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Buckling Resistance of Bolted Steel Angles–Numerical Study and Design Rules

Sofia Antonodimitraki[1,2] | Pavlos Thanopoulos[2] | Ioannis Vayas[2] | Jan Maesschalck[3] | Jean-Francois Demonceau[1]

Correspondence

Sofia Antonodimitraki
University of Liege,
Faculty of Applied Sciences
ArGEnCo Department
Quartier Polytech 1, Allée de la découverte, 9, B52/3,
4000 Liège, Belgium
Email: santonodimitraki@uliege.be

[1] Steel and Composite Construction, UEE Research Unit, Liège University, Liège, Belgium
[2] Institute of Steel Structures, National Technical University of Athens, Athens, Greece
[3] Elia Transmission Belgium SA/NV, Brussels, Belgium

Abstract

Steel angle members bolted through one leg at their extremities are commonly met in lattice towers, where they are mainly used as bracings. Despite their practical relevance, their design relies on empirical expressions due to the incomplete understanding of their structural behaviour. This paper investigates the influence of the end joints on the buckling resistance of such members. A numerical model, validated against experimental results is employed for a parametric study to evaluate the impact on the ultimate buckling load of key parameters, including the size of the profile, the slenderness of the member and the number of the bolts at the end connections. The results highlight the significant influence of the number of the bolts, of the joint plasticity and the amplitude of imperfections on the buckling resistance. Comparisons with the predictions of the European normative standards for lattice towers, i.e. EN1993-3-1 and EN 50341-1 as well as the new version prEN 1993-3, reveal considerable discrepancies, with the normative predictions being both conservative and unsafe. The outcomes of the study highlight areas where existing standards may be insufficient and provide a basis for improving design recommendations to enhance safety and optimise material use.

Keywords

steel angles, lattice towers, buckling, bolted joints, high strength steel, Eurocode 3

1 Introduction

Owing to their simple geometry, ease of fabrication, high connectivity, and compactness in storage and transportation, steel angles are highly popular in the construction sector—particularly in power transmission and telecommunication lattice towers. When used as bracing members, they are typically connected through one leg to the adjoining structural elements. This eccentric connection, oblique to the principal axes of the cross-section, further complicates their already complex structural behaviour. The response of such members under compression is still not fully comprehended, therefore their design remains largely empirical, as reflected in the relevant European design standards, namely EN 1993-3-1 [1] and EN 50341-1 [2].

The expansion of renewable energy sources and 5G technology has increased demands on power transmission and telecommunications infrastructure. Heavier conductors and larger antennas require towers with greater bearing capacity. As a result, the use of high strength steel in tower construction is becoming increasingly appealing—if not essential—for the industry. However, research into the behaviour and design of high strength steel angles with one leg-supports like those of bracing members in lattice towers is still very limited [3].

To address this gap, the present study investigates the behaviour of one-leg bolted high strength steel angles under compression and evaluates the accuracy of the buckling verification expressions provided by existing normative documents. Specifically, a finite element model is developed, validated against experimental data, and used for a parametric study to expand the database of buckling resistances. Parameters affecting the resistance of these members are discussed, and the accuracy of the design procedures, outlined in EN 1993-3-1 [1] and EN 50341-1 [2] as well as in the new draft version of the former, prEN 1993-3 [4], is assessed through comparisons with the numerical results. The findings highlight areas where existing standards may be insufficient, providing a basis for improving design recommendations to enhance safety and

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optimize material use.

2 Finite element modelling

The numerical model was developed with the commercial software ABAQUS [5], replicating the actual setup of the specimens during the experimental campaign [6]. It was composed of the angle member, two gusset plates and two or four bolts depending on the joint typology. Figure 1 illustrates (a) the experimental setup of a specimen with one-bolt end joints and (b) the geometry of the numerical models both with one and two-bolt end joints. As shown, the supporting angles were welded to the end plates which were rigidly connected to the test rig. No relative rotation between them was allowed, therefore only the free leg of the supporting angles was modelled as a gusset plate.

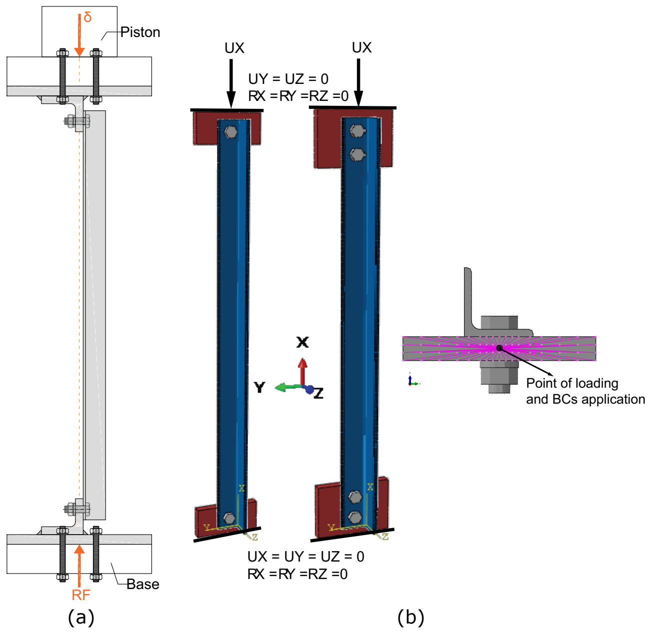

Figure 1 (a) Experimental setup, (b) Numerical models

The angle members, having a nominal cross-section of L80x80x8 and lengths according to Table 1, were modelled considering the actual dimensions measured in the laboratory. The measured dimensions were also used for the gusset plates, while the nominal ones were used for the bolts (M24).

All parts were modelled with a linear elastic – perfectly plastic material model without strain hardening but with a nominal slope for numerical stability (model (b) of EN 1993-1-14 [7]). The measured values of material properties, obtained from coupon tests, were used for the angle members and the gusset plates. For the bolts, in absence of measured data, the nominal material properties were considered. The engineering material properties of all parts were translated into the form of true stress and true stain, which were used as input into the FE model.

The solid hexahedral C3D8I element from the software library was used for all the parts of the model. The maximum dimension of the elements in the longitudinal direction was 18 mm. The mesh density in the joint regions was increased, with the maximum element dimension being 7 mm. Along the thickness of the plates, 3 elements were used both for the gusset plates and the profile legs. Through mesh sensitivity studies, it was found that by doubling the mesh density, the difference in the ultimate resistance was less than 1%; so, the selected mesh density deemed to be acceptable.

The interactions between the different parts of the model were represented realistically by defining several contact surfaces. The hard contact formulation was used in the normal direction. In the tangential direction, the penalty formulation was adopted with a friction coefficient equal to 0.2 corresponding to untreated steel surfaces [8].

All nodes of the outer surfaces of the gusset plates were coupled to the central node, where the loading and the boundary conditions were introduced (see Figure 1b). The boundary conditions were defined in accordance with those realised in the experiments. Since the end plates of the supports were rigidly connected to the test rig, all degrees of freedom at the support points were restrained except for the translation along the longitudinal axis at the top support point on which the loading was applied. The loading was introduced as an imposed displacement parallel to the longitudinal axis of the member as done during the physical test. The application of the loading and the boundary conditions is illustrated in Figure 1b.

The residual stresses were modelled using the 4-point model proposed in [9] with an amplitude equal to the maximum average measured value (Figure 2). This model was adopted instead of the measured distribution of the residual stresses, because the latter was not in self-equilibrium due to inherent inaccuracies in the laboratory measurements. The 4-point model was preferred over the more traditional 3-point one because it better matches the experimental data [6].

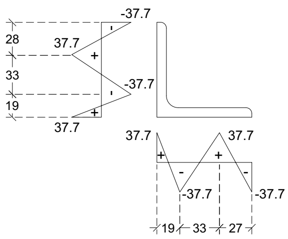

Figure 2 Residual stresses model

The geometric imperfections were introduced in the model using the shapes of the relevant buckling modes. In most cases, the 1st buckling mode was the most representative. The amplitude of the imperfection was defined as the sum of the maximum measured global imperfection of the member and the additional displacement associated to an out-of-squareness imperfection observed at the supports.

Two types of analysis were executed: a linear buckling analysis (LBA) in order to obtain the shape of the geometric imperfections and a geometrically and materially nonlinear analysis with imperfections (GMNIA) from which the ultimate load and the failure mode were identified. For the analyses the implicit solver of the software was employed.

3 Validation of the numerical model

The accuracy of the numerical model was validated against experimental data collected by the authors and presented in [6]. The main properties of the tested specimens are presented in Table 1. The validation was conducted in terms of ultimate resistance, failure mode and initial stiffness.

Table 1 Lengths and number of bolts per joint of the tested specimens

Specimen	S1-1	S1-2	S1-3	S1-4	S2-1	S2-2	S2-3
Bolts / joint	1				2		
Member length [mm]	1400	1715	2270	2880	1300	2090	3020

Figure 3 displays the comparison of the experimental and numerical equilibrium paths presented as load-axial displacement curves. It is demonstrated that the numerical model can adequately reproduce the global response of the specimens.

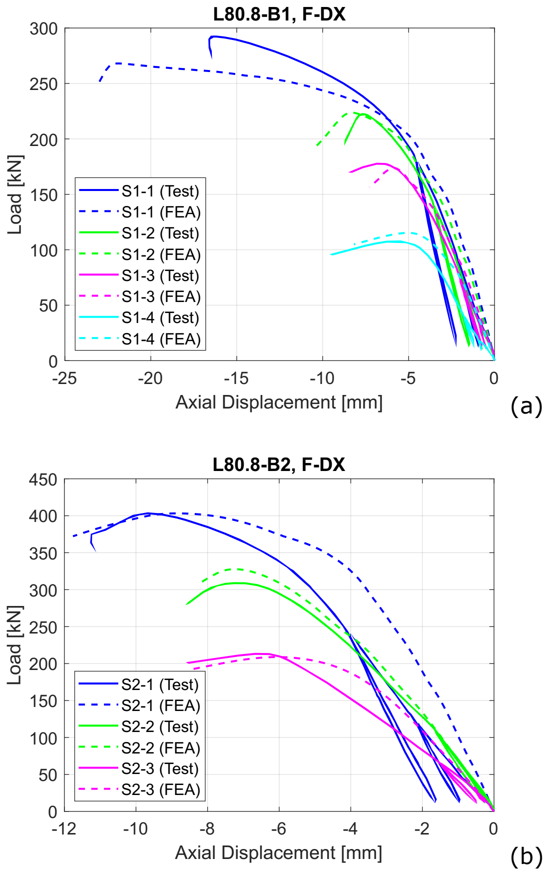

Figure 3 Experimental and numerical equilibrium paths of the specimens with (a) one-bolt and (b) two-bolt end joints

The deformation at the failure of the least and most slender specimens examined (S2-1 and S1-4, respectively) as obtained from both the experiments and simulations along with the numerically calculated distribution of the v. Mises stresses are presented in Figure 4. A qualitative comparison between the experimental and numerical results reveals that the numerical model can capture the failure modes both of slender and non-slender members. Similar agreement between experimental and numerical failure modes was observed for all other specimens as well.

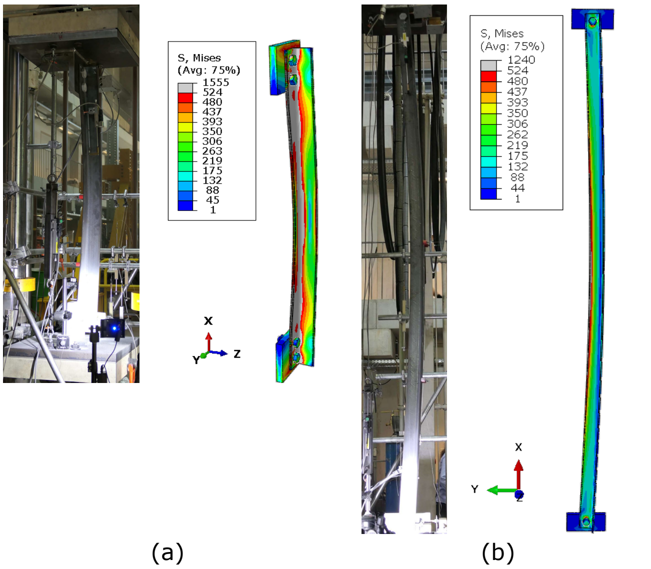

Figure 4 Experimental and numerical failure modes of (a) the least (S2-1) and (b) the most (S1-4) slender specimen

As shown in Figure 5, the experimental and numerical resistances of the specimens—presented in a non-dimensional form (member resistance normalised by section resistance)—exhibit a strong correlation. The mean value of the ratio of the numerical to experimental resistance is 1.00, with a coefficient of variation (C.O.V.) of 4.9%. The maximum deviation is 8%, and the correlation coefficient is 0.992 indicating the very good accuracy of the model. Any deviations are due to simplifications made in the simulation of the geometric imperfections.

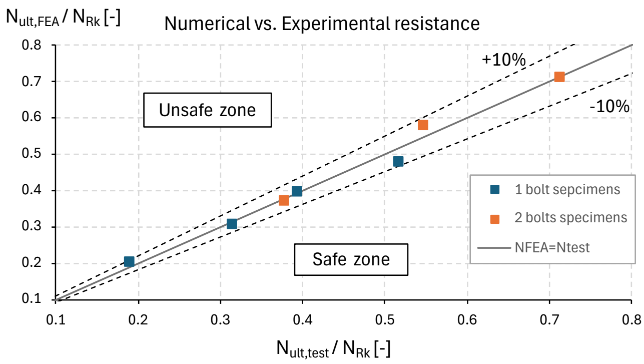

Figure 5 Correlation between and experimental and numerical resistance

4 Numerical parametric study

The parametric study aimed to extend the experimental database on the buckling resistances by investigating a wider range of cross-sections and slenderness ratios. The examined members had the same configurations as those studied experimentally. The study considered members made of S460 steel with three different cross-sections (L45x45x5, L80x80x8, L150x150x13), six different slenderness ratios ranging from 50 to 180, and two types of joints with one and two bolts. The end joints consisted of thick gusset plates—three times the thickness of the angle leg—made of S460 steel and the largest possible for each profile 10.9-steel bolts. The use of thick, and thus stiff, gusset plates is justified by the fact that, in real structures, bending deformation in the gusset plates is typically limited, as both tension and compression bracing members are commonly connected at the same joint.

The modelling assumptions were the same as those described in section 2 with the following modifications:

a. Nominal material properties according to EN 10025-2:2019 [10] were used.
b. The residual stresses were modelled with a 3-point model with an amplitude equal to 70 MPa in accordance with the recommendations [11] on which the European design standards are based.
c. The amplitude of the geometric imperfections was taken equal to the out-of-straightness tolerance prescribed by the product norm [12]. Three different geometric imperfection shapes were considered: the shape of the first buckling eigenmode with positive and negative sign and the shape of the failure mode as obtained by a geometrically and materially nonlinear analysis without imperfections (GMNA). The ultimate resistance was taken equal to the minimum resistance derived by applying those three imperfection shapes.
d. The material of the bolts and the regions of the angle member around the bolt holes (grey regions in Figure 6) was modelled using an elastic steel material. This modelling choice was made to suppress failure modes related to joint resistance, thereby allowing the investigation of buckling behaviour even in relatively short members.

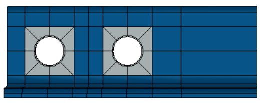

Figure 6 Regions around bolt holes (marked with grey) where elastic material was assigned

Figure 7 presents the results of the parametric study in a non-dimensional form. The vertical axis denotes the normalised ultimate resistance, while the horizontal axis shows the normalised reference length, which corresponds to the slenderness ratio under the assumption of pin-ended boundary conditions. The reference length is defined as the distance between the (external) bolts of the end joints.

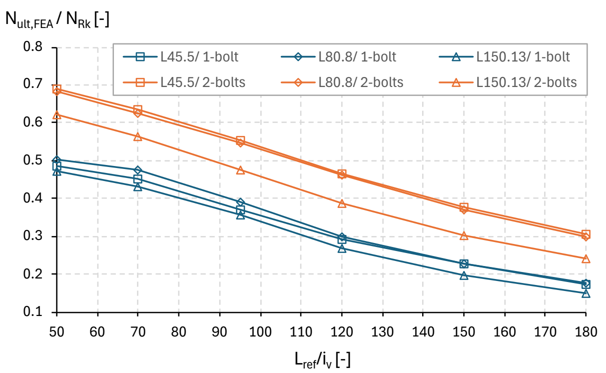

Figure 7 Results of the parametric study

The numerical analyses indicate that the buckling reduction factor varies between χ = 0.15 and 0.50 for members with one-bolt end joints, and between χ = 0.24 and 0.69 for those with two-bolt end joints. Replacing one-bolt with two-bolt connections results in a resistance increase of approximately 30% for short members and up to 75% for slender ones.

5 Assessment of the normative design rules

In Europe the design of steel angles in lattice towers is governed by two standards: EN 1993-3-1 [1] and EN 50341-1 [2]. Both adopt the effective slenderness method, treating angles with bolted connections as pin-ended, centrally loaded members. End restraint and connection-induced eccentricity are addressed through an effective slenderness. This section assesses the predictive accuracy of the design rules outlined in Annex J of EN 50341-1 [2] and its Belgian National Annex NBN EN 50341-2-2 [13], in EN 1993-3-1 [1], and in Annex C of the draft revision prEN 1993-3 [4] against the results of the numerical parametric study.

The key distinction between the current and new version of EN 1993-3 lies in the choice of buckling curve and the treatment of flexural–torsional buckling (FTB). EN 1993-3-1 [1] prescribes curve *b* for angle profiles, independent of steel grade, and requires a separate FTB check. In contrast, prEN 1993-3 [4] assigns curve *a* to angles made of S460 or higher steel grade and considers FTB to be implicitly covered by the local buckling verification, so no additional check is required.

Comparisons between the resistances predicted by the design standards and those obtained numerically are presented in Figure 8 to 10. Cases in which the joint resistance is lower than the member's buckling resistance—and thus governs the design—are highlighted in grey in the figures.

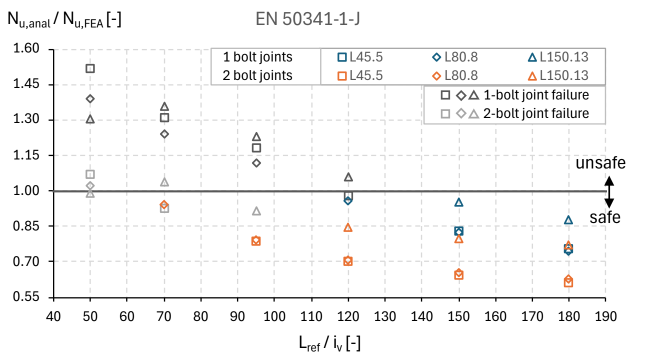

Figure 8 Comparison between numerical and analytical resistance according to EN 50341-1-Annex J

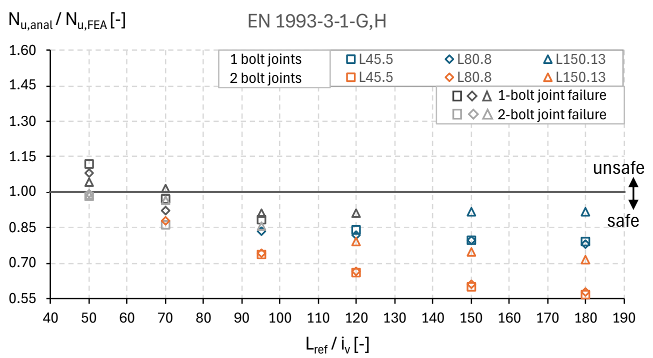

Figure 9 Comparison between numerical and analytical resistance according to EN 1993-3-1-Annex G,H

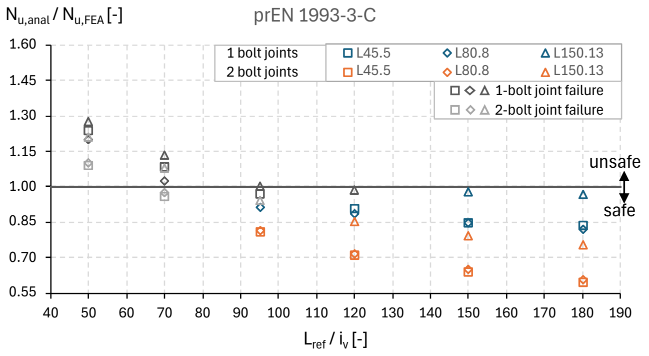

Figure 10 Comparison between numerical and analytical resistance according to prEN 1993-3-Annex C

The comparison between the numerical and analytical resistances indicates that the accuracy of all examined standards is inconsistent. Depending on the end joint configuration and the member's slenderness, the standards may yield either unsafe or overly conservative results. In particular, the buckling design rules of all standards are unsafe for short members with one-bolt end joints. All but EN 1993-3-1 [1] are also unsafe for short members with two-bolt joints. However, when both member and joint resistance are considered, all standards provide safe estimates. The unsafety of the buckling design rules gradually reduces with increasing slenderness. For highly slender members, all standards become conservative, with the conservatism being more pronounced for members with two-bolt joints. At high slenderness, the normative resistance of members with one-bolt end joints can be as low as 75% of the numerical resistance, while for those with two-bolt joints, it may drop to only 56%.

Although all standards exhibit a similar trend—overestimating the resistance of short members and underestimating that of longer ones—they yield discernibly differences in their predictions. These discrepancies are more pronounced for members with one-bolt end joints and low slenderness. Among the standards considered, EN 1993-3-1 [1] is the most conservative, whereas EN 50341-1 [2] is the least conservative for members with one-bolt joints, and prEN 1993-3 [4] for those with two-bolt joints.

6 Discussion

The findings of the assessment presented in the previous section are inherently tied to the assumptions underpinning the numerical parametric study. As mentioned earlier, the use of thick gusset plates intended to reflect the stabilising effect provided by co-existent tension members in real structures, where typically both tension and compression members are connected at the same joint. However, this may have led to stiffer end joints than those encountered in practice, particularly when tensile and compressive forces are not fully balanced. As a result, the normative design rules may appear more conservative in this study than they would be in a real tower scenario.

Furthermore, the imperfection amplitude adopted in the numerical analyses—derived from the relevant product norm [12]—is four times greater than the imperfections implicitly assumed in the European buckling curves [8,11,14]. While this choice ensures a conservative buckling resistance estimation, it also amplifies the demonstrated non-conservatism of the buckling resistance rules of the design standards.

The use of elastic material in specific regions of the end joints may also influence the outcomes of the parametric study. Consequently, the comparison between the design standards and the numerical results should be considered accurate only when the influence of joint plasticity on the members' buckling resistance remains limited. In other cases, the comparisons should be regarded as indicative.

This section provides a closer investigation into the influence of imperfection amplitude and joint plasticity on the buckling resistance of one-leg bolted angles. The parametric study was repeated for members made of L80x80x8. First, an imperfection amplitude of L/1000—consistent with the assumptions of the European design standards [1,2,4]—was adopted. Subsequently, elastoplastic material properties were assigned to the entire model.

Figure 11 illustrates the comparison of the normalised resistances for members with imperfection amplitudes of L/250 and L/1000. Considerable resistance differences are observed—particularly for members with one-bolt joints—reaching up to 24% at intermediate slenderness ratios. For members with two-bolt joints the maximum deviation is about 13% and occurs at high slenderness.

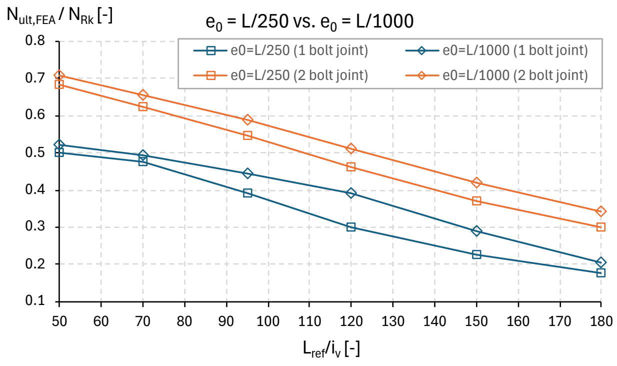

Figure 11 Normalised resistances of L80x80x8 members with imperfection amplitudes of L/250 and L/1000

Figure 12 compares the resistance of members with elastic and elastoplastic end joints. The results show that the joint plasticity primarily affects members with one-bolt joints, reducing their resistance by up to 15%. In contrast, its influence on members with two-bolt joints is minimal, with a maximum reduction in resistance of only 3%.

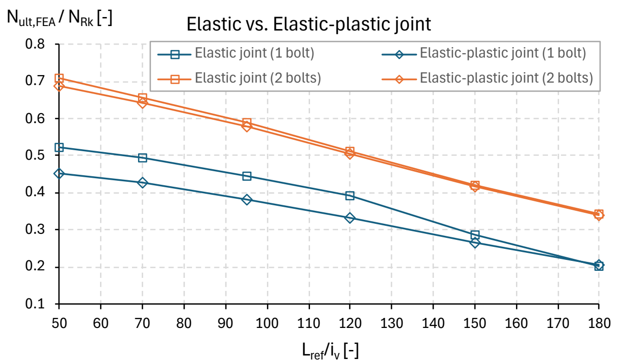

Figure 12 Normalised resistances of L80x80x8 members with elastic and elastoplastic end joints (geometric imperfection amplitude: L/1000)

7 Conclusions and perspective

This study numerically investigated the buckling resistance of steel angles which are bolted through one leg at their extremities and assessed the accuracy of the relevant design rules prescribed by the European design standards.

A sophisticated finite element model was developed, and its accuracy was validated against experimental data, demonstrating its ability to accurately predict the ultimate load, failure mode, and the initial stiffness of angles with bolted ends. A parametric study using this validated model revealed the significant influence of the number of bolts on buckling resistance. It also highlighted the unfavourable effect of joint plasticity, particularly in the resistance of members with one-bolt joints. Furthermore, this study identified an inconsistency between the product norm [12] and design standards [1,2,4] regarding the amplitude of tolerated geometric imperfections, which results in considerable variations in buckling resistance.

The accuracy of the European standards EN 50341-1-Annex J [2], EN 1993-3-1-Annex G,H [1], and prEN 1993-3-Annex C [4] was assessed. The buckling resistance rules of these standards were found to provide inconsistent predictions—unsafe for short members and overly conservative for slender ones. When joint resistance was considered alongside member resistance, all normative predictions were on the safe side. However, this probably would not be the case if members with more than two bolts at the end joints were examined.

The overconservatism of the existing design standards, particularly for the slender members, prevents the full potential of high-strength steel from being exploited. Therefore, improvements to current design rules are necessary. The authors are developing a new methodology for the design of angles which are bolted through one leg at their extremities, comprising four key steps: (1) estimation of joint stiffness, (2) evaluation of bending moments due to eccentricities, (3) determination of member slenderness, and (4) verification of the member resistance under compression and biaxial bending using an interaction formula similar to those proposed in ANGELHY [15].

8 Acknowledgement

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