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EFFECTS OF JOINTS ON THE BUCKLING RESISTANCE OF ANGLE MEMBERS MADE OF S460 STEEL - EXPERIMENTAL INVESTIGATIONS

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ABSTRACT

Energy production is undergoing radical changes in Europe, with the development of more sustainable production means. These changes require a complete overhaul of the power transmission network, with the construction of new lines and, in particular, new transmission towers with higher load-bearing capacities than in the past. To meet this demand, the design of new towers, typically made of steel angles, requires increasingly the use of high-strength S460 steel. In the literature several studies provide experimental evidence regarding the buckling resistance of pin-ended angle members made of high strength steel, allowing the validation of the relevant design procedures. However, this is not the case for high strength steel angles with bolted connections on one leg at their extremities, like those used for bracing members in lattice towers. To fill this gap, a research project called "New Steel" has been recently launched. This project involves investigations aiming to study the influence of the structural joints at the extremities of S460 angle members on their buckling resistance through experimental, numerical, and analytical studies. The present paper firstly describes an experimental campaign conducted in the framework of this project and summarises the obtained results. A critical discussion of the findings follows and finally, an assessment of the accuracy of the design methods as recommended in the existing normative documents (such as EN 50341-1 and prEN 1993-3) is presented through comparisons to the experimental results.

Keywords: Angle profile, high-strength steel, bracing members, lattice towers.

1 INTRODUCTION

Despite their geometrical simplicity, equal leg angles exhibit specificities, which distinguish them from other common, primarily double symmetric, cross-sections and lead to a complex structural behaviour. These features mainly originate from the monosymmetric nature of the angle profile and the non-coincidence of its shear centre with the centroid of the section. Moreover, due to the intersection of the profile's legs at a single point, angles possess negligible warping stiffness and consequently minute torsional stiffness, so they are highly susceptible to torsional effects.

Angles are widely used as bracing members in a variety of structures. Especially in lattice towers, they have extensive application due to their easy transportation and connectivity, qualities that facilitate their erection on rugged terrain where they frequently need to be built. Commonly, the connections of angle bracings are realised by bolting one of their legs to gusset plates or other structural members with one or multiple bolts. This type of connections provides elastic supports to the angle bracings, oblique to the principal axes of their cross-section and induces load eccentricities rendering the behaviour of such members even more complicated.

In Europe, the design of these members is traditionally executed using the European buckling curves with a corrected effective slenderness, as reflected in two relevant European normative standards, i.e. the prEN 1993-3 (1) and the EN 50341-1 (2). However, this method can lead in some cases to inaccurate results, particularly for members with one bolt connections (3–5). A new design approach, based on the interaction formulae method, has been recently developed in the framework of the ANGELHY RFCS project (6, 7) and included as Annex F in the new version of EN 1993-3-1 (8). Although promising, this method cannot be directly applied for the design of angle bracings since there is no guidance on how to evaluate the required critical buckling load (preliminary recommendations for the calculation of an effective length are given in (9)) and the developing bending moments, both greatly affected by the support conditions.

Recently, several research studies have dealt with angle members with bolted connections on one leg. The influence of these connections on the buckling resistance of the angles is examined through numerical and analytical methods in (3, 5), experimental evidence is provided by tests conducted on single angles in (4, 10–12) and on subassemblies of towers made of angle members in (13, 14), and design recommendations for such members are proposed in (9, 15).

Nevertheless, to the knowledge of the authors, there are no studies examining the behaviour of high strength steel angles with support conditions like those used for bracing members in lattice towers. Indeed, the above-mentioned references are not addressing the use of high strength steels except for Qu et al. (11) who conducted tests on S460 angles, but with support conditions at the extremities of the members which are not representative of those of angle bracings. The use of high strength steel, though, in lattice towers has become, if not necessary, at least attractive for the industry due to the rising need for erection of new transmission towers with increased bearing capacities when compared to the past, to cover the increased power transport requests. By using high strength steel, the new towers could bear higher loads without increasing substantially their dimensions and thus their visual impact on the landscape, so they can be more easily accepted by the citizens.

To fill this gap, a research project called "New steel" has been recently launched, funded by Elia Group (Transmission Operator Belgium) and ArcelorMittal, and developed in collaboration with the University of Liège. This project includes investigations on the buckling behaviour of S460 angles bolted on one leg at their extremities through experimental, numerical, and analytical studies. In the present paper, the experimental campaign conducted at the University of Liège in the framework of the aforementioned project is first described. Then, the main findings are summarised and discussed. Finally, the accuracy of the design methods as recommended in existing normative documents (1, 2) is assessed.

2 EXPERIMENTAL STUDY

The goals of this experimental study were firstly to improve the understanding of the buckling behaviour of angle members bolted on one leg at their extremities and secondly to generate reliable data of buckling resistances covering the use of the high strength S460 steel. This data will allow the validation of a numerical model which will be subsequently used in order to extend the database of buckling resistances.

To achieve these goals, the experimental campaign included three parameters of investigation: (a) the Class of the cross-section, (b) the number of the bolts at the end joints and (c) the slenderness of the member. More specifically, the investigation involved a compact (Class 2) L80x80x8 and a slender (Class 4) L90x90x6 cross-section, two types of joints with one or two bolts and three or four different lengths, ranging between 1105 and 3400 mm, for each cross-section and type of joint.

Table 1. Matrix of the experimental campaign

	Specimen	S11	S12	S13	S14	S21	S22	S23	S31	S32	S33	S34	S41	S42	S43
Investigation parameters	Profile	L80 x 80 x 8							L90 x 90 x 6						
	Number of bolts per joint	1				2			1				2		
	Member length [mm]	1400	1715	2270	2880	1300	2090	3020	1250	1840	2550	3260	1105	2340	3400

The buckling tests were complemented with detailed measurements of the actual dimensions of the specimens, coupon tests for material characterisation, measurements of the initial geometric imperfections and of the residual stresses.

The present paper will focus on the tests on the members made of the compact L80x80x8 cross-section.

2.1 Dimensions of the specimens

The dimensions of the cross-sections were measured at three positions along the member length, i.e. at L/4, L/2 and 3L/4. The minimum and maximum values of leg length and thickness were 78.2 and 79.4 mm and 8 and 8.5 mm, respectively, which are both within the acceptable tolerances prescribed in EN 10056-2 (16). The actual lengths of the specimens were also measured; the maximum difference from the nominal values was 1.6 mm or 0.1%. So, it was confirmed that the dimensions of the specimens were very close to the nominal ones.

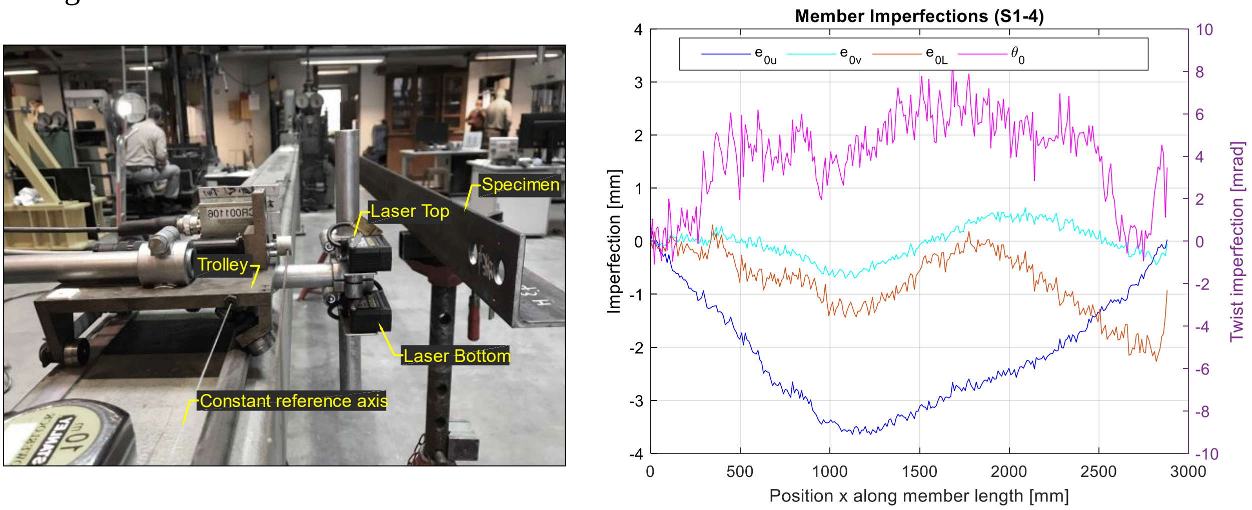
2.2 Material properties

Since all the members tested in this campaign were from the same production batch, two coupon tests, one for each leg of the profile, were carried out for the determination of the material properties. The testing procedure was in accordance with ISO-6892-1-2019 (17). The minimum measured yield stress and tensile strength were equal to 524 MPa and 645 MPa, respectively, conforming to the requirements of EN 10025-2(18) for steel grade S460. The mean measured modulus of elasticity was found equal to 214 GPa.

2.3 Measurement of initial imperfections

The initial geometric imperfections were measured with an in-house laser system which is shown in Fig. 1(a). For the determination of the imperfections of the whole profile, two scans per specimen were required, since in each scan the system measured the imperfections on two points, located near the tip and the heel, on one of the profile’s legs. The measurement step was approximately equal to 10 mm. The imperfection at each measurement point along the specimen was evaluated as the distance between this point and the vertical plane defined by two reference points near the heel of the profile, one at each end of the specimen.

The distribution of the various components of the global as well as the local imperfection along each specimen were derived through post-processing of the laser measurements. Indicative distributions of the imperfection components for a long member, where the influence of the imperfections on its buckling resistance is expected to be more significant, are presented in Fig. 1(b). The values of e_{0u} and e_{0v} correspond to the global flexural imperfections along the major and the minor axes of the cross-section respectively, θ_0 is the global torsional imperfection and e_{0L} is the local out-of-squareness imperfection. The out-of-straightness imperfection, defined as the resultant of the two global flexural imperfections, was quite small for all specimens, less than 4L/1000, thus within the acceptable range defined by EN 10056-2 (16). On the other hand, the out-of-squareness imperfection was for some of the specimens larger than the normative limit of 1mm (16). These deviations, though, are not expected to affect significantly the resistance of the members since being of Class 2 are not prone to local buckling.



(a) (b)

Fig. 1. (a) Geometric imperfections measuring system, (b) Distribution of geometric imperfection components along a slender specimen

2.4 Residual stresses

The residual stresses were measured following the sectioning method as described by Tegebte (19). Their distribution, derived by averaging the measurements on the two faces of the profile, is presented in Fig. 2. In the same Figure the measured residual stresses are compared to the distribution model proposed in (20) on which the normative standards (1, 2) are based and to an alternative 3-point model in which the reference value is not given as a function of the yield stress but is taken equal to 70 MPa. The latter model is considered more appropriate since the residual stresses are independent of the yield stress of the material. The agreement between the measured values and the theoretical models is not very good with the actual residual stresses being less severe than the theoretical ones.

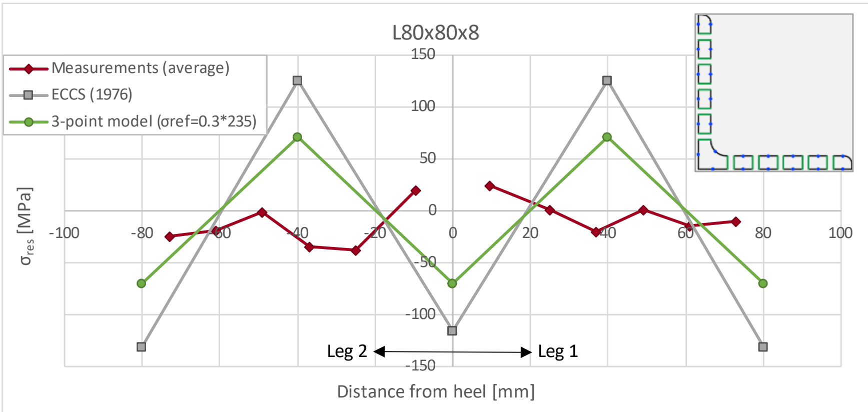
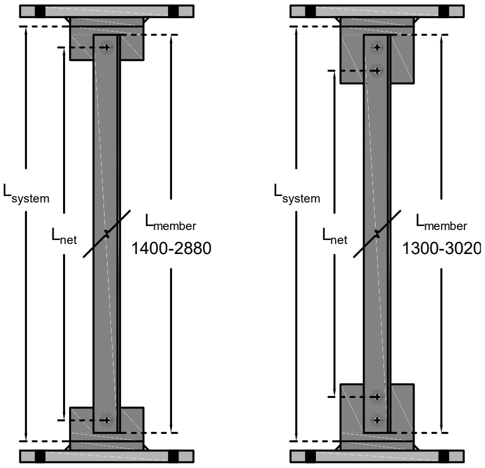


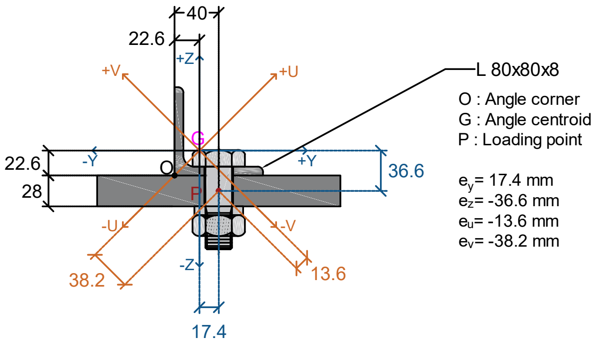
Fig. 2. Distribution of the measured residual stresses and comparison to theoretical models

2.5 Buckling tests

The specimens were bolted at their extremities to two thick (28 mm) supporting angles, serving as gusset plates, with one or two bolts as illustrated in Fig. 3. Thick gusset plates were selected in order to have limited bending deformation, which reflects the behaviour in real structures, where both tension and compression bracing members are commonly connected in the same joint. So, even though the gusset plates in real structures are much thinner, the coexistence of both tension and compression members in the same joint cancels out the developing bending moments due to the eccentric connections. The supporting angles were welded to end plates which were bolted to the testing machine with four bolts. Therefore, no relative rotation between the supporting angles and the machine was allowed.



(a)

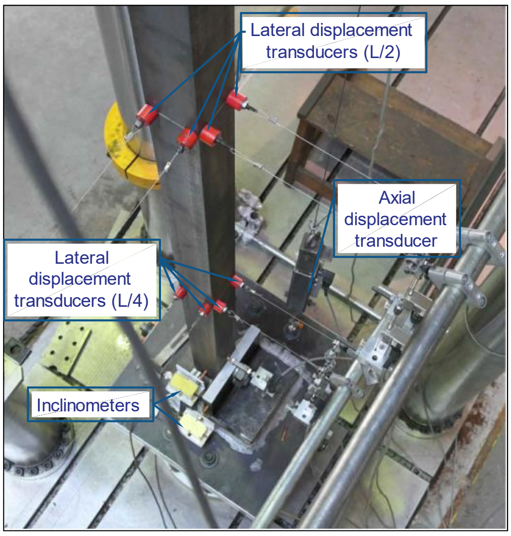


(b)

Fig. 3. Geometry of the specimens

The tests were carried out with a servo-hydraulic machine (Schenk Hydropuls, Germany) whose maximum static force capacity is 2000 kN and which can accommodate specimens with lengths in the range of 0.3 – 5.2 m (Fig. 4a). The load was applied at the top of the specimen under displacement control and the response of the specimens throughout the tests was monitored with several displacement transducers and inclinometers (Fig. 4b). The main values recorded during the tests were the applied load, the axial shortening, the lateral displacements at the mid-height and the quarter of the length and the rotations at the ends of the specimens. On both mid- and quarter-height cross-sections four displacement transducers were placed, so both the flexural displacements and the torsional twists could be measured. Inclinometers were placed both on the member ends and on the gusset plates, so their relative rotation could be evaluated. The inclinometers installed on the member recorded both in-plane and out-of-plane rotations, while those installed on the gusset plates recorded only out-of-plane rotations, since the plates were assumed rigid in their plane (the mentioned directions of the rotations refer to the plane of the gusset plates). The equivalent applied load was measured as reaction force at the bottom of the specimen with a load cell.

(a)



(b)

Fig. 4. (a) Test setup and b) instrumentation

The data obtained from the measurements were post-processed in order to derive the key deformation components (displacements along the principal axes and torsional rotations), which allow the characterisation of the failure mode. The response of four characteristic specimens in terms of equilibrium paths and deformed shapes of the critical cross-section, i.e. the cross-section at the mid-length of the specimen, is indicatively presented in Fig. 5. The equilibrium paths given in Fig. 5 correspond to the lateral displacements of the shear centre of the cross-section along the major (DU) and minor (DV) axis and the torsional twist (RX). The lateral displacements are positive if their direction is the same with the direction of the positive axes U and V, while the torsional twist is positive if it has a counterclockwise direction. The presented slenderness of the specimens has been calculated as the ratio of the system length (see Fig. 3a), which is 50 larger than the member length, over the radius of gyration of the cross-section, calculated taking into account its actual dimensions.

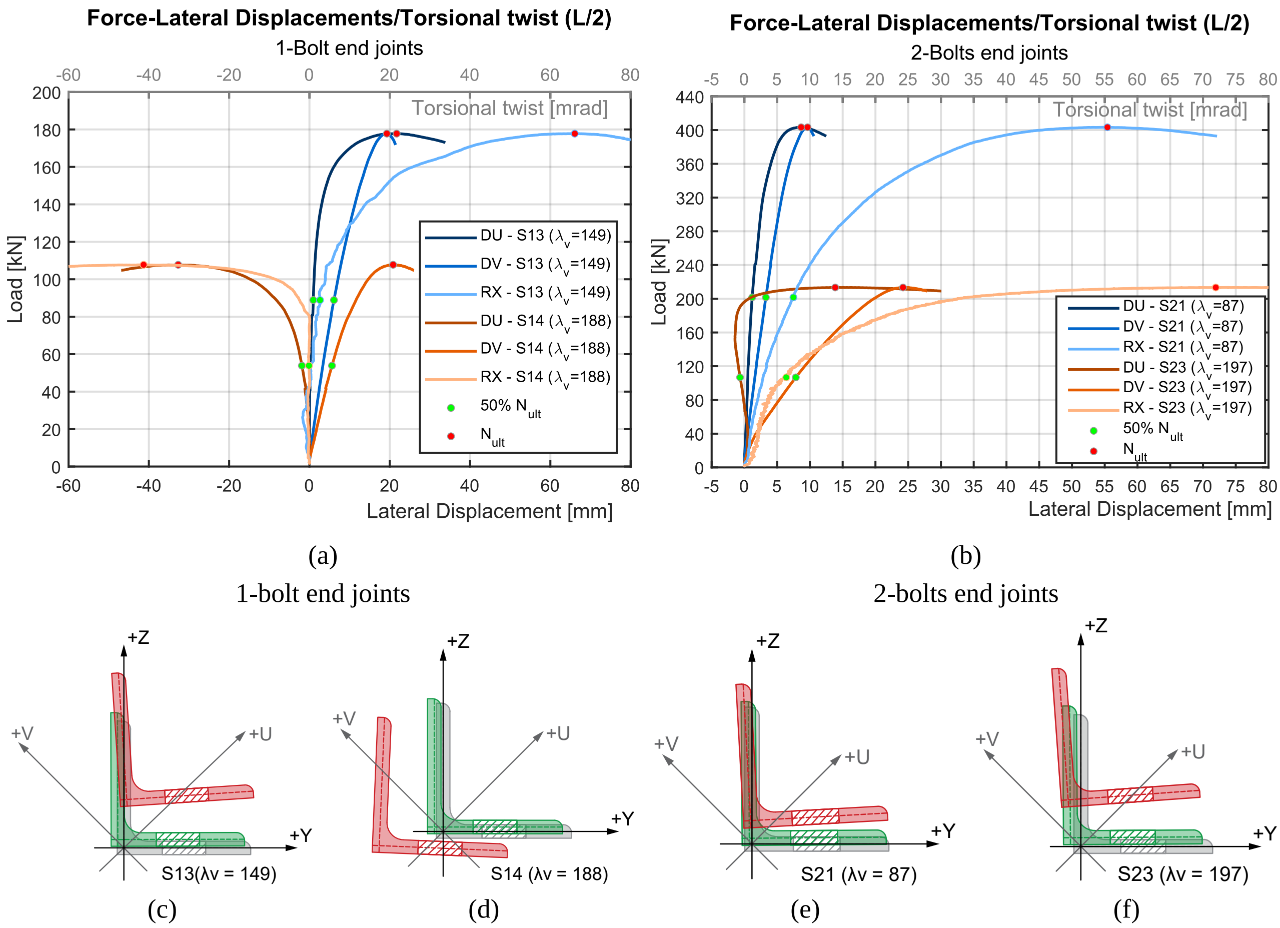


Fig. 5. Equilibrium paths (a, b) and deformed shapes of the mid-length cross-section (c - f) of the two longest specimens with 1-bolt end joints and of the shortest and longest specimens with 2-bolts end joints

Even from the beginning of the loading, lateral displacements along both principal axes appear. These displacements mainly result from the biaxial bending due to the loading eccentricity. Since the eccentricity along the minor axis is almost three times larger than the eccentricity along the major axis, the minor axis displacements were initially larger than the major axis ones. This was the case for all the specimens, except for the longest one with 1-bolt end joints, up to 99% of the ultimate load. However, just before the collapse, the displacements along the major axis increased rapidly due to the second order effects, especially for the longer members, which can be justified by the much lower flexural stiffness of the specimens about the minor axis. Finally, at collapse, the magnitudes of the lateral displacements along both principal axes were comparable for most of the specimens and, for three of them, the major axis displacement was larger than the minor axis one. In addition to lateral displacements, torsional twist started to increase since the beginning of the loading for all specimens.

This can be attributed to the eccentric nature of the end connections. The equilibrium path concerning torsional twist becomes highly nonlinear as the loading increases and, at collapse, significant twist appears for all specimens, even for the longer ones. For the specimens with 2-bolts end joints the torsional twist at collapse increases with the slenderness of the member. However, this is not the case for the 1-bolt end joints specimens.

Although all specimens displayed at collapse considerable lateral displacements along both principal axes as well as significant torsional twist, two distinct deformed shapes can be distinguished. In the first one, exhibited only by specimens with 1-bolt joints, the lateral displacement along the Y axis, parallel to the bolted leg of the specimen, prevailed as shown in Fig. 5(d). In the second one, which appeared in both 1-bolt and 2-bolts end joint specimens, the lateral displacement along the Z axis, parallel to the free leg of the specimen, was dominant (Fig. 5 c, e, f). Characteristic pictures of the specimens at the collapse are given in Fig. 6.

As expected from their design prior to testing, all the specimens failed due to buckling, except for the shortest one with 1-bolt end joints for which the joint resistance was exceeded before the failure of the member. This last specimen was decided to be tested, even though the normative joint resistance was lower than the buckling one, because there was evidence from the literature (4, 10) that EN 50341-1-J (2) overestimates the buckling resistance of short members with 1-bolt end connections. So, it was expected that the buckling failure would prevail, which was not the case, though, for the S460 angles tested in this campaign.

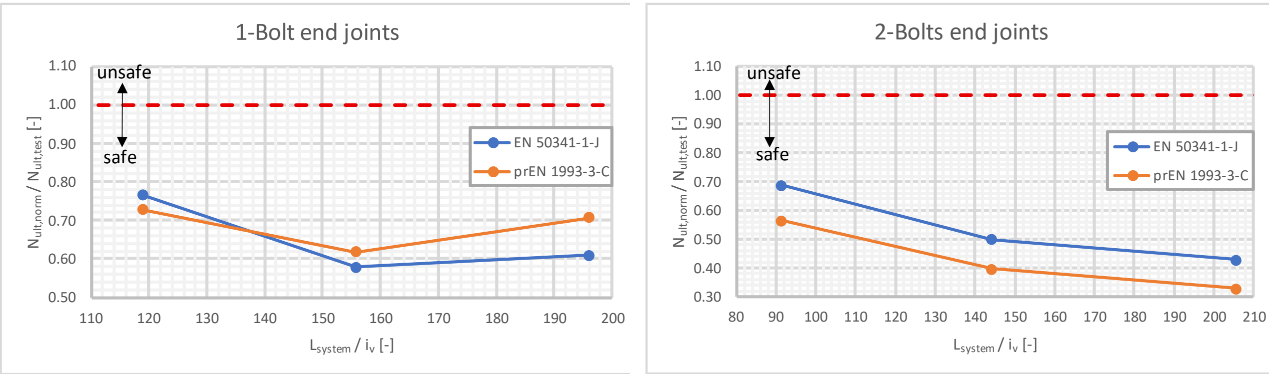
(a) (b) (c) (d)

Fig. 6. Pictures at collapse of the two longest specimens with 1-bolt end joints (a:S13, b:S14) and of the shortest (c:S21) and the longest (d:S23) specimens with 2-bolts end joints

3 ASSESSMENT OF EXISTING DESIGN RULES

In this section, the existing design rules provided by EN 50341-1-Annex J (2) in combination with the corresponding Belgian Annex (21) and by prEN 1993-3-Annex C (1) are assessed against the obtained experimental results. For the analytical calculations, the actual, measured, material properties and dimensions of the cross-sections are taken into account, while the system length (L_{system} = L_{member} + 50 mm, see Fig. 3a), is considered as reference length of the members. This length is deemed more representative of the distance between the end nodes of a structural beam element with which a bracing member is commonly modelled for tower design in accordance with code provisions.

The comparison between the analytical and experimental resistances is given in Fig. 7. Both codes provide safe prediction of the buckling resistance of the tested S460 angle members either they are connected with one or two bolts at their extremities ($N_{ult,norm}$ / $N_{ult,test}$ <1). However, the resistance predictions of the codes are quite conservative in the whole slenderness range, especially for members with 2-bolts end joints.



(a) (b)

Fig. 7. Comparison of the experimental to the analytical resistances for the specimens with (a) 1-bolt joints and (b) 2-bolts joints

4 CONCLUSIONS

An experimental campaign to investigate the buckling resistance of S460 angle members with one leg bolted connections at their extremities was carried out. Complementary measurements of the initial geometric imperfections and of the residual stresses of the tested members were also performed. It was found that the global out-of-straightness imperfections were quite smaller than the normative acceptable values (16). The local out-of-squareness imperfections of most of the specimens exceeded the normative limit (16), nevertheless, their impact on the buckling resistance of the members seems to be limited. The distribution of the residual stresses along the angle legs did not closely follow the model adopted by the design codes (1, 2); however, the maximum recorded residual stress was quite lower than the normative proposed value (20).

All the tested specimens failed due to buckling except for the shortest member with 1-bolt end joints whose joints resistance was exceeded prior the member failure. At collapse, all the members displayed considerable lateral displacements along both principal axes as well as torsional rotations, independently of their slenderness and the number of the bolts in their end joints. Two distinct deformed shapes at collapse were revealed: one for which the lateral displacement along the axis parallel to the bolted leg of the angle was leading and one for which the lateral displacement along the axis parallel to the free leg of the angle was dominant. The first one appeared only for members connected with one bolt at their ends, while the second appeared both in members with one and two-bolts end joints.

Finally, the accuracy of the design rules proposed in the relevant European design codes was assessed. Both prEN 1993-3-Annex C (1) and EN 50341-1-Annex J (2) seems to provide quite conservative resistance predictions in the whole investigated slenderness range for S460 angles either they are connected with one or two bolts, when compared to these experimental results.

The authors are working towards the improvement of the design rules concerning angles with one leg bolted end connections, aiming to the extension of the interaction formulae method included in the Annex F of prEN 1993-3 (1) in order to cover the design of such members. This is intended to be achieved by developing analytical formulae for the calculation of the critical buckling load and the developing bending moments due to the eccentricities accounting for the actual stiffness of the joints of the members.

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