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CYCLIC BEHAVIOUR OF HOLLOW AND FILLED AXIALLY-LOADED MEMBERS

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SUMMARY

Hollow section members are often employed as bracing elements, for both structural and aesthetic reasons. This paper describes an investigation of the response of such members to cyclic axial loading. The influences of concrete or mortar infill and of member slenderness are addressed.

Steel and composite members employing three different section sizes were subjected to monotonic and cyclic axial displacements in the inelastic range. The monotonic test specimens had an aspect ratio of three to promote local buckling, while preventing overall lateral buckling. On the other hand, the cyclic tests considered members of two different overall lengths: 1100mm and 3300mm. In the case of the monotonic test specimens and the shorter cyclic test specimens, both hollow and filled specimens were tested. The normalised slendernesses of the cyclic test specimens varied from 0.4 to 3.2, to cover the possible practical range, and both elastic and inelastic buckling was observed. Failure of the less slender specimens was initiated by local buckling in compression, followed by rupture in tension. The presence of concrete infill was observed to influence this mode of failure. Local tensile end-failures were observed with some specimens of relatively small cross-section. The ductility capacities and energy dissipation of the individual specimens are compared, and the effect of slenderness and infill are quantified.

To aid interpretation of the experimental results, the general purpose structural analysis software LUSAS was employed to study local inelastic behaviour. This showed that under tension loading, the presence of the concrete infill can lead to non-negligible hoop stresses in the steel section, which in turn affect the ductility capacity of the member. Design predictions of the buckling strength and post-buckling resistance of the specimens are compared with the experimental results. The implications of the results on seismic design procedures for structures incorporating these types of members are also presented and discussed.

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INTRODUCTION

Rectangular (RHS) and square (SHS) hollow sections are often employed as bracing members in earthquake-resistant frames, for structural as well as aesthetic reasons. As part of the capacity design philosophy, energy is dissipated through critical members and components, which are expected to undergo inelastic cyclic deformations without suffering significant loss of strength. Clearly, in the case of braced frames, these critical members are the diagonal braces, for which a detailed assessment of the cyclic response is fundamental to the seismic design process.

During the last few decades, several studies have been undertaken to examine the inelastic behaviour of bracing members. Earlier tests by Popov et al [1] reported that once a member buckled during cyclic testing, its compressive capacity in subsequent cycles decreased and that the axial stiffness of a specimen during tension loading deteriorated with each cycle to larger amplitude displacements. Further work by Jain et al [2] suggested that the post-buckling reduction in compressive strength could be expressed as a function of the effective slenderness of the member. More recently, Remennikov & Walpole [3] established that the post-buckling compressive capacity of steel bracing members can vary between 20% and 100% of that in the first cycle depending on the slenderness ratio of the brace, with slender braces suffering the greater reduction. Tremblay [4] found that fracture of rectangular hollow bracing members depends strongly on their slenderness ratio and, to a lesser extent, on the width-to-thickness ratio of the cross-section and the imposed displacement history. It was reported that slender braces sustained higher ductility levels prior to fracture, most likely because the strain demand in the plastic hinge reduces with brace slenderness. Tremblay's results suggest that slender braces perform well during cyclic testing, which agrees with the findings of Jain et al [2].

Compared to other forms of steel member, hollow sections are very effective at resisting axial loads. However, when such sections possess thin walls, they are susceptible to local buckling at high compressive strains. The onset of local buckling reduces the ductility of the brace member and may lead to brittle failure. This has been noted in a number of experimental studies on cold-formed hollow section bracing members (for example, [1] – [4]). It was envisaged that filling hollow steel sections with concrete or mortar would improve their squash load, but more importantly, it would delay the onset of local buckling and improve the member's post-buckling response. In addition, the inherent improvement in fire resistance provided by composite members may also lend this member type as a viable option in practical design situations.

A large amount of research has been performed into the response of concrete-filled RHS stub columns, much of which is summarised in [5] and [6]. It was found that the presence of concrete infill eliminated or delayed local buckling of steel hollow sections, and increased significantly the ductility of the section. At certain values of longitudinal strain, the infill begins to increase in volume due to microcracking, which induces concrete confinement by the steel tube. The confining pressure is less, and the material degradation greater, for square rather than circular sections.

Comparatively less research has been performed on void-filled RHS braces subjected to cyclic axial loading. Earlier experimental work by Liu & Goel [7] on the cyclic behaviour of cold-formed steel RHS bracing members filled with concrete found that the infill improved the specimens buckling, post-buckling and tensile capacities, reduced the severity of local buckling and hence delayed cracking of the steel, and ultimately increased the member's ductility. The presence of concrete infill led to the greatest improvement in performance for specimens with larger width-to-thickness ratios and smaller overall slenderness ratios, as these members are more susceptible to local buckling.

Zhao et al [8] tested fixed-ended cold-formed steel rectangular hollow sections filled with normal and lightweight concrete under axial cyclic loading. As expected, it was found that filling the hollow sections increased their first cycle peak load, post peak residual strength, ductility and energy absorption capabilities. Similar to Liu & Goel [7], they found the improvements were more significant in sections with thinner walls, for which the first cycle peak compression loads and the energy dissipated increased by up to 100% and 85%, respectively. The improvements were also more significant for the members filled with normal concrete rather than light-weight concrete.

Due to uncertainties related to the actual contribution of the infill during inelastic cyclic response, the use of composite members as dissipative diagonals in concentrically braced frames has not been addressed within the current Eurocode 8 provisions [9]. This paper therefore describes an experimental programme undertaken as part of a wider investigation in support of EC8 development, focusing on the comparative behaviour of hollow and filled bracing members [10].

SPECIMEN DETAILS AND TEST SET-UP

All test specimens comprised rectangular hollow sections (RHS) or square hollow sections (SHS) manufactured from cold-formed steel S235JRH, with a nominal yield strength of $235N/mm^2$ and an ultimate strength of between $360N/mm^2$ and $510N/mm^2$ [11]. The actual material characteristics of the steel specimens were determined from three tensile coupons taken from each length of hollow section, and tested in accordance with the European Standard BS EN 10002-1:2001 [12]. The average measured yield strength of the material was 309.2MPa with a coefficient of variation (COV) of 0.17. Detailed results and discussion of the material properties are presented elsewhere [13].

Three cross-sectional geometries were considered, namely 50x25x2.5RHS, 40x40x2.5SHS and 20x20x2.0SHS. With respect to local buckling, international codes ([14], [15], [16], [17]) classify the walls of these sections as Plastic or Class 1. Such sections should form plastic hinges capable of sustaining the moment capacity at significant levels of rotation. The measured d/t ratios of these specimens are given in Table 1, in which d is defined as by Eurocode 3 [18] as h – 3t, where h is the greater overall dimension of the section parallel to the principal axis and t is the wall thickness.

Three series of cyclic tests were conducted. The first and second series were on hollow and filled specimens, respectively, with overall lengths (L_T) of 1100mm and unstiffened lengths of 850mm. This allowed a direct comparison of the influence of infill on the performance of brace specimens. To enable a wider investigation on the influence of brace slenderness on the response of bare steel brace specimens a further set of tests, namely test series 3, was carried out on hollow steel specimens of overall length (L_T) 3300mm. This gives a range of normalised slenderness $\overline{\lambda}$ between 0.4 and 3.2, where $\overline{\lambda}$ is defined by Eurocode 3 [16] and Eurocode 4 [18] for non-slender cross-sections as $(N_{pl,R}/N_{cr})^{0.5}$, in which $N_{pl,R}$ and N_{cr} are the plastic section capacity and theoretical elastic (Euler) buckling load, respectively. Note that Eurocode 4 takes account of the additional strength provided by the concrete ($A_c f_{ck}$) in $N_{pl,R}$ and uses an equivalent stiffness $(EI)_e$ in estimating N_{cr}.

Each specimen tested was assigned a unique designation from which the load and specimen type could be identified:

- The first group of letters describes the loading type and length of specimen:
  CyIS = Cyclic intermediate length specimen (1.1m); CyLS = Cyclic long specimen (3.3m)
- The number before the dash indicates the test number in that particular series of tests.
- The number immediately after the dash is the nominal depth (D) of the hollow steel section.
- The letters ‘H’ and ‘F’ indicate hollow specimens and specimens filled with mortar, respectively.

As outlined in Table 1, twenty-one cyclic tests were performed in total – six on filled specimens and fifteen on unfilled specimens. Monotonic tensile and compressive tests on short specimens, all with unstiffened length-to-width ratios of three, were also carried out. These included four monotonic tension tests and four monotonic compression tests on short 40x40x2.5SHS specimens, two of each of which were filled with mortar.

Table 1. Cyclic test programme and specimen resistances.

Test ID	Section Size	$\bar{\lambda}$	*d/t*	Measured			Buckling Capacity Predictions		
				F_y (kN)	F_{max} (kN)	F_c (kN)	$\frac{F_c}{N_{EC,mat}}$	$\frac{F_c}{N_{EC,\sec t}}$	$\frac{F_c}{N_{AISC}}$
Series1	(Hollow; L_T = 1100mm)								
CyIS1-40H	40x40x2.5	0.40	13.1	100.4	112.9	104.3	1.25	1.02	1.25
CyIS2-40H	40x40x2.5	0.40	13.1	101.5	112.4	106.6	1.28	1.04	1.28
CyIS3-20H	20x20x2.0	0.88	6.7	26.0	45.4	27.0	1.05	1.14	1.03
CyIS4-20H	20x20x2.0	0.88	6.7	29.8	46.0	31.9	1.24	1.35	1.22
CyIS5-50H	50x25x2.5	0.64	17.1	78.1	111.5	82.3	1.09	0.99	1.08
CyIS6-50H	50x25x2.5	0.64	17.1	83.1	111.5	85.1	1.12	1.03	1.12
Mean							1.17	1.10	1.16
COV							0.08	0.12	0.09
Series2	(Filled; L_T = 1100mm)								
CyIS1-40F	40x40x2.5	0.56	12.8	181.2	202.2	190.1	1.19	0.99	1.22
CyIS2-40F	40x40x2.5	0.56	12.8	162.3	202.2	168.5	1.05	0.88	1.08
CyIS3-20F	20x20x2.0	1.00	6.8	37.0	52.5	37.9	1.21	1.13	1.24
CyIS4-20F	20x20x2.0	1.00	6.8	36.7	52.4	38.1	1.22	1.13	1.25
CyIS5-50F	50x25x2.5	0.73	17.3	95.7	118.2	100.1	1.05	0.94	1.09
CyIS6-50F	50x25x2.5	0.73	17.3	98.6	119.0	102.8	1.08	0.96	1.12
Mean							1.14	1.00	1.17
COV							0.07	0.10	0.07
Series3	(Hollow; L_T = 3300mm)								
CyLS1-40H	40x40x2.5	1.30	12.9	123.1	143.9	53.5	1.15	1.14	1.12
CyLS2-40H	40x40x2.5	1.31	12.9	124.1	143.8	53.1	1.14	1.13	1.11
CyLS3-40H	40x40x2.5	1.31	12.9	126.5	142.5	50.4	1.08	1.07	1.05
CyLS4-20H	20x20x2.0	3.17	6.5	63.1	70.2	5.7	1.12	1.05	1.34
CyLS5-20H	20x20x2.0	3.00	6.0	64.4	69.3	5.6	1.04	0.98	1.24

CyLS6-20H	20x20x2.0	3.00	6.0	63.6	72.3	5.8		1.09	1.03	1.30
CyLS7-50H	50x25x2.5	1.95	17.3	99.4	117.7	18.9		1.30	0.87	1.00
CyLS8-50H	50x25x2.5	2.18	16.9	162.6	187.7	17.6		1.44	0.77	0.92
CyLS9-50H	50x25x2.5	2.18	16.9	165.4	190.1	15.1		1.48	0.66	0.78
Mean								1.20	0.97	1.10
COV								0.13	0.17	0.17

In all composite specimens, the hollow steel members were filled with mortar composed of 22.8% and 77.2% cement and fine aggregate (sand), respectively, of dry mass and a water/cement ratio of 0.51. The mix design also contained shrinkage reducing and self-compacting admixtures. The mix was carefully designed to have high strength, low shrinkage and good compaction [19]. Its average compressive and tensile splitting strengths at 28 days were 24N/mm^2 and 2.53N/mm^2, respectively.

As shown in Figure 1, fixed end restraints were employed for all test specimens and stiffener plates were placed to provide an adequate length of weld for the required transfer of force between the specimen and the supports, and to encourage uniaxial flexural buckling of the cyclic test specimens. The specimens were manufactured in a rigid jig to ensure that the base plates of the specimens were parallel, the holes aligned properly and all test specimens within each group had the same length. In the composite specimens, the area surrounding the stiffener was filled with a high-strength glass fibre resin ("Plastic Padding Glass Fibre Resin"). This was activated with approximately 3% hardener paste (Dibenzoyl Peroxide, paste with plasticizer). The properties of the resin are described in more detail in [13]. The ends of the specimens were then ground flat prior to welding the base plates.

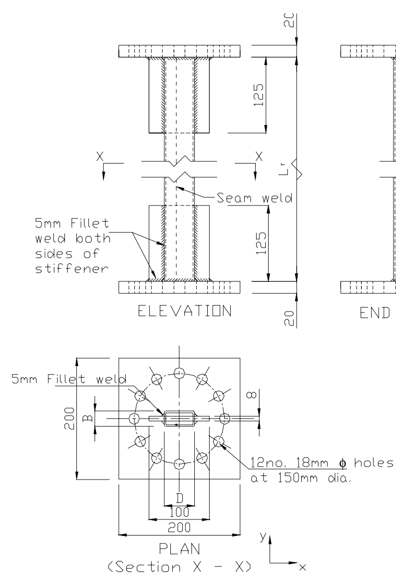


Figure 1. Test specimen.

Two separate test rigs were employed, one for the monotonic tensile tests and cyclic tests of intermediate length (1100mm) specimens, and another to test the long (3300mm) specimens. Both test rigs comprised of four solid columns supporting an upper loading platen, with loading being applied through a hydraulic actuator attached to the lower platen. More detailed descriptions of the test rigs employed are given elsewhere [13, 20]. For the cyclic tests, loading was applied according to the provisions of the ECCS [21].

The recommended complete testing procedure was followed, for which the axial deformation history is shown in Figure 2. The yield displacements measured in the monotonic tensile tests on short specimens were used to determine the amplitudes of the cycles.

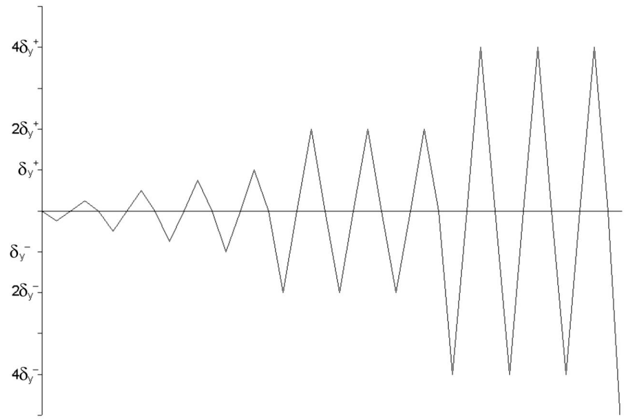


Figure 2. Cyclic displacement waveform for ECCS procedure.

MONOTONIC TEST RESULTS

Four monotonic compression tests were performed on short 40x40x2.5SHS specimens (L_T = 370mm), two of which were filled with mortar. Figure 3 compares the compression load-displacement responses of one filled and one unfilled 40x40x2.5SHS specimen. The other specimens displayed very similar responses. To directly compare the influence of the mortar, the load-displacement curves are normalised by the yield strength obtained from the tensile coupon tests (f_y) times the actual cross section area of steel (A_s). It is evident that the infill increases the buckling capacity of the strut, but more so the post-buckling capacity of the strut. In fact, the maximum compressive resistance of both composite struts was increased by 26% due to the presence of the mortar. This is notably higher than the Eurocode 4 [18] prediction of 13 percent, which suggests that confinement of the mortar by the steel section significantly increased its compressive resistance (by about 70 %). Both inward and outward local buckling occurred in the hollow compression specimens, whereas the mortar in the filled specimens prevented inward local buckling.

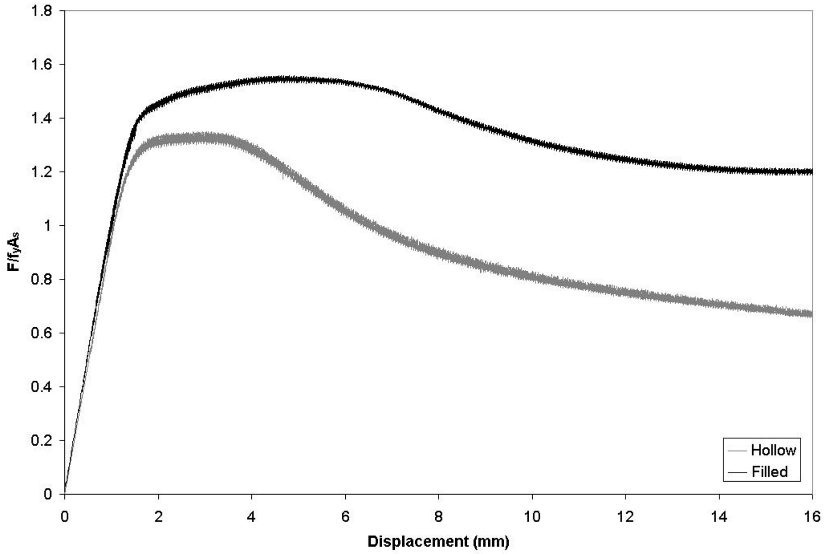


Figure 3. Monotonic compression load-displacement curves

The affect of infill on the behaviour of steel hollow sections in tension was also investigated in two pairs of monotonic tensile tests. The difference in the axial stiffness and ultimate resistance of the hollow and filled specimens was less than 6%. However, as expected, after yielding the filled specimens did not exhibit necking, as observed in the hollow sections. As a result, the ductility capacity of the filled specimens under monotonic tension was on average about 35% lower than that of the hollow counterparts. Nonetheless, under cyclic loading other parameters also influence ductility capacity. These are investigated in subsequent parts of this paper.

Numerical analyses which replicated the monotonic tensile tests were performed using the finite element software package LUSAS. Non-linear thick shell elements, with elastic-plastic material properties, were employed to model the steel hollow section. For simplicity, the corner radii were ignored. The mortar infill was modeled using brick elements and was assumed to remain elastic throughout the analysis. The steel and mortar were assumed to be fully debonded and thus, friction was ignored in the analysis. The lateral inward deformation of the wall of the steel section at mid-height is plotted against the applied axial displacement in Figure 4. Up to yield, the presence of the infill medium reduces necking to less than one quarter of that observed in the hollow member. After yield, necking increases substantially in the hollow specimen, but reduces in the filled specimen. The reduction of necking in the composite section results in much greater hoop stresses, which increase with axial elongation of the specimen, as shown in Figure 5. In comparison, the maximum hoop stress in the hollow section occurs just before yielding and is less than 0.3MPa (less than 1% of that experienced by the filled specimen at yield). This large increase in hoop stress affects the critical longitudinal strain at fracture, as observed in the monotonic tensile tests.

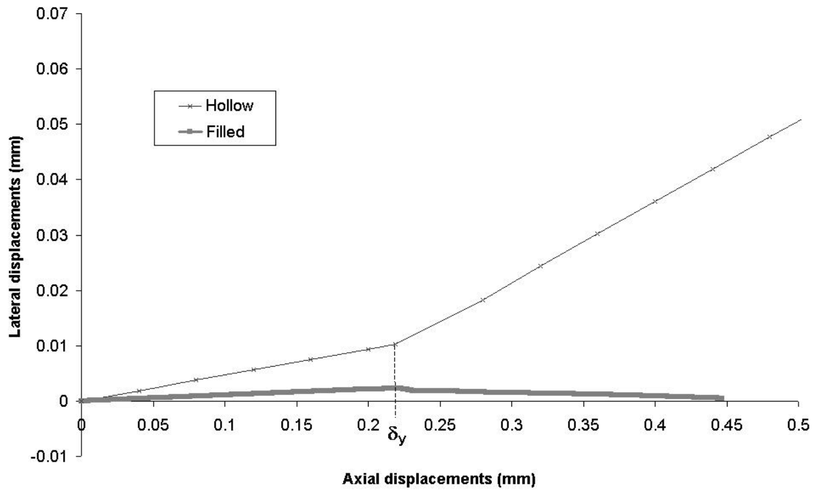


Figure 4. Estimated necking of hollow and filled tensile specimens.

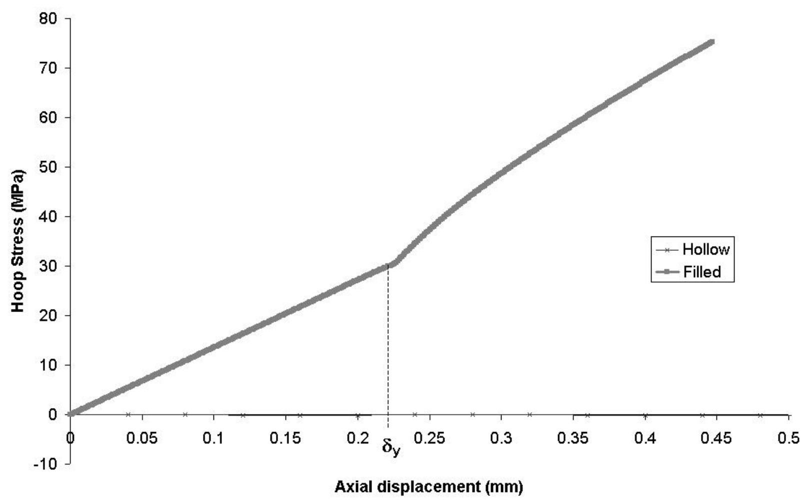
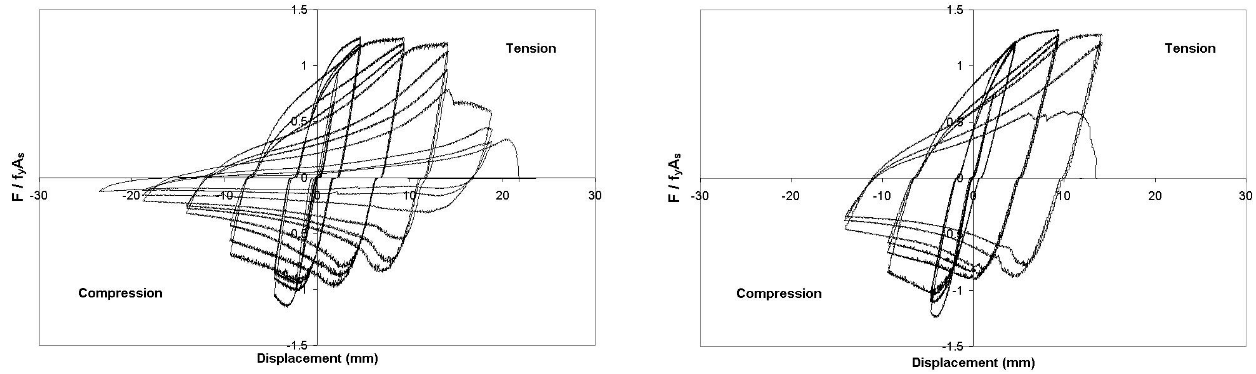


Figure 5. Hoop stresses in hollow and filled tensile specimens estimated using LUSAS.

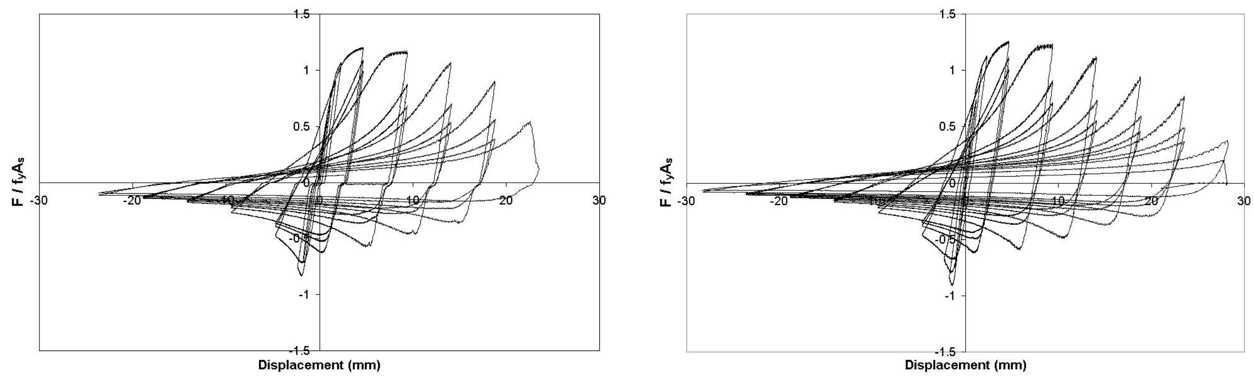
CYCLIC TEST RESULTS

The measured resistance properties F_y, F_{max}, and F_c of the cyclic test specimens are summarised in Table 1, where F_y is the yield strength of the specimens, F_{max} the maximum tensile strength, and F_c the first buckling load in compression. Series 1 and 2 refer to the 1100mm long hollow and filled specimens, respectively and Series 3 refers to the 3300mm long hollow specimens. In both Series 1 and 2, the 20x20x2.0SHS specimens experienced biaxial buckling, while the less slender specimens experienced uniaxial buckling. Typical hysteretic curves obtained from both test series are presented in Figures 6, in which the load values are normalised by the yield capacity of the section (f_yA_s). All specimens in these two test series were tested to failure. In Series 3, all square hollow sections experienced biaxial buckling, and only the 50x25x2.5RHS specimens buckled uniaxially. Typical normalised hysteretic curves obtained from test Series 3 are shown in Figure 7. No sign of fracture was evident in the 40x40x2.5SHS and 50x25x2.5RHS specimens of Series 3 up to displacements exceeding 40mm in both directions, where the stroke of the actuator was reached. However, local buckling occurred at mid-height was observed. In contrast, the 20x20x2.0SHS specimens of Series 3 all failed suddenly in tension close to one end of the unstiffened length, as shown in Figure 8.



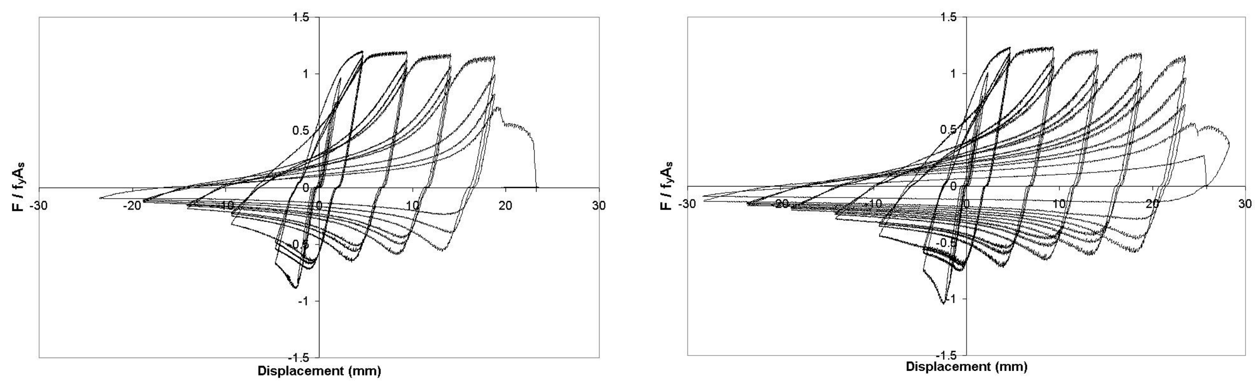
(a) CyIS1-40H (40x40x2.5SHS)

(d) CyIS1-40F (40x40x2.5SHS)



(b) CyIS4-20H (20x20x2.0SHS)

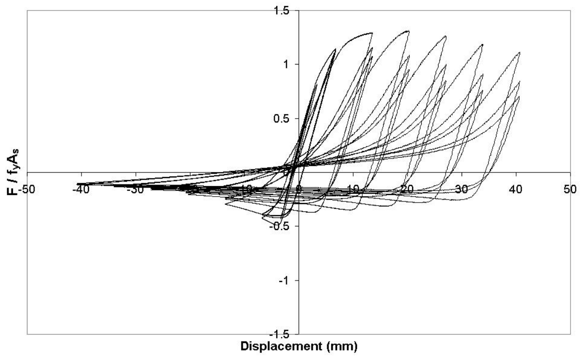
(e) CyIS3-20F (20x20x2.0SHS)



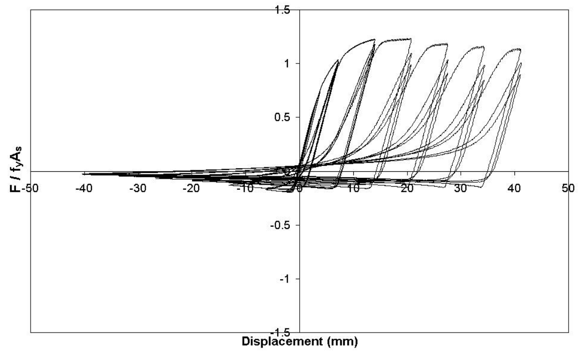
(c) CyIS5-50H (50x25x2.5RHS)

(f) CyIS5-50F (50x25x2.5RHS)

Figure 6. Experimental load-displacement response of 1100mm long specimens: (a) – (c) hollow specimens, and (d) – (f) filled specimens.



(a) CyLS1-40H (40x40x2.5SHS)



(b) CyLS7-50H (50x25x2.5RHS)

Figure 7. Experimental load-displacement response of 3300mm long hollow specimens.

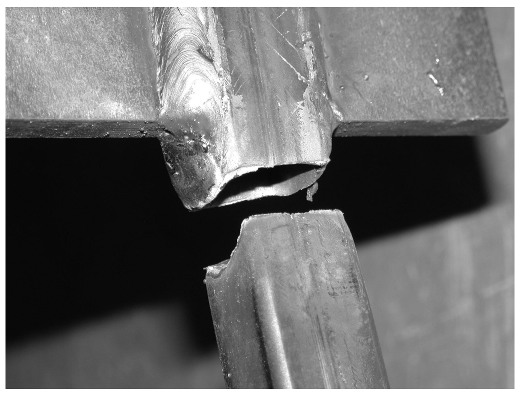
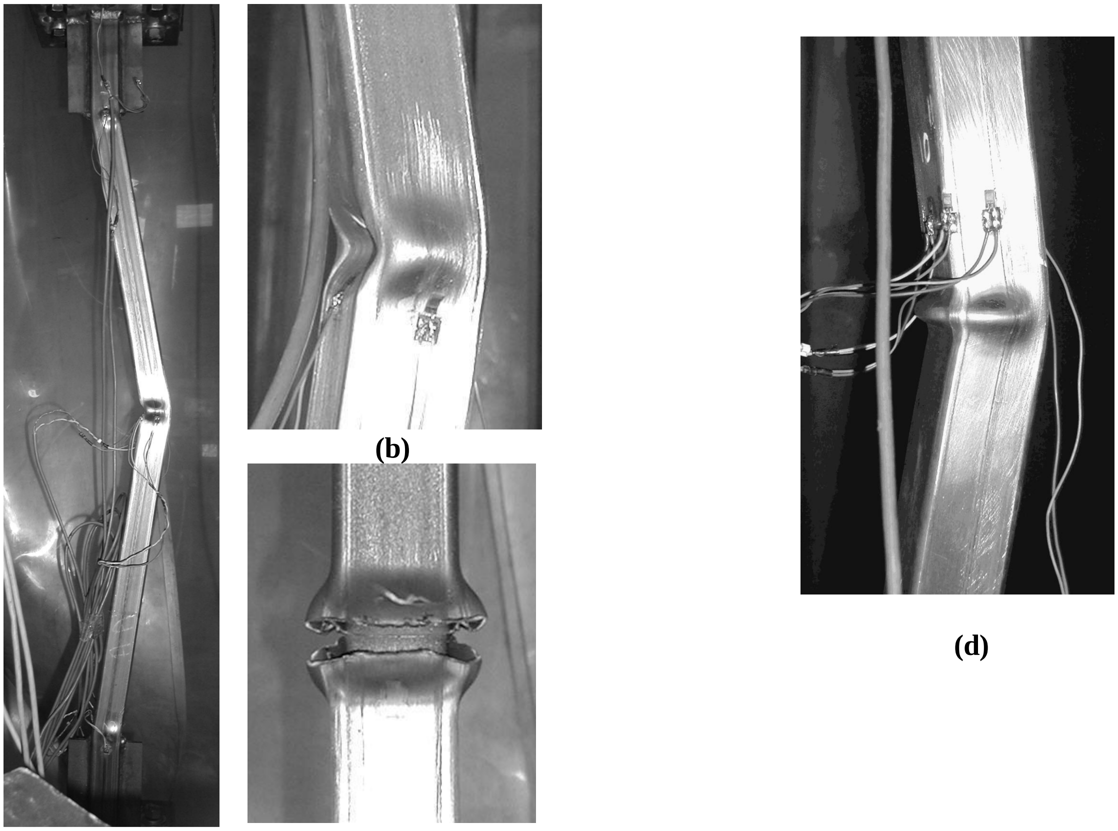


Figure 8. Typical failure of 20x20x2.0SHS long specimen (Series 3)

In Series 1, the hollow steel members comprising sections with relatively large width-to-thickness ratios (i.e. 40x40x2.5SHS and 50x25x2.5RHS specimens) all experienced inward and outward local buckling in compression (as shown in Figure 9(b)) and necking in tension, whereas the smaller 20x20x2.0SHS specimens experienced necking only. These local phenomena (i.e. local buckling and necking) occurred both at mid-height of the specimens and close to the end stiffeners (Figure 9(a)). Local buckling caused progressive strain localisation which was accentuated with each compressive cycle. When the steel was subsequently stretched in tension, small cracks formed at these locations, and repeated cycling eventually caused failure. Fracture occurred at mid-height (as shown in Figure 9(c)), in all specimens except CyIS4-20H which fractured close to an end stiffener.

Unlike the equivalent hollow sections, in Series 2 inward local buckling was not observed in the filled 40x40x2.5SHS specimens, as shown in Figure 9(d). However, both of the filled specimens of this size failed suddenly, in a more brittle manner than the other specimens. The filled 20x20x2.0SHS specimens behaved similarly to the equivalent unfilled specimens, with the exception that the corner on the concave side of the laterally buckled Specimen CyIS3-20F buckled locally outwards at mid-height, while the opposite corner was indented over a height of approximately 40mm. Inward and outward local buckling were observed at mid-height of the filled 50x25x2.5RHS specimens, but this local buckling occurred at larger longitudinal deformations than in the equivalent hollow sections.



(a) (b) (c) (d)

Figure 9. (a) Local and overall buckling, (b) local buckling, and (c) fracture at mid-height of a 40x40x2.5 hollow steel specimen; (d) local buckling of 40x40x2.5 filled specimen

Buckling capacity

In Table 1, the experimental initial buckling loads for each specimen type (F_c) are compared with unfactored design strengths predicted using European (N_{EC}) (Eurocode 3 [16], Eurocode 4 [18]) and American (N_{AISC}) [14] specifications. For cold-formed members, Eurocode 3 allows the use of either the basic material yield strength in conjunction with curve 'b', or the yield strength from full section tensile tests in combination with curve 'c'. For the 'simplified method of design', Eurocode 4 [18] recommends using design curve 'a' for filled steel hollow sections.

Material properties obtained from the tensile coupon tests were used with buckling curves 'a' and 'b' for the filled and hollow specimens, respectively. Good agreement is observed between these design values ($N_{EC,mat}$) and those predicted using the American specification (N_{AISC}), most of which underestimate the actual buckling resistance of the specimens (Table 1). For comparison, the measured section yield strengths from the tensile tests on short specimens were also used to predict the buckling capacity ($N_{EC,sect}$) of the filled and hollow specimens using buckling curves 'a' and 'c', respectively. These design values obtained using the full section strength gave closer estimations to the actual buckling strengths for most specimens. In fact, the buckling capacity is slightly overestimated for the filled 40x40x2.5SHS and 50x25x2.5RHS specimens in Series 2 and the 3300mm long 50x25x2.5RHS specimens in Series 3. In Figure 10, the buckling resistance of each specimen has been normalised by the plastic section capacity $N_{pl,R}$ to allow a direct comparison to be made between the experimental results and those predicted by international standards. The North American standard [14] employs a curve similar to Curve 'b' in Eurocode 3 [16]. These design curves predict the buckling resistance well, with curves 'a' and 'c' of Eurocode 3 providing reasonable upper and lower bounds.

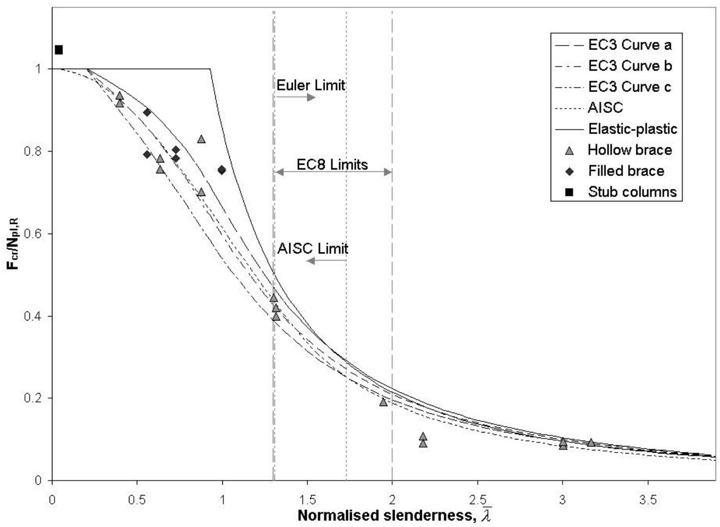
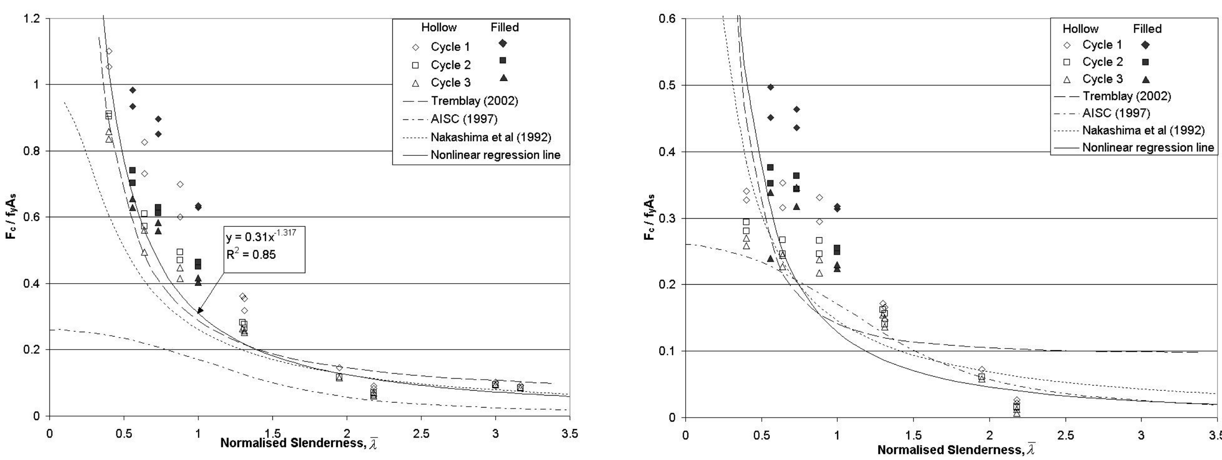


Figure 10. Normalised buckling resistances.

Post-buckling compressive strength

In all tests, once first buckling of the specimens had occurred, their compressive resistance decreased upon applying larger compression deformations or during the second and third cycle at a given displacement amplitude (Figures 6 and 7). Figures 11(a) and (b) show the relationship between the compressive strength (normalised using the yield strength determined from the coupon tests) at ductility levels (i.e. δ/δ_y) of 2 and 5, respectively, and the normalised slenderness ($\bar{\lambda}$). The infill does not appear to greatly improve the post buckling capacity of the specimens at a ductility of 2 (Figure 11(a)), probably because local buckling is not significant at this stage. However, at higher ductility levels local buckling becomes more pronounced in the hollow specimens but is limited somewhat in the filled specimens, thus improving their post-buckling resistance, especially for the stockier specimens (Figure 11(b)).



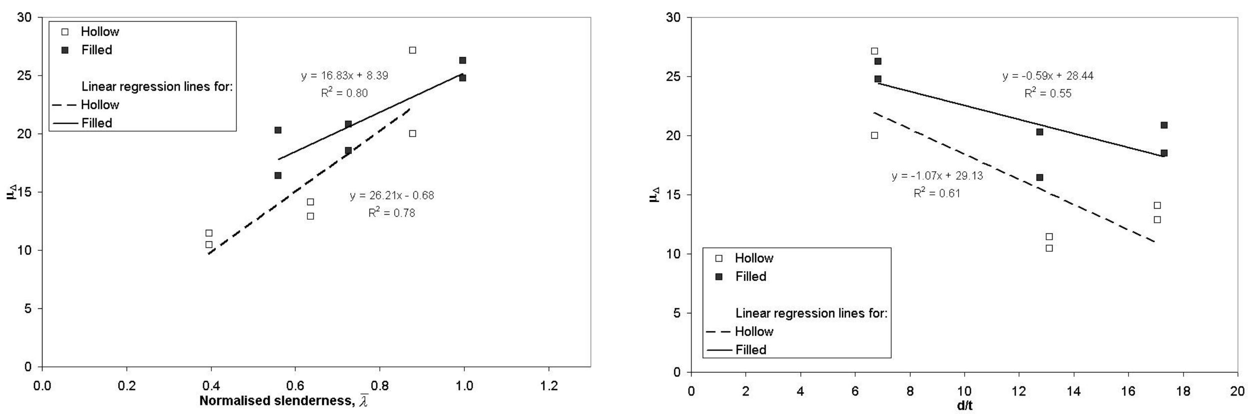
(a) **(b)**

Figure 11. Normalised compressive strength verses slenderness at ductility levels of (a) 2 and (b) 5.

An accurate estimate of the post-buckling resistance of brace members is important as it can determine the maximum base shear and column loads in some brace configurations [22] and also influences the overall energy dissipation capabilities of the brace member. The experimental results are compared with predictions by Tremblay [4] and Nakashima et al [23] in Figure 11. These predictions are based on nonlinear regression of results data from cyclic tests on bare steel specimens. The predictions are in general agreement with the experimental results for the steel hollow specimens from this study. However, they slightly underestimate the post-buckling resistance of the filled specimens, in particular during the first cycle at higher ductility levels. On the other hand, the American seismic provision [15] assumes that the post-buckling brace capacity is $0.26P_n$, where P_n is the predicted maximum axial compression strength, regardless of the ductility level. This provides a good lower bound for the compressive strength of both filled and hollow specimens at a ductility of 5 (Figure 11(b)), but significantly underestimates their compressive strength at a ductility of 2 (Figure 11(a)).

Ductility and energy dissipation

The ductility capacities of the brace specimens are plotted against member and section slenderness in Figures 12(a) and (b), respectively. The ductility ratios presented in these figures allow for the influence of the actual (measured) yield strength of each specimen. A detailed assessment of the influence of the variation in measured yield strength on the ultimate strain of the steel material was carried out by testing a large number of coupons and steel sections under monotonic tensile loading [13]. As expected, this indicated a clear relationship between the ultimate strain and yield strength of steel. In this paper, ductility is defined as $\mu_\Delta = \frac{\partial_u}{\delta_y} + \alpha \frac{f_y - f_{y,nom}}{f_{y,nom}}$, where δ_u is the maximum axial deformation, δ_y is the axial deformation at yield, α is a constant to take account of the relationship between ultimate strain and material strength, f_y is the actual (measured) material yield strength, and $f_{y,nom}$ is the nominal material yield strength. Linear regression of monotonic tensile section test results determined α to be 14.04 [13].



(a) (b)

Figure 12. (a) Ductility versus member slenderness, and (b) ductility versus section slenderness.

The ductility capacity of both hollow and filled specimens increases with member slenderness (Figure 12(a)). This trend has been observed in previous studies on bare steel members ([2], [4]). The variation of ductility capacity with section width-to-thickness (b/t) ratio does not show a clear trend (Figure 12(b)). On the other hand, the ductility capacities of the 50x25x2.5RHS specimens are higher than that of the 40x40x2.5SHS specimens because the stiffer webs of the rectangular section improve the local buckling resistance of the flanges. The influence of the infill on ductility can also be seen from Figures 12(a) and (b), with the filled specimens generally displaying higher capacities. No improvement is observed for the most slender specimens because local buckling did not occur due to their low section slenderness (Figure 12(b)). In addition, these specimens have higher member slenderness, resulting in their buckling behaviour being dominated by elastic rather than plastic response, as is the case for stockier specimens.

Although the ductility under monotonic tension may be lower for the filled members, under cyclic loading local buckling plays a role in initiating and determining failure. Since the infill delays local buckling, it increases ductility capacity. Clearly, this would only be useful for members with relatively low overall slenderness, or relatively high d/t, or both. On the other hand, if d/t is low (e.g. <<10) particularly when the overall slenderness is high, the infill would not make a positive contribution to seismic behaviour, and may in fact result in inferior performance since failure become more dominated by steel fracture in tension (rather than due to the cracks initiated at the local buckling regions).

Assessment of the amount of energy dissipated by brace specimens indicated that there is no significant difference between the hollow and filled members. On the other hand, the amount of energy dissipated by both filled and hollow specimens decreased with increase in slenderness, as shown in Figure 13. In Figure 13, the energy index is defined as the area under the load-axial deflection curve in both tension and compression regions during the first cycle at a ductility level of 4 normalised to the elastic energy of the strut.

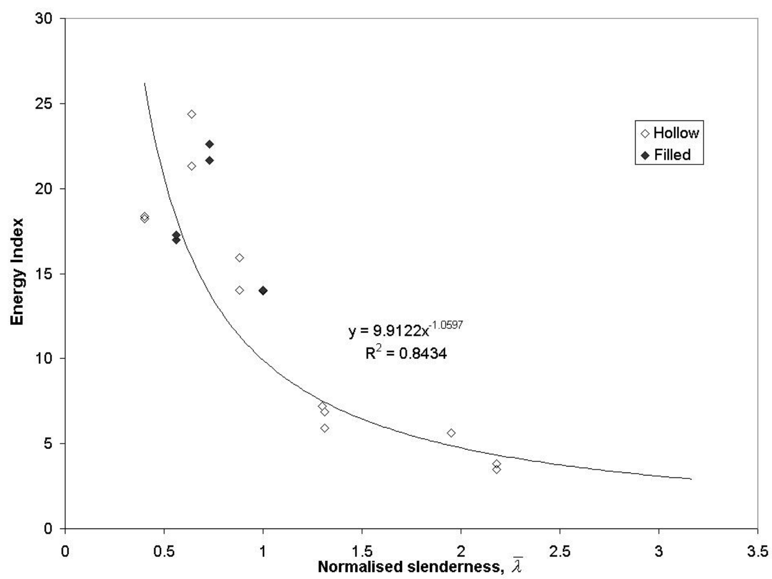


Figure 13. Energy index verse slenderness for the 1st cycle at a ductility of 4.

CONCLUSIONS

An experimental study has been described in which the response of hollow and filled cold-formed hollow steel bracing members to cyclic loading was investigated. The observed experimental bucking capacities of both hollow and filled specimens agreed well with the values employed in European and American design codes. This implies that cyclic loading did not affect the contribution made by the infill to the compression resistances of the composite struts. Moreover, at relatively high ductility levels, the post-buckling compression resistances of the filled specimens was noticeably higher than those of the equivalent hollow specimens, confirming that the infill continued to enhance brace strength at larger displacements.

Predictions based on buckling curves 'a' and 'c' in Eurocode 3 for filled and hollow specimens, respectively, in conjunction with the actual section strength gave the closest approximation to the observed buckling capacities of the brace members. In contrast, predictions based on measured material strengths tended to underpredict experimentally observed values, which if replicated in practice could lead to an underestimate of design base shear. For composite members, the section resistance approach slightly overestimated the buckling resistance of most specimens, and hence represents the upper bound required in capacity design. For the hollow specimens, a similar upper bound could be achieved by employing curve 'b' with section, rather than material, strength.

At low ductility demand levels, the post-buckling compression resistance of a brace can influence the maximum seismic forces acting on the braced frame, while at high ductility levels, it affects the axial compression forces in column members. A design rule in the American seismic provision [15] appears to provide a safe, if conservative, value when considering the latter issue, but it grossly underestimates the resistance at low ductility levels. At low slenderness, the normalised post-buckling resistances of filled specimens were greater than those of hollow specimens, implying that relationships proposed in previous studies on steel struts ([4], [13] and [23]) may underestimate the resistance of composite braces.

To prevent inadvertent yielding in non-dissipative parts of the structure, an accurate estimate of the ultimate tensile capacity of brace members is essential. The ultimate tensile resistance of the cyclic test specimens generally exceeded the yield stengths of the cross-section by about 20%. Moreover, in both monotonic and cyclic tests, this margin was always slightly greater for the composite specimens. Hence, additional caution, expressed in terms of the overstrength factor employed, should be exercised if the design tension capacity of a filled brace is based on the resistance of the steel section only. This issue has received relatively little previous attention, and is worthy of further study.

Under monotonic tension loading, the presence of the infill can lead to reduced ductility capacity. Numerical analysis has shown that by preventing ductile necking, the infill causes non-negligible hoop stresses to arise in the steel section, and that these increase rapidly after yield. These hoop stresses will in turn affect the longitudinal fracture strain at failure.

Under cyclic loading, the ductility capacities of both hollow and filled specimens were observed to be proportional to their overall slenderness, and inversely proportional to their section slenderness and yield strength. In comparing the performance of hollow and filled specimens, it is evident that the mortar infill prevented inward local buckling in most cases, and delayed it in others. This in turn meant that when allowance was made for the different yield strengths of the individual specimens, the infill was observed to improve ductility capacity, especially at low slenderness values. Generally, the presence of infill reduced the sensitivity of the ductility capacity to member and section slenderness.

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