Structural design of high strength austenitic stainless steel

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Abstract

Efficient use of material is an important factor in achieving economical and sustainable structures. Typically, annealed austenitic stainless steel has a material strength of around 220 N/mm², somewhat lower than that of common structural carbon steel grades. This lower strength, coupled with the higher material cost, puts stainless steel at a significant disadvantage when considering material selection, despite its other desirable properties. However, the strength of stainless steel may, at relatively low expense, be considerably enhanced through modification of the chemical composition and through the process of cold-working due to the strain hardening nature of the material. This strength enhancement has not generally been utilised in practice due to a lack of knowledge of the structural behaviour of this high strength material. Given the high material cost of stainless steel, the need to optimise the efficiency of design methods and to develop the performance, availability and diversity of the current product range is clear. To this end, this paper describes tests, numerical modelling and the development of design guidance for high strength stainless steel members in a range of structural configurations. Material tensile tests, member tests in compression and member tests in bending have been described. The results of the tests have been successfully replicated numerically, and subsequent sensitivity studies and parametric studies have been performed. Test and numerical results have been compared against two design methods developed for standard strength material (Eurocode 3 Part 1.4 and the deformation capacity based design method). The comparisons have revealed that both design methods provide a similar level of reliability to that offered for standard strength material, and thus, extension of both design methods to the high strength grades has been recommended.

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Keywords

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1. Introduction

The use of stainless steel in construction is growing due to the material's excellent corrosion resistance, ease of maintenance, attractive appearance, ductility, toughness and fire resistance [1]. Stainless steel is a sustainable material because it is highly durable, may be readily recycled and does not degrade when reprocessed. Protective coatings, often harmful to the environment, are not needed with stainless steel due to its inherent corrosion resistance, even when located in harsh surroundings.

The austenitic grades of stainless steel most widely used in construction are grade 1.4301 (304) and grade 1.4401 (316). Grade 1.4301 is suitable for rural, urban and light industrial sites whilst grade 1.4401 is a more highly alloyed grade and will perform well in marine and industrial sites. Both grades have a minimum specified design strength of between 210 and 240 N/mm² depending on material thickness (controlled by process route), which is lower than that for the common structural carbon steel grades, although their ductility is approximately twice that of these carbon steel grades. The lower strength, coupled with the higher material cost, puts stainless steel at a considerable disadvantage when considering material selection despite its other desirable properties.

However, the strength of stainless steel may be considerably enhanced, at relatively low expense, through modification of the chemical composition and through the process of cold-working. When stainless steel is cold-worked, it undergoes substantial strain hardening leading to a significant strength enhancement, whilst adequate ductility is still retained. PrEN 1993-1-4 (2005) [2] specifies a number of cold-worked levels, which are defined either in terms of minimum yield strength or minimum ultimate tensile strength and are taken from the European material standard for stainless steel, EN 10088-2 (2005) [3]; for example, CP350 and CP500 have minimum yield strengths of 350 and 500 N/mm² respectively, and C700 and C850 have minimum ultimate tensile strengths of 700 and 850 N/mm², respectively.

For members where design is governed by strength rather than stiffness, this increase in yield strength enables designers to specify a lighter section with a reduced wall thickness or reduced overall dimensions. Rectangular hollow sections made from cold-worked material cost only slightly more than sections from material in the annealed state. More highly alloyed stainless steels (known as duplex grades) with higher strength are available, though these grades are more expensive and are not as widely available as structural sections. Material in the cold-worked condition is available in the grades of stainless steel typically used in structural applications in a range of product forms including plate, sheet, coil, strip, bar and rectangular hollow sections. An increased use of the high strength material is anticipated in the areas of transportation and building. The material examined in the present study was grade 1.4318 (X2CrNi18-7), which has similar corrosion-resistant properties to grade 1.4301. In its annealed state, grade 1.4318 stainless steel has a yield strength of 330-350 N/mm², which can be further enhanced through the process of cold-working. In this study, grade 1.4318 material in two conditions was examined: annealed (similar to strength level C700) and cold-worked (strength level C850).

High strength stainless steel has not generally been utilised in practice due to the lack of knowledge of the structural behaviour of material in this condition. Previous studies on the cross-section properties of high strength cold-worked stainless steel have been performed [4], and numerical investigations of member behaviour have also been carried out [5]. However, a recent European project studied the behaviour of high strength stainless steel structural members and connections with the aim of developing comprehensive structural design guidance to facilitate economic structural design of members from high strength material. The findings of this project relating to structural hollow sections are described herein.

2. Testing

2.1 Introduction

Laboratory tests were performed in a range of structural configurations to investigate the local and overall member response of cold-worked stainless steel rectangular hollow sections. Material tests, compression tests (stub columns and flexural buckling tests) and bending tests were carried out. All the tests were performed on four rectangular hollow

sections (RHS 80×80×3, 100×100×3, 120×80×3 and 140×60×3) of austenitic stainless steel of grade 1.4318 (301 LN). The nominal and measured dimensions of the cross-sections (height h , breadth b , thickness t and internal corner radius r) are given in Table 1. The sections were fabricated from (solution) annealed material (similar to strength level C700) and from cold-worked material (strength level C850). Full details of the test programme have been reported [6].

The material properties of the test sections are give in Table 2. Two tensile coupons were cut from each section – one from the flange (face 1) and one from the web (face 2). Results of the tensile coupon tests (Young's modulus E , 0.2% proof strength $\sigma_{0.2}$ and ultimate tensile strength σ_u) are presented in Table 2. For the 80×80×3 section, tensile corner coupons were also measured, the corner regions having a mean 0.2% proof strength and ultimate strength of 614 and 941 N/mm² respectively for the annealed material, whilst the same properties for the C850 material were 807 and 1162 N/mm² respectively.

2.2 Stub column tests

Stub column tests were carried out on each of the four section sizes, RHS 80×80×3, 100×100×3, 120×80×3 and 140×60×3. For each section size, two specimens pertaining to the annealed and C850 strength levels were tested. Each specimen was 400 mm long and the ends of the column were milled flat and square to allow accurate seating of the ends to the rigid plates of the testing machine. The failure loads F_u and the 0.2% proof stresses $\sigma_{0.2}$ for the stub column tests are given in Table 3. It should be noted that for the RHS 120×80×3 and RHS 140×60×3 stub columns, local plate buckling occurred at or below the 0.2% proof strength; the actual 0.2% proof strengths would clearly be greater than the recorded values in the absence of plate buckling.

2.3 Flexural buckling tests

Flexural buckling tests were carried out on RHS 80×80×3 and 100×100×3 of different member slenderness ratios. The nominal lengths of the RHS 80×80×3 members were 1.15 m, 1.85 m and 2.85 m. For each slenderness ratio, two specimens, corresponding to the annealed and C850 strength levels, were tested. For the RHS 100×100×3, the nominal

lengths of the members were 1.45 m, 2.25 m and 3.55 m. Again, for each slenderness ratio, tests were performed on two specimens corresponding to the annealed and C850 strength levels. Endplates of dimensions 150×200×30 mm were welded to both ends of the columns. The endplates were grooved and supported on wedges to provide pinned end conditions. The groove was generally positioned in the centre of the cross-section, except where the measured initial global imperfection of the column was less than 1 mm, in which case an eccentricity of 1 mm was set. All measured global imperfections of the columns were less than 1.5 mm.

The buckling loads F_u for the RHS 80×80×3 and 100×100×3 sections are shown in Table 4, where the letter A after the section designation refers to the specimens formed from annealed material, whilst C850 relates to the specimens formed from cold-worked material. Table 4 also contains the numerically predicted buckling loads $F_{u,FE}$, as described in Section 3 of this paper. The buckled specimens for RHS 80×80×3 are shown in Fig. 1, where C1 and C2 within the identification number refers to the strength level of the specimen (C1 relates to annealed material and C2 relates to C850 material).

2.4 Bending tests

A total of six simply-supported bending tests were performed. The section sizes were RHS 100×100×3, 120×80×3 and 140×60×3 with two specimens for each section size pertaining to the two considered strength levels – annealed and C850. The overall length of the specimens was nominally 2 m, with equal overhangs at each end of 100 mm beyond the centrelines of the supports. Point loads were applied at one-third locations between the supports. Web crippling was prevented by inserting wooden blocks inside the RHS at the loading points and the supports. The failure moments M_u for the bending tests are shown in Table 5, which also contains the numerically predicted failure moments $M_{u,FE}$, as described in Section 3 of this paper. The deformed RHS 100×100×3 – annealed test specimen after failure is illustrated in Fig. 2.

2.5 Internal support (web crippling) tests

Tests on a total of six short span simply-supported beams were carried out to determine the resistance of the RHS to localised point loading. Section sizes, as for the bending

tests described above, were RHS 100×100×3, 120×80×3 and 140×60×3 with two specimens for each section size pertaining to the two considered strength levels – annealed and C850. The overall length of the specimens was nominally 0.8 m, with equal overhangs at each end of 100 mm beyond the centrelines of the supports. A single point load was applied at the centre of the beam through a 50 mm steel pad. Wooden blocks were inserted inside the RHS at the support points only to prevent web crippling. The test failure loads F_u and corresponding FE failure loads $F_{u,FE}$ for the internal support tests are shown in Table 6. Deformed internal support test specimens are shown in Fig. 3.

3. Numerical modelling

3.1 Introduction

Numerical modelling of the above tests on cold-worked high strength stainless steel hollow sections was performed. Full details of the numerical modelling programme have been reported [7]. Initial analyses were conducted to simulate the tests on the 12 pin-ended columns, 6 simply supported beams proportioned to failure by flexure and 6 simply supported beams proportioned to fail at the internal support. Further studies were conducted to investigate the sensitivity of the models to variations in key parameters, and following successful replication of the experiments and validation of the models, parametric studies were performed to provide additional results. The general-purpose finite element (FE) software package ABAQUS [8] was employed throughout the study.

3.2 Modelling parameters

The elements chosen for the stub column models were 9-noded, reduced integration shell elements with five degrees of freedom per node, designated as S9R5 in the ABAQUS element library. This element has been shown to perform well in similar applications involving the modelling of stainless steel SHS and RHS flexural members [9] and the local and global buckling of stainless steel SHS, RHS and CHS columns [10]. S9R5 is characterised as a ‘thin’ shell element and is not recommended for modelling cases where transverse shear flexibility is important. Transverse shear flexibility is said to become important when the shell thickness is more than about 1/15 of a characteristic length on its surface [8].

The curved geometry at the corners of the cross-sections was modelled using curved S9R5 shell elements. Convergence studies were conducted to determine an appropriate mesh density, with the aim of achieving suitably accurate results whilst minimising computational time. For the modelling of flexural buckling, linear elastic eigenmode simulations were conducted to provide buckling modes to be used as initial imperfections in subsequent non-linear analyses. The modified Riks method [8] was employed to solve the geometrically and materially non-linear stub column models. The modified Riks method is an algorithm that enables effective solutions to be found to unstable problems (e.g. post-ultimate response of compression or flexural members), and adequately traces non-linear unloading paths.

ABAQUS requires that material behaviour be specified by means of a multi-linear stress-strain curve, defined in terms of true stress and log plastic strain. Points to define this multi-linear stress-strain curve were taken from a compound two-stage Ramberg-Osgood material model fitted to the measured stress-strain data from the tensile coupon tests. The concept of adopting a two-stage model was originally devised by Mirambell and Real [11]. A more complete description is provided by Gardner [12] and further analysis was carried out by Rasmussen [13].

Residual stresses are induced into cold-formed stainless steel hollow sections by deformations during the forming process and by non-uniform cooling following welding. The deformationally induced residual stresses (largely resulting in through-thickness bending) also exist in the material coupons cut from within the finished cross-sections. The effects of these are therefore inherently present. Previous FE simulations [10,5] have indicated that the effects of weld-induced residual stresses on cold-formed stainless steel tubular members are relatively small. A simple assumed residual stress pattern (comprising a tensile stress equal to the material 0.2% proof stress $\sigma_{0.2}$ in tension acting on a central portion of one-fifth of the plate width and an equilibrating compressive stress of $\sigma_{0.2}/4$ over the remainder of the plate) was adopted on the welded face in all flexural buckling models. No residual stresses were specified in the bending or internal support tests.

Initial geometric imperfections comprising a superposition of the lowest local and global elastic eigenmodes were included in the column models. Local imperfections only were included in the bending and internal support models.

The magnitude of the local imperfections ω_0 was taken from a previous study [10] where ω_0 is defined in terms of the material and geometric properties of the plate elements, as given by Eq. (1). The global imperfection magnitude was taken as the column length divided by 2000 (i.e. $L/2000$). A brief study is described in a later section to assess the sensitivity of the columns to variation in global imperfection amplitude.

$$\omega_0 / t = 0.023(\sigma_{0.2} / \sigma_{cr}) \quad (1)$$

where ω_0 is the magnitude of the initial imperfection, t is the plate thickness, $\sigma_{0.2}$ is the material 0.2% proof stress and σ_{cr} is the elastic critical buckling stress of the plate, assuming simply supported boundary conditions.

4. Comparison between test and FE results

This section compares the key results from the tests with those generated by FE modelling. The results are presented by test type in the following sub-sections: flexural buckling tests; bending tests; and internal support tests.

4.1 Flexural buckling

A total of 12 flexural buckling tests were conducted as part of the experimental study. Each of these tests was modelled using the described parameters above and a comparison between the test and FE results is presented in Table 4.

The results indicate that on average the FE models predict failure loads 1% higher than the test failure loads, and upon examination of the individual results it can be seen that the scatter is relatively small. It may therefore be concluded that FE modelling parameters found to be suitable for normal strength stainless steel sections are also applicable to the higher strength (cold-worked) sections. A study is conducted in the next section to assess

the sensitivity of column buckling FE models to variation in global imperfection amplitude, and parametric studies are carried out to extend the range of results over a wider range of non-dimensional column slenderness $\bar{\lambda}$.

In all cases the FE failure mode and the general form of the load-lateral deflection curves was similar to those observed in the tests. A comparison of test and FE load-lateral deflection response for the 100×100×3-A annealed column (length = 3546 mm) is shown in Fig. 4.

4.2 Bending

A total of 6 bending tests were conducted as part of the experimental study. Each of these tests was modelled using the described parameters above and a comparison between the test and FE results is presented in Table 5.

Table 5 shows that the FE prediction of the bending moment at failure is, on average, 9% lower than the test failure bending moment. It also shows that there is good consistency between the results. An explanation for the general under-prediction of strength by the FE model lies in the distribution of material properties around the cross-sections. This matter is analysed in the next section.

Fig. 5 shows a deformed FE model of the RHS 120×80×3 annealed simply-supported beam. In the test arrangement, wooden blocks were positioned inside the bending specimens at the loading points and at the supports to avoid local crippling of the webs. This was modelled by constraining the out-of-plane deformation of the webs (to a common value) beneath the loading points, though from Fig. 5 some web deformation adjacent to the points of load application is still evident at large deflections. Only half of the cross-section for the bending and internal support specimens was modelled and symmetry boundary conditions applied.

The general form of the test and FE bending moment versus vertical deflection at mid-span curves were similar in all cases. Some variation in the deflection at ultimate moment was observed, but this may be expected since the slope of the curve is low in this region.

A typical comparison between test and FE bending moment versus vertical deflection at mid-span is shown for the RHS 120×80×3-C850 beam specimen in Fig. 6.

4.3 Internal support (web crippling)

Table 6 presents a comparison between the test and FE results for the six internal support test specimens. The results demonstrate very good agreement in terms of magnitude of failure load with a mean value of FE failure load divided by test failure load of 0.96, and there is little scatter in the results.

Fig. 7 shows a deformed FE model of the RHS 100×80×3-A internal support specimen. In the test arrangement, wooden blocks were positioned inside the cross-sections at the supports to avoid local crippling of the webs, but not under the point of load application. This was modelled by constraining the out-of-plane deformation of the webs at the supports, whilst providing no constraint to web in the region of load application. The load was introduced into each test specimen through a 50 mm wide steel plate. This was modelled by constraining a 50 mm width of the loaded flange to displacement vertically in unison. The failure mode of the FE model was similar to that observed in the tests.

5. Sensitivity and parametric studies

Sensitivity and parametric studies were conducted to investigate the response of the FE models to variation in key input parameters. Variation in global imperfection amplitude for columns and distribution of material properties in beams were assessed.

5.1 Global imperfection amplitude

A comparison between the test and FE results for flexural buckling was presented in Table 4. Although there is good overall agreement between test and FE results, it can be seen that the strength of the most slender columns is generally slightly over-predicted and the strength of the least slender columns is generally slightly under-predicted. Sensitivity of the models to variation in the global imperfection amplitude was assessed. For the three RHS 80×80×3-A members, buckling loads for FE models with global imperfections of $L/2000$ and $L/1000$ are presented in Table 7.

The increase in FE global imperfection magnitude from L/2000 to L/1000 resulted in a mean reduction in buckling load for the three RHS 80×80×3-A columns of 3.9%. This represents relatively low imperfection sensitivity and as a general parameter a global imperfection magnitude of L/2000 is deemed more appropriate.

5.2 Distribution of material properties

The results of Table 5 show that although the predicted (FE) bending strengths are conservative and display low variability, they are also consistently lower than the corresponding test results. An explanation for this may lie in the distribution of material strength around the cross-section. It is clear that the bending specimens, where the extreme fibres in the flanges are far more highly stressed than the web regions, are likely to be more sensitive to variation in material distribution than the compressed specimens where the cross-sections are more uniformly loaded.

For the comparison between FE and test results given in the section above, average tensile material properties have been uniformly distributed around the cross-sections, with the exception of the corner regions where enhanced strengths have been specified. For this study, tensile material properties measured from the narrow faces have been applied to the two narrow faces of the cross-section and those measured from the wide faces have been applied to the two wide faces of the cross-section. It should be noted that the welds always appear on one of the two narrow faces and the test specimens were configured such that the weld was positioned on the underside of specimens; therefore for the square cross-sections, the material properties from the welded and opposite faces were applied to the extreme faces in tension and compression. The results are shown in Table 8.

Table 8 indicates that more accurate distribution of material properties around the cross-section of the FE models leads to closer agreement with the test results. It is often the case that only one material test is conducted for each member. However, where detailed material property data are available, based on the findings of this study, it is recommended that properties be applied to the specific face from which measurements were taken.

For flexural buckling, parametric studies were conducted to generate results for columns over a range of non-dimensional slenderness $\bar{\lambda}$. This was achieved by employing the 100×100×3-A and 100×100×3-C850 cross-section and material properties and varying only the length the column. (The cross-sectional dimensions of the shortest of the three tested columns were adopted for all parametric models). The results are given in Table 9, and compared in the following section with existing design guidance.

6. Comparison with existing design guidance

This section presents a comparison of the test results with existing design rules from Eurocode 3 Part 1.4 [2] and the deformation capacity based design method [14,15]. As with the FE modelling, comparison has been made according to test type in the following three sections: flexural buckling, bending and web crippling. In all comparisons, resistances from the two design methods have been generated using measured geometry and measured (weighted average) tensile material properties. All partial safety factors have been set to unity to enable a direct comparison.

6.1 Flexural buckling

Comparison between the flexural buckling test results and the results predicted by the two considered design methods is given in Table 10. The Eurocode formulations for the determination of flexural buckling resistance are given below, whilst those for the deformation capacity based design method are described in [14,15].

In Eurocode 3 Part 1.4, flexural buckling resistance is determined from Eq. (2).

$$N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}} \quad \text{for Class 1, 2 and 3 cross-sections} \quad (2a)$$

$$N_{b,Rd} = \frac{\chi A_{eff} f_y}{\gamma_{M1}} \quad \text{for Class 4 cross-sections} \quad (2b)$$

where A is the gross cross-sectional area, A_{eff} is the effective area of Class 4 cross-sections, χ is the reduction factor accounting for buckling determined from Eq. (3), and γ_{M1} is a partial safety factor and set equal to unity for this comparison.

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \quad \text{but } \chi \leq 1.0 \quad (3)$$

where the intermediate factor Φ and the non-dimensional member slenderness $\bar{\lambda}$ are defined by Eqs. (4) and (5), respectively.

$$\Phi = 0.5[1 + \alpha(\bar{\lambda} - \bar{\lambda}_0) + \bar{\lambda}^2] \quad (4)$$

$$\bar{\lambda} = \sqrt{\frac{A f_y}{N_{\text{cr}}}} \quad \text{for Class 1, 2 and 3 cross-sections} \quad (5a)$$

$$\bar{\lambda} = \sqrt{\frac{A_{\text{eff}} f_y}{N_{\text{cr}}}} \quad \text{for Class 4 cross-sections} \quad (5b)$$

in which α is an imperfection factor (taken as 0.49 for the sections investigated herein), $\bar{\lambda}_0$ is the limiting slenderness taken as 0.4, and N_{cr} is the elastic critical buckling force for the relevant buckling mode based on the gross properties of the cross-section.

A graphical comparison of the test and finite elements results with the Eurocode design curve is given in Fig. 8. The normalised column buckling load has been derived by dividing the test or FE failure load F_u by the cross-sectional resistance $N_{c,Rd}$, calculated according to prEN1993-1-4 (and utilising the effective area as appropriate). The three test points that lie below the Eurocode buckling curve are from the RHS 80×80×3-A cross-section. These three specimens were tested three months after the others, but there is thought to be no differences in the test procedure or measurement equipment.

6.2 Bending

Bending test results and the results predicted by the two considered design methods are compared in Table 11. Again, the Eurocode design expressions are given below whilst those for the deformation capacity based design method have been reported in [14,15]. The hollow sections studied herein were not susceptible to lateral torsional buckling ($\bar{\lambda}_{LT}$ was less than 0.4 in all cases), and thus only in-plane cross-section resistance was considered (Eq. (6)).

$$M_{c,Rd} = \frac{W_{pl} f_y}{\gamma_{M0}} \quad \text{for Class 1 or 2 cross-sections} \quad (6a)$$

$$M_{c,Rd} = \frac{W_{el} f_y}{\gamma_{M0}} \quad \text{for Class 3 cross-sections} \quad (6b)$$

$$M_{c,Rd} = \frac{W_{eff} f_y}{\gamma_{M0}} \quad \text{for Class 4 cross-sections} \quad (6c)$$

where W_{pl} , W_{el} and W_{eff} are the plastic, elastic and effective section moduli respectively, and γ_{M0} is a partial safety factor and set equal to unity for this comparison.

6.3 Web crippling at internal support

Comparison between the web crippling test results and the results predicted by the prEN 1993-1-4 are shown in Table 12. It should be noted that for web crippling, prEN 1993-1-4 refers the user to Eurocode 3 Part 1.3 [16] and this is therefore used as the basis for the comparison. The deformation capacity based design method has not been developed to cover web crippling.

Eurocode 3 Part 1.3 [16] does not contain explicit rules for the determination of web crippling resistance for RHS. These sections are therefore dealt with assuming coefficients for sheeting – this is the same assumption made by Talja and Salmi [17]. Thus, for two webs, crippling resistance is given by Eq. (7).

$$R_{\alpha,Rd} = 1.02t^2 \sqrt{Ef_y} (1 - 0.1\sqrt{r/t})(0.5 + \sqrt{0.02I_a/t}) / \gamma_{M1} \quad (7)$$

where t is the web thickness, r is the internal corner radii, E is the material Young's modulus, f_y is the material 0.2% proof strength, l_a is the length over which the concentrated load is applied, and γ_{M1} is a partial safety factor set equal to unity for this comparison

The comparison shows that the Eurocode predicts, on average, 83% of the failure load for web crippling, with a relatively small scatter. These results are approximately in line with those calculated for standard strength material [17].

7. Recommendations for design guidance

Comparison of the test results with the Eurocode design method and the deformation capacity based design method [14,15] has revealed similar predicted/ test ratios as were observed from an extensive comparison for standard strength stainless steel specimens [14,12].

Based on the recent testing programme on cold-worked structural stainless steel members, the Eurocode design rules that are currently limited to standard strengths of stainless steel may therefore safely be extended in scope to additionally cover high strength sections. The deformation capacity based design method demonstrates a similar level of improvement over the Eurocode approach through more accurate material modelling and section classification as for standard strength specimens.

8. Conclusions

The strength of stainless steel can be considerably enhanced through modification of the chemical composition and through the process of cold-work. High strength material has not previously been included in European structural stainless steel design standards and hence the strength enhancement has not generally been utilised in practice. This paper has presented the results of a test programme on high strength stainless steel structural hollow sections. Material tensile tests, member tests in compression and member tests in bending have been described. Accurate replication of test behaviour for flexural buckling, bending and web crippling has been achieved using the FE package ABAQUS. Improved agreement between test and FE results was achieved by using face specific material

properties rather than weighted average material properties on all faces. Test and numerical results have been compared against two design methods developed for standard strength material (Eurocode 3 Part 1.4 and the deformation capacity based design method). The comparisons have revealed that both design methods provide a similar level of reliability to that offered for standard strength material, and thus, extension of both design methods to the high strength grades has been recommended.

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Table 1: Nominal and measured dimensions of test specimens

Section	Nominal dimensions			Measured dimensions							
				Annealed				C850			
	h (mm)	b (mm)	t (mm)	h (mm)	b (mm)	t (mm)	r (mm)	h (mm)	b (mm)	t (mm)	r (mm)
RHS 80×80×3	80	80	3	80.1	80.3	3.08	4.0	80.4	80.0	3.06	3.0
RHS 100×100×3	100	100	3	100.0	100.1	3.07	2.5	100.3	100.0	3.05	3.0
RHS 120×80×3	120	80	3	120.0	79.8	3.07	4.0	120.3	80.4	3.05	4.0
RHS 140×60×3	140	60	3	139.9	60.4	3.07	4.5	139.8	60.2	3.06	4.0

Table 2: Measured tensile material properties of test sections

Section	Annealed			C850		
	$\sigma_{0.2}$ (N/mm ²)	σ_u (N/mm ²)	E (N/mm ²)	$\sigma_{0.2}$ (N/mm ²)	σ_u (N/mm ²)	E (N/mm ²)
RHS 80x80x3 – Face 1	571	864	190000	713	1048	173000
RHS 80x80x3 – Face 2	469	805	185000	590	1001	173000
RHS 100x100x3 – Face 1	530	830	195000	666	971	183000
RHS 100x100x3 – Face 2	432	782	195000	549	914	184000
RHS 120x80x3 – Face 1	619	886	197000	763	1025	188000
RHS 120x80x3 – Face 2	459	796	202000	553	916	192000
RHS 140x60x3 – Face 1	619	886	190000	741	1038	187000
RHS 140x60x3 – Face 2	492	808	203000	561	956	185000

Table 3: Failure loads and 0.2% proof stresses for stub column tests

Section	Annealed		C850	
	Failure load F_u (kN)	$\sigma_{0.2}$ (N/mm ²)	Failure load F_u (kN)	$\sigma_{0.2}$ (N/mm ²)
RHS 80×80×3	598	505	755	617
RHS 100×100×3	609	483	656	541
RHS 120×80×3	595	501 ^b	645	555
RHS 140×60×3	560	478 ^a	642	523

Notes: ^a plate buckling occurred below $\sigma_{0.2}$

^b plate buckling occurred at $\sigma_{0.2}$

Table 4: Test and FE results for flexural buckling specimens

Cross-section	Length (mm)	Test failure load F_u (kN)	FE failure load $F_{u,FE}$ (kN)	FE/ Test
RHS 80x80x3-A	1148	407	423	1.04
RHS 80x80x3-A	1850	267	293	1.10
RHS 80x80x3-A	2849	150	172	1.15
RHS 80x80x3-C850	1147	518	512	0.99
RHS 80x80x3-C850	1847	332	332	1.00
RHS 80x80x3-C850	2848	162	175	1.08
RHS 100x100x3-A	1447	560	492	0.88
RHS 100x100x3-A	2250	406	377	0.93
RHS 100x100x3-A	3546	220	229	1.04
RHS 100x100x3-C850	1447	634	571	0.90
RHS 100x100x3-C850	2250	427	418	0.98
RHS 100x100x3-C850	3552	222	227	1.02
Mean:				1.01

Table 5: Test and FE results for bending specimens

Cross-section	Test failure moment M_u (kNm)	FE failure moment $M_{u,FE}$ (kNm)	FE/ Test
RHS 100×100×3-A	23.3	21.0	0.90
RHS 120×80×3-A	29.8	27.7	0.93
RHS 140×60×3-A	34.6	30.0	0.87
RHS 100×100×3-C850	26.7	25.0	0.93
RHS 120×80×3-C850	33.7	32.8	0.97
RHS 140×60×3-C850	39.0	34.3	0.88
Mean:			0.91

Table 6: Test and FE results for internal support specimens

Cross-section	Test failure load F_u (kN)	FE failure load $F_{u,FE}$ (kN)	FE/ Test
RHS 100×100×3-A	107.1	110.4	1.03
RHS 120×80×3-A	108.3	102.6	0.95
RHS 140×60×3-A	107.5	98.4	0.92
RHS 100×100×3-C850	119.2	119.2	1.00
RHS 120×80×3-C850	118.2	117.0	0.99
RHS 140×60×3-C850	126.7	112.2	0.89
Mean:			0.96

Table 7: Sensitivity of column FE models to variation in global imperfection amplitude

Cross-section	Length (mm)	FE Failure load for L/2000 (kN)	FE failure load for L/1000 (kN)	Reduction in failure load (%)
RHS 80x80x3-A	1148	423	411	2.8
RHS 80x80x3-A	1850	293	282	3.8
RHS 80x80x3-A	2849	172	163	5.2
Mean:				3.9

Table 8: Bending FE models with uniform and distributed material properties

Cross-section	Test failure moment (kNm)	FE (uniform properties)/ Test failure moment (kNm)	FE (distributed properties)/ Test failure moment (kNm)
RHS 100×100×3-A	23.3	0.90	0.99
RHS 120×80×3-A	29.8	0.93	0.97
RHS 140×60×3-A	34.6	0.87	0.89
RHS 100×100×3-C850	26.7	0.93	1.01
RHS 120×80×3-C850	33.7	0.97	0.99
RHS 140×60×3-C850	39.0	0.88	0.90
Mean:		0.91	0.96

Table 9: Parametric studies for flexural buckling

Cross-section	Length (mm)	$\bar{\lambda}$	FE Failure load (kN)
RHS 100x100x3-A ¹	1447	0.56	492
RHS 100x100x3-A	1800	0.70	448
RHS 100x100x3-A ¹	2250	0.87	377
RHS 100x100x3-A	2600	1.01	335
RHS 100x100x3-A	3000	1.17	287
RHS 100x100x3-A ¹	3546	1.38	229
RHS 100x100x3-A	4000	1.55	190
RHS 100x100x3-C850 ¹	1447	0.62	571
RHS 100x100x3-C850	1800	0.77	510
RHS 100x100x3-C850 ¹	2250	0.96	418
RHS 100x100x3-C850	2600	1.11	356
RHS 100x100x3-C850	3000	1.28	294
RHS 100x100x3-C850 ¹	3552	1.51	227
RHS 100x100x3-C850	4000	1.70	184

Note: ¹ Models of tests

Table 10: Comparison between tests and design guidance for flexural buckling

Cross-section	$\sigma_{0.2}$ (N/mm ²)	E (N/mm ²)	Length (mm)	Test failure load (kN)	Eurocode / Test failure load	Deformation based / Test failure load
80x80x3-A	520	187500	1148	407	1.01	1.05
80x80x3-A	520	187500	1850	267	1.05	1.03
80x80x3-A	520	187500	2849	150	1.01	0.99
80x80x3-C850	653	173000	1147	518	0.83	0.92
80x80x3-C850	653	173000	1847	332	0.82	0.85
80x80x3-C850	653	173000	2848	162	0.87	0.89
100x100x3-A	487	195000	1447	560	0.78	0.90
100x100x3-A	487	195000	2250	406	0.81	0.87
100x100x3-A	487	195000	3546	220	0.82	0.86
100x100x3-C850	594	183500	1447	634	0.69	0.87
100x100x3-C850	594	183500	2250	427	0.73	0.83
100x100x3-C850	594	183500	3552	222	0.74	0.82
Mean:					0.85	0.91

Table 11: Comparison between tests and design guidance for bending

Cross-section	$\sigma_{0.2}$ (N/mm ²)	E (N/mm ²)	Test failure moment (kNm)	Eurocode / Test failure moment	Deformation based / Test failure moment
100x100x3-A	487	195000	23.3	0.71	0.86
120x80x3-A	521	200000	29.8	0.68	0.93
140x60x3-A	529	199100	34.6	0.62	1.00
100x100x3-C850	594	183500	26.7	0.72	0.84
120x80x3-C850	638	190400	33.7	0.73	0.97
140x60x3-C850	621	185600	39.0	0.64	1.00
Mean:				0.70	0.94

Table 12: Comparison between tests and design guidance for web crippling

Cross-section	$\sigma_{0.2}$ (N/mm ²)	E (N/mm ²)	Test failure load (kN)	Eurocode / Test failure load
100x100x3-A	487	195000	107.1	0.85
120x80x3-A	521	200000	108.3	0.86
140x60x3-A	529	199100	107.5	0.88
100x100x3-C850	594	183500	119.2	0.80
120x80x3-C850	638	190400	118.2	0.86
140x60x3-C850	621	185600	126.7	0.76
Mean:				0.83

Fig. 1: Deformed RHS 80x80x3 columns from flexural buckling tests

Fig. 2: Deformed RHS bending specimen

Fig. 3: Deformed RHS specimens from internal support tests

Fig. 4: Comparison of test and FE load-lateral deflection response for RHS 100×100×3-A (length = 3546 mm) column

Fig. 5: Deformed FE model of the RHS 120×80×3-A simply-supported beam

Fig. 6: Comparison of test and FE bending moment versus vertical deflection at mid-span curve for RHS 120×80×3-C850 bending specimen

Fig. 7: Deformed RHS 100x100x3-A internal support specimen

Fig. 8: Comparison between tests, FE results and prEN 1993-1-4 (2005) buckling curve