Izmit Bay Bridge, Steel Towers

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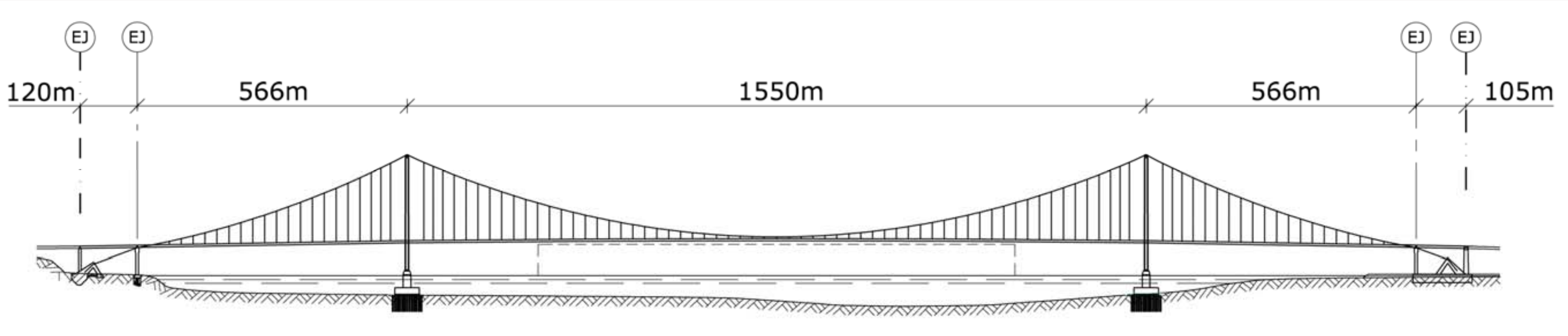
COWI, Bridges - International

Izmit Bay Bridge, Steel Towers

- Fourth longest suspension bridge
- 1550m main span
- 250m high towers in steel
- Extremely high seismic load
- Short construction period
- Orthotropic steel girder with 3 lanes of road traffic
- Total steel quantity for towers, cables and bridge girder is 70000t
- COWI is responsible for the detailed design


Izmit Bay Bridge, Steel Towers
İstanbul
Beşiktaş
Beyoğlu
Bağcılar
Esenler
Küçükçekmece
Bakırköy
Ümraniye
Kadıköy
Maltepe
Kartal
Samandira
Sultanbeyli
Kınalıada
Adalar
Sea of
Marmara
Gebze
Darıca
Çayırova
Şekerpınar
Dilovası
Hersek
Altınova
Karamürsel
Degirmendere
Gölcük
Körfez
İzmit
Yalova
Çınarcık
Taşköprü
Şile
Ağva
Merkez Mh.
Cumaköy
Tepecik
Mollafeneri

Izmit Bay Bridge, Steel Towers



- Navigational clearance profile 64.3 x 1000m
- Tower foundations at 40m water depth with base isolation
- Steel towers
- Bridge deck continuous trough towers with no vertical supports
- Bridge deck supported in transverse direction by wind bearings

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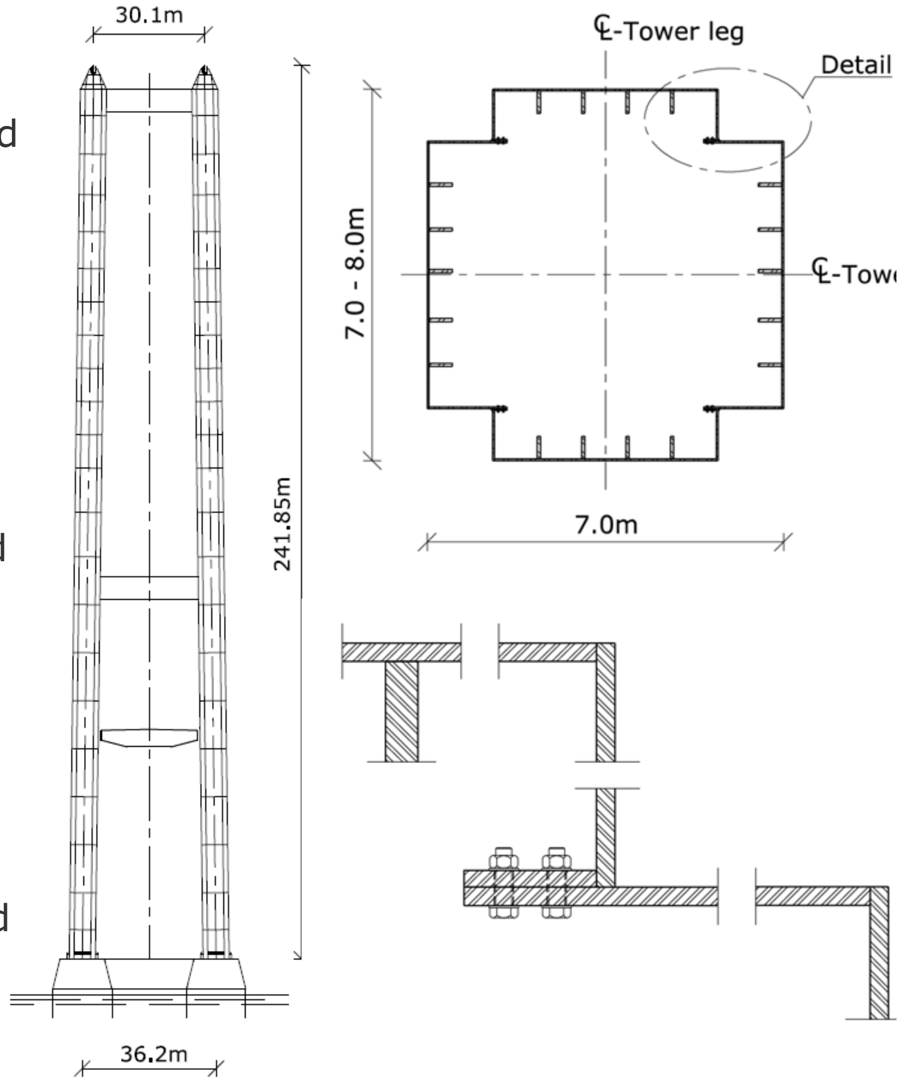
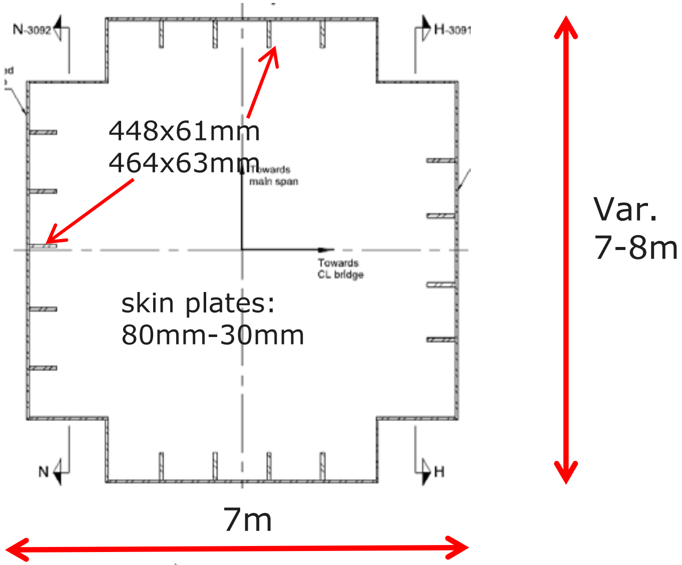
Steel Towers, facts

- Steel towers gain from low weight and increased flexibility in seismic loading
- Fast construction
- Preferable to design for normal ULS combinations and then verify the towers for seismic load combinations
- The extreme seismic demands show however that plastic design is necessary for an optimized and economical tower design (to keep weight down and maintain high flexibility)
- Constructed by 22 prefabricated blocks
- Horizontal joints by combined welding and bolting
- Vertical joints welded for block 1-11, rest bolted



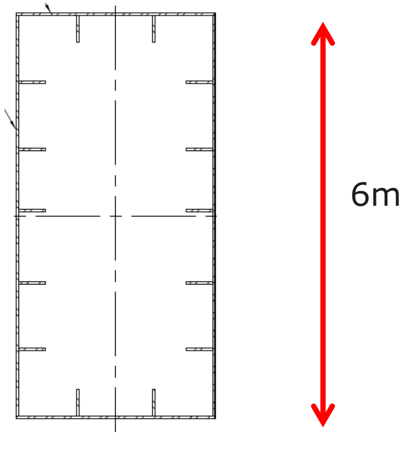
Towers, layout

Longitudinal steel in legs:

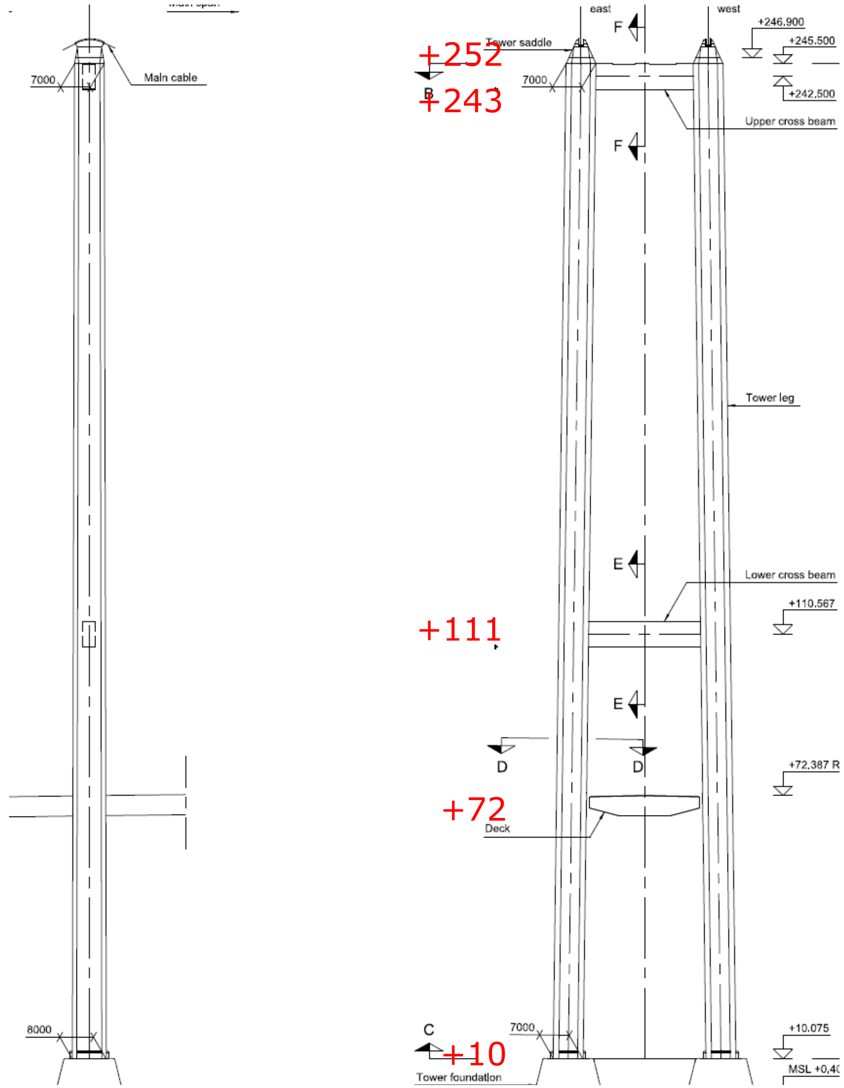


All steel:
S460

Longitudinal steel in cross beams:

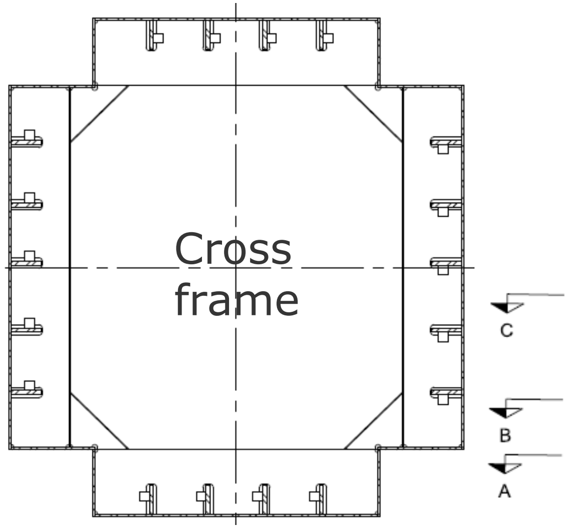


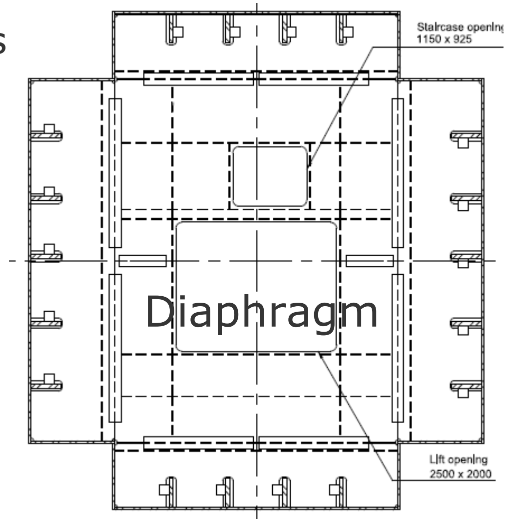
Stiffeners:
330x35mm (upper)
400x42mm (lower)
Skin plate:
20mm – 80mm

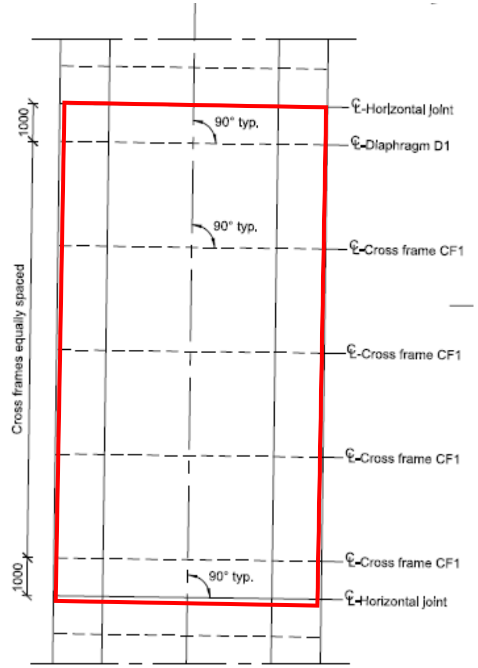


Towers, layout

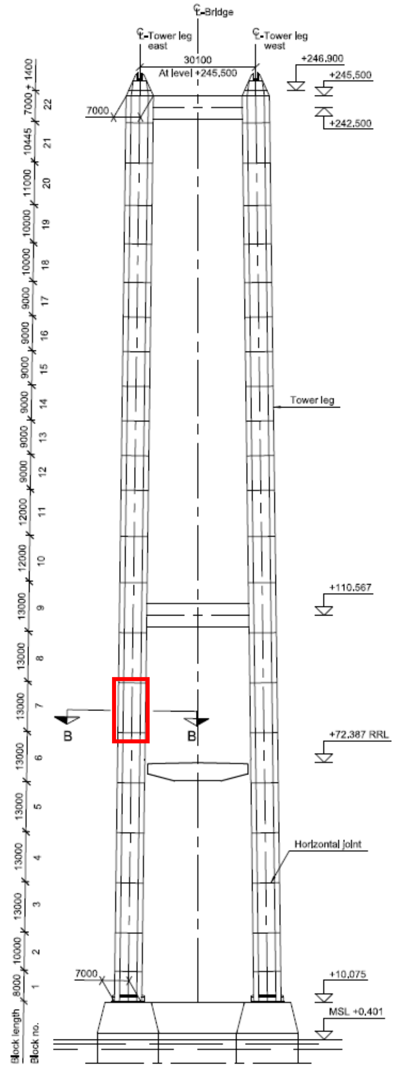
› Typical block length of 10-13m
› A typical block consists of 4 cross frames and 1 diaphragm at the top
› The diaphragm is also used as working platform during erection



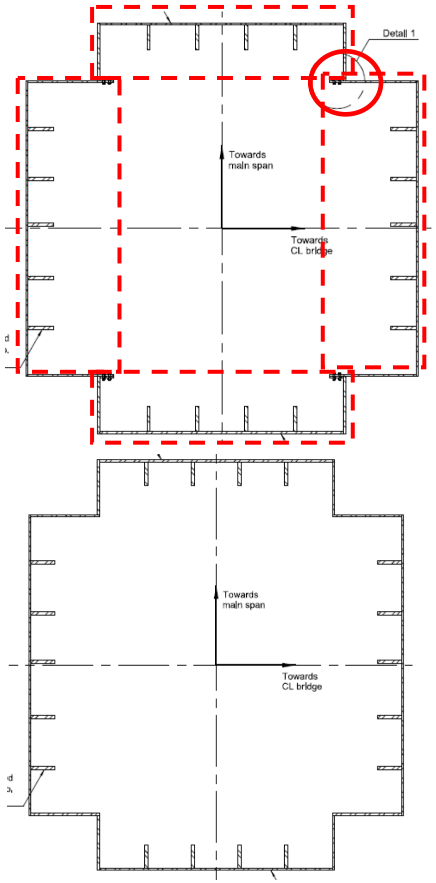


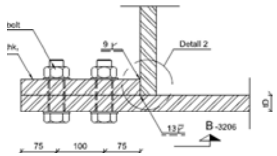


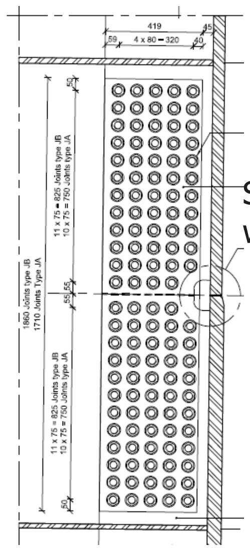
Typical leg block



Towers, layout





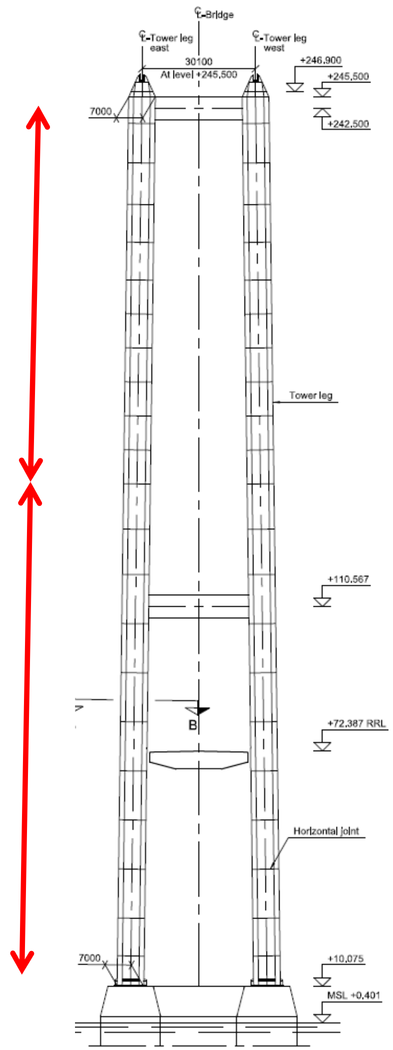


Skin plate welded

Bolt friction connection in stiffeners

Panel erection
Block no. 12-22
Tower crane, 40t

Block erection
Block no. 1-11
Floating crane, 300t



Steel towers, production by Cimtas

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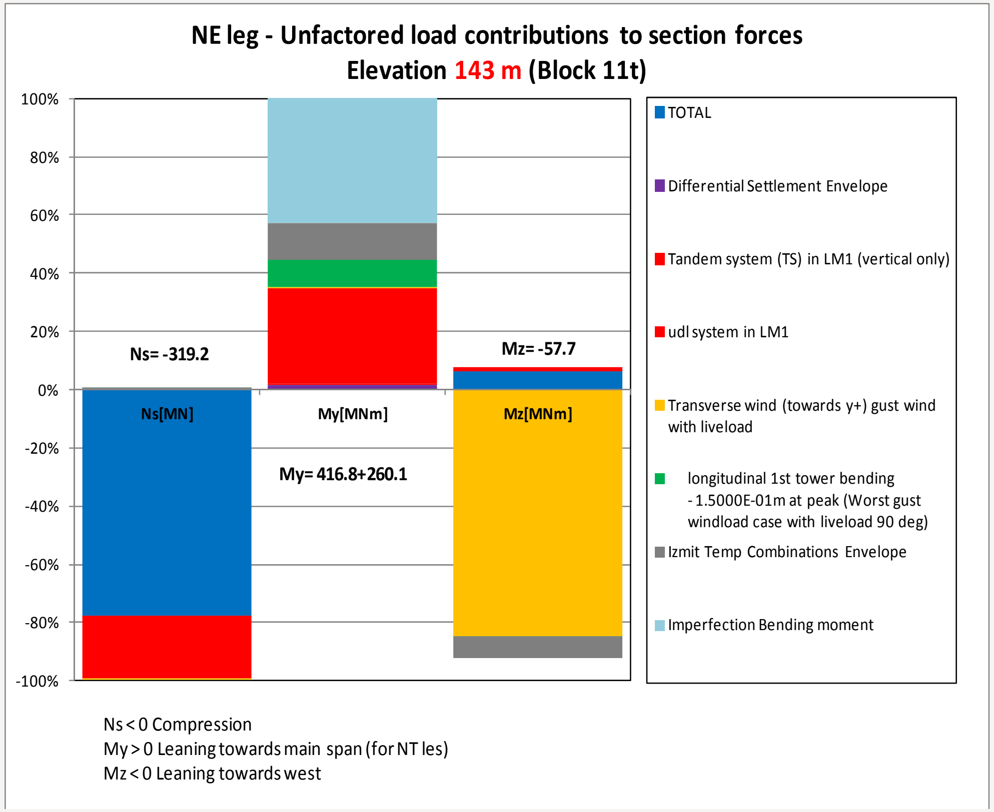
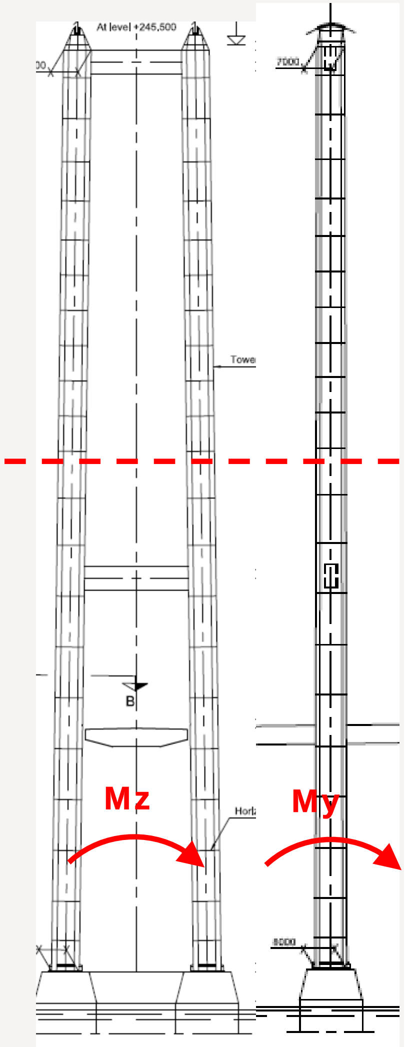
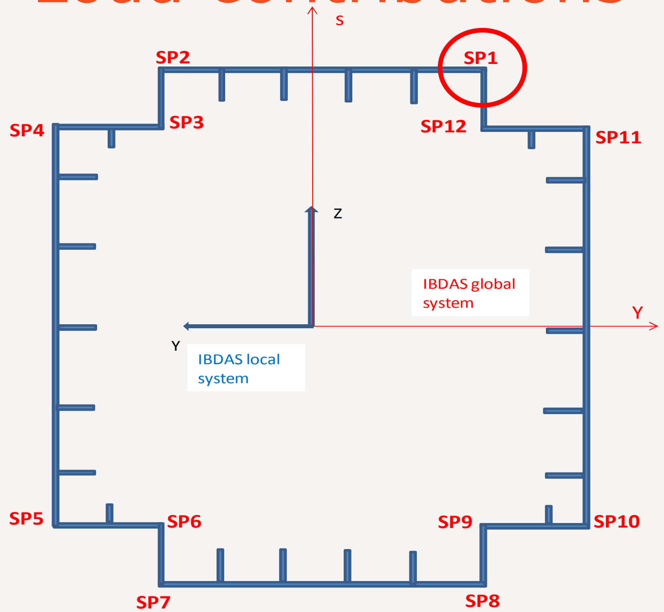
Steel towers, load contributions

- Load contribution in elevation 143m
- N: Mainly from dead load and traffic, only little from transverse wind
- My: Mainly from traffic and imperfection
- Mz: Mainly from transverse wind

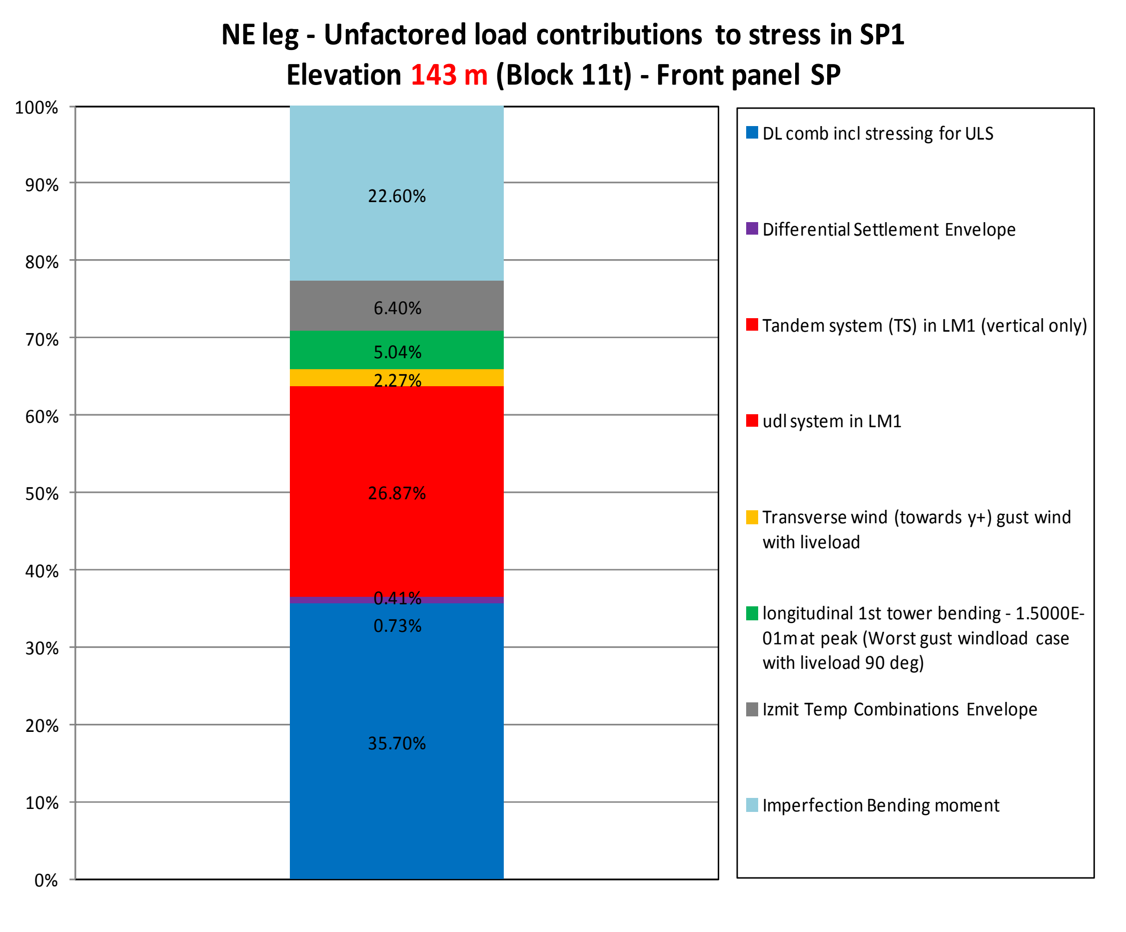




Load contributions



- Stress contribution in elevation 143m
- σ: Mainly from dead load, traffic load and imperfection



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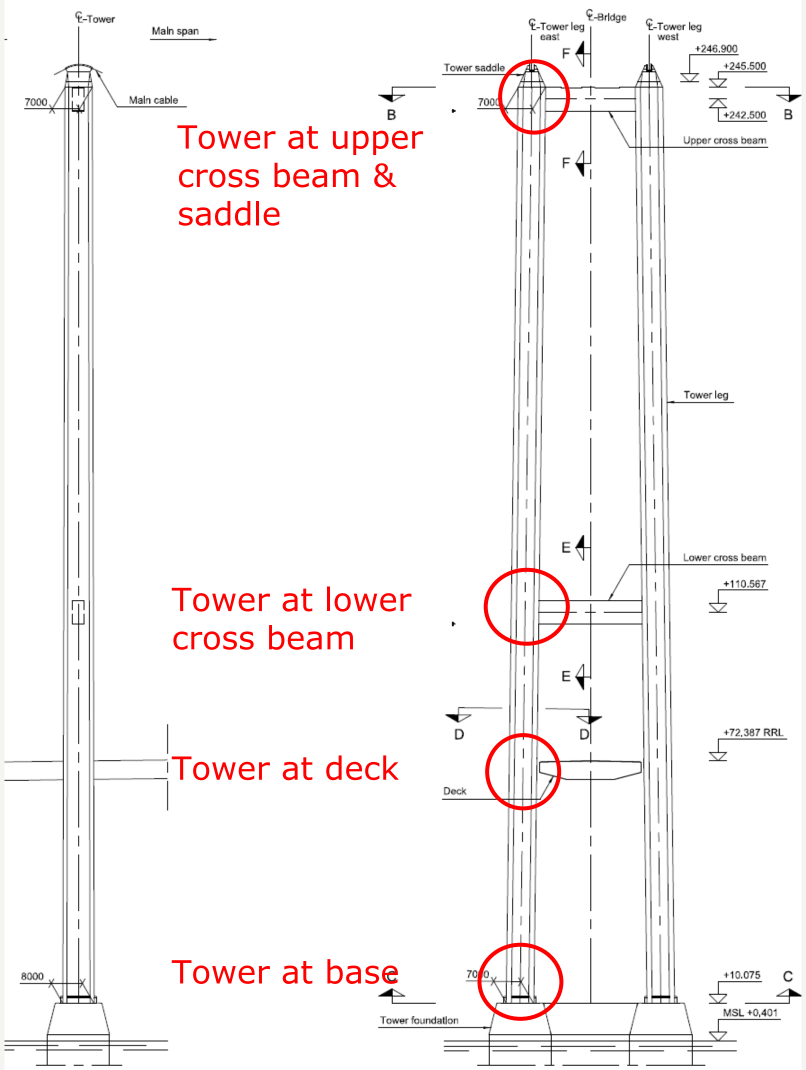
Towers, special areas

- The special areas are in general verified through local shell models built into the global FE-model. Thereby no boundary conditions are necessary and 2.order effects and plasticity are automatically accounted for



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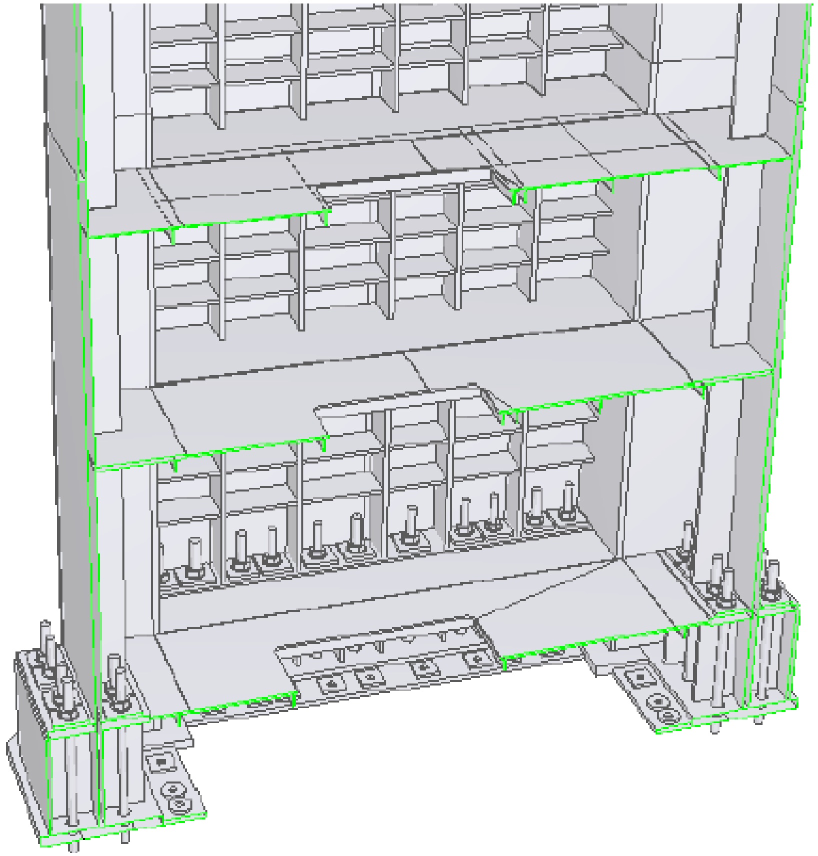
Tower at base

- 84 no. M110 anchor rods class 10.9, L=11m, 6MN preload
- Shear resistance achieved by friction and 34 no. M115 shear rods
- Local ship impact of 10.5MN over 1m2
- 5 diaphragms installed due to ship impact



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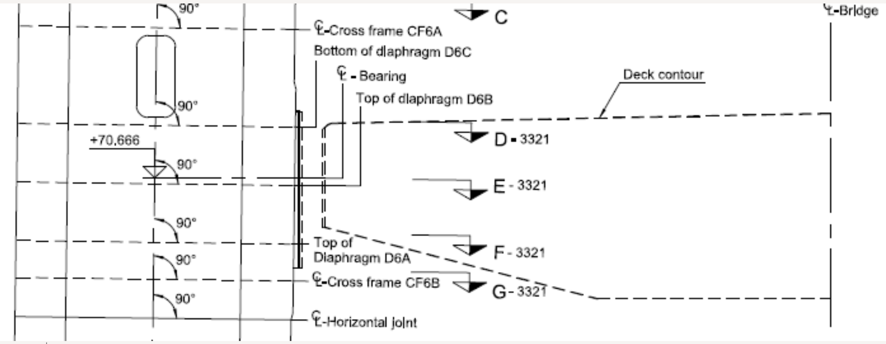
Tower at base, Cimtas

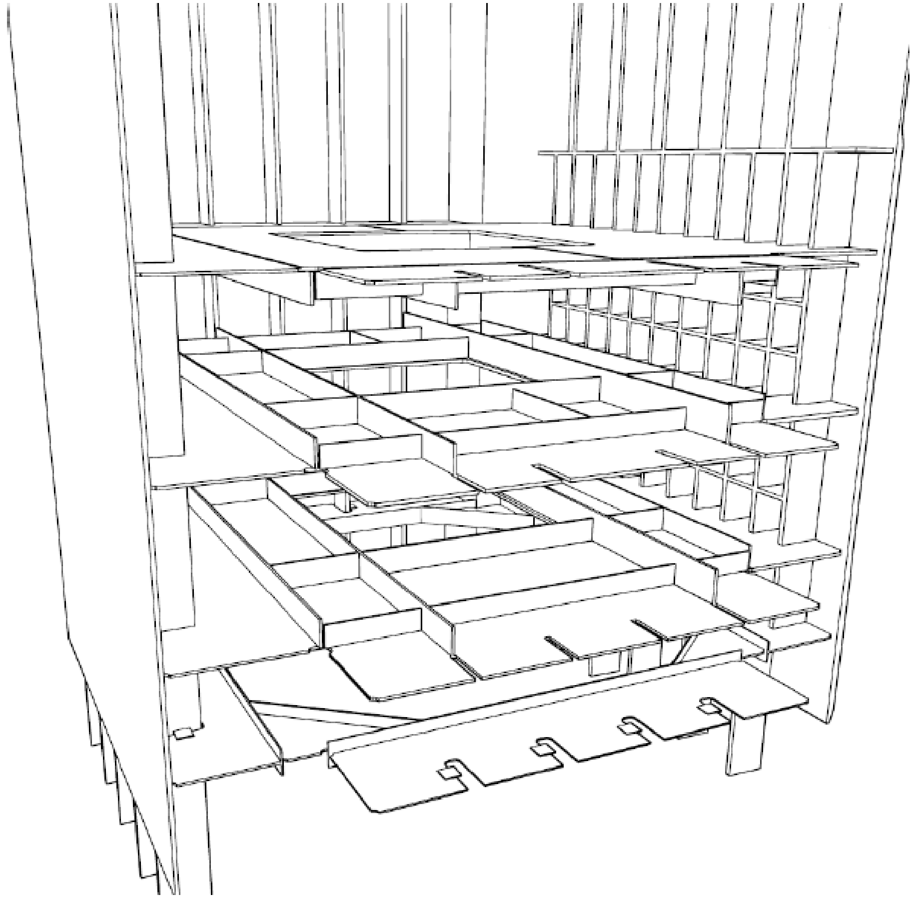
Tower at deck

› Bridge deck is "floating" through the tower legs

› Transverse loading is taken by one bearing on each leg, designed for 13 MN in normal ULS (primary due to wind) and 35 MN in seismic load combination

› Large movements in both longitudinal and vertical directions during seismic events





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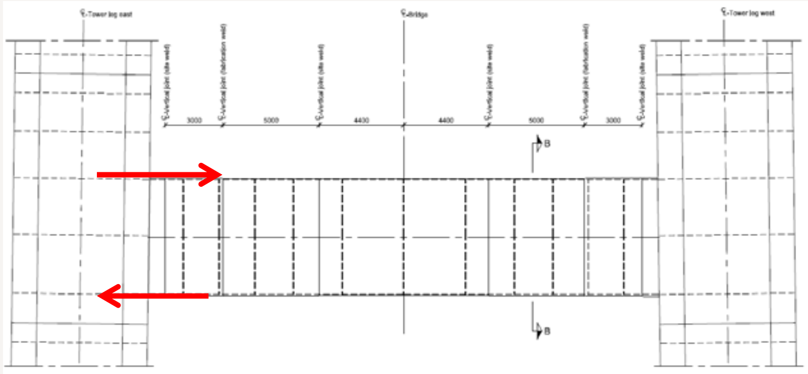
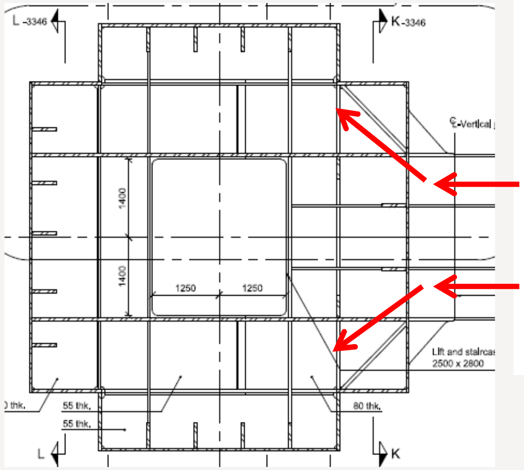
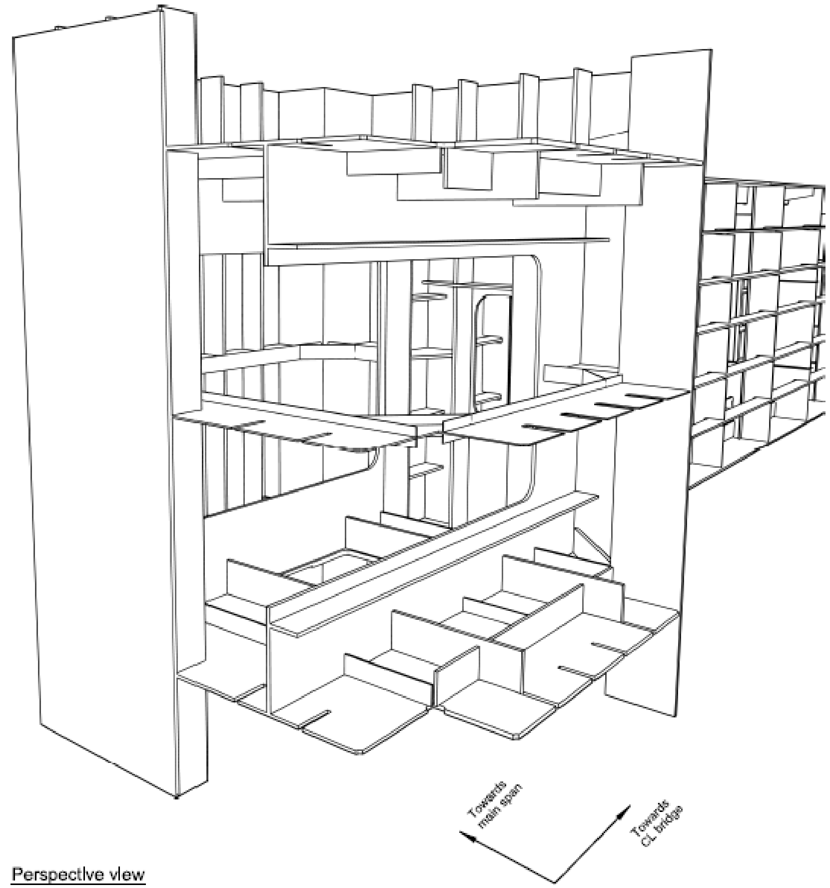
Tower at lower cross beam



› Large bending moments My due to transverse wind loading and seismic load combination





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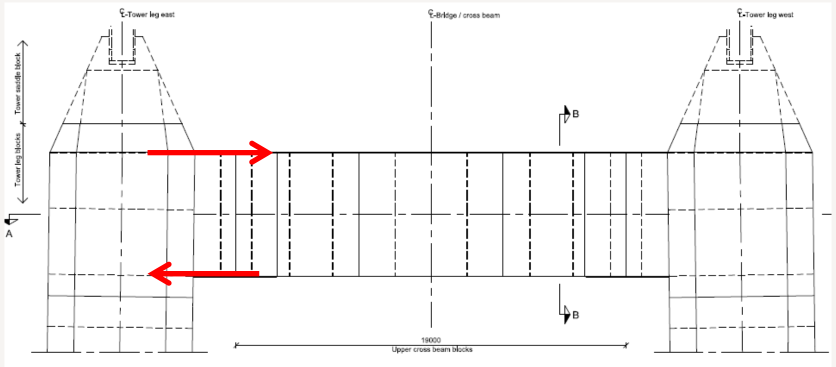
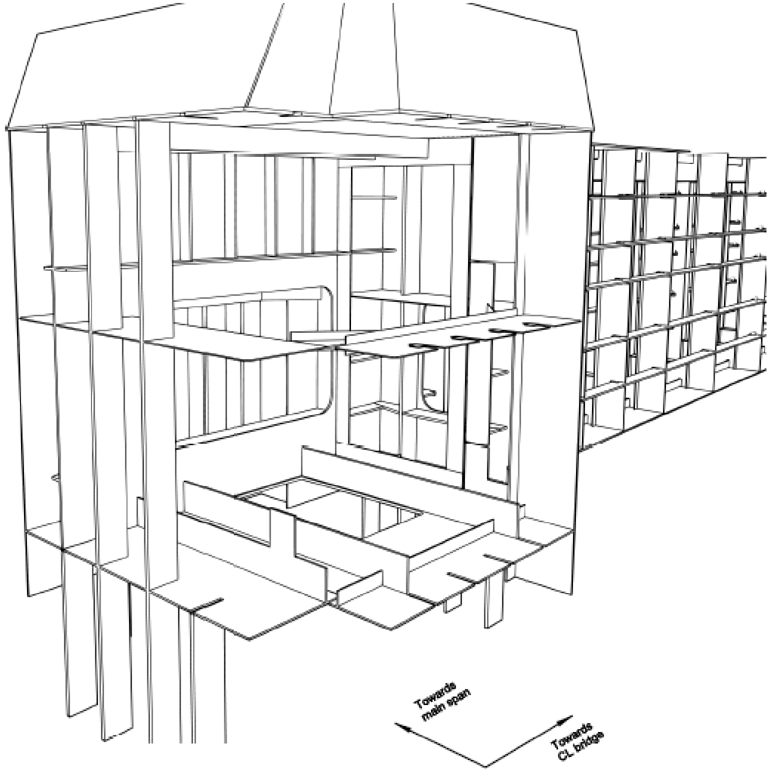
Tower at upper cross beam



- Large bending moments Mz due traffic loading giving uneven deflection of the two tower legs in longitudinal direction
- Others as for lower cross beam



Perspective view

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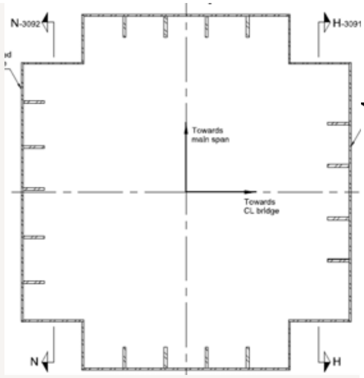
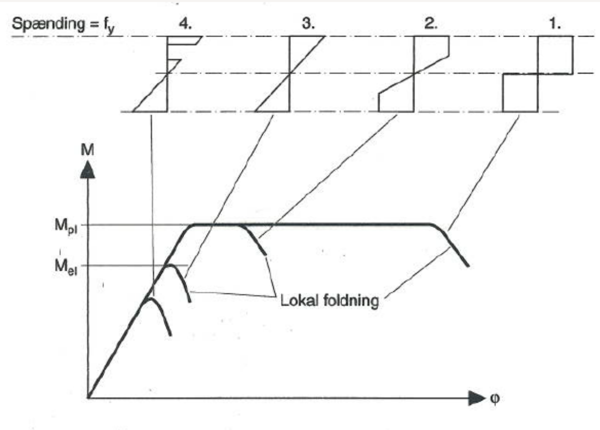
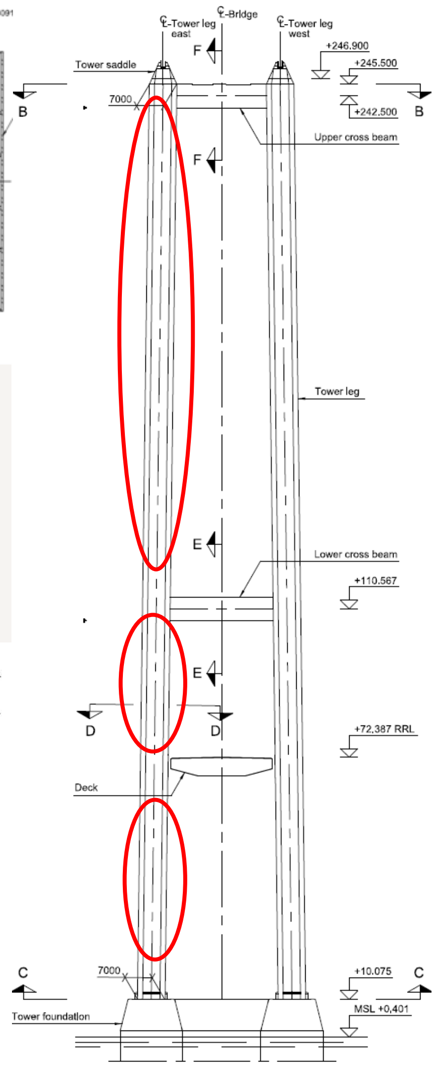
Towers, design of leg members



- Cross section class 1 to 3 are generally economical for compression members
- Section class 1 and 2 have post elastic capacity
- Initial proportioning made to correspond approximately to class 2 in regions with high seismic demands
- Class 2 cross-sections are those which can develop their plastic moment resistance, but have limited rotation capacity because of local buckling
- Cross section class definition applies for members where local buckling is controlled by plate proportioning only
- For stiffened plates, transverse stiffener spacing must also be considered

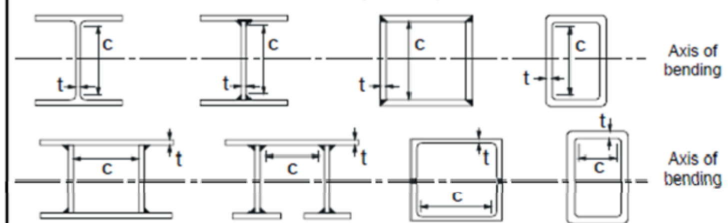




Towers, design of leg members

Table 5.2 (sheet 1 of 3): Maximum width-to-thickness ratios for compression parts

Internal compression parts



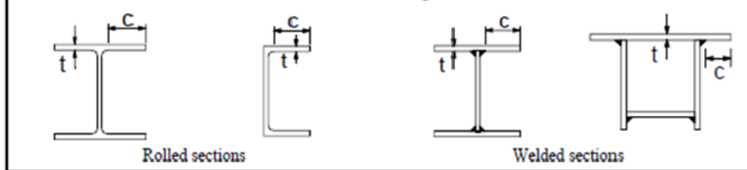
Class	Part subject to bending	Part subject to compression	Part subject to bending and compression
Stress distribution in parts (compression positive)			
1	$c/t \le 72\varepsilon$	$c/t \le 33\varepsilon$	when $\alpha > 0{,}5$: $c/t \le \frac{396\varepsilon}{13\alpha - 1}$ when $\alpha \le 0{,}5$: $c/t \le \frac{36\varepsilon}{\alpha}$
2	$c/t \le 83\varepsilon$	$c/t \le 38\varepsilon$	when $\alpha > 0{,}5$: $c/t \le \frac{456\varepsilon}{13\alpha - 1}$ when $\alpha \le 0{,}5$: $c/t \le \frac{41{,}5\varepsilon}{\alpha}$
Stress distribution in parts (compression positive)			
3	$c/t \le 124\varepsilon$	$c/t \le 42\varepsilon$	when $\psi > -1$: $c/t \le \frac{42\varepsilon}{0{,}67 + 0{,}33\psi}$ when $\psi \le -1$*): $c/t \le 62\varepsilon(1-\psi)\sqrt{(-\psi)}$

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

*) $\psi \le -1$ applies where either the compression stress $\sigma \le f_y$ or the tensile strain $\varepsilon_y > f_y/E$

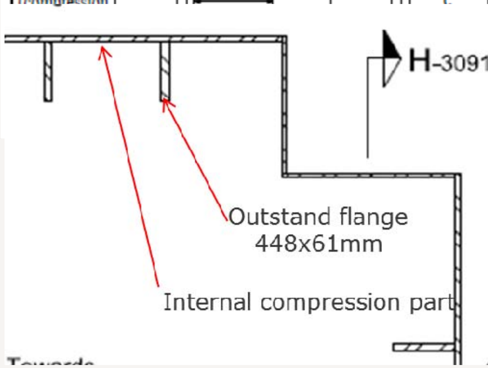
Table 5.2 (sheet 2 of 3): Maximum width-to-thickness ratios for compression parts

Outstand flanges



Class	Part subject to compression	Part subject to bending and compression: Tip in compression	Part subject to bending and compression: Tip in tension
Stress distribution in parts (compression positive)			
1	$c/t \le 9\varepsilon$	$c/t \le \frac{9\varepsilon}{\alpha}$	$c/t \le \frac{9\varepsilon}{\alpha\sqrt{\alpha}}$
2	$c/t \le 10\varepsilon$	$c/t \le \frac{10\varepsilon}{\alpha}$	$c/t \le \frac{10\varepsilon}{\alpha\sqrt{\alpha}}$
Stress distribution in parts (compression			
			$c/t \le 21\varepsilon\sqrt{k_\sigma}$ k_σ see EN 1993-1-5

355	420	460
0,81	0,75	0,71



Towers, design of leg members

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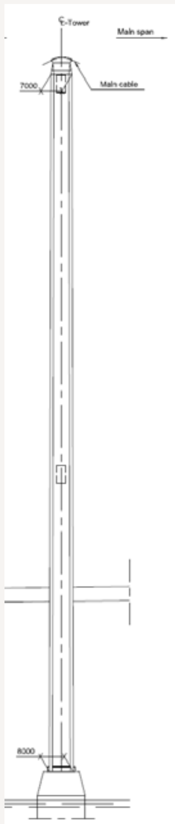
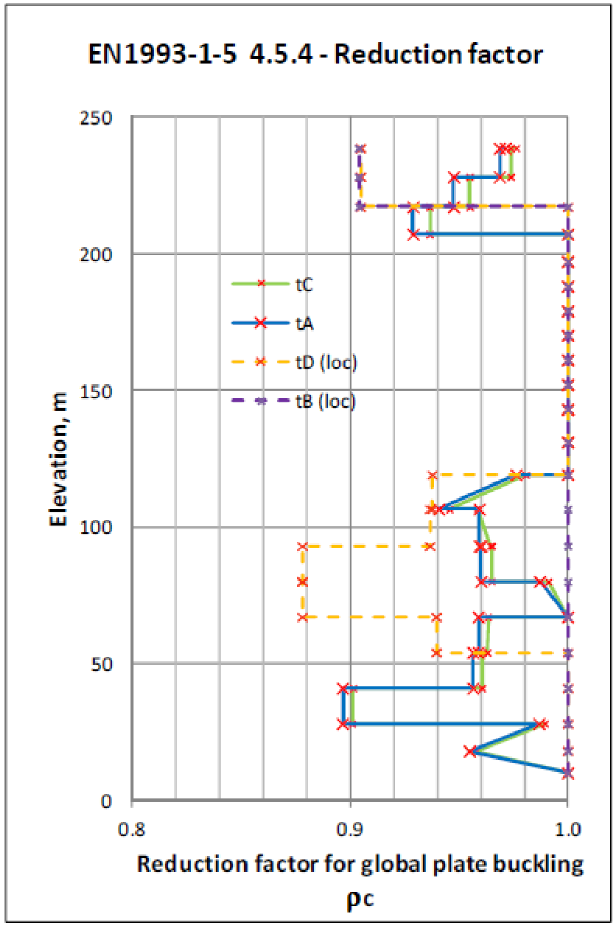
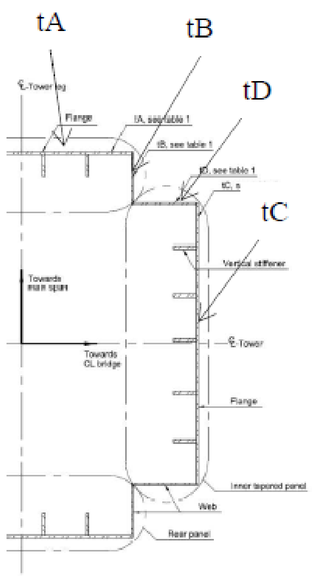
Towers, design of leg members

- From the figure is seen that no reduction in capacity shall be made within the level from 120m up to 205m







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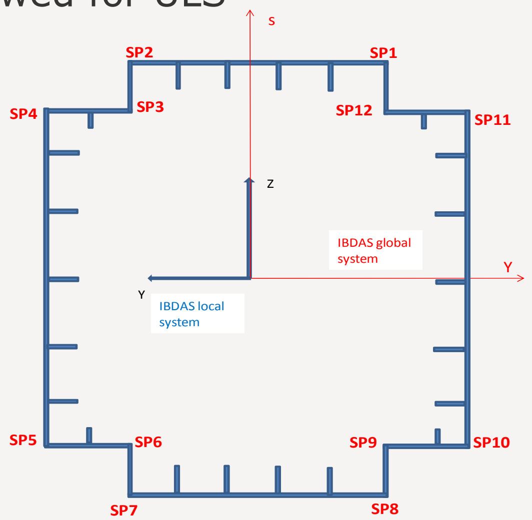
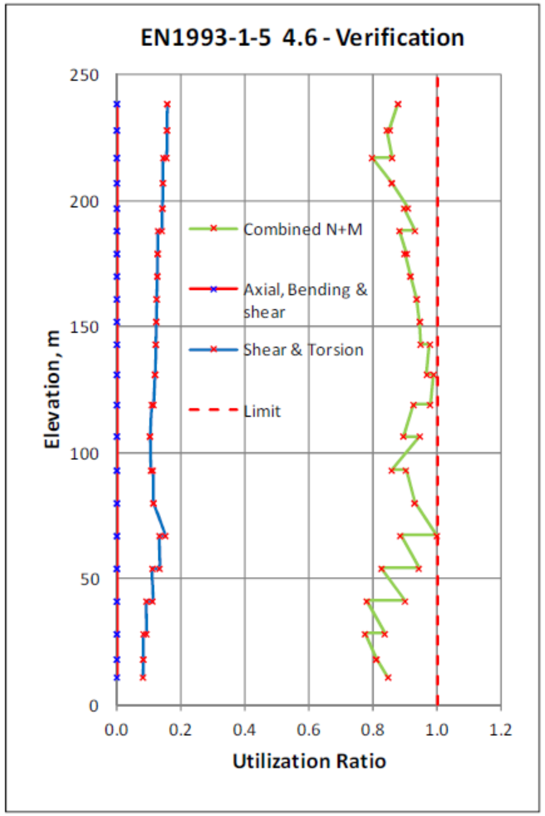
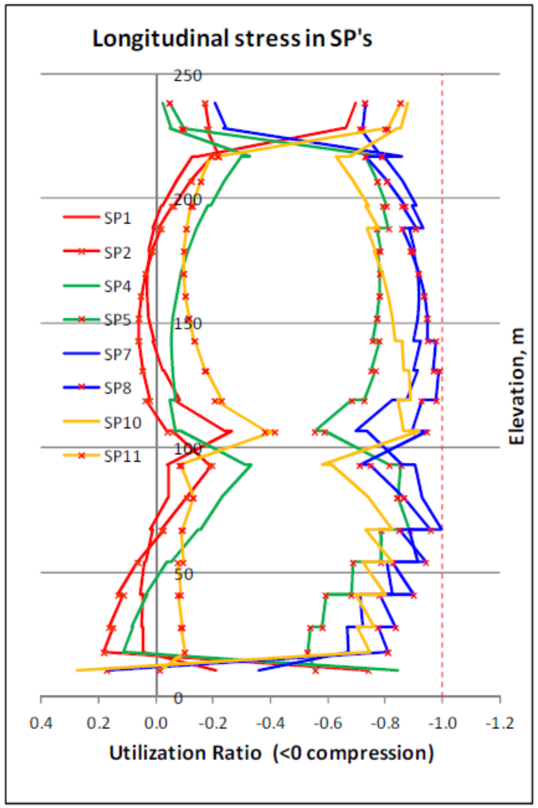
Towers, ultimate limit state (ULS)

- Verification according to EN1993-1-5 considering 12 stress points in the cross section
- All stresses are below yielding
- Plastic design generally not allowed for ULS







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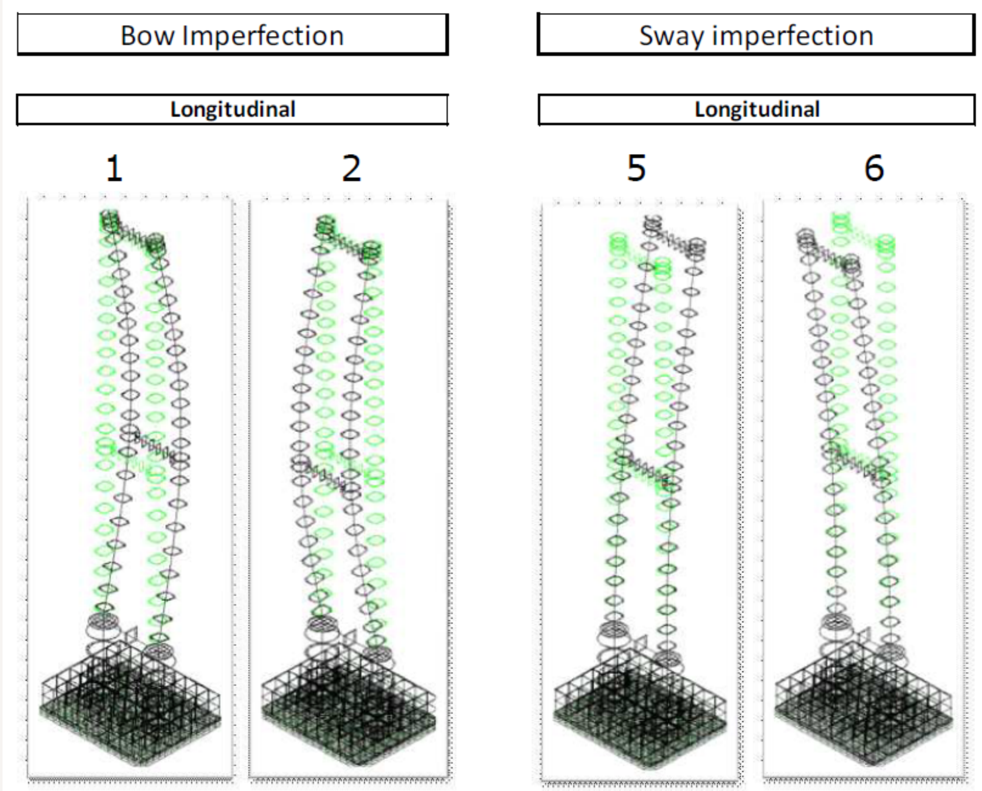
Towers, seismic limit states

- Seismic load case analyses in IBDAS:
  - Non-linear time history analysis
  - Elastic or plastic material
  - Road traffic corresponds to 20% of full traffic load
  - Second order effects included by means of global geometric imperfections

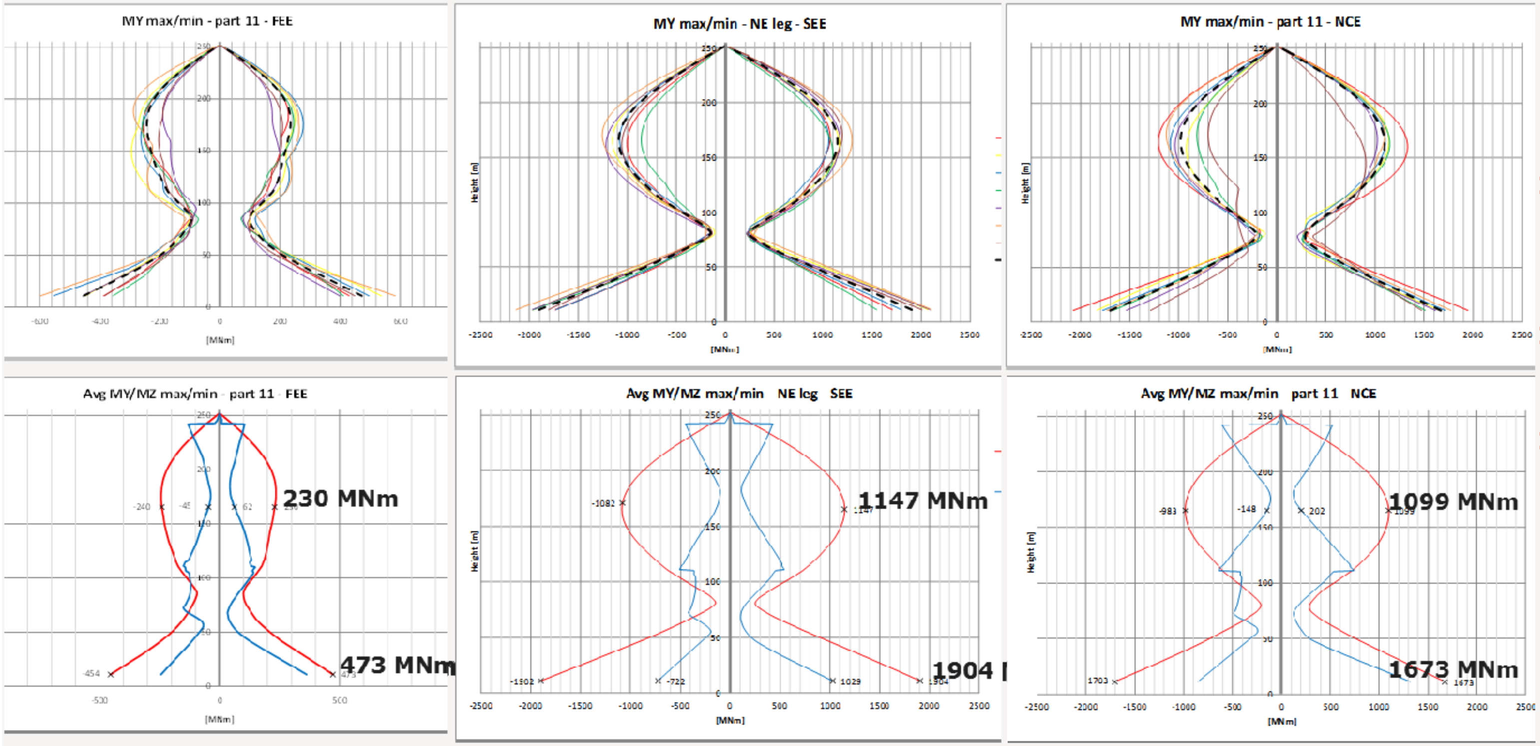


Towers, seismic limit states

Functional evaluation earthquake (FEE) – 150 years

Safety evaluation earthquake (SEE) – 1000 years

No collapse earthquake (NCE) – 2475 years



- 3 seismic events are shown:
  150 years (FEE)
  1000 years (SEE)
  2475 years (NCE)
- Longitudinal bending moments for north tower
- Average of 7 time histories
- South tower less onerous due to different soil conditions

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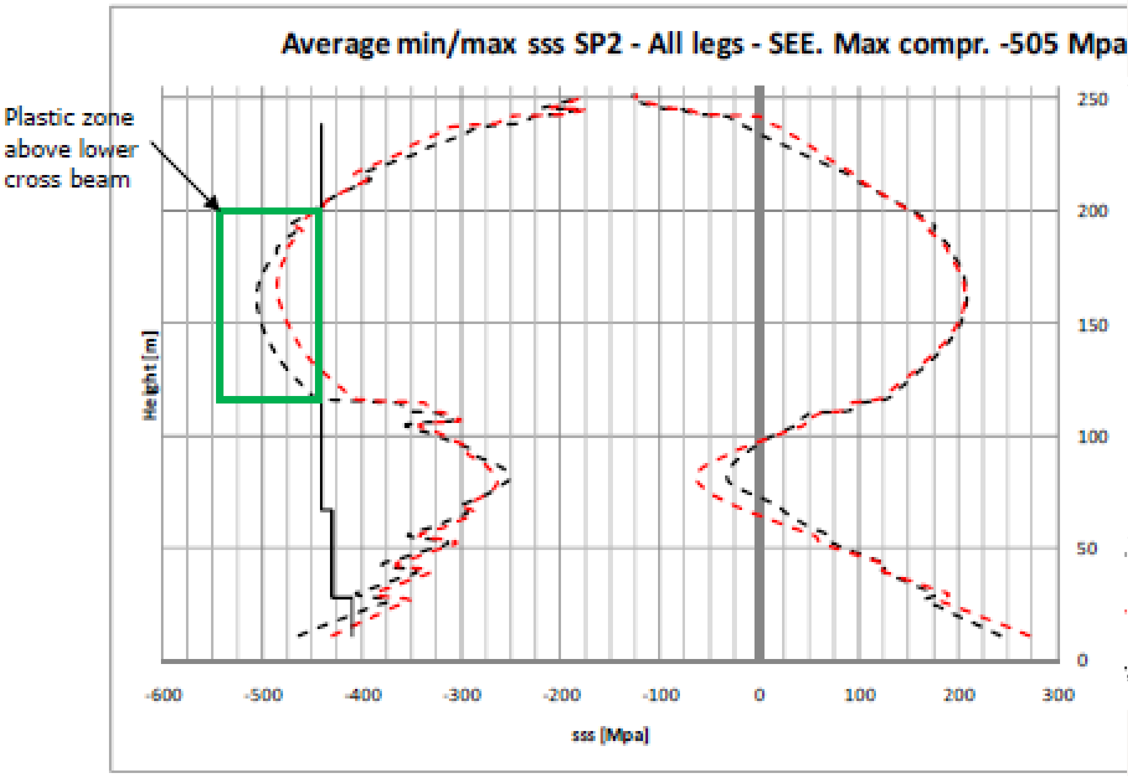
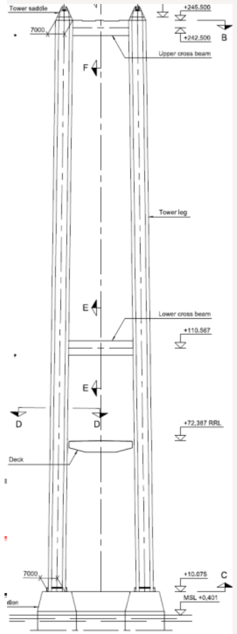
Towers, seismic limit states





- Elastic design not sufficient to verify north tower legs
- For 1000 and 2475 year events, inelastic response is acceptable
- Limited damage, so structure can be restored essentially to its pre-seismic conditions

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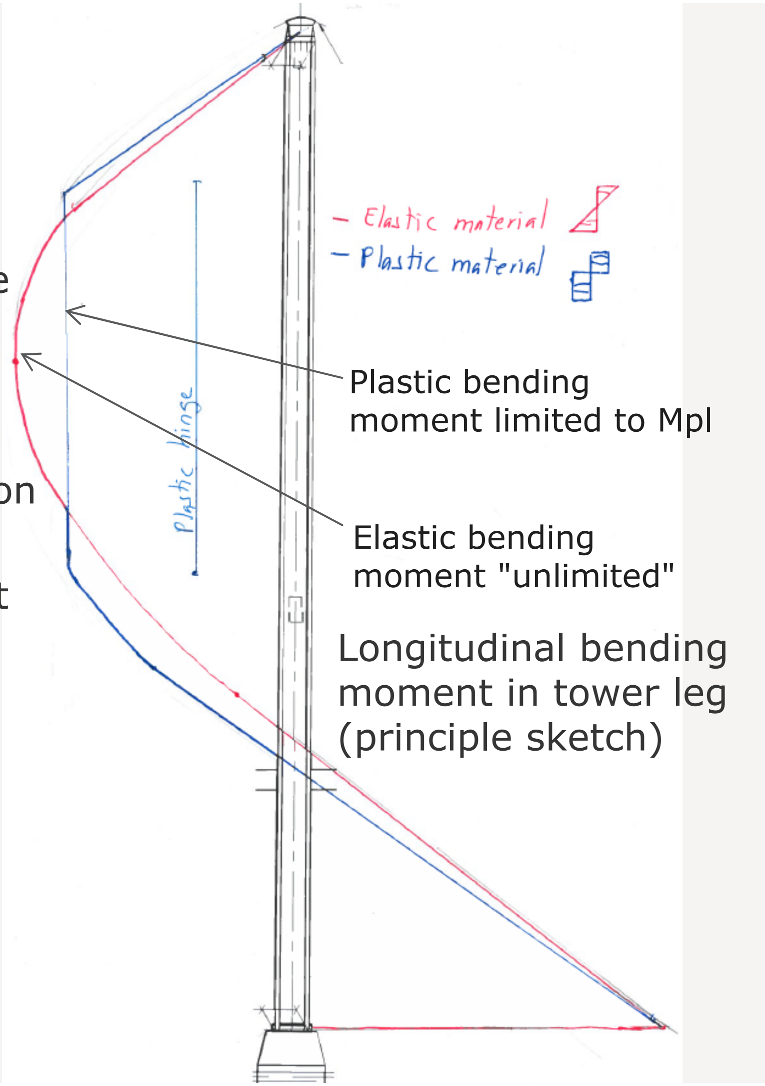
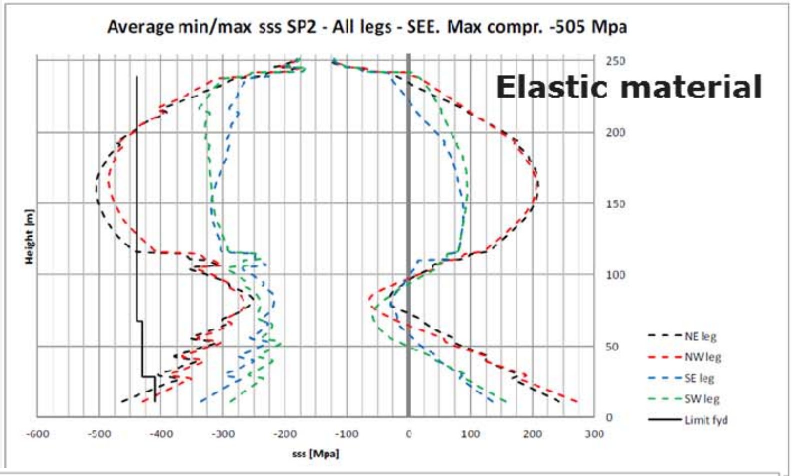
Towers, seismic limit states

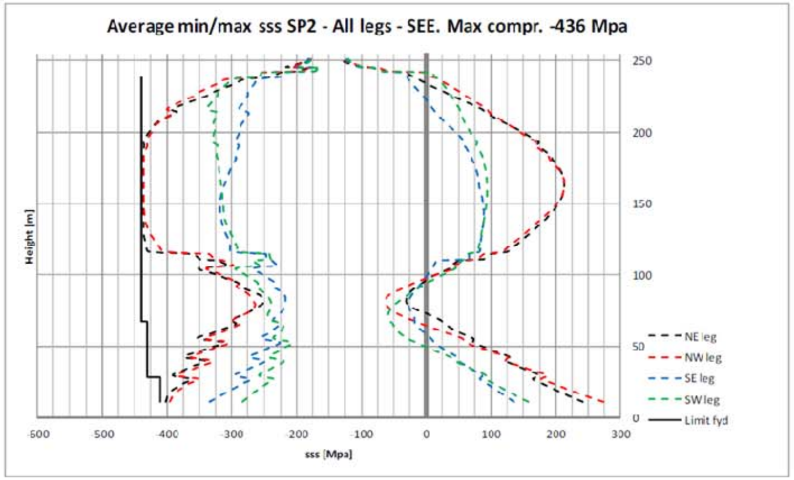
- Force demands calculated by IBDAS nonlinear, 2nd order analysis using plastic material properties
- Realistic value of yield strength to be applied to ensure that the benefit from reduction in peak moment is not taken prior to the actual formation of the plastic hinge
- Seismic design items to be verified:
  - Verification of plastic section capacity - cross section to sustain force demands
  - Verification of rotation capacity - code requirement for ensuring ductility
  - Verification of global integrity – done in global FE-model ensuring no global buckling collapse
  - Verification of permanent deformations – "Repairable damage" after SEE, NCE and to be restored to pre-seismic conditions



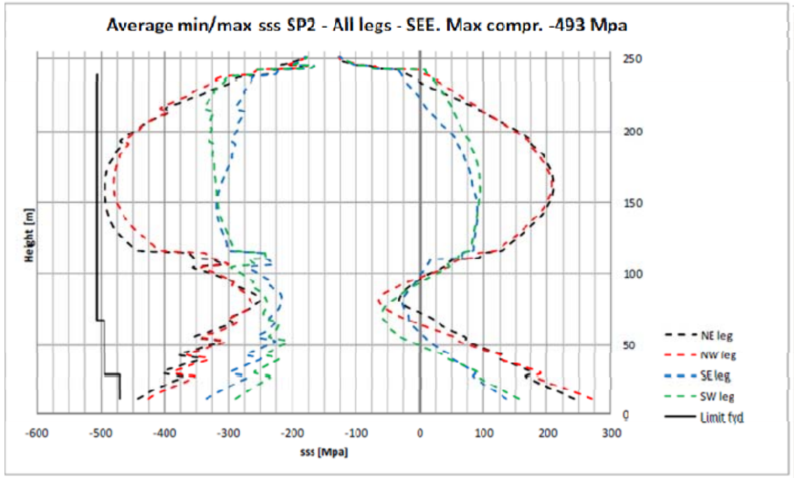
Longitudinal bending moment in tower leg (principle sketch)

Towers, seismic limit states





Plastic material – characteristic yield stress



Plastic material – realistic yield stress

- Plastic verification according to EN1993 part 1-1
- No guidance whether formulas are applicable for plated structural elements
- Need for FE modelling to prove plastic section capacity of tower leg cross section

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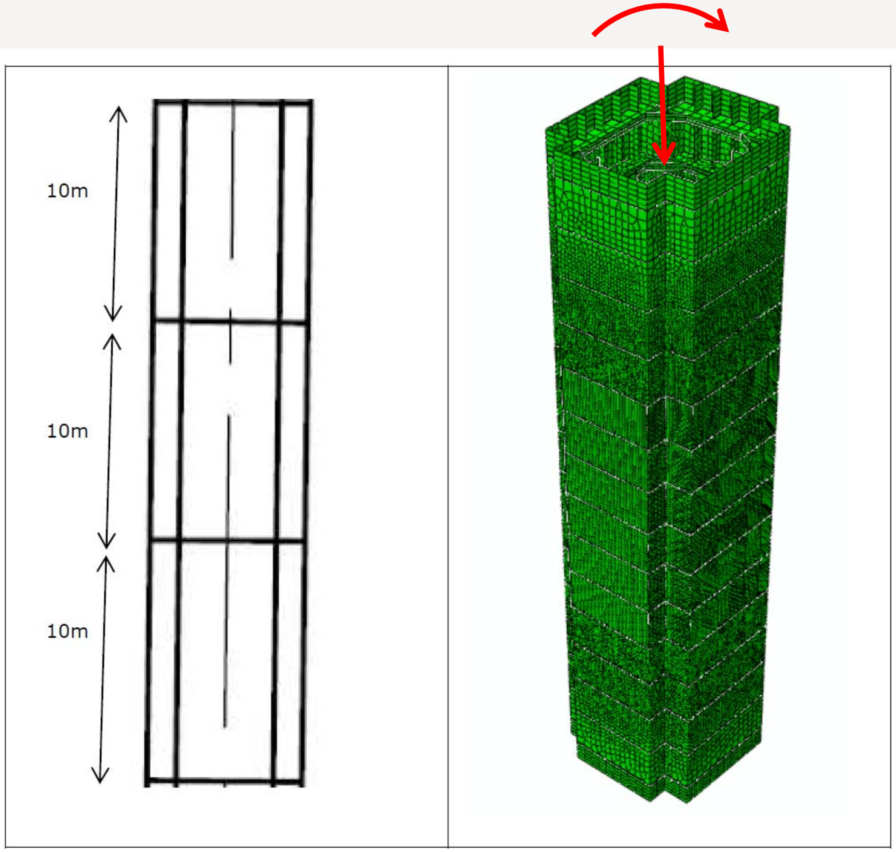
Towers, Abaqus FE-model

- Scope of model
  - Derive plastic section capacity
  - Derive plastic rotation capacity
- Basic model description
  - Three blocks
  - Load (N+M) applied at the top, bending moment increased until failure
- Equivalent imperfections
  - Accounting for structural and geometrical imperfections
- Analyses
  - Stain hardening
  - 2.order with large deformations



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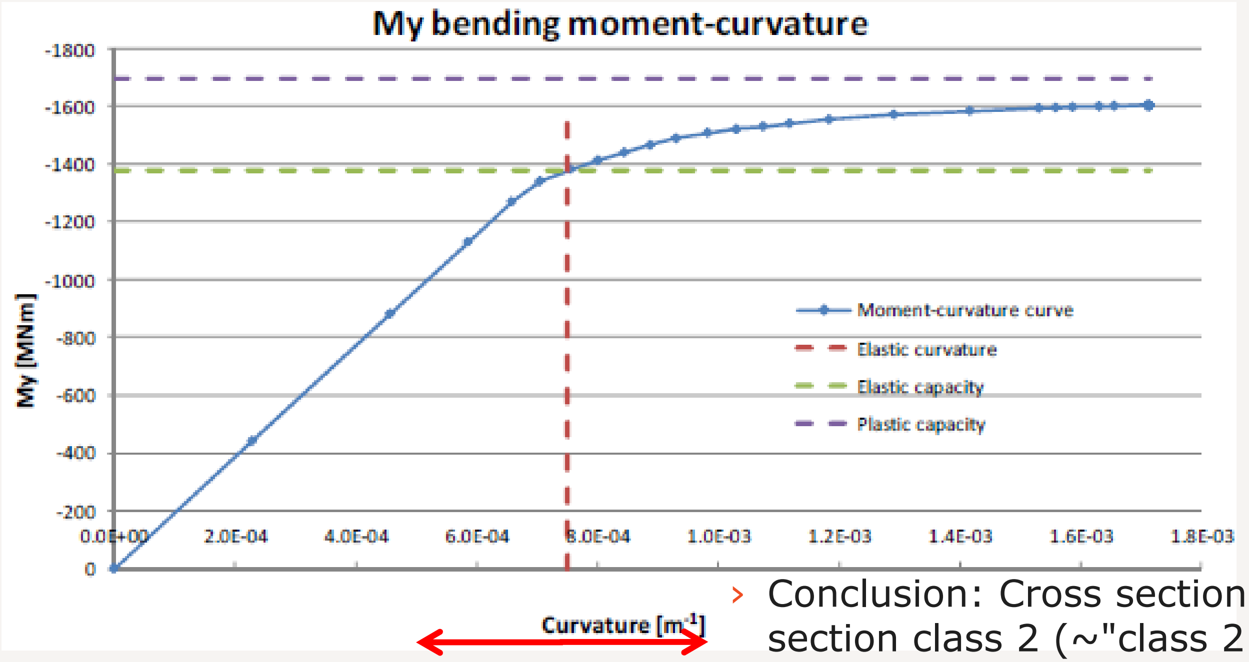
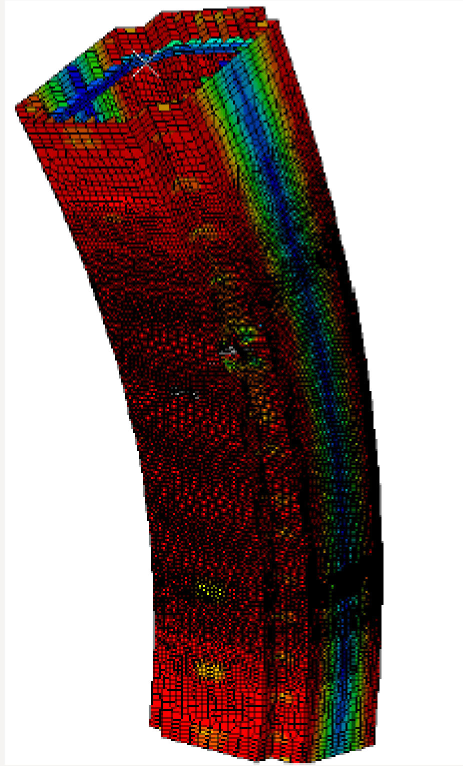
Towers, Abaqus FE-model

› Longitudinal bending moment only:



Elastic Plastic

› Conclusion: Cross section is almost section class 2 (~"class 2.2")



VMIS stresses - yielding at both sides

$$F_{My} = \frac{M_{y_R}}{f_y \cdot W_{el_y}} = \frac{1603\mathrm{MNm}}{430\mathrm{MPa} \cdot 3.13\mathrm{m}} = 1.19$$

$$\frac{W_{pl\_y}}{W_{el\_y}} = 1.24$$

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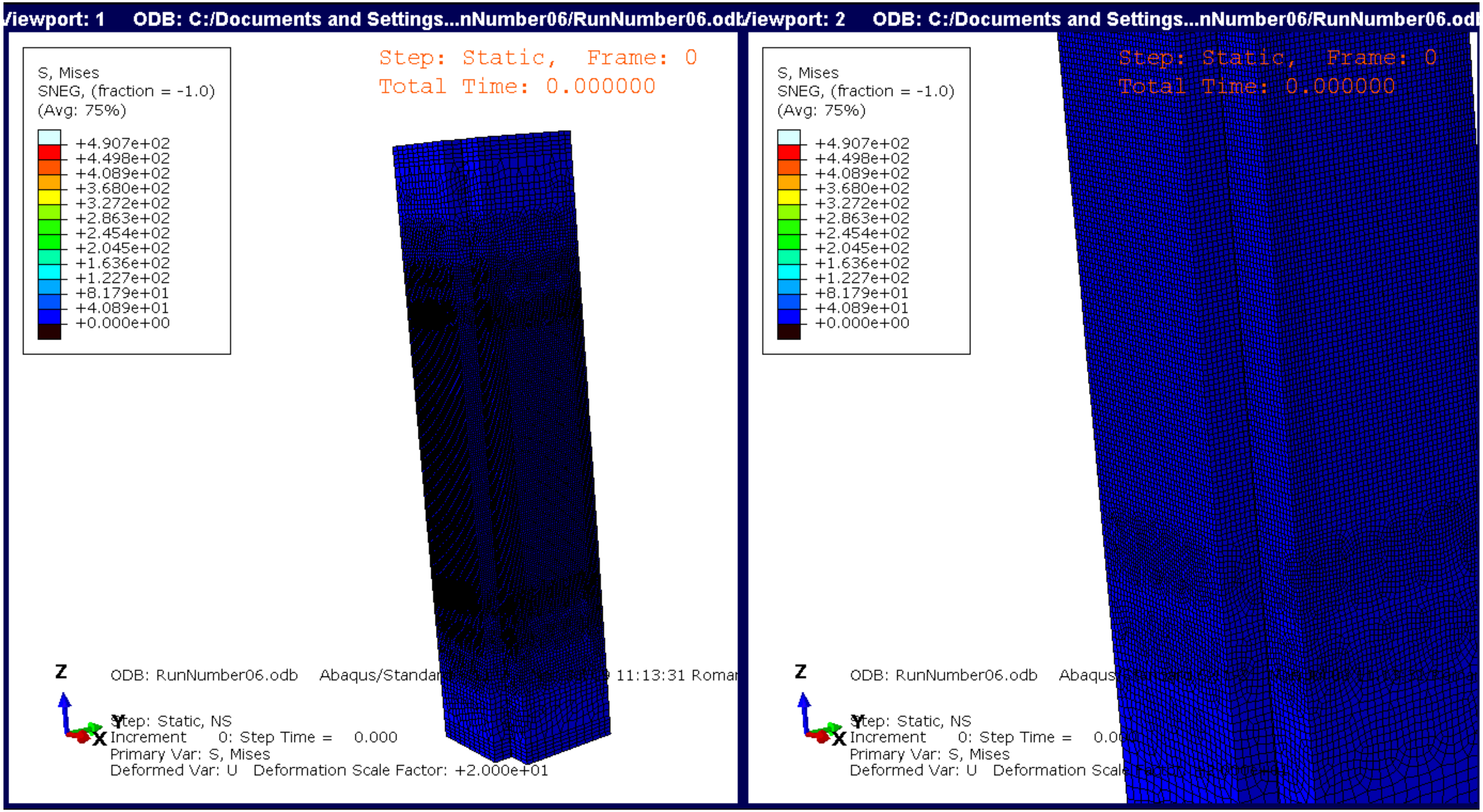
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S, Mises
SNEG, (fraction = -1.0)
(Avg: 75%)
+4.907e+02
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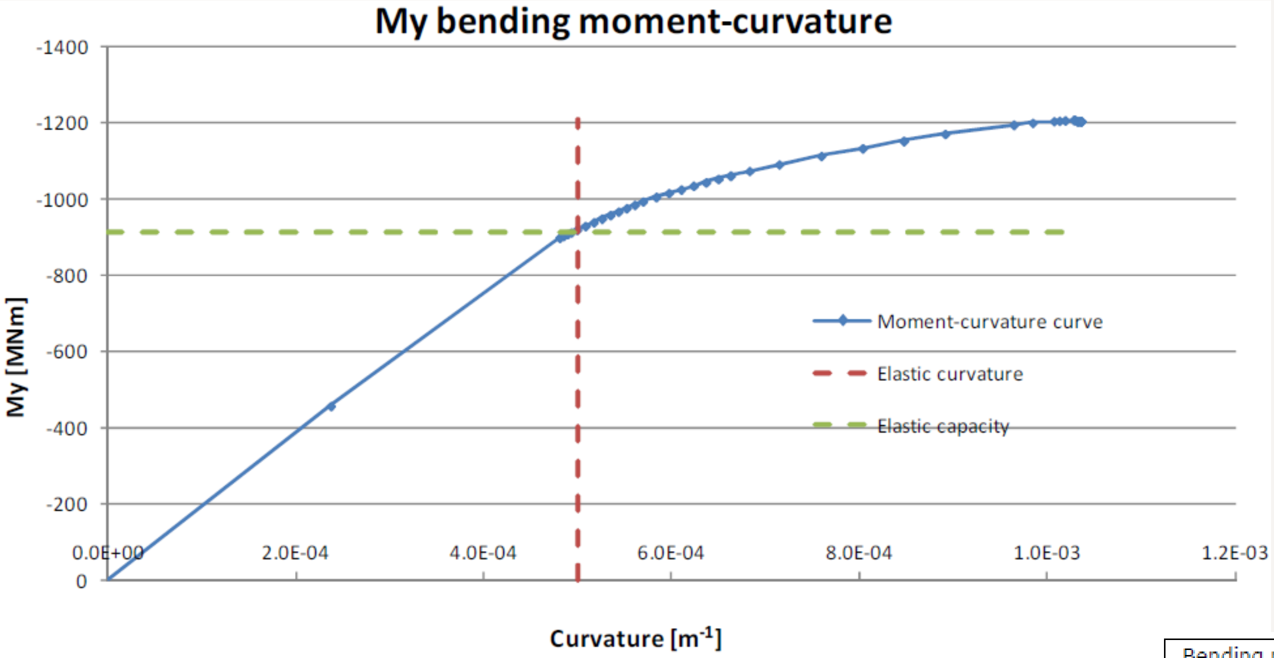
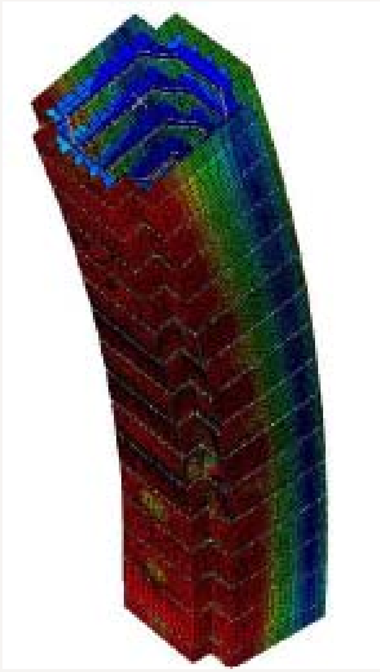
+4.089e+02
+3.680e+02
+3.272e+02
+2.863e+02
+2.454e+02
+2.045e+02
+1.636e+02
+1.227e+02
+8.179e+01
+4.089e+01
+0.000e+00
Step: Static, Frame: 0
Total Time: 0.000000
Z
Y
X
ODB: RunNumber06.odb Abaqus/Standar
11:13:31 Roma
Step: Static, NS
Increment 0: Step Time = 0.000
Primary Var: S, Mises
Deformed Var: U Deformation Scale Factor: +2.000e+01
Viewport: 2 ODB: C:/Documents and Settings...nNumber06/RunNumber06.odb
S, Mises
SNEG, (fraction = -1.0)
(Avg: 75%)
+4.907e+02
+4.498e+02
+4.089e+02
+3.680e+02
+3.272e+02
+2.863e+02
+2.454e+02
+2.045e+02
+1.636e+02
+1.227e+02
+8.179e+01
+4.089e+01
+0.000e+00
Step: Static, Frame: 0
Total Time: 0.000000
Z
Y
X
ODB: RunNumber06.odb Abaqus
Step: Static, NS
Increment 0: Step Time = 0.0
Primary Var: S, Mises
Deformed Var: U Deformation Scale

Towers, Abaqus FE-model

› Longitudinal bending and axial force





VMIS stresses - yielding at one side only

The bending moment capacity is considerably above the elastic moment capacity of approximately 900MNm (green line):

$$\sigma = \frac{N}{A} + \frac{M_{y_{el}}}{W_{el_y}} = \frac{237}{1.6} + \frac{900}{3.13} = 148 + 288 = 436MPa$$

Bending moment resistance [MNm]	Plastic curvature capacity [m^{-1}]
1206	5.3 x10^{-4} (=1.07 x $\kappa_{elastic}$)

COWI

Towers, seismic limit states

$$UR = \frac{M_{IBDAS}}{M_{Abaqus}} \cdot \alpha_u = \frac{M_{IBDAS}}{M_{Abaqus}} \cdot 1.05$$

(load magnification factor acc. to EN1993-1-5 annex C)

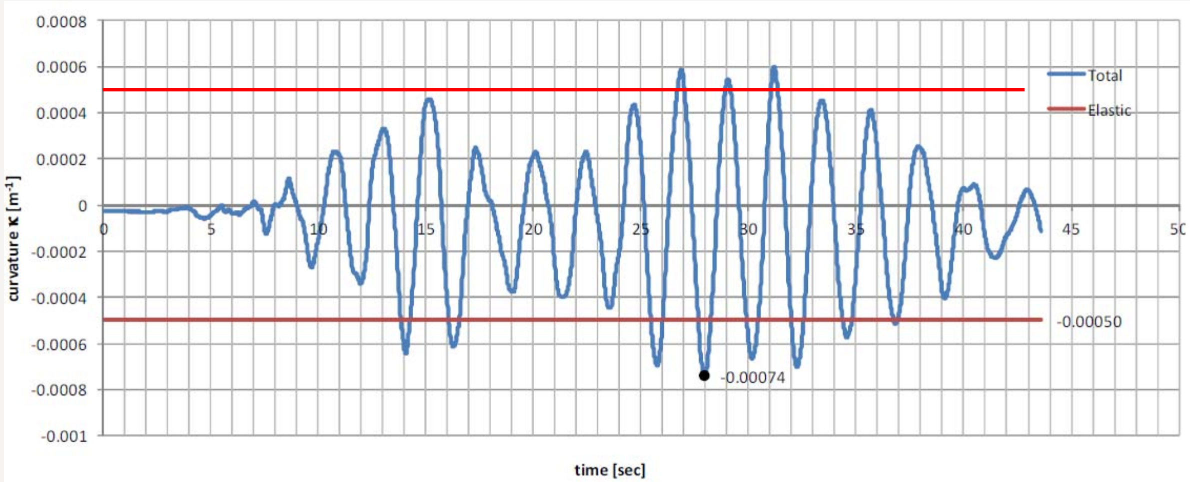
Analysis no.	Description	Imperfections	Applied axial force [MN]	ABAQUS bending moment capacity [MNm]	Bending moment capacity adjusted * [MNm]	IBDAS bending moment demand [MNm]	Section utilization [-]
6	N+M	Single bow between adjacent cross frames (2000/400=5mm) & twist (1/50) Double bow & twist	-237	1206	1248	1150	0.97

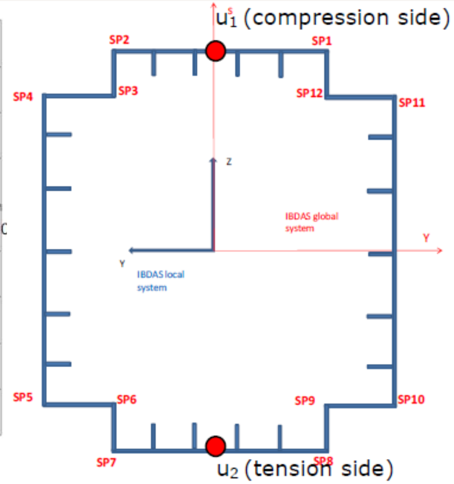
*) The bending moment capacity from Abaqus is increased by factor 1.035 as not all plates are fully modeled

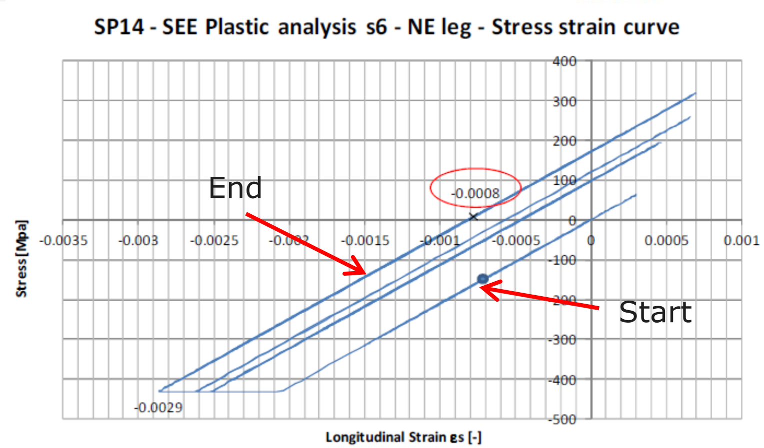
- Elastic UR =1.15 (peak stress) reduced to UR~0.97 (moment capacity) by deriving the plastic capacity in Abaqus

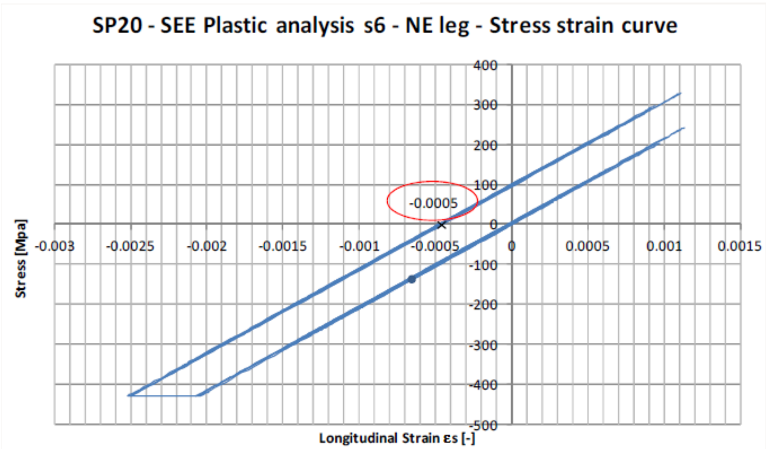
COWI

Towers, seismic limit states









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Towers, seismic limit states

› It must be verified that the plastic rotation demands are less than the plastic rotation capacities divided by 1.4:

$$\kappa_{plastic,E} \leq= \frac{\kappa_{plastic,u}}{1.4}$$

$$0.49 \cdot \kappa_{elastic,E} \leq= \frac{0.9 \cdot \kappa_{elastic}}{1.4} = 0.65 \cdot \kappa_{elastic}$$

0.9 is a conservative value, found to 1.07 previously

$$UR = 0.49/0.65 = 0.75$$

(for single most critical time history)

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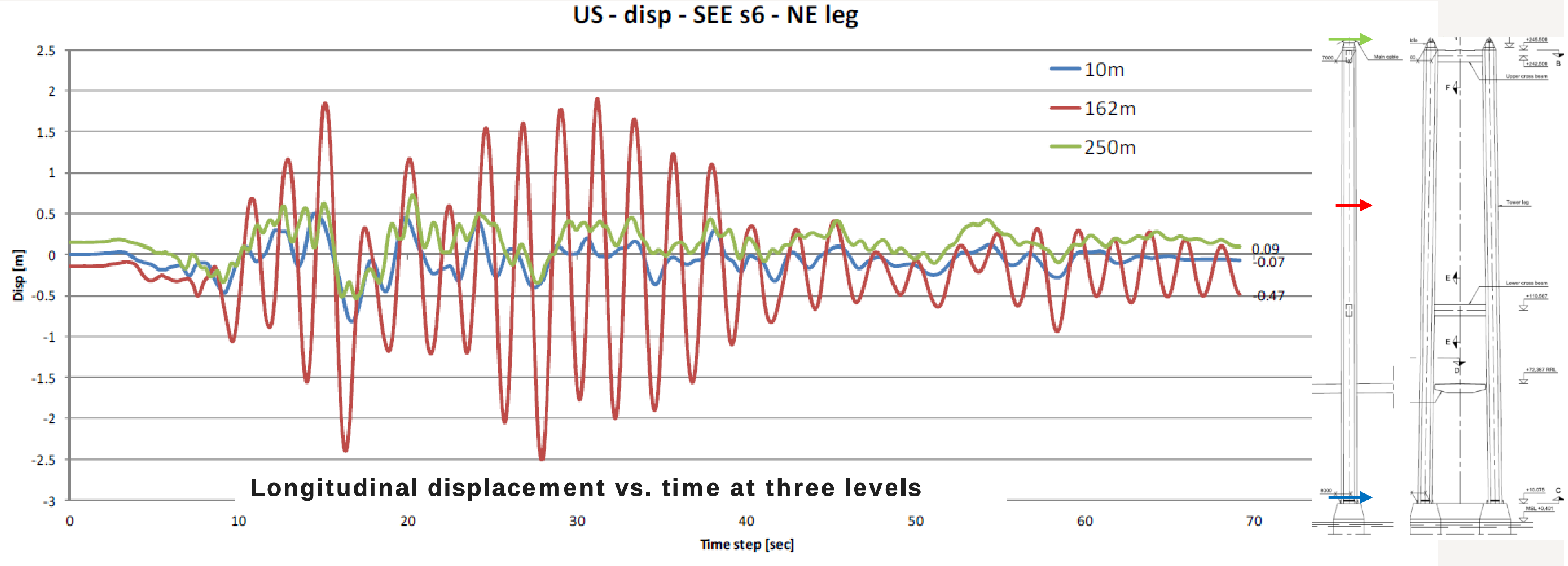
Towers, seismic limit states

- Tower to have "repairable damage" after event and to be restored to pre-seismic conditions
- Result to be taken as average of 7 time histories
- Permanent deformations becomes 50mm or 1/4850xtower height - acceptable



Longitudinal displacement vs. time at three levels

Summary of seismic checks

Item	Criteria	Demand calculation	Capacity calculation	Result
1) Verification of plastic capacity	Safety factor of 1.05	Global FE-model (IBDAS)	Abaqus	UR = 0.97
2) Verification of rotation capacity	Safety factor 1.4	Global FE-model (IBDAS)	Abaqus	UR = 0.75
3) Verification of global integrity	No buckling collapse failure for "average time history"	Global FE-model (IBDAS)	IBDAS	All time histories pass
4) Verification of permanent deformations	"Repairable damage". Permanent deformations less than initial tower imperfections	IBDAS	-	Only 50mm permanent deformation UR = 0.20

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Izmit Bay Bridge, Steel Towers
Thank you for your attention