

Design of Short-to-Intermediate Slender Built-up Flanged Cruciform Columns

Jelena Dobrić^{a,*}, Nina Gluhović^a, Aljoša Filipović^a, Mileva Samardžić-Petrović^a, Xiongfeng Ruan^b,
Barbara Rossi^c

^a University of Belgrade, Faculty of Civil Engineering, Serbia

^b KU Leuven, Department of Chemical Engineering, Sint-Katelijne-Waver, Belgium

^c University of Oxford, Department of Engineering Science, Oxford Sustainable Metal Structures Research Group, Oxford, United Kingdom

*Corresponding author: jelena@imk.grf.bg.ac.rs

Abstract

This paper explores the structural behaviour of cold-formed austenitic stainless steel flanged cruciform section columns through a combination of experimental and numerical methods, focusing on the influence of discrete fasteners on their flexural and torsional stiffness. The columns are designed as doubly symmetrical built-up sections, utilizing four star-oriented lipped equal-leg angle profiles interconnected with M8 bolts of class 8.8. Four intermediate slender built-up cruciform section columns, each measuring 2250 mm in length and featuring various fastener arrangements, were tested under semi-rigid support conditions and subjected to pure axial compression. The specimens failed due to a combination of torsional and local buckling. Following the experimental campaign, a parametric study was conducted based on experimentally validated numerical models to investigate key variables such as cross-sectional dimensions, column length, and bolt spacing. The experimental and numerical findings regarding the failure modes of the columns and their corresponding buckling resistances were compared with theoretical predictions. The recently proposed design procedures for carbon steel columns, based on the Direct Strength Method for built-up sections and the established Perry-Robertson design curve for solid cruciform sections, were evaluated. Results indicated that these procedures exhibited insufficient accuracy and significant data scatter, which necessitated modifications for improved predictions. The new design equations enhance accuracy and reliability, laying the groundwork for their potential incorporation into future standards.

Keywords

Flanged cruciform sections; Fastener interconnections; Local buckling; Torsional buckling; DSM- and EC3-based design.

1 Introduction

Steel structural elements with flanged cruciform-shaped sections are traditionally used in buildings. The orthogonal position of their Tee-legs enables simple connections with floor and roof beams, making these structural elements an ideal choice for the columns of three-dimensional (moment-resisting) frame systems. Conventional flanged cruciform section columns usually involve post-production processes, often by welding hot-rolled plates and I-profiles. Cold-formed flanged cruciform section columns, however, are generally built-up columns, i.e. fabricated by connecting single components into a composite unit, using various fastening techniques, employing bolts, screws, or spot welds. To maximise the structural efficiency of the column, different single cross-section shapes, such as plain or lipped channel- and equal-leg sections, are adequately grouped and appropriately connected, see Fig. 1. Such cold-formed flanged cruciform section members combine lightness and ease of fabrication.

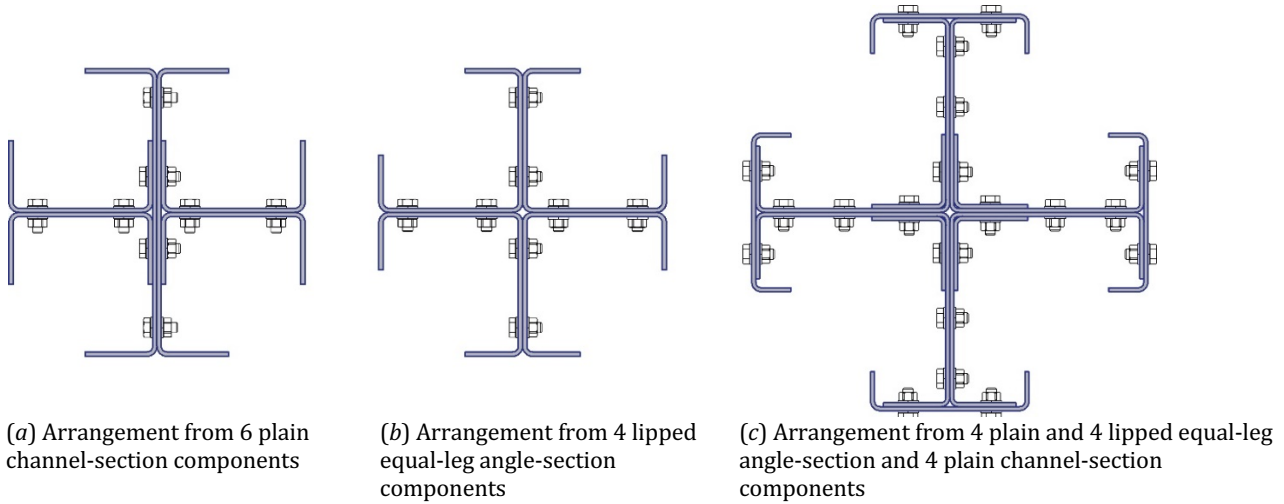


Fig. 1 Cold-formed built-up flanged cruciform-shaped sections.

A distinguishing feature of thin-walled cruciform section columns is their high susceptibility to torsional buckling. The inherent double symmetry of the cross-section, coupled with the coinciding centroid and shear centre, results in the absence of flexural displacements in the pure torsional buckling mode; under these conditions, all four cross-section legs exhibit identical twisting behaviour. Plain cruciform section columns, having plate legs (walls) intersecting at the centroid, exhibit no primary warping rigidity (the warping constant I_w can be considered to be zero), negligible torsional GI_t and low flexural EI rigidity. By introducing flanges as edge-stiffeners (which reduces the leg transverse bending), the biaxial flexural EI_y, EI_z , and torsional stiffness $GI_t + \pi^2 EI_w / L^2$ of these columns can be significantly increased (depending on the flange width-to-thickness ratio).

The ultimate resistance and instability of the cruciform section column (under axial compression) is governed by one of the following failure modes: (i) yielding or local buckling (LB) of cross-section legs for stocky columns, (ii) combined local-torsional modes with dominant local mode contribution, or pure torsional buckling (TB) for short to intermediate slenderness and (iii) pure flexural buckling (FB) for very slender column. The ultimate response of thin-walled cruciform section columns is primarily determined by their elastic buckling capacity, often leading to the interaction of local and global instability modes (depending on the cross-section and column slenderness). Each Tee-leg of a flanged cruciform section acts like an edge-stiffened plate, longitudinally supported along the leg junction. The boundary conditions along the longitudinal leg junction significantly depend on the rotational restraints stemming from the flexibility (slenderness) of the other 3 Tee-legs. When the rotational restraint is insufficient, the cross-section Tee-leg exhibits “rigid-body” like rotation with very little (or no) transverse bending (torsional mode). Conversely, sufficient rotational restraint entails local buckling modes with multiple half-waves featured by transverse bending of the Tee-leg webs. The local instability mode of the Tee-leg webs also depends on the flange slenderness. Elastically restrained outstanding flanges of Tee-legs typically demonstrate transverse bending and tend to buckle in multiple half-waves (rotational restraint provided by the other cross-section elements triggers the flange local instability).

Determining the instability modes that trigger plastic mechanisms and, ultimately, failure becomes more complex in the case of cold-formed built-up cruciform section columns, considering the discreteness of its cross-section. Under pure axial compression, the individual components experience relative shear displacements at the fastener interfaces in the longitudinal (load) direction. Therefore, the built-up sections are characterised by limited composite action of the individual components and, consequently, partial flexural and warping rigidity. The appropriate arrangement of fasteners, including their spacing and grouping along the column length and the leg

widths, ensures the composite integrity of built-up members affected by instability-induced deformations. Unlike connections realized by spot welds or screws, bolted connections typically feature a bolt-to-hole clearance that allows for a greater relative slipping to occur between the interconnected components, thereby increasing flexibility within the built-up column assembly. The fasteners are most effective in areas with the greatest shear deformations. In the case of axially compressed pin-ended built-up column (the column end cross-sections are free to undergo global flexural deformations while being restrained from torsional rotations), the shear deformation increases from mid-height towards the ends; thus, the effectiveness of the fasteners is optimized when they are positioned at the column ends, which is a widely recognized finding that has influenced design requirements in the American National Standards ANSI/AISC 360-22 [1] and AISI S100-16 (2020) w/S3-22 [2]. However, this structural behaviour depends on the support conditions at the column ends. In general, fixed-ended supports of axially compressed built-up columns inhibit longitudinal shear displacements at their ends; therefore, in this case, the end fastener groups are ineffective (this is also valid for pin-ended columns with load transfer through rigid end plates that prevent longitudinal shear displacements at the column ends). Nevertheless, in the case of semi-rigid column bases, a certain level of partial restraint against longitudinal shear deformations between the individual components at the column ends, arising from the rotational flexibility of the support conditions, exists. Several factors determine the ability of individual components to avoid separation or independent deformations: the axial stiffness of each component, the in-plane stiffness of the interconnected regions in contact, the shear and bearing strengths of the fasteners and the specific fastener arrangement.

There is virtually no scientific literature on built-up columns experiencing torsional instability mode. Recent research on carbon, stainless steel and high-strength cold-formed steel built-up structural elements has offered valuable insights into their buckling behaviours, including local, distortional, flexural, and flexural-torsional instabilities, see [3] to [31]. The structural performances and design considerations of cruciform columns have attracted significant interest from several researchers, see [32] to [40]. Significant research efforts have focused on examining the buckling and post-buckling behaviour of cruciform columns, aiming to develop new design guidelines or improve existing ones to predict their ultimate resistances more accurately. A detailed review of the scientific literature is provided in [41], [42].

A lack of experimental studies on built-up cruciform columns restricts the ability to conduct numerical simulations and reliable parametric analyses to grasp their overall structural behaviour. Dobrić et al. [41], [42] recently conducted both experimental and numerical investigations focusing on cruciform section columns made of austenitic stainless steel, considering two cross-section configurations: built-up plain cruciform section columns made from four equal-leg angle sections and flanged cruciform section columns composed of six plain channel sections. They investigated the effects of fastener arrangement on the column instability modes in different slenderness ranges.

The objectives of this paper are to examine the effectiveness of adding fasteners along the column lengths and across the leg widths as a means of reducing shear deformations and enhancing the buckling load for short to intermediate-length columns, and to perform accuracy assessments of the recently proposed design procedures for carbon steel columns, i.e. (1) the Direct Strength Method (DSM)-based design rules for built-up sections composed of 3 or more interconnected elements that experience failure due to FB or flexural-torsional buckling (FTB), proposed by Rasmussen et al. [15] and Phan et al. [20], [21], and (2) the modified Eurocode 3 design (based on Perry-Robertson formulation) for solid cruciform section proposed by Behzadi-Sofiani et al. [40] containing a key parameter related to the torsional slenderness and the torsional-to-flexural buckling load ratio.

In this paper, an experimental investigation of four intermediate-length cold-formed austenitic stainless steel (CFSS) built-up flanged cruciform section columns is presented. These columns were constructed from four star-oriented lipped equal-leg angles. The experimental results are used to develop and validate a finite element (FE) model. Following this, an FE parametric analysis to examine how variations in the spacing between interconnections (fastener rows) affect the ultimate buckling resistance of these columns across various lengths and cross-sectional geometries is conducted. The design procedures referenced in [20], [21] and [40] are outlined and assessed critically (using statistical metrics) in comparison to the established data; based on the evaluation results, directions for improvements are proposed.

2 Experiment

2.1 Specimens design and test preparation

This section describes the material and geometry characteristics of the column specimens, followed by a description of the test setup and procedure. Finally, the experimental results are presented.

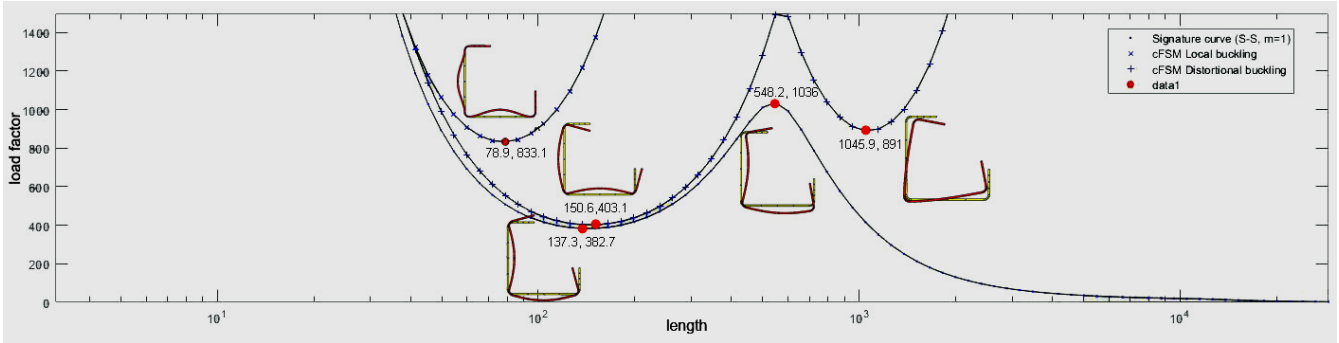
The experimental campaign comprised four global buckling tests on built-up flanged cruciform section specimens subjected to pure axial compression. Each specimen was constructed from four star-oriented lipped equal-leg angle section components, with nominal dimensions of $50 \times 135 \times 3$ mm (“lip width” $\times$ “leg width” $\times$ “wall thickness”) and an internal corner radius of 6.5 mm. The angle sections were press-braked from a cold-rolled sheet of austenitic stainless steel grade EN 1.4301 (UNS S30400), X5CrNi18-10 [43]. According to the slenderness limits outlined in EN 1993-1-4 [44], the angle sections are classified as Class 4. The angle components were interconnected with M8 bolts, property class 8.8. The hole diameter was 9 mm, allowing for a clearance of 1 millimetre between the bolt and the hole. The connection was designed as Category A (bearing type), per EN 1993-1-8 [45].

The Conventional and Constrained Finite Strip Method (CUFSM) [46] was utilized to establish initial predictions for the non-composite lower limits of built-up columns—based on the elastic buckling curve of individual angle components—and the fully composite upper limits, which consider the elastic buckling behaviour of cruciform columns with fully connected angle wall surfaces along their lengths (the wall thicknesses of the individual angle sections in contact were effectively combined to form a single wall that is double the thickness of an individual angle).

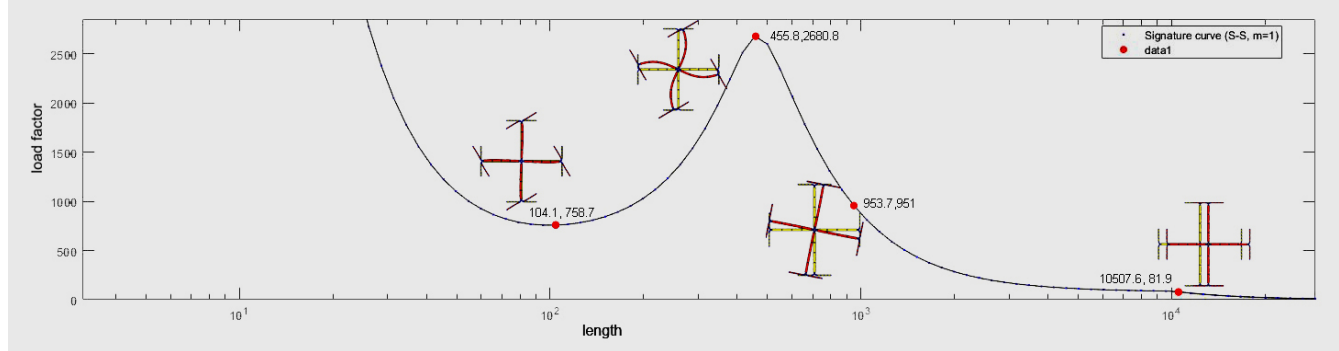
For built-up cruciform sections, the presence of fasteners leads to a partial composite action between individual angle components. The partially composite conditions among individual angle components—factoring in nodal constraints at fastener locations—were not analysed using the CUFSM; this was instead tackled with a Linear Bifurcation Analysis (LBA) through the FE software ABAQUS [47].

The signature curves for axially compressed columns with lipped equal-leg angle sections, having nominal cross-section dimensions of $50 \times 135 \times 3$ mm and pinned (simple-simple) supports, obtained using the software CUFSM [46], are depicted in Fig. 2a. The effects of fastener (bolt) holes were not accounted in this analysis. The half-wavelength for LB is 78.9 mm, less than the half-wavelength for distortional buckling (DB), which equals 1045.9 mm. The signature curve for a pin-ended built-up cruciform section (displayed in Fig. 2b), assuming full composite action among the angle components, reveals distinct minima for LB in Tee-leg flanges and also for the entire Tee-legs, along with a representation of global buckling modes, including TB and FB. The half-wavelength for LB of the Tee-leg flanges is 104,1 mm, whereas it is 455,8 mm for a local instability mode of the Tee-leg walls. The signature curve for a column with fixed (clamped-clamped) supports does not materialize distinctly specific minima for the

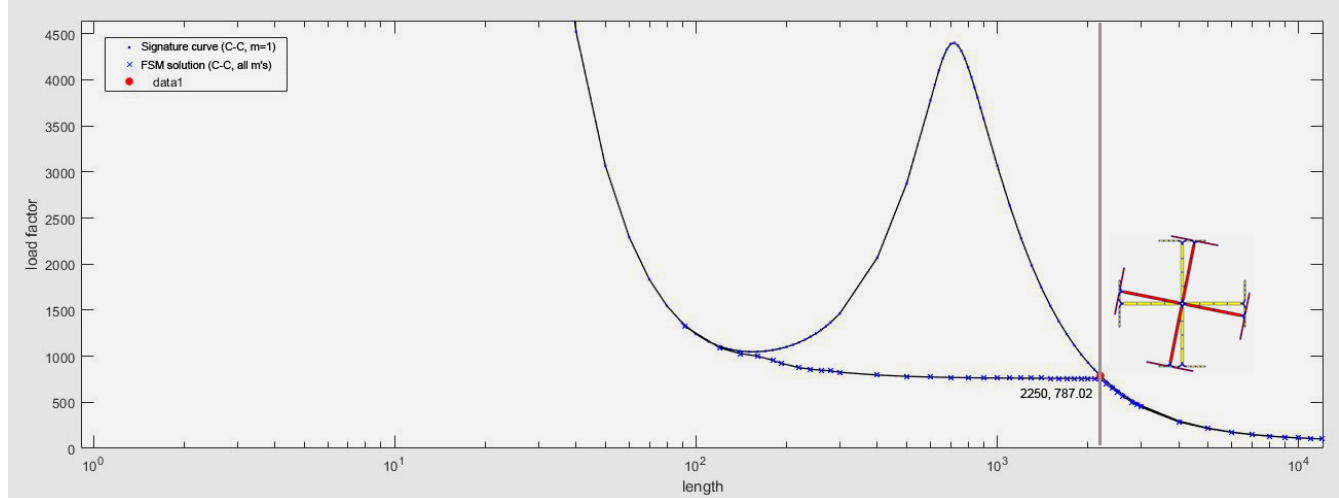
1 global buckling modes. To resolve this, the constrained finite strip method was employed to dissect the contribution
 2 of TB mode along the signature curve; the half-wavelength for the TB mode is found to be 2250 mm, as illustrated
 3 in Fig. 2c.



(a) Signature curve for lipped equal-leg angle section 50×135×3 mm



(b) Signature curve for built-up flanged cruciform section presuming full composite action among individual angles



(c) Signature curve and finite strip method solution for built-up flanged cruciform section considering fixed boundary conditions and full composite action between individual angles

Fig. 2 CUFSM Elastic Buckling Analysis.

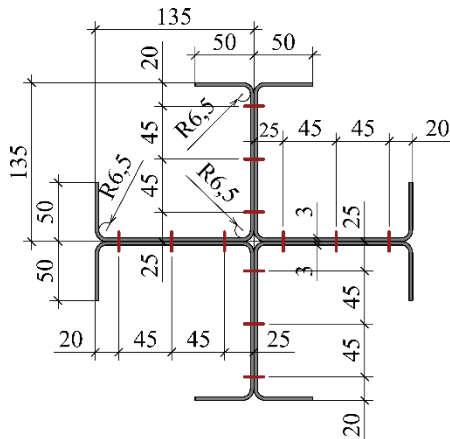
4
 5 In the intermediate slenderness region of built-up cruciform section columns, there is a smooth transition from the
 6 pure LB mode to a combined TB and LB mode (or pure TB), depending on the fastener arrangement [42]. To capture
 7 this structural behaviour, the test matrix was developed drawing insights from the results obtained through CUFSM
 8 [46]. Four specimens were designed in the intermediate slenderness region, with a length of 2250 mm and different
 9 fastener spacings, see Fig. 3. For all specimens, bolt hole-to-end distances of 25 mm were chosen, facilitating
 10 consistent spacings of 50 mm along the component length (i.e. lower than the half-wavelength for LB of individual
 11 angle components). To begin with, the individual components were completely interconnected along their entire
 12 length, as illustrated in Fig. 3c. Afterwards, to investigate how fastener spacing influences the instability mode,

fasteners were set in every alternate hole with a spacing of 100 mm along the component length and/or cross-section Tee-leg width in the second, third, and fourth specimens (the fastener spacing of 100 mm exceeds the half-wavelength for LB of individual angle components); this arrangement excluded the ends of the columns, as illustrated in Fig. 3d-f.

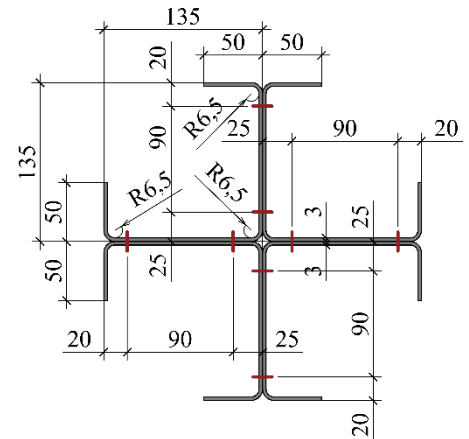
Furthermore, the fastener spacings satisfy the inequality $(a/r_i) \leq 0.5(KL/r)$ [2], where (KL/r) represents the overall slenderness of the built-up section relative to the axis of the built-up member, assuming that full composite action is achieved. Here, a denotes the longitudinal centre-to-centre distance between adjacent rows of fasteners, while r_i is the minimum radius of gyration for the entire unreduced cross-sectional area of a single section. Finally, K is defined as the buckling length factor.

End Fastener Groups (EFGs) were used on the second and fourth specimens; each EFG includes three fastener rows placed at 50 mm spacings (the EFG length is 100 mm). Note that the geometry of the EFG does not meet the requirements of AISI S100 [2]: the length of the EFG should be equal to 1.5 times the maximum dimension of the built-up cross-section (in this case, it is 405 mm) with longitudinal fastener spacing not larger than 4 times the fastener diameter (in this case it is 32 mm). Our approach aims to capture the effect of fastener row numbers at the column ends on the column's failure mode and strength.

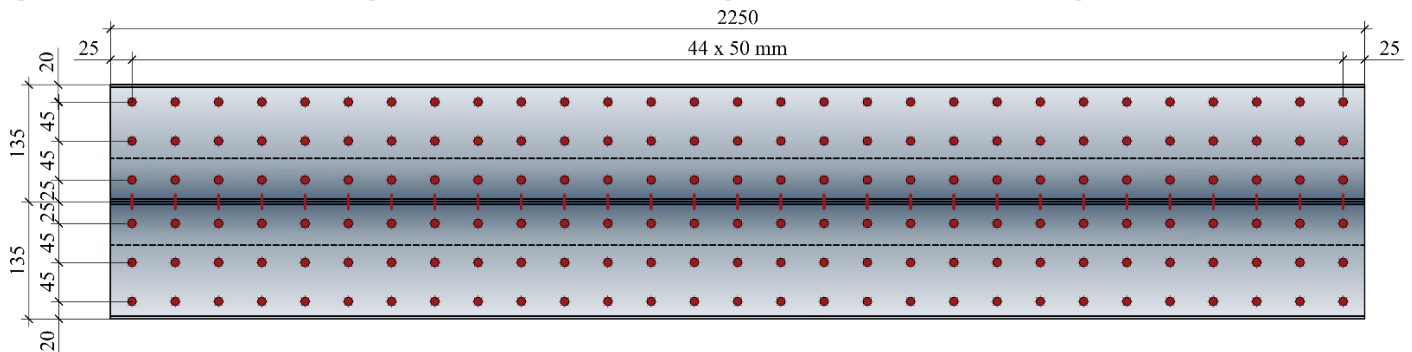
The labelling format for the test specimens follows the convention "N-4L50×135×3-2250-a-b", where "N" denotes the specimen's sequence number, the label "4L50×135×3" specifies the amount and dimensions of the individual angle components that make up the built-up cross-section ("lip width"×"leg width"×" wall thickness"), the number 2250 represents the length of the built-up column in mm, the letter "a" indicates the centre-to-centre spacing between two adjacent interconnections along the column length, whereas "b" denotes the centre-to-centre spacing between two adjacent interconnections along the cross-section Tee-leg width.



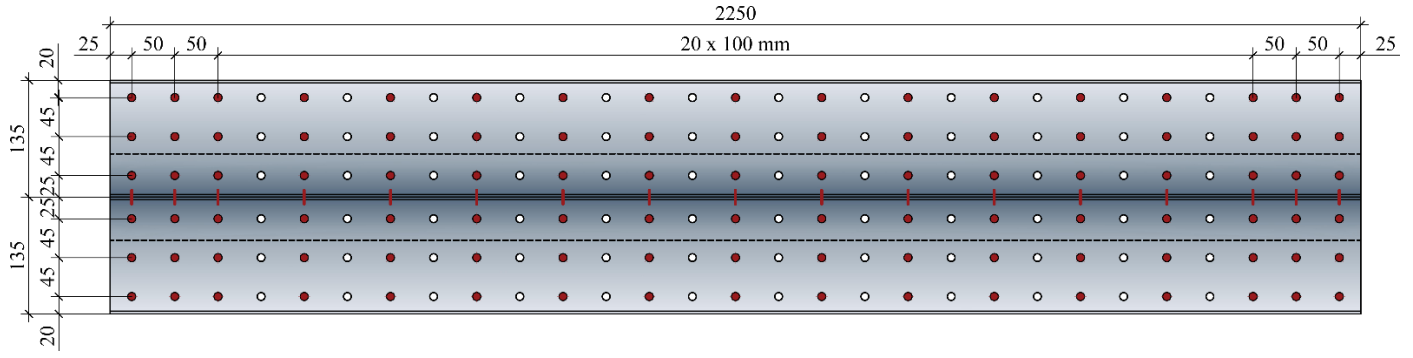
(a) Cross-section dimensions and fastener arrangement for specimens 1 and 2, at the bolt-hole positions



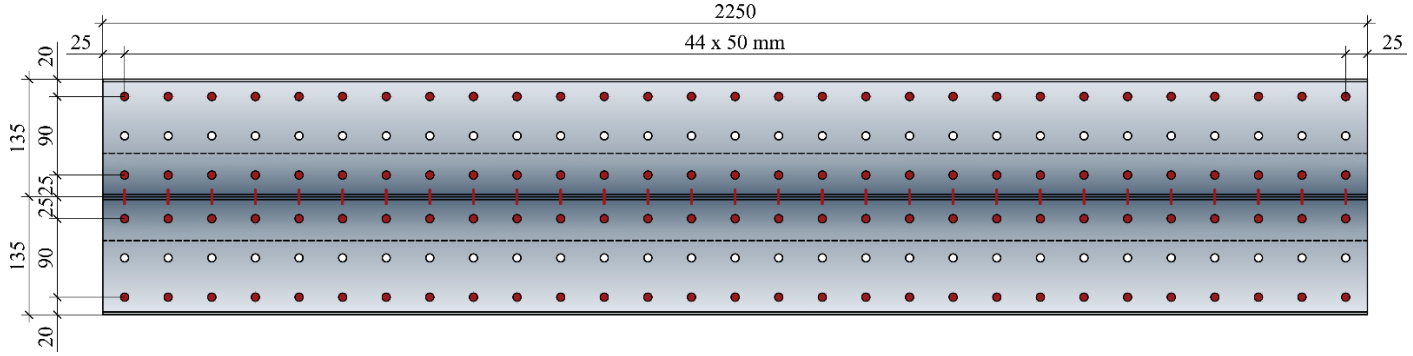
(b) Cross-section dimensions and fastener arrangement for specimens 3 and 4, at the bolt-hole positions



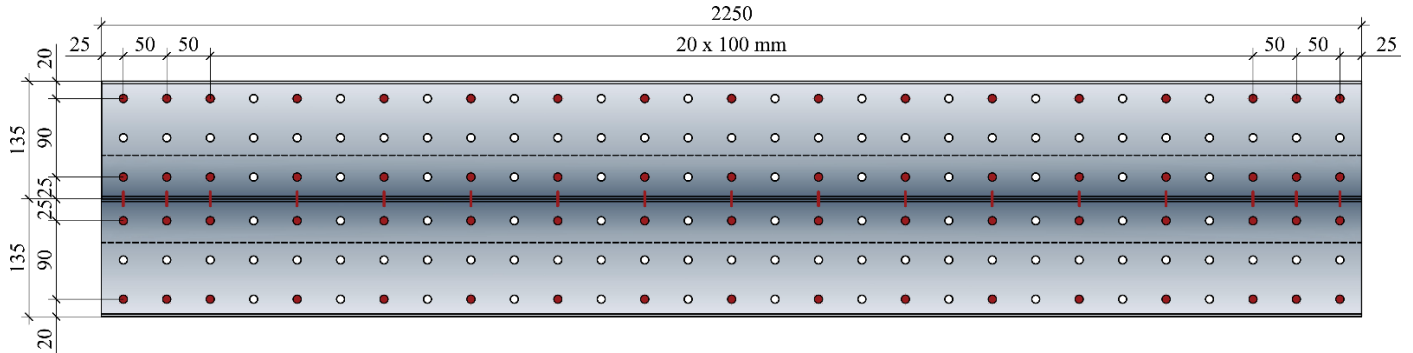
(c) Fastener arrangement for built-up flanged cruciform specimen 1-4L50×135×3-2250-50-45



(d) Fastener arrangement for built-up flanged cruciform specimen 2-4L50×135×3-2250-100-45



(e) Fastener arrangement for built-up flanged cruciform specimen 3-4L50×135×3-2250-50-90



(f) Fastener arrangement for built-up flanged cruciform specimen 4-4L50×135×3-2250-100-90

Fig. 3 Geometric characteristics of tested specimens (all dimensions are in mm).

The mechanical properties of the structural austenitic stainless steel grade EN 1.4301, used in the production of the column specimens, were experimentally determined through strain-controlled tensile tests, conducted on a Shimadzu AG-Xplus Universal Testing Machine with a maximum capacity of 300 kN. Flat and corner dog-bone coupons, extracted from the final press-braked angle sections, were tested according to the criteria outlined in EN ISO 6892-1 [48]. The key material properties are summarised in Table 1 as follows: E is the modulus of elasticity, f_y is the 0.2% proof stress, $\sigma_{0.01}$, $\sigma_{0.05}$, $\sigma_{1.0}$ are the proof stresses related to 0.01, 0.05 and 1.0% respectively, f_u is the ultimate tensile strength, ϵ_u is the strain at the ultimate tensile strength, ϵ_f is the total strain at fracture, while n and m are the strain hardening exponents used in the Ramberg–Osgood model for nonlinear metallic materials [49].

Table 1 Summary of the average material properties obtained from tensile coupon tests.

Stainless steel grade	Coupon	f_y (N/mm ²)	$\sigma_{0.01}$ (N/mm ²)	$\sigma_{0.05}$ (N/mm ²)	$\sigma_{1.0}$ (N/mm ²)	f_u (N/mm ²)	E (N/mm ²)	ϵ_u (%)	ϵ_f (%)	Strain hardening parameters	
										n	m
EN 1.4301	Flat	316.9	211.1	264.2	349.1	629.1	198445	49	56	8.3	2.2
	Corner	457.9	252.8	342.7	520.1	688.3	194540	37	45	4.9	2.6

To measure the geometric imperfections of specimens, the photogrammetric method was used (see Fig. 4). Photographs were captured using a Canon EOS 6D camera under optimal lighting conditions, positioned 1 meter

away from the specimen surfaces. Prior to taking the photographs, non-reflective measuring control targets were affixed to the columns to establish a scale reference. A total of 145 photographs were taken on each longitudinal side (wall) of the specimen, strategically spaced along its length and height. Both orthogonal and oblique camera angles were considered, ensuring at least 90% photo overlap in the layout.

The captured images were then processed using Agisoft Metashape software [50] to create a 3D point cloud for each specimen wall. This process included photo alignment and point cloud generation, all executed with ultra-high processing parameters. Each resulting model, comprising 3 to 4 million points, was scaled using the control targets, with measured discrepancies between the targets and the lengths derived from the point cloud being less than 0.2 mm. To model the imperfections, a plane was fitted to each model surface and the distance of normal points from it was calculated.



Fig. 4 The photogrammetric method used for measurement of specimen imperfections.

Geometric data specific to a set of cross-sections along the length of the specimen was collected in relation to the global coordinate system, which allowed for the classification of the measured imperfections into four distinct types as illustrated in Fig. 5 (also indicating the positive sign convention utilized). The local imperfections were assessed by determining the perpendicular distances from the relevant ideal plane to the measured section wall. The maximum deviation was utilized as the local imperfection amplitude, specifically denoted as d_{f0} for the flange distortion d_f , and d_{w0} for the web buckling d_w . The flexural imperfections oriented perpendicular to the $y - y$ axis (bow imperfections δ_y with an amplitude of δ_{y0} at mid-span), along with the imperfections oriented perpendicular to the $z - z$ axis (camber imperfections δ_z with an amplitude of δ_{z0} at mid-span), were measured by locating the centroid of each evaluated cross-section and comparing it to the nominal section. It was presumed that the centroids of the end cross-sections and the nominal centroids align. Therefore, both bow and camber imperfections are relative to the corresponding idealized planes passing through the end cross-sections' centroids. The twist imperfections, referred to as θ , and the twist angle of the entire specimen were evaluated by establishing a mid-span cross-section as the null reference section. The twist angle, defined as the differential rotation between the two ends, was utilized to determine the maximum twist imperfection, indicated as θ_0 . The data summarizing the measured imperfection amplitudes at mid-span are presented in Table 2.

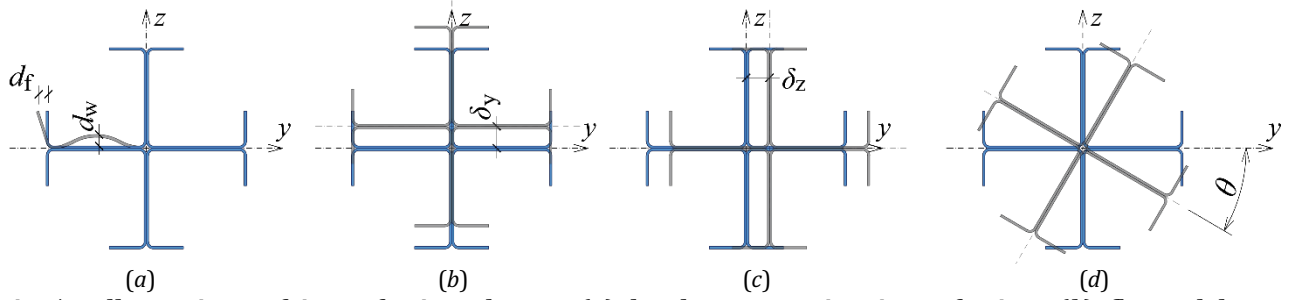
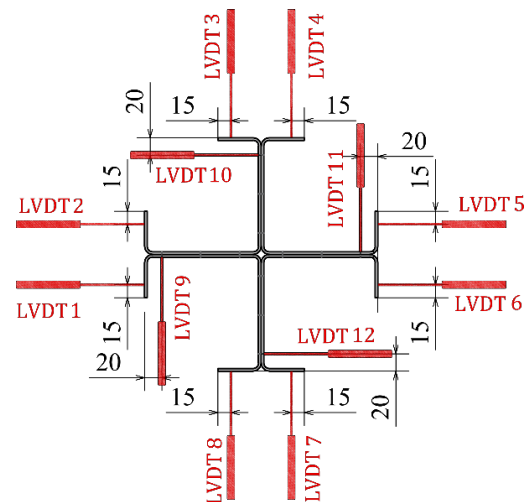


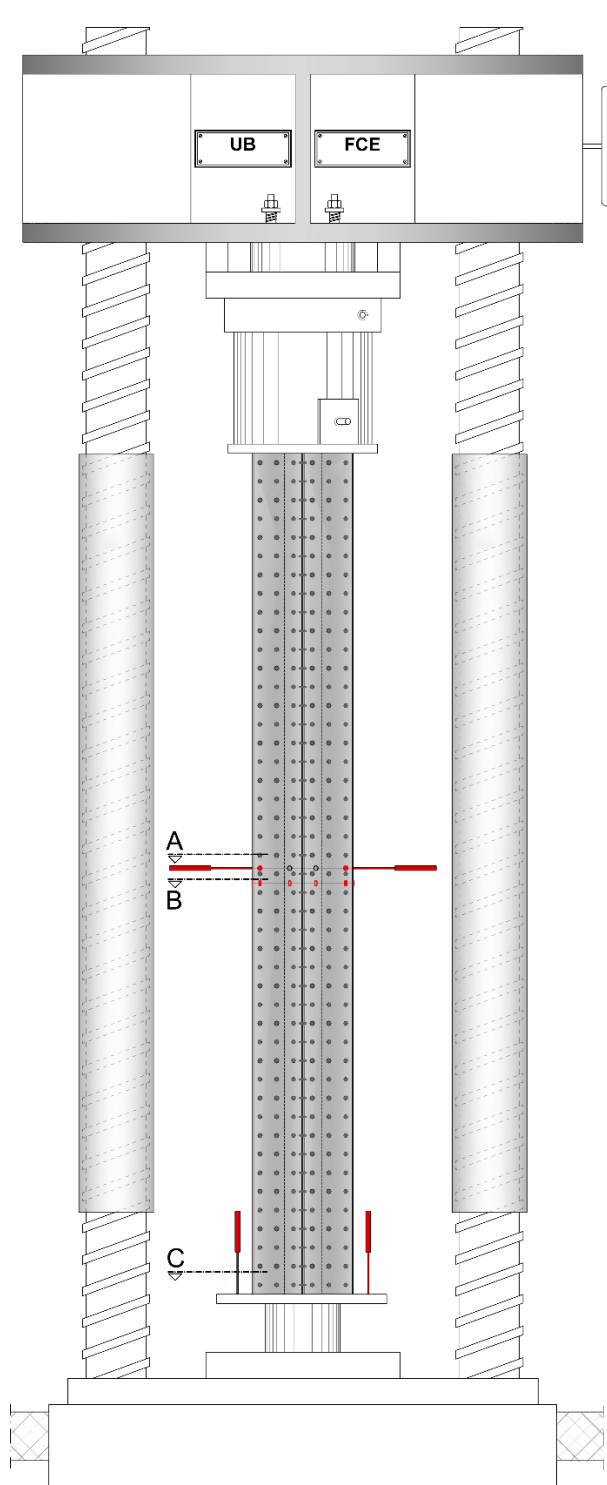
Fig. 5 Illustrations of imperfection shapes: (a) local cross-section imperfection; (b) flexural bow imperfection; (c) flexural camber imperfection; (d) twist imperfection (positive clockwise).

Table 2 Amplitudes of measured geometric imperfections.

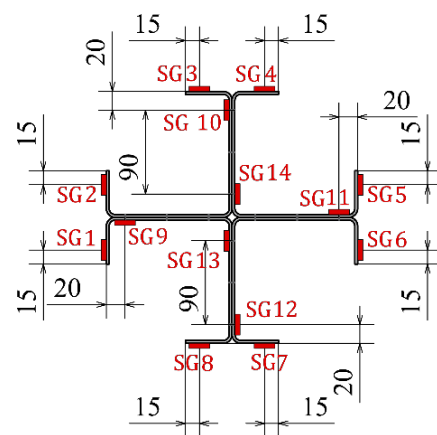
Test #	d_{f0} (mm)	d_{w0} (mm)	δ_{y0} (mm)	δ_{z0} (mm)	θ_0 (°)
1	0.567	0.135	0.894	1.004	0.986
2	0.958	0.101	-0.548	-0.481	-0.558
3	-0.498	0.202	0.924	0.785	0.899
4	0.325	0.255	0.625	0.324	0.444

The column buckling tests were performed with an Amsler 2000 kN hydraulic testing machine under displacement control. Fig. 6 presents schematic drawings of the test setup, including the orientation and position of the specimen. The specimens were secured within sliding steel clamps and positioned on the testing machine loading plates. This setup provided partial rotational restraint at the column ends, allowing for relative end shear deformations between the individual angle components. This underscores the key function of the fastener rows at the column ends. Each specimen was geometrically aligned to ensure the longitudinal axis was collinear with the load application, with positional measurements taken in both $y - z$ axes. Throughout the tests, lateral bi-planar displacements and the twist deformations of the built-up column were monitored using 12 linear variable displacement transducers (LVDTs) strategically placed at mid-height on each Tee-leg flange and web (see Fig. 6, section A-A). Additionally, 14 linear strain gauges (SGs) were affixed at mid-height to capture axial compressive strains at the cross-section extreme fibres (see Fig. 6, section B-B). Four LVDTs were installed on the upper platen of the testing machine to monitor end shortening (see Fig. 6, section C-C). A calibrated load cell, Force Transducer C6A, was utilized to measure the applied loads during testing. Data was acquired through a system that recorded readings from the load cell, LVDTs, and SGs. After ensuring geometric alignment, all tests were performed under a controlled displacement regime, with a maximum loading rate limited to 0.01 mm/s until reaching the maximum load capacity. Tests were continued in the post-buckling regime, maintained at approximately 70% of the maximum load attained before cessation.

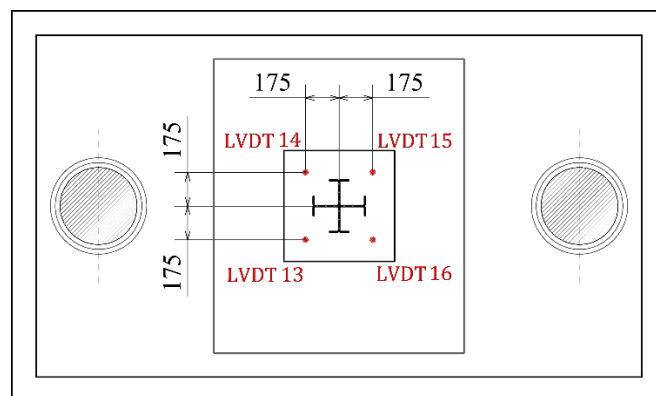




Section A-A



Section B-B



Section C-C

Fig. 6 Schematic drawings of test setup and instrument configuration (all dimensions in mm).

2.2 Test results and discussion

The specimen buckling patterns with the plastic yield line mechanisms, corresponding to the post-buckling regime at approximately 70% of the post-peak load, are illustrated in Fig. 7. Additionally, Fig. 7 includes numerically simulated counterparts (see Section 3.1) that correspond to the same stress state.

Table 3 provides key test results observed at the ultimate state: the ultimate buckling resistance — $P_{b,u,exp}$, the ratio of the buckling resistance to squash load — $P_{b,u,exp}/P_y$, the lateral deflections at mid-height concerning the minor and major principal (and geometric) axes — $d_{u,y}$ and $d_{u,z}$, and the twist rotations at mid-height concerning the centroid of the built-up cross-section — φ_u . The value of the squash load P_y is calculated by multiplying the nominal cross-sectional area by the measured average yield stress (see Table 1). The table also provides values for the elastic critical load associated with LB denoted as $P_{cr,LB}$, as well as the elastic critical load for TB indicated as $P_{cr,TB}$. These values were derived using LBA performed with the FE software ABAQUS [47].

The lateral displacements recorded at the mid-height cross-section by LVDTs were converted into bi-planar displacements within the y – z plane, as well as twist rotations around the longitudinal axis of the column. This conversion followed the simplified mathematical model established by Landesmann et al [51]. The mid-height lateral displacements are assessed according to the conventions for the y and z axes, while the twist rotations are considered positive when they occur in a clockwise direction. Fig. 8a illustrates both the undeformed and deformed configurations of the built-up section within the specified y – z plane. Positive readings for lateral displacements signify movement away from the initial positions of the LVDTs, whereas negative readings signify movement towards their starting positions, as shown in Fig. 8b. The locations of the LVDTs and SGs on the specimens are detailed in Fig. 6.

Fig. 9 displays the relationship between load and axial shortening for all tests (alongside the corresponding numerical curves). In Annex i, graphs in Fig. 13 to Fig. 20, depicting the relationship between load and transformed cross-sectional displacements are included, as well as graphs depicting the relationship between load and axial strains for each specimen tested.

The use of additional fasteners seems to enhance the composite action, resulting in higher ultimate buckling loads for specimens 1, 2, and 3 compared to specimen 4—specifically, a 6.8% increase (see Table 3). However, it is evident that all specimens display remarkably similar overall behaviours. The primary failure mechanism observed in all specimens was global TB, coupled with LB of the Tee-legs, occurring within the elastic stress range $P_{b,u,exp}/P_y < 1$. There was no visible separation between the component areas in contact. The critical cross-sections exhibited torsional rotations akin to rigid-body motion, coupled with transverse bending of the Tee-leg flanges. Notably, the twist rotations displayed distinct directional characteristics: specimens 1, 3, and 4 experienced negative (counterclockwise) rotations, while specimen 2 showed positive (clockwise) rotations. The deformed configurations, illustrated in Fig. 7, confirm the predominance of twist deformations in a single half-wave mode; this behaviour is corroborated by the relationship between load and lateral displacements in Fig. 13, Fig. 15, Fig. 17 and Fig. 19. Compressive axial strains were observed at mid-height across the entire loading regime. As illustrated in Fig. 14, Fig. 16, Fig. 18 and Fig. 20, the ascending branches of the load-axial strain response exhibited increasing divergence prior to reaching the peak load, alongside rising applied load and torsional displacements. The twisting action at critical cross-sections induced tensile strains at the tips of each flange of the section legs, as evidenced by strain gauges SG1, SG3, SG5, and SG7 for specimens 1, 3, and 4, and SG2, SG4, SG6, and SG8 for specimen 2. The plastic hinge mechanism predominantly developed in the compression flanges at the critical mid-height cross-

section, although this phenomenon also manifested at various locations along the lengths of the specimens; the mechanism became more pronounced beyond the peak load regime.

As presented in Table 3, the variation in fastener spacing along the column length—from 50 mm in specimen 1 to 100 mm in specimen 2—demonstrates a negligible effect on the ultimate load capacity, with differences remaining under 1%. In specimen 2, LB is influenced not only by the transverse bending of the Tee-leg flanges but also by the transverse bending of the Tee-leg webs between transverse fastener rows. Notably, the elastic critical load for LB in specimen 2 is 5% lower compared to specimen 1.

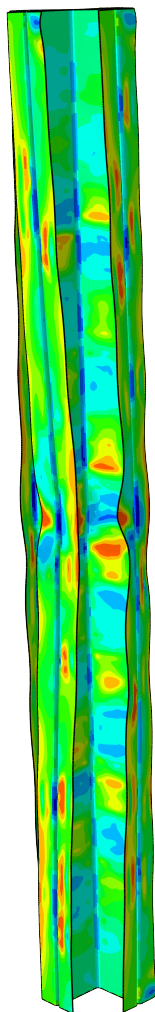
Additionally, specimen 3, which lacks longitudinal fastener restraints in the middle of the Tee-leg, exhibits failure load values closely comparable to those of specimen 1, indicating that the absence of these restraints does not significantly affect their structural responses (note that the longitudinal distance between interconnections is smaller than the LB half-wavelength of the cross-section walls for both specimens). Additionally, the elastic critical load for LB in specimen 3 shows negligible reduction compared to specimen 1. Referring to the work of Rasmussen et al. [15], this study demonstrates that uniform torsion in open sections can occur without inducing fastener shear deformations. Thus, the contributions of each individual component can be accounted for in the estimation of the torsional stiffness of the built-up section.

The failure mechanism observed in specimen 4 (which lacks every other transverse fastener row and the middle longitudinal row on the Tee-legs) exhibited significant LB across the entire cross-section. The behaviour accounts for both the transverse bending of the Tee-leg flanges and the webs between the fastener restraints (see Fig. 7d). Specimen 4 demonstrated a noticeable lower ultimate strength, with its elastic critical load for LB being 9% lower than that of specimen 1.

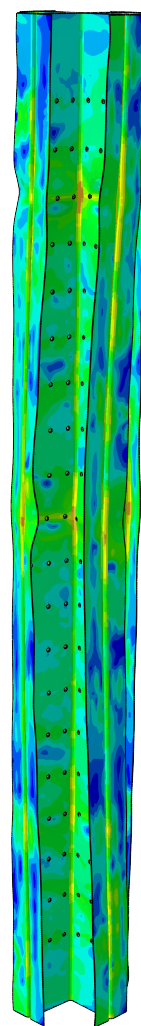
The load-axial strain curves for specimen 4 diverge from those of the other specimens and show almost twice less axial displacement at peak load (see Fig. 9). This behaviour stems from the fastener configuration within the selected intermediate slenderness regime, which triggers a transition from the pure LB mode to a pure TB mode, with both becoming prominent and interacting. Specifically, the structural response of the built-up cruciform columns with the selected length and cross-section geometry is significantly affected by the rotational restraints associated with the slenderness and inherent flexibility of each leg. This emphasises the critical role of fastener placement along the column length and across the Tee-leg width in maintaining the structural integrity of built-up columns subjected to twisting. As evidenced by comparisons of the elastic critical forces for TB, the arrangement of fasteners is a determining factor in the overall torsional stiffness of cruciform section columns. Hence, it is imperative to ensure that the spacing between fastener rows is less than the LB half-wavelength of the cross-section plate walls, as illustrated in Fig. 2a.

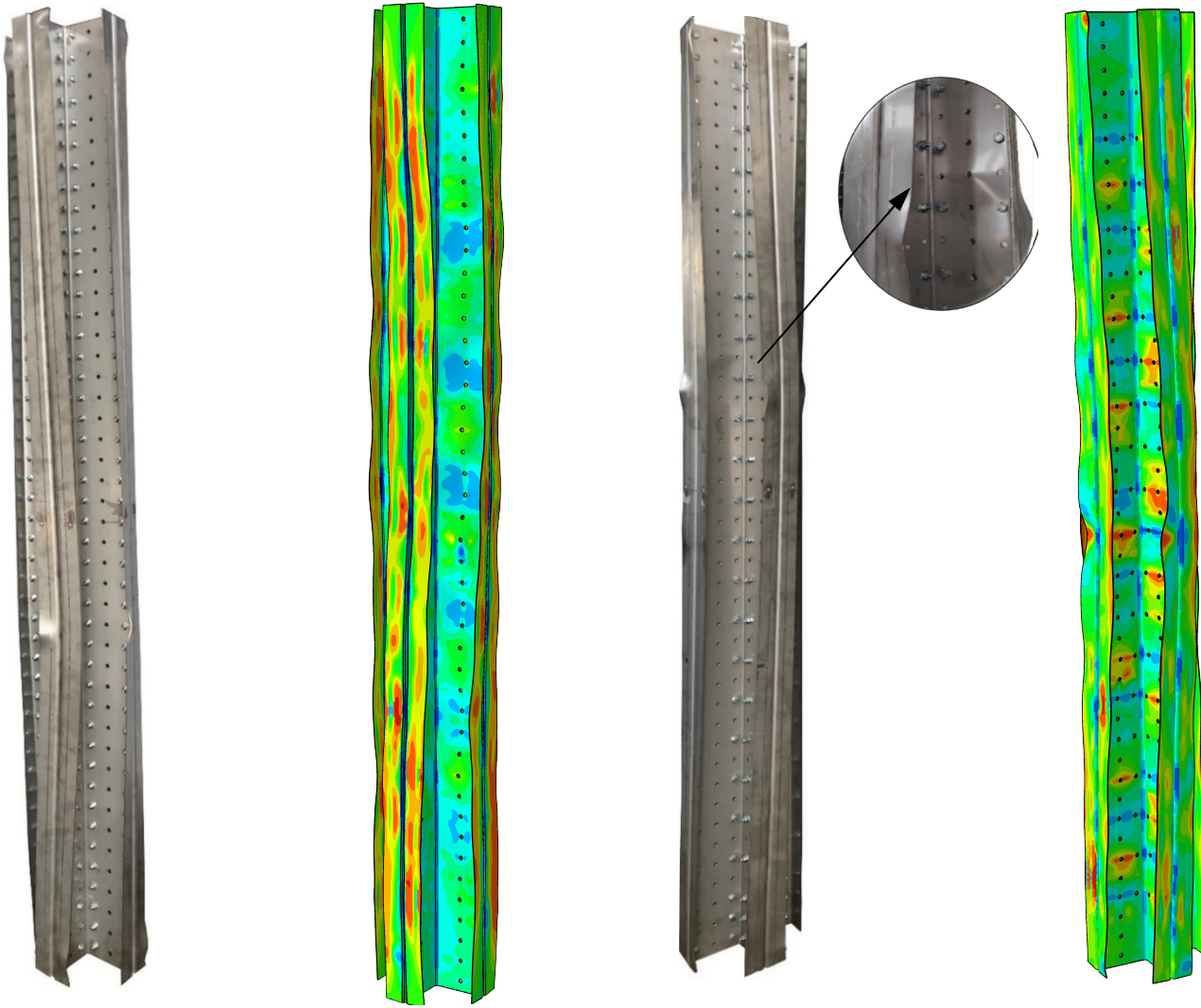


(a) 1-4L50×135×3-2250-50-45



(b) 2-4L50×135×3-2250-100-45





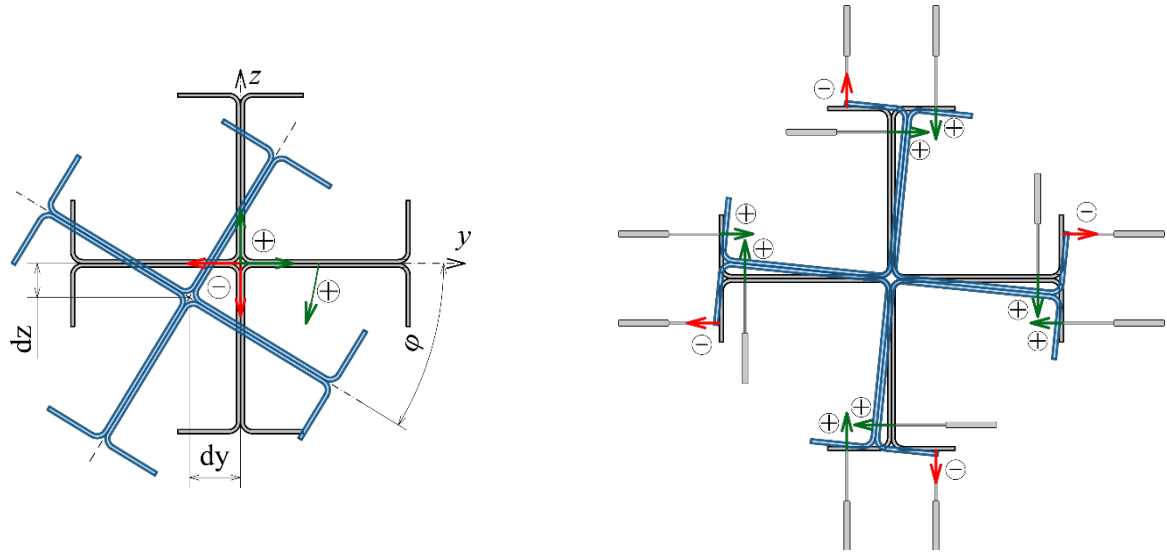
(c) 3-4L50×135×3-2250-50-90

(d) 4-4L50×135×3-2250-100-90

Fig. 7 Failure mode of specimens and their numerical counterparts (deformed patterns in the post-buckling regime at approximately 70% peak-load).

Table 3 Summary of buckling test results and critical buckling loads for LB and TB obtained in LBA.

Test #	Governing failure mode	$P_{b,u,exp}$ (kN)	$d_{u,y}$ (mm)	$d_{u,z}$ (mm)	ϕ_u (deg)	$P_{b,u,exp}/P_y$	$P_{cr,LB}$ (kN)	$P_{cr,TB}$ (kN)
1	TB+LB	863.72	-0.135	-0.349	5.066	0.65	1751.6	2870.4
2	TB+LB	861.62	0.323	-0.240	-3.527	0.65	1666.4	2761.7
3	TB+LB	863.21	0.187	-0.395	3.727	0.65	1750.8	2873.7
4	TB+LB	808.72	0.269	-0.269	3.352	0.61	1604.2	2702.3



(a) Sign convention for lateral deflections and torsional rotations with respect to the y - z coordinate system (b) Sign convention for lateral deflections with respect to the initial LVDT positions

Fig. 8 Schematic representation of the cross-section in its undeformed and deformed position.

3 Finite element study

The FE software ABAQUS [47] was employed to develop numerical models that replicate the tests outlined in Section 3.2, and predict aspects of the specimen structural behaviour. The validation of the models is leveraged by previous experience with analogous FE modelling [41], [42], which is detailed in Section 3.1.

An FE parametric study aimed at investigating the buckling failure modes of short to intermediate length CFSS built-up flanged cruciform section columns is presented in Section 3.2. The study considers three cross-section geometries, different fastener arrangements and pin-ended boundary conditions. The outcomes pertaining to the columns' ultimate structural responses are discussed in Section 3.3. The computed ultimate buckling loads are employed to assess the applicability of two recent design proposals: the DSM-based procedure [15], [20], [21], and the modified Eurocode 3 procedure [40].

3.1 Modelling and validation

The FE modelling assumptions used in research [41], [42] were employed in this analysis. The four-node shell element S4R, featuring hourglass control and reduced integration, was selected from the ABAQUS element library to represent the measured geometries of the individual components in the built-up specimens accurately. After evaluating various mesh sizes, a 4 mm square mesh was chosen for discretizing both the flat and corner regions of the cross-sections, balancing result accuracy with convergence efficiency. In the bolt point contacts, where the bolts are represented as linear springs, a refined mesh size of 1 mm was implemented within a circular region of 10 mm in diameter. The same mesh refinement was adopted around the empty bolt holes in the Tee-legs of the FE models for specimens 2, 3, and 4 being explicitly modelled.

The measured stress-strain curves from the flat and corner tensile coupon tests were integrated into the FE models. Notably, the corner material model was constrained to the corner region, as previous research [52] demonstrated that there were no significant strength enhancements beyond the curved areas of the press-braked sections. The true stress-strain curves, converted from the measured engineering stress-strain curves, were integrated into the ABAQUS plasticity model. A modulus of elasticity of $E = 200,000$ MPa and a Poisson's ratio of $\nu = 0.3$ were assumed in all cases.

In the modelling of the bolts, linear springs were employed through a mesh-independent connector element, integrating the shear properties of the bolt connections derived from tests outlined in [41]. To account for the bolt physical radius, kinematic coupling was implemented for all nodes situated on the radius, effectively linking them to the endpoints of the connector elements.

The measured geometric imperfections were used in the validation process. To account for these imperfections, the eigenmode shape derived from LBA was incorporated into the corresponding FE model, scaled appropriately to match the absolute maximum amplitude measured.

To replicate the supporting conditions of the tested specimens, the loading plates of the testing machine were represented as 2D rigid bodies with reference points established at their centroids. The slave nodes were permitted to translate only along the longitudinal axis of the column, with movement restricted to one end where the failure load was applied as a controlled nodal displacement (set at the reference point). A surface-to-surface general contact formulation was employed to accurately capture the interactions between the ends of the columns and the loading plates of the testing machine, as well as between the adjacent surfaces of individual components. For these interactions, the normal behaviour was modelled using a hard contact formulation, while a penalty-based friction formulation governed the tangential behaviour. A uniform friction coefficient of 0.35 was applied across all contact interfaces, and this value was calibrated based on experimental data.

The geometrically and materially non-linear analysis with imperfections (GMNIA) was conducted as a quasi-static simulation. To improve computational efficiency, a dynamic explicit solver was utilized alongside the variable non-uniform mass scaling technique, with a time increment set to 5×10^{-6} s. The explicit dynamic method in Abaqus generates a smooth loading amplitude, thereby mitigating excessive inertial forces during the quasi-static analysis.

Table 4 compares the computed ultimate loads $P_{b,u,FEM}$ to the experimental ones $P_{b,u,exp}$. The ratio of numerical to experimental ultimate load averages 1.001, with a coefficient of variation of 0.561%. To facilitate a comparative analysis between the FE and experimental results, the axial load is plotted against end shortening in Fig. 9, while the numerically observed failure modes are illustrated in Fig. 7. These comparisons indicate a reasonable correlation between the FE and experimental results, thereby affirming the validity of the FE models for conducting subsequent parametric studies.

Table 4 Comparison between numerical and experimental ultimate loads.

Test #	$P_{b,u,FEM}$ (kN)	$P_{b,u,FEM} / P_{b,u,exp}$
1	867.5	1.005
2	863.1	1.002
3	866.7	1.003
4	802.7	0.992
Mean		1.001
Coefficient of variation (%)		0.561

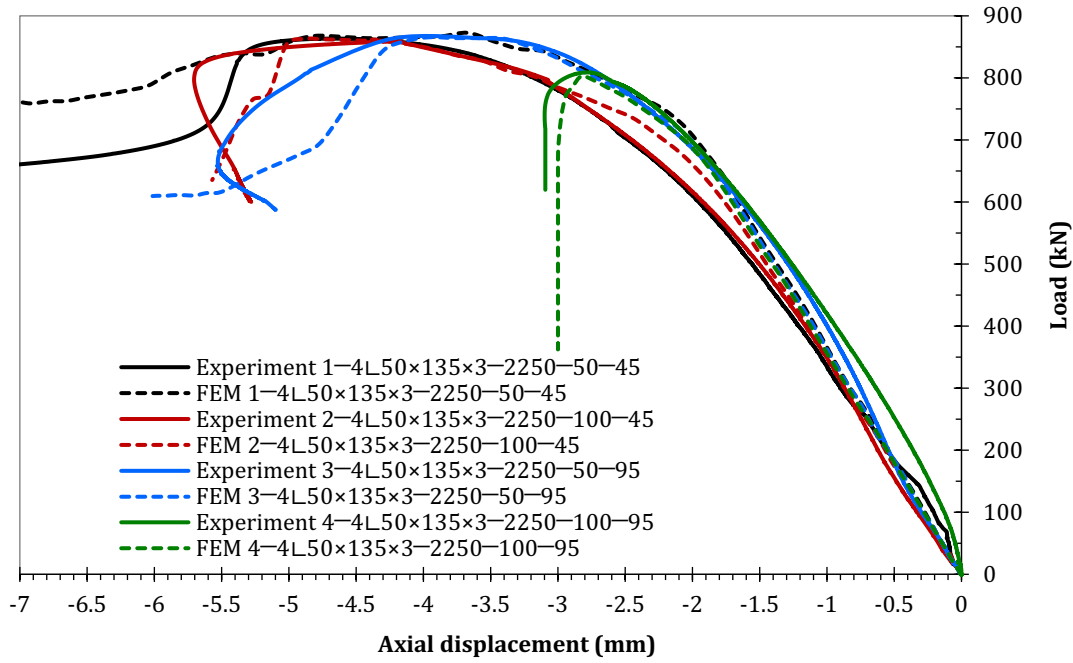
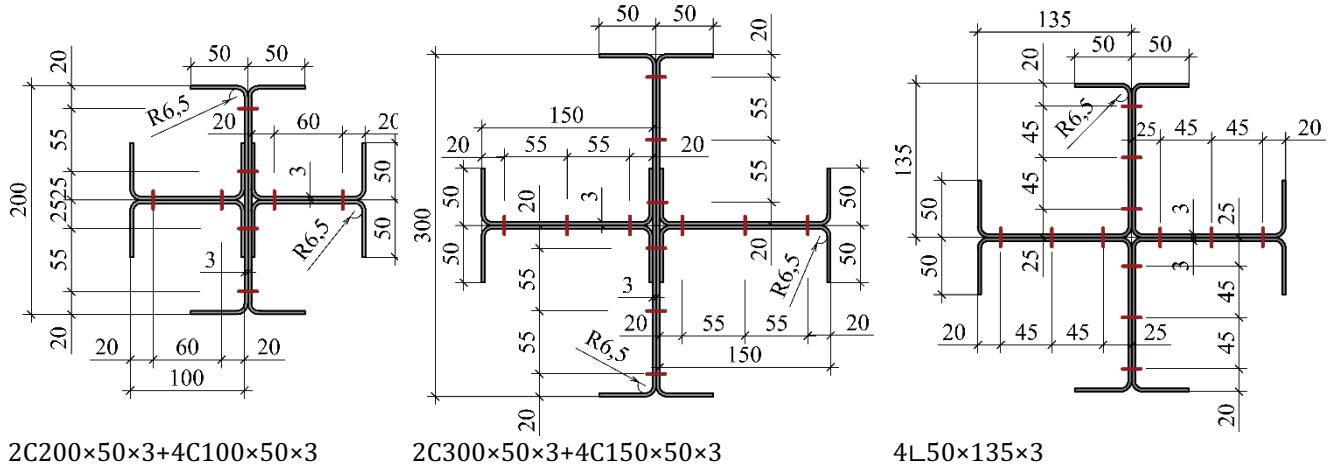


Fig. 9 Load versus axial shortening – comparisons between FEM and experiment.

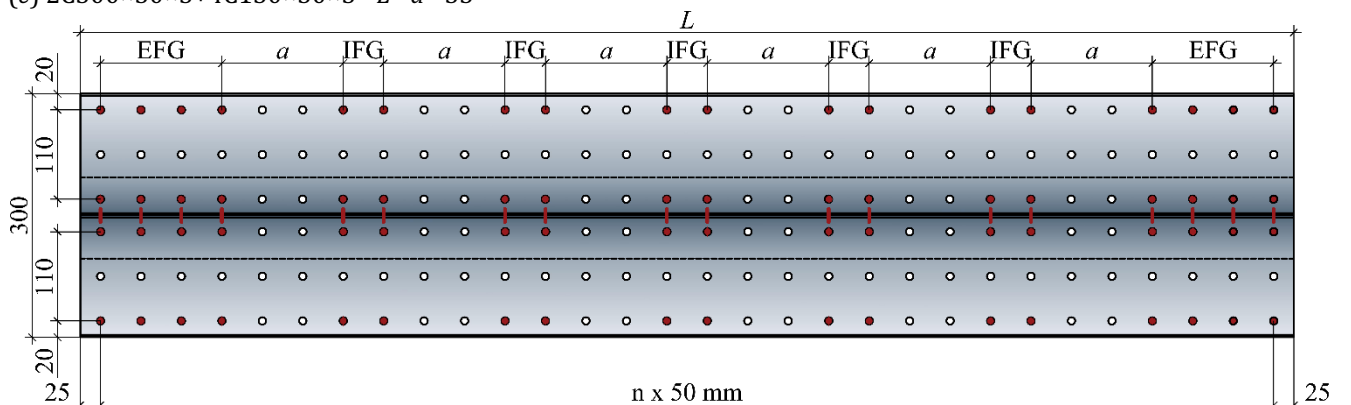
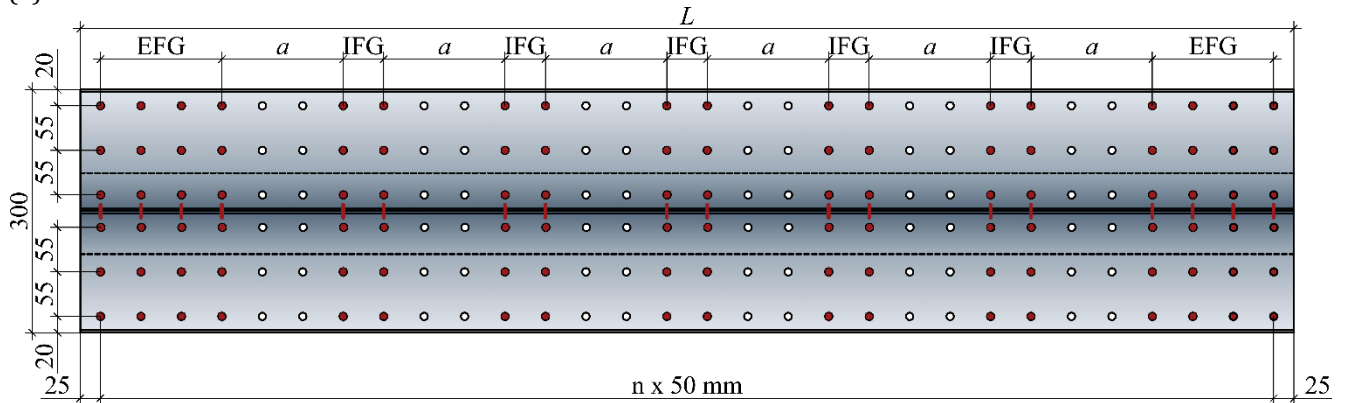
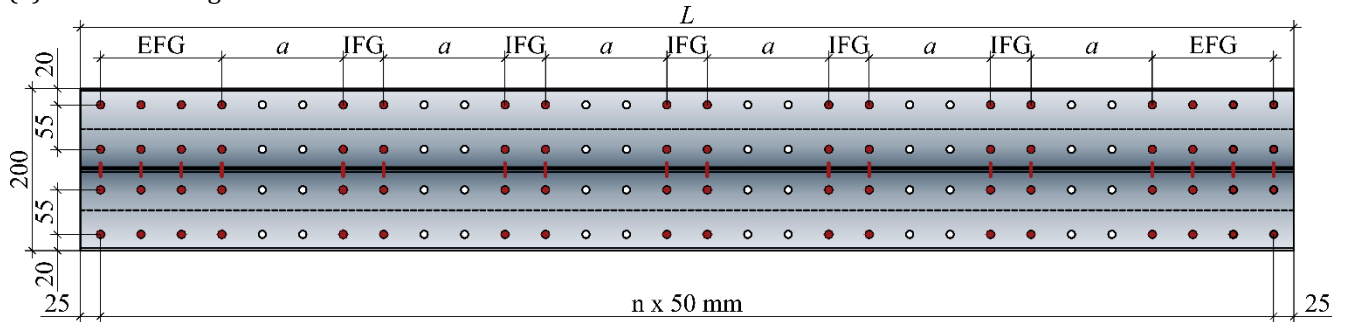
3.2 Parametric study

In order to generate data on the buckling failures and corresponding ultimate resistances of cold-formed built-up flanged cruciform columns, a wide-ranging parametric study was conducted utilizing the validated FE models. Based on the geometries described in Section 2 and in the recently published research [42], the study encompasses a variety of column lengths spanning from short to intermediate where the governing failure modes are LB or/and TB.

The specific role of the fastener arrangement has been the focus of the study. Three cruciform section geometries were considered: (i) configuration using six plain channels with nominal dimensions $100 \times 50 \times 3$ mm and $200 \times 50 \times 3$ mm, denoted as $2C200 \times 50 \times 3 + 4C100 \times 50 \times 3$ [42], (ii) the same cross-section configuration, but with channel geometry $150 \times 50 \times 3$ mm and $300 \times 50 \times 3$ mm, denoted as $2C300 \times 50 \times 3 + 4C150 \times 50 \times 3$, (iii) configuration using four lipped equal-leg angles with nominal dimensions of $50 \times 135 \times 3$ mm (as the tested built-up specimens), denoted as $4L50 \times 135 \times 3$. All cross-sections are classified as slender, i.e. Class 4 [44]. The individual components were interconnected using M8 bolts, arranged along the column length (transverse fastener rows) and across the Tee-leg width (longitudinal fastener rows). In addition to the column geometries, the following interdependent influencing parameters were examined in the study: the number of fastener rows within the EFGs and the Internal Fastener Groups (IFGs), the centre-to-centre spacing, denoted as “ a ”, between two adjacent fastener connections along the column length and the centre-to-centre distance spacing, denoted as “ b ”, between two adjacent fastener connections across the Tee-leg width (for columns $2C300 \times 50 \times 3 + 4C150 \times 50 \times 3$, $b_1 = 55$ mm, $b_2 = 100$ mm; for columns $4L50 \times 135 \times 3$, $b_1 = 545$ mm, $b_2 = 90$ mm), as shown in Fig. 10.



(a) Cross-section geometries



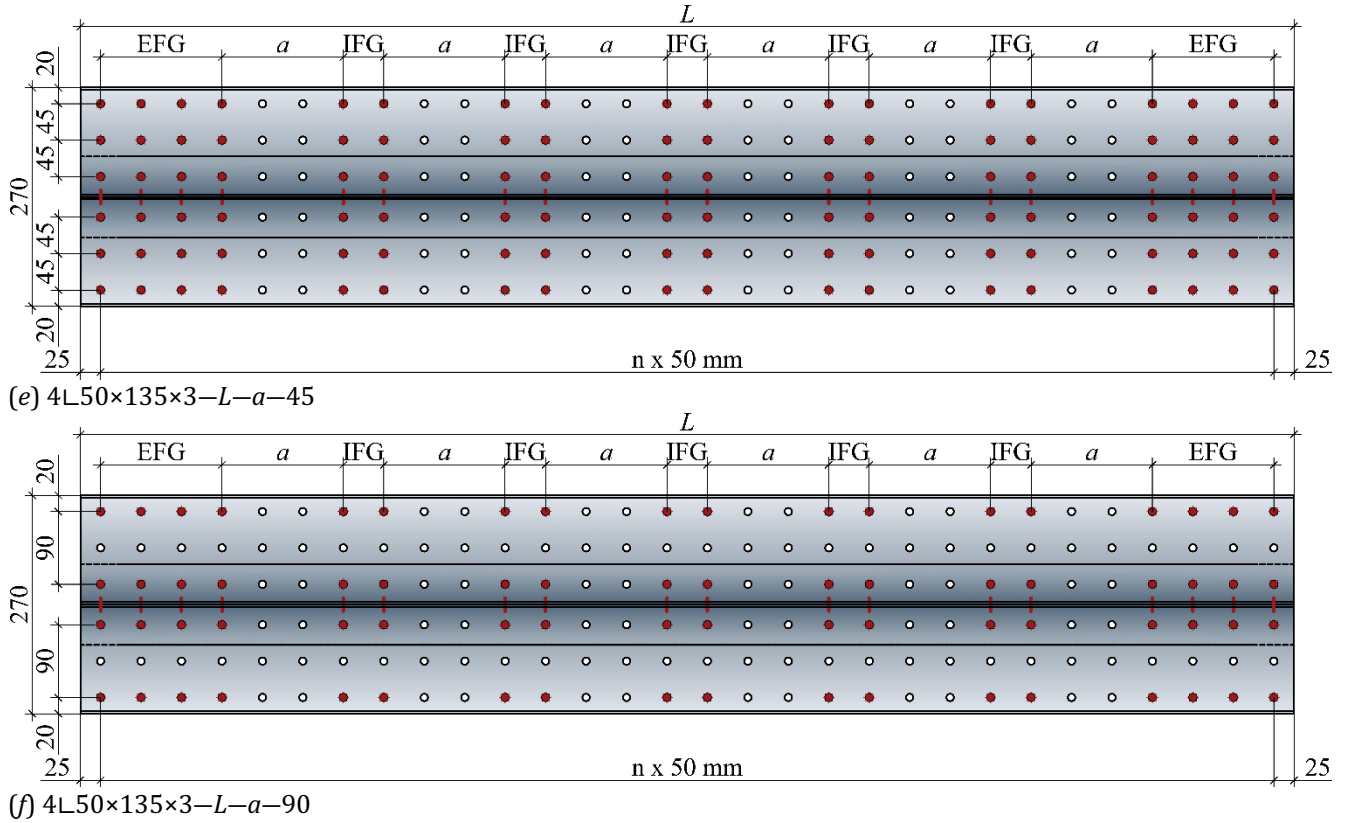


Fig. 10 Geometric characteristics and parameters designation of FE models in the parametric study (all dimensions in mm).

All FE models include: (i) non-linear material stress-strain relationship based on the two-stage Ramberg–Osgood material model [49], utilized in conjunction with standardized material properties specifically designed for both flat and corner regions of press-braked sections made from austenitic stainless steel grades, as outlined in [53], (ii) pinned boundary conditions at both column end cross-sections (with warping and torsional rotations prevented) are modelled using kinematic coupling constraints that link their rotational and translational degrees of freedom to reference points located at the centroid of the end section, (iii) initial geometric imperfections combining a critical local cross-sectional component with an amplitude of $c/125$ (c is the Tee-leg width) [54], torsional (twist) component with an amplitude of $\tan^{-1}(L/1000c)$ at the column mid-height [40] and critical flexural component about both principal axes with an amplitude of $L/1000$ at the column mid-height [55]. In each FE model, a combination of initial geometric imperfections was incorporated. Depending on the slenderness of the column and cross-section, one of the imperfections was selected as the primary one, while the less significant imperfections were reduced by a factor of 0.7, as per [55]. A preliminary LBA, performed with the same mesh employed, characterized each buckling mode shape for the subsequent GMNIA.

3.3 Failure modes and ultimate resistances

The analysis of the computed results indicates the following conclusions:

- Depending on the column slenderness and the fastener amount along the column length and across the Tee-leg width, the built-up cruciform section columns failed in either a mode exhibiting predominantly localised cross-section buckling deformations (short column lengths) or a mode combining global twisting deformations about the column longitudinal axis, often accompanied by LB (intermediate column lengths).
- For pin-ended boundary conditions, installing EFGs can significantly enhance the effective torsional (and flexural) rigidity, thereby augmenting their buckling resistances.

- The distance between interconnections and the arrangement of fastener rows in EFGs significantly influences the ultimate resistances of stocky columns. Excessive spacing of fasteners triggers LB, leading to separation and independent responses of individual components. Therefore, it is advisable to maintain fastener spacing below the LB half-wavelength while ensuring EFGs effectively mitigate longitudinal shear deformations between the components.

- The collapse mechanism of intermediate-length columns shifts between LB and TB, with one mode becoming dominant depending on the rotational restraint provided by the Tee-legs. The flexibility of the Tee-legs is influenced not only by their cross-sectional geometry (slenderness) but also by the positioning of the fasteners along the column length and across the Tee-leg width. Consistent with previous findings, the numerical results indicate that a denser arrangement of fasteners—specifically, spacing that is smaller than the LB half-wavelength—positively impacts the column's ultimate resistance, leading to the occurrence of the global TB mode.

- Using more fastener rows in long-length columns failing by global TB failure yields marginal strength improvements; this indicates that the open cross-sections maintain a uniform torsional response without causing significant shear deformations at the fasteners.

4 Design recommendations

Following the provisions outlined in Section I1.2, guidelines for the design of axially compressed built-up columns are addressed in Section E of AISI S100-16 (2020) w/S3-22 [2]. The impact of increased shear deformations between the individual components of the built column is accounted for by reducing the cross-section rigidity used in calculating the elastic critical buckling stress. In earlier editions of the AISI S100 Specification, the discrete characteristics and partial composite action of built-up columns were accounted for through a modified slenderness ratio approach. However, the 2016 edition with Supplement 3 [2] shifts this approach by considering cross-section rigidity reduction through a reduced moment of inertia (mechanics-based approach [15]), as specified in Section I1.2.2.1-1 [2].

Hence, when the global buckling mode involves shear forces in the fasteners between individual components, the moment of inertia I , for the built-up column about the axis of flexural buckling should be replaced by the reduced moment of inertia I_r , to determine the elastic critical global buckling load F_{cre} :

$$I_r = I \frac{(KL/r)^2}{(KL/r)^2 + (a/r_i)^2} \quad (1)$$

where I is the moment of inertia of the built-up column about the axis of FB, KL is the effective length of built-up column for the axis of FB, r is the radius of gyration of the full unreduced cross-sectional area of the built-up column about the axis of FB, a is the centre-to-centre spacing between two adjacent interconnections along the column length, r_i is the minimum radius of gyration of the full unreduced cross-sectional area of an individual section in a built-up column. The connection spacing and strength should adhere to the following criteria: (i) for intermediate connection spacing, the slenderness ratio of each individual component should not exceed 50% of the slenderness ratio of the entire built-up member, i.e. $(a/r_i) \leq 0.5(KL/r)$ (the factor 0.5 accounts for a degree of ineffectiveness of certain fastener rows); (ii) for intermediate connection strength, interconnections made by fasteners or welds should be designed to accommodate and transmit 2.5% of the nominal compression resistance of the built-up member; and (iii) for end connection strength, the fasteners or welds at the ends of the column effective length need to be capable of transmitting the required shear strength between individual components, as determined by Eq. [I1.2.2.1-2] [2].

Appendix 2 of AISI S100-16 (2020) w/S3-22 [2] outlines the guidelines for determining elastic buckling stresses in thin-walled cold-formed steel members. This can be done using either numerical methods—such as the finite strip method, finite element method, or generalized beam theory—or through analytical solutions based on linear elastic theory.

Section 6.4 of EN 1993-1-1 [56] outlines a specific procedure for pin-ended uniform battened or laced built-up columns. This approach involves substituting the built-up column with a continuous equivalent characterised by a smeared shear stiffness S_V that reflects the structural behaviour of the built-up configuration. $N_{cr,V}$ is the critical buckling load of an equivalent continuous column, in which the influence of shear deformations is significant (based on the differential equation of equilibrium conditions of an axially compressed member in the deformed state):

$$N_{cr,V} = \frac{1}{\frac{1}{N_{cr}} + \frac{1}{S_V}} \quad (2)$$

where N_{cr} is the critical Euler load for the same built-up column neglecting shear deformations. To effectively limit shear deformations between the connected components, at least two IFGs should be provided along the length of the column. Clause 6.4.4 provides criteria for the design of closely spaced built-up members—if the conditions listed in Table 6.9 are met, these members can be treated as continuous (solid) for design purposes, allowing the neglect of shear deformations; if these conditions are not satisfied, the design should adhere to the guidelines established for battened members. There are no particular guidelines concerning the construction details specifics for interconnections. EN 1993-1-4 [44] establishes additional guidelines for designing stainless steel structures, supplementing the provisions in EN 1993-1-1 [56] applicable to carbon steel structures.

To address the discreteness of built-up sections and the limited composite action of their individual components, developing a methodology for modelling built-up cross-sections to predict elastic critical buckling responses—encompassing failure modes and the corresponding elastic critical buckling stresses—has posed a challenge in numerous scientific studies [4], [8], [9], [10], [11], [19], [23], [24], [26]. The following modelling approaches have generally gained acceptance: (i) for global buckling modes—such as flexural, torsional, or flexural–torsional—the elastic critical stress f_{cre} should be evaluated using a full built-up cross-section model, wherein the wall thicknesses of the individual contacting sections are combined as a single wall with double the thickness of a single section; (ii) the elastic critical buckling stresses for local f_{cr1} and distortional buckling f_{crd} of the built-up member should be determined using a single (individual component) cross-section model; and (iii) all elastic critical buckling load predictions (local, distortional, and global) should be calculated as the product of the corresponding elastic critical buckling stress and the gross cross-sectional area of the built-up section.

To effectively evaluate the influence of partial fastener restraints on longitudinal shear deformations between components and the interactions of local and global instabilities in built-up columns, researchers have proposed revisions to the AISI S100 design guidelines. In addition to the established DSM procedure [2], Selvaraj and Madhavan [27] conducted a comprehensive evaluation of the accuracy of several DSM-based methodologies [9], [17], [18], [23], [25] concerning built-up closed cross-section columns under conditions of global, local, and local-global interactive buckling, while notably excluding distortional or interactive distortional buckling considerations. Their findings revealed a significant degree of data scatter across all examined design methodologies. Predictions tend to be conservative for columns with intermediate fastener spacing less than the corresponding half-wavelength of the LB but become non-conservative for those with greater fastener spacing due to the effects of local-global interactive buckling [27].

This paper evaluates the applicability of two recently proposed procedures: (i) DSM-based approach that addresses both flexural and flexural-torsional buckling modes while considering interactions with cross-sectional buckling modes; it incorporates parameters such as effective flexural and torsional rigidities and limits on fastener spacing [15], [20], [21], and (ii) a modified Eurocode 3 procedure specifically designed for solid plain carbon steel cruciform section columns, introducing a novel parameter that establishes a relationship between torsional slenderness and the torsional-to-flexural buckling load ratio [40]. Additionally, further refinement of the formula proposed by Behzadi-Sofiani et al. [40] is warranted, taking into account the discrete characteristics of built-up cross-section columns and their vulnerability to LB. This adjustment involves the integration of an effective (reducing) rigidity approach [15] and effective cross-sectional properties specifically for Class 4 sections.

In the design predictor equations of both proposed procedures, the elastic critical stresses for global and local buckling modes of the cruciform built-up columns are determined in accordance with the guidelines provided in Appendix 2, AISI S100 [2], but with the notation: (i) the elastic critical global stresses are determined using a built-up cross-section model featuring multiple component sections — in the equations for elastic critical stresses, the flexural EI , torsional GJ and warping rigidity GI_w of a full built-up cross-section (with double thickness at the overlapping section walls) should be replaced by corresponding effective (reduced) rigidity EI_{eff} , GJ_{eff} , $GI_{w,eff}$ as seen in Eqs. (21)-(24) in this study, (ii) the elastic critical buckling stresses for local and distortional buckling of the built-up sections are determined considering a single component section model, as seen in Eqs. (25) and (26) in this study.

The elastic critical FB stress of a full built-up section was calculated as per Sections 2.3.1.1 and 2.3.1.1.1. The elastic critical FTB stress of a full built-up section was calculated as per Sections 2.3.1.1 and 2.3.1.1.2. The elastic TB stress was determined as per Sections 2.3.1.1 and 2.3.1. All global elastic critical buckling stress predictions were calculated as the ratio of the corresponding elastic critical buckling load and the gross cross-sectional area of the built-up section. The elastic local buckling stress was obtained as outlined in Section 2.3.2.1. For calculating the elastic critical distortional buckling stress, one may refer to the guidelines from Section 2.3.3.1 or employ numerical methods. It is important to note that the codified provisions for distortional buckling apply to Z-, C-, Hat, and similar open cross-sections that have edge-stiffened flanges, as outlined in AISI S100, Section E4 [2].

All columns were considered as pin-ended, which established the effective length equal to the actual column length, $KL = L$. Additionally, the safety factor used in predicting the column ultimate resistances for each design procedure was taken equal to 1.0.

The results of the parametric study were carefully analysed to discern the prevailing failure modes. The study focused primarily on failures governed by LB or TB to assess the appropriateness of the design equations. In the presented parametric study, none of the FE columns exhibited a distortional buckling mode, which is consistent with expectations due to their cruciform geometry. A comparison between the generated FE data and calculated data is presented in Fig. 11 and Fig. 12, distinguishing between the various design procedures employed. Following this, reliability assessments are conducted to ascertain the partial safety and resistance factors for the built-up columns examined in this study.

4.1 DSM-based design procedure for built-up sections

In light of the findings from investigations on mechanical models of built-up members conducted by Rasmussen et al. [15], Phan et al. [20], [21] have recently introduced and validated a novel DSM-based approach specifically designed for fixed-ended and pin-ended built-up columns that are susceptible to FB and FTB modes. This procedure

highlights the application of effective (reduced) flexural, torsional, and warping rigidities in conjunction with DSM equations for axially compressed members specified in AISI S100 [2]. Additionally, a fastener spacing limit $(a/r_i)/(KL/r) \leq 0.5$ is recommended.

According to AISI S100 [2], the nominal axial resistance P_n of axially compressed member should be determined as the minimum among the nominal strengths addressing global buckling P_{ne} (which includes flexural, torsional, and flexural-torsional modes), local buckling P_{nl} , and distortional buckling P_{nd} .

The nominal global buckling column resistance should be determined following the analytical methods and criteria provided in Section E2:

$$P_{ne} = A_g F_n \quad (3)$$

where A_g is the gross cross-sectional area, and F_n is the compressive stress that should be calculated as follows:

$$F_n = \begin{cases} (0.658^{\lambda_c^2}) f_y & \text{for } \lambda_c \leq 1.5 \\ \left(\frac{0.877}{\lambda_c^2} \right) f_y & \text{for } \lambda_c > 1.5 \end{cases} \quad (4)$$

where the slenderness is given by:

$$\lambda_c = \sqrt{f_y / F_{cre}} \quad (5)$$

and f_y is the yield strength.

The global buckling stress F_{cre} should be calculated as follows:

$$F_{cre} = P_{cre} / A_g \quad (6)$$

where P_{cre} is the minimum of the applicable elastic global (flexural, torsional, and flexural-torsional) buckling loads of the column, determined in accordance with Appendix 2, Sections 2.3.1.1.1 to 2.3.1.1.3.

The elastic FB load, P_{cre} is determined as follows:

$$P_{cre} = \frac{\pi^2 EI}{(KL)^2} \quad (7)$$

where I is the moment of inertia of the entire unreduced cross-section about the axis of buckling (for example $y - y$ or $z - z$).

The elastic FTB load P_{cre} is determined as follows:

$$P_{cre} = \frac{1}{2\beta} \left[(P_{ey} + P_t) - \sqrt{(P_{ey} + P_t)^2 - 4\beta P_{ey} P_t} \right] \quad (8)$$

where:

$$\beta = 1 - (y_o/r_o)^2 \quad (9)$$

r_o is the polar radius of gyration of the cross-section about the shear centre:

$$r_o = \sqrt{r_y^2 + r_z^2 + y_o^2 + z_o^2} \quad (10)$$

P_{ey} is the elastic FB load about y -axis (for singly-symmetric sections, y is the axis of symmetry):

$$P_{ey} = \frac{\pi^2 EI_y}{(K_y L_y)^2} \quad (11)$$

1 P_t is the elastic TB load:

$$P_t = \frac{1}{r_0^2} \left[GJ + \frac{\pi^2 EI_w}{(K_t L_t)^2} \right] \quad (12)$$

2 r_y, r_z are the radii of gyration of the cross-section about the centroidal principal axes, y_0, z_0 are the shear centre
3 coordinates with respect to the centroid of the gross cross-section (for the cruciform section $y_0 = z_0 = 0$), K_y is the
4 effective length factor considering bending about $y - y$ axis, L_y is the unbraced member length considering bending
5 about $y - y$ axis, G is the shear modulus, J is the Saint-Venant torsion constant, I_w is the torsional warping
6 constant, K_t is the effective length factor for twisting, L_t is the unbraced member length for twisting [2].

7 The nominal LB axial resistance P_{nl} , accounting for potential reductions in resistance arising from the interplay of
8 combined yielding or global and local buckling, is addressed using the DSM equations outlined in Section E3.2:

$$P_{nl} = \begin{cases} P_{ne} & \text{for } \lambda_l \leq 0.776 \\ \left[1 - 0.15 \left(\frac{P_{crl}}{P_{ne}} \right)^{0.4} \right] \left(\frac{P_{crl}}{P_{ne}} \right)^{0.4} P_{ne} & \text{for } \lambda_l > 0.776 \end{cases} \quad (13)$$

9 where the slenderness:

$$\lambda_l = \sqrt{P_{ne}/P_{crl}} \quad (14)$$

10 The elastic critical local buckling load P_{crl} should be based on the lowest buckling stress among cross-section
11 elements as follows:

$$P_{crl} = A_g F_{crl} \quad (15)$$

12 The elastic critical LB stress F_{crl} (corresponding to the smallest local buckling stress of all cross-section walls) can
13 be determined as per Appendix 2, Section 2.3.2.1:

$$F_{crl} = k \frac{\pi^2 E}{12(1 - \mu^2)} \left(\frac{t}{w} \right)^2 \quad (16)$$

14 where w is the flat plate width, t is the plate thickness, k is the plate buckling coefficient (for uniformly compressed
15 unstiffened elements, outstand flanges $k = 0.43$; for uniformly compressed stiffened elements, internal parts $k =$
16 4), μ is Poisson's ratio.

17 The nominal column resistance for distortional buckling P_{nd} is determined with the DSM equations outlined in
18 Section E4:

$$P_{nd} = \begin{cases} P_y & \text{for } \lambda_d \leq 0.561 \\ \left[1 - 0.25 \left(\frac{P_{crl}}{P_y} \right)^{0.6} \right] \left(\frac{P_{crl}}{P_y} \right)^{0.6} P_y & \text{for } \lambda_d > 0.561 \end{cases} \quad (17)$$

19 Where the slenderness:

$$\lambda_d = \sqrt{P_y/P_{crl}} \quad (18)$$

20 P_y is the squash load in compression

$$P_y = A_g f_y \quad (19)$$

1 The elastic critical distortional buckling load P_{crd} is determined as follows:

$$P_{\text{crd}} = A_g F_{\text{crd}} \quad (20)$$

2 where the elastic critical distortional buckling stress F_{crd} can be determined in accordance with Appendix 2, Section
3 2.3.3.1 or using the numerical procedures.

4 To compute the nominal axial strength P_n of the built-up column, the effective second moments of area $I_{y,\text{eff}}$ ($I_{u,\text{eff}}$)
5 and $I_{z,\text{eff}}$ ($I_{v,\text{eff}}$) for both the major and minor principal axes should be taken into account:

$$I_{\text{eff}} = \alpha I_{\text{full}} \quad (21)$$

6 The effective warping constant $I_{w,\text{eff}}$ should be obtained as follows:

$$I_{w,\text{eff}} = \alpha I_{w,\text{full}} \quad (22)$$

7 while the parameter α in Eqs. (21) and (22) should be determined as follows:

$$\alpha = \frac{(KL/r)^2}{(KL/r)^2 + (a/r_i)^2} \quad (23)$$

8 It is important to note that the parameter α in Eq. (23) corresponds to the second term in Eq. (1), which pertains to
9 the reduced moment of inertia I_r of built-up section.

10 The effective torsion constant J_{eff} and the effective local $F_{\text{crl, eff}}$ and distortional $F_{\text{crd, eff}}$ buckling stresses should be
11 obtained as follows:

$$J_{\text{eff}} = n J_{\text{ind}} \quad (24)$$

$$F_{\text{crl, eff}} = F_{\text{crl, ind}} \quad (25)$$

$$F_{\text{crd, eff}} = F_{\text{crd, ind}} \quad (26)$$

12 Subscripts "*ind*" and "*full*" denote the values, respectively, corresponding to an individual (single) section and the
13 built-up section under the assumption of full composite action. Additionally, n represents the count of individual
14 sections that make up the built-up section.

15 The nominal global buckling column resistance of the built-up column is determined through Eq. (4), utilizing the
16 substitution of parameters EI_{eff} , $EI_{w,\text{eff}}$, GJ_{eff} and EI , EI_w , GJ in the computation of the elastic FB, FTB and TB loads,
17 outlined in Eqs. (7), (8), and (12). The minimum value among these calculations yields the elastic global buckling
18 stress. The radius of gyration r in Eqs. (10) and (23) is determined as the square root of the ratio of the effective
19 second moment of area I_{eff} and full unreduced built-up cross-sectional area A_g ($A_g = n A_{g,\text{ind}}$). The reduction of the
20 column resistance due to LB should not be considered using the Effective Width Method (EWM) because it is already
21 taken into account via the DSM equations. The nominal LB column resistance is calculated using Eqs. (13), (15), and
22 (16), while the nominal DB column resistance is determined through Eqs. (17) and (20). The elastic critical LB and
23 DB stresses are computed based on a single (individual) cross-section model, where the smallest resistance of the
24 individual component cross-section is taken into consideration, as shown in Eqs. (25) and (26). Finally, the nominal
25 axial resistance of the built-up column is established as the minimum among the nominal strengths related to global
26 buckling, LB, and DB resistance.

4.2 Design procedure for cruciform sections based on Perry-Robertson formulation

The design procedure for plain carbon steel cruciform section columns, proposed by Behzadi-Sofiani et al. [40], utilizes the Perry-Robertson formulation for the imperfection parameter $\eta = \alpha\beta(\bar{\lambda} - \bar{\lambda}_0)$. In this context, α and $\bar{\lambda}_0$ are constants that represent the effects of geometric imperfections and residual stresses on the column buckling resistance, while β is a factor accounting for the influence of TB and FB mode interaction [40].

The design expression of the TB resistance of cruciform section columns ($N_{cr,T} \leq N_{cr,F}$) is:

$$N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}} \quad (27)$$

where f_y is the material yield strength and γ_{M1} is the partial safety factor for the resistance of members subject to instability. It is proposed to use the column gross cross-sectional area A in all TB resistance calculations, regardless of the cross-section's class.

A new parameter Δ_F , which accounts for the variation of the buckling reduction factor χ with $N_{cr,T}/N_{cr,F}$ should be determined as follows:

$$\Delta_F = \left(1 - \frac{N_{cr,T}}{N_{cr,F}}\right)^2 \quad (28)$$

where $N_{cr,T}$ is the elastic critical torsional buckling load, leading to the relative slenderness $\bar{\lambda}_T$:

$$\bar{\lambda}_T = \sqrt{\frac{A f_y}{N_{cr,T}}} \quad (29)$$

and $N_{cr,F}$ is the elastic critical flexural buckling load, leading to the relative slenderness $\bar{\lambda}$:

$$\bar{\lambda} = \sqrt{\frac{A f_y}{N_{cr,F}}} \quad (30)$$

The buckling reduction factor χ can be obtained using Δ_F in conjunction with the LB design curve for compressed outstand flanges from EN 1993-1-5 [55]:

$$\chi = \chi_F + \Delta_F(\chi_T - \chi_F). \quad (31)$$

in which χ_F is the FB reduction factor based on a newly proposed flexural buckling curve, see Eq. (34) and χ_T is the TB reduction factor (the procedure assumes that torsional and local buckling are essentially identical modes for cruciform sections):

$$\chi_T = \frac{\bar{\lambda}_T - 0.188}{\bar{\lambda}_T^2} \text{ where } \chi_T \leq 1.0 \quad (32)$$

The design expression of the FB resistance of cruciform section columns ($N_{cr,T}/N_{cr,F} > 1.0$) is:

$$N_{b,Rd} = \frac{\chi_F A f_y}{\gamma_{M1}} \quad (33)$$

where:

$$\chi_F = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} \text{ and } \chi_F \leq 1.0 \quad (34)$$

in which the normalised slenderness $\bar{\lambda}$ is given by Eq. (30); the parameter ϕ and the imperfection factor η have their usual meaning, i.e.

$$\phi = 0.5(1 + \eta + \bar{\lambda}^2) \quad (35)$$

$$\eta = \alpha\beta(\bar{\lambda} - \bar{\lambda}_0)^\beta \quad (36)$$

with β accounting for the influence of mode interaction:

$$\beta = 1.6 - 0.4 \frac{N_{cr,T}}{N_{cr,F}} \text{ where } 1.0 \leq \beta \leq 1.2 \quad (37)$$

The value of 1.2 is retained for all cases in which torsional buckling is critical (i.e. $N_{cr,T}/N_{cr,F} \leq 1.0$).

The design buckling resistances of all columns studied in the parametric study were determined using an imperfection factor $\alpha = 0.49$ in conjunction with a non-dimensional limiting slenderness $\bar{\lambda}_0 = 0.2$.

It is proposed to replace the gross cross-sectional area A with the effective area A_{eff} (determined by EN 1993-1-5 [55]) in Eqs. (30) and (33) for Class 4 cross-sections when FB is critical. For cases when TB is critical, it is proposed to use the gross area in all calculations, regardless of the cross-section class.

The cross-section resistance of columns failing by LB is given by:

$$N_{c,Rd} = \frac{Af_y}{\gamma_{M0}} \quad (38)$$

where A is the gross cross-sectional area (i.e. summation of the gross areas of the individual component cross-sections), taken as the gross cross-sectional area for Class 1, 2 and 3 sections or the effective cross-sectional area A_{eff} for Class 4 sections (i.e. summation of the effective areas of the individual components with their reduced effective widths), and γ_{M0} is the partial safety factor.

Proposed modifications considering the reduction in column buckling resistance due to the discrete nature of built-up cross-section

To account for the discrete nature of the built-up cross-section columns, the effective flexural, torsional and warping rigidity approach developed in [15], [12], [13] is introduced in the procedure proposed in [40].

The elastic critical FB load, $N_{cr,F}$ in Eqs. (28), (30) and (37) is therefore:

$$N_{cr,F} = \pi^2 EI_{eff} / (KL)^2 \quad (39)$$

And the elastic critical TB load, $N_{cr,T}$ in Eqs. (28), (29) and (37) is:

$$N_{cr,T} = \frac{1}{r_0^2} \left(GJ_{eff} + \frac{\pi^2 EI_{w,eff}}{(K_t L_t)^2} \right) \quad (40)$$

The design model proposed in the research by Behzadi-Sofiani et al. [40] has been developed for solid cruciform sections predominantly failing by torsional deformations in a single half-wave mode, similar to the critical LB mode. However, this structural behaviour is not always specific to built-up cross-sections; moreover, the TB failure of built-up sections can be accompanied by a localized failure mode of individual components with multiple half-waves, i.e. with transverse bending of the component cross-section walls (internal and outstand compression parts).

In light of this, the comparative analysis between the numerical and calculated loads is carried out considering two cases:

- (1) The design TB resistances of the columns are determined using the gross cross-sectional area A in Eqs. (27) and (29) for all classes, as it is proposed in the research of Behzadi-Sofiani et al [40].
- (2) The design TB resistances of the columns are then determined using the effective cross-sectional area A_{eff} for slender cross-sections (class 4).

4.3 Comparisons

Comparisons of the numerically and experimentally obtained ultimate resistances P_u with the design resistances $P_{u,pred}$ predicted by the DSM-based design procedure and the Perry-Robertson formulation-based design procedure are graphically shown in Fig. 11. Key performance indicators, including average values, Coefficient of Variation (CoV), and maximum/minimum metrics for 240 generated data, are presented for each design approach in Table 5. Furthermore, the efficiency and precision of these design procedures are illustrated in Fig. 12, with the resistance ratios plotted against the non-dimensional relative slenderness for TB, $\bar{\lambda}_T$.

The subsequent observations can be drawn:

- Both design procedures result in unreliable and unsafe predictions when the requirement regarding the fastener spacing limit $(a/r_i)/(KL/r) \leq 0.5$ [2] is not met; in this work, these columns were not considered in our statistical evaluations and reliability analyses.
- The failure loads are generally excessively underestimated (by a large margin) and largely overestimated in a few cases by the proposed DSM-based design approach.
- While the DSM-based procedure better predicts the failure modes (LB or TB), the design approach based on the Perry-Robertson formulation leads to significantly less scatter but generally overestimates the resistance (< 1.0) across almost the entire slenderness range. This tendency to overestimate can be attributed to the treatment of LB and TB as equivalent failure mechanisms. Consequently, when TB is a critical concern, the approach employs the gross cross-sectional area for all column capacity calculations, irrespective of the cross-section class classification.
- To account for LB, the failure loads were recalculated using the effective cross-sectional area A_{eff} for slender Class 4 cross-sections, incorporating the effective rigidity approach. This adjustment enhances the accuracy of the Perry-Robertson formulation, resulting in safer and more reliable results, although it may introduce a slight increase in scatter.
- The greatest scatter, accompanied by a significant conservative assessment of the ultimate resistances, appears for (i) intermediate-to-long length columns where the failure mode shifts from torsional to flexural instability, i.e. $N_{cr,T} \rightarrow N_{cr,F}$ and (ii) columns configured with a fastener arrangement that approaches the maximum allowable fastener spacing limits, i.e. $(a/r_i)/(KL/r)_0 \rightarrow 0.5$.

Table 5 Statistical indicators comparing FE and test results with design resistances for flanged cruciform section columns.

Design procedure	Statistical indicators			
	Average	CoV	Max	Min
DSM-based design [15], [20], [21]	1.594	0.362	3.47	0.82
Perry-Robertson formulation-based design [40] with gross cross-sectional area	1.042	0.259	2.61	0.81
Modified Perry-Robertson formulation-based design [40] with effective cross-sectional area	1.404	0.272	3.66	0.97

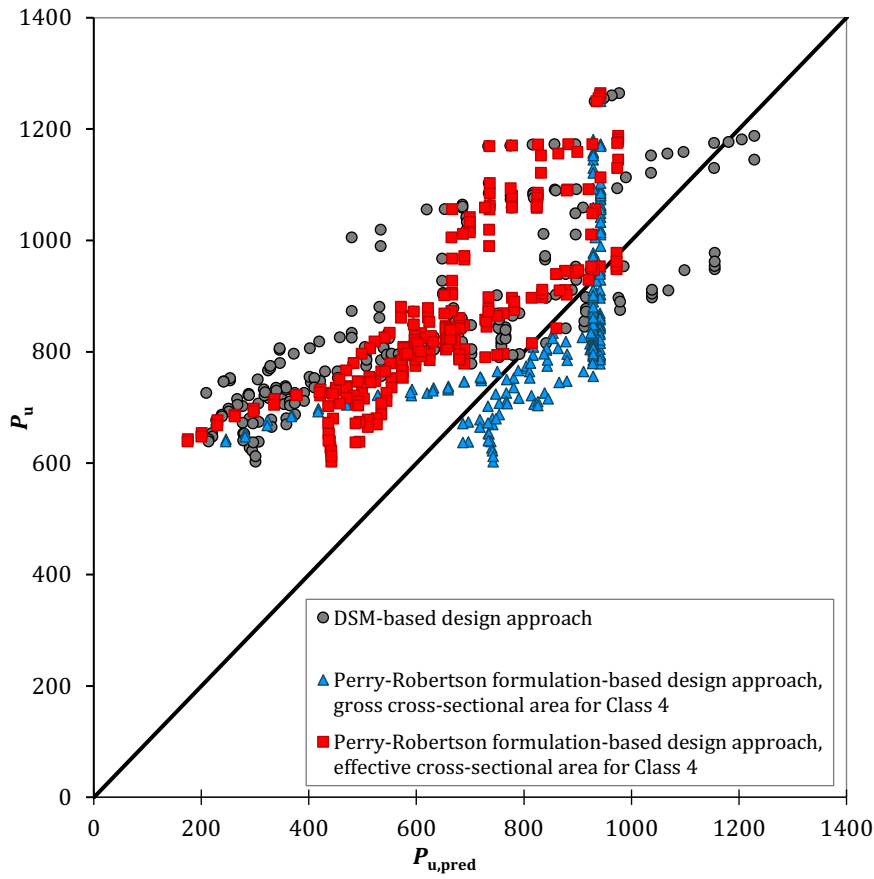
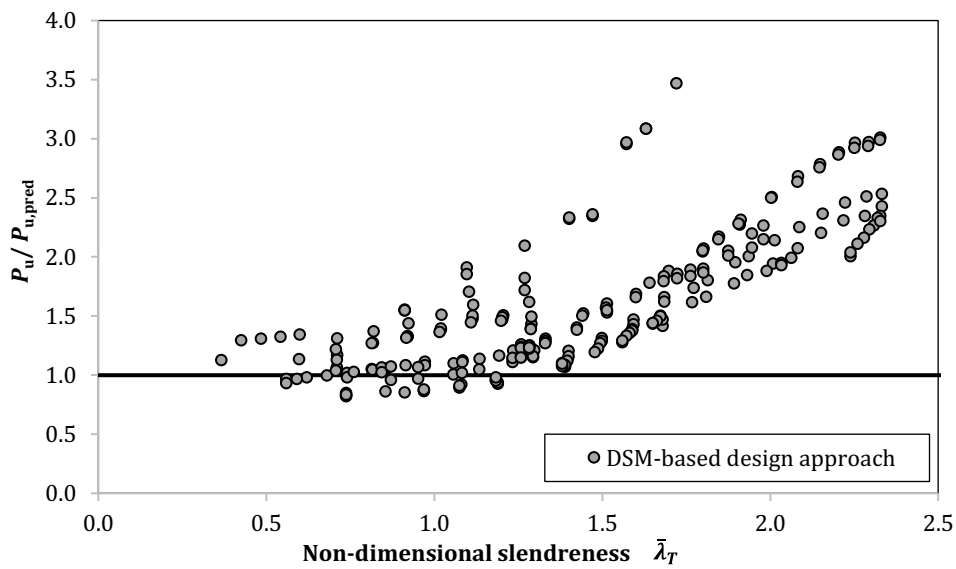
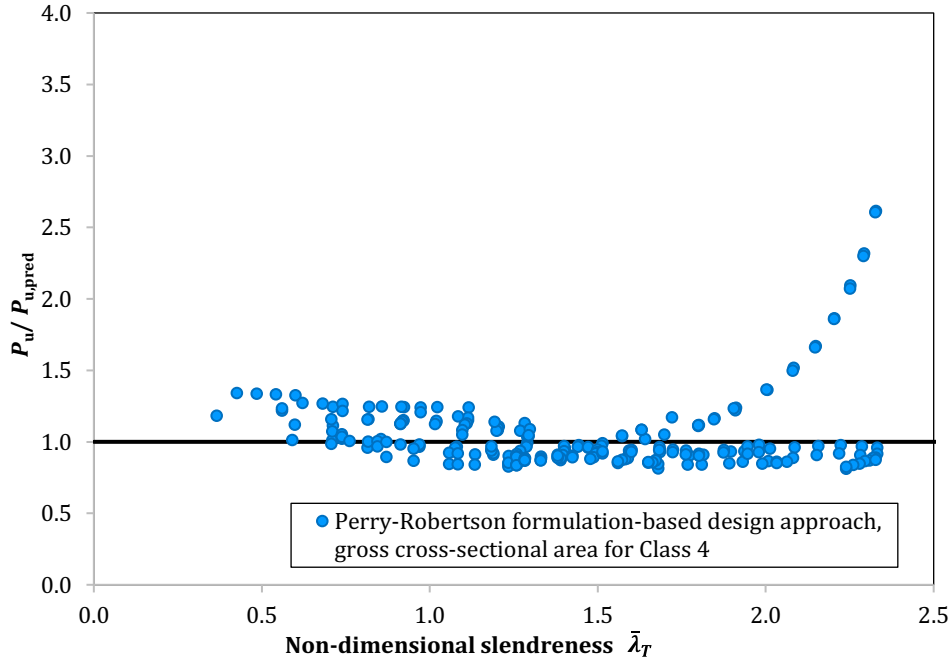


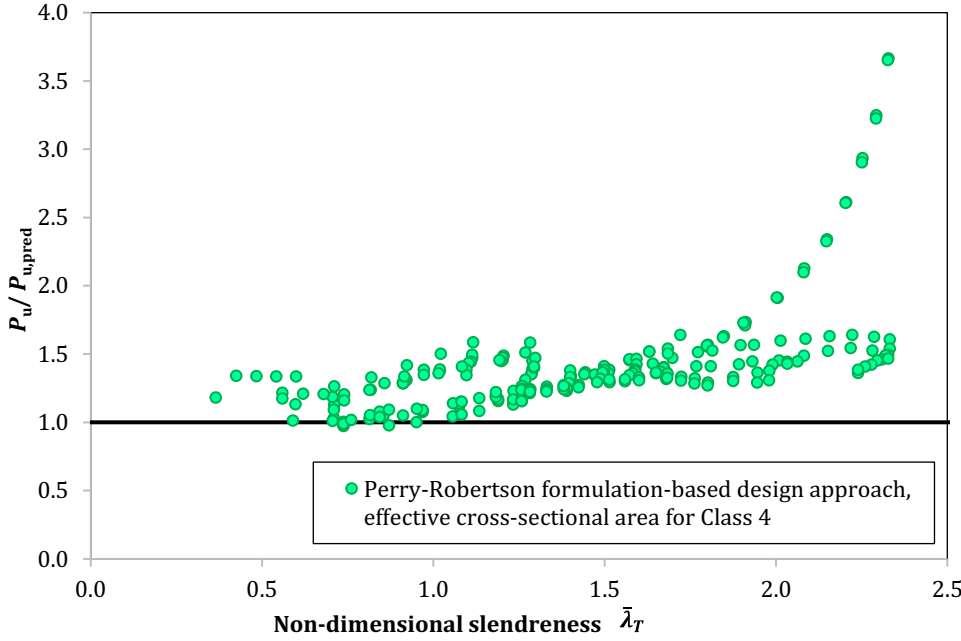
Fig. 11 Comparisons of the ultimate resistances obtained from FE analysis and test results, along with the design resistances.



(a) DSM-based design [15], [20], [21]



(b) Perry-Robertson formulation-based design [40] that accounts for gross cross-sectional area



(c) Perry-Robertson formulation-based design [40] that accounts for effective cross-sectional area

Fig. 12 Assessment of the proposed design procedures.

5 Reliability assessment

5.1 Safety factor γ_{M1}^*

This section evaluates the reliability of the modified formulation proposed by Behzadi-Sofiani et al. [40]. Considering the modifications introduced in this paper to address LB, the failure loads are computed using the effective cross-sectional area A_{eff} for slender Class 4 sections. Statistical analyses were conducted in accordance with the procedures outlined in Annex D of EN 1990 [57], utilizing previous numerical and experimental data from the parametric studies detailed in Section 3.2. For austenitic stainless steel grade, the material over-strength factor was taken as 1.3 with a CoV of 6%, whereas the CoV of the variability of the geometric properties was taken as 5%, as recommended in [58]. The key statistical parameters of the reliability analyses are summarised in Table 6, where b and V_δ are the average and CoV of the column resistances generated both numerically and experimentally, in

relation to the predictions from the resistance model, respectively. Additionally, V_r denotes the combined variability that arises from the resistance model along with the basic variables. The acceptance limit f_a , as provided in [59], is given by:

$$f_a = 1.03 + 0.75(V_r - 0.04) \text{ but } 1.03 \leq f_a \leq 1.15 \quad (41)$$

The required value of the partial safety factor γ_{M1}^* was determined following the procedure described in [58]; according to EN 1993-1-4 [44] the target partial safety factor value γ_{M1} is equal to 1.1.

The ratio $\gamma_{M1}^*/\gamma_{M1}$ should satisfy:

$$\gamma_M^*/\gamma_M \leq f_a \quad (42)$$

for the method to be deemed reliable.

The modified Perry-Robertson formulation yields a final resistance ratio $\gamma_{M1}^*/\gamma_{M1}/f_a$ that approaches 1.0, thereby satisfying the reliability requirements. However, the considerable variability in the buckling response of intermediate-length columns—particularly those transitioning between torsional and flexural buckling modes—has resulted in generated-to-predicted resistance ratios that produce a value of b greater than customary.

Table 6 Reliability analysis results and safety factor γ_{M1}^* for the Perry-Robertson-based formulation [24], [15], [20], [21]

Source	Critical mode	b	V_δ	V_r	γ_{M1}^*	$\gamma_{M1}^*/\gamma_{M1}$	f_a	$\gamma_{M1}^*/\gamma_{M1}/f_a$
EN 1.4301	LB and TB	1.311	0.211	0.216	1.15	1.045	1.043	1.001

5.2 Resistance factor ϕ

This section focuses on assessing the Load and Resistance Factor Design (LRFD) resistance factor linked to the modified Perry-Robertson formulation. In accordance with the guidelines in AISI S100 [2], the resistance factor ϕ is calculated using equations K2.1.1-2 and K2.1.1-4, which can be found in section K2.1.1:

$$\phi = C_\phi (M_m F_m P_m) e^{-\beta_0 \sqrt{V_M^2 + V_F^2 + C_P V_P^2 + V_Q^2}} \quad (43)$$

$$C_P = \left(1 + \frac{1}{n}\right) \frac{m}{m-2} \quad (44)$$

where $M_m = 1.10$ and $F_m = 1.00$ are the mean values of the material and fabrication factor, respectively, $\beta_0 = 2.5$ is the target reliability index for structural members in LRFD, $C_\phi = 1.52$ is a calibration coefficient in LRFD, $V_F = 0.05$, $V_M = 0.10$ and $V_Q = 0.21$ are the coefficients of variation of the fabrication factor, material factor, and load effect, respectively, C_P is a correction factor that depends on the number of tests n and degrees of freedom $m = n - 1$. In order to assess the resistance factor ϕ , it is necessary to calculate the factors P_m and V_P that correspond to mean value and coefficient of variation of the ratios $P_u/P_{u,pred}$.

The resistance factor ϕ for the modified Perry-Robertson formulation [24], [15], [20], [21] has been calculated on the basis of the statistics shown in Table 5. Table 7 shows the n , C_P , P_m , V_P and ϕ values obtained for the column buckling resistance predictions provided by the modified Perry-Robertson formulation for the whole failure data – FE and test results. Considering all data, the computed resistance factor is $\phi = 0.97$. Therefore, the results of the reliability analysis support the recommendation that the resistance factor $\phi = 0.85$ specified in AISI S100 [2] for

compression members in LRFD can be effectively applied to cold-formed stainless steel built-up cruciform section columns designed using the modified Perry-Robertson formulation.

Table 7 LRFD reliability analysis results and resistance factor ϕ for the Perry-Robertson-based formulation [24], [15], [20], [21]

Source	Critical mode	n	C_P	P_m	V_P	ϕ
EN 1.4301	LB and TB	240	1.01	1.43	0.27	0.97

6 Conclusions

The paper begins by presenting the results of compression tests conducted on austenitic stainless steel built-up cruciform columns with intermediate slenderness ($L = 2250$ mm) and four different fastener configurations. The columns were tested under semi-rigid support conditions and axial compression. To create the doubly symmetrical flanged cruciform section columns, four press-braked lipped equal-leg angles (with nominal dimensions of $50 \times 135 \times 3$ mm and an internal corner radius of 6.5 mm) were assembled in a star orientation. The interconnections were made using M8 bolts of class 8.8. The angle components of the first specimen were connected with bolts placed every 50 mm. For the second specimen, bolts were used at every other hole (i.e., every 100 mm), with reinforcement provided by three-bolt rows at the ends. In the third specimen, the angle components were connected with bolts spaced every 50 mm along the length. However, connections were made only at the longitudinal edges across the Tee-leg width—specifically at the Tee-leg junction and the web-to-flange junction. Lastly, the angle components of the fourth specimen were connected at every other hole (i.e., every 100 mm) along the longitudinal edges of the Tee-leg width. The experimental program aimed to differentiate between local and torsional buckling responses and quantify the degree of composite action among the individual components influenced by the fastener arrangements.

A FE model was developed to numerically simulate the tested specimens' structural behaviour and failure modes. A parametric study followed, focusing on built-up flanged cruciform section columns predominantly failing due to LB or TB, or their interactions. The FE parametric study comprised 250 pin-ended built-up cruciform section columns made from austenitic stainless steel alloy, covering a slenderness range from 0.3 to 3.5.

The ultimate buckling loads derived from the FE parametric study were utilized to assess the applicability of recent design proposals: DSM-based procedure for designing cold-formed carbon steel built-up columns [15], [20], [21] and the Perry-Robertson formulation-based procedure for the design of solid carbon steel cruciform section columns [40].

The following main conclusions could be drawn from the study:

- For pin-ended boundary conditions, installing EFGs can substantially increase the effective torsional and flexural rigidity of the built-up column and, in turn, substantially augment its buckling capacity.
- The ultimate capacity of short-length columns is significantly influenced by the spacing of interconnections and the number of fastener rows in EFGs. It is advisable to maintain the fastener spacing at less than the half-wavelength required for LB of the cross-sectional plate elements, while also ensuring that the EFGs effectively prevent longitudinal shear deformations between the component elements; similar findings have been reported in scientific research [13], [22], [25] investigating the buckling behaviour of built-up columns with various cross-sectional geometries.
- Increasing the fastener spacing along the column length and across the Tee-leg width leads to the most significant reductions in strength for specimens of intermediate lengths; this phenomenon corresponds to

the critical buckling mode that shifts between local and torsional modes. Likewise, arranging fasteners in a denser configuration, with spacing smaller than the LB half-wavelength, enhances the ultimate capacity of the columns, increasing the likelihood of global TB mode.

- Using additional fastener rows slightly increases the buckling strength of slender columns predominantly experiencing global TB failure, showing that uniform torsion of open sections can occur without causing significant shear deformation of the fasteners.
- The DSM-based procedure tends to produce highly scattered predictions; this method often results in overly conservative predictions, particularly within the intermediate slenderness range $\bar{\lambda}_T > 1.2$, while appearing unconservative in the low-to-intermediate slenderness range $0.5 < \bar{\lambda}_T < 1.2$.
- Lastly, the procedure based on the Perry-Robertson formulation has been modified to effectively address the discrete nature of thin-walled built-up cruciform section columns and their susceptibility to both local and torsional buckling modes. The revised approach yields significantly more accurate and reliable predictions of resistance compared to existing codified provisions, ultimately offering design guidelines for built-up compression elements failing by torsional buckling.

Acknowledgements

This investigation is supported by the Serbian Ministry of Education, Science and Technological Development through the 200092 project.

The authors are grateful to companies AluBuild Engineering doo Belgrade, CO-Designing Belgrade and Institute for Testing of Materials Belgrade for their support.

References

- [1] Specification for Structural Steel Buildings. An American National Standard ANSI/AISC 360-22, American Institute of Steel Construction (AISC), Chicago, August 1, 2022.
- [2] AISI S100-16(2020) w/S3-22. North American Specification for the Design of Cold-Formed Steel Structural Members, 2016 Edition (Reaffirmed 2020) with Supplement 3, 2022 Edition.
- [3] F. Bleich, Buckling Strength of Metal Structures, McGraw-Hill Book Company, 1952.
- [4] B. Young, J. Chen, Design of cold-formed steel built-up closed sections with intermediate stiffeners, J Struct Eng., 134(5) (2008) 727–37.
- [5] J. Becque, K.J.R. Rasmussen, Experimental investigation of the interaction of local and overall buckling of stainless steel I-columns, J. Struct. Eng. 135 (11) (2009) 1340–1348.
- [6] D.C. Fratamico, S. Torabian, K.J.R. Rasmussen, B.W. Schafer, Buckling and collapse behavior of screw-fastened, built-up cold-formed steel columns of varying cross-section size: experimental investigation, Proceedings of the Annual Stability Conference, Structural Stability Research Council, San Antonio, Texas, March 21–24, 2017.
- [7] D.C. Fratamico, S. Torabian, X. Zhao, K.J.R. Rasmussen, B.W. Schafer, Experimental study on the composite action in sheathed and bare built-up cold-formed steel columns, Thin-Walled Structures. 127 (2018) 290–305.
- [8] JH Zhang, B. Young, Experimental investigation of cold-formed steel built-up closed section columns with web stiffeners. J Constr Steel Res., 147 (2018) 380–92.
- [9] JH Zhang, B. Young, Finite element analysis and design of cold-formed steel built-up closed section columns with web stiffeners, Thin-Walled Struct., 131 (2018) 223–37.
- [10] L. Wang, B. Young, Behaviour and design of cold-formed steel built-up section beams with different screw arrangements. Thin-Walled Struct., 131 (2018) 16–32.
- [11] S. Selvaraj, M. Madhavan, Structural Design of Cold-formed Steel face-to-face Connected Built-up beams using Direct Strength Method, Journal of Constructional Steel Research 160 (2019) 613–628.
- [12] F.J. Meza, J. Becque, I. Hajirasouliha, Experimental study of the cross-sectional capacity of cold-formed steel built-up columns, Thin-Walled Structures 155 (2020) 106958.
- [13] F.J. Meza, J. Becque, I. Hajirasouliha, Experimental study of cold-formed steel built-up columns, Thin-Walled Structures. 149 (2020), 106291.

- [14] B. Chen, K. Roy, A. Uzzaman, G. Raftery, J.B.P. Lim, Axial strength of back-to-back cold-formed steel channels with edge-stiffened holes, un-stiffened holes and plain webs, *Journal of Constructional Steel Research* 174 (2020) 106313.
- [15] K.J.R. Rasmussen, M. Khezri, B.W. Schafer, H. Zhang, The mechanics of built-up cold-formed steel members, *Thin-Walled Structures*. 154 (2020) 106756.
- [16] J. Dobrić, N. Gluhović, Z. Marković, D. Buđevac, Stability design criteria for closely spaced built-up stainless steel columns, *Building Materials and Structures*, vol. 64 (4) (2021) ISSN (online) 2335-0229, 235-250.
- [17] ST Vy, M. Mahendran, Behaviour and design of slender built-up nested cold-formed steel compression members, *Eng Struct.*, 241 (2021) 112446.
- [18] X. Zhou, Y. Xiang, Y. Shi, L. Xu, Y. Zou, Simplified design method of cold-formed steel columns with built-up box sections, *Eng Struct.*, 228 (2021) 111532.
- [19] S. Selvaraj, M. Madhavan, Design of cold-formed steel built-up columns subjected to local-global interactive buckling using direct strength method, *Thin-Walled Struct.*, 159 (2021) 107305.
- [20] D.K. Phan, K.J.R. Rasmussen, B.W. Schafer, Tests and design of built-up section columns, *J. Construct. Steel Res.* 181 (2021) 106619.
- [21] D.K. Phan, K.J.R. Rasmussen, B.W. Schafer, Numerical investigation of the strength and design of cold-formed steel built-up columns, *J. Construct. Steel Res.* 193 (2022) 107276.
- [22] H.D. Craveiro, R. Rahnavard, L. Laím, R.A. Simões, A. Santiago, Buckling behavior of closed built-up cold-formed steel columns under compression, *Thin-Walled Structures*. 179 (2022), 109493.
- [23] Q. Li, B. Young, Experimental and numerical investigation on cold-formed steel built-up section pin-ended columns, *Thin-Walled Structures*. 170 (2022) 108444.
- [24] S. Selvaraj, M. Madhavan, Experimental investigation and design considerations on cold-formed steel built-up I-section columns subjected to interactive buckling modes, *Thin-Walled Structures*. 175 (2022) 109262.
- [25] S. Selvaraj, M. Madhavan, Design of cold-formed steel built-up closed section columns using direct strength method, *Thin-Walled Structures*. 171 (2022), 108746.
- [26] S. Selvaraj, M. Madhavan, Proposal to Improve AISI S100 DSM Design Standards for Cold-Formed Steel Built-Up Closed Cross-Section Columns Subjected to Local-Global Interaction Buckling. *Journal of Structural Engineering (ASCE)*, 149 (10) (2023), <https://doi.org/10.1061/JSENDH/STENG-11916>.
- [27] S. Selvaraj, M. Madhavan, Structural Behaviour of Cold-Formed Steel Built-Up Closed Cross-section Columns - Assessing the Influence of Parameters and Design Methods, *Engineering Structures* 294 (2023) 116600.
- [28] K. Jiang, O. Zhao, Stainless steel built-up section stub columns: Testing, numerical modelling and design, *Thin-Walled Structures*. 191 (2023) 111070.
- [29] Y. Jing, K. Jiang, O. Zhao, L. Gardner, Web crippling of stainless steel built-up I-sections under End-Two-Flange loading: Tests, simulations and design, *Engineering Structures*. 304 (2024), 117576.
- [30] Y. Cui, J. Zhang, C. Ma, M. Niu, K. Jiang, S. Li, A. Su, Testing, numerical modelling and design of G550 high strength cold-formed steel built-up section columns, *Thin-Walled Structures*. 196 (2024), 111529.
- [31] C. Ma, C. Wei, K. Jiang, O. Zhao, A. Su, Experimental and numerical investigations of high-strength cold-formed steel multi-limb built-up section columns, *Engineering Structures*. 309 (2024), 118012.
- [32] F. Nishino, L. Tall, T. Okumura, Residual stress and torsional buckling strength of H and cruciform columns, *Trans. Jpn. Soc. Civ. Eng.* 1968 (160) (1968) 75-87.
- [33] R. Dabrowsi, On torsional stability of cruciform columns, *Journal of Constructional Steel Research*, 9(1) (1988) 51-59.
- [34] K.J. Rasmussen, G.J. Hancock, Plate slenderness limits for high strength steel sections, *J. Construct. Steel Res.* 23 (1-3) (1992) 73-96.
- [35] G. Chen, NS Trahair, Inelastic torsional buckling strengths of cruciform columns, *Engineering Structures*, 16(2) (1994) 83-90.
- [36] P.B. Dinis, D. Camotim, N. Silvestre, On the local and global buckling behaviour of angle, T-section and cruciform thin-walled members, *Thin-Walled Struct.* 48 (10-11) (2010) 786-797.
- [37] N.S. Trahair, Strength design of cruciform steel columns, *Engineering Structures*, 35 (2012) 307-313.
- [38] M. Komeda, J. Nie, K. Ono, T. Miyashita, S. Okada, An experimental study on buckling behavior of cruciform columns made of SBHS400, in: *The 29th International Ocean and Polar Engineering Conference, OnePetro*, 2019.
- [39] J. Nie, H. Sugimoto, K. Ono, T. Miyashita, M. Matsumura, S. Okada, An experimental study on the local buckling strength of an SBHS700 stub column with cruciform section: With an overview of the mechanical properties of SBHS700, *Steel Constr.* 12 (2) (2019) 82-90.
- [40] B. Behzadi-Sofiani, L. Gardner, M. Ahmer Wadee, Behaviour, finite element modelling and design of cruciform section steel columns. *Thin-Walled Structures* 182, (2023), 110124.
- [41] J. Dobrić, N. Gluhović, Ivanović J, Rossi B, Design procedures for cold-formed stainless steel built-up columns assembled from equal-leg angles, *J. Construct. Steel Res.* 212 (2024), 108263.
- [42] J. Dobrić, N. Gluhović, N. Fric, X. Ruan, M. Bock, B. Rossi, Buckling strengths of cold-formed built-up cruciform section columns under axial compression, *Thin-Walled Structures*, 200 (2024), 111879.
- [43] EN 10088-4: Stainless steels - Part 4: Technical delivery conditions for sheet/plate and strip of corrosion resisting steels for construction purposes. Brussels, Belgium, CEN 2009.

- [44] Eurocode 3: Design of steel structures – part 1-4: General rules – supplementary rules for stainless steels, including amendment A1 (2015), EN 1993-1-4:2006+A1:2015, Brussels, Belgium, CEN 2015.
- [45] Eurocode 3, Design of Steel Structures – Part 1-8: Design of joints EN 1993-1-8, CEN, Brussels, Belgium, 2005.
- [46] Schafer BW, Ádány S. Buckling analysis of cold-formed steel members using CUFSM: conventional and constrained finite strip methods. In: Proceedings of the eighteenth international speciality conference on cold-formed steel structures. Orlando (USA); 2006.
- [47] ABAQUS User Manual, Version 6.12, Providence, RI, USA: DS SIMULIA Corp; 2012.
- [48] EN ISO 6892-1. Metallic materials – Tensile testing. Part 1: Method of test at room temperature. Brussels, Belgium, CEN 2009.
- [49] I. Arrayago, E. Real, L. Gardner, Description of stress-strain curves for stainless steel alloys, Materials and Design. 87 (2015) 540-552.
- [50] Agisoft Metashape Professional (Version 2.2.0) 2024 Software Retrieved from: <https://www.agisoft.com>
- [51] A. Landesmann, D. Camotim, P.B. Dinis, R. Cruz, Short-to-intermediate slender pin-ended cold-formed steel equal-leg angle columns: Experimental investigation, numerical simulations and DSM design, Engineering Structures 132 (2017) 471–493.
- [52] B. Rossi, S. Afshan, L. Gardner, Strength enhancements in cold-formed structural sections – Part II: Predictive models, J. Constr. Steel Res., 83 (2013) 189–196.
- [53] S. Afshan, O. Zhao, L. Gardner, Standardised material properties for numerical parametric studies of stainless steel structures and buckling curves for tubular columns. J Construct Steel Res., 12 (2019) 2–11.
- [54] prEN 1993-1-14: Eurocode 3: Design of steel structures — Part 1-14: Design assisted by finite element analysis, draft version, Brussels, Belgium, CEN.
- [55] Eurocode 3: Design of steel structures – Part 1-5: Plated structural elements EN 1993-1-5, Brussels, Belgium, CEN 2006.
- [56] Eurocode 3: Design of steel structures – Part 1-1: General rules and rules for buildings EN 1993-1-1, Brussels, Belgium, CEN 2005.
- [57] EN 1990, Eurocode – Basis of structural design, European Committee for Standardisation (CEN), Brussels, Belgium, 2005.
- [58] S. Afshan, P. Francis, N. Baddoo, L. Gardner, Reliability analysis of structural stainless steel design provisions, J. Construct. Steel Res. 114 (2015) 293–304.
- [59] L.S. da Silva, T. Tankova, L. Marques, C. Rebelo, A. Taras, Standardisation of safety assessment procedures across brittle to ductile failure modes (SAFEBRITILE), European Commission, Directorate-General for Research and Innovation, 2018, Final document.

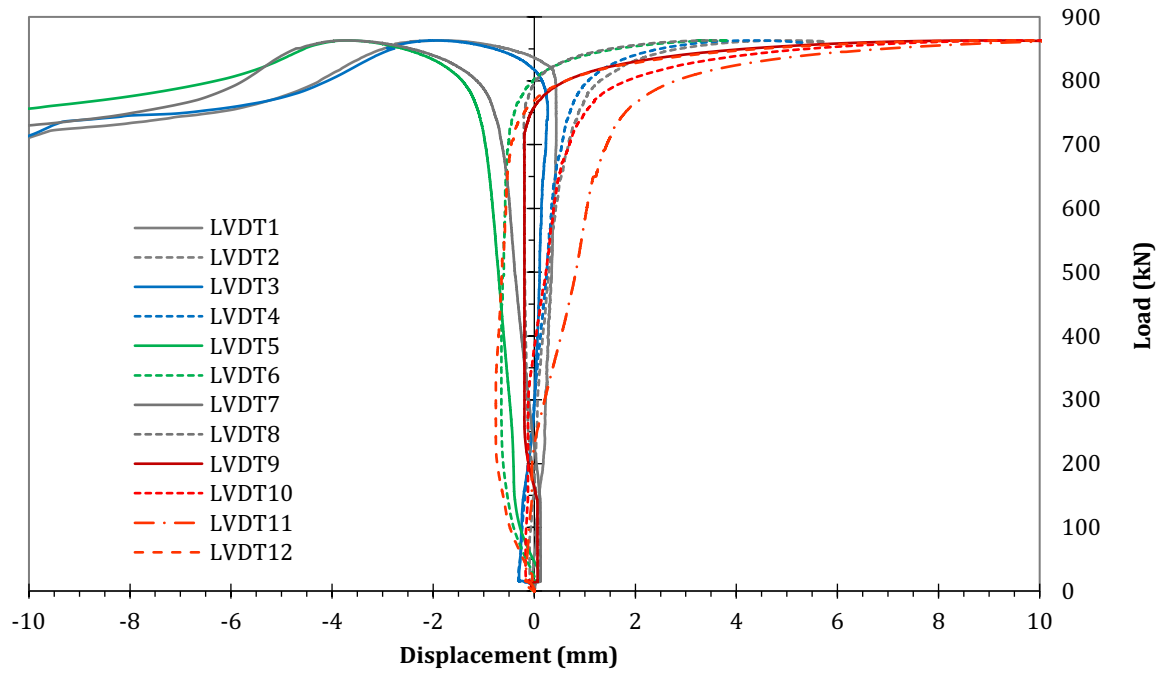


Fig. 13 Load—mid-height lateral displacement curves for specimen 1-4L50×135×3-2250-50-45.

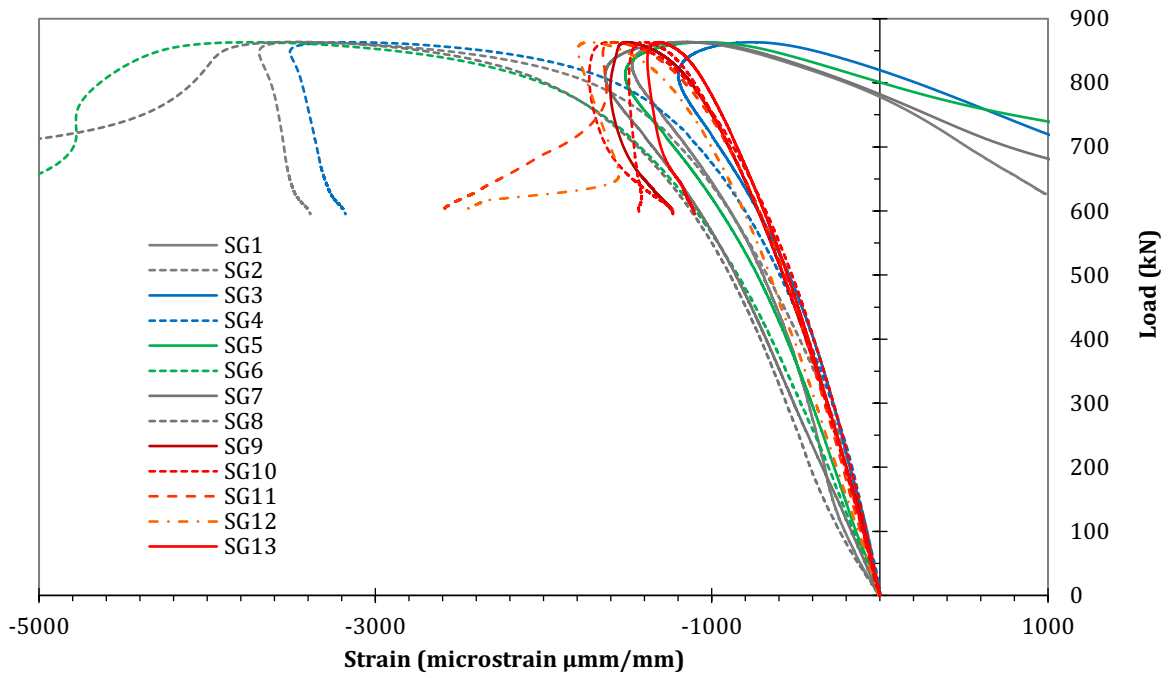


Fig. 14 Load—mid-height axial strain curves for specimen 1-4L50×135×3-2250-50-45.

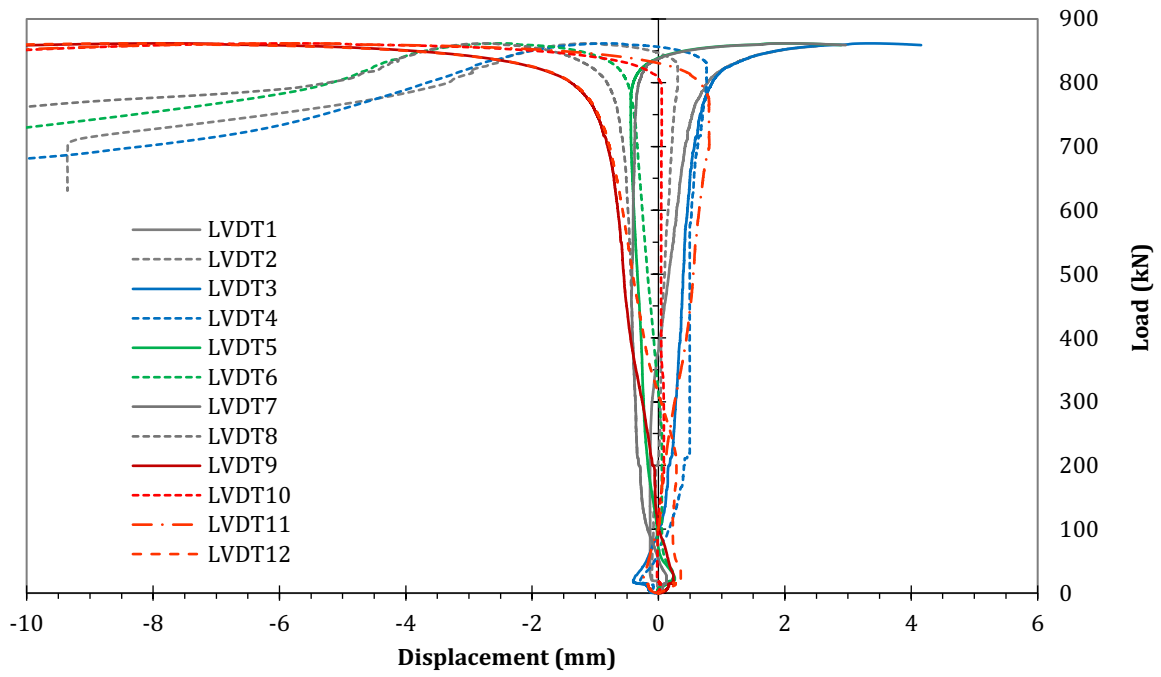


Fig. 15 Load—mid-height lateral displacement curves for specimen 2-4L50×135×3-2250-100-45.

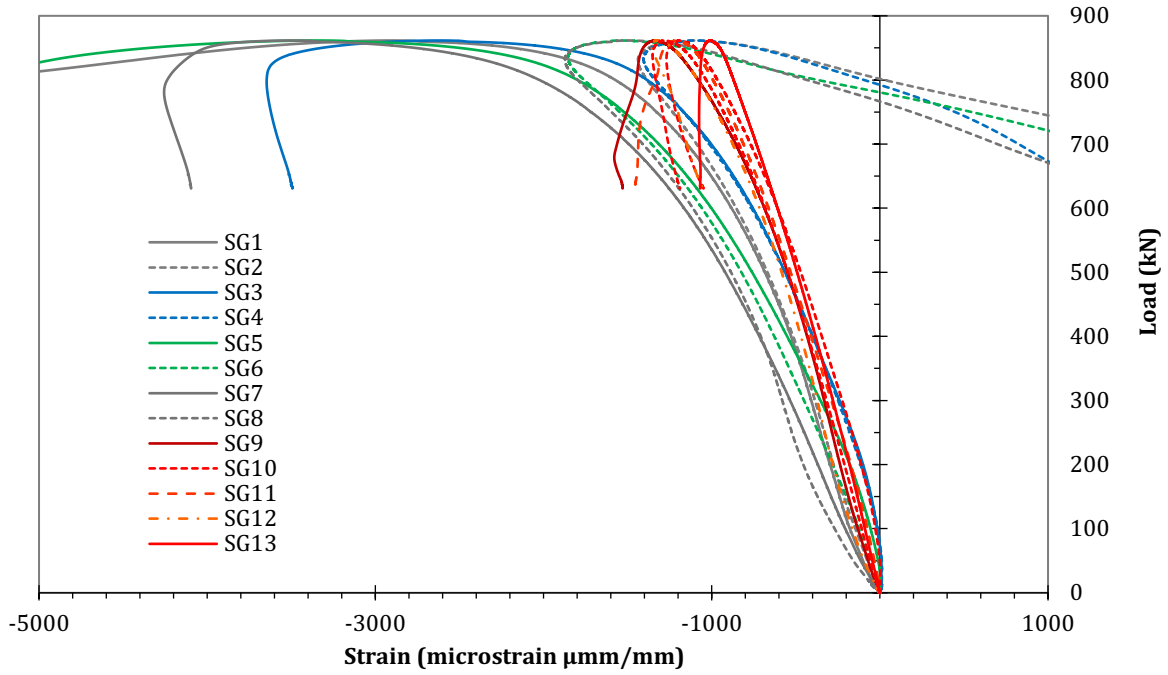


Fig. 16 Load—mid-height axial strain curves for specimen 2-4L50×135×3-2250-100-45.

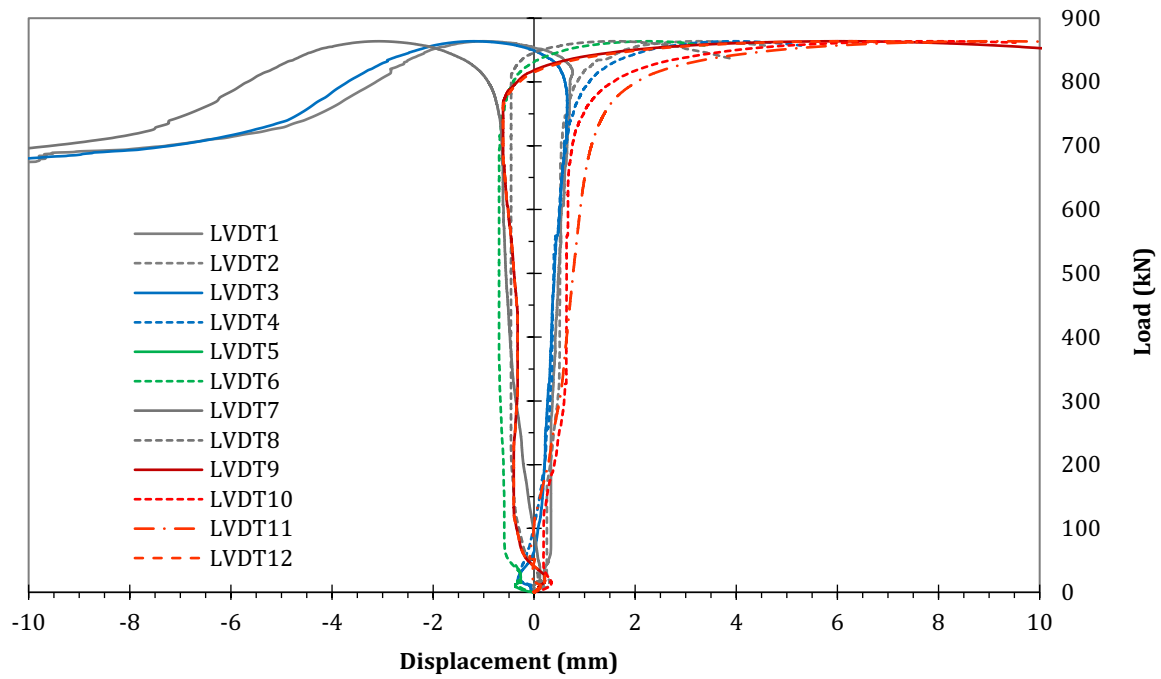


Fig. 17 Load–mid-height lateral displacement curves for specimen 3–4L50×135×3–2250–50–90.

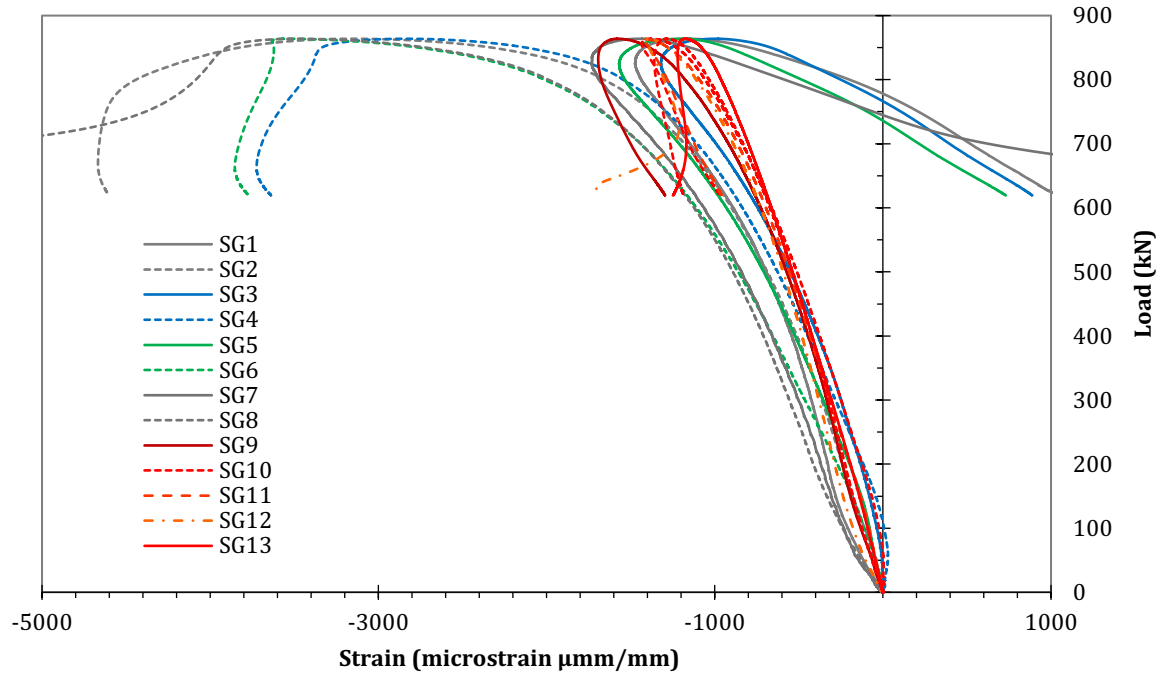


Fig. 18 Load–mid-height axial strain curves for specimen 3–4L50×135×3–2250–50–90.

