Testing and numerical modelling of lean duplex stainless steel hollow section columns

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Abstract

Stainless steels are employed in a wide range of structural applications. The austenitic grades, particularly EN 1.4301 and EN 1.4401 and their low-carbon variants EN 1.4307 and EN 1.4404 are the most commonly used within construction and these typically contain around 8-11% nickel. The nickel represents a large portion of the total material cost and thus high nickel prices and price volatility have a strong bearing on both the cost and price stability of stainless steel. While austenitic stainless steel remains the most favourable material choice in many applications, greater emphasis is now being placed on the development of alternative grades with lower nickel content. In this study, the material behaviour and compressive structural response of a lean duplex stainless steel (EN 1.4162), which contains approximately 1.5% nickel are examined. A total of eight stub column tests and twelve long column tests on lean duplex stainless steel square (SHS) and rectangular hollow sections (RHS) are reported. Precise measurements of material and geometric properties of the test specimens were also made, including the assessment of local and global geometric imperfections. The experimental studies were supplemented by finite element analysis and parametric studies were performed to generate results over a wider range of cross-sectional and member slenderness. Both the experimental and numerical results were used to assess the applicability of the Eurocode 3: Part 1-4 provisions regarding the Class 3 slenderness limit and effective width formula for internal elements in compression and the column buckling curve for hollow sections to lean duplex structural components. Comparisons between the structural performance of lean duplex stainless steel and that of other more commonly used stainless steel grades are also presented, showing lean duplex to be an attractive choice for structural applications.

Keywords: Buckling, Classification, Columns, Experiments, Finite element, Hollow section, Lean Duplex, Numerical modelling, Stainless steel, Stub columns.

1. Introduction

There is a wide variety of grades of stainless steels, providing a range of material characteristics to suit the demands of numerous, diverse engineering applications. Both overall, and within the construction industry, the austenitic grades feature most prominently [1]. The most commonly employed grades of austenitic stainless steel are EN 1.4301/1.4307 and EN 1.4401/1.4404, which contain around 8-11% nickel [2]. Nickel stabilises the austenitic microstructure and therefore contributes to the associated favourable characteristics such as formability, weldability, toughness and high temperature properties. However, nickel also represents a significant portion of the cost of austenitic stainless steel and this has led, particularly in recent years to the development and evaluation of alternative grades of stainless steel with low nickel content.

Appropriate material selection, taking due account of in-service performance, economics and environmental conditions, involves matching the material characteristics to the particular demands of the application. Within construction, although austenitic stainless steels are the most widely specified, their strengths are often not fully utilised; a recently developed ‘lean duplex’ stainless steel, containing approximately 1.5% nickel, may offer a more appropriate balance of properties for structural applications. The particular grade considered in this study is EN 1.4162, which is generally less expensive and possesses higher strength than the familiar austenitics, while still retaining good corrosion resistance and high temperature properties [3], together with adequate weldability [4] and fracture toughness [5]. Examples of the use of lean duplex stainless steel in construction have already emerged [6], including footbridges in Förde, Norway and Siena, Italy; the latter is shown in Fig. 1.

Despite early applications of lean duplex stainless steel, its structural properties remain largely unverified as no test data on structural components have been reported to date. A research project is underway at Imperial College London to address these shortcomings, focusing initially on cold-formed hollow sections. Material properties derived from tensile and compressive coupon tests, slenderness limits for cross-section classification and effective

width formulae for slender sections have been recently developed by the authors [7]. This paper examines the compressive behaviour of lean duplex stainless steel square and rectangular hollow sections (SHS and RHS respectively). Eight stub column tests and twelve flexural buckling tests have been carried out and are reported in detail herein. The test results were used to validate finite element (FE) models, which were thereafter employed in parametric studies, to expand the range of available structural performance data, studying the influence, in particular, of cross-section and member slenderness. Both the experimental and numerical results were used to assess the applicability of the European structural design provisions for stainless steel EN 1993-1-4 [8] to lean duplex stainless steel structural components. Comparisons with recent proposals made by the authors regarding the classification of stainless steel cross-sections [9], as well as comparisons with the structural performance of other commonly used structural stainless steel grades are also included.

2. Experimental investigation

2.1 Introduction

An experimental investigation into the structural performance of lean duplex stainless steel (grade EN 1.4162) SHS and RHS was conducted in the Structures Laboratory at Imperial College London. The laboratory testing program comprised tensile and compressive tests on flat coupons and tensile tests on corner coupons extracted from the cold-formed sections, eight three-point bending tests, eight stub column tests and twelve flexural buckling tests. The chemical composition of the tested material as given in the mill certificates is displayed in Table 1. A detailed description of the experimental set-up and the experimental results of the material tests and three-point bending tests is given in [7], whereas the stub column tests and flexural buckling tests are reported in detail in this section.

2.2 Material properties

The material properties derived from the coupon tests, which are used in the assessment of the member test results and the development of the FE models are given in Tables 2-4 for tensile flat, compressive flat and tensile corner material, respectively. The reported material

parameters are the Young's modulus E , the 0.2% and 1% proof stresses $\sigma_{0.2}$ and $\sigma_{1.0}$, respectively, the ultimate tensile stress σ_u , the plastic strain at fracture ε_f (based on elongation over the standard gauge length $5.65\sqrt{A}$, where A is the cross-sectional area of the coupon), and the strain hardening exponents n and $n'_{0.2,1.0}$ used in the compound Ramberg-Osgood material model [10-13], which is a two-stage version of the basic Ramberg-Osgood model [14, 15]. The key minimum specified material properties for grade EN 1.4162 stainless steel cold-rolled strip, as defined in [16], and included in EN 10088-4 [2], are as follows: $\sigma_{0.2}=530$ N/mm², $\sigma_u=700-900$ N/mm², $\varepsilon_f=30\%$ (over a gauge length $5.65\sqrt{A}$).

2.3 Stub column tests

Four section sizes were employed for the stub column tests, namely SHS 100×100×4, SHS 80×80×4, SHS 60×60×3 and RHS 80×40×4. Two repeated concentric compression tests were carried out for each of the cross-section sizes to enable the determination of a suitable Class 3 limit for lean duplex stainless steel internal elements in compression. All specimens were cold-rolled and seam welded. A stub column length equal to four times its mean nominal cross-sectional width was chosen, which is deemed long enough to include a representative pattern of residual stresses and geometric imperfections, yet short enough to avoid overall flexural buckling [17].

Measurements of the basic geometry and initial geometric imperfections of the specimens were conducted prior to testing. The geometric imperfections measurements followed the procedure reported by Schafer and Peköz in [18]. Local geometric imperfections were measured only over the middle half of each specimen's length in order to eliminate the effect of end flaring, which results from the release of residual stresses following cutting operations [19]. The same approach has been successfully adopted in previous studies [20, 21]. The maximum measured local geometric imperfection w_0 for each nominal stub column dimension is given in Table 5. Table 5 also includes the measured geometry (see Fig. 2) of each stub column specimen, where L is the stub column length, B is the section width, H is the section depth, t is the thickness and r_i is the internal corner radius.

The ends of the stub columns were milled flat and square and were compressed between parallel platens in a self contained 300 T Amsler hydraulic testing machine as depicted in

Fig. 3. The instrumentation consisted of four LVDTs to measure the end shortening between the flat plattens, a load cell to accurately record the applied load and four strain gauges, affixed at the mid-height at each specimen in the configuration shown in Fig. 3. The strain readings were used initially to verify that the load was being applied concentrically and later to eliminate the effect of the elastic deformation of the plattens [12, 20, 22]. To verify concentricity of loading, a small alignment load, approximately equal to 10% of the estimated failure load was applied and the variation in strain around the cross-section was checked; in all cases, individual strain gauge readings varied less than 5% from the average strain, which was deemed acceptable. All data (load, voltage, strains and displacements) were recorded at two seconds intervals using the data acquisition system DATASCAN.

Tests were continued beyond the ultimate load-carrying capacity of the stub columns, and the post-ultimate response was recorded. The ultimate load and the corresponding end shortening at ultimate load are given in Table 6, while the full load-end shortening curves for the tested specimens are depicted in Fig. 4. Note that the reported end shortening curves and the end-shortening values corresponding to ultimate load given in Table 6 refer to the true stub column shortening, which is obtained on the basis of the recorded LVDT and strain readings according to the procedure recommended in [22]. Failure was due to local buckling though often after considerable plastic deformation; typical failure modes are depicted in Fig. 5.

2.4 Flexural buckling tests

Having established the basic material and cross-sectional response, twelve flexural buckling tests were carried out in order to obtain ultimate load carrying capacity data and assess the suitability of the current codified buckling curve for hollow sections [8] for lean duplex stainless steel SHS and RHS. The tests were conducted on pin-ended columns with nominal cross-sectional dimensions of $80 \times 80 \times 4$, $60 \times 60 \times 3$ and $80 \times 40 \times 4$, in a similar fashion to the tests described in [20]. Both minor and major axis buckling were considered for the RHS $80 \times 40 \times 4$ specimens. The specimen lengths were chosen such that the buckling lengths (i.e. total distance between knife edges) were equal to 800 mm, 1200 mm, 1600 mm and 2000 mm. This provided a range of non-dimensional member slendernesses, defined through by Eq. (1), in accordance with Eurocode 3: Part 1.4 [8], from 0.57 to 2.02.

$$\bar{\lambda} = \sqrt{A\sigma_{0.2}/N_{cr}} \quad (1)$$

where A is the cross-sectional area, $\sigma_{0.2}$ is the 0.2% proof stress and N_{cr} is the elastic critical buckling load of the column.

All tests were carried out in a 600 kN Instron capacity rig under displacement control. Knife edges were employed to achieve the pin-ended boundary conditions, as shown in Fig. 6, where the test rig is also depicted. A close-up of the knife edges is depicted in Fig. 7. The employed instrumentation may also be seen in Fig. 6 and consisted of a load cell attached to the top knife edge, two pairs of LVDTs at each end of the column measuring end rotations and end shortening and two string pots attached at the mid-height of the columns measuring the lateral deflection of the specimens.

Measurements of the specimen geometry, including initial global geometric imperfections e_0 were conducted prior to testing and are reported in Table 7. The measured overall geometric imperfections were generally small and hence the load was applied eccentrically at the ends such that the combined effects of initial bow and loading eccentricity gave a total eccentricity at mid-height of $L/1500$, where L is the pin-ended column buckling length. This value is the statistical mean of geometric imperfections in steel structural members [23].

All columns failed by flexural buckling without any visible sign of local buckling. The full load-lateral displacement curves were recorded and are shown in Figs. 8 and 9 for SHS and RHS columns respectively. The key results from the column tests, including the ultimate load and the lateral displacement at ultimate load are reported in Table 8. All obtained test results have been used in the validation of the numerical models, as described in Section 3, and are analysed and discussed in detail in Section 4 of the present paper.

3. Numerical modelling

3.1 Basic modelling assumptions

The tests reported in the previous section have been utilised to validate FE models and generate additional results by means of parametric studies, thus enabling a thorough assessment of the key parameters affecting the structural response of lean duplex stainless steel compression members. The general purpose finite element analysis package ABAQUS [24] was used for all numerical studies reported in this paper. The FE simulations followed the proposals regarding numerical modelling of stainless steel components reported in [25, 26].

Measured geometric properties reported in Tables 5 and 7 for stub columns and long columns, respectively, have been employed in the FE models. Owing to the thin-walled nature of tubular sections, and in line with similar previous investigations [7, 20, 25, 26, 27], shell elements were employed to discretise the models. The 4-noded doubly curved shell element with reduced integration S4R [24] has been utilised in this study. As discussed later, it was assumed that the corner properties, as derived from the corner coupon tests, extended up to a distance equal to two times the material thickness into the flat region of each face of the models on either side of the corners. Two elements were utilised to discretise each of these flat parts adjacent to the corners and hence, in order to maintain a uniform mesh size within all flat parts of the models, an element size equal to the material thickness was required for all models. A coarser, non-uniform mesh was shown to yield results of similar accuracy but given the low computational cost associated with the finer mesh size, a uniform mesh was employed. Regarding the root radii, three elements were used to discretise them, assuming that their geometry is approximated by circular arcs.

Geometry, boundary conditions, applied loads and failure modes of the tested components were observed to be symmetric. The displayed symmetry was exploited in the finite element modelling with suitable boundary conditions applied along the symmetry axes, enabling significant savings in computational time. Regarding the stub columns, only a quarter of the section was modelled, whereas for the long columns, half of the cross-section was discretised. For both stub columns and long columns the full component length was modelled. All degrees of freedom were restrained at the end cross-sections of the stub column models, apart from vertical translation at the loaded end, which was constrained via kinematic coupling to follow the same vertical displacement. Similar boundary conditions were applied to the flexural buckling models, with the only difference lying in the rotational degree of freedom about the

axis of buckling of the end cross-sections, which was not restrained, thus enabling the modelling of the pin-ended boundary conditions.

It has been experimentally verified that the cold-forming process induces strength enhancements in the corner regions of cold-formed components for carbon steel [28] and stainless steel [29]. The enhanced strength also extends beyond the curved corner regions into the flat parts of the cross-section. A quantitative assessment of the effect of cold-forming on the stress-strain response of lean duplex stainless steel can be found in [7]. Previous studies [25, 26] suggest that the best agreement between experimental and FE results for cold-rolled stainless steel hollow sections is obtained when the corner properties extend into the flat regions by a distance equal to two times the material thickness. This has been verified by experimental observations in the corner regions [29], and this approach has been followed in the present study. The material properties derived from tensile corner tests (as reported in Table 4) were assigned to the corner regions of the models and the adjacent flat regions up to two times the material thickness, whereas compressive material properties (as reported in Table 3) were assigned to the remainder of the sections.

Residual stresses in cold-formed tubular sections may be categorised as (1) bending residual stresses that vary through the thickness of the sections and arise as a result of plastic deformation during forming and (2) membrane residual stresses that are induced during the seam-welding operation to complete the tube. Careful measurements [30] have shown the latter to be relatively insignificant in stainless steel hollow sections and largely swamped by the dominant bending residual stresses. Furthermore, the effect of the bending residual stresses is inherently present in the material stress-strain properties [30, 31] since the residual stresses that are released during the cutting of the coupons (causing longitudinal curvature) are essentially reintroduced by straightening of the coupons during testing. Residual stresses were not therefore explicitly introduced into the described models, but their influence was present in the material modelling.

As mentioned in Section 2.1, a compound version [10-13] of the basic Ramberg-Osgood material model [14, 15] was employed to simulate the stress-strain response of lean duplex stainless steel, with the respective material parameters given in Tables 3-5. For incorporation into the FE analyses, this material model was approximated with a multilinear curve, the points of which were distributed proportionally to curvature of the original continuous curve

[32], following a procedure described in [33], in order to minimise the error introduced by the approximation. These points were thereafter converted into true stress σ_{true} and log plastic strain $\varepsilon_{\text{ln}}^{\text{pl}}$, as defined by Eqs. (2) and (3)

$$\sigma_{\text{true}} = \sigma_{\text{nom}}(1 + \varepsilon_{\text{nom}}) \quad (2)$$

$$\varepsilon_{\text{ln}}^{\text{pl}} = \ln(1 + \varepsilon_{\text{nom}}) - \frac{\sigma_{\text{true}}}{E} \quad (3)$$

where σ_{nom} and ε_{nom} are the engineering stress and strain respectively and E is the Young's modulus.

Based on the aforementioned modelling assumptions, a series of FE models were generated. Linear eigenvalue buckling analyses using the subspace iteration method were initially performed to extract the buckling mode shapes. These served as initial geometric imperfection patterns used in the subsequent geometrically and materially non-linear analyses. The modified Riks method [24], which is essentially a variation of the classical arc-length method, was employed for the non-linear analyses to enable the full load-deflection response, including into the post-ultimate range to be simulated.

The lowest local buckling mode shape was utilised to perturb the geometry of the stub columns, while both the first local and first global mode shapes were introduced as geometric imperfections in the flexural buckling models. Four variations of the local imperfection amplitude were considered in the non-linear analyses; the maximum measured imperfection reported in Table 5, 1/10 and 1/100 of the cross-sectional thickness and the imperfection amplitude derived from the predictive model of [34] as adapted for stainless steels [25], given by Eq. (4)

$$w_0 = 0.023 \left(\frac{\sigma_{0.2}}{\sigma_{\text{cr}}} \right) t \quad (4)$$

where $\sigma_{0.2}$ is the tensile 0.2% proof stress given in Table 2 and σ_{cr} is the elastic critical buckling stress of the most slender of the constituent plate element in the section, determined

on the basis of the flat width of the element. For the global imperfection amplitudes, four fractions of the respective buckling length were considered, namely $L/500$, $L/1000$, $L/1500$ and $L/2000$, noting that $L/1500$ represents the experimental imperfection.

3.2 Validation of models and parametric studies

In this section the results of the numerical simulations and the tests are compared, and the sensitivity of the models to the key modelling parameters, particularly the imperfection amplitudes, are examined. Comparisons with the test results are made to assess the accuracy of the models and verify their suitability for performing parametric studies.

Table 9 presents the ratios of the numerical to experimental ultimate loads and corresponding displacements at ultimate load for the varying imperfection amplitudes. The ultimate load is generally well-predicted for the measured imperfection amplitude, the amplitude predicted by the Dawson and Walker model (Eq. (4)) and $t/100$, whereas the use of the $t/10$ value results in a clear underestimation of the load carrying capacity of the stub columns. The end shortening at ultimate load appears to be more sensitive to the initial imperfection amplitude and is best predicted when an imperfection amplitude from the Dawson and Walker model or $t/100$ is used. The Dawson and Walker model predicts imperfection amplitudes on the basis of both geometric and material properties of cross-sections. It has been shown, as in the current study, to provide suitable local imperfections for inclusion in numerical models to accurately simulate tests [25-27], and to provide a means of predicting measured imperfection amplitudes directly [19, 25]. This model was therefore employed in the parametric studies described in this paper to derive local imperfection amplitudes for both the stub columns and long columns.

Overall excellent agreement between the experimental stub column results and those obtained from the FE simulations was achieved; the compressive response was accurately predicted throughout the full loading history, including initial stiffness, ultimate load, displacement at ultimate load and post-ultimate response. Figs. 10 and 11 depict the experimental and numerical load-end shortening curves using the imperfection amplitude predicted by the Dawson and Walker model for the $80 \times 40 \times 4$ -SC2 and $80 \times 80 \times 4$ -SC2 stub columns, whereas a comparison of experimental and numerical failure modes is displayed in Fig. 12.

Good agreement between test and numerical results is also displayed for the flexural buckling specimens. Comparisons are shown in Table 10, where it may be seen, as expected, that the ratio of the numerically predicted ultimate buckling load to the experimental buckling load is clearly influenced by the assumed initial global imperfection amplitude. The most accurate and consistent prediction of test response is obtained for an imperfection amplitude of $L/1500$, which coincides with the total imperfection amplitude (initial bow imperfection plus eccentricity) present in the tests. Comparisons between experimental and FE results in terms of load versus lateral deflection are depicted in Figs. 13, 14 and 15 for an SHS column, an RHS column buckling about the major axis and an RHS column buckling about the minor axis, respectively. The FE failure modes also compare well with the test failure modes, as displayed in Fig. 16.

Upon validation of the FE models for both stub columns and long column parametric studies have been conducted. The generated models adhere to the basic modelling assumptions stated in Section 3.1. The material properties adopted in the FE parametric studies were based on the averaged experimental material stress-strain curves; flat compressive and corner tensile material properties were assigned to the respective parts of the models. Local geometric imperfections in the form of the lowest buckling mode shape with an amplitude derived from Eq. (4) were incorporated for both stub column and flexural buckling models, whereas the global imperfection amplitude of the long columns was taken as $L/1500$.

All cross-sections considered in the parametric studies had an outer width B equal to 100 mm and an outer height H equal to either 100 mm or 200 mm, thereby resulting into aspect ratios of 1.0 and 2.0. The length of the stub column models was set equal to four times their mean outer dimension, hence 400 mm for the SHS and 600 mm for the RHS models, while their thickness varied from 1.6 mm to 13.0 mm to encompass a wide range of cross-sectional slendernesses. The cross-section slenderness was defined as $c/t\varepsilon$ in accordance with Eurocode 3: Part 1-4 [8], where c is the flat element width, t is the element thickness and $\varepsilon = \sqrt{(235/f_y)(E/210000)}$. Regarding the flexural buckling models, constant thicknesses of 4.75 mm and 9.50 mm were selected for the 100×100 and 100×200 cross-sections respectively, resulting in Class 3 cross-sections according to the slenderness limits given in [8] - the actual $c/t\varepsilon$ ratio was 30, compared to the Class 3 slenderness limit of 30.7. The buckling length of the columns was varied to cover a wide spectrum of member slendernesses

ranging from 0.4 to 2.4. The results of the parametric studies are discussed in the following section.

4. Analysis of results and design recommendations

4.1 Introduction

In this section, the applicability of the provisions of Eurocode 3: Part 1.4 [8], including the Class 3 slenderness limit and effective width formula for internal elements in compression and the buckling curve for hollow section columns to lean duplex stainless steel structural components is assessed on the basis of both the experimental and numerical results reported in this paper. Furthermore, the modified slenderness limits and effective width formulae for stainless steel cross-sections, proposed by the authors on the basis of a significantly larger experimental data pool than was available at the time of development of Eurocode 3: Part 1.4 in [9], are also assessed. Finally, comparisons of the structural performance of lean duplex stainless steel with that of the more common stainless steel grades in construction are made. In all code comparisons, the measured tensile material properties derived for each cross-section from flat tensile coupon tests were utilised.

4.2 Class 3 slenderness limit for elements in compression

The obtained test and FE data were used to assess the applicability of the codified slenderness limits to lean duplex stainless steel elements. For all experimental and numerical stub column results, the ultimate load divided by the squash load, $F_u/A\sigma_{0.2}$, is plotted against the slenderness of the most slender constituent element in the cross-section in Fig. 17, where the respective Class 3 limits for carbon steel and stainless steel specified by Eurocode 3: Part 1.1 [35] and Eurocode 3: Part 1.4 [8], as well as the Class 3 limit proposed in [9] are also included.

As shown in Fig. 17, the RHS ($H/B=2.0$) display superior load carrying capacity to their SHS ($H/B=1.0$) counterparts of equal cross-sectional slenderness (i.e. $c/t\epsilon$). This is due to the higher level of restraint offered by the narrow faces to the wider (more slender) faces of the

RHS and the potential for stress redistribution once local buckling of the wider face plates occurs. In order to maintain simplicity, the effect of element interaction on the cross-sectional response is not accounted for in [8] or [35] and a conservative cross-section classification approach is specified, according to which all elements are treated in isolation and the cross-sectional response relates to its most slender element. More advanced approaches accounting for element interaction have been derived for hot-rolled carbon steel H-sections [36], cold-formed carbon steel sections [37] and cold-formed stainless steel sections [38].

Within the current cross-section classification approach codified in [8], the Class 3 limit (i.e. the limit below which an element can be assumed to be fully effective) of 30.7ε is conservative and could be relaxed to 37ε , as proposed by the authors [9] for other grades of stainless steel. The respective carbon steel Class 3 limit of 42ε is marginally unconservative and does not provide adequate reliability as assessed by the statistical analysis conducted in [9], according to Annex D of EN 1990 [39].

4.3 Effective width formula

Slender (Class 4) cross-sections are treated in Eurocode 3: Part 1-4 [8] following the Von Karman effective width approach, as modified according to experimental data of Winter [40-42], to account for the occurrence of local buckling prior to reaching the 0.2% proof strength. The effective width equation for internal elements given in Eurocode 3: Part 1.4 is compatible with the corresponding codified Class 3 limit of 30.7ε , which has been shown to be rather conservative. For consistency with the revised limit of 37ε , a revised effective width equation was proposed [9], as given by Eq. (5):

$$\rho = \frac{0.772}{\bar{\lambda}_p} - \frac{0.079}{\bar{\lambda}_p^2} \leq 1 \quad (5)$$

where ρ is the reduction factor for local buckling and $\bar{\lambda}_p$ is the element slenderness, as defined in [8]. The Class 3 limits set out in [8] and [9] and the F_u/F_y (ultimate load normalised by the squash load) ratios predicted according to the respective effective width equations are plotted together with the F_u/F_y data points derived from parametric studies against the $c/t\varepsilon$ ratio of the most slender plate element in Fig. 18. The results confirm the adequacy but

conservatism of the current Eurocode 3: Part 1.4 provisions and the applicability of the proposed revised formula (Eq. (5)) [9].

4.4 Flexural buckling

The applicability of the buckling curve specified in Eurocode 3: Part 1.4 for hollow sections to lean duplex stainless steel tubular columns is assessed by comparing the column test and numerical data with the respective codified predictions. For both experimental and FE results, the ultimate load has been normalised by the corresponding squash load (defined as $A\sigma_{0.2}$) and plotted against the non-dimensional slenderness $\bar{\lambda}$ in Fig. 19, where the stub column test data are also included. The effect of the aspect ratio is insignificant for slender columns, but becomes increasingly important with decreasing member slenderness, because of the increasing influence of cross-sectional behaviour (i.e. local buckling). Good agreement between the test data and code predictions is observed and hence application of the current buckling curve ($\bar{\lambda}_0 = 0.4$ and $\alpha = 0.49$) to lean duplex stainless steel SHS and RHS columns is proposed in the present paper.

4.5 Comparison of lean duplex with other stainless steel grades

The initial material cost of stainless steel comprises two components: the basic manufacturing cost and the alloy adjustment factor, which depends on the alloying elements used and hence varies markedly between grades. Lean duplex stainless steel only contains approximately 1.5% nickel, resulting in a relatively low alloy adjustment factor and hence a competitive initial material cost [43]. In Figs. 20 and 21 the structural response of stub columns and long columns of the most commonly adopted structural stainless steel grades (i.e. austenitic and duplex grades) is compared with the corresponding lean duplex test data reported herein. The stub column data included in Fig. 20 have been reported in [12, 44-49], whereas the flexural buckling data were taken from [45-48, 50, 51]. In the determination of the slenderness parameter plotted on the horizontal axis of Figs. 20 and 21, only geometric properties have been included (c/t for stub columns and L_{cr}/i , where L_{cr} is the buckling length and i is the radius of gyration, for long columns), so that the effect of material is accounted for only in the vertical axis. In the high slenderness regime all stainless steel grades exhibit similar structural capacities since failure is governed principally by stiffness. However, for stockier cross-

sections and members the lean duplex and conventional duplex structural components behave similarly and exhibit superior performance to their austenitic counterparts of similar geometric slenderness, since their higher strength can be fully utilized.

5. Conclusions

Eight stub column tests and twelve flexural buckling tests on lean duplex stainless steel SHS and RHS have been reported in detail in the present paper. The results of the experimental investigation were supplemented by numerically generated data. Upon validation of the FE models, parametric studies were conducted to investigate the structural response over a wide range of cross-sectional slenderness for the stub columns and member slenderness for the long columns. Based on both experimental and numerical data, the provisions of Eurocode 3: Part 1-4 for the classification and local buckling treatment of internal elements in compression and buckling for stainless steel hollow section columns, were assessed. Both the class 3 limit and the corresponding effective width equation for internal elements in compression was shown to be adequate but conservative and the adoption of the more favourable slenderness limits and effective width formulae [7] for stainless steel elements is supported herein. Regarding the flexural buckling response of lean duplex stainless steel columns, the current buckling curve for stainless steel hollow sections is deemed suitable. Overall, lean duplex stainless steel is shown to offer superior structural performance compared to the austenitic grades and at a lower cost [43], which represents a significant economic advantage and renders lean duplex stainless steel an attractive choice for structural applications.

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Table 1: Chemical composition (% by weight) of test material

Section	C (%)	Si (%)	Mn (%)	P (%)	S (%)	Cr (%)	Ni (%)	N (%)	Mo (%)	Cu (%)
60×60×3	0.025	0.8	4.99	0.02	0.001	21.64	1.5	0.209	0.3	0.31
80×80×4 and 80×40×4	0.028	0.7	4.85	0.021	0.001	21.4	1.6	0.229	0.26	0.29
100×100×4	0.019	0.64	5.05	0.02	0.001	21.41	1.57	0.227	0.28	0.34

Table 2: Tensile flat material properties

Cross-section	E (N/mm ²)	$\sigma_{0.2}$ (N/mm ²)	$\sigma_{1.0}$ (N/mm ²)	σ_u (N/mm ²)	ϵ_f %	Compound R-O coefficients	
						n	n' _{0.2,1.0}
SHS 100×100×4	198800	586	632	761	47	9.0	2.8
SHS 80×80×4	199900	679	736	773	42	6.5	4.2
SHS 60×60×3	209800	755	819	839	44	6.0	4.3
RHS 80×40×4	199500	734	785	817	50	10.1	3.4

Table 3: Compressive flat material properties

Cross-section	E (N/mm ²)	$\sigma_{0.2}$ (N/mm ²)	$\sigma_{1.0}$ (N/mm ²)	Compound R-O coefficients	
				n	n' _{0.2,1.0}
SHS 100×100×4	198200	560	642	8.3	2.6
SHS 80×80×4	197200	657	770	4.7	2.6
SHS 60×60×3	206400	711	845	5.0	2.7
RHS 80×40×4	204000	607	734	4.6	2.9

Table 4: Tensile corner material properties

Cross-section	E (N/mm ²)	$\sigma_{0.2}$ (N/mm ²)	$\sigma_{1.0}$ (N/mm ²)	σ_u (N/mm ²)	ϵ_f %	Compound R-O coefficients	
						n	n' _{0.2,1.0}
SHS 100×100×4	206000	811	912	917	32	6.3	4.1
SHS 80×80×4	210000	731	942	959	24	5.6	3.7
SHS 60×60×3	212400	885	1024	1026	22	6.3	4.0
RHS 80×40×4	213800	831	959	962	26	4.4	4.0

Table 5: Measured dimensions of stub columns

Specimen	L (mm)	B (mm)	H (mm)	t (mm)	r_i (mm)	A (mm ²)	w_0 (mm)
100×100×4-SC1	400.0	101.0	102.0	3.93	3.8	1495.2	0.071
100×100×4- SC2	400.0	102.0	103.0	3.97	3.9	1524.7	0.071
80×80×4- SC1	319.7	80.0	80.5	3.88	3.8	1147.4	0.080
80×80×4- SC2	332.2	80.0	80.0	3.81	3.6	1125.0	0.080
60×60×3- SC1	239.8	60.0	60.0	3.09	2.3	683.0	0.062
60×60×3- SC2	240.0	60.0	60.0	3.17	2.1	700.4	0.062
80×40×4- SC1	239.9	39.0	79.5	3.76	3.5	799.8	0.058
80×40×4- SC2	237.8	39.6	79.5	3.81	4.3	808.8	0.058

Table 6: Summary of test results for stub columns.

Specimen	Ultimate Load F_u (kN)	End shortening at ultimate load δ_u (mm)
100×100×4-SC1	1022	3.63
100×100×4- SC2	1037	4.01
80×80×4- SC1	923	4.13
80×80×4- SC2	915	3.88
60×60×3- SC1	613	4.09
60×60×3- SC2	616	3.69
80×40×4- SC1	709	4.33
80×40×4- SC2	710	4.12

Table 7: Measured geometric properties of columns

Specimen	Axis of buckling	H (mm)	B (mm)	t (mm)	r_i (mm)	A (mm ²)	Buckling length L_{cr} (mm)	Global imperfection amplitude e_0 (mm)
80×80×4-2000	-	79.6	79.5	3.80	3.4	1116.7	1999.0	0.41
80×80×4-1200	-	79.3	79.6	3.72	3.8	1091.0	1199.5	0.10
60×60×3-2000	-	60.0	60.0	3.13	2.7	689.1	1999.0	0.31
60×60×3-1600	-	59.6	60.0	3.15	2.4	692.4	1599.0	0.32
60×60×3-1200	-	60.0	60.0	3.13	2.4	689.8	1199.0	0.26
60×60×3-800	-	60.0	60.0	3.13	2.4	690.8	799.0	0.23
80×40×4-MI-1600	Minor	39.0	79.2	3.80	4.3	800.4	1600.0	0.03
80×40×4-MJ-1600	Major	79.5	39.3	3.95	4.0	835.8	1599.5	0.25
80×40×4-MI-1200	Minor	40.0	79.2	3.80	3.8	811.3	1199.0	0.15
80×40×4-MJ-1200	Major	79.6	39.5	3.96	3.6	842.4	1199.5	0.13
80×40×4-MI-800	Minor	39.5	79.4	3.80	3.6	810.0	797.2	0.22
80×40×4-MJ-800	Major	79.9	39.5	3.93	4.1	835.6	799.0	0.28

Table 8: Key results from flexural buckling tests

Specimen	Non-dimensional slenderness $\bar{\lambda}$	Ultimate load F_u (kN)	Lateral deflection at F_u (mm)
80×80×4-2000	1.21	361.9	20.0
80×80×4-1200	0.73	672.5	4.7
60×60×3-2000	1.66	162.3	19.5
60×60×3-1600	1.34	231.7	15.4
60×60×3-1200	0.99	326.9	10.4
60×60×3-800	0.66	445.9	5.9
80×40×4-MI-1600	2.02	160.4	4.1
80×40×4-MJ-1600	1.14	406.3	3.8
80×40×4-MI-1200	1.47	237.4	9.9
80×40×4-MJ-1200	0.86	497.7	7.7
80×40×4-MI-800	0.99	366.6	9.0
80×40×4-MJ-800	0.57	546.2	6.3

Table 9 Comparison of the stub column test results with FE results for varying imperfection amplitudes

Stub column designation	Measured amplitude w_0		$t/10$		$t/100$		Dawson and Walker model	
	FE F_u / Test F_u	FE δ_u / Test δ_u	FE F_u / Test F_u	FE δ_u / Test δ_u	FE F_u / Test F_u	FE δ_u / Test δ_u	FE F_u / Test F_u	FE δ_u / Test δ_u
100×100×4-SC1	0.95	0.71	0.86	0.61	0.98	0.78	0.97	0.73
100×100×4- SC2	0.96	0.64	0.87	0.50	0.98	0.70	0.98	0.69
80×80×4- SC1	1.00	0.68	0.92	0.45	1.01	0.75	1.02	0.80
80×80×4- SC2	1.02	0.81	0.95	0.57	1.05	0.96	1.06	0.98
60×60×3- SC1	0.97	0.86	0.90	0.54	0.98	0.91	0.98	0.91
60×60×3- SC2	0.99	0.89	0.93	0.57	1.00	0.99	1.00	1.02
80×40×4- SC1	1.00	0.83	0.90	0.55	1.03	0.93	1.03	0.93
80×40×4- SC2	0.97	0.76	0.89	0.61	1.01	1.03	1.01	1.04
Mean	0.98	0.77	0.90	0.55	1.00	0.88	1.01	0.89
COV	0.02	0.12	0.03	0.10	0.03	0.14	0.03	0.15

Table 10 Comparison of the column test results with FE results for varying imperfection amplitudes

Specimen	FE F_u / Test F_u			
	L/500	L/1000	L/1500	L/2000
80×80×4-2000	0.96	1.03	1.06	1.08
80×80×4-1200	0.89	0.94	0.95	0.96
60×60×3-2000	0.94	1.00	1.03	1.04
60×60×3-1600	0.93	0.99	1.02	1.04
60×60×3-1200	0.94	1.00	1.03	1.04
60×60×3-800	0.99	1.01	1.02	1.03
80×40×4-MI-1600	0.81	0.87	0.89	0.90
80×40×4-MJ-1600	0.85	0.90	0.93	0.94
80×40×4-MI-1200	0.87	0.93	0.96	0.97
80×40×4-MJ-1200	0.90	0.94	0.96	0.98
80×40×4-MI-800	0.92	0.97	0.99	1.00
80×40×4-MJ-800	1.01	1.05	1.07	1.08
Mean	0.92	0.97	0.99	1.01
COV	0.06	0.06	0.05	0.05

Fig. 1: Lean duplex stainless steel footbridge in Siena, Italy.

Fig. 2: Section labelling convention and location of flat and corner coupons.

Fig. 3: Stub column testing apparatus.

Fig. 4: Load-end shortening curves for stub columns.

Fig. 5: Typical stub column failure modes (from left to right: 60×60×3-SC1, 80×80×4-SC1, 80×40×4-SC1).

Fig. 6: Test setup for flexural buckling tests.

Fig. 7: Knife edge detail.

Fig. 8: Load-lateral displacement curves for SHS columns.

Fig. 9: Load-lateral displacement curves for RHS columns.

Fig. 10: Experimental and numerical load-end shortening curves for 80×40×4-SC2.

Fig. 11: Experimental and numerical load-end shortening curves for 80×80×4-SC2.

Fig. 12: Experimental and FE failure modes for SHS 80×80×4-SC2.

Fig. 13: Experimental and numerical load-lateral displacement cures for SHS60×60×3-L=2000 mm column.

Fig. 14: Experimental and numerical load-lateral displacement cures for 80×80×4-MJ-L=1200 mm column.

Fig. 15: Experimental and numerical load-lateral displacement cures for 80×80×4-MI-L=1200 mm column.

Fig. 16: Experimental and FE failure modes for SHS 80×80×4-L=1600 mm column.

Fig. 17: Current and proposed Class 3 slenderness limit for internal elements in compression.

Fig. 18: Assessment of EC3: Part 1.4 and proposed effective width formulae for internal elements.

Fig. 19: Normalised test and FE column results and assessment of EC3 buckling curve.

Fig. 20: Performance of stub columns of various stainless steel grades.

Fig. 21: Performance of columns of various stainless steel grades.