

Buckling resistance of equal-leg hot-rolled angles: an improved imperfection factor

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Abstract

In recent years, the construction of increasingly taller and more efficient structures, especially in the field of towers and masts, has required the development of high-performance steel profiles that can withstand new loads. In response to this growing demand, high-strength hot-rolled angles up to S460 have been introduced to the steel market. However, despite the existence of evidence demonstrating that the detrimental influence of geometrical and material imperfections decreases with increasing yield strength, the same buckling curve is frequently recommended for different steel grades. For instance, in the forthcoming design specifications for hot-rolled sections in the next generation of Eurocodes, only the S460 grade benefits from a higher buckling curve while the same buckling curve is recommended for grades up to S420. In a first step, this paper illustrates the beneficial effect of the yield strength on the stability of steel angles subjected to pure compression and highlights the limitations of the current stepwise evolution of the imperfection factor in the design rules, based on a review of existing buckling tests and a series of numerical simulations on hot-rolled equal-leg angle members. In a second step, a new modified imperfection factor evolving with the steel grade is proposed and validated, in order to better reflect the beneficial effect of the yield strength on buckling resistance, for both existing and future emerging steel grades.

Keywords:

Angle sections; imperfection factor; buckling resistance; flexural buckling; EN1993-1-1.

28 **1 Introduction**

29 Angle sections are extensively used in structural engineering applications such as buildings, bridges,
30 lattice towers and masts. The ease of fabrication, transport and on-site assembly using bolted
31 connections makes them an attractive solution for practitioners. However, equal-leg angle sections
32 exhibit several specific features that distinguish them from doubly symmetric profiles. They are
33 monosymmetric sections with very low torsional and warping stiffness, and their bending resistance
34 around the weak axis is significantly smaller than around the strong axis. In addition, their plastic
35 resistance is substantially greater than the elastic one, and their legs may be prone to local buckling.

36 Although angle sections are already available in the steel market in a wide range of dimensions and
37 steel grades, up to S460, the steel market is continuously the emergence of new, ever-stronger steel
38 [1]. This is driven by improvements in the operational capabilities of the steelmaking process, which
39 have enabled the production of much heavier and thicker sections with enhanced material properties.

40 These materials offer high strength-to-weight ratios and enable optimisation of structural designs,
41 potentially leading to significant weight, cost and carbon savings, as widely reported in the literature
42 [2] [3] [4] [5] [6] [7] [8] [9] [10]. However, appropriate design rules accounting for higher yield
43 strengths are required to fully exploit these advantages [2].

44 A large number of experimental and numerical studies have investigated the buckling behaviour of
45 angle members composed of both mild and high-strength steels. These include tests on large and mid-
46 size angles, analyses of load eccentricity effects, and investigations of interaction between global and
47 local buckling modes [11] [12] [13] [14] [15] [16] [17] [18]. Complementary analytical studies have
48 also addressed the inelastic response and the accuracy of existing design rules [19] [20] [21]. More
49 recently, numerical parametric studies have highlighted the influence of parameters such as load
50 eccentricity, residual stresses and imperfection modelling on the buckling resistance of angle
51 members [22] [23] [24]. Furthermore, several studies have shown that the dominant failure mode of

pin-ended equal-leg members is weak-axis flexural buckling, sometimes combined with local effects [11] [16] [21] [25] [26] [27] [28] [29].

Although design rules for angle sections are provided in several parts of Eurocode 3, i.e. EN1993-1-1:2005 [30], EN1993-1-1:2022 [31], EN1993-3-1 [32], prEN 1993-3 [33] and EN 1993-1-5 [34], inconsistencies remain between the different provisions, as highlighted in [35]. In particular, the RFCS-funded project ANGELHY [36] has led to a comprehensive reassessment of the design of hot-rolled equal-leg angles, resulting in a consistent and validated set of rules for pin-ended members [29] [37]. Additional studies have proposed further improvements for different boundary conditions [38] [39] [40] [41].

Nevertheless, while the validity of current recommendations has been demonstrated for S460 grade, the assessment of the existing rules for intermediate grades remains limited. Ensuring continuity between grades and introducing yield strength-dependent design rules could lead to significant material, cost and carbon savings. In this context, the present study investigates the flexural buckling of hot-rolled equal-leg angles under concentric loading, with the aim of deriving a modified imperfection factor as a function of the yield strength, applicable to both existing and future emerging steel grades.

2 Research background

2.1 Current design recommendations

The European design standard EN1993-1-1 [30] [31] provides the Ayrton-Perry formulation for the resistance of a member subjected to pure compression, as expressed in Eq. (1).

$$N_{b,Rd} = \frac{\chi \cdot \beta \cdot A \cdot f_y}{\gamma_{M1}} \geq N_{Ed} \quad (1)$$

where:

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} \quad (2)$$

$$\phi = 0.5 \cdot [1 + \alpha \cdot (\bar{\lambda} - 0.2) + \bar{\lambda}^2] \quad (3)$$

$$\bar{\lambda} = \sqrt{\frac{\beta \cdot A \cdot f_y}{N_{cr}}} \quad (4)$$

73 $N_{b,Rd}$ is the design buckling resistance, N_{Ed} is the design compression load, χ is the reduction factor
 74 to account for the relevant global buckling mode, β is the reduction factor to account for local
 75 buckling, A is the cross-sectional area, f_y is the material yield strength, γ_{M1} is the partial safety
 76 coefficient, $\bar{\lambda}$ is the relative slenderness, N_{cr} the Euler's load and α is the imperfection factor.
 77 The latter parameter accounts for the effect of the initial geometrical and material imperfections on
 78 the member stability and leads to the definition of the appropriate European buckling curves. These
 79 buckling curves have been established on the basis of more than 1000 experimental tests in the 1960s
 80 and are designated by the following letters a_0 , a , b , c , d [42] [43] [44] [45] [46]. Regarding hot-rolled
 81 steel angles, the recommendation for the S460 grade changes between the two versions EN1993-1-
 82 1:2005 [30] and EN1993-1-1:2022 [31] as illustrated in Table 1.

83 **Table 1. Selection of buckling curve for the flexural buckling of hot-rolled equal-leg angles according to the two**
 84 **versions of EN1993-1-1**

Cross-section		Limits	Buckling about axis	EN1993-1-1:2005		EN1993-1-1:2022	
				S235-S420	S460	S235-S420	S460 up to S700 inclusive
L-sections		Rolled sections	<i>any</i>	b	b	b	a

Indeed, in accordance with the recommendations set out in the EC3 selection table for other hot-rolled sections, the same buckling curve is proposed for grades from S235 to S420 while, for grades S460 and above, a different buckling curve is recommended for hot-rolled angles. This can be attributed to the fact that the detrimental influence of initial imperfections on the buckling resistance decreases as the yield strength increases. As exposed by Meng et al. [47], when considering an imperfect column with an out-of-straightness of $L/1000$ and without residual stresses, the imperfection parameter can be expressed as a function of the material yield strength (see Eq. (4)).

$$\eta = \frac{e_0 \cdot A}{W_{el}} = \frac{L}{1000} \cdot \frac{A}{W_{el}} = \bar{\lambda} \cdot \pi \cdot \sqrt{\frac{E \cdot I}{A \cdot f_y}} \cdot \frac{A}{1000 \cdot W_{el}} = \left(\frac{\pi}{1000} \cdot \frac{\sqrt{EIA}}{W_{el}} \right) \cdot \frac{\bar{\lambda}}{\sqrt{f_y}} \quad (4)$$

where η is the generalized initial imperfection factor, e_0 is the maximum value of the initial imperfection at mid-height, A is the cross-sectional area, W_{el} is the elastic bending modulus, L is the member length, $\bar{\lambda}$ is the relative slenderness, E is the Young's modulus, and I is the moment of inertia. The relation in Eq. (4) can be written $\eta = f(1/\sqrt{f_y})$ which demonstrates the relationship between the imperfection parameter and the yield strength in the idealised case without residual stresses. However, as the yield strength increases, the relative importance of residual stresses is also reduced as the amplitude of the residual stresses is not affected by the yield stress, in particular for hot-rolled sections (see Section 0). This second beneficial impact on the member stability is difficult to be evaluated as it highly depends on the residual stress distribution and thus on the cross-section typology. The following subsections aim at providing evidence of the benefit in developing a new modified imperfection factor for hot-rolled steel angles. A state-of-the-art overview of the existing modified imperfection factors for other cross-sectional shapes is presented as the basis for the developments set out in this publication. Indeed, according to the design recommendations for the intermediate steel grades (S275 to S420), the buckling curve “b” initially specified for S235 must be considered up to S420, but a progressively decreasing imperfection factor value could be suggested

so as to recognise the fact that the detrimental effect of the residual stresses on the actual buckling load reduces when the yield stress increases.

2.2 Existing data

A review of existing GMNIA simulations and experimental investigations on the column buckling capacity is presented in Fig. 1.

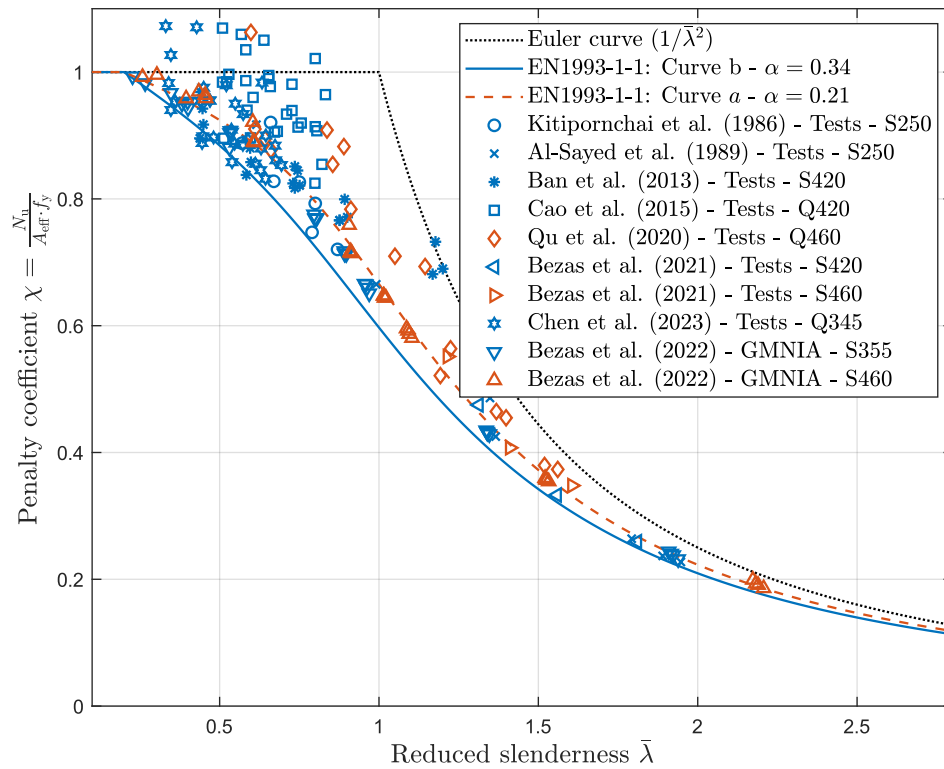


Fig. 1. Comparison between experimental test results and column buckling resistances according to EN1993-1-1

Before 2000, the reference tests on hot-rolled equal-leg angles were carried out by Kitipornchai et al. [27], Al-Sayed et al. [48] and Adluri et al. [49]. They studied the inelastic behaviour of angles made of mild steels and a buckling curve for steel angles was derived based on those tests [50]. Ban et al. [12] tested six different cross-section sizes L125x125x8, L140x140x10, L140x140x12, L160x160x10, L180x180x12 and L200x200x14 made of 420MPa steel to evaluate the suitability of the existing design curves for this particular high-strength grade. They affirm that the overall buckling

strength of S420 columns significantly exceeds the buckling curve resistance prescribed in Eurocode because the detrimental impact of initial imperfections is reduced. Cao et al. [51] confirm that the Eurocode buckling curve is too conservative for Q420, a conclusion supported by the results of 90 buckling tests on 30 large and high-strength angle sections with widths of 220 mm and 250 mm and various thicknesses between 20 mm and 30 mm. Qu et al. [52] tested 48 pin-ended columns made of Q460, they evaluated that the prediction of the compressive load-bearing capacity of high-strength steel angle sections is relatively accurate. As can be seen in Fig. 1, the numerical simulations as well as the buckling tests for the S420 grade are closely aligned with the buckling curve that has been prescribed for the S460. Finally, the 96 Q345 tests from the research of Chen et al. [25] on similar angle sections demonstrate analogous results for this regular grade. Bezas et al [11] [29] confirm this trend through experimental and numerical evidence on large-section angles made of S420 and S460 grades. Consequently, this comparison demonstrates the optimisation potential of developing a buckling curve selection that is a continuous function of the yield strength instead of having a stepwise evolution of the imperfection factor as currently recommended in design guidelines.

2.3 Existing modified imperfection factors

In previous years, a number of modified imperfection factors that continuously account for the material strength were proposed. Maquoi [53] was the first author to propose a modified imperfection factor α^* , as expressed in Eq. (5):

$$\alpha^* = \alpha \cdot \left(\frac{235}{f_y} \right)^n \quad (5)$$

With α the coefficient recommended for the S235 steel grade. He proposed an exponent $n = 0.8$ to keep the gap of one buckling curve between the curve recommended for S235 and the one for the former high-strength steel grade, S430. Indeed, for instance, if a curve “b” is recommended for S235 ($\alpha = 0.34$) and a curve “a” ($\alpha = 0.21$) for S430, the modified imperfection factor with the exponent

$n = 0.8$, i.e. $\alpha^* = 0.34 \cdot \left(\frac{235}{430}\right)^{0.8} = 0.21$, respects the gap of one buckling curve between the two reference grades. As an alternative, Johansson [54] proposed to consider an exponent $n = 1.0$. In the framework of the European project RUOSTE [55], Somodi et al. [56] [57] proposed the same expression but with an exponent $n = 0.5$ for cold-formed hollow sections and $n = 0.6$ for welded I-sections. Subsequently, Saufnay et al. [2] [58] proposed to consider an exponent $n = 0.7$ for hot-rolled I- or H-sections as well as for hot-rolled hollow sections. For all types of hot-rolled sections, the forthcoming version EN1993-1-1:2022 [31] suggests a gap of one buckling curve between grades below S460 and grades equal or higher than S460. Therefore, they proposed an exponent $n = 0.7$ that fits with the current design recommendation for the high-strength steel grade S460 ($\alpha^* = 0.34 \cdot \left(\frac{235}{460}\right)^{0.7} = 0.21$).

As an alternative to the above-mentioned formulation, in the framework of the European project STROBE [59], a reduction in the length of the plastic plateau to 0.1 and a modified imperfection factor function of the “ $\varepsilon = \sqrt{235/f_y}$ ” coefficient for hot-rolled and cold-formed hollow sections [47] as well as for welded I-sections [60] have been proposed.

Finally, the last proposal that can be found in the literature consists of applying this “ ε ” coefficient to the relative slenderness, thereby rendering the length of the plastic plateau a function of the yield strength. This approach has been proposed by Jönsson et al. [61] for hot-rolled I- and H-shaped sections and by Fang et al. [62] for cold-formed hollow sections. All existing modified imperfection factors are summarised in Table 2.

Table 2. Existing modified imperfection factors in the literature.

Cross-section typology		Imperfection parameter	Imperfection factor	Author, year (Project)
Rolled I- or H-sections		$\alpha^* \cdot (\bar{\lambda} - 0.2)$	$\alpha^* = \alpha \cdot \left(\frac{235}{f_y}\right)^{0.8}$	Maquoi, 1982
		$\alpha^* \cdot (\bar{\lambda} - 0.2)$	$\alpha^* = \alpha \cdot \left(\frac{235}{f_y}\right)^{1.0}$	Johansson, 2005
		$\alpha^* \cdot (\bar{\lambda} \cdot \varepsilon - 0.2)$	$\alpha^* = \alpha$	Jönsson et al., 2016
		$\alpha^* \cdot (\bar{\lambda} - 0.2)$	$\alpha^* = \alpha \cdot \left(\frac{235}{f_y}\right)^{0.7}$	Saufnay et al., 2024
Welded I-sections		$\alpha^* \cdot (\bar{\lambda} - 0.2)$	$\alpha^* = \alpha \cdot \left(\frac{235}{f_y}\right)^{0.6}$	Somodi et al., 2017 (RUOSTE)
		$\alpha^* \cdot (\bar{\lambda} - 0.1)$	$\alpha^* = 0.45 \cdot \left(\frac{235}{f_{y,flange}}\right)^{0.5}$ for $y - y$	Yun et al., 2023 (STROBE)
			$\alpha^* = 0.55 \cdot \left(\frac{235}{f_{y,flange}}\right)^{0.5}$ for $z - z$	
Hollow sections	Hot-finished	$\alpha^* \cdot (\bar{\lambda} - 0.1)$	$\alpha^* = 0.24 \cdot \left(\frac{235}{f_y}\right)^{0.5}$	Meng et al., 2020
		$\alpha^* \cdot (\bar{\lambda} - 0.2)$	$\alpha^* = \alpha \cdot \left(\frac{235}{f_y}\right)^{0.7}$	Saufnay et al., 2024
	Cold-formed	$\alpha^* \cdot (\bar{\lambda} - 0.2)$	$\alpha^* = \alpha \cdot \left(\frac{235}{f_y}\right)^{0.5}$	Somodi et al., 2017 (RUOSTE)
		$\alpha^* \cdot (\bar{\lambda} \cdot \varepsilon - 0.2)$	$\alpha^* = \alpha$	Fang et al., 2018
		$\alpha^* \cdot (\bar{\lambda} - 0.1)$	$\alpha^* = 0.56 \cdot \left(\frac{235}{f_y}\right)^{0.5}$	Meng et al., 2020

As can be clearly seen in Table 2, the extant literature does not address the subject of hot-rolled equal-leg angles yet. The objective of the present paper is therefore to derive an expression of the modified imperfection factor for this profile typology.

3 Methodology

3.1 Research scope

This paper focuses on the flexural buckling behaviour of pin-ended members consisting of hot-rolled equal-leg angles made of mild or high-strength steel grades (up to S460), as well as possible future emerging steel grades (from S500 to S690) as a prospective study, when subjected to concentric axial loading.

The studied profiles were selected to cover both compact and slender cross-sections; their geometrical properties are listed in Table 3. Moreover, the criteria used to classify these profiles for steel grades S460 and S690 are the ones proposed by the ANGELHY project [37], where the inconsistencies between the existing classification rules given in EN1993-1-1:2022 [31] have been erased; thus, equal-leg angles are classified as class 3 when $c/t \leq 14\epsilon$. It should be mentioned that both L70x70x5 and L250x250x20 are classified as class 4 sections as far as they are made of a steel grade equal or higher than S420.

Table 3. Specifications of the cross-sections selected in the present paper.

Cross-section designation	h (mm)	t (mm)	r (mm)	A (mm ²)	Class in S460	Class in S690	Type of FE
L70x70x5	70	5	9	684	4	4	Solid
L70x70x7	70	7	9	940	3	3	Beam
L150x150x18	150	18	18	5100	3	3	Beam
L250x250x20	250	20	18	9600	4	4	Solid

3.2 Description of the numerical model

Two different finite elements (see Table 3) were used in the geometrical and non-linear analyses with imperfections (GMNIA) to simulate the selected cross-sections. Initially, the FineLg software [63] with beam finite elements has been used to model sections that are not prone to local buckling. However, in order to extend the range of this study and simulate sections on which such phenomena

could occur, the Abaqus software [64] with solid finite elements has been used, as such finite elements are not available in FineLg. The FineLg software has already been used in many studies [58] [65] [66] [67], including some dealing with the buckling behaviour of angle profiles [11] [29] [68] while Abaqus is the most commonly used software to simulate the flexural buckling of members [29] [47] [56] [60] [69].

For the beam finite element models, 10 beam elements were assessed as suitable for evaluating the buckling resistance of a member subjected to pure compression, the same assumption was validated in [58]. The GMNIA analyses were solved by the arc-length method in order to capture the decreasing slope after the occurrence of instability phenomena. The loading is applied through an imposed load at the gravity centre in the end-section of the angle member.

For the solid finite element models, the three-dimensional solid 8-node hexahedral element with incompatible modes (C3D8I) from the Abaqus element library was selected to simulate the member, including the curved parts of the cross-section. Three solid elements over the thickness of the section have been used; the accuracy of this mesh density has been validated in [70]. Furthermore, in order to distribute uniformly the external applied load, but also to avoid any local failure at the point of application of the load, end plates at the extremities of the samples have been considered indirectly through a specific constraint.

To model a buckling curve, 13 simply supported elements corresponding to 13 relative slendernesses were selected from $\bar{\lambda} = 0.2$ to $\bar{\lambda} = 2.6$ for both compact and slender sections. The buckling lengths L_{cr} associated with those relative slendernesses are computed as expressed in Eq. (6).

$$\bar{\lambda} = \sqrt{\frac{\beta \cdot A \cdot f_y}{N_{cr}}} \rightarrow L = L_{cr} = \frac{i \cdot \pi \cdot \bar{\lambda}}{\sqrt{\frac{\beta \cdot f_y}{E}}} \quad (6)$$

The Euler's load N_{cr} is analytically computed considering the geometrical properties of the selected gross cross-sections. In the buckling procedure, the β coefficient reduces the plastic resistance to account for the risk of local buckling. Obviously, this coefficient is equal to 1.0 for class 3 profiles, while, for class 4 profiles, this coefficient was numerically computed using a rather short member with relative slenderness of about 0.15, for which global buckling phenomena are not relevant. This approach aims to eliminate the conservatism of the existing design recommendations of EN1993-1-1 for class 4 sections sensitive to local buckling [37].

For each relative slenderness, the reduction factor χ_{FEM} obtained by numerical simulations is evaluated as expressed in Eq. (7) in such a way to draw the numerical buckling curve.

$$\chi_{FEM} = \frac{N_u}{\beta_{FEM} \cdot A \cdot f_y} \quad (7)$$

with N_u the peak axial load obtained through GMNIA simulations, $\beta_{FEM} = N_{FEM,CS}/A \cdot f_y$ the reduction factor for local buckling obtained numerically where $N_{FEM,CS}$ is the ultimate axial load obtained through GMNIA simulations of the short members, A the cross-sectional area, and f_y the material strength.

The yield strengths f_y considered in the numerical models are the nominal values provided in Table 5.1 of EN1993-1-1:2022 [31]. However, the latter specifications for the nominal yield strength only consider a reduction of the yield strength for thicknesses above 40 mm, while the corresponding product standards prescribe a reduction from 16 mm. Therefore, it has been decided to perform some numerical simulations considering the yield strength reduction prescribed in products standards, in order to ensure that similar conclusions could also apply to steels complying with the relevant product standards, i.e., EN10025-2 [71] and EN10025-4 [72] for hot-rolled steel angles. The results related to these yield strengths are indicated with the index EN10025 hereafter.

In each numerical model, a linear elastic – perfectly plastic material model with a nominal plateau slope for numerical stability has been chosen, similarly to what was already done in previous research works [2] [11] [29] [58] [73] [74]. The material law is illustrated in Fig. 2.

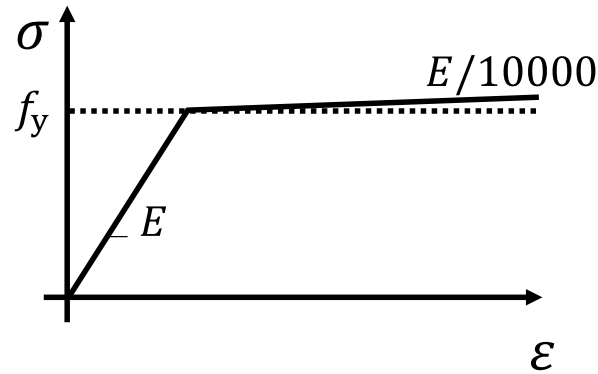


Fig. 2. Material law for numerical simulations

3.3 Geometrical and material imperfections

According to the modelling recommendations from prEN1993-1-14:2025 [75], an initial bow imperfection with a magnitude equal to $L/1000$ with a deformation shape similar to the first pure weak-axis flexural buckling mode (see Fig. 3a) has been implemented in the numerical models. The selection of the initial geometrical imperfection shape is justified by the outcomes of the ANGELHY project [36], where it has been demonstrated that the dominant failure mode of a pinned compressed equal-leg angle member is weak axis flexural buckling. In fact, it has been observed experimentally [11] that, even if in some cases the member twists at the first steps of loading, it finally buckles in a weak axis flexural buckling. This failure mode can be characterised as flexural-torsional mode and is also often observed in short to intermediate slenderness members [36]. However, it has been demonstrated and validated numerically that the ultimate resistance of these members exhibiting a flexural-torsional mode (obtained also through a linear buckling analyses) should be calculated based on the pure weak-axis flexural mode [29]. In fact, as it is demonstrated in [21] [29], considering flexural-torsional imperfections for these short-to-intermediate slenderness members leads to higher

resistances than adopting pure weak axis flexural imperfections. Similar observations regarding the dominant flexural failure mode have been independently made by other researchers [25] [26] [27] [28]. Furthermore, a comparison study has been performed by Gomes et al. [76] where different design approaches for equal-leg angles are examined including ANGELHY design rules, existing Eurocode provisions as well as Behzadi-Sofiani design approach. Through this study, it has been further validated that the use of flexural modes (ANGELHY proposal) leads to safe and less conservative results than considering flexural torsional buckling modes (Behzadi-Sofiani approach). All these references justify the selection of the imperfections to be considered in the numerical study. Since the load is applied concentrically, the member always buckles toward the side of the imposed minor-axis imperfection, meaning that the sign of this imperfection does not affect the predicted resistance.

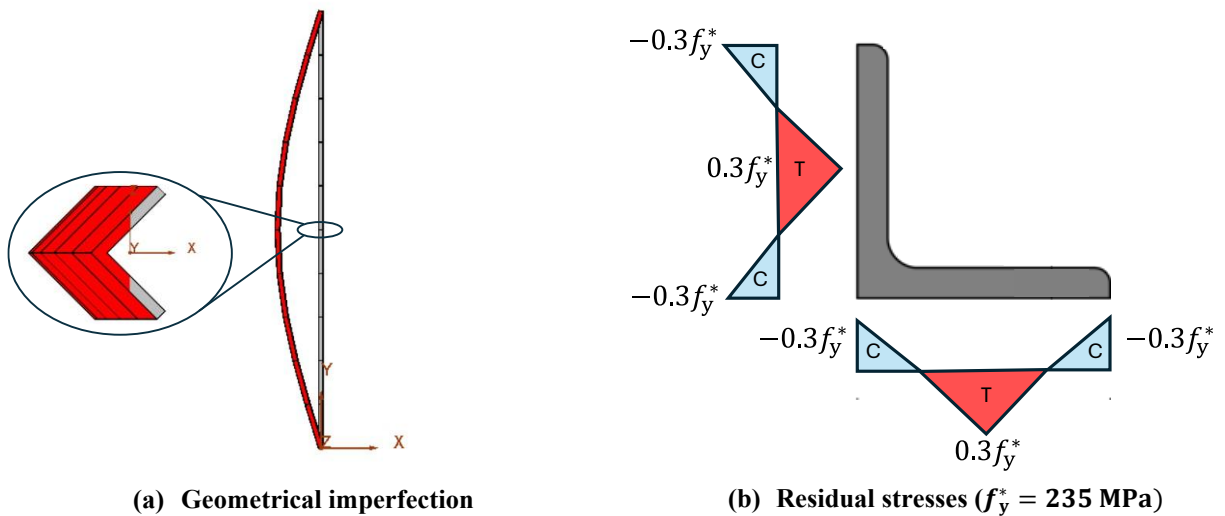


Fig. 3. Geometrical and material imperfections.

Researchers in Europe and USA carried out a series of residual stress measurements in the 1960s and 70s aiming at deriving an analytical model of residual stresses distribution for hot-rolled equal-leg angles, which has been adopted later in various international codes and design recommendations such as the ECCS [77] for Europe. It consists of a bi-linear model function of the material yield strength f_y

times the parameters β_1, β_2 and β_3 , that are defined as the residual stress factors and obtained by dividing the residual stress by the yield strength [78]. The magnitudes of those factors are often discussed in the literature based on experimental measurements [23] [24] [77] [78] [79] [80] when evaluating the bearing capacity of steel angle members under compression, as the measurements generally present large scatter. Sometimes, even the shape of the residual stress pattern is discussed to have a better agreement with the experimental measurements, for instance a trilinear model instead of a bilinear one [24].

The assumption adopted in the present study is consistent with recent findings reported for conventional hot-rolled structural sections. Several investigations have shown that residual stress distributions are primarily governed by cross-sectional geometry and cooling conditions rather than by the material yield strength itself [81] [82]. Although these studies were conducted on hot-rolled I- and H-sections, the underlying mechanisms responsible for residual stress formation are identical for hot-rolled angle sections, since residual stresses mainly originate from non-uniform cooling during the rolling process. The stability of members in compression made of large hot-rolled angles has been investigated by L. Zhang et al. [23] and P. Moze et al. [24] in 2014 for which a series of residual stress measurements were carried out on S275, S355 and S420 specimens through the sectioning method. It was shown that, similarly to other hot-rolled sections, the residual stress pattern of hot-rolled angles is unaffected by the material yield strength. The adopted residual stress pattern corresponds to peak residual stresses of 70.5 MPa ($0.3f_y^*, f_y^* = 235$ MPa), irrespective of the steel grade – see Fig. 3b. Consequently, in the absence of any experimental evidence indicating a different behavior for hot-rolled angles, it is reasonable to assume that the relative influence of residual stresses decreases with increasing yield strength, while their absolute magnitude remains largely unaffected by the steel grade.

To extend this observation to high-strength steel grades, some authors [78] [79] conducted residual stress measurements on S420 steel grade. It was observed that their magnitude is lower than that prescribed by a model function of the material strength, the magnitudes were approximately equal to the ones of mild steel grades meaning that they are unrelated to the steel grade. Indeed, Ban et al. [79] proposed residual stress factors of $0.15f_y$ for Q420, which are lower in magnitude than $0.3f_y^*$. More recently, some researchers [18] [83] investigated the residual stress distributions of angle profiles made of S460 steel. They clearly highlighted the fact that the measured stresses are lower in magnitude than $0.3f_y^*$. This literature review serves to corroborate the assumption that the residual stress distributions are independent of the yield strength, which was expected, as they result from the cooling process that is similar irrespective of the steel grade. All these reference residual stress measurements are gathered in Fig. 4 as well as the classically prescribed distribution to highlight the suitability of the pattern with peaks of $0.3f_y^*$ to model the residual stresses regardless of the yield strength considered.

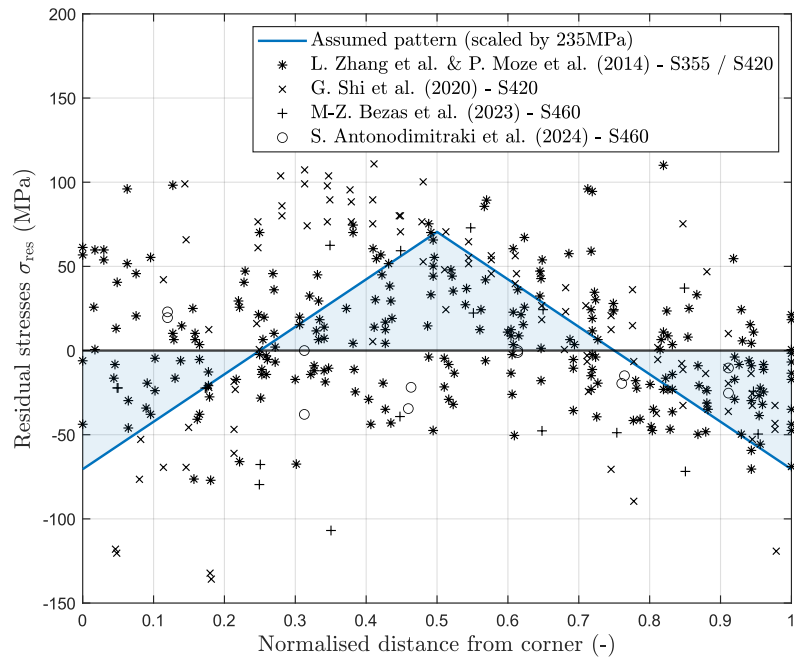


Fig. 4. Residual stress measurements in the literature for various steel grades

Fig. 4 illustrates the large scatter of residual stress measurements for 3 different steel grades: S355, S420 and S460. It however appears that, the compression residual stresses at the leg tips are always lower in magnitude than those given by the recommended pattern scaled for 235 MPa, meaning that the same distribution can be conservatively considered whatever the steel grade under concern, as these stresses have the greatest influence on flexural buckling resistance. Consequently, the residual stress pattern shown in Fig. 3b, scaled by 235 MPa, is adopted in the parametric study for all considered steel grades. It is also assumed to be appropriate for non-existing grades (grades above S460) in a prospective context.

The adopted pattern is directly implemented in FineLg, where it is available as a built-in residual stress distribution and is therefore used without modification in the finite element analyses. In Abaqus, however, residual stresses are introduced indirectly through an equivalent geometric imperfection, accounting simultaneously for both geometrical and material imperfections.

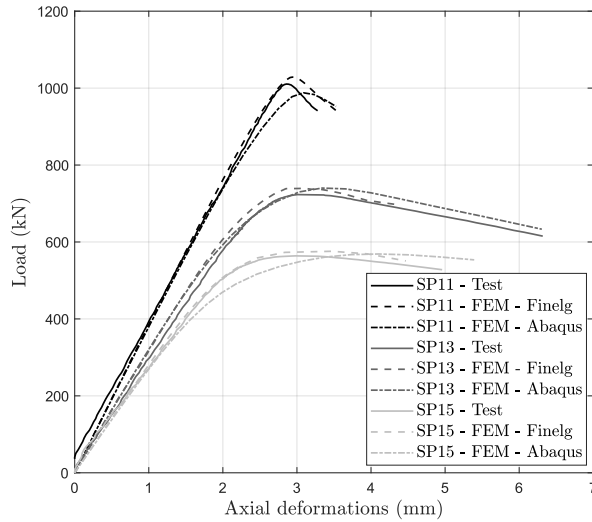
4 Numerical validation

Farhoud et al. [21] already performed numerical investigations by means of the software Abaqus with solid elements, considering both material and geometrical non-linearities. The model used in their parametric study has been validated based on the experimental tests of Spiliopoulos et al. on L70x70x7 made of S235 steel [13]. They concluded on the adequacy of the software to properly reproduce the experimental tests using measured initial geometrical bow imperfections, residual stresses as those described in section 3.3 and an elastoplastic material law coming from coupon tests. Therefore, the use of solid finite elements as well as the assumptions made in section 3 are relevant in the framework of this publication. Finally, it has been observed that a flexural initial imperfection gives a safer resistance up to 7% compared to a torsional-flexural imperfection, even though the first elastic buckling mode is a local or a torsional-flexural one. Therefore, it confirms the adequacy of the adoption of a flexural buckling mode as geometrical initial imperfection for the parametric study.

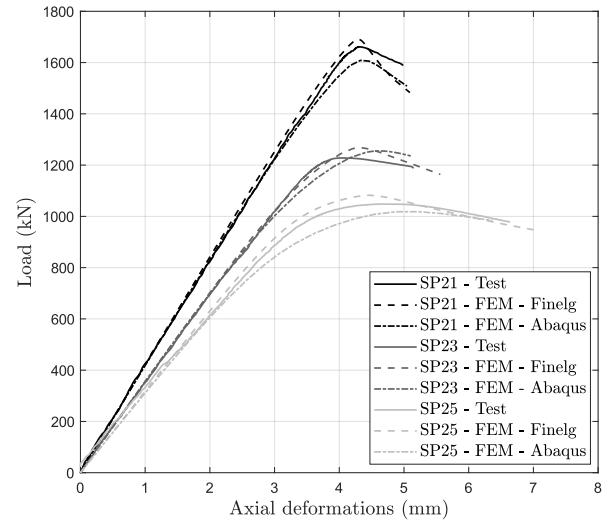
In the framework of the ANGELHY project [36], experimental buckling tests were performed to evaluate the resistance of S460 steel angles. In particular, in this framework, Bezas et al. [11] conducted experimental tests on L150x150x18 and L200x200x16 profiles made of S420 and S460 steel grades. Within this study, they also successfully reproduced the observed structural behaviour by using the FineLg software with beam finite elements, considering again both material and geometrical non-linearities. This demonstrates that the latter software is also adequate for simulating the flexural buckling of high-strength steel angle profiles, validating also the considerations made in section 3 regarding the initial geometrical imperfections, the residual stresses, and the material law. The comparison values and charts are reported in Table 4 and Fig. 5 for concentrically loaded angles made of S420 and S460 steel grades. These results are coming from a previous research of Bezas et al. [11] in which the failure modes and deformation patterns are discussed in more details. For the validation studies reported in Table 4 and Fig. 5, the measured material properties and geometrical imperfections reported in Bezas et al. [11] were adopted in the numerical models.

Table 4. Comparison between buckling tests, numerical results using both FE models and Eurocode predictions.

Specimen	Profile	Measured f_y (MPa)	Length (mm)	$N_{u, test}$ (kN)	GMNIA-FineLg (kN)	GMNIA-Abaqus (kN)	EC3 (kN)
SP11	L150x150x18	417,2	2500	1010,6	1028,6	987,8	894,8
SP13		425,8	3000	723,2	739,3	740,2	698,5
SP15			3500	563,9	575,8	568,8	542,7
SP21	L200x200x16	487,6	3000	1661,5	1690,6	1608,4	1337,2
SP23			3500	1228,0	1267,4	1255,2	1088,6
SP25			4000	1048,1	1082,2	1018,4	984,1



(a) L150x150x18 - S420



(b) L200x200x16 - S460

Fig. 5. Numerical validation of buckling tests using both numerical models.

The validation results were further quantified by comparing the ratios $N_{u, \text{test}}/N_{u, \text{FEM}}$. The mean values are 0.98 and 1.01 for FineLg and Abaqus, respectively, while the corresponding coefficients of variation (COV) are 0.64% and 2.58%. These results confirm the excellent agreement between the experimental tests and both finite element models.

Consequently, based on the literature review and the comparison reported in Table 4 and Fig. 5, it appears that both the nonlinear Abaqus software with solid elements and the FineLg software with beam elements are suitable to perfectly reproduce the buckling behaviour of concentrically loaded steel angles made of regular and high-strength steel grades.

5 Numerical results

5.1 Assessment of the design rules for reference grades

As reported in Table 1, EN1993-1-1:2022 [31] specifies a gap of one buckling curve between mild steel grades (up to S420) and high-strength steel grades (from S460), whereas the same buckling

curve is specified for both in the current EN1993-1-1:2005 [30]. Consequently, the first step of the parametric study consisted of validating the design rules for the reference grades, i.e., S235 and S460. Comparisons between the Eurocode design rules and the numerical simulations are reported in Fig. 6, (a) for sections non-affected by local buckling and (b) for sections prone to local buckling.

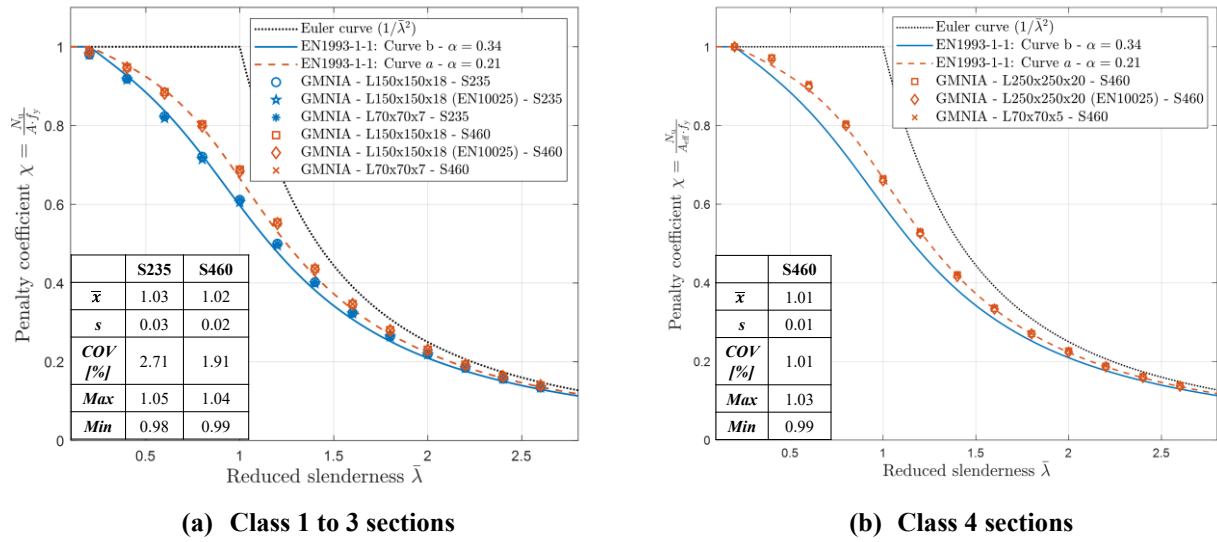


Fig. 6. Assessment of the existing rules for reference grades

Fig. 6 shows that the gap of one buckling curve between the two reference grades is reliable, confirming that the detrimental influence of initial imperfections is significantly reduced when moving from a S235 grade to a S460 grade.

The statistical evaluation reported in Fig. 6 indicates a good correspondence between numerical simulations and design rules. A buckling curve is assumed to be suitable when the mean value, noted $\bar{\chi}$ is above 1.0, this limit corresponds to the partial safety coefficient for stability design calculations ($\gamma_{M1} = 1.0$). To complete the statistical evaluation, the standard deviation s , the coefficient of variation COV and the maximum and minimum values are also provided.

It can be seen in Fig. 6a that the numerical data for a relative slenderness $\bar{\lambda} = 0.2$ are below the plastic plateau. This feature has already been observed in many studies and led to some modified imperfection factors with a different length for the plastic plateau [47] [60], which is even sometimes

different depending on the steel grade [61] [62]. However, this limit has been fixed in the past based on a statistical evaluation of the wide experimental campaign, including angles. In addition, according to the numerical simulations for slender sections in Fig. 6b, reduction factors higher than 1.0 are obtained when the section is prone to local buckling. Indeed, through the linear buckling analysis, only local buckling modes are obtained for the first buckling modes for relative slenderness smaller or equal to 0.2. Therefore, it has been assumed that the length of the plateau may be conserved up to $\bar{\lambda} = 0.2$ in the framework of this research.

In addition, the numerical results are seen to be unaffected by the geometrical properties, similarly to other research on angles [26]. But it should be reminded that, in the present study, the β coefficient to predict the effective cross-section has been computed numerically. Using the effective cross-section properties computed according to the recommendations of EN1993-1-5:2005 [34] would result in slightly different buckling resistances depending on the geometrical properties as it has been demonstrated that, the slender the section legs are, the more conservative the local buckling resistance predictions [35] [37]. In addition, the numerical simulations conducted considering the nominal yield strength from product standards are slightly below the values obtained by considering the yield strength from design standard. Indeed, for the S235 steel grade for instance, the design rule EN1993-1-1:2022 [31] recommends the use of a yield strength of 235 MPa for thicknesses up to 40 mm while the associated EN10025-2 & -4 [71] [72] recommend a reduction to 225 MPa from 16mm. The profile L150x150x18 is concerned by this particular feature as reported in Fig 4a. Nevertheless, according to the statistical evaluations made in Fig. 6, there is a good agreement between the recommended curves in EN1993-1-1:2022 [31] and the numerical simulations, meaning that it is appropriate to conserve these design recommendations for those grades in an eventual improvement proposal of the existing design rules.

5.2 Assessment of the existing rules for intermediate grades

The previous subsection illustrates how the impact of initial imperfections is reduced when moving from the S235 grade to the S460 grade. However, intermediate grades such as S355 and S420 are gathered in the same category as the S235 grade (see Table 1) while it is likely that the detrimental impact of initial imperfections is reduced for these grades too. The same applies to the S500 grade, which has the same prescribed buckling curve as the S460 grade. The comparison between numerical simulations and the currently prescribed buckling curves for intermediate grades is shown in Fig. 7 – (a) for classes 1-3 cross-sections and (b) for class 4 cross-sections. Although L250x250x20 was classified as class 4 for a steel grade of S420 (see §3.1), when considering a reduction of the yield stress that the product norm EN10025 implies for profiles thicker than 16 mm, this profile is transferred to class 3, and thus, it is not reported in Fig. 7(b).

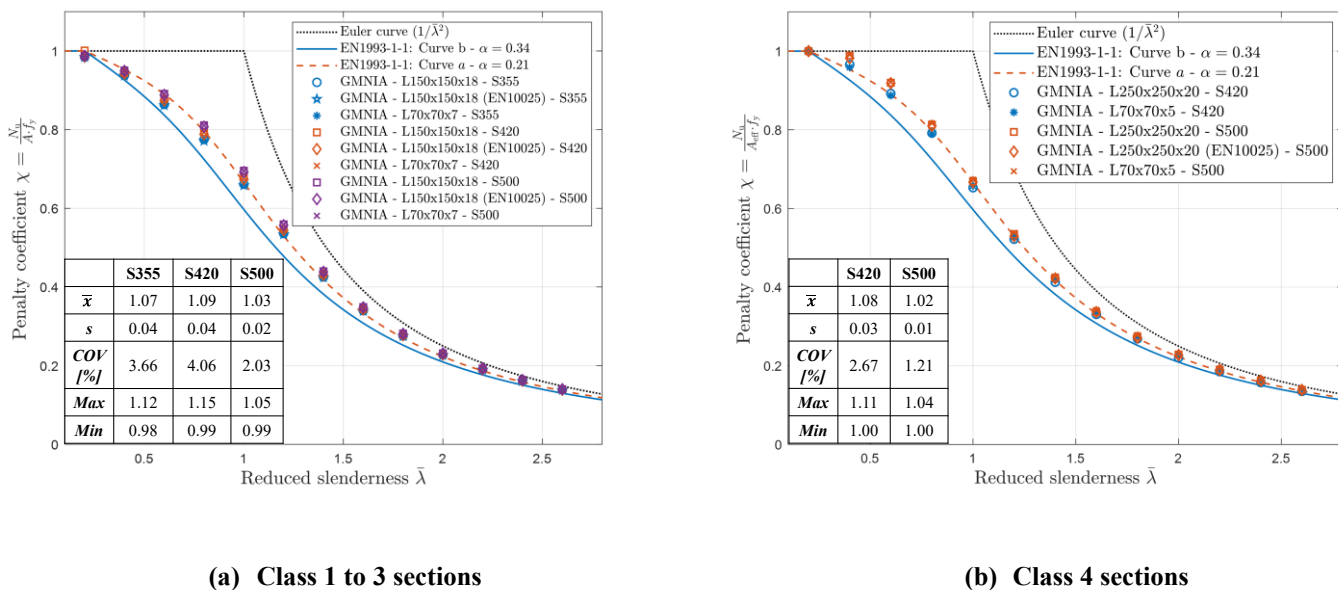


Fig. 7. Assessment of the existing rules for intermediate grades

The current design recommendations for the flexural buckling of intermediate grades are too conservative. Indeed, as it can be observed in Fig. 7, the buckling resistance of intermediate grades are clearly underestimated when considering curve b. It confirms the observations made from

experimental evidence in Section 2, highlighting the usefulness of having a modified imperfection factor that accounts for the reduction of the detrimental influence of the initial imperfections with the steel grade on the buckling resistance.

5.3 Proposal for a new modified imperfection factor

Based on the literature review presented in Section 2.3 and the numerical conclusions in Section 5.1 and 5.2, the objective is to derive a modified imperfection factor that respects the gap of one buckling curve between the reference grades, i.e. S235 and S460, and continuously accounts for the beneficial impact of the yield strength. The modified imperfection factor which has already been proposed for hot-rolled H-shaped and hollow sections [2] [58] is reported in Eq. (8)).

$$\alpha^* = \alpha \cdot \left(\frac{235}{f_y} \right)^{0.7} \quad (8)$$

This proposal offers the advantage of conserving the current buckling curves for S235 and S460 as they were evaluated as suitable in Section 5.1. Indeed, for S235, $\alpha^* = 0.34 \cdot \left(\frac{235}{235} \right)^{0.7} = 0.34$ corresponding to a buckling curve “b” while for S460, $\alpha^* = 0.34 \cdot \left(\frac{235}{460} \right)^{0.7} = 0.21$ corresponding to buckling curve “a” as currently prescribed (see Table 1). The proposal therefore respects the gap of one buckling curve between the two reference grades. Consequently, an extensive parametric study covering grades from S235 up to S690 was carried out by means of the FineLg software for Class 1 to 3 cross-sections and grades from S420 (for the reasons explained in §3.1) to S690 by means of the Abaqus software for Class 4 cross-sections and comprised approximately 500 numerical simulations. The present proposal is valid for steel grades ranging from S235 to S690, corresponding both to the scope of numerical investigation and to the steel grades currently covered by EN 1993-1-1:2022 [26]. It should nevertheless be noted that experimental evidence supporting the adopted residual stress assumptions is currently limited to steel grades up to S460. No residual stress measurements or

structural test data are available for higher-strength hot-rolled angle sections, as such products are not currently manufactured as standard hot-rolled angles.

For each numerically produced buckling curve, an imperfection factor α has been calculated to fit with the numerical curve. The imperfection factor α obtained is evaluated as appropriate when the mean value is above 1.0 while limiting the maximum overestimation of the load-bearing capacity to 2%. Those numerically obtained imperfection factors, the current recommendations of the two versions of EN1993-1-1 [30] [31] as well as the proposed modified imperfection factor are represented in Fig. 8.

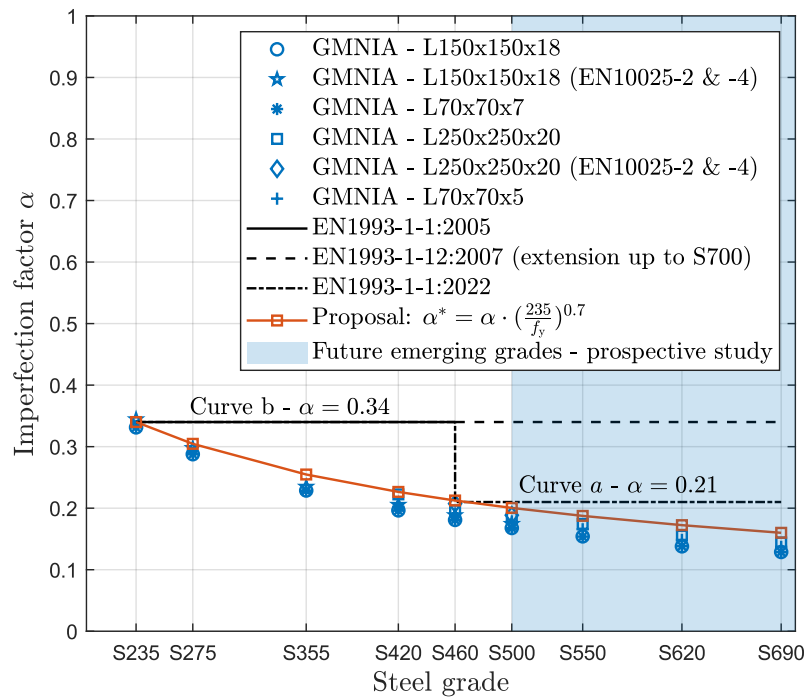


Fig. 8. Proposal for a new modified imperfection factor

Fig. 8 confirms the reliability of the proposal expressed in Eq. (8) for hot-rolled equal-leg angles. In order to illustrate the benefit of this proposal, comparisons between the numerical simulations and the new buckling curves obtained through the modified imperfection factor for S355, S420, S500 and S690 are represented in Fig. 9. The statistical evaluation is performed based on the current EN1993-

1-1:2022 recommendations [31] (noted as α in the tables) and on the modified imperfection factor (noted as α^* in the tables).

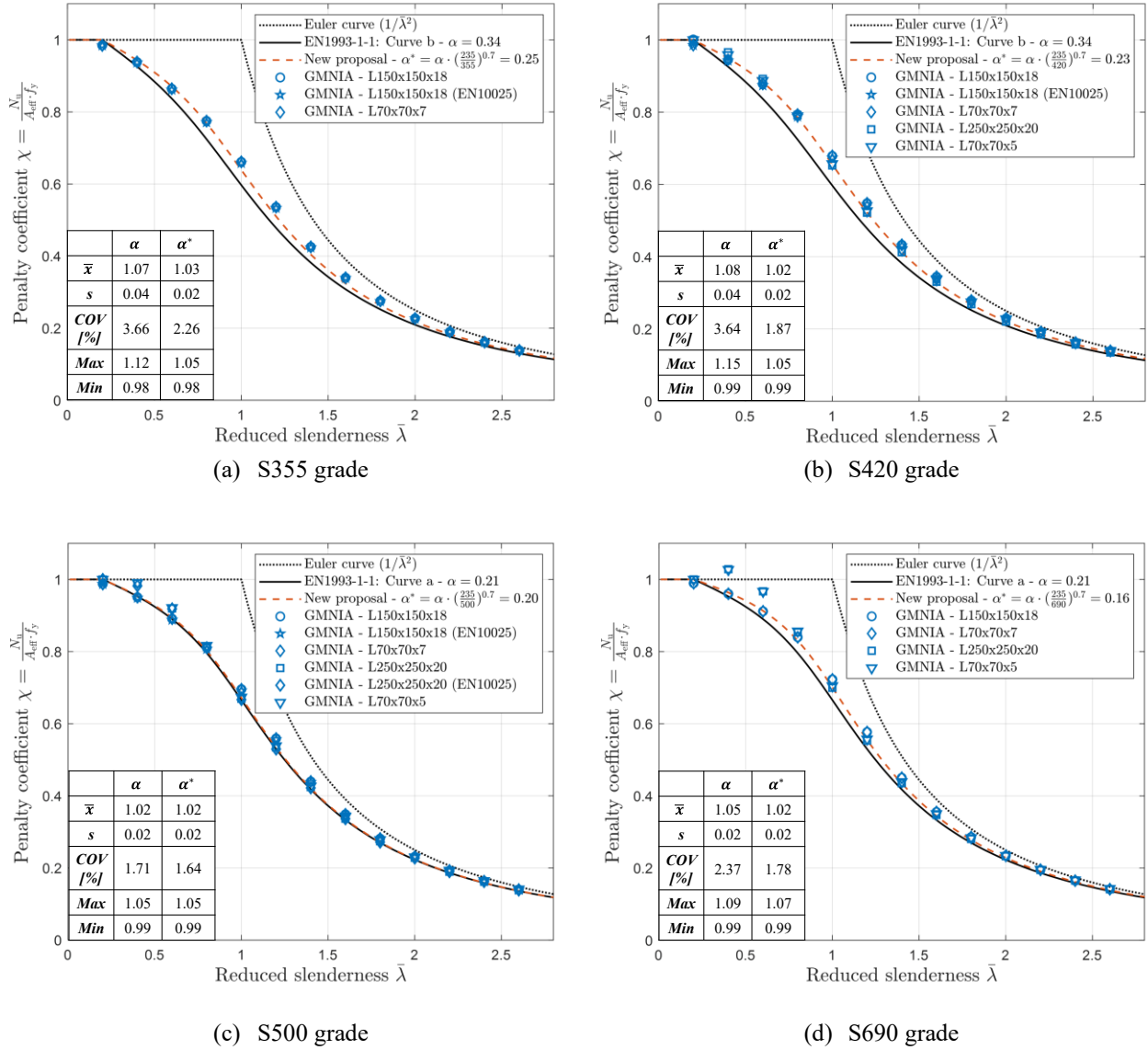


Fig. 9. Buckling curves resulting from the modified imperfection factor for S420 and S690 grades

Fig. 9 illustrates that the modified imperfection factor is particularly beneficial for intermediate grades as well as for future emerging grades. It can be seen that class 4 profiles exhibit slightly higher resistances at the low slenderness range, due to the rotation of the member at the first steps of loading, that initially tends to buckle in a flexural-torsional mode, while at the end flexural buckling governs. This behaviour has been detailed in [11] [16] [70].

6 Reliability analysis & resulting gains

The reliability analysis of the flexural buckling design recommendations of EN1993-1-1:2022 [31] and the proposed modified imperfection factor is performed according to the standard procedure set out in EN1990:2023 [84]. This standard establishes the principles and requirements for the safety assessment of structures and provides guidelines for structural reliability. In particular, Annex D of EN1990:2023 [84] provides a procedure for evaluating the reliability of a design rule modification based on testing and / or finite element simulations. The first step of this procedure consists of comparing the experimental / numerical values with the analytically computed ones. This comparison is reported in Fig. 10a when considering the predictions from EN1993-1-1:2022 [31] and in Fig. 10b for the improved resistance predictions of the new proposal.

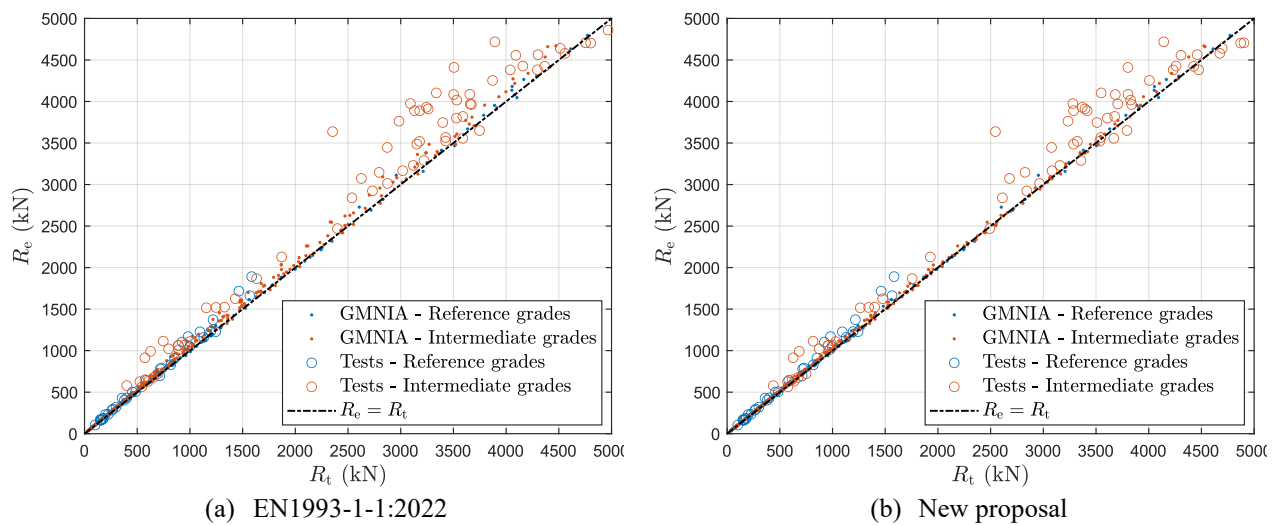


Fig. 10. Comparison between experimental tests / GMNIA simulations with predicted buckling resistances in accordance with (a) EN1993-1-1:2022, (b) the new modified imperfection factor

It should be noted that Fig. 10 has been plotted by distinguishing reference grades, i.e., S235 and S460, and intermediate grades, as the resistance predictions of the latter have been evaluated as too conservative with the current design recommendations. So, as expected, the new proposal is not affecting the scatter of data for reference grades, while the scatter is reduced for intermediate grades

as the new proposal improves the resistance predictions for those grades. Indeed, the data of Fig. 10b lie closer to the line with an angle of $\pi/4$ than in Fig. 10a, meaning that the modified imperfection factor is more accurate when predicting the resistance of hot-rolled equal-leg angles.

The procedure detailed in Annex D of EN1990-2023 [84] is then followed to assess the required partial factor γ_M^* so as to confirm the adequacy of the new proposal. The simplified procedure “P2” proposed by Tankova et al. [85] is adopted in the present paper to simplify the global procedure. It was proved that the simplified procedures are always safe-sided, compared to a strict application of Annex D procedure [85]. In this simplified approach, the variability of the yield stress (V_{fy}), the variability of the cross-sectional area (V_A), as well as the variability of the Young’s modulus (V_E) are accounted for to evaluate the variability of the basic variables ($V_{rt}^2 = \sum V_x^2$). Residual stresses and geometrical imperfections could have also been considered as random variables, but they are usually only considered implicitly in the resistance model due to the complexity of accounting their variability [85]. Annex E of the forthcoming EN1993-1-1:2022 [31] provides the material overstrength, i.e., the ratio between the mean value and the nominal value of the yield strength) with the corresponding COV for each steel grade. For instance, the material overstrength is 1.2 for a S355 grade and the coefficient of variation is 5.0%. The reference values proposed in Annex E were considered to account for the variability of the yield stress (V_{fy}) within the present study. The Annex E [31] also provides the COV of the Young’s modulus (V_E), which is 3%. The COV of the cross-sectional area is computed based on the COV of geometric parameters proposed in Annex E; so, the COV is 3.3% for L150x150x18 and has been assumed as a constant value for all other cross-section geometries. The results of the reliability analysis, including the number of elements in the sample “ n ”, the slope of the resistance function “ b ”, the coefficient of variation of the errors V_δ and the recommended partial factor γ_{M1}^* , are gathered in Table 5 for the current design recommendations of

EN1993-1-1:2022 [31] as well as for the new modified imperfection factor presented in this paper (see Eq. (8)).

Table 5. Reliability analysis of the current design recommendations and the new proposal according to the procedure set out in Annex D of EN1990-2023.

Design approach	Data	n	b	V_δ	γ_{M1}^*
EN1993-1-1:2022	GMNIA	554	1.03	3.2%	1.02
	Tests	118	1.12	10%	1.08
New proposal	GMNIA	554	1.01	2.0%	1.01
	Tests	118	1.08	8.4%	1.07

The recommended partial factors reported in Table 5 for the new proposal are slightly greater than 1.0 but smaller than those obtained for the current design recommendations in [31]. The coefficients of variation of the errors are smaller with the new proposal which demonstrates the reliability of this proposal compared to the existing design rules. In addition, as a reminder, the procedure “P2” used in the present reliability analysis leads to conservative γ_{M1}^* values due to the consideration of a “too large” value of V_{rt} . Therefore, the value of γ_{M1} could reasonably be conserved in case of incorporating the proposal in the future Eurocode 3.

The resistance gains resulting from the use of the modified imperfection factor instead of the current imperfection factor of EN1993-1-1:2022 [31] is represented in Fig. 11 for each steel grade as a function of the relative slenderness.

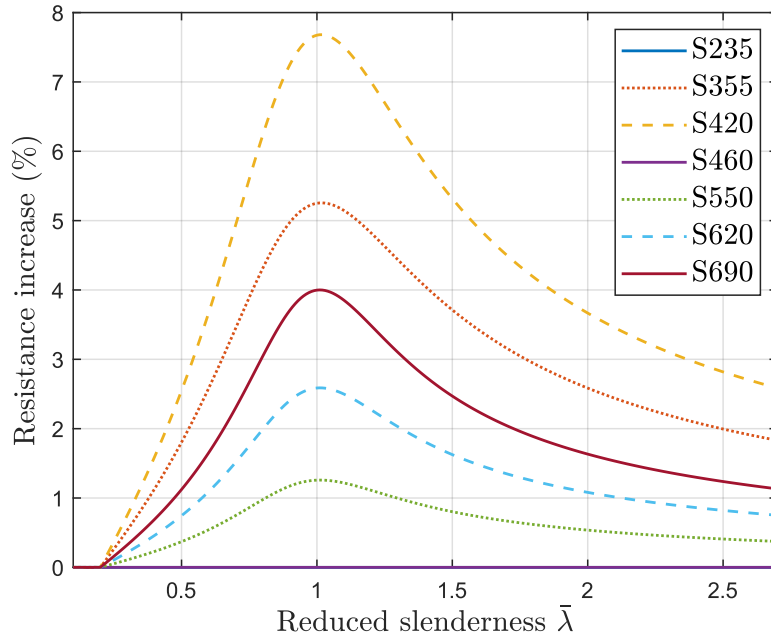


Fig. 11. Impact of the proposal on the buckling resistance capacity

As can be seen in Fig. 11, the most significant benefits are observed at a relative slenderness of $\bar{\lambda} = 1.0$, where initial imperfections exert the greatest influence on buckling resistance. The new proposal is particularly advantageous for intermediate steel grades such as S355 and S420 grades, which are the most underestimated grades despite S355 is seen as the standard steel grade in contemporary design. Saufnay et al. [10] already demonstrate that this modified imperfection factor increases the benefit in using high-strength steels to larger slenderness ratios for hot-rolled sections. In the context of the “build clever” strategy within the hierarchy to net-zero [86] [87], the implementation of appropriate design rules is essential not only to enable designers to exploit material performance efficiently and achieve weight savings, but also to embed sustainability at the core of the design process. By optimizing structural efficiency and minimizing resource use, designers contribute to reducing embodied carbon and environmental impact, aligning engineering decisions with broader sustainability goals like achieving net-zero by 2050.

7 Conclusions

This paper addresses the necessity of implementing appropriate design rules, in particular to improve the prediction of the flexural buckling resistance for hot-rolled equal-leg angles subjected to concentric loading. Through the performed developments, it has been demonstrated that the resistance for conventional grades S355 and S420 is underestimated in current design recommendations for the reasons summarised below. Based on this observation, a modified imperfection factor has been derived through a wide numerical campaign, offering the potential for significant weight savings for conventional grades as well as for those which could emerge as a prospective study. The main outcomes of this paper can be summarised as listed here below:

- The higher the yield stress, the less the detrimental impact of initial imperfections
- The forthcoming recommendations for reference grades (S235 and S460) in EN1993-1-1:2022 [31] fit well with numerical simulation predictions
- The buckling resistances for intermediate grades between S235 and S460 as well as future emerging grades (above S460) are clearly underestimated
- The scope of the modified imperfection factor proposed by Saufnay et al. [2] [58], initially developed for hot-rolled H-shaped and hollow sections, has been extended to include hot-rolled angle sections

The proposal of modification for the existing flexural buckling procedure is summarised in Fig. 12. As future work, a full statistical approach based on a Monte Carlo analysis should be carried out to assess the reliability of this new modified imperfection factor with a unitary partial safety coefficient $\gamma_{M1} = 1.0$. The proposed modified imperfection factor has been validated for steel grades up to S690, corresponding to the grades currently covered by EN 1993-1-1:2022 [31]. However, experimental validation of the underlying residual stress assumptions is presently available only up to S460. For

higher steel grades, no residual stress measurements or structural test results are available for hot-rolled angle sections, since such products are not yet manufactured as standard hot-rolled sections. Consequently, the results obtained beyond S460 should be regarded as prospective. Additional investigations would be required to confirm the applicability of the proposed formulation to future high-strength angle sections, particularly in the context of the future development of EN 1993-1-12, which is expected to extend the design provisions for structural steels up to S960.

The reliability of the proposal should be evaluated for unequal leg as well as welded angles to generalise the proposal to all steel angle profiles. In addition, Table 2 illustrates that there are different formulations in the literature for this modified imperfection factor; it would be of great interest to generalize and choose one single form for all types of elements to propose an amendment of the design rules.

$$\eta = \alpha \cdot (\bar{\lambda} - 0.2)$$

Cross-section		Limits	Buckling about axis	Buckling curves	
				S235 S275 S355 S420	S460 up to S700 inclusive
L-sections		Rolled sections	any	b	a

Table 8.3 of EN1993-1-1: 2022 [31]

→

$$\eta = \alpha^* \cdot (\bar{\lambda} - 0.2), \text{ with } \alpha^* = \alpha \cdot \left(\frac{235}{f_y}\right)^{0.7}$$

Cross-section		Limits	Buckling about axis	Buckling curves
L-sections		Rolled sections	any	b

Proposal of modification

Fig. 12. Modification proposal for the future version of EN1993-1-1

561 **References**

- 562 [1] “World Steel Association.” [Online]. Available: <https://worldsteel.org/>
- 563 [2] L. Saufnay, “High-strength steels and innovative design approaches for sustainable steel structures,”
- 564 University of Liège, Liège, 2025.
- 565 [3] L. Saufnay and J.-F. Demonceau, “Economic and environmental assessment of high-strength steel
- 566 grades,” *ce/papers*, vol. 6, no. 3–4, pp. 527–532, 2023.
- 567 [4] L. Saufnay and J.-F. Demonceau, “Establishment of reliable relative price predictions for high-strength
- 568 steel members,” *Steel Construction*, 2023, doi: <https://doi.org/10.1002/stco.202300013>.
- 569 [5] G. L. Griffis, G. Axmann, B. V Patel, C. M. Waggoner, and J. Vinson, “High-strength steel in the long-
- 570 span retractable roof of reliant stadium,” *2003 NASCC Proc., Baltimore*, 2003.
- 571 [6] N. Baddoo and A. Chen, “High Strength Steel Design and Execution Guide,” *SCI (the Steel*
- 572 *Construction 397 Institute)*, vol. 398, 2020.
- 573 [7] K. Mela and M. Heinisuo, “Weight and cost optimization of welded high strength steel beams,” *Eng.*
- 574 *Struct.*, vol. 79, pp. 354–364, Nov. 2014, doi: 10.1016/J.ENGSTRUCT.2014.08.028.
- 575 [8] L. Cederfelt and J.-O. Sperle, “High strength steel in the roof of swedbank arena savings in weight,
- 576 cost and environmental impact.,” in *Nordic Steel Construction Conference*, Oslo, Norway, Jan. 2012.
- 577 [9] M. Sansom and R. M. Lawson, “STRONGer Steels in the Built Environment (STROBE) - Deliverable
- 578 5.1: WP 5: Life cycle assessment of comparative conventional strength & HSS designs,” 2021.
- 579 [10] L. Saufnay and J.-F. Demonceau, “High-strength steels in steel structures: How to select the appropriate
- 580 steel grade?,” *Structures*, vol. 82, p. 110820, Dec. 2025, doi: 10.1016/J.ISTRUC.2025.110820.
- 581 [11] M.-Z. Bezas, J.-F. Demonceau, I. Vayas, and J.-P. Jaspart, “Experimental and numerical investigations
- 582 on large angle high strength steel columns,” *Thin-walled structures*, vol. 159, p. 107287, 2021, doi:
- 583 10.1016/j.tws.2020.107287.
- 584 [12] H. Y. Ban, G. Shi, Y. J. Shi, and Y. Q. Wang, “Column buckling tests of 420 MPa high-strength steel
- 585 single equal angles,” *International Journal of Structural Stability and Dynamics*, vol. 13, no. 02, p.
- 586 1250069, 2013, doi: 10.1142/S0219455412500691.
- 587 [13] A. Spiliopoulos, M.-E. Dasiou, P. Thanopoulos, and I. Vayas, “Experimental tests on members made
- 588 from rolled angle sections,” *Steel Construction*, vol. 11, no. 1, pp. 84–93, 2018, doi:
- 589 <https://doi.org/10.1002/stco.201710023>.
- 590 [14] M. Kettler, G. Lichtl, and H. Unterweger, “Experimental tests on bolted steel angles in compression
- 591 with varying end supports,” *J. Constr. Steel Res.*, vol. 155, pp. 301–315, 2019, doi:
- 592 <https://doi.org/10.1016/j.jcsr.2018.12.030>.
- 593 [15] L. Saufnay, A. Beyer, J.-P. Jaspart, and J.-F. Demonceau, “Experimental and Numerical Investigations
- 594 on Closely Spaced Built-Up Angle Members,” *Journal of Structural Engineering*, vol. 149, no. 4, p.
- 595 4023012, 2023, doi: 10.1061/JSENDH.STENG-11642.
- 596 [16] M.-Z. Bezas, J.-F. Demonceau, I. Vayas, and J.-P. Jaspart, “Compression tests on large angle columns
- 597 in high-strength steel,” *Steel Construction*, vol. 15, no. 1, pp. 43–47, 2022, doi:
- 598 <https://doi.org/10.1002/stco.202100051>.
- 599 [17] Z. Huang, H. Liu, H. Liu, and Z. Li, “Experimental study on stability behavior of equal-leg angle steel
- 600 columns,” *Thin-Walled Structures*, vol. 166, p. 108042, 2021, doi:
- 601 <https://doi.org/10.1016/j.tws.2021.108042>.
- 602 [18] M.-Z. Bezas *et al.*, “Buckling resistance of star-battened angle members made of high-strength steel,”
- 603 *Structures*, vol. 58, p. 105274, 2023, doi: <https://doi.org/10.1016/j.istruc.2023.105274>.
- 604 [19] I. Vayas, A. Charalampakis, and V. Koumouisis, “Inelastic resistance of angle sections subjected to
- 605 biaxial bending and normal forces,” *Steel Construction*, vol. 2, no. 2, pp. 138–146, 2009, doi:
- 606 <https://doi.org/10.1002/stco.200910018>.
- 607 [20] N. Schillo and M. Feldmann, “Buckling resistance of L-profiles in towers, masts and open line
- 608 constructions,” *Stahlbau*, vol. 84, no. 12, pp. 946–954, 2015, doi:
- 609 <https://doi.org/10.1002/stab.201510335>.

- [21] H. Farhoud, M.-Z. Bezas, J.-F. Demonceau, J.-P. Jaspart, and I. Vayas, "Buckling Strength of Rolled Angles: Comparison between International Codes," *Journal of Structural Engineering*, vol. 150, no. 9, p. 4024122, 2024, doi: 10.1061/JSENDH.STENG-12984.
- [22] Y. Sun, D. Song, S. Sun, and Y. Guo, "Behavior of large-size and high-strength steel angle subjected to eccentric load," *Structures*, vol. 57, p. 105161, 2023, doi: <https://doi.org/10.1016/j.istruc.2023.105161>.
- [23] L. Zhang and J. P. Jaspart, "Stability of members in compression made of large hot-rolled and welded angles," *Université de Liège*, 2013.
- [24] P. Može, L. G. Cajot, F. Sinur, K. Rejec, and D. Beg, "Residual stress distribution of large steel equal leg angles," *Eng. Struct.*, vol. 71, pp. 35–47, Jul. 2014, doi: 10.1016/J.ENGSTRUCT.2014.03.040.
- [25] H. Chen, H. Chen, K. Cao, and Y. Guo, "Study on Axial Compression Stability of Q345 Large-Section Angle Steel Columns," *Buildings*, vol. 13, no. 4, 2023, doi: 10.3390/buildings13041030.
- [26] H. Chen, D. Zeng, K. Cao, and Y. Guo, "Buckling behavior and design of large-section angle steel columns under axial compression," *Journal of Building Engineering*, vol. 78, p. 107578, Nov. 2023, doi: 10.1016/J.JOBE.2023.107578.
- [27] S. Kitipornchai and H. W. Lee, "Inelastic experiments on angle and tee struts," *J. Constr. Steel Res.*, vol. 6, no. 3, pp. 219–236, Jan. 1986, doi: 10.1016/0143-974X(86)90035-0.
- [28] S. Chen and X. Wang, "Buckling Strength of Single Angle Struts Part 1: Angles Subject to Axial Compression," *Advances in Structural Engineering*, vol. 16, no. 6, pp. 1129–1137, 2013, doi: 10.1260/1369-4332.16.6.1129.
- [29] M.-Z. Bezas, J.-F. Demonceau, I. Vayas, and J.-P. Jaspart, "Design rules for equal-leg angle members subjected to compression and bending," *J. Constr. Steel Res.*, vol. 189, p. 107092, 2022.
- [30] CEN, "EN1993-1-1:2005: Eurocode 3: Design of steel structures - Part 1-1: General rules and rules for buildings," Brussels, 2005.
- [31] CEN, "EN1993-1-1:2022: Eurocode 3: Design of steel structures - Part 1-1: General rules and rules for buildings," Brussels, 2022.
- [32] CEN, "EN 1993-3-1: Design of steel structures - Part 3-1: Towers, musts and chimneys,," Brussels, 2005.
- [33] CEN, "prEN 1993-3: Design of steel structures - Part 3: Towers, musts and chimneys," Brussels, 2022.
- [34] CEN, "EN1993-1-5: Eurocode 3: Design of steel structures – Part 1-5: Plated structural elements," Brussels, 2005.
- [35] M.-Z. Bezas, J.-P. Jaspart, I. Vayas, and J.-F. Demonceau, "New Consistent Set of Design Equations for Equal-leg Angle Sections and Members," *ce/papers*, vol. 5, no. 4, pp. 874–881, 2022, doi: <https://doi.org/10.1002/cepa.1830>.
- [36] Vayas et al., "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 (ANGELHY): Final report," 2021.
- [37] M.-Z. Bezas, J.-F. Demonceau, I. Vayas, and J.-P. Jaspart, "Classification and cross-section resistance of equal-leg rolled angle profiles," *J. Constr. Steel Res.*, vol. 185, p. 106842, 2021, doi: <https://doi.org/10.1016/j.jcsr.2021.106842>.
- [38] S. Antonodimitraki et al., "Buckling resistance of angle bracing members with one-leg bolted end connections," *ce/papers*, vol. 6, no. 3–4, pp. 1157–1162, 2023, doi: <https://doi.org/10.1002/cepa.2432>.
- [39] M. Kettler, A. Taras, and H. Unterweger, "Member capacity of bolted steel angles in compression – Influence of realistic end supports," *J. Constr. Steel Res.*, vol. 130, pp. 22–35, 2017, doi: <https://doi.org/10.1016/j.jcsr.2016.11.021>.
- [40] B. Behzadi-Sofiani, L. Gardner, M. A. Wadee, P. B. Dinis, and D. Camotim, "Behaviour and design of fixed-ended steel equal-leg angle section columns," *J. Constr. Steel Res.*, vol. 182, p. 106649, 2021, doi: <https://doi.org/10.1016/j.jcsr.2021.106649>.
- [41] D. Camotim, P. B. Dinis, and A. Landesmann, "Behavior, Failure, and Direct Strength Method Design of Steel Angle Columns: Geometrical Simplicity versus Structural Complexity," *Journal of Structural Engineering*, vol. 146, no. 11, p. 4020226, 2020, doi: 10.1061/(ASCE)ST.1943-541X.0002742.
- [42] J. Strating and H. Vos, "Computer Simulation of the ECCS Buckling Curve using a Monte-Carlo Method," in *Construction Métallique*, vol. 2, 1973, pp. 23–39.

- [43] J. Rondal and R. Maquoi, "Single equation for SSRC column-strength curves," *Journal of the Structural Division*, vol. 105, no. 1, pp. 247–250, 1979.
- [44] R. Maquoi and J. Rondal, "Mise en équation des nouvelles courbes européennes de flambement," 1978.
- [45] D. Sfintesco, "Fondement expérimental des courbes européennes de flambement," *Construction métallique*, vol. 3, pp. 5–12, 1970.
- [46] H. Beer and G. Schulz, "Bases théoriques des courbes européennes de flambement," *Construction Métallique*, vol. 3, no. 485, pp. 37–57, 1970.
- [47] X. Meng and L. Gardner, "Behavior and design of normal-and high-strength steel SHS and RHS columns," *Journal of Structural Engineering*, vol. 146, no. 11, p. 4020227, 2020.
- [48] S. H. Al-Sayed and R. Bjorhovde, "Experimental study of single angle columns," *J. Constr. Steel Res.*, vol. 12, no. 2, pp. 83–102, 1989.
- [49] S. M. R. Adluri and M. K. S. Madugula, "Flexural Buckling of Steel Angles: Experimental Investigation," *Journal of Structural Engineering*, vol. 122, no. 3, pp. 309–317, 1996, doi: 10.1061/(ASCE)0733-9445(1996)122:3(309).
- [50] S. M. R. Adluri and M. K. S. Madugula, "Development of Column Curve for Steel Angles," *Journal of Structural Engineering*, vol. 122, no. 3, pp. 318–325, 1996, doi: 10.1061/(ASCE)0733-9445(1996)122:3(318).
- [51] K. Cao, Y. J. Guo, and D. W. Zeng, "Buckling behavior of large-section and 420 MPa high-strength angle steel columns," *J. Constr. Steel Res.*, vol. 111, pp. 11–20, Aug. 2015, doi: 10.1016/J.JCSR.2015.03.014.
- [52] S. Qu, J. Wang, Y. Guo, J. Hao, and X. Yang, "Experiment and code-based study on high-strength steel pin-ended angles under axial compression," *Thin-Walled Structures*, vol. 149, p. 106541, Apr. 2020, doi: 10.1016/J.TWS.2019.106541.
- [53] R. Maquoi, "Some improvements to the buckling design of centrally loaded columns," in *Structural Stability Research Council, Proceedings of the Annual Meeting*, 1982.
- [54] B. Johansson, "Buckling Resistance of Structures of High Strength Steel," in *Use and application of high-performance steels for steel structures*, IABSE., International Association for Bridge and Structural Engineering (IABSE), 2005, ch. 5.3, pp. 120–128.
- [55] M. Feldmann *et al.*, "Rules on high strength steels (RUOSTE): Final report," 2016. Accessed: Jul. 13, 2021. [Online]. Available: <https://op.europa.eu/en/publication-detail/-/publication/515285b0-c820-11e6-a6db-01aa75ed71a1>
- [56] B. Somodi and B. Kövesdi, "Flexural buckling resistance of cold-formed HSS hollow section members," *J. Constr. Steel Res.*, vol. 128, pp. 179–192, 2017.
- [57] B. Somodi and B. Kövesdi, "Flexural buckling resistance of welded HSS box section members," *Thin-Walled Structures*, vol. 119, pp. 266–281, 2017.
- [58] L. Saufnay, J.-P. Jaspart, and J.-F. Démonceau, "Improvement of the prediction of the flexural buckling resistance of hot-rolled mild and high-strength steel members," *Eng. Struct.*, vol. 315, p. 118460, 2024.
- [59] N. Baddoo *et al.*, "Stronger steels in the built environment (STROBE): Final report," 2021.
- [60] X. Yun, Y. Zhu, X. Meng, and L. Gardner, "Welded steel I-section columns: Residual stresses, testing, simulation and design," *Eng. Struct.*, vol. 282, p. 115631, 2023, doi: <https://doi.org/10.1016/j.engstruct.2023.115631>.
- [61] J. Jönsson and T.-C. Stan, "European column buckling curves and finite element modelling including high strength steels," *J. Constr. Steel Res.*, vol. 128, pp. 136–151, 2017.
- [62] H. Fang, T.-M. Chan, and B. Young, "Structural performance of cold-formed high strength steel tubular columns," *Eng. Struct.*, vol. 177, pp. 473–488, 2018, doi: <https://doi.org/10.1016/j.engstruct.2018.09.082>.
- [63] Greisch office and University of Liège, "FineLg: non-linear finite element analysis program: user's manual," Sep. 2018, Liège.
- [64] ABAQUS, "Abaqus 6.14: User's manual," *Simulia*, no. Version 6.14, p. v6, 2014.
- [65] N. Boissonnade and H. Somja, "Influence of imperfections in FEM modelling of lateral torsional buckling," in *Proceedings of the Annual Stability Conference*, 2012, pp. 18–21.

- [66] N. Boissonnade, H. Degée, J. Naumes, and M. Oppe, “Experimental and numerical investigations for I-girders in bending and shear stiffened by trapezoidal stiffeners,” *Advanced Steel Construction*, vol. 4, no. 1, pp. 1–12, 2008.
- [67] V. de Goyet, “L’analyse statique non linéaire par la méthode des éléments finis des structures spatiales formées de poutres à section non symétrique,” *Université de Liège*, 1989.
- [68] M. Z. Bezas, J. P. Jaspart, I. Vayas, and J. F. Demonceau, “Design recommendations for the stability of transmission steel lattice towers,” *Eng. Struct.*, vol. 252, p. 113603, Feb. 2022, doi: 10.1016/J.ENGSTRUCT.2021.113603.
- [69] J. Wang and L. Gardner, “Flexural buckling of hot-finished high-strength steel SHS and RHS columns,” *Journal of Structural Engineering*, vol. 143, no. 6, p. 4017028, 2017.
- [70] M.-Z. Bezas, *Design of lattice towers from hot-rolled equal leg steel angles*. Université de Liège (Belgium), 2021.
- [71] CEN, “EN10025-2: Hot rolled products of structural steels - Part 2: Technical delivery conditions for non-alloy structural steels,” Brussels, 2019.
- [72] CEN, “EN10025-4: Hot rolled products of structural steels - Part 4: Technical delivery conditions for thermomechanical rolled weldable fine grain structural steels,” Brussels, 2019.
- [73] T. Tankova, L. S. da Silva, and F. Rodrigues, “Buckling curve selection for HSS welded I-section members,” *Thin-Walled Structures*, vol. 177, p. 109430, 2022.
- [74] R. C. Spoorenberg, H. H. Snijder, L.-G. Cajot, and N. Popa, “Buckling curves for heavy wide flange QST columns based on statistical evaluation,” *J. Constr. Steel Res.*, vol. 101, pp. 280–289, 2014.
- [75] CEN, “FprEN1993-1-14: Eurocode 3: Design of steel structures — Part 1-14: Design assisted by finite element analysis,” Brussels, Apr. 2024.
- [76] J. O. Gomes Jr, H. Carvalho, L. S. da Silva, J. O. Ferreira Filho, and A. Lavall, “Assessment of design procedures for the buckling resistance of hot-rolled steel equal leg angles under concentric and eccentric compression,” in *Structures*, 2023, p. 105308.
- [77] ECCS, *Manual on stability of steel structures*. S. l: [s. n.], 1976.
- [78] G. Shi, Z. Zhang, L. Zhou, and Y. Gao, “Experimental study and modeling of residual stresses of Q420 large-section angles,” *J. Constr. Steel Res.*, vol. 167, p. 105958, 2020, doi: <https://doi.org/10.1016/j.jcsr.2020.105958>.
- [79] H. Ban, G. Shi, Y. Shi, and Y. Wang, “Residual Stress Tests of High-Strength Steel Equal Angles,” *Journal of Structural Engineering*, vol. 138, no. 12, pp. 1446–1454, 2012, doi: 10.1061/(ASCE)ST.1943-541X.0000585.
- [80] J. V Bhilawe and L. M. Gupta, “Experimental investigation of steel equal angle subjected to compression,” *Engineering Structures and Technologies*, vol. 7, no. 2, pp. 55–66, 2015, doi: 10.3846/2029882X.2015.1113892.
- [81] L. N. Dissanayake, J. Yang, F. Walport, and L. Gardner, “Residual stress model for hot-rolled steel I-sections,” *Eng. Struct.*, vol. 366, p. 123378, Nov. 2026, doi: 10.1016/J.ENGSTRUCT.2026.123378.
- [82] A. Skiadopoulos, A. de C. e Sousa, and D. G. Lignos, “Experiments and proposed model for residual stresses in hot-rolled wide flange shapes,” *J. Constr. Steel Res.*, vol. 210, p. 108069, 2023.
- [83] S. Antonodimitraki, M.-Z. Bezas, J. Maesschalck, O. Anwaar, I. Vayas, and J.-F. Demonceau, “Effect of Joints on the Buckling Resistance of Angle Members Made of S460 Steel – Experimental Investigations ,” Jun. 2024, *The Swedish Institute of Steel Construction*. doi: 10.5281/zenodo.12384699.
- [84] CEN, “EN1990: Eurocode - basis of structural and geotechnical design,” Brussels, Mar. 2023.
- [85] T. Tankova, L. Simões Da Silva, L. Marques, C. Rebelo, and A. Taras, “Towards a standardized procedure for the safety assessment of stability design rules,” *J. Constr. Steel Res.*, vol. 103, pp. 290–302, Dec. 2014, doi: 10.1016/J.JCSR.2014.09.010.
- [86] E. Halliwell, “Putting the net-zero hierarchy into practice: Build clever,” *The Structural Engineer: journal of the Institution of Structural Engineer*, vol. 102, no. 3, pp. 10–11, 2024.
- [87] The Institution of Structural Engineers (IStructE), “The hierarchy of net zero design.” Accessed: Aug. 23, 2024. [Online]. Available: <https://www.istructe.org/resources/blog/the-hierarchy-of-net-zero-design/>