European Commission
Research Programme of the Research Fund for Coal and Steel

ANGELHY

Innovative solutions for design and strengthening of telecommunications and transmission lattice towers using large angles from high strength steel and hybrid techniques of angles with FRP strips

WORK PACKAGE 1 – DELIVERABLE 1.4

Comparison between EN 1993-1-1, EN 1993-3-1 and EN 50341-1

Coordinator:

National Technical University of Athens - NTUA, Greece

Beneficiaries:

ArcelorMittal Belval & Differdange SA - AMBD, Luxembourg

Université de Liege - ULG, Belgium

COSMOTE Kinites Tilepikoinonies AE - COSMOTE, Greece

Centre Technique Industriel de la Construction Métallique - CTICM, France

SIKA France SAS - SIKA, France

Grant Agreement Number: 753993

01/06/2018

AUTHORS:

CTICM
Steel Construction Research Division
Espace Technologique – Immeuble Apollo
L'Orme des Merisiers – F-91193 Saint Aubin
Authors: André Beyer, Alain Bureau

NATIONAL TECHNICAL UNIVERSITY OF ATHENS
Institute of Steel Structures
15780 Athens, Greece
Authors: Pavlos Thanopoulos, Ioannis Vayas, Dimitrios Vamvatsikos

ARCELORMITTAL BELVAL&DIFFERDANGE SA
Global R&D
66, rue de Luxembourg
L-4009 Esch-sur-Alzette, Luxembourg
Authors: Mike Tibolt

UNIVERSITE DE LIEGE
ARGENCO
Allée de la Découverte, 9
4000 Liège, Belgium
Authors: Jean-Pierre Jaspart

Critical assessment of EN rules for lattice towers and design assisted by testing

1 Introduction

In Europe, lattice towers may be designed according to different standards. In general, the design of steel lattice towers should be performed according to the Eurocodes and, in particular, EN 1993-1-1 – Eurocode 3 Part 1-1 [1] (providing general rules and rules for buildings), EN 1993-1-8 – Eurocode 3 Part 1-1 [2] (providing rules for connections) and EN 1993-3-1 – Eurocode 3 Part 3-1 [4] (providing specific rules for towers and masts). Yet, it can be noted that there are noticeable differences between these two standards concerning the design of angle sections. Additionally, the CENELEC standard EN 50341-1 [5] provides specific rules for lattice towers used in the field of overhead electrical lines exceeding 1 kV in alternating current (AC). The rules given in the EN 50341-1 address specific problems linked to the use of lattice towers in overhead lines but this standard also provides rules that define specific methods for the verification of the lattice tower and its constituting parts. Especially, the design methods for angle sections may diverge from the rules provided in the Eurocodes. Therefore, this report aims at reviewing the design methods provided in the three cited standards in order to quantify the resulting differences in the design of lattice towers and angle sections.

First, the structure of EN 50341-1 is outlined in order to highlight the paragraphs of this standard that are in concurrence to the design rules provided in the Eurocodes. Then, the methods provided for the structural analysis and the design of angle sections are compared and the resulting differences are quantified.

The last main chapter of this report concerns design assisted by testing. Indeed, according to EN 50341-1 one single test can be applied to validate the calculation hypotheses used for design of the lattice tower. In general, the exploitation of one single test cannot be done using the classical statistical methods defined in of EN 1990 defining the provisions for design assisted by testing in the framework of the Eurocodes. A critical review of the test exploitation performed in EN 50341-1 is therefore necessary.

2 Principal notations

Latin capital letters

A_{gv}: gross area resisting shear in EN 50341-1 bolt tearing resistance model (see Table 5)

A_{net}: net area for the ultimate tension resistance of angle sections according to EN 1993-1-8 (see Table 2)

A_{nt}: net area resisting tension in the bolt tearing resistance model (see Table 5)

A_{nv}: net area resisting shear in EN 1993-1-8 bolt tearing resistance model (see Table 5)

$F_{b,Rd}$: bolt bearing resistance (see Table 7)

$N_{u,Rd}$: ultimate tension resistance (see Table 2)

$N_{t,Rd}$: design tension resistance (of the angle section)

$V_{eff,1}$: block tearing resistance according to EN 1993-1-8 (see Table 5)

$V_{eff,2}$: block tearing resistance according to EN 50341-1 (see Table 5)

Latin small letters:

a: distance between packing plates for closely spaced built-up members (see Figure 31)

b: leg width of an angle section

b_1: width of the connected leg of an angle section (see Figure 4)

b_2: width of the unconnected leg of an angle section (see Figure 4)

d: diameter of the bolt (see Table 7)

d_0: diameter of the bolt hole (see Figure 4)

e_1: distance measured parallel to the load between the bolt and the edge (see Figure 4)

e_2: distance measured perpendicular to the load between the bolt and the edge (of the angle section leg – see Figure 4)

f_u: ultimate tension resistance

f_y: yield stress

h: width of the larger leg for unequal leg angles

i_{min}: minimum radius of gyration

i_v: radius of gyration about the v-axis

i_z: radius of gyration about the z-axis

k_1: factor in the bolt bearing resistance model (see Table 7)

m: number of packing plates (see Eq. 5.16)

n: number (of bolts, of tests, etc.)

p_1: bolt pitch measured parallel to the force acting in the angle section (see Figure 4)

t: thickness (see Figure 4)

Greek small letters:

α: factor in the bolt bearing resistance model (see Table 7)

β_2: factor in the EN 1993-1-8 ultimate tension resistance model (see Table 2)

β_3: factor in the EN 1993-1-8 ultimate tension resistance model (see Table 2)

χ_T: reduction factor for torsional buckling (see Eq. 5.7)

ε: factor equal to $\sqrt{235/f_y}$

γ_{mi}: partial factor

η_i: factor in the EN 50341-1 bolt bearing resistance model (see Table 7)

$\bar{\lambda}$: relative slenderness (see Eq. 5.8)

$\bar{\lambda}_T$: relative slenderness for torsional buckling (see Eq. 5.4)

$\bar{\lambda}_{eff}$: modified relative slenderness for buckling of angle sections (see Eq. 5.9)

φ: factor in the buckling resistance model (see Eq. 5.6)

ρ: reduction factor accounting for the effect of local buckling (see Figure 18 and Eq. 5.7)

3 Structure of EN 50341-1

The CENELEC standard EN 50341-1 is composed of 12 main chapters and 16 annexes as summarised in Figure 1. The numbered items in Figure 1 correspond to actual chapters whereas the other items are only introduced in order to represent the functional structure of this standard. From a most general point of view, the standard is divided into requirements specifically defined for overhead lines. These contain electrical requirements for the lines that are entirely out of the scope of the structural Eurocodes but they also contain some specific regulations for the load cases to be considered. However, they are in line with the principles of the Eurocode 1. In particular, the calculation of wind loads is based on the applicable provisions of EN 1991-1-4.

The second main part of the standard EN 50341-1 concerns the design of the components of the line as the supporting structure and the equipment used for the transportation of electricity (for example the conductors). The poles or towers used as supporting structure may be fabricated from concrete, wood or steel. Depending on the material used, the CENELEC standard defines specific design methods that partially overlap the individual parts of the Eurocodes treating structures fabricated from these materials (EN 1992 for concrete structures; EN 1995 for wooden structures; EN 1993 for steel structures). In the frame of ANGELHY, §7.3 and Annex J (highlighted in blue in Figure 1) are of special interest as they define specific rules for the design of steel lattice towers and angle sections. Nonetheless, in general, §7.3 refers to the provisions of the relevant parts of EN 1993 (for example Part 1-1 for the cross-section classification, Part 1-8 for the resistance of connections) as reference method. Slightly different provisions are defined for the structural analysis (see chapter 4 for more details). Additionally, EN 50341-1 allows the use of alternative methods as defined in its Annex J for the design of lattice towers. A detailed analysis of these methods is given in chapter 5 of this report. Finally, §7.3.9 of EN 50341-1 addresses the topic of “design assisted by testing”. The corresponding provisions are presented and discussed in chapter 6 in order to analyse the compatibility with EN 1990.

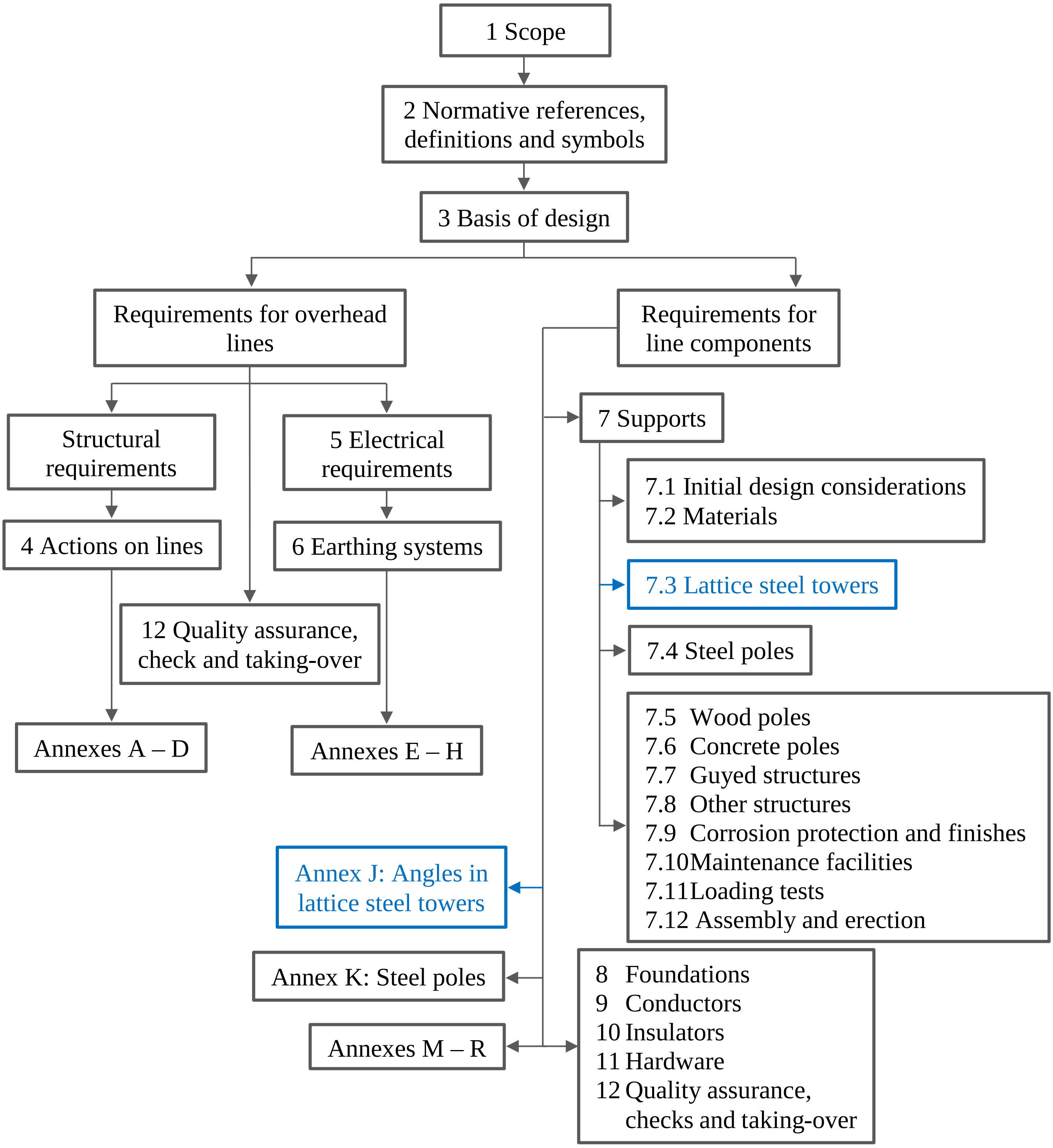


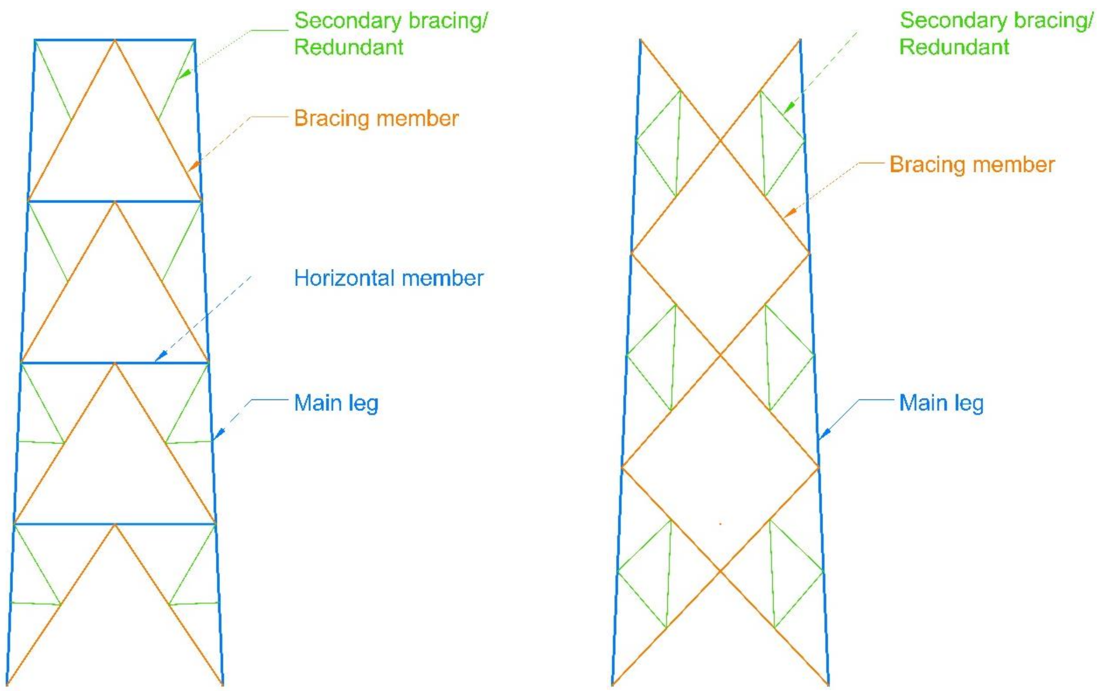
Figure 1: Structure of EN 50341-1

4 Structural modelling and structural analysis

4.1 Modelling of lattice towers

A typical tower basically consists of main legs or chords as well as bracings and secondary bracings (also referred to as "redundants") as shown in Figure 2 for two widely used typologies of transmission towers. Deliverable D1.1 – Tower Structural Typologies present more details concerning the typical design of lattice towers. In practice:

- the main legs are modelled considering continuity over their total length;
- the bracing members and horizontal members are considered as pinned at their ends connected to the main legs and, in the case shown in Figure 2a), as pinned to the horizontal members;
- the secondary bracing elements are also considered as pinned at their ends.



a) b)

Figure 2: Examples of a lattice tower

These practical habits are valid according to EN 1993-1-1 and EN 1993-3-1 provided that the stiffness respects the criteria defined in EN 1993-1-8 for nominally pinned and nominally rigid joints. Nonetheless, it may be noted that there are no specific criteria for the stiffness of joints in lattice structures. The CENELEC standard EN 50341-1 accepts the hypotheses concerning the joint's behaviour without referring to the design criteria of EN 1993-1-8. Owing to the practical design of lattice towers made from angle sections, the omission of the joint stiffness criteria is however acceptable for this specific structure.

Additionally to the provisions concerning the behaviour of the joints, EN 50341-1 proposes two specific rules for the modelling of lattice towers made of angle sections:

- it is considered that the redundants only act as stabilizing elements for the main bracings. They are therefore supposed to be unstressed under the applied external loads and consequently, they may be neglected in a first order elastic analysis (see paragraph 4.2);
- bending moments at the element ends of single angle sections resulting from the eccentric introduction of the axial force may be neglected for the design of the joints and of the angle sections in compression as the method provided for the buckling resistance of the angle sections already accounts for the bending moments (see paragraph 5.4). The elements may therefore be modelled without eccentricities at the nodes in the numerical model.

Especially, the second point may influence the global behaviour and the resistance of lattice towers. A more detailed analysis of possible modelling hypotheses is performed in WP 2 – Task 2.6 of the ANGELHY project.

Besides the modelling of the geometry, the introduction of loads is of special interest when analysing lattice towers. In fact, EN 50341-1 allows designers to neglect bending moments resulting from wind applied to the individual members. The wind loads can therefore be considered to be applied only to the nodes of the lattice tower. Neither EN 1993-1-1 nor EN 1993-3-1 address the application of wind loads explicitly. Nonetheless, it is common practice to apply the loads on the members, i.e. for the case of lattice towers the wind loads are modelled as distributed loads on the members including, tower legs, horizontal members and tower diagonals. Admittedly, the exposed area of the tower members as well as the resulting bending moment is generally small. However, as the bending moment resistance of angle sections is small, too, the bending moments acting in the tower members may be relevant for the design of angle sections. Consequently, it is appears of economic interest to exploit the plastic bending moment capacities and particularly the plastic nonlinear interaction between bending moments and axial forces. This specific problem is addressed in detail in Task 2.2 of the ANGELHY project.

4.2 Structural analysis

EN 1993-1-1 defines in detail the approach that has to be adopted for the determination of the internal forces and moments acting in a structure. In general, Part 1-1 of Eurocode 3 organises the structural analysis in several steps as represented in Table 1 in a simplified manner.

Table 1: Steps for structural analysis according to EN 1993-1-1

Step	Comment
1) Modelling of structure	Reliable hypotheses of the joint behaviour have to be considered according to the stiffness criteria provided in EN 1993-1-8 (pinned, semi-rigid or rigid).
2) Application of loads	Relevant parts of Eurocode 1 have to be applied; additional information on ice loads and wind loads on lattice towers are provided in EN 1993-3-1.
3) Choice of the material behaviour	The structural analysis may be performed based on an elastic or an elastic-plastic material behaviour. However, plastic analysis is authorised only for structures of class 1 sections.
4) Check the influence of sway second order effects	Depending on the lateral stiffness of the structure and the applied vertical loads, sway second order effects may increase the internal forces and moments. Their influence should be accounted for for a given load combination if the corresponding critical load amplification factor associated with global instability α_{cr} is less than 10.
5) Check the influence of sway imperfection	If the lateral loads acting on the structure (effect of wind for example) are low compared to the gravity loads, an additional sway imperfection should be taken into account in order to ensure a minimum lateral stiffness of the structure.
6) Check the influence of member imperfection and member second order effects	In particular cases, a low stiffness of an individual member may influence the global behaviour of the structure. If the critical load amplification factor associated with flexural buckling of an individual column is less than 4, it may be necessary to include a member imperfection and to consider sway and member second order effects simultaneously during the analysis.

Owing to the structural particularities of lattice towers, EN 1993-3-1 as well as EN 50341-1 propose a simplified procedure. In particular:

- an elastic material behaviour should be considered;
- lattice towers may be analysed according to first order theory;
- sway imperfection and member imperfection may be neglected.

Nonetheless, it appears that these simplifications are entirely justified and consequently, it may be concluded that the standards EN 1993-3-1 and EN 50341-1 providing rules for lattice towers are in accordance with Part 1-1 of Eurocode 3 defining the general rules for the design of steel structures.

5 Design of angle sections

5.1 General

Paragraph 4 of this report has shown that the provision for the structural modelling and analysis of lattice towers defined in EN 1993-1-1, EN 1993-3-1 and EN 50341-1 are in good agreement. Inversely, it is shown next, that the provision defined for the design of angle sections may differ greatly from one standard to another. The different design methods are compared on a qualitative and quantitative basis in order to highlight again the research needs that are focussed on in the framework of ANGELHY.

The axes referred to in the following are defined in Figure 3.

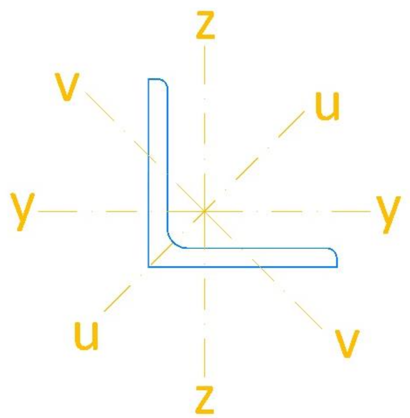


Figure 3: Definition of axes for angle sections

5.2 Angle sections in tension connected on one leg only

In general, diagonals of the main and secondary bracings as well as the horizontal members are connected on one single leg at their ends. Most frequently, only one single bolt is used per connection but, in special cases, one may also find connections with several bolts in practice. The geometry of the connection studied here is recalled in Figure 4.

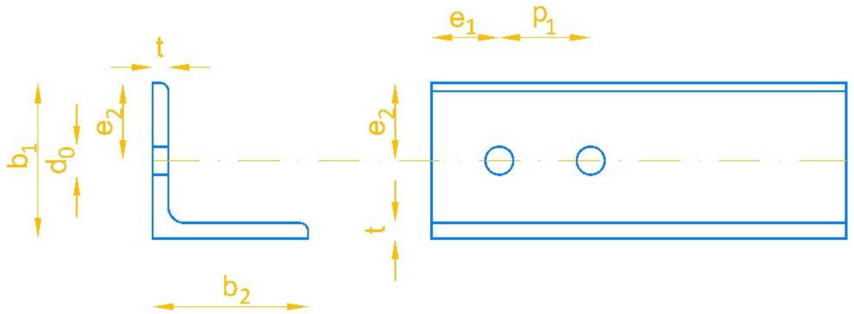


Figure 4: Geometry of a single leg angle connection

The resistance of the angle section is determined by the ultimate resistance $N_{u,Rd}$ of the cross-section reduced by the influence of the bolt holes. Even, if the basic resistance criterion is identical in the three studied standards, the expression of the ultimate resistance $N_{u,Rd}$ differs between the Eurocodes EN 1993-1-1 (the design expression for angle sections is actually defined in EN 1993-1-8) and EN 1993-3-1 on one hand and the CENELEC standard EN 50341-1 on the other hand. The expressions for the ultimate resistance $N_{u,Rd}$ are summarised in Table 2 and

Table 3.

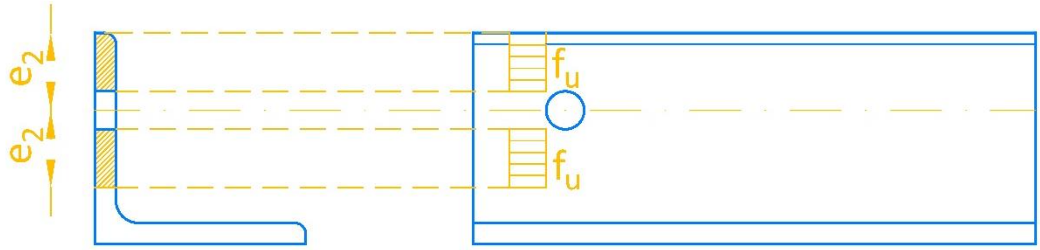
Table 2: Expressions of $N_{u,Rd}$

Case	EN 1993-1-1/EN 1993-1-8 and EN 1993-3-1	EN 50341-1
Connection with 1 bolt	$N_{u,Rd} = 2(e_2 - 0{,}5d_0)t\, {}^{f_u}/_{\gamma_{M2}}$	$N_{u,Rd} = (b_1 - d_0)t\, {}^{f_u}/_{\gamma_{M2}}$
Connection with 2 bolts	$N_{u,Rd} = \beta_2 A_{net}\, {}^{f_u}/_{\gamma_{M2}}$	$N_{u,Rd} = \left(b_1 - d_0 + \frac{b_2}{2}\right) t\, {}^{f_u}/_{\gamma_{M2}}$
Connection with 3 and more bolts	$N_{u,Rd} = \beta_3 A_{net}\, {}^{f_u}/_{\gamma_{M2}}$	

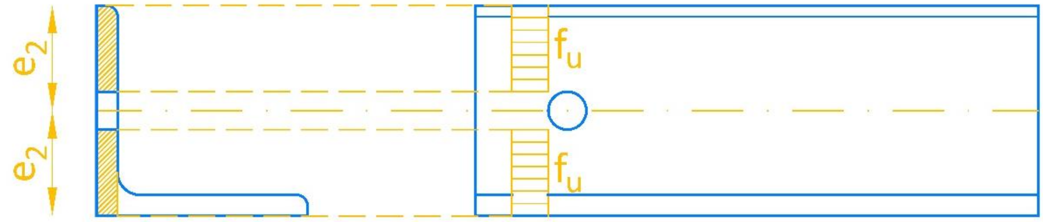
Table 3: Expression of β_2 and β_3 according to EN 1993-1-8

Condition on the bolt pitch p_1		
$p_1 \leq 2{,}5\ d_0$	$2{,}5\ d_0 \leq p_1 \leq 5{,}0\ d_0$	$p_1 \geq 5{,}0\ d_0$
$\beta_2 = 0{,}4$	$\beta_2 = 0{,}4 + 0{,}3\frac{p_1 - 2{,}5d_0}{2{,}5d_0}$	$\beta_2 = 0{,}7$
$\beta_3 = 0{,}5$	$\beta_3 = 0{,}5 + 0{,}2\frac{p_1 - 2{,}5d_0}{2{,}5d_0}$	$\beta_3 = 0{,}7$

Table 2 shows that the provisions given in EN 1993-1-8 are more complex than the resistance model defined in EN 50341-1. It may be noted that this complexity does not always yield more economic results but EN 1993-1-8 may predict lower ultimate resistances depending on the joint configuration. This fact is highlighted qualitatively in Figure 5 and Figure 6. Indeed, the resisting area predicted by EN 50341-1 is independent from the position of the bolt along the leg and it is equal to the area of the connected leg reduced by the bolt hole (see Figure 6). Inversely, EN 1993-1-8 “limits” the resisting area to twice the distance e_2 (distance between centroid of the bolt hole and the closest edge). If the bolt is situated near the edge, it is therefore not possible to mobilise the entire area of the leg. In order to consider the entire area of the connected leg, the bolt has to be situated at mid-width of the leg. Moving the bolt closer to the heel reduces again the area that can be considered to resist the applied axial tension force.



a) Bolt is situated above the half width of the leg



b) Bolt is situated at the half width of the leg

Figure 5: Resisting area for a one-leg connection with one bolt according to EN 1993-1-8

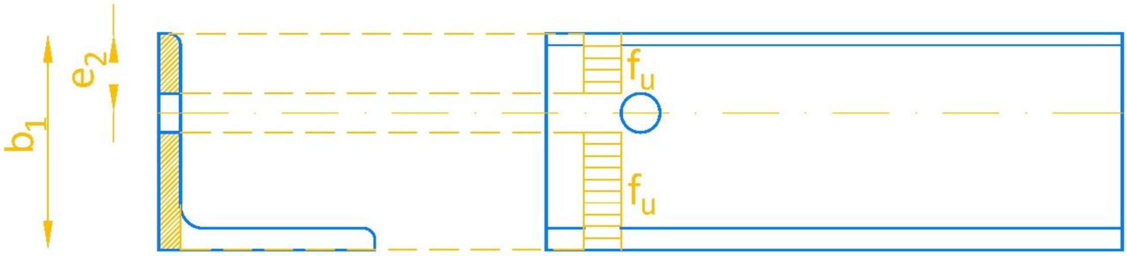


Figure 6: Resisting area for a one-leg connection with one bolt according to EN 50341-1

In order to obtain a quantitative evaluation of the observed difference, the diagram of Figure 7 represents the ratio between the ultimate tension resistance predicted by EN 1993-1-8 and the ultimate tension resistance predicted by EN 50341-1. A value lower than one therefore indicates that the Eurocode provisions are more conservative than the provisions given in EN 50341-1. For large angle sections, the difference may attain up to 70%. Yet, one may note that the bolt should be situated as far as possible from the intersection between the minor-axis v and the leg of the angle section so as to avoid the generation of bi-axial bending due to the eccentricity of the axial force (this appears to be even more important for angle sections in compression). The distance between the heel and the point of intersection of the v-v axis with the leg is approximatively equal to 0,47b. The dotted green line in Figure 7 represents this distance. If the bolt is situated at the interaction between the v-v axis and the leg, the difference between the resistance models reduces to approximatively 8%. In this case, both resistance models are therefore rather close.

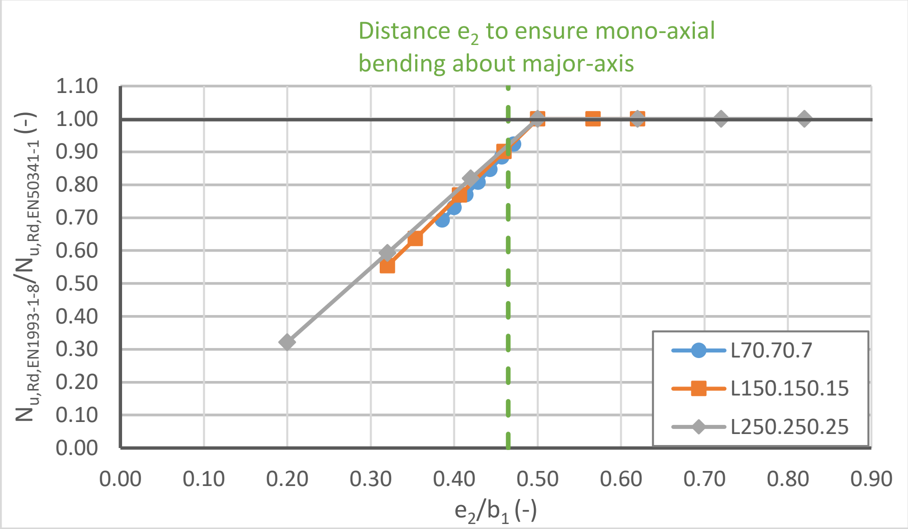


Figure 7: Ultimate tension resistance of a one-leg connection with one bolt – comparison between EN 1993-1-8 and EN 50341-1

Next, it is interesting to discuss the differences between EN 1993-1-8 and EN 50341-1 for connections with more than one bolt. The current version of EN 1993-1-8 [2] distinguishes two cases: connections with two bolts and connections with at least three bolts. In both cases, the resisting area corresponds to the net section reduced by a factor β_i.

Inversely, EN 50341-1 only considers the case of connections made of two and more bolts and defines the resisting area of the angle section equal to the area of the connected leg reduced by the bolt hole and half the area of the second leg.

Figure 8 compares again the provisions of EN 1993-1-8 and the provisions of EN 50341-1. As for connections with only one bolt, the resistance predicted by EN 1993-1-8 is very conservative. Only for values of p_1 approaching $5{,}0d_0$ the difference becomes lower than 10%.

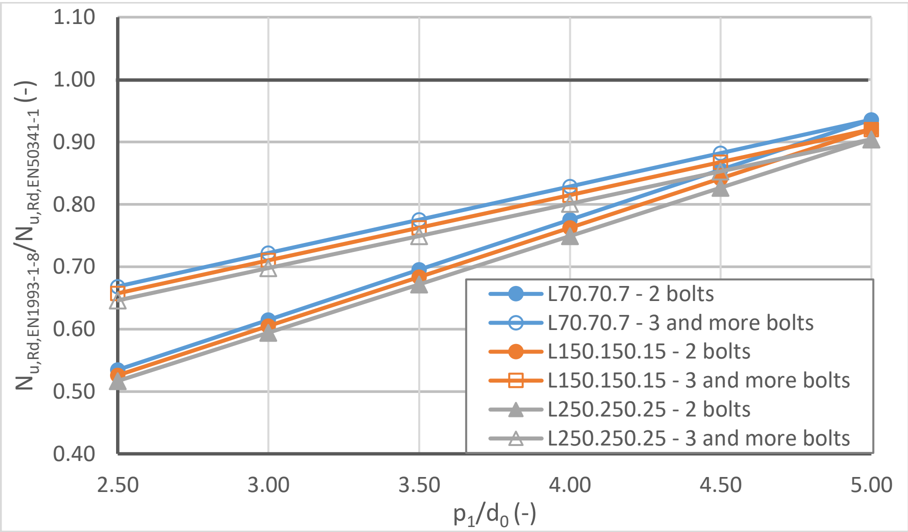


Figure 8: Ultimate tension resistance of a one-leg connection with one bolt – comparison between EN 1993-1-8 and EN 50341-1

It should be noted that the distinction between connections with two bolts and connections with at least three bolts introduced in EN 1993-1-8 has been criticised because it leads to a major inconsistency (see for example reference [12]). Indeed, if one considers a connection made of two bolts possessing a distance of $5d_0$, one might be tempted to increase its ultimate tension resistance by introducing an additional bolt in between. The addition of the third bolt in the middle of the two others, divides the distance p_1 by two. Consequently, the β_3 factor equals 0,5 (three bolts with a distance $p_1 = 2{,}5d_0$) whereas the value of β_2 related to the original connection with two bolts distanced by $p_1 = 5{,}0d_0$ is 0,7. Therefore, adding a bolt leads to a reduction of the ultimate tension resistance in this case. In order to understand this inconsistency, it is necessary to go back to the drafting of EN 1993-1-8. Indeed, the expressions given for the ultimate tension resistance (see Table 2) are intended to cover the effect of block tearing that has been introduced as supplementary check in the last draft of the current version of EN 1993-1-8 as recalled in reference [12]. Consequently, the check of the net section ultimate resistance by the expressions given in Table 2 and the check of block tearing are redundant in EN 1993-1-8. In the future version of EN 1993-1-8 [3] this inconsistency is eliminated and new design criteria for bolted connections and in particular for connections of angle sections at one leg only are introduced. As the final draft of EN 1993-1-8:2020 is available at the time of writing of the present report, the following comparisons are based on the new version of Eurocode 3 Part 1-8 [3]. Consequently, the comparisons are not biased by the inconsistencies in the current version of Eurocode 3 Part 1-8 [2].

In the following, all possible failure modes are considered in the comparisons of the design criteria for bolted angle sections in tensions. Depending on the connection geometry, one of the failure modes represented in Table 4 becomes relevant. Also, one may note that the relevant failure mode for the same connection geometry may be different in both standards as the provided expressions for a given failure mode are different.

Table 4: Failure modes considered for the tension resistance of angle sections

Failure mode	EN 1993-1-8	EN 50341-1
Ultimate tension resistance	see Table 6	see Table 2 and Table 6
Block tearing	see Table 5	see Table 5
Bearing	see Table 7	see Table 7

The expressions associated with the failure modes given in Table 4 are represented in Table 5 for the block tearing resistance check, Table 6 the ultimate tension resistance check and Table 7 for the bearing resistance check. The three tables show that the resistance models for these failure modes are similar but possess slight differences. For example, the expression given in Eurocode 3 Part 1-8:2020 for the ultimate tension resistance considers 75% of the net area of the angle section as resisting. EN 50341-1 proposes an equation that is presented differently, but yields nearly the same result as the term between the parentheses tends to 75% of the net area especially for equal leg angle sections.

Regarding Table 5, one may notice that EN 1993-1-8 considers the net area for the terms linked to the ultimate tension resistance f_u whereas EN 50341-1 allows the designer to use the gross area in shear. Yet, a supplementary reduction is applied through the factors 0,5x0,8 compared to $1/\sqrt{3}$. Obviously, a quantitative evaluation is necessary to obtain a global view of the differences.

Table 5: Block tearing resistance according to EN 1993-1-8:2020 and EN 50341-1

	EN 1993-1-8:2020	EN 50341-1
Block tearing resistance	$V_{eff,1,Rd} = \left[f_u A_{nt} + min\left(\frac{f_y A_{gv}}{\sqrt{3}}; \frac{f_u A_{nv}}{\sqrt{3}}\right)\right] / \gamma_{M2}$	$V_{eff,2,Rd} = 0{,}8\left(\frac{f_u A_{nt}}{\gamma_{M2}} + 0{,}5\frac{f_u A_{gv}}{\gamma_{M2}}\right)$
Area resisting tension	$A_{nt} = t(e_2 - 0{,}5d_0)$	
Net area resisting shear	$A_{nv} = t(e_1 + [n-1]p_1 - [n-0{,}5]d_0)$	
Gross area resisting shear	$A_{gv} = t(e_1 + [n-1]p_1)$	

Table 6: Ultimate tension resistance

Number of bolts	EN 1993-1-8:2020	EN 50341-1
1	$N_{u,Rd} = \frac{2(e_2 - 0{,}5d_0)tf_u}{\gamma_{M2}}$	$N_{u,Rd} = (b_1 - d_0)t\, {}^{f_u}/_{\gamma_{M2}}$
2 and more	$N_{u,Rd} = min\left(\frac{0{,}75A_{net}f_u}{\gamma_{M2}}; V_{eff,1,Rd}\right)$ with: $A_{net} = A - nd_0t$	$N_{u,Rd} = \left(b_1 - d_0 + \frac{b_2}{2}\right)t\, {}^{f_u}/_{\gamma_{M2}}$

Finally, Table 7 represents the bolt bearing resistance models. As for the two previous failure modes, the bolt bearing resistance model proposed in the two standards are similar. One may note that the recommended value for the η_i factors, used in EN 50341-1, is 1,0. Consequently, it is possible to attain a maximum of $3f_u$ as reference stress in the EN 50341-1 resistance model. This is also the maximum value that may be attained by the Eurocode 3 Part 1-8 resistance model. It is interesting to note that EN 50341-1 introduces the influence of the edge distance e_2 (perpendicular to the load application). A similar criterion provided in the 2005 version of Eurocode 3 Part 1-8 has been suppressed in EN 1993-1-8:2020. Nonetheless, for the angle sections studied hereafter, it is considered that the distance between the heel and the bolt hole is equal to 0,47b in order to ensure mono-axial bending resulting from the eccentricity of the applied axial force. Consequently, the resulting e_2/d_0 ratios are based on an edge distance e_2 of 0,53b. The corresponding values of the ratio e_2/d_0 are 2,10, 2,79 and 4,65 for the angle section L70.70.7, L150.150.15 and L250.250.25, respectively. Owing to the high values of the ratio e_2/d_0 the criterion linked to the edge distance e_2 does not become relevant in the following. Finally, one may note that the influence of the ratio between the ultimate tension resistance of the bolt f_{ub} and the ultimate tension resistance f_u of the angle section is considered in EN 1993-1-8:2020. This criterion may become relevant if bolts of low strength (of class 4.6 for example) are used with high strength steel section. Hereafter, it is considered that the ratio f_{ub}/f_u is always higher than one and consequently, the associated criterion does not become relevant.

Table 7: Bearing resistance according to EN 1993-1-8 and EN 50341-1

	EN 1993-1-8	EN 50341-1
Bearing resistance	$F_{b,Rd} = dt\dfrac{\alpha k_1 f_u}{\gamma_{M2}}$	
Factor α	For outer bolts: $\alpha = Min\left(\dfrac{e_1}{d_0}; 3\dfrac{f_{ub}}{f_u}; 3\right)$ For inner bolts: $\alpha = Min\left(\dfrac{p_1}{d_0} - \dfrac{1}{2}; 3\dfrac{f_{ub}}{f_u}; 3\right)$	For outer bolts: $\alpha = Min\begin{pmatrix} 3\eta_1; \\ 1{,}2\dfrac{e_1}{d_0}\eta_2; \\ 1{,}85\left[\dfrac{e_1}{d_0} - 0{,}50\right]\eta_3; \\ 2{,}3\left[\dfrac{e_2}{d_0} - 0{,}50\right]\eta_5 \end{pmatrix}$ For inner bolts: $\alpha = Min\begin{pmatrix} 3\eta_1; \\ 0{,}96\left[\dfrac{p_1}{d_0} - 0{,}50\right]\eta_4; \\ 2{,}3\left[\dfrac{e_2}{d_0} - 0{,}50\right]\eta_5 \end{pmatrix}$
Factor k_1	$k_1 = 1$ for steel grades lower than S460	$k_1 = 1$

First, the overall tension resistance $N_{t,Rd}$ accounting for all failure modes given in Table 4 are compared for angle sections connected with **one bolt** in Figure 9.

This figure indicates that EN 50341-1 gives more economic results for connections with small and intermediate edge distances that are of most practical interest. For section L150.150.15 and 250.250.25, both standards predict bearing failure based on α resulting from the condition on the ratio e_1/d_0 other the whole range of the diagram. The observed differences consequently yield directly from the condition represented in Table 7.

- for e_1/d_0 = 1,2: α is equal to 1,2 according to EN 1993-1-8:2020 and α is equal to 1,295 according to EN 50341-1;
- for e_1/d_0 = 1,2 to 2,5: α is equal is equal to the ratio e_1/d_0 according to EN 1993-1-8:2020 and α is equal to $1{,}2e_1/d_0$ according to EN 50341-1 leading to the observed difference of 17% (=1-1/1,2);
- from e_1/d_0 = 2,5: α is equal to the maximum value of 3 according to EN 50341-1, according to EN 1993-1-8:2020 it still increases as the limit value of 3 is only attained for e_1/d_0 = 3.

For the L70.70.7 section, the differences between both standards are slightly different as starting from a ratio e1/d0 of 1,92, the block tearing resistance becomes relevant for the tension resistance according to EN 50341-1. As the block tearing resistance increases more slowly than the bearing resistance, the predictions of EN 1993-1-8:2020 becomes favourable starting from e_1/d_0 = 2,60. As the Eurocode 3 bearing resistance attains its maximum for e1/d0 = 3, the difference between both standards decreases again as the block tearing resistance continuous to increase according to EN 50341-1 for higher e_1/d_0 ratios.

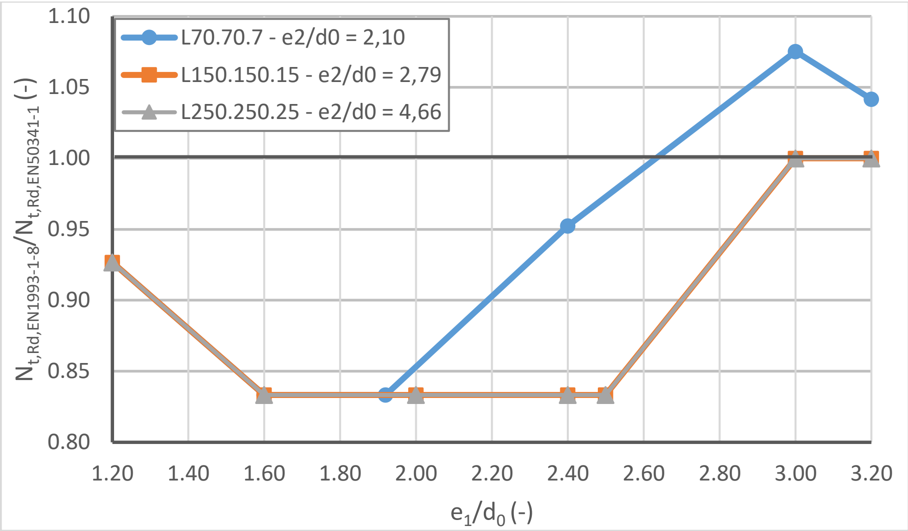


Figure 9: Tension resistance of a one-leg connection with one bolt – comparison between EN 1993-1-8 and EN 50341-1

Next, connections with two and three bolts are studied. First, Figure 10, Figure 11 and Figure 12 present the comparison between EN 50341-1 and EN 1993-1-8 for two bolt connections. For the three studied sections, it is supposed that the bolts are situated at the theoretical intersection between the connected leg and the v-v axis in order to ensure that a sole major-axis bending moment results from the eccentricity of the bolts.

The results presented in the following figures show again that the provisions given in the studied standards lead to differences of approximatively ±10% in the final resistance of the connections depending on their geometry.

Observing Figure 10, Figure 11 and Figure 12, one may remark that the results depend on the studied section. Nonetheless, the tendencies are similar. Indeed:

- For sections L70.70.7 and L150.150.15 and high ratios e_1/d_0 of 2,4, 2,8 and 3,2 block tearing becomes relevant in both standards over an important range of the bolt pitches. In this case the Eurocode 3 Part 1-8 provisions are more economic, especially for low values of the ratio p_1/d_0; with increasing ratio p_1/d_0, the results become closer. For very high bolt pitches the Eurocode 3 Part 1-8 resistance becomes limited by the maximum value of bolt bearing resistance (corresponding $\alpha = 3$ for inner bolts). In these case EN 50341-1 still predicts block tearing failure whose associated resistance increases with the bolt pitch and hence the curves abruptly descend;

- For sections <u>L70.70.7 and L150.150.15 and lower ratios e_1/d_0</u> (=1,2 … 2,0 for L70.70.7 and 1,2 … 2,4 for L150.150.15), one may observe three zones:
  1) for low values of the ratio p_1/d_0 the curves linked to the resistances increase and the Eurocode provisions become more favourable: In this range Eurocode 3 Part 1-8 predicts bearing failure whereas EN 50341-1 predicts block tearing failure. As the bearing resistance increases faster than the block tearing resistance, the difference between both standards increases;
  2) for intermediate values of the ratio p_1/d_0, both standards predict block tearing failure. As the EN 50341-1 resistance predictions increases slightly faster than the Eurocode predictions, the curves slightly decrease;
  3) for higher values of the ratio p_1/d_0, the Eurocode 3 Part 1-8 resistance is limited by the maximum bearing resistance becomes independent from the ratio p_1/d_0 (α reaches 3 for inner bolts), inversely EN 50341-1 continuous to predict block tearing failure and consequently, the predicted resistance continuous to increase explaining the descending branch of the curves in this range;
  4) finally, the EN 50341-1 predicted resistance becomes limited by bolt bearing failure for $e_1/d_0 = 1,2$ and consequently a plateau is reached because both the Eurocode resistance and the EN 50341-1 predicted resistance become independent from the ratio p_1/d_0.
- For <u>section L250.250.25</u>, one may observe that the curves are divided into two parts:
  1) for low values of the ratio e_1/d_0, bolt bearing failure is relevant for both standards and over the whole range of the bolt pitches p_1; for these case the Eurocode 3 strength predictions is for low values of the bolt pitch lower than the strength prediction of EN 50341-1 but it increases faster. However, the Eurocode 3 provisions reach the limit value of the bolt bearing resistance for a lower value of the bolt pitch (for a ratio p_1/d_0 of 3,50) than the EN 50341-1 provisions. Consequently, the resistance predicted by the CENELEC standard continuous to increase whereas the Eurocode strength prediction is constant starting from $p_1/d_0 = 3,50$. Hence the curves decrease for higher p_1/d_0 ratios;
  2) for bolt pitches corresponding to a ratio $p_1/d_0 = 3,63$, the bolt bearing resistance predicted by EN 50341-1 also reaches its maximum value and becomes independent from the ratio p_1/d_0. Therefore, the difference between both standards is constant for higher bolt pitches.

It is interesting to note that the curves are shifted downwards with increasing edge distance for ratios e_1/d_0 of 1,2 to 2,4. For ratios e_1/d_0 of 2,8 and 3,2 the curves appear to be shifted upwards. Indeed, starting from a ratio e1/d0 of 2,5, the EN 50341-1 predicted resistance becomes independent from the edge distance e1 as the a value for outer bolts attains its maximum value of 3. Inversely, the Eurocode 3 Part 1-8 predicted bearing distance still increases with the edge distance as the maximum a value for outer bolts is only attained for the e_1/d_0 ratio of 3,0. Finally, the predicted resistances are exactly identical for a e_1/d_0 ratio of 3,0 starting from a bolt pitch corresponding to p_1/d_0 of 3,63 as for this connection geometry the a values for both inner and outer bolts attains its maximum value of 3,0 according to both standards.

It should be noted, that for the studied cases of two bolt connections, the ultimate tension resistance does not become relevant neither for EN 1993-1-8 nor for EN 50341-1. Nonetheless, it is interesting to remark that the values of the ultimate tension resistance are very close for all studied sections:

- for L70.70.7: $N_{u,Rd}$ = 175,8 kN according to EN 1993-1-8 and $N_{u,Rd}$ = 175,4 kN according to EN 50341-1;
- for L150.150.15: $N_{u,Rd}$ = 834,8 kN according to EN 1993-1-8 and $N_{u,Rd}$ = 846,7 kN according to EN 50341-1;
- for L250.250.25: $N_{u,Rd}$ = 2413,8 kN according to EN 1993-1-8 and $N_{u,Rd}$ = 2491,2 kN according to EN 50341-1.

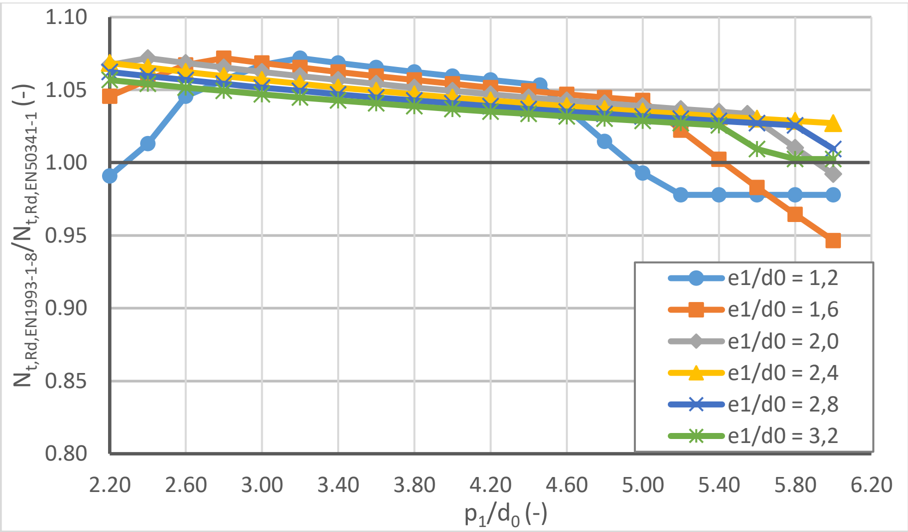


Figure 10: Tension resistance of a one-leg connection with <u>two</u> bolts for L70.70.7 angles – e_2/d_0 = 2,10

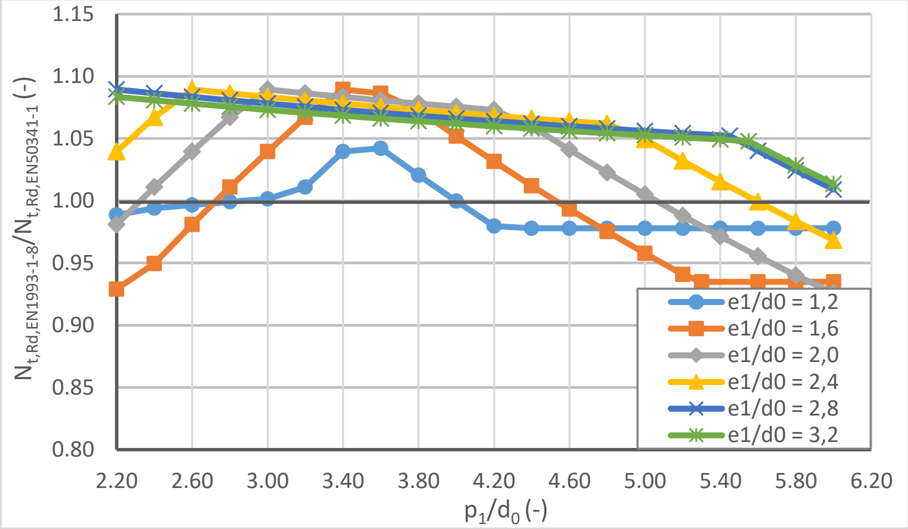


Figure 11: Tension resistance of a one-leg connection with <u>two</u> bolts for L150.150.15 angles – e_2/d_0 = 2,79

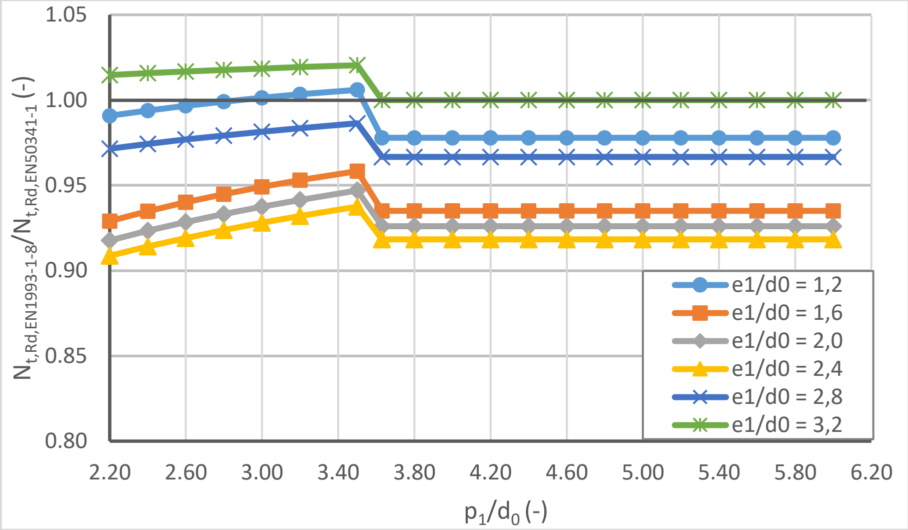


Figure 12: Tension resistance of a one-leg connection with <u>two</u> bolts for L250.250.25 angles – e_2/d_0 = 4,66

Next, Figure 13, Figure 14 and Figure 15 represent the results obtained for three bolt connections. Again, one may observe that the curves for sections L70.70.7 and L150.150.15 are very similar. For lower values of the bolt pitch block tearing is relevant according to both standards. Hence, the Eurocode 3 provisions are slightly favourable but the difference decrease with increasing bolt pitch. At a given bolt pitch, the ultimate tension resistance becomes relevant. However, it becomes relevant at a smaller bolt pitch if Eurocode 3 Part 1-8 is applied. Therefore, the curves decrease and finally attain a plateau if the ultimate tension resistance becomes relevant according to EN 50341-1, too.

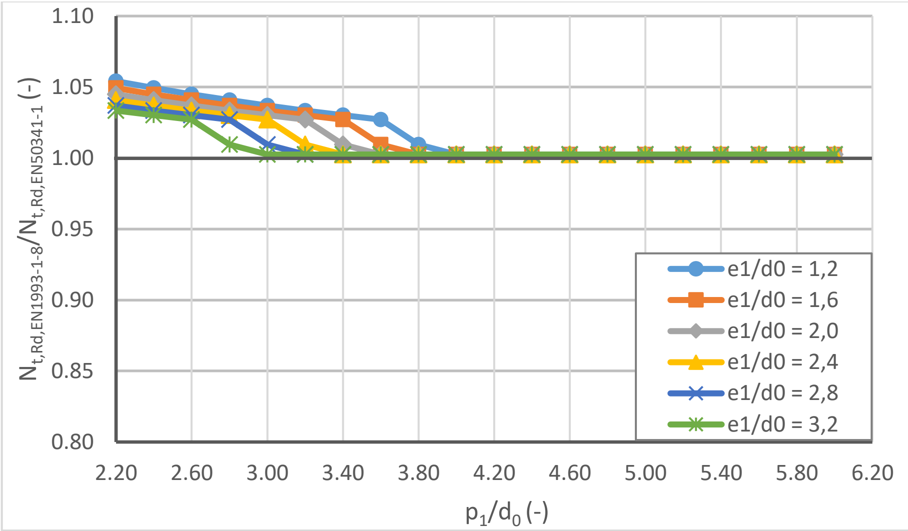


Figure 13: Tension resistance of a one-leg connection with three bolts for L70.70.7 angles – e_2/d_0 = 2,10

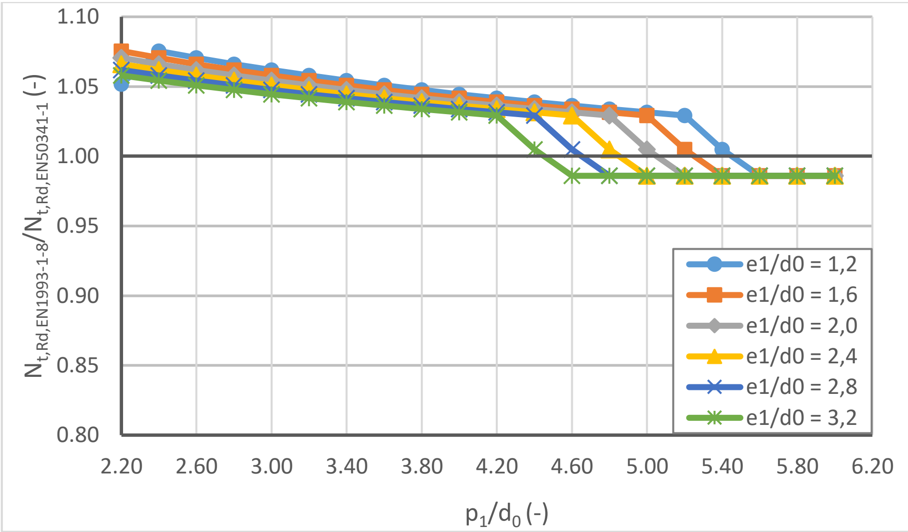


Figure 14: Tension resistance of a one-leg connection with three bolts for L150.150.15 angles – e_2/d_0 = 2,79

The comparison represented in Figure 15 for section L250.250.25 is slightly different compared to Figure 13 and Figure 14. As the area of the section is much higher than the one of the smaller angle sections, the ultimate tension resistance does not become relevant. Indeed, the resistance of the three bolt connection of the L250.250.25 angle section is similar to the resistance of the two bolt connection of the L150.150.15 section represented in Figure 11:

- for small bolt pitches the Eurocode 3 provisions predict bolt bearing failure whereas EN 50341-1 predicts block tearing failure;
- for intermediate values of the bolt pitch both standards predict block tearing failure;
- for higher values of the bolt pith, the Eurocode 3 predictions become limited by the bearing resistance with an α value of 3,0 for inner bolts whereas the strength predictions of EN 50341-1 continuous to increase as this standard still predicts block tearing failure;
- finally for high values of the bolt pitch the curves attain a plateau as bolt bearing failure becomes also relevant for EN 50341-1. The α value for inner bolts is

constant and equal to 3,0. For small e_1/d_0 ratios (1,2 … 2,4), the difference between both standards increase as the α value according to EN 50341-1 is more favourable than the one given in EN 1993-1-8. Starting from e1/d0 = 2,5, the a value of outer bolts attains its maximum value of 3,0 according to EN 50341-1 whereas it still increases according to EN 1993-1-8. Therefore, the difference between both standards decrease again. Finally, both standards predict the same resistance for e_1/d_0 = 3,2 and high values of the bolt pitch as both standards predict the limit value of 3,0 for outer and inner bolts.

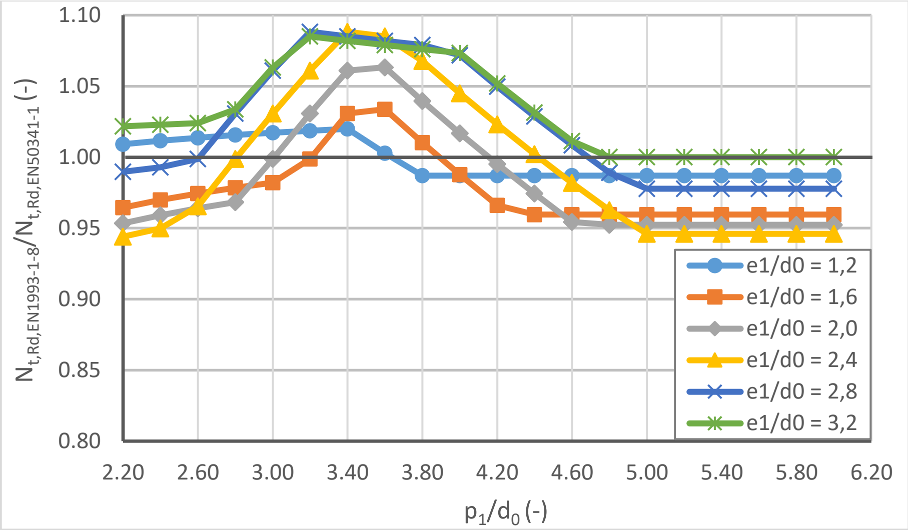


Figure 15: Tension resistance of a one-leg connection with three bolts for L250.250.25 angles – e_2/d_0 = 4,66

Up to this point, it has been supposed that the bolts are situated at the intersection between the minor axis v and the connected leg. Yet, in general, the bolt may be situated elsewhere on the leg and in particular the bolts may be closer to the edge. In these case, bolt bearing failure may be influenced by the condition related to the ratio e_2/d_0. In order to quantify the differences between both standard for such configurations, Figure 16 represents the results for a L150.150.15 section connected with two bolts situated at 40 mm from the edge (e_2 = 40 mm – e_2/d_0 = 1,38).

Figure 16 shows that the resistance predictions are rather close up to a ratio p_1/d_0 of approximatively 4,6 to 5,0. Up to this value of the bolt pitch both standards predict block tearing failure. It may be noted that for small edge distances e_1 (e_1/d_0 = 1,2 and e_1/d_0 = 1,6) and small ratios p_1/d_0 the shear resisting part of the block tearing resistances results from shear failure of the gross section $\left(f_y A_{gv}/\sqrt{3}\right)$. It may be noted that the resistance associated with the shear failure of the gross section increases faster than the shear failure of the net section $\left(f_u A_{nv}/\sqrt{3}\right)$. Therefore, the curve is increasing in this part of the diagram. When the shear failure of the net section becomes relevant, the curves slightly decrease owing to the differences in the reduction factors (see Table 5). Depending on the ratio e_1/d_0, the kink in the curves is located at different values of the bolt pitch. Indeed, the Eurocode 3 provisions become favourable when the EN 50341-1 predicts bolt bearing failure limited by the condition on the edge distance e_2:

- For e_1/d_0 = 1,2: the kink is situated at p_1/d_0 = 4,6 and the strength predictions of Eurocode 3 becomes up to 15% favourable as the EN 50341-1 α value for the inner bolt is limited to 2,070 (condition 2,3 (e_2/d_0 - 0,5) – see Table 7). For these configurations the α value for outer bolts are close ($\alpha_{EN\ 1993\text{-}1\text{-}8}$ = 1,20 and $\alpha_{EN\ 50341\text{-}1}$ = 1,295).

- For e_1/d_0 = 1,6 and e_1/d_0 = 2,0: the α values according to EN 50341-1 are higher and equal to 1,92 (limited by condition on e_1) and 2,07 (limited by condition on e_2) and consequently the bolt bearing failure becomes relevant only for very high values of the bolt pitch;
- For higher values of the ratio e_1/d_0, the α value for both outer and inner bolts is limited by the condition on e_2 resulting in α = 2,07 according to EN 50341-1. The tension resistance of the angle section becomes equal to 482,9 kN independently from the edge distance e_1 and from the bolt pitch p_1. Inversely, as there is no condition on the edge distance e_2 according to EN 1993-1-8:2020 for the bolt bearing resistance, the Eurocode 3 predicted resistance increases as it results from block tearing.

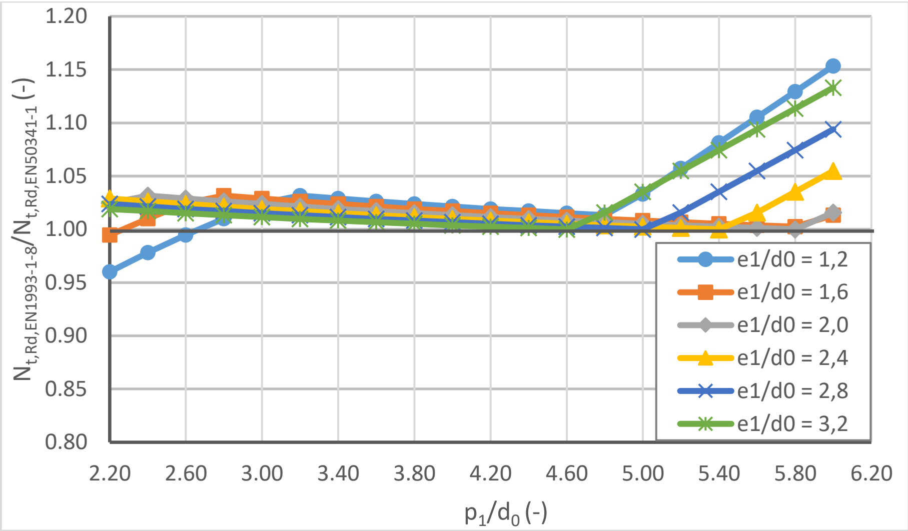


Figure 16: Tension resistance of a one-leg connection with <u>two</u> bolts for L150.150.15 angles – <u>e_2/d_0 = 1,38</u>

Throughout this paragraph, the tension resistance of simple angle sections connected on one leg only has been discussed and the differences between EN 1993-1-8 and EN 50341-1 have been quantified. It has been shown that the predicted resistances are rather close for both standards even if the resistance models diverge on a first sight. The differences are in the range of 10% for the studied examples. Major differences only result from the fact that EN 50341-1 introduces a limitation of the bolt bearing resistance linked to the edge distance e_2 (perpendicular to the load application). If the associated criterion for the determination of α becomes relevant, the strength predictions of Eurocode 3 may become very favourable. Nonetheless, the limitation of the bolt bearing resistance linked to e_2 is only relevant for extreme cases that are of less practical interest.

5.3 Angle sections in compression – Cross section resistance

Depending on the compactness of the legs, the cross-section may attain its full compression resistance (angle sections of class 3) or less due to the effect of local buckling (angle sections of class 4). According to Eurocode 3 Part 1-1, an angle section in compression is of class 3 if it respects the following two conditions simultaneously:

$$\frac{h}{t} \leq 15\varepsilon \quad \text{and} \quad \frac{b+h}{2t} \leq 11{,}5\varepsilon \qquad \textbf{Eq. 5.1}$$

It should be noted that h and b are overall dimensions considering the total width of the leg.

For equal leg angle sections, the second condition is always relevant whereas both conditions may become relevant for unequal angle sections depending on the ratio between the width of the two legs as shown in Figure 17. One may observe that the first criterion of Eq. 5.1 becomes only relevant for unequal leg angles whose bigger leg is at least twice as wide as its smaller leg. Figure 17 also shows the class 3-4 limit provided in EN 1993-3-1 and EN 50341-1. These two standards provide one single limit that is based on the reduction curve for outstand flanges in compression given in EN 1993-1-5. Clearly, the limits provided in EN 1993-3-1 and EN 50341-1 are favourable compared to EN 1993-1-1. One should note that the specific standards for towers do not base the classification on the overall dimensions of the legs, b and h, but they use **the outstand widths** (h-2t) and (b-2t). The orange line in Figure 17 shows the resulting class 3-4 limit in terms of ratio h/t. One may wonder why the outstand width used in EN 1993-3-1 and EN 50341-1 is not equal to h-t-r and b-t-r, respectively. Yet, for hot-rolled angle sections, the fillet radius is close to the thickness and therefore, the provided expressions may be a simplification.

Finally, the specific limit provided in the North American standard AISC360-10 [6] for angle sections in compression is shown by the violet line in Figure 17. As Eurocode 3 Part 1-1, AISC360-10 bases the classification on the overall width of the leg. It appears that the North American standard is more severe than the European provisions; in particular, it is more severe than EN 1993-3-1 and EN 50341-1. For slightly non-equal leg and for equal leg angle sections it becomes however more favourable than Part 1-1 of Eurocode 3. It should also be mentioned that, amongst the studied standards, only AISC360-10 provides b/t limits in bending:

Class 2-3 limit: b/t ≤ 16,14ε

Class 3-4 limit: b/t ≤ 27,20ε

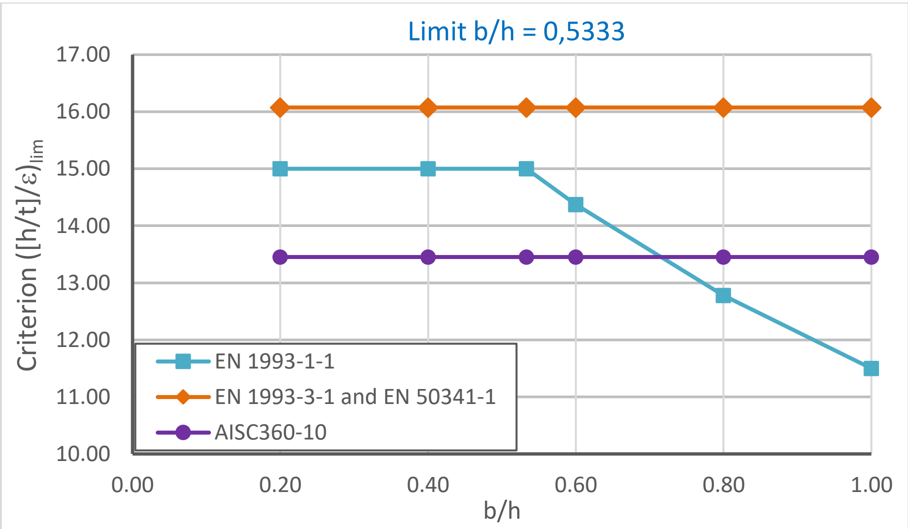


Figure 17: Limit between class 3 and class 4 for angle sections in compression

The important difference in between the three European standards may be surprising. Yet, one may not that the final compression resistance of the angle section will be the same if the ratio (h-2t)/t of the wider flange is less than 13,93ε for the studied section. Indeed, a designer using EN 1993-1-1 may conclude that the section is of class 4. Consequently, he has to use EN 1993-1-5 to determine the effective area A_{eff} of the section. Yet, according to EN 1993-1-5 the reduction factor ρ is equal to 1,0 if the outstand flange in uniform compression respects the limit of:

$(h-2t)/t \leq 13{,}93\varepsilon$

or in terms of ratio h/t:

$h/t \leq 16{,}07\varepsilon$

Therefore, the cross-section limits provided in EN 1993-1-1 for angle sections in uniform compression appear to be inconsistent with the only Eurocode rules that may be used to determine the effective cross-section properties of hot rolled angles, i.e. the rules provided in EN 1993-1-5. Additionally, it seems clear that the rules of EN 1993-1-5 were not developed for angle sections and the precision of the effective width method seems therefore doubtful. Consequently, it is interesting to compare the results of the Eurocode effective width method to the other reduction curves proposed specifically for angle sections in the past.

It should be recalled that the relative slenderness, used as basic variable for the strength reduction, can be determined by Eq. 5.2 according to EN 1993-1-5.

$$\bar{\lambda} = \frac{c/t}{28{,}4\varepsilon\sqrt{k_\sigma}} \qquad \textbf{Eq. 5.2}$$

For outstand flanges in uniform compression, the buckling coefficient k_σ becomes 0,43. Consequently, Eq. 5.2 can be simplified to obtain Eq. 5.3.

$$\bar{\lambda} = \frac{c/t}{18{,}6\varepsilon} \qquad \textbf{Eq. 5.3}$$

Figure 18 shows the Euler curve $1/\lambda^2$ as reference. Yet, it is recalled that the different reference widths “c” may be considered. As discussed above, EN 1993-3-1 and EN 50341-1 define c as b-2t. Other international references as ECCS publication n°39 [13] and the North American standard AISC360-10 [6] define c explicitly as the overall leg width b. Consequently, the resulting slenderness λ is different for the same angle section. This is highlighted by the two yellow curves in Figure 18. Indeed, the continuous yellow curve is based on a reference width “c” of b-2t whereas the discontinuous yellow line is based on c=b.

In addition to the local buckling reduction curve proposed in EN 1993-1-5 for outstand flanges, Figure 18 also provides the specific reduction curves for angle sections are defined in ECCS publication n°39 [13] as well as in the North American standard AISC360-10 [6]. One may observe that the specific rules lead to much lower resistances than the application of EN 1993-1-5. Nonetheless, this observation may be understood as the effective width method of

EN 1993-1-5 takes benefit of the stabilising effect resulting from adjacent plates. At least in case of equal leg angle sections in uniform compression, this stabilising effect is certainly negligible as both legs are equally slender. Figure 18 shows that the North American provisions become more favourable than those proposed in ECCS publication n°39 [13] starting from a b/t ratio of 20. The most recent version of AISC360 published in 2016 [6], proposes a new design rule for local buckling based on the effective width method. Yet, these new rules are not specific to angle sections and they are therefore not represented in Figure 18.

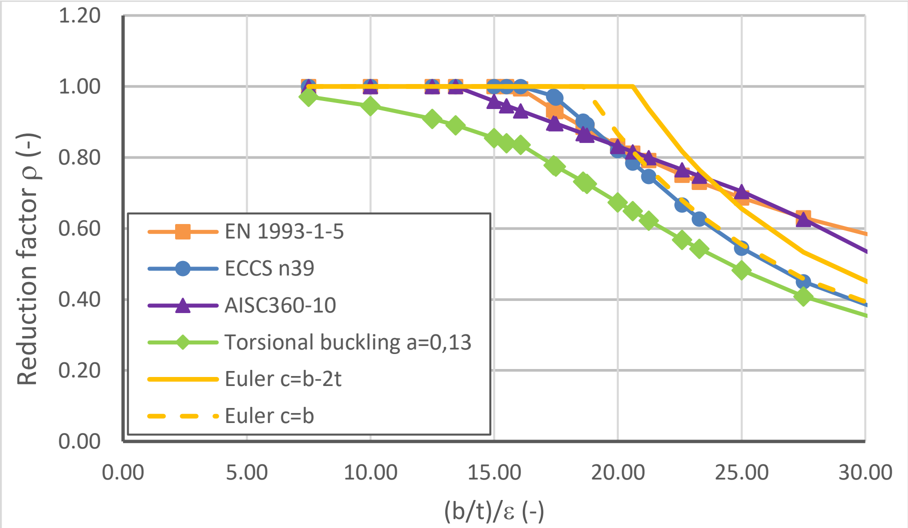


Figure 18: Comparison of reduction curves for equal leg angle sections in uniform compression

Last, it can be noted that EN 1993-3-1, EN 50341-1 and ECCS publication n°39 consider that torsional buckling is covered by the local buckling check. Therefore, it seems interesting to compare the reduction curve for torsional buckling to the reduction curves applied for local buckling of equal leg angle sections in uniform compression. Here, the buckling curve of EN 1993-1-1 [1] is applied with an imperfection factor α equal to 0,13. The relative slenderness for torsional buckling used as key parameter in the buckling curve is determined based on an equal leg angle section possessing a width of the legs of 70 mm. The thickness is determined based on the b/t ratio. Also, it is considered that the fillet radius at the root is equal to t. The torsional constant and the other relevant cross-section properties necessary to calculate the slenderness are determined numerically based on these simplified assumptions. Finally, the relative slenderness for torsional buckling is expressed according to Eq. 5.4.

$$\bar{\lambda}_T = \sqrt{\frac{A f_y}{N_{cr,T}}} \qquad \textbf{Eq. 5.4}$$

With

$$N_{cr,T} = \frac{1}{i_p^2}(G I_t) \qquad \textbf{Eq. 5.5}$$

The reduction curve for torsional buckling is recalled in Eq. 5.6 and Eq. 5.7.

$$\boldsymbol{\varphi = 0,5\left(1 + \alpha\left(\bar{\lambda}_T - 0,2\right) + \bar{\lambda}_T^2\right)} \qquad \textbf{Eq. 5.6}$$

$$\boldsymbol{\rho = \chi_T = \frac{1}{\varphi + \sqrt{\varphi^2 - \bar{\lambda}_T^2}}} \qquad \textbf{Eq. 5.7}$$

Observing Figure 18, one may conclude that the effective width method for local buckling applied according to EN 1993-3-1 and EN 50341-1 does not at all cover the effect of torsional buckling for angle sections addressed with the European buckling curve. Nonetheless, it has been shown in reference [14] that the format of the European buckling curve is not adequate to describe the resistance of double symmetric I sections failing in a torsional buckling mode. Therefore, the application of this buckling curve appears also doubtful for angle sections. A deeper investigation on local and torsional buckling of single angle sections is provided in Deliverable 2.2 – Design rules for single angle members.

The present paragraph can be summarised as follows:

- The cross section classification of angle sections in compression according to Part 1-1 of Eurocode 3 is inconsistent with the reduction curve accounting for the effect of local buckling given in EN 1993-1-5;
- Both, EN 1993-3-1 and EN 50341-1 base the classification of angle sections in compression on the reduction curve provided in EN 1993-1-5 leading to a consistent approach;
- AISC360-10 provides a specific but more severe class 3 limit for angle sections in compression than the tower standards EN 1993-3-1 and EN 50341-1;
- Amongst the studied standards, only AISC360-10 gives b/t limit ratios for angle sections in bending;
- The reduction curve provided in EN 1993-1-5 accounts for the favourable effect of the redistribution of stresses to the restrained plate edges. Yet, this effect may be questioned at least for equal leg angle sections in compression as both legs are equally slender;
- Specific reduction curves for angle sections in compression are provided in AISC360-10 and ECCS publication n°39. Both standards provide curves that are much more conservative than the effective width method of EN 1993-1-5.

5.4 Angle sections in compression – Member buckling resistance

5.4.1 General

Owing to the design of the joints of angle sections, their buckling behaviour is rather complex. Indeed, as single angle sections, used as web members of lattice towers, are generally connected on one leg only, bending moments arise at the member ends in addition to the axial force. The interaction between bending and axial force is difficult to capture by an analytical approach. Consequently, most standards introduce a modified buckling slenderness in order to account for the bending moments without including them directly in the design approach. The general format of the design method is given in Eq. 5.8 to Eq. 5.11.

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} \qquad \text{Eq. 5.8}$$

$$\bar{\lambda}_{eff} = k\bar{\lambda} \qquad \text{Eq. 5.9}$$

$$\varphi = 0,5\left(1 + \alpha\left(\bar{\lambda}_{eff} - 0,2\right) + \bar{\lambda}_{eff}^2\right) \qquad \text{Eq. 5.10}$$

$$\chi = \frac{1}{\varphi + \sqrt{\varphi^2 - \bar{\lambda}_{eff}^2}} \qquad \text{Eq. 5.11}$$

Depending on the specific design of the connections, the resulting bending moments may be more or less important. Therefore, EN 1993-3-1 and EN 50341-1 that have been developed explicitly to address the strength of towers, contain a multitude of cases and corresponding expressions for the factor k. It may be noted that EN 50341-1 refers to the method of EN 1993-1-1 and EN 1993-3-1 for the design of angle sections in compression. As an alternative EN 50341-1 provides specific buckling rules in its Annex J that may only be applied if full-scale tower tests have been performed (see paragraph 6 and reference [8]). Obviously, if overhead transmission towers are designed according to the general procedure, i.e. by applying the design methods of Eurocode 3, the resulting tower strength is identical for both standards. Inversely, the application of the alternative procedure provided in Annex J of EN 50341-1 may lead to significant differences in the design strength of towers as is shown throughout the next paragraphs.

It may be noted that EN 1993-1-1 also provides specific rules for the buckling of angle sections. Yet, this standard only addresses three cases whereas both specific tower standards provide a multitude of cases (see Table 8, Table 13 and Table 14). One might therefore suppose that the rules given in EN 1993-1-1 may become too safe-sided in some cases. It is therefore important to compare the strength predictions obtained for buckling of single angle sections members and to quantify the resulting differences. Hereafter, paragraph 5.4.2 addresses the buckling of diagonals and then paragraph 5.4.3 studies the strength predictions for leg members.

5.4.2 Buckling of diagonals

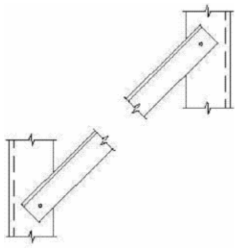
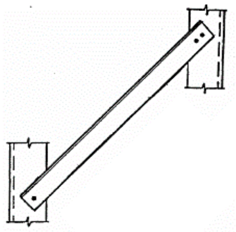
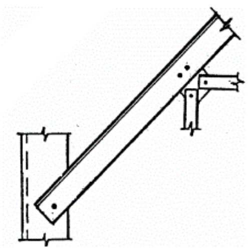
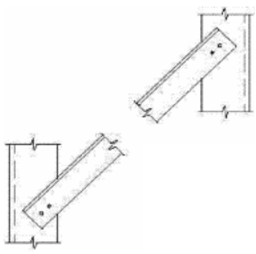
The following diagrams compare the strength predictions for the different joint configurations presented in Table 8. The corresponding expressions for the factor k are given in Table 13 of Annex A.

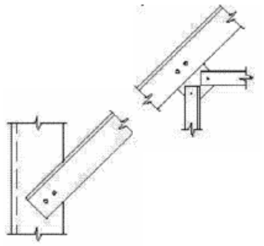
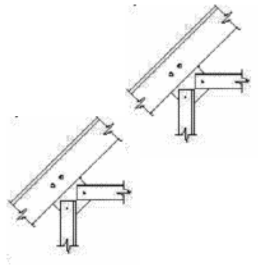
According to EN 1993-1-1, it is not possible to use the modified slenderness approach for case 1 as well as 2a and 2b. If the member is connected with only one bolt at one end at least, it is necessary to account explicitly for the resulting bending moment in the design. Yet, EN 1993-1-1 does not state how this should be done. Obviously, the designer may perform a second order analysis (elastic or plastic) accounting for the influence of imperfections. An alternative method would be the application of interaction factors. However, equations (6.61) and (6.62) of the current version of Eurocode 3 Part 1-1 were not developed for such kind of section. Consequently, their precision is questionable. Still, for the comparisons represented in this paragraph, equations (6.61) and (6.62) of EN 1993-1-1 are applied together with Annex A of the same standard (for the determination of the interaction factors) for cases for which only one bolt is used at one member end. Both EN 1993-3-1 and EN 50341-1 provide modified slenderness expressions even for cases 1, 2a and 2b. In order to account for the fact that only one bolt is used, EN 1993-3-1 introduces a supplementary reduction factor η equal to 0,8 for case 1 and equal to 0,9 for case 2a and 2b. The final reduction factor used for the strength verification is the given by Eq. 5.12.

$$\chi_{EN1993-3-1} = \eta\chi \qquad \textbf{Eq. 5.12}$$

EN 50341-1 does not introduce this supplementary factor η. One might suppose that EN 50341-1 completely introduces the influence of the connection configuration into the modified slenderness.

Table 8: Specific joint configurations for the buckling of diagonals

Case	Configuration	Description
1		Member without intermediate restraint connected with one bolt at its ends.
2a		Member without intermediate restraint connected with one bolt at one end and at least two bolts at the other end.
2b		Member with intermediate restraint connected with one bolt at its end and at least two bolts at the restraint.
3a		Member without intermediate restraint connected with at least two bolts at its ends.

3b		Member with intermediate restraint connected at least two bolts at its end and at the restraint.
3c		Intermediate segment of a continuous member connected with at least two bolts at the restraint.

In the following, the strength predictions resulting from the different standards are compared for single angle members made of L70.70.7 section. For other sections, the results are similar.

First, Figure 19 compares the strength predictions for case 1. In case of EN 50341-1 and EN 1993-3-1, the reduction factor may be determined directly with the approach represented in Eq. 5.8 to Eq. 5.11. Inversely, as only one bolt is used, it is necessary to account explicitly for the bending moment according to EN 1993-1-1. This is done hereafter by applying the interaction equations (6.61) and (6.62) of EN 1993-1-1. The reduction coefficient represented in Figure 19 is derived as the ratio between the axial force leading to an utilisation ratio of 1,0 and the cross section resistance to the axial force Af_y (class 4 sections are not considered for the comparisons). Last, one may note that the Eurocode 3 Part 1-1 interaction equations are applied based on the elastic section modulus $W_{u,el}$ (curve noted as EN 1993-1-1 – EL) and based on the plastic section modulus $W_{u,pl}$ (curve noted as EN 1993-1-1 – PL).

Figure 19 shows that the approach of EN 1993-1-1 is highly conservative compared to the modified slenderness approaches of EN 1993-3-1 and EN 50341-1. One may suppose that the effect of the structure on the buckling of the element is more precisely accounted for in the specific standards for towers, EN 1993-3-1 and EN 50341-1, rather than in Eurocode 3 Part 1-1 providing general rules for steel structures. Yet, in this case, it is astonishing that there is a considerable difference (up to 35%) in the strength predictions between EN 1993-3-1 and EN 50341-1. The observed difference mainly results from two points:

- EN 1993-3-1 considers buckling curve “c” whereas EN 50341-1 considers buckling curve “a_0”;
- EN 1993-3-1 imposes the use of the supplementary reduction factor η (=0,8 for case 1 – see Table 13 and Eq. 5.12).

The origin of the EN 50341-1 provisions can be found in reference [13]. In this reference, the modified slenderness expressions are justified with tests performed on complete lattice towers and sub-structures of lattice towers. The origins of the EN 1993-3-1 provisions are not clear at this stage.

The conservatism of the EN 1993-1-1 approach is independent from the section modulus used. Indeed, the difference between the curves linked to the plastic and the elastic section modulus attains only approximatively 15%. One reason for this low difference is the supposed form of the plastic cross-section interaction in the interaction equations. The real form of bending moment-axial force interaction is clearly more non-linear for angle sections (see for example reference [15]) than for I sections (for which the interaction equations have been developed). Accounting rigorously for the plastic interaction would certainly increase the difference between both Eurocode 3 Part 1-1 curves. Nonetheless, even if the exact plastic interaction was applied, the difference between the Eurocode 3 Part 1-1 approach and the strength predictions resulting from EN 1993-3-1 and EN 50341-1 would be of the same magnitude. This has been shown in reference [16] in which the resistance has been calculated numerically through elastic and plastic second order simulations.

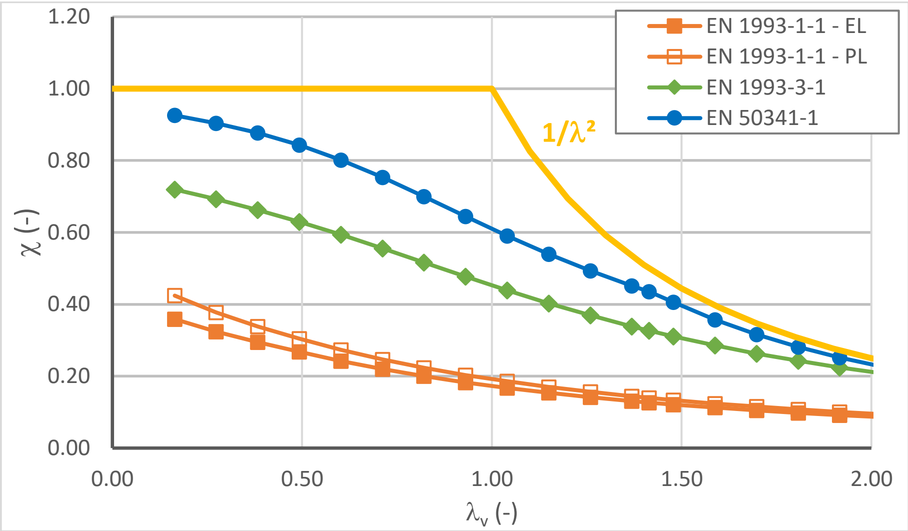


Figure 19: Strength prediction for minor-axis buckling of L70.70.7 member – case 1 of Table 8

Figure 20 shows strength predictions for buckling about the z-z/y-y axis for case 1. Only the provisions of EN 1993-3-1 and EN 50341-1 are considered. Again the difference between both standards attains up to 35% for intermediate values of the relative slenderness due to the difference in the applied buckling curve (EN 1993-3-1: buckling curve "c"; EN 50341-1: buckling curve "a_0") and the factor η to be applied on the reduction factor χ according to EN 1993-3-1 (see Eq. 5.12). Nonetheless, one may note that buckling about the z-z/y-y axis is certainly of less interest for case 1 as it only becomes relevant in case of intermediate restraints about the minor-axis. In presence of intermediate restraints, case 2b should however be applied.

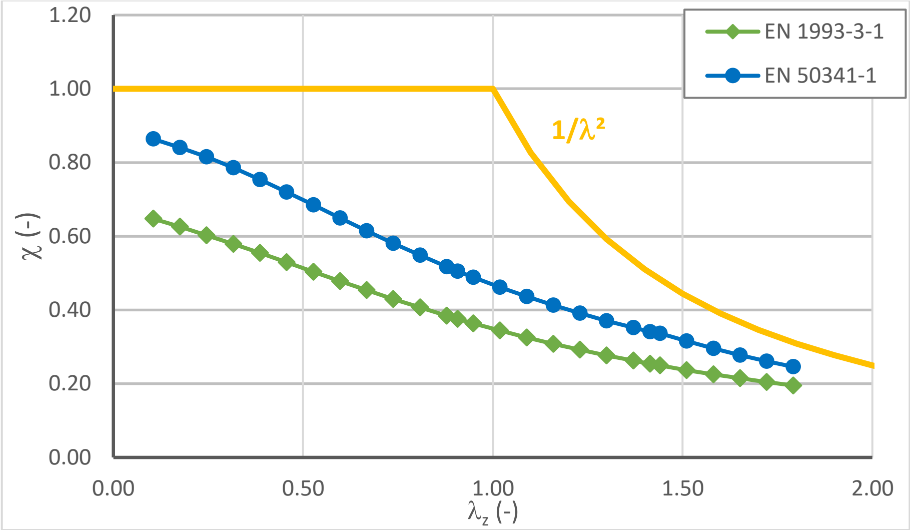


Figure 20: Strength prediction for z-z/y-y axis buckling of L70.70.7 member – case 1 of Table 8

Next, cases 2a and 2b are considered. In both cases, the member is connected with one bolt at one end and with two bolts at the other end. The second end may be the physical extremity of the angle diagonal (case 2a) or an intermediate restraint (2b). Again, one may observe the high conservatism of the EN 1993-1-1 approach in Figure 21. Inversely, it appears that the difference between EN 1993-3-1 and EN 50341-1 decreases owing to the factor η that becomes 0,9 according to EN 1993-3-1 compared to η = 0,8 for angle members connected with one single bolt at both ends.

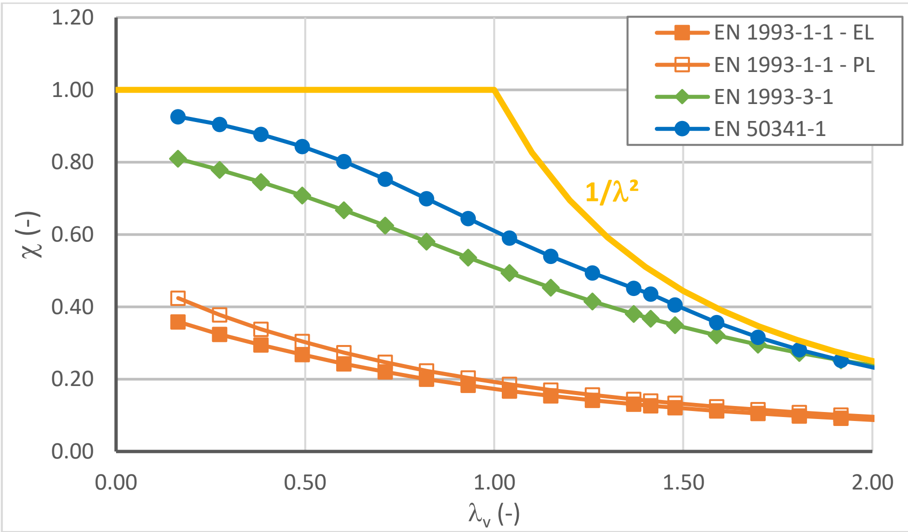


Figure 21: Strength prediction for minor-axis buckling of L70.70.7 member – case 2a and 2b of Table 8

Figure 22 represents the strength predictions for z-z/y-y axis buckling resulting from EN 1993-3-1 and EN 50341-1. Especially, case 2b is of practical interest as the member may buckle about the z-z/y-y axis if it possesses an intermediate restraint reducing the buckling length about the minor-axis. Up to a slenderness of 1,41 (=$\sqrt{2}$), EN 1993-3-1 and EN 50341-1 give very similar results. Yet, at 1,41 for the relative slenderness, there exists a discontinuity in the strength curve provided by EN 50341-1. It should be noted that the strength curves defined in EN 50341-1 are divided into two sections limited by a relative slenderness of $\sqrt{2}$ as shown for the example represented in Figure 22 by Eq. 5.13 to Eq. 5.15.

$$\bar{\lambda}_{z,eff} = k\bar{\lambda}_z \qquad \text{Eq. 5.13}$$

with:

$$k = 0,65 + \frac{0,71}{\bar{\lambda}_z} \qquad \text{if } \bar{\lambda}_z \leq \sqrt{2} \qquad \text{Eq. 5.14}$$

$$k = 1 \qquad \text{if } \bar{\lambda}_z > \sqrt{2} \qquad \text{Eq. 5.15}$$

Obviously, the effective slenderness $\bar{\lambda}_{z,eff}$ should be identical for a value of $\bar{\lambda}_z = \sqrt{2}$ in order to obtain a continuous reduction curve. However by applying Eq. 5.14, one obtains an effective slenderness of 1,63 and with Eq. 5.15, one obtains an effective slenderness of 1,41 (=$\sqrt{2}$). Consequently, one obtains different reduction factors at the limit between both expressions.

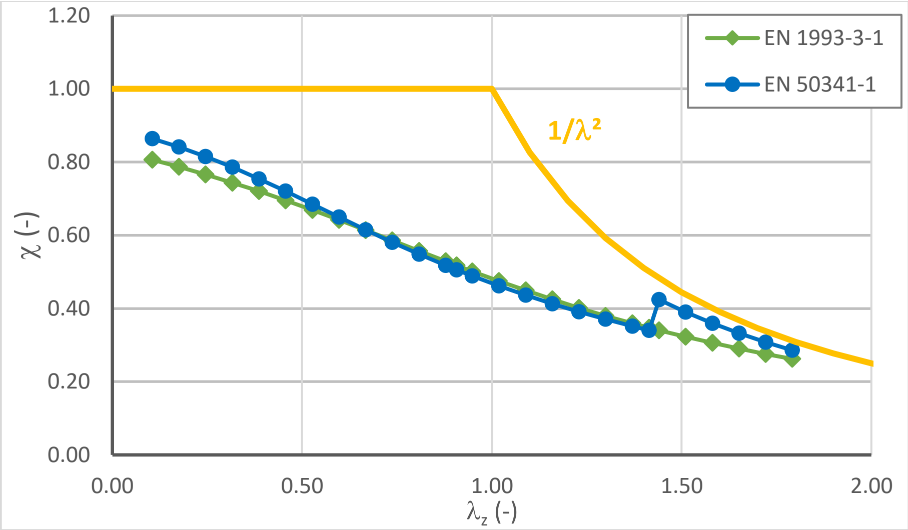


Figure 22: Strength prediction for z-z/y-y axis buckling of L70.70.7 member – case 2a and 2b of Table 8

Clearly, the discontinuity shown in Figure 22 is a drawback when EN 50341-1 is applied. One should note that the expressions provided for the effective slenderness and the factor k have originally been proposed in reference [13]. In this reference, it is stated that generally the bolt resistance becomes relevant for connections containing one single bolt. One might therefore wonder whether the first part of the EN 50341-1 reduction curve for case 2b is relevant for the design or not. It is understandable that starting from a certain relative slenderness (and hence a certain value of the reduction factor), the strength of the angle member is sufficiently reduced so that the bolt resistance is not design relevant anymore but buckling of the member. Observing the strength curve of EN 50341-1, it may be supposed that this limit slenderness is equal to $\sqrt{2}$. Nonetheless, at this stage of the study, a single value of this limit appears questionable as it is certainly influenced by the size of the angle section and the bolts used for the connection.

Finally, Figure 23, Figure 24 and Figure 25 represent the results obtained for angle diagonals attached with at least two bolts at both member ends. For this case, the strength predictions obtained by the different standards are much closer than for case 1, 2a and 2b. Indeed, for minor-axis buckling EN 1993-1-1 and EN 1993-3-1 give identical expressions for the effective slenderness as shown in Figure 23. The provisions of EN 50341-1 lead to slightly higher resistances owing to the favourable buckling "a_0" used in this standard.

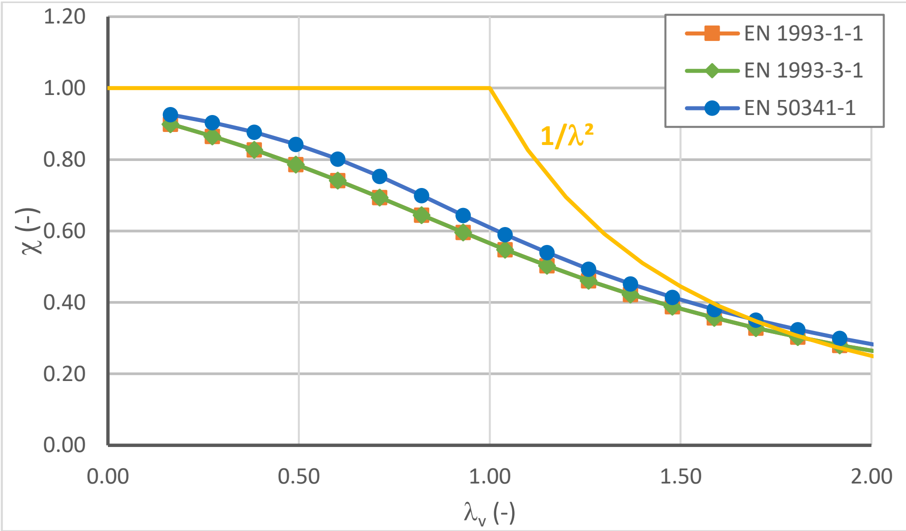


Figure 23: Strength prediction for minor-axis buckling of L70.70.7 member – case 3a, 3b and 3c of Table 8

Figure 24 represents the results for z-z/y-y buckling of diagonals without intermediate restraint. For this case, minor-axis buckling is generally relevant. Nonetheless, it is interesting to observe that for case 3a the strength provisions of EN 1993-1-1 and EN 50341-1 are nearly identical whereas EN 1993-3-1 is slightly favourable. The differences are however rather low and attain only of about 12%.

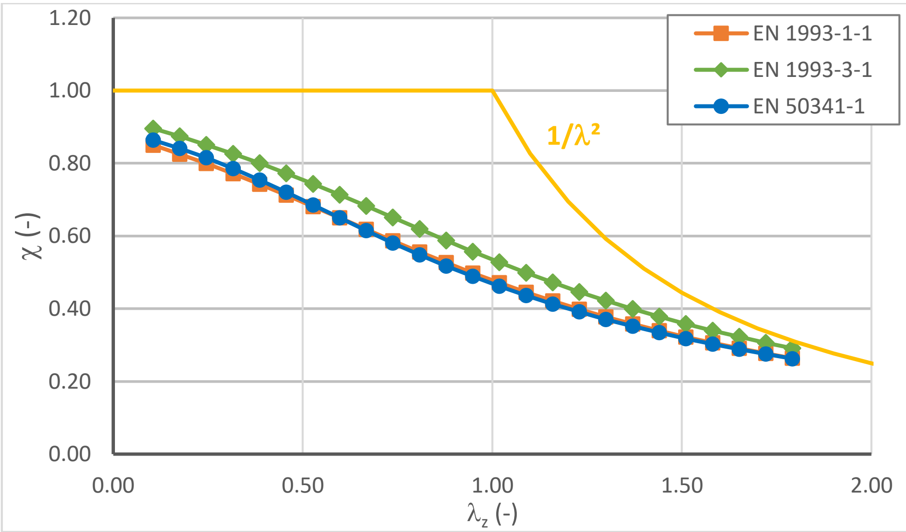


Figure 24: Strength prediction for z-z/y-y axis buckling of L70.70.7 member – case 3a of Table 8

Last, the strength predictions for z-z/y-y buckling for cases 3b and 3c are presented in Figure 25. Again, one should underline that z-z/y-y buckling is of practical interest for members possessing intermediate lateral restraints as those covered by cases 3b and 3c. Therefore, it is important to compare the resulting strength predictions. As for case 3a, the results are rather close. Also, up to the limit slenderness of $\bar{\lambda}_z = \sqrt{2}$, EN 1993-1-1 and EN 50341-1 can be considered as equivalent. Nonetheless, as for cases 2a and 2b, EN 50341-1 appears to predict a discontinuity. For the cases presented in Figure 25, one can clearly not attribute this discontinuity to the fact that the bolt resistance is relevant before the limit slenderness is attained (as a non-defined number of bolts can be used). The reason for this discontinuity is therefore unclear.

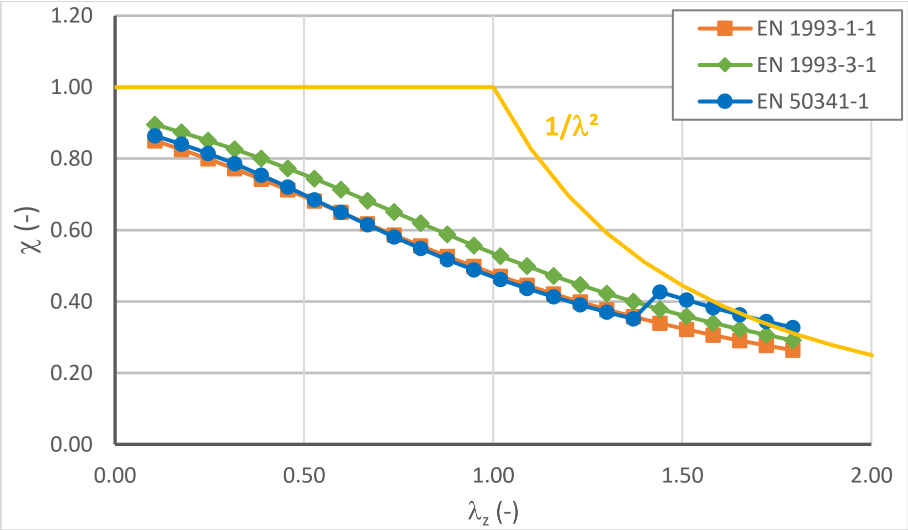


Figure 25: Strength prediction for z-z/y-y axis buckling of L70.70.7 member – cases 3b and 3c of Table 8

Throughout this paragraph, the buckling resistance of single angle diagonals predicted by different standards has been studied. All cases concerned the section L70.70.7. Nonetheless, one may note that, for other sections, the results are equivalent. The previous comparisons lead to the following conclusions:

- The EN 1993-1-1 strength predictions are highly conservative for single angle diagonals connected with only one bolt at one member end at least (cases 1, 2a and 2b of Table 8). Indeed, the rules of EN 1993-3-1 and EN 50341-1 may lead to resistances that are partially **three times (!)** as high as the prediction of EN 1993-1-1. Reasons for such high differences might be attributed to system effects arising in lattice towers that are naturally not covered in EN 1993-1-1 providing general rules for steel structures.
- EN 1993-3-1 and EN 50341-1 lead to a difference from 15% to 30% for cases of diagonals connected with one single bolt. The reason for this difference is mainly the supplementary reduction factor η (between 0,8 and 0,9) that has to be applied to the buckling reduction factor χ according EN 1993-3-1 in order to account for the interaction between the bending moment and the axial force acting along the member.

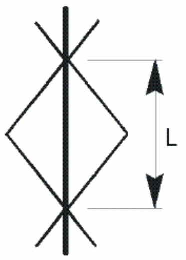
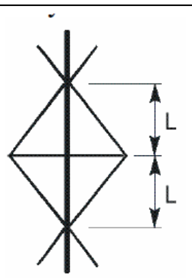
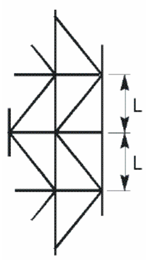
- For members connected with at least two bolts at their ends, the rules provided in the three different standards are much closer. The maximum difference attains only about 15%.
- The strength curves provided in EN 50341-1 for z-z/y-y axis buckling for case 2a, 2b, 3b and 3c contain a discontinuity whose physical origin is not clear. This is a clear drawback, especially for cases 2b, 3b and 3c that are of practical interest.

In reference [13], that constitutes the basis of the buckling rules provided in EN 50341-1 for the elements of lattice towers, the rules are justified with reference to tests on complete lattice towers and sub-structures of lattice towers highlighting the influence of the system effects. Consequently, it appears that the hypotheses concerning modelling and analysis of lattice towers have to be consistent with the design methods used for the verification of the individual members. This problem will be analysed in detail in WP 2 of the ANGELHY project based on full-scale tests and numerical simulations of steel lattice towers (deliverables 2.5 and 2.6).

5.4.3 Buckling of leg members

As for buckling of diagonals, EN 1993-3-1 and EN 50341-1 define slenderness modification factors (denoted k in Eq. 5.8 to Eq. 5.11) for particular cases of leg members shown in Table 9. Inversely, Part 1-1 of Eurocode 3 does not address specifically the case of leg members.

Table 9: Slenderness modification factor for buckling of leg members

<table>
<tr><th>Case</th><th colspan="2">Configuration</th><th>Description</th></tr>
<tr><td>1</td><td colspan="2">
</td><td>Leg is connected to primary bracing at both ends (symmetrical case).
Minor-axis buckling only has to be considered.</td></tr>
<tr><td rowspan="2">2</td><td>symmetric</td><td>asymmetric</td><td rowspan="2">Primary bracing at one end and secondary bracing at the second end of the leg.
Minor-axis buckling only has to be considered.</td></tr>
<tr><td>
</td><td>
</td></tr>
</table>

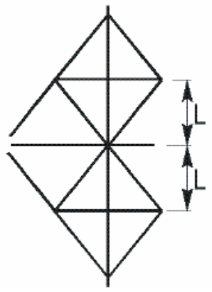
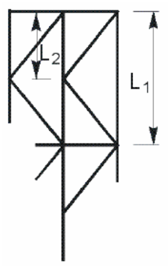
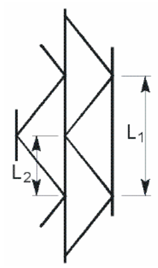
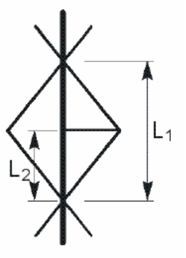
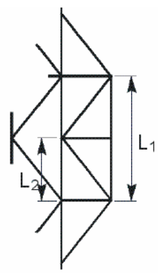
3			Secondary bracing at both ends of the leg. Minor-axis buckling only has to be considered.
4	With horizontal members	Without horizontal members	Leg connected to primary bracing at both ends but in a single plane (unsymmetrical case). Buckling about z-z/y-y axis has to be considered over the length L_1. Only for unequal leg angles, minor-axis buckling over the length L_2 should be considered.
			
5	With horizontal members	Without horizontal members	Leg connected to primary bracing at both ends but in a single plane (unsymmetrical case) and at an intermediate position it is connected to secondary bracing. Buckling about z-z/y-y axis has to be considered over the length L_1. Only for unequal leg angles, minor-axis buckling over the length L_2 should be considered.
			

Figure 26 to Figure 30 represent the strength predictions obtained for the leg members of steel lattice towers. As before, the comparisons are based on a single angle section L70.70.7. Nonetheless, the results are representative for all equal leg angle sections.

The following figures show that the tendencies are similar for all cases of Table 9. In particular, one observes that:

- In most cases, EN 50341-1 is favourable. In particular, for low and intermediate values of the relative slenderness, EN 50341-1 predicts higher resistances owing to the use of the favourable buckling curve "a_0" compared to buckling curve "c" that is used in EN 1993-1-1 and EN 1993-3-1.
- Generally, EN 1993-3-1 is more favourable than EN 1993-1-1 owing the k factor that reduces the relative slenderness.
- Case 4 of Table 9 is the only exception of the two previous tendencies. Indeed, the k factor for case 4 leads to an increased relative slenderness. It may be noted that the buckling restraint is only situated in one single plane at the member ends

in this case. Consequently, the restraint against minor-axis buckling may not be as efficient at the member ends as in case of restraints in the both perpendicular axis y-y and z-z. The k factor might therefore consider the effect of an interactive buckling mode about the z-z/y-y axis and about the minor-axis.

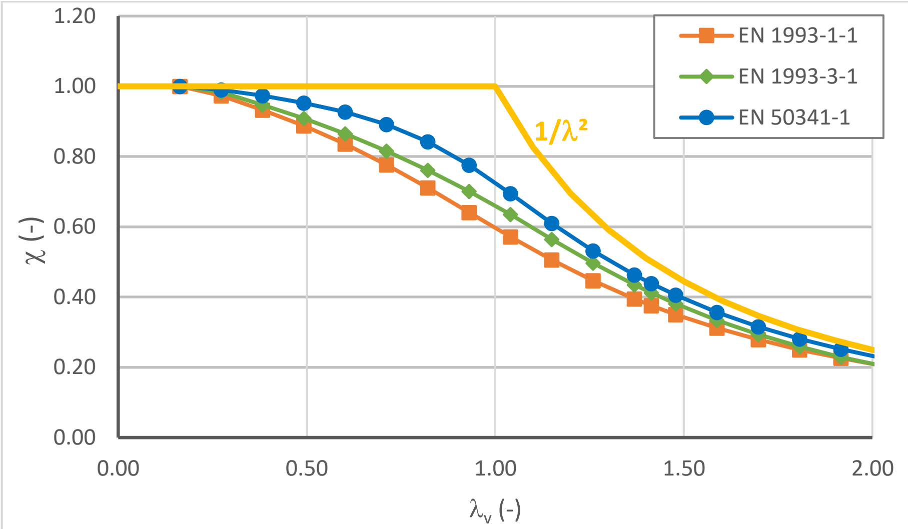


Figure 26: Strength prediction for minor-axis buckling of L70.70.7 member – case 1 of Table 9

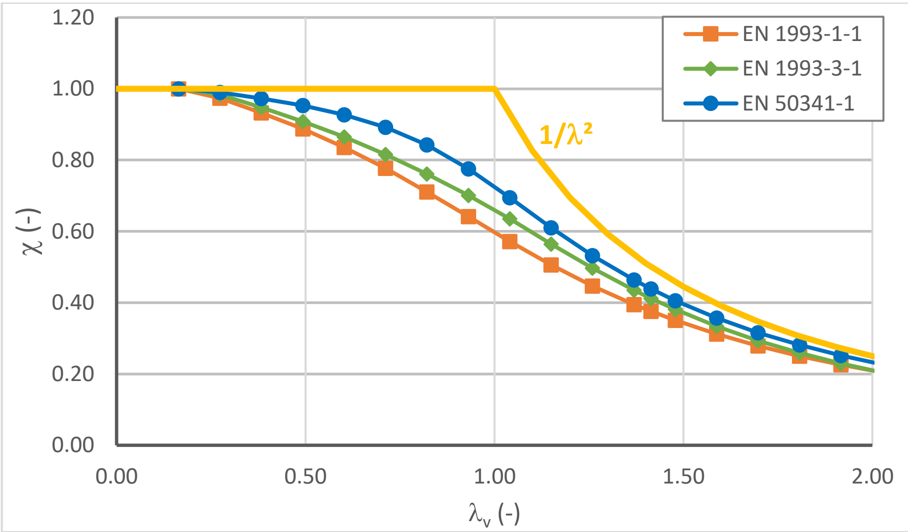


Figure 27: Strength prediction for minor-axis buckling of L70.70.7 member – case 2 of Table 9

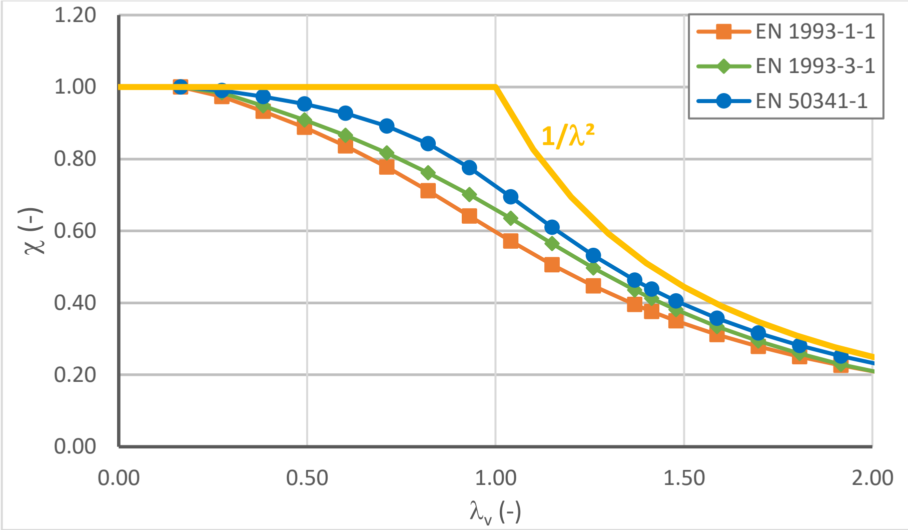


Figure 28: Strength prediction for minor-axis buckling of L70.70.7 member – case 3 of Table 9

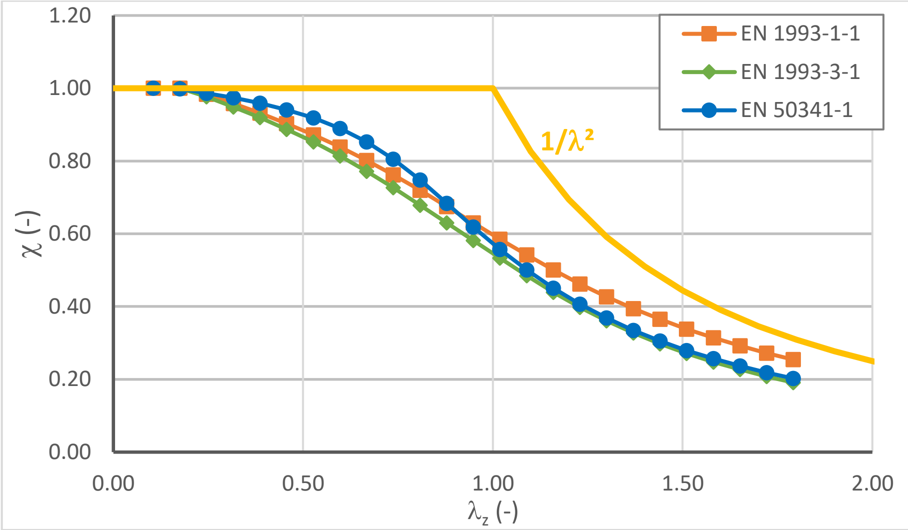


Figure 29: Strength prediction for y-y/z-z axis buckling of L70.70.7 member – case 4 of Table 9

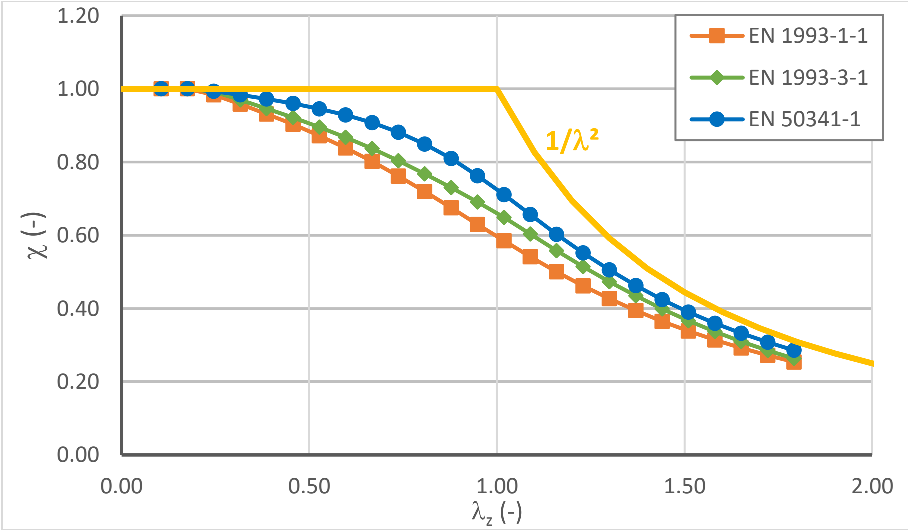


Figure 30: Strength prediction for y-y/z-z axis buckling of L70.70.7 member – case 5 of Table 9

As for the buckling resistance of single angle diagonals discussed in paragraph 5.4.2, it is shown here that the provisions for the buckling resistance of leg members provided in the studied standards can lead to considerable differences in terms of resistance. System effects may again have a significant influence on the leg resistance highlighting the important link between modelling and analysis of the lattice tower and design of its constituting parts.

5.5 Closely-spaced built-up sections in compression

The introduction of Eurocode 3 Part 1-1 changed considerably the habit concerning the design of closely spaced built-up members in many countries of Europe. Indeed, in the past it had been accepted to treat back-to-back connected angles as a single member for the buckling about the z-z axis (see Figure 31) if the distance between the packing plates "a" was lower than $50i_{min}$ (i_{min} = the minimum radius of gyration of an individual angle section). The current version of Eurocode 3 Part 1-1 reduces the limit to $15i_{min}$. For star battened built-up members the Eurocode limit for the distance "a" is equal to $70i_{min}$. Provided that the distance "a" is lower than the limit distance, the out-of-plan buckling resistance of the built-up member is obtained with the European buckling curve and an imperfection factor <u>α equal to 0,34</u> (buckling curve b).

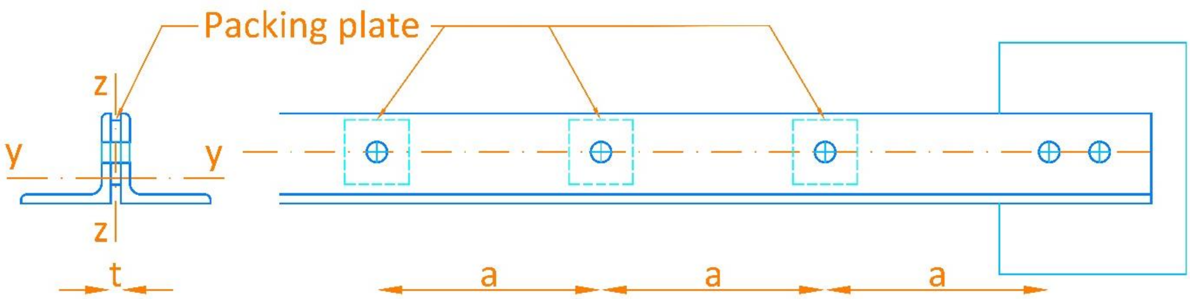


Figure 31: Back-to-back connected angle sections

If the limit distances between the packing plates are not respected, it is necessary to consider the influence of the shear stiffness in the design of the built-up members. Yet, Part 1-1 of Eurocode 3 does not specify how this should be done. For the following comparisons, the influence of the shear stiffness is accounted for by applying the complete design method proposed for battened columns (not closely spaced) in §6.4.3 of EN 1993-1-1.

The results of this procedure are presented in Figure 32 for two back-to-back connected L70.70.7 sections. The thickness of the packing plate is equal to 8 mm for this example.

Figure 32 also represents the resistance obtained for the studied members according to the provisions of the CENELEC standard EN 50341-1. This standard introduces an explicit method accounting for the influence of the packing plate connection (the same method is provided in ECCS publication n°39). Indeed, it is proposed to determine an equivalent geometric slenderness according to Eq. 5.16. The reduction factor is calculated with the Eurocode buckling curve based on this geometric slenderness and an imperfection factor <u>α equal to 0,13</u> (buckling curve a_0).

$$\boldsymbol{\lambda_{zi}} = \sqrt{\boldsymbol{\lambda_z^2} + \boldsymbol{\lambda_1^2}\frac{\boldsymbol{m}}{\boldsymbol{2}}} \qquad \textbf{Eq. 5.16}$$

With:

λ_z: geometric slenderness of the built-up member considered as uniform - $\lambda_z = L/i_z$

λ_1: geometric slenderness of an individual angle section between packing plates - $\lambda_1 = a/i_v$

m: number of angle sections

Finally, it is interesting to note that the French National Annex to EN 1993-3-1 limits the distance "a" again to $50i_{min}$ for the built-up members (of back-to-back connected angle sections) treated as uniform.

Figure 32 compares the results of the different design methods for different distances "a" between the packing plates. EN 1993-3-1 is applied with the French National Annex. Consequently, only one single curve is represented and this curve overlaps the reduction curve resulting from the provisions of EN 1993-1-1 with "a" equal to $15i_v$.

It can be observed that the provisions of EN 50341-1 lead in general to higher resistances than the Eurocode rules. One of reasons is the use of the favourable buckling curve a_0 in EN 50341-1 (compared to buckling curve b in EN 1993-1-1). Consequently, even for distances "a" reaching $50i_v$ EN 50341-1 predicts higher resistances than the Eurocode 3 Part 1-1 for built-up members respecting the limit of $15i_v$ (and consequently neglecting the influence of the packing plate connection). Only, if the distance "a" attains approximatively $80i_v$, EN 50341-1 becomes less favourable than the uniform built-up member treated with Eurocode. Yet, EN 50341-1 limits the distance "a" to $50i_v$ and therefore this last case is out of the scope of the CENELEC standard. Here it is only represented for the comparisons.

Finally, Figure 32 shows the resistance predictions of EN 1993-1-1 if the complete method provided for battened columns is applied. Besides its complexity for hand calculations, this method clearly leads to the most conservative results.

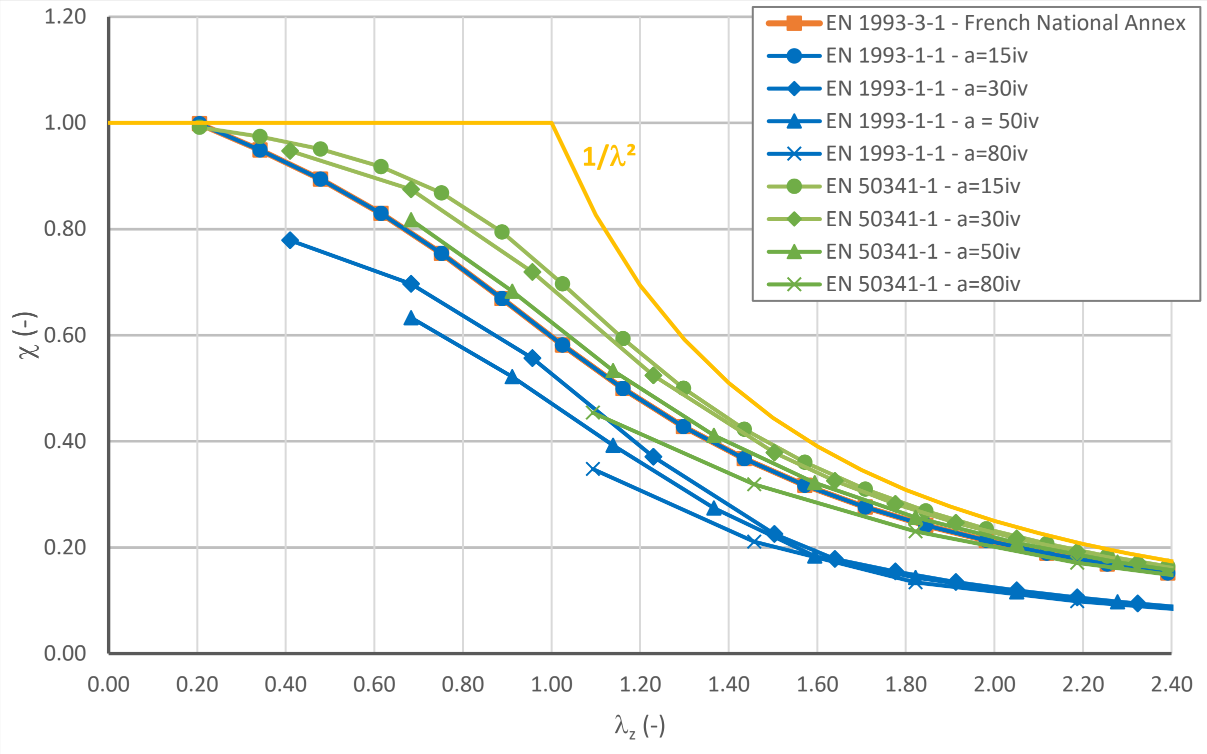


Figure 32: Resistance curve for two L70.70.7 connected back-to-back

In this paragraph, only a single example of back-to-back connected angle sections is treated. Nonetheless, for other sections, the results are comparable. Additionally, one may note that EN 50341-1 does not introduce a specific method for star battened angle sections.

The investigations provided in this paragraph concerning the out-of-plane buckling can be summarised as follows:

- The design method for closely-spaced built-up members provided in EN 50341-1 is based on ECCS publication n°39;
- According to EN 50341-1, the influence of the packing plates have to be accounted for, independently from the distance between each other;
- EN 50341-1 accounts for the influence of the packing plate connection by an interaction between the overall slenderness of the uniform built-up member and the slenderness of a chord between the packing plates;
- The resistance predicted by EN 50341-1 is higher than the resistance predicted by EN 1993-1-1 for the uniform member as the latter uses buckling curve b and EN 50341-1 takes benefit of buckling curve a_0;
- Applying the design method proposed in EN 1993-1-1 for battened columns on closely spaced built-up members is highly unfavourable compared to the design method proposed in EN 50341-1.

Finally, this paragraph emphasized the need of supplementary research in order to develop a consistent, safe-sided and economic design rule. This task is performed in WP 3 of the ANGELHY project.

5.6 Conclusions

Throughout paragraph 5 of this report, the design of the constituting parts of lattice towers – diagonals under compression and tension as well as leg members under compression – is addressed and the different design methods provided in the three standards EN 1993-1-1, EN 1993-3-1 and EN 50341-1 are compared. In particular, it has been shown that the specific tower standards EN 1993-3-1 and EN 50341-1 lead to more economic results than EN 1993-1-1. Especially for the buckling resistance of diagonals and leg members, one might suppose that the methods of the two tower standards are more economic as system effects are, at least partially, accounted for. Nonetheless, the present paragraph has highlighted that there seems to be no clear background that completely justifies the rules. Additionally, certain methods appear to be somewhat inconsistent (see discontinuity in Figure 25, for example). This highlights the need of further research work concerning the link between modelling, analysis and design of lattice towers. Therefore, the work to be provided in WP 2 (concerning design of single angle sections and modelling and analysis of lattice towers) as well as in WP 3 (concerning the resistance of closely spaced built-up members) will be valuable in order to obtain the information necessary to propose consistent design methods.

6 Design assisted by testing according to EN 50341-1

6.1 General

The design methods provided in EN 50341-1 for steel lattice towers have been derived from and calibrated to physical tests on complete towers and sub-structures of towers (see reference [13]). Yet, owing to the complex functioning of steel lattice towers (influence of eccentricities in connections, system effects and redistribution of forces), the precision of the design methods is not always clear, especially when a new tower design is studied. Consequently, testing of a complete tower may give valuable results in order to verify the design methods and the capability of the tower to resist the applied loads. For example, it is recalled that the favourable buckling design methods of Annex J of EN 50341-1 may only be applied if full-scale tower tests have been performed before. As the specific design methods of EN 50341-1 may lead to much more favourable results compared to Eurocode 3, it is of high importance that the tower tests are performed following a well specified procedure and that the exploitation of the test results is done considering all influences that may increase the tower resistance (for example use of steel with higher yield strength than the nominal value).

According to EN 50341-1, the tests on towers of overhead transmission lines should be performed following the provisions given in EN 60652 – Loading tests on overhead line structures [7].This standard distinguishes two types of tests:

- <u>Design tests (§4.1 of EN 60652):</u> The objectives of a design test mainly concern the validation of a new procedure. In particular:
  a) Validation of a new tower design;
  b) Verify the compliance of the tower with specifications;
  c) Develop and validate a new design procedure;
  d) Develop and validate a new fabrication process.
- <u>Sample tests (§4.2 of EN 60652):</u> Sample tests are performed with the objective to check the constant quality of fabrication during the production of a batch of towers.

Nonetheless, it may be noted that sample tests are generally not performed in practice according to the information collected in reference [17]. Inversely, design tests are of practical interest. Observing the objectives of design tests, it is possible to link these tests to certain tests defined in EN 1990. In particular, design tests may be classed as tests of:

- <u>Type a)</u>: “tests to establish directly the ultimate resistance or serviceability properties of structures and structural members for given loading conditions”;
- <u>Type d)</u>: “tests to reduce uncertainties in parameters used in resistance models”.

EN 1990 states that it is necessary to derive the design value of resistance for these types of tests by accepted statistical techniques. In general, it is therefore necessary to perform several tests on the same (or very similar) elements in order to obtain the statistical information needed for the analysis. Yet, for obvious economical reasons, EN 50341-1 as well as EN 60652 do not introduce a minimum number of tests to be realised. In many cases, <u>one single test is therefore performed</u> in order to validate the adequacy of the applied design methods and to obtain the resistance of the tower. Nonetheless, EN 1990 allows the use of a single test for design assisted by testing provided that extensive prior information is available. In this case, the so-called "Bayesian procedure" is to be applied according to ISO 12491 – "Statistical methods for quality control of building materials and components" [9] (see §D6 (2) of EN 1990).

In general, the statistical methods account for the scatter of material and geometrical characteristics of the components used for the fabrication of a structure or of a structural component. The result of the application of statistical methods is a "partial factor" that converts the result of the tests to a design value of, for example, the resistance of the tested structure/component. According to EN 50341-1, this partial factor is considered equal to 1,05 as the test is deemed successful if the tested tower withstands the test load $F_{test,R}$ given by.

$$\boldsymbol{F_{test,R} = 1,05 F_{R,d}} \qquad \textbf{Eq. 6.1}$$

With:

$F_{R,d}$: the calculated load for the Ultimate Limit State

It should be noted that EN 60652 does not define how the resistance $F_{R,d}$ should be determined. It appears obvious that the result of the physical test can only be exploited for the tower design if $F_{R,d}$ is determined under the same loading conditions used in the test. In order to be consistent with EN 50341-1, it is also necessary that the determination of the tower resistance $F_{R,d}$ is based on the buckling design methods defined in Annex J of EN 50341-1.

If the tower test carried on up to failure, it is possible to determine a real design resistance based on Eq. 6.1 as expressed by Eq. 6.2:

$$\boldsymbol{F_{R,d} = \frac{F_{test,R}}{\gamma_M} = \frac{F_{test,R}}{1,05}} \qquad \textbf{Eq. 6.2}$$

In the following, it is of special interest to study if the factor of 1,05 is in compliance with the reliability requirements formulated in EN 1990. In order to do so, the test procedure defined in EN 60652 is first presented in paragraph 6.2. Then, paragraph 6.3 addresses the exploitation of the tower tests on a probabilistic basis in order to analyse the origin of the value 1,05. Finally, the compliance of this factor with the provisions given in EN 1990 is discussed and recommendations are derived.

6.2 Test procedure according to EN 60652

Prior to the test, a test program has to be defined. According to EN 60652, this test program should contain all necessary information concerning the test procedure and in particular, it should detail:

- the position of all measuring devices (dynamometers, load cells, displacement transducers, etc.);
- the test rigging arrangement and attachment details;
- the method and position of load application;

- the load cases (with the maximum load $F_{test,R}$ for each load case) to be considered and the sequence of loading;
- the loading rate;
- the load steps and holding period.

Several points of the previous list are detailed further more in EN 60652. Of course, the load application is of most importance for the results. Therefore, it is interesting to describe these points hereafter according to the provision of the standard:

Choice of load cases and position of load application:

The load cases tested should represent most critical cases (for example leading to the highest axial compression respectively tension force in different members). These cases have to be defined by the client. Generally, all load cases are applied to the same tower specimen. Consequently, it is obvious that only the last load case may be conducted up to failure and give information about the real resistance of the tower for this tested load case. The tests not performed up to failure should be conducted at least up to the specified test load $F_{test,R}$. They therefore can be used to validate if the tower resistance is higher than the specified loads (loads at ultimate resistance). The load application should sufficiently precisely represent the real loads (mostly resulting from wind).

Load steps and holding period:

EN 60652 recommends to apply the loads in steps of 50%, 75%, 90%, 95% and 100% of the specified test load $F_{test,R}$. For each load step, the loads should be kept constant for about 5 to 10 minutes (see also reference [17]).

If the tower resists the final load step at least 1 minute without any failure of its components, it is considered acceptable with respect to the tested load case. If the tower fails before it resists 1 minute to 100% of the specified loads, the following procedure should be applied in case of design tests:

- For failure at less than 95% of $F_{test,R}$: The failed component is to be replaced and the test should be repeated up to 100% of $F_{test,R}$;
- For failure between 95% and 100% of $F_{test,R}$: The tower should be modified and tested again.

Nonetheless, it appears that the general practice is to review the failed component and to modify its design (modify section, modify joints) independently from the failure load level. The modified tower is then tested again. Additionally, the habit consists in testing only the load cases that have not been passed before failure of a component (see reference [17]).

When the tower has passed all load cases successfully, it is necessary to check whether its constituting parts (members, gusset plates, bolts) respect the specifications provided in the relevant product and execution standards. Concerning steel lattice towers, relevant provisions are for example given in:

- EN 10025-1…-6 for material properties of hot rolled steel;
- EN 10056-1…-2 for dimensions and tolerances of hot rolled steel angles.

In order to validate the conformity of the components of the steel tower, tests are performed on randomly selected elements. The general practice is to select five to eight elements to be tested [17].

The testing procedure described here according to the provisions of EN 60652 is in conformity to the specifications given in EN 1990 for carrying out physical tests. Nonetheless, in order to enable the designer the exploit the test result reliably, some precisions should be introduced into the test procedure as recommended in paragraph 6.4.

At this point, it is therefore of most interest to address the problematic of the exploitation of the test result. It has been accepted in the Overhead Transmission Line industry that a single test is not representative of the characteristic strength of a batch of towers (a tower population). Therefore, reference [17] introduces a probabilistic approach that is deemed to be in accordance with ISO 12491 and IEC 60826 – "Design criterion of overhead transmission lines" that addresses the design of overhead lines on a reliability based approach. In paragraph 6.3, the proposed probability based exploitation of the tower tests are presented and its compliance to the provisions of EN 1990 is discussed.

6.3 Exploitation of test results – Reliability analysis

6.3.1 Types of tests

The method used to address the reliability of the full-scale test should be adapted to the objective of the tower test. Indeed, it is recalled, that the tower test may be used in order to develop new design methods, to validate a new tower design, etc. … (see paragraph 6.1) but it can also be used only in order to enable the designer to use the specific design methods of Annex J of EN 50341-1. Depending of the objective of the test the exploitation of the result should be performed as follows:

1) Test performed in order to validate the application of Annex J:

   It has to be supposed that the design methods of Annex J are well calibrated with reference to physical tests. Therefore, if the tested tower is sufficiently similar to the towers that have been used for the calibration of Annex J it is not necessary to perform a reliability analysis. However, it has to be verified that the experimentally determined tower resistance exceeds the tower resistance obtained by applying the design procedures defined in EN 50341-1. Obviously, the applied design procedures should be based on the same data as the tower test and in particular on measured yield stress, measured cross section dimensions and the internal forces and moments resulting from the test load. The influence of possible variations of the yield strength and the geometric dimensions compared to nominal values is included in the partial factor used in the design method. In this case the factor of 1,05 may be understood to ensure a supplementary margin of safety to cover uncertainties in the testing procedure (conversion factor).

2) Test performed to develop a new tower design/a new design method:

   If it appears that the tower design may not be assimilated to the towers used for the calibration of the design procedures or if a new design procedure is to be developed, it is necessary to perform a reliability analysis based on statistical methods as defined in EN 1990 in order to exploit the test result. This second case is considered hereafter in paragraphs 6.3.2 and 6.3.3.

6.3.2 Recall of EN 1990 procedure

EN 1990 proposes the two following methods to derive design results for a resistance based on results of physical tests:

- Method A: The characteristic value is assessed and then divided by an appropriate partial factor γ_M and multiplied by a conversion factor to obtain the design value;
- Method B: The design value is directly determined by accounting, implicitly or explicitly, for the total required reliability.

It may be recalled that EN 1990 defines the characteristic value of the resistance as the 5% fractile value. Also, one may note that Method A is recommended in EN 1990 provided that the value of the partial factor is known (for example, as defined by the relevant part of Eurocode).

Independently from the applied methods the evaluation of the physical tests should account for:

- scatter of test data;
- statistical uncertainty associated with the number of tests;
- prior (statistical) knowledge;
- required level of reliability (for Method B).

Based on the necessary information associated with the previous four points, the characteristic value and the design value of the resistance may be determined with "standard" probabilistic methods. Nonetheless, in order to apply these methods, a minimum number of three tests is necessary for the evaluation. Yet, EN 1990 also refers to the use of fewer tests. In this case, the so-called "Bayesian approach" should be applied according to ISO 12491. This procedure absolutely needs prior knowledge of statistical distributions (mean value, standard deviation) of the property to be measured in the tests, i.e. resistance.

Based on the hypothesis that the statistical distribution of the resistances previously observed and that the statistical distribution of the actually performed tests is of Gaussian (normal) type, it is possible to obtain an updated statistical distribution that is then used to obtain the design value of the resistance of the tested tower.

The updated mean value μ_{up} is given by:

$$\boldsymbol{\mu_{up} = \frac{\mu_0 \sigma_0^2 + n\mu\sigma_{\mu 0}^2}{\sigma_0^2 + n\sigma_{\mu 0}^2}} \qquad \textbf{Eq. 6.3}$$

Where:

μ_{up}: is the updated value of the mean

μ_0: is the mean value of the population (of previous tests)

σ_0: is the standard deviation of the population (of previous tests)

$\sigma_{\mu 0}$: is the standard deviation of the mean value of the population (of previous tests)

μ: is the mean value of the performed tests n: is the number of performed tests

According to the Bayesian approach, the updated value of the standard deviation σ_{up} can be obtained by:

$$\boldsymbol{\sigma_{up}} = \sqrt{\frac{\sigma_0^2 \sigma_{\mu 0}^2}{\sigma_0^2 + n\sigma_{\mu 0}^2}} \qquad \textbf{Eq. 6.4}$$

With the aid of the updated standard deviation and the updated mean value, it is possible to determine the design value of resistance of the tested tower according to EN 1990 with Eq. 6.5.

$$\boldsymbol{R_d = \eta_d(\mu_{up} - k_{x,n}\sigma_{up})} \qquad \textbf{Eq. 6.5}$$

Where:

η_d: is the conversion factor accounting for sources of scatter not covered by the tests;

$k_{x,n}$: is the fractile factor for a sample size n.

It is very delicate to choose a general value of the conversion factor, as it highly depends on the objective of the test and the test arrangement used to obtain the result. For the following comparison the conversion factor is supposed to be equal to η_d = 1,0.

The factor $k_{x,n}$ has to account for the number of performed tests and for the target failure probability. For example, EN 1990 bases the determination of the design resistance approximatively on the 0,1% fractile (reliability index is equal to 3,04) of the distribution function (this value corresponds to reliability class RC2). The corresponding value of the factor $k_{x,n}$, noted as $k_{d,n}$ for the 0,1% fractile, is given in Table 10. At this point it should be underlined that the failure probability of 0,1% is of purely theoretical nature. Indeed, the physical probability of failure for an existing structure is much lower due to effects that are not covered in the resistance model. Nonetheless, the term "failure probability" is used hereafter for simplicity.

Table 10: Values of $k_{d,n}$ according to EN 1990

n	1	2	3	4	5	6	8	10	20	30	∞
V_x known	4,36	3,77	3,56	3,44	3,37	3,33	3,27	3,23	3,16	3,13	3,04
V_x unknown				11,40	7,85	6,36	5,07	4,51	3,64	3,44	3,04

For other failure probabilities (different than 0,1%) one may find values of the $k_{x,n}$ factor in ISO 12491 [9]. In this standard the $k_{x,n}$ factor is linked to t_p fractile (corresponding to the failure probability) of the t-distribution with ν degrees of freedom (ν is the sample size "n" minus 1). The t-distribution is recalled in Eq. 6.6.

$$\boldsymbol{f(t,v)} = \frac{\boldsymbol{\Gamma}\left(\frac{v+1}{2}\right)}{\sqrt{v\pi}\,\boldsymbol{\Gamma}\left(\frac{v}{2}\right)}\left(\mathbf{1}+\frac{t^2}{v}\right)^{-\frac{v+1}{2}} \qquad \textbf{Eq. 6.6}$$

Consequently, the $k_{x,n}$ factor may be determined through Eq. 6.7.

$$t_p = \int_{-\infty}^{k_{x,n}} f(t,v)\,dt \qquad \textbf{Eq. 6.7}$$

Hereafter, the Bayesian approach is applied according to ISO 12491 in order to assess the reliability of full-scale tests used to obtain a design resistance for the tested tower according to EN 50341-1.

6.3.3 Application of the Bayesian approach

The previous paragraph highlighted, that the use of very few tests is possible for the determination of a design resistance that concerns a total population of towers. Yet, the Bayesian approach assumes that valuable prior knowledge of the statistical distribution of the overall tower population is available (resulting from prior tests). References [18] and [19] describe two studies that have been performed in order to obtain a statistical distribution of the tower resistances determined through full-scale tests. The studies have shown that the statistical distribution of the resistance is approximatively of Gaussian type (normal distribution) with a mean value of μ = 104,6%, a standard deviation of σ = 8,9% and a standard deviation of the mean of σ_0 = 4,45% (see also reference [17]). The given values correspond to the ratio between the resistance obtained through the tower tests, noted as F_{test} hereafter, and the resistance obtained through a design model, noted as $F_{R,d}$. It should be noted that the reference value $F_{R,d}$ used for the evaluations in reference [19] has been determined with the "ASCE Guide for the Design of Lattice Transmission Towers" [20]. This point is of high importance as the ratio between the resistance obtained in a full-scale test and the resistance obtained through a design model obviously depends on the design model that is used.

In addition to the statistical distribution, it is necessary to define a target reliability level. It is recalled that the failure probability of a structure for a reference period of 50 years should be less than 0,1% according to EN 1990 (for reliability class RC2).

Based on the statistical distribution of the tower resistances and the target failure probability, a reduction factor linking the test results to a design value of resistance for the overall tower population can be determined. In a first step, the design resistance linked to the statistical data of past tower tests is determined without considering the influence of new test results. The calculation is based on the distribution presented in reference [19]. The value of $k_{d,n}$ to be used in Eq. 6.5 is equal to 3,04 as a sufficiently high number of tests have been performed in the past leading to the cited mean value and standard deviation of tower resistances. The design resistance becomes:

$$R_d = \mu - k_{x,n}\sigma = 1,046 - 3,04 \times 0,089 = 0,775 \qquad \textbf{Eq. 6.8}$$

Consequently, the design resistance should be equal to 77,5% of the resistance obtained from calculation following the provisions of [20] in order to respect a failure probability of 0,1%. In other words, a design resistance equal to the calculated resistance $F_{R,d}$ would be attained if the mean value of the resistance obtained from past tests was 1,29 $F_{R,d}$ instead of 1,046 $F_{R,d}$.

Yet, if a new test is performed, one obtains an updated mean value and an updated standard deviation according to Eq. 6.3 and Eq. 6.4 In references [7] and [17], it is recommended that a tower test is considered successful only if at least the reference load is attained. This reference load is defined in EN 50341-1 as equal to 1,05 times the calculated resistance $F_{R,d}$. Therefore, a successful tower test leads at least to a measured resistance equal to 105% of the calculated resistance. It is consequently possible to obtain a lower estimate of the new distribution with the aid of the Bayesian approach.

If at least two tests are performed, one obtains the following results:

$$\boldsymbol{\mu_{up}} = \frac{\boldsymbol{\mu_0\sigma_0^2 + n\mu\sigma_{\mu 0}^2}}{\boldsymbol{\sigma_0^2 + n\sigma_{\mu 0}^2}} = \frac{\mathbf{1,046 \times 8,9^2 \ + 2 \times 1,05 \times 4,45^2}}{\mathbf{8,9^2 \ + 2 \times 4,45^2}} = \mathbf{1,0473} \qquad \textbf{Eq. 6.9}$$

$$\boldsymbol{\sigma_{up}} = \sqrt{\frac{\boldsymbol{\sigma_0^2\sigma_{\mu 0}^2}}{\boldsymbol{\sigma_0^2 + n\sigma_{\mu 0}^2}}} = \sqrt{\frac{\mathbf{8,9^2 \ \times 4,45^2}}{\mathbf{8,9^2 \ + 2 \times 4,45^2}}} = \mathbf{0,0363} \qquad \textbf{Eq. 6.10}$$

Eq. 6.5 is again applied with the updated values of Eq. 6.9 and Eq. 6.10. Hereafter, ISO 12491 is applied to obtain the $k_{x,n}$ factor corresponding to 2 tests and a target failure probability of 0,1%. Applying Eq. 6.6 and Eq. 6.7, one obtains a value of 318! Obviously, it is not possible to attain the failure probability of 0,1% using the Bayesian approach with only two tests. Nonetheless, it is to be noted that the value of the $k_{x,n}$ factor is very sensitive to the number of tests as shown in Table 11. In this table it is shown that at least **11 tests**, leading to a mean value of 1,05 $F_{test}/F_{R,d}$, have to be performed in order to consider the tower resistance equal to $F_{R,d}$ with a failure probability lower than 0,1%. Obviously, it is not economically viable to perform eleven full-scale tests. Generally, only one or two full-scale tests are realised up to failure. Nonetheless, it may also be possible to accept a higher failure probability leading to a lower value of the factor $k_{x,n}$. Indeed, the factor $k_{x,n}$ is also very sensitive to the target failure probability as shown in Table 12.

Table 11: Ratio between F_{test} and $F_{R,d}$ as a function of the number of tests

Number of tests	Mean value of the ratio $F_{test}/F_{R,d}$ obtained through new tests - μ	Updated mean value of the tower population μ_{up}	Updated standard deviation of the tower population σ_{up}	$k_{x,n}$ according to ISO 12491	$F_{test}/F_{R,d}$
2	1,05	1,0473	0,0363	318,3	-
4	1,05	1,0480	0,0315	10,22	1,376
7	1,05	1,0485	0,0268	5,207	1,100
11	1,05	1,0489	0,0230	4,14	1,049

The results shown in Table 12 are calculated based on the assumption that two tests have been performed leading to a mean value of 1,05 for the ratio $F_{test}/F_{R,d}$. One may observe that a failure probability of 11,6% has to be accepted in order to obtain the design resistance of 1,0 $F_{R,d}$ based on the results of two full-scale tests.

Table 12: Ratio between F_{test} and $F_{R,d}$ as a function of the failure probability

Failure probability	$k_{x,n}$	$F_{test}/F_{R,d}$
5,00 10^{-2}	6,314	1,223
1,00 10^{-1}	3,078	1,069
1,16 10^{-1}	2,613	1,050

Clearly, a failure probability of 11,6% appears to be much higher than the target failure probability of approximately 0,1% defined in EN 1990 for the resistance models. Nonetheless, one may note that the obtained value is close to the “exclusion limit” of 10% (corresponding to the failure probability according to EN 1990) defined in IEC 60826 – Design criteria of overhead transmission lines [10]. The value of 1,05 used in Eq. 6.1 is therefore roughly in accordance with the IEC standard for overhead transmission lines if at least two tests are performed. Yet, it appears to be impossible to justify a minimum reliability level based on one single tower test.

One may note that a single tower test actually consists of several “sub-tests” corresponding to different load cases. Obviously, only the last sub-test may be conducted up to failure. However, in order to consider a test as successful, the tower should at least resist to 1,05 $F_{R,d}$ according to EN 50341-1 and EN 60652. Depending on the number of “sub-tests” an acceptable reliability level may therefore be attained. As shown in Table 11, it is even possible to attain a failure probability of less than 0,1% if 11 load cases are studied.

Last, it should be noted that the statistical data of tower resistances used throughout this paragraph to evaluate the reliability level of full-scale tower tests has been determined with reference to the resistance model defined in “ASCE. Guide for Design of Steel Transmission Towers” [20]. An evaluation based on other standards may lead to different results. In order to highlight this statement it appears interesting to cite reference [21] presenting an evaluation of the reliability of the Australian standard “AS 3995: Design of steel lattice towers and masts” [11]. This publication primarily concerns the evaluation of the design methods applied to communication towers. Nonetheless, even if the geometry of the communication towers is different from the geometry of transmission towers, the global behaviour and failure modes of both tower types are similar. Indeed, reference [21] studies, amongst others, the accuracy of the design model proposed for the buckling resistance of angle leg members. The evaluation has revealed a mean value μ of 1,11 (ratio $F_{test}/F_{R,d}$) and a standard deviation σ of 0,178 with reference to experimental data. These values are clearly different from those determined in reference [19] comparing the American tower standard and full-scale tests (reference [19] obtains $\mu = 1{,}046$ and $\sigma = 0{,}089$). Admittedly, the statistical data presented in both studies cannot be compared directly as buckling of the tower legs is only one of many possible failure modes. Nonetheless, the comparison indicates that different standards can lead to different reliability levels. Therefore, it seems necessary to evaluate full-scale tower tests with reference to Eurocode 3 Part 3-1 in order to obtain a complete view on the reliability level of full-scale tests in the context of the Eurocodes.

6.4 Conclusions and recommendations

Section 6 gives an overview of the evaluation of full-scale tests for the determination of the design resistance of transmission line towers. It is recalled that EN 1990 allows designers to perform very few tests to obtain a corresponding design resistance if they possess sufficient knowledge concerning the statistical distribution of the resistance. These data have to be obtained from prior tests. For overhead transmission towers, valuable statistical data were published in the past and can be used for the evaluation of the design resistance. Yet, it has been shown that the target reliability level defined in EN 1990 for resistance models (corresponding to a failure probability of approximatively 0,1%) cannot be attained with one or two tests. Indeed, it is necessary to perform at least 11 tests in order to ensure that the failure probability is less than 0,1% provided that the tests attain at least $1{,}05F_{R,d}$ as imposed by EN 50341-1. Yet, if at least two tests are performed the target reliability level defined in IEC 60826 is attained. In this case, the failure probability is approximately equal to **10%**. Consequently, it appears that there exists a clear discrepancy between minimum reliability requirements defined in EN 1990 and IEC 60826. Nonetheless, a full-scale tower test generally consists in several sub-tests in order to cover several load cases and design situations. Each sub-test has to attain at least 1,05 times the design resistance. Consequently, it is possible to attain a reliability level similar to the target defined in EN 1990 if each sub-test is considered in the evaluation. It should also recalled that the statistical data of tower resistances used for the evaluation has been obtained with reference to the American design guide for towers published in 1971. The statistical data may certainly be different if the tests are evaluated with reference to other standards. Therefore, it appears interesting to re-evaluate full-scale tests performed in the past with reference to EN 1993-3-1 in order to study if the statistical data are similar to the one published in references [18] and [19].

At this point, it seems necessary to underline that the statistical evaluation of the tower tests seems only necessary if a new tower design (sufficiently different from standard designs) or a new design procedure is developed. If the test is performed on a tower that is similar to towers used in the past for the validation of the used design procedures, it appears not necessary to perform a statistical evaluation. Indeed, in this case, the scatter of key parameters (yield strength, geometric dimensions) is already included in the partial factor applied in the design method.

Independently from the objective of the tower test, it has been pointed out that the test procedure defined in EN 60652 needs some precisions in order to allow the designer to exploit the result. In particular, it should be defined that:

- The design load $F_{R,d}$ is to be calculated under the loads applied in the physical test and based on the measured yield strength and measured geometric characteristics;
- The yield strength should be measured for all casts used to fabricate the profiles used in the tower;
- The material used for the tensile tests should be extracted before the tower test is performed;
- The geometric dimensions should be measured on all relevant profiles, i.e. profiles that are susceptible to lead to the failure of the tower.

7 References

Standards

[1] CEN (2005), “Eurocode 3. Design of steel structures. Part 1-1: General rules and rules for buildings”, Brussels.

[2] CEN (2005), “Eurocode 3. Design of steel structures. Part 1-8: Design of joints”, Brussels.

[3] CEN (2020), “Eurocode 3. Design of steel structures. Part 1-8: Design of joints”, Brussels.

[4] CEN (2007), “Eurocode 3. Design of steel structures. Part 3-1: Towers, masts and chimneys – Towers and masts”, Brussels.

[5] CEN/CENELEC (2015), “EN 50341-1: Overhead electrical lines exceeding AC 1 kV - Part 1: General requirements - Common specifications”, Brussels.

[6] ANSI (2010), “ANSI/AISC 360-10: Specification for Structural Steel Buildings”.

[7] ANSI (2016), “ANSI/AISC 360-16: Specification for Structural Steel Buildings”.

[8] CEN (2004), “EN 60652: Loading tests for overhead line structures”, Brussels.

[9] ISO (1997), “ISO 12491: Statistical methods for quality control of building materials and components”.

[10] IEC (2017), “IEC 60826: Design criteria of overhead transmission lines”.

[11] Standards Australia (1991), “Interim Standard for the Design of Steel Lattice Towers and Masts, Australian Standard AS 3995”, Sydney.

Publications

[12] Moze, P. (2017), “Angles connected by one leg in tension”, Proceedings of the conference EUROSTEEL 2017, Copenhagen, Denmark.

[13] ECCS TC 8 (1985), “Recommendations For Angles in Lattice Transmission Towers”, Brussels.

[14] Taras, A (2011), “Contribution to the development of Consistent Stability Design Rules for Steel Members”, PhD thesis, TU Graz.

[15] Vayas, I., Charalambakis, A., Koumousis, V. (2009), “Inelastic resistance of angle sections subjected to biaxial bending and normal forces”, Steel Construction – Design and Research, 2, 138-146.

[16] Kettler, M., Unterweger, H. (2017), “Bolted steel angles in compression – Influence of realistic end supports”, Proceedings of the conference EUROSTEEL 2017, Copenhagen, Denmark.

[17] Cigré – Working group B2.08 (2009), “Publication n°399 – Improvement of the tower testing methodology”.

[18] Paschen, R., Pezard, J., Zago, P. (1988), “Probabilistic evaluation of test results of transmission line towers”, report 22-13, Cigré session 1988.

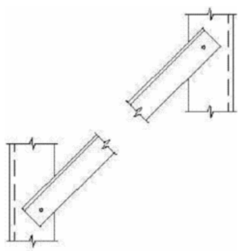
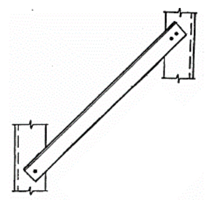
[19] Riera, J.D, de Menezes, R.C.R., da Silva, V.R., Ferreira da Silva, J.B.G. (1990), “Evaluation of the probability distribution of the strength of transmission line steel towers based on tower test results”, report 22-305, Cigré session 1990.

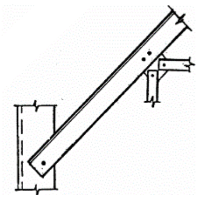
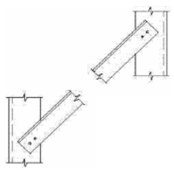
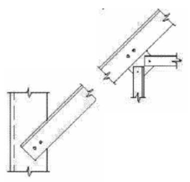
[20] ASCE (1971), “Guide for Design of Steel Transmission Towers”.

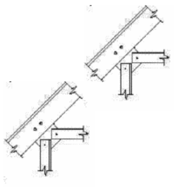
[21] Pham, L., Holmes, J.D., Yang, J. (1992), “Reliability Analysis of Australian Communication Lattice Towers”, Journal of Constructional Steel Research 23,255-272.

Annex A – Slenderness modification factor for buckling of diagonals

Table 13: Slenderness modification factor for buckling of diagonals

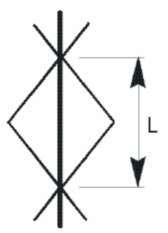
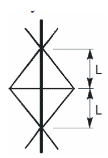
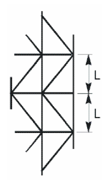
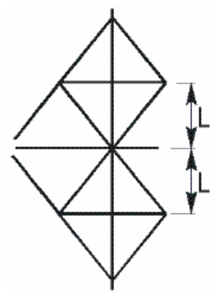
<table>
<tr><th rowspan="2">Cases</th><th colspan="3">EN 1993-1-1</th><th colspan="4">EN 1993-3-1</th><th colspan="7">EN 50341-1</th></tr>
<tr><th>Imperfection factor α</th><th>Axi s</th><th>Factor k</th><th>Imperfection factor α</th><th>Axis</th><th>Factor k</th><th>Reduction of η applied to χ</th><th>Imperfection factor α</th><th>Axis</th><th>Slenderness</th><th>Load eccentricity</th><th>Continuity of the element</th><th>Number of bolts at the non-continuous end</th><th>Factor k</th></tr>
<tr><td rowspan="4">1
</td><td colspan="3" rowspan="4">Simplified method cannot be applied. The angle has to be checked by applying a method considering the bending moment introduced at the member ends (for example interaction equations (6.61) and (6.62))</td><td rowspan="4">0,34</td><td rowspan="2">v-v</td><td rowspan="2">$0,7+\frac{0,35}{\bar{\lambda}_v}$</td><td rowspan="4">0,8</td><td rowspan="4">0,13</td><td rowspan="2">v-v</td><td>$\bar{\lambda}_v \le \sqrt{2}$</td><td>at both ends</td><td>-</td><td>-</td><td>$0,65+\frac{0,50}{\bar{\lambda}_v}$</td></tr>
<tr><td>$\bar{\lambda}_v > \sqrt{2}$</td><td>-</td><td>no</td><td>-</td><td>1</td></tr>
<tr><td rowspan="2">z-z/ y-y</td><td rowspan="2">$0,7+\frac{0,58}{\bar{\lambda}}$</td><td rowspan="2">z-z/ y-y</td><td>$\bar{\lambda} \le \sqrt{2}$</td><td>at both ends</td><td>-</td><td>-</td><td>$0,65+\frac{0,71}{\bar{\lambda}}$</td></tr>
<tr><td>$\bar{\lambda} > \sqrt{2}$</td><td>-</td><td>no</td><td>-</td><td>$0,86+\frac{0,40}{\bar{\lambda}}$</td></tr>
<tr><td rowspan="4">2a
</td><td colspan="3" rowspan="4">Simplified method cannot be applied. The angle has to be checked by applying a method considering the bending moment introduced at the member ends (for example interaction equations (6.61) and (6.62))</td><td rowspan="4">0,34</td><td rowspan="2">v-v</td><td rowspan="2">$0,7+\frac{0,35}{\bar{\lambda}_v}$</td><td rowspan="4">0,9</td><td rowspan="4">0,13</td><td rowspan="2">v-v</td><td>$\bar{\lambda}_v \le \sqrt{2}$</td><td>at both ends</td><td>-</td><td>-</td><td>$0,65+\frac{0,50}{\bar{\lambda}_v}$</td></tr>
<tr><td>$\bar{\lambda}_v > \sqrt{2}$</td><td>-</td><td>no</td><td>-</td><td>1</td></tr>
<tr><td rowspan="2">z-z/ y-y</td><td rowspan="2">$0,7+\frac{0,40}{\bar{\lambda}}$</td><td rowspan="2">z-z/ y-y</td><td>$\bar{\lambda} \le \sqrt{2}$</td><td>at both ends</td><td>-</td><td>-</td><td>$0,65+\frac{0,71}{\bar{\lambda}}$</td></tr>
<tr><td>$\bar{\lambda} > \sqrt{2}$</td><td>-</td><td>no</td><td>-</td><td>1</td></tr>
</table>

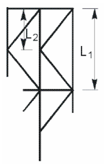
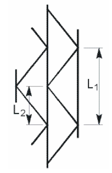
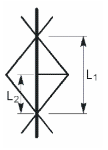
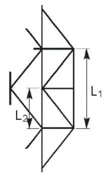
2b 	Simplified method cannot be applied. The angle has to be checked by applying a method considering the bending moment introduced at the member ends (for example interaction equations (6.61) and (6.62))			0,34	v-v	$0{,}7+\frac{0{,}35}{\bar{\lambda}_v}$	0,9	0,13	v-v	$\bar{\lambda}_v \leq \sqrt{2}$	at both ends	-	-	$0{,}65+\frac{0{,}50}{\bar{\lambda}_v}$
										$\bar{\lambda}_v > \sqrt{2}$	-	at one end	1	1
					z-z/ y-y	$0{,}7+\frac{0{,}40}{\bar{\lambda}}$			z-z/ y/y	$\bar{\lambda} \leq \sqrt{2}$	at both ends	-	-	$0{,}65+\frac{0{,}71}{\bar{\lambda}}$
										$\bar{\lambda} > \sqrt{2}$	-	at one end	1	1
3a 	0,34	v-v	$0{,}7+\frac{0{,}35}{\bar{\lambda}_v}$	0,34	v-v	$0{,}7+\frac{0{,}35}{\bar{\lambda}_v}$	1	0,13	v-v	all	at both ends	-	-	$0{,}65+\frac{0{,}50}{\bar{\lambda}}$
											-	no	-	
		z-z/ y-y	$0{,}7+\frac{0{,}50}{\bar{\lambda}_v}$		z-z/ y-y	$0{,}7+\frac{0{,}40}{\bar{\lambda}}$			z-z/ y-y	all	at both ends	-	-	$0{,}65+\frac{0{,}71}{\bar{\lambda}_v}$
											-	-	-	
3b 	0,34	v-v	$0{,}7+\frac{0{,}35}{\bar{\lambda}_v}$	0,34	v-v	$0{,}7+\frac{0{,}35}{\bar{\lambda}_v}$	1	0,13	v-v	$\bar{\lambda}_v \leq \sqrt{2}$	at both ends	-	-	$0{,}65+\frac{0{,}50}{\bar{\lambda}_v}$
										$\bar{\lambda}_v > \sqrt{2}$	-	at one end	2	
		z-z/ y/y	$0{,}7+\frac{0{,}50}{\bar{\lambda}_v}$		z-z/ y/y	$0{,}7+\frac{0{,}40}{\bar{\lambda}}$			z-z/ y-y	$\bar{\lambda} \leq \sqrt{2}$	at both ends	-	-	$0{,}65+\frac{0{,}71}{\bar{\lambda}}$
										$\bar{\lambda} > \sqrt{2}$	-	at one end	2	$0{,}65+\frac{0{,}50}{\bar{\lambda}}$

3c 	0,34	v-v	$0,7 + \frac{0,35}{\overline{\lambda_v}}$	0,34	v-v	$0,7 + \frac{0,35}{\overline{\lambda_v}}$	1	0,13	v-v	$\overline{\lambda_v} \leq \sqrt{2}$	at both ends	-	-	$0,65 + \frac{0,50}{\overline{\lambda_v}}$
										$\overline{\lambda_v} > \sqrt{2}$	-	at both ends	-	
		z-z/ y/y	$0,7 + \frac{0,50}{\overline{\lambda_v}}$		z-z/ y/y	$0,7 + \frac{0,40}{\overline{\lambda}}$			z-z/ y-y	$\overline{\lambda} \leq \sqrt{2}$	at both ends	-	-	$0,65 + \frac{0,71}{\overline{\lambda}}$
										$\overline{\lambda} > \sqrt{2}$	-	at both ends	-	1

Annex B - Slenderness modification factor for buckling of leg members

Table 14: Slenderness modification factor for buckling of leg members

Cases	EN 1993-1-1	EN 1993-3-1			EN 50341-1		
		Imperfection factor α	Axis	Factor k	Imperfection factor α	Axis	Factor k
1 Primary bracing at both ends 	No specific rule provided	0,34	v-v	$0,8+\frac{\overline{\lambda_v}}{10}$ and: $0,9 \leq k \leq 1,0$	0,13	v-v	1
			z-z/ y-y	Not to be checked		z-z/ y-y	Not to be checked
2 Primary bracing at one end and secondary bracing at the second end symmetric  asymmetric 	No specific rule provided	0,34	v-v	$0,8+\frac{\overline{\lambda_v}}{10}$ and: $0,9 \leq k \leq 1,0$	0,13	v-v	1
			z-z/ y-y	Not to be checked		z-z/ y-y	Not to be checked
3 Secondary bracing at both ends 	No specific rule provided	0,34	v-v	$0,8+\frac{\overline{\lambda_v}}{10}$ and: $0,9 \leq k \leq 1,0$	0,13	v-v	1
			z-z/ y-y	Not to be checked		z-z/ y-y	Not to be checked

<table>
<tr><td colspan="2">4
Primary bracing at both ends – non symmetric</td><td rowspan="3">No specific rule provided</td><td rowspan="3">0,34</td><td rowspan="2">v-v</td><td rowspan="2"><u>Only for unequal leg angles:</u>
$1,2\left(0,8+\frac{\overline{\lambda_v}}{10}\right)$
and:
$1,08 \le k \le 1,2$
Buckling over length <u>L₂</u></td><td rowspan="3">0,13</td><td rowspan="2">v-v</td><td rowspan="2">Not to be checked</td></tr>
<tr><td>a)</td><td>b)</td></tr>
<tr><td></td><td></td><td>z-z/
y-y</td><td>$1,2\left(0,8+\frac{\overline{\lambda}}{10}\right)$
and:
$1,08 \le k \le 1,2$
Buckling over length <u>L₁</u></td><td>z-z/
y-y</td><td>1,2</td></tr>
<tr><td colspan="2">5
Primary bracing at both ends and intermediate secondary bracing – non symmetric</td><td rowspan="3">No specific rule provided</td><td rowspan="3">0,34</td><td rowspan="2">v-v</td><td rowspan="2"><u>Only for unequal leg angles:</u>
$0,8+\frac{\overline{\lambda_v}}{10}$
and:
$0,9 \le k \le 1,0$
Buckling over length <u>L₂</u></td><td rowspan="3">0,13</td><td rowspan="2">v-v</td><td rowspan="2">Not to be checked</td></tr>
<tr><td>a)</td><td>b)</td></tr>
<tr><td></td><td></td><td>z-z/
y/y</td><td>$0,8+\frac{\overline{\lambda}}{10}$
and:
$0,9 \le k \le 1,0$
Buckling over length <u>L₁</u></td><td>z-z/
y-y</td><td>1</td></tr>
</table>