Published in : Steel Construction (2022), vol. 15, n°1, pp. 43-47
DOI: 10.1002/stco.202100051
Status : Postprint (Author's version)

COMPRESSION TESTS ON LARGE ANGLE COLUMNS IN HIGH-STRENGTH STEEL

Nominated for Eurosteel 2021 Best Paper Award

Marios-Zois Bezas, Jean-François Demonceau, Ioannis Vayas, Jean-Pierre Jaspart

Dr. Marios-Zois Bezas (corresponding author)
Marios-Zois.Bezas@uliege.be
University of Liege
Steel and Composite Construction
Allée de la Découverte 9 B52/3 Liege 4000, Belgium

Professor Jean-François Demonceau jfdemonceau@uliege.be
University of Liege
Steel and Composite Construction
Allée de la Découverte 9 B52/3 Liege 4000, Belgium

Professor Ioannis Vayas vastahl@central.ntua.gr National Technical University of Athens
Institute of Steel Structures
9 Iroon Polytechneiou str., Zografou Campus
Athens, 15780, Greece

Professor Jean-Pierre Jaspart jean-pierre.jaspart@uliege.be University of Liege
Steel and Composite Construction
Allée de la Découverte 9 B52/3
Liege 4000, Belgium

KEYWORDS : STABILITY; HIGH-STRENGTH STEEL; ANGLE CROSS-SECTIONS; BUCKLING; FINITE ELEMENTS ANALYSIS (FEM)

The paper addresses the instability of large angle columns in high-strength steel subjected to compression loads. It presents a number of compression tests on such columns with different eccentricities and slenderness and the resulting different buckling failure modes observed. The experimental campaign is intended to widen our understanding of the behaviour of highstrength steel columns with large angle sections in compression and bending and so to complement previous experimental studies. The tests were accompanied by numerical analyses and calculations of the load carrying capacities based on current Eurocode 3 design recommendations. The numerical simulations were conducted using the FEM software FINELG, which takes into account the influence of geometrical and material non-linearities. The numerical and analytical results are compared with the corresponding experimental ones. The experimental campaign and the numerical simulations were conducted as part of the ANGELHY project funded by the European Commission's Research Fund for Coal and Steel (RFCS).

Published in : Steel Construction (2022), vol. 15, n°1, pp. 43-47
DOI: 10.1002/stco.202100051
Status : Postprint (Author's version)

1. Introduction

Angles have been used since the very beginning of steel construction due to their easy transportation and on-site erection. They are extensively used in lattice towers and masts for telecommunication or electricity transmission, and in a wide range of civil engineering applications including buildings and bridges; they are also used to strengthen existing structures. Recent developments have led to a wider application of large angle sections made of high-strength steel, but there is a lack of consistent European rules for the design of members made of such angle profiles.

To widen our understanding of the instability of steel columns made from angle cross-sections subjected to compression and bending, twelve buckling tests on columns with large angle steel profiles were performed. The experiments were limited to S460M high-strength steel only, given the fact that a number of compression tests on angles with lower steel grades [1, 2] have already been performed by others. The test campaign was conducted at the University of Liège.

Details about the experimental campaign, such as the measurements before and during the tests as well as the test results, are presented in this report. Numerical simulations which take account of initial imperfections as well as geometrical and material non-linearities were performed using FINELG software, and the results were compared with the experimental ones. Finally, the ultimate test resistance was compared with the Eurocode predicted capacity. More details about the experimental campaign, the numerical and the analytical developments can be found in the original paper [3]. This research is part of the ANGELHY project [4] funded by the European Commission's Research Fund for Coal and Steel (RFCS).

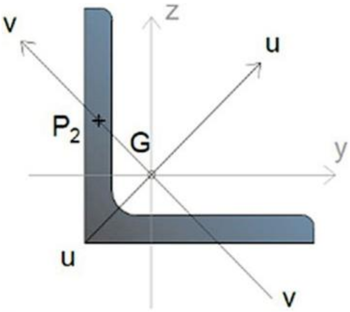
2. Details of test specimens

Two profiles from large angle cross-sections made of highstrength steel were selected for the experimental campaign. For each profile, six column tests were performed with three different lengths per profile and with two load application positions for each length. The selected load application points (Fig. 1) are the centre of gravity (G), which corresponds to pure compression in the angle, and the middle of the wall (P_2), which represents the position of the connecting bolt in actual structures. Tab. 1 provides details of the specimens.

To simplify the placement of the specimen in the test rig, constant dimensions were used for the end plates (made of S355) welded at the extremities of the angle members in all tests.

Published in : Steel Construction (2022), vol. 15, n°1, pp. 43-47
DOI: 10.1002/stco.202100051
Status : Postprint (Author's version)

Fig. 1 *Location of the load application points and definition of the axes*



G centre of gravity
P_2 middle of the wall
u-u major principal axis (weak axis displacement)
v-v minor principal axis (strong axis displacement)
y,z geometrical axes

Tab. 1 *Specimen details*

Specimen ID	Profile	Steel grade	Length [mm]	Eccentricity [mm]
Sp11	*L* 150×150×18	S460M	2500	0.00
Sp12	*L* 150×150×18	S460M	2500	e_v = 48.74
Sp13	*L* 150×150×18	S460M	3000	0.00
Sp14	*L* 150×150×18	S460M	3000	e_v = 48.74
Sp15	*L* 150×150×18	S460M	3500	0.00
Sp16	*L* 150×150×18	S460M	3500	e_v = 48.74
Sp21	*L* 200×200×16	S460M	3000	0.00
Sp22	*L* 200×200×16	S460M	3000	e_v = 66.64
Sp23	*L* 200×200×16	S460M	3500	0.00
Sp24	*L* 200×200×16	S460M	3500	e_v = 66.64
Sp25	*L* 200×200×16	S460M	4000	0.00
Sp26	*L* 200×200×16	S460M	4000	e_v = 66.64

3. Associated measurements

Before the tests, the actual geometrical dimensions of each angle section – the width and thickness of each leg – were measured at three points along the member length: 1/4, 1/2 and 3/4 of the total length.

Then, two measurements (M1, M2) on each external face and along the entire column length were taken to evaluate the initial deformation of the specimens. Fig. 2 shows the configuration of the test set-up. A measurement was taken every 50 mm along the column.

Published in : Steel Construction (2022), vol. 15, n°1, pp. 43-47
DOI: 10.1002/stco.202100051
Status : Postprint (Author's version)

An accurate comparison between the actual measured imperfections of the specimens and those assumed in the Eurocode is quite difficult to achieve. EN 1090-2 [5] prescribes that the deviation from straightness should be $\Delta \leq L$ [mm]/750, while prEN 1993-1-14 [6] states that 80 % of the geometric fabrication tolerances given in [5] should be applied. This leads to an initial bow imperfection of magnitude approximately equal to $L/1000$ and a deformation shape assumed to be similar to the first member instability mode, when in reality the shape is more complex. However, a rough check of the values indicates that for all specimens the measured imperfections are smaller than the geometrical tolerances prescribed in European regulations.

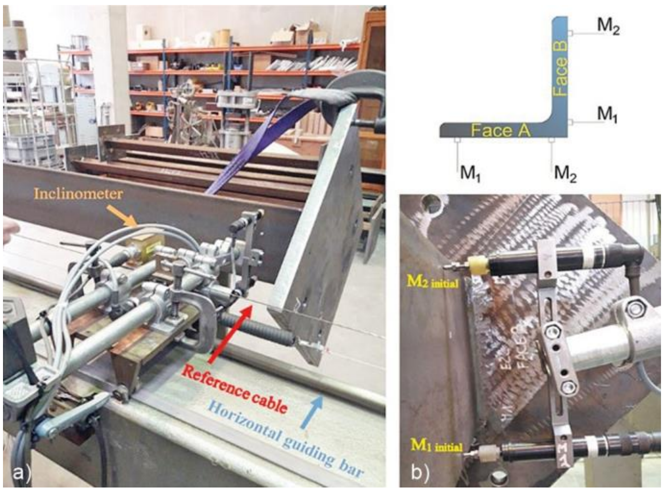
The tests were performed with the Amsler 500 testing machine (Fig. 3) which can apply a compression load up to 5000 kN. During the tests, the following displacements illustrated in Fig. 3 were measured:

- the vertical displacement C_1,
- four horizontal displacements C_2, C_3, C_4 and C_5 at the mid cross-section (1st pos.),
- four horizontal displacements C_6, C_7, C_8 and C_9 at the lower cross-section (2nd pos.).

In addition, strains at four points (l_1 to l_4) of the crosssection at mid-height of the columns were measured.

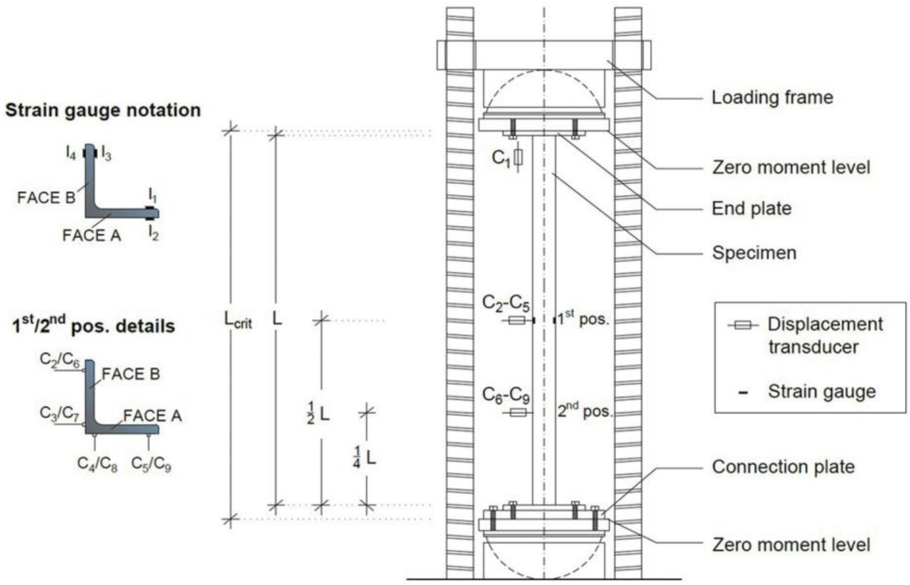
Finally, coupon tests were performed in accordance with ISO 6892-1 [7]. The samples were extracted from one of the extremities of the angle member after the buckling tests as recommended in ISO 377 [8]. It can be seen from the coupon tests that specimens Sp2# were made of grade S460 steel as initially planned, but specimens Sp1# were finally made of S420.

Fig. 2 *a) Measurement system for geometrical imperfections, b) detail and position of the displacement transducers*



Published in : Steel Construction (2022), vol. 15, n°1, pp. 43-47
DOI: 10.1002/stco.202100051
Status : Postprint (Author's version)

Fig. 3 *Sketch of Amsler 500 test machine and measurements during a test*



4. Results, comparisons and discussion

4.1. RESULTS OF THE EXPERIMENTAL TESTS

Figs. 4, 5 show the measured axial displacement versus the applied load for profiles L 150×150×18 and L 200×200×16 resp. Both figures indicate that the experimental results are in line with the physical expectations (influence of member length and of the eccentricity on the member stiffness and resistance properties).

For all the centrally loaded tests, the deflections along the weak axis increased significantly with the load until failure was reached by weak axis buckling. Specimens Sp11, Sp13 and Sp15 failed in a pure flexural buckling mode, while in specimens Sp21, Sp23 and Sp25, twist rotations were recorded in addition to weak axis deflections, thus revealing a flexural-torsional buckling mode.

The eccentrically loaded specimens were subjected initially to compression and strong axis bending. However, at larger load levels their tendency to buckle towards the weak axis led them to deflect towards both principal axes. In specimens Sp22, Sp24 and Sp26, these deflections were accompanied by significant twist rotations, clearly indicating failure with a flexural-torsional buckling mode. In contrast, twist rotations were small for specimens Sp12, Sp14 and Sp16, indicating a mixed mode between flexural and flexural-torsional buckling. Local buckling was not visibly observed in any specimen. More details about the results (initial measurements, imperfections, deflections, strains, twists) can be found in [9].

Published in : Steel Construction (2022), vol. 15, n°1, pp. 43-47
DOI: 10.1002/stco.202100051
Status : Postprint (Author's version)

***Fig. 4** Load vs. axial deformation of tested profiles L 150x150x18*

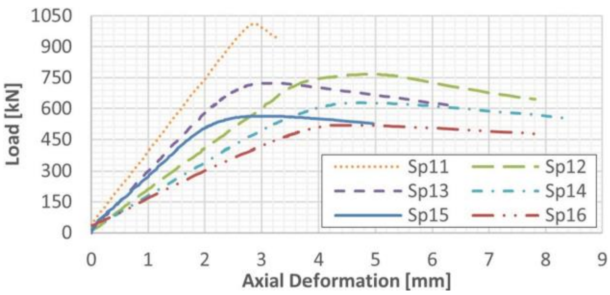
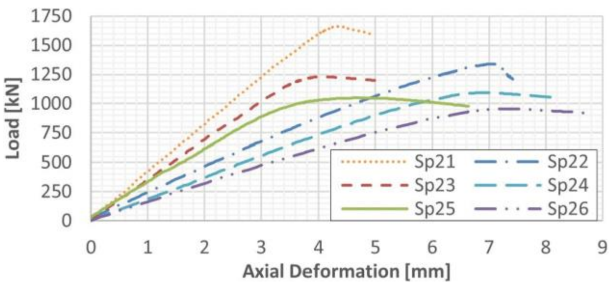


***Fig. 5** Load vs. axial deformation of tested profiles L 200x200x16*



4.2. COMPARISON WITH NUMERICAL SIMULATIONS

A comparison of the stiffness and ultimate resistance of the members obtained experimentally and by means of numerical simulations that take account of relevant imperfections as well as geometrical and material non-linearities is presented below. The numerical analyses of the test specimens were performed with the FINELG [10] non-linear finite element software using beam elements with seven DOF (including warping), given the fact that no local buckling took place during the tests. Only the column was simulated, while the end plates were considered indirectly (Fig. 6b). The columns are assumed to be pin-end members (as in the test rig) with free rotations at their extremities; only torsional rotation and warping were prevented.

The FINELG finite element analyses performed acc. to the GMNIA method taking into account:

– an initial member imperfection with shape and magnitude in accordance with the measured ones
– residual stresses (the pattern is shown in Fig. 6a and is derived from previous studies [11])
– a material law in accordance with the measured ones.

Published in : Steel Construction (2022), vol. 15, n°1, pp. 43-47
DOI: 10.1002/stco.202100051
Status : Postprint (Author's version)

Fig. 6 *a) Assumed distribution of residual stresses, b) FEM model used for Fig. 5 Load vs. axial deformation of tested profiles L 200×200×16 the numerical simulations*

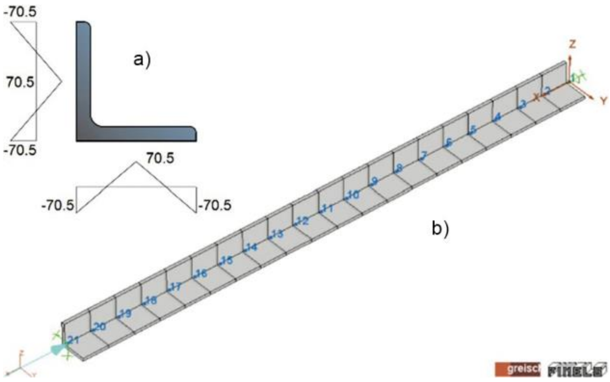


Fig. 7 *Comparison between test and FEM results for Sp1#*

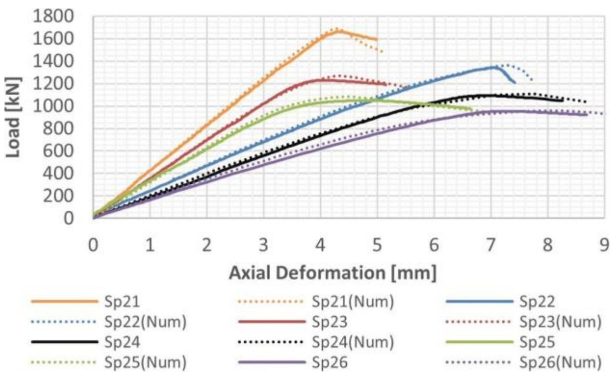
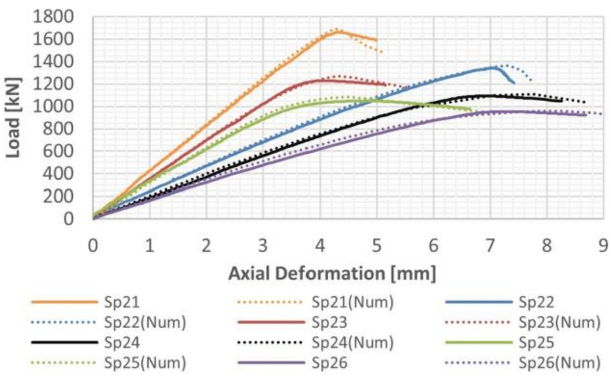


Fig. 8 *Comparison between test and FEM results for Sp2#*



4.3. COMPARISON WITH EUROCODE PREDICTIONS

A comparison between the ultimate resistance obtained by the tests and through EN1993-1-1 [12] design formulae was also undertaken for centrally loaded columns only. According to [12], the first profile (*L* 150×150×18) is classified as Class 1 and the second one (*L* 200×200×16) as Class 4. The procedure described in EN1993-1-5 [13] was followed to evaluate the effective cross-section of the

Published in : Steel Construction (2022), vol. 15, n°1, pp. 43-47
DOI: 10.1002/stco.202100051
Status : Postprint (Author's version)

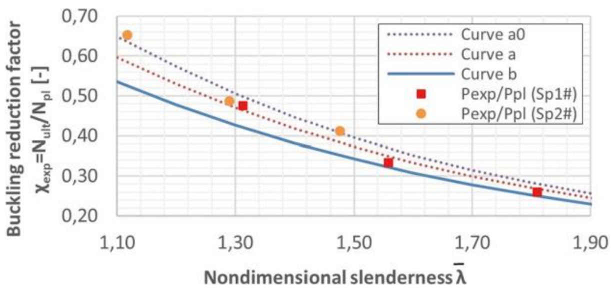
Class 4 profile, although as already stated, no plate buckling has been reported during the tests. Fig. 9 presents the experimental results compared with those obtained through the present recommendations of EN1993-1-1; reference buckling curves a_0, *a* and *b* are reported too**.** The non-dimensional slenderness is given by Eq. (1):

$$\bar{\lambda} = \sqrt{\frac{N_{pl}}{N_{cr}}} \tag{1}$$

where N_{cr} is the elastic critical load for the relevant buckling mode (i. e. minimum eigen value associated to all flexural and flexural-torsional modes). A pure torsional mode cannot be obtained for a centrally loaded angle column as explained in [9].

In EC 3, the buckling curve *b* was selected for axially loaded equal-leg angle columns. It has been found that the experimental results are in line with this curve or above. It can also be easily observed that the ultimate resistance is higher than the Eurocode predictions; the latter is deemed to provide safe evaluations, especially for specimens Sp2#, where the detrimental effects of local buckling are possibly overestimated.

Fig. 9 *Comparison of experimental results with buckling curves of EN1993-1*



5. Conclusions

In this paper the stability of steel columns with large angle high-strength steel profiles were investigated through twelve buckling tests. From this experimental programme and the accompanying numerical/analytical studies, the following conclusions may be drawn**:**

- The centrically loaded specimens with class 1 cross-section and the eccentrically loaded specimens with class 4 cross-section failed very clearly in a pure weak axis flexural buckling mode and correspondingly in a flexural-torsional one.
- The centrically loaded specimens with class 4 cross-section and the eccentrically loaded specimens with class 1 cross-section failed more or less in a flexural-torsional buckling mode, which was more pronounced in the former.
- Local buckling was not observed in any specimen, although some of them were categorized as class 4 acc. to EN1993-1-1.
- A very good agreement was found between the numerical GMNIA simulations and the experimental results in terms of stiffness and ultimate resistances.

Published in : Steel Construction (2022), vol. 15, n°1, pp. 43-47
DOI: 10.1002/stco.202100051
Status : Postprint (Author's version)

- The design resistance of the specimens based on EN1993-1-1 and EN1993-1-5 is on the safe side, especially for the second profile for which the local buckling reduction effects seem to be overestimated by Eurocode.

For information, further investigations achieved in the ANGELHY project have led to the proposal of a new consistent set of design formulae for cross-section classification and member stability; more details may be found in [14-16].

Acknowledgment

The work presented here has been carried out within the framework of a European Research project entitled ANGELHY "Innovative solutions for design and strengthening of telecommunications and transmission lattice towers using large angles from high-strength steel and hybrid techniques of angles with FRP strips", with a financial grant from the European Community's Research Fund for Coal and Steel (RFCS). The authors gratefully acknowledge this financial support.

Published in : Steel Construction (2022), vol. 15, n°1, pp. 43-47
DOI: 10.1002/stco.202100051
Status : Postprint (Author's version)

References

[1] Spiliopoulos, A.; Dasiou, M.-E.; Thanopoulos, P.; Vayas, I. (2018) Experimental tests on members made from rolled angle sections. Steel Construction – Design and Research 11, no. 1, pp. 84-93. https://doi.org/10.1002/stco.201710023

[2] Ban, H. Y.; Shi, G.; Shi, Y. J.; Wang, Y. Q. (2013) Column buckling tests of 420MPa high strength steel single equal angles. International Journal of Structural Stability and Dynamics 13, no. 2, 1250069.

[3] Bezas, M. Z.; Demonceau, J. F.; Vayas, I.; Jaspart, J. P. (2021) Experimental and numerical investigations on large angle high strength steel columns. Thin-walled structures 159C, pp. 107287.

[4] Vayas, I. et al. (2021) Telecommunication and transmission lattice towers from angle sections – the ANGELHY project. ce/papers 4, no. 2–4, pp. 210-217.

[5] ISO 1090-2 (2008) Technical requirements for the execution of steel structures. Berlin: Beuth.

[6] prEN1993-1-14 (2019) Eurocode 3, Design of steel structures. Berlin: Beuth.

[7] ISO 6892 – 1 (2016) Metallic materials – Tensile testing – Part 1: Method of test at room temperature. Berlin: Beuth.

[8] EN ISO 377 (1997) Steel and steel products – Location and preparation of samples and test pieces for mechanical testing. Berlin: Beuth.

[9] Bezas, M.-Z. (2021) Design of lattice towers from hot-rolled equal leg steel angles [PhD thesis]. University of Liège & National Technical University of Athens. http://hdl.handle. net/2268/262364

[10] Greisch Engineering [eds.] (2003) FINELG Non-linear finite element analysis program, User's manual, Version 9.0.

[11] Zhang, L.; Jaspart, J. P. (2013) Stability of members in compression made of large hot-rolled and welded angles. Université de Liège.

[12] EN1993-1-1 (2005) Design of steel structures. Part 1-1 General rules and rules for buildings. Berlin: Beuth.

[13] EN1993-1-5 (2006) Design of steel structures. Part 1-5 Plate structural elements. Berlin: Beuth.

[14] Bezas, M.-Z.; Demonceau, J.-F.; Vayas, I.; Jaspart, J.-P. (2021) Classification and cross-section resistance of equalleg rolled angle profiles. Journal of Constructional Steel Research 185, 106842. https://doi.org/10.1016/j.jcsr. 2021. 106842

[15] Bezas, M.-Z.; Demonceau, J.-F.; Vayas, I.; Jaspart, J.-P. (2022) Design rules for equal-leg angle members subjected to compression and bending. Journal of Constructional Steel Research 189, 107092. https://doi.org/10.1016/j.jcsr.2021.107092

[16] Bezas, M.-Z.; Jaspart, J.-P.; Vayas, I.; Demonceau, J.-F. (2022) Design recommendations for the stability of transmission steel lattice towers. Engineering Structures 252C, 113603. https://doi.org/10.1016/j.engstruct.2021.113603