51

Table I show that for pure moment, a ductility ratio of 7·5 is reduced to 4·5 when high-strength concrete of 80 MPa is used in place of normal-strength concrete of 32 MPa. Furthermore, a ductility ratio of 2·9 results from the use of high-strength steel under pure moment. Other comparisons can be made from the table. It can be concluded that the application of high-strength concrete and high-strength steel result in a reduction in ductility. Table I shows that for high levels of axial force, as would be expected in a column, the ductility is considerably less than that for pure bending. However, the values of these ratios for high-strength concrete and steel are fairly similar.

6. STRENGTH INTERACTION DIAGRAMS OF SHORT COLUMNS

Strength interaction diagrams have been determined from the thrust–moment–curvature diagrams developed herein. The peak moment from each moment–curvature graph is then plotted together with

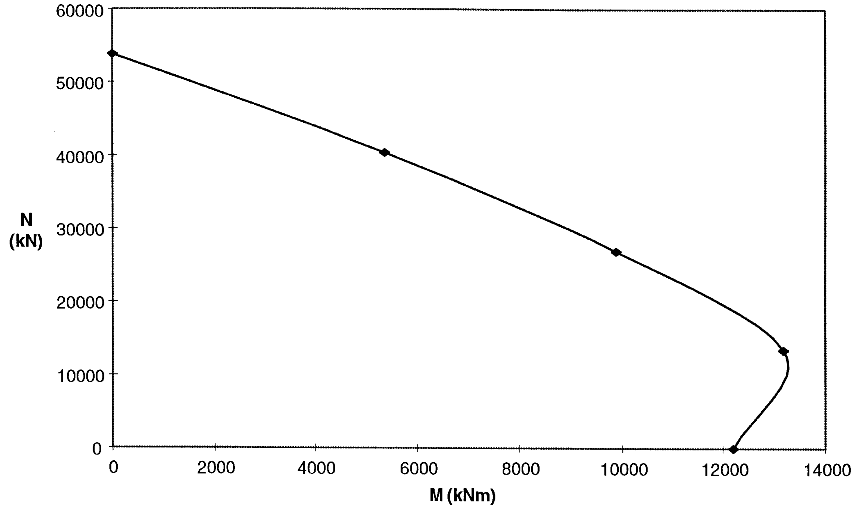


Figure 15. Strength interaction diagram for normal strength concrete

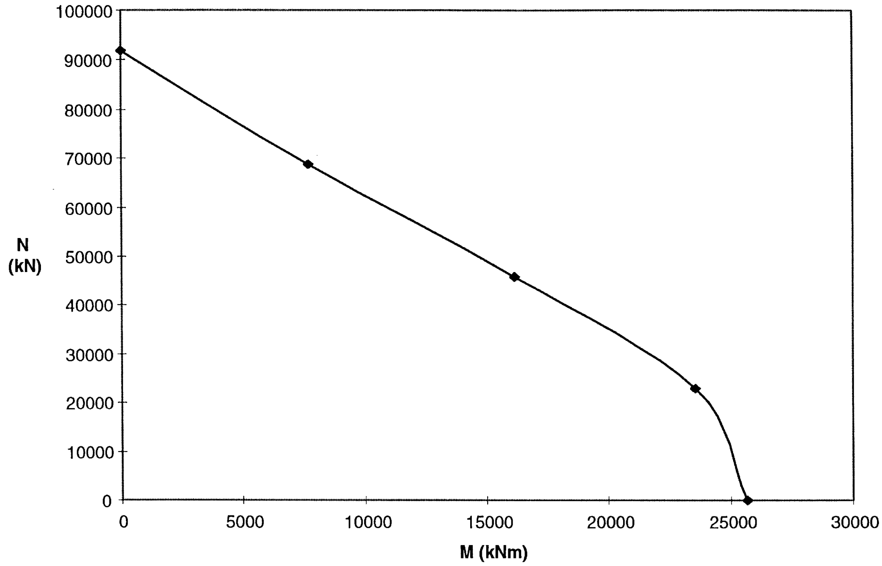


Figure 16. Strength interaction diagram for high-strength structural steel

the applied axial force. These are useful for the design of short concrete-filled steel box columns and can be used for determining the adequacy of slender columns for stability. The strength interaction diagrams for various cross-sections considered in this paper are shown in Figures 15, 16 and 17. These

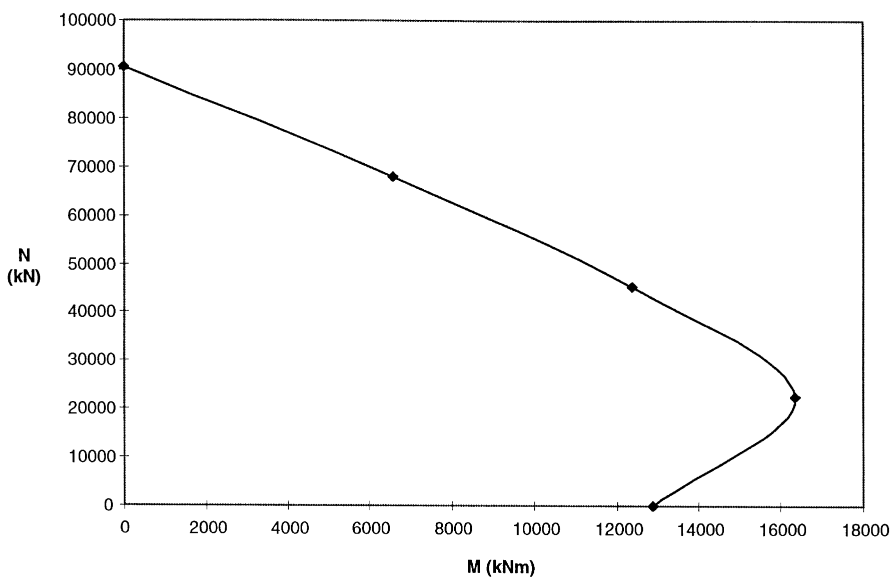


Figure 17. Strength interaction diagram for high-strength concrete

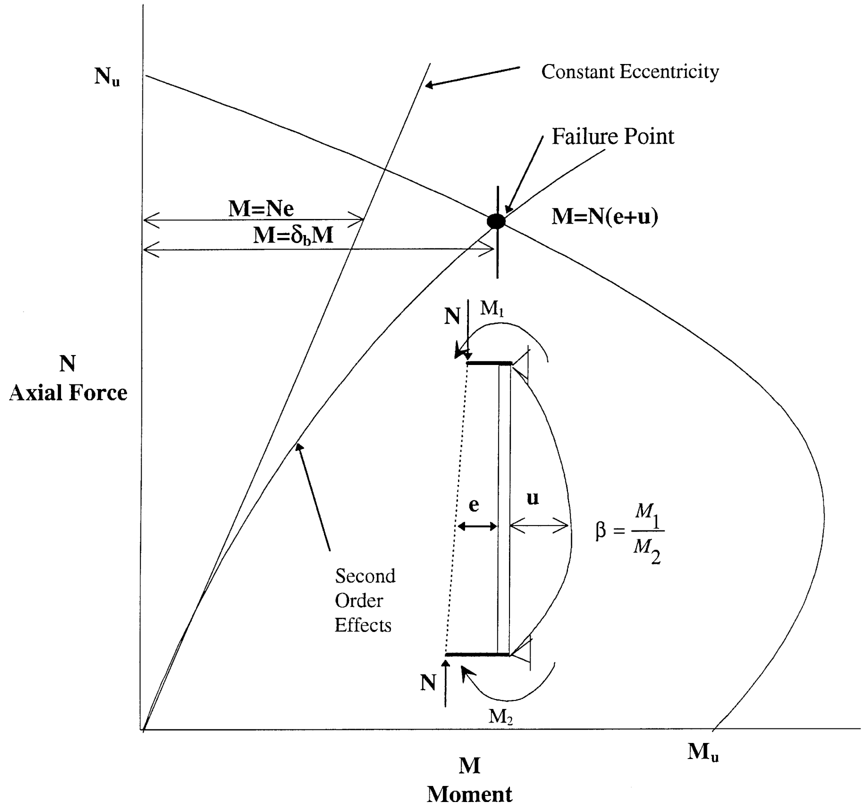


Figure 18. Strength interaction diagram

strength interaction diagrams are for normal-strength steel and concrete, high-strength steel and high-strength concrete, respectively.

The strength interaction diagrams are useful in determining the behaviour of a cross-section. However, for design a capacity reduction factor must be applied to the cross-section. The capacity reduction factor depends on the type of failure, which may be either primary compression or primary tension. Also, the type of material being used influences the value of the capacity reduction factor. These capacity reduction factors are outlined in international design codes and it is suggested for design that the capacity reduction factors are left to the reader to determine according to their respective international codes.

7. STABILITY OF SLENDER COLUMNS

The concept of fabricated steel box columns filled with concrete has been developed for use in tall buildings where standard hot-rolled I-sections are too small to be suitable for the loads applied. The cross-sectional size of the members used is generally fairly large, so that the slenderness ratio of the columns is generally small. This therefore suggests that the main design criteria is strength, since

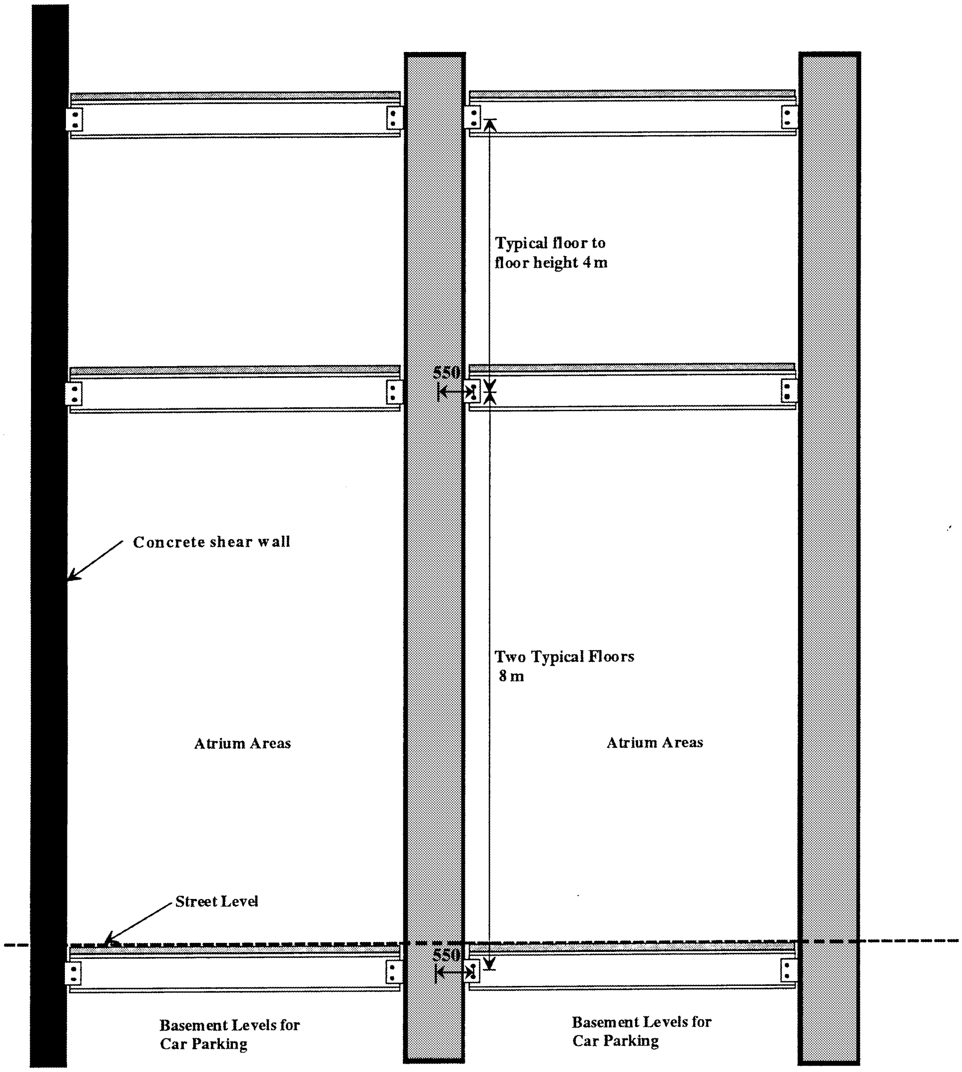


Figure 19. Slender atrium column for tall building

stability and second-order effects will generally be negligible. Slender columns are defined in the Australian Concrete Structures Code AS 3600[29] as those which have a value of slenderness ratio.

$$\frac{L}{r} > 25 \tag{9}$$

for a braced column, which is the case for a composite frame braced by a shear wall in a tall building, where

$$r = 0.25D \tag{10}$$

for a typical box column filled with concrete. The method developed in this study is very useful in the

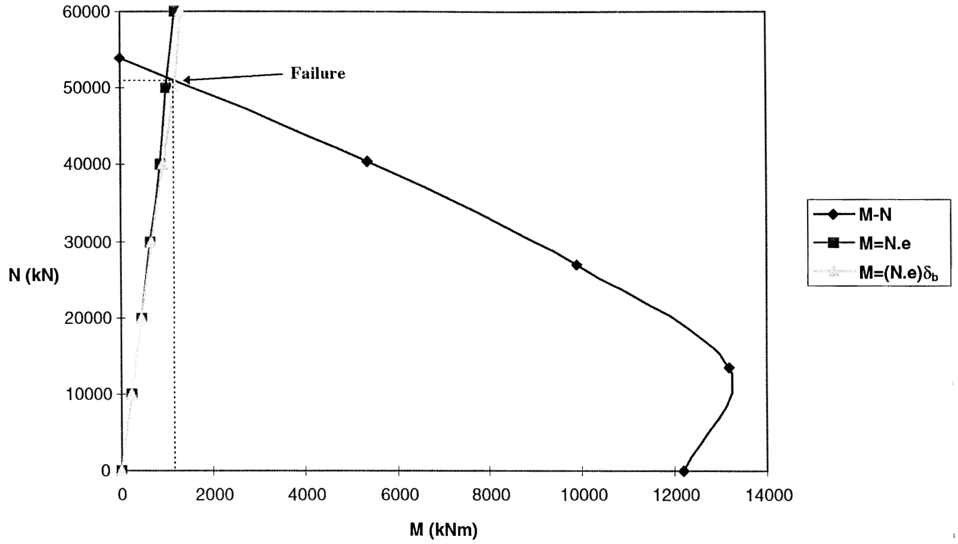


Figure 20. Strength interaction diagram and slenderness effects

design of short concrete-filled steel box columns and the strength interaction diagrams can be used when checking a slender column using the curves developed for the loading line. This has been outlined by Oehlers and Bradford[6] and Bradford[30] and is described here. A slender column which is subjected to axial force and bending moment will suffer from second-order effects which can be determined by the use of a moment magnifier. The moment magnifier for a braced column outlined in Eurocode 4,[31] is determined using the following expression:

$$\delta_b = \frac{k_m}{1 - \dfrac{N}{N_{crit}}} \tag{11}$$

where

$$k_m = 0.66 + 0.4\beta \geq 0.44 \tag{12}$$

and

$$\beta = \frac{M_1}{M_2} \tag{13}$$

where M_1 and M_2 are the smaller and larger end moments, respectively, and the critical buckling or bifurcation load is calculated as

$$N_{crit} = \frac{\pi^2 (EI)_e}{L^2} \tag{14}$$

where the effective rigidity is calculated as

$$(EI)_e = E_s I_s + 0.8 E_c I_c \tag{15}$$

The method for determination of the failure load is shown in Figure 18 as outlined by Oehlers and Bradford.[6] The strength interaction diagram is shown together with the line for constant eccentricity.

The loading line incorporating second-order effects is also shown and where it crosses the interaction diagram that determines the failure load of moment and axial force.

8. DESIGN EXAMPLE

A design example is given here to illustrate the use of the strength interaction diagrams and the stability analysis suggested by Bradford.[29] This example is shown in Figure 19 and represents a typical design example for the base level of a tall building where an atrium column may be unsupported for buckling for 2 levels. Determine the capacity of the atrium concrete filled box column shown in Figure 19. The box column has the following properties:

(i) *steel*

$$b = 1000\text{mm}; t = 25\text{mm}; E_s = 200\,000 \text{ MPa}; \sigma_y = 300 \text{ MPa}$$

(ii) *concrete*

$$f_c' = 32 \text{ MPa}; E_c = 28\,500 \text{ MPa}$$

Now one must firstly determine whether the column requires slenderness effects to be taken into account

$$L = 8000\text{mm}$$

$$r = 0{\cdot}25D = 0{\cdot}25(1000) = 250\text{mm}$$

$$L/r = 32 > 22$$

Thus the column is considered slender and second-order effects will need to be considered. The uncracked second moment of area of the concrete is determined as

$$I_c = \frac{950^4}{12} = 68 \times 10^9 \text{mm}^4$$

and the second moment of area of the steel box is calculated as

$$I_s = \frac{1000^3 \times 1000}{12} - \frac{950^3 \times 950}{12} = 15{\cdot}5 \times 10^9 \text{mm}^4$$

One can determine the effective rigidity in using equation (15) as

$$(EI)_e = 4650 \times 10^{12} \text{mm}^4$$

The critical buckling load is determined from equation (14)

$$N_{crit} = \frac{\pi^2 \times 4650 \times 10^{12}}{8000^2} = 717\,150 \text{ kN}$$

The moment magnifier for a column in a braced frame is determined from equation (11), where for uniform bending or single curvature the maximum value for moment gradient gives $\beta = 1{\cdot}0$ and $k_m = 1{\cdot}06$ so that the value of the moment magnifier is

$$\delta_b = \frac{1{\cdot}06}{1 - \dfrac{N}{717\,150}}$$

The loading line is therefore plotted and the initial eccentricity is calculated as

$$e_{total} = \frac{M_{level}}{N_{total}} = \frac{1000 \times 550}{25\,000} = 22\text{mm}$$

where

$$M_{level} = N_{level} \times e_{level}$$

as shown in Figure 19. The axial force from each level was calculated as 1000 kN. The eccentricity of the load on this level was 550 mm and the total axial force from 25 levels is 25000 kN. The loading line and eccentricity is magnified due to the presence of slenderness effects as shown in Figure 20. The failure load is characterized by the intersection of the loading line with the strength interaction diagram at N = 50 500 kN and M = 1150 kNm. The effects of slenderness are negligible in this example, which is typical of a multistorey building.

9. CONCLUSIONS

The method developed here has shown the influence of various material and geometric properties on both the strength and ductility of concrete-filled steel box columns. These are currently being validated with a set of experiments which will be reported in a future paper. A more rigorous numerical model to look at slender column behaviour will be developed in the future, incorporating both material and geometric nonlinearities. However, the importance of the study is that structural engineers and designers utilizing high-strength materials should make an assessment of ductility so that the structural response to overload may be determined.

This study has concentrated on the behaviour of the ductility, strength and stability of concrete-filled steel box columns. Design procedures are left to the individual, as these will be different in various codes of countries throughout the world as suggested by Bradford.[30]

10. FURTHER RESEARCH

Further research is required by conducting extensive experiments, which include determining slender column behaviour and the influence of composite steel–concrete connections on the effective lengths of these columns. Thus, adequate test data on composite connection properties is first necessary, and then the results of this may be used in nonlinear finite-element analyses which incorporate both material and geometric nonlinearities to identify overall framework behaviour.

Other areas of research include the fire behaviour of these unreinforced columns, which can be further identified by full-scale load testing and numerical models should be calibrated with such tests in order to allow the design and determination of concrete-filled steel columns for fire resistance.

ACKNOWLEDGEMENTS

The author would like to thank the Australian Research Council Small Grants Scheme for financial assistance with this project. This work is part of an ongoing research project on the behaviour of steel–concrete composite structural members at the University of Wollongong.

REFERENCES

1. R. M. Kowalczyk, R. Sinn and M. B. Kilmister (eds), *Structural Systems for Tall Buildings,* Council on Tall Buildings and Urban Habitat. McGraw-Hill, New York, U.S.A., 1993.

2. R. Q. Bridge and J. Webb, 'Thin walled circular concrete filled steel tubular columns', *Composite Construction in Steel and Concrete II, Proc. Engineering. Foundation Conf.*, 1992, pp. 634–649.
3. H. F. Austmeier, 'Extension of the Commerzbank Headquarters Frankfurt/Main', *Proc. Engng. Foundation Conf. on Composite Construction III*, Irsee, Germany, 1996, pp. 181–196.
4. C. M. Wise, H. W. Bridges, S. R. Walsh, C. J. Smith, P. T. Cross and P. A. Bailey, 'The new Commerzbank headquarters, Frankfurt, Germany', *The Structural Engineer, J. Instn Struct. Engrs, UK,* **74** (7), 111–122 (1996).
5. L. J. O'Brien and A. Di Scerni, 'Steel framed buildings—construction lessons learnt', *Australasian Structural Engineering Conf.*, Sydney, Australia, September 1994.
6. D. J. Oehlers and M. A. Bradford, *Composite Steel and Concrete Structural Members, Fundamental Behaviour,* Pergamon, 1995.
7. B. Uy and S. Das, 'Wet concrete loading of thin-walled steel box columns during construction of a tall building' *J. Construct. Steel Res.,* **42** (2), 95–119 (1997).
8. K. B. Watson and L. J. O'Brien, 'Tubular composite columns and their development in Australia', *Instn Engrs Australia, Structural Engineering Conf. Adelaide,* Australia, 1990, pp. 186–190.
9. B. Uy and S. Das, 'Bracing of thin-walled steel box columns during pumping of wet concrete in tall buildings', *Thin Walled Struct.,* Submitted for publication.
10. P. J. Terrey, M. A. Bradford and R. I. Gilbert, 'Creep and shrinkage in concrete filled steel tubes', *Tubular Structures VI, Proc. Sixth Int. Symp. on Tubular Structures,* 1994, pp. 293–298.
11. S. Morino, J. Kawaguchi and Z. S. Cao, 'Creep behavior of concrete-filled steel tubular members', *Proc. Engng. Foundation Conf. on Composite Construction III,* Irsee, Germany, 1996, pp. 514–525.
12. H. Nakai, A. Kurita and L. H. Ichinose, 'An experimental study on creep of concrete filled steel pipes', *Proc. 3rd Int. Conf. on Steel–Concrete Composite Structures,* 1991, pp. 55–60.
13. B. Uy and S. Das, 'Time effects in concrete filled steel box columns in tall buildings', *Struct. Design Tall Build.,* **6** (1), 1–22 (1997).
14. B. Uy, 'Strength and ductility of fabricated steel–concrete filled box columns', *Proc. Engng. Foundation Conf. on Composite Construction III,* Irsee, Germany, 1996, pp. 616–629.
15. B. Uy and M. A. Bradford, 'Elastic local buckling of steel plates in composite steel–concrete members', *Engng Structures, Int. J.,* **18** (3), 193–200, (1996).
16. B. Uy and M. A. Bradford, 'Local buckling of thin steel plates in composite construction: experimental and theoretical study', *Struct. & Build., Proc. Instn Civil Engrs,* **110,** 426–440 (1995).
17. B. Uy, 'Local and post-local buckling of concrete filled steel welded box columns', *To appear in Journal of Constructional Steel Research, An International Journal,* (In press).
18. R. Q. Bridge, M. D. O'Shea, P. Gardner, R. Grigson and J. Tyrell, 'Local buckling of square thin-walled steel tubes with concrete infill', *Proc. Int. Conf. Structural Stability and Design,* Sydney, Australia, 1995, pp. 307–314.
19. B. Uy and M. A. Bradford, 'Ductility and member behaviour of profiled composite beams—Part 1: Experimental study', *J. Struct. Engng, ASCE,* **121** (5), 876–882 (1995).
20. K. J. R. Rasmussen and G. J. Hancock, 'Plate slenderness limits for high strength steel sections', *J. Construct. Steel Res.,* **23,** 73–96, (1992).
21. K. J. R. Rasmussen and G. J. Hancock, 'Tests of high strength steel columns', *J. Construct. Steel Res.* **34,** 27–52 (1997).
22. Comité Européen du Béton—Fédération Internationale de la Précontrainte, *International Recommendations for the Design and Construction of Concrete Structures* 1970.
23. R. P. Johnson, *Composite structures of steel and concrete, Vol. 1, Beams, Slabs, Columns and Frames for Buildings,* 2nd edn, Blackwell Scientific, 1995.
24. M. A. Bradford, 'Design of short concrete-filled RHS sections', *Australian Civil Engng Trans.,* **33** (3), 189–194, (1991).
25. Structural Steel Development Group Newsletter, 'Hi tech erection for hi tech steel columns', *Structures, BHP Steel*, September 1989.
26. B. Uy and S. B. Patil, 'Concrete filled high strength steel box columns for tall buildings: behaviour and design', *Int. J. Struct. Design Tall Build.* **5** (2), 75–94, (1996).
27. H. Mochuziki, T. Yamashita, K. Kanaya and T. Fukasawa, 'Application of HT780 high strength steel plate to structural member of super high rise building—Part 1: Development of high strength steel with heavy gauge and welding process', *Proc. Fifth East Asia—Pacific Conf. on Structural Engineering and Construction, Building for the 21st Century,* Gold Coast, Australia, 1995, pp. 2283–2288.
28. Y. Hagiwara, A. Kadono, T. Suzuki, I. Kubodera, T. Fukasawa and Y. Tanuma, 'Application of HT780 high

strength steel plate to structural member of super high rise building—Part 2: Reliability inspection of the structure'.*Proc. Fifth East Asia–Pacific Conf. on Structural Engineering and Construction, Building for the 21st Century*, Gold Coast, Australia, 1995, pp. 2289–2294.
29. Standards Association of Australia, *AS3600 Concrete Structures*, Sydney, Australia, 1994.
30. M. A. Bradford, 'Design strength of slender concrete-filled rectangular steel tubes', *ACI Struct. J.* **93** (2), 229–235 (1996).
31. *Eurocode 4 (1994) Design of composite Steel and Concrete Structures*, DDENV 194-1-1, Draft for development.

