Structural assessment of composite seam-welded rectangular hollow columns

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Structural elements made of seam-welded hollow sections are considerably less expensive than their seamless equivalents. This is a strong incentive for their use, but the current steel design standards still contain recommendations for seam-welded hollow sections based on test results performed with sections produced using outdated steel grades and fabrication processes. These design procedures are even more conservative when applied to composite tubular seam-welded elements. This paper presents an experimental campaign performed to investigate the structural behaviour of steel- and concrete-filled composite columns made with seam-welded rectangular hollow profiles. The experiments involved columns with only steel sections, concrete-filled and concrete-filled plus additional reinforcing bars. The main aim of the tests was to identify the contribution of each component in the overall structural response. The results enable an evaluation of the design recommendations present in Eurocode 4 aiming to determine less conservative buckling curves to be used for the investigated composite columns made of seam-welded rectangular hollow sections.

Notation

A_a	steel cross-sectional area
A_c	concrete cross-sectional area
A_s	reinforcing steel cross-sectional area
E	Young's modulus
E_{cm}	concrete secant elastic modulus
f_{cd}	concrete compressive strength design value
f_{ck}	concrete compressive strength
f_s	reinforcing steel yield stress
f_{sd}	reinforcing steel yield stress design value
f_u	steel ultimate stress
f_y	steel yield stress
f_{yd}	steel yield stress design value
$N_{b,Rd}$	composite column compressive buckling resistance
N_{EC4}	Eurocode 4 composite column compressive resistance
N_{exp}	column experimental ultimate load
$N_{pl,Rd}$	composite column compressive plastic resistance
α_{cc}	concrete confinement coefficient
$\bar{\lambda}$	normalised slenderness ratio
χ	buckling reduction factor

1. Introduction

Novel building construction strategies and techniques are constantly being developed to achieve greater heights of structures with economic efficiency. One of the most popular solutions brings together reinforced concrete and structural steel, usually known as composite structures.

These composite structures have many advantages, such as: minimising the corrosion of the concrete reinforcing steel bars

(in steel hollow sections filled with concrete); protection of the structural steel profile when it is encased in concrete; better response to fire conditions; and reduction of the column cross-section, among others. Many composite columns were initially used to enhance the structure's fire resistance; however, the development of high-strength steel and concretes, alongside the pressing need to construct higher buildings, have together boosted the adoption of composite columns design (Calado and Santos, 2013).

Recent studies (Silva *et al.*, 2016; Tenchini *et al.*, 2014) have highlighted the advantages of using composite columns in moment-resisting frames ('dual steel') characterised by the combined use of high-strength steel (HSS) in non-dissipative members and mild carbon steel (MCS) in dissipative zones, especially for inducing the seismic response of multi-storey buildings to be controlled by overall ductile mechanism responses.

When composite seam-welded columns are considered, reports presented in the literature highlight the need for a better understanding of their global behaviour and the influence of effects such as residual stresses and concrete confinement.

Schneider (1998) presented an experimental and analytical study of short, concrete-filled steel tube columns concentrically loaded in compression up to failure. This study involved 14 compressive axial tests of circular, square and rectangular composite columns. All steel tubes were made of cold-formed carbon steel with a specified yield strength of 317 MPa. All tubes were seam welded and annealed to relieve residual stresses. The main conclusions were: the square and rectangular

tube walls did not show a significant concrete nucleus confinement after the yield load of the composite column and the AISC specification (ANSI/AISC, 2010) provided a conservative prediction for the axial strength of the composite columns.

Han (2002) investigated the behaviour of 24 concrete-filled rectangular hollow section (RHS) stub columns subjected to axial compression. The tubes were all manufactured from mild steel sheet, with four plates cut from the sheet, tack welded into a rectangular shape and then welded with a single bevel butt weld at the corners. On these tests, the typical failure mode was local (outward folding) failure mechanism. The section capacities predicted using the Eurocode 4 (BSI, 2004) design methods were compared with the stub column test results. The results predicted by Eurocode 4 were 10% lower than the tests with a mean value of 0.879 and a coefficient of variation (COV) of 0.0991.

Sakino *et al.* (2004) tested 114 hollow and concrete-filled steel tubular (CFT) short column specimens under axial compression. The main test parameters were: (a) tube shape, (b) tube tensile strength, (c) tube diameter-to-thickness ratio and (d) concrete strength. The circular steel tubes were cold formed from a flat plate by press bending and seam welding. The square steel tubes were fabricated by welding together two pieces of channel section, which were cold formed from a flat plate. For circular composite columns, the authors concluded that the design formula in the Architectural Institute of Japan recommendations (AIJ, 1997) predicts with good accuracy the axial load capacity. However, for square composite columns, the cited design code led to a slightly conservative value for columns with small B/t ratio where B and t are the width and the thickness of the square cross-section, respectively. According to the authors, the main reason for this difference can be associated with strain hardening effects in the steel tubes.

Nardin and El Debs (2006) reported an experimental study of CFT columns. Six short columns subjected to concentric load were tested. Columns of square (CFTS), circular (CFTC) and rectangular (CFTR) cross-sections with two steel tube thicknesses were tested. The square tubes for CFTS and the rectangular tubes for CFTR were constructed by seam welding two U-shaped, cold-formed steel plates. The circular tubes for CFTC were hot-rolled structural steel sections. The experimental results showed that the ductility of high-strength concrete increased when confined by a steel tube, but the load-carrying capacity remained the same. The confinement effect did not raise the capacity of the columns because the axial strain of the high-strength concrete core was significantly not mobilised up to the column ultimate load. A model to predict the load against strain behaviour was proposed and the pre-peak and post-peak behaviour of concrete-filled steel columns when axially loaded could be predicted from the response of the steel tube and concrete core. As a result, Eurocode 4 and

American Institute of Steel Construction (AISC) underestimate the axial load capacity of the specimens with circular cross-section where the highest difference reached was 28%.

Han *et al.* (2008) produced 32 stub composite columns experiments with end plates, divided into circular and square sections, submitted to axial compression. The tubes were all manufactured from mild steel sheet (2.83 mm thick with a yielding strength of 362.9 MPa), with the plate being cut from the sheet, tack welded into a circular or square shape and then welded with a single bevel butt weld. The measured compressive cube strength (f_{cu}) was 74.3 MPa. The columns exhibited a ductile response during the tests. The resistance was controlled by the steel tube and, as the endplates' thickness increased, the steel tube ends deformation, restrained by a thick endplate, enhanced the concrete confinement effects.

Oliveira *et al.* (2009) performed an experimental analysis of the confinement effects in steel-concrete composite columns centred on two parameters: concrete compressive strength and column slenderness. Sixteen CFT columns with circular cross-section were tested under axial loading. The tested columns were filled by concrete with compressive strengths of 30, 60, 80 and 100 MPa, and had length/diameter ratios of 3, 5, 7 and 10. According to the results, Eurocode 4 (BSI, 2004) and CAN/CSA (2001) presented the lowest differences between the predicted and the experimental ultimate load results, 2.4% and 2.3% on average.

Recently, Han *et al.* (2014) presented some developments and advanced applications of CFT structures focusing on the member behaviour. The authors showed a comparison of the measured results between a circular steel (S) stub column, a reinforced-concrete (RC) stub column and a concrete-filled steel tubular (CFST) stub column without steel reinforcement. As expected, the ultimate strength for a concrete-filled steel tube is even larger than the summation of the strength of the steel tube and the RC column, which is described as '1 (steel tube) + 1 (concrete core) greater than 2 (simple summation of the two materials)'. Finally, the authors concluded that, for the CFST column with various column lengths, the Eurocode 4 (BSI, 2004) led to the highest prediction when the slenderness ratio λ was less than 80, whereas the ANSI/AISC360 (ANSI/AISC, 2010) led to the lowest estimate when λ was less than 30.

Concerning the residual stress in cold-formed, thick-walled, square hollow sections, Tong *et al.* (2012) performed an experimental investigation in sections with thickness greater than 6 mm. Square hollow sections were formed using two different forming processes: (a) a 'direct square' where the section was formed by rolling an annealed flat strip directly into a square hollow section, which was then welded at the edges; (b) 'indirect way from circular to square' where the section was formed by rolling an annealed flat strip into a

circular hollow section first, which was then welded at the edges and the process was completed by further rolling into a square hollow section. The authors reported that the longitudinal residual stresses were in tension at the outer surface and in compression at the inner surface, with non-linear distributions, which resemble a 'sine' curve along the section thickness.

Dundu (2016) tested 29 square composite columns made of hot-rolled square hollow sections, with variable length and cross-sections ranging from $60 \times 60 \times 3.0$ mm to $150 \times 150 \times 4.5$ mm. The concrete strength was 30 MPa for all the specimens and the yield strength of the steel profiles varied from 355 MPa to 522 MPa according to the tensile tests. The test procedure was to apply an axial load at a rate of 2 mm/min until failure, using a boundary condition with pinned ends. The general failure mode shows an overall member buckling, caused especially by the large slenderness ratio. As expected, all the specimens showed a reduction in strength as the slenderness ratio increased. The Eurocode 4 (BSI, 2004) predicted a more accurate compressive resistance than the South African design code SANS 10162-1 (SANS, 2001), which overestimated the columns' capacity. The author concluded that square concrete-filled steel tubes provide an insignificant concrete core confinement effect.

Based on this scenario, the present paper presents the results of experiments performed to study the structural response of composite columns made with seam-welded rectangular hollow profiles. The experiments involved columns with: only steel sections (S), concrete-filled (S&C) and concrete-filled with additional reinforcing bars (S&RC). The objective of the tests was to identify the contribution of each component to the overall structural response while enabling an assessment of the design recommendations given in Eurocode 4 (BSI, 2004). The outcomes of the present investigation is centred on establishing less conservative buckling curves for the investigated composite columns made of seam-welded rectangular hollow sections produced by a high-frequency induction welding process.

2. Steel and composite column design

The structural columns are predominantly loaded by axial compression or by axial compression and bending moment. According to Eurocode 4 (BSI, 2004), distinct composite cross-sections may be considered: steel sections fully encased with concrete; steel sections partially encased with concrete; or steel square, rectangular and circular tube sections filled with concrete.

According to Eurocode 4 (BSI, 2004), the interaction between the two materials (steel and concrete) in composite sections only occurs if the strains for both steel yield stress and concrete compressive stress are similar. It is well known that the strain for the concrete compressive ultimate strength of 3.5‰ is

about two times higher than the strain for the yield stress (1.75‰) for the usual steel grades. Thus, for composite action the steel must have the capacity to reach plastic strains, without local buckling, that is, class 1 or 2 sections are required (CEN, 2003b). The composite columns' structural behaviour can be affected by: creep, shrinkage, concrete adherence and confinement, the concrete strength, the slenderness of the composite column cross-section, the steel profile to total cross-sectional area ratio, among others.

The buckling curves of Eurocode 3 (BSI, 2005) and Eurocode 4 (BSI, 2004) distinguish between profiles of hot-rolled (curve a) and cold-formed (curve c) sections. However, with current manufacturing processes, seam-welded tubes are manufactured by a high-frequency welding process, which gives them mechanical properties similar to seamless hot-rolled sections.

The investigated composite columns were designed according to Eurocode 4 (BSI, 2004). The steel profiles used in the tests were seam-welded rectangular hollow sections made of ASTM A36 steel grades (equivalent to the steel grades between S235 and S275) and a 30 MPa concrete strength. The Eurocode 4 presents two design methods, the general method and the simplified method; the latter was used in this paper. This method limits the normalised slenderness maximum value to $\lambda \leq 2.0$.

According to Eurocode 4 (BSI, 2004), a full interaction of concrete, steel profile and reinforcement bars is the aim. Both reinforcement bars and steel profile cross-sections must reach the yield stress design value (f_{yd} and f_{sd}), and the concrete cross-section must reach the compressive stress design value ($0.85f_{cd}$). Thus, the compressive plastic resistance $N_{pl,Rd}$ is

$$1. \quad N_{pl,Rd} = A_a f_{yd} + 0.85 A_c f_{cd} + A_s f_{sd}$$

In the case of the cross-sections of steel hollow profile filled with concrete, Eurocode 4 (BSI, 2004) specifies that the concrete compressive failure stress should be increased from 0.85 up to 1.0 (f_{cd}) due to the concrete confinement effect. Therefore, the compressive plastic resistance $N_{pl,Rd}$ can be rewritten as

$$2. \quad N_{pl,Rd} = A_a f_{yd} + A_c f_{cd} + A_s f_{sd}$$

3. Experimental programme

In total, 12 tests were performed in this work, divided into three distinct groups: the first included only the $100 \text{ mm} \times 80 \text{ mm} \times 6.3 \text{ mm}$ seam-welded steel profiles (S). The second incorporated the same steel profiles now filled with concrete (S&C). The last group was also filled with concrete (S&RC),

Table 1. Experiments and Eurocode 4 design predictions comparison

Test	N_{exp} : kN	$N_{pl,Rd}$: kN	$N_{b,Rd}$: kN	N_{EC4} : kN	N_{exp}/N_{EC4}
TR-03_1000_S	962.34	805.73	642.82	642.82	1.50
TR-04_1000_S	839.08	805.73	642.82	642.82	1.31
TR-01_1000_S&C	885.13	958.30	822.90	822.90	1.08
TR-06_1000_S&C	1059.60	958.30	822.90	822.90	1.29
TR-02_1000_S&RC	936.17	1020.64	869.76	869.76	1.08
TR-05_1000_S&RC	994.84	1020.64	869.76	869.76	1.14
TR-07_500_S	1032.41	805.73	—	805.73	1.28
TR-09_500_S	1022.36	805.73	—	805.73	1.27
TR-08_500_S&C	1125.69	958.30	—	958.30	1.17
TR-10_500_S&C	1178.59	958.30	—	958.30	1.23
TR-11_500_S&RC	1271.72	1020.64	—	1020.64	1.25
TR-12_500_S&RC	1310.79	1020.64	—	1020.64	1.28
Mean					1.24
Standard deviation					0.11

while presenting four 6.3 mm dia. longitudinal steel reinforcing bars and stirrups of the same diameter (only present at the supports and mid-span to tie up the reinforced bars).

The other variable investigated in the experiments was the column height, that is, two column heights were tested at 1000 mm and 500 mm, as can be observed in Table 1. Their associated normalised slenderness $\bar{\lambda}$ indicated that, for the 1000 mm composite columns ($\bar{\lambda} \approx 0.60$), the flexural global buckling controlled the column design, whereas for the 500 mm composite columns ($\bar{\lambda} \approx 0.20$), the failure was associated with the full section yielding.

In relation to the necessary concrete amount to fill the prototypes, the compression specimen tests and the concrete compressive strength to be reached, the mass proportion was as follows: 1: 2.17: 2.44 (28.11 kg of cement, 61.00 kg of fine aggregate, 68.59 kg of coarse aggregate, 15.46 kg of water and 0.028 kg of superplasticiser) and a water–cement factor of 0.55 were utilised. The stress plotted against strain curves obtained according to the Brazilian standard NBR 8522 (ABNT, 2008) made possible the determination of the concrete secant elastic modulus, $E_{cm} = 24.16$ GPa. Compression specimen tests led to a concrete compressive strength, $f_{ck} = 26.42$ MPa. This compressive strength was less than the pre-defined value of 30 MPa, probably due to the superplasticiser addition and/or by the non-reduction of the water–cement factor during the concrete casting.

The steel A36 hollow profiles' mechanical characterisation was made according to NBR 6152 (ABNT, 2002). Five coupon tests were performed leading to: a mean Young's modulus of 193.52 GPa with a standard deviation of 12.02 MPa; a mean yield stress of 382.17.52 MPa with a standard deviation of 5.04 MPa; and a mean ultimate stress of 441.03 MPa with a standard deviation of 2.89 MPa. Additional details can be found in Castanheira (2016).

Special plates with slots in which cylinders were inserted were fabricated and adopted at the column ends to simulate simple support conditions, close to $k = 1.0$, and increase the columns' buckling effective length as presented in Figure 1. This strategy was used in the profile lower inertia direction to induce the flexural buckling to occur in this direction. Figure 2 shows the columns assembly in the hydraulic press during the tests, with the linear variable displacement transducers (LVDTs) positioned at eight different points. Figure 2 also depicts the strain gauges (S) that were adopted and used to determine the maximum strains achieved at four different points along the column.

4. Experimental results

The 500 mm columns tests (stub columns) were simpler, as there was no need to simulate the simple support conditions because the full section yield was expected. Alternatively, the 1000 mm high columns (intermediate columns) test assembly was more complex due to the hinge simulation arrangement to simulate the column flexural buckling response. Table 1 shows the ratio between the tests' maximum loads and the values predicted by Eurocode 4 (BSI, 2004) evaluated according to the material mechanical properties obtained from the specimens. The Eurocode 4 predictions (N_{EC4}) considered the possibilities of full section yielding ($N_{pl,Rd}$) and flexural buckling ($N_{b,Rd}$).

Figure 3(a) depicts a comparison between the 500 mm high column tests to assess the influence of the concrete and additional reinforcing bars. All tests presented the full cross-section yielding and a subsequent local buckling at mid-height and/or close to the supports. The steel columns (S) present curves with lower ultimate load values, a steeper load decreasing branch and a smaller ductility capacity. The curves for steel columns with concrete (S&C) and with reinforced concrete (S&RC) reach higher ultimate loads; this occurred, among other reasons, owing to the presence

Figure 1. End conditions with pinned supports: (a) top support, before test; (b) bottom support, maximum rotation at ultimate load

Figure 2. Test configuration and LVDT layout – TR03_1000_S: (a) rear view; (b) front view

of the concrete and longitudinal reinforcement bars that increase the columns' load-carrying capacities. These curves also presented a gradual load decreasing branch and higher ductility capacities.

Figure 3(b) presents a similar comparison for the 1000 mm high columns. It can be observed that all the column failures were controlled by flexural buckling, with a well-defined ultimate load followed by an abrupt unloading branch. Once

Figure 3. Steel and composite columns load plotted against displacement curves: (a) 500 mm high columns; (b) 1000 mm high columns

Figure 4. Columns' deformed configurations: (a) TR-07_500_S; (b) TR-08_500_S&C; (c) TR-12_500_S&RC; (d) TR-04_1000_S; (e) TR-01_1000_S&C; (f) TR-05_1000_S&RC

more the steel columns (S) presented curves with lower maximum load values, a steeper load decreasing branch and a smaller ductility capacity. The concrete-filled columns (S&C) and the reinforced composite columns (S&RC) achieved variable ultimate loads values. One column presented a value

almost equal to the steel column and another with a higher value, probably due to the concrete strength and the presence of longitudinal reinforcement bars. These curves also presented a gradual load decreasing branch and higher ductility capacities. An inspection of the curves' initial slope made it

Figure 5. Lateral displacement evolution along the column height, TR-03_1000_S test

possible to conclude that, as expected, for steel columns the initial stiffness is lower than for all the composite steel columns.

Figure 4 illustrates the typical columns' deformed configurations. The 500 mm columns' deformed shapes presented in Figures 4(a)–4(c) are characterised by the presence of plastic hinges and local buckling (the latter occurred after the full section yielded) at the column centre and supports. The 1000 mm columns' deformed configurations presented in Figures 4(d)–4(f) are all characterised by a maximum lateral displacement close to the column's centre, due to the flexural buckling that occurred in the column's minor axis.

In order to evaluate the actual boundary conditions that occurred in the tests, the results obtained with the displacement transducers are presented and discussed. Figure 5 illustrates the results of the lateral displacements of three selected points along the column height for various applied load magnitudes. These results were acquired with the aid of six displacement transducers positioned at three points along the column height: 750 mm, 500 mm and 250 mm measured from the column base. Figure 5 illustrates these curves for the 1.0 m high non-composite column test TR-03_1000_S. In this figure it is possible to observe that, before the ultimate load (962.34 kN) is reached, the curves exhibit a similar behaviour associated with a column configuration with fixed and hinged supports, that is, $k=0.7$. As the load increases, the horizontal displacements change, coming into line with a column with both hinged supports, that is, $k=1.0$, while presenting a typical global buckling collapse.

Figure 6 presents the normalised load plotted against normalised slenderness curves for the investigated columns groups (S, S&C and S&RC). For each group the Eurocode 3 (BSI, 2005) and Eurocode 4 (BSI, 2004) curves associated with seamless steel profiles (curve a) and seam-welded steel profile (curve c) are presented alongside the test results, making possible an assessment of the actual curve that better fits the tested configurations.

The TR-09_500_S and TR-07_500_S, TR-08_500_S&C and TR-10_500_S&C, TR-11_500_S&RC and TR-12_500_S&RC tests were all stub columns with normalised slenderness values, $\bar{\lambda}$, less than 0.2, and they were strongly affected by strain hardening. Figure 6(b) presents the results of tests TR-08_500_S&C, TR-10_500_S&C, TR-01_1000_S&C and TR-06_1000_S&C. This figure also incorporates the 29 square composite columns results by Dundu (2016), which were made of hot-rolled square hollow sections, with variable length and cross-sections ranging from $60 \times 60 \times 3.0$ mm to $150 \times 150 \times 4.5$ mm. The Dundu (2016) tests adopted in all specimens a concrete strength of 30 MPa and a steel profiles yield strength varying from 355 MPa to 522 MPa. It may be observed that the Dundu (2016) tests with normalised slenderness ranging from 0.6 and 1.5 overestimated composite columns' resistances predicted by Eurocode 4 curve a.

The TR-03_1000_S and TR-04_1000_S, TR-01_1000_S&C and TR-06_1000_S&C, TR-02_1000_S&RC and TR-05_1000_S&RC tests can be classified as simple supported intermediate columns with height of 1000 mm, that is, $k=1$, controlled by flexural buckling, where the normalised slenderness ratios are

Figure 6. Normalised column load plotted against normalised slenderness curves for: (a) steel columns; (b) steel and concrete columns; (c) steel and reinforced-concrete columns

calculated taking into account the materials involved (steel, concrete and reinforcement bars). All tested columns with normalised slenderness ratios ranging from 0.2 and 0.6 are situated above Eurocode 4 (BSI, 2004) curve a. This affirmative was also observed in the tests performed by Dundu (2016). This strongly indicates that the conservatism is associated with the design of these seam-welded profiles in this normalised slenderness range, when, in fact, they could be designed as seamless profiles. Obviously, more tests must be performed with other cross-sections and other normalised slenderness, especially in the 0.6–1.5 range, to fully validate such an affirmation.

5. Final considerations

Table 2 shows the comparison of the gain in percentage associated with each composite column material. Equations are also presented taking into account the percentage associated with each resistance contribution, for both experiments and Eurocode 4 (CEN, 2004) design values. The results shown in this table enable the determination of the individual contributions of the steel profile (S), concrete (S&C minus S) and reinforcing bars (S&RC minus S&C) assessed from the experiments and Eurocode 4 predictions. The experiments and Eurocode 4 predictions indicated that the steel contribution was responsible for 80% of the column

Table 2. Assessment of the individual contributions to the column load-carrying capacity

Mean experimental resistances: kN			% total
Steel	$S =$	1027.39	80.0
Concrete	$S\&C - S =$	124.76	9.0
Reinforcing bars	$S\&RC - S\&C =$	139.12	11.0
Total	$S + S\&C + S\&RC =$	1291.26	100.0
Eurocode 4 predictions: kN			
Steel	$S =$	805.73	79.0
Concrete	$S\&C - S =$	152.57	15.0
Reinforcing bars	$S\&RC - S\&C =$	62.34	6.0
Total	$S + S\&C + S\&RC =$	1020.64	100.0

load-carrying capacity. This was not the case for the other two components. The experiments and Eurocode 4 predictions for the concrete contribution to the column capacity were, respectively, 9% and 15%, whereas for the reinforcing bar contribution to the column capacity they were, respectively, 11% and 6%.

The tests with columns 1000 mm high (intermediate slenderness) were controlled by flexural buckling, while the columns 0.5 m high (stub columns) failed by the section yielding followed by local buckling at the column centre and near its supports. The results obtained from the lateral displacement transducers indicated that the hinged support configuration performed as expected, while the effect of possible eccentricities created during each test installation and profile initial imperfections have not led to any discrepancies in the results.

The load plotted against vertical displacement curves indicated a real resistance increase in all composite columns in relation to the non-composite (only steel tubes) columns. It could be observed that the columns controlled by section yielding were associated with curves that had less defined ultimate loads, while also presenting higher ductility capacities. It could also be noted that, in all of the 1000 mm high columns, the failure modes presented a well-defined ultimate load followed by an abrupt unloading branch. An inspection of the curves' initial slope made it possible to conclude that, as expected, for steel columns the initial stiffness is lower than for all the composite steel columns.

Unfortunately, according to the authors' knowledge, there are still very few references dealing with seam-welded profiles. However, all tested columns with seam-welded profiles and normalised slenderness ranging from 0.2 to 0.6 indicated the conservatism of Eurocode 4 (BSI, 2004) curve a. This is a strong indication of the under-design of these seam-welded profiles and that they could be designed as seamless profiles. Obviously, more tests must be performed with other cross-sections and other normalised slenderness values, especially in the 0.6–1.5 range, to fully validate these results.

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