BEHAVIOUR AND DESIGN OF FABRICATED HIGH STRENGTH STEEL COLUMNS SUBJECTED TO BIAXIAL BENDING PART I: EXPERIMENTS

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ABSTRACT: This paper considers the behaviour of hollow and concrete filled steel columns fabricated with high strength structural steel plate and subjected to biaxial bending. The columns considered in this paper are fabricated from high strength quenched and tempered structural steel plate of nominal yield stress of 690 (N/mm^2). An extensive series of experiments on short and slender hollow and concrete filled steel composite columns has been carried out under biaxial bending. The principal parameter that has been considered in the test program is the slenderness of the component plates. The experiments did reveal the beneficial effect of the use of concrete infill on delaying the onset of local buckling and thereby the post-local buckling behaviour of the columns. This set of experiments is part of an on-going program of research which has been conducted at the Universities of Wollongong and New South Wales over the last decade. Further research will include the consideration of these types of members as part of overall structural assemblages, as well as utilising these members under high temperature and cyclic loading conditions to simulate fire or blast loading conditions respectively.

Keywords: biaxial loading, buckling, composite structures, high strength steel, local buckling, steel columns, steel construction, tall buildings, welded columns

1. INTRODUCTION

Recent developments in the metallurgical qualities of high strength steel have seen it become extremely attractive for the design and construction of tall buildings. The benefits of the use of high strength steel can be utilised in a braced frame of a tall building where the external spandrel frame is used to resist predominantly gravity loads. 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. The term high strength steel is meant to describe structural steel of about 600-700 N/mm^2 nominal yield stress in tension.

A very effective method for optimising tall building gravity load systems is to reduce the cross-sectional dimensions of the vertical columns. This paper will summarise previous research findings for the use of high strength quenched and tempered structural steel. A brief summary of previous applications of high strength steel in tall buildings throughout the world is also given. A detailed experimental program was conducted to evaluate the behaviour of short and slender high strength steel and steel-concrete composite columns loaded biaxially. The major application for this would be in assessing corner columns in buildings and also compressive struts in trusses which are subjected to lateral loads about two orthogonal axes. These experiments will be fully detailed herein. The paper concludes by highlighting future research that is required to be conducted in order to promote the future use of these columns and for the development of international standards.

2. PREVIOUS RESEARCH

A brief history of research of high strength steel sections is given herein, which mainly focuses on quenched and tempered structural steel plate used for fabricated steel and steel-concrete composite sections.

Rasmussen and Hancock [1, 2] conducted tests on both high strength steel fabricated I-sections and box sections with a nominal yield stress of 690 (N/mm^2). These tests established local buckling slenderness limits for 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 4100-1998 [3]. It was found that providing these local buckling slenderness limits were adhered to, then the slender column behaviour could be predicted using this standard developed specifically for mild structural steel. Sivakumaran and Yuan [4] considered slenderness limits and ductility of steel sections fabricated with high strength steel with nominal yield stresses between 300 and 700 (N/mm^2) respectively. The test programme involved testing twelve W shaped stub column sections with the objective being to determine the compression flange strength and strain ductility of sections of different steel grades.

Uy [5] presented the results of steel and composite sections using high strength structural steel of nominal yield stress 690 (N/mm^2). These sections constructed as stubby columns were subjected to concentric axial compression. A theoretical model to predict the axial strength of these columns was provided and shown to be in good agreement with the models suggested by Eurocode 4 [6] for encased and concrete filled sections. Uy [7] conducted an extensive experimental programme on short concrete filled steel box columns, which incorporated high strength structural steel of Grade 690 (N/mm^2). The Eurocode 4 approach [6], which employs the rigid plastic analysis method, was found to over predict the strength of the cross-sections. A modified 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 utilising high strength structural steel, particularly when large flexural loads are present.

Mursi and Uy [8] conducted further research on high strength steel box columns filled with concrete. The study consisted of four short columns and four slender columns to consider both the strength and stability aspects of steel-concrete composite high strength columns. The results of this study, showed that further refinement or adjustments need to be made to the Eurocode 4 approach, to allow for the effects of high strength steel particularly when large flexural loads are present. More recently, Mursi, Haedir and Uy [9] completed an experimental program on short steel sections and concrete filled high strength steel short columns subjected to biaxial bending. The experiments were compared with the existing Australian, European and American Standards and found to be in good agreement. In this paper a more extensive experimental program has been completed on slender steel sections and concrete filled high strength steel columns subjected to biaxial bending under slender member condition. The experiments were compared with the existing Australian, European Standards and the LRFD specification and found to be in good agreement in the companion paper.

3. EXPERIMENTS

This experimental study looks into the behaviour of hollow and concrete-filled high strength steel fabricated columns. The aim was to study the coupled effects of local and global buckling in hollow and composite columns. An extensive series of experiments were carried out composed of high strength structural steel. Columns were tested to the peak and post peak load under biaxial bending with load eccentricities applied to the column ends in both directions, (See Figure 8). Longitudinal fillet welds of 5 mm throat thickness along the full length of the columns were utilised to fabricate the steel casings. Details of the short and slender columns and equipment used to test the columns are provided. Material property tests for both the steel and concrete are also outlined herein. The following sections provide an explanation of the detailed experimental program of hollow and composite columns under biaxial bending loading conditions.

3.1 Testing Procedures Used for the High Strength Steel Columns

All short and slender columns were tested in a vertical self-straining compression testing machine with a 5000 kN capacity loading jack in the Heavy Structures Laboratory at The University of New South Wales. A load eccentricity was applied to each column. Column specimens tested in the testing frame were subjected to a deformation-controlled position of the ram for all tests using this machine. This enabled the ascending and descending portions of the load deformation graphs to be recorded. Digital recording facilities were available to record the applied shortening and other deformations of the specimens.

3.1.1 Orthogonal knife edge assembly

To prevent friction at the column ends, the load was applied by a set of loading plates with orthogonal knife edges. The use of knife edge end supports was to ensure free rotations in the two orthogonal planes to be achieved in the experiments. Johnson and May [10] used a knife edge assembly to conduct biaxial experiments on columns, whilst Kilpatrick and Rangan [11] used a knife edge assembly to apply uniaxial loading on columns. They determined that the use of the knife edge in column tests was important to define the line of action of the force accurately. When placing the column and the knife edge assemblies, special attention was required to verify the correct position of the column before load was applied. The design work for the knife edge system was undertaken as part of the research project for the column testing device. The use of orthogonal knife edges in this project is unique, and distinguishes itself from previous research projects which use spherical bearings. These orthogonal knife edges were designed for working loads of up to 5000 kN and to provide rotational freedom about two axes of bending of the column. Detailed drawings of the assembly at the bottom of the column and an isometric view of the crossed knife edge assemblies are provided in Figures 1, 2 and 3 respectively.

3.1.2 Instrumentation

Electric resistance strain gauges were used to measure the surface strains of the steel section for each column. In all tests, strains were recorded at the mid-length of the column. A total of ten strain gauges were used with the objective to plot the strain distribution across the section at different load levels. Three strain gauges were accurately positioned on each compression side and two strain gauges were positioned on each tension side. There were more strain gauges placed on the compression side as local buckling was expected to occur in these areas. Figure 4 shows the location of the strain gauges around the perimeter of the steel casing. Both the electronic strain gauges and LVDTs were connected to a datalogging system which was controlled by a personal computer. The load was applied to the top of the short columns by a hydraulic jack. The applied load was recorded by a matching load cell, which was connected to the datalogging equipment. LABVIEW Software was used to graphically monitor the relationships between loading and other parameters. All readings and electrical output information were recorded.

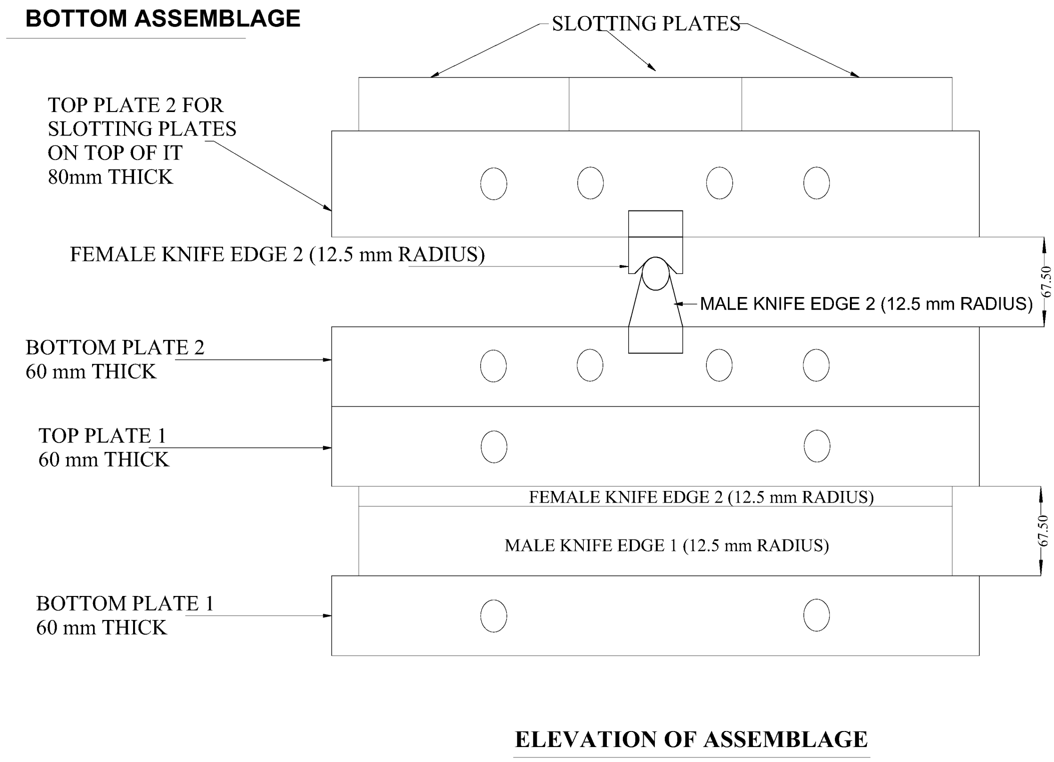

Figure 1. Orthogonal Knife Edge Assembly for Biaxial Bending

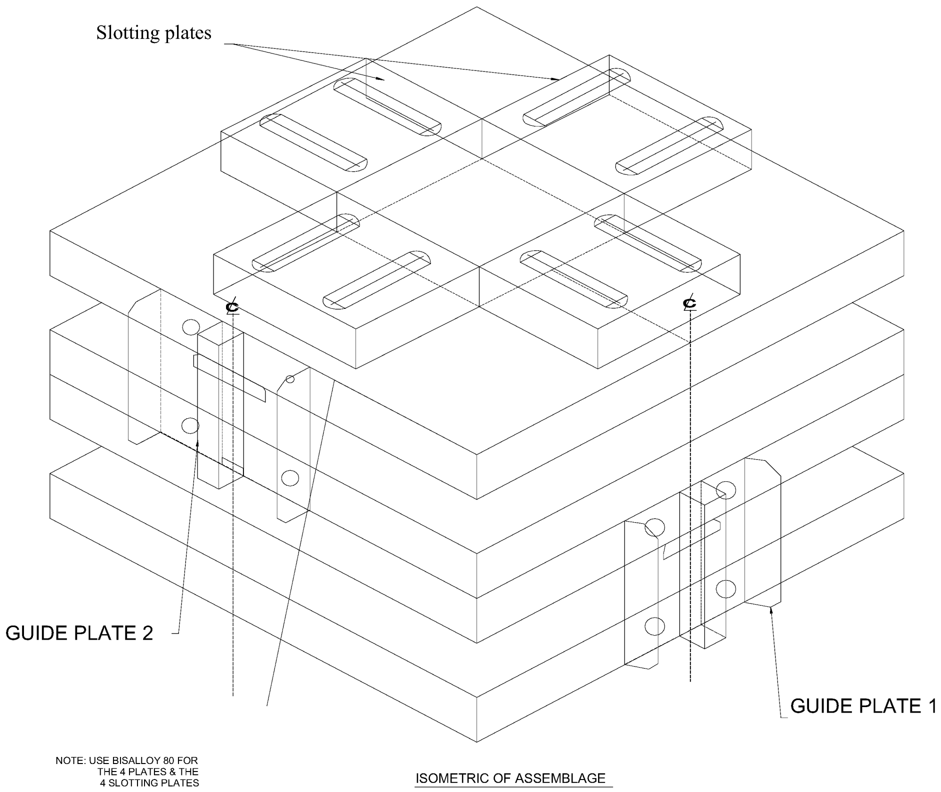

Figure 2. Isometric View of Orthogonal Knife Edge Assembly

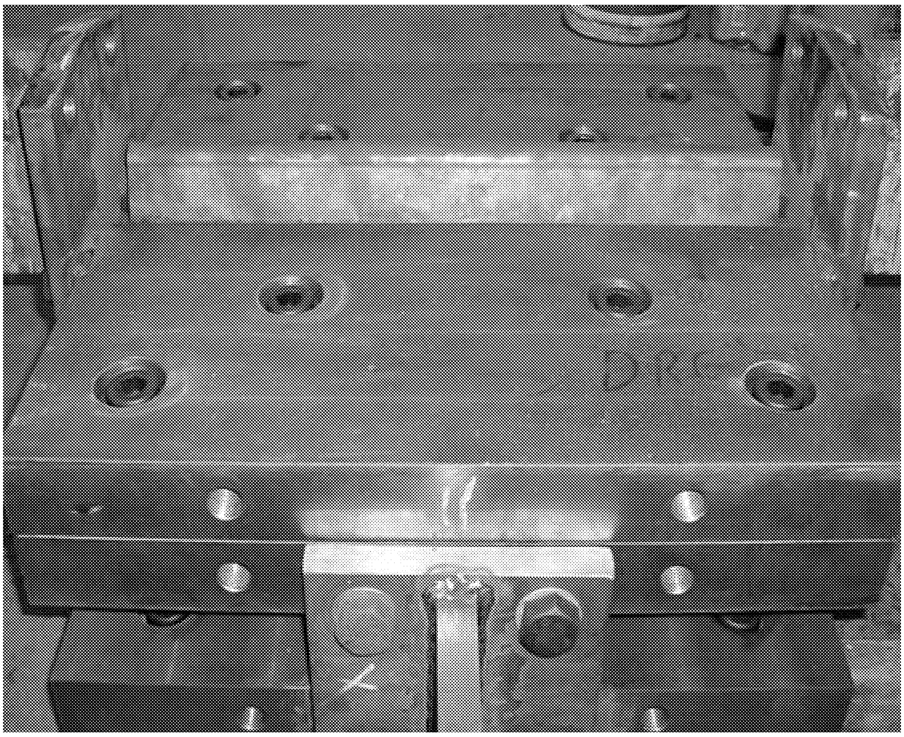

Figure 3. Orthogonal Knife Edge Bottom Assembly

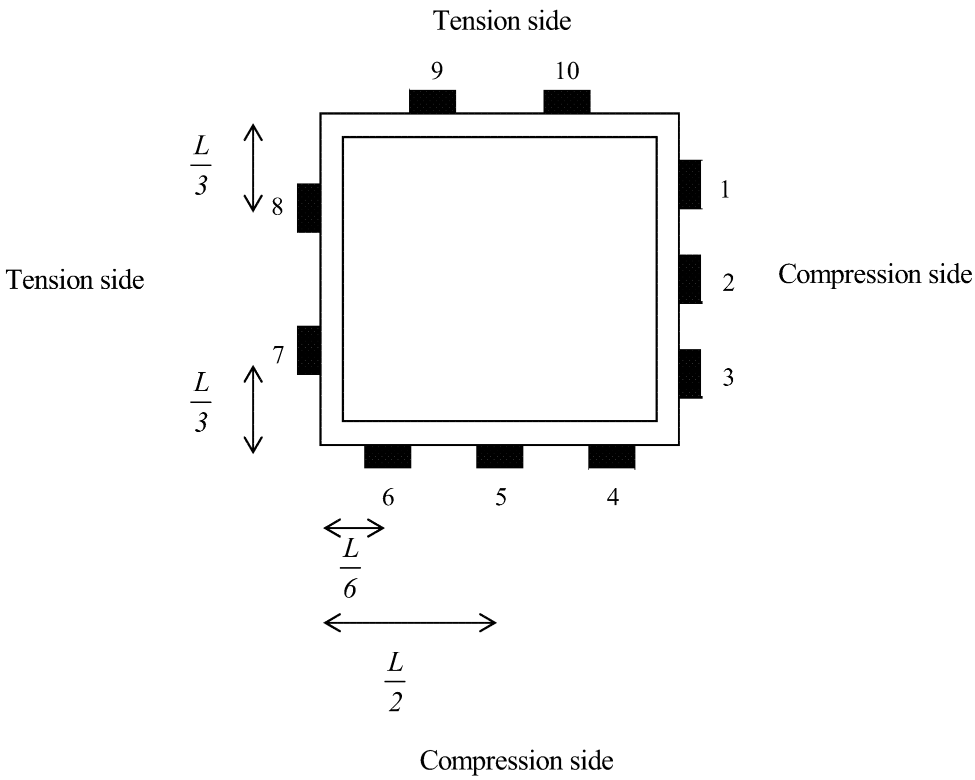

Figure 4. Arrangement of Strain Gauges on the Short and Slender Columns

3.2 Short High Strength Steel Column Test Series

The experimental program consisted of fabricating, constructing and testing eight high strength steel sections. Four of the sections were hollow and denoted as SH-H, whilst four of the sections were concrete filled and denoted as SH-C. The variation of the plate slenderness was a major consideration in the test series. For all of the hollow sections the non-dimensional plate slenderness was equal to or greater than 35 and thus all of the sections were considered slender. For the concrete filled steel sections the plate slenderness provides both compact and slender cross-sections and thus the effects of local buckling could be properly examined. Table 1 provides full details of all eight tests, with nominal width to thickness ratios *(b/t)* of 25, 35, 45 and 55. The actual *(b/t)* ratios for the component steel plates of the columns are stated in Table 1. All columns had a length to depth ratio (*L/d*) between 3 and 4. A value of greater than three was chosen so that local buckling could be adequately taken into account in the short column behaviour. Furthermore, all of the columns had an eccentricity to depth ratio between 8 and 9 % as illustrated in Table 1.

Table 1. Geometric Properties for Short Hollow and Concrete Infill Columns

Specimen	b (mm)	d (mm)	L (mm)	t (mm)	e	$\frac{b}{t}$	$\frac{b}{t}\sqrt{\frac{f_y}{250}}$	$\frac{L}{d}$	$\frac{e}{d}$
SH – H 110	119.9	119.7	430.0	5	10	24.0	39.9	3.59	0.084
SH – H 160	169.9	170.3	579.3	5	15	34.0	56.5	3.41	0.088
SH – H 210	219.6	220.6	730.5	5	20	43.9	72.9	3.31	0.090
SH – H 260	269.8	269.7	880.5	5	25	54.0	89.6	3.26	0.093
SH – C 110	119.9	120.1	431.5	5	10	24.0	39.9	3.58	0.083
SH – C 160	170.0	169.5	581.8	5	15	34.0	56.5	3.42	0.088
SH – C 210	220.0	220.2	730.5	5	20	44.0	73.1	3.32	0.091
SH – C 260	269.3	270.1	880.5	5	25	53.9	89.5	3.26	0.093

3.3 Material Properties

3.3.1 Concrete cylinder compressive strength

This section reports the results of the concrete compressive strength obtained from the concrete cylinder tests. The tests were carried out in accordance with the provisions of AS 1012.9 [12]. Three concrete cylinders were tested at 14, 21 and 28 days respectively. The average concrete cylinder strength is given in Table 2.

It is noted that the concrete compressive strength is quite low and it was chosen as not to provide significant axial strength, but rather to provide post-local buckling strength to the steel section and also to increase the stiffness of the section. Furthermore, the use of lower strength concrete also provides greater strain compatibility at the peak stress for the two materials. The use of high strength concrete may not allow the peak stress of the steel to be achieved and thus render the high strength steel ineffective for the strength limit state.

Table 2. Concrete Compressive Strength Tests

Sample	Diameter (mm)			Area	Load (kN)	f_c (N/mm^2)	Average
	D_1	D_2	Average	A (mm^2)			f_c (N/mm^2)
1	152.1	152.5	152.3	18222.3	304	16.68	
2	152.2	152.0	152.1	18164.9	320	17.62	17.31
3	152.5	152.5	152.5	18275.0	322	17.62	
4	152.4	152.2	152.3	18217.5	354	19.43	
5	152.5	152.0	152.3	18210.4	370	20.32	19.85
6	152.4	151.9	152.1	18172.1	360	19.81	
7	152.2	152.0	152.1	18164.9	386	21.25	
8	151.9	152.2	152.0	18153.0	386	21.26	21.03
9	152.1	152.2	152.2	18181.7	374	20.57	
10	152.2	152.2	152.2	18188.8	416	22.87	
11	152.1	152.1	152.1	18169.7	416	22.90	22.88

3.3.2 Tensile coupon tests

Four tensile coupon tests were conducted to determine the tensile stress-strain relationship of high strength steel, and the results are summarised in Table 3. In accordance with the provisions of AS1391 – 1991 [13] the strain rate specified is to be within the range of 2.5 x 10^{-4} to 2.5 x 10^{-3} s^{-1}. Hence, a cross-head rate of 5 mm/min was adopted for the subsequent tensile coupons tests and the strain rate was calculated to be 1.67 x 10^{-3} s^{-1}.

Table 3. High Strength Steel Tensile Coupons Test

Specimen	Cross-sectional area, A_{tc} (mm^2)	Yield stress, σ_y (N/mm^2)	Ultimate stress, σ_u (N/mm^2)
TC 1	191.5	685.2	731.1
TC 2	191.6	699.3	747.0
TC 3	192.9	733.2	780.4
TC 4	190.7	687.2	758.3
Mean	191.7	701.2	754.2
Standard deviation	0.77	19.2	17.9

Figure 5 shows the stress-strain relationships for high strength structural steel in tension. The curves show an increase in stress after yielding and an average yield stress value was calculated as 701 N/mm^2. The mean ultimate stress of the tensile coupons was determined to be 754 N/mm^2. The average yield strain ε_y was determined as 3466 $\mu\varepsilon$ for the nominal 5 mm thick high strength steel plate.

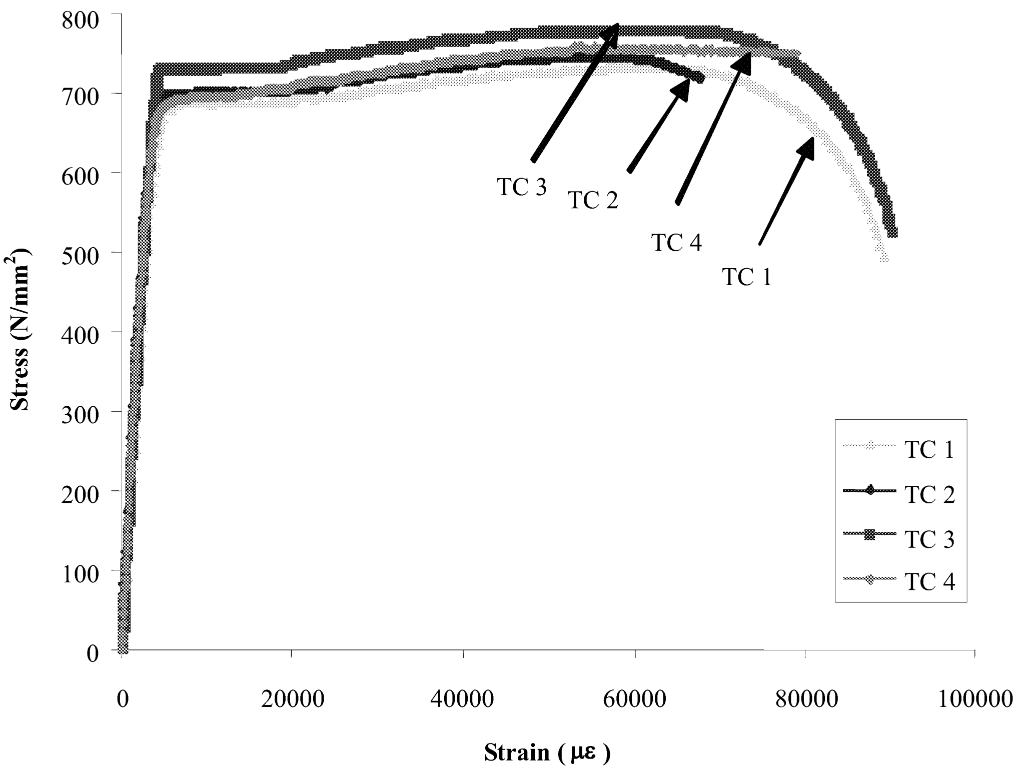

Figure 5. Stress-strain Relationships for High Strength Steel Tensile Cupons

3.3.3 Compression tests of stub hollow columns

The stub columns were initially tested under load control before reaching their peak load. During the tests, the strains registered on the data logger were recorded manually at different load levels. After reaching their ultimate load, the specimens were tested under a displacement rate in order to achieve the post-failure behaviour of the specimens with greater accuracy. The displacement rate was set at 0.5 mm/min and then to 1 mm/min. From the stress-strain curves, the compressive yield stresses can be determined as the 0.2% proof stress.

Table 4. Material Properties of the Steel Plate in Compression

Specimen	Test type	Actual values			
		b, d (mm)	t (mm)	E (N/mm^2)	F_{yc} (N/mm^2)
ST 1 - 50	Compression	59.6	5	234223	755.8
ST 2 - 50	Compression	60.0	5	261697	762.5
ST 3 - 50	Compression	60.1	5	239428	765.0
ST 4 - 50	Compression	59.7	5	182669	775.6
ST 1 - 70	Compression	79.0	5	171736	733.8
ST 2 - 70	Compression	79.9	5	231596	772.1
Mean				220225	760.8

In Figure 6, the stress-strain relationships of the stub columns are illustrated. The compressive yield stresses of the specimens were determined as the 0.2% proof stresses using the stress-strain curves. In Figure 7, the compressive and tensile yield stresses of the steel plate obtained are 760 N/mm^2 and 701 N/mm^2, respectively. Rasmussen and Hancock [1], quoted yield stresses of 750 N/mm^2 and

670 N/mm^2 were obtained for the compression coupon and tension coupon tests, respectively. Table 4 summarises the material properties established from the compressive tests. As seen from the table, the mean of the Young's modulus obtained in each series of the stub column tests is 220, 225 N/mm^2.

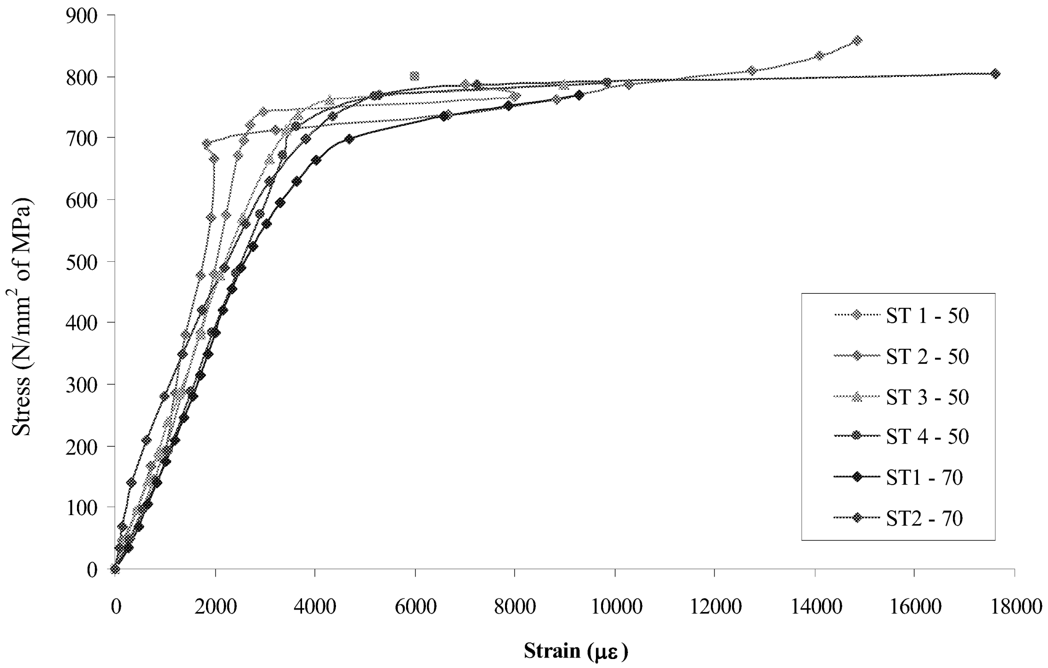

Figure 6. Stress-strain Curves for Stub Columns in Compression

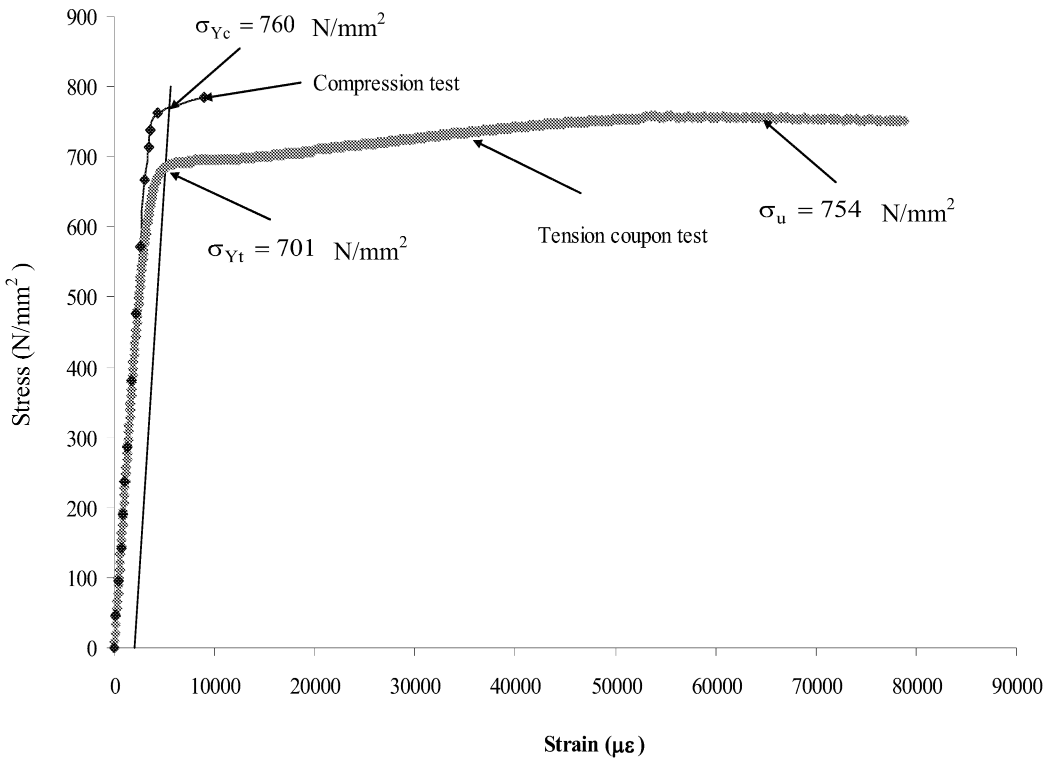

Figure 7. Stress-strain Curves for High Strength Steel Plate in Tension and Compression

3.4 Test Set-up

The test set-up for the short columns is illustrated in Figure 8, which highlights the specimen, loading plates for applying the load about two orthogonal axes, together with the jack and instrumentation. All the loading plates in this experimental program were heat treated and case hardened in order to allow them to achieve a load capacity of approximately 5000 kN.

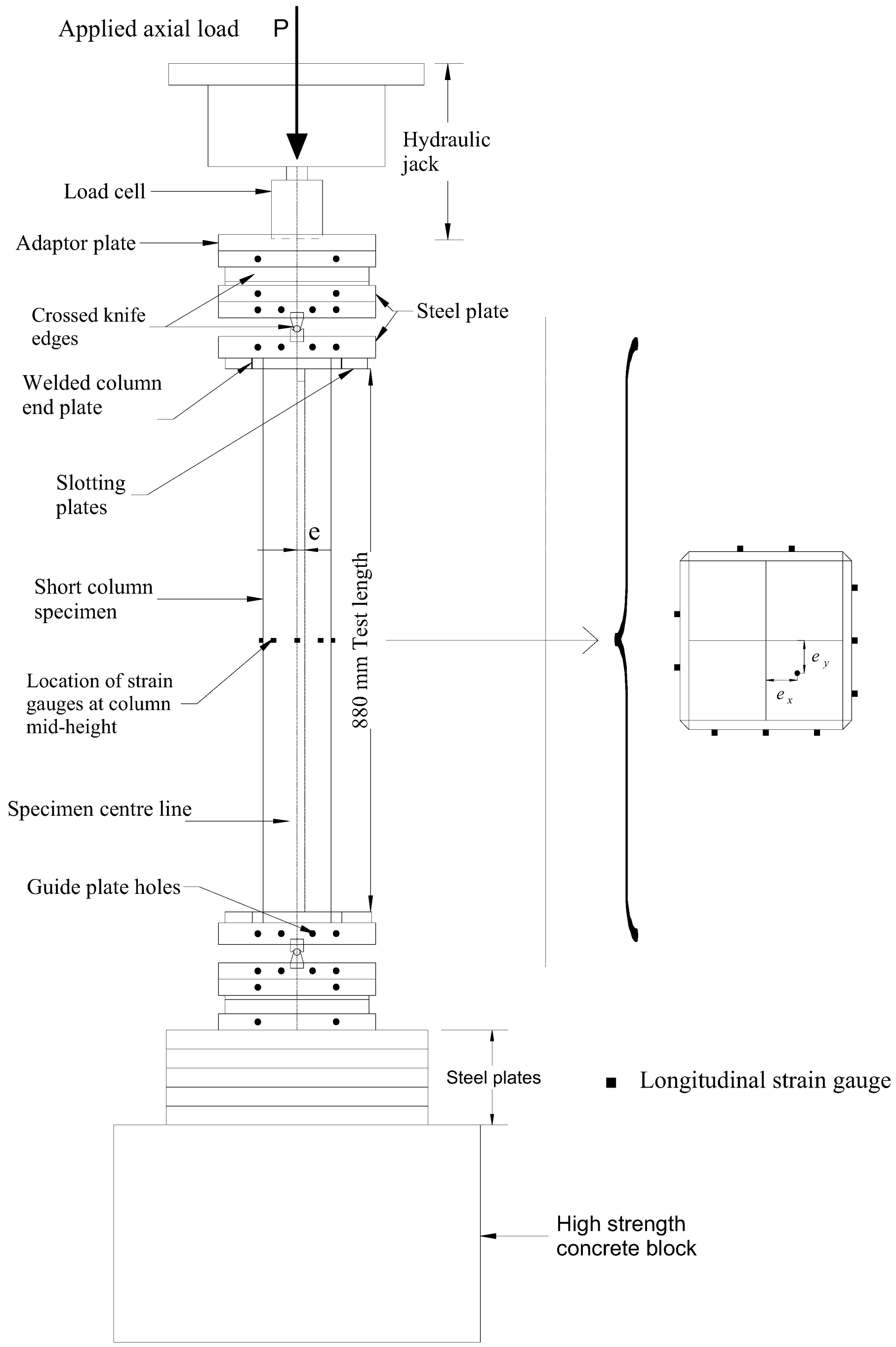

Figure 8. Loading Machine Set Up with Crossed Knife Edge Assemblies for Biaxial Bending

3.5 Short High Strength Steel Column Test Results

A total of eight short columns consisting of four hollow and four composite column specimens were tested under combined compression and biaxial bending. The short columns were tested under a uniform displacement rate. During the whole loading procedure, a computerised data recording system recorded a large amount of data including the shortening and strains at mid-height of the short columns plotted against the recorded load levels. The columns were tested until they reached their post-failure strength at increasing deformations.

3.5.1. Axial load-shortening relationship of short columns

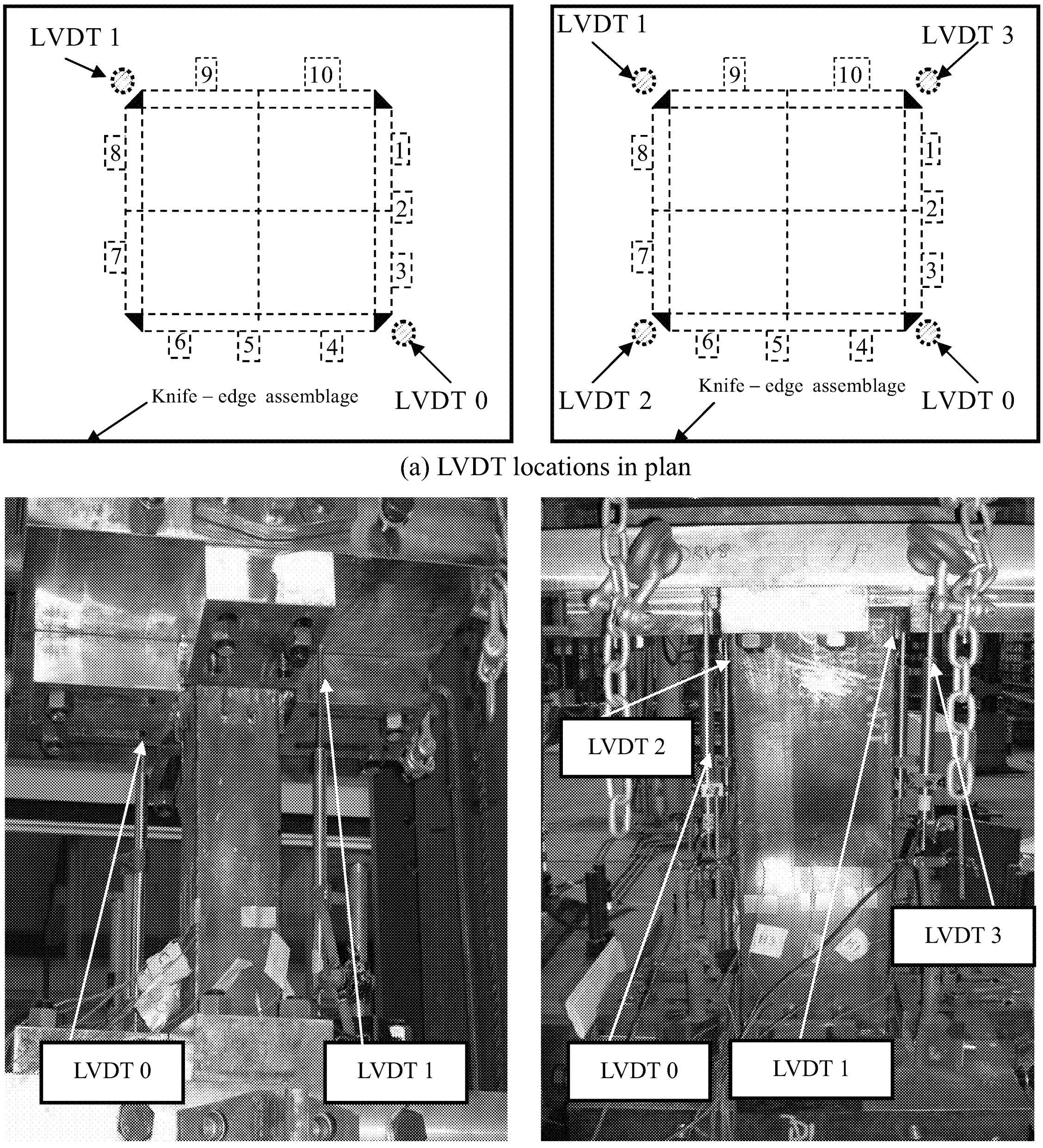

Figure 9. LVDT Locations for a Short Column

In this series of experiments, two LVDTs were used to measure the shortening of the columns tested. They were placed near to the corner of the column end plates, as shown in Figure 9. Figures 10 to 14 present the results of the axial load-shortening for all short columns tested. The axial load-shortening diagrams provide useful information in identifying the point of yielding, the ultimate load and the post-peak behaviour of the columns. They also show the effect of the shortening of each individual column as the load and eccentricity increases. Thus whilst some results show shortening measured as a negative deformation, some of the results are positive, which illustrate some extension, which is as a result of tension in the column caused by bending deformations. Nevertheless, the curves in Figures 10 to 14 give a general perspective of the failure of the columns. They exhibit the ductile nature of either the hollow or composite sections. As illustrated in the figures, the axial load-shortening relationships are seen to be fairly linear prior to reaching the peak load. Additionally, it was observed during the experiments that concrete-filled column specimens displayed a superior performance to hollow sections in terms of sectional strength and initial stiffness. The ultimate load capacity of the composite sections was greatly increased due to the delayed local buckling of the steel casing in the compression flange.

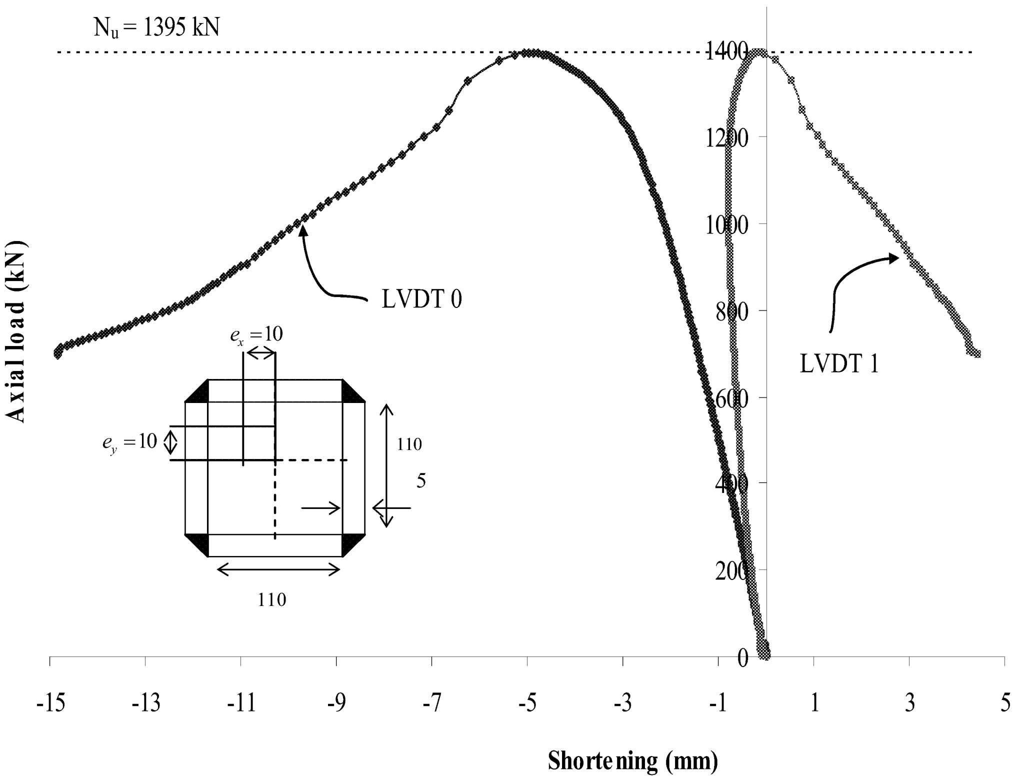

Figure 10. Axial Load-shortening of SH – H 110

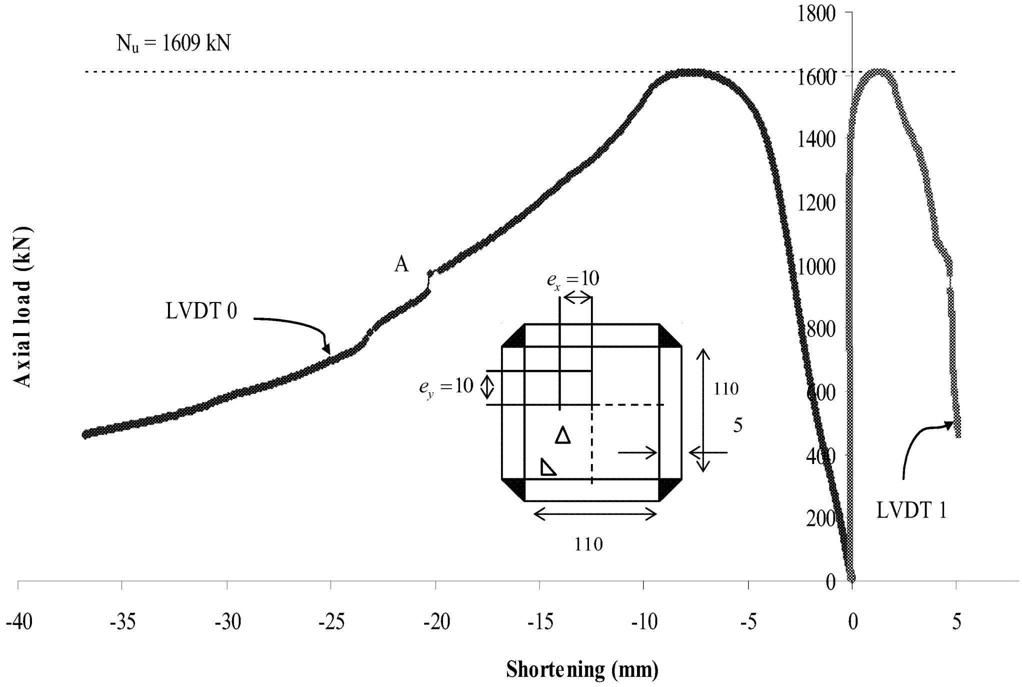

Figure 11. Axial Load-shortening of SH – C 110

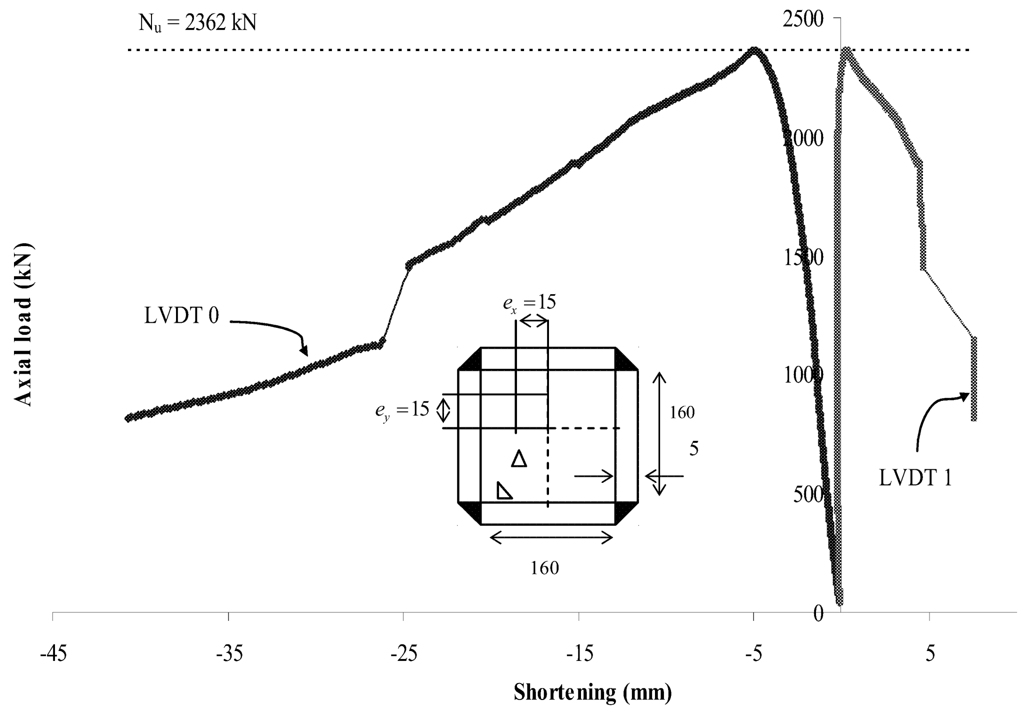

Figure 12. Axial Load-shortening of SH – C 160

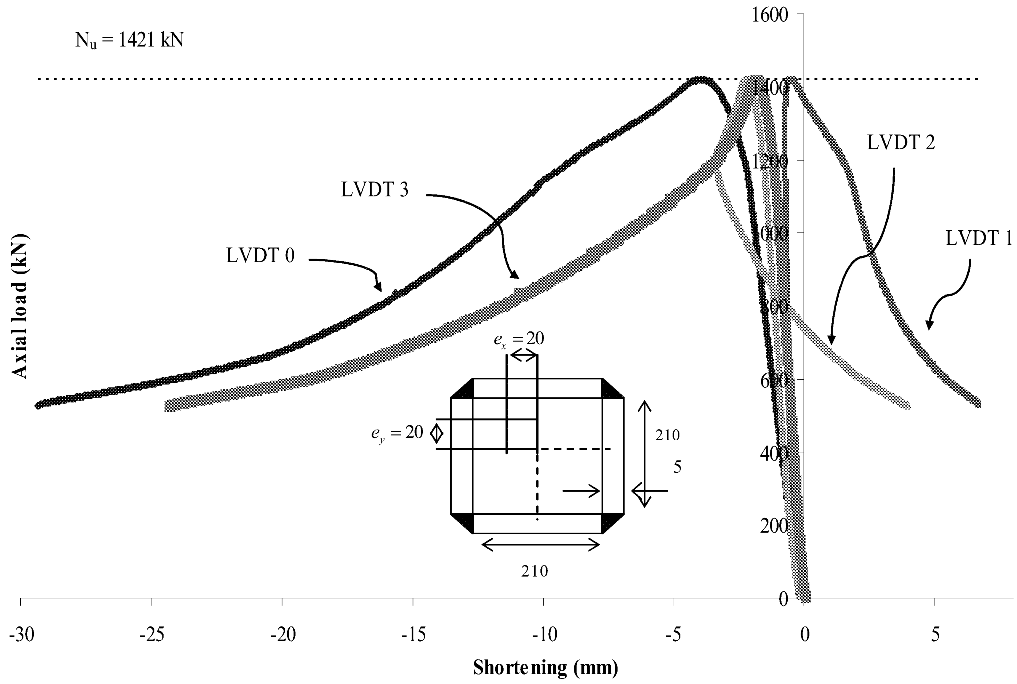

Figure 13. Axial Load-shortening of SH – H 210

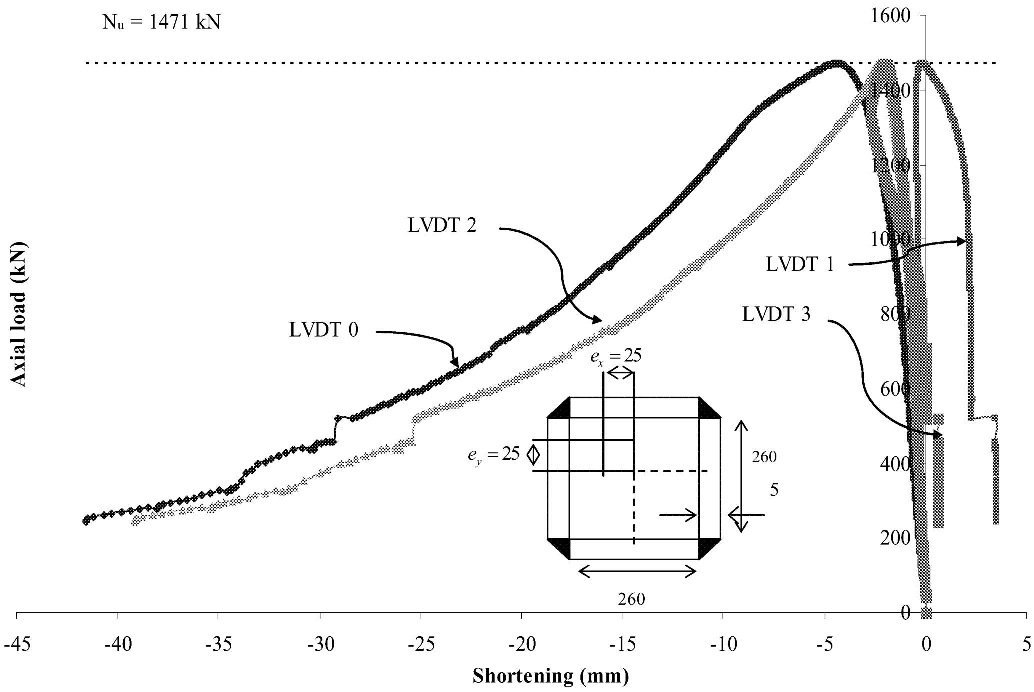

Figure 14. Axial Load-shortening of SH – H 260

3.5.2. *Failure modes*

The failure modes of the columns have also been noted. In particular the difference between the hollow sections and concrete filled steel sections is well worth highlighting. Figure 15 and Figure 16 illustrates the failure modes for the same size cross-sections which were both hollow and concrete filled. Quite evidently the major difference is that the local buckling shape is restrained by the concrete and this has a marked effect on the post-local buckling reserve of strength of the steel.

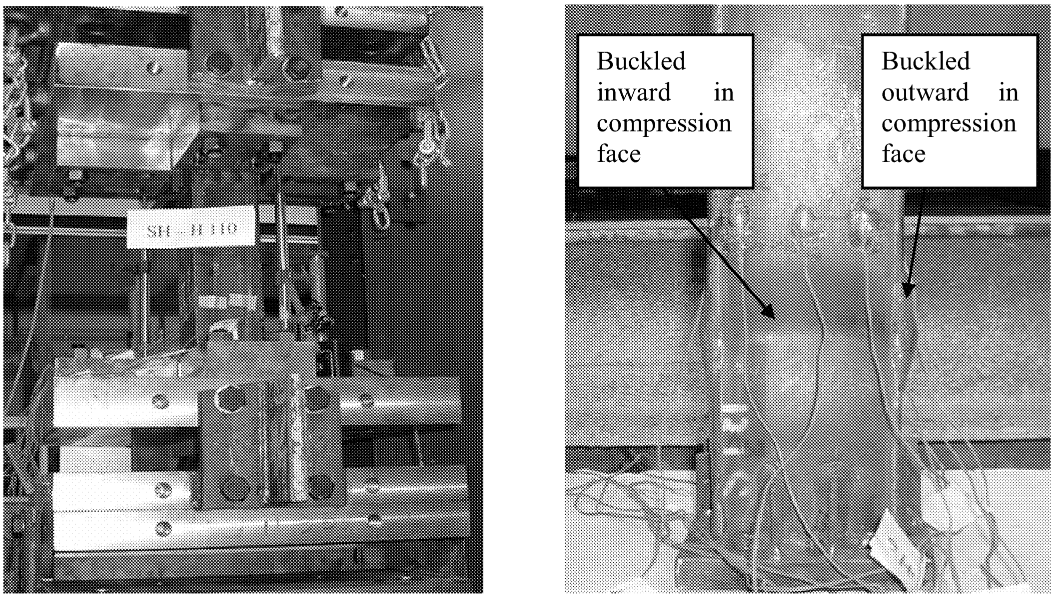

Figure 15. Failure Mode of Column SH – H 110

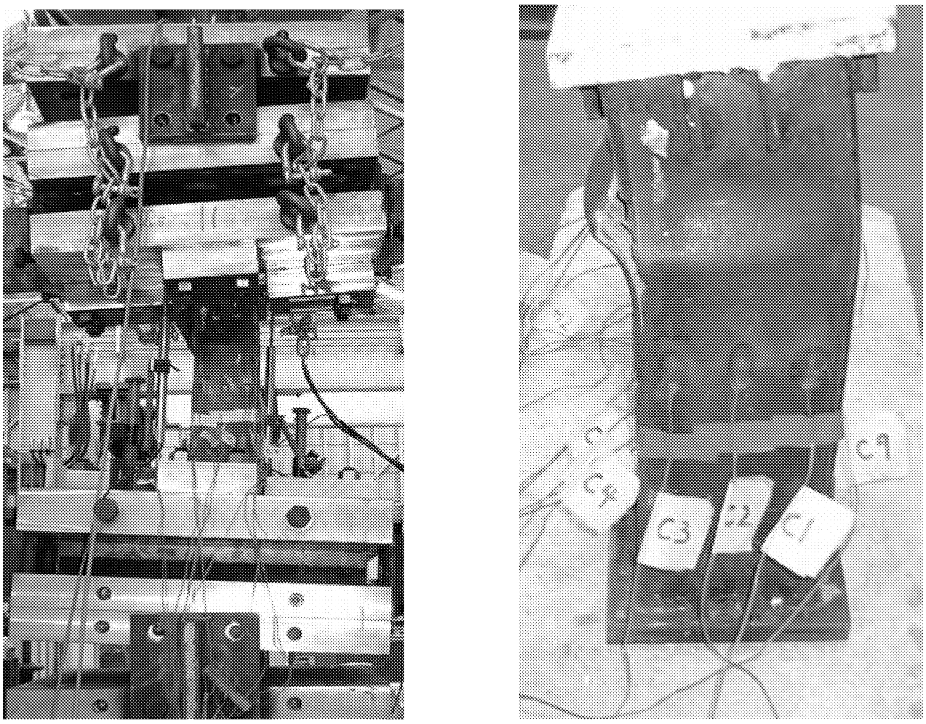

Figure 16. Failure Mode of Column SH – C 110

3.5.3 Maximum loads

Table 5. Details of Short Columns with Ultimate Test Loads

Specimen	N_c (kN)	N_s (kN)	N_u (kN)	N_{test} (kN)	N_{test}/N_u
SH – H110	0	1540	1540	1395	0.91
SH – H160	0	2240	2240	-	-
SH – H210	0	2940	2940	1421	0.48
SH – H260	0	3640	3640	1471	0.40
SH – C110	278	1540	1818	1609	0.89
SH – C160	588	2240	2828	2362	0.84
SH – C210	1014	2940	3954	-	-
SH – C260	1555	3640	5195	-	-

Note: - = Indicates that no data is registered due to testing error

Table 5 provides a summary of the maximum loads achieved by each of the columns in the test series. Furthermore, the table also provides a determination of the ultimate capacity of the cross-sections, assuming that the columns are compact and loaded in pure compression. The ultimate capacity of a composite column, using the principle of superposition was calculated as $N_u=N_c+N_s$, where N_c is the capacity of the concrete section assuming the mean cylinder strength can be used and N_s is the capacity of the steel section assuming full yield is achieved. Table 5 thus highlights quite clearly how load eccentricity and plate slenderness of the steel section can have a marked effect on the ultimate capacity of the section.

3.6 Slender High Strength Steel Column Test Results

An experimental programme was conducted on columns of a slender nature fabricated with high strength steel plate, formed into boxes and filled with concrete. All columns were constructed with four equal width plates and joined at the vertices by means of a fillet weld as illustrated in Figure 17. The dimensions and other salient features of the specimens from the testing are summarised for the experiments in Table 6. In particular, the nominal width to thickness ratio of 25, 35, 45 and 55 was used in the program. The columns were tested in a vertical self-straining compression-testing machine. The slender columns were tested with a small eccentricity, *e* about each axis equivalent to 10% of the cross-sectional width of the specimen. An additional specimen was conducted for the hollow and concrete filled sections which had an eccentricity *e* about each axis equivalent to 20% of the cross-sectional width of the specimen, namely SL40-H210 and SL40-C210. The test set-up for the columns is illustrated in Figure 17, these highlight the specimen, loading plates for applying the load about two axes, the jack and the location of strain gauges and LVDTs to measure respectively the strain and the lateral deflection of the column in each plane of bending.

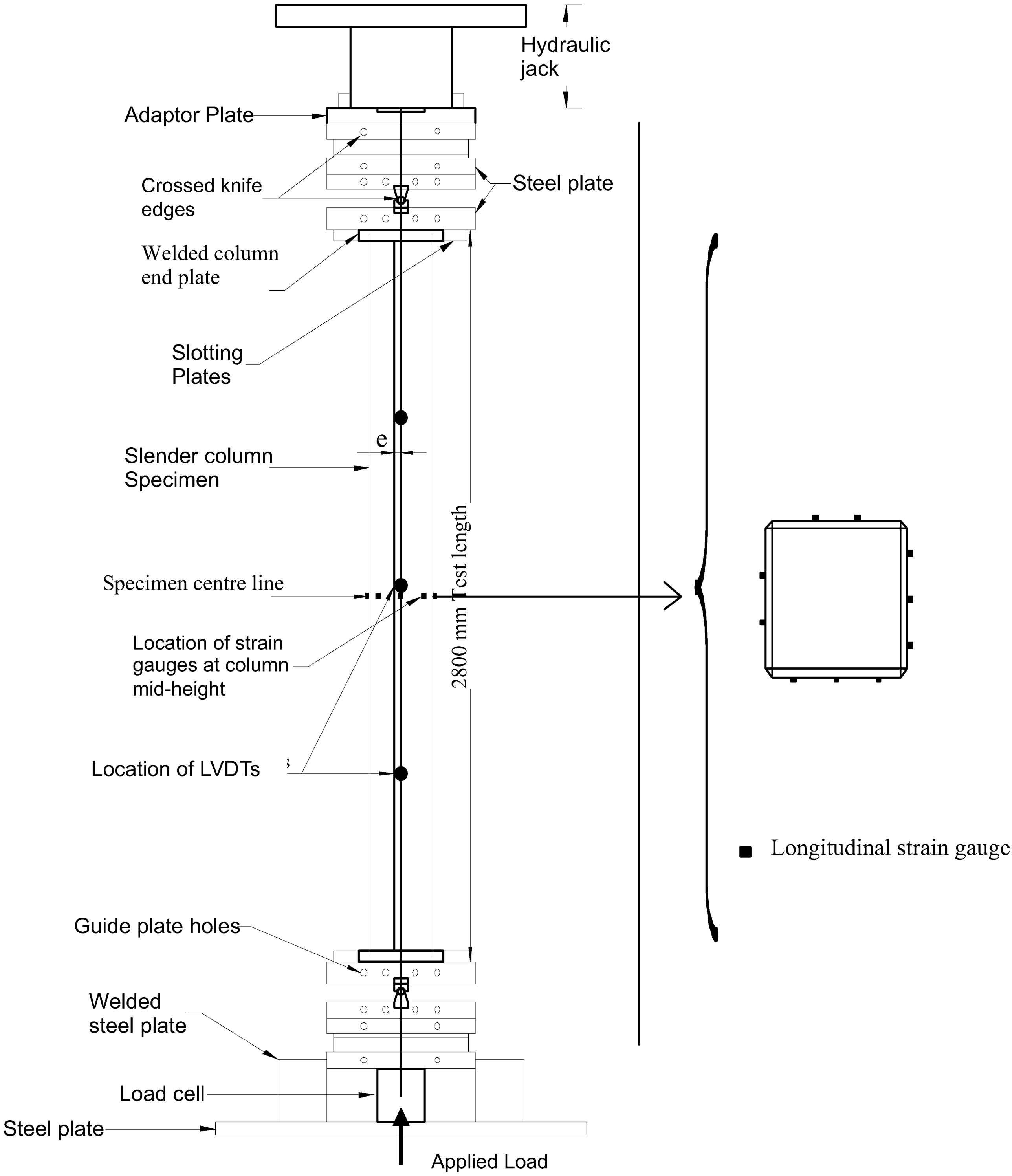

Figure 17. Loading Machine Set Up with Crossed Knife Edge Assemblies for Slender Column Under Biaxial Bending

Table 6. Details for the Testing of Slender Columns

Specimen	b (mm)	d (mm)	L (mm)	t (mm)	e (mm)	$\frac{b}{t}$	$\frac{b}{t}\sqrt{\frac{f_y}{250}}$	$\frac{e}{d}$
SL – H110	120.0	120.0	3430	5	10	24.0	39.9	0.084
SL – H160	170.0	170.0	3430	5	15	34.0	56.5	0.088
SL – H210	220.0	220.0	3430	5	20	44.0	72.9	0.090
SL40 – H210	220.0	220.0	3430	5	40	44.0	72.9	0.182
SL – H260	270.0	270.0	3430	5	25	54.0	89.6	0.093
SL – C110	120.0	120.0	3430	5	10	24.0	39.9	0.083
SL – C160	170.0	170.0	3430	5	15	34.0	56.5	0.088
SL – C210	212.0	220.0	3430	5	20	44.0	73.1	0.091
SL40 – C210	220.0	220.0	3430	5	40	44.0	73.1	0.182
SL – C260	270.0	270.0	3430	5	25	54.0	89.5	0.093

3.6.1. Load-deflection relationship of slender columns

The lateral deflection of the column from the original state was registered by using LVDTs. These LVDTs were located perpendicularly to the two adjacent sides of the column and three of them were positioned vertically in each plane of bending. The planes of bending were defined by the two major axes of the section along the length of the column perpendicular to its sides.

The position of the LVDTs located at the top quarter of the column, midheight and at the bottom quarter of the column. The horizontal deflections were registered by means of a computer system, which records the data instantly to an input file. Figures 18 to 27 illustrate the lateral deflections of the column at the position of the LVDT in accordance with the biaxial bending load capacity.

3.6.2. Failure modes

Local buckling takes place right at the peak followed by overall buckling for the slender composite column SL-C260, which has a slender section. Whilst local buckling occurred after the peak load in the two columns SL-C210, SL40-C210 which have slender sections. This gives a reason for further discussion of the slenderness limits provided by the design codes. Failure modes for the composite column SL-C160 which has a compact section demonstrated overall buckling followed by local buckling in the plastic range or following the peak. The slender composite column SL-C110 which had a compact section did not reveal any local buckling failure and it exhibited a ductile overall buckling failure when compared to the slender hollow column SL-H110.

For the slender hollow columns SL-H260, SL-H210, SL40-H210, all of which had slender sections, local buckling took place before the peak followed by overall buckling. Whilst local buckling coincides with the peak load for the slender hollow column SL-H160 which had a slender section according to the design codes. Local buckling occurred after the peak for the hollow column SL-H110 which also had a slender section. Figures 18 - 27 illustrated the load deflection curves for the same size cross-sections of columns which were both hollow and concrete filled. Table 7 provides a summary of the maximum loads achieved by each of the columns in the test series.

Table 7. Experimental Load for Slender Hollow and Composite Columns

Specimen	$N_{biaxial}$ (kN)
SL – H 110	642
SL – H 160	1230
SL – H 210	1300
SL40 – H 210	1077
SL – H 260	1342
SL – C 110	765
SL – C 160	1858
SL – C 210	2784
SL40 – C 210	2188
SL – C260	3634

4. CONCLUSION

This paper has presented the background behind the use of high strength steel columns in landmark tall buildings. The paper has also highlighted the need to investigate the behaviour of these columns under generalized loading and boundary conditions.

A novel experimental program involving the testing of eighteen high strength steel columns has been carried out. The experiments have highlighted the benefits of the use of very low concrete strength in improving the overall performance of the steel section. This is as a result of the restraint offered by the concrete for the local and post-local buckling of the steel casing. These experiments will give designers improved confidence as it will provide evidence of a very general nature as to the suitability of existing design procedures which are considered in a companion paper.

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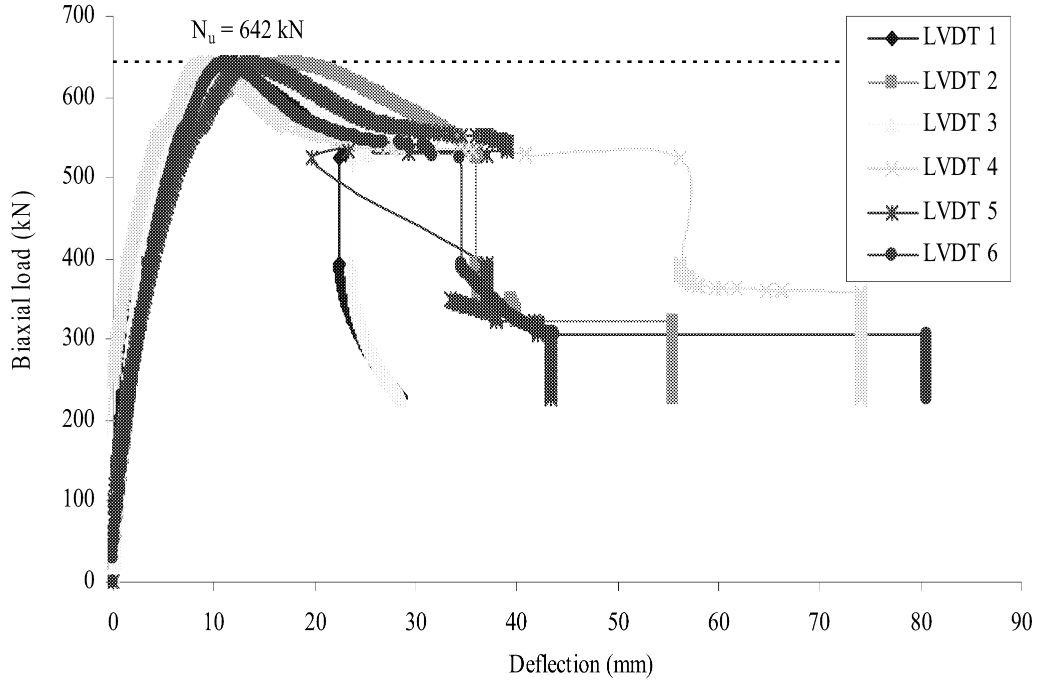

Figure 18. Load Deflection for Column SL – H110

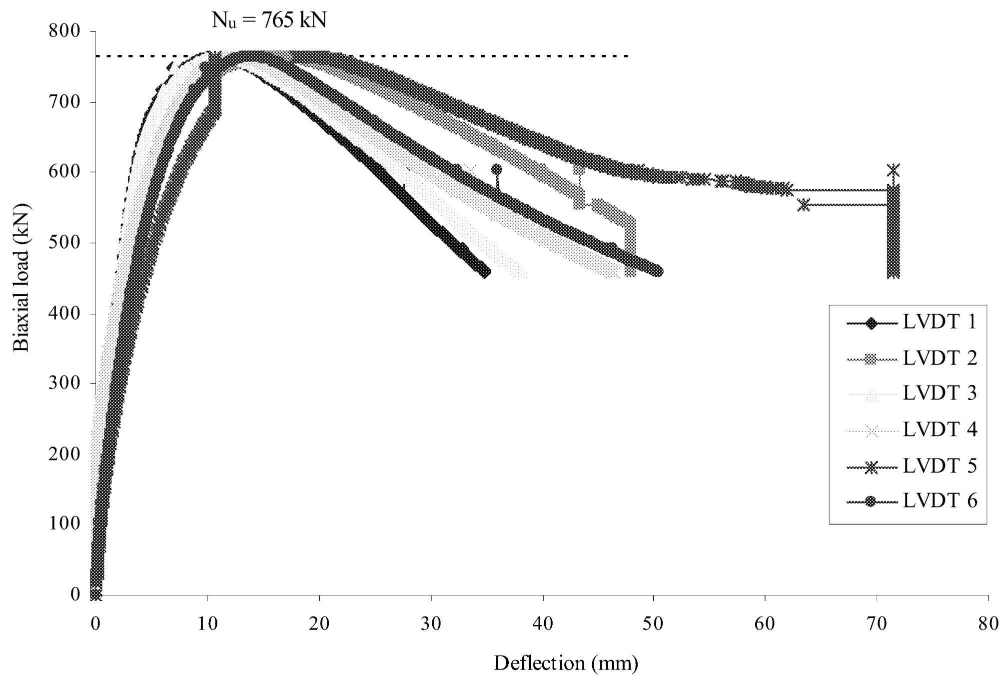

Figure 19. Load Deflection for Column SL – C110

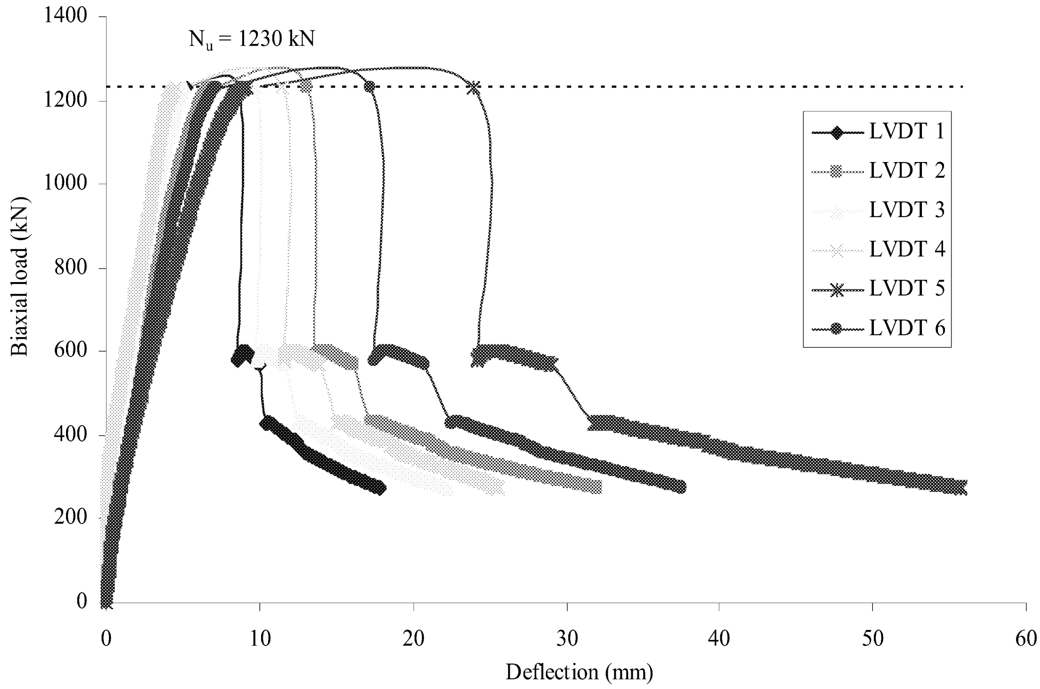

Figure 20. Load Deflection for Column SL – H160

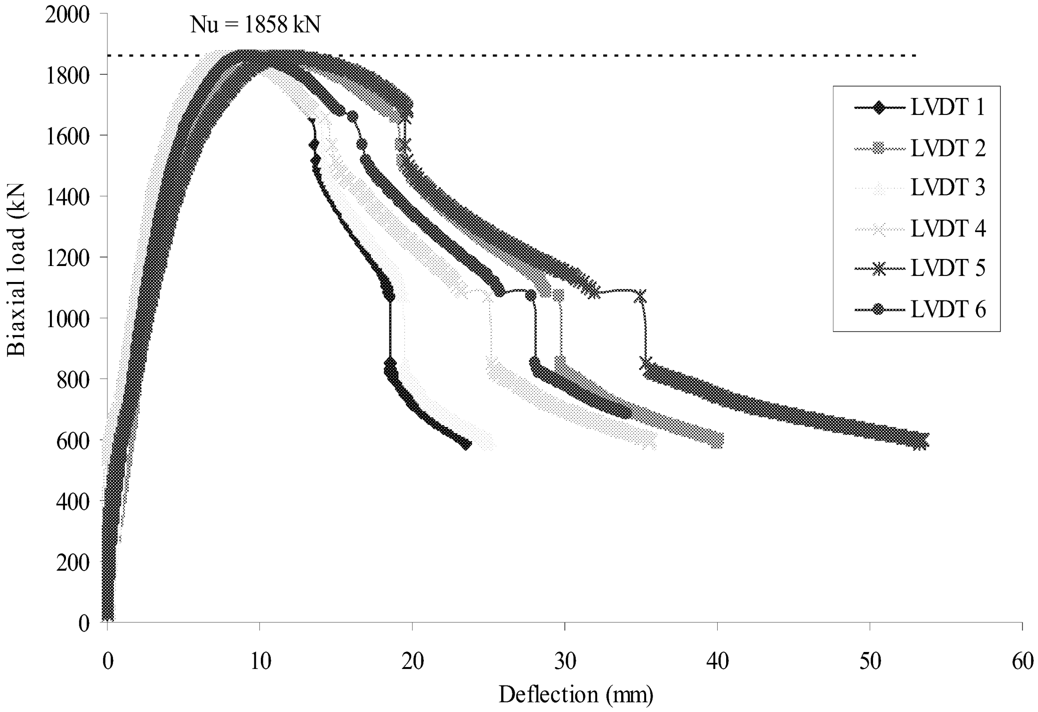

Figure 21. Load Deflection for Column SL – C160

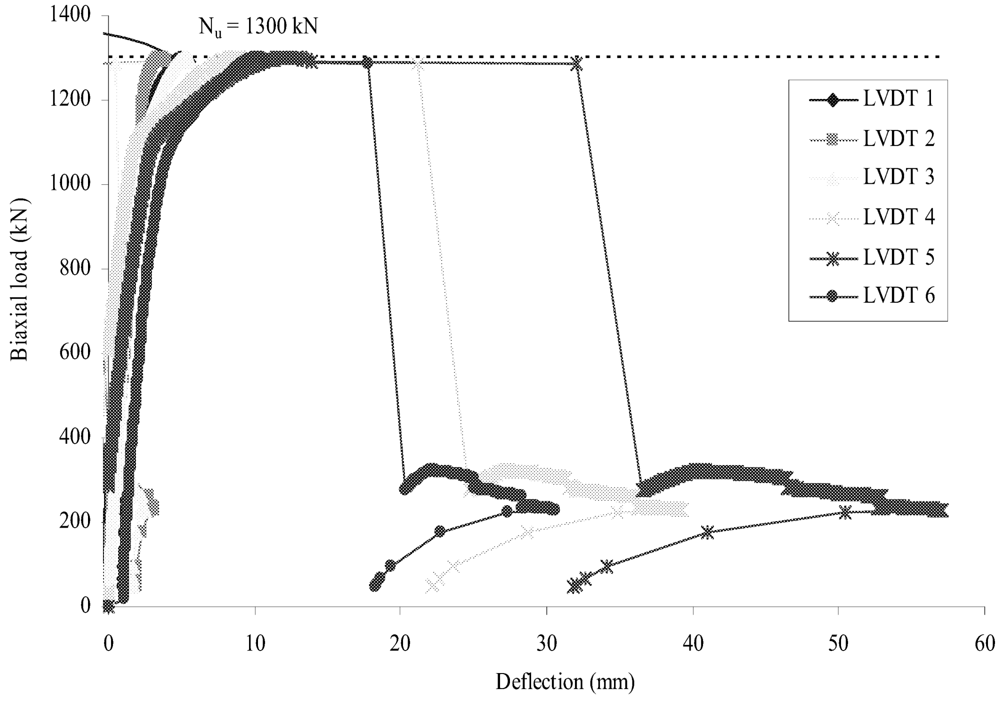

Figure 22. Load deflection for column SL – H210

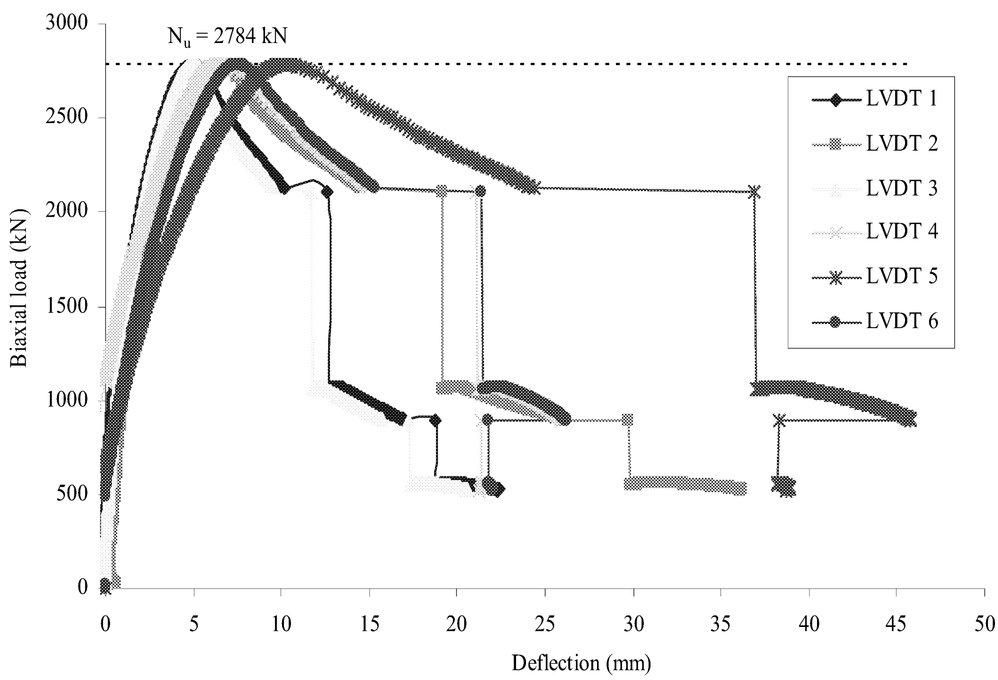

Figure 23. Load deflection for column SL – C210

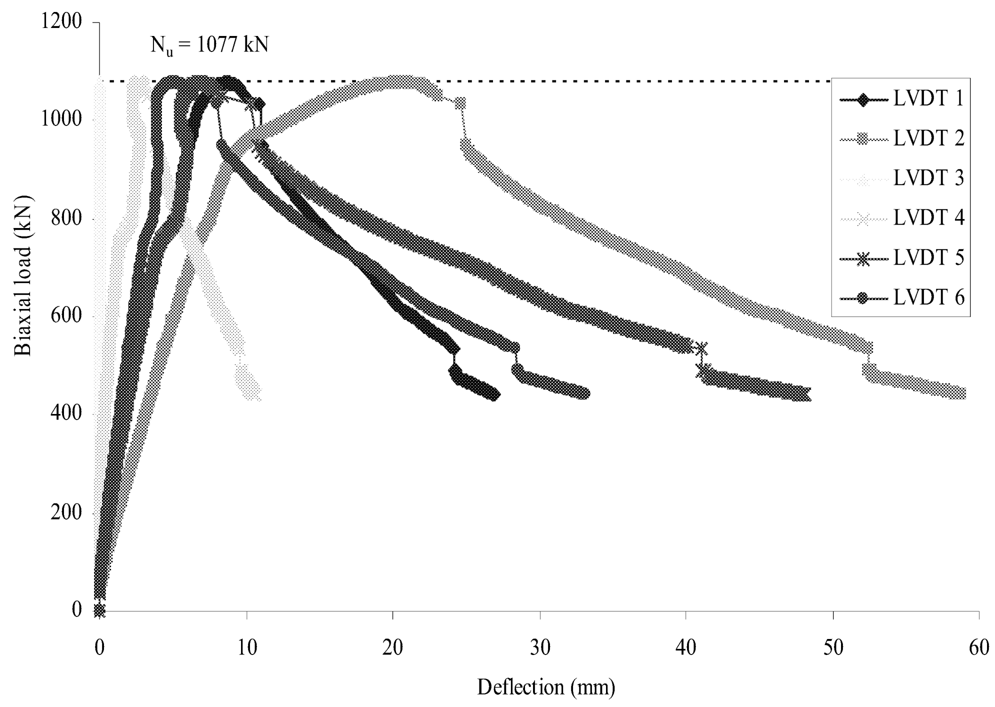

Figure 24. Load Deflection for Column SL40 – H210

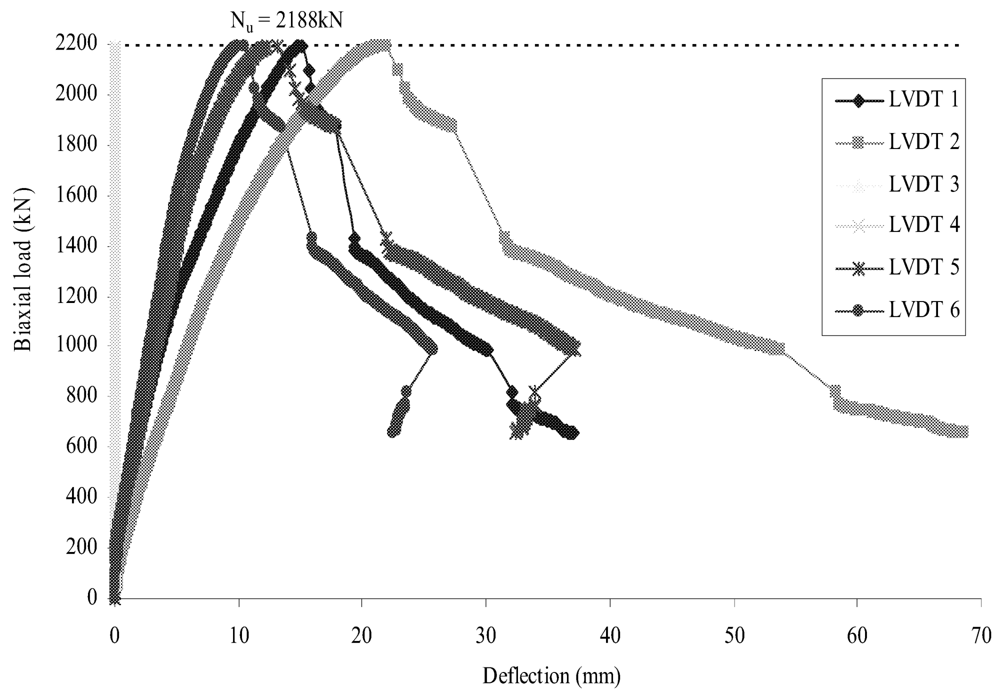

Figure 25. Load Deflection for Column SL40 – C210

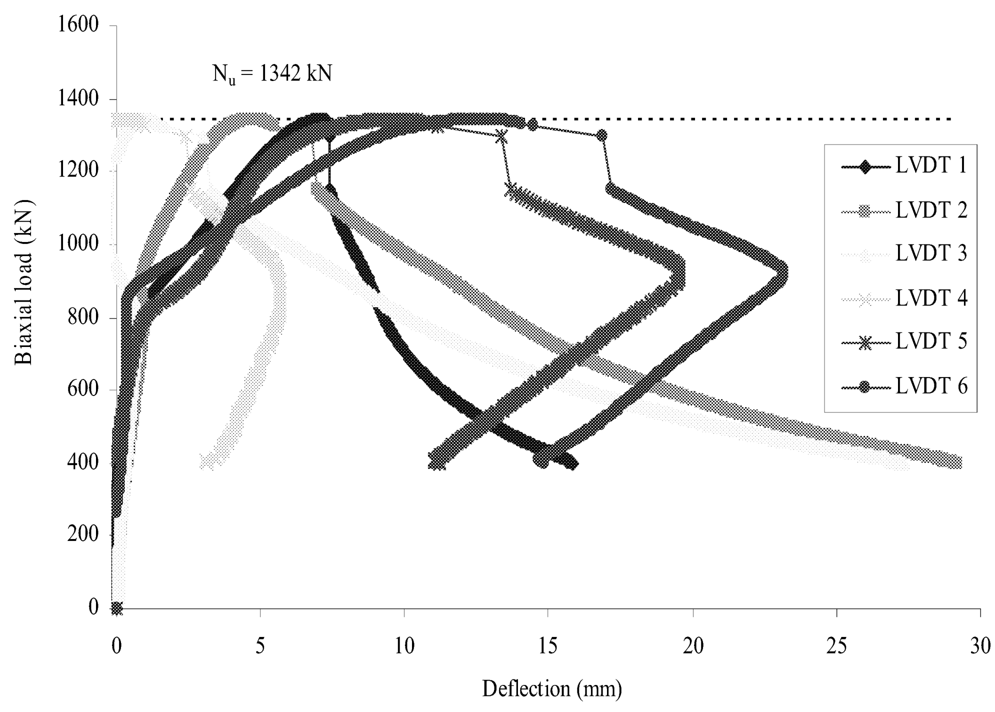

Figure 26. Load Deflection for Column SL – H260

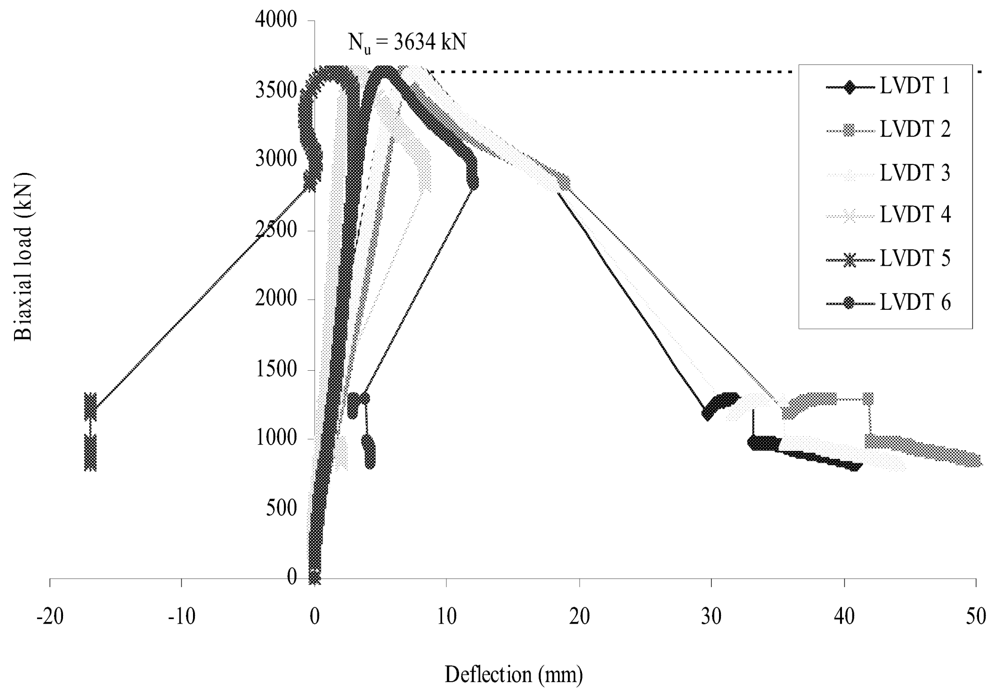

Figure 27. Load Deflection for Column SL – C260

Figure 28. Failure Mode of Hollow Slender Column SL-H260

Figure 29. Failure Mode of Composite Slender Column SL-C260

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NOTATION

A = area, mm^2

b = width of column section, mm

b/t = plate slenderness

d = depth of column section, mm

e = eccentricity of applied load, mm

e/d = eccentricity to depth ratio

E_s = elastic modulus of steel

f_c = compressive cylinder strength of concrete

F_{yc} = compressive yield stress of steel

L = length of column, mm

L/d = length to depth ratio

N_c = axial capacity of the concrete component of the section

N_s = axial capacity of the steel component of the section

N_u = ultimate section axial capacity of composite column

N_{test} = short column testing capacity under biaxial load

$N_{biaxial}$ = slender column testing capacity under biaxial load

t = thickness of steel plate, mm

σ_y = tensile yield stress of steel, N/mm^2

σ_u = ultimate tensile stress of steel, N/mm^2