ORIGINAL ARTICLE

Buckling resistance of angle bracing members with one-leg bolted end connections

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Abstract

Steel angle members are widely used for the construction of lattice towers, mainly because of easy assembly on site and compact transport. Bracing members, especially, are connected directly or through gusset plates to the adjacent members with one or more bolts on one leg only. This type of connection provides some rotational restraint at the ends of the bracing, but also induces eccentricities. This paper presents the European normative framework covering the design against buckling of angle bracing members in lattice towers and identifies the differences between the different design rules available in existing normative documents. These differences are quantified through worked examples and the normative design procedures are evaluated against experimental results from the literature. The influence on the buckling resistance of various parameters, such as the support conditions and the steel grade, covering members made of both conventional mild steel as well as high strength (S460) steel, is also studied and quantified. Finally, useful conclusions are drawn about the accuracy of the available design rules.

Keywords

steel angles, lattice towers, design rules, buckling resistance, bolted joints, high strength steel

1 Introduction

Angle members have a wide application as bracings in a variety of structures such as towers, buildings and bridges. Especially in lattice towers, they are used extensively because of their easy transportation and assembly on site. Usually, bracing members are connected at their ends to gusset plates or directly to other members with one or multiple bolts through one of their legs. This type of connection provides some rotational restraint which is beneficial for the buckling resistance of the members, but also induces eccentricities causing the development of bending moments which negatively affect the resistance. The existing normative design procedures consider the influence of the supports, but in most cases implicitly through auxiliary coefficients. In the literature, studies examining the behaviour of angle members connected through one leg are also available [1]–[4], some of them [5] proposing new design models as alternative to the normative ones.

This work compares the design procedures available in the European normative documents to each other and to experimental results found in the literature in order to evaluate their accuracy. Moreover, the influence of various support conditions and of the steel grade on the buckling resistance of angles is examined. The focus is given on single span members without intermediate restraints. Finally, some conclusions are drawn based on the aforementioned investigations and the subsequent research activities planned by the authors are described.

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2 Available design rules

For bracing members made of angle sections in lattice transmission towers, two European standards provide guidelines and rules for their design: prEN 1993-3 [6] (new draft of the EN 1993-3-1 [7]) and EN 50341-1 [8]. Both standards propose lattice towers to be modelled as trusses and to be analysed using a linear elastic approach. However, for the verification of the angle members two distinct methodologies can be distinguished.

In the first one, which is adopted by the Annex C of prEN 1993-3 [6] and Annex J of EN 50341-1 [8], angle members in lattice towers are considered as concentrically loaded, thus subjected exclusively to axial forces, and pin jointed at their both ends, so their buckling length is considered equal to their system length. The actual support conditions and the eccentricities are considered implicitly using an adapted effective slenderness. More precisely, in the expression of the effective relative slenderness, the beneficial effect of the actual end restraints is recognised by multiplying the relative slenderness of the member by a reduction factor, while the detrimental effect of the eccentricities is considered by adding a constant to the above-mentioned reduced slenderness term. According to the design procedure prescribed in Annex C of prEN 1993-3 [6] and Annex J of EN 50341-1 [8], this effective relative slenderness is used in combination with a design formula for members in compression only, through Eq. 1, where N_{Ed} is the applied axial force and $N_{b,Rd}$ the buckling resistance of the member.

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1 \tag{1}$$

Although Annex C of prEN 1993-3 [6] and Annex J of EN 50341-1 [8] follow the same concept, they also exhibit some differences:

(a) Each standard deals with the flexural-torsional buckling differently. The main text of prEN 1993-3 [6] specifies that flexural-torsional buckling can be disregarded for equal leg angles and suggests verification only against flexural buckling. Respectively, the main text of EN 50341-1 [8] states that torsional and flexural-torsional buckling of equal leg angles are covered by the provisions for plate buckling, so no additional check is required. However, the Belgian National Annex of EN 50341 [9] stipulates separate verifications against torsional and flexural-torsional buckling. Since torsional buckling can only appear when the member is made of a doubly symmetric cross-section and is loaded on the shear centre, which is not the case for angle bracing members, only flexural and flexural-torsional buckling should be considered when applying the design procedure of EN 50341-1 [8].

(b) Although for the calculation of the effective relative slenderness both standards provide expressions which have the same format, the expressions are different. Additionally, EN 50341-1 [8] defines a limit relative slenderness value, equal to $\sqrt{2}$ (based on observations from experimental tests conducted in the 80's), above and below of which the expression for the calculation of the effective relative slenderness differs for the same support conditions, while prEN 1993-3 [6] employs the same expression over the whole slenderness range.

(c) Each standard considers different buckling curves. More specifically, prEN 1993-3 [6] proposes buckling curve b to be used for angle sections made of steel up to S420 and curve a for higher steel grades, while EN 50341-1 [8] suggests buckling curve a_0 independently of the steel grade for flexural buckling. For flexural-torsional buckling, the Belgian National Annex of EN 50341 [9] proposes the use of buckling curve b.

(d) Annex C of prEN 1993-3 [6] introduces an additional reduction factor for the buckling resistance of bracing members in the cases where at least one end of the member is connected with one bolt only.

(e) The buckling design rules of the Annex J of EN 50341-1 [8] can only be applied provided that they are accompanied by full-scale tests.

The second methodology for the design of angle members, developed in the framework of the European RFCS project ANGELHY [10], [11] and recently incorporated in Annex F of prEN 1993-3 [6], approaches the problem in a different and more rational perspective. In fact, prEN 1993-3 (Annex F) [6] is the only European standard providing interaction formulae for the buckling verification of pinned ended angle members under combined axial compressive force and biaxial bending. The interaction formulae for angle members with profiles of Class 1-3 are given by Eq. 2. This way, the eccentricities at the extremities of the member are considered explicitly by including the resulting bending moments in the design process. On the other hand, the restraining effect of the actual end supports can be accounted for through appropriate buckling lengths, provided appropriate expressions for such effective lengths are available. So, different relative slenderness values according to the buckling planes could potentially be integrated as the restraining effects can be different in these planes. However, the standard does not provide guidance on the evaluation of the buckling lengths due to the current lack of knowledge; some preliminary proposals can be found in the literature [12]. Finally, it can be mentioned that flexural-torsional buckling need not be considered when the design procedure according to Annex F of prEN 1993-3 [6] is followed.

$$\left[\frac{N_{Ed}}{N_{bu,Rd}} + k_{uu}\frac{M_{u,Ed}}{M_{bu,Rd}}\right]^{\xi} + k_{uv}\frac{M_{v,Ed}}{M_{v,Rd}} \leq 1 \tag{2a}$$

$$\left[\frac{N_{Ed}}{N_{bv,Rd}} + k_{vu}\frac{M_{u,Ed}}{M_{bu,Rd}}\right]^{\xi} + k_{vv}\frac{M_{v,Ed}}{M_{v,Rd}} \leq 1 \tag{2b}$$

where N_{Ed}, $M_{u,Ed}$ and $M_{v,Ed}$ are the applied axial force and bending moments about the major and minor principal axes, respectively, $N_{bu,Rd}$ and $N_{bv,Rd}$ the flexural buckling resistance about each principal axis, $M_{bu,Rd}$ the lateral torsional buckling resistance, $M_{v,Rd}$ the minor bending moment resistance of the cross-section, k_{uu}, k_{uv}, k_{vu}, k_{vv} interaction factors and ξ an interaction factor depending on the cross-section class.

3 Comparison of the normative design procedures

In this section, the previously discussed design procedures are compared quantitatively through worked examples and evaluated against experimental results coming from tests performed at the University of Graz [2]. These tests examined angles with an L80x80x8 profile made of S275 (nominal steel grade) steel. The members were bolted to thick gusset plates (t_{gusset} = 25 mm) at their extremities with one or two bolts on one leg only, which were located at 45 mm from the angle heel. Both preloaded and non-preloaded bolts were used. The gusset plates were fixed at their external edge or had a knife edge type support preventing the rotation of the gusset plate in its plane but allowing the one out of its plane. The geometry of the specimens is shown in Fig. 1. The load during the test was applied on the centre of the upper gusset plate (point P in Fig. 1).

For comparison reasons, the same steel and geometry of the members were adopted in the worked examples. Additionally, supports with thin gusset plates were also considered in order to catch the full range of buckling resistances for a certain member length. In total, six different support conditions were examined in the worked examples. The designation of the support conditions is in the form of "X-Y-Z", where X indicates the number of the bolts in each end joint (X = 1B or 2B for one or two bolts respectively), Y indicates the thickness of the gusset plate (Y = TkG or TnG for thick and thin gusset plate) and Z indicates the support conditions of the gusset plate (Z = F or KE for fixed and knife edge support conditions, respectively).

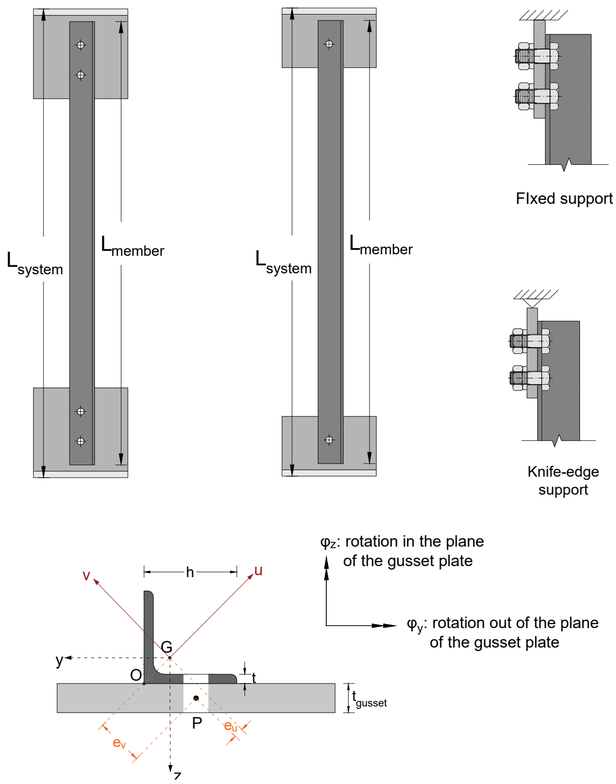

Figure 1 Geometry of the examined members

It has to be mentioned that only the design procedure according to Annex F of prEN 1993-3 [6] can take into account all these support conditions. Annex C of prEN 1993-3 [6] and Annex J of EN 50341-1 [8] provide different coefficients for the calculation of the effective slenderness for end joints with one or two bolts, but do not account for the stiffness of the gusset plate and its support neither the different eccentricity of the applied load depending on the thickness of the gusset plate. So, for the calculations according to these two design procedures all the examined members were considered pin-ended and axially loaded, while the buckling length was considered equal to the system length.

On the contrary, for the application of Annex F of prEN 1993-3 [6], simplified but realistic support conditions were considered for the analysis of the examined members. More specifically, it was assumed that only the number of bolts in each joint determines the rigidity of the joint in the plane of the gusset plate; if two bolts are used in each joint the rotation in the plane of the gusset plate (φ_z) is assumed fixed, while if only one bolt is used φ_z rotation is assumed free. Respectively, the rigidity of the joint out of the plane of the gusset plate is assumed to be determined by the thickness of the gusset plate and the support at its external edge. So, if the gusset plate is thick enough and fixed at its external edge the rotation out of the plane of the gusset plate (φ_y) is assumed fixed, while if it has a knife edge type support this φ_y rotation is assumed free. If the gusset plate of the joint is thin, the rotation out of its plane is assumed free, independently of the type of its support. In lack of analytical expressions providing the bending moments and the buckling length of eccentrically loaded members with oblique supports according to the geometric and not the principal axes of the cross-section, numerical analyses performed for their calculation using SOFiSTiK software [14]. More specifically, the bending moments due to support eccentricity were calculated through a Linear Elastic Analysis and the buckling length accounting for the support restraint was calculated through a Linear Buckling Analysis taking into account the 1st mode, where flexural buckling dominates.

The experimentally obtained resistances and the analytically calculated ones are compared in Fig. 2 and 3 for members connected with one or two bolts to each of their supports, respectively. In these figures, the Cross-Section Resistance (CSR) for each support condition (which affects the development of bending moments) calculated according to the simple interaction criterion proposed in [13] is also presented. Firstly, it can be observed that the resistance according to the examined design procedures vary significantly, especially for low slenderness members. The experimental results for members with 1-bolt end joints seem to follow the corresponding curves calculated according to Annex F of prEN 1993-3 [6]. On the contrary, the experimental results for the members with 2-bolts end joints do not match with any of the curves obtained with the analytical design procedures. The deviations in the experimentally obtained resistances for members with same length and geometry of the end joints is due to the preloading or not of the bolts in the end joints, which, as expected, affects more the slender members.

Comparing to the experimental resistances, Annex F of prEN 1993-3 [6] provides safe and quite accurate resistance predictions for all the specimens connected with one bolt to the gusset plate, either it has fixed or knife edge support conditions. For the specimens connected with two bolts to the gusset plate, Annex F of prEN 1993-3 [6] is unsafe for all the members with a fixed gusset plate and for the slenderest member connected to gusset plates with knife edge support conditions. This can be explained by the modelling assumptions. As mentioned above, the rotation in the plane of the gusset plate was considered fixed for members connected with two bolts per joint. However, in reality, this connection is not fully rigid, which affects unfavourably the resistance of the members. The authors are presently working on the development of accurate formulae for the prediction of the actual stiffness of common joints in lattice towers.

Concerning Annex C of prEN 1993-3 [6] and Annex J of EN 50341-1 [8], they both provide safe resistance predictions for specimens connected with two bolts to fixed gusset plates. They are also safe for the slender specimens connected with one bolt to fixed gusset plates but unsafe for those with low slenderness. When the gusset plate has knife edge support conditions, both codes are unsafe. This could be justified by the fact that the development of these two codes has been based on experimental results from tests on members in lattice towers sub-structures, where knife edge support conditions may be not realistic enough.

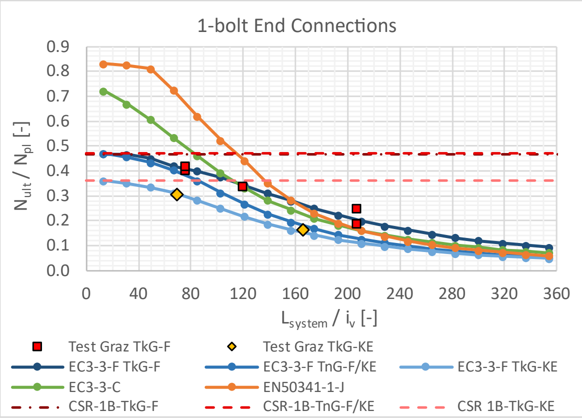

Figure 2 Buckling resistance of angle bracings with 1-bolt end joints according to various design procedures – comparison with experimental results [2] (N_{ult}: ultimate resistance)

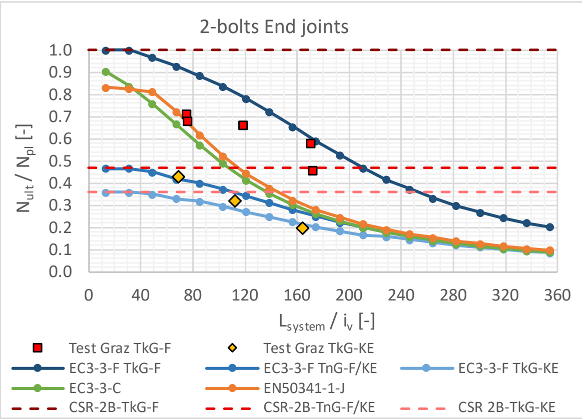

Figure 3 Buckling resistance of angle bracings with 2-bolts end joints according to various design procedures - comparison with experimental results [2] (N_{ult}: ultimate resistance)

4 Influence of the support conditions

The support conditions affect greatly the buckling resistance of eccentrically loaded angle members bolted on one leg only; they not only affect the buckling length of the member but also the developing bending moments. This section evaluates quantitatively, through the relevant standards, the influence of the end support conditions on the buckling resistance. Fig. 4-7 present the differences caused by the number of bolts, the thickness of the gusset plate and its support conditions, respectively, for three steel grades i.e., S235, S355 and S460. Generally, it can be observed that the differences in the resistance of the member due to the various support conditions are high even for short members and, as expected, they become even higher as the slenderness of the member increases. This is because the sensitivity of the buckling resistance to the support conditions increases with the slenderness of the member. Moreover, the differences are not the same for all examined steel grades. In most of the cases, the difference is higher for the higher steel grade.

The influence of the number of bolts is recognised by all the examined normative design procedures. As expected, the resistance is increased if two bolts are used in each end joint instead of only one. However, as shown in Fig. 4, the amount of increase depends (a) on the length of the member and (b) on the stiffness and the support of the gusset plate. Moreover, the increase predicted by each design procedure is also different. According to Annex F of prEN 1993-3 [6], the highest increase, reaching 170%, is obtained when the gusset plate is thick and fixed at the external edge. For these support conditions the maximum benefit appears for members with medium to high slenderness. For the cases where the gusset plate is thin or it has knife edge support conditions, the benefit in resistance increases with the slenderness of the member and reaches a maximum value of about 55%. Similarly, according to Annex J of EN 50341-1 [8], the gain in the resistance increases with the slenderness of the member and reaches the value of 40%. According to prEN 1993-3 Annex C, and on the contrary to the other two design approaches, the difference in the resistance between the members with one or two bolts end joints is independent of the slenderness of the member and equals to 25%.

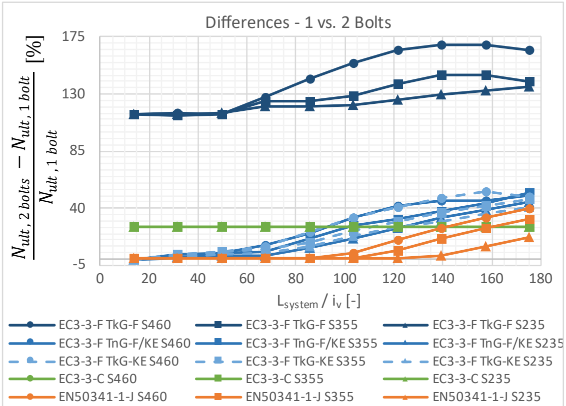

Figure 4 Differences in resistance between members connected to gusset plates with one or two bolts

The thickness of the gusset plate to which the angle bracings are bolted affects also significantly their buckling resistance. As shown in Fig. 5, when the gusset plate is fixed at its external edge, the thicker it is the higher the resistance of the member becomes. The difference in resistance increases with the slenderness of the member and can reach the values of 55% and 175% for end joints with one and two bolts, respectively. This increase in the resistance is due to the higher restraint provided by the joint since the bending stiffness of the gusset plate increases with the thickness, which reduces the buckling length of the member and the developing bending moments. These effects seem to surpass the negative influence of the increase of the eccentricity of the applied load associated to the increase of the gusset plate thickness.

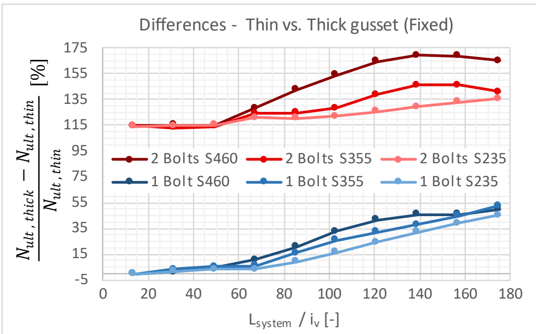

Figure 5 Differences in resistance between members connected to thick or thin gusset plates with fixed support conditions – Calculation according to prEN 1993-3 Annex F

On the contrary, when the gusset plate has knife edge support conditions, the resistance of the member reduces with the increase of the gusset plate thickness (Fig. 6) both for one bolt and two bolts end joints. The reason is that, in this case, only the eccentricity of the applied load increases with the increase of the gusset plate thickness. The negative effect of increasing the thickness of a gusset plate with knife edge support conditions on the resistance of the member, which can decrease the resistance up to 25%, reduces with the slenderness of the member.

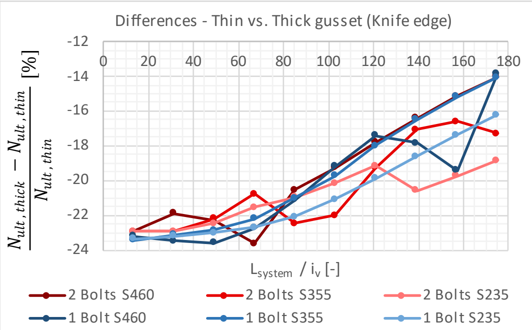

Figure 6 Differences in resistance between members connected to thick or thin gusset plates with knife edge support conditions – Calculation according to prEN 1993-3 Annex F

Finally, the influence of the support of the gusset plate on the resistance of the member is examined. As illustrated in Fig. 7, the resistance can be increased up to 220% for connections with two bolts and up to 80% for one bolt connections if the support of the gusset plate is fixed instead of knife edge.

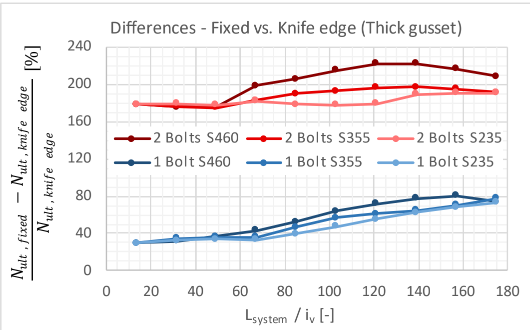

Figure 7 Differences in resistance between members connected to thick gusset plates with fixed or knife edge support conditions – Calculation according to prEN 1993-3 Annex F

5 Influence of steel grade

The successive increase of yield strength between the commonly used in lattice towers steel grades S235, S355 and the recent S460 is about 50% and 30%, respectively. However, it is expected that the benefit from upgrading the steel quality in the buckling resistance diminishes as the slenderness of the member increases. Apart from the slenderness of the member, the benefit from upgrading the steel quality depends on the support conditions and the design procedure according to which the resistance is evaluated, as shown in Fig. 8-11.

More specifically, if the resistance is calculated according to Annex F of prEN 1993-3 [6], the benefit from upgrading the steel quality from S235 to S355 ranges approximately between 12-51% (Fig. 8). Respectively, the benefit from upgrading S355 to S460 ranges between 6-32% (Fig. 9). The gain in resistance from upgrading the steel quality varies for different support conditions, but no clear trend can be identified.

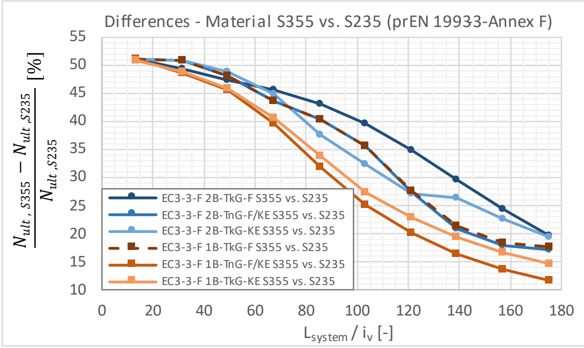

Figure 8 Differences in resistance between members made of S235 and S355 steel for various support conditions – Calculation according to prEN 1993-3 Annex F

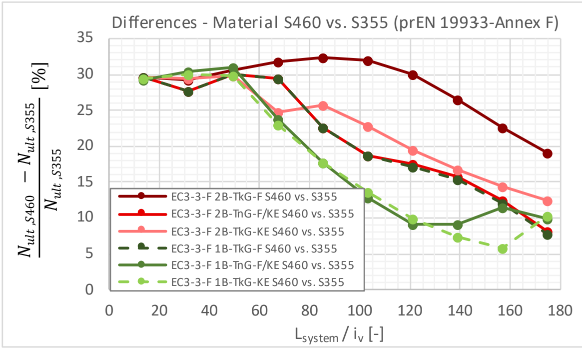

Figure 9 Differences in resistance between members made of S355 and S460 steel for various support conditions – Calculation according to prEN 1993-3 Annex F

If the resistance of the member is calculated according to Annex C of prEN 1993-3 [6], the benefit from upgrading the steel quality from S235 to S355 ranges between 13-50%, while the corresponding range from upgrading S355 to S460 is 13-36% (Fig. 10). This benefit is independent of the number of bolts in the end joints, because the latter is considered only through the constant factor ω (ω=0.8 or 1.0 for 1 or 2-bolts end joints, respectively).

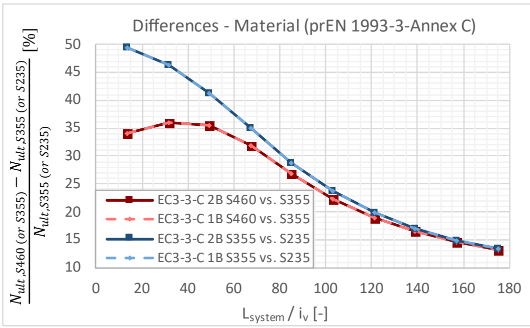

Figure 10 Differences in resistance between members made of S235, S355 and S460 steel for various support conditions – Calculation according to prEN 1993-3 Annex C

Finally, if the resistance of the member is calculated according to Annex J of EN 50341-1 [8], the benefit from upgrading the steel quality from S235 to S355 ranges approximately between 2-39% and 14-39% for members with one and two-bolts end joints, respectively. The corresponding ranges from upgrading the steel quality from S355 to S460 are 1-20% and 7-20%.

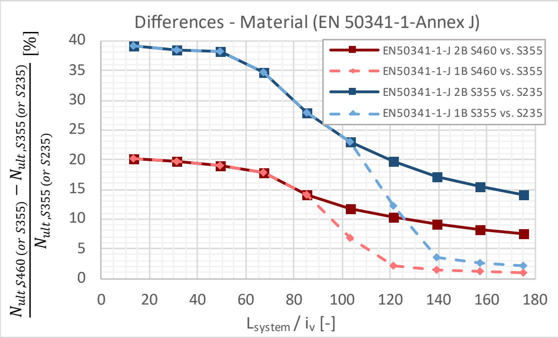

Figure 11 Differences in resistance between members made of S235, S355 and S460 steel for various support conditions – Calculation according to EN 50341-1 Annex J

From the above investigations, it can be concluded that by upgrading the steel quality the buckling resistance can be enhanced efficiently only for members of medium to low slenderness (L_{system}/i_v <100). To increase the resistance of slenderer members, alternative measures, like increasing the thickness of the profile or addition of redundant members, could be more competent.

6 Conclusions

In the presented study, various design procedures for angle members connected on one leg were compared and evaluated against experimental results found in the literature [2]. Moreover, the influence of various parameters on the resistance of angle members has been investigated such as the number of bolts per end joint, the thickness and support conditions of the gusset plates, the slenderness of the member and the angle steel grade.

Among the examined design procedures, the one given in Annex F of prEN 1993-3 [6] is more adaptable to different support conditions and seems to provide more accurate predictions compared to the experimental results. The unsafe resistance predictions for the members connected with two bolts to fixed gusset plates can be justified by the assumption of fully rigid support end conditions while this does not properly reflect the reality. The authors are working on the development of appropriate formulae to predict the rotational stiffness of commonly used joints in lattice towers, with the objective to integrate the actual support conditions in the design procedure. This can be achieved through the proposal of accurate evaluation methods of (i) the buckling length and (ii) the bending moments developing at the member ends and the implementation of these two values into the design procedures given in Annex F of prEN 1993-3 [6].

For the considered worked examples, Annex C of prEN 1993-3 [6] and Annex J of EN 50341-1 [8] provide safe, but in many cases very conservative, predictions of the resistance for members connected with two bolts to fixed gusset plates. Conversely, they are unconservative for all the members connected to gusset plates with knife edge support conditions. For members connected with one bolt to fixed gusset plates, the resistance predictions obtained by applying these design procedures are safe for slender members, but unsafe for those members with low slenderness.

Following the first study presented within this paper, the authors will experimentally investigate the influence of the number of bolts of the end joints on the resistance of S460 angle profiles considering members of varying slenderness, while the other parameters will be examined numerically.

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