Advanced stability analysis and design of a new Danube archbridge

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Subject of the lecture

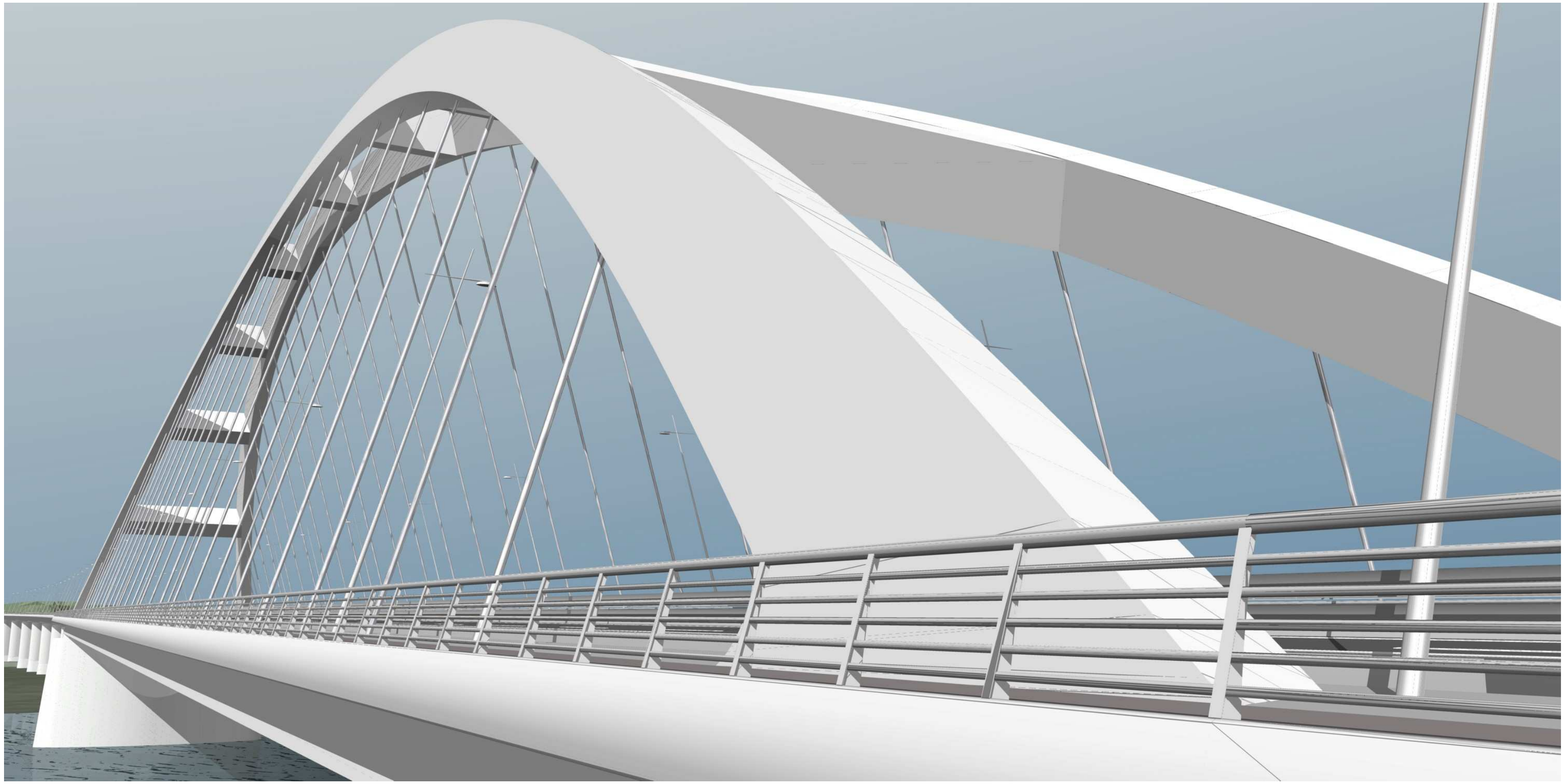
Buckling of steel tied arch

Buckling of orthotropic steel plates

ANALYSIS – DESIGN METHODS – APPLICATION

Contents

Dunaújváros Danube bridge



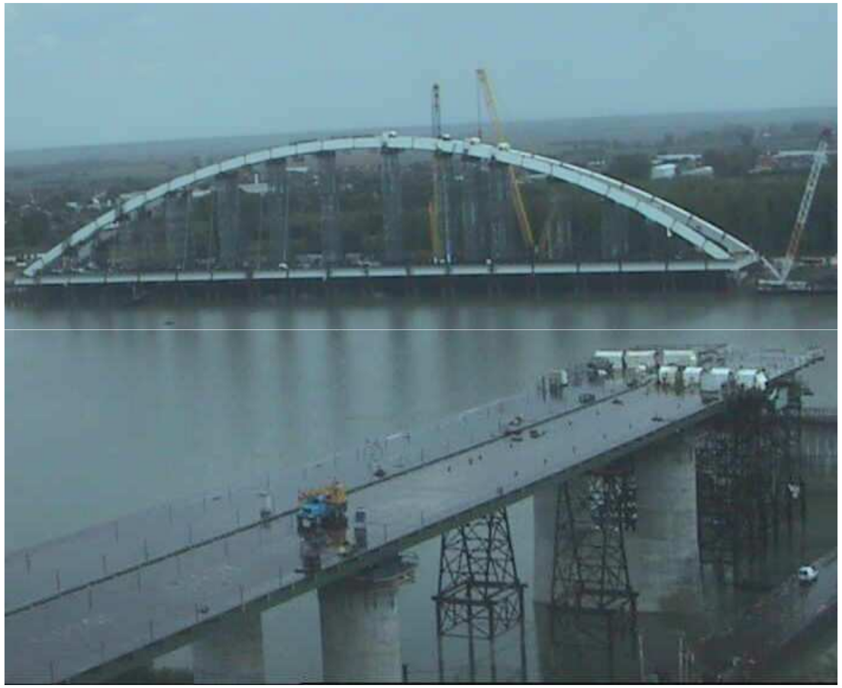
Location


Wien
Bratislava
Győr
Tatabánya
Budapest
Salgótarján
Miskolc
Eger
Nyíregyháza
Debrecen
Szombathely
Veszprém
Székesfehérvár
Szolnok
Dunaújváros
Zalaegerszeg
Békéscsaba
Kaposvár
Szekszárd
Szeged
Pécs

Geometry

Total length of the bridge:	1780 m
Main span; tied arch bridge:	307.8 m
arch height:	48 m
steel box:	2 x 3.8 m

Current stage



Webcam: http://www.dunaujhid.hu/webcam.html

Tasks of the Department

Preliminary phase: advisor for designer

Design phase: research on design methods
- model test – arch stability
- wind tunel test on section model

analysis and design
- stability, fatigue,
- earthquake, aerodynamic

Construction phase: erection method
- structural design for erection

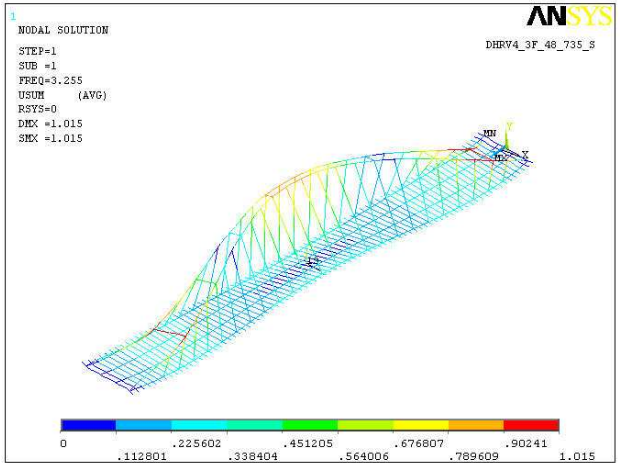
Model test on arch stability

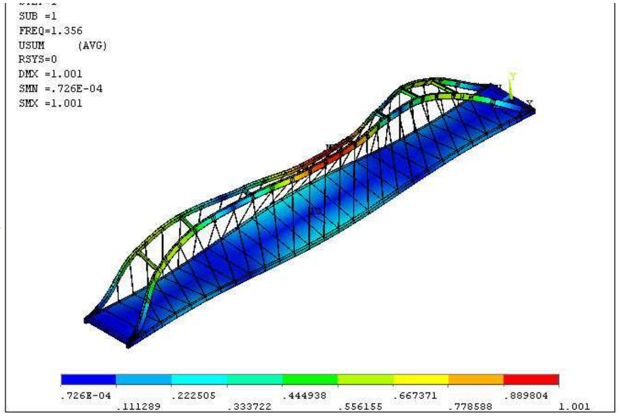
Purpose

Experimental test

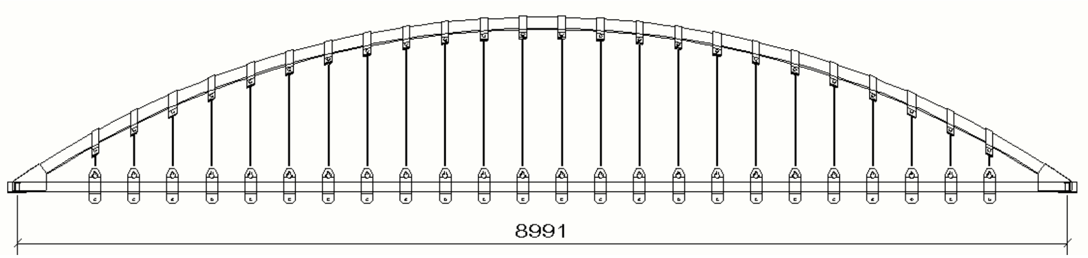
- design → equivalent global arch stability
- aims

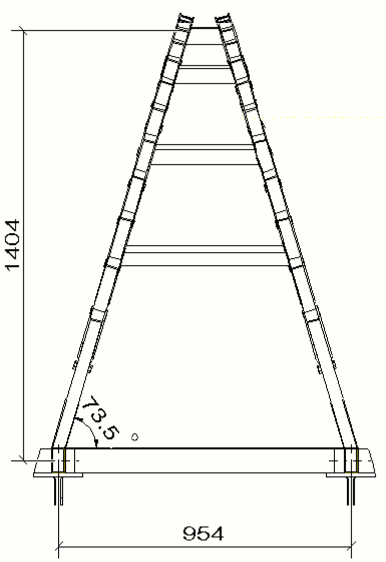
1) check the safety of the standard design methods
2) verify advanced numerical model and calibrate imperfection sizes

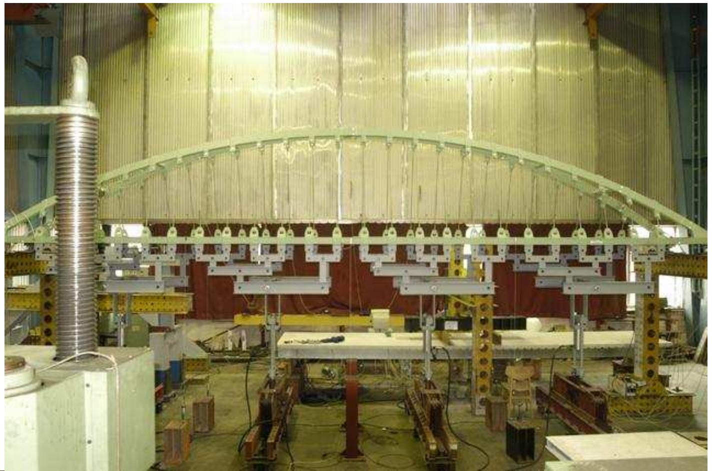




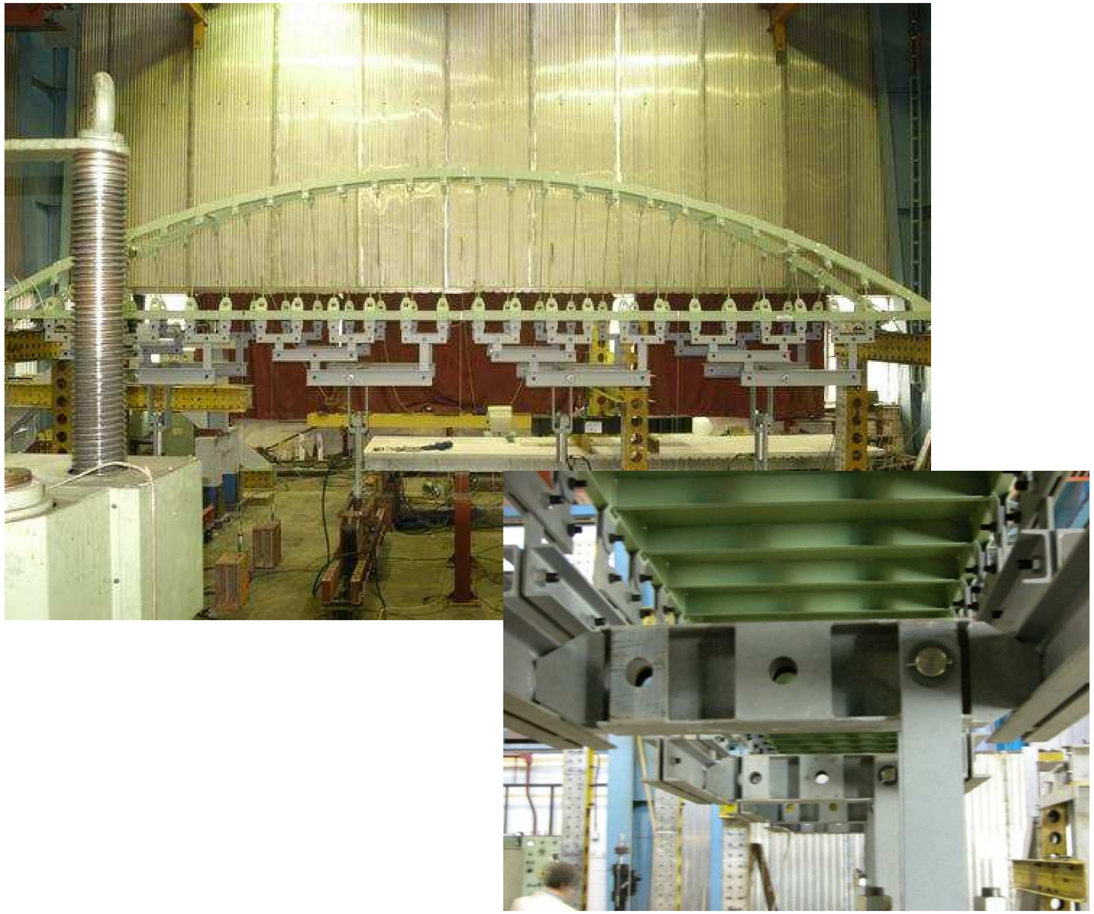
Bridge model M=1:34



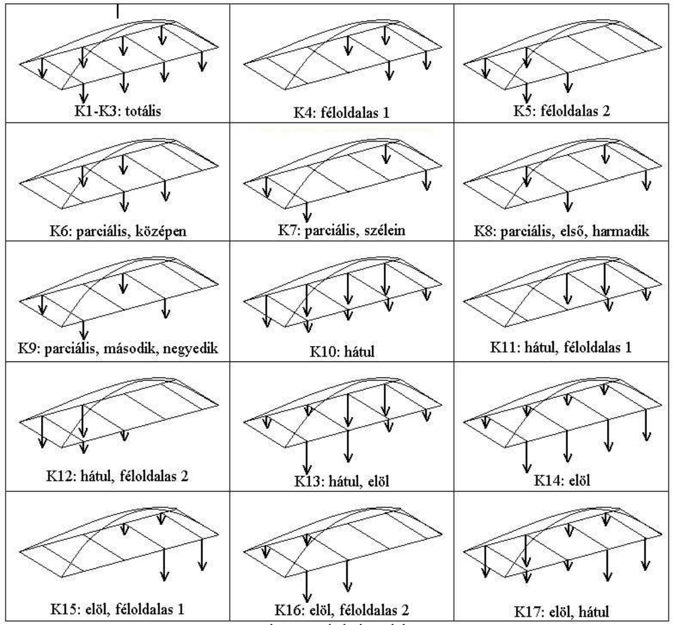




Loading system

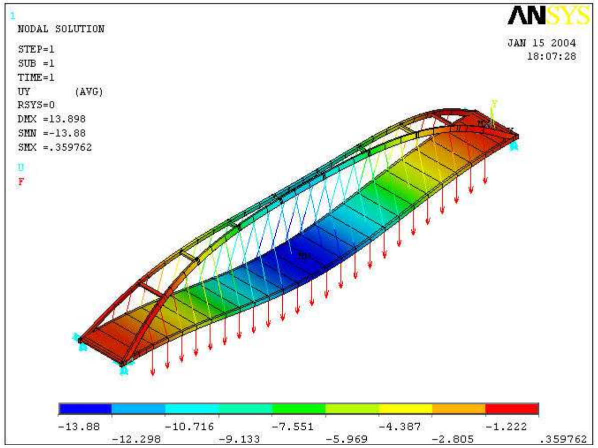


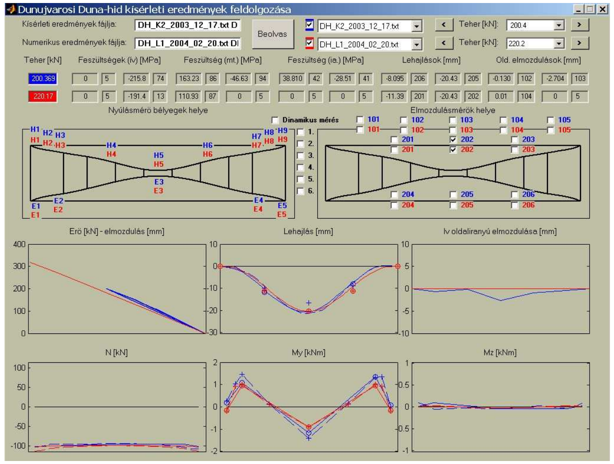
15 load cases

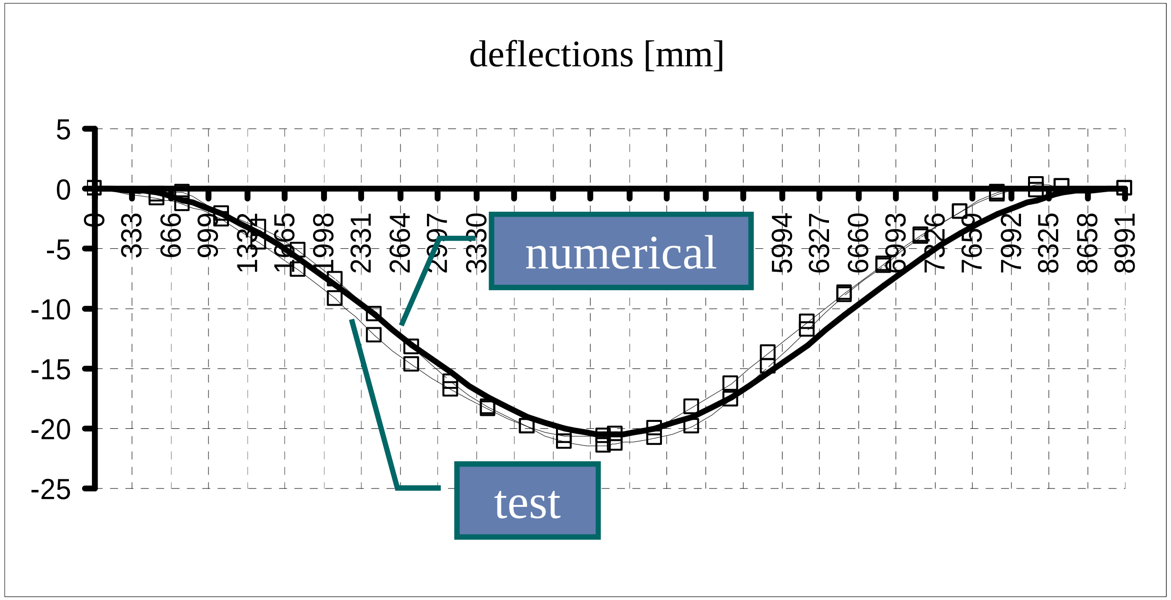


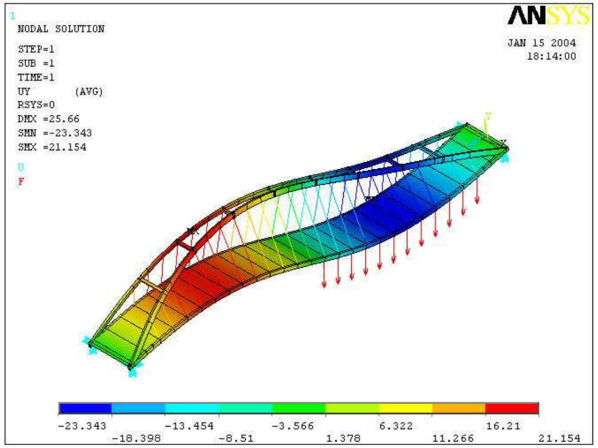
Total loading: Σq=220 kN

Self-weight + 75 x 40 t trucks

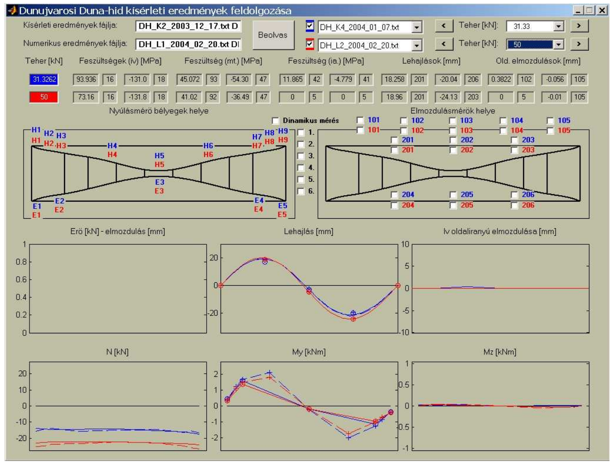


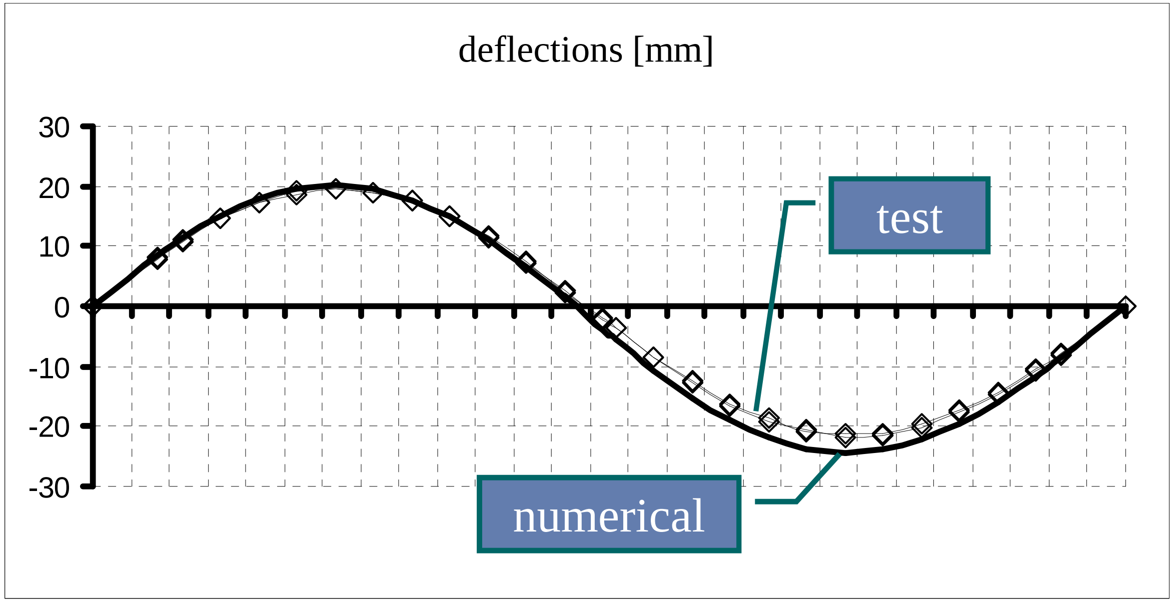






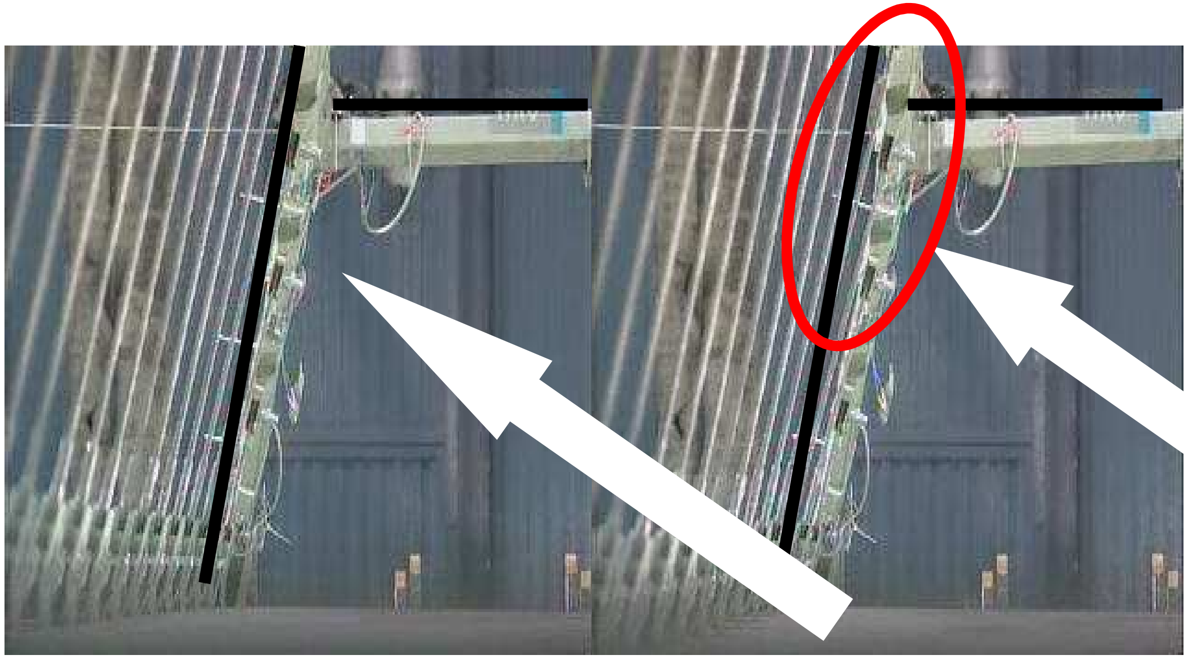
partial half-sided loading: Σq=50 kN





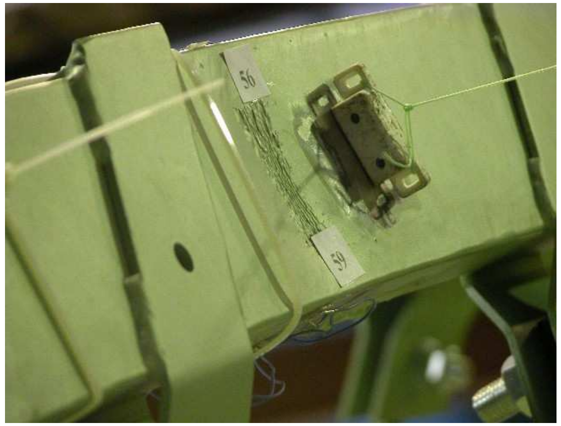
Failure test – 1

total loading: 320 kN out-of-plane buckling



out-of-plane buckling of the arch

local plate buckling



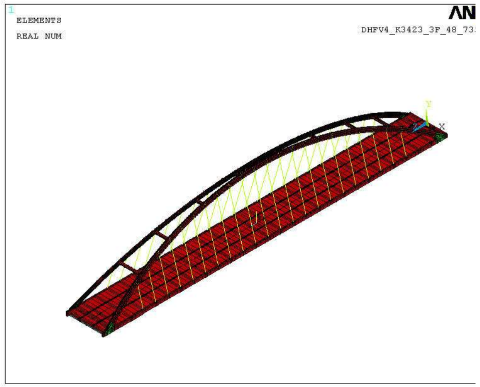
Failure test – 2

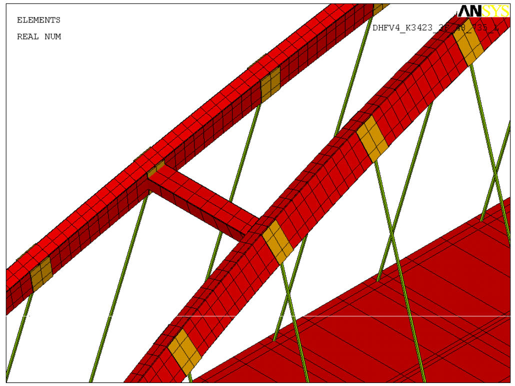
half-sided loading: 110 kN

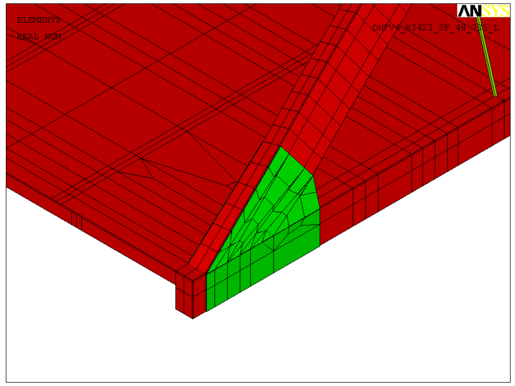
in-plane buckling of the arch

Numerical model

Model data	Ansys beam model	Ansys shell model
Element type	BEAM44 LINK10	SHELL181 LINK10
number of elements	~6 000	~17 000
number of nodes	~12 000	~17 000







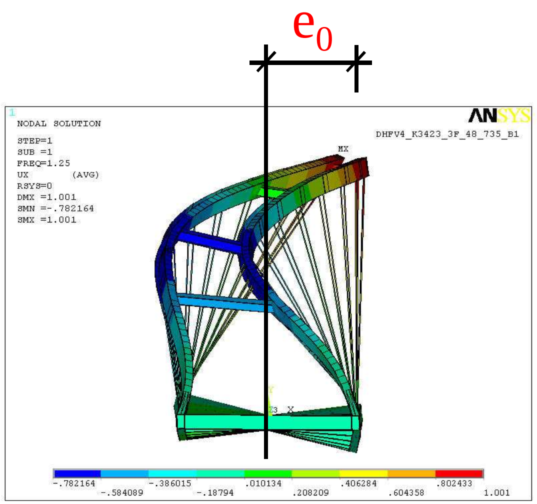
Analysis	
Linear	material and geometrical linearity
Instability	Block Lanczos buckling analysis
Geometrically nonlinear	geometrical nonlinearity, imperfect model
Virtual experiment	material and geometrical nonlinearity, imperfect model

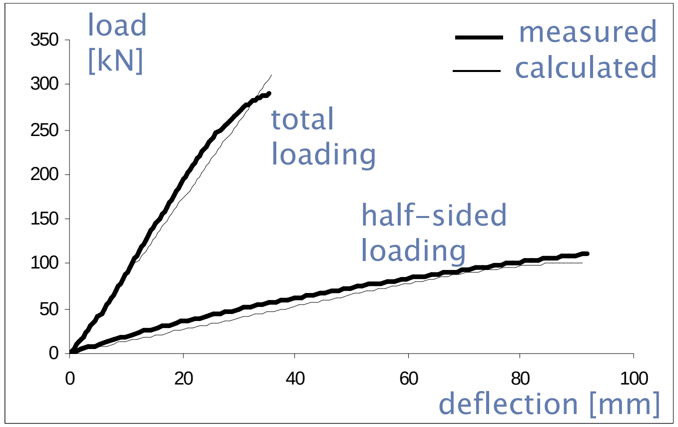
Verification

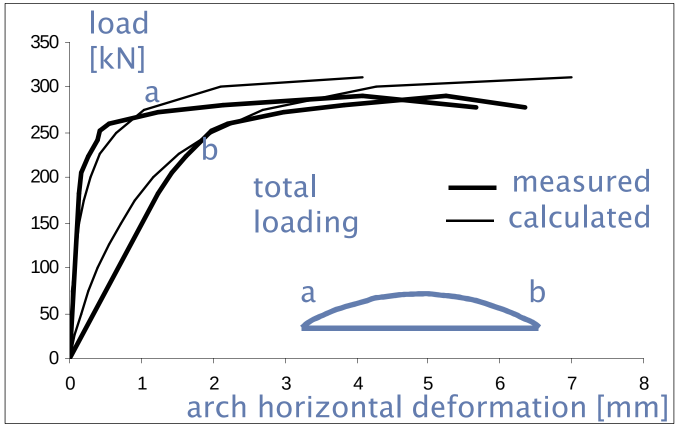
material and geometrical nonlinearity

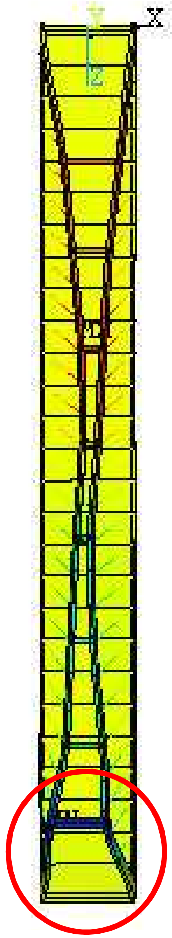
imperfection: $e_0 = 2$ mm

on the half side of the model ⇒ non-symmetrical behaviour









Design methods

Hungarian Standard:

HS

(1) $$\frac{N}{N_e} + \psi_y \frac{M_y}{M_{e,y}} + \psi_z \frac{M_z}{M_{e,z}} \leq 1$$ strength check

+

(2) $$\frac{N}{N_{ke,z}} \leq 1$$ buckling check

Japanese Standard:

JSHB

(3) $$\frac{N}{N_{ke,z}} + \psi_y \frac{M_y}{M_{e,y}} + \psi_z \frac{M_z}{M_{e,z}} \leq 1$$ linear interaction

Eurocode:

EC3

(4) $$\frac{N}{N_{ke,y}} + k_{yy} \frac{M_y}{M_{e,y}} + k_{yz} \frac{M_z}{M_{e,z}} \leq 1$$

(5) $$\frac{N}{N_{ke,z}} + k_{zy} \frac{M_y}{M_{e,y}} + k_{zz} \frac{M_z}{M_{e,z}} \leq 1$$

(4)–(5): combined interaction

Solution method

Experimental test → ultimate load

Numerical model ↓

Internal forces: N, M_y, M_z from analyses:

1. linear + second order modification factor
2. geometrically nonlinear – equivalent imperfection 1
3. geometrically nonlinear – equivalent imperfection 2

standard ultimate load ⟷ experimental ultimate load

Equivalent imperfection size 1

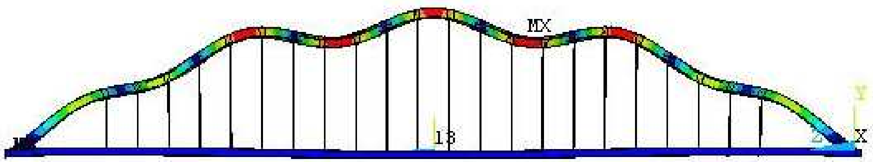
Eurocode 3 – Part 1.1: shape of the elastic critical buckling mode: η_{cr}

$$e_{0,d} = \alpha(\bar{\lambda} - 0.2)\frac{M_{Rk}}{N_{Rk}} \frac{1 - \frac{\chi\bar{\lambda}^2}{\gamma_{M1}}}{1 - \chi\bar{\lambda}^2}$$

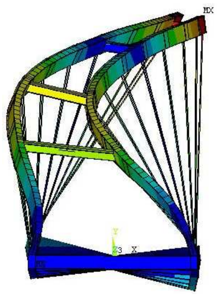
$$\eta_{init} = \frac{e_{0,d}}{\bar{\lambda}^2} \frac{N_{Rk}}{EI\eta''_{cr,\max}} \eta_{cr}$$

second order analysis

in-plane buckling mode:



out-of-plane buckling mode:



In-plane	Out-of-plane
3.44 mm	17.30 mm

Equivalent imperfection size 2

Eurocode 3 – Part 2:

(Design of bridges)

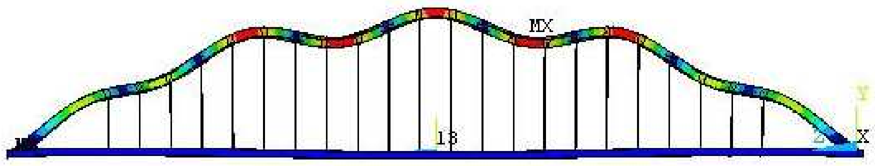
$$\eta_{0,z} = \frac{l}{500}$$

in-plane

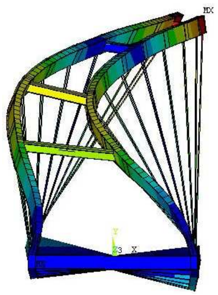
$$\eta_{0,y} = \frac{l}{250}$$

out-of-plane

in-plane buckling mode:



out-of-plane buckling mode:



In-plane	Out-of-plane
17.98 mm	35.96 mm

Comparison of classical design methods

	total load	half-sided load
HS	2.25	3.06
JSHB	3.07	3.28
EC3	2.20	1.87

experimental ultimate load / standard ultimate load

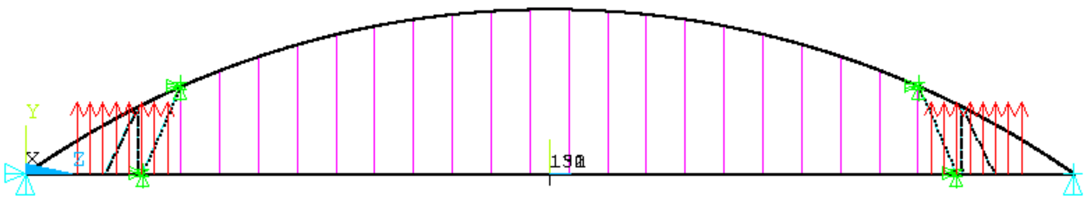
Comparison of Eurocode approaches

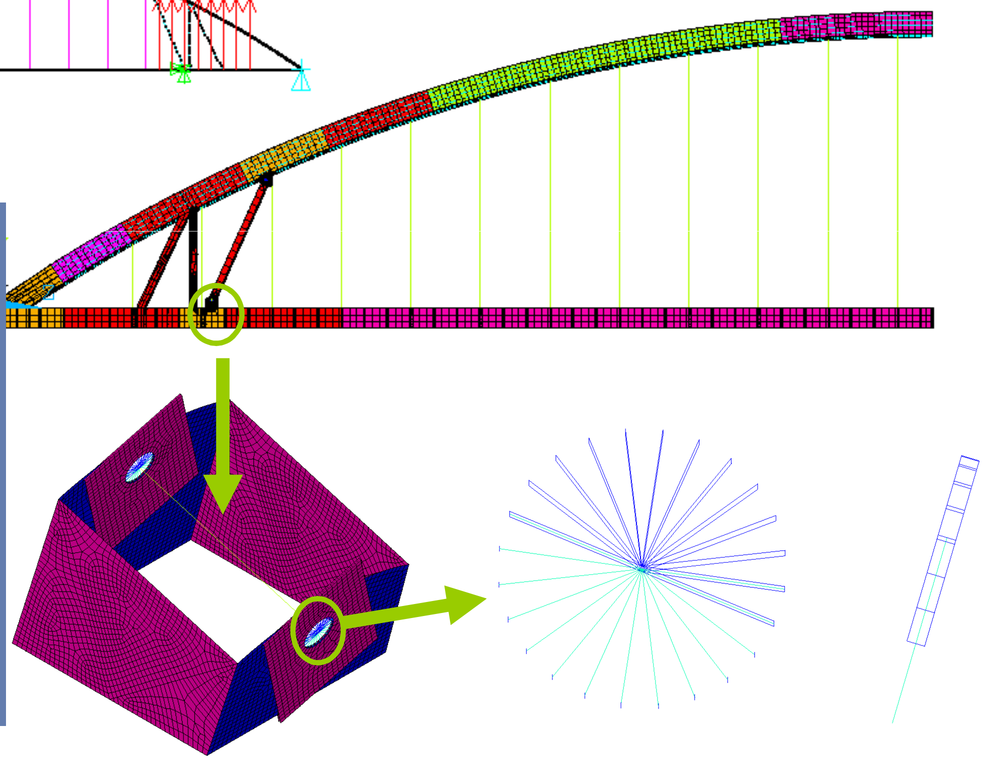
	total load	half-sided load
EC3 – linear	2.20	1.87
EC3 – eqv. geom. imp. 1	1.45	1.84
EC3 – eqv. geom. imp. 2	2.29	2.15

experimental ultimate load / standard ultimate load

Arch bridge erection

Finite element model

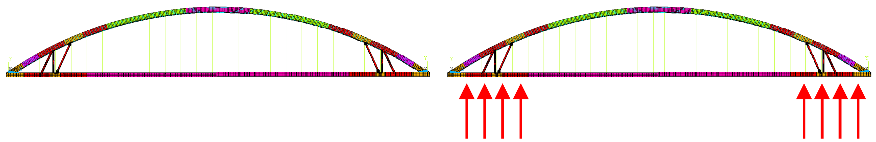




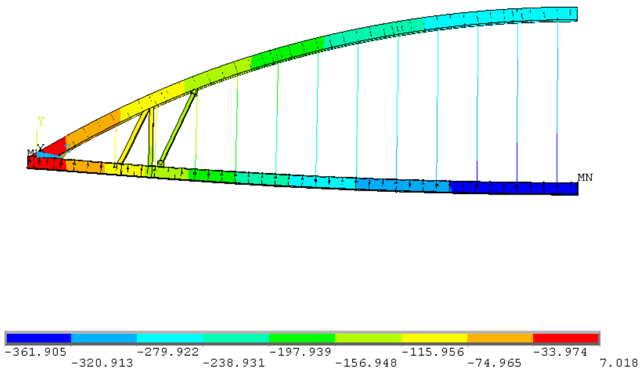
Model data	Ansys beam model	Ansys shell model
Element type	BEAM44 LINK10	SHELL181 LINK10
number of elements	~29 000	~170 000
number of nodes	~57 000	~170 000
DOF	~340 000	~1 000 000

Erection phases

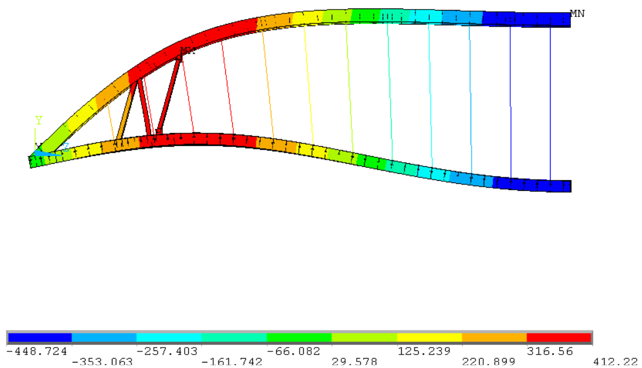
1. Bridge is on the riverbank on a rack system
2. The cables are stressed to the self weight
3. Additional bars are built in the bridge
4. The bridge is palced on barges



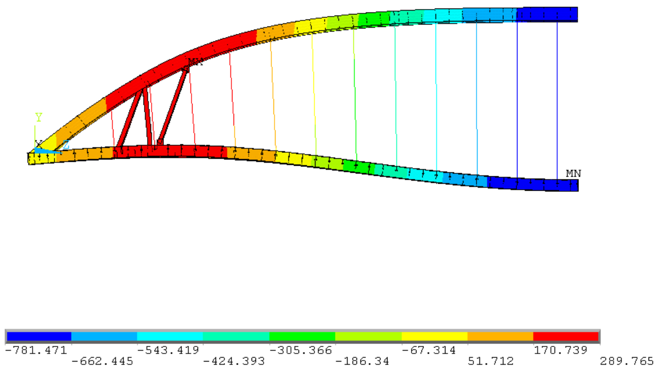
Stress analysis



1. Load case: self-weight



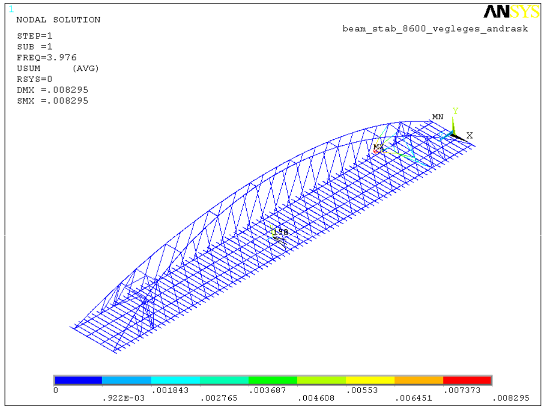
2. Load case: ship reaction forces



3. Load case: 1. load case + 2. load case

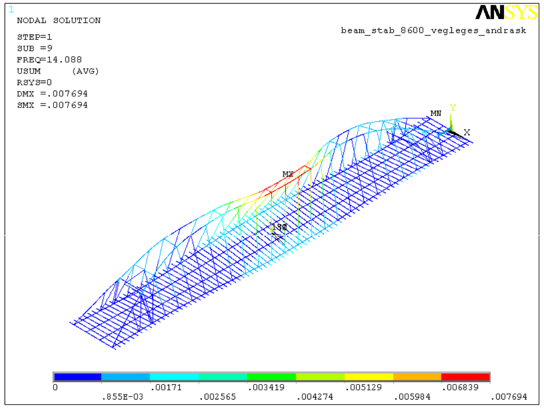
Instability analysis

Stiffening bar buckling



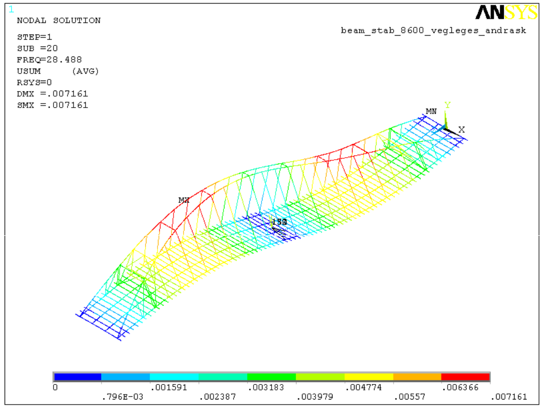
$\alpha_{cr} = 3.97$

Out-of-plane buckling



$\alpha_{cr} = 14.088$

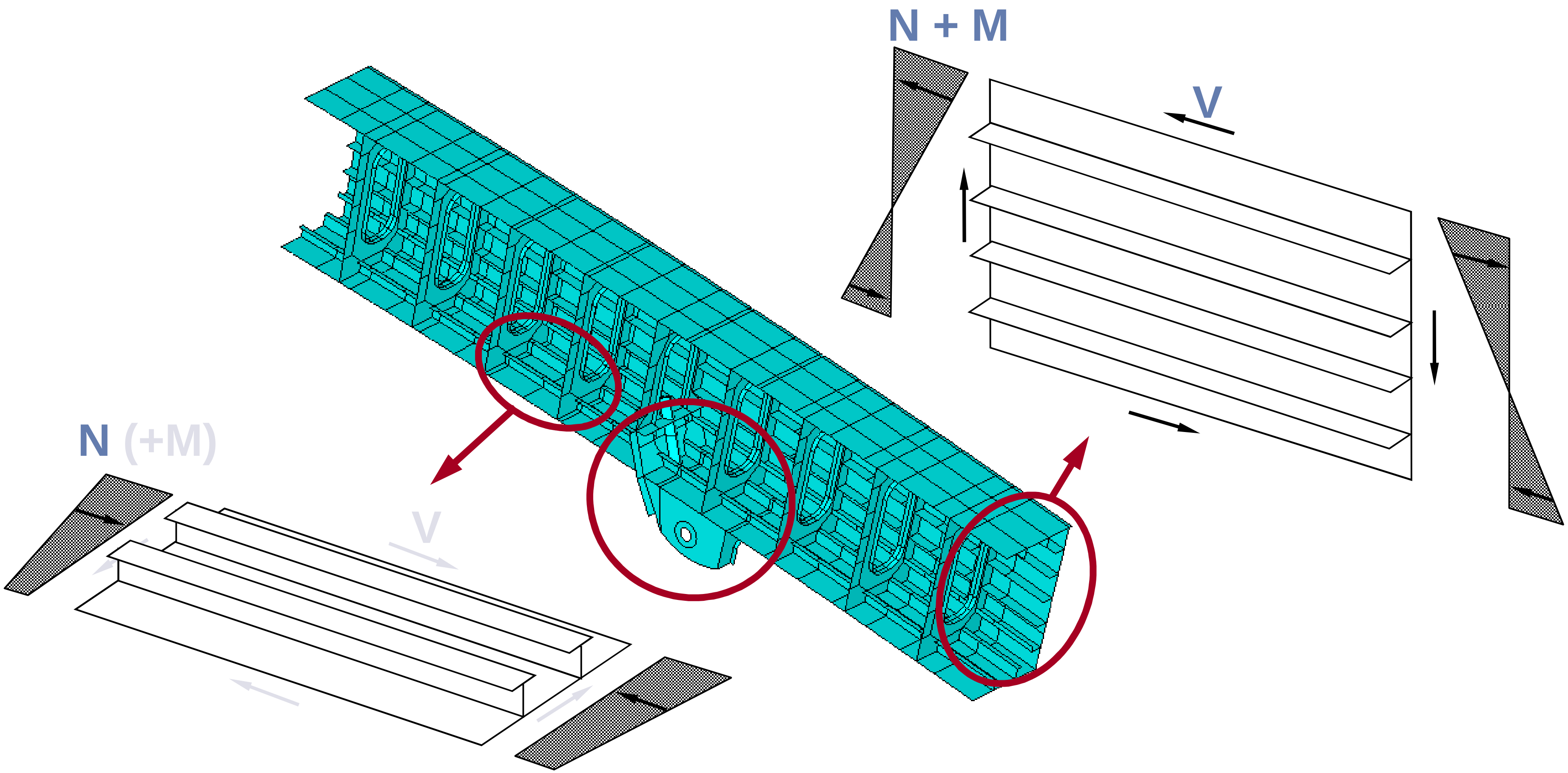
In-plane buckling



$\alpha_{cr} = 28.488$

Orthotropic plate buckling

Orthotropic plates



Design methods – Hungarian Standard

- allowable stress design

$$f_y = 460\ MPa \longrightarrow \sigma_e = 300\ MPa$$

factor ~1.47

- dominantly compressed stiffened plates or plate parts
  → (1) buckling of fictive column stub

- stiffened plate subject to complex stress field
  → (2) orthotropic plate check

- irregular configuration and stress field – ??? no rule given
  → (3) generalized plate check

(1) Buckling of fictive column stub

- fictive column = stiffener + adjacent plating
- column slenderness (λ) and reduction factor (ϕ)

a) flexural buckling

$$\lambda_1 = L_{kt} \cdot \sqrt{\frac{A_b}{I_b + b_e \cdot \frac{t_f^3}{11} \cdot \left(2 \cdot \frac{L_{kt}^2}{4 \cdot b_s^2} + \frac{L_{kt}^4}{16 \cdot b_s^4}\right)}}$$

$$\phi_1 = \beta - \sqrt{\beta^2 - \frac{1}{(\lambda_1 / \lambda_E)^2}}$$

b) torsional buckling

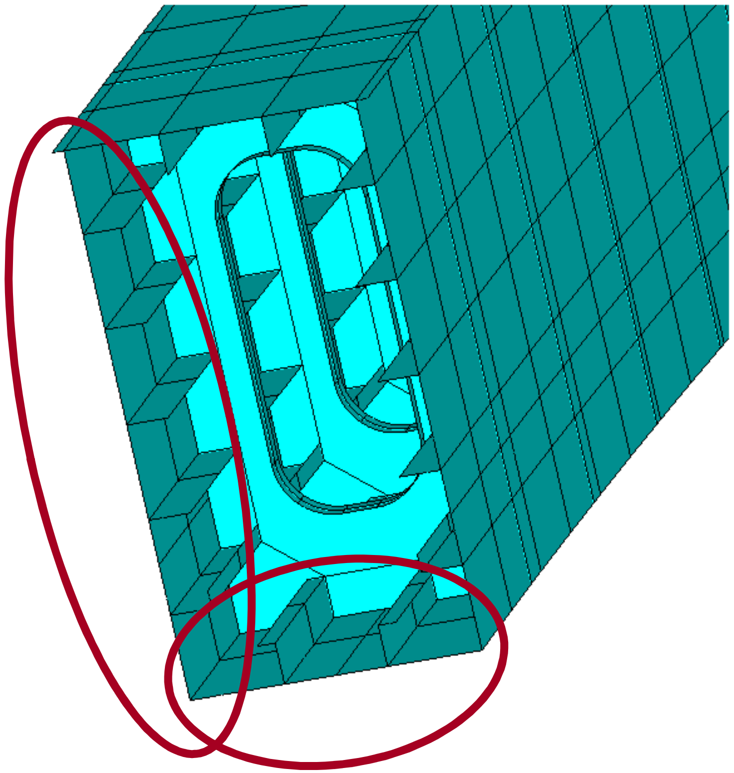
$$\lambda_2 = L_{kt} \cdot \sqrt{\frac{I_u + I_\eta}{0.04 \cdot L_{kt}^2 I_T + I_\eta \cdot h^2}}$$

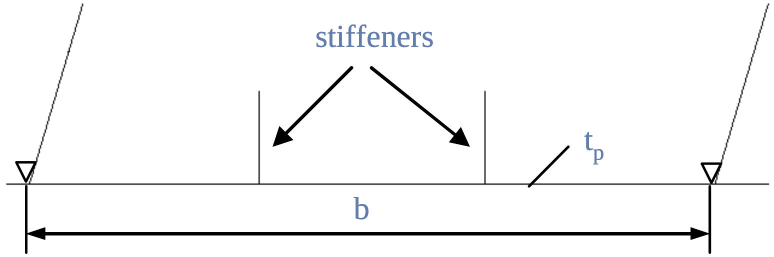
$$\phi_2 = \left(\frac{1}{1 + (\lambda_2 / \lambda_E)^5}\right)^{0.4}$$

$\phi \cdot \sigma_e$ (allowable stress)

- check: $\frac{N_b}{A_b} \leq \phi \cdot \sigma_e$

Actual calculations on the Danube bridge





Case Nr.	t_p [mm]	b [m]	a [m]	stiffener
1	40	2	4.56	2 x 280-22
2	30	3.8	4.56	5 x 280-22
3	50	2	2.125	2 x T270-150-22
4	20	3.8	3.9	5 x 280-22
5	16	3.8	3.86	5 x 280-22
6	20	2	3.86	2 x 280-22

t_p – plate thickness; b – plate width; a – plate length between transverse stiffeners or diaphragms

(2) Orthotropic plate subject to complex stress field

- plate-type behaviour
- plate slenderness (λ_0) and reduction factor (ϕ_b)

$$\lambda_0 = \frac{3.3}{\sqrt{k_{red}}} \frac{b}{t}$$

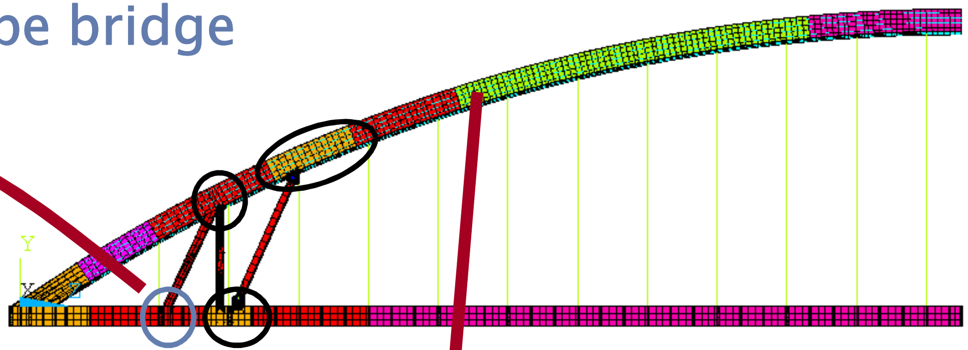
k_{red}: buckling coefficient, e.g. Klöppel-Scheer-Möller (overall plate buckling of horizontally and longitudinally stiffened plates)

↓

ϕ_b reduction factor for plates

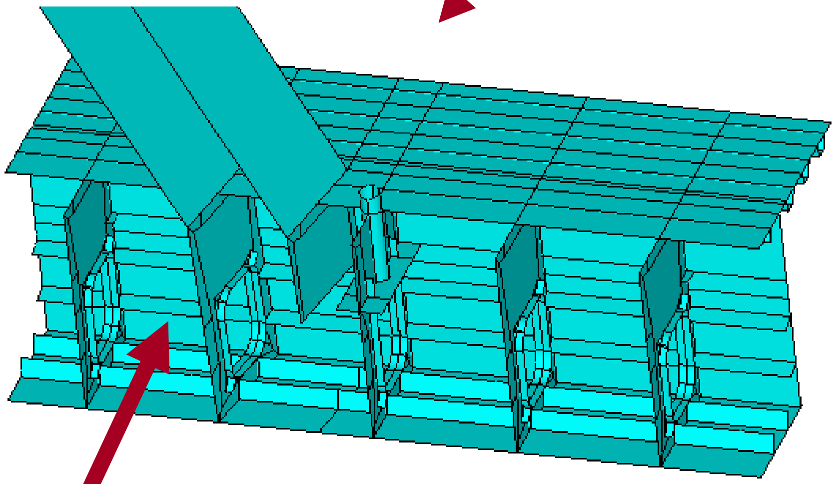
- check: $\sigma_{red} = \sqrt{\sigma^2 + 3\tau^2} \leq \phi_b \cdot \sigma_e$

Actual calculations on the Danube bridge



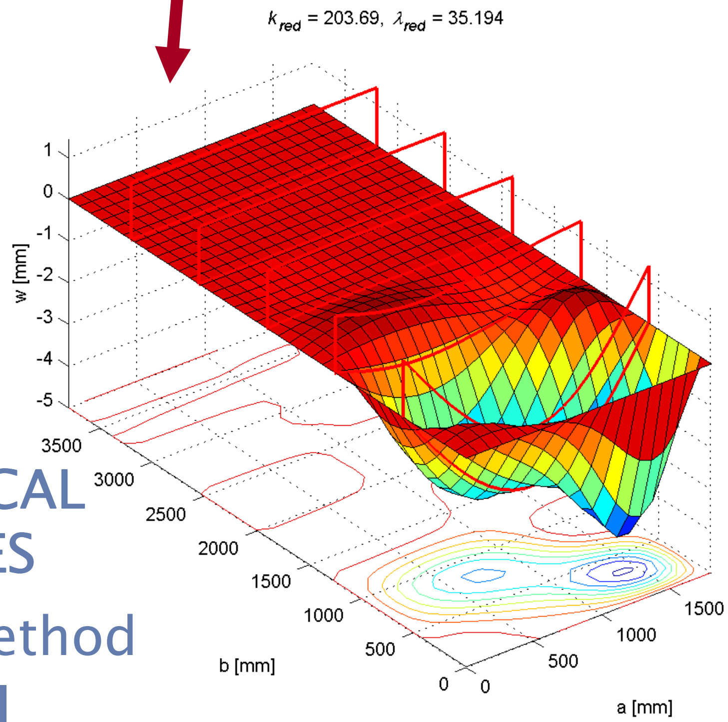
B) SUBMODELS

– FEM

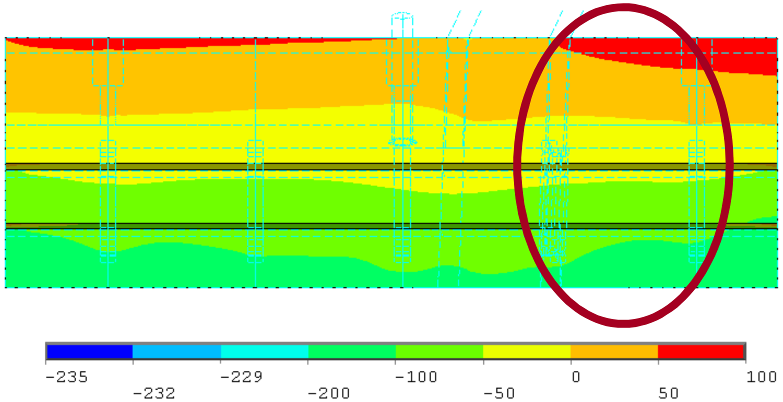


A) TYPICAL PLATES

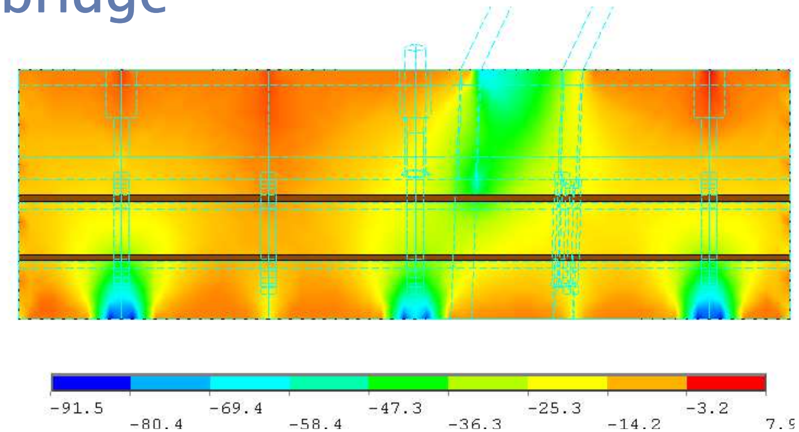
energy method



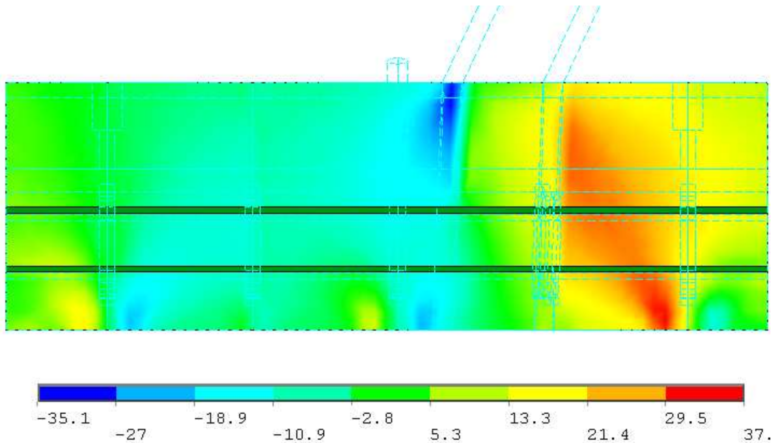
Actual calculations on the Danube bridge



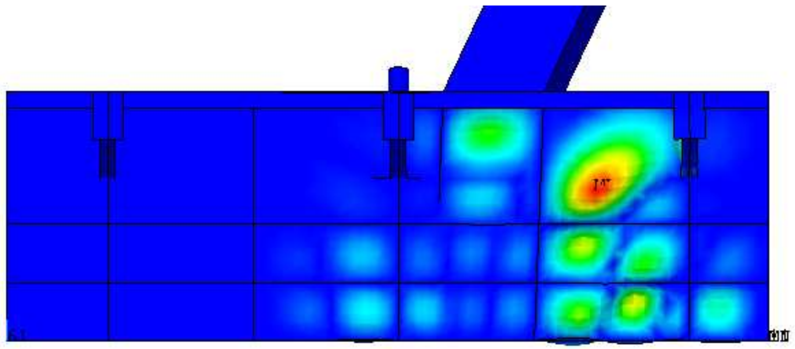
axial stresses [MPa]



vertical stresses [MPa]



shear stresses [MPa]



$\alpha_{cr} = 8.332$

buckling shape

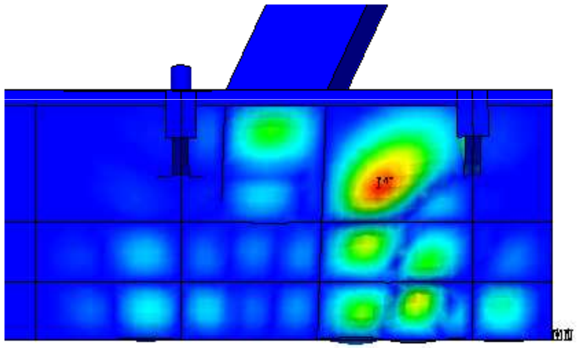
(3) Irregular configuration and stress field – ??? no rule given

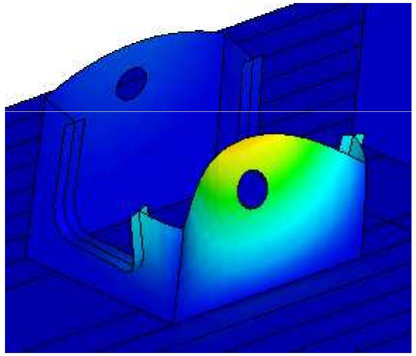
- assume plate-type behaviour → generalized
- plate slenderness (λ_0) and reduction factor (ϕ_b)

$$\sigma_{red,cr} = \alpha_{cr}\sigma_{red,\max}$$

α_{cr}: critical load factor

$$\lambda_0 = \sqrt{\frac{\pi^2 E}{\sigma_{red,cr}}}$$





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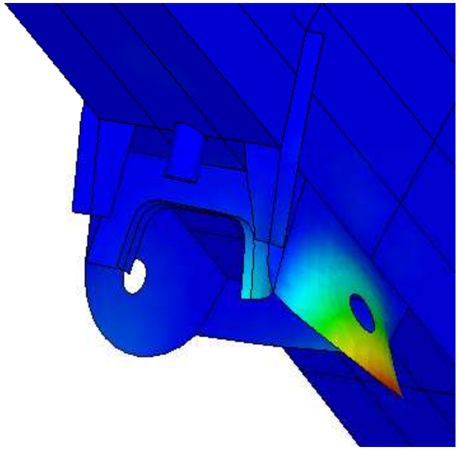
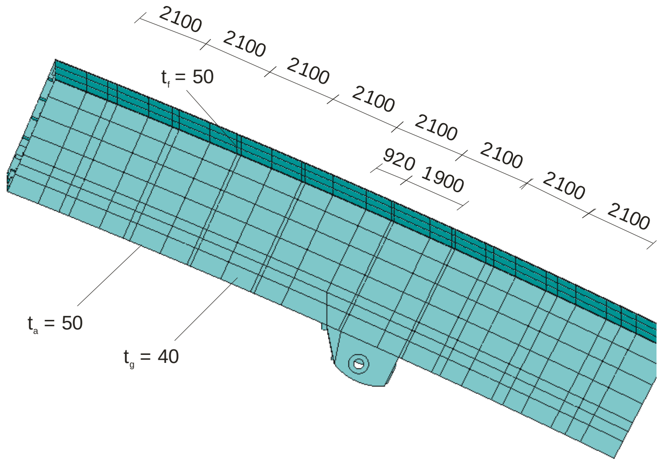
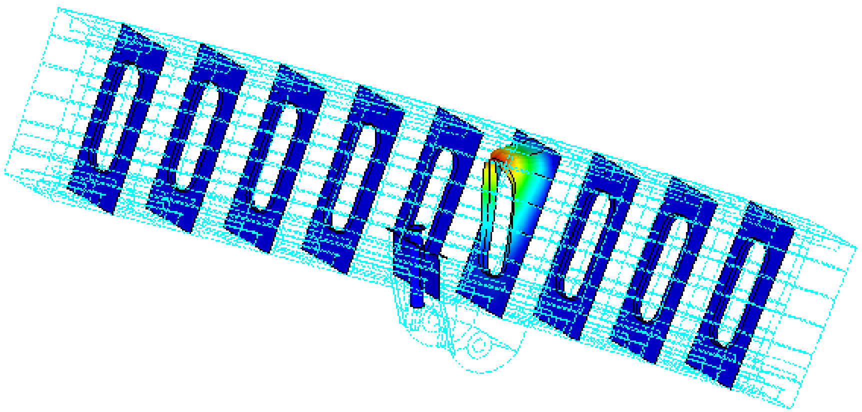
ϕ_b reduction factor for plates

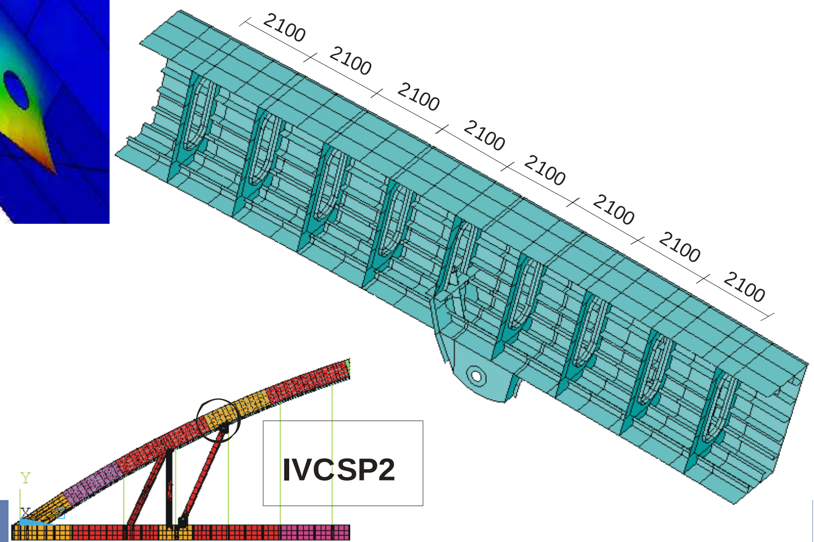
- check: $\sigma_{red} = \sqrt{\sigma^2 + 3\tau^2} \leq \phi_b \cdot \sigma_e$

Actual calculations on the Danube bridge









Design methods – Eurocode 3 Part 1-5

(1) basic procedure for stiffened plates in complex stress fields (no use of numerical models)

(2) partial use of FEM: plate slenderness from bifurcation analysis

(3) reduced stress method

(4) finite element analysis based design (full numerical simulation)

(1) Basic procedure (no use of numerical models)

- consideration of both plate-type and column-like buckling

a) plate-type:

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

$\sigma_{cr,p}$ crit. stress for overall buckling e.g. from orthotropic plate theory

$\downarrow$ ρ

b) column-like:

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

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

$\downarrow$ χ_c

c) interpolation:

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

- cross-section resistance – with effective area

$$N_{c,Rd} = \frac{A_{c,eff} f_y}{\gamma_{M0}}$$

- check: $N_{Ed} \le N_{c,Rd}$

(1) Basic procedure (no use of numerical models)

- consideration of both plate-type and column-like buckling

a) plate-type:

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

$\sigma_{cr,p}$ crit. stre
e.g. fron

ρ

b) column-like:

$$\frac{{}^2 EI_{sl,1}}{A_{sl,1} a^2}$$

- proposal for modification (Maquoi, Skaloud):

$\rho_c = \chi_c$ but not smaller than $\rho_{p,\infty}$

c) interpolation:

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

- cross-section resistance – with effective area

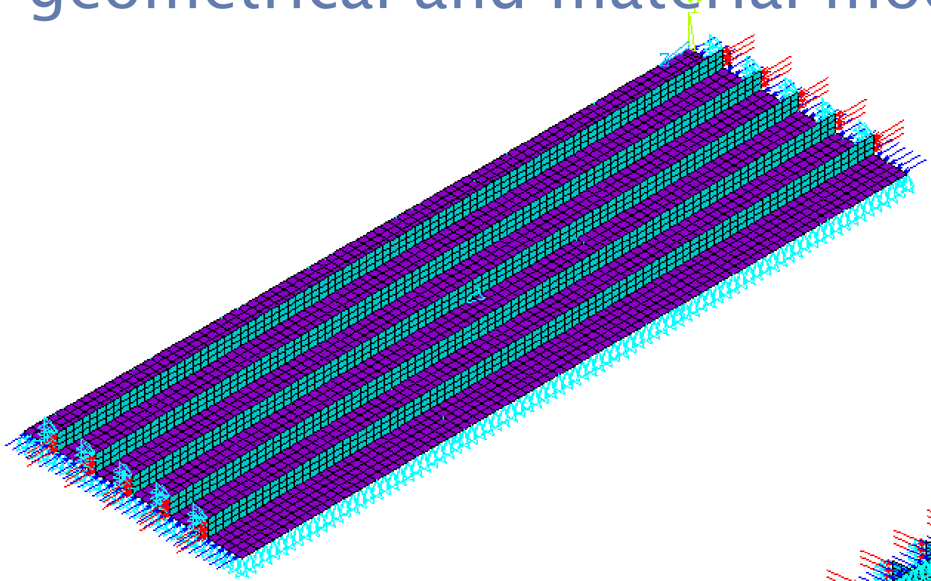
$$N_{c,Rd} = \frac{A_{c,eff} f_y}{\gamma_{M0}}$$

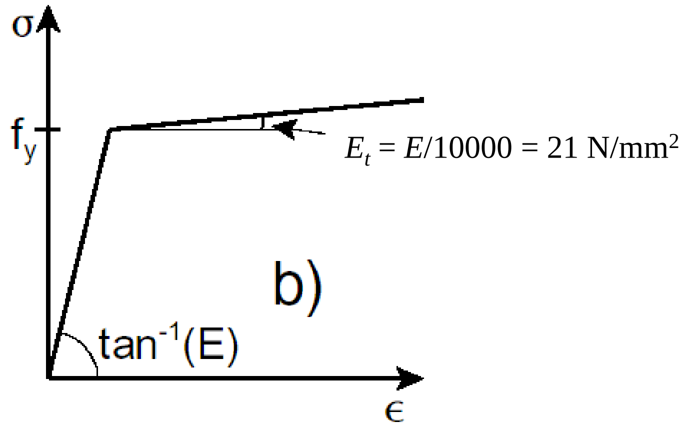
- check: $N_{Ed} \le N_{c,Rd}$

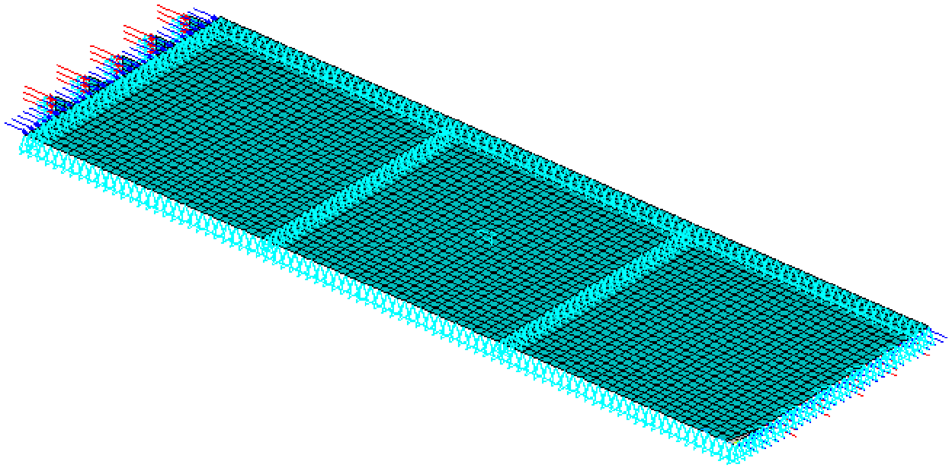
(4) Finite element analysis based design

- geometrical and material non-linearity
- equivalent geometric imperfections
- non-linear simulation

- geometrical and material model



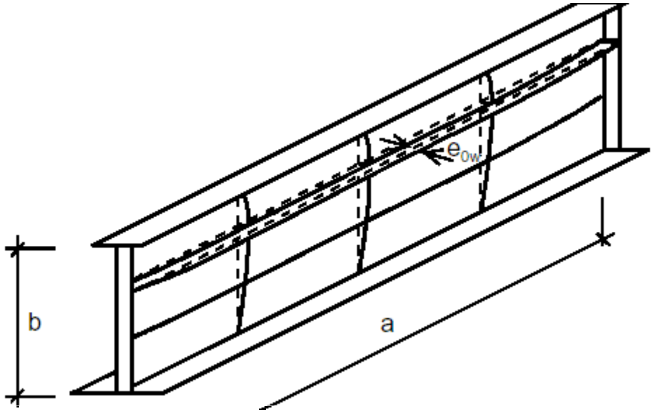




Case Nr.	t_p [mm]	b [m]	a [m]	stiffener
1	40	2	4.56	2 x 280-22
2	30	3.8	4.56	5 x 280-22
3	50	2	2.125	2 x T270-150-22
4	20	3.8	3.9	5 x 280-22
5	16	3.8	3.86	5 x 280-22
6	20	2	3.86	2 x 280-22

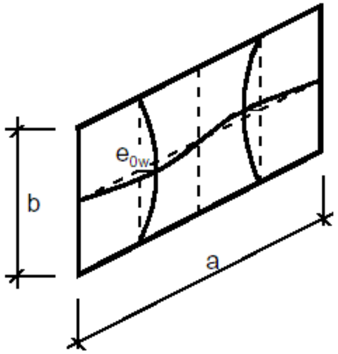
t_p – plate thickness; b – plate width; a – plate length between transverse stiffeners or diaphragms

- equivalent geometric imperfections



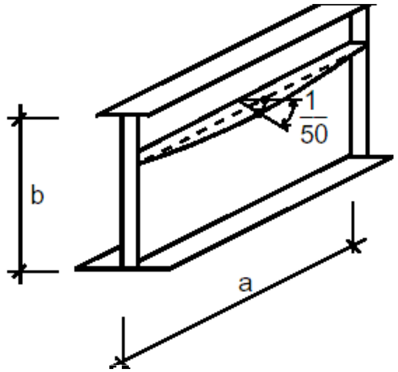
$$e_{0w} = \min(a/400, b/400)$$

a) global imperfection of stiffener



$$e_{0w} = \min(a/400, b/400)$$

b) imperfection of subpanel



$$\phi_0 = 1/50$$

c) local imperfection of stiffener

~ alternatively, relevant buckling shapes, i.e.
a) overall buckling,
b) local buckling of subpanels,
c) torsion mode of the stiffener

- equivalent geometric imperfections

combination of the imperfections:

leading (100%) + others (70%)

↓

PROBLEM when using
buckling shapes as imperfections:

overall/local plate buckling usually
accompanied by the torsion of stiffener

the requirements for the imperfection
amplitudes are difficult to satisfy

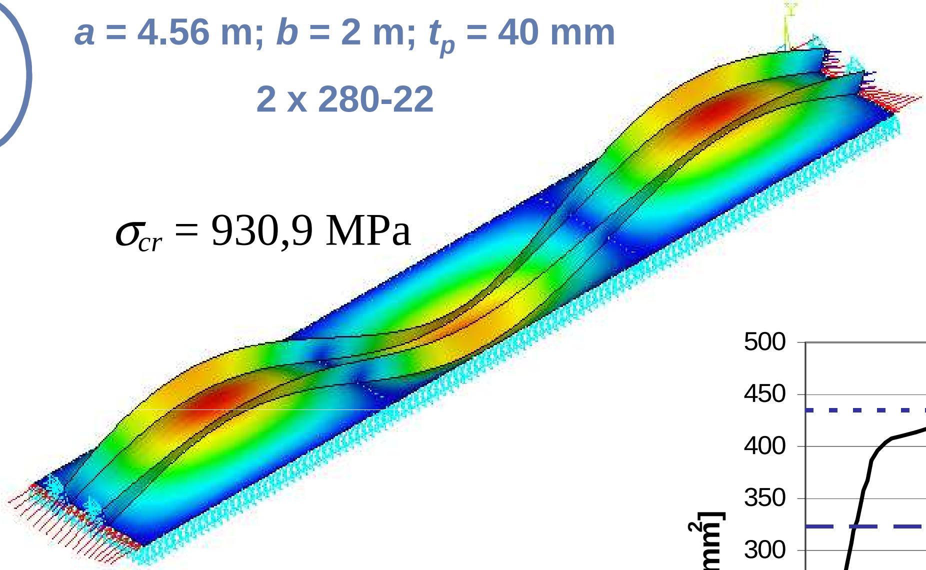
Actual calculations on the Danube bridge

1

***a* = 4.56 m; *b* = 2 m; t_p = 40 mm**

2 x 280-22

σ_{cr} = 930,9 MPa

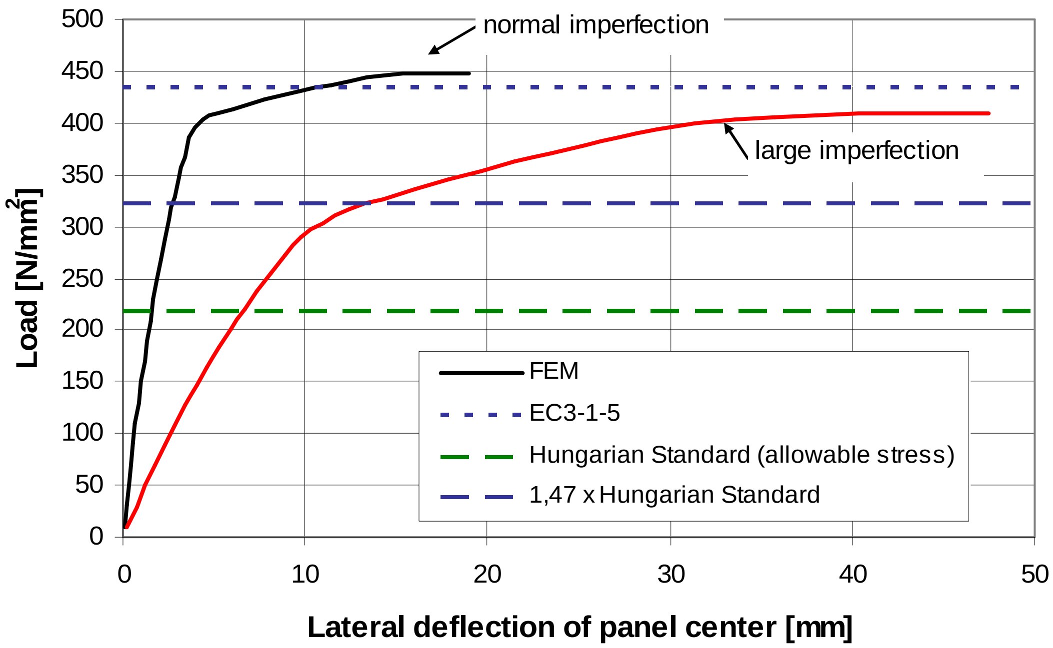


$$e_{0w,1} = \min(a/400, b/400) = 2000/400 = 5\text{mm}$$

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

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

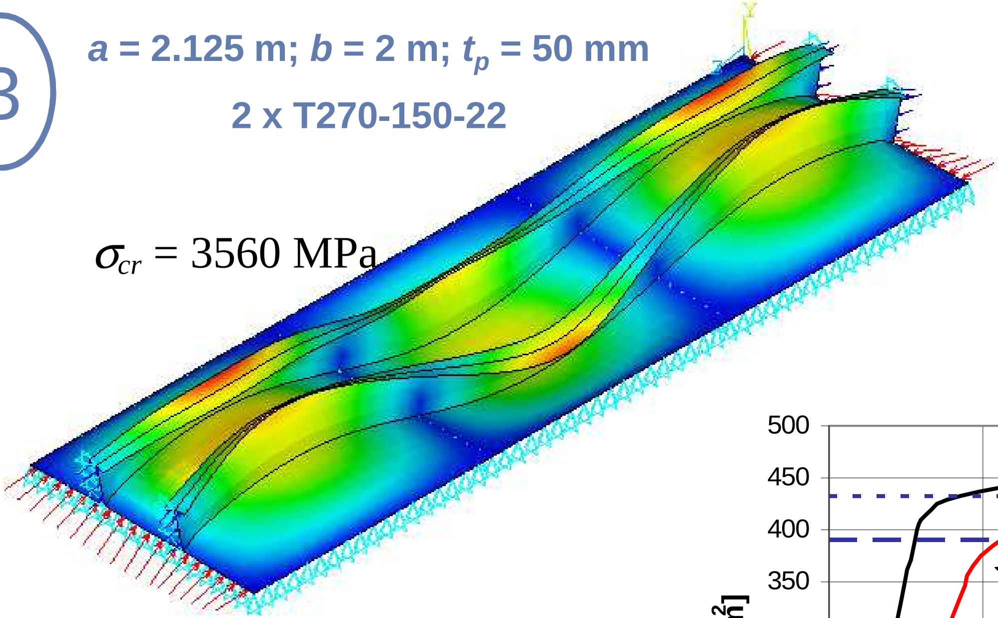
$$\alpha_{0,2} = \frac{5{,}6}{0{,}661 \cdot 10^{-3}} = 8472$$



3

a = 2.125 m; b = 2 m; t_p = 50 mm

2 x T270-150-22



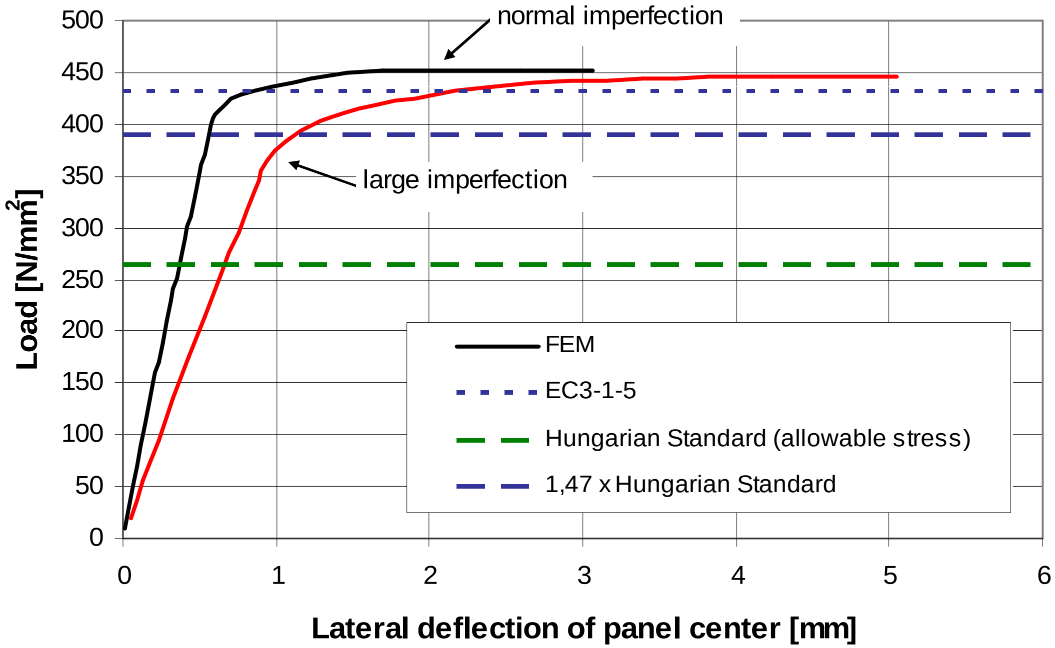
σ_{cr} = 3560 MPa

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

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

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

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

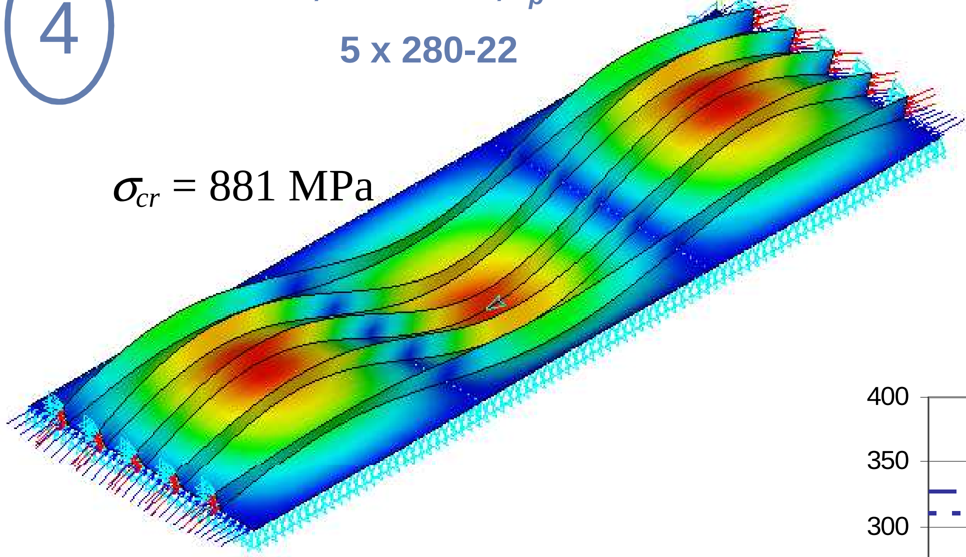


4

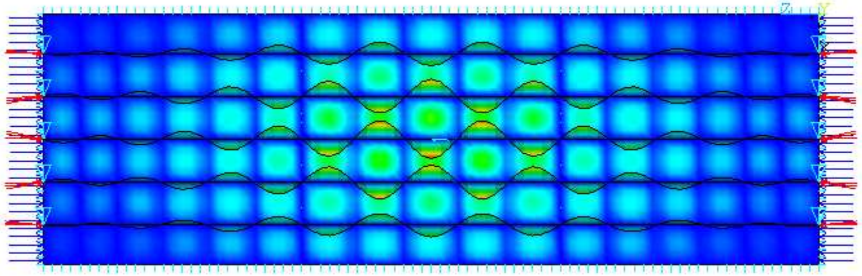
a = 3.9 m; b = 3.8 m; t_p = 20 mm

5 x 280-22

σ_{cr} = 881 MPa



σ_{cr} = 745 MPa



$$e_{0w,1} = \min(a/400, b/400) = 3800/400 = 9{,}5\text{mm}$$

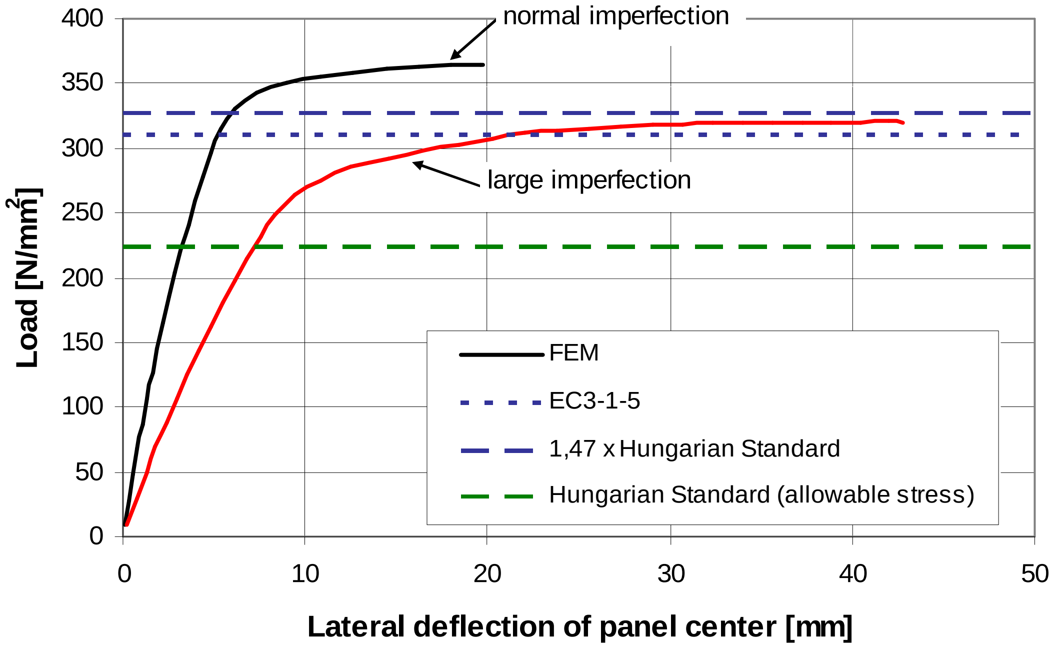
$$e_{0x,2} = h_b\phi_0 = 280 \cdot \frac{1}{50} = 5{,}6\text{mm}$$

$$e_{0w,3} = 650/400 = 1{,}625\text{mm}$$

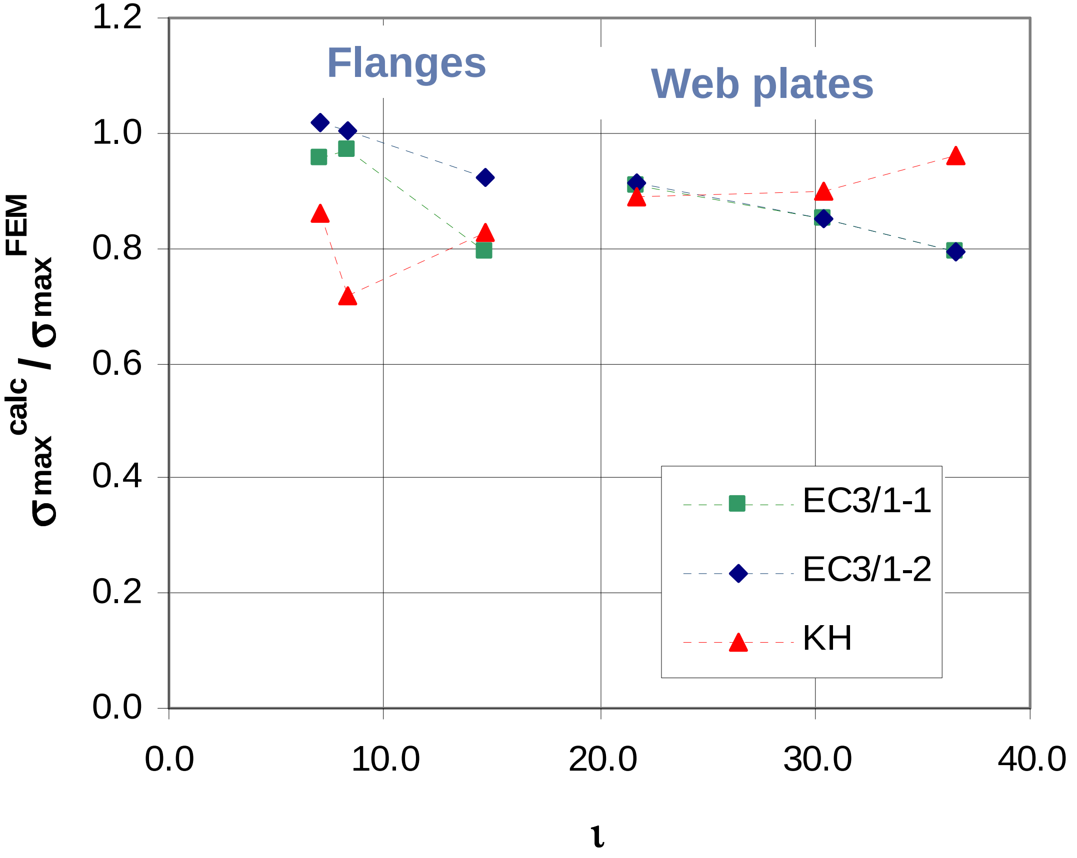
$$\alpha_{0,1} = \frac{9{,}5}{2{,}278 \cdot 10^{-3}} = 4170$$

$$\alpha_{0,2} = \frac{5{,}6}{0{,}577 \cdot 10^{-3}} = 9705$$

$$\alpha_{0,3} = \frac{1{,}625}{0{,}813 \cdot 10^{-3}} = 1999$$



Comparison



$$\iota = \frac{\sqrt{\delta}}{\sqrt[4]{\gamma}} \frac{b_f}{t_f}$$

Concluding remarks

Tied arch bridge project

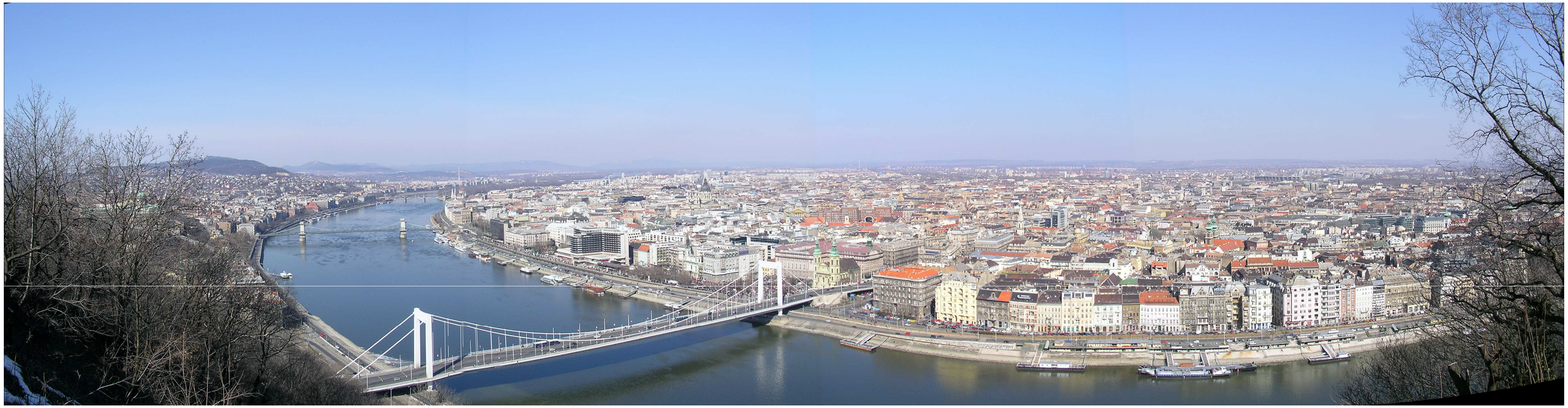
Studies on global stability of tied arch

- Model test
- Evaluation of classical and advanced design methods in comparison to the test ultimate loads.

Studies on the buckling of orthotropic plates

- Design methods – classical and advanced
- Comparison of different design methods to FE simulation based results.

Application



Thank you for your attention!