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Ultimate response of stainless steel continuous beams

M. Theofanous [a,*], N. Saliba [b], O. Zhao [a], L. Gardner [a]

[a] *Department of Civil and Environmental Engineering, Imperial College London, United Kingdom*
[b] *Department of Civil Engineering, University of Balamand, Lebanon*

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ABSTRACT

An experimental study of stainless steel continuous beams not susceptible to lateral torsional buckling is reported in this paper and the applicability of plastic design methods to such structures is considered. A total of 18 two-span continuous beams were tested. Three cross-section types – cold-formed square hollow sections (SHS), cold-formed rectangular hollow sections (RHS) and welded I-sections, and two material grades – austenitic EN 1.4301/1.4307 and lean duplex EN 1.4162, were considered. The geometric and material properties of the continuous beam test specimens were carefully recorded and supplemented by tests on simply supported specimens of the same cross-sections. The test specimens covered a wide range of cross-section slendernesses and two different loading positions were adopted. The experimental results were used to assess the degree of moment redistribution in indeterminate stainless steel structures and the applicability of both conventional and novel plastic design methods, including an extension of the continuous strength method (CSM). Comparisons indicated that conventional plastic design is applicable to stainless steel structures, while greater efficiency can be achieved by considering strain-hardening through the CSM.

1. Introduction

Efficient design of metallic structures often involves the exploitation of the inelastic range of the material's stress–strain curve, provided sufficient ductility is available. Modern structural design codes determine the extent to which this exploitation is allowed through the process of cross-section classification. The European structural design codes for carbon steel [1] and stainless steel [2] specify four behavioural classes of cross-sections according to their susceptibility to local buckling. Indeterminate carbon steel structures comprising Class 1 cross-sections classified as Class 1 are assumed to possess sufficient deformation capacity to allow plastic design. However, despite the high material ductility of structural stainless steels [3] and the existence of a Class 1 limit in EN 1993-1-4 [2], plastic design is not currently permitted for stainless steel structures.

The absence of suitable guidance for the design of indeterminate stainless steel structures can be partly attributed to a lack of relevant experimental research. The majority of previously published test data relate to individual stainless steel components rather than complete structures, although some tests on continuous stainless steel beams that allow the assessment of moment redistribution have been reported [4,5]. This paper substantially increases the pool of available test data on indeterminate stainless steel structures, by reporting an experimental investigation on 18 two-span continuous beams. Both cold-formed hollow sections (SHS and RHS) and welded I-sections are examined. Additionally, tests on simply supported beams with the same cross-sections as the continuous beam specimens are reported and the experimental results are utilised in analysing the continuous beam test results. The experimental response of both the simply supported beams and the continuous beams is then compared with the predictions of EN 1993-1-4 [2]. Analysis of the results reveals that current design provisions are overly conservative, since they do not account for material strain-hardening or the significant moment redistribution (in the case of the continuous beams) taking place before collapse. Hence material savings can be achieved if inelastic design procedures are followed at both cross-section level and system level. To this end, the continuous strength method (CSM), originally developed for stainless steel determinate structures [6–8], which allows for the actual material response at cross-sectional level, is adapted to stainless steel indeterminate structures, resulting in more favourable and accurate strength predictions.

2. Experimental studies

An experimental investigation into the structural response of stainless steel simple and continuous beams has been carried out in the Structures Laboratory at Imperial College London. The employed cross-sections were SHS and RHS in grade EN 1.4301/

* Corresponding author.
E-mail address: m.theofanous@imperial.ac.uk (M. Theofanous).

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1.4307 and welded I-sections in grade EN 1.4162. Following material coupon tests, five 3-point bending tests on SHS and RHS, four 3-point bending tests on I-sections and four 4-point bending tests on I-sections were initially performed, to extract fundamental flexural performance data. These were utilised to assess the suitability of current design provisions in EN 1993-1-4 [2]. Subsequently 18 two-span continuous beam tests (five-point bending) were conducted, which enabled the study of stainless steel indeterminate structures and an assessment of the accuracy of current codified provisions. Performing both simply supported and continuous beam tests on the same cross-sections enables the study of the effect of moment redistribution on ultimate capacity of indeterminate structures, since the effect of strain-hardening at cross-sectional level is captured in the 3-point bending tests. A full account of the performed experimental investigations can be found in [9–12].

2.1. Material coupon tests

From each of the cold-formed hollow sections employed in the beam tests, both flat and corner coupons were extracted and tested in tension and the key results are summarised in Tables 1 and 2 for flat and corner coupons respectively. Similarly, Table 3 reports the average tensile properties exhibited by the stainless steel plates from which the welded I-sections were fabricated. All tensile tests were conducted in accordance with EN 10002-1 [13]. In Tables 1–3, E is Young's modulus, $\sigma_{0.2}$ is the 0.2% proof stress, $\sigma_{1.0}$ is the 1.0% proof stress, σ_u is the ultimate tensile stress, ε_f is the plastic strain at fracture based on elongation over the standard gauge length $(5.65\sqrt{A_0})$ and n and $n'_{0.2,1.0}$ are strain-hardening exponents for the compound Ramberg–Osgood model [4,14] as modified in [6]. The measured stress–strain curves are reported in [10–12].

2.2. Bending tests on simply supported beams

A total of 13 tests on simply supported beams were conducted on the same cross-sections as those employed for the continuous beam tests. The tests were used to quantify the effect of cross-section slenderness on the bending resistance and rotation capacity of the tested beams and to assess the suitability of the European codified slenderness limits as well as revised slenderness limits proposed elsewhere [15]. Moreover the simply supported beam tests were utilised subsequently in the analysis of the continuous beam tests, in order to quantify the relative contribution of strain-hardening at cross-sectional level and moment redistribution at system level to the overstrength displayed by the continuous beams compared to the codified predictions.

One test was conducted for each of the three SHS employed in the 3-point bending configuration (Fig. 1), whilst two tests were conducted for the RHS 60 × 40 × 3 specimen, one about the major axis and one about the minor axis. The RHS and SHS beams had a total length of 1200 mm and were simply supported between rollers, which allowed axial displacement of the beams' ends. The rollers were placed 50 mm inward from each beam end. For the RHS 60 × 40 × 3-MA specimen the face containing the weld was the web, whilst in all other cases the face containing the weld was the bottom (tension) flange. A wooden block was inserted at the location of load application to prevent web crippling. All tests were carried out at a rate of 3.0 mm/min. Loads, end rotations, displacements at the points of load application (and at mid-span for the four point bending tests) and strains at a distance of 100 mm from mid-span were all monitored and recorded at one-second intervals using the data acquisition system DATASCAN. Prior to testing, measurements of the geometry of the specimens were taken, which are summarised in Table 4 along with the ultimate moment resistance and the deformation capacity achieved by each specimen. The adopted labelling convention of the cross-section geometry is shown in Fig. 2. The rotation capacity was defined according to Eq. (1), where the θ_u is the total rotation at mid-span when the moment–rotation curve falls back below the plastic moment capacity M_{pl} as obtained from the test results and θ_{pl} is the elastic component of the rotation when M_{pl} is reached defined as $\theta_{pl}=M_{pl}L/2EI$ (I being the second moment of area and L the beam length) as shown in Fig. 3. The rotation at mid-span was assumed to equal the sum of the end rotations

$$R=\frac{\theta_u}{\theta_{pl}}-1 \tag{1}$$

All specimens failed by local buckling of the compression flange and the upper part of the web, as shown in Fig. 4. The recorded mid-span moment–rotation (at plastic hinge) responses of the tested beams are depicted in Fig. 5 in a non-dimensional format; the recorded moment has been normalised by the respective plastic moment resistance, while the rotation at plastic hinge has been normalised by θ_{pl}, to facilitate comparison between the specimens.

For each of the four welded I-section geometries considered herein one simply supported beam was tested in the 3-point configuration and one in the 4-point configuration as depicted

Table 1
Tensile flat material properties for SHS and RHS.

Cross-section	E (N/mm^2)	$\sigma_{0.2}$ (N/mm^2)	$\sigma_{1.0}$ (N/mm^2)	σ_u (N/mm^2)	Modified R–O coefficients	
					n	$n'_{0.2,1.0}$
SHS 50 × 50 × 3	198,000	552	608	798	5.50	2.90
SHS 60 × 60 × 3	197,730	483	546	745	5.25	2.90
SHS 100 × 100 × 3	201,300	419	470	725	5.25	2.25
RHS 60 × 40 × 3	191,690	538	592	753	5.00	3.50

Table 2
Tensile corner material properties for SHS and RHS.

Cross-section	E (N/mm^2)	$\sigma_{0.2}$ (N/mm^2)	$\sigma_{1.0}$ (N/mm^2)	σ_u (N/mm^2)	Modified R–O coefficients	
					n	$n'_{0.2,1.0}$
SHS 50 × 50 × 3	195,000	723	918	927	4.56	3.76
SHS 60 × 60 × 3	193,440	614	776	855	4.75	4.25
SHS 100 × 100 × 3	189,520	694	829	839	5.50	3.50
RHS 60 × 40 × 3	198,530	741	968	984	4.67	4.00

Table 3
Longitudinal tensile material properties for plates comprising I-sections.

Nominal plate thickness (mm)	E (N/mm^2)	$\sigma_{0.2}$ (N/mm^2)	$\sigma_{1.0}$ (N/mm^2)	σ_u (N/mm^2)	Modified R–O coefficients	
					n	$n'_{0.2,1.0}$
6	193,500	516	557	727	10.70	2.20
8	203,000	504	545	727	12.10	2.20
10	216,500	501	557	768	11.70	2.20
12	205,500	456	506	723	10.50	2.40