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ABSTRACT

This paper describes a series of compression tests performed on long columns fabricated from cold-formed high strength steel plates with nominal yield stress of 550 MPa (80 ksi). The steel is classified as G550 to Australia Standard AS1397. The test results presented in this paper are the second stage of an Australian Research Council research project entitled "Compression Stability of High Strength Steel Sections with Low Strain-Hardening". A total of 28 long columns, which were made from two thicknesses of sheet steel (0.42 mm and 0.6 mm) (0.017 in. and 0.024 in.), were tested. A box shaped section was tested between pinned ends over a range of lengths.

This paper shows the comparison of loads obtained experimentally with those predicted on the basis of AS/NZS4600 and the AISI specification including Supplement No.1, 1999. The finite element program, ABAQUS, was also used to simulate the column behaviour. For sections which undergo local instability at loads significantly less than the ultimate loads, the column design rules in AS/NZS 4600 and the AISI Specification are unconservative if used in their current form for G550 steel. Proposals for improved column design of high strength slender sections are proposed in this paper.

INTRODUCTION

Cold-formed sections used in steel framed houses as roof and wall structural members are normally made from coil by means of roll-forming. The use of high strength cold-reduced steels with yield stress values up to 550 MPa (80 ksi) is increasing rapidly, particularly for steel framed houses with sections as thin as 0.42 mm (0.017 in.). Steels with high yield stress usually have

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little or no strain hardening in the stress-strain curve, and low ductility unlike conventional 250 MPa and 350 MPa (36 ksi and 50 ksi) structural steels that are highly ductile and strain harden. Strain hardening is may be important in the stability of thin-walled sections and so the high strength steels may have their stability affected by the lack of strain hardening. In the design of thin-walled box columns, an optimum choice of section must be made to ensure that the maximum strength is obtained for a given weight of material.

It is customary to consider that a column may buckle in either one of two ways: (a) by plate buckling of its component webs and flanges in shorter half-waves (local or plate buckling) or (b) by deflection of the entire column in a half-wave of length equal to the effective column length (overall buckling). For a given column, buckling is supposed to occur at the lower of the two critical stresses, local or overall. In reality, however, there is an interaction between these two modes of buckling, so that the actual buckling stress will be smaller than either of the buckling stresses. Imperfections play a significant role in interaction buckling. Column strength is characterized by the maximum axial force that can be supported without excessive lateral deformations. In view of the fact that post-buckling strength of a flat plate is available for structural members to carry additional load, cold-formed steel sections are normally designed on the basis of the post-buckling strength of the plate elements rather than based on the local buckling stress.

Columns and their strength and behaviour constitute a subject area that has received much study and discussion over the years. Experimental and theoretical investigations have been performed to study the interaction of local and overall buckling. Bijlard and Fisher (1952) first studied the column strength of square tubes in the post-buckling range of component plates. Graves-Smith (1967) presented a theory for predicting the ultimate strength in compression of thin-walled box columns which buckle locally. Van der Neut (1969) used a mathematical model to study the interaction of local buckling and overall buckling of thin walled compression members. Hancock (1981) used the finite strip method to analyze the nonlinear response of box and I-sections under axial compression and obtained the interaction buckling loads for them by a overall bifurcation analysis of a locally buckled section.

For high strength cold-reduced steel sections made from thin galvanized cold-reduced steel to Australian Standard AS 1397-1993 and similar ASTM Standards such as A792, no specific investigation has been performed. Mainly due to lack of knowledge on their structural behavior, the 1996 Australian/New Zealand Standard AS/NZS 4600 for Cold-Formed Steel Structures and the 1996 American Iron and Steel Institute (AISI) Specification for Cold-Formed Structural Members have limited the design stress for high strength steels to 75 percent of their yield stress or tensile strength as applicable. The AISI Specification has recently been revised in Supplement (1999) to allow values higher than 75 percent for multiple web configurations, the value depending mainly on plate slenderness, but excludes its use for structural members such as columns.

A research project on these steels in tension, which was carried out by Rogers and Hancock (1996), has shown that they have substantially reduced ductility but this may not affect the net section strength of perforated sections. Steels of this type are similar to Structural Grade 80 steels in the USA according to the ASTM A653 (1997) Specification. A research project led by Professor W-W Yu at the University of Missouri-Rolla to investigate the strength of these ASTM steels when formed into decking sections and subject to bending has demonstrated that their local and post-local buckling capacities may be significantly influenced by the lack of strain hardening. In particular, the ultimate moments of panels with slender sections (b/t>100) were lower than the design moments calculated on the basis of a conventional effective section model. However no significant definitive testing has been performed for structural members composed of AS 1397 steel in compression. The AS 1397 steel may be zinc coated or aluminium-zinc coated. The steel studied in this paper is aluminium-zinc coated similar to ASTM A792 (1994).

Research on these steels as stub columns in compression, which was carried out by Yang and Hancock (2002) as the first stage of this research has shown that the greatest effect of the low strain hardening was for the stockier sections where strain hardening play an important role. For the more slender sections where elastic local buckling and post-local buckling are more important, the effect of low strain hardening does not appear to be as significant which is contrary to recent design proposals in the USA where it was believed that the more slender sections had been also influenced. The aim of this paper is to present the second stage test results for long columns. The tests were performed on lipped-box shaped specimens (LB-section) fabricated from G550 steel sheets to AS1397 with 0.42 mm and 0.60 mm (0.017 in. and 0.024 in.) thickness similar to those in Yang and Hancock (2002). In this paper, two series of long column tests (LB-section) with pin-ends are described. The purpose of the tests was to investigate whether high strength steel members with low strain hardening can be designed according to existing standards of AS4600 or the AISI Specification or whether these standards need to be modified for high strength steel.

TEST SPECIMENS

General

In the first stage report (Yang & Hancock, 2002), a series of stub column tests was presented on sections fabricated from G550 steel sheets in 0.42 mm and 0.60 mm (0.017 in. and 0.024 in.) thickness. For cold-formed steel sections, which generally have thin-walled plate elements, the stub-column test is aimed at determining the effect of local buckling as well as the effect of cold-forming, residual stress and yielding on the section capacity in compression.

In order to determine the capacity of long columns in compression, local and overall buckling and the interaction between them should be considered. For the high strength steel columns assembled from thin plates, the section configuration is an important factor affecting their behaviour. Thin plate elements will generally continue to carry load after local buckling into the post-buckling range so that local buckling does not mean failure of the whole column. However, a singly-symmetric section may have a neutral axis shift after local buckling occurs resulting in an additional moment. To eliminate this problem, in this test program, doubly-symmetric sections were chosen. These sections are the LB-sections previously described by Yang & Hancock (2002) for stub columns.

Table 1. Measured Dimensions of Long Columns (0.60 mm) & Test Loads

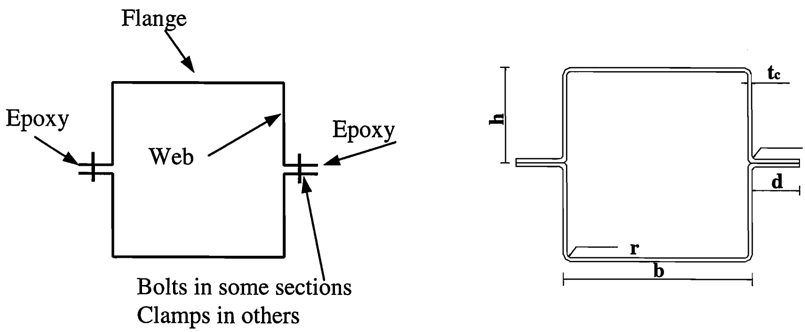
Specimen	Flange	Web	Thickness		Radius		Lip	Thickness of lip	Length	Ultimate test load	Elastic buckling test load
	h (mm)	b (mm)	t_b (mm)	t_c (mm)	r (mm)	R (mm)	d (mm)	hl (mm)	L (mm)	P_t (kN)	P_{cr} (kN)
060LB20P450a	10.5	22.3	0.60	0.65	0.60	0.60	6.0	1.5	448.5	16.4	N/A
060LB20P450b	10.3	20.6	0.60	0.65	0.60	0.60	5.8	1.6	449.0	21.3	N/A
060LB20P900a	10.7	21.4	0.60	0.65	0.60	0.60	5.2	1.9	896.5	8.1	N/A
060LB20P900b	11.5	20.5	0.60	0.65	0.60	0.60	5.9	1.6	896.0	N/A	N/A
060LB30P450a	15.5	32.0	0.60	0.65	0.60	0.60	8.2	1.7	448.8	N/A	N/A
060LB30P450b	15.1	31.4	0.60	0.65	0.60	0.60	8.1	1.4	449.0	35.5	N/A
060LB30P900a	15.0	30.8	0.60	0.65	0.60	0.60	8.7	1.5	898.3	21.9	N/A
060LB30P900b	15.2	30.1	0.60	0.65	0.60	0.60	8.5	1.6	897.5	20.3	N/A
060LB40P450a	19.8	41.9	0.60	0.65	0.60	0.60	10.8	1.6	448.8	47.6	31.6
060LB40P450b	20.0	41.9	0.60	0.65	0.60	0.60	10.7	1.5	448.6	45.5	32.6
060LB40P900a	19.8	41.5	0.60	0.65	0.60	0.60	11.4	1.7	897.5	28.7	24.4
060LB40P900b	19.8	41.2	0.60	0.65	0.60	0.60	11.1	1.6	898.5	30.1	27.3

Note: 1in.=25.4 mm; 1kip=4.448 kN

Table 2. Measured Dimensions of Long Columns (0.42 mm) & Test Loads

Specimen	Flange	Web	Thickness		Radius		Lip	Thickness of lip	Length	Ultimate test load	Elastic buckling test load
	h (mm)	b (mm)	t_b (mm)	t_c (mm)	r (mm)	R (mm)	d (mm)	hl (mm)	L (mm)	P_t (kN)	P_{cr} (kN)
042LB30P550a	15.4	32.5	0.41	0.45	0.60	0.60	5.8	1.1	550.1	17.50	10.10
042LB30P550b	15.9	32.4	0.41	0.45	0.60	0.60	5.8	1.2	550.0	17.38	9.70
042LB30P1100a	15.7	31.7	0.41	0.45	0.60	0.60	6.1	1.1	1099.8	10.78	9.90
042LB30P1100b	15.8	32.6	0.41	0.45	0.60	0.60	5.9	1.2	1100.1	11.70	10.40
042LB40P550a	21.2	41.5	0.41	0.45	0.60	0.60	5.9	1.0	550.0	20.90	9.20
042LB40P550b	21.3	41.2	0.41	0.45	0.60	0.60	5.9	1.1	549.5	23.00	9.60
042LB40P1100a	21.3	41.4	0.41	0.45	0.60	0.60	6.1	1.4	1099.8	15.00	8.80
042LB40P1100b	21.2	41.6	0.41	0.45	0.60	0.60	6.0	1.2	1099.5	14.20	8.50
042LB40P1700a	20.4	41.2	0.41	0.45	0.60	0.60	6.0	1.3	1698.0	8.40	6.20
042LB40P1700b	20.5	41.4	0.41	0.45	0.60	0.60	5.9	1.1	1697.0	8.79	7.60
042LB50P550a	26.4	51.0	0.41	0.45	0.60	0.60	5.9	1.2	550.0	23.40	6.60
042LB50P550b	26.5	50.8	0.41	0.45	0.60	0.60	6.2	1.2	550.1	23.60	6.80
042LB50P1100a	26.5	50.3	0.41	0.45	0.60	0.60	6.6	1.5	1099.8	19.70	7.20
042LB50P1100b	26.5	50.7	0.41	0.45	0.60	0.60	6.2	1.3	1100.1	17.50	6.50
042LB50P1700a	25.3	52.0	0.41	0.45	0.60	0.60	6.2	1.2	1697.0	11.20	6.50
042LB50P1700b	25.3	51.7	0.41	0.45	0.60	0.60	6.0	1.2	1698.0	10.70	6.20

Note: 1in.=25.4 mm; 1kip=4.448 kN



(a) Test Section (LB)

(b) Dimension Nomenclature

Fig. 1 Test Section (LB-Section)

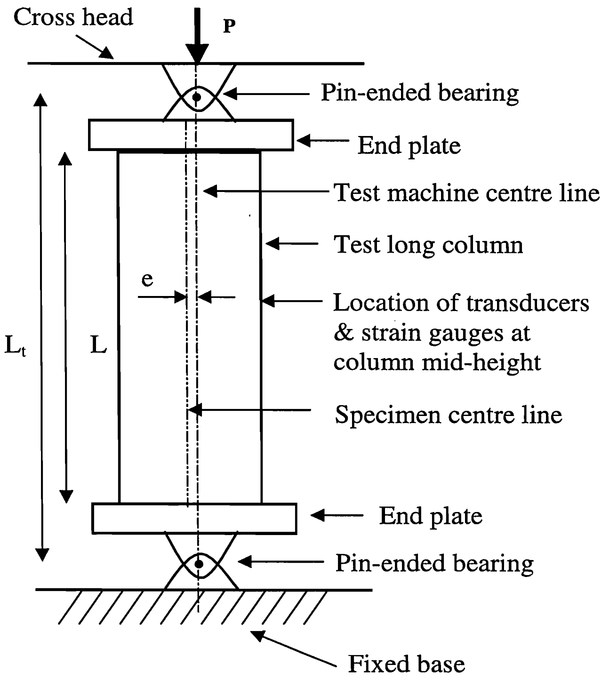


Fig. 2 Test Arrangement

In this paper, a long column is defined as a member whose length is considerably larger than any of its cross-sectional dimensions and which is subjected to compression in the longitudinal direction. A centrally loaded column means that the resultant compressive force is approximately coincident with its longitudinal centroidal axis (Galambos, 1988).

The tests were performed on closed sections brake pressed from aluminium/zinc-coated Grade G550 structural steel sheet to AS1397 in 0.42 mm and 0.6 mm (0.017 in. and 0.024 in.) thickness. The sections tested are shown in Fig. 1. Epoxy was used to close the LB-sections. The lengths of the LB sections ranged from 450 mm to 1100 mm (17.7 in. to 43.3 in.) for the 0.60 mm (0.024 in.) sheet steel and 550 mm to 1700 mm (21.7 in. and 66.9 in.) for the 0.42 mm (0.017 in.) sheet steel. The cross-section dimensions of the specimens are shown in Tables 1 & 2, for the nomenclature shown in Fig. 1. The cross-section dimensions are the average of the measured values. The base metal thickness (t_b) was measured by removing the zinc coating by acid-etching. The ends of each specimen were milled flat and parallel to ensure full contact between specimens and end bearing. Bolts & clamps were also used on the LB-sections as shown in Fig. 1. Further details are given in Yang, Hancock and Rasmussen (2002).

Labelling

The LB-sections were divided into two different series: one for the 0.60 mm (0.024 in) sheet steel and one for the 0.42 mm (0.017 in) sheet steel. The test specimens were labelled such that the thickness of steel sheet, type of section, nominal width of specimen and specimen number were expressed by the label.

For example, the label "060LB20P450a" defines the following specimen:

- The first three numbers indicate that the specimen is fabricated from 0.60 mm (0.024 in) steel sheet.
- Th fourth and fifth letters (LB) indicate that the specimen is a lipped box
- The "20" indicates that the nominal width of specimen is 20 mm (0.79 in)
- The "P" indicates that the specimen is pin-ended
- The "450" indicates that the nominal length of specimen is 450 mm (17.7 in)
- The last letter "a" indicates that the specimen was the first tested (alternative b)

Geometric Imperfection Measurements and Material Properties

Geometric imperfections were measured for all of the specimens. The full set of measurements and the maximum overall imperfections are given in Yang, Hancock and Rasmussen (2002). The values of the measured imperfections vary from 0.2 mm to 1.5 mm (0.008 in. to 0.06 in.). The material and its properties are fully described in Yang & Hancock (2002) and Yang, Hancock and Rasmussen (2002). The measured 0.2% proof stresses of the steel were 690 MPa and 711

MPa (100 ksi and 103 ksi) for 0.42 mm and 0.60 mm (0.017 in. and 0.024 in.) thickness respectively.

LONG COLUMN TESTS

Testing

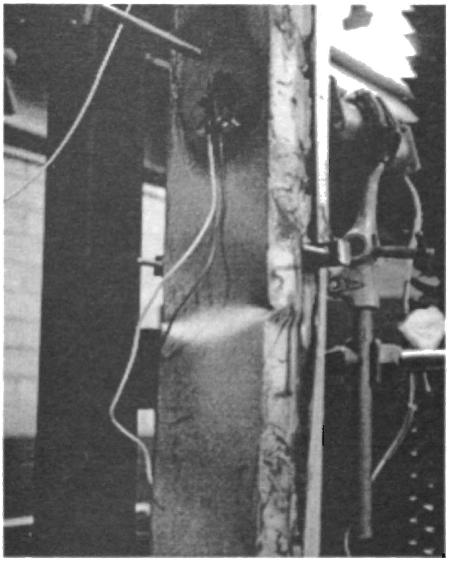
The arrangement is shown in Fig. 2. The rig consisted of the Sintech/MTS-300kN (67kips) testing machine with pin-ended bearings. The load and shortening were recorded using the Sintech data acquisition system. The compressive deformation rate was 0.05 mm/min (0.002 in./min).

The measured specimen lengths are given in Table 1. The measured specimen lengths (L) which were different from the lengths of the pin-ended column (L_t) were obtained as the sum of the specimen length and the distance from the platen face to the centre of the end bearings (L_t=L+150 mm (5.9 in.) for 0.60 mm (0.024 in.) series and L_t=L+108 mm (4.3 in.) for 0.42 mm (0.017 in.) series). The first column was found to fracture in the epoxy at loads very close to the ultimate load. It was therefore decided to drill 3 mm (0.012 in.) holes in the corner lips and to place small diameter bolts and nuts to ensure that the corners did not come apart for the first series. These bolts were only located at the ends (four for each) and were only used for the 0.60 mm (0.024 in.) sections. Clamps were used on the lips of 0.42 mm (0.017 in.) sections as shown in Fig. 1.

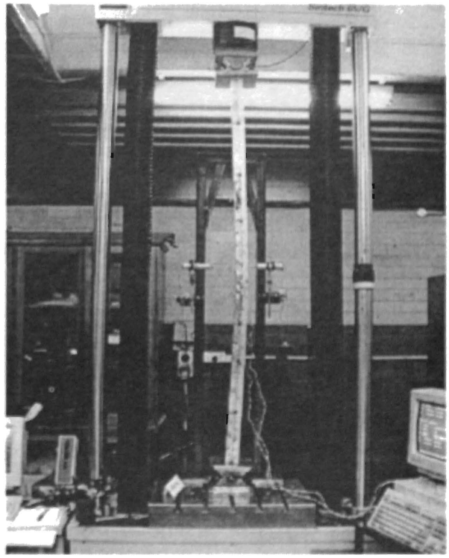
The central deflections were measured using two transducers on two opposite sides of the LB-sections. Two strain gauges were attached at the centre of each side of each column. The transducers and strain gauges were connected to the SPECTRA data acquisition system. During the test, the trial axial load which was 1/15 or 1/10 of the estimated ultimate load was applied and readings of the strain gauges were obtained. Based on a calculation of the location of the initial central eccentricity (e) of the action line as shown in Fig. 2, the location of the column was adjusted at each end. This procedure was repeated until the value of the initial eccentricity (e) was approximately equal to the nominal value of L/1000 which is the maximum normally specified in structural design standards. The measured values and the method of calculation are given in Yang, Hancock and Rasmussen (2002).

Measurement of flexural rigidity (E*I)

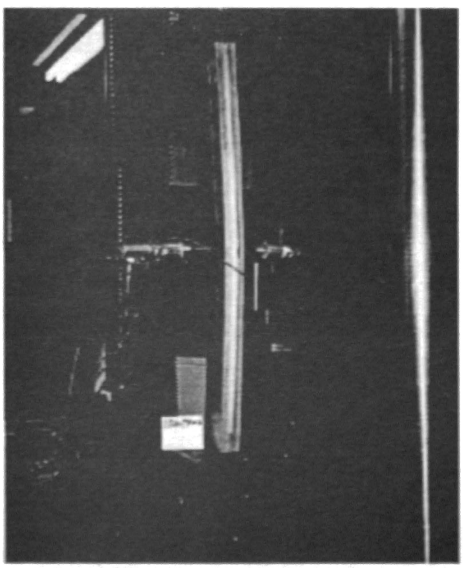
For the LB sections in 0.60 mm (0.024 in.) thickness, the flexural rigidities (E*I) (minor axis) were measured. The ends of each specimen were simply supported and sag of the specimen between supports under self-weight was permitted. A point load was then applied at mid-length. The mid-length deflections were measured with increasing load. The measured flexural rigidities



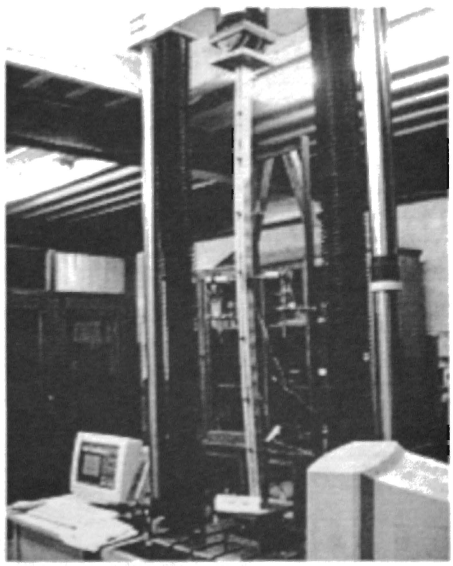
(a) Local buckle at failure



(b) Overall buckle



(c) Local & Overall buckling (Bolts at ends)



(d) Local & Overall buckling (Clamps)

Fig.3 Buckling Mode

and values obtained from the computer program THIN-WALL (Papangelis and Hancock, 1998) using the measured dimensions are shown in Yang, Hancock and Rasmussen (2002).

The experimental results are in good agreement with the theoretical results except for the 060LB40P450 specimen. The mean ratio of the experimental values to the theoretical values was approximately 1.09. This short wide section is very sensitive to a deviation of measured deflection, which may be the main reason for the large difference between the experimental results and theoretical results. The accurate rigidity measurements mean that the epoxy was effective in causing the two hat-shaped halves to act in a fully integral manner.

Test specimen behavior and ultimate load-capacity

Initially the columns remained elastic with slope of the load-shortening diagram approximately constant after some initial take-up. The ultimate loads (P_t) are given in Tables 1 & 2 for each specimen. For the 060LB20 and 060LB30 series, the ultimate load (P_t) was less than the theoretical local buckling load (N_{ol}) so that the slope kept approximately constant until the ultimate load was reached. The 450 mm (17.7 in.) specimens in 0.60 mm (0.024 in.) material and the 550 mm (21.7 in.) specimens in 0.42 mm (0.017 in.) material buckled inelastically soon after the ultimate load with a sudden drop in load. The 900 mm (35.4 in.) specimens in 0.60 mm (0.024 in.) material and the 1100 mm and 1700 mm (43.3 in. and 66.9 in.) specimens in 0.42 mm (0.017 in.) material generally had a more gradual decrease in load until inelastic local buckling occurred at which point there was a sudden drop in load as for the 450 mm (17.7 in.) specimens. Only the 042LB30P1100 specimens appeared to inelastically locally buckle before the ultimate load caused by overall deformation was reached. For the intermediate length columns in 0.42 mm (0.017 in.) thickness and the 060LB40P450 column, the slope reduced continuously from the local buckling load to reaching the ultimate load.

By observation of the surface of the specimens, the local buckling behaviour can be observed. Initially, elastic local buckling occurred with many half-wavelengths occurring along the length of the specimens. Eventually the columns entered the elastic-plastic state and a local plastic mechanism formed accompanied by a sudden drop in load. The specimens (in 0.60 mm (0.024 in.) thickness) with lower web slenderness (b/t) (from 33 to 50) developed roof-shaped mechanisms. However, all the others developed the so-called flip-disc mechanisms at about mid-length on one side as shown in Fig.3(a). All of the test columns are shown in Yang, Hancock and Rasmussen (2002).

The experimental local buckling loads were evaluated from the plots of load-deflection and load-shortening. For the plots of load-deflection, the P-w^2 method (Ventaramaiah, K.R., Roorda, J.) was used to obtain the elastic local buckling load. For the two series of pin-ended tests, the experimental local buckling loads were obtained except for the 060LB20 and 060LB30 specimens whose ultimate loads were lower than the elastic local buckling loads of the section. The experimental local buckling loads (P_{cr}) are given in Table 1 & 2.

As shown in Figs. 3 (c) & (d) taken during the test, after the local buckling load was reached, elastic local buckling can be observed on the concave side of the columns. Due to the small eccentricity of the load, one side of the column was in greater compression than the other and modulation of the local buckling occurred on this side. Finally, interaction of local and overall buckling resulted in failure of column.

It is noticeable that the column strength increases as the plate slenderness (b/t) becomes larger and the overall slenderness becomes smaller for a given column length as shown in Yang, Hancock and Rasmussen (2002).

ANALYSES

Elastic local buckling analyses

The theoretical elastic local buckling loads (N_{ol}) were obtained using the THIN-WALL program (Papangelis and Hancock,1998). The average measured cross-section dimensions of the specimens for each series as well as the measured values of base metal thickness and Young's modulus taken from the coupon tests were used to determine the theoretical local buckling loads. The mean ratio of the experimental local buckling loads (P_{cr}) to the theoretical local buckling loads (N_{ol}) was approximately 1.06 and 0.95 respectively for the two series. These results indicate that the THIN-WALL program was in good agreement with the experimental local buckling load as detailed in Yang, Hancock and Rasmussen (2002).

Finite Element Nonlinear Analyses

The finite element non-linear analysis program "ABAQUS" was used to simulate the behavior of the columns. Further details are given in Yang and Hancock (2002) and Yang, Hancock and Rasmussen (2000), such as element type, material behavior, loading procedure, boundary conditions and geometrical imperfection.

ABAQUS Results

The ABAQUS ultimate loads are given as AB in Yang, Hancock and Rasmussen (2002) where the ratios AB/P_t are computed. The average difference between the ABAQUS results and test results is approximately 6%. For most specimens, the ABAQUS results are slightly higher than the test results. The differences including variability are most likely related to assumed imperfection in the ABAQUS model. Further investigation of the imperfections to use in ABAQUS is required.

TEST RESULTS AND ABAQUS COMPARED WITH THE DESIGN STANDARDS

General

Figs. 4a, 4b & 4c show the test strength (P_t) and the ABAQUS values (AB) compared with a range of design curves for the 30 mm, 40 mm and 50 mm (1.2 in., 1.6 in. and 2.0 in.) sections in 0.42 mm (0.017 in.) material respectively. Figs. 5a, 5b & 5c show same for the 20 mm, 30 mm and 40 mm (0.79 in., 1.2 in. and 1.6 in.) sections in 0.60 mm (0.024 in.) material respectively. In Figs. 4 & 5, the test strengths (P_t) have been non-dimensionalised with respect to the squash load (P_y) as computed for the measured yield stress and dimensions in Table 1 & 2.

Dashed and dash-dotted curves are plotted in Figs. 4 & 5. The dashed curve is the ratio of N_{cRb}/P_y against the column length (l_x). N_{cRb} was calculated using the AISI Specification based on a yield stress R_bf_y as included in Section A3.3.2 of the AISI Specification Supplement No.1(1999). The dash-dotted curve is the ratio of $N_{c0.75}/P_y$ against column length (l_x). $N_{c0.75}$ was calculated based on AS/NZS 4600 with a yield stress of $0.75f_y$ as included in Clause 1.5.1.5(b) of AS/NZS 4600.

The horizontal dashed line represents the ratio of the theoretical local buckling load to the squash load (N_{ol} /P_y) against the column length (l_x). The horizontal solid line represents the ratio of the section capacity to the squash load ($N_{s0.90}/P_y$) against the column length (l_x). $N_{s0.90}$ was calculated based on AS/NZS 4600 with a yield stress of $0.90f_y$ as proposed by Yang and Hancock (2002).

The heavy dash-dot curve is the proposed design curve, which is the ratio of the nominal reduced member capacity to the squash load (N_{cred}/P_y) against the column length (l_x). The proposed design method is described following.

Although each column in 0.60 mm (0.024 in.) thickness had 2 holes in the lips at both ends, the effect of the holes on the ultimate load was not taken into account in the calculation of the section or the member capacity. The numerical values of the strength for each column are given in Tables in Yang, Hancock and Rasmussen (2002)

Test results comparisons with design standards

It can be seen for all sections and lengths that when the elastic local buckling loads (N_{ol}) are lower than the test results (P_t), the test results are generally lower than the design curves at intermediate columns lengths where local and Euler buckling interact.

As shown in Figs. 4, for the columns in 0.42 mm (0.017 in.) thickness, the results are lower than the curves based on AS/NZS 4600 and the AISI Specification. The low values for the

060LB20P450 specimen is probably due to a large load eccentricity. The test was repeated and gave a higher result.

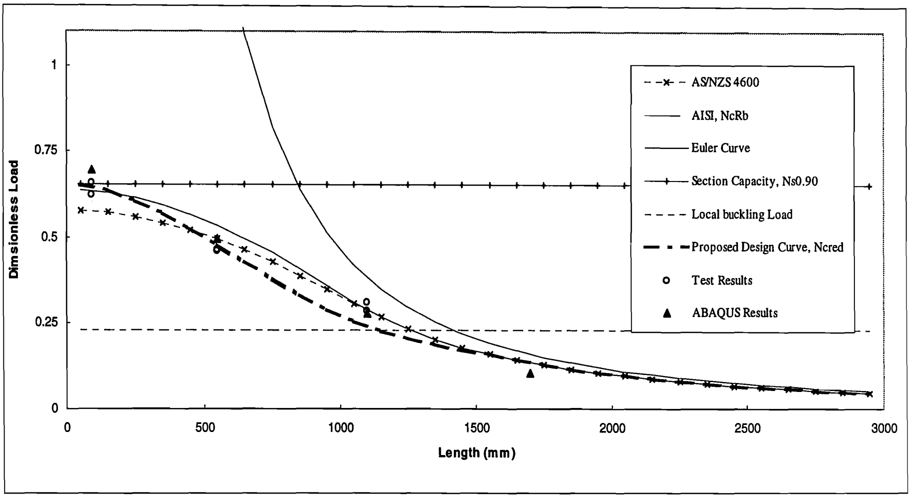


Fig.4a Comparison of Test & ABAQUS Results with Design Standard (0.42 mm) 30x30 mm Sections

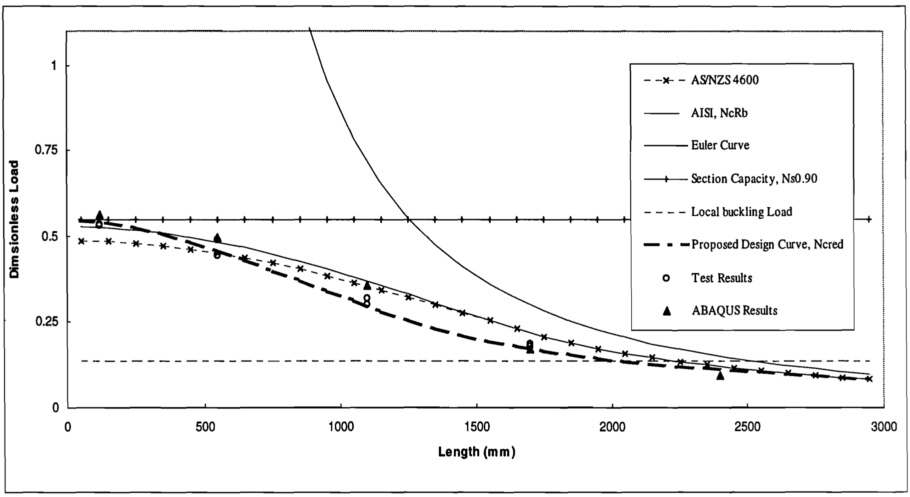


Fig.4b Comparison of Test & ABAQUS Results with Design Standard (0.42 mm) 40x40 mm Sections

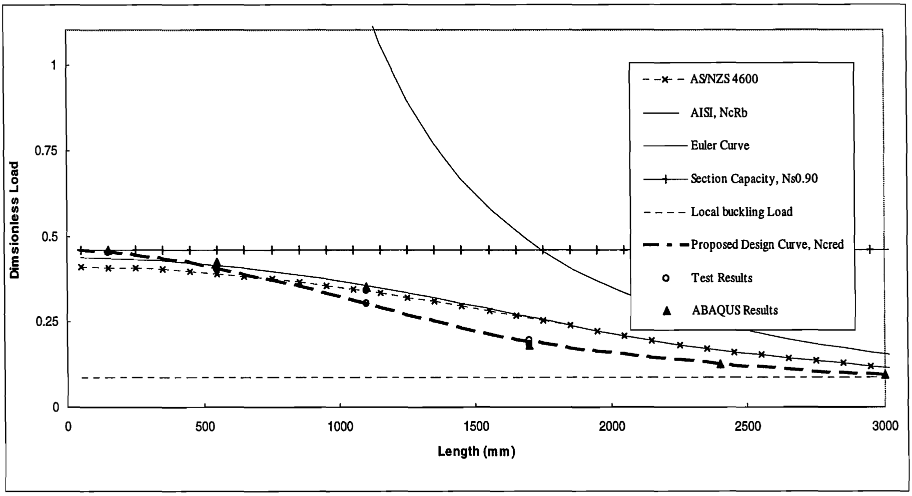


Fig.4c Comparison of Test & ABAQUS Results with Design Standard (0.42 mm)
50x50 mm Sections

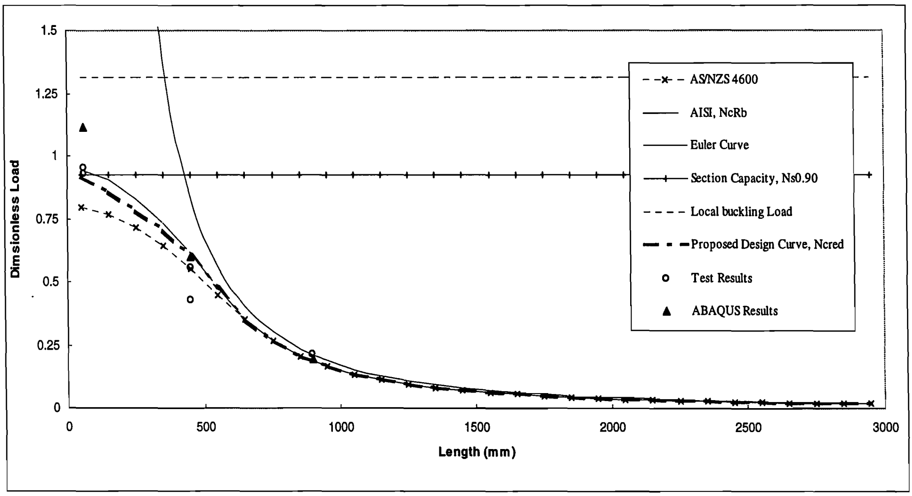


Fig.5a Comparison of Test & ABAQUS Results with Design Standard (0.60 mm)
20x20 mm Sections

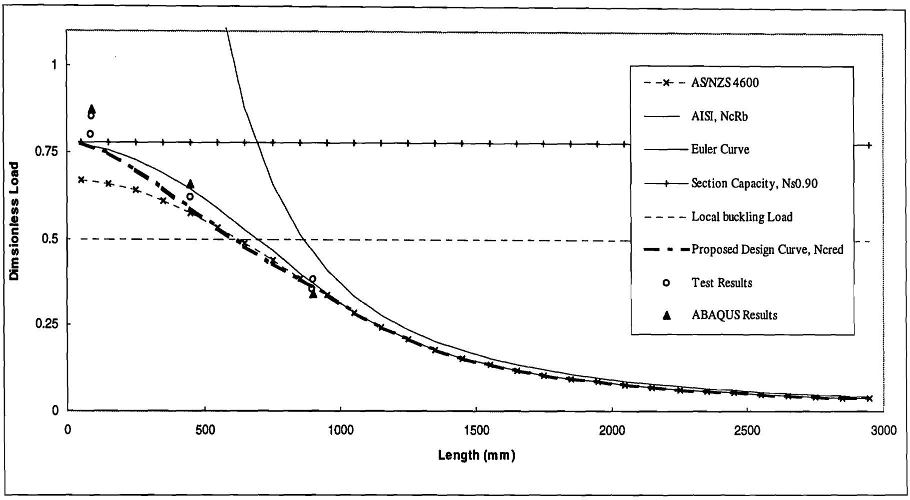


Fig.5b Comparison of Test & ABAQUS Results with Design Standard (0.60 mm)
30x30 mm Sections

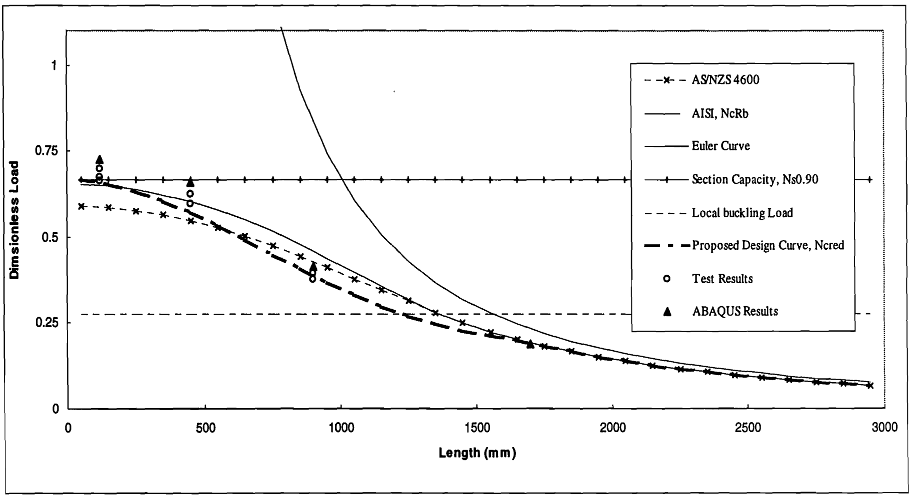


Fig.5c Comparison of Test & ABAQUS Results with Design Standard (0.60 mm)
40x40 mm Sections

For the sections of stockier cross-section (060LB20 and 060LB30), the AS/NZS 4600 design curve based on $0.75f_y$ and the AISI design curve based on a stress R_bf_y are generally slightly conservative.

The conclusion is that a modified design method is required for sections with low local buckling loads as may occur in high strength steel columns with slender plate elements.

ABAQUS results comparisons with design standards

The ABAQUS results show a similar trend to the test results when compared with the design curves although, as detailed above, they are slightly higher probably due to assumption with the imperfections.

For the largest 50 mm (2.0 in.) section in 0.42 mm (0.017 in.) material, two more long columns were simulated using ABAQUS in order to obtain a complete graph of variation with length and to make up the shortage where no test could be performed on such long columns. The lengths were 2400 mm and 3000 mm (94.5 in. and 118.1 in.). The dimensions of cross-section used are the same as 042LB50P1700 (94.5 in. and 118.1 in.) as detailed in Table 2. The ABAQUS results are 7.27 kN and 5.42 kN (1.63 kips and 1.22 kips) for 2400 mm and 3000 mm (94.5 in. and 118.1 in.) respectively. It can be seen that the results are 7% and 4% lower than the curves based on AS/NZS 4600 and the AISI Specification. The longer the column, the less the difference becomes for a given size of cross-section.

MODIFIED DESIGN CURVE

The effect of local buckling on overall buckling behaviour has been studied in several research projects. On the basis of tests and analytical studies, DeWolf, Pekoz, Winter (1974) and Kalyanaraman, Pekoz, Winter (1977) conclude that a satisfactory approach is to calculate the overall buckling load using the effective radius of gyration and the effective area, both calculated at the overall buckling stress. The unified approach proposed by Pekoz (1986) was adopted by the AISI Specification (1986, 1991, 1996) and AS/NZS 4600 (1996) to determine the strength of columns where local and Euler buckling interact. This proposed approach does not take account of the reduction of the radius of gyration (flexural rigidity) resulting from local buckling which was proposed by the earlier researchers. However, for the high strength steel sections, the effect of the reduction in the radius of gyration on the interaction of local buckling and overall buckling is significant, especially buckling of these very slender sections at loads above the local buckling load.

To fit the test data, a new design approach is proposed. The proposed approach consists of two steps. Firstly, the reduced yield point $0.90f_y$, which was determined by Yang and Hancock (2002) for stub columns, is used to replace the yield stress in Clause 3.4 of AS/NZS 4600 and

Section C4 of the AISI specification. Secondly, a reduction factor γ is applied to the radius of gyration as defined in Eq.1. It is a function of the length varying from some limit γ_0 at length $l_x=0$ (here γ_0 taken as 0.65) to 1.0 at length $l_x=1.10*l_{x0}$. Here, l_{x0} is the length where the local buckling load equals the Euler buckling load as defined by Eq. 2. The reduction factor γ accounts for the loss of flexural rigidity due to local buckling. The value of reduced radius of gyration $\gamma*r_x$ is used in Clause 3.3.3.2(a) of AS/NZS 4600 to replace the normal radius of gyration r_x (Section C3.1.2, Eq. C3.1.2-8 of the AISI Specification). It can be seen in Figs 4 & 5 that the proposed design curves based on the reduced the radius of gyration γ fit the test data well.

$$\gamma=\gamma_0+(1-\gamma_0)*l_x/(1.1*l_{x0}) \qquad (1)$$

$$l_{x0}=\pi * r_x * \sqrt{E/f_{ol}} \qquad (2)$$

CONCLUSIONS

Pin-ended column tests with box-sections and constructed from high strength G550 steel have been successfully performed. The plate slenderness (b/t) ranged from 33 to 119 and the column slenderness (L/r_x) ranged from 27 to 148. A load eccentricity which produced a column response equivalent to L/1000 was used for all tests. ABAQUS simulations of the test results with local imperfections and overall eccentricity were made.

- The ABAQUS results (AB) were generally in good agreement with the test results (P_t). The difference was on average less than 6% for all columns although the ABAQUS results were higher. More detailed investigation of imperfections to use in ABAQUS is required.

- The columns with stockier plate elements, which had high local buckling stresses (f_{ol}), failed by overall buckling. The test results and the ABAQUS results were close to the curves based on AS/NZS 4600 and the AISI Specification. For very long columns with slender plate elements which had lower local buckling stresses (f_{ol}), failure was still governed by overall buckling although local buckling occurred. However, for the intermediate length columns, the failure mode was governed by the interaction of local and overall buckling. The interaction of local and overall buckling reduced the column strength and made the test results lower than the design curves. The worst case had a difference between test results and the results based on AS/NZS 4600 and the AISI Specification of about 14%,-which means that for the slender sections AS/NZS 4600 and the AISI method are unconservative.

- To account for the loss of flexural rigidity due to local buckling for the slender sections, a reduction of the radius of gyration is needed to take account of interaction buckling in the design curve. The proposed design curve based on a reduction factor (γ) fits the test data well. So the reduced radius of gyration $\gamma*r_x$ may be used in the design curve for the slender sections.

- Since ABAQUS results were in reasonably good agreement with test results, ABAQUS can be used to simulate long column tests which could not be performed in the available machine.

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REFERENCES

American Iron and Steel Institute. (1997). "1996 Edition of the Specification for the Design of Cold-Formed Steel Structural Members", Washington, DC, USA.

American Iron and Steel Institute. (2000). "1996 Edition of the Specification for the Design of Cold-Formed Steel Structural Members, Supplement 1, July 1999”, Washington, DC, USA.

American Society for Testing and Materials A611. (1997). "Standard Specification for Steel Sheet, Carbon, Cold-Rolled, Structural Quality", Philadelphia, PA, USA

American Society for Testing and Materials A653. (1997). "Standard Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process", Philadelphia, PA, USA.

American Society for Testing and Materials A792. (1994). "Standard Specification for Steel Sheet, 55% Aluminum-Zinc Ally-Coated by the Hot-Dip Process", Philadelphia, PA, USA

Bijlaard P.P., and Fisher G.P., "Interaction of column and local buckling in compression members", NACA TN2640 1953.

Dewolf, J.T., Pekoz, T., and Winter, G., “Local and overall buckling of cold formed steel members”, Journal of the Structural Division, ASCE, Oct., 1974.

Galambos, T.V., “Guide to stability design criteria for metal structures”, John Wiley & Sons, Inc., 1988.

Graves-Smith, T.R., "The ultimate strength of columns of arbitrary length", Symposium on thin - walled steel structures, Swansea, 1967.

Hancock, G.J., "Nonlinear analysis of thin sections in compression" Journal of Structural Engineering, Vol.107, No.ST3, 1981

Hibbitt, Karlsson & Sorensen, Inc., "ABAQUS/Standard User's Manual", Ver. 5.7, 1997

Kalyanaraman, V., Pekoz, T., and Winter, G., "Unstiffened compression elements", Journal of the Structural Division, ASCE, Sept., 1977.

Levy, S. Wooley, R.M., and Kroll, W.D., "Instability of simply supported square plate with reinforced circular hole in edge compression", Research Paper RP1849, Vol.39, Dec. 1947

Mathcad 2000 Professional, (1999), ©1989-1999 Mathsoft Inc.

McAdam, J.N, Brockenbrough, R.A., LaBoube, R.A., Pekoz, T, and Schneider, E.J., "Low strain hardening ductile steel cold-formed members", 9th International Specialty Conference on Cold-Formed Steel Structures, St Louis, Missouri, Nov 1988.

Miller T.H., Pekoz T., "Unstiffened strip approach for perforated wall studs", Journal of Structural Engineering, Vol.120, No.2, 1994

Papangelis, J.P., Hancock, G.J. THIN-WALL 2.0 (1998), Centre for Advanced Structural Engineering, Department of Civil Engineering, University of Sydney.

Pekoz, T., "Development of a unified approach to the design of cold-formed steel members", Research Report, American Iron and Steel Institute, 1986.

Pekoz, T., "Development of a unified approach to the design of cold-formed steel members", 8th International Specialty Conference on Cold-Formed Steel Structures, St. Louis, Missouri, Nov., 1986

Rogers, C.A., Hancock, G.J. (1997), "Ductility of G550 sheet steels in tension", Journal of Structural Engineering, ASCE, Vol. 123, No. 12, 1586-1594.

Rhodes J., Harvey J.M., "Examination of plate post-buckling behavior", Journal of the Engineering Mechanics Division, Vol.103, No. EM3, June, 1977.

Roger, C.A., Hancock, G.J., " Ductility of G550 sheet steels in tension-elongation measurements and perforated tests", Research Report No. R735, School of Civil and Mining Engineering, University of Sydney

Shanmugam, N.E. Chiew S. and Lee S., "Strength of thin-walled square steel box columns", Journal of Structural Engineering, ASCE, Vol. 113, No. 4, 818-831.

Standards Australia / Standards New Zealand. (1996). "Cold-formed steel structures - AS/NZS 4600", Sydney, NSW, Australia

Standards Australia. (1993). "Steel sheet and strip - Hot-dipped zinc-coated or aluminium/zinc coated - AS 1397", Sydney, NSW, Australia

Van der Neut, A., "The interaction of local buckling and column failure of thin-walled compression members", Proceedings, 12th International congress applied mechanics, Springerverlag, Germany, 1969.

Walker, A. C., "Design and analysis of cold-formed sections", International Textbook Company Limited, 1975

Wu, S., Yu, W.W, and LaBoube, R.A. (1996a), "Strength of flexural members using structural Grade 80 of A653 Steel (deck panel tests)", Second Progress Report, Department of Civil Engineering, University of Missouri-Rolla, November.

Wu, S., Yu, W.W, and LaBoube, R.A. (1996b), "Flexural members using structural Grade 80 of A653 steel (deck panel tests)", 13th International Specialty Conference on Cold-Formed Steel Structures, St Louis, Missouri, Oct 1996, pp. 255-274.

Yang, D., Hancock, G.J., "Compression tests of cold-reduced high strength steel stub column", Research Report No. R815, School of Civil and Mining Engineering, University of Sydney, 2002

Yang, D., Hancock, G.J., and Rasmussen, K.J.R. "Compression tests of cold-reduced high strength steel long column", Research Report No. R816, School of Civil and Mining Engineering, University of Sydney, 2002

NOTATION

AB	ultimate load from ABAQUS
b	flange width
d	lip width
e	eccentricity
E	Young's modulus of elasticity
E*I	minor axis flexural rigidity
f_y	yield stress
h	web width
hl	glued lips thickness
l_x, l_{x0}	length of column
L	length of column
L_t	length of pin-ended column
N_{cred}	reduced member capacity
N_{ol}	elastic buckling load
N_s	nominal section compression capacity
P_{cr}	elastic buckling load of test
P_e	Euler load
P_t	ultimate load of test
P_y	squash load
r	radius of corner
R	radius of corner
r_x	radius of gyration of the cross-section
t_b	thickness of base metal
γ_0, γ	reduction factor of radius of gyration