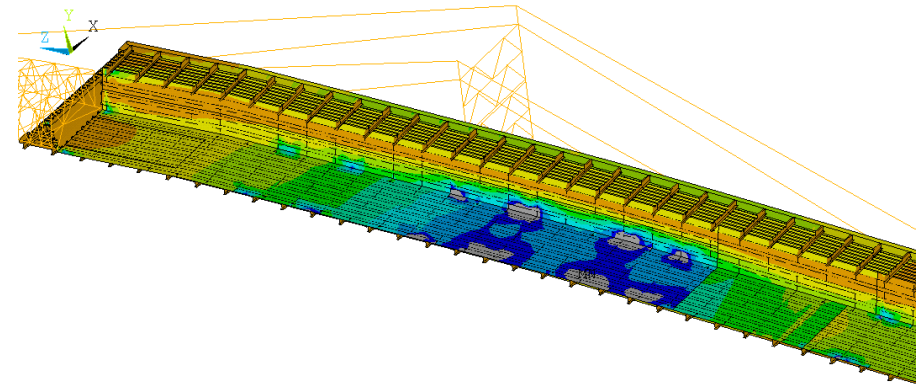


FEM based design of structures 2

Bridges



Balázs Kövesdi, PhD
Associate Professor
BME Dept. of Structural Engineering

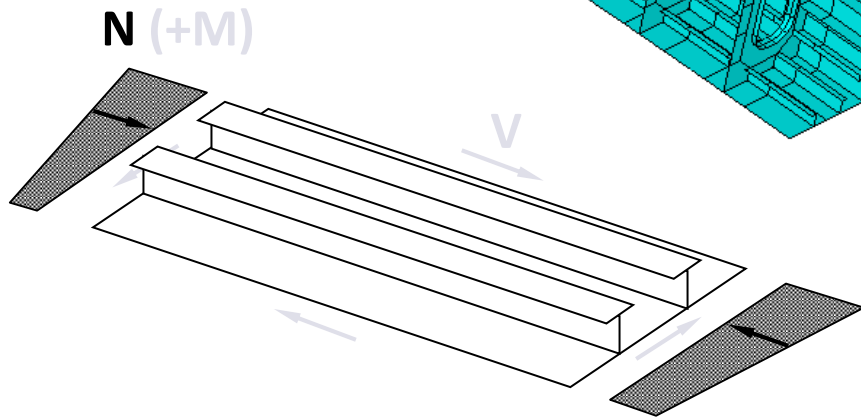
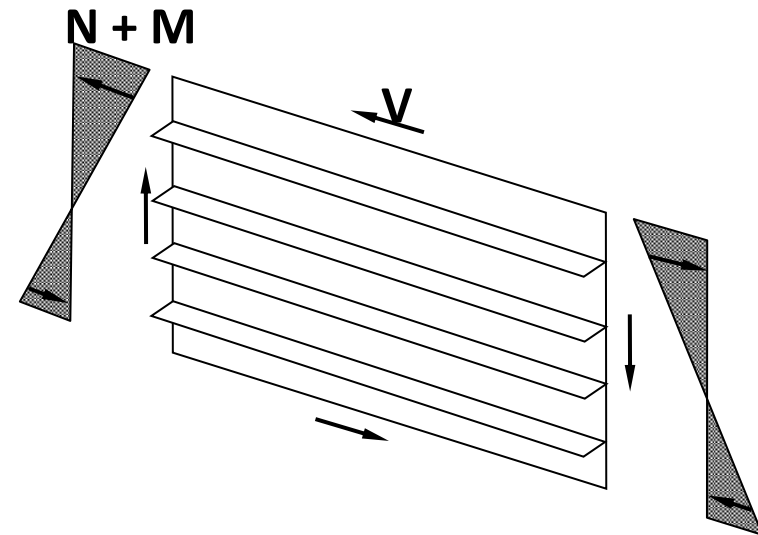
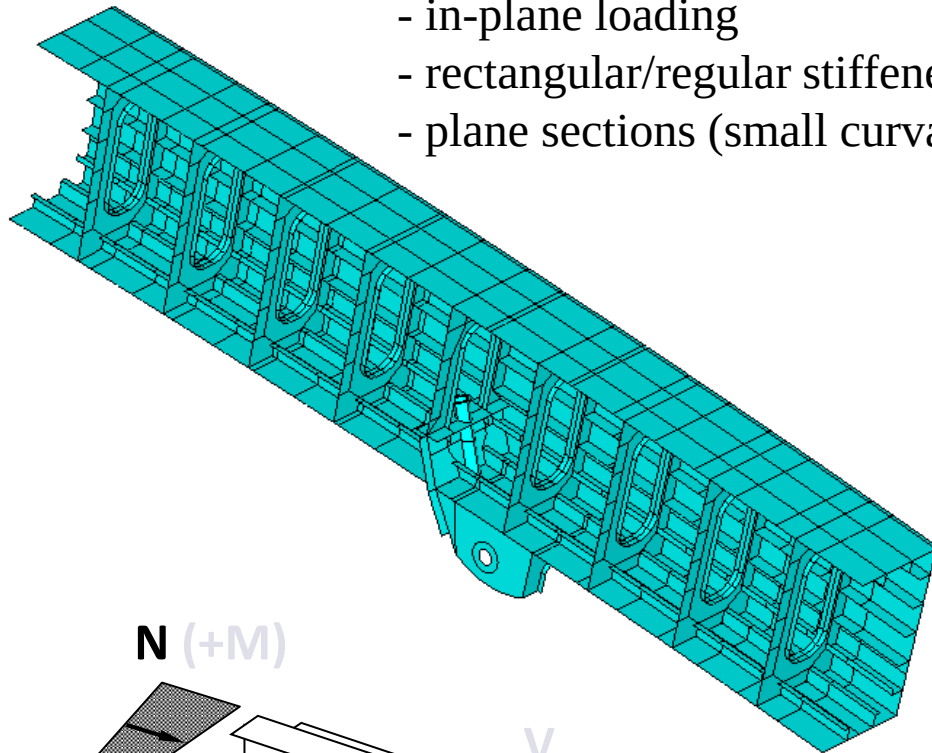


Welded Plated Stiffened Structures



Subdivision of cross section components

- Regular elements:
- regular/ rectangular geometry
 - in-plane loading
 - rectangular/regular stiffeners
 - plane sections (small curvature)



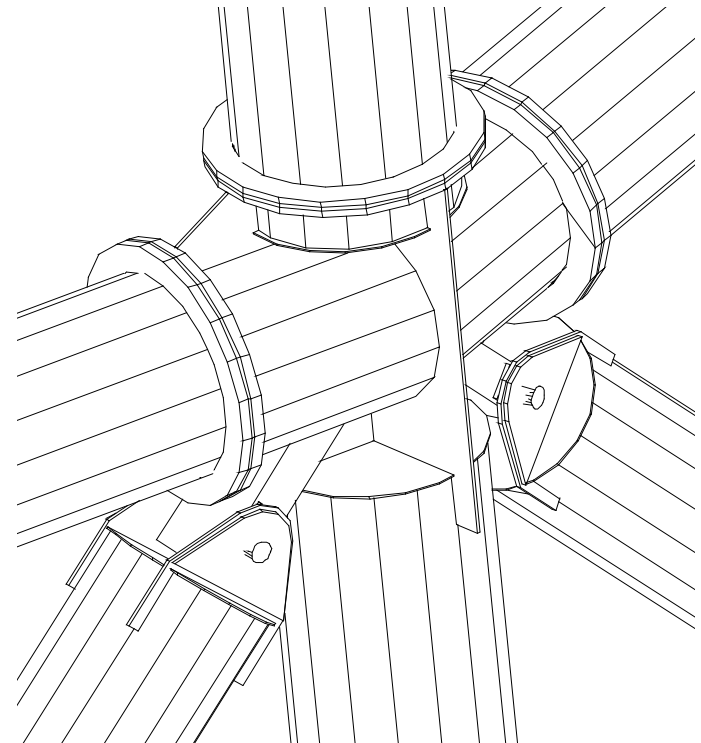
Subdivision of cross section components

- Irregular elements:
- joint regions
 - irregular plate geometries
 - large curvatures (circular tubes)

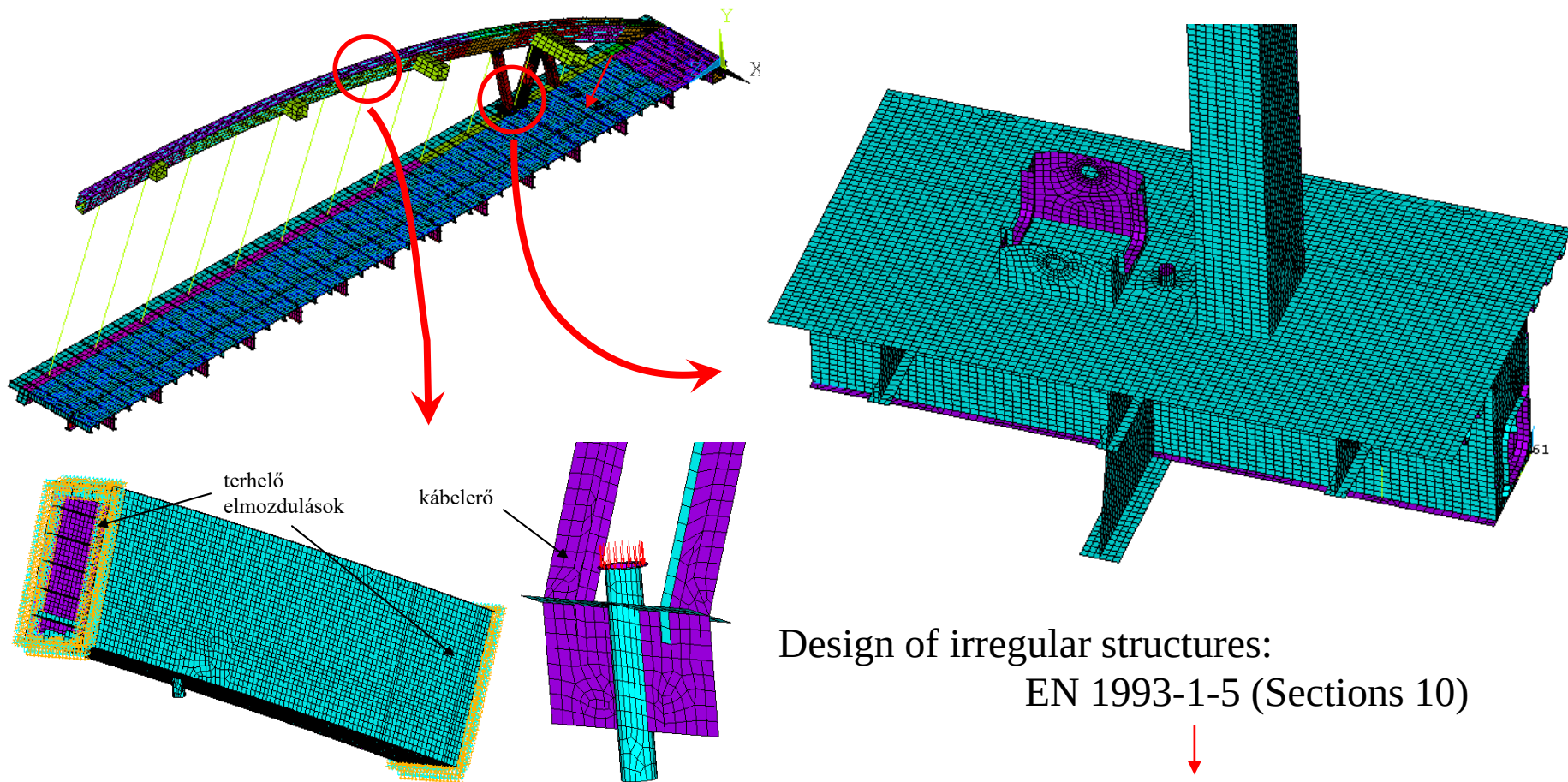
EN 1993-1-5 (Plated structures)



EN 1993-1-6 (Shell structures)



Design aspects



Design of **regular structures**: EN 1993-1-5 (Sections 1-9)
Effective width method

Design of **irregular structures**:
EN 1993-1-5 (Sections 10)

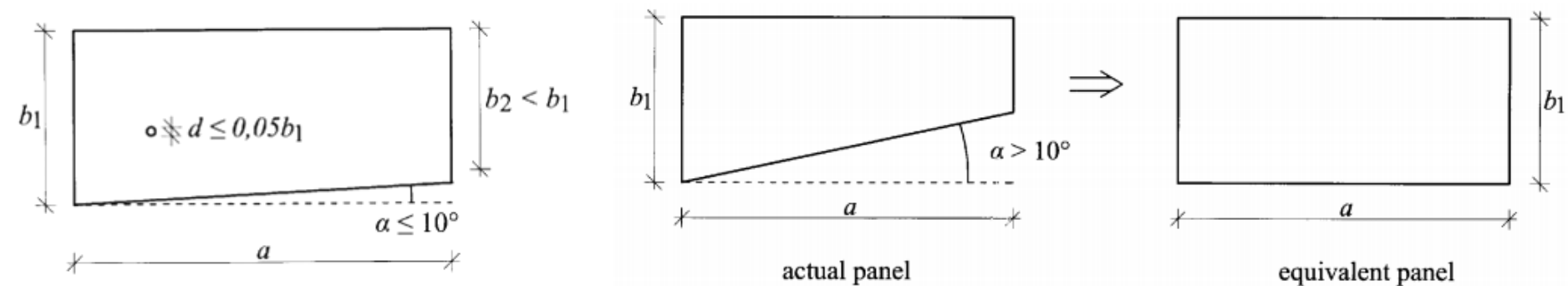
Reduced stress method

Scope of EN 1993-1-5

Application restrictions:

- shape of the panel is rectangular (flanges are parallel) or almost rectangular (angle α between longitudinal edges should not exceed 10°);
- stiffeners in direction of the normal stresses and/or perpendicular to them (longitudinal/transversal);
- size of unstiffened hole or cut-out is negligible, $d/b < 0.05$;
 d diameter of the hole,
 b width of the plate,
- plane plate, if the curvature $r > b^2/t$;
- stiffeners (that are included in calculation) should have minimal relative stiffness:

$$\gamma_{sl,i} = \frac{10.92 \cdot I_{sl,i}}{b \cdot t^3} \geq 25$$



Design specialties

In-plane loading by compression/bending



Local plate buckling



Global plate buckling



Interaction of both

Column like behaviour

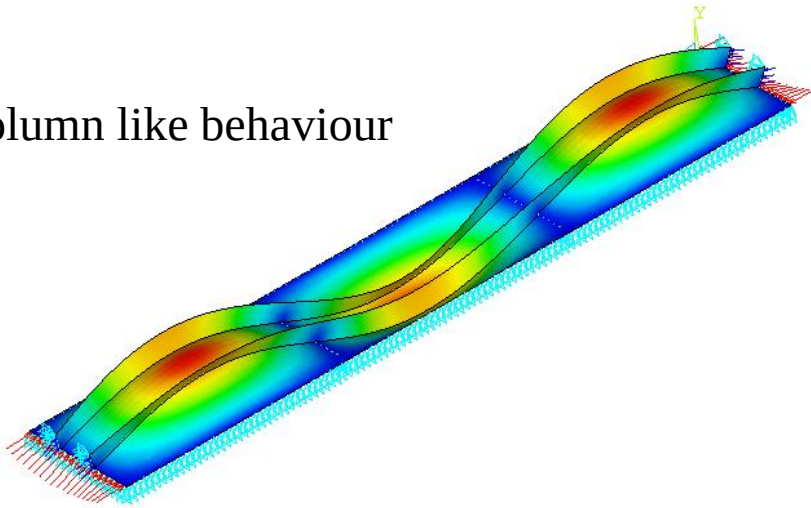
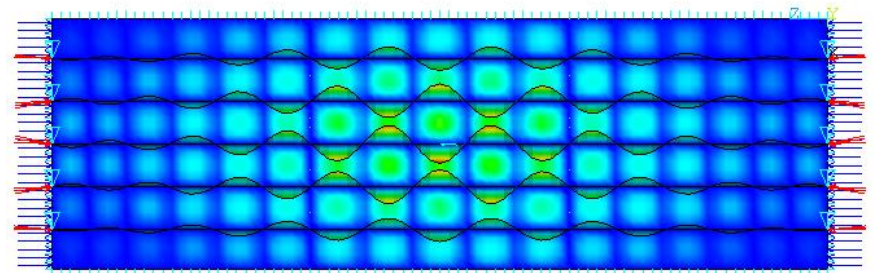
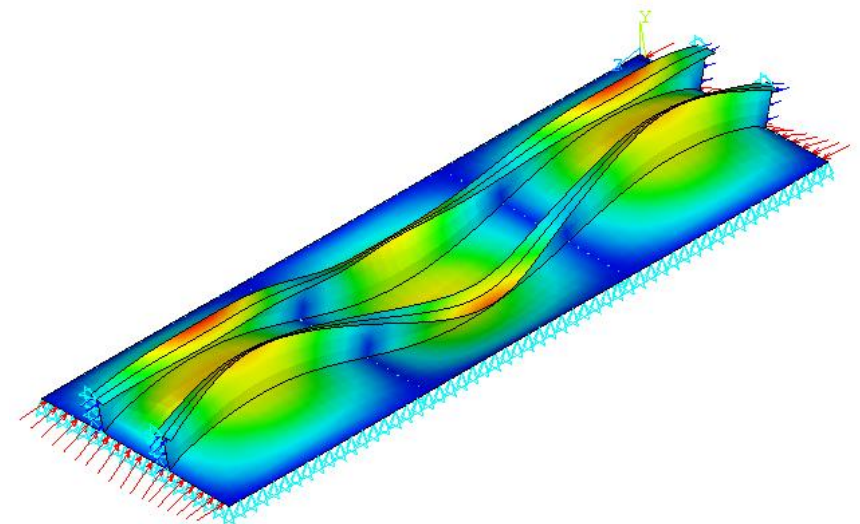
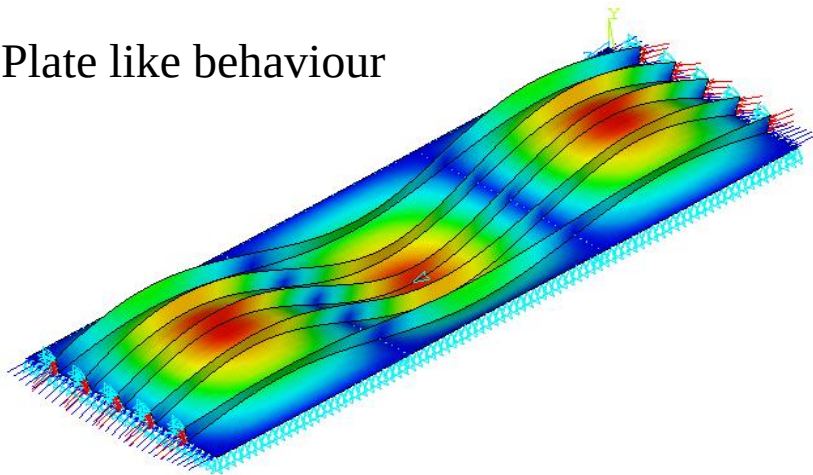


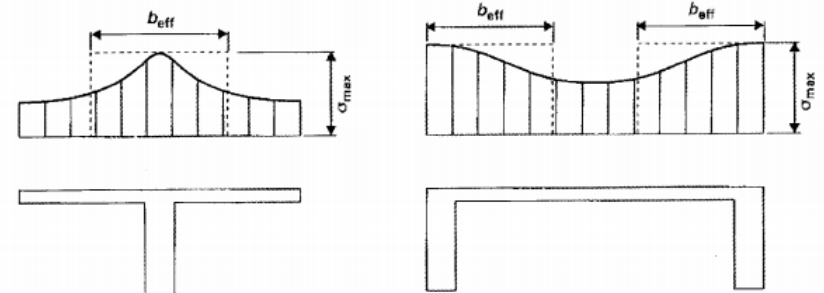
Plate like behaviour



Design specialties

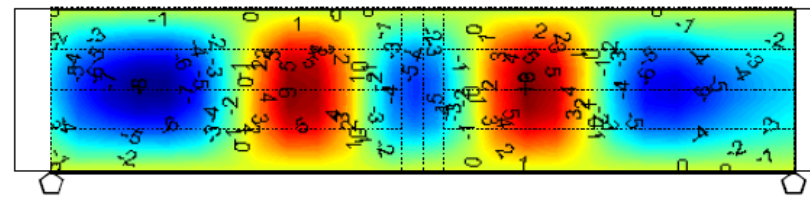
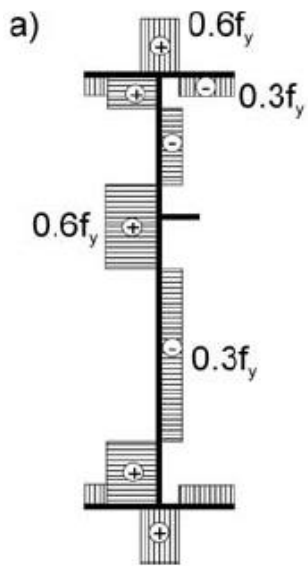
Wide plates stiffened by longitudinal and/or transversal stiffeners

Shear-lag effect

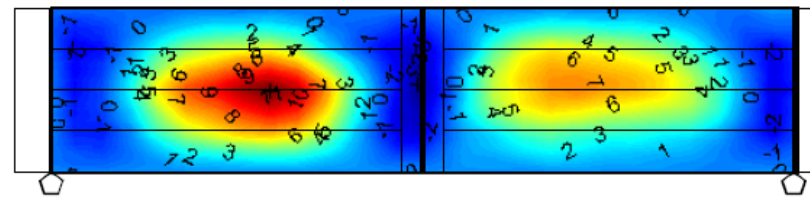


Design specialties

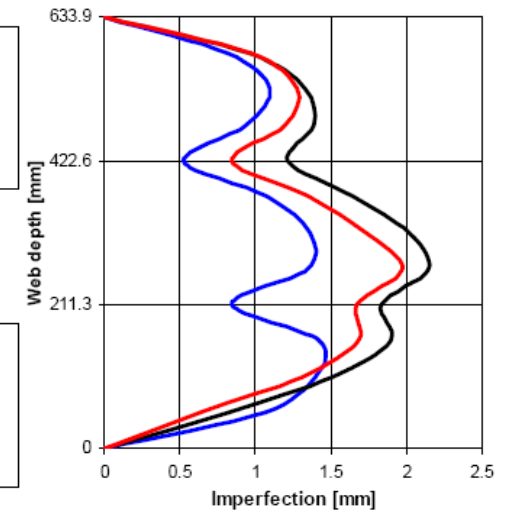
Manufacturing specialty: Residual stresses coming from welding



a) before welding horizontal stiffeners (girder S8)



b) after welding horizontal stiffeners (girder S8)



c) web cross-sections (girder J1)

Manufacturing specialty: Geometrical imperfections

less severe than for cold formed structures, or column/beam behaviours

imperfection shape, direction important

magnitude has smaller effect

Design according to EN1993-1-5

- (1) Effective width method - applicable for all sub-panels and whole section
- (2) Reduced stress method (GNB) – according to Section 10
- (3) Advanced methods: FEM based design; GMNI simulation, virtual tests
according to EN 1993-1-5 Annex C**

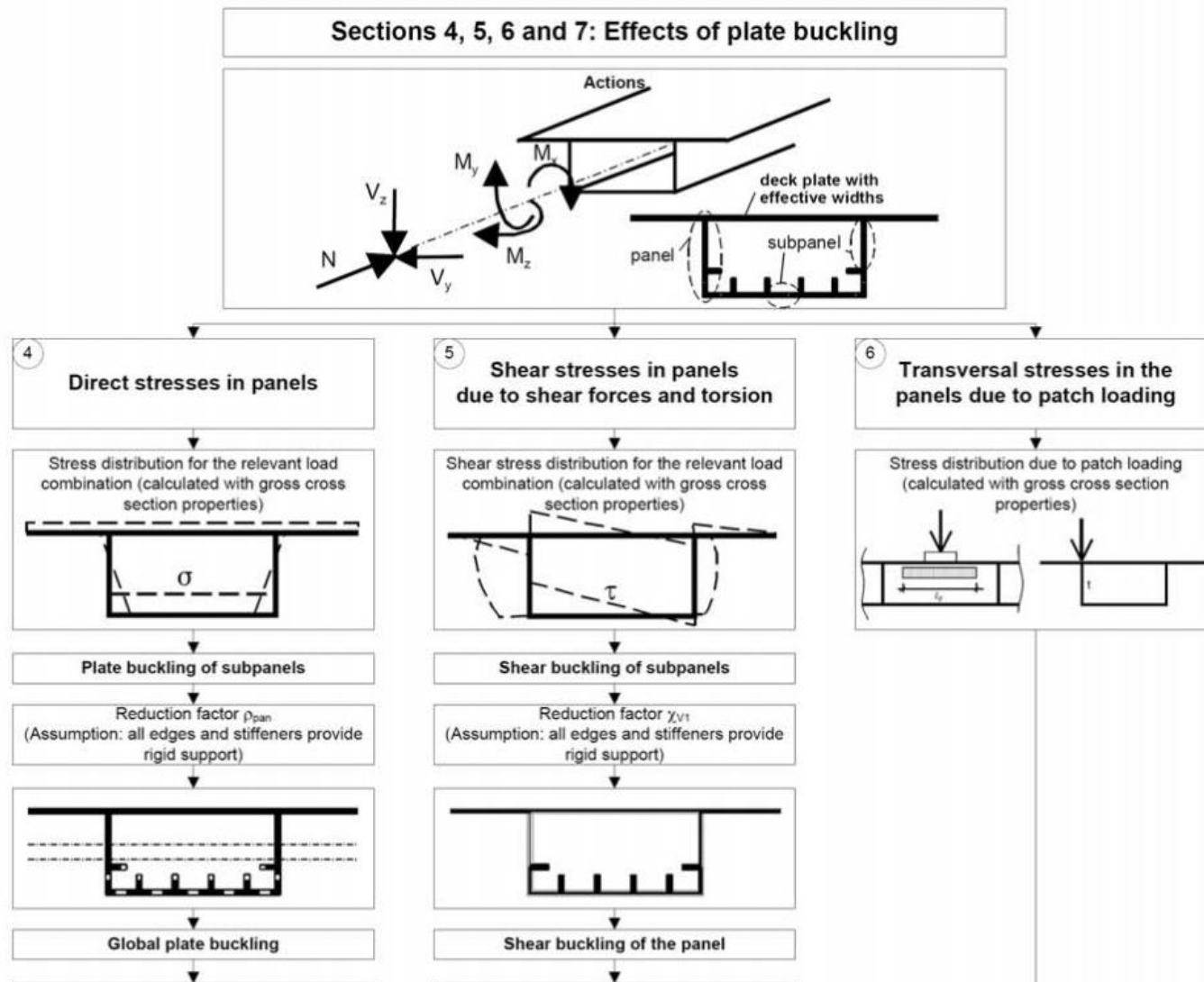
Effective width method :

- Theorie: similar as for unstiffened girders with increasing factors
 - Buckling factor
 - Sub-panels – whole panel check
- Longitudinal stresses: effective section → resistance
- Shear, transverse load: reduction factors

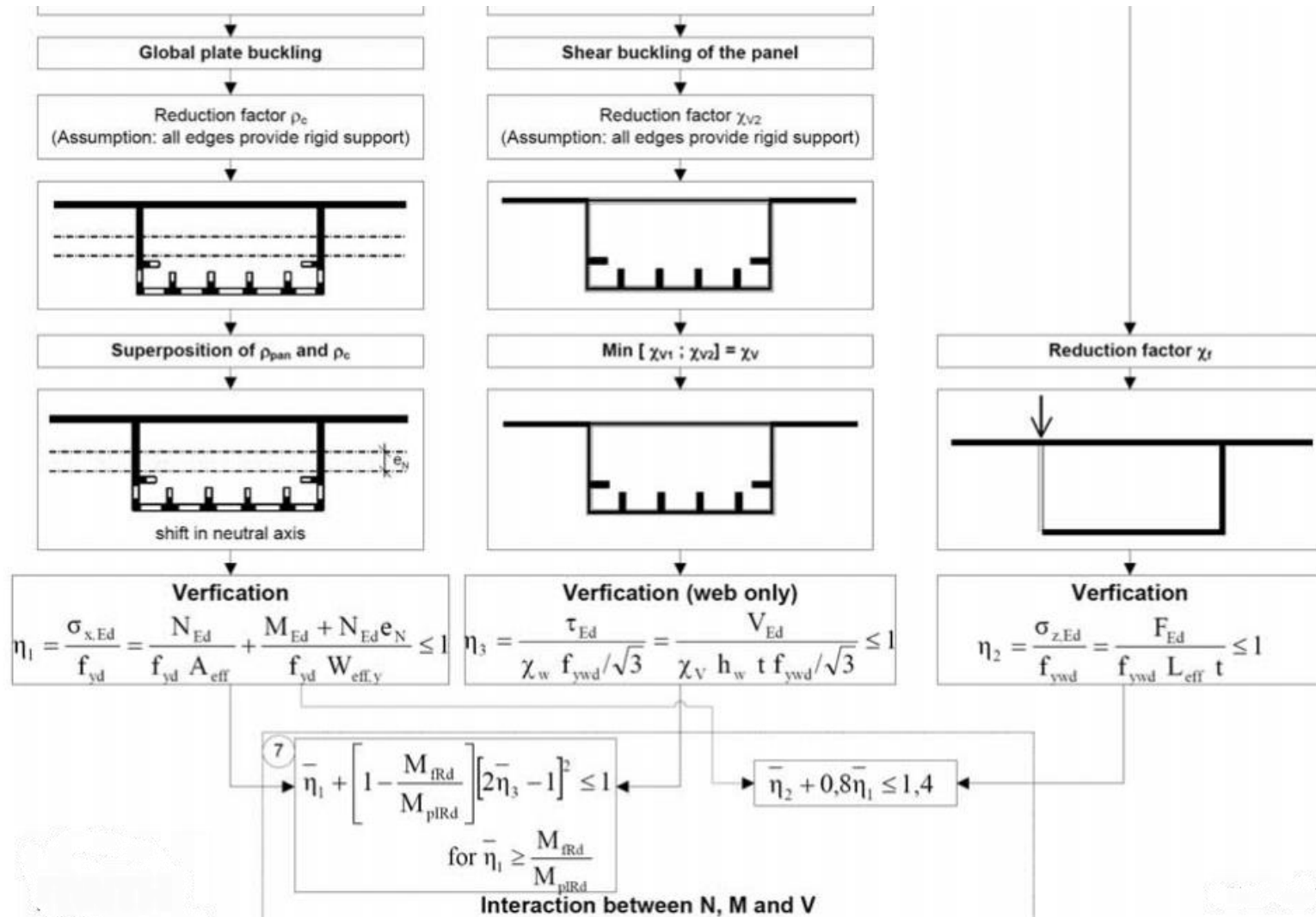
Design according to EN1993-1-5

- Effective width method:
 - static check at cross section level
 - distinct analyses for normal-, shear stresses and transverse load (patch loading)
 - additional analysis for interactions
 - basis: $\sigma_n \leq f_y$ at the extreme fiber, effect of plate buckling included in A_{eff} (4th class cross sections)
- Reduced stress method (GNB):
 - cross section properties remain the same,
 - maximum stresses are reduced
 - static check by the Von Mises stresses $\leftrightarrow$ reduced (allowed) stresses
- FEM based design:
 - numerical simulation based calculation
 - characteristic value of load bearing capacity (equivalent geometric imperfection included)
 - GMNI analysis is applied

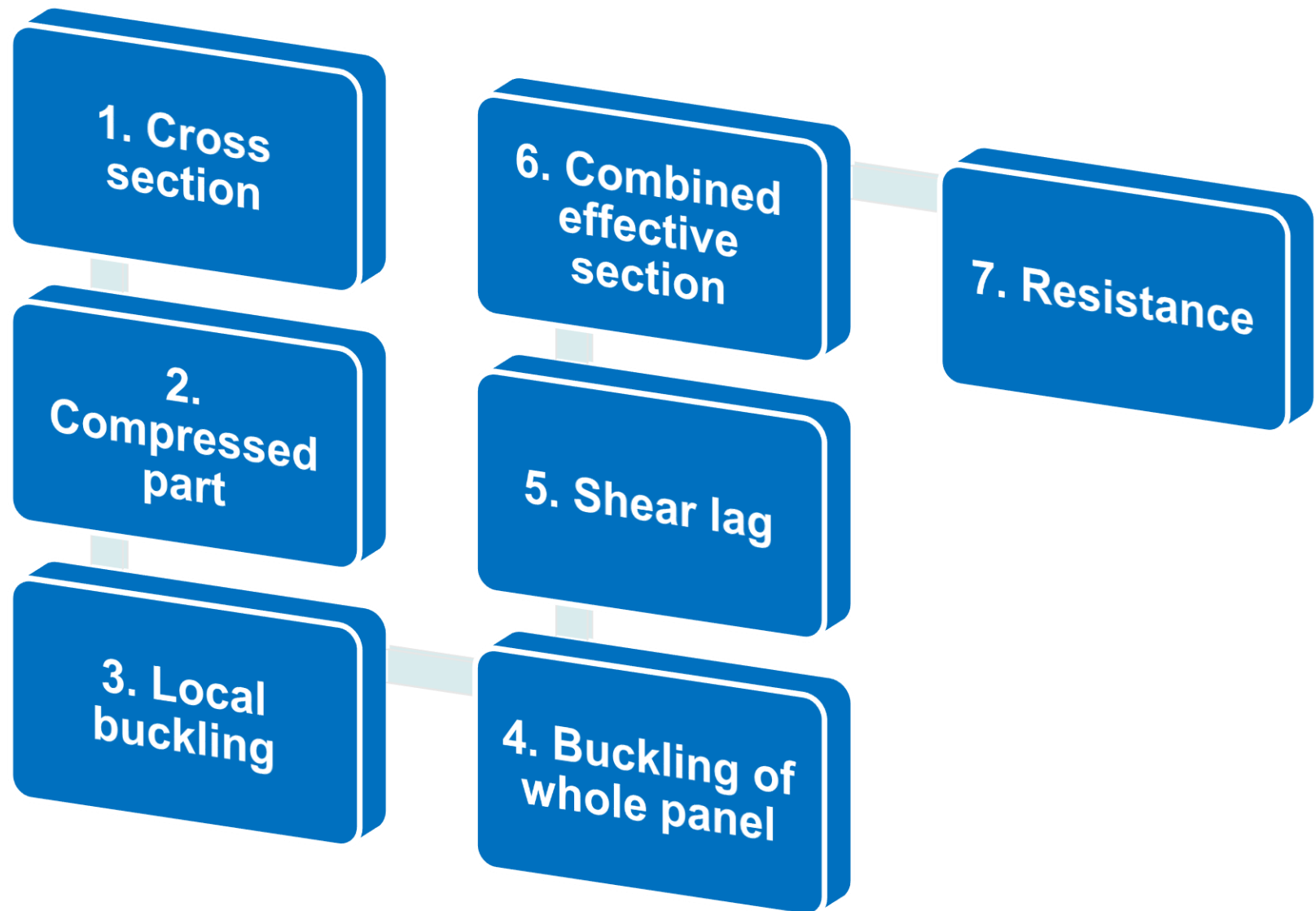
Effective width approach



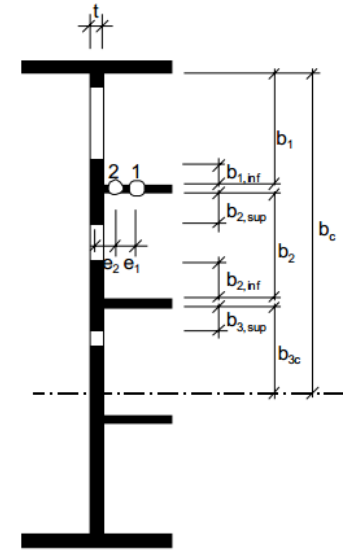
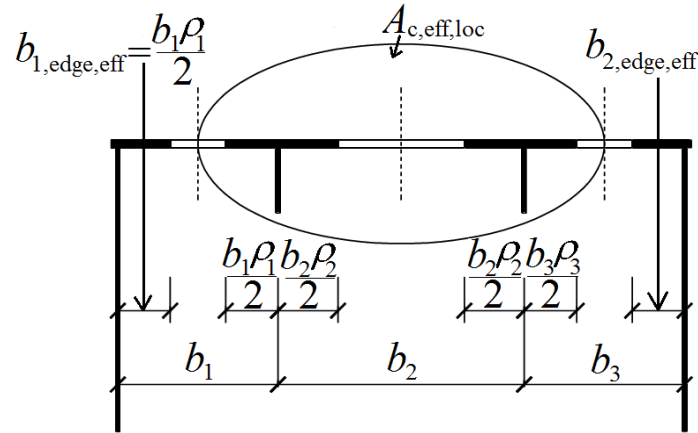
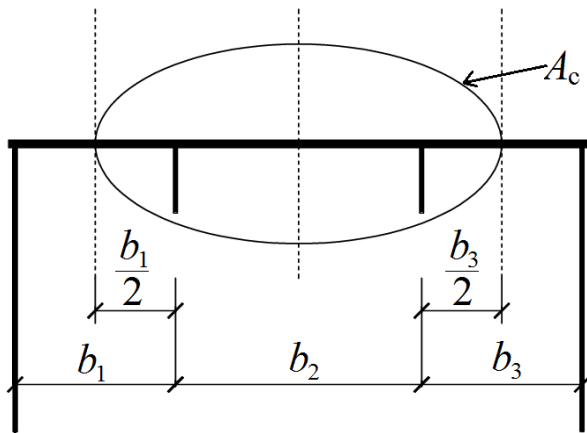
Design by EN 1993-1-5



Longitudinal normal stresses



Effective cross section



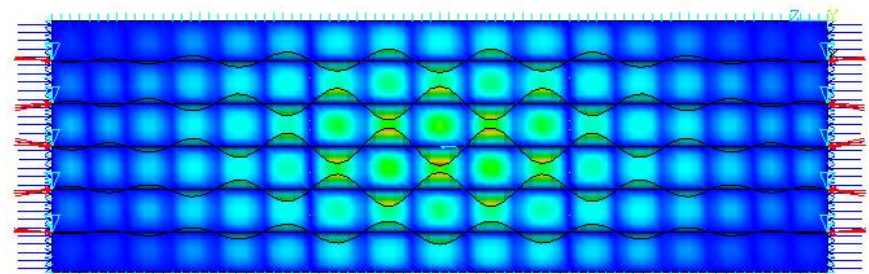
$$A_{c,eff} = \rho_c \cdot A_{c,eff,loc} + \sum b_{edge,eff} \cdot t$$

global buckling / stiffener buckling

$$A_{c,eff,loc} = A_{sl,eff} + \sum \rho_{loc} \cdot b_{c,loc} \cdot t$$

always effective part

Local buckling within stiffeners

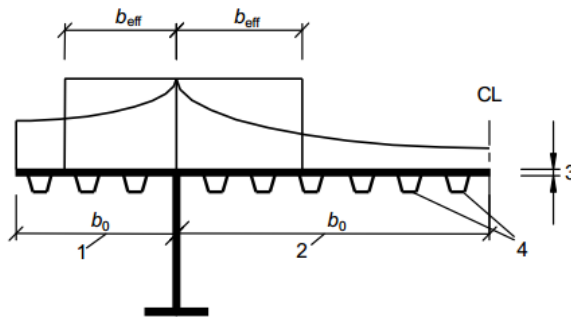


Shear leg effect

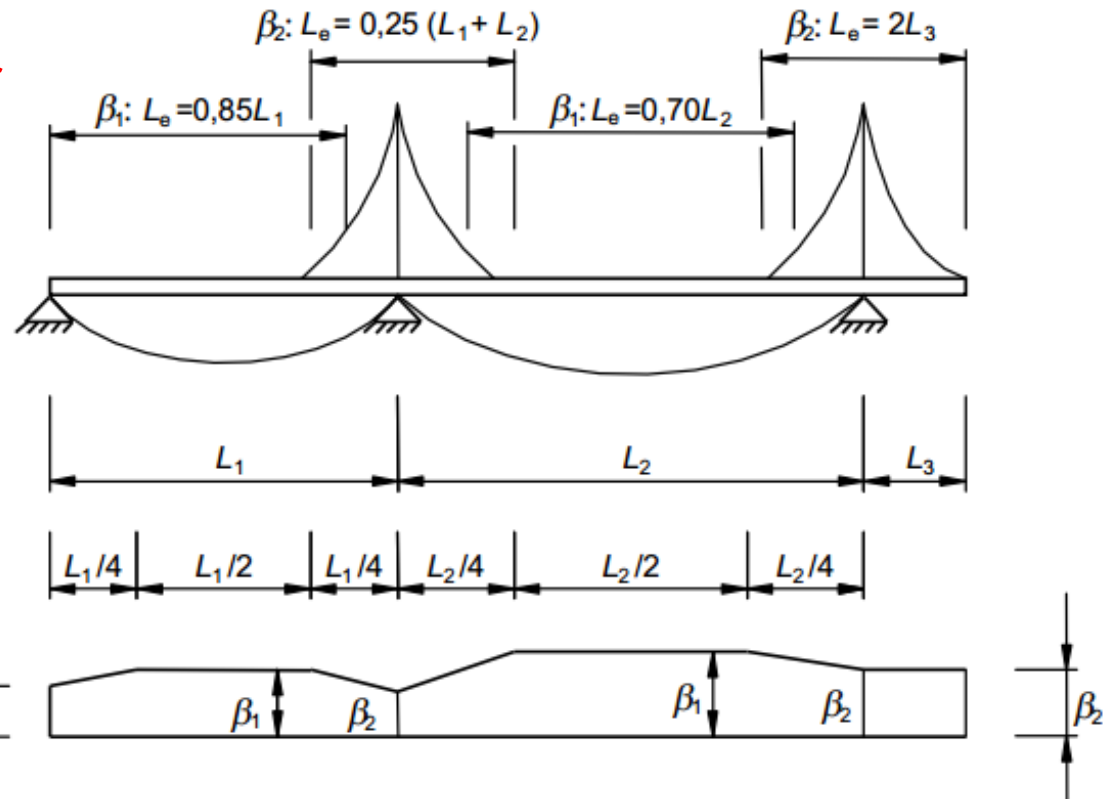
$$A_{eff} = A_{c,eff} \cdot \beta_{ult}$$

$$\alpha_0 = \sqrt{\frac{A_{c,eff}}{b_0 \cdot t}}$$
~~$$\alpha_0 = \sqrt{1 + \frac{A_{st}}{b_0 t}}$$~~

Specialty for stiffened elements

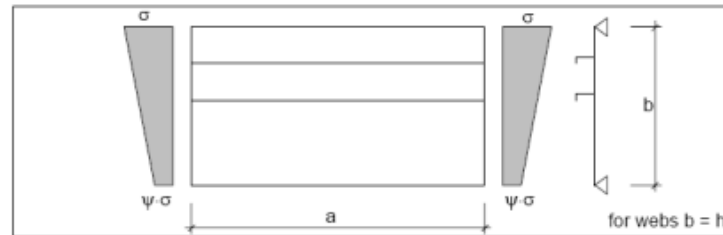


- 1 for flange outstand
- 2 for internal flange
- 3 plate thickness t
- 4 stiffeners with $A_{st} = \sum A_{sti}$



Effective width calculation

Section 4: Effective cross section properties with or without longitudinal stiffeners loaded in compression



Account for local plate buckling: $\beta_{A,c} = \frac{A_{c,eff,loc}}{A_c}$ (column) $\frac{A_{st,eff,loc}}{A_{st}}$ (plate)

Column buckling

$$\bar{\lambda}_c = \sqrt{\beta_{A,c} \frac{f_y}{\sigma_{cr,c}}}$$

$$\alpha_e = 0,34(0,49) + \frac{0,09}{i/e}$$

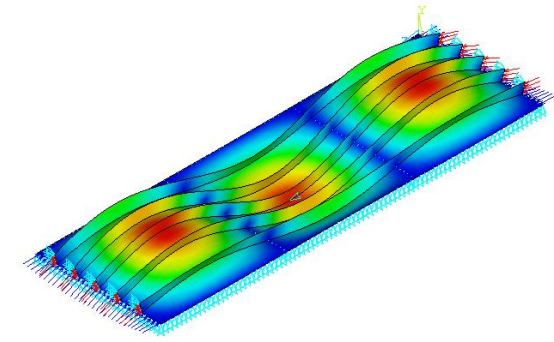
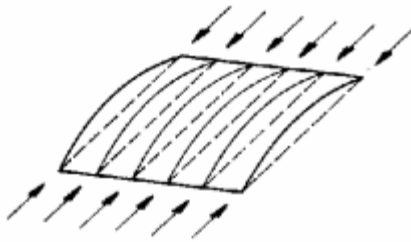
$$\varphi = 0,5 \left[1 + \alpha_e (\bar{\lambda}_c - 0,2) + \bar{\lambda}_c^2 \right]$$

$$\chi_c = \frac{1}{\varphi + \sqrt{\varphi^2 - \bar{\lambda}_c^2}}$$

Plate buckling

$$\bar{\lambda}_p = \sqrt{\beta_{A,c} \frac{f_y}{\sigma_{cr,p}}}$$

$$\rho_p = \frac{\bar{\lambda}_p - 0,055(3 + \psi)}{\bar{\lambda}_p^2}$$



Effective width calculation

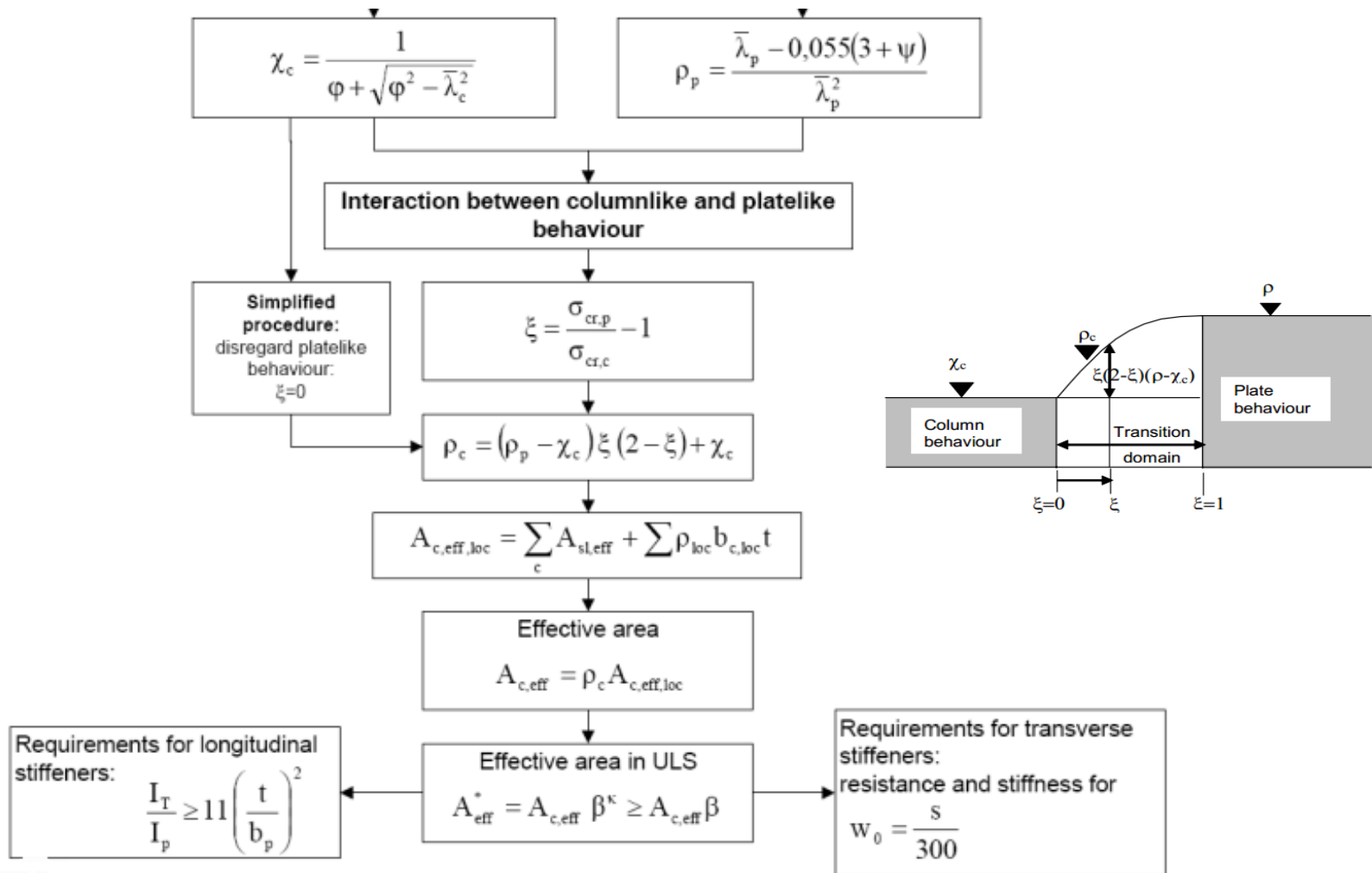
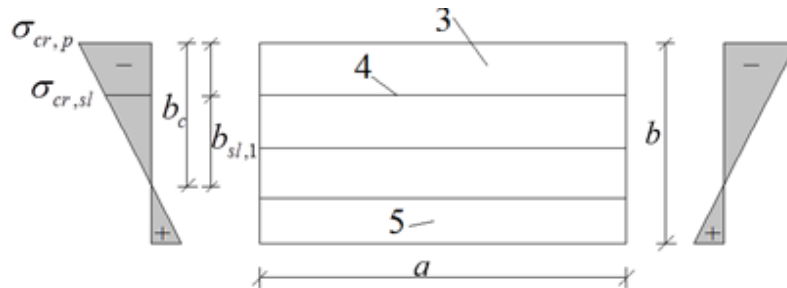
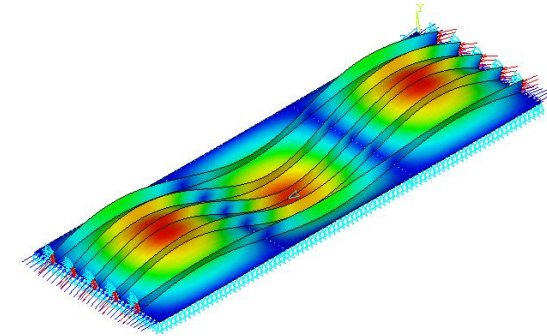


Plate-like behaviour

$$\sigma_{cr,p} = k_{\sigma,p} \cdot \sigma_E$$

$$\sigma_E = \frac{\pi^2 \cdot E \cdot t^2}{12 \cdot (1 - \nu^2) \cdot b^2} = 190000 \cdot \left(\frac{t}{b} \right)^2$$



- 1 median of longitudinal stiffener
- 2 median of longitudinal stiffener and contributive plating
- 3 sub-panel
- 4 longitudinal stiffener
- 5 plate thickness t

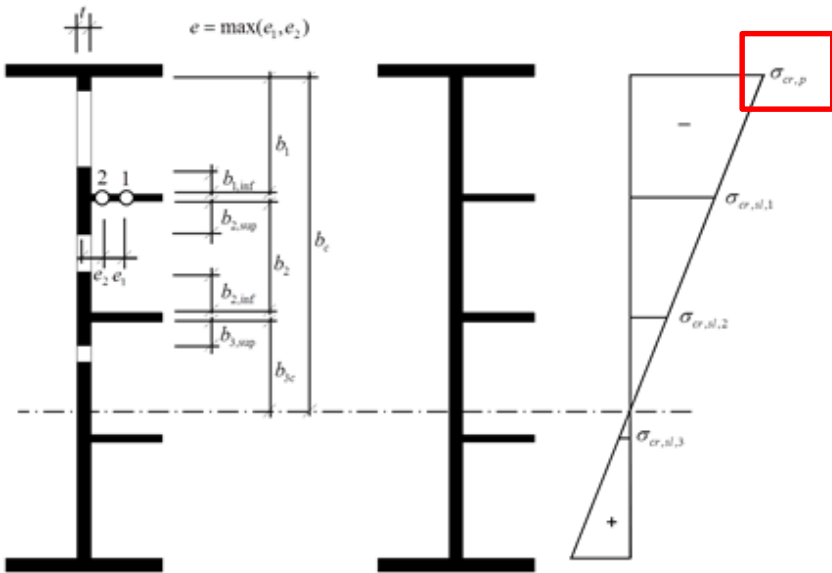
	total width	effective width	condition for Ψ_i
$b_{1,inf}$	$\frac{3-\psi_1}{5-\psi_1} b_1$	$\frac{3-\psi_1}{5-\psi_1} b_{1,eff}$	$\psi_1 = \frac{\sigma_{cr,sl,1}}{\sigma_{cr,p}} > 0$
$b_{2,sup}$	$\frac{2}{5-\psi_2} b_2$	$\frac{2}{5-\psi_2} b_{2,eff}$	$\psi_2 = \frac{\sigma_{cr,sl,2}}{\sigma_{cr,sl,1}} > 0$
$b_{2,inf}$	$\frac{3-\psi_2}{5-\psi_2} b_2$	$\frac{3-\psi_2}{5-\psi_2} b_{2,eff}$	$\psi_2 > 0$
$b_{3,sup}$	$0,4b_{3c}$	$0,4b_{3c,eff}$	$\psi_3 = \frac{\sigma_{cr,sl,3}}{\sigma_{cr,sl,2}} < 0$

Plate-like behaviour

$$k_{\sigma,p} = \frac{2\left((1+\alpha^2)^2 + \gamma - 1\right)}{\alpha^2(\psi+1)(1+\delta)} \quad \text{if } \alpha \leq \sqrt[4]{\gamma}$$

$$k_{\sigma,p} = \frac{4(1+\sqrt{\gamma})}{(\psi+1)(1+\delta)} \quad \text{if } \alpha > \sqrt[4]{\gamma}$$

with: $\psi = \frac{\sigma_2}{\sigma_1} \geq 0,5$

$$\gamma = \frac{I_{sl}}{I_p}$$

$$\delta = \frac{\Sigma A_{sl}}{A_p}$$

$$\alpha = \frac{a}{b} \geq 0,5$$

where: I_{sl} is the second moment of area of the whole stiffened plate;

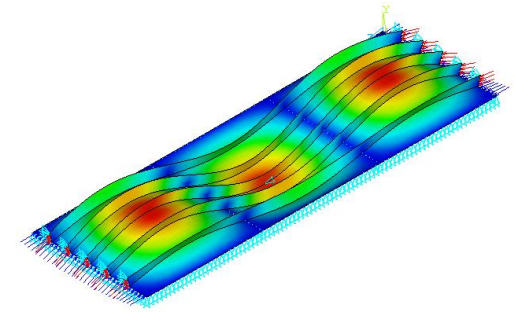
$$I_p \text{ is the second moment of area for bending of the plate } = \frac{bt^3}{12(1-\nu^2)} = \frac{bt^3}{10,92};$$

ΣA_{sl} is the sum of the gross areas of the individual longitudinal stiffeners;

A_p is the gross area of the plate $= bt$;

σ_1 is the larger edge stress;

σ_2 is the smaller edge stress;



$$\bar{\lambda}_p = \sqrt{\frac{A_{c,eff,loc} \cdot f_y}{A_c \cdot \sigma_{cr,p}}} = \sqrt{\frac{\beta_{A.c} \cdot f_y}{\sigma_{cr,p}}}$$



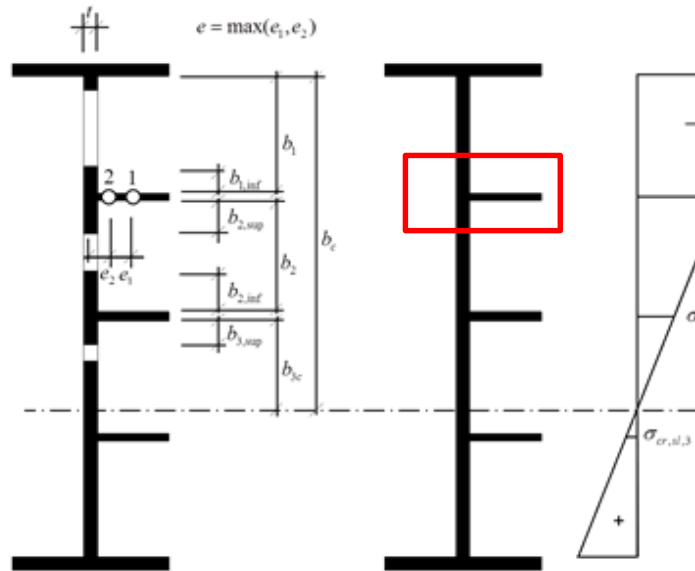
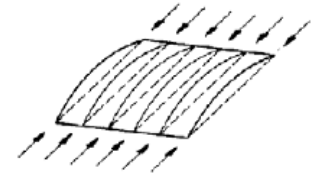
$$\rho = \frac{\bar{\lambda}_p - 0.055(3 + \psi)}{\bar{\lambda}_p^2}$$

Column-like behaviour

$$\sigma_{cr,sl} = \frac{\pi^2 \cdot E \cdot I_{sl,1}}{A_{sl,1} \cdot a^2}$$



Critical stress of the compressed stiffener



$$\sigma_{cr,c} = \sigma_{cr,sl} \cdot \frac{b_c}{b}$$

extrapolation to the plate edge

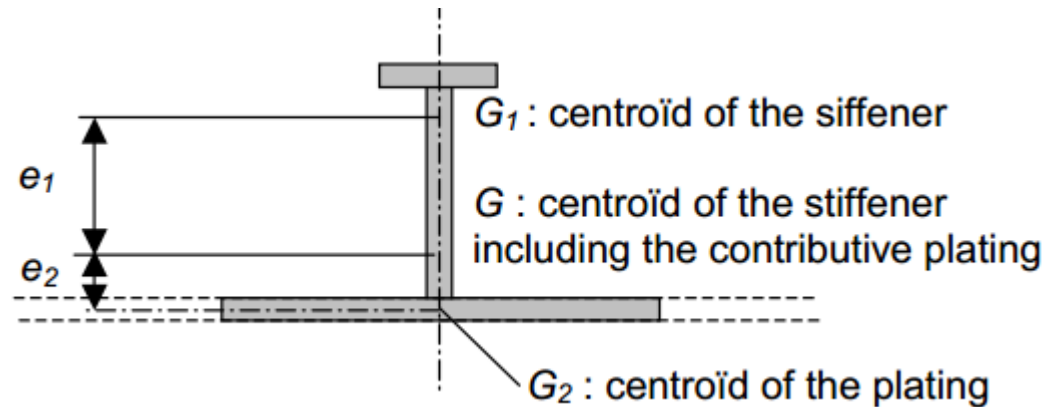
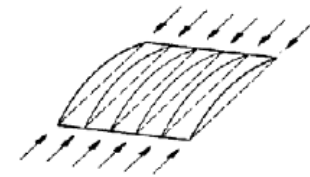
$$\bar{\lambda}_c = \sqrt{\frac{\beta_{A,c} \cdot f_y}{\sigma_{cr,c}}}$$

$$\beta_{A,c} = \frac{A_{sl,1,eff}}{A_{sl,1}}$$

$$\chi_c = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}_c^2}}$$

Reduction factor from flexural buckling curves with increased imperfection factor

Column-like behaviour



$$\alpha_e = \alpha + \frac{0,09}{i/e}$$

with
$$i = \sqrt{\frac{I_{sl,1}}{A_{sl,1}}}$$

$e = \max(e_1, e_2)$ is the largest distance from the respective centroids of the plating and the one-sided stiffener (or of the centroids of either set of stiffeners when present on both sides) to the neutral axis of the effective column, see Figure A.1;

$\alpha = 0,34$ (curve b) for closed section stiffeners;

$= 0,49$ (curve c) for open section stiffeners.



Interpolation between plate and column-like buckling

a) Plate like behaviour:

$$\lambda_p = \sqrt{\frac{\beta_{A,c} f_y}{\sigma_{cr,p}}}$$

$\sigma_{cr,p}$

ρ

b) Column like behaviour:

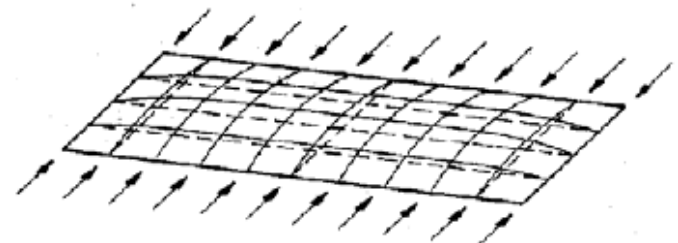
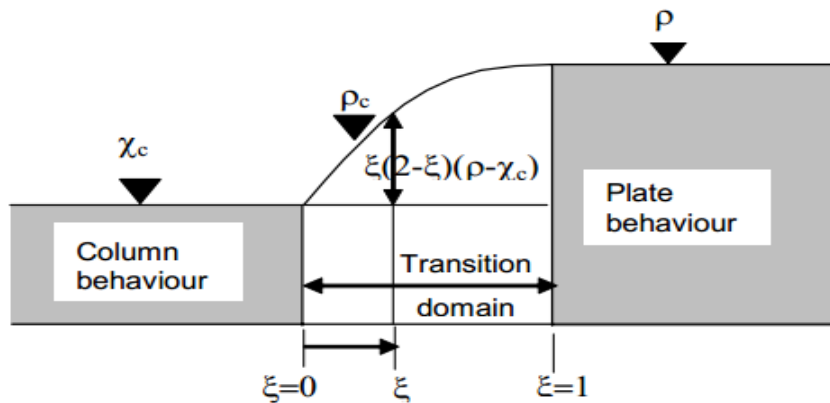
$$\lambda_c = \sqrt{\frac{\beta_{A,c} f_y}{\sigma_{cr,c}}}$$

$$\sigma_{cr,c} = \frac{\pi^2 EI_{sl,1}}{A_{sl,1} a^2}$$

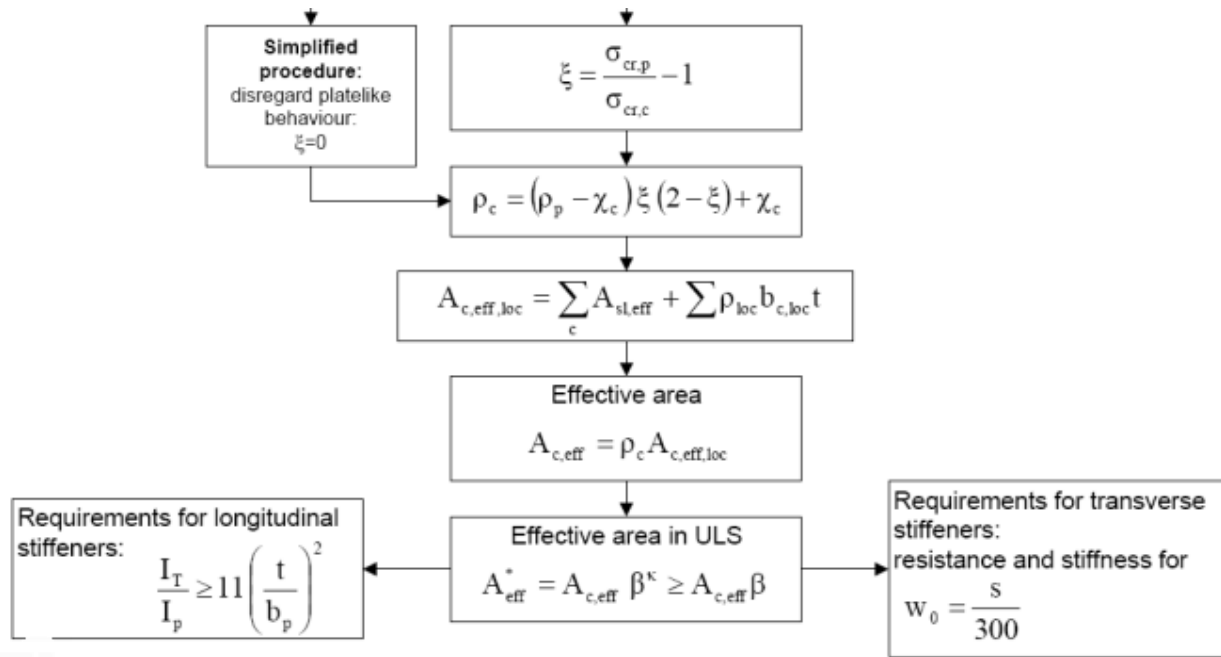
χ_c

c) interpolation:

$$\rho_c = (\rho - \chi_c)\xi(2 - \xi) + \chi_c \quad \xi = \frac{\sigma_{cr,p}}{\sigma_{cr,c}} - 1 \quad (0 \leq \xi \leq 1)$$



Verification



$$\eta_1 = \frac{\frac{N_{Ed}}{f_y A_{eff}}}{\gamma_{M0}} + \frac{\frac{M_{Ed} + N_{Ed} e_N}{f_y W_{eff}}}{\gamma_{M0}} \leq 1,0$$

Shear buckling

$$\bar{\lambda}_w = 0,76 \sqrt{\frac{f_{yw}}{\tau_{cr}}}$$

$$\tau_{cr} = k_\tau \sigma_E$$

1) Sub-panel:

$$\bar{\lambda}_w = \frac{h_{wi}}{37,4 t \varepsilon \sqrt{k_{ti}}}$$

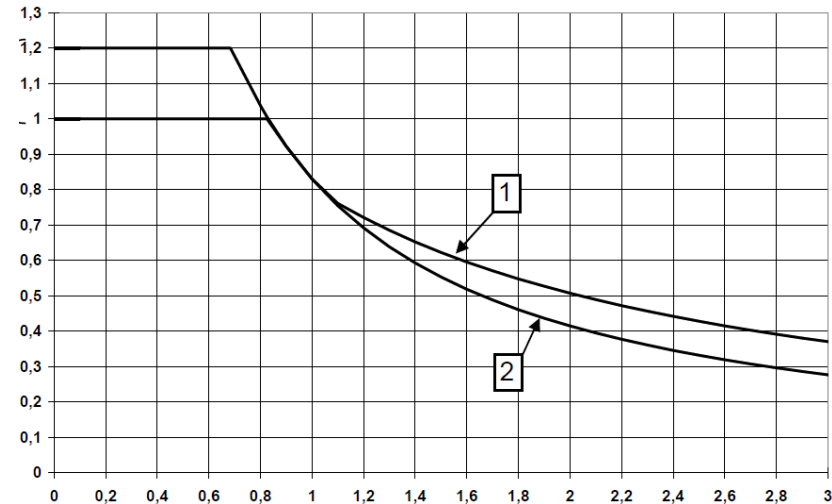
2) Whole panel:

$$\bar{\lambda}_w = \frac{h_w}{37,4 t \varepsilon \sqrt{k_\tau}}$$

$$k_\tau = 5,34 + 4,00 (h_w / a)^2 + k_{\tau st}$$

$$k_\tau = 4,00 + 5,34 (h_w / a)^2 + k_{\tau st}$$

additional factor
covering stiffness



Shear buckling factor

$$\text{where } k_{\tau\ell} = 9 \left(\frac{h_w}{a} \right)^2 \sqrt[4]{\left(\frac{I_{s\ell}}{t^3 h_w} \right)^3} \quad \text{but not less than } \frac{2,1}{t} \sqrt[3]{\frac{I_{s\ell}}{h_w}}$$

a is the distance between transverse stiffeners (see Figure 5.3);

$I_{s\ell}$ is the second moment of area of the longitudinal stiffener about the z-z axis, see Figure 5.3 (b).

For webs with two or more longitudinal stiffeners, not necessarily equally spaced, $I_{s\ell}$ is the sum of the stiffness of the individual stiffeners.

NOTE: No intermediate non-rigid transverse stiffeners are allowed for in equation (A.5).

(2) The equation (A.5) also applies to plates with one or two longitudinal stiffeners, if the aspect ratio $\alpha = \frac{a}{h_w}$ satisfies $\alpha \geq 3$. For plates with one or two longitudinal stiffeners and an aspect ratio $\alpha < 3$ the shear buckling coefficient should be taken from:

$$k_{\tau} = 4,1 + \frac{6,3 + 0,18 \frac{I_{s\ell}}{t^3 h_w}}{\alpha^2} + 2,2 \sqrt[3]{\frac{I_{s\ell}}{t^3 h_w}} \quad (\text{A.6})$$

M-V interaction

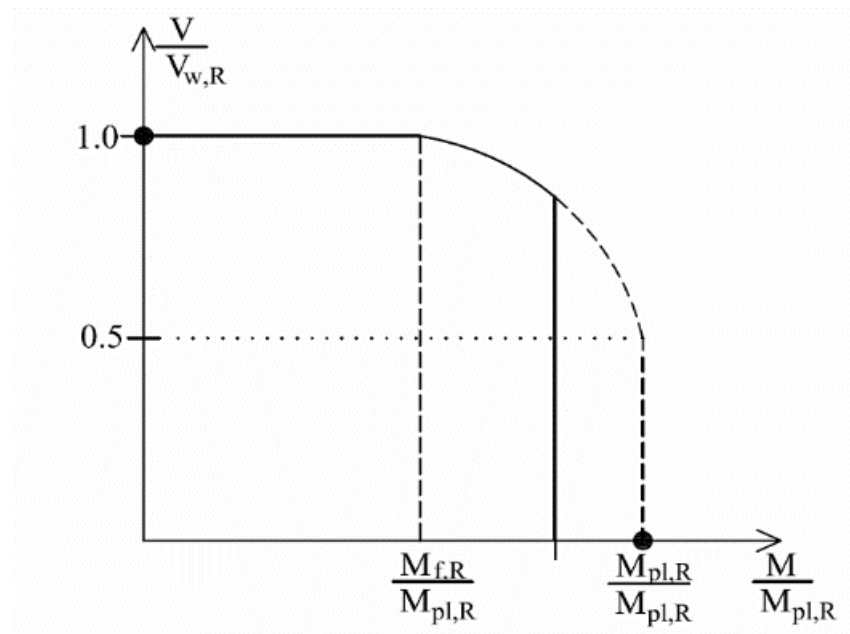
$$\bar{\eta}_1 + \left(1 - \frac{M_{f,Rd}}{M_{pl,Rd}}\right) (2\bar{\eta}_3 - 1)^2 \leq 1,0 \quad \text{for } \bar{\eta}_1 \geq \frac{M_{f,Rd}}{M_{pl,Rd}} \quad (7.1)$$

where $M_{f,Rd}$ is the design plastic moment of resistance of the section consisting of the effective area of the flanges;

$M_{pl,Rd}$ is the design plastic resistance of the cross section consisting of the effective area of the flanges and the fully effective web irrespective of its section class.

$$\bar{\eta}_1 = \frac{M_{Ed}}{M_{pl,Rd}}$$

$$\bar{\eta}_3 = \frac{V_{Ed}}{V_{bw,Rd}}$$



M-F interaction

$$\eta_2 + 0.8 \cdot \eta_1 \leq 1.4$$

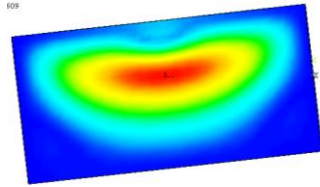
$$\eta_1 = \frac{N_{Ed}}{f_y \cdot A_{eff}} + \frac{M_{Ed} + N_{Ed} \cdot e_N}{f_y \cdot W_{eff}} \leq 1.0$$

$$\gamma_{M0}$$

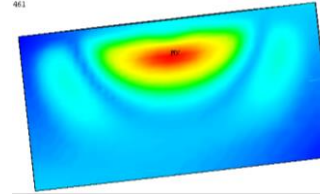
$$\eta_2 = \frac{F_{Ed}}{f_{yw} \cdot L_{eff} \cdot t_w} \leq 1.0$$

$$\gamma_{M1}$$

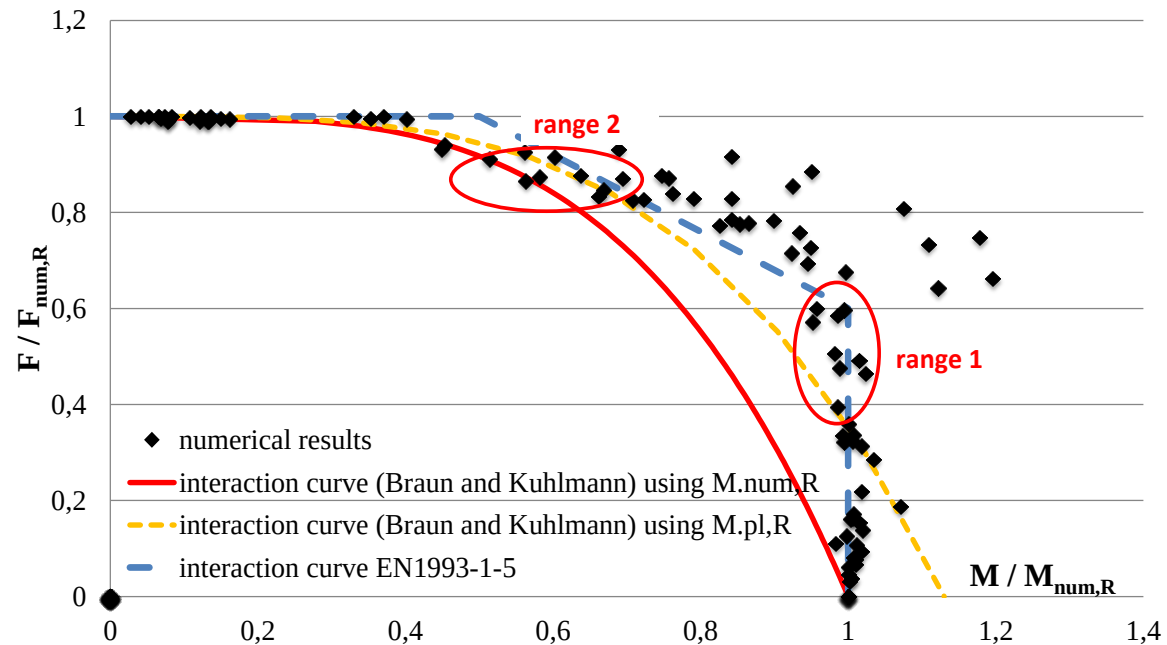
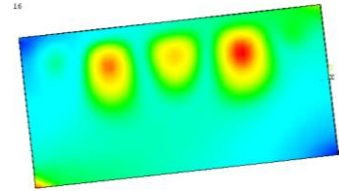
Patch loading



Interaction



Bending





FEM based design

Current design rules – EN 1993-1-5 Annex C

Annex C is informative

C.1 General

(1) Annex C gives guidance on the use of FE-methods for ultimate limit state verification of plated structures.

(2) The choice of the FE-method depends on the problem to be analyzed and based on the following assumptions:

Table C.1: Assumptions for FE-methods

No	Material behaviour	Geometric behaviour	Imperfections, see section C.5	Example of use
1	linear	linear	no	elastic shear lag effect, elastic resistance
2	non linear	linear	no	plastic resistance in ULS
3	linear	non linear	no	critical plate buckling load
4	linear	non linear	yes	elastic plate buckling resistance
5	non linear	non linear	yes	elastic-plastic resistance in ULS

Current design rules – EN 1993-1-5 Annex C

C.2 Use

(1) In using FEM for design special care should be taken to

- the modelling of the structural component and its boundary conditions;
- the choice of software and documentation;
- the use of imperfections;
- the modelling of material properties;
- the modelling of loads;
- the modelling of limit state criteria;
- the partial factors to be applied.

Current design rules – EN 1993-1-5 Annex C

C.3 Modelling

- (1) The choice of FE-models (shell models or volume models) and the size of mesh determine the accuracy of results. **For validation sensitivity checks** with successive refinement may be carried out.
- (2) The FE-modelling may be carried out either for:
 - the **component as a whole** or
 - a **substructure** as a part of the whole structure.
- (3) The boundary conditions for **supports**, interfaces and applied loads should be chosen such that results obtained are conservative.
- (4) **Geometric properties** should be taken as **nominal**.
- (5) **All imperfections** should be based on the shapes and amplitudes as given in section C.5.
- (6) **Material properties** should conform to C.6(2).

Current design rules – EN 1993-1-5 Annex C

C.4 Choice of software and documentation

- (1) The software should be suitable for the task and be proven reliable (benchmark tests, examples).
- (2) The mesh size, loading, boundary conditions and other input data as well as the output should be documented in a way that they can be reproduced by third parties.

Current design rules – EN 1993-1-5 Annex C

C.5 Use of imperfections

- (1) Where imperfections need to be included in the FE-model these imperfections should include both **geometric and structural imperfections**.
- (2) Unless a more refined analysis of the geometric imperfections and the structural imperfections is carried out, **equivalent geometric imperfections** may be used.

Geometric imperfections may be based on the shape of the critical plate buckling modes with amplitudes given in the National Annex. 80 % of the geometric fabrication tolerances is recommended.

- (3) The **direction** of the applied **imperfection** should be such that the lowest resistance is obtained.
- (4) For applying equivalent geometric imperfections Table C.2 and Figure C.1 may be used.
- (5) In combining imperfections a **leading imperfection** should be chosen and the **accompanying imperfections** may have their values reduced to 70%.

Current design rules – EN 1993-1-5 Annex C

Imperfections

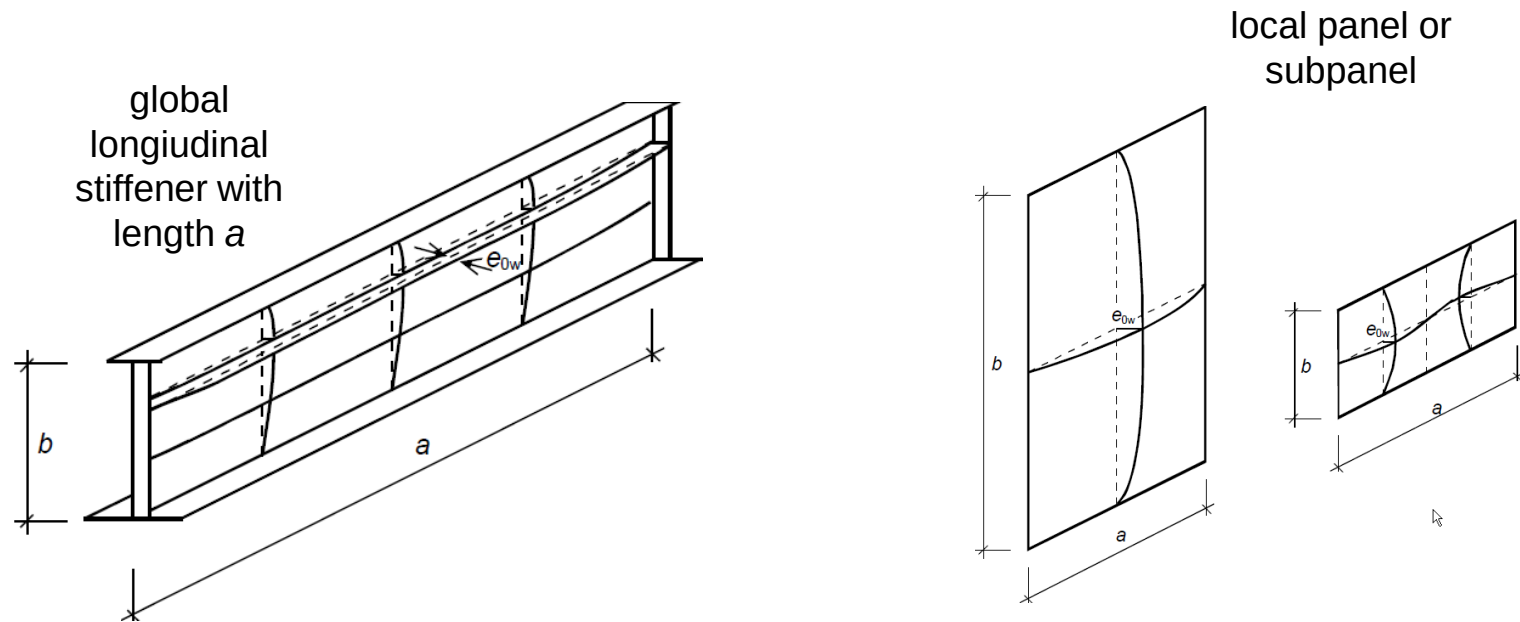


Table C.2: Equivalent geometric imperfections

Type of imperfection	Component	Shape	Magnitude
global	member with length ℓ	bow	see EN 1993-1-1, Table 5.1
global	longitudinal stiffener with length a	bow	$\min (a/400, b/400)$
local	panel or subpanel with short span a or b	buckling shape	$\min (a/200, b/200)$
local	stiffener or flange subject to twist	bow twist	$1 / 50$

Current design rules – EN 1993-1-5 Annex C

C.6 Material properties

(1) Material properties should be taken as characteristic values.

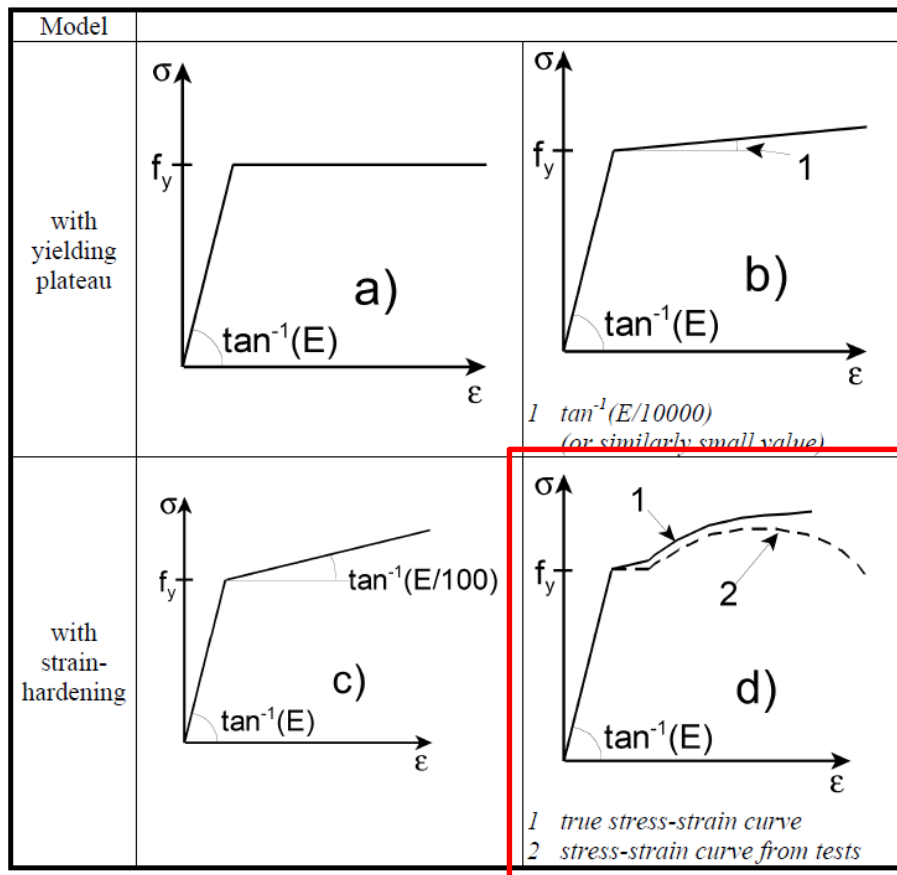


Figure C.2: Modelling of material behaviour

Current design rules – EN 1993-1-5 Annex C

C.7 Loads

(1) The loads applied to the structures should include relevant load factors and load combination factors. For simplicity a single load multiplier α may be used.

C.8 Limit state criteria

(1) The ultimate limit state criteria should be used as follows:

1. for structures susceptible to buckling:
attainment of the maximum load.
2. for regions subjected to tensile stresses:
attainment of a limiting value of the **principal membrane strain (5%)**.

Current design rules – EN 1993-1-5 Annex C

C.9 Partial factors

- (1) The load magnification factor α_u to the ultimate limit state should be sufficient to achieve the required reliability.
- (2) The magnification factor α_u should consist of two factors as follows:
 1. α_1 to cover the model uncertainty of the FE-modelling used. It should be obtained from evaluations of test calibrations, see Annex D to EN 1990;
 2. α_2 to cover the scatter of the loading and resistance models. It may be taken as γ_{M1} if instability governs and γ_{M2} if fracture governs.
- (3) It should be verified that:

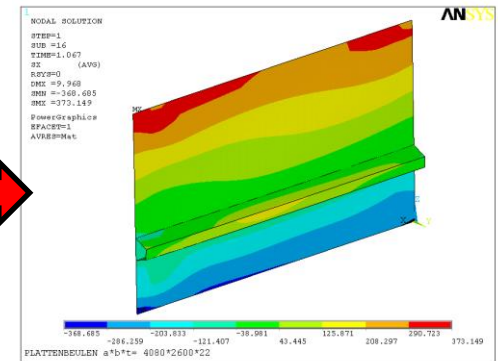
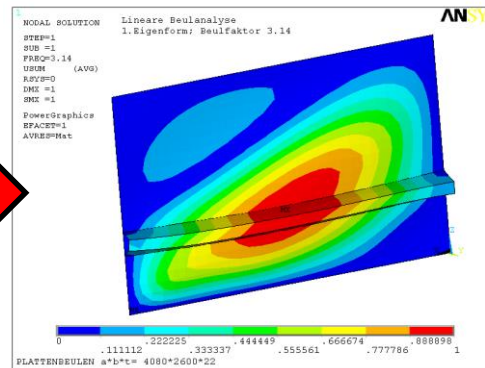
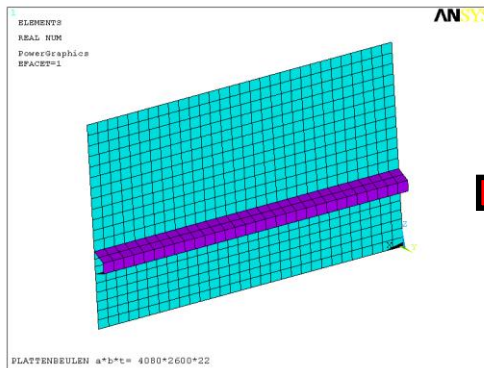
$$\alpha_u > \alpha_1 \cdot \alpha_2$$

Current design rules – EN 1993-1-5 Annex C

1. Linear Analysis (LA)

2. Linear Buckling Analysis (LBA)

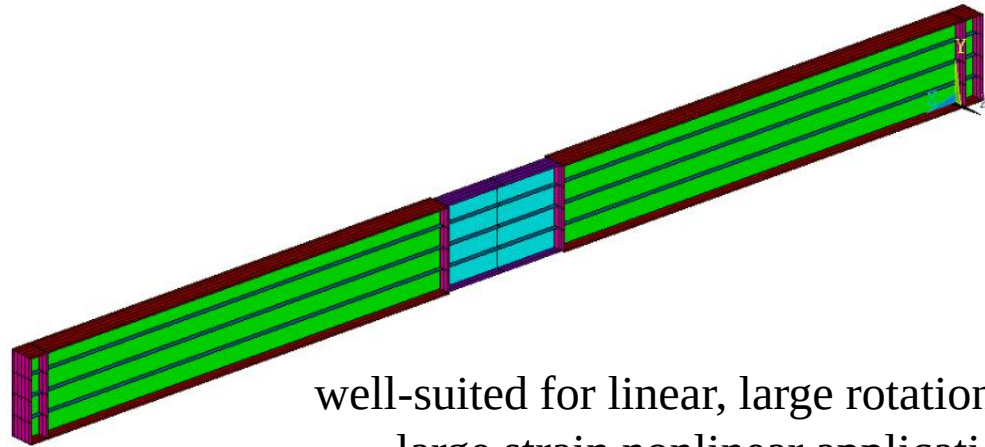
3. Geometric and Material Nonlinear Analysis with Imperfections (GMNIA)



LA + LBA

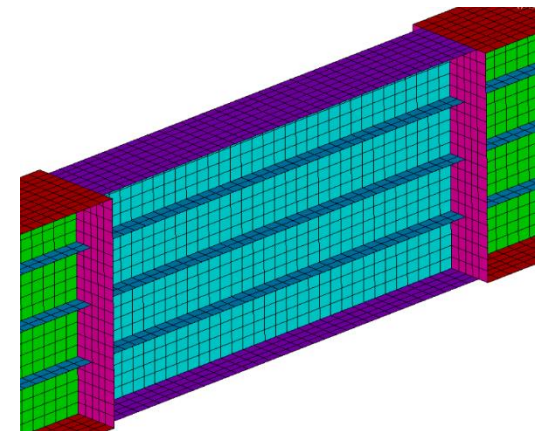
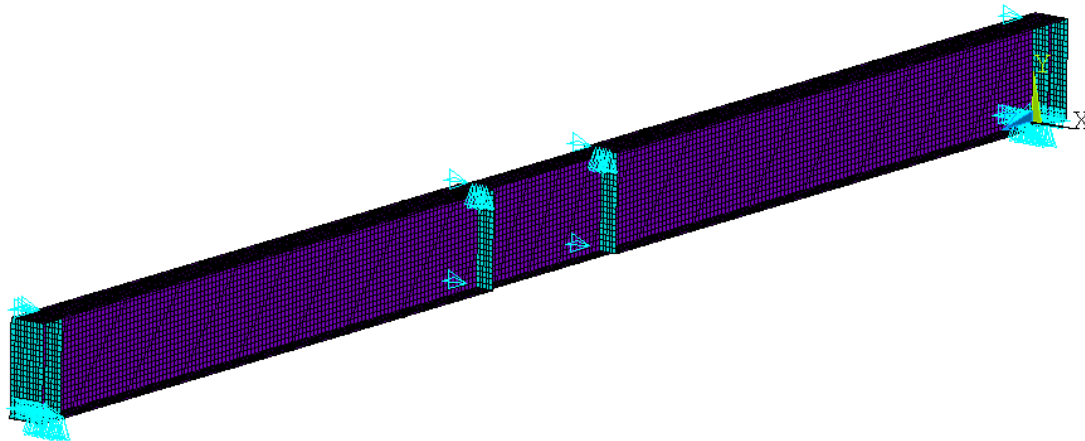
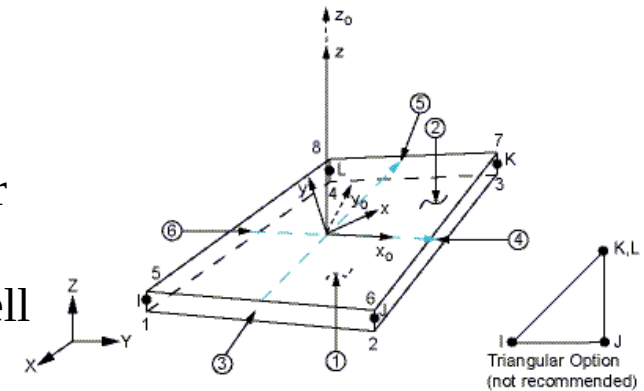
GMNIA
(Imperfections affine to
modeshapes from LBA)

Developed numerical model

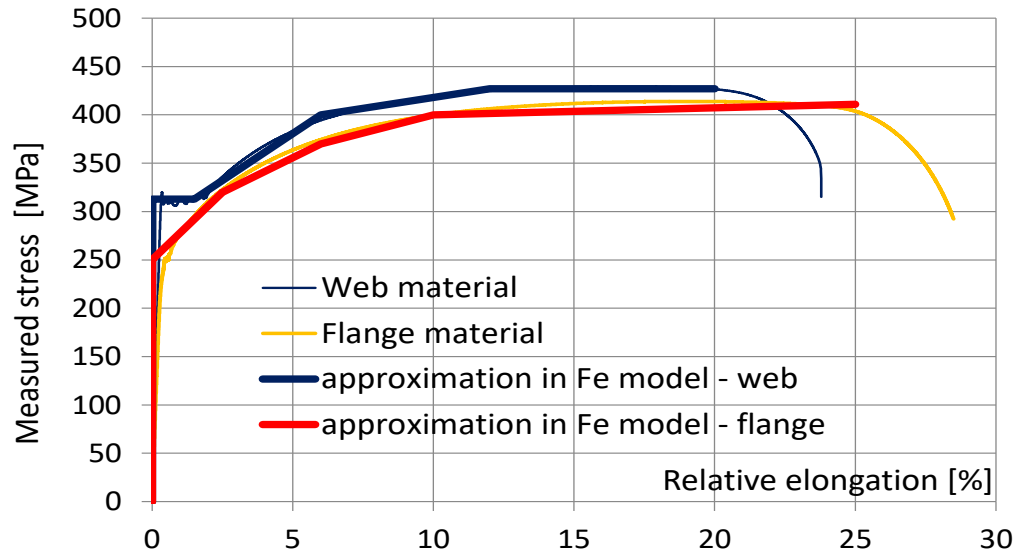


- full shell model
- eight node thin shell elements

well-suited for linear, large rotation, and/or large strain nonlinear applications;
capable to model thin and thick plates as well



Applied material model and boundary conditions

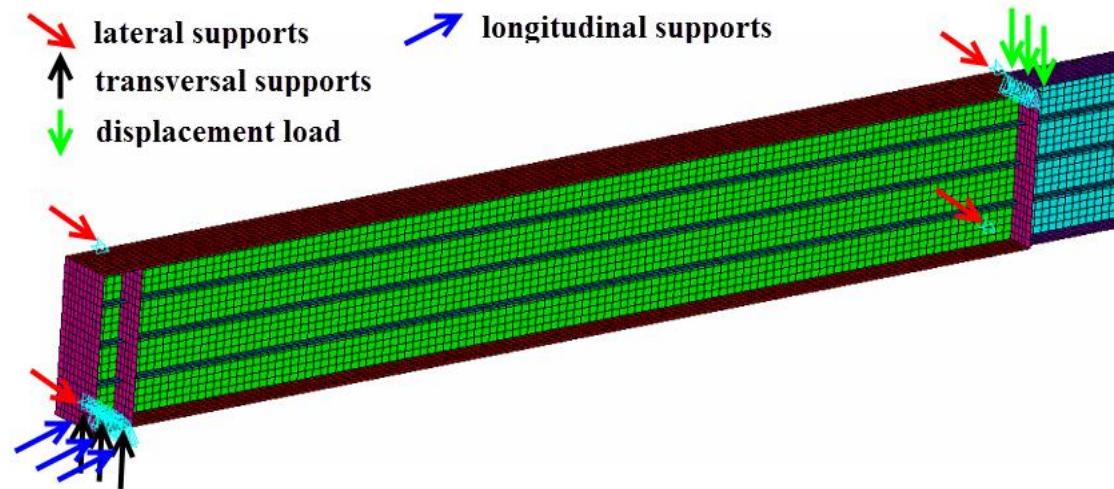


Material models

linear elastic – hardening plastic material;
multi-linear approach of the real material
behaviour;
isotropic hardening rule is used with
von Mises yield criterion;
separated model for web and flange

Boundary conditions

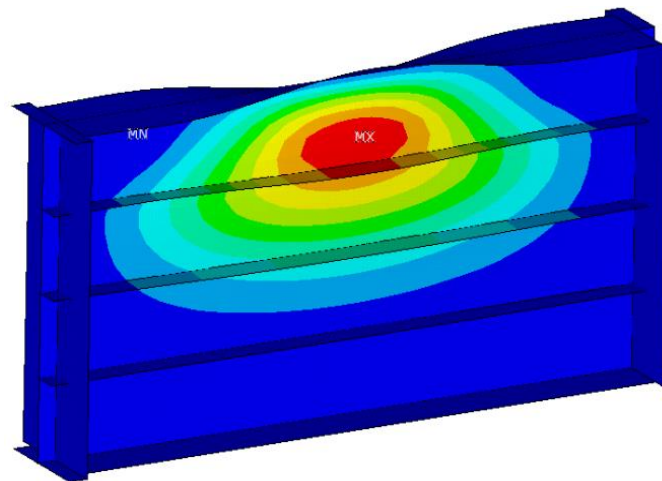
identical with the supports
used in the test



Applied imperfections

Three different approaches, according to the consideration of the imperfections

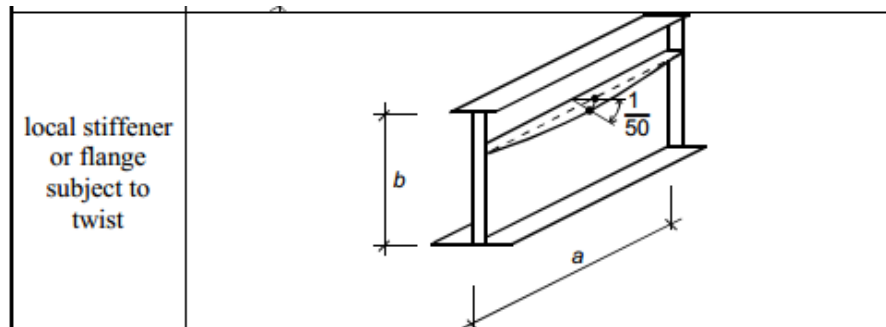
Approach 1: At first the **first eigenmode shape** is applied as equivalent geometric imperfection, with an amplitude of **$h_w/200$** . The reason of the imperfection magnitude selection is that the indicated failure mode based on the eigenmode shape is global buckling of the web. The first eigenmode shape contains accompanying flange local buckling and rotation of the longitudinal stiffeners. The first eigenshape is scaled-up to have the maximum amplitude as proposed by the EN1993-1-5



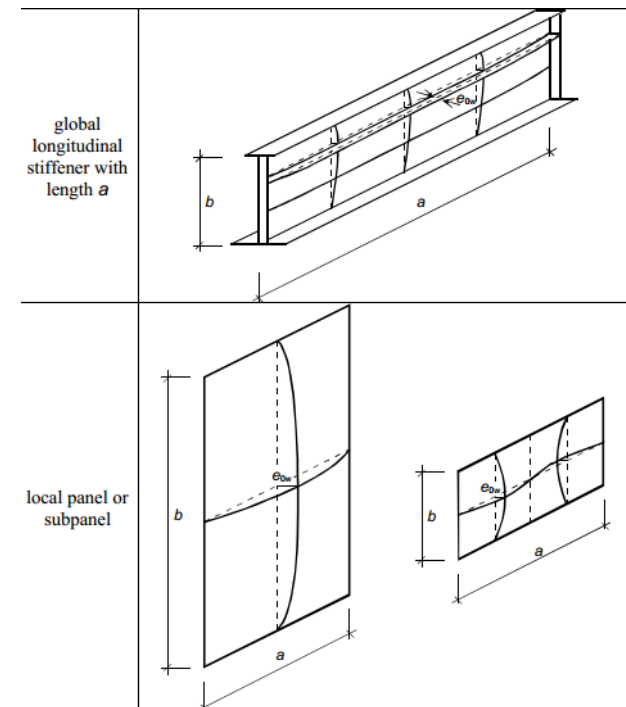
Applied imperfections

Approach 2: The second applied imperfection is the **equivalent geometric imperfections** using hand defined imperfections shapes. Three different hand-defined imperfection shapes are applied in the numerical model:

- global imperfection of the web panel ($h_w/200$),
- local imperfection of the web sub-panels ($b_1/200$),
- rotation of the longitudinal stiffeners ($1/50$).

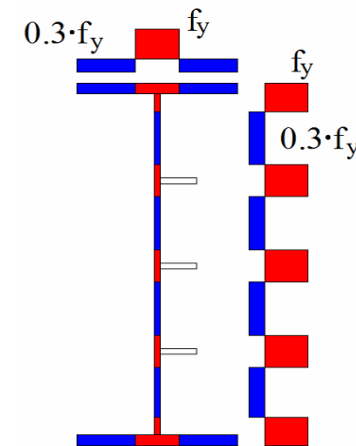
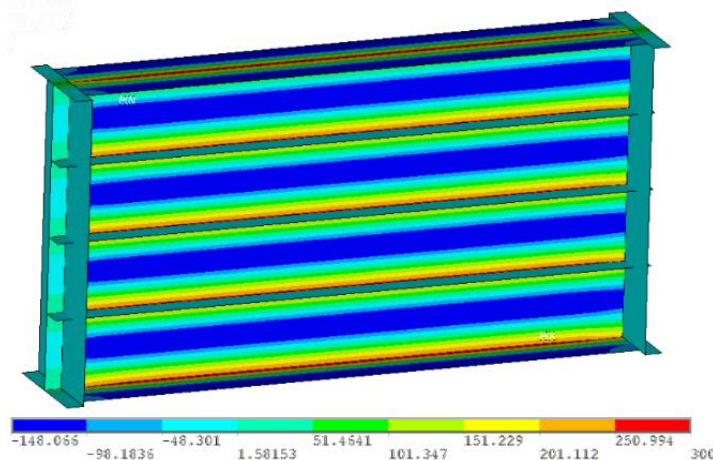


The global imperfection is selected as leading imperfection, thus the failure mode is the closest to this imperfection shape. Another two imperfection magnitudes are taken by their 70%.



Applied imperfections

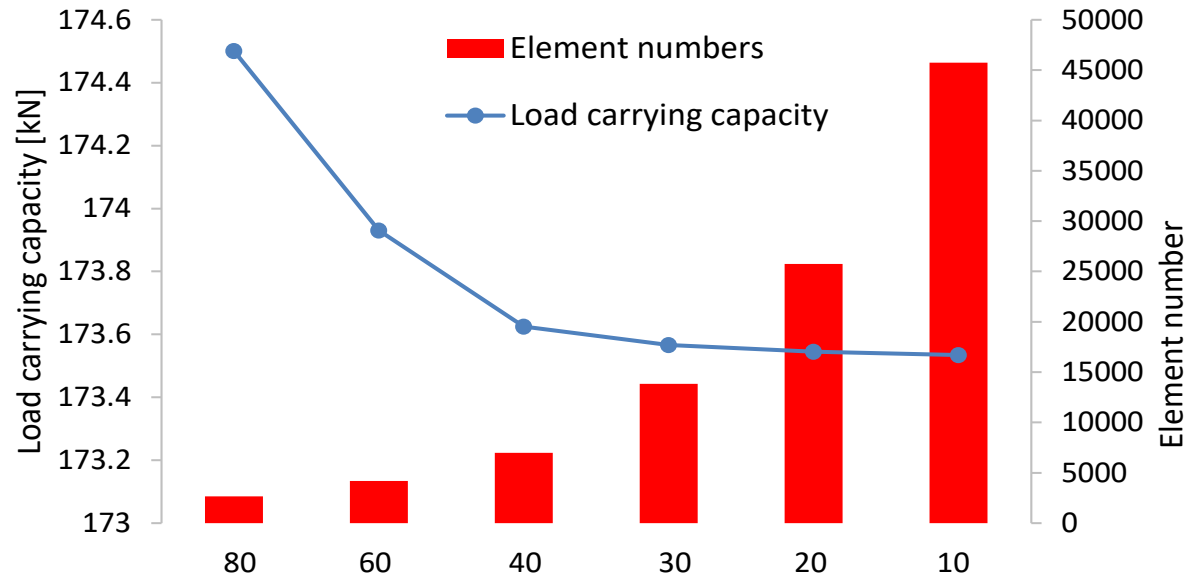
Approach 3: The third applied imperfection type is geometric imperfections with accompanying residual stresses. The residual stress pattern shown here is applied in the numerical model.



Beside the residual stresses geometric imperfections are also applied in the numerical model with magnitudes representing the manufacturing tolerances, as follows:

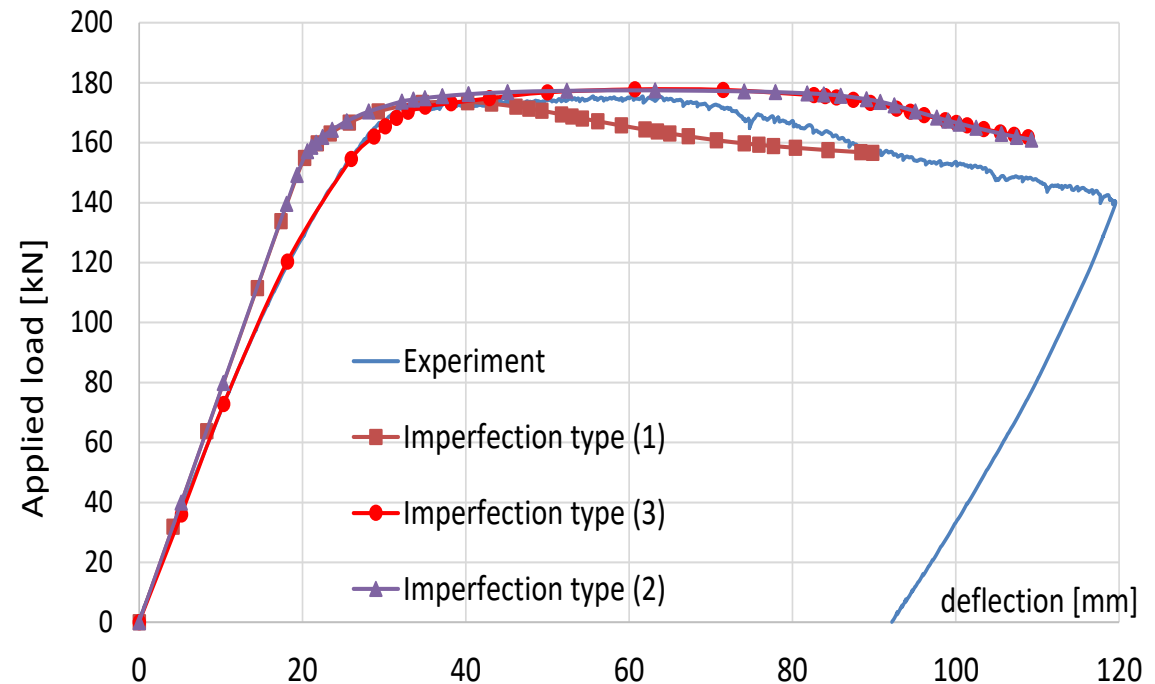
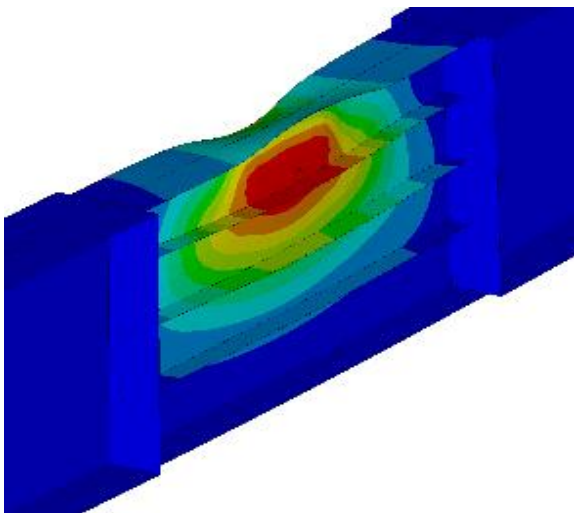
- global imperfection of the web panel ($h_w/1000$),
- local imperfection of the web sub-panels ($b_1/1000$),
- rotation of the longitudinal stiffeners ($1/500$).

Mesh sensitivity analysis



The ultimate loads are determined by geometrical and material nonlinear analysis using equivalent geometric imperfections. Mesh convergence study is also executed to determine the appropriate finite element sizes.

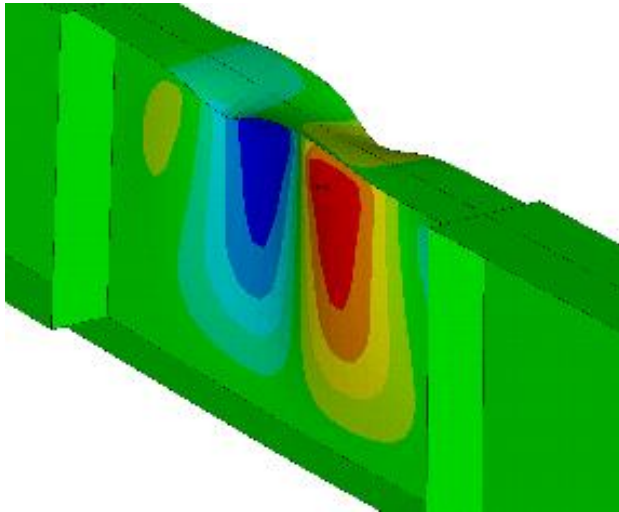
Model validation



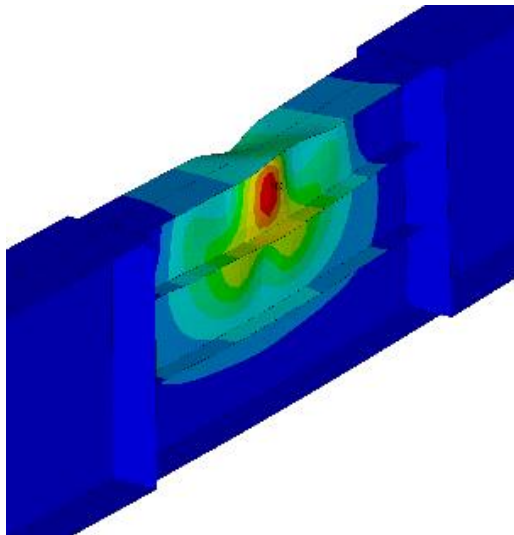
The results show that the ultimate load can be calculated using all the three imperfection types with high accuracy. There are differences, however, in the load-displacement diagrams depending on the applied imperfection shape.

Model validation

Unstiffened girder

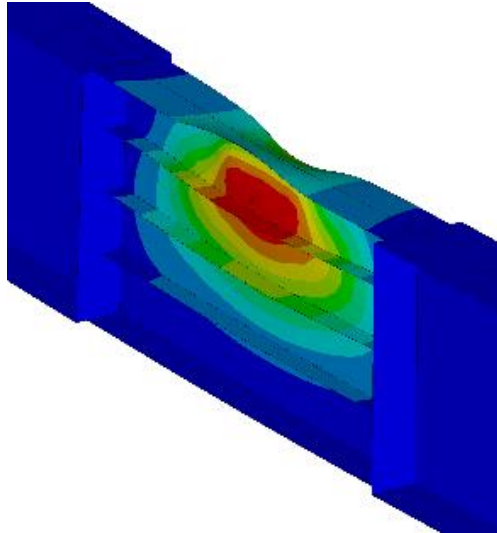


Two stiffeners

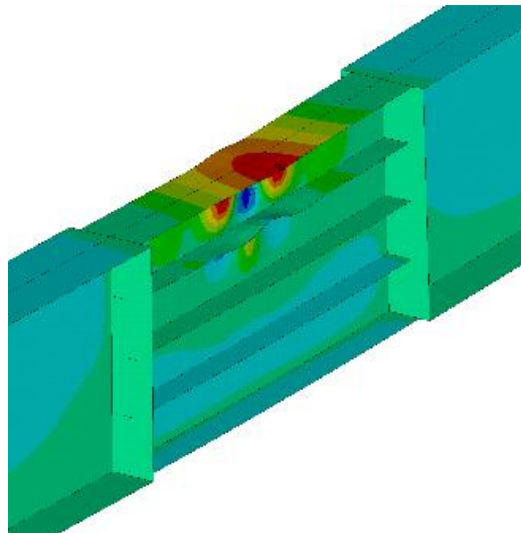


Model validation

Three stiffeners – V1

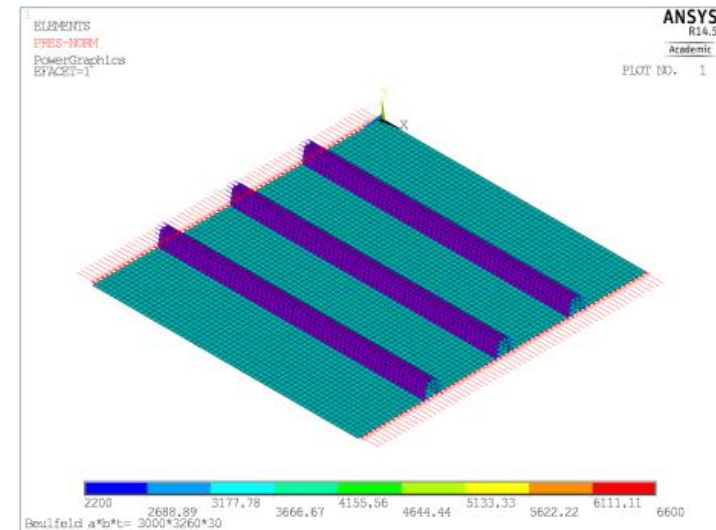
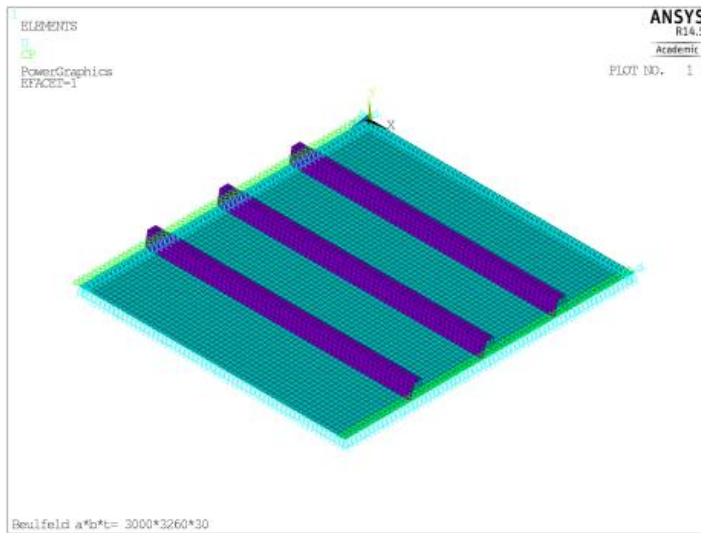


Three stiffeners – V2



Example 2

Linear Analysis



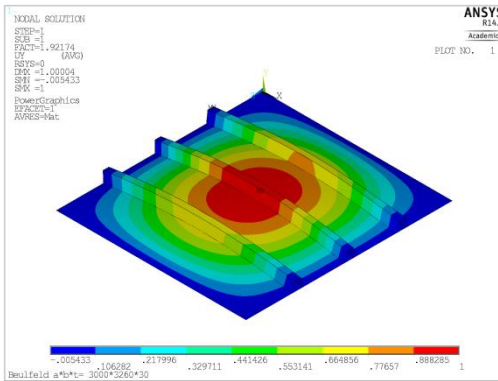
Mesh, boundary conditions and loading.

The meshing of the geometry is done by using 4 node shell elements, 6 degrees of freedom (DOF) at each node and full integration (SHELL181, KEYOPT 3=2).

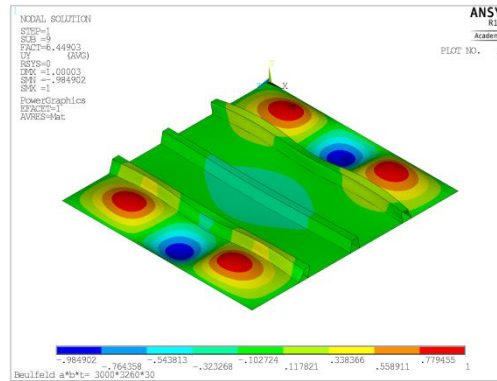
Imperfection combinations

Some interesting modeshapes from LBA

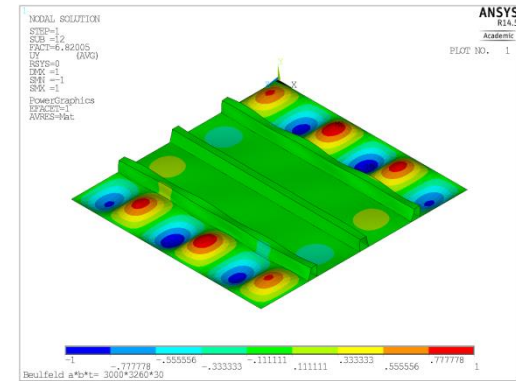
→ Leading imperfection and one (or more ?) accompanying imperfections.



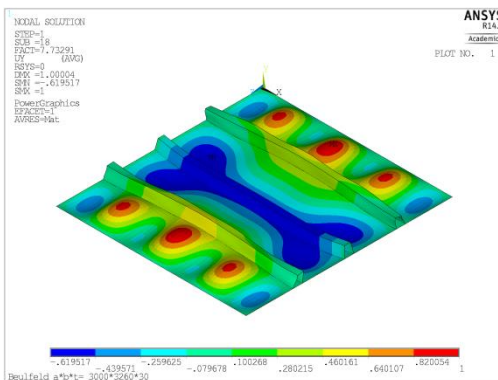
Mode 1



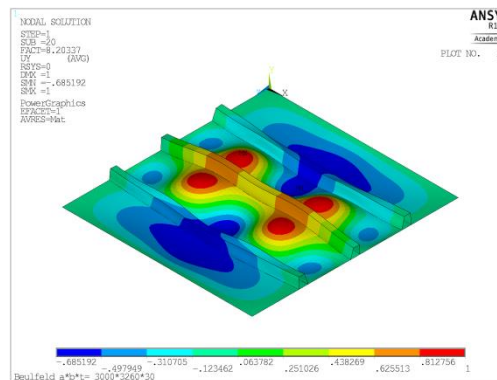
Mode 9



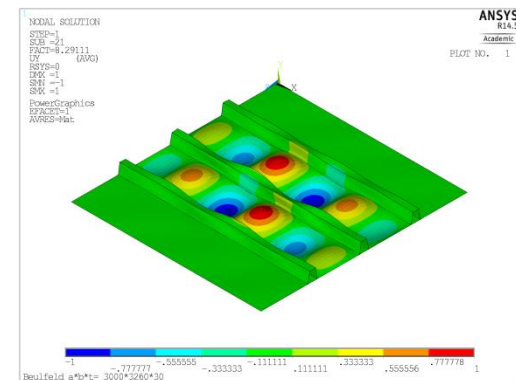
Mode 12



Mode 18



Mode 20



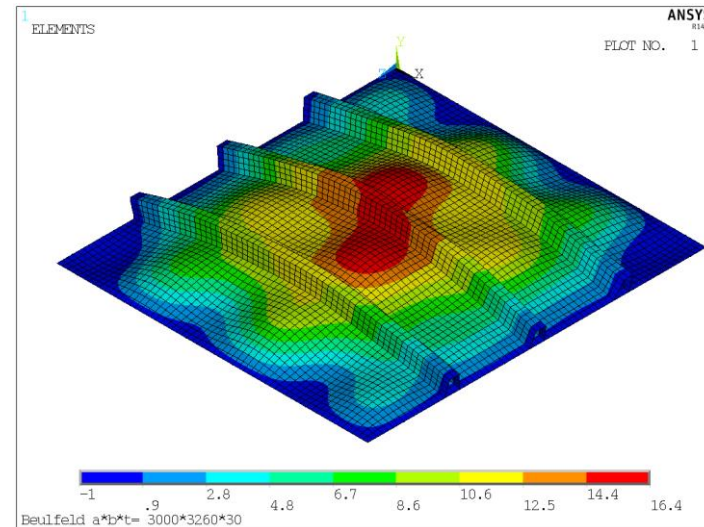
Mode 21

Imperfection combinations

Nonlinear buckling analysis (GMNIA)

$$e_{global} = \min\left(\frac{a_{global}}{200}, \frac{b_{global}}{200}\right) = \frac{3000\text{mm}}{200} = 15.0\text{mm}$$

$$e_{local} = \min\left(\frac{a_{local}}{200}, \frac{b_{local}}{200}\right) = \frac{743\text{mm}}{200} = 3.7\text{mm}$$



Due to the high number of possible combinations, 4 plausible sets of combinations are tested:

set 1: $1.0 \times \text{Mode 1 (+15.0mm)} + 0.7 \times \text{Mode 9 (-2.6mm)} + 0.7 \times \text{Mode 20 (+2.6mm)}$

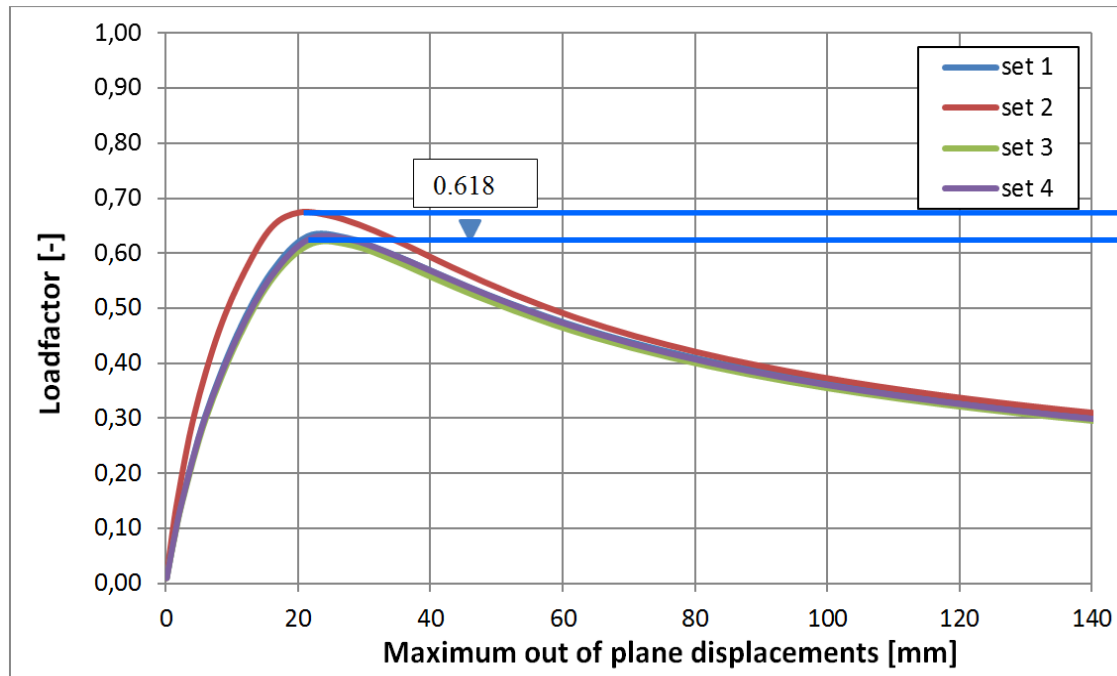
set 2: $0.7 \times \text{Mode 1 (+10.5mm)} + 1.0 \times \text{Mode 9 (-3.7mm)} + 1.0 \times \text{Mode 20 (+3.7mm)}$

set 3: $1.0 \times \text{Mode 1 (+15.0mm)} + 0.7 \times \text{Mode 12 (+2.6mm)} + 0.7 \times \text{Mode 21 (-2.6mm)}$

set 4: $1.0 \times \text{Mode 1 (+15.0mm)} + 0.7 \times \text{Mode 18 (+2.6mm)} + 0.7 \times \text{Mode 20 (+2.6mm)}$

Effect on ultimate resistance

Nonlinear buckling analysis (GMNIA)



$$\frac{\square 0.68}{\square 0.62} = \square 1.10$$

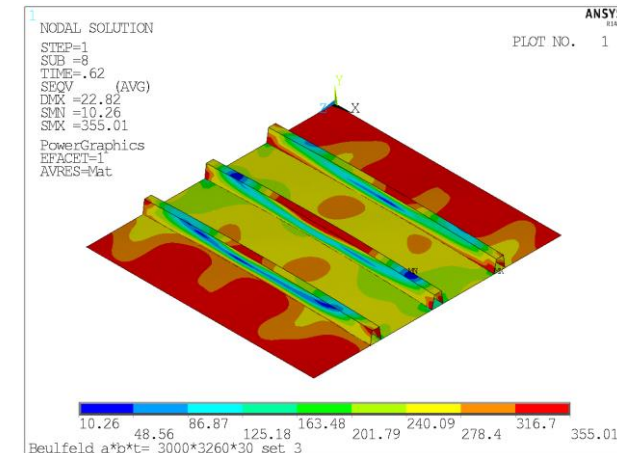
Deviation ~10%

$$\alpha_u = 2.0 \cdot 0.62 = 1.24$$

$$\sigma_{x,\max,Ed} = \frac{\alpha_u}{\alpha_1 \cdot \alpha_2} \cdot \sigma_{x,Ed} = 248 \text{ N/mm}^2$$

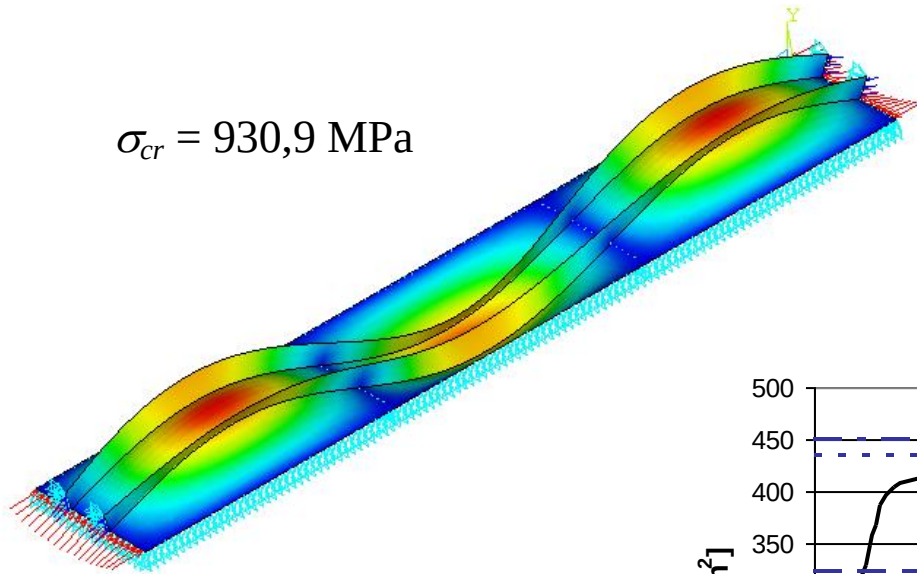
$$\alpha_u = 1.24 > \alpha_1 \cdot \alpha_2 = 1.05 \cdot 1.10 = 1.55$$

Verification o.k.



Example 3

$\sigma_{cr} = 930,9 \text{ MPa}$

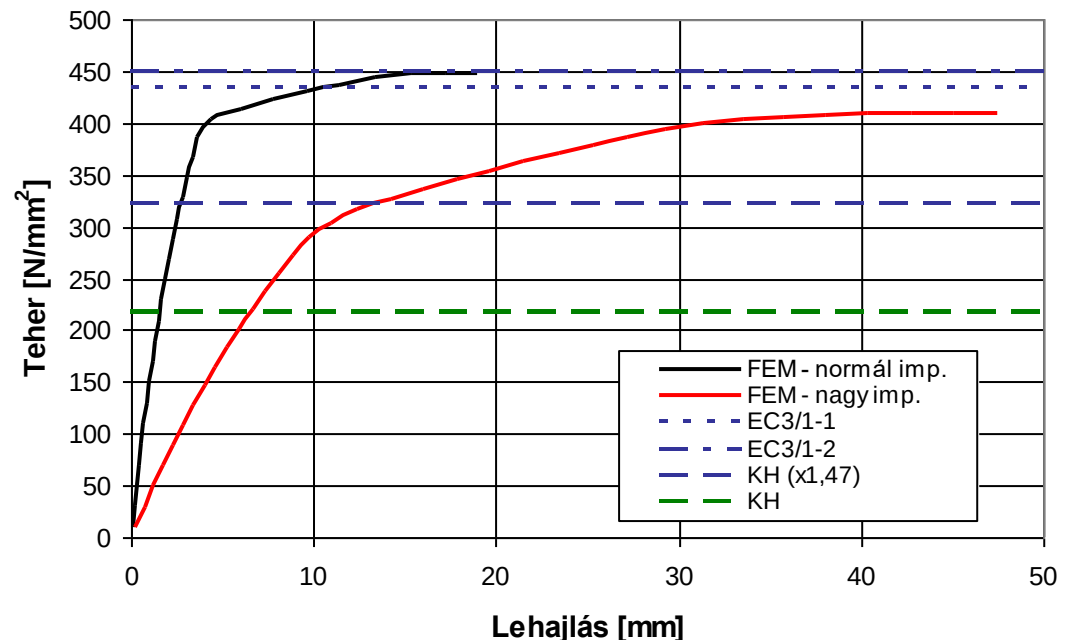


$$e_{0w,1} = \min(a/400, b/400) = 2000/400 = 5\text{mm}$$

$$e_{0x,2} = h_b \phi_0 = 280 \cdot \frac{1}{50} = 5,6\text{mm}$$

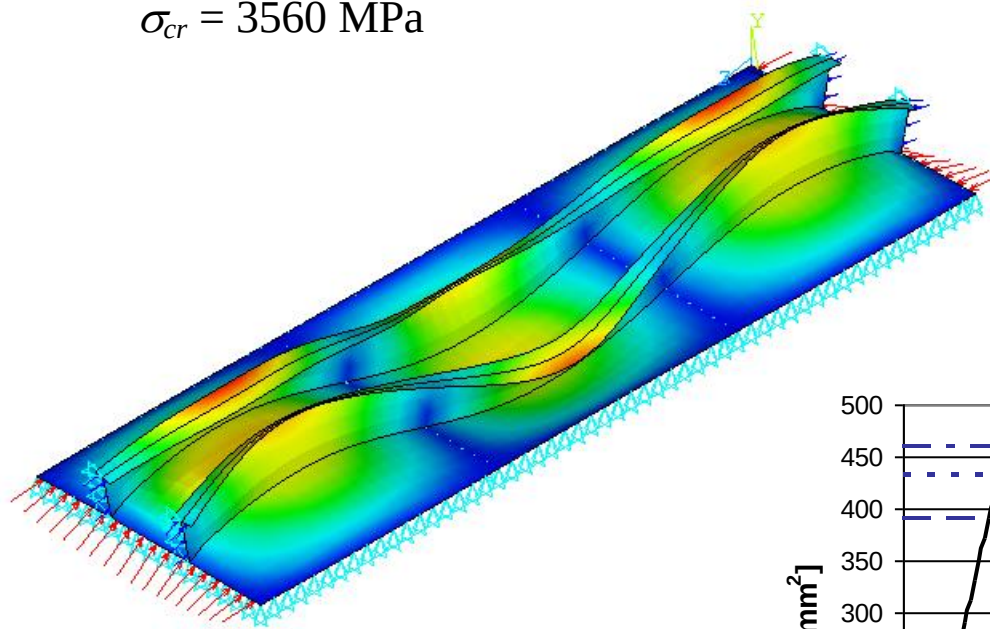
$$\alpha_{0,1} = \frac{5,00}{2,588 \cdot 10^{-3}} = 1932$$

$$\alpha_{0,2} = \frac{5,6}{0,661 \cdot 10^{-3}} = 8472$$



Example 4

$$\sigma_{cr} = 3560 \text{ MPa}$$

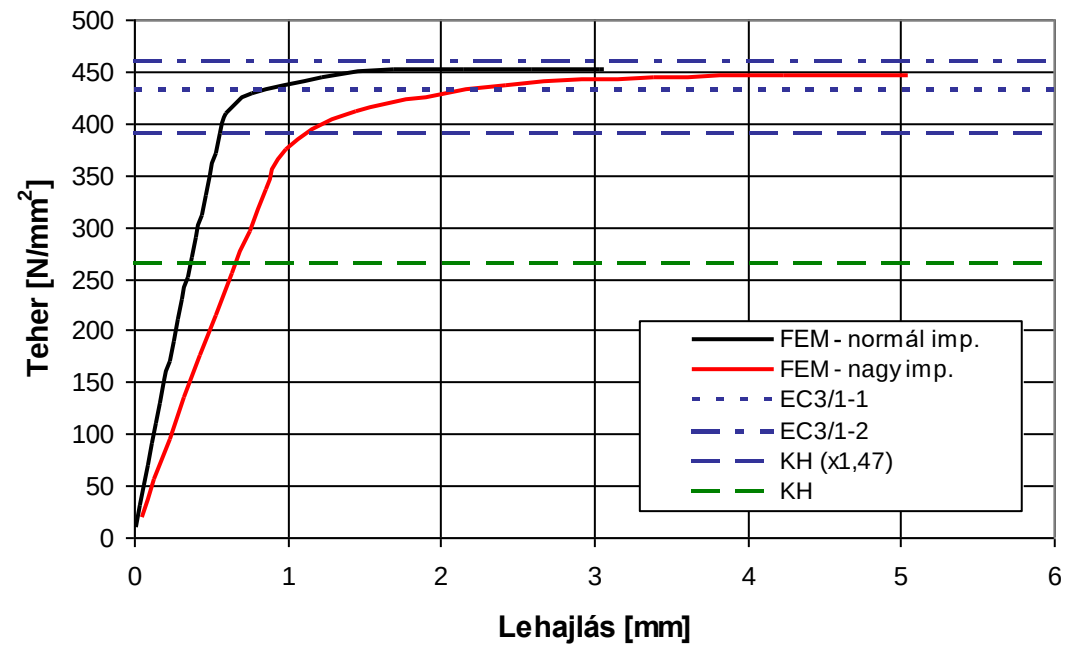


$$e_{0w,1} = \min(a/400, b/400) = 2000/400 = 5\text{mm}$$

$$e_{0x,2} = h_b \phi_0 = 270 \cdot \frac{1}{50} = 5,4\text{mm}$$

$$\alpha_{0,1} = \frac{5,00}{0,756 \cdot 10^{-3}} = 6614$$

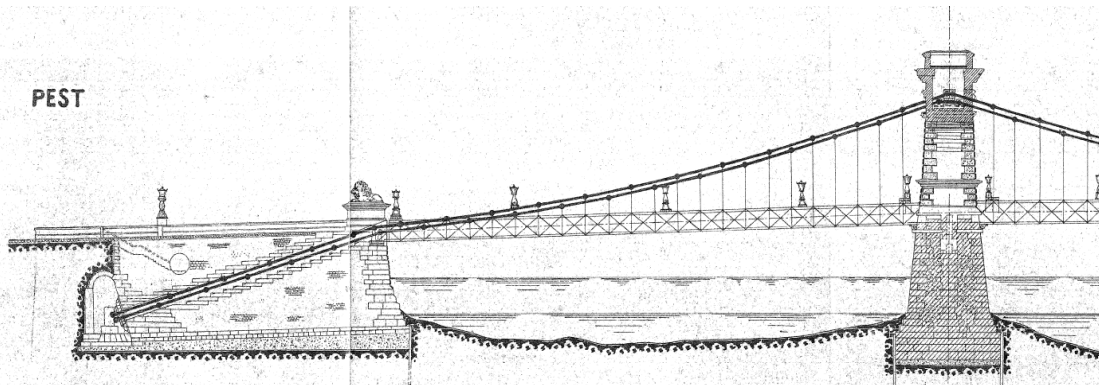
$$\alpha_{0,2} = \frac{5,4}{0,445 \cdot 10^{-3}} = 12135$$



Independent static check of the historical Széchenyi Chain Bridge



Structure of the Chain Bridge



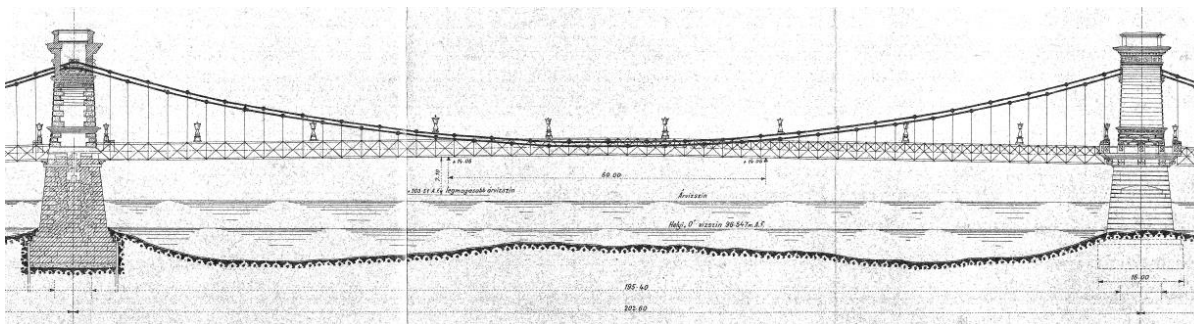
Suspension bridge with truss stringer

Symmetric arrangement

Side span: 88,71 m

Middle span: 202,60 m

Total length: 380,02 m



Chain pile:

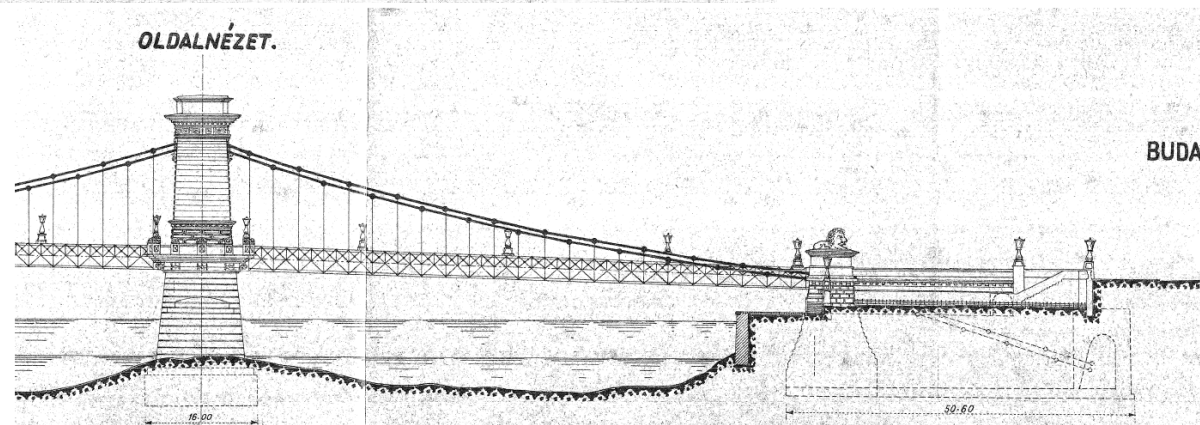
moving and fix bearer system

hinged anchorage

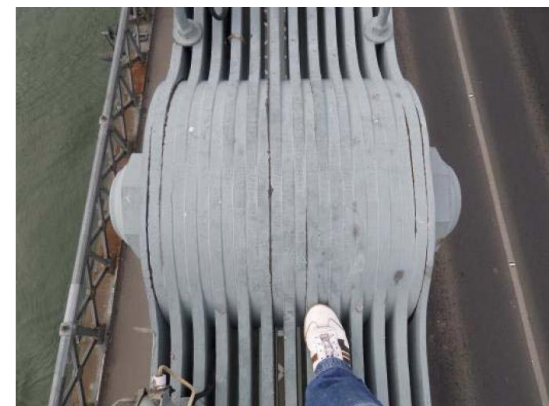
Stiffening girder:

fix abutment bearers

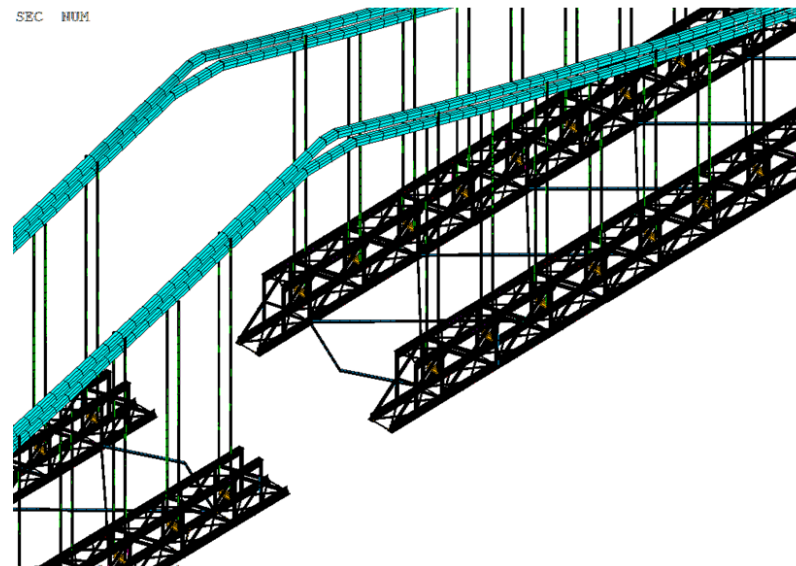
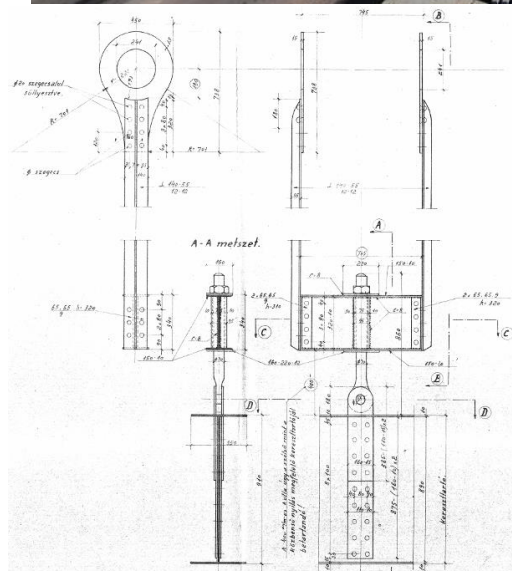
hinged bearers at piers



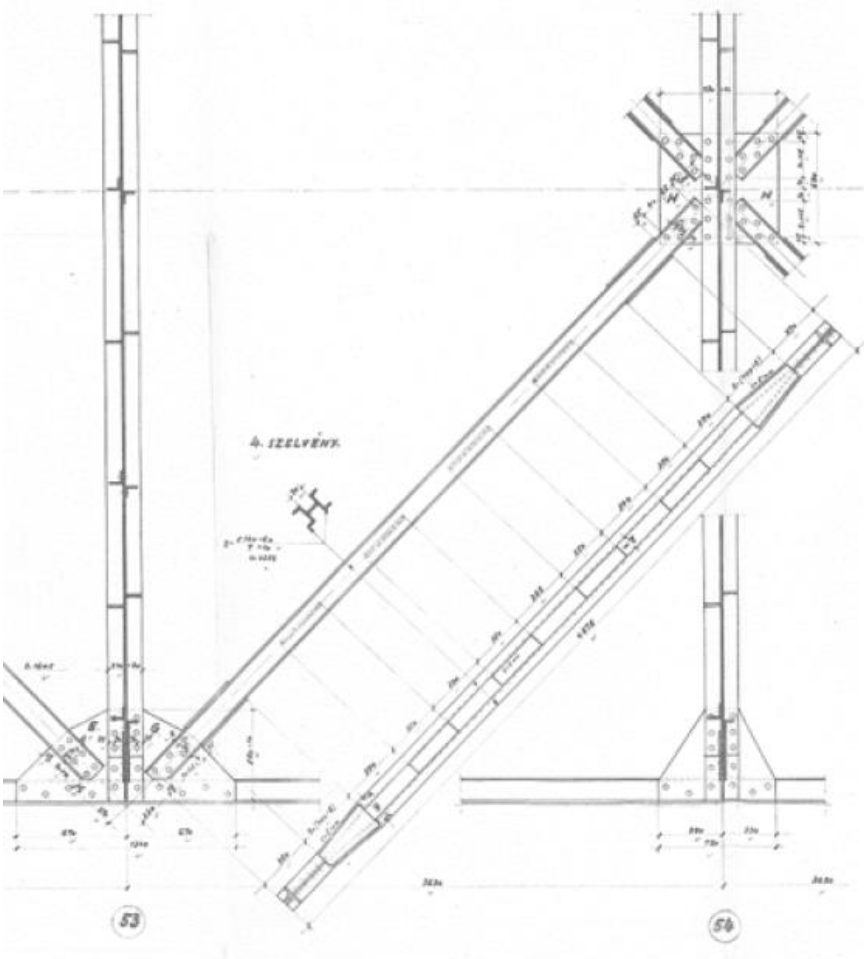
Structure of the Chain Bridge



25 chains in one intersection

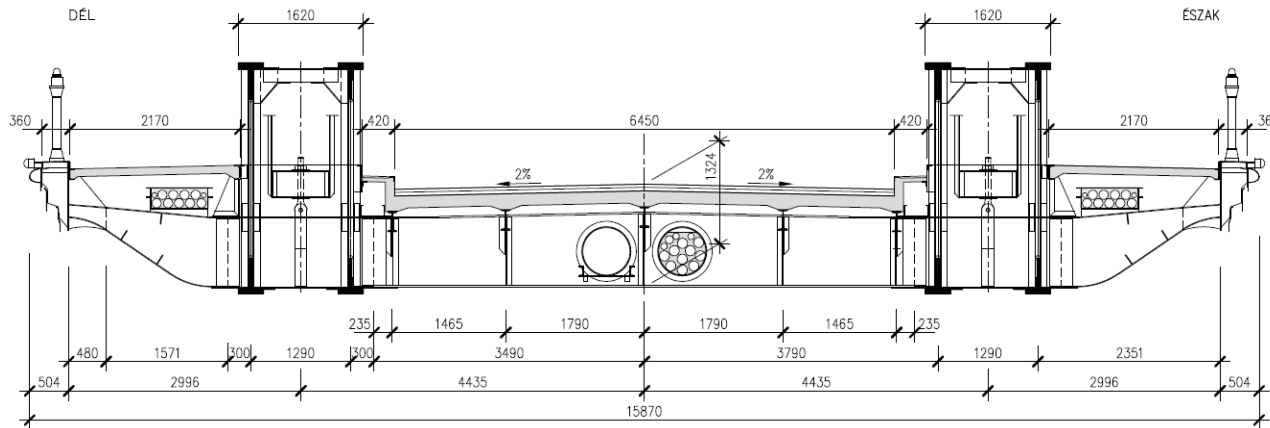


Structure of the Chain Bridge



Refurbishment on the bridge superstructure

Original cross-section



Pavement:

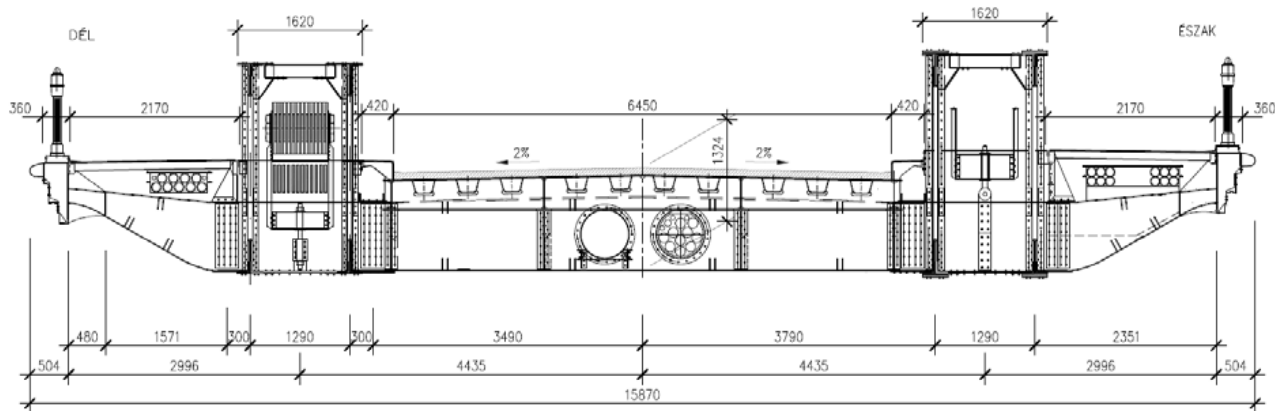
RC deck on 5 steel girders

Causes of refurbishment:

- 100 years old structure in its actual form
- corrosion on longitudinal stringers (loss of cross section up to 40 %)
- RC deck in critical state (especially at dilatations and kerbs)
- pavement soaks through

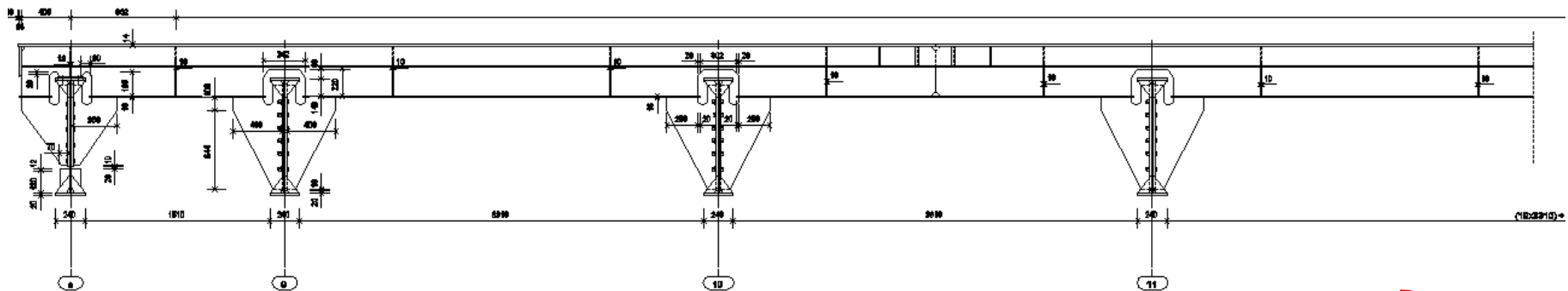
Refurbishment on the bridge superstructure

New cross-section

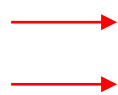


pavement:

orthotropic steel deck supported by 4 steel girders



Reduced self weight
decrease 8-10%



internal forces from self weight decrease
slight change in dynamic characteristics

not
significant

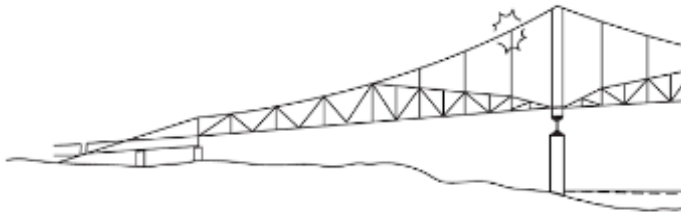
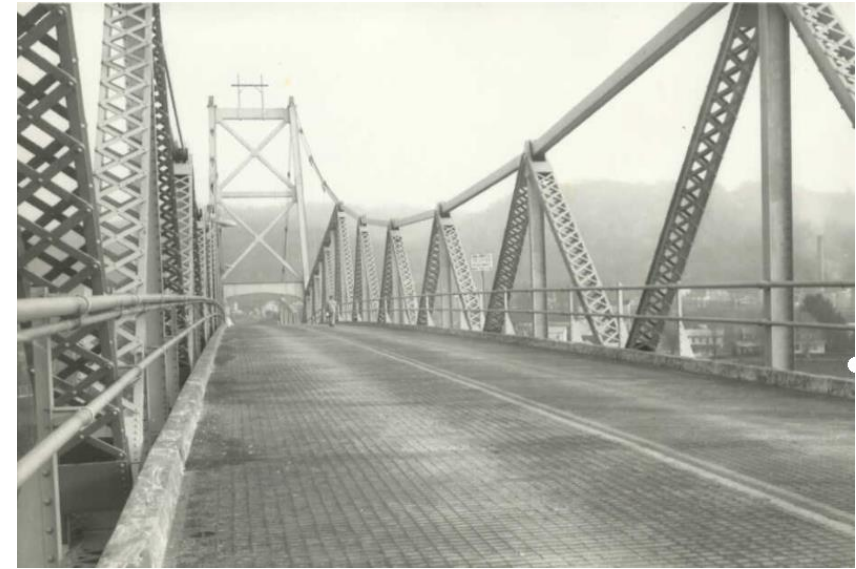
Stringer loses its stress-free state due to self weight

Failures on similar bridge structures

Silver bridge: 1928 - 1967

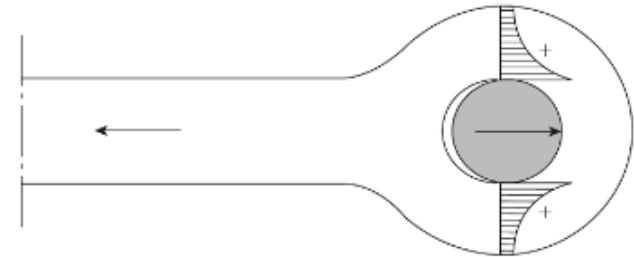
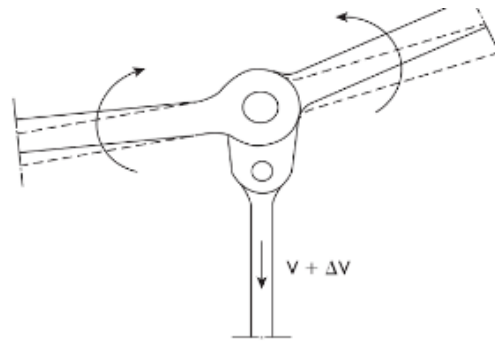
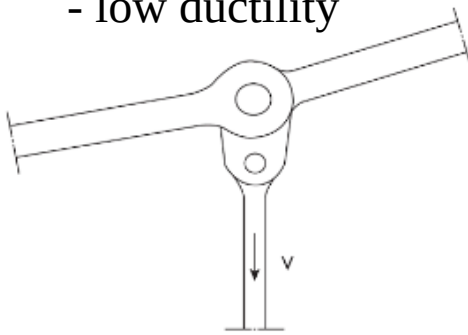


- three-span chain bridge
- over the Ohio river
- plummeted in the river in 1967



reason: - pin becomes fixed due to corrosion
- low redundancy level
- low ductility

→ fatigue crack (51 mm plate thickness)



Special focus on that in our analyses

Material properties

archive data:

A táncszínház 1913-14-ben gyártott
korrózióálló acél
korrózióálló acél (gyapjúszálak)

Szállási határ	Darab	Szállási határ	nyúlás %	Contractio %	Munka			
		Átlag	Átlag	Átlag	Átlag			
4400 - 5000	9	2670 - 3000	2840	195 - 265	237	40 - 59	50.8	1170
5000 - 5100	15	2650 - 3000	2850	18 - 29	24.0	36 - 58	51.1	1210
5100 - 5200	11	2650 - 3090	2980	21 - 29	24.2	42 - 54	50.6	1220
5200 - 5300	6	2740 - 3080	2860	18 - 26	21.4	35 - 51	45.5	1170
5300 - 5400								
5400 - 5500	4	2690 - 2970	2830	16 - 20	18.9	44 - 48	44.5	1010
5500 - 5600								
5140	47.4	2880	231	49.5	1190			

49.5% átlag, 23.1% átlag, 49.5% átlag

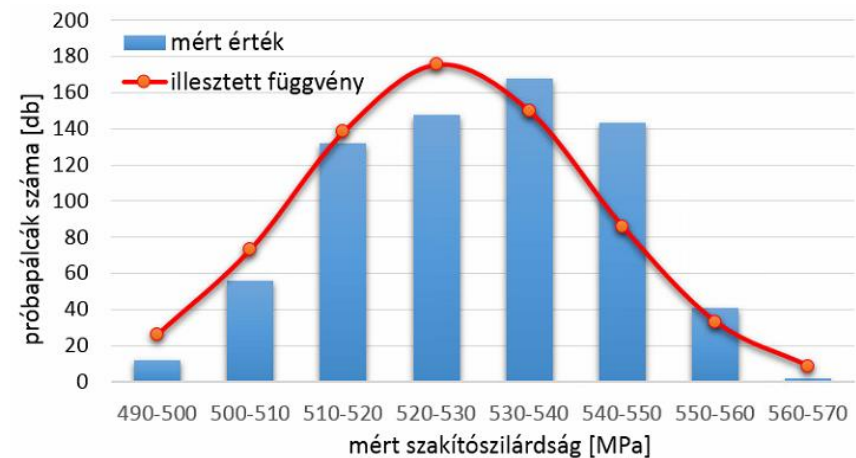
Total of 702 db tensile specimen's results on the material of the eyebar chains

Experimental based material properties:

Eyebars material	$f_{y,mean}$ [MPa]	$f_{y,k}$ [MPa]	$f_{u,mean}$ [MPa]	$f_{u,k}$ [MPa]
Based on measurements in 1913	296	281	529	503
Based on measurements in 1948	326	309	555	527

76% of eyebars material from the original bridge from 1915

24% made of carbon steel, produced in 1947-49

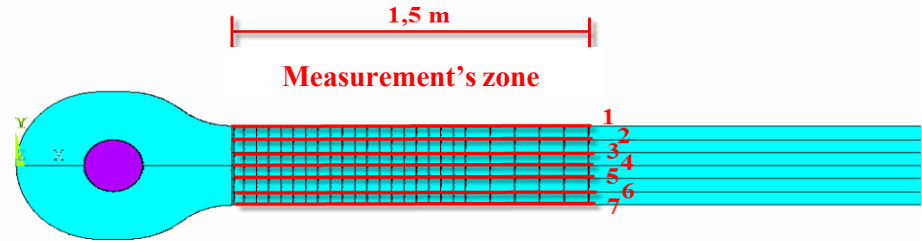


Allowed stresses in
I. loadgroup

Eyebars material	t=29mm
σ_{eng} [MPa]	187
τ_{eng} [MPa]	109

Corrosion analysis' reports

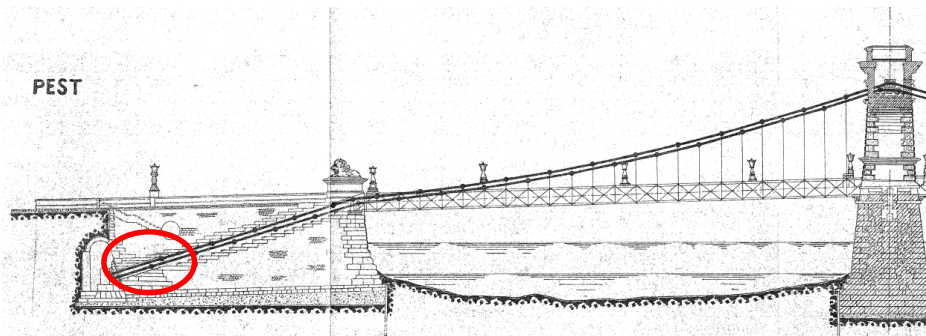
In 2002 at the Department of Structural Engineering



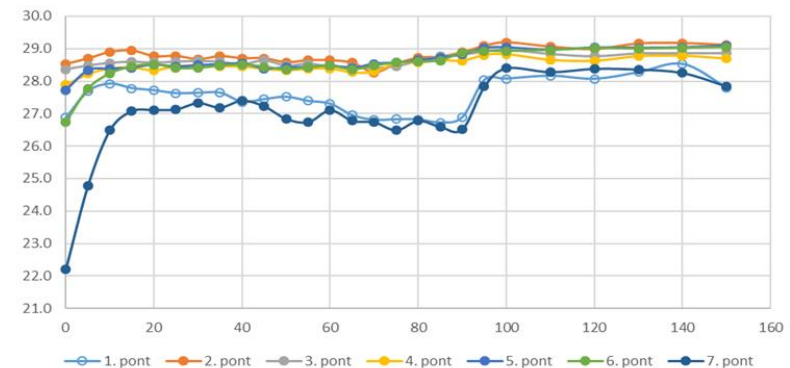
8 x 13 = 104 eyebar
on 1,5 m length
on 7 measurement paths
in 5 cm raster

plate thickness
measurement

Measurement's location

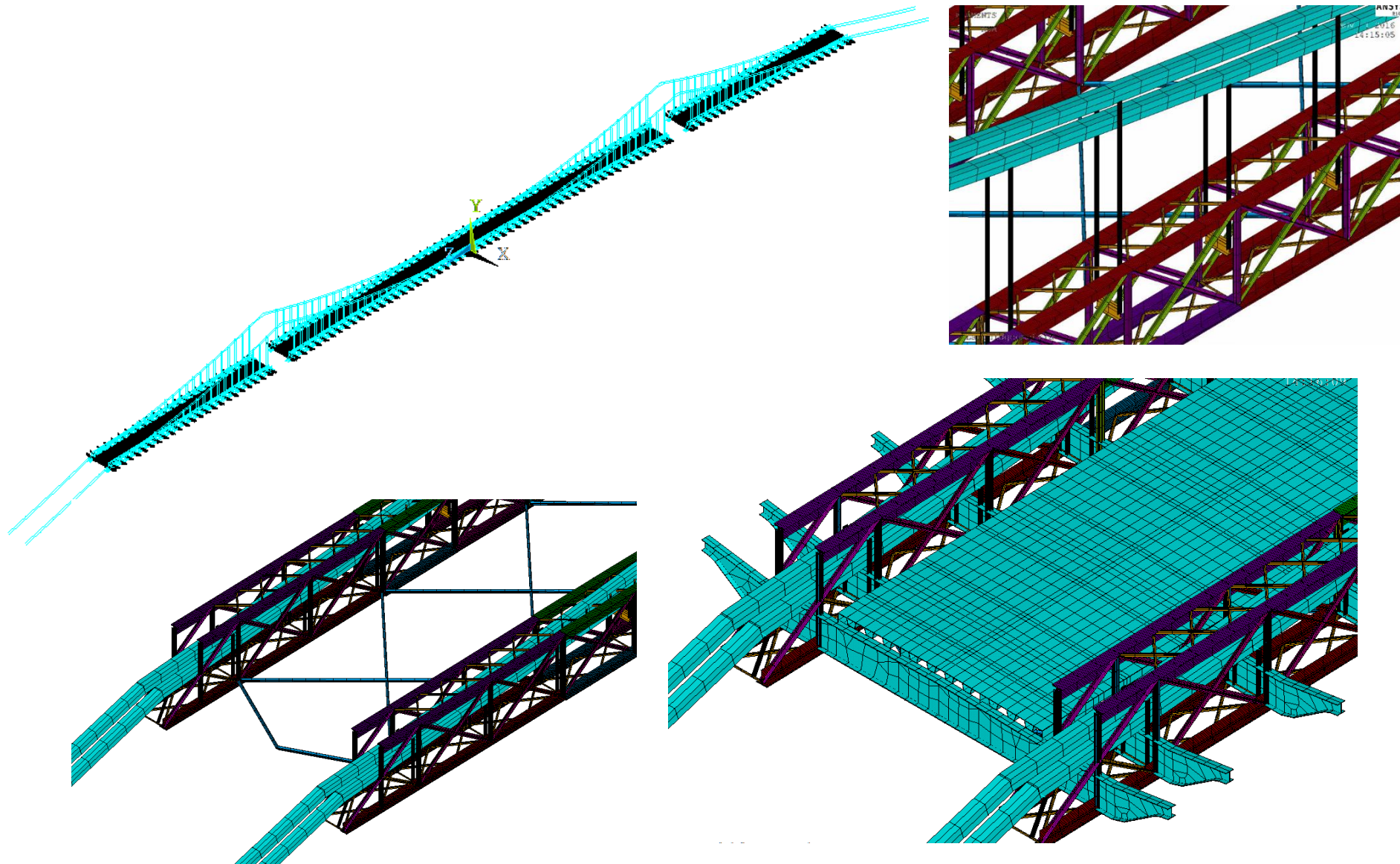


Measurement's results: 8-10% loss of material



Critical at the plate edges and at the anchorage

Applied numerical model



Applied numerical model

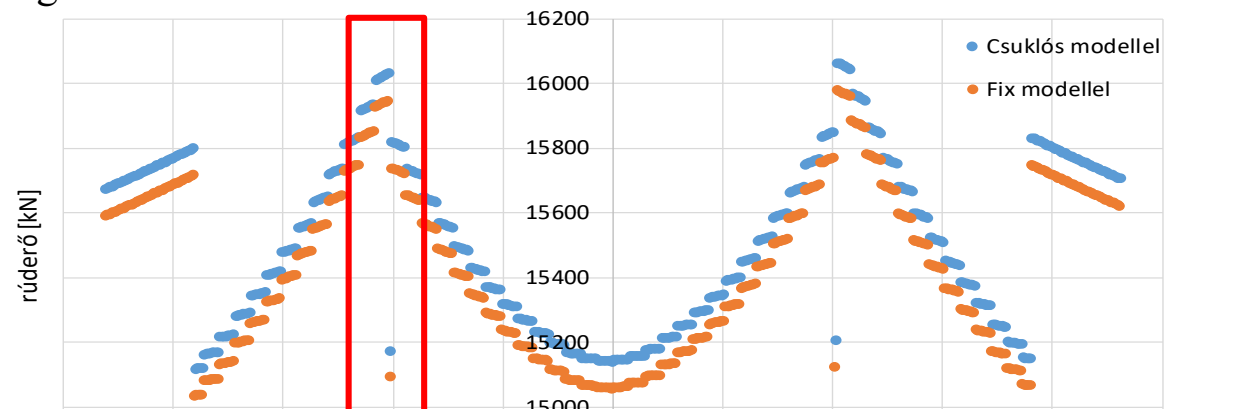
Specialties of the calculation:

- takes in consideration the **original bridge erection order**
- **different statical skeleton in different stages**
- force level setup on suspension bars by previous measurements
- pursue wrecking and refurbishment in time
- takes in consideration the **eyebars' actual shape and force level**
- **Second order analysis** in every loadcase
- modelling of pinned/fixed eyebar connection (corrosion)
- Modelling of **material loss due to corrosion**

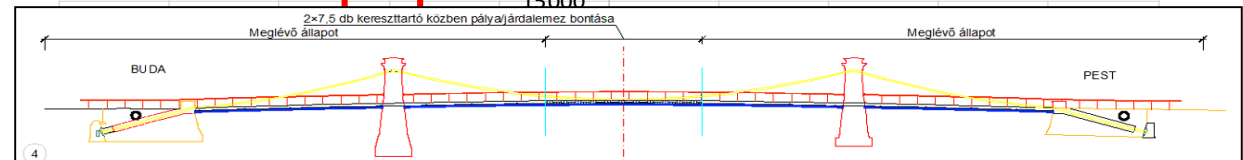
Lot of numerical problems

Calculation's results

Chain's design normal force diagram



Chain's design bending moment diagram



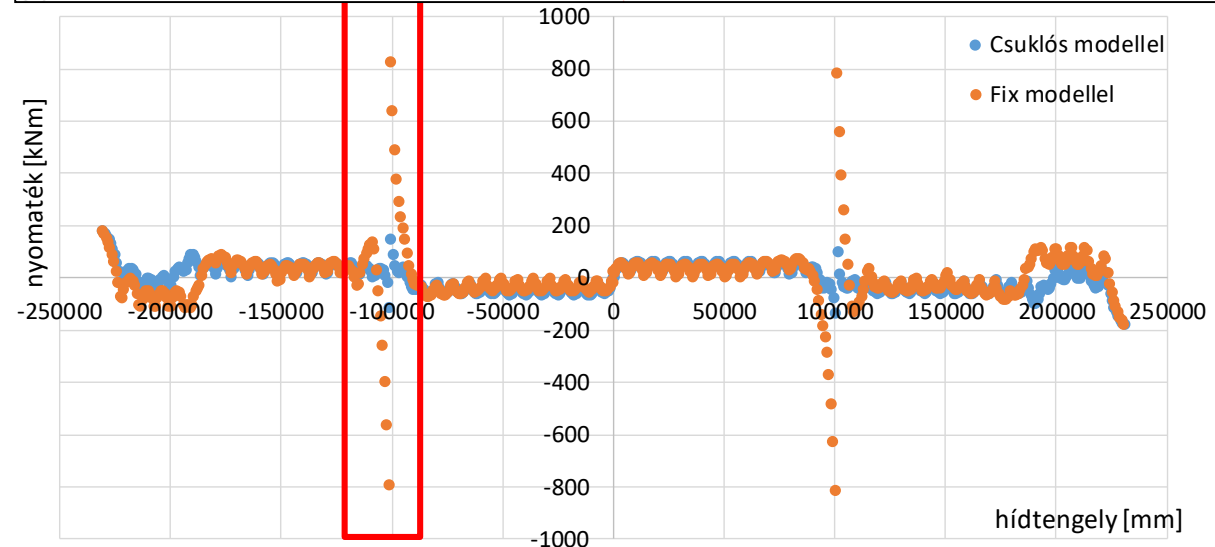
combined $N_{\max}$ and $M_{\max}$



does not fulfil the first yielding limit state by eÚT standard

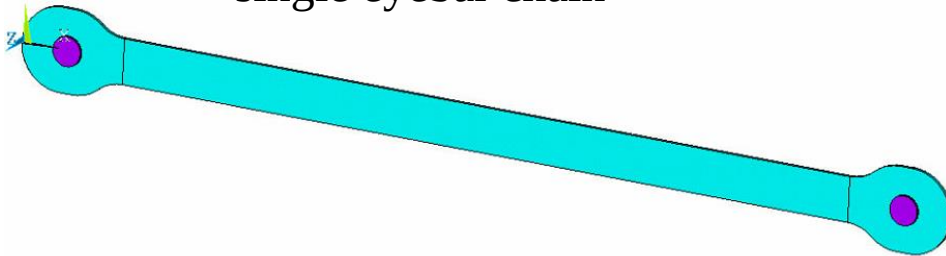


Refined numerical model

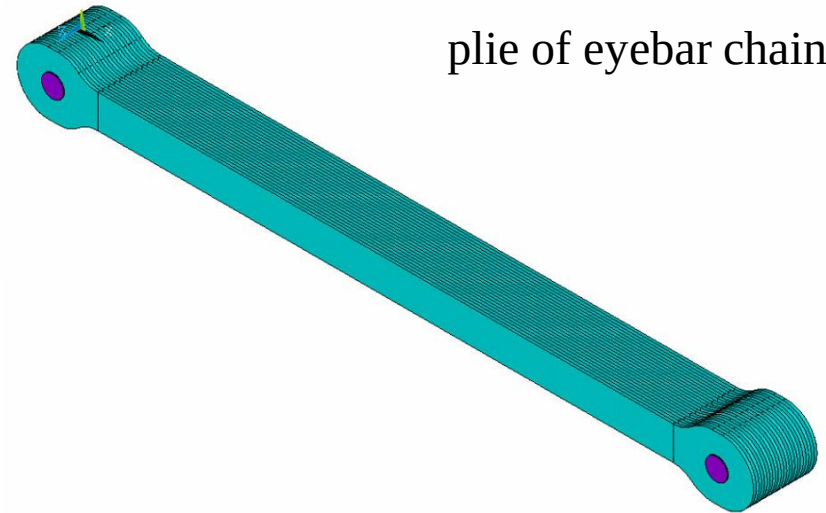


Alkalmazott numerikus modellek

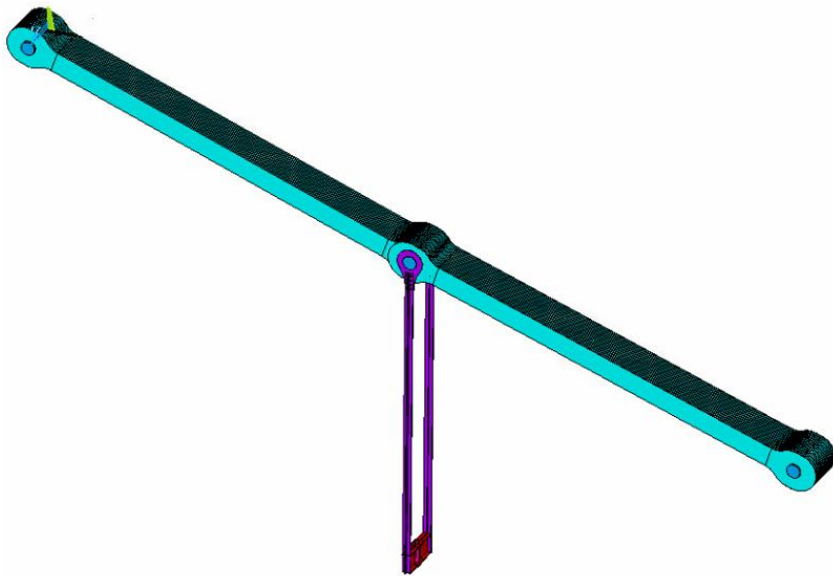
single eyebar chain



pile of eyebar chain



suspension bar's connection

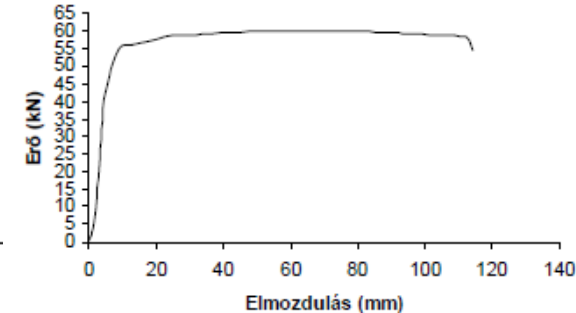
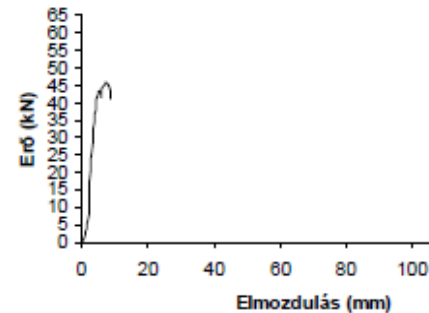
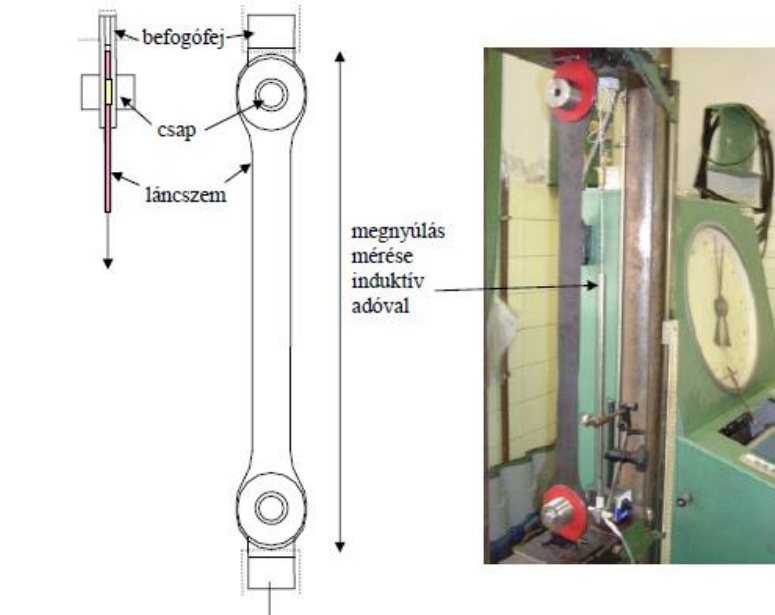


- **load carrying capacity of a single eyebar chain** (special focus on the relevancy of the eyebar-head's load carrying capacity)
- load carrying capacity of an eyebar for **combined N+M stresses**, effect of the connection's transformation from pinned to fixed
- **load carrying capacity of the eyebar pile**, analysis of random lack of uniformity in loading
- determine the loss of load carrying capacity, based on **corrosion** measurement
- eyebar's **fatigue** analysis, considering the extra loading from friction **of the pins**
- check of **pin's connection**
- analysis of **suspension bars' connections**

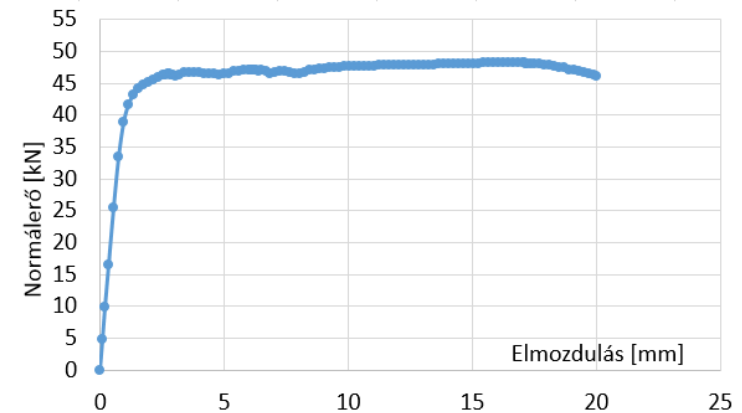
Numerical model's validation

By the PhD dissertation of Vértés K.

experimental load-displacement diagram

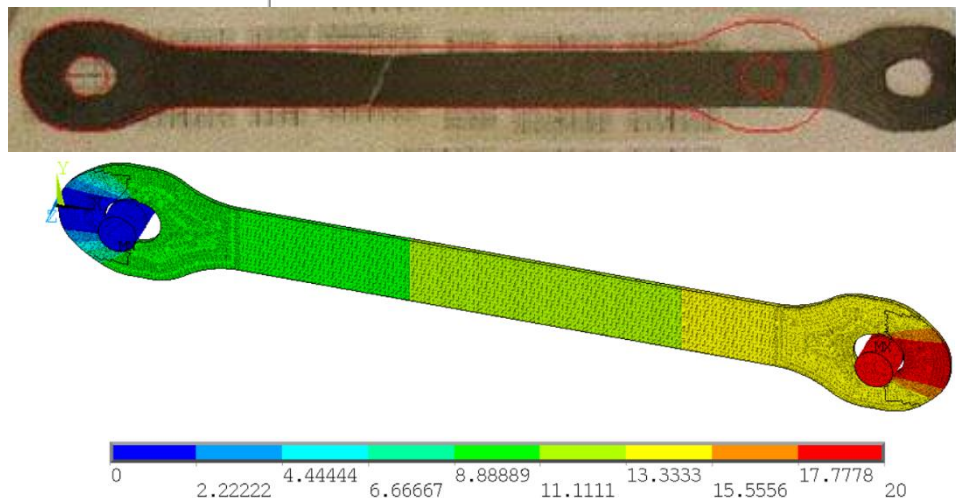


numerical simulation



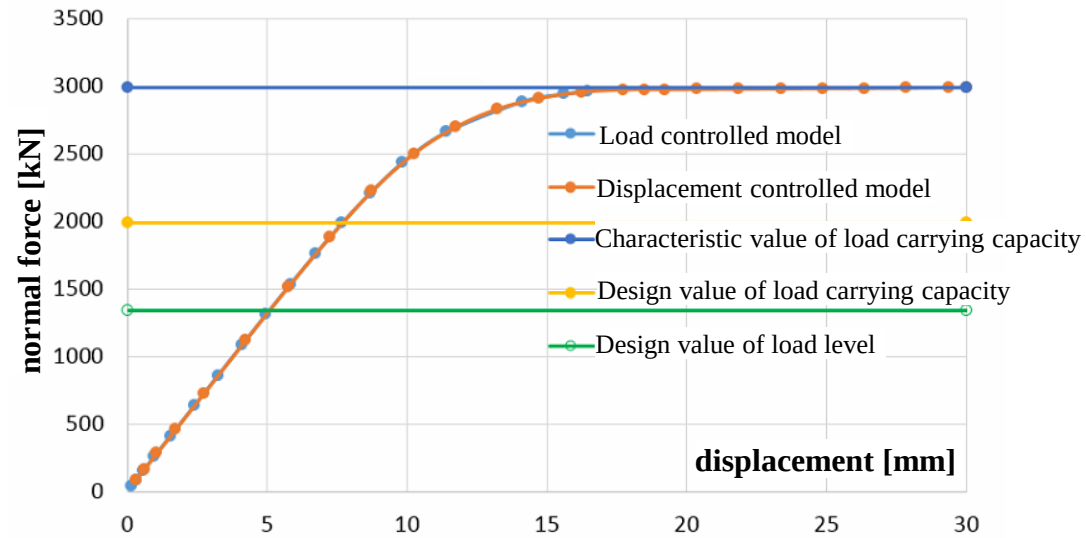
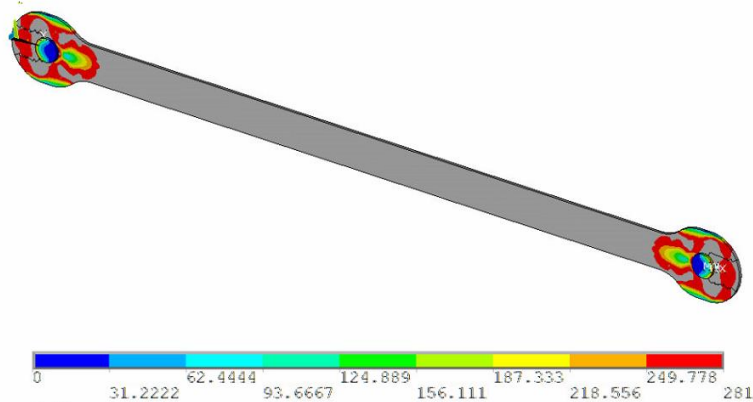
Conclusion:

- precision of the model is adequate,
- potential failure on the shank (straight part).

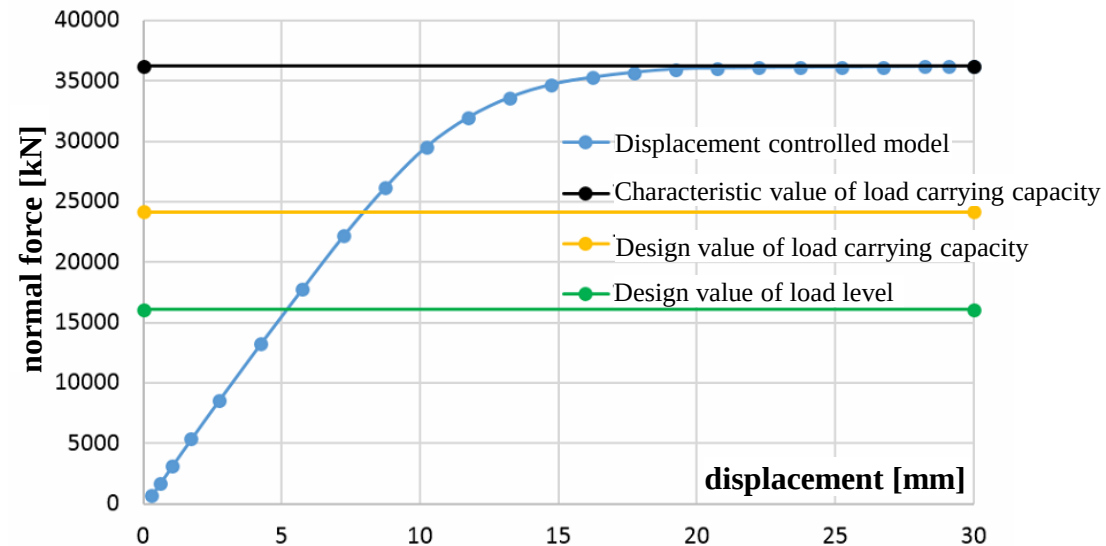
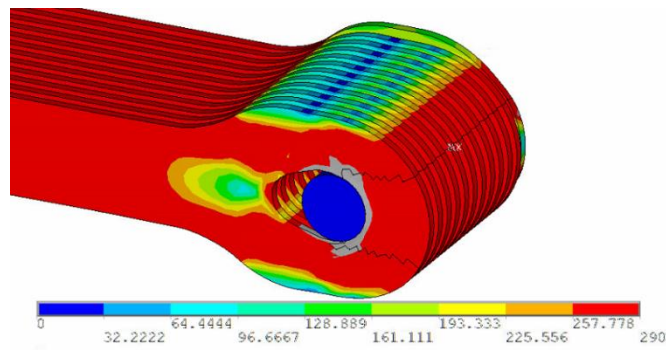


Láncok húzási ellenállása

Single eyebar



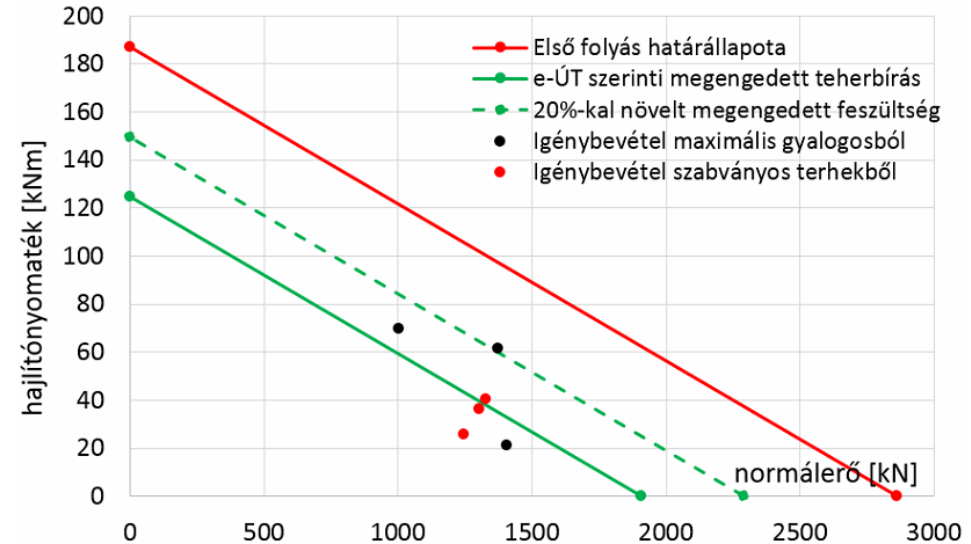
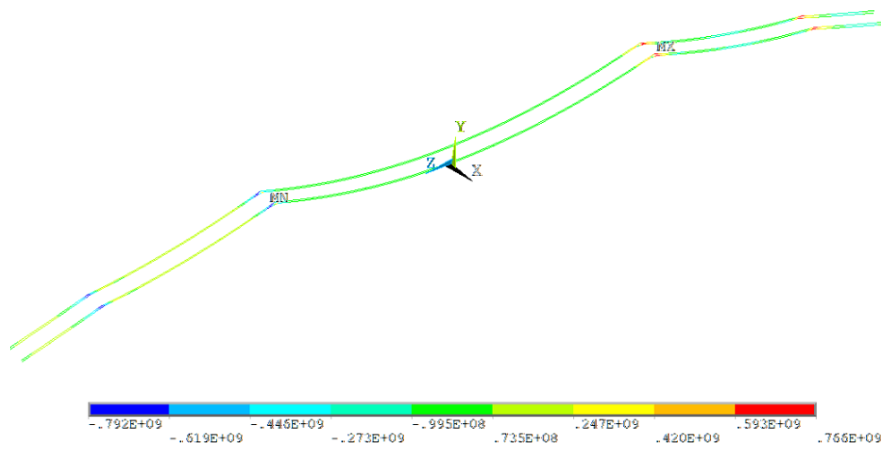
Eyebar pile



N-M interaction's analysis

Design loading

	loading	Normal force	Bending moment
1.	total pedestrian	$N_{\max} = 16\,405\text{ kN}$	$M_{\text{egy}} = 260\text{ kNm}$
2.	pedestrian at midspan	$N_{\text{egy}} = 16\,021\text{ kN}$	$M_{\text{egy}} = 741\text{ kNm}$
3.	pedestrian at sidespan	$N_{\text{egy}} = 11\,575\text{ kN}$	$M_{\max} = 840\text{ kNm}$
4.	reduced pedestrian	$N_{\text{egy}} = 15\,121\text{ kN}$	$M_{\text{egy}} = 443\text{ kNm}$
5.	standardized vehicle	$N_{\text{egy}} = 14\,821\text{ kN}$	$M_{\text{egy}} = 397\text{ kNm}$
6.	BKK vehicle	$N_{\text{egy}} = 14\,121\text{ kN}$	$M_{\text{egy}} = 285\text{ kNm}$

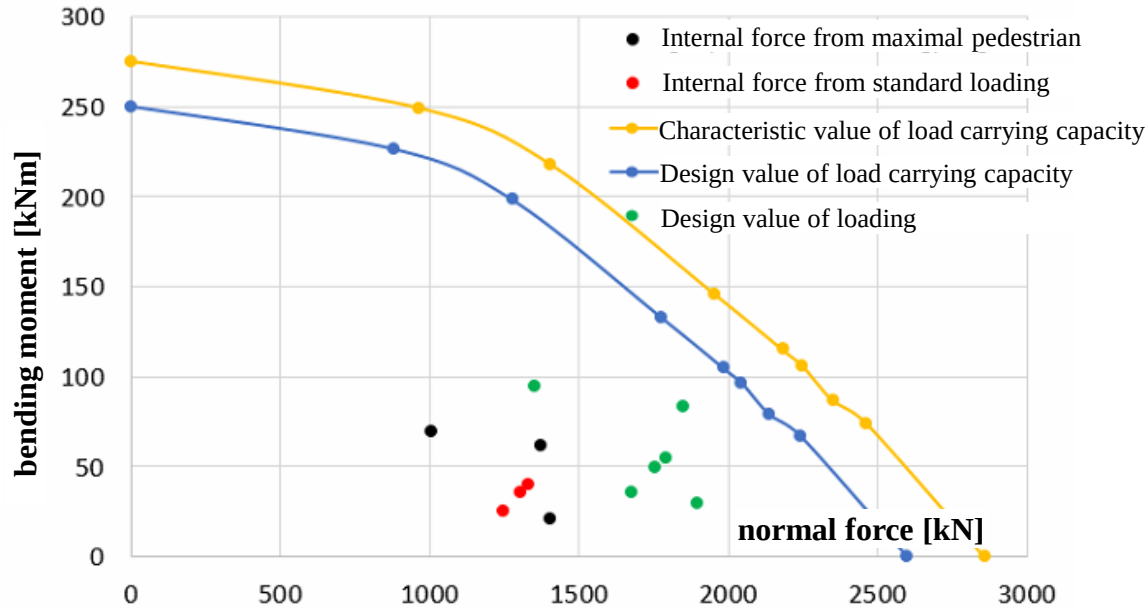


Possibility of raising allowed stresses:

- stress concentration zone
- transition from pinned to fixed joint → joint' side-stress

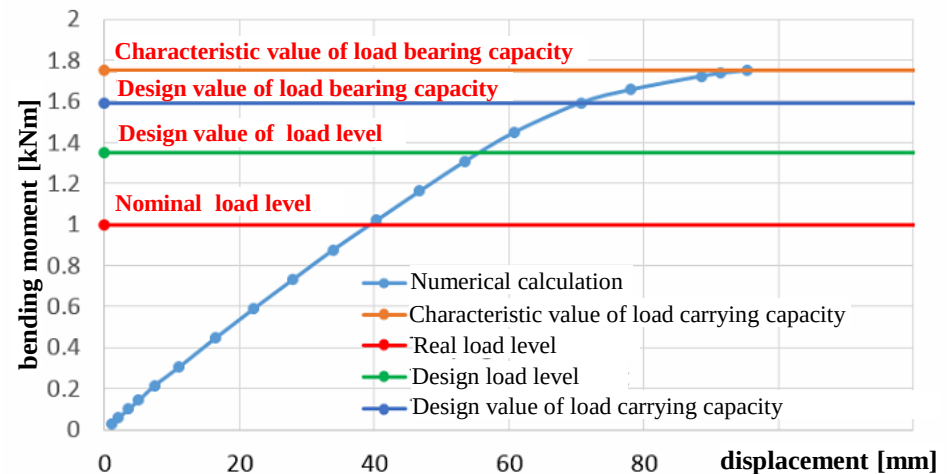
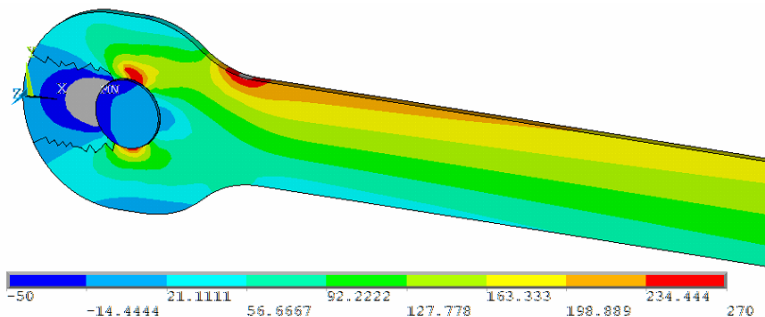
Eurocode based design

Load bearing capacity



Maximum rate of use
85%

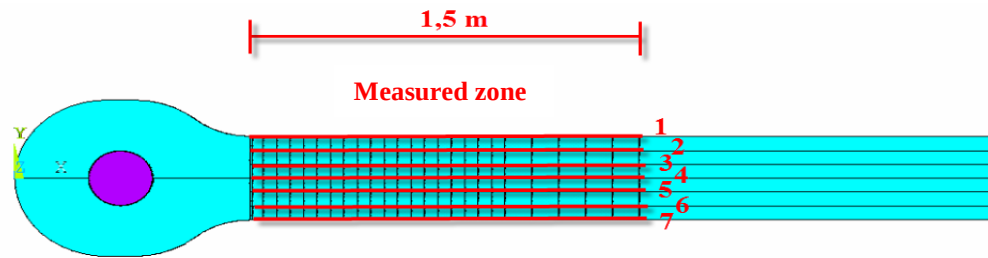
Check-up of linear elastic behaviour



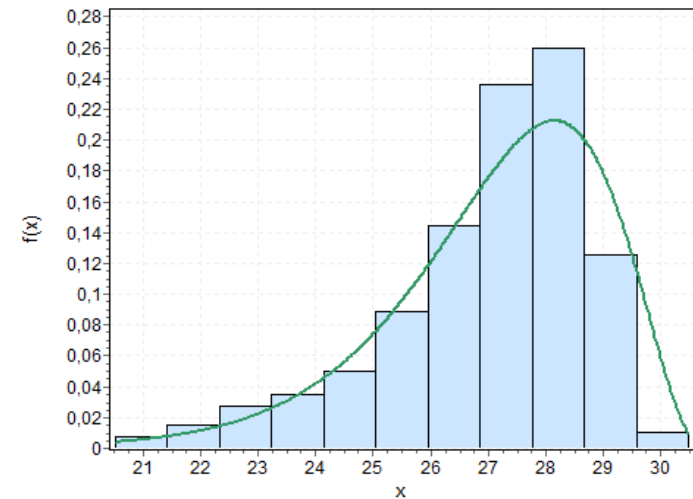
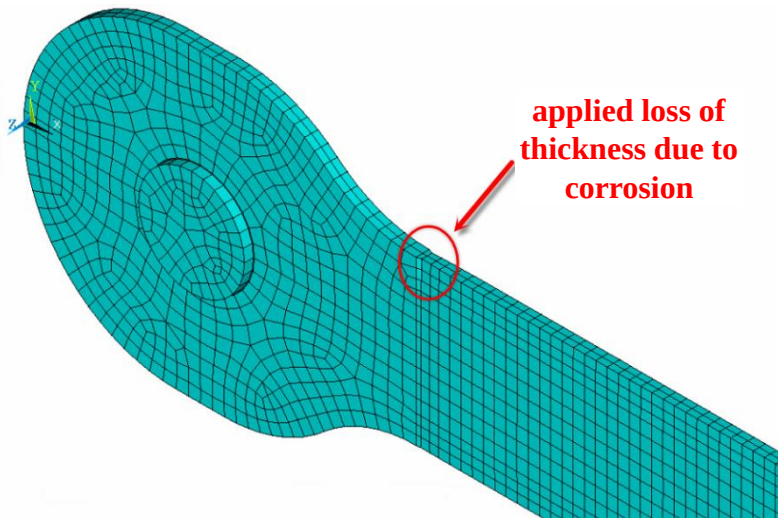
Corroded eyebar's analysis for tension

Consideration of corrosion:

- by measurement data
- full probability analysis



Application in the numerical model:



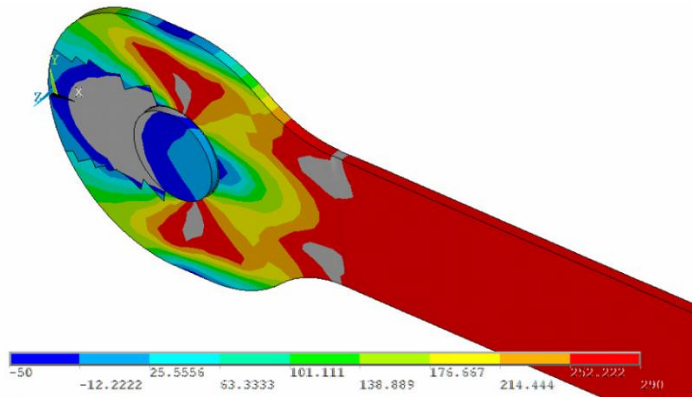
Probabilistic variables:

- Height of eyebar's straight part,
- Eyebar's thickness,
- Material's yield stress,
- Parameter that influences the corrosion's magnitude.

Corrosion surface:

- median surface
- considering the measured scatter

Corroded eyebar's load bearing capacity for tension



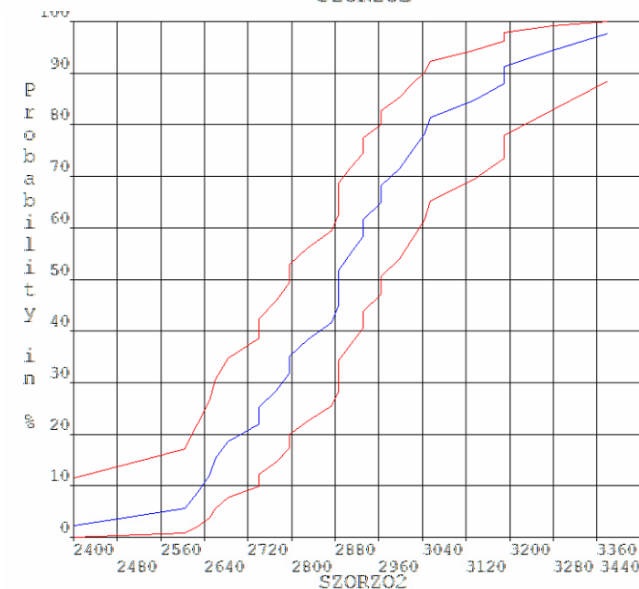
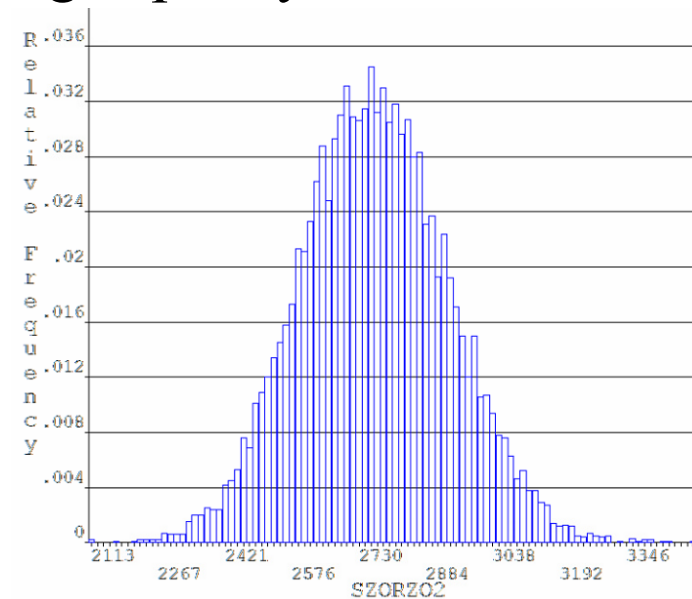
Average load carrying capacity: 2767 kN
(7% average loss of resistance)

5% quantile values: 2495 kN

Design resistance by e-Út : 1661 kN

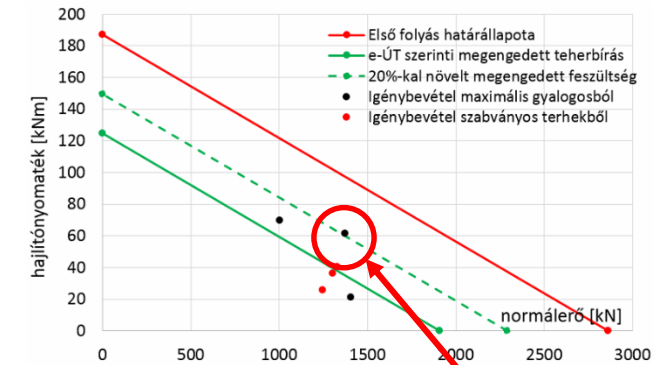
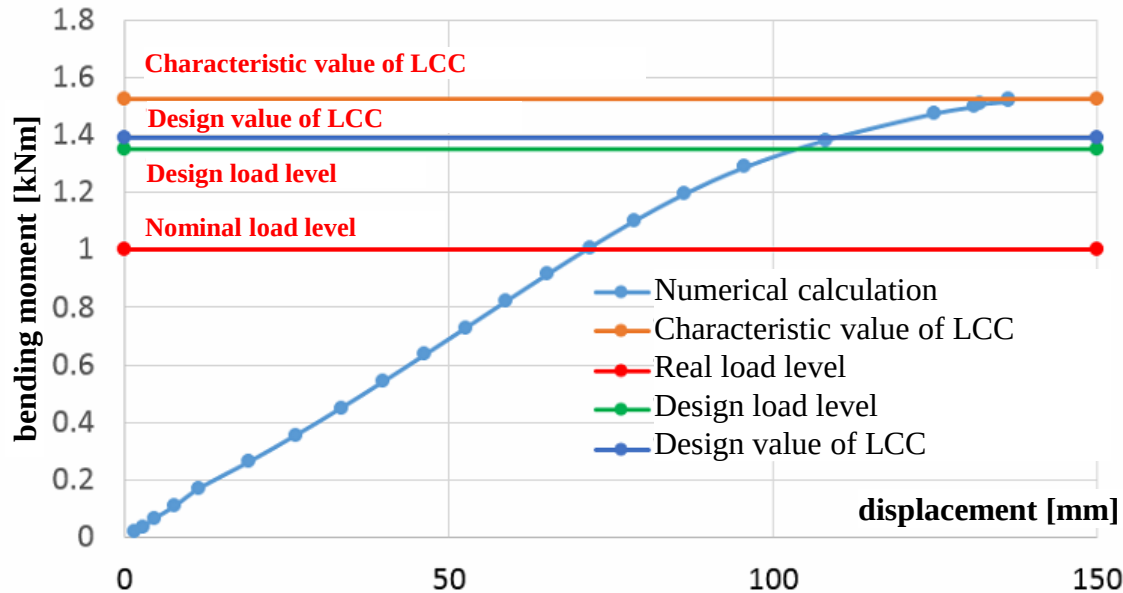


Design load level: 1333 kN



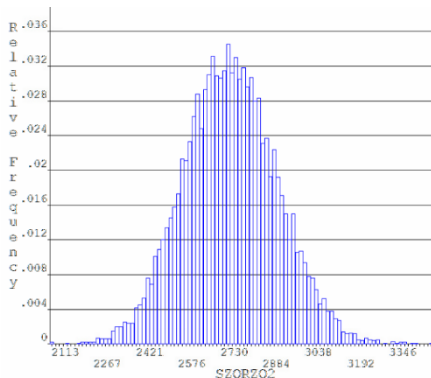
Corroded eyebar's N-M interaction analysis

Load carrying capacity by numerical simulation



analysed N-M pair

Load carrying capacity by full probabilistic analysis



average load multiplier: 1.58

5%-os lower quantile: 1.48

1%₀ lower quantile : 1.38

design load level: 1.35

Utilisation ratio: 98%





Thank you for your attention!