NATIONAL TECHNICAL UNIVERSITY OF ATHENS

LIÈGE université
Sciences Appliquées

Design of lattice towers from hot-rolled equal leg steel angles

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Introduction - State of the art

Which is the relevant Code that should I use?

EN 1993-1-1

EN 1993-1-5

EN 1993-3-1

EN 50341

Various codes and norms that may be used for the design of angles!

Let's classify a typical profile L70x70x6 with S355 in pure compression!

c/ εt = (70-6-9)/6ε = 11,26

Normative document	Limit for class 3 to 4	Class
EN 1993-1-1 (Table 5.2 Sheet 3)	c/εt ≤ 9,2	4
EN 1993-1-1 (Table 5.2 Sheet 2)	c/εt ≤ 14	1
EN 1993-1-5	c/εt ≤ 11,1	4
EN 1993-3-1	c/εt ≤ 13,9	1
EN 50341	c/εt ≤ 13,9	1

Inconsistencies between the codes!

Angles sections are common profiles! Not?

(i) open profiles with very small section constants in torsion and warping;

(ii) monosymmetrical sections;

(iii) bending capacity and radius of gyration around the weak axis are substantially lower than around strong axis;

(iv) their legs are prone to local buckling as external plate elements;

(v) higher plastic resistances than their elastic ones;

(vi) due to eccentric connection in one leg, they are subjected also to bending in addition to axial force.

Existing design rules have been mainly developed for doubly symmetric sections!

There is a need of a full consistent set of formulae to cover the design of angles!!

Objectives of the dissertation

Develop **Rules** and **Recommendations** for the design of lattice towers

- Classification and <u>cross-section</u> resistance
- Resistance and stability of <u>members</u>
- Global structural analysis of <u>towers</u>

- Numerical studies
- Analytical developments
- Laboratory tests

Classification of equal leg angles profiles

Objective:

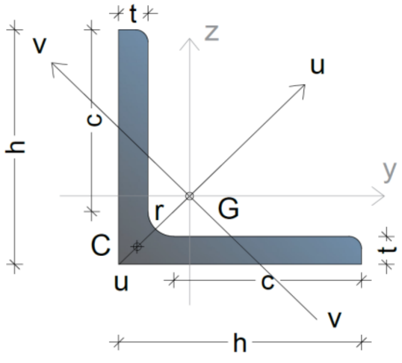
- Develop-validate classification criteria for angle cross-sections in:
  - Compression
  - Strong axis bending M_u
  - Weak axis bending M_v

Why?

- Various normative documents (compression) → sometimes in contradiction.
- Considering outstand plate elements or associated local buckling with tortional one.
- For bending → only EN 1993-1-1 (2/2).
- References to different ratios: **h/t** or **c/t** or **(h-2t)/t**.

How?

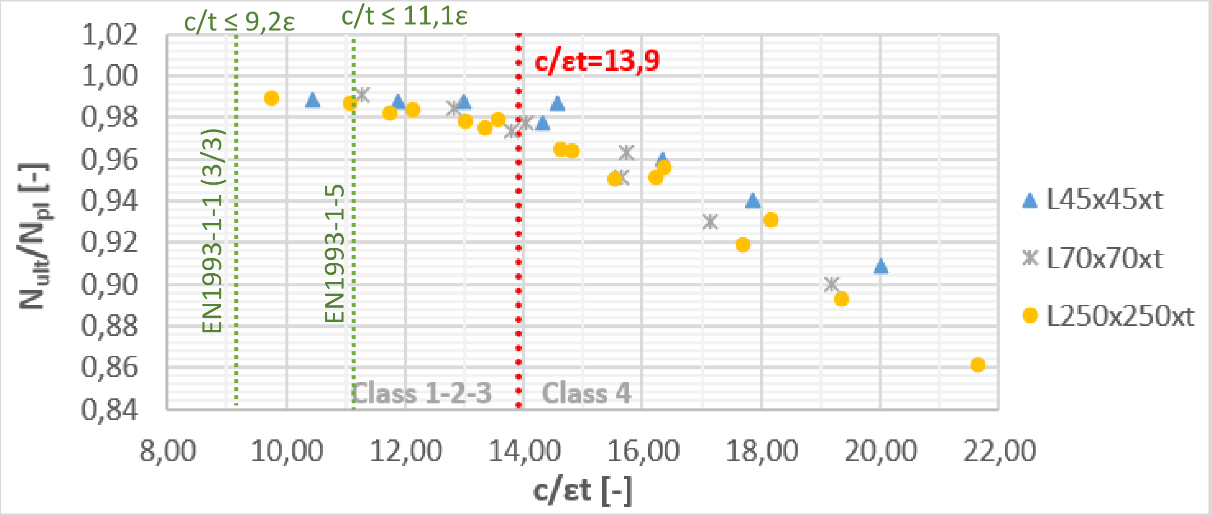
- Numerical studies to define **c/t** ratios for the corresponding loadings.
- Analytical developments → validation of the numerical results.



Normative document	Limit for class 3 to 4	Class
EN 1993-1-1 (Table 5.2 Sheet 3)	$c/\varepsilon t \leq 9{,}2$	4
EN 1993-1-1 (Table 5.2 Sheet 2)	$c/\varepsilon t \leq 14$	1
EN 1993-1-5	$c/\varepsilon t \leq 11{,}1$	4
EN 1993-3-1	$c/\varepsilon t \leq 13{,}9$	1
EN 50341	$c/\varepsilon t \leq 13{,}9$	1

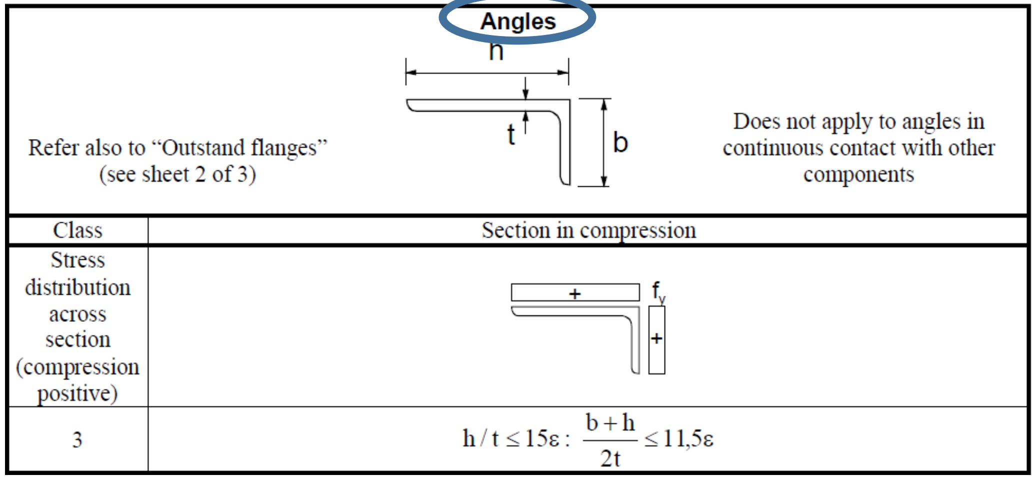
Classification of equal leg angles profiles

Classification to compression



EN 1993-1-1: 2005 (E)

Table 5.2 (sheet 3 of 3): Maximum width-to-thickness ratios for compression parts



	Angles
Refer also to "Outstand flanges" (see sheet 2 of 3)	Does not apply to angles in continuous contact with other components
Class	Section in compression
Stress distribution across section (compression positive)	
3	$h/t \leq 15\varepsilon : \frac{b+h}{2t} \leq 11{,}5\varepsilon$

This condition (**c/t ≤ 13,9ε**) is less conservative than EN 1993-1-1 (3/3) or EN 1993-1-5 (h/t), and is in line with :

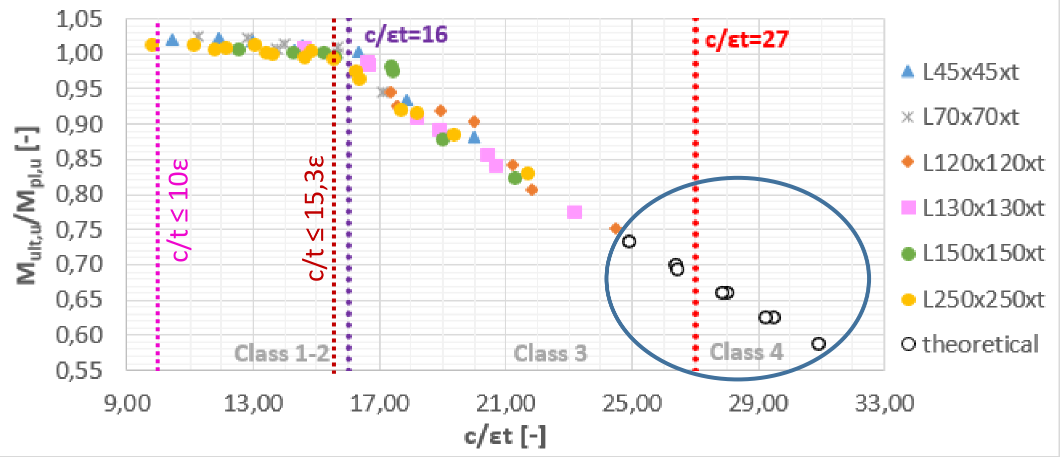
- EN 1993-1-1 (2/3) for outstand elements (c/t ≤ 14ε);
- EN 50341 (c/t ≤ 13,9ε);
- EN 1993-1-5, in which $\bar{b} = c$ instead of h (c/t ≤ 13,9ε);
- EN 1993-3-1 (c/t ≤ 13,9ε), in which the (h-2t)/t is suggested but is not so far from the exact value c=h-t-r.

$$\rho = 1{,}0 \rightarrow \bar{\lambda}_p = \frac{\bar{b}/t}{28{,}4\varepsilon\sqrt{k_\sigma}} = 0{,}748 \rightarrow \frac{h/t}{\varepsilon} = 13{,}9$$

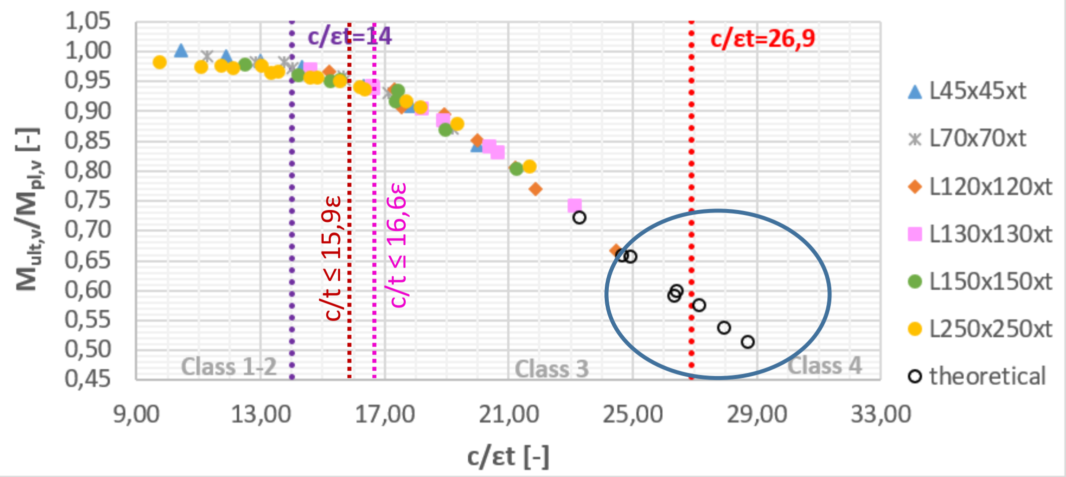
Classification of equal leg angles profiles

Classification to strong and weak axis bending

Strong axis



Weak axis



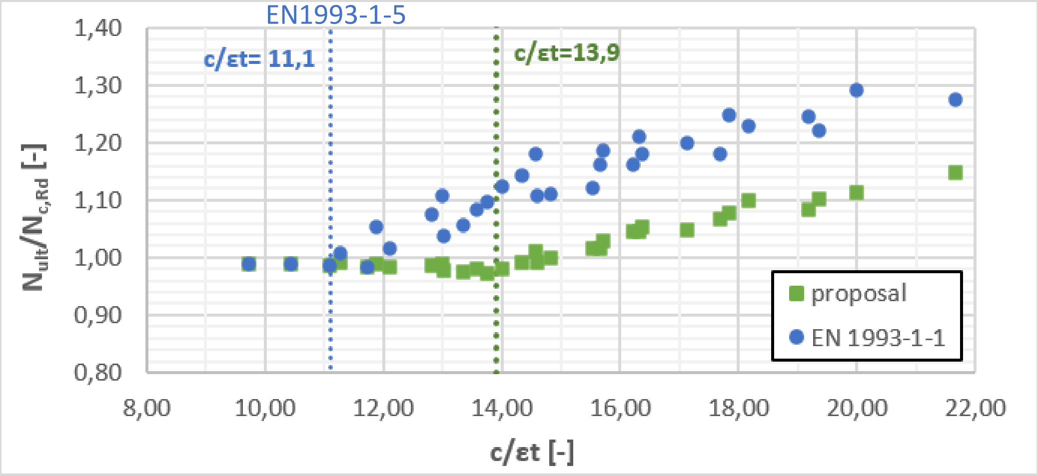
- Rules can be found only in EN 1993-1-1 → leg is an outstand plate element simply supported.
- The tension leg has enough stiffness to restrain the compression one.
- Plate stability factor (k_σ) may be improved → a higher value has been adopted, considering clamped support conditions → fits well with the numerical results.

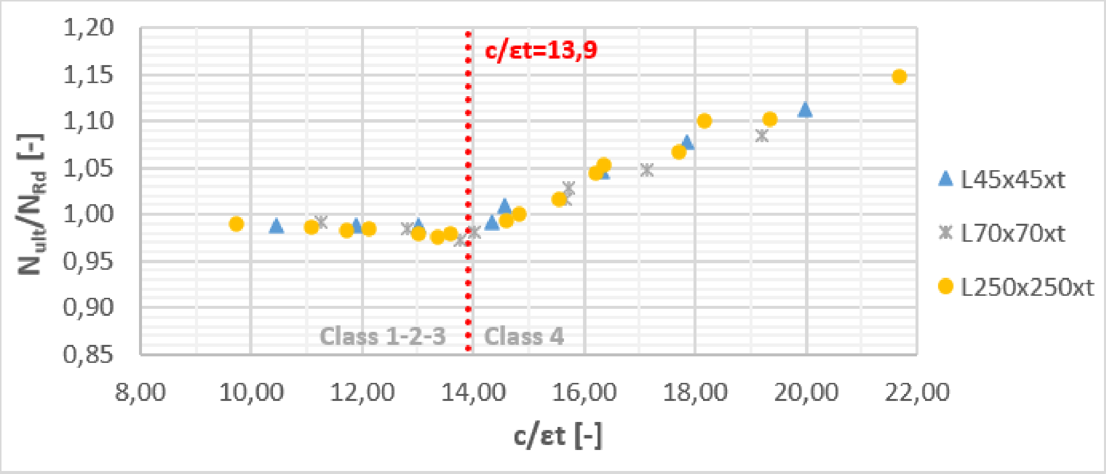
- **The limits proposed by EC3 are in contradiction!**

Design resistances of angle cross-sections

1. Axial compression

- The design resistance is following the provisions of EN 1993-1-1 (classes 1-3) and EN 1993-1-5 (class 4).
- The plate buckling slenderness may be given by: $\bar{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{cr}}} = \frac{c/t}{28{,}4\varepsilon\sqrt{0{,}43}} = \frac{c/t}{18{,}6\varepsilon}$





- The response is not influenced by the cross-section size.
- For stocky class 1 to 3 legs → small overestimation of, but is counterbalanced by strain hardening effect that is not considered in the FEM.
- Class-4 sections → the proposed rules are largely on the safe side.

Design resistances of angle cross-sections

2. Strong axis bending Mu

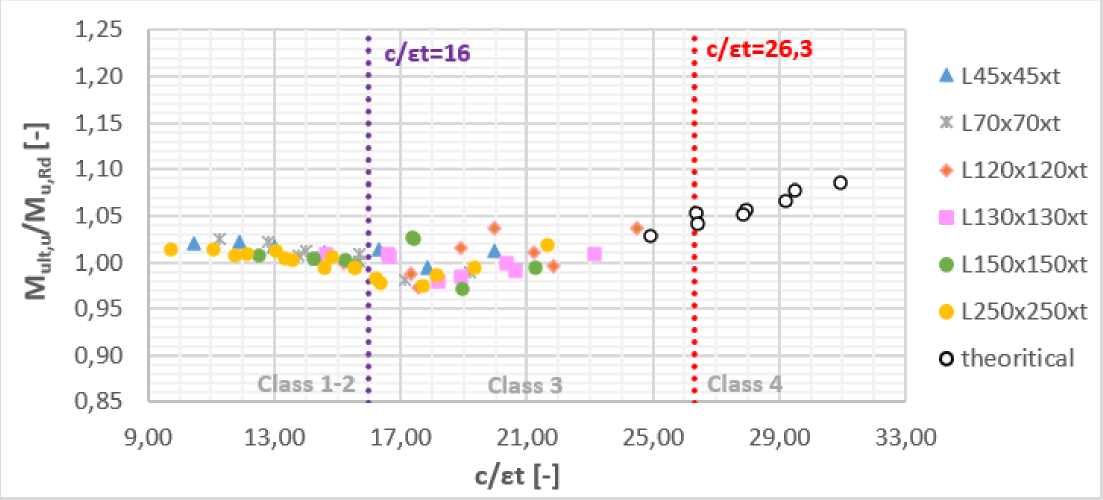
- The design resistance may be determined from: $M_{u,Rd} = W_u \frac{f_y}{\gamma_{M0}}$

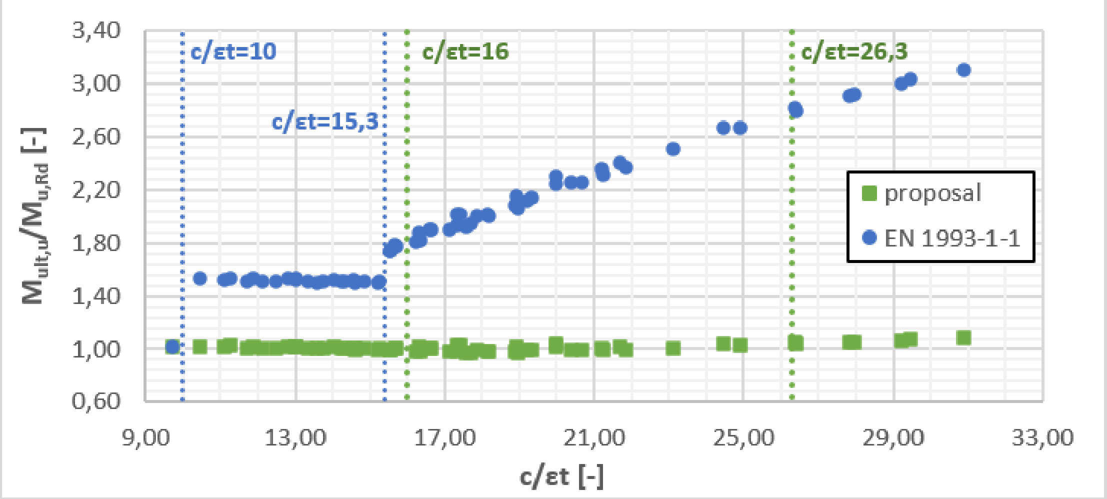
  where $W_u = \alpha_{i,u} W_{el,u}$ is the section modulus about u axis:

  $\alpha_{2,u} = 1{,}5$ for class 1 or 2

  $\alpha_{3,u} = \left[1 + \left(\frac{26{,}3\varepsilon - c/t}{26{,}3\varepsilon - 16\varepsilon}\right) \cdot (1{,}5 - 1)\right]$ for class 3

  $\alpha_{4,u} = W_{eff,u} / W_{el,u}$ for class 4





- The response is not influenced by the cross-section size.
- Class 1 and 2 legs → proposed formulae predict almost exactly the resistance.
- Class-3 sections → small overestimation of resistance in a very limited range of $c/\varepsilon t$-ratios.
- Class-4 sections → proposed rules are always on the safe side.

Design resistances of angle cross-sections

3. Strong axis bending Mv – tip in compression

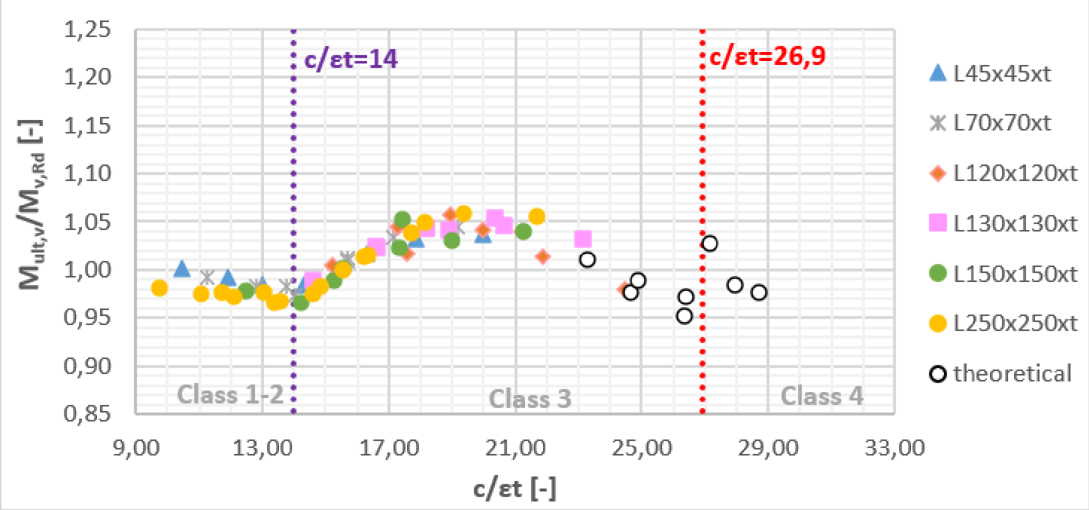
- The design resistance may be determined from: $M_{v,Rd} = W_v \frac{f_y}{\gamma_{M0}}$

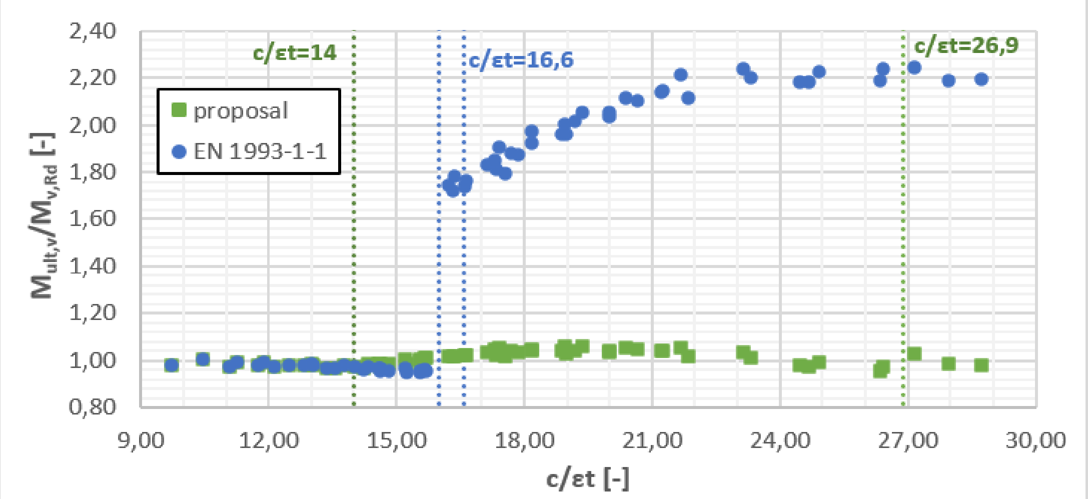
  where $W_v = \alpha_{i,v} W_{el,v}$ is the section modulus about v axis:

  $\alpha_{2,v} = W_{pl,v} / W_{el,v}$ for class 1 or 2

  $\alpha_{3,v} = \left[1 + \left(\frac{26{,}9\varepsilon - c/t}{26{,}9\varepsilon - 14\varepsilon}\right) \cdot (\alpha_{2,v} - 1)\right]$ for class 3

  $\alpha_{4,v} = W_{eff,v} / W_{el,v}$ for class 4

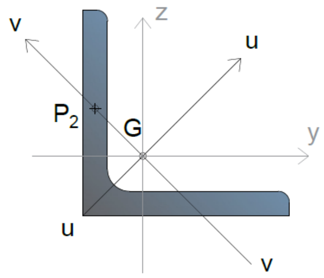




- The response is not influenced by the cross-section size.
- Class 1 and 2, entering even in class-3 → a small overestimation of resistance → counterbalanced by strain hardening effect.
- This is also observed for large *c/εt*-ratios in the border between class 3 and 4.

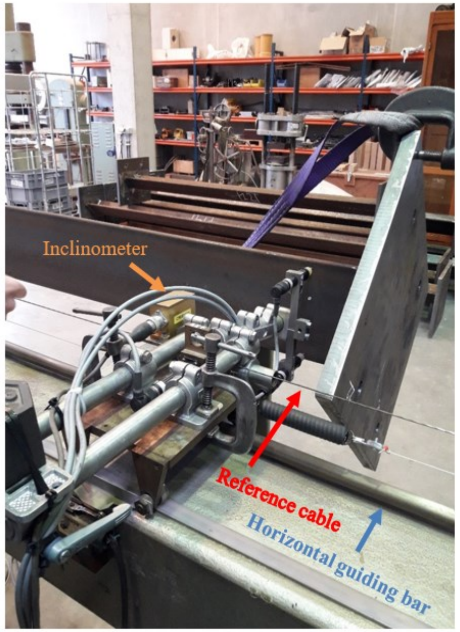
Compression tests on HSS angle columns

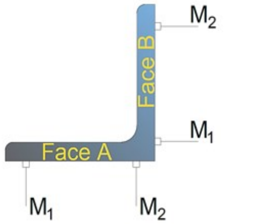
ID of Specimen	Profile	Steel grade	Length of angle member L [mm]	Eccentricity [mm]
Sp11	L 150x150x18	S460M	2500	0,00
Sp12	L 150x150x18	S460M	2500	$e_v = 48{,}74$
Sp13	L 150x150x18	S460M	3000	0,00
Sp14	L 150x150x18	S460M	3000	$e_v = 48{,}74$
Sp15	L 150x150x18	S460M	3500	0,00
Sp16	L 150x150x18	S460M	3500	$e_v = 48{,}74$
Sp21	L 200x200x16	S460M	3000	0,00
Sp22	L 200x200x16	S460M	3000	$e_v = 66{,}64$
Sp23	L 200x200x16	S460M	3500	0,00
Sp24	L 200x200x16	S460M	3500	$e_v = 66{,}64$
Sp25	L 200x200x16	S460M	4000	0,00
Sp26	L 200x200x16	S460M	4000	$e_v = 66{,}64$

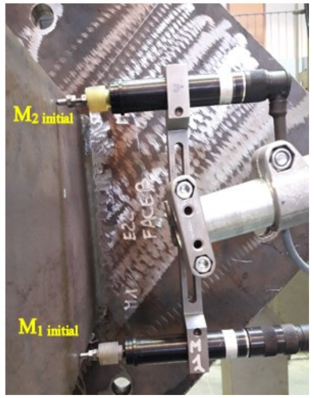


Test measurements:

- Actual dimensions of the cross-sections (h_1, h_2, t_1, t_2, e_v, L)
- Initial geometrical imperfections
- Coupon tests for the material properties

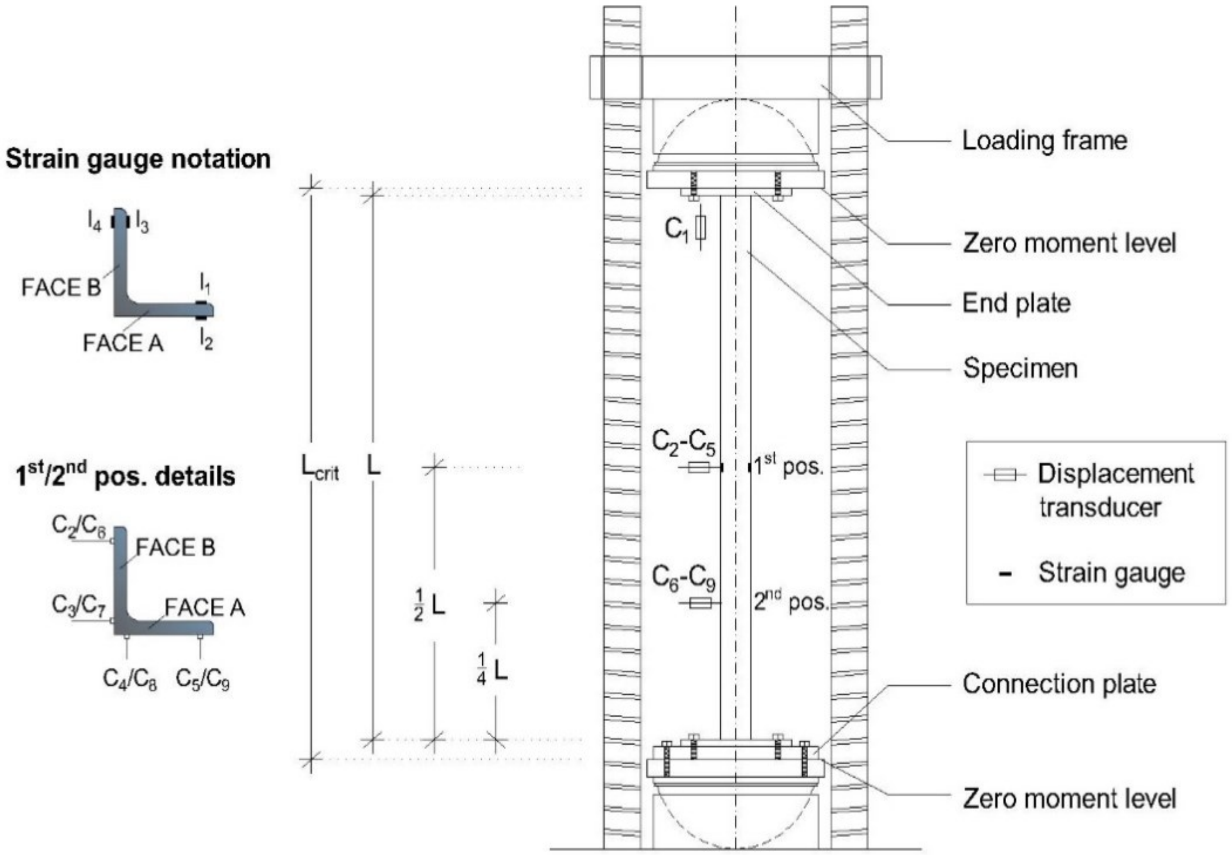




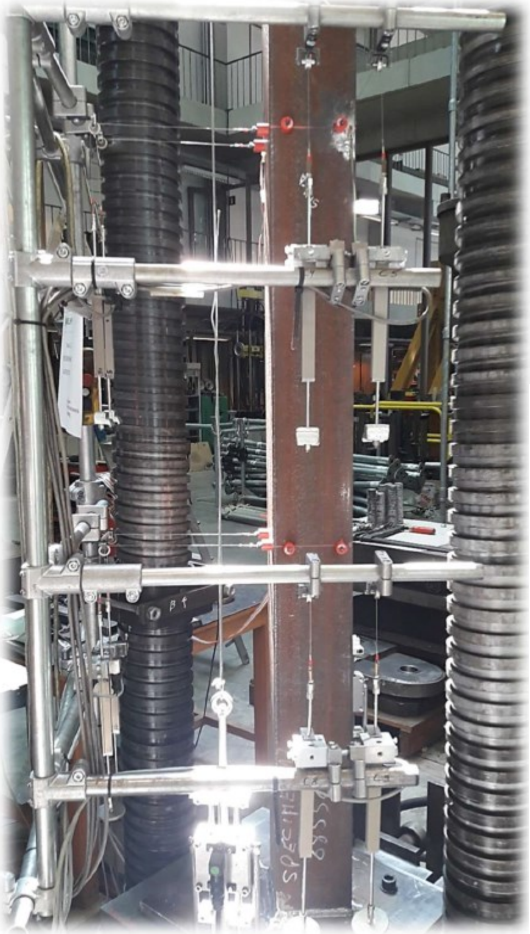
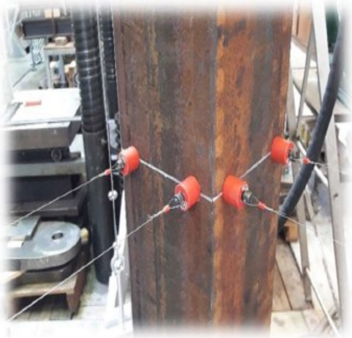


ID of material	E [MPa]	Measured yield stress f_y [MPa]	Measured ultimate stress f_{ult} [MPa]	Measured strain at failure [%]	Nominal yield stress $f_{y,nom}$ [MPa]	$f_y/f_{y,nom}$ [-]	Characterized specimens
S 460/1	203155	425,8	572,50	14,3	460,0	0,93	Sp12, Sp13, Sp14, Sp15, Sp16
S 460/2	208947	487,6	604,64	13,7	460,0	1,06	Sp21, Sp22, Sp23, Sp25, Sp26
S 460/3	197317	417,2	560,87	14,3	460,0	0,91	Sp11
S 460/4	203797	472,6	587,21	13,8	460,0	1,03	Sp24

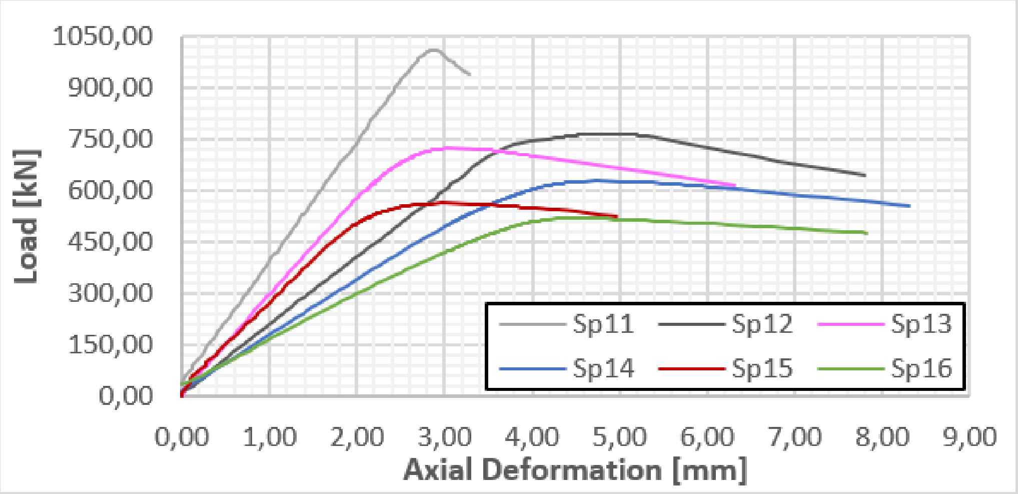
Compression tests on HSS angle columns

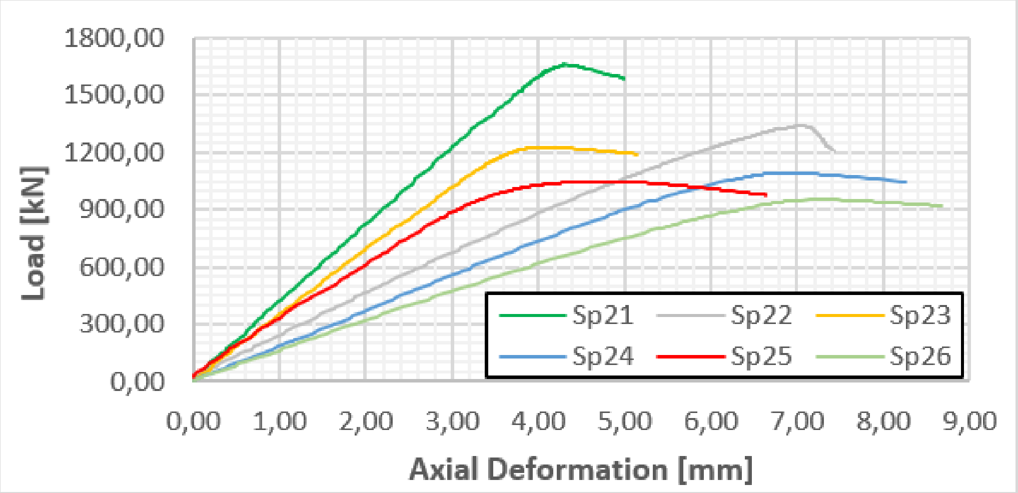


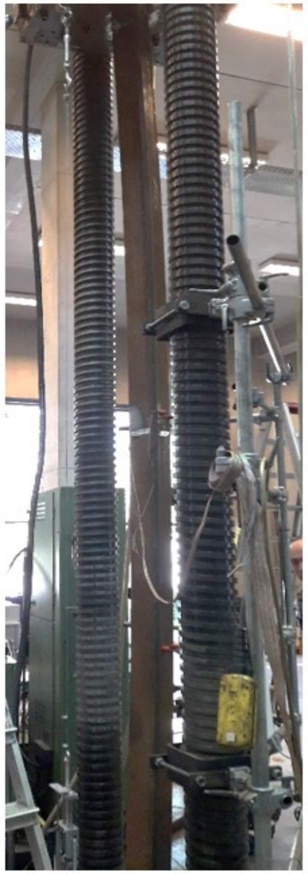
Amsler 500 testing machine, compression load up to 5000 kN

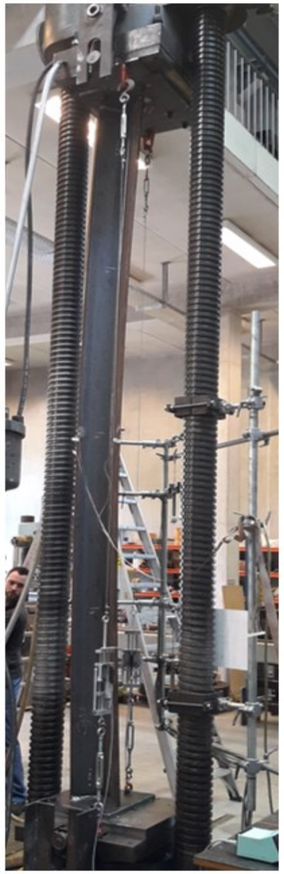




Compression tests on HSS angle columns





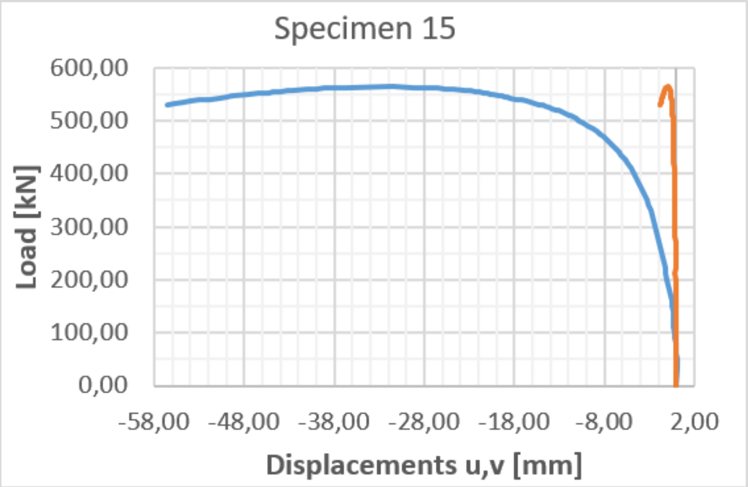


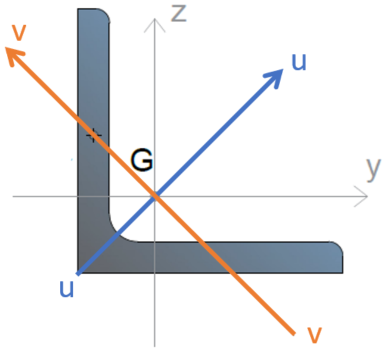


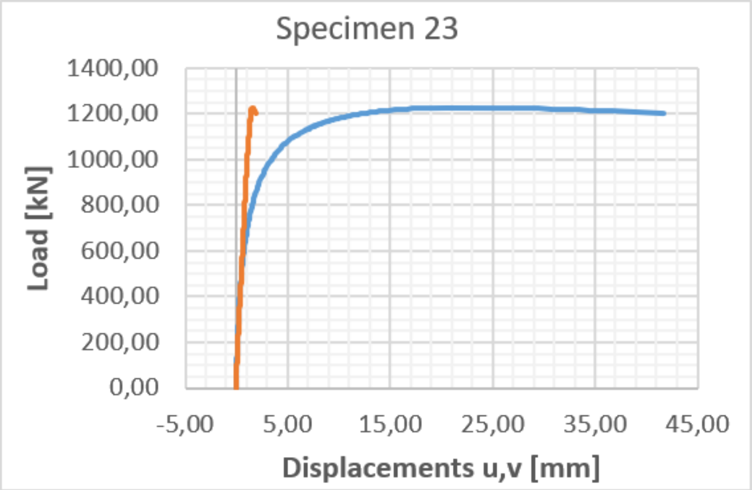
The tests results are in line with the physical expectations -> influence of the member length or the load eccentricity on the member stiffness and resistance.

Compression tests on HSS angle columns

- Specimens without nominal eccentricity: some showed nearly zero deflections along v-v axis and some very small ones → limited unintentional eccentricity resulting from installation tolerances.







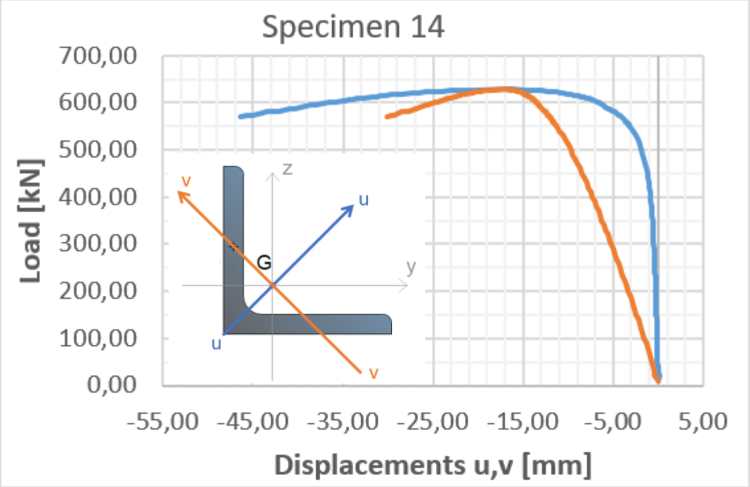
- Centrally loaded tests → deflections along u-u axis increased significantly until failure be reached by <u>weak axis buckling</u>.

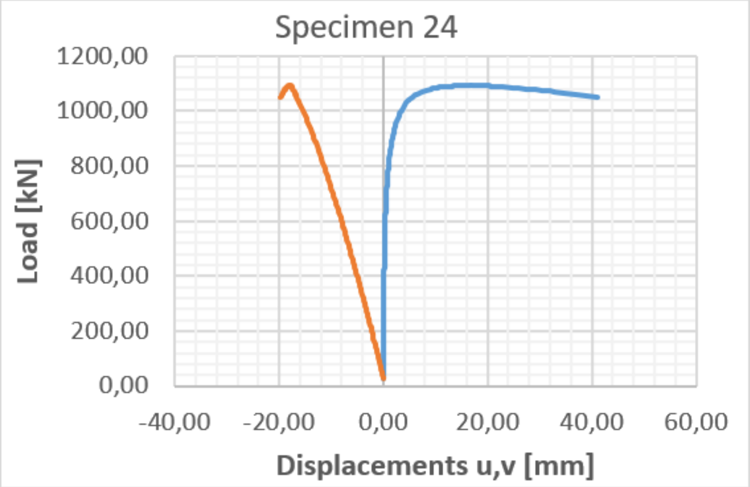
- Sp11, Sp13, Sp15 failed in a <u>pure flexural buckling mode.</u>

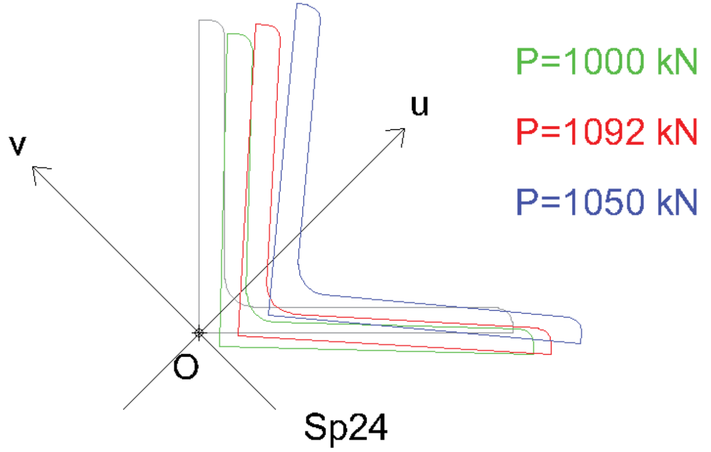
- Sp21, Sp23, Sp25 in a <u>flexural-torsional buckling</u> mode (twist rotations and weak axis deflections).

Compression tests on HSS angle columns

- Eccentrically loaded specimens → $N+M_u$.
- Low loading levels → deflections along v-v axis were high but very small in u-u.
- Higher loading levels → deflections along u-u axis grew quickly and prevailed at failure.





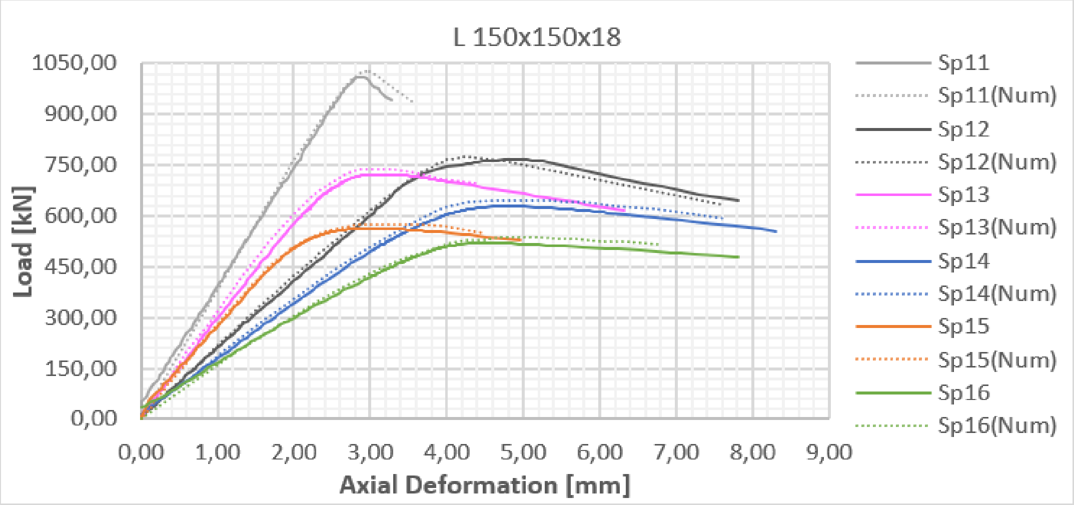


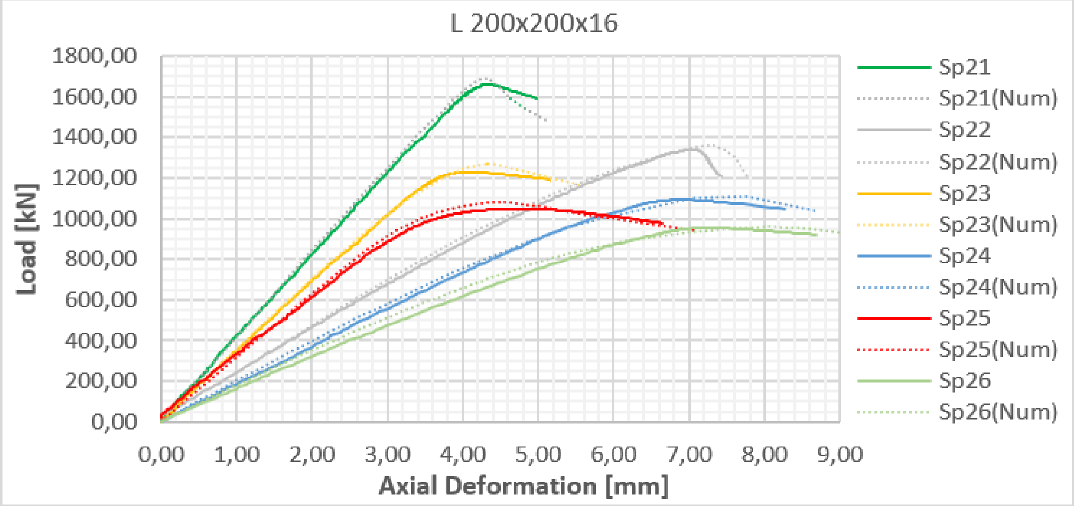
- Sp22, Sp24, Sp26 failed in a <u>flexural-torsional buckling mode</u>, while Sp12, Sp14, Sp16 failed in a <u>mixed mode between flexural and flexural torsional buckling</u>.

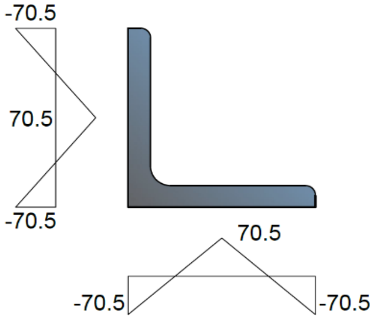
- Local buckling was not visibly observed in any specimen although Sp2# specimens are categorized as class 4.

Compression tests on HSS angle columns

- Comparison of the **stiffness** and the ultimate **resistance** of the members through FEM analyses (FINELG).
- GMNIA analyses were performed, considering (i) initial member imperfections, (ii) residual stresses, (iii) actual material properties.







ID of Specimen	N_{exp}/N_{FEM}
Sp11	0,98
Sp12	0,99
Sp13	0,98
Sp14	0,97
Sp15	0,98
Sp16	0,97
Sp21	0,98
Sp22	0,99
Sp23	0,97
Sp24	0,99
Sp25	0,97
Sp26	0,99

Design rules for members made of angles

Objective:

- Develop efficient design rules for angle members in:
  - Axial compression
  - Strong axis bending M_u
  - Weak axis bending M_v (the member resistance coincides with the cross-section resistance)
  - Combined axial force and bi-axial bending

Why?

- Most of the existing rules have been developed for doubly symmetric sections → the proposed rules are adapted for angle sections.
- To remove existing inconsistencies of the codes and “clear” the design process.

How?

- Numerical studies and experimental test → validation of the design rules.

Design rules for members made of angles

1. Buckling Resistance to compression

- The design resistance may be determined from:

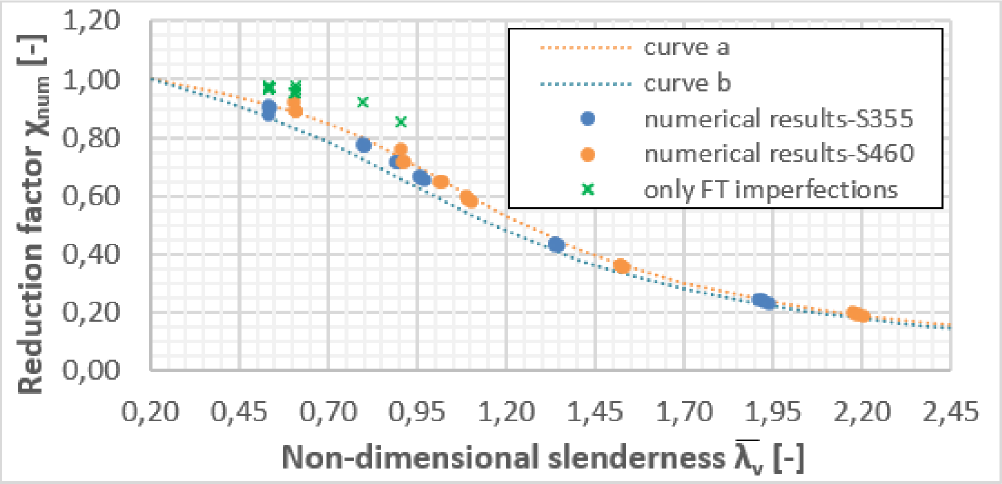
$$N_{b,Rd} = \begin{cases} \chi_{min} \dfrac{Af_y}{\gamma_{M1}} & \text{for class 1,2 and 3 profiles} \\ \chi_{min} \dfrac{A_{eff} f_y}{\gamma_{M1}} & \text{for class 4 profiles} \end{cases}$$

where $\chi_{min} = min\{\chi_u; \chi_v\}$ and $A_{eff} = A - 2ct(1-\rho)$

- The buckling reduction factor χ_{min} is determined as a function of the relative slenderness $\overline{\lambda_u} = \sqrt{\dfrac{Af_y}{N_{cr,u}}}$ and $\overline{\lambda_v} = \sqrt{\dfrac{Af_y}{N_{cr,v}}}$ of the compression member for the flexural buckling modes only.

- Buckling curve b for steel grades S235-S420, and buckling curve a for higher steel grades (≥ S460) have been selected.

- The plate buckling slenderness may be given by: $\bar{\lambda}_p = \sqrt{\chi_{min}} \dfrac{c/t}{18,6\varepsilon}$



- Class 1 and Class 4 profiles
- Flex and Flex-Tor relevant buckling modes

$$\chi_{num} = N_{ult}/N_{pl}$$

For samples with FT eigenmode:
(i) numerical results reported with blue/orange points have been evaluated using $N_{ult} = N_{ult,\,F,\,imp}$
(ii) results presented with green points using $N_{ult} = N_{ult,\,FT,\,imp}$.

Design rules for members made of angles

2. Lateral torsional buckling resistance to strong axis bending

- The design resistance may be determined from: $M_{u,Rd} = \chi_{LT} W_u \frac{f_y}{\gamma_{M0}}$

  where $W_u = \alpha_{i,u} W_{el,u}$ and $\alpha_{i,u}$ as defined for the CS resistance

- The plate buckling slenderness may be given by: $\bar{\lambda}_p = \sqrt{\chi_{LT}} \frac{c/t}{35{,}6\varepsilon}$

- The factor χ_{LT} should be determined as a function of the relative slenderness of the member: $\overline{\lambda_{LT}} = \sqrt{\frac{W_u f_y}{M_{cr}}}$ where $M_{cr} = C_b \frac{0{,}46 \cdot E \cdot h^2 \cdot t^2}{l}$

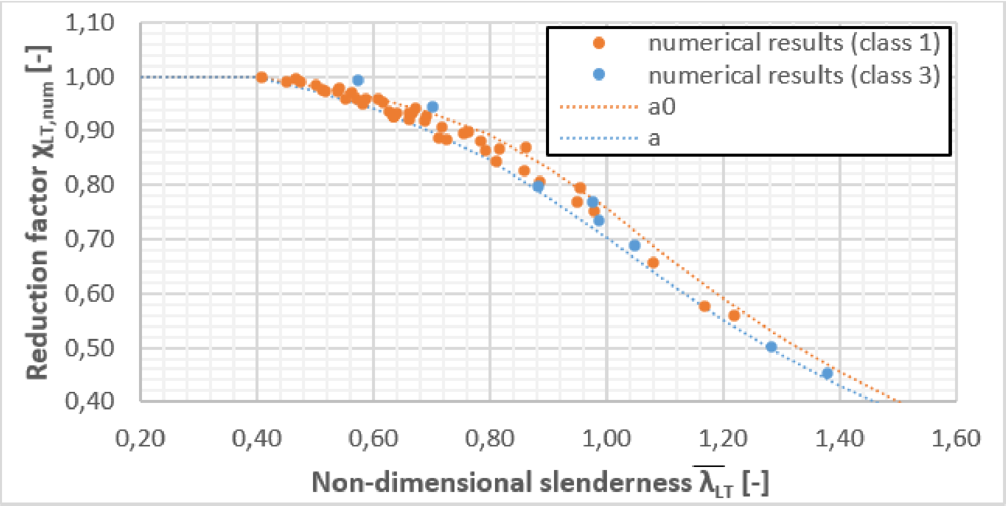
- Should be derived from buckling curve a, using following equations

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \bar{\lambda}_{LT}^2}} \quad \text{but} \begin{cases} \chi_{LT} \leq 1{,}0 \\ \chi_{LT} \leq 1/\bar{\lambda}_{LT}^2 \end{cases}$$

$$\Phi_{LT} = 0{,}5\left[1 + a_{LT}\left(\bar{\lambda}_{LT} - 0{,}4\right) + \bar{\lambda}_{LT}^2\right]$$

- LTB may be ignored when one of the following conditions apply:

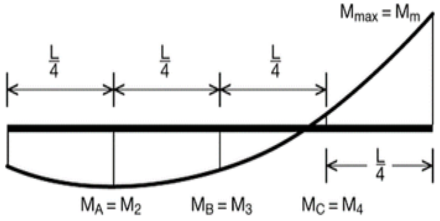
$$\bar{\lambda}_{LT} \leq \bar{\lambda}_{LT,0} \ \text{(where } \bar{\lambda}_{LT,0} = 0{,}4\text{)} \quad \text{or} \quad \frac{M_{Ed}}{M_{cr}} \leq \bar{\lambda}_{LT,0}^{\ 2}$$



The mean value $M_{cr,numerical}/M_{cr,analytical}$ is about 0,99

General case:

$$C_b = \frac{12{,}5 M_{max}}{2{,}5 M_{max} + 3M_A + 4M_B + 3M_C} \leq 1{,}5$$



For linear moment distribution:

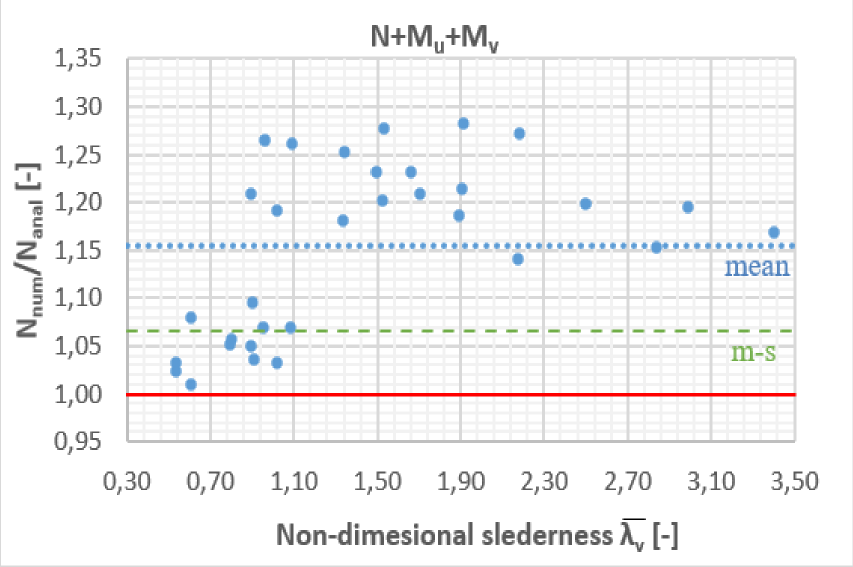
$$C_b = \frac{12{,}5}{7{,}5 + 5\psi} \quad \text{with} \quad -1 \leq \psi = \frac{M_2}{M_1} \leq 1$$

Design rules for members made of angles

3. Buckling resistance to bending and axial compression

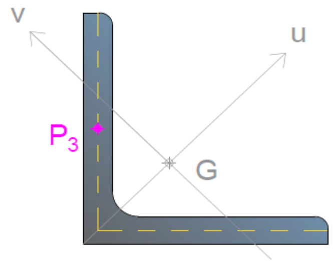
- Two conditions for buckling around one or the other principal axis should be satisfied for angle members subjected to N+M_u+M_v. Torsional buckling is included in the local buckling check.

  - strong axis check : $\left(\frac{N_{Ed}}{N_{bu,Rd}} + k_{uu}\frac{M_{u,Ed}}{M_{u,Rd}}\right)^{\xi} + k_{uv}\frac{M_{v,Ed}}{M_{v,Rd}} \leq 1$
  - weak axis check: $\left(\frac{N_{Ed}}{N_{bv,Rd}} + k_{vu}\frac{M_{u,Ed}}{M_{u,Rd}}\right)^{\xi} + k_{vv}\frac{M_{v,Ed}}{M_{v,Rd}} \leq 1$



4. Buckling resistance N+M – General method

- The general method (EN1993-1-1) has been adapted through numerical and experimental validations to fit better with the response of angle members → tendency of angles to buckle along weak axis;

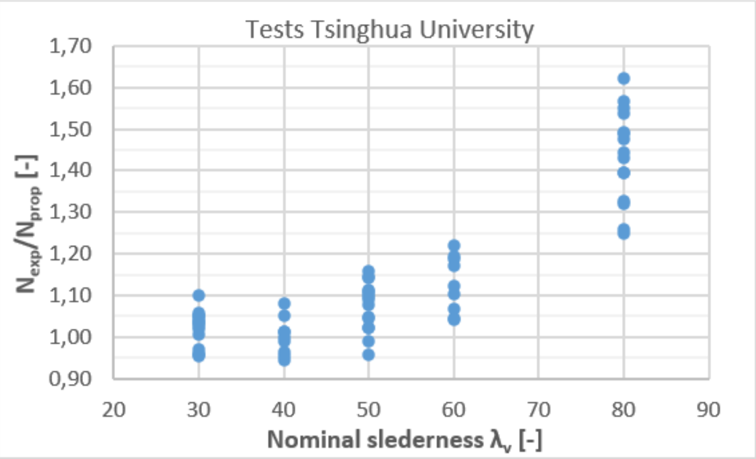


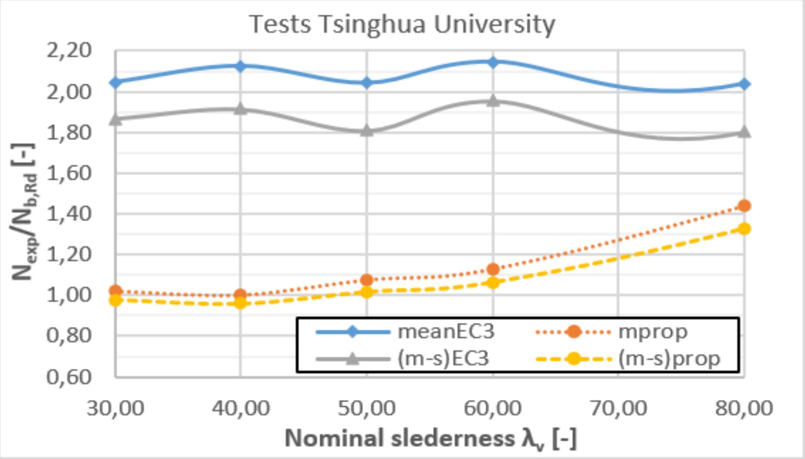
Design rules for members made of angles

EN1993-1-1

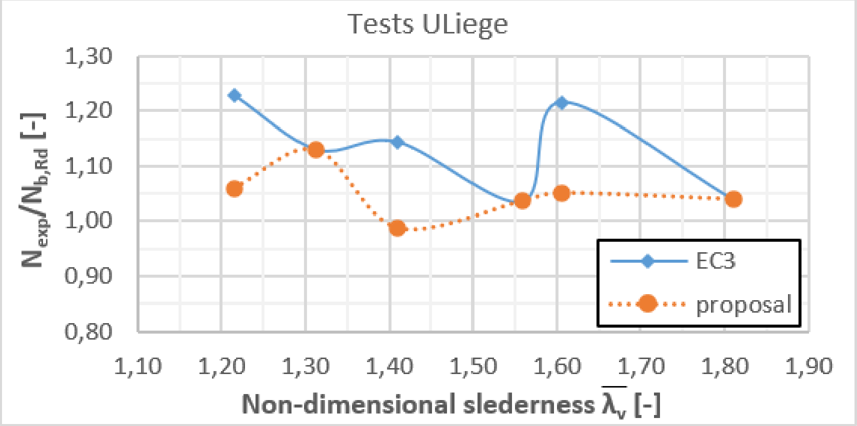
4. Experimental validations

Institute	Tsinghua University
Type of test	66 centrally loaded tests
Profiles	from L125x125x8 to L200x200x14
Classifications	Class 4
Steel grade	S420
Boundary conditions	pin-ended columns





Institute	ULiège
Type of test	6 centrally axial loaded tests
Profiles	L150x150x18 & L200x200x16
Classifications	Class 1 & Class 4
Steel grade	S420 & S460
Boundary conditions	pin-ended columns

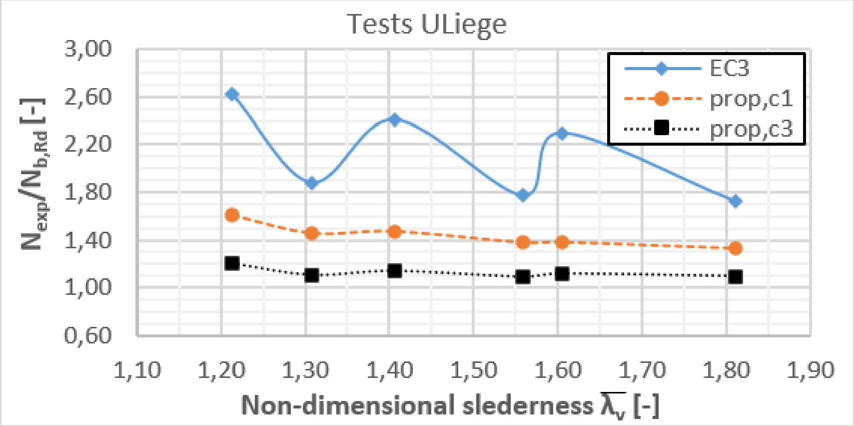


Design rules for members made of angles

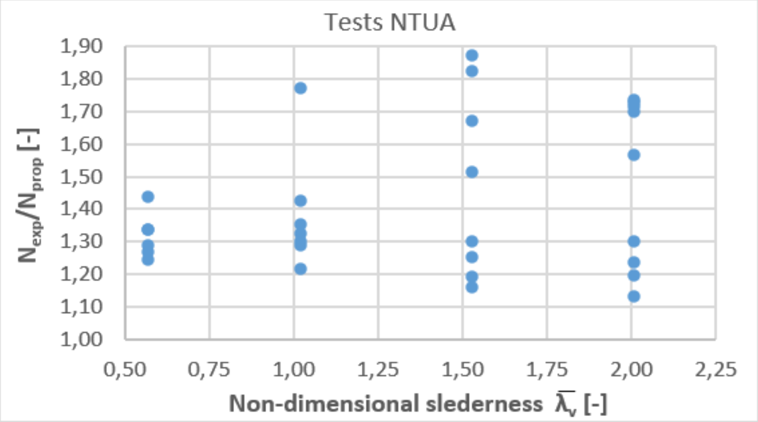
EN1993-1-1

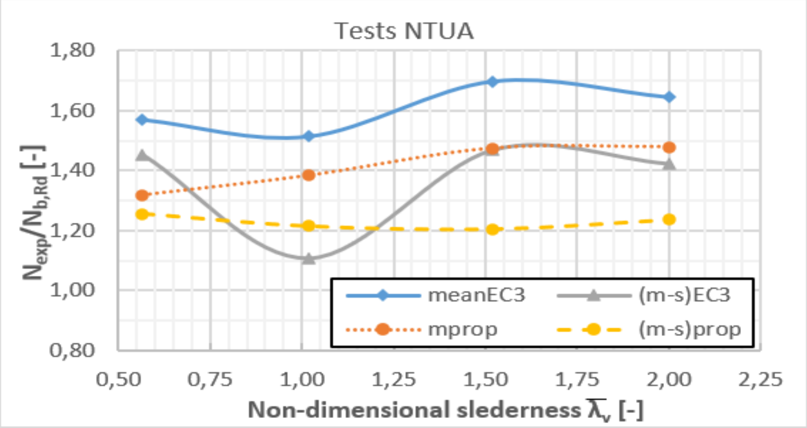
4. Experimental validations

Institute	ULiège
Type of test	6 eccentrally axial loaded tests
Profiles	L150x150x18 & L200x200x16
Classifications	Class 1 & Class 4
Steel grade	S420 & S460
Boundary conditions	pin-ended columns



Institute	NTUA
Type of test	33 eccentrally loaded tests
Profiles	L70x70x7
Classifications	Class 1
Steel grade	S275
Boundary conditions	pin-ended columns





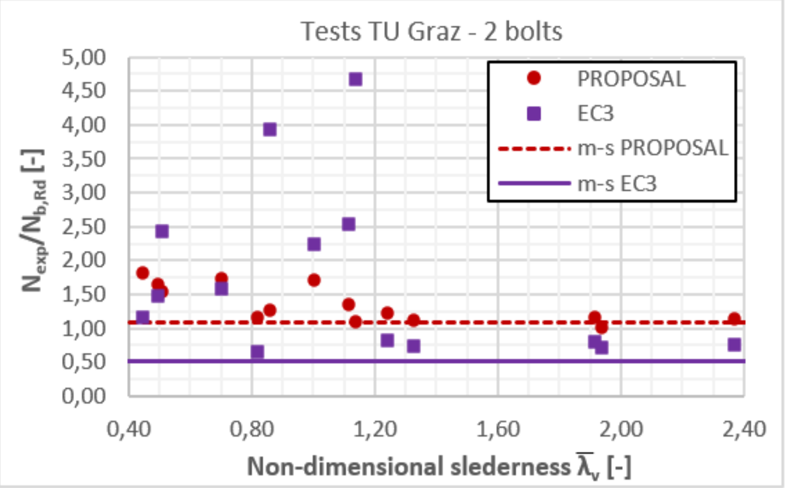
Design rules for members made of angles

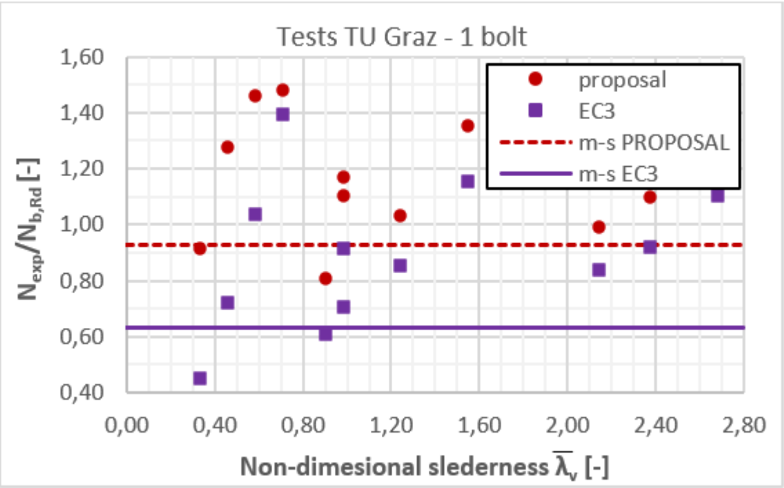
EN1993-3-1

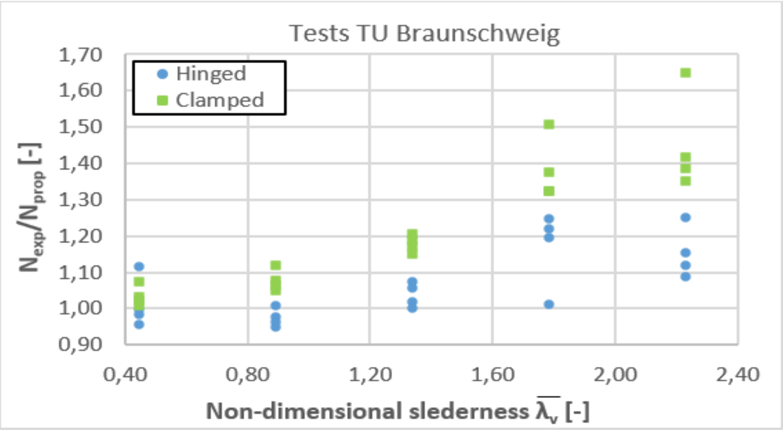
4. Experimental validations

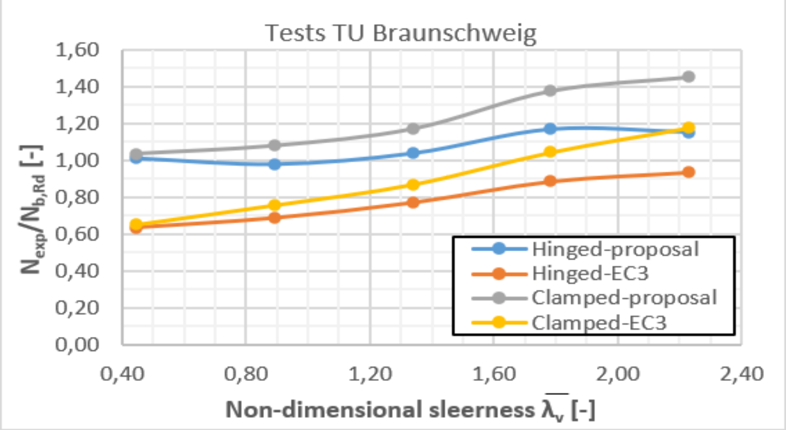
Institute	TU Graz
Type of test	27 compression tests through 1 or 2 bolts
Profiles	24 on L80x80x8 and 3 on L120x120x12
Classifications	Class 1
Steel grade	S275
Boundary conditions	(i) clamped supports, (ii) knife supports allowing rotation in the loading plane, (iii)fully hinged support

Institute	TUBraunschweig
Type of test	40 compression tests through 1 bolt
Profiles	L50x50x5
Classifications	Class 1
Steel grade	S355
Boundary conditions	(i) clamped supports, (ii) fully hinged support









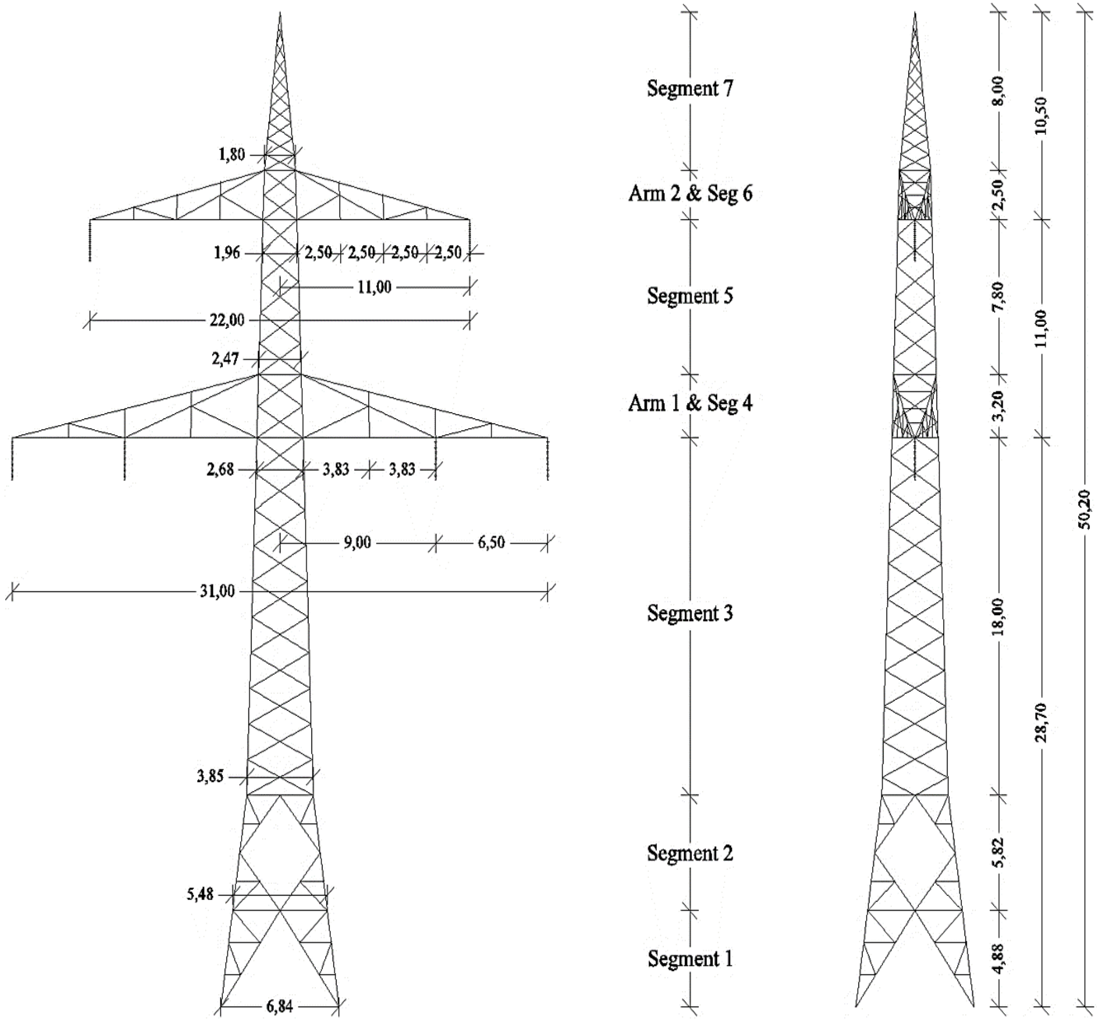
Assessment of the design of a lattice tower

- Current design approach: **EN 1993-3-1** and/or **EN 50341**: linear elastic analysis of a truss structure (disregarding bending moments).

- A suspension Danube tower (supposed to be in Germany), made of equal-leg angles and S355J2 has been selected. Only the tower is modelled - not the entire line.

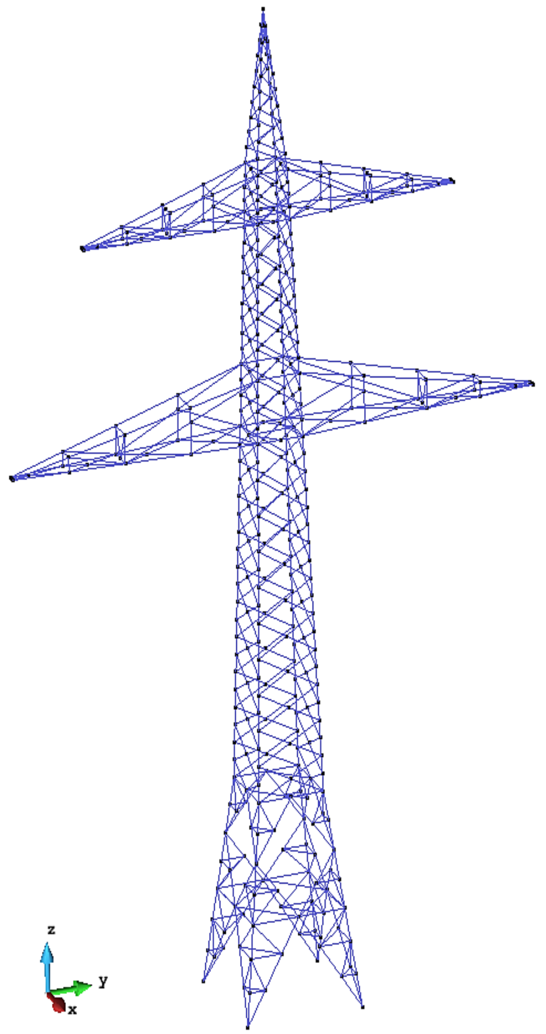
- Initial design (TOWER) for G+W according to EN 50341-1. The eccentricities of the connections are not modelled, but their influence is considered via λ_{eff} in the member buckling checks.

- Geometry + material = fixed → cross-sections size



Assessment of the design of a lattice tower

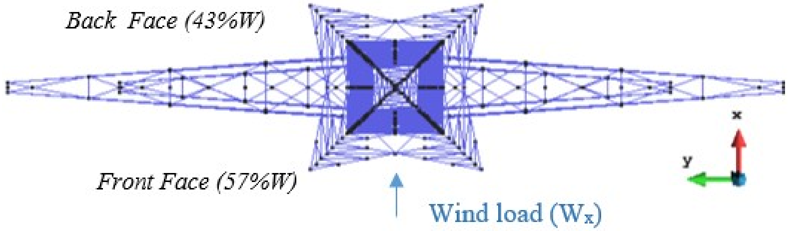
- Assessment of the initial design → FINELG - beam elements (7 DOF).
- Every bar in its real position (eccentricity, rotation) → elements subjected to N+M.
- The connections not modelled directly but have been simulated through appropriate constraints at the extremities of the elements; their self-weight is also considered.
- The tower structure is modelled using the following assumptions:
  - main legs → considering continuity over their total length;
  - Diagonals, horizontals and secondary bracing members → pin-ended;



Assessment of the design of a lattice tower

- Loads: Self-weight and wind forces acting on tower, conductors, earth wire and insulators.
- Mean wind load (W_x, W_y) for segments → distribution to faces (front / back) → constant linear loads along the bars.
- TOWER (EN 50341) while FINELG (EN 1993-3-1) for wind loads → differences but W_{tot} per direction differs < 5%.
- The way that the loads are applied on the pylon (linear vs concentrate) influences more the response of the tower, but the assumption made in FINELG is closer to reality.
- 12 load combinations were considered for the initial design. The two most critical ones are selected for the assessment:
    - X direction: $1{,}35G + 1{,}35W_x$
    - Y direction: $1{,}35G + 1{,}35W_y$

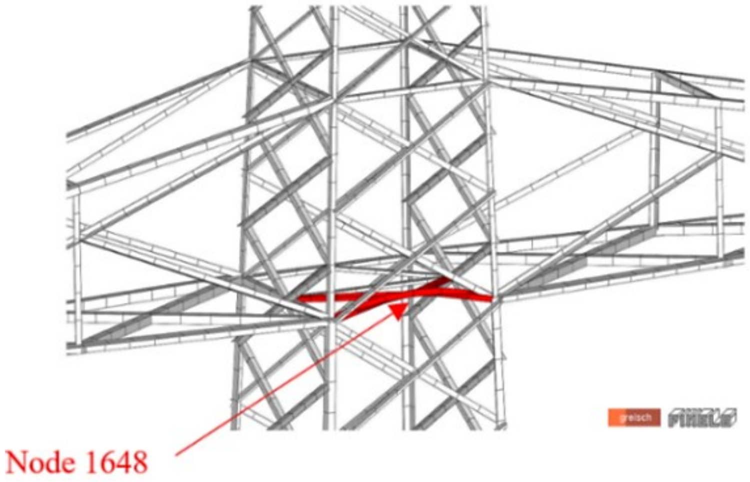
  $\gamma_G = \gamma_W = 1{,}35$ for unfavourable actions (EN 50341)
- Load sequence: 1,35G+α1,35W.
- Comparison of both software in the elastic range → results in good agreement.



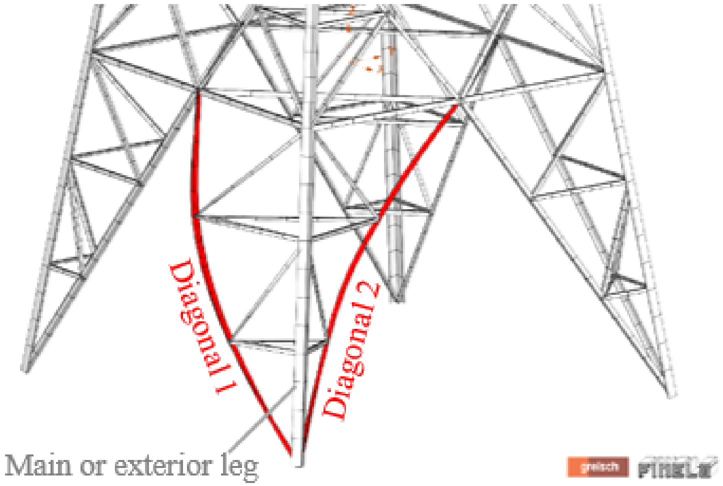
Assessment of the design of a lattice tower

1. Linear buckling analyses

$1{,}35G+\alpha_{cr}1{,}35W_x$



$1{,}35G+\alpha_{cr}1{,}35W_y$



Load combination	$1{,}35G+\alpha_{cri}1{,}35W_x$		$1{,}35G+\alpha_{cr}1{,}35W_y$	
No of mode	Load factor α_{cr}	Type of instability	Load factor α_{cr}	Type of instability
1st	3,056	Member	1,015	Segment
2nd	5,853	Member	1,179	Member
3rd	6,764	Member	1,205	Member

Assessment of the design of a lattice tower

2. Second order elastic analyses

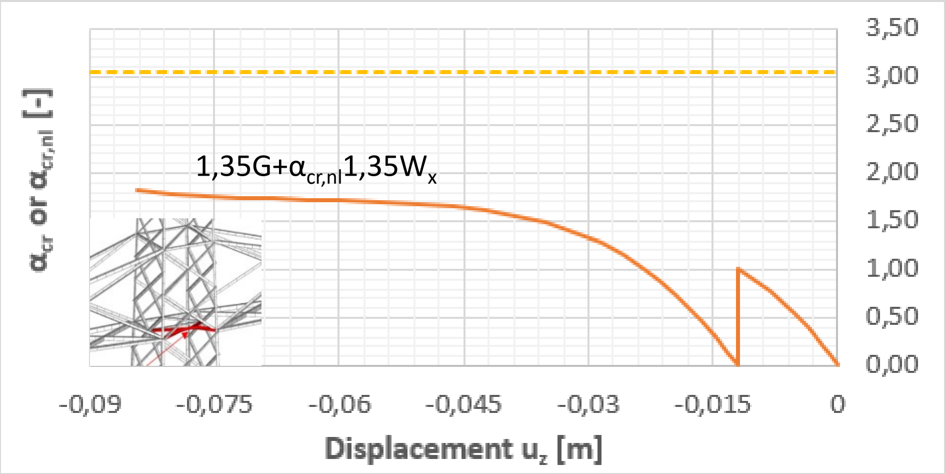
- Geometrically non-linear elastic analysis with elastic material law, without considering initial imperfections → to complement the elastic buckling analysis.

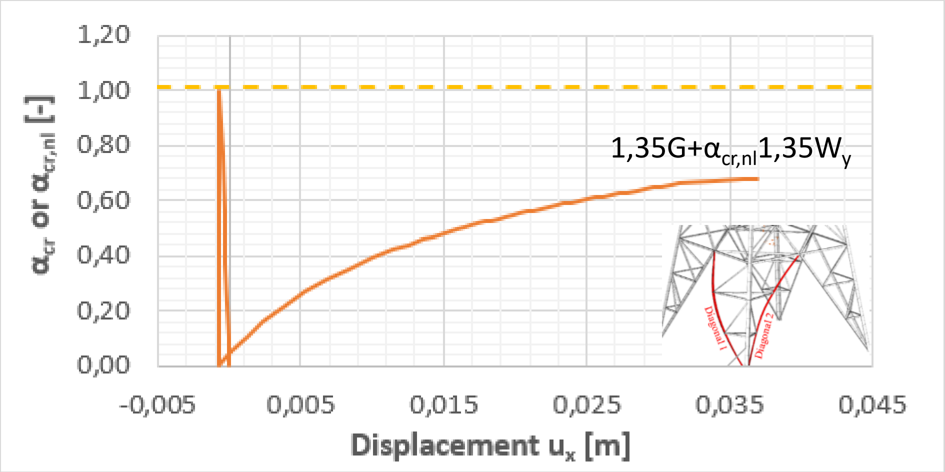
- α_{cr} significantly higher than the maximum $\alpha_{cr,nl}$.

- Internal forces at node 1648 → failure occurs for two different triplets of $N+M_u+M_v$.

- P-δ effects are significantly influencing the internal forces of the members →should be considered in the design.

Internal forces/Type of analysis	Elastic instability analysis	2nd order linear elastic analysis without initial imperfection
Axial N [kN]	-266,92	-177,10
Torsion M_T [kNm]	0,05	0,413
Bending M_u [kNm]	3,56	10,19
Bending M_v [kNm]	-0,24	-5,08
Load factor α_{cr} or $\alpha_{cr,nl}$	3,06	1,71



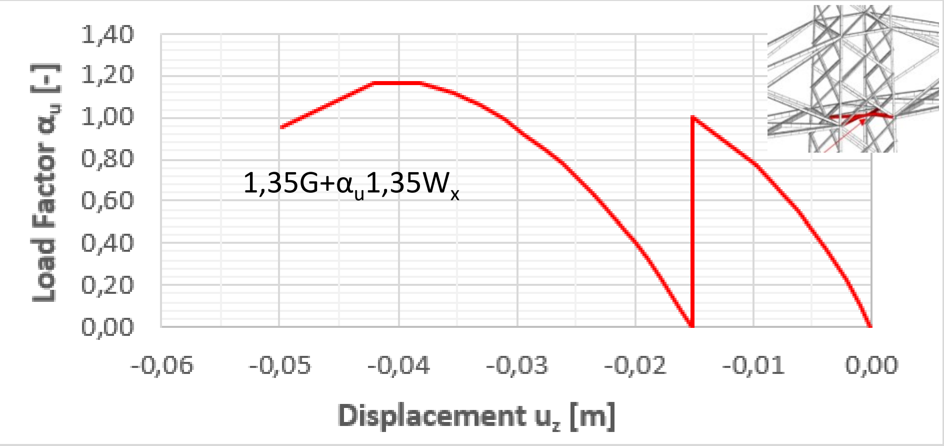


Assessment of the design of a lattice tower

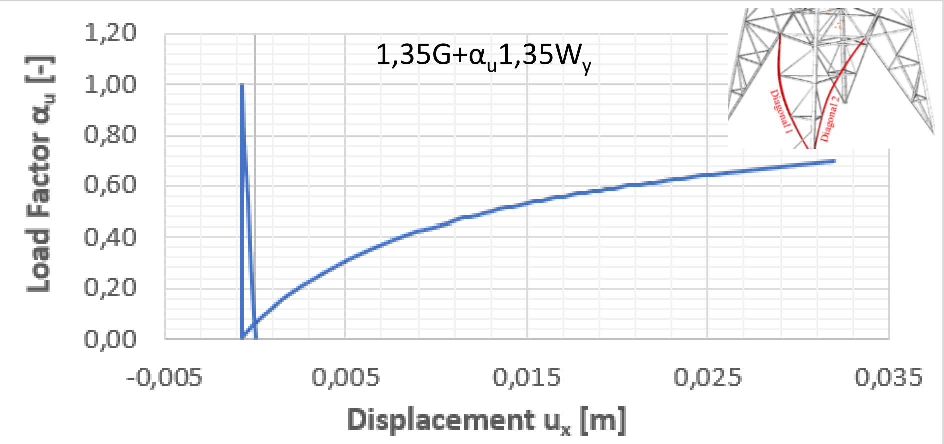
3. Full non-linear analyses

- GMNIA analysis.

- <u>X direction</u>: α_u = 1,17 > 1,0
  → (i) safe initial design;
  (ii) $\alpha_u \leq$ 1,0, assumption of elastic behavior confirmed.



- <u>Y direction:</u> $\alpha_u \approx$ 0,66 < 1,0
  → (i) initial design is insufficient
  (ii) development of segment instability → not covered by TOWER and existing recommendations

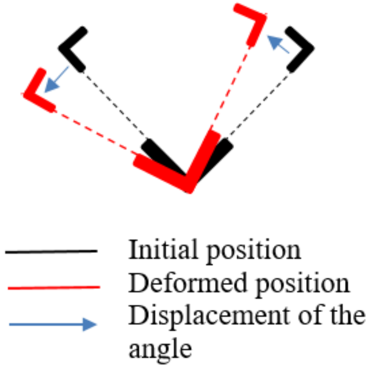


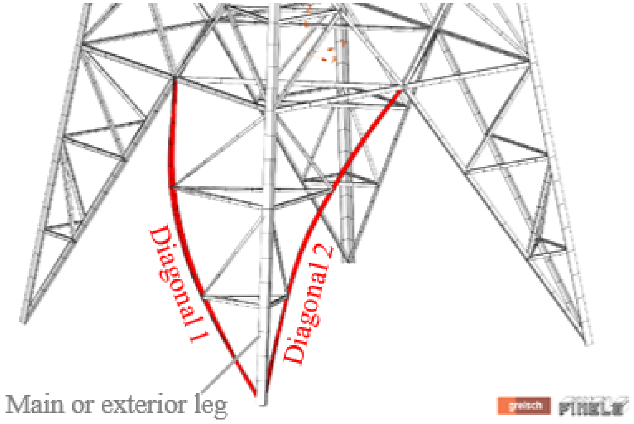
The segment instability mode

Definition:

- **Segment instability** → global instability mode associated to buckling of more than one member forming a segment.
- The members are individually stable.
- Simultaneous buckling of both diagonals over the whole leg height + a longitudinal rotation of the main member → represents a "new mode".

- A horizontal cut indicates:
  - Diagonals → moves laterally and bends about a geometrical axis;
  - Main leg → rotates about its longitudinal axis;
  - Secondary horizontal bracings → they are just translated – no deformation.





- The objective is to develop and validate an analytical formula for the evaluation of the critical load of such a type of instability.

The segment instability mode

1. Simplified model (SM)

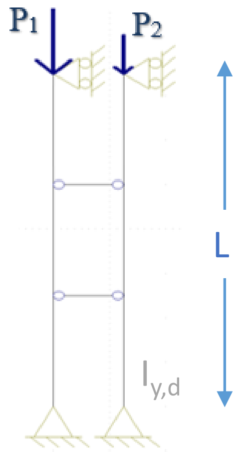
- Equivalent model: vertical members represent the diagonals, horizontal members represent the horizontal elements.

- Extremities of the vertical members → pinned; it is what expected at the foundation level. The very small restraining effect resulting from the actual continuity of the diagonals at the top is neglected.

- $N_{cr} = \frac{2\pi^2 E I_{y,d}}{L^2}$

- $a_{cr} = \frac{N_{cr}}{P_1 + P_2}$

- **This model is independent of the number of horizontal "rigid triangles", and therefore may be generally used for segments with pyramidal configuration.**

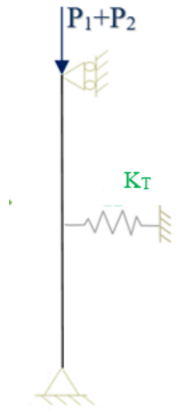


Equivalent model of the leg

The simplified equivalent model disregards the rotational restraint of the main leg member as well as the continuity of the diagonals above the leg level

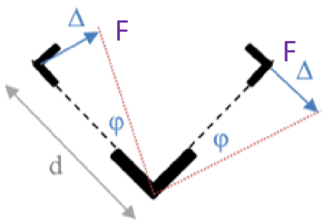
The segment instability mode

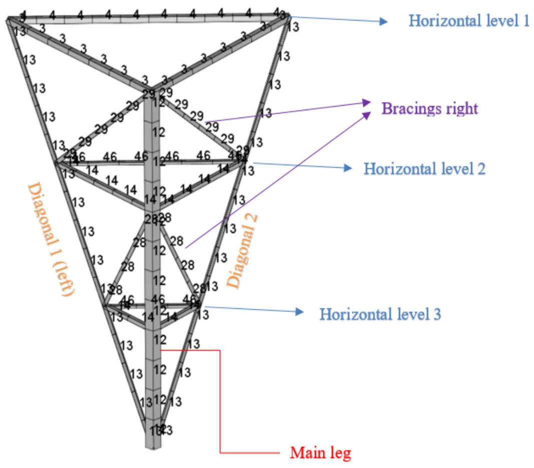
2. Refined model (RM)



Equivalent refined model

- The beneficial effect of the torsional stiffness of the exterior leg is considered.
- Simplified formulae based on the geometry, cross-section and material properties.
- $N_{cr} = \frac{\pi^2 EI}{L^2} + \frac{3}{16} K_T L$
- Stiffness of spring restraint: $K_T = \frac{4}{m^2}(2R_{mean})$
- Lateral restraint of diagonals: $R_{mean} = \frac{3C}{2L_{ext}} \cdot \frac{1}{n} \sum_{i=1}^{n} \frac{1}{d_i^2}$





The segment instability mode

3. Ultimate resistance of the leg

- Is determined by the current provisions of EN 1993-1-1
- Buckling reduction factor χ is determined by the relative slenderness $\overline{\lambda_{seg}} = \sqrt{\frac{2N_{pl}}{N_{cr}}} = \sqrt{\frac{2 \cdot Ad \cdot fy}{N_{cr}}}$ using buckling **curve d.**

4. Numerical validations

- Validation → through comparisons to results obtained from 2D numerical simulations of the proposed models (OSSA2D software)

  → then through the use of the whole tower model, using FINELG.

- Lower values obtained with SM when compared to RM, results from the fact that the rotational restraint of the main leg, as well as the continuity of the diagonals above the leg level, are disregarded.

P_1 [kN]	P_2 [kN]	Simplified model		Refined model	
		$\alpha_{cr,OSSA2D}$ [-]	$\alpha_{cr,anal,s}$ [-]	$\alpha_{cr,OSSA2D}$ [-]	$\alpha_{cr,anal,r}$ [-]
30	0	1,19	1,21	1,33	1,33
30	15	0,80	0,80	0,90	0,89
30	20	0,72	0,72	0,81	0,80
30	30	0,59	0,60	0,67	0,67

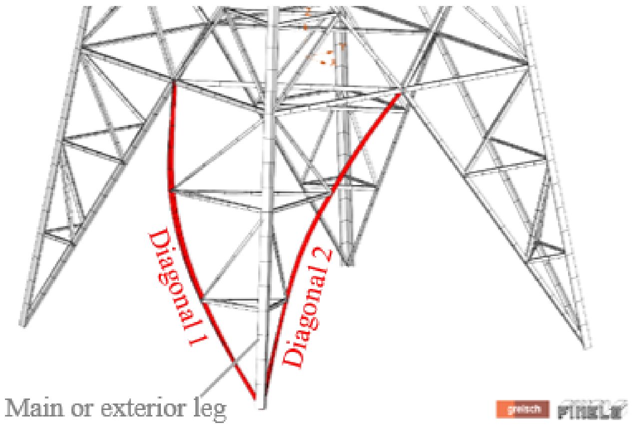
Load combination	P_1+P_2 [kN]	$\alpha_{cr,FIN}$ [-]	No of eigenmode	$\alpha_{cr,s}$ [-]	$\alpha_{cr,r}$ [-]	$\alpha_{cr,s}/\alpha_{cr,FIN}$ [-]	$\alpha_{cr,r}/\alpha_{cr,FIN}$ [-]
$G+W_y$	30,00	1,37	1	1,21	1,33	0,881	0,973
$G+W_x$	9,77	4,28	4	3,70	4,10	0,866	0,957
G_{tower}	1,83	23,99	12	19,75	21,84	0,823	0,910
Wx	7,15	6,42	1	5,06	5,60	0,788	0,872
Wy	33,05	1,48	1	1,10	1,21	0,740	0,818
Mean value	---	---	---	---	---	0,820	0,906

The segment instability mode

5. Application of the design models

Elastic critical instability	Buckling mode	Load factor α_{cr} [-]	Corresponding compression load [kN]	Level of accuracy
FINELG ($\alpha_{cr,FIN}$)	Segment	1,02	41,88	EN 1993-3-1: $\alpha_{cr,EC3}$ / $\alpha_{cr,FIN}$ = 1,66 Proposed simplified model: $\alpha_{cr,anal,s}$ / $\alpha_{cr,FIN}$ = 0,86 Proposed refined model: $\alpha_{cr,anal,r}$ / $\alpha_{cr,FIN}$ = 0,96
FINELG ($\alpha_{cr,FIN,diag}$)	Diagonal in between restraints – weak axis	1,66	66,30	
EN 1993-3-1 ($\alpha_{cr,EC3}$)		1,69	67,41	
Segment inst. models: Simpl. model ($\alpha_{cr,anal,s}$) Refined model ($\alpha_{cr,anal,r}$)	 Segment Segment	 0,87 0,97	 36,19 40,01	

Load factors and critical loads for **elastic critical instability**



From a 2nd order elastic analysis without initial imperfections to correspond to a force in the diagonals equal to the ultimate ones

Ultimate state	Buckling mode	Load factor α_u	Corresponding compression load [kN]	Level of accuracy
FINELG ($\alpha_{u,FIN}$)	Segment	0,66	(4,49+31,55)=36,04	EN 1993-3-1: $\alpha_{u,EC3}/\alpha_{u,FIN}$ = unknown Proposed simplified model: $\alpha_{u,anal,s}$ / $\alpha_{u,FIN}$ = 0,86 Proposed refined model: $\alpha_{u,anal,r}$ / $\alpha_{u,FIN}$ = 0,92
EN 1993-3-1 ($\alpha_{u,EC3}$)	Diagonal in between restraints – weak axis	See comment below.[1]		
Segment inst. models: Simpl. model ($\alpha_{u,anal,s}$) Refined model ($\alpha_{u,anal,r}$)	 Segment Segment	 0,57 0,61	 0,072·409,17=29,46 0,079·409,17=32,32	

[1]This value cannot be evaluated through a second order elastic analysis, as the segment instability occurs before the diagonal buckles. But it may be seen that, when segment instability occurs (α_u = 0,66), the force in diagonal 2 is equal to 31,50 kN while the ultimate buckling resistance between intermediate restraints according to EN 1993-3-1 (using buckling curve b for a slenderness 1,742) is equal to $N_{Rd}=\chi N_{pl}$=0,27·204,59=55,24 kN. Subsequently, the unconservative character of the present EN 1993-3 is seen to be rather significant.

Load factors and ultimate loads **at the ultimate state**

Conclusions & Research contribution and innovation

- Results of **experimental tests on large angle high strength steel columns** were presented, providing qualitative understanding and quantitative evaluation of the member response.
- Detailed **numerical simulations of the experimental tests** were performed, demonstrating useful modelling features that can prove beneficial for researchers.
- **Existing European specifications on hot-rolled equal angle sections were critically reviewed**, highlighting the inconsistencies and the lack in the design approaches in these normative documents.
- A complete and full consistent **set of design rules covering all aspects of design for angles** was developed, clearing thus the design process. They include cross section **classification**, **cross section resistance** for all types of loading as well as **rules for members** design to individual and combined internal normal forces and bending moments.
- Extensive experimental, analytical and numerical studies were conducted to **validate the proposed set of design rules**. The validated rules can be directly applied in structural engineering design practice involving angle profile members.
- Appropriate **buckling formulas** and **corresponding buckling curves** were proposed for **flexural** and **lateral-torsional buckling of angles**. The buckling formulas and curves can be reliably implemented in the structural design practice according to modern structural design standards.

Conclusions & Research contribution and innovation

- An **assessment of the current design approach used for lattice transmission towers** was achieved through numerical studies. Results can be useful for designing appropriate lattice towers.
- **An instability mode for lattice towers not properly covered by the norms was detected** and defined. **Two analytical models for the prediction of the critical load of the new buckling mode** were developed and validated numerically. Both design models are easy to apply, clearly indicate the required check to perform and fill the gap in the existing provisions of the European normative documents.
- **All** the developed **rules** of the present dissertation **were written in Eurocode 3** format to allow a direct possible inclusion in forthcoming drafts and are included in prEN1993-3.

Perspectives for future research

Based on the present dissertation, some suggestions for future research are summarized next:

- Numerical parametrical studies to find out if the application of the proposed design rules developed for equal-leg angles, could be extended to unequal leg angles too.
- Investigations are still required to better account for the beneficial effect of the restraint due to bolted connections at the extremities of the angle members, that is currently covered by the provisions of EN 1993-3 through the definition of an equivalent bucking length.

- The segment instability mode detected here need further examination. First, the selection of the buckling curve could be improved, as now it is suggested to use the lowest one (curve d) due to the lack of studies showing that a higher one could be safely used.
- The segment instability mode detected in the framework of the thesis, was associated with a certain tower configuration and has been observed in the tower's leg. Consequently, further numerical and experimental investigations are needed to check if a similar instability mode could occur in other parts of the tower (for instance in the arms), and how this could be affected by the configuration of the tower. Finally, the accuracy of the proposed models for other possible segment instabilities may be checked.

Publications

<u>Peer reviewed journals</u>

1. Bezas M.-Z., Demonceau J.-F., Vayas I., Jaspart J.-P., (2021), **Experimental and numerical investigations on large angle high strength steel columns**, *Thin-Walled Structures* 159C, 107287, https://doi.org/10.1016/j.tws.2020.107287.
2. 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, https://doi.org/10.1016/j.jcsr.2021.106842.
3. Bezas M-Z, Demonceau J-F, Vayas I., Jaspart J-P, (2021), **Compression Tests on Large Angle Columns in High Strength Steel**, Ernst & Sohn, *ce/papers*, Special Issue: EUROSTEEL 2021 Sheffield — Steel's coming home, Vol. 4, Issue 2 – 4, pp. 1523-1529, https://doi.org/10.1002/cepa.1451.
4. Tibolt M., Bezas M.-Z., Vayas I., Jaspart J.-P., (2021), **The design of a steel lattice transmission tower in Central Europe,** Ernst & Sohn, *ce/papers*, Special Issue: EUROSTEEL 2021 Sheffield — Steel's coming home, Vol. 4, Issue 2 – 4, pp. 243-248, https://doi.org/10.1002/cepa.1288.

<u>International Conferences:</u>

1. Bezas M.Z., Demonceau J.-F., Vayas I., Jaspart J.-P., (2021), **Compression tests on large angle columns in high strength steel**, EUROSTEEL 2021 conference, Sheffield, UK.
2. Tibolt M., Bezas M.Z., Jaspart J.-P., Demonceau J.-F., (2021), **The design of a steel lattice transmission tower in Central Europe**, EUROSTEEL 2021 conference, Sheffield, UK.
3. Beyer A., Bureau A., Jaspart J.-P., Demonceau J.-F., Bezas M.-Z. (2021), **Torsional, flexural and torsional-flexural buckling of angle section members – an analytical approach**, International Conference on Modelling and Simulation, ICMS 2021, Poznan, Poland.
4. Bezas M.Z., Tibolt M., Jaspart J.-P., Demonceau J.-F., (2019), **Critical assessment of the design of an electrical transmission tower**, 9th International Conference on Steel and Aluminum Structures, ICSAS19, Bradford, UK.

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