ORIGINAL ARTICLE

# Load carrying capacity of star-battened angle members made of S460 steel in compression

Marios-Zois Bezas[1] | Sofia Antonodimitraki[1] | Andre Beyer[2] | Muhammad-Omer Anwaar[3] | Jan Maesschalck[4] | Jean-Pierre Jaspart[1] | Jean-François Demonceau[1]

**Correspondence**

Dr.-Ing. Marios-Zois Bezas
University of Liege
Steel and Composite Construction
Allée de la découverte 9 B52/3
Liege 4000, Belgium
Email: marios-zois.bezas@uliege.be

[1] University of Liege, Liege, Belgium
[2] CTICM, Saint Aubin, France
[3] ArcelorMittal Global R&D, Luxembourg, Luxembourg
[4] Elia Transmission Belgium SA/NV, Brussels, Belgium

**Abstract**

Lattice towers are extensively built in Europe and worldwide to serve power transmission purposes. Their members mainly consist of single angle profiles, while built-up sections, such as star-battened ones, are usually preferred for the strengthening of individual members or in case of very high compression loads. Recent upwards developments of the power grid requests also lead to the need to use high strength steels (HSS) for the design and construction of higher and heavier loaded steel lattice transmission towers. Within the present paper, investigations conducted on the bearing capacity of a star-battened member made of two S460 L250x250x28 angles are presented, even if the conclusions may be extended to any angle size and steel grade.

Firstly, a critical review of the current European normative documents (EN 50341, EN1993-1-3, prEN1993-3) is made and the buckling resistance of the above-mentioned member is evaluated. Secondly, the results of an experimental compression test performed on the studied member are presented; these results are complemented by a full non-linear finite element numerical simulation. All the obtained results are finally compared and discussed.

**Keywords**

Buckling, Star-battened section, Built-up section, Angle profile, High-strength steel

## 1 Introduction

In order to face the challenges of the energy transition needs, the transmission capacity of high voltage lines and towers must be significantly risen in the future years. So, the existing conductors should be multiplied or replaced by bigger higher grade conductors, and this has a direct impact on the structural stability of the towers as the loads to be supported by the latter are increased. Existing towers must therefore be reinforced and new towers need to be designed for higher loads, where usually built-up star-battened profiles are used. Currently, all high voltage lattice towers are made of S355 steel grade, which was introduced in the early 1960s and is still used in new towers up to today. But the use of high strength steel (HSS) is seen as necessary in order to meet the new structural performance requirements with the goal of minimum visual impact and CO2 footprint. Among them, S460 stands out with a yield strength up to 30% higher than S355, while the weight of an S460 tower could be relative compared to its S355 counterpart by optimizing the section of the profiles, leading thus to lighter structures. Therefore, S460 appears quite promising for the sustainable strengthening of existing towers and for the design of new ones.

Lattice transmission towers in Europe are currently designed according to EN 1993-3-1 [1] with references to other parts of Eurocode 3 such as EN 1993-1-1 [2], EN 1993-1-5 [3] and EN 1993-1-8 [4], or in accordance with the CENELEC standard EN 50341-1 [5] which is exclusively dedicated to the design of overhead electrical transmission lines. However, the design methods given in EN 50341-1 sometimes diverge from the rules provided in the Eurocodes as discussed in Ref. [6]. Additionally, even if this is not strictly prevented, the application of EN 50341-1 to S460 steel grades remains quite questionable. It is also remarkable that, amongst all these normative documents, a number of inconsistencies have been identified concerning different aspects of design as detailly addressed in Ref. [7]. Although new steels have been recently developed, the lack of evolution concerning their use in electrical pylons is due to the slow evolution of the extremely strict normative context to which the design, fabrication and production of steel structures (EC3) and in particular steel structures for overhead electrical lines (EN 50341) are subjected. However, in the framework of the recently finished European RFCS project ANGELHY [6], existing European specifications on single and star-battened angle sections were reviewed, experimental, analytical and numerical studies and a complete set of design rules for pin-ended members considering steel grades up to S460 has been developed. The proposed design rules have been included nowadays as Annex F in the latest version of prEN 1993-3 [8]. However, they are not accounting for the effects of the restrains due to bolted connections at the extremities of the angle members.

More globally, extensive research has been carried out in the recent years to study the behavior of single or built-up sections made of angle profiles, but focusing mainly on regular steel grades. Schillo et al. [9] examined the rules of the European standards to predict the buckling resistance of S355 rolled angles and compared them to test results and numerical investigations considering various types of initial imperfections. Bezas et al. [10]-[11] proposed a complete and duly validated set of design rules for angles covering all aspects of their design (classification, cross-section and member resistance); these rules have been validated for steel grade up to S460. Beyer et al. [12] performed a numerical sensitivity analysis of the bearing capacity of built-up S355 members on different geometrical parameters. More recently, Saufnay et al. [13] investigated experimentally and numerically the behavior of closely spaced built-up star-battened angles members made of S355 equal and unequal angle profiles. Further experimental studies of closely spaced built-up members can be found in Ref. [14]-[15], but none of them were considering high strength steel.

This paper focuses on the buckling resistance of a star-battened member constituted of large angle sections made of HSS, but the conclusions may be extended to any angle size and steel grade. Firstly, a critical review of the current European normative documents such as EN 50341, EN 1993-1-3 as well as its latest version prEN 1993-3 is made. In order to make a more concrete comparisons between the norms, a star-battened member made of two S460 L250x250x28 angles subjected to pure compression is considered as study case, and its buckling resistance is evaluated using the provisions of all the above-mentioned norms. The member of the study case has also been tested experimentally, in agreement with Annex J of EN 50341 requesting an experimental validation. Furthermore, a full non-linear finite element numerical simulation by means of the Ansys software [16], considering material and geometrical non-linearities, has been conducted; the main obtained results are presented herein. Experimental, numerical and analytical results are finally compared and discussed. The studies presented in this paper are part of an ongoing project entitled "New steel" funded by Elia and ArcelorMittal industrial partners and involving the University of Liège.

## 2 Buckling resistance to compression: a code review

As indicated above, the normative documents under concern are (i) EN 50341, (ii) EN 1993-3-1 and (iii) prEN 1993-3. As two over three partners of this project are located and act in Belgium, it has been decided to follow also the relevant National Annexes of Belgium when it was necessary [17]-[18]. The classification and cross-section resistance follows the provisions of single angle profiles that have been given for the three considered standards in Ref.[7]. Subsequently, only the design rules for the member resistance of star-battened members subjected to axial compression are briefly addressed and discussed in the following sections.

### 2.1 EN 50341

According to EN 50341, compression members shall be designed using the provisions of Annex G and Annex H of EN 1993-3-1, or in accordance with the provisions of Annex J.4 of EN 50341, only if full-scale tests are performed. For the latter, it is also indicated that the experimental resistance should be at least 5% higher than the analytically determined design load for the ultimate limit state. Besides that, even if this is not strictly prevented, the application of EN 50341 to S460 steels remains questionable; in different sections of the document, specific rules are provided for S235 and S355 steels, but not for S460. This being, Annex J.4 will anyway be applied in this paper, for comparison with the other design procedures, and this application will be complemented by an experimental test. Therefore, the design buckling resistance of a compression star-battened member should be taken as:

$$N_{b,Rd} = \begin{cases} 2\chi A f_y/\gamma_{M1} & \text{for Class 1, 2 and 3 cross} - \text{sections} \\ 2\chi A_{eff} f_y/\gamma_{M1} & \text{for Class 4 cross} - \text{sections} \end{cases} \tag{1}$$

where $A$ or $A_{eff}$ is the gross or the effective cross-section area of one angle profile in the built-up section. The reduction factor $\chi$ is determined as a function of the effective slenderness $\overline{\lambda_{eff}}$, which is evaluated for the relevant buckling mode, i.e. for a flexural (index $F$) about v-v or u-u axis – see Figure 1 for the definition of the axis – or a torsional buckling mode (index $T$), as the member cross-section shown in Figure 1 being a double symmetric one. Additionally, a specific formula for the calculation of the slenderness of compound members is also provided in J.4.3.4.3 (index $u$) and should be considered for the design. Therefore, $\overline{\lambda_{eff}} = \max\{\overline{\lambda_{eff,F}}, \overline{\lambda_{eff,T}}, \overline{\lambda_{eff,u}}\}$. For all the above-mentioned cases, buckling curve $a_0$ is used ($\alpha$ = 0,13).

### 2.2 EN 1993-3-1

As in EN 50341, reference is also made to Annexes G and H in EN 1993-3-1. The design buckling resistance is given by eq. (1), while the reduction factor $\chi$ is again determined as a function of the effective slenderness $\overline{\lambda_{eff}}$, which is evaluated again for the relevant buckling mode. Here however, the specific check for compound members is not addressed, while the imperfection factor $\alpha$ should be taken as equal to 0,34 (buckling curve *b)* for angle sections according to EN 1993-1-1.

### 2.3 prEN 1993-3

The resistance of closely spaced built-up members is checked according to EN 1993-1-1 for flexural buckling about both principal axes using buckling curve *b*. The slenderness of star battened members for buckling about the u-u axis may be given by:

$$\bar{\lambda}_{Sv} = \sqrt{\frac{N_{Rk}}{N_{cr,Sv}}} \tag{2}$$

while buckling about v-v axis is evaluated as for the other two considered normative documents. For the latter, index F is used in order to be in line with the notations used in the previous sub-sections. The critical axial force of the built-up member $N_{cr,Sv}$ may be given, depending on the connection type, as follows:

$$N_{cr,Sv} = \frac{1}{\frac{1}{N_{cr}}+\frac{1}{S_v}} \tag{3}$$

where: $N_{cr} = EI_{max}\left(\frac{\pi}{L_{cr}}\right)^2$ and $S_v = \frac{1}{\frac{1}{24EI_{v,ch}}}$

For the determination of the reduction factor $\chi$, the slenderness should be taken as $\overline{\lambda_{max}} = \max\{\bar{\lambda}_{Sv}, \bar{\lambda}_F\}$. Indeed, it has been illustrated in the framework of the ANGELHY project [19] that torsional or flexural-torsional modes are not relevant for star-battened profiles.

## 2.4 Comparison of the standards

EN 50341:2012 Annex J looks to provide a quite advantageous evaluation of the $\chi$ factor, but also quite questionable. The use of curve $a_0$ for buckling about the u-u axis is quite optimistic and could never be scientifically justified, as it has been shown in the ANGELHY project. But, on the other hand, it is based on an evaluation of the relative slenderness where flexural-torsional effects have been accounted for. Being so, EN 50341 leads to a safe estimation of the relative slenderness, which is further compensated by the unsafe selection of curve $a_0$. Moreover, curve $a_0$ applies also in EN 50341 for buckling about the v-v axis. This is again quite surprising as this buckling mode is the one of two simple independent angles, for which, even EN 1993-1-1: Table 6.2 recommends curve *b*. Although the provisions of this standard are based on thousands of experimental full-scale tests on pylons, those were made of S235 and S355, and so may not be extrapolated without further investigations to higher steel grades.

Compared to EN 50341, the application of EN 1993-1-1 leads to lower resistance as a consequence of the combination of (i) the use of curve *b* and (ii) the consideration of flexural-torsional detrimental effects. In prEN 1993-3 Annex F, curve b is used for buckling about u-u and the v-v axes but is based on a more correct value of the relative slenderness in which torsional and flexural-torsional aspects are not considered; this approach has been numerically and experimentally validated within the ANGELHY project [19]. As a conclusion, the use of prEN 1993-3 appears today as the only scientifically founded way to estimate the buckling capacity of a star-battened member made of high-strength steel.

## 3 Study case – geometry of the studied member

To draw solid conclusions on the analysed standards, a member has been selected as a study case and its buckling resistance has been evaluated experimentally, numerically and analytically. The cross-section of the member consists of two L250x250x28 angle profiles made of S460 steel grade, as shown in Figure 1. The L-profiles are interconnected by 6 plates (PL1 246x264x16 mm) using non-preloaded bolts M30x90 (8.8), while they are welded at their extremities to end plates (PL2 596x596x40 mm). The nominal total member length (including the end plates) equals L = 3464 mm while the distance between the intermediate connection plates is 941 mm, as illustrated in Figure 2. Table 1 summarizes the geometrical properties of the star-battened built-up section that are required for the calculation of the member resistance.

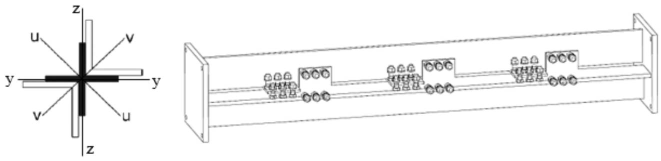


**Figure 1** General layout of the member and definition of axes

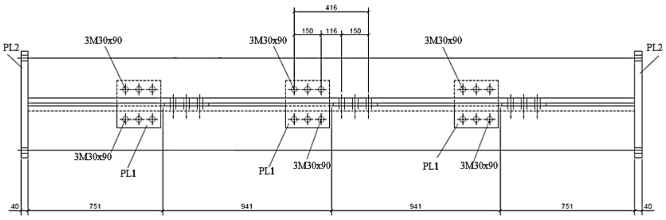


**Figure 2** Geometry of the studied member

**Table 1** Built-up star-battened cross-section properties

| **Property** | **Value** | |
|---|---|---|
| Area ASB | 26600 | mm$^2$ |
| Moment of inertia $I_{max,SB}$ | 40585,84 | cm$^4$ |
| Radius of gyration $i_{max,SB}$ | 123,5 | mm |
| Radius of gyration $i_{min,SB}$ | 95,4 | mm |
| Radius of gyration $i_{x,SB}$ | 110,4 | mm |
| Torsion constant $I_{T,SB}$ | 6907562,7 | mm$^4$ |
| Warping constant $I_{w,SB}$ | $3{,}206\cdot10^6$ | mm$^6$ |

According to the normative document EN 10025 [20] for hot-rolled products, the minimum yield strength $f_y$ for the considered cross-section, due to its thickness, equals 440 MPa. In EN 50341, EN 1993-3 and prEN 1993-3, which all refer to EN 1993-1-1, an alternative variation of the yield strength with the plate thickness is suggested in which no reduction of the yield strength is recommended for thicknesses lower than 40 mm. The use of these thickness variation laws can be set in the National Annexes. In Belgium [21], reference must be made to product norm EN 10025, and a yield strength of 440 MPa is therefore adopted here. Afterwards, three situations have been considered in terms of material properties; the first two cases adopt the nominal values (i.e. 440MPa and 460 MPa) while the third one considers the actual measured one. The upper value of the yield stress was equal to 478,5 MPa while the engineering stress $f_y$ may be determined by the mean value of the yield plateau which was 445 MPa.

## 4 Experimental test

The experimental test of the specimen described in §3 is presented hereafter. All the details regarding the measurements of the initial imperfections, the actual geometrical properties, the coupon tests and the evaluation of residual stresses can be found in Ref. [22]-[23]. The buckling test has been carried out in the OCAS Tubular Testing System which has a capacity of 1580 kN in both tension and compression. To attach the specimen to the test rig, two swivel joints were used at its extremities. Being so, the point of rotation was laying outside the specimen, which extends the buckling length of the test sample to $L_{cr}$ = 3878 mm. During the test, the total shortening as well as the lateral displacements of the mid-section have been measured as schematized in Figure 3. Furthermore, three sets of 4 strain gauges have been placed at 1/4, 1/2 and ¾ of the member's length.

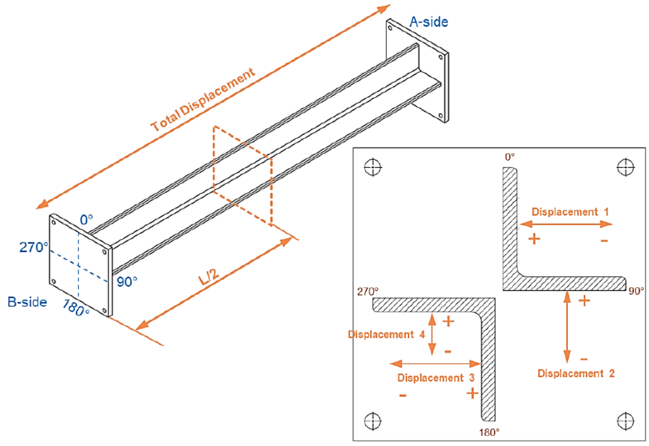


**Figure 3** Location of the displacement sensors

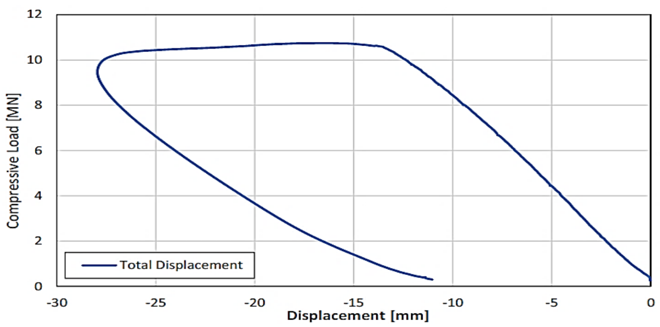


**Figure 4** Axial deformation (total displacement) of the specimen vs the compressive load

Figure 4 and Figure 5 respectively show the total axial deformation and the lateral displacements of the mid-section versus the compressive load. After reaching a compressive load of 10,6 MN, yielding is initiated. The applied force was further increased up to 10,75 MN, where buckling failure occurred. Both in the horizontal and the vertical plane, the sample appears to be deformed in an S-shape as can be observed in Figure 6 while local buckling of the legs appears at L/4 and 3L/4. Therefore, the member failed due to a combination of both global and local buckling.

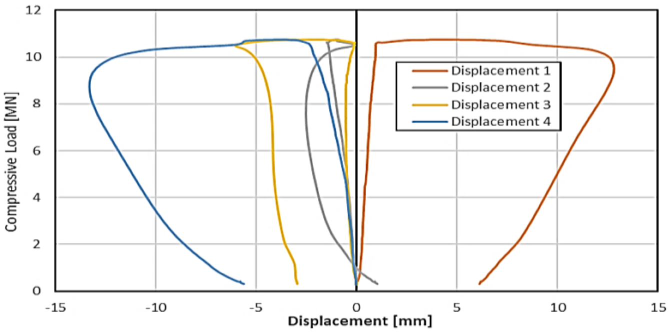


**Figure 5** Lateral displacements of the mid-section vs the compressive load

**Figure 6** Final deformation of the specimen

## 5 Analytical resistance of the member

In this section, the member buckling resistance is evaluated according to the three considered standards described in §2. The profile is categorized in class-3 according to EN 1993-3-1 and prEN 1993-3 but in class-4 based on EN 50341. Despite the inconsistencies in between EN 1993-1-1, EN 1933-1-5 and EN 50341 for the classification and the evaluation of the effective area [7], if a strict application of the existing provisions is followed, then ρ is equal to 1,0; thus no reduction has been implemented in the area for the rest of the calculations. Table 2 summarizes the analytical calculations and the evaluation of the code resistances. Checks that are not required by the considered standard are indicated through dashes in the table.

**Table 2** Analytically obtained resistances

| | EN 50341 | | | EN 1993-3-1 | | | prEN 1993-3 | | |
|---|---|---|---|---|---|---|---|---|---|
| fy [MPa] | 440 | 445 | 460 | 440 | 445 | 460 | 440 | 445 | 460 |
| $\overline{\lambda}_{eff,F}$ [-] | 0,592 | 0,592 | 0,605 | 0,592 | 0,592 | 0,605 | 0,592 | 0,592 | 0,605 |
| $\overline{\lambda}_{Sv}$ [-] | --- | --- | --- | --- | --- | --- | 0,524 | 0,524 | 0,536 |
| $\overline{\lambda}_{eff,T}$ [-] | 0,716 | 0,715 | 0,732 | 0,716 | 0,715 | 0,732 | --- | --- | --- |
| $\overline{\lambda}_{eff,u}$ [-] | 0,537 | 0,537 | 0,550 | --- | --- | --- | --- | --- | --- |
| $\overline{\lambda}_{eff}$ [-] | 0,716 | 0,715 | 0,732 | 0,716 | 0,715 | 0,732 | 0,592 | 0,592 | 0,605 |
| $N_{b,Rd}$ [MN] | 10,42 | 10,54 | 10,82 | 9,07 | 9,17 | 9,34 | 9,84 | 9,96 | 10,21 |
| $N_{exp}/N_{b,Rd}$ [-] | 1,03 | 1,02 | 0,99 | 1,19 | 1,17 | 1,15 | 1,09 | 1,08 | 1,05 |

Amongst the three standards, EN 1993-3-1 looks to be the most conservative one, while EN 50341 predicts quite well the experimental resistance; prEN 1993-3 gives intermediate values. However, according to Section 7.3.9 of EN50341, the experimental result should be 5% higher than the resistance value got through the application of Annex J.4 rules. And if this condition is not satisfied, further calculations have to be achieved so as to derive a new $f_y$ value to be used in Annex J.4. On the contrary, prEN 1993-3 is always on the safe side but not too conservative, especially if we count on $f_y$ = 460MPa as it is allowed in some countries, while its background is scientifically and experimentally validated for high strength steels.

## 6 Numerical simulation

The numerical simulations were carried out with ANSYS software using solid finite elements and taking into account the actual measured imperfections, material properties and residual stresses. Furthermore, to represent the real stiffness of the built-up section, several contact regions were defined, as shown in Figure 7(a). For the contact between the nut and the angle and the contact between the packing plates and the angle, a friction coefficient equal to 0,2 is applied; this value is considered as a lower bound. Nonetheless, as the bolts are not preloaded, the value of the friction coefficient has been shown to have very low influence [12]. In addition, a clearance of 2 mm in bolt holes has been modelled, in order to account for the sliding before contact. The boundary conditions applied in the numerical model are represented schematically in Figure 7(b). It should be noted that the rotations about the axis y (noted $w_{,x}$ in Figure 7(b)) and z (noted $v_{,x}$ in Figure 7(b)) are free, while the torsional rotations are restrained.

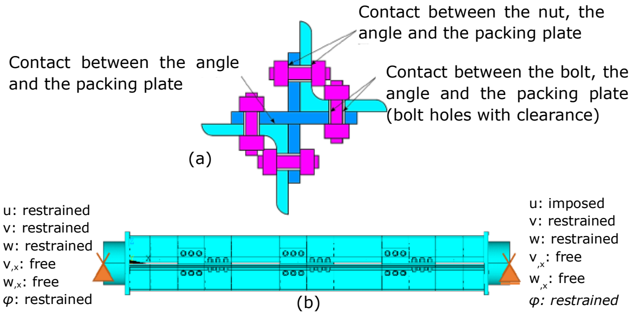


**Figure 7** (a) Contact regions in the built-up section and (b) schematic representation of the support conditions of the numerical model

A full non-linear analysis has been performed increasing the longitudinal displacement *u* on one of the supports up to failure, so as to determine the failure load of the member. The comparison between the curves of the load versus the axial shortening of the specimen as obtained in laboratory (orange solid line) and through the numerical analysis (blue solid line) is shown in Figure 8. Despite that both ultimate axial forces are quite similar, the stiffness and the value of the displacement at which the ultimate axial force is reached differ. In order to further analyse the difference between laboratory test and numerical simulation, the axial strains have been investigated. First, the strain at which the ultimate axial force is reached should be close to the elastic one $\varepsilon_y = f_y/E = 0{,}21$ %. The value of this strain may also be determined approximately based on the axial shortening of the member that is $\delta/L$=7,7/3384 =0,23% for the numerical simulation and $\delta/L$=13/3384 =0,38% for the laboratory test (see Figure 8). A final comparison is given in Figure 8 where the load-displacement experimental curve has been back calculated from the mean strain values at the three locations along the member (i.e. axial displacements 1, 2 and 3 are based on mean axial strains at ¼L, ½L and ¾L respectively). The obtained estimation of the evolution of axial displacements with the applied loads seems to confirm the initial stiffness of the numerical model.

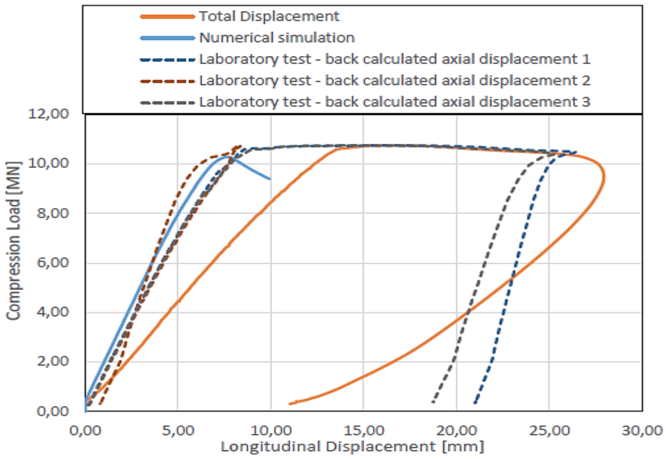


**Figure 8** Load displacement curves with back calculated axial displacements

Finally, it has been observed that the numerical sample fails due to a global flexural buckling with the presence of a slight local buckling of the legs (outstand flange). Indeed, the local displacements along a leg were low compared to the mid-span displacements of the member. However, the mid-span displacements themselves remain low at failure. As the member is rather stocky, flexural buckling is not very significant (relative slenderness is low and strength reduction due to instability is low as well). In conclusion, the numerical model gave a value of the ultimate resistance to the axial force equal to 10,28 MN which is about 4% lower than the laboratory test result. Furthermore, in both cases, the failure mode of the built-up member seems to be a combination of the global buckling of the member with a contribution of local buckling of the flanges. Globally, one may however conclude that the numerical simulation is very close to the result of the single laboratory test that has been carried out.

## 7 Conclusions

Two main European normative documents cover nowadays the design of star-battened members in compression: EN 50341:2012 and EN 1993-3-1:2005. Through a careful analysis of these documents, some lacks, contradictions and inconsistencies have been revealed, in particular – but not exclusively – for their application to S460. On the contrary, in a recent European project entitled ANGELHY, a particular attention has been paid to S460 where a base set of formulae has been proposed and has been recently implemented in the new forthcoming version of EN 1993-3-1:2005, named prEN 1993-3:2021.
A star-battened member has been considered as a case study and its resistance has been evaluated analytically, experimentally and numerically. The application of prEN

1993-3:2021 to the studied member has been achieved and the results have been compared with those provided by EN 50341:2012 and EN 1993-3-1:2005. EN 1993-3-1 looks to be the most conservative, while EN 50341 predicts quite well the experimental resistance; prEN 1993-3 gives intermediate values. The estimation provided by EN 50341 is also quite precise but remains questionable as long as EN 50341 has never been validated for S460 steel. To validate the normative analytical approaches, a buckling test has been performed. Its ultimate experimental resistance was equal to 10,75 MN and the member fails due to a combination of global member buckling and local buckling of the legs of the angle profiles. It has also been seen that, for the specimen considered in the case study, EN50341 Annex J.4 cannot be applied as the test resistance is not higher than 5% of the analytical one and so further calculations are required to satisfy this 5% criterion.
Finally, a full non-linear finite element model with volume elements has been analysed to simulate the response of the member in compression until failure. The numerical ultimate resistance equals 10,28 MN, which is about 4% lower than the laboratory test result. In both cases, the failure mode seems to be a combination of global and local buckling.

**References**


[1] EN 1993-3-1: Design of steel structures - Part 3-1: *Towers, musts and chimneys. Tower and musts*, Brussels, Comité Européen de Normalisation, 2005.

[2] EN 1993-1-1: Design of steel structures - Part 1-1: *General rules and rules for buildings*, Brussels, Comité Européen de Normalisation, 2005.

[3] EN 1993-1-5: Design of steel structures - Part 1-5: *Plate structural elements*, Brussels, Comité 000.Europeen de Normalisation, 2006.

[4] EN 1993-1-8: Design of steel structures - Part 1-8: *Design of joints*, Brussels, Comité Européen de Normalisation, 2005.

[5] EN 50341-1: Overhead electrical lines exceeding AC 1 kV - Part 1: *General requirements - Common specifications*, 2012.

[6] Vayas, I.; Jaspart, J.P.; Bureau, A.; Tibolt, M.; Reygner, S.; Papavasiliou, M. (2021) *Telecommunication and transmission lattice towers from angle sections – the ANGELHY project*, Ernst & Sohn, ce/papers, Vol. 4, Issue 2 – 4, pp 210-217.

[7] Bezas, M.-Z. (2021) *Design of lattice towers from hot-rolled equal leg steel angles*, PhD thesis, http://hdl.handle.net/2268/262364, University of Liège & National Technical University of Athens.

[8] prEN 1993-3: Design of steel structures - Part 3: *Towers, musts and chimneys*, Brussels, Comité Européen de Normalisation, 2021.

[9] Schillo, N.; Feldmann, M. (2015) *Buckling resistance of L-profiles in towers, masts and open line Constructions,* Stahlbau 84, Heft 12, 946-954.

[10] Bezas, M.Z.; Demonceau, J.F.; Vayas, I.; Jaspart, J.P. (2021) *Classification and cross-section resistance of equal-leg rolled angle profiles*, Journal of Constructional Steel Research, Vol. 185, 106842.

[11] Bezas, M.Z.; Demonceau, J.F.; Vayas, I.; Jaspart, J.P. (2022) *Design rules for equal-leg angle members subjected to compression and bending*. Journal of Constructional Steel Research, Vol. 189, 107092.

[12] Beyer, A.; Bureau, A.; Jaspart, J.P.; (2021) *Buckling resistance of compression members with back-to-back connected angle sections*, Ernst & Sohn, ce/papers, Vol. 4, Issue 2 – 4, pp. 2132–2139.

[13] Saufnay, L.; Beyer, A.; Jaspart, J.P.; Demonceau, J.F.; (2023) *Experimental and numerical investigations on closely spaced built-up members*. J. Struct. Eng, Vol. 149, Issue 4.

[14] Botelho, I.S.; da Silva Vellasco, P.C.G.; de Lima, L.R.O.; Rodrigues, M.C.; da Silva, A.T.; (2019) *An assessment of starred rolled stainless steel angle columns*. 9th ICSAS.

[15] Lu, C.; Ma, X.; Mills, J.E.; (2014) *The structural effect of bolted splices on retrofitted transmission tower angle members*. Journal of Constructional Steel Research, Vol. 95, pp. 263-278.

[16] Ansys® Academic Research Mechanical, Release 18.2.

[17] NBN EN 50341-2-2: Overhead electrical lines exceeding AC 1 kV - Part 2-2: *National Normative Aspects (NNA) for Belgium*, 2019.

[18] NBN EN 1993-3-1 ANB: Design of steel structures - Part 3-1: Towers, masts and chimneys. Tower and musts, Belgian National Annex, 2011.

[19] Beyer, A.; Delacourt, G.; Bureau, A. (2020) *Deliverable 3.4: Development of design rules for closely spaced built-up angle sections*. Research Report-ANGELHY project, CTICM France.

[20] EN 10025-2: Hot rolled products of structural steels - Part 2: *Technical delivery conditions for non-alloy structural steels*, Comité Européen de Normalisation, 2019.

[21] NBN EN 1993-1-1 ANB: Design of steel structures - Part 1-1: General rules and rules for buildings, Brussels, Belgian National Annex, 2018.

[22] Wittenberghe, J. (2022) *Buckling Test on Transmission Pole for AM R&D Esch ocas*, New steel project report, ArcelorMittal, Belgium.

[23] Bezas, M.Z.; Demonceau, J.F.; Jaspart, J.P. (2022) *Measurements of residual stresses and characterisation of material properties*, New steel project report, Universite de Liege, Belgium.