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Axial compressive strength of short steel and composite columns fabricated with high stength steel plate

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Abstract. The design of tall buildings has recently provided many challenges to structural engineers. One such challenge is to minimise the cross-sectional dimensions of columns to ensure greater floor space in a building is attainable. This has both an economic and aesthetics benefit in buildings, which require structural engineering solutions. The use of high strength steel in tall buildings has the ability to achieve these benefits as the material provides a higher strength to cross-section ratio. However as the strength of the steel is increased the buckling characteristics become more dominant with slenderness limits for both local and global buckling becoming more significant. To arrest the problems associated with buckling of high strength steel, concrete filling and encasement can be utilised as it has the affect of changing the buckling mode, which increases the strength and stiffness of the member. This paper describes an experimental program undertaken for both encased and concrete filled composite columns, which were designed to be stocky in nature and thus fail by strength alone. The columns were designed to consider the strength in axial compression and were fabricated from high strength steel plate. In addition to the encased and concrete filled columns, unencased columns and hollow columns were also fabricated and tested to act as calibration specimens. A model for the axial strength was suggested and this is shown to compare well with the test results. Finally aspects of further research are addressed in this paper which include considering the effects of slender columns which may fail by global instabilities.

Key words: columns; composite construction; high strength steel; steel structures; tall buildings.

1. Introduction

The design of tall building gravity load systems is greatly influenced by the ability to resist axial force with the smallest cross-sectional sizes available. Recent developments in the quality of high strength steel have seen it become extremely attractive for the design and construction of tall buildings in Australia and future landmark buildings are earmarked to utilise high strength steel in Japan. The benefits of the use of high strength steel can be utilised in a braced frame where the external spandrel frame is used to resist gravity loads alone. High strength steel is most efficient when it is allowed to develop its full yield stress. Thus high strength steel is efficient when local and overall buckling can be eliminated in a column design. This paper will summarise the previous applications of high strength steel in tall buildings in major cities throughout the world. The summary includes a description of the column type and grade of steel used to illustrate the methods in which high strength structural steel is

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being used. Whilst the term high strength steel is meant to describe structural steel of about 600-700 MPa, there is work reported in Europe, which refers to high strength steel being of the order of 450-500 MPa nominal yield strength in tension.

Based on the previous applications, a detailed experimental program was conducted which reflected current and future uses of high strength steel in composite columns. The experiments were based on the behaviour of high strength steel columns in pure compression and consisted of both bare steel sections and composite sections. These experiments will be described here and a numerical model for the calculation of the pure compressive strength will be presented and shown to provide a conservative estimate of the column cross-section strength.

The paper will conclude by highlighting the way forward and outline future research that is required to be conducted in order to allow the use of these columns in the development of international standards. This includes extending the present work to consider slender columns subjected to both uniaxial and biaxial bending in addition to compression.

2. Completed and planned projects

Previous projects, which have been completed and planned, are summarised in Table 1. This list will

Table 1 Projects utilising high strength structural steel

Building	City	Year completed	Number of storeys	Column type	Steel grade (MPa)
Grosvenor Place	Sydney	1988	50	Encased	690
Central Park	Perth	1989	50	Encased	690
300 Latrobe st.	Melbourne	1990	30	Encased	690
Star City	Sydney	1997	20	Encased	690
Shimizu SSHR	Tokyo	Proposed	120	Filled	690

Fig. 1 Star city, Sydney

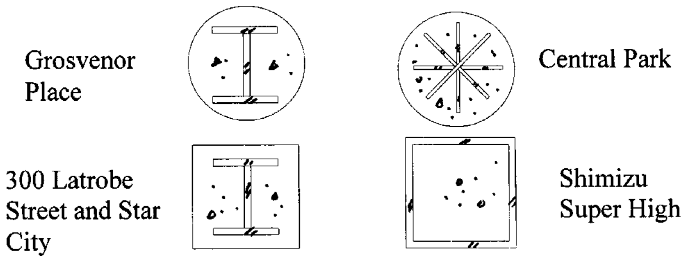

Fig. 2 High strength steel composite cross-sections

identify the type of projects and the potential benefits achieved from the use of high strength steel. In particular, this table reflects tall building projects in Australia where high strength steel has been used. In the design of Star City, Sydney the largest building project in Sydney since the Sydney Opera House, the major benefits derived from the use of high strength steel were in providing additional car space in the basement levels of the building, which is illustrated in Fig. 1. This was a mandatory requirement for the project specified by the Sydney City Council, (Davie 1995). The use of high strength steel in the other Australian buildings was justified in reducing column sizes and thus providing additional floor area and car park spaces in the building. This was therefore used on projects in Sydney, Melbourne and Perth in notable buildings such as Grosvenor Place, 300 Latrobe Street and Central Park, (Structural Steel Development Group 1989 and 1990). The Shimizu Super High Rise (SSHR), which is a proposed project in Tokyo, Japan, will use high strength steel in box columns for the exterior spandrel frame, (Council on Tall Buildings and Urban Habitat 1993). Fig. 2 also illustrates the cross-section geometries utilised for each of these projects.

3. Previous research

Previous research of high strength steel in flexural members has been undertaken by Haaijer (1961), Frost and Schilling (1964) and Suzuki *et al*. (1994). Each of these studies were undertaken on beams where the flanges and webs were constructed of materials of different strength characteristics. These studies were all concerned with bare steel beams and Uy and Sloane (1998) considered the use of high strength steel in composite beams.

Previous research into high strength structural steel for columns has been mainly carried out in the regions where it has been applied in practice and this includes research in both Australia and Japan. Firstly Rosier and Croll (1987) considered the benefits of high strength quenched and tempered steel being applied in structures such as bridges, buildings and silos. This study included consideration of the economics of the material over conventional mild structural steel and showed the significant advantages that could be derived from its use.

Rasmussen and Hancock (1992 and 1995) conducted tests on both high strength steel fabricated I-sections and box sections. These tests established local buckling slenderness limits for these high strength steel sections. Furthermore, slender columns were tested and the behaviour of these was compared with the slender column curves of the existing Australian Standard AS 4100-1990 (Standards Australia 1990). It was found that providing the local buckling slenderness limits were adhered to, then the slender column behaviour could be described using this standard developed specifically for mild structural steel.

Hagiwara *et al.* (1995) and Mochizuki *et al*. (1995) considered the behaviour of high strength structural steel for the application in super high-rise buildings in Japan. These studies considered the reliability inspection and the welding process for heavy gauge steel plate. These studies are pertinent to the application of the use of high strength steel in projects such as the Shimizu Super High Rise in Tokyo, Japan.

Uy (1996) considered the behaviour of concrete filled steel box columns filled with concrete. These studies considered the advantages derived from filling the sections with concrete to increase the local buckling stresses. Furthermore, the members were considered under combined bending and compression to assess the strength of short columns. The results of these columns were compared with columns designed with normal strength structural steel, to show the reduced cross-sectional dimensions able to be achieved. Furthermore, comparisons of the cross-sectional ductility were made and showed that composite members composed of high strength structural steel still had a large degree of reserve of strength after peak loading conditions were attained.

Uy (2001) conducted an extensive experimental programme on short concrete filled steel box columns, which incorporated high strength structural steel of Grade 690 MPa. The experiments were then used to calibrate a refined cross-sectional analysis method, which considered both the non-linear material properties of the steel, and concrete coupled with the measured residual stress distributions in the steel. The model and experiments were then compared with the existing approach of Eurocode 4 and it was found that certain modifications were necessary. The Eurocode 4 approach, which employs the rigid plastic analysis approach, was found to over predict the strength of the cross-sections. A modified analysis technique known as a mixed analysis was therefore developed and found to be in good agreement with both the test results and the refined analysis procedure. This model considers the concrete to be plastic and the steel to be elastic-plastic and provides a much more realistic design approach for sections utilizing high strength structural steel.

Bergmann and Puthli (2000) conducted an extensive experimental programme on short and slender high strength steel encased sections of 460 MPa grade steel subjected to combined compression and bending. These tests were then compared with the Eurocode 4 approach, which was found to be suitable for predicting the ultimate load for short columns. However, the results of the slender column tests proved to be inconclusive.

4. Experiments

This section outlines the test program undertaken which included column tests and extensive material property tests. The test set-up for the columns will be described and the results will then be presented. A general review and description of the failure modes will then be provided. The test program consisted of eight columns, of which four columns were fabricated I sections and four were fabricated box sections. The columns and their pertinent dimensions are illustrated in Fig. 3.

Each of the specimens were designed to satisfy the yield slenderness criteria for international steel codes. For the I sections the flange and web plate slenderness was given as

Flange Slenderness (I-Section)

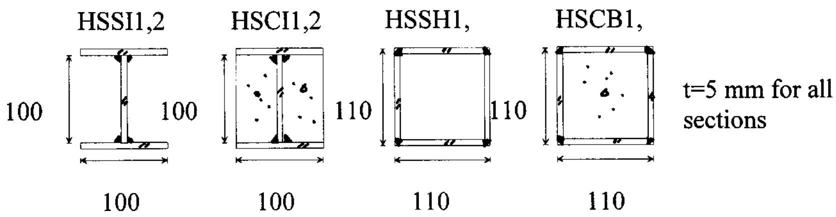

Fig. 3 Column cross-sections

$$\frac{b}{t}\sqrt{\frac{f_y}{250}} = \frac{47.5}{5}\sqrt{\frac{690}{250}} = 15.8$$

Web Slenderness (I-Section)

$$\frac{b}{t}\sqrt{\frac{f_y}{250}} = \frac{100}{5}\sqrt{\frac{690}{250}} = 33.2$$

For the box sections the plates slenderness was calculated as

Flange and Web Slenderness (Box Section)

$$\frac{b}{t}\sqrt{\frac{f_y}{250}} = \frac{100}{5}\sqrt{\frac{690}{250}} = 33.2$$

Now the limiting plate slenderness suggested by international codes for bare steel sections such as AS4100-1998 (Standards Australia) is

Flange Slenderness (I-Section)

$$\frac{b}{t}\sqrt{\frac{f_y}{250}} = 8$$

Web Slenderness (Box and I-Section)

$$\frac{b}{t}\sqrt{\frac{f_y}{250}} = 40$$

Whereas the limiting plate slenderness suggested by Uy and Bradford (1996) if one takes due account of the restraint offered by the concrete is

Flange Slenderness (I-Section)

$$\frac{b}{t}\sqrt{\frac{f_y}{250}} = 17.5$$

Web Slenderness (Box and I-Section)

Table 2 Tensile coupon tests

Specimen number	Yield stress, σ_y (MPa)	Ultimate stress σ_u (MPa)
1	765.3	809.7
2	781.3	816.8
3	796.4	808.9
4	793.7	830.9
Mean	784.2	816.6
Standard deviation	14.2	10.0

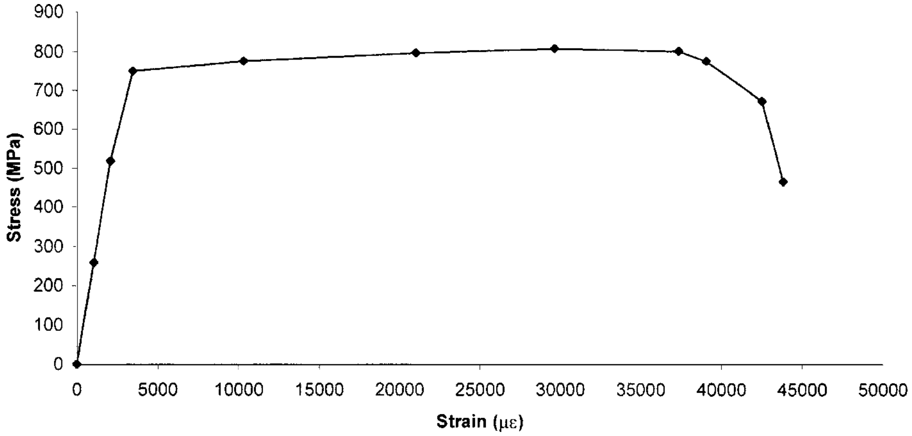

Fig. 4 Typical stress-strain curve for tensile coupon

$$\frac{b}{t}\sqrt{\frac{f_y}{250}} = 65$$

Thus based on the sections shown in Fig. 3, all sections obey the suggested slenderness criteria with the exception of the bare steel I-Sections.

4.1. Tensile coupon tests

To determine the stress-strain characteristics of the steel plate in tension a series of tensile coupons were produced from the virgin steel plate and tested in an Instron uniaxial testing machine. Pertinent data for these test coupons is provided in Table 2. Four tests were conducted with a mean value for yield stress of 784 MPa being established. Whilst high strength steel is not considered to have a defined strain-hardening region, the tests revealed an increase in stress after yielding and the mean ultimate stress of the material in tension was determined to be 817 MPa. A typical stress-strain diagram is given in Fig. 4 and the failure modes of the tensile coupons are shown in Fig. 5. Ductility can be observed both by the pronounced necking of the specimens, which achieved ultimate strains in excess of 40,000 $\mu\varepsilon$ (4%) in all cases.

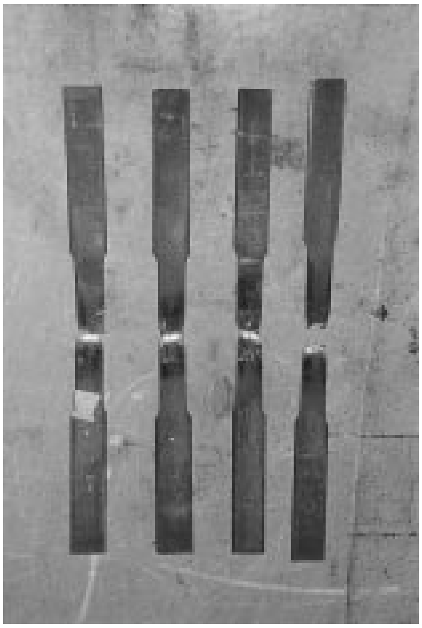

Fig. 5 Typical failure mode for tensile coupon

Table 3 Stub column tests

Specimen number	Yield stress, σ_y (MPa)	Ultimate stress σ_u (MPa)
1	757.3	833.0
2	757.3	839.1
3	750.0	779.0
Average	754.9	817.0
Standard deviation	4.2	33.1

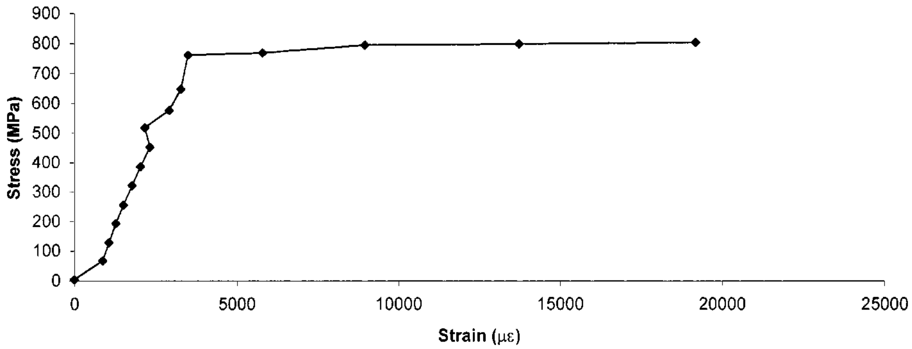

Fig. 6 Typical stress-strain curve for stub column

4.2. Compressive stub column tests

The stress-strain characteristics can vary in tension and compression, which can be due to the effects of residual stresses. To try and identify these differences a series of stub column tests were undertaken.

Fig. 7 Typical stub column failure

Table 4 Concrete compressive strengths

Test specimen	Age in days	Av. diameter (mm)	Failure Load (kN)	Compressive strength (MPa)	Av. Compressive strength (MPa)
1	7	99.5	332	42.3	
2	7	102.0	302	38.5	40.4
3	14	101.5	385	47.6	
4	14	101.0	404	50.4	49.0
5	28	101.5	385	47.2	
6	28	102.0	457	55.9	51.6

These stub column tests were able to establish a clear reduction in the mean yield stress of at least 30 MPa, which could be attributed to the effects of residual stresses. However, the ultimate stress of the stub column tests was virtually identical to the tensile coupon tests with a mean value of 817 MPa being attained in both. Table 3 summarises the results of the stub-column tests. Fig. 6 shows a typical stress-strain diagram for the tests and a typical specimen is shown in Fig. 7. The initial slope of the stress-strain diagram is caused through the initial setting of the specimen in the plaster. However, the measured strains in the stub column specimens are unaffected by this after the plaster has been compressed.

4.3. Concrete cylinder tests

Concrete cylinders were cast and tested to help ascertain the characteristic compressive strength of the concrete. In total six cylinders were tested and the relevant parameters and results are summarised in Table 4. The mean compressive strength of the concrete was thus determined at 7, 14 and 28 days. The mean compressive strength of the columns at the time of testing was estimated as 50 MPa as all the tests were conducted between 14 and 28 days after the concrete was cast.

4.4. Column tests

Eight columns were tested in pure compression in an Amsler 5,000 kN capacity compression testing

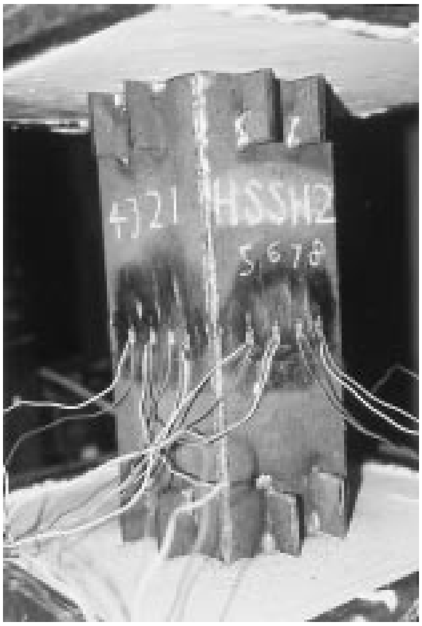

Fig. 8 Photograph of column test set-up

facility. The column test set-up is illustrated in Fig. 8, which shows the general characteristics of the testing machine plattens and the instrumentation used in the testing. The test set-up highlights the end conditions, which were provided to ensure a uniform loading surface to the column. Using steel plates with a recessed edge, the plates were filled with a very stiff plaster and a small preload was applied until the plaster cured and reached an appropriate stiffness. In addition to this strain gauges were used on the steel surfaces, which were useful in tracing the load-strain characteristics for both the determination of yielding and local buckling of the steel plates. Furthermore, linear varying displacement transducers (LVDT's) were used to measure the load-axial shortening characteristics, which was also useful in the determination of yielding and ultimate loading of the column members.

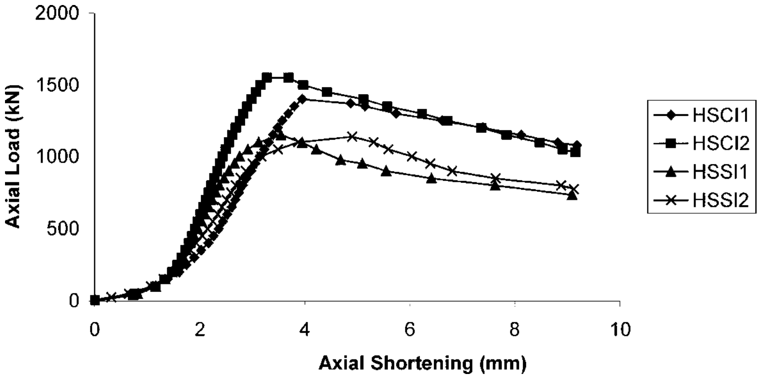

Fig. 9 Load-deflection results of fabricated I-section columns

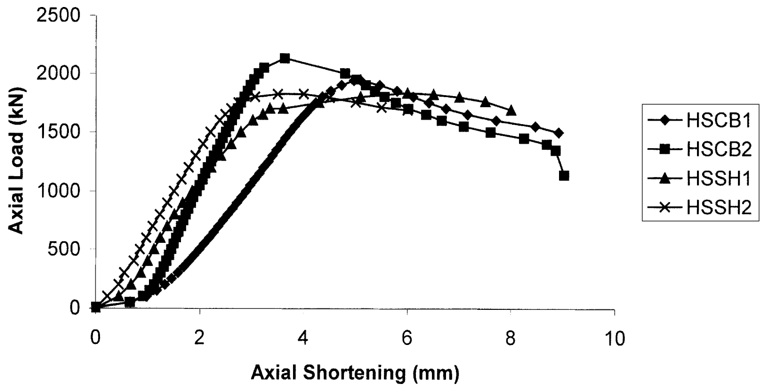

Fig. 10 Load-deflection results of fabricated box section columns

4.5. Load-deflection results

The axial load-axial shortening of each column was recorded and these results were useful in being able to ascertain the point at which yielding took place and the point of ultimate failure, which was usually characterised by concrete crushing and softening. Figs. 9 and 10 illustrate typical load-deflection results for the columns tested. These diagrams also show the initial set of the plaster which is made apparent by the low slope of the sections after initial loading. For both the fabricated I section and

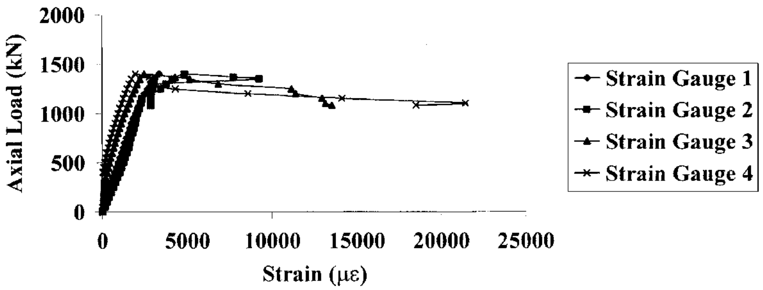

Fig. 11 Load-strain results of HSCI1

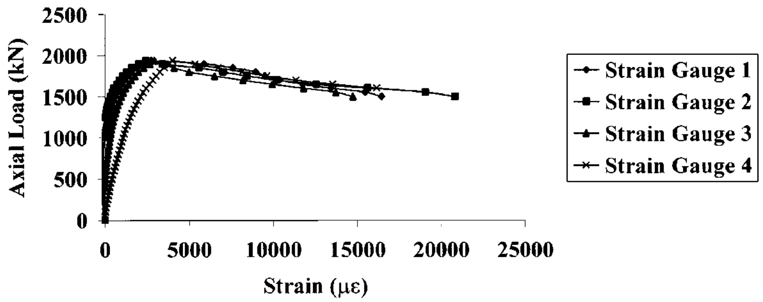

Fig. 12 Load-strain results of HSCB1

Fig. 13 Failure modes of columns

box section columns, one can see that the composite sections have a larger stiffness, as well as achieving a larger ultimate capacity. Furthermore, the apparent ductility of both the bare still sections and the composite sections is shown to be quite adequate with a significant post peak reserve of strength being displayed.

4.6. Load-strain results

The load strain results were used to identify yielding of the steel sections in compression, and the strain gauges also proved useful in identifying the onset of inelastic local buckling. Figs. 11 and 12 show a set of typical load-strain results for the columns tested. The fabricated I section columns were designed so that the plates were compact for local buckling, however inelastic local buckling was still evident after the ultimate load was reached and this is defined by the erratic behaviour of the strain gauges as shown in Figure 11. The fabricated box columns were also designed with compact plates, however, inelastic local buckling was more controlled and greater stress redistribution capability was noted by the smooth nature of the strain gauges after the peak load was reached as shown below.

4.7. Failure modes

All columns were tested in pure compression and thus failure was essentially a primary compression mode. Failure was initiated by compressive yield since most plate sections were compact. Once yielding began, concrete crushing and inelastic local plate buckling of the plate elements usually followed this. Whilst the failure mode was primary compressive each of the members displayed a significant reserve of strength and thus highlighted a ductile failure plateau. Fig. 13 highlights the failure mode for the bare steel I section and the concrete filled I section. This diagram shows the presence of local buckling and the reduced wavelength produced by the concrete infill. Fig. 14 shows the failure modes for the hollow steel box column and Fig. 15 shows the failure modes for six of the

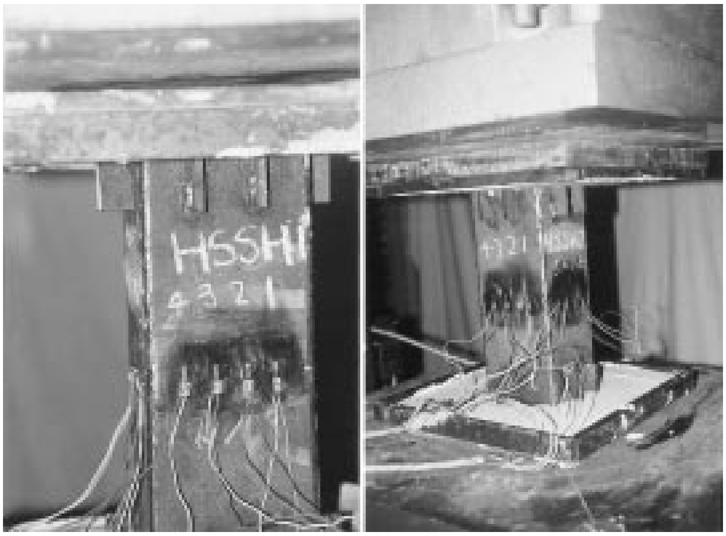

Fig. 14 Failure modes of columns

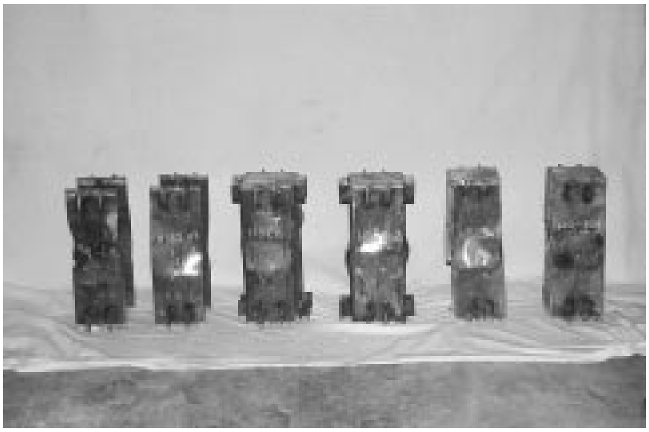

Fig. 15 Failure modes of columns

columns tested in this paper. These photographs highlight the local buckling modes for both the bare steel and composite sections as well as highlighting the concrete crushing for the composite sections.

5. Compressive strength model

All the columns in this paper were tested in pure compression and in order to predict the strength of these members an axial compressive strength model is proposed which considers both the steel and concrete contributions to axial strength. The model shown in Eq. (1) was used to provide a prediction of the axial strength of each of the columns.

Table 5 Strength comparisons

Test	Name	A_c (mm^2)	A_s (mm^2)	f_c (MPa)	f_y (MPa)	$N_{u.test}$ (kN)	$N_{u.theory}$ (kN)	$N_{u.test}/N_{u.theory}$
1	HSSI1	0	1,500	NA	750	1,163	1,125	1.03
2	HSSI2	0	1,500	NA	750	1,140	1,125	1.01
3	HSCI1	9,500	1,500	50	750	1,408	1,600	0.88
4	HSCI2	9,500	1,500	50	750	1,590	1,600	0.99
5	HSSHI1	0	2,100	NA	750	1,644	1,575	1.04
6	HSSHI2	0	2,100	NA	750	1,561	1,575	0.99
7	HSCB1	10,000	2,100	50	750	1,940	2,075	0.93
8	HSCB2	10,000	2,100	50	750	2,132	2,075	1.03
							Mean	0.99
							Standard Deviation	0.06

NA-not applicable

$$N_u = N_{uc} + N_{us} \tag{1}$$

where N_u is the ultimate axial strength of the composite column, $N_{uc} = f_c.A_c$ is the concrete contribution to axial strength and $N_{us} = f_y.A_s$ is the steel contribution to axial strength. This is very similar to the suggested model of Eurocode 4 (British Standards Institution 1992) without the use of partial safety factors.

6. Comparisons

Table 5 summarises the specimen names, pertinent geometric and material properties and results of the tests. The strength of each of the tests, $N_{u.test}$ was determined from the peaks of the load-deflection graphs, whilst the theoretical value for ultimate load $N_{u.theory}$ was determined using Eq. (1). The ratio of the test results to theoretical results is also calculated in Table 5 for each specimen.

The theoretical results generally provide a conservative estimate of strength for the steel section columns. Furthermore, the composite section columns are also well represented by the model. There is a slight value of non-conservatism in the strength determination of the composite columns, which could be due to the assumed maximum concrete stress. Eurocode 4 (British

Table 6 Strength comparisons

Test	Name	A_c (mm^2)	A_s (mm^2)	E_c (MPa)	E_s (GPa)	k_{test} (kN/mm)	$k_{average}$ (kN/mm)
1	HSSI1	0	1,500	NA	200	700	
2	HSSI2	0	1,500	NA	200	550	625
3	HSCI1	9,500	1,500	35,500	200	650	
4	HSCI2	9,500	1,500	35,500	200	950	800
5	HSSHI1	0	2,100	NA	200	760	
6	HSSHI2	0	2,100	NA	200	690	725
7	HSCB1	10,000	2,100	35,500	200	1040	
8	HSCB2	10,000	2,100	35,500	200	900	970

NA-not applicable

Standards Institution 1994) suggests a maximum stress equivalent to the cylinder stress be used and this generally accounts for some confinement effect. The maximum stress utilised herein was equal to the cylinder strength and thus illustrates that some confinement may be present. However, it may be necessary to impose a factor to account for creep in the composite columns and thus a value less than the cylinder strength may need to be applied for design. The mean value of the ratio of strength shows that the model overestimates the strength of the columns by 1% and there is a 6% standard deviation associated with the model. The model is therefore shown to be quite acceptable for use in design.

The compressive stiffness of the sections can be determined for both the steel sections and the composite sections. Experimentally the axial stiffness is determined by calculating the slope in the linear part of the load-deflection curve. The results of experimental stiffnesses are given in Table 6. Whilst one may be able to determine the theoretical stiffness, this has not been undertaken here since an accurate determination of concrete elastic modulus was not obtained and the localised shortening was not measured. The test set up shown was able to measure relative cross-head shortening which is not an accurate method for determining the stiffness. Table 6 shows the calculated test stiffnesses and these are presented to give an indication of the increase obtained by concrete infill. For both the I section and box-section members an increase of 28 and 34% respectively was achieved when concrete infill was used. A theoretical determination of the axial stiffness will reveal that the increased stiffness is expeeted to be approximately between 80-100% for the different sections and thus suggests that movement in the plaster and plattens of the test set up prevents an accurate determination of this. A comparison between experimental and theoretical stiffness is thus ommitted from the discussion.

7. Conclusions

This paper has described the advantages of the use of high strength steel in tall buildings. A brief overview of projects to utilise these structural forms has been provided and the reasons for their use have been given. In particular the use of high strength steel plate in composite column forms was identified as a potentially useful application. An extensive experimental program was conducted to consider the axial compressive behaviour of both high strength steel columns and high strength steel composite columns. A compressive strength model was then proposed and found to be conservative in its prediction of the axial compressive strength of all the columns tested. Further research is however necessary to consider the behaviour of these columns in combined bending and compression. Furthermore the effects of interaction buckling due to local and global buckling also need to be considered as part of future research in this area.

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