DEGREE OF FREEDOM

Brenneriveien 5, 0182 Oslo

Prosjekt	Kammen gang- og sykkelbru - Åndalsnes
Fase	Teknisk godkjenning
Tittel	Beregningsrapport
Oppdragsgiver	Rauma kommune
Dato	28-02-2023
Prosjekt- / Rapport ID	21130-R02

Sammendrag

Dette dokumentet er beregningsrapport for konstruksjonssikkerhet for Kammen gang- og sykkelbru i Åndalsnes (Rauma).

Rapporten leses i sammenheng med 21130-R01 Prosjekteringsforutsetninger og de tilhørende vedlegg.

04	31-01-24	Kommentarer fra BN innarbeidet	92	FA	FI	BO
03	12-04-23	Avvik fra UK innarbeidet	89	JB	SA	BO
02	28-02-23	Kommentarer fra BN innarbeidet	89	JB	SA	BO
01	05-09-22	Rapport oversatt til norsk	67	FA	BO	BO
00	07-04-22	First submission	58	JB	SA	BO
REVISJON	DATO	BESKRIVELSE	SIDER	UTF.	KONTR.	GODKJ.

Revisjon	Dato	Utf.	Kontr.	Godkj.
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INNHOLDSFORTEGNELSE

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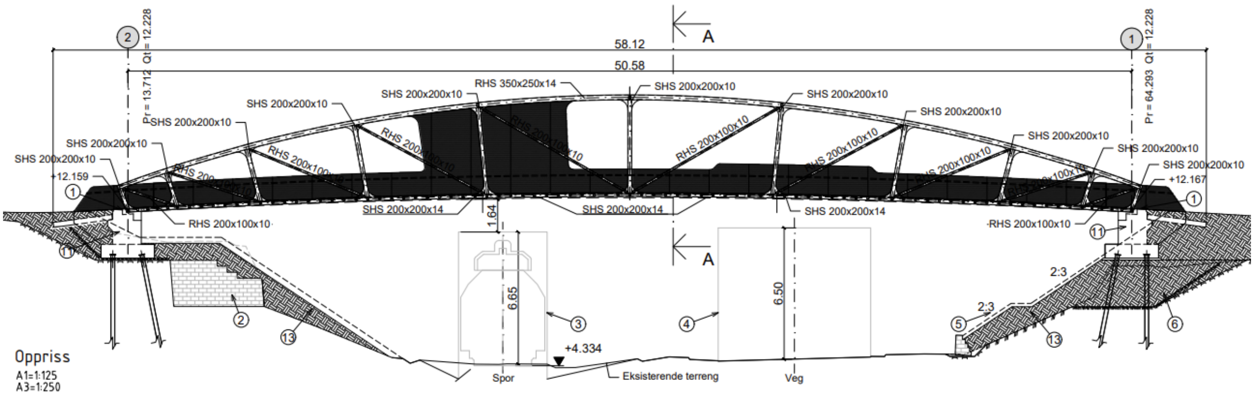
1 INTRODUKSJON

1.1 BESKRIVELSE AV BRUA

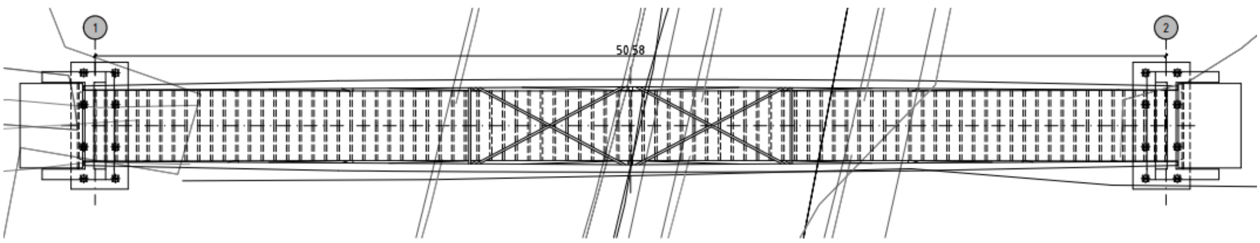
Nye Kammen bru skal erstatte den gamle brua som fikk store skader etter en ulykke. For å minimere grensesnittet mot vei og jernbane som passerer under, skal brua utføres med ett spenn.

Konstruksjonen består av et stålfagverk med en vertikal kurve som gir tilstrekkelig klaring, og samtidig skaper et konsept med visuell kontinuitet. Dekket utføres som en kontinuerlig 8mm plate som spenner i bruas lengderetning mellom horisontale bjelker c/c 0,6m. Staver og gurter RHS-stålprofiler (HUP) som sveises sammen med buttsveis (full gjennomsveising).

Den foreslåtte brukonstruksjonen er vist nedenfor med oppriss, plan og typisk tverrsnitt.



Figur 1-1: Oppriss



Figur 1-2: Plan

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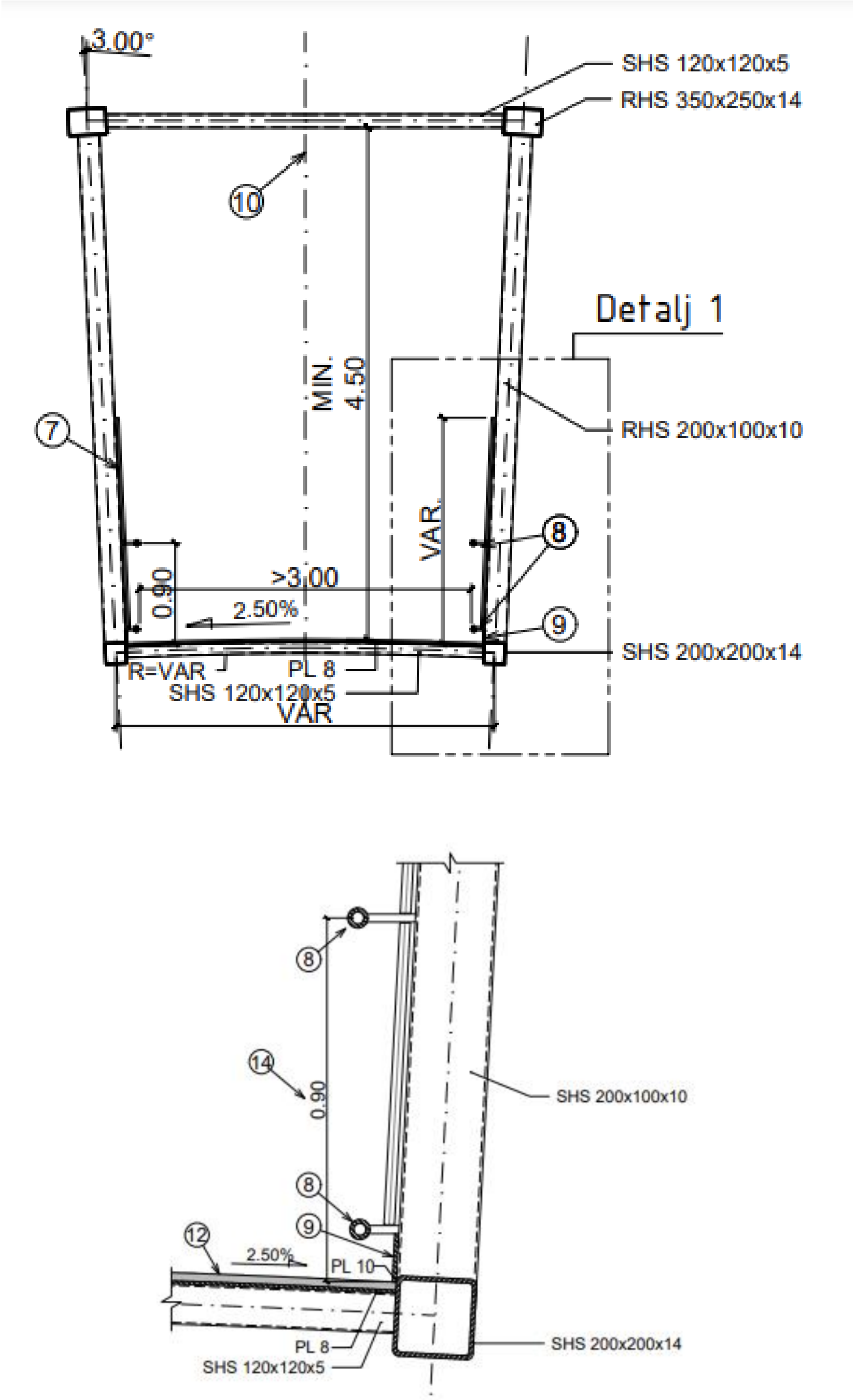

Figur 1-3: Typisk snitt

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1.2 FORMÅL OG OMFANG AV RAPPORTEN

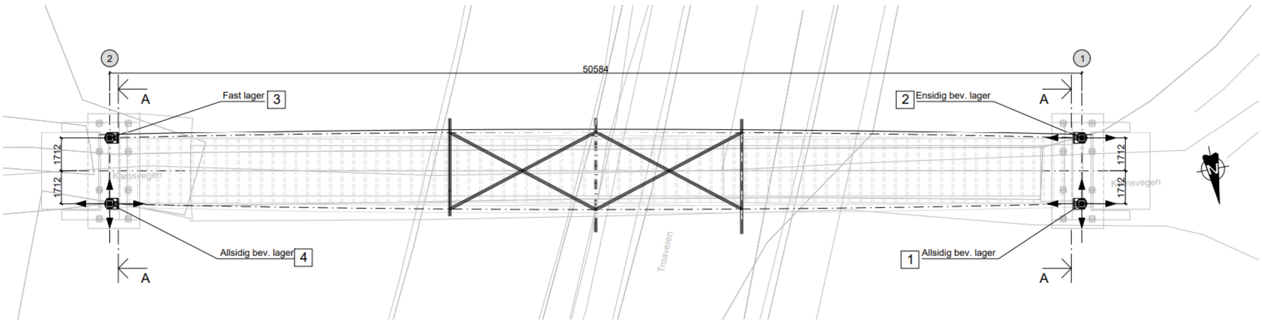
Formålet med denne rapporten er å beskrive analyseprosedyren som er brukt for å prosjektere den nye Kammen gangbro, samt presentere kapasitetskontrollene som er utført basert på denne analysen.

Prosjekteringsforutsetninger og referanse dokumenter:

1. 21130 – Kammen bru - R01 Prosjekteringsforutsetninger (07-04-2022).
2. Designers' Guide to EN 1991-1-4 Eurocode 1_ Actions on Structures, General Actions_ Wind Actions. N Cook.
3. 22WP004-GEW001-10-R-001-01-00A_Kammen Bru - Geotechnical Works, Geotechnical assessment.
4. EUR 23984 EN, Design of lightweight footbridges for human induces vibrations.

2 STATISK MODELL

Brua har kun 1 spenn og er fastholdt i lengderetning i akse 2. Hele overbygg er en stålkonstruksjon som er koblet til betonglandkar ved hjelp av to lagre og fuge i hver akse. Fastholdning i tverretning skjer kun på to lagre, en per landkar. Dette er for å minimere tvang og krefter mot kvikkleir. Landkarene er store for å kompensere horisontale krefter og unngå strekk i pelene.



Figur 2-1: Lagerplan

Gulv blir modellert som en plate slik at overflatelaster blir påført selve gulvplater. Gulvplate anses til å ikke være innspent i kantente og eksentrisitet i forhold til bjelkene neglisjeres. Punktlaster blir påført bjelker som er mest konservativt uansett. Vindlaster for trau blir påført undergurter og resten blir påført staver, for detaljer, se prosjektforutsetninger.

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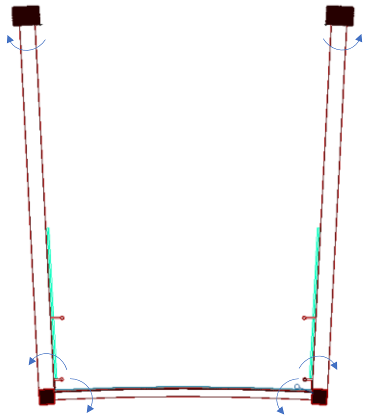
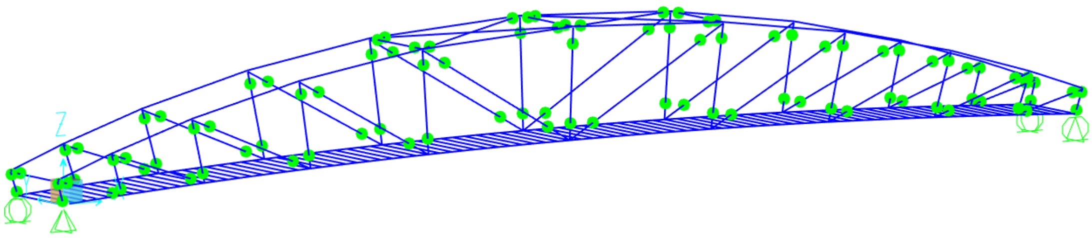


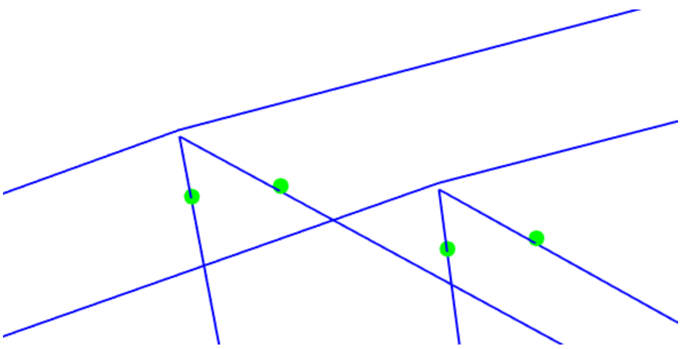
Figur 2-2: Ingen ledd i retninger bjelker får krefter i

I tverretning blir alle bjelkene innspent. I praksis vil de være krefter på grunn av vind som må tas gjennom moment. Derfor blir det feil å modellere stavene som leddet. Dimensjonering vil ta hensyn til det. Det er uansett ikke tillatt av Eurokode å ikke ta hensyn til moment ved tverrlaster.

I lengderetning eller i fagverkets plan, vil eksterne effekter tas opp gjennom aksialkrefter og moment i overgurt/undergurt. Derfor anses stavene i fagverket til å være leddet. Dette gjelder også vindfagverk. Dette er iht. NS-EN 1993-1-8 Tabell 5.3.



Figur 2-3: Bjelkene er leddet i fagverkets plan



Figur 2-4: Eksentrisiteter ved overgurter

Eksentrisiteter ved undergurter er iht. NS-EN 1993-1-8 5.1.5 (5) og derfor ikke tatt hensyn til i beregningene. Eksentrisiteter ved overgurter er blitt tatt hensyn til.

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3 ANALYSE – GLOBALMODELL

Analyse av brua under de forskjellige lastvirkningene en utført ved modellering av ulike globale og lokale FE-modeller, støttet av håndberegninger (se Vedlegg 1) der det er nødvendig. Den globale modellen gir grunnlaget for videre kontroller og beregninger. Derfor presenteres den først.

For fullstendig beskrivelser av Finite Element Modellene (FEM) se vedlegg 2.

3.1 AKSESYSTEMER

Det benyttes forskjellige aksesystemer som oppsummeres her:

Retning (vei)	Analysemodell	Reaksjoner	Nodekrefter
Lengderetning	X	H long.	1
Tverretning	Y	H trans.	2
Vertikalt	Z	V	3

4 LASTER OG LASTKOMBINASJONER

Påførte laster er oppsummert i vedlegg 3.

5 GLOBAL OPPFØRSEL

Beregningen som ble utført viste at broen har nok kapasitet mot knekking og andre globale bruddmekanismer. Den detaljerte beskrivelsen av broens globale oppførsel er beskrevet i Vedlegg 4.

6 RESULTATER FRA FEM

Opptredende krefter er beskrevet i vedlegg 5.

7 KAPASITETSKONTROLL STÅL

Beregningen som ble utført viser at brua har nok kapasitet i brudd- og bruksgrensetilstand. De detaljerte resultatene av kapasitetskontrollen for stål er beskrevet i Vedlegg 6.

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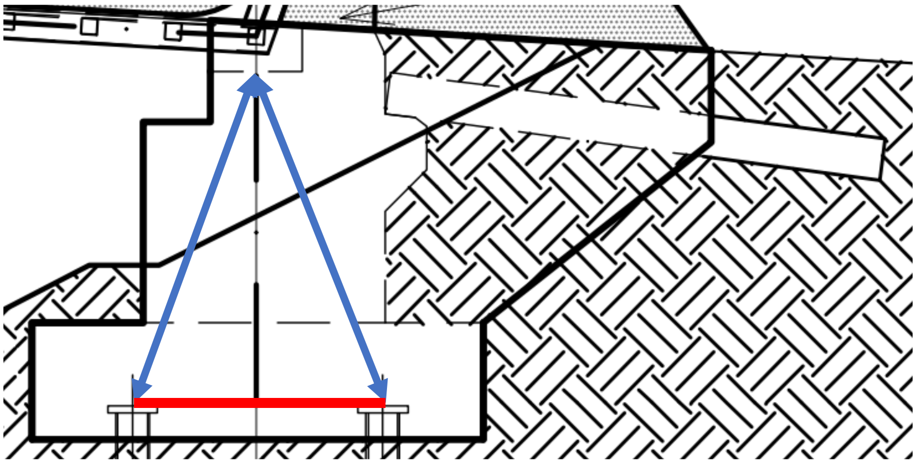
8 LANDKAR

Konseptet for landkarene er å ha nok betongmasse for å overføre kreftene til de 8 pelene, samt kompensere for oppadgående vertikale krefter.

Peler er del av geoteknisk rapport, og redegjøres for der.



Figur 8-1: Prinsipp for fordeling av krefter

Prosjekteringen av landkar er beskrevet i Vedlegg 7.

8.1 JEKKER

Jekkekrefter er beregnet fra analysemodellen. Jekking forutsettes med maksimal vindhastighet og ingen trafikk på gangbrua. Eksentrisiteten er den samme som for lagrene, derfor beregnes kreftene direkte fra modellen. Lagrene kan fastholdes midlertidig horisontalt under jekking ettersom alle lagerpunktene har samme armering.

Minste diameter på jekken vil være D=90 mm.

9 BRUUTSTYR

9.1 LAGER OG FUGER

For å bestemme dimensjonerende belastninger på lagre/skjøter og deres bevegelser benyttes relevante lastkombinasjoner. Temperaturvirkninger for maksimal ekspansjon og sammentrekning beregnet i henhold til NS-EN 1991-1-5 som følger:

$\Delta T_{N,exp}$ + 20° = +62° / $\Delta T_{N,con}$ - 20° = -58°

To pottelagre i hvert landkar.

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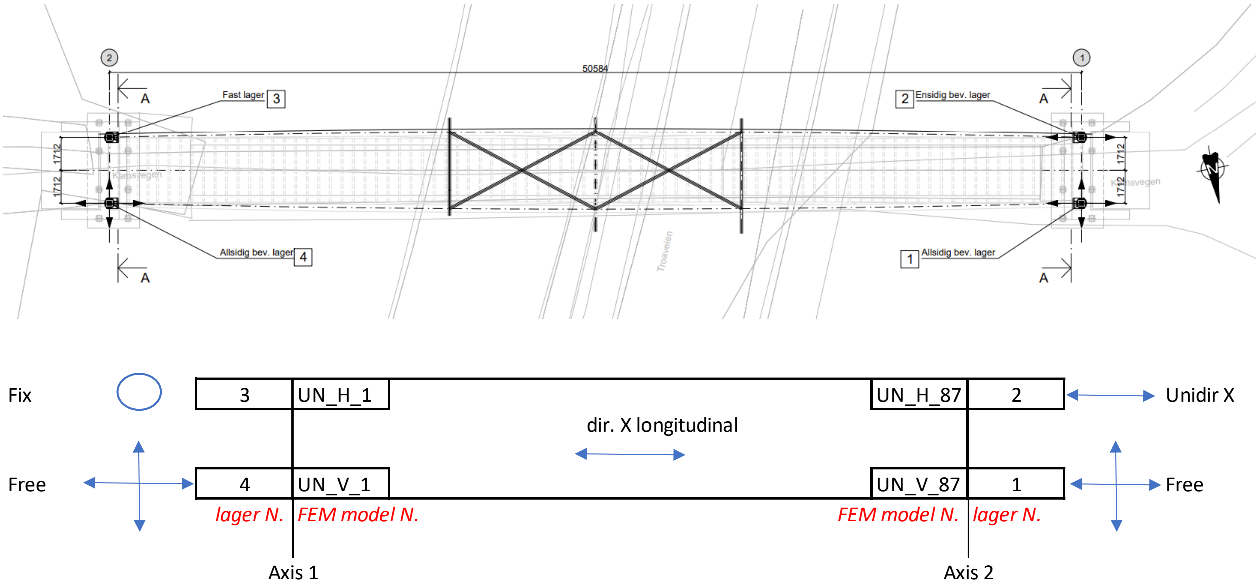

Figur 9-1: Lagerplan

Maksimum/minimum horisontale/vertikale laster, forskyvninger og rotasjoner er hentet fra den globale FE-modellen for de aktuelle lastkombinasjonene.

9.1.1 Lagerkrefter

ULS LOADS ENVELOPE

Lager	type	Fz	Fx (long.)	Fy (transv)	Zone
-	-	kN	kN	kN	-
Lager 4	Free	607	±12.6*	±12.6*	North-west
Lager 3	Fix	612	195 / -107	296 / -276	South-west
Lager 2	Unidir X	603	±12.6*	307 / -298	South-east
Lager 1	Free	604	±12.6*	±12.6*	North-east

SLS LOADS ENVELOPE

Lager	type	Fz	Fx (long.)	Fy (transv)	Zone
-	-	kN	kN	kN	-
Lager 4	Free	451	±8.4*	±8.4*	North-west
Lager 3	Fix	453	136/ -79	187 / -173	South-west
Lager 2	Unidir X	448	±8.4*	194 / -187	South-east
Lager 1	Free	448	±8.4*	±8.4*	North-east

* Values due to friction at bearing, friction need to be added to the overall load acting on the foundation.

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9.1.2 Bevegelser

From standard FEM model:

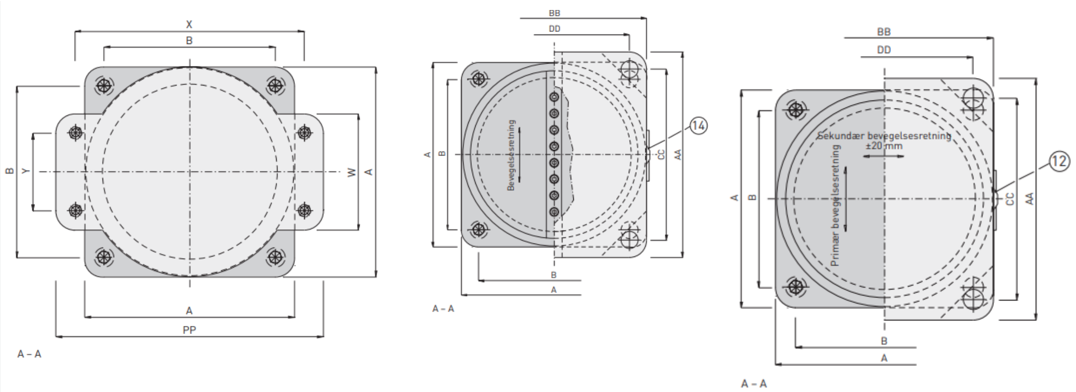
Joint	OutputCase	Lager	type	U1	U2	U3	R
Text	Text	Text	Text	mm	mm	mm	%
UN_H_1	SLS	Lager 3	fix	0.0	0.0	0.0	1.05
UN_H_1	SLS	Lager 3	fix	0.0	0.0	0.0	0.95
UN_V_1	SLS	Lager 4	free	13.0	2.5	0.0	0.89
UN_V_1	SLS	Lager 4	free	-13.9	-2.2	0.0	0.46
UN_H_87	SLS	Lager 2	Max	44.6	0.0	0.0	0.81
UN_H_87	SLS	Lager 2	Min	-28.4	0.0	0.0	1.18
UN_V_87	SLS	Lager 1	Max	36.5	2.6	0.0	0.58
UN_V_87	SLS	Lager 1	Min	-19.1	-2.2	0.0	0.81

From bearing FEM model (DT+20 degrees):

Joint	OutputCase	Lager	type	U1	U2	U3	R
Text	Text	Text	Text	mm	mm	mm	%
UN_H_1	SLS	Lager 3	fix	0.0	0.0	0.0	0.94
UN_H_1	SLS	Lager 3	fix	0.0	0.0	0.0	0.77
UN_V_1	SLS	Lager 4	free	15.0	2.8	0.0	1.13
UN_V_1	SLS	Lager 4	free	-15.0	-2.7	0.0	0.70
UN_H_87	SLS	Lager 2	Max	57.8	0.0	0.0	0.59
UN_H_87	SLS	Lager 2	Min	-41.0	0.0	0.0	1.06
UN_V_87	SLS	Lager 1	Max	48.6	2.8	0.0	0.84
UN_V_87	SLS	Lager 1	Min	-31.2	-2.7	0.0	1.00

9.2 LAGRE

Vi dimensjonerer med TOBE type 20. Avstand B mellom bolter 232mm.



Belastningstabell fast lager i bruddgrensetilst

Type		10	20	30	40	50
V_{max}	kN	520	1590	3040	4690	6090
V_{min}	kN	60	380	1210	2070	2790
H_{max}	kN	220	420	720	950	1130

Belastningstabell ensidig bevegeli

Type		10	20	30
V_{max}	kN	520	1590	3040
V_{min}	kN	80	380	1170
H_{max}	kN	160	420	720

Belastningstabell allsidig bevegelig

Type		10	20	30
Vmax	kN	520	1590	3040

Figur 9-2: Fast, ensidig og allsidig lager

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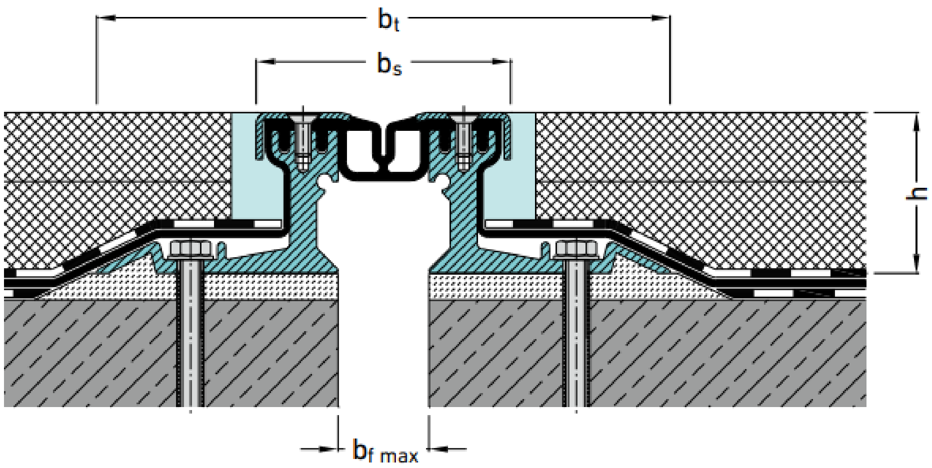
9.3 FUGER

For fuger forutsettes Migutan FP(G) .../60 S NI l eller tilsvarende. Vi benytter:

FP 110/60 S SI ls ved akse 1

FP 155/60 S SI ls ved akse 2

Expansion joint cover	Joint width max. $b_{f\,max}$ [mm]	Total movement Δb_f [mm]	Visible width b_s [mm]	Joint width total b_t [mm]	Joint height h [mm]	Load bearing capacity (car) [kN]	Load bearing capacity (truck) [kN]	Load bearing capacity (forklift) [kN]	Load bearing capacity solid plastic tyres [kg/mm]
FP 80/60 S NI ls	35	20 (±10)	82	201	60	35	600	130	6,5
FPG 80/60 S NI ls	35	16 (±8)	82	201	60	35	600	130	6,5
FP 90/60 S NI ls	50	40 (±20)	95	214	60	35	600	130	
FPG 90/60 S NI ls	50	20 (±10)	95	214	60	35	600	130	4,3
FP 110/60 S NI ls	65	60 (±30)	111	230	60	35	600	130	
FPG 110/60 S NI ls	65	40 (±20)	111	230	60	35	600	130	
FP 130/60 S NI ls*	90	90 (±45)	133	252	60	35	600	130	
FP 155/60 S NI ls */**	110	120 (±60)	155	274	60	35	300	70	



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10 VEDLEGG 1 – HÅNDBEREGNINGER

10.1 VINDLASTER

Kun statiske effekter ettersom periode er mindre enn 2s.

Annex 1.1 Wind forces

Wind forces

Calculations according to NS-EN 1991-1-4

Input data

Peak wind pressure (from design basis) $q_p := 1{,}41\ \frac{kN}{m^2}$

$d_{tot} := 1{,}1\ m$ $\quad b := 4\ m$

$A_{xref} := d_{tot} \cdot 2$ Value on the safe side

Wind on bridge (perpendicular)

$\frac{b}{d_{tot}} = 3{,}6364$

$c := 1{,}3$ Figure 8.3, value on the safe side

$F_{1c} := \frac{q_p \cdot c \cdot A_{xref}}{2} = 2{,}0163\ \frac{kN}{m}$ Two forces one on each chord

Wind on bridge (vertical)

$c_{fz} := 0{,}9$

$\frac{c_{fz} \cdot b \cdot q_p}{2} \cdot \frac{3}{4} = 1{,}9035\ \frac{kN}{m}$ Force closer to wind direction

$\frac{c_{fz} \cdot b \cdot q_p}{2} \cdot \frac{1}{4} = 0{,}6345\ \frac{kN}{m}$ Force further from wind direction

Wind on bridge (longitudinal)

50% of forces in the longitudinal direction 8.3.4

$\frac{q_p \cdot c \cdot A_{xref}}{4} = 1{,}0082\ \frac{kN}{m}$ Two forces one on each chord

Wind on members

Upper chord

$d := 350\ mm$ $\quad b := 250\ mm$ $\quad r := 18{,}8\ mm$

$\frac{r}{b} = 0{,}0752$ $\quad \psi_r := \text{linterp}\left(\begin{bmatrix} 0 \\ 0{,}2 \\ 0{,}4 \end{bmatrix}; \begin{bmatrix} 1 \\ 0{,}5 \\ 0{,}5 \end{bmatrix}; \frac{r}{b}\right) = 0{,}812$ Figure 7.24

$\frac{d}{b} = 1{,}4$ $\quad c_{f,0} := \text{linterp}\left(\begin{bmatrix} 0{,}2 \\ 0{,}7 \\ 5 \end{bmatrix}; \begin{bmatrix} 2 \\ 2{,}4 \\ 1 \end{bmatrix}; \frac{d}{b}\right) = 2{,}1721$ Figure 7.23

$F_{uc} := c_{f,0} \cdot \psi_r \cdot b \cdot q_p = 0{,}6217\ \frac{kN}{m}$

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Annex 1.1 Wind forces

Diagonal bracing

$d := 200\ \text{mm}$ $\quad b := 200\ \text{mm}$ $\quad r := 9{,}4\ \text{mm}$

$\frac{r}{b} = 0{,}047$ $\quad \psi_r := \text{linterp}\left(\begin{bmatrix} 0 \\ 0{,}2 \\ 0{,}4 \end{bmatrix}; \begin{bmatrix} 1 \\ 0{,}5 \\ 0{,}5 \end{bmatrix}; \frac{r}{b}\right) = 0{,}8825$ Figure 7.24

$\frac{d}{b} = 1$ $\quad c_{f,0} := \text{linterp}\left(\begin{bmatrix} 0{,}2 \\ 0{,}7 \\ 5 \end{bmatrix}; \begin{bmatrix} 2 \\ 2{,}4 \\ 1 \end{bmatrix}; \frac{d}{b}\right) = 2{,}3023$ Figure 7.23

$F_{db} := c_{f,0} \cdot \psi_r \cdot b \cdot q_p = 0{,}573\ \frac{\text{kN}}{\text{m}}$

$d := 200\ \text{mm}$ $\quad b := 100\ \text{mm}$ $\quad r := 9{,}4\ \text{mm}$

$\frac{r}{b} = 0{,}094$ $\quad \psi_r := \text{linterp}\left(\begin{bmatrix} 0 \\ 0{,}2 \\ 0{,}4 \end{bmatrix}; \begin{bmatrix} 1 \\ 0{,}5 \\ 0{,}5 \end{bmatrix}; \frac{r}{b}\right) = 0{,}765$ Figure 7.24

Figure 7.23

$\frac{d}{b} = 2$ $\quad c_{f,0} := \text{linterp}\left(\begin{bmatrix} 0{,}2 \\ 0{,}7 \\ 5 \end{bmatrix}; \begin{bmatrix} 2 \\ 2{,}4 \\ 1 \end{bmatrix}; \frac{d}{b}\right) = 1{,}9767$

$F_{db} := c_{f,0} \cdot \psi_r \cdot b \cdot q_p = 0{,}2132\ \frac{\text{kN}}{\text{m}}$

Additional forces on chords

Force due to higher plexiglass using formulae for vertical walls

$c_{pD} := 1$ $\quad c_{pE} := 0{,}5$

$A_{ref} := 7\ \text{m} \cdot 3{,}4\ \text{m}$

$L := 7\ \text{m}$

$$F_{ucfr} := \frac{c_{pD} \cdot A_{ref} \cdot q_p}{L} \cdot \frac{1}{2} + F_{uc} = 3{,}0187\ \frac{\text{kN}}{\text{m}}$$

$$F_{lcfr} := \frac{c_{pD} \cdot A_{ref} \cdot q_p}{L} \cdot \frac{1}{2} + F_{lc} = 4{,}4133\ \frac{\text{kN}}{\text{m}}$$

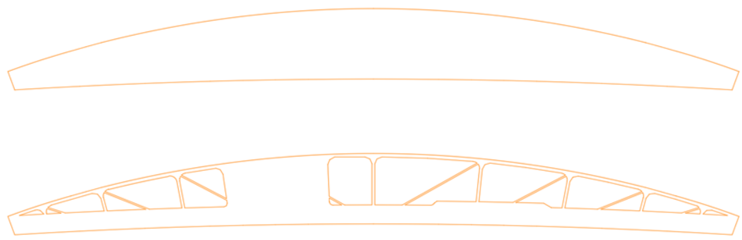
$$F_{ucba} := \frac{c_{pE} \cdot A_{ref} \cdot q_p}{L} \cdot \frac{1}{2} + F_{uc} = 1{,}8202\ \frac{\text{kN}}{\text{m}}$$

$$F_{lcba} := \frac{c_{pE} \cdot A_{ref} \cdot q_p}{L} \cdot \frac{1}{2} + F_{lc} = 3{,}2148\ \frac{\text{kN}}{\text{m}}$$

Bridge as truss

Chapter 7.11

Areas from CAD



$A := 200,5\ \mathrm{m}^2$

$A_c := 116\ \mathrm{m}^2$

$\phi := \frac{A_c}{A} = 0,5786$

$v := \sqrt{\frac{2 \cdot q_p}{\rho}} = 47,4974\ \frac{\mathrm{m}}{\mathrm{s}}$

$\nu := 15 \cdot 10^{-16}\ \frac{\mathrm{m}}{\mathrm{s}^2}$

$l := 225\ \mathrm{mm}$

$Re := \frac{v \cdot l}{\nu} = 7,1246 \cdot 10^{15}\ \mathrm{m\ s}$

$c_f := 1,8$

Figure 7.34

$A_c \cdot c_f \cdot q_p = 294,408\ \mathrm{kN}$

Less than 338kN (total force in SAP2000), considering a truss is not conservative

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10.2 SJEKK AV PELER

Input

$N_{Ed} := 350\ kN$ $\quad M_{yEd} := 44\ kN\ m$ $\quad M_{zEd} := 26\ kN\ m$

$\gamma_{M0} := 1,1$ According to 1993 -2 NA

$f_y := 355\ MPa$

Pile resistance 250x85

$h := 254\ mm$ $\quad b := 260\ mm$ $\quad t_f := 14,4\ mm$ $\quad t_w := 14,4\ mm$

$\Delta t := 2\ mm$ Loss of thickness accordig to NS-EN 1993 -5 Table NA.4.1 100 years

$h_w := h - 2 \cdot t_f$ $\quad b := b - 2 \cdot \Delta t$ $\quad t_f := t_f - 2 \cdot \Delta t$ $\quad t_w := t_w - 2 \cdot \Delta t$

$A := h_w \cdot t_w + 2 \cdot b \cdot t_f = 77\ cm^2$

$$W_{ely} := \frac{2 \cdot b \cdot t_f \cdot \left(\frac{h_w}{2} + \frac{t_f}{2}\right)^2 + 2 \cdot \frac{t_w}{3} \cdot \left(\frac{h_w}{2}\right)^3}{\frac{h}{2}} = 660\ cm^3$$

$$W_{elz} := 8 \cdot \frac{\frac{t_f}{3} \cdot \left(\frac{b}{2}\right)^3}{b} = 230\ cm^3$$

$$N_{Rd} := \frac{A \cdot f_y}{\gamma_{M0}} = 2474,31\ kN \qquad M_{yRd} := \frac{W_{ely} \cdot f_y}{\gamma_{M0}} = 212,9228\ kN\ m \qquad M_{zRd} := \frac{W_{elz} \cdot f_y}{\gamma_{M0}} = 73,32\ kN\ m$$

$$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{yEd}}{M_{yRd}} + \frac{M_{zEd}}{M_{zRd}} = 0,7027$$

EN 1993 -1 -1 6.2.1 (7)

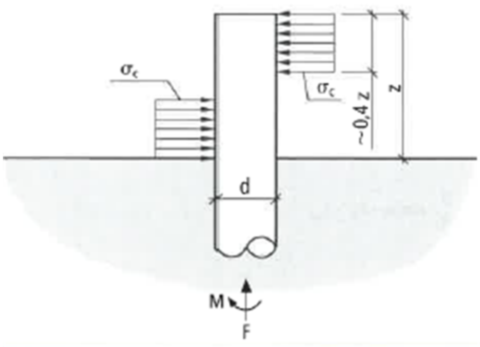
Pile cap

Peleveiledningen $11 - 6$

$d := h$ $\quad z := 0,26\ m$

$$M_{yRd} := 0,24 \cdot d \cdot z^2 \cdot 45 \cdot \frac{MPa}{1,5} = 123,6269\ kN\ m$$

$$\frac{M_{yEd}}{M_{yRd}} = 0,3559$$



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11 VEDLEGG 2 – GLOBAL FEM MODELL, BESKRIVELSE

11.1 GEOMETRY

Global geometry of FEM follows the real geometry at the end of the design process and as defined in the drawings.

Some deviations from the reality are:

- Circular beams are modelled with straight elements.
- The continuous steel plate is modelled as straight plates spanning from horizontal beam to horizontal beam. This results in a slightly higher unsupported span than in reality. (0,6m vs. 0,5m)

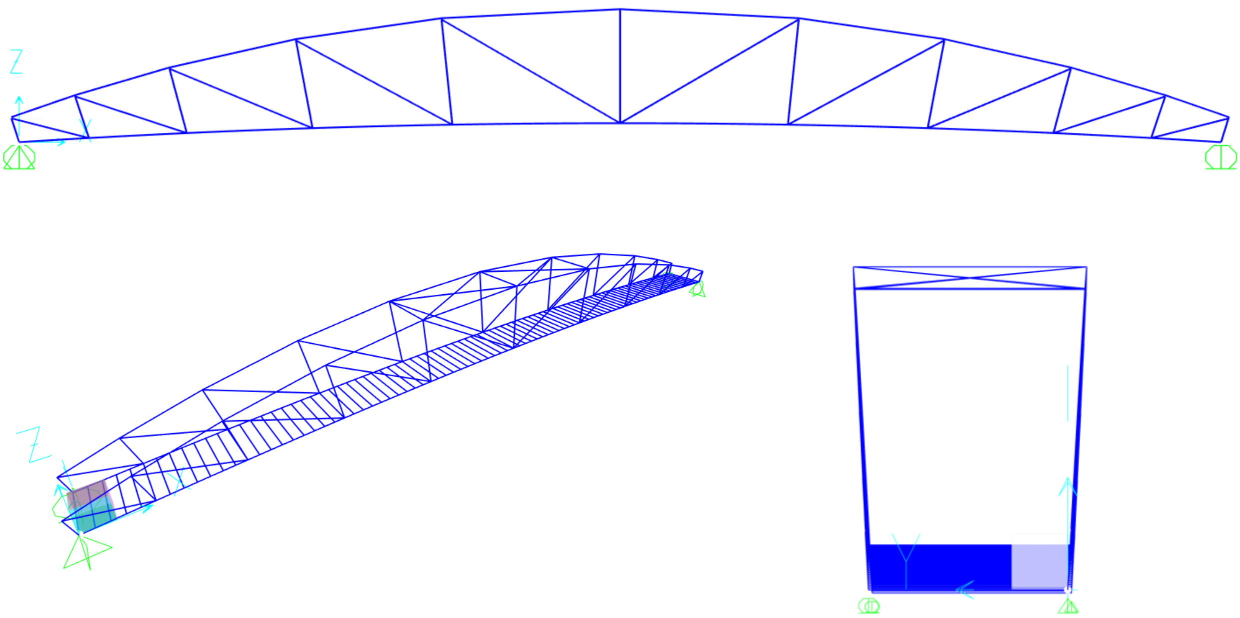


Figure 11-1 BIM geometry – Z-levels at key points

The origin is set at the southwest corner of the bridge.

The following criteria are set for naming the beams:

Region	Prefix
Southern upper chord	OV_H_
Northern upper chord	OV_V_
Southern lower chord	UN_H_
Northern lower chord	UN_V_
Southern main bracing	DI_H_
Northern main bracing	DI_V_
Horizontal members	HO_
Wind bracing	VA_

11.2 ANALYSIS METHOD

The performed analyses for beam/joint design are linear first order analyses. Additionally, some cases have been studied separately. Theses are modal analysis (for frequencies), buckling analysis and local plate analysis. The combinations with the highest load on the upper chord have been run again.

Dynamic analyses have been run too in order to verify the dynamic behavior of the structure.

No long-term effects are considered to have a significant impact on the bridge.

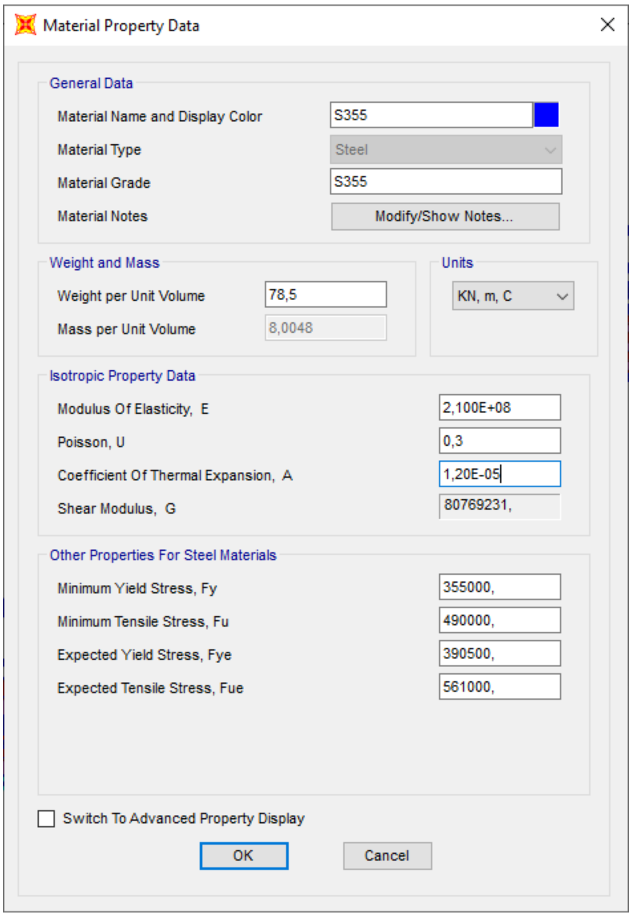
11.3 MATERIALS

Detailed data about defined materials can be found at the design basis.

11.3.1 Structural steel

Since no plates or thicknesses are above 40mm, there is only one material definition.

The steel weight reflects the real structural weight. In order to account for connection plates, the weight has been increased in the load definition, keeping the material definition according to the design basis.



11.3.2 Concrete

Concrete is not included in the analysis model but calculated separately.

11.4 SECTIONS

11.4.1 Steel cross sections

The criteria for the choice of cross sections has been the limits defined in NS-EN 1993-1-8 for RHS joints:

Table 7.8: Range of validity for welded joints between CHS or RHS brace members and RHS chord members

<table>
<tr><th rowspan="3">Type of joint</th><th colspan="6">Joint parameters [$i = 1$ or 2, j = overlapped brace]</th></tr>
<tr><th rowspan="2">b_i/b_0
or
d_i/b_0</th><th colspan="2">b_i/t_i and h_i/t_i or d_i/t_i</th><th rowspan="2">h_0/b_0
and
h_i/b_i</th><th rowspan="2">b_0/t_0
and
h_0/t_0</th><th rowspan="2">Gap or overlap
b_i/b_j</th></tr>
<tr><th>Compression</th><th>Tension</th></tr>
<tr><td>T, Y or X</td><td>$b_i/b_0 \geq 0,25$</td><td rowspan="2">$b_i/t_i \leq 35$
and
$h_i/t_i \leq 35$
and
Class 2</td><td rowspan="3">b_i/t_i
≤ 35
and
h_i/t_i
≤ 35</td><td rowspan="3">$\geq 0,5$
but
$\leq 2,0$</td><td>≤ 35
and
Class 2</td><td>–</td></tr>
<tr><td>K gap
N gap</td><td>$b_i/b_0 \geq 0,35$
and
$\geq 0,1 + 0,01\, b_0/t_0$</td><td>$\leq 35$
and
Class 2</td><td>$g/b_0 \geq 0,5(1-\beta)$
but $\leq 1,5(1-\beta)$ 1)
and as a minimum
$g \geq t_1 + t_2$</td></tr>
<tr><td>K overlap
N overlap</td><td>$b_i/b_0 \geq 0,25$</td><td>Class 1</td><td>Class 2</td><td>$\lambda_{ov} \geq 25\%$
but $\lambda_{ov} \leq 100\%$ 2)
and $b_i/b_j \geq 0,75$</td></tr>
<tr><td>Circular brace member</td><td>$d_i/b_0 \geq 0,4$
but $\leq 0,8$</td><td>Class 1</td><td>d_i/t_i
≤ 50</td><td colspan="3">As above but with d_i replacing b_i and d_j replacing b_j.</td></tr>
<tr><td colspan="7">1) If $g/b_0 > 1,5(1-\beta)$ and $g/b_0 > t_1 + t_2$ treat the joint as two separate T or Y joints.
2) The overlap may be increased to enable the toe of the overlapped brace to be welded to the chord.</td></tr>
</table>

Figure 11-2 Criteria to be satisfied with the section design

Table 7.9: Additional conditions for the use of Table 7.10

Type of brace	Type of joint	Joint parameters	
Square hollow section	T, Y or X	$b_i/b_0 \le 0{,}85$	$b_0/t_0 \ge 10$
	K gap or N gap	$0{,}6 \le \dfrac{b_1 + b_2}{2b_1} \le 1{,}3$	$b_0/t_0 \ge 15$
Circular hollow section	T, Y or X		$b_0/t_0 \ge 10$
	K gap or N gap	$0{,}6 \le \dfrac{d_1 + d_2}{2d_1} \le 1{,}3$	$b_0/t_0 \ge 15$

Figure 11-3 Criteria to be satisfied with the section design

Notice that the main truss will have overlap joints while the upper horizontal truss will have gap joints.

The following cross sections have been chosen for the bridge

Region	Prefix
Upper chord	RHS 350x250x14
Lower chord	SHS 200x200x14
Lower chord (central)	SHS 200x200x14
Main bracing (diagonal)	RHS 200x100x8
Main bracing (vertical)	SHS 200x200x8
Horizontal members	SHS 120x120x5
Horizontal members (at both ends)	SHS 120x120x6
Wind bracing	SHS 120x120x5

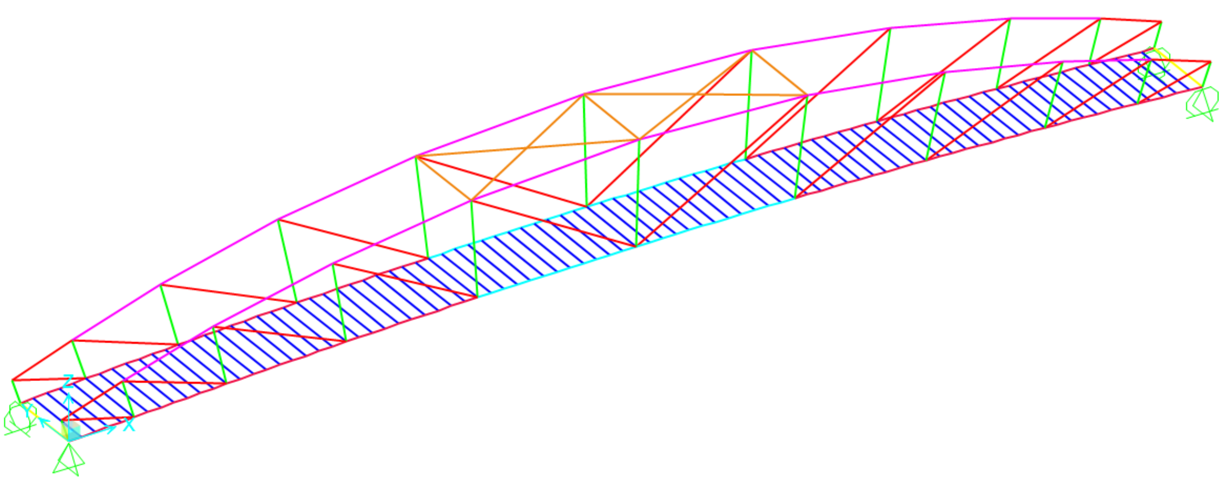


Figure 11-4 Steel section grouping

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The sections have been rotated to have the proper inertia and releases. The angle is verified with the "extruded" option of SAP2000.

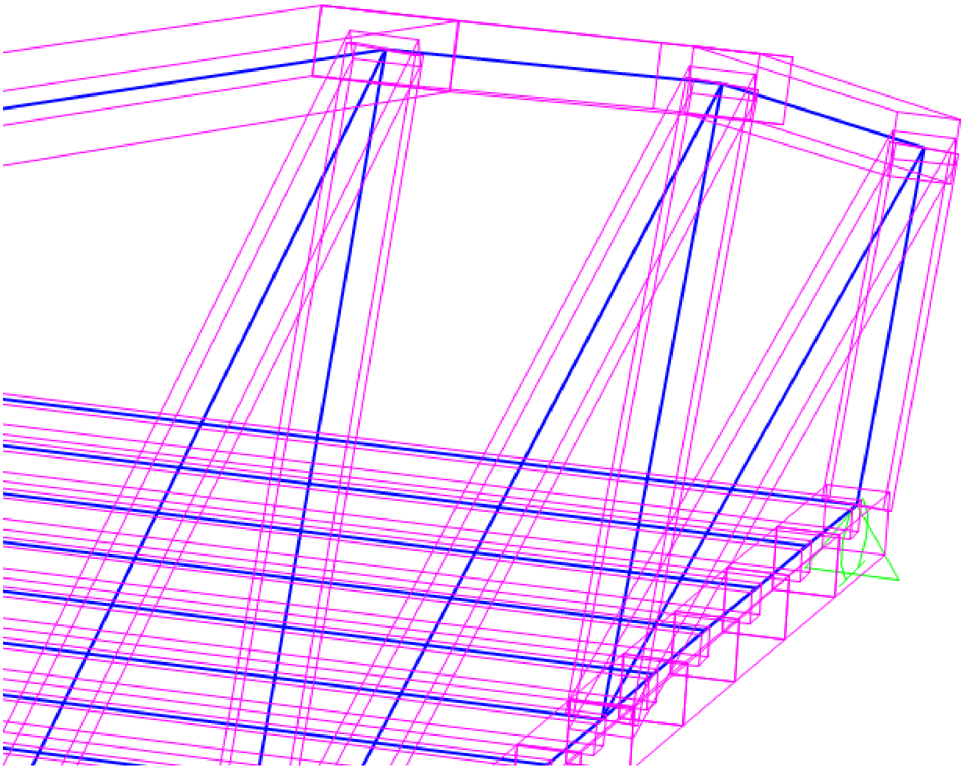

Figure 11-5 Upper chord and main bracing oriented along the wind

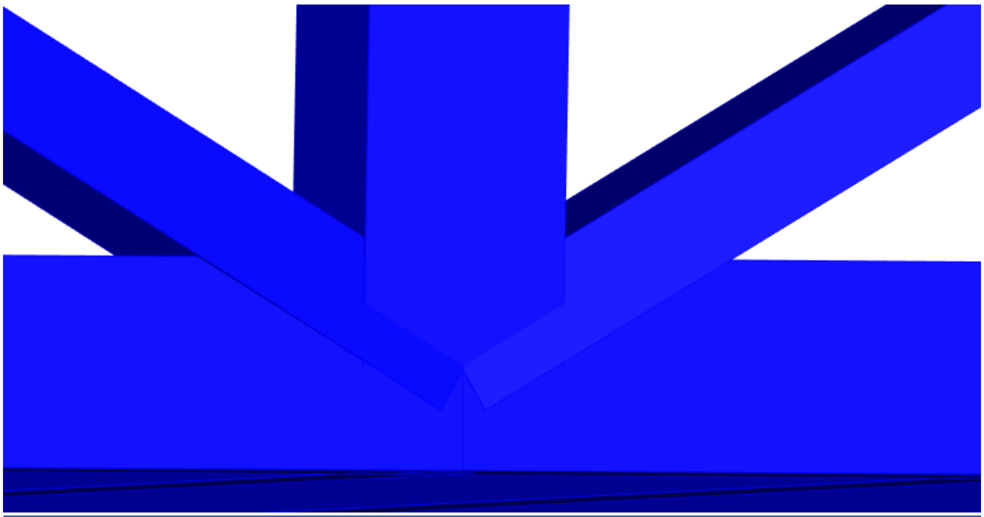

Figure 11-6 Main bracing diagonals oriented along the wind

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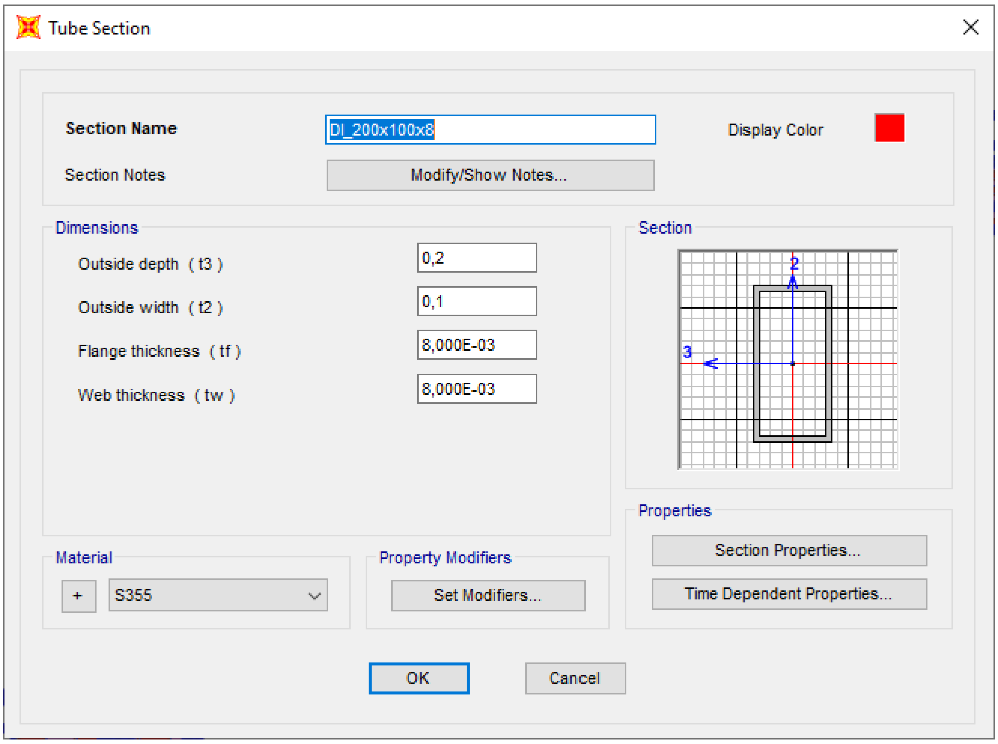
The sections have been defined with the standard definition in SAP2000.



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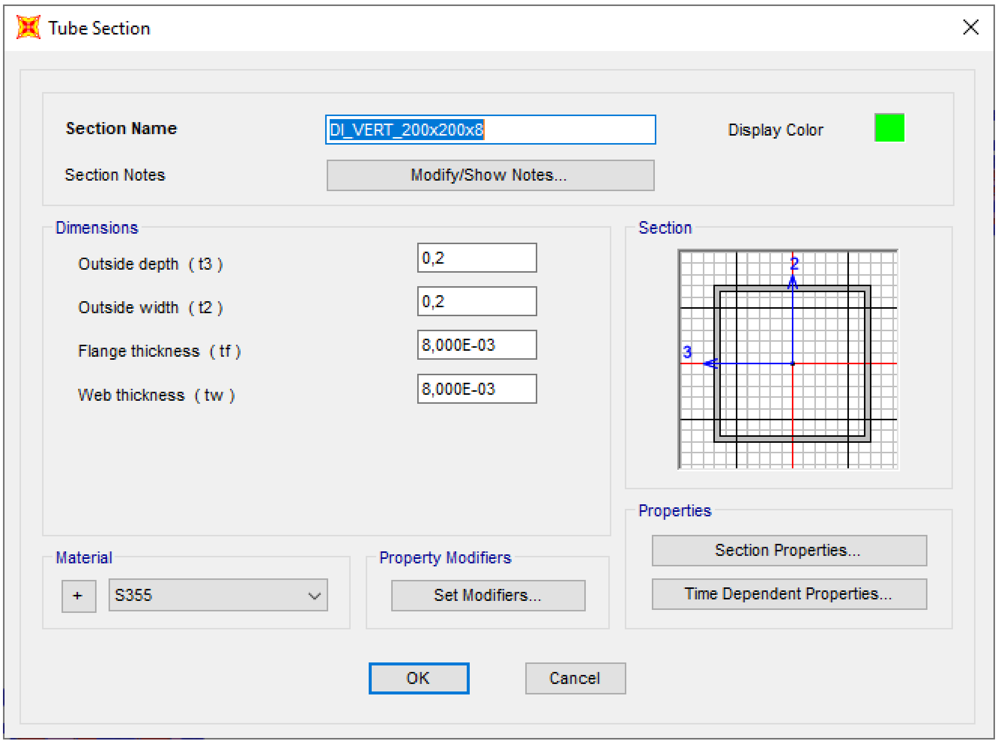

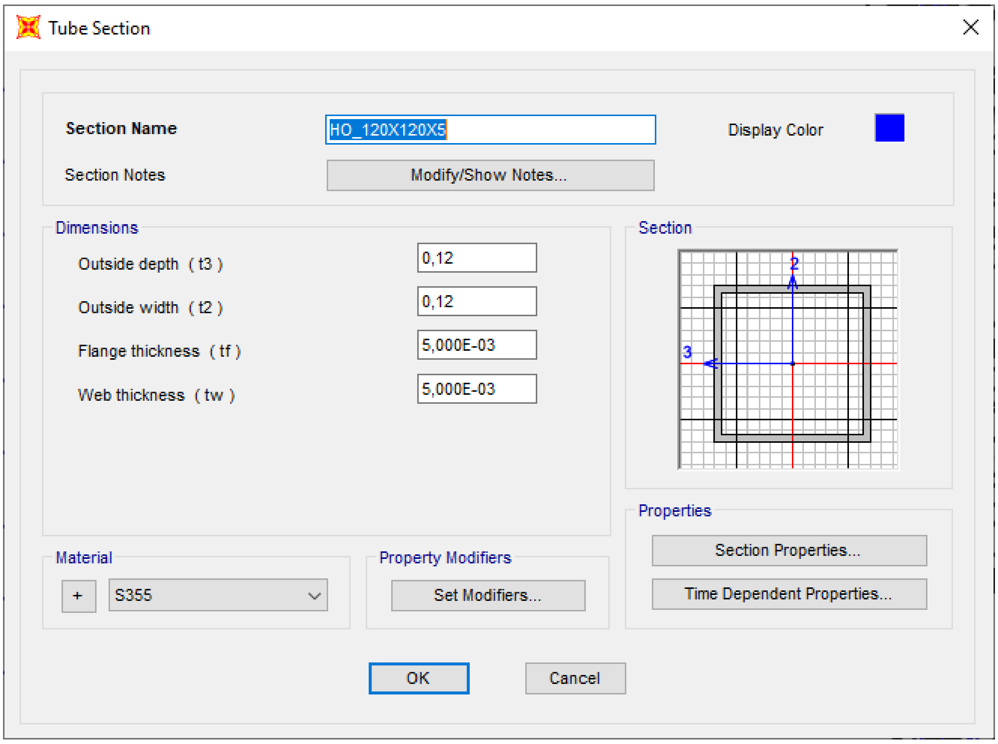

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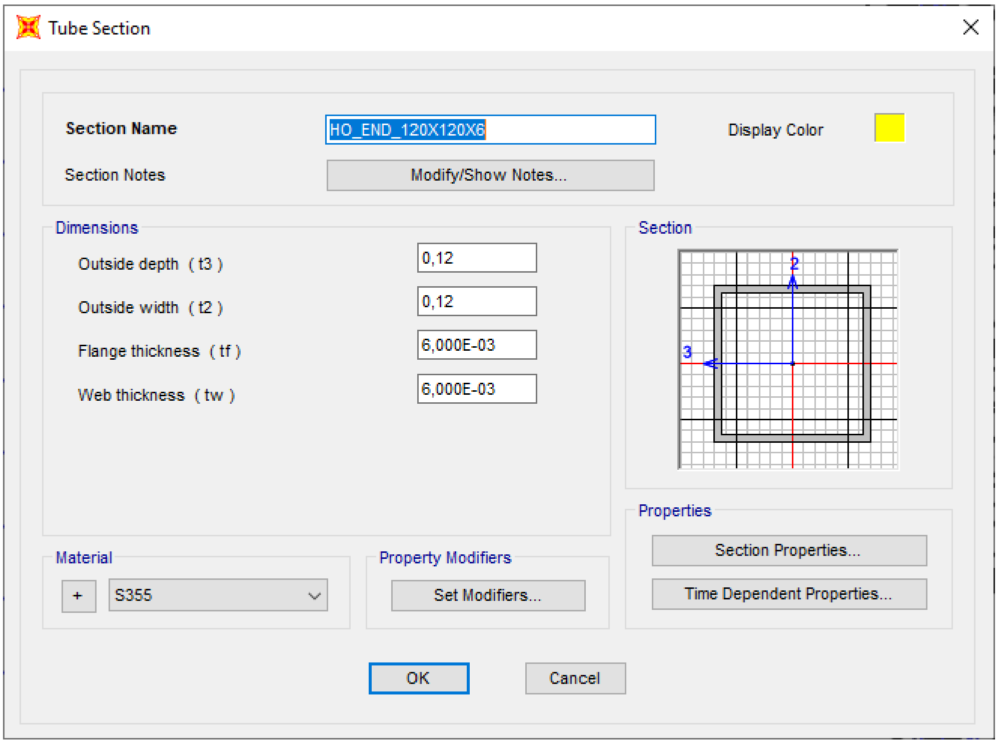

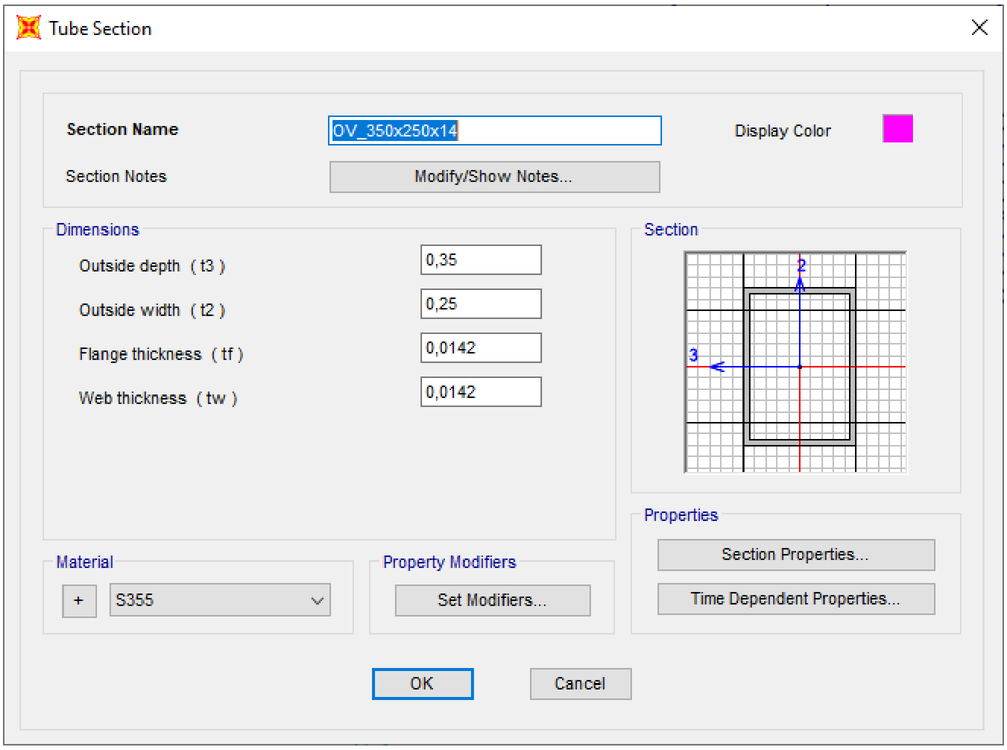

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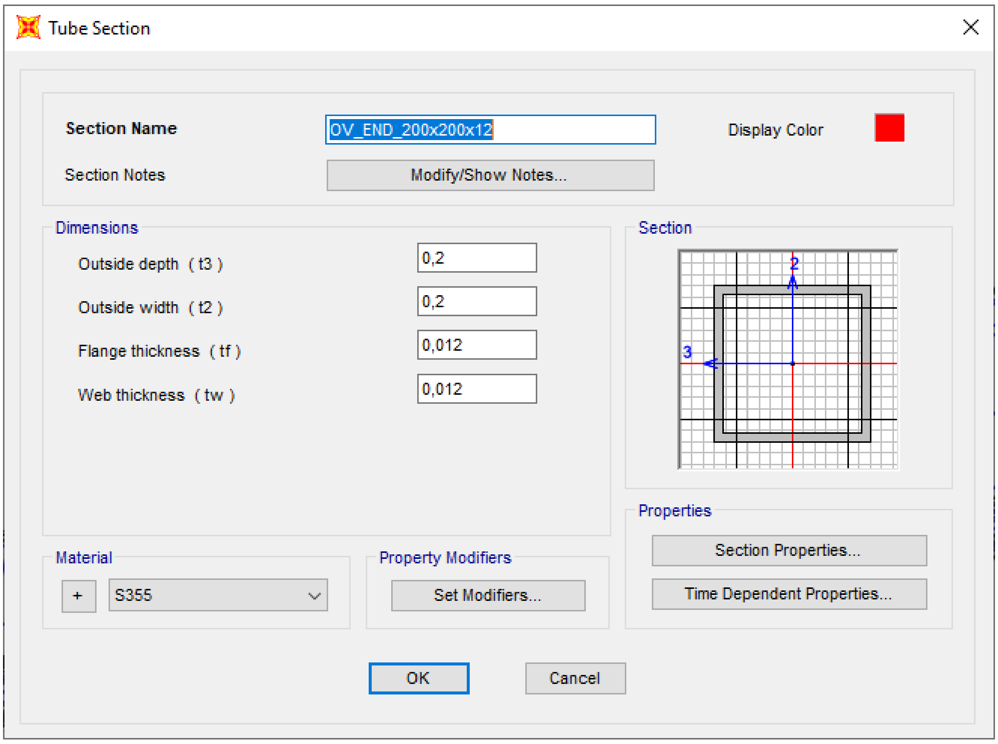

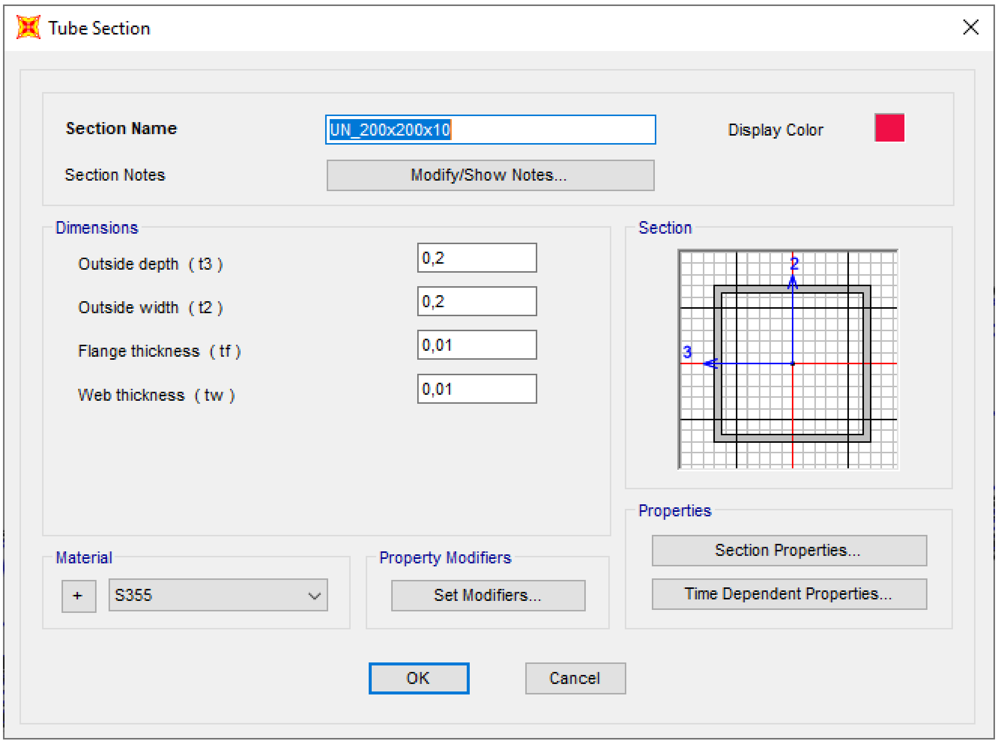

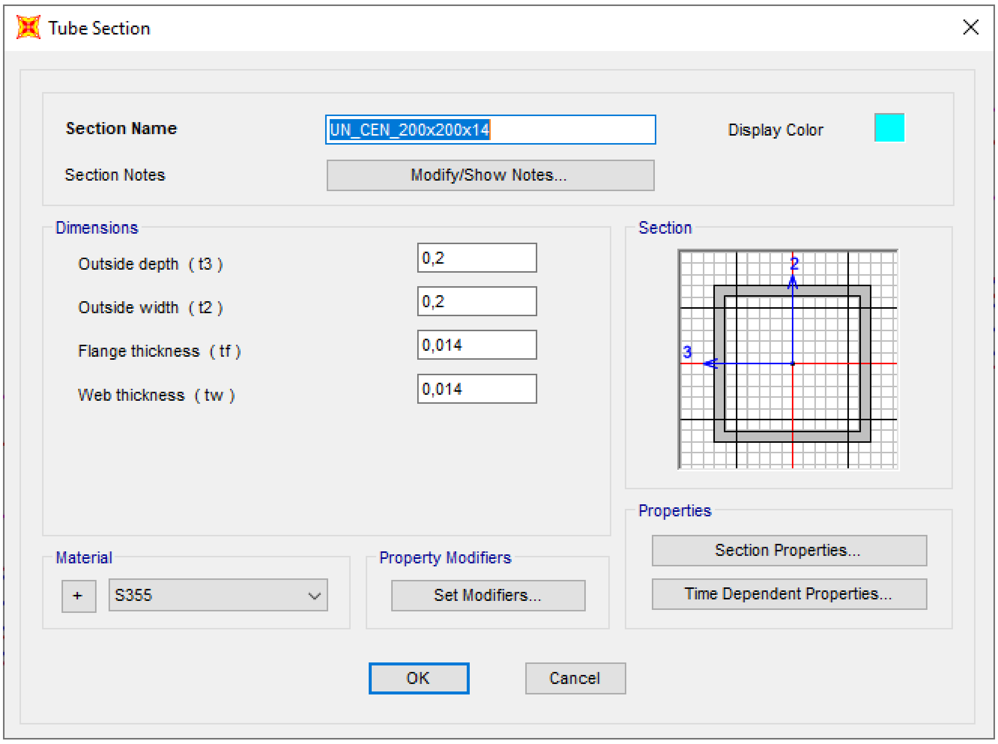

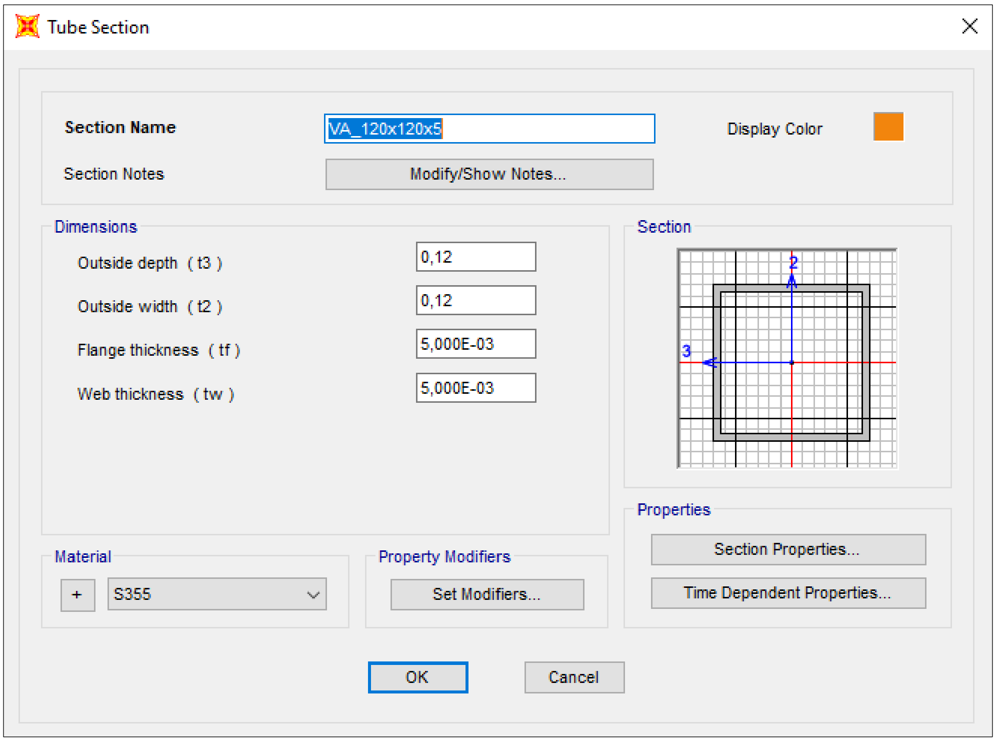

Figure 11-7 Section definitions in SAP2000 model

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11.4.2 Plate cross sections

The only plate section is a 8mm S355 section. In order to better analyze the local effects a separate model was used with a much finer meshing.

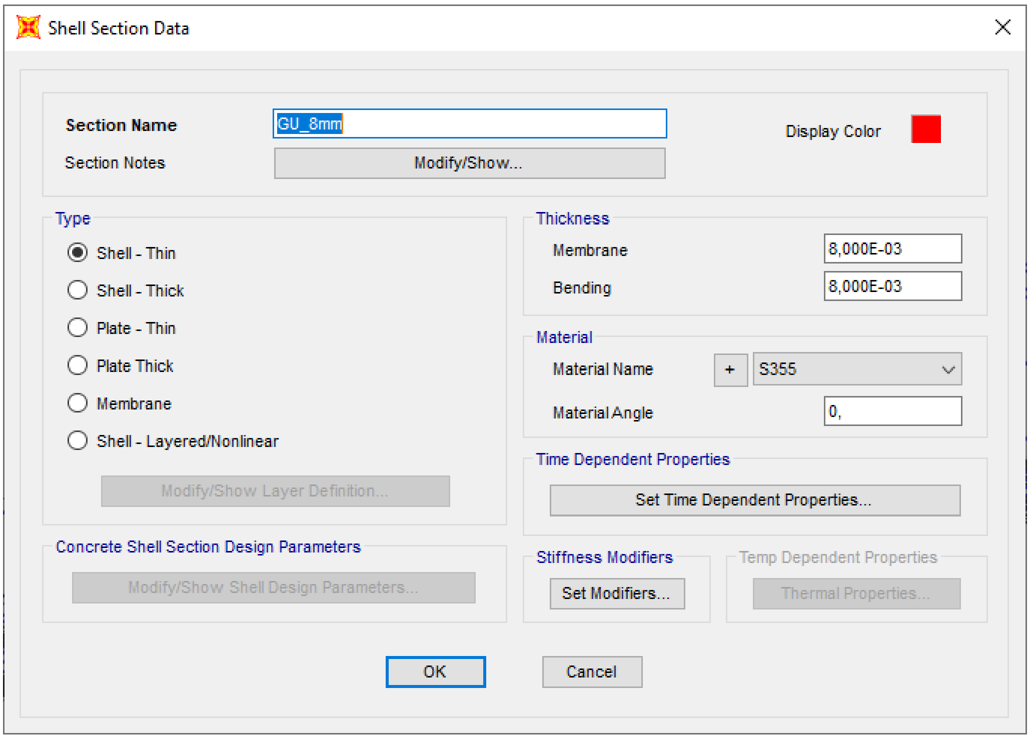

Figure 11-8 Plate cross sections definitions in SAP2000

11.5 OFFSETS

Bearings are offset to account for eccentricity.

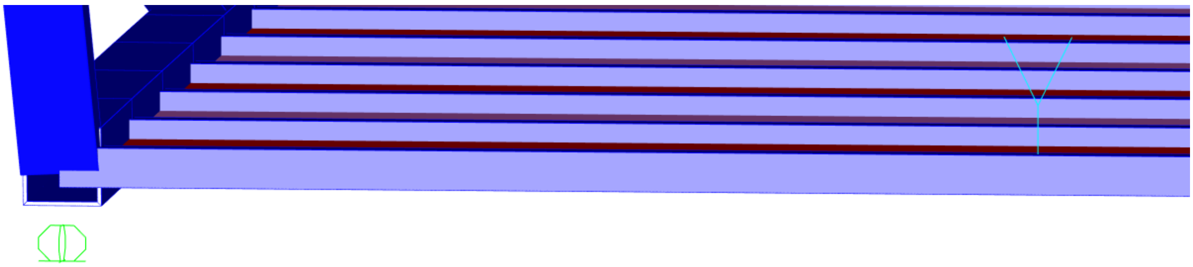

Figure 11-9 Beam offsets to better reflect the structural situation

No eccentricities are considered for either main bracing (lower end) or wind bracing. NS-EN 1993-1-8 5.1.5 (5) allows us to not consider the eccentricity so the starting point of the braces is adjusted accordingly due to the gap/overlap being smaller than the limit. For the compression chord, eccentricities are introduced in the model.

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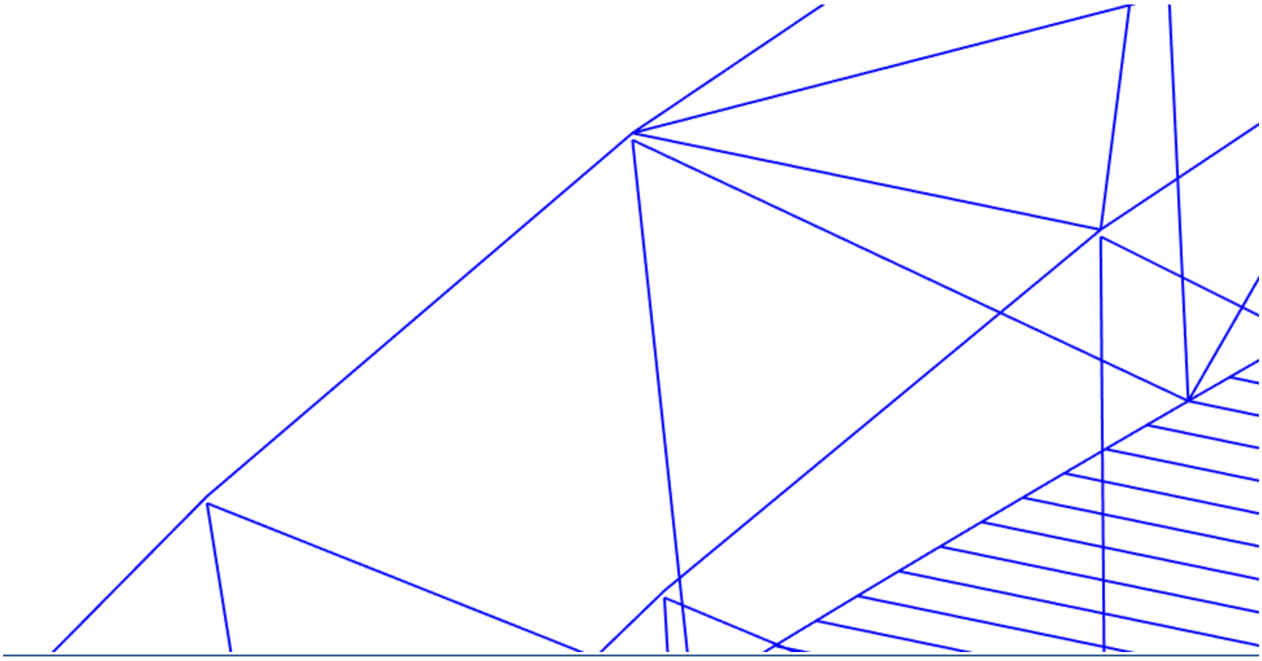

Figure 11-10 top chord and main bracing are meeting eccentrically

11.6 RELEASES

According to NS EN 1993-1-8 the RHS joints in the bracing can be modelled as pinned joints for external forces. Forces acting on the bars themselves, need to be taken into account accordingly. Therefore beams in bracings (both vertical and wind bracing) have been modelled as pinned only in the bracing plane. The bracing plane is the vertical plane for the main bracing and the horizontal plane for the wind bracing. Moments due to wind forces acting on the braces will therefore be taken into account.

The lower floor beams have not been modelled as pinned as they will be transferring moment for which they are designed.

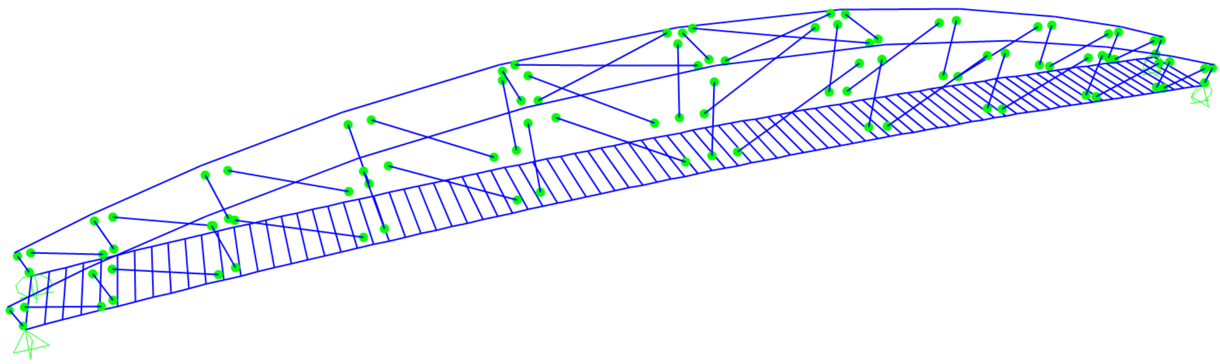

Figure 11-11 Beam releases in the lattice plane.

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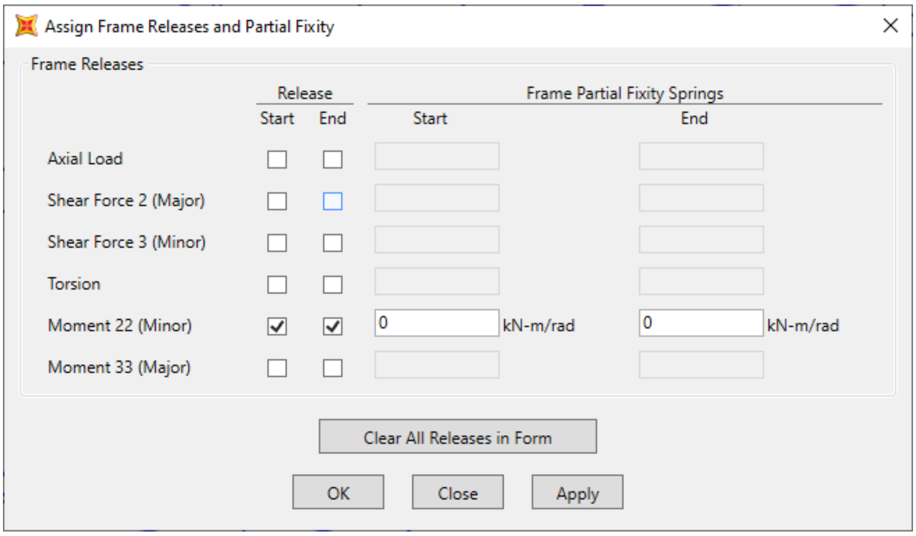

Figure 11-12 Bracing moment realeases only on the bracing plane.

12 VEDLEGG 3 – FEM – LASTER PÅFØRT I MODELLEN

The loads applied are summarized in the following table:

12.1 PERMANENT LOADS

12.1.1 Self-weight (DEAD)

Self-weight of steel is automatically calculated by the software. An additional 5% is added in order to account for welds and plates.

12.1.2 Superimposed dead load (SDL)

1kN/m as described in the design basis. Applied in the upper chord as it is more unfavorable with regards to buckling. The pavement load of 0,15kN/m2 is applied directly on the shells.

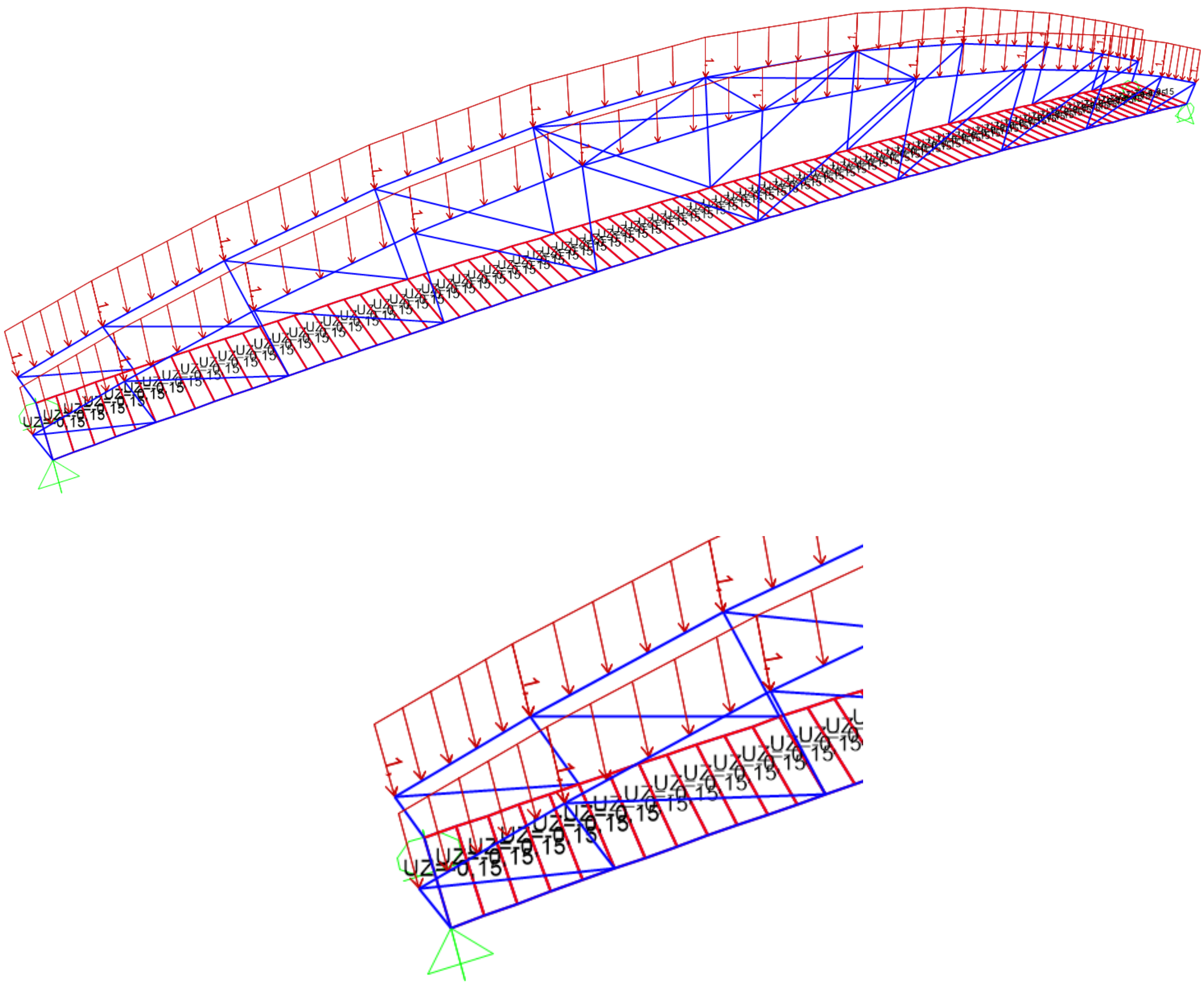

Figure 12-1 SDL loads.

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12.2 VARIABLE LOADS – WIND

Wind loads are obtained from basic values given at Basis of Design, as follows.

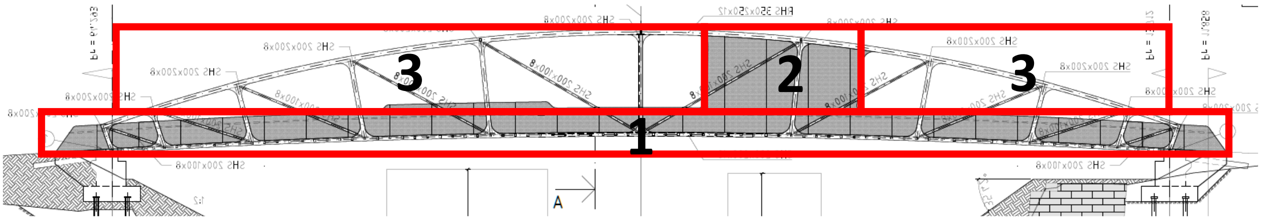

Figure 12-2 Areas for wind calculation

The part 8 of the wind Eurocode is applicable for bridges with a near constant section. We have an intermediate case as the structure is neither a lattice structure, nor a bridge.

The strategy therefore has been to divide the structure in two parts and calculate the wind loads separately.

The loads corresponding to the part upper from the lower plexiglass panel have been calculated as loads on individual members on both sides.

This results in a conservative estimate of the wind load, as there will be some wake effects that reduce the actual wind load.

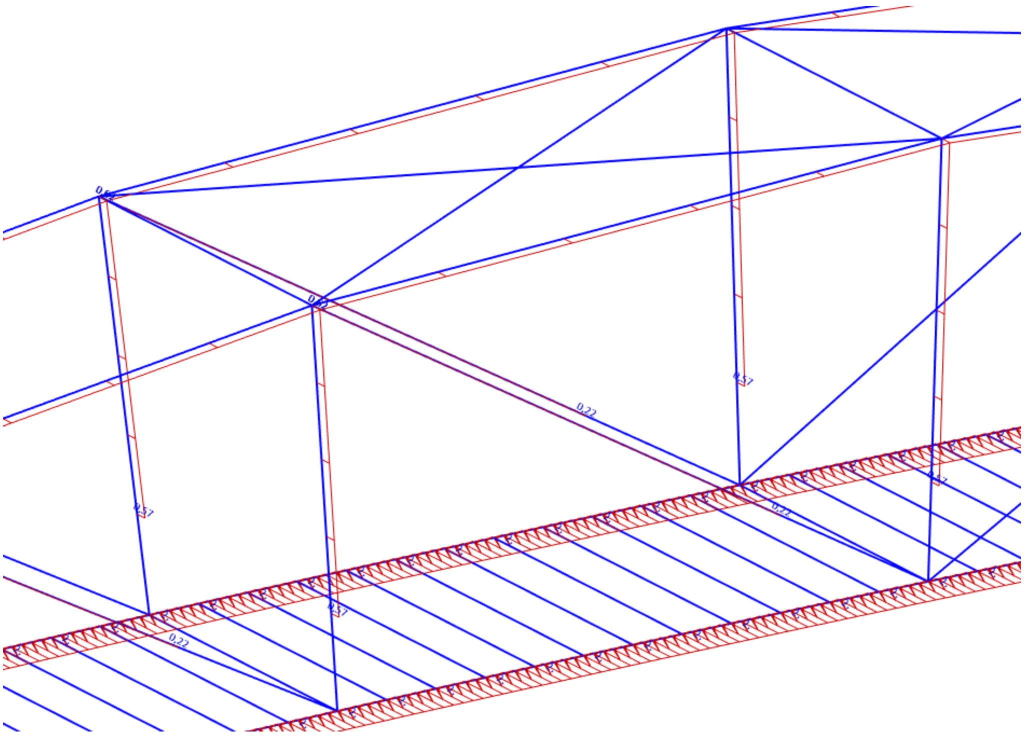

Figure 12-3 Application of northward wind on beams and bridge deck

The vertical plexiglass surface is added directly on the chords as an additional load.

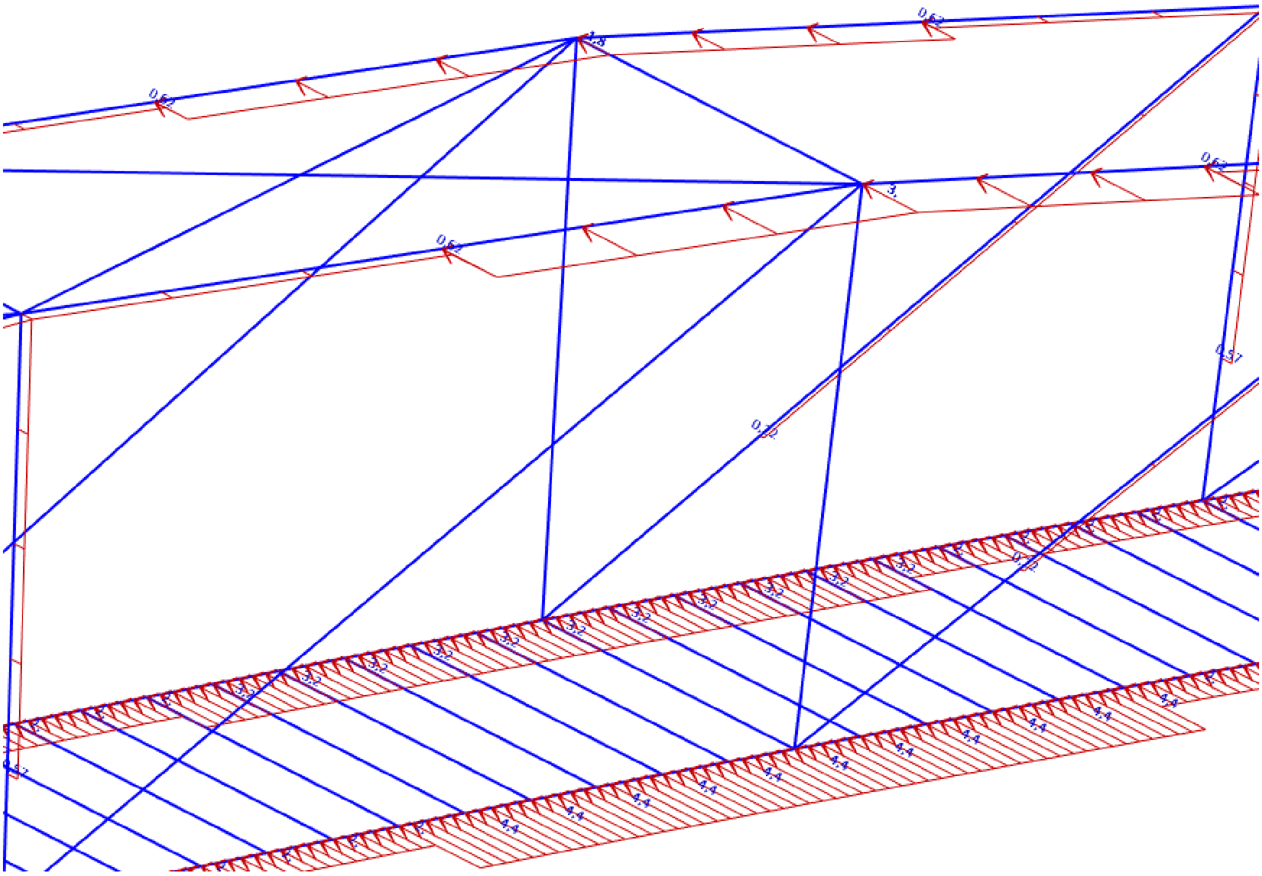

Figure 12-4 Application of northward wind on beams and bridge deck

The deck loads are applied on the deck while the beam loads are applied directly on the beams.

The vertical wind forces on the deck, that can act both upwards and downwards, are applied on the chords taking into account the direction of the incoming wind as it will change the point of application. The resulting force has an equivalent point of application corresponding to b/4

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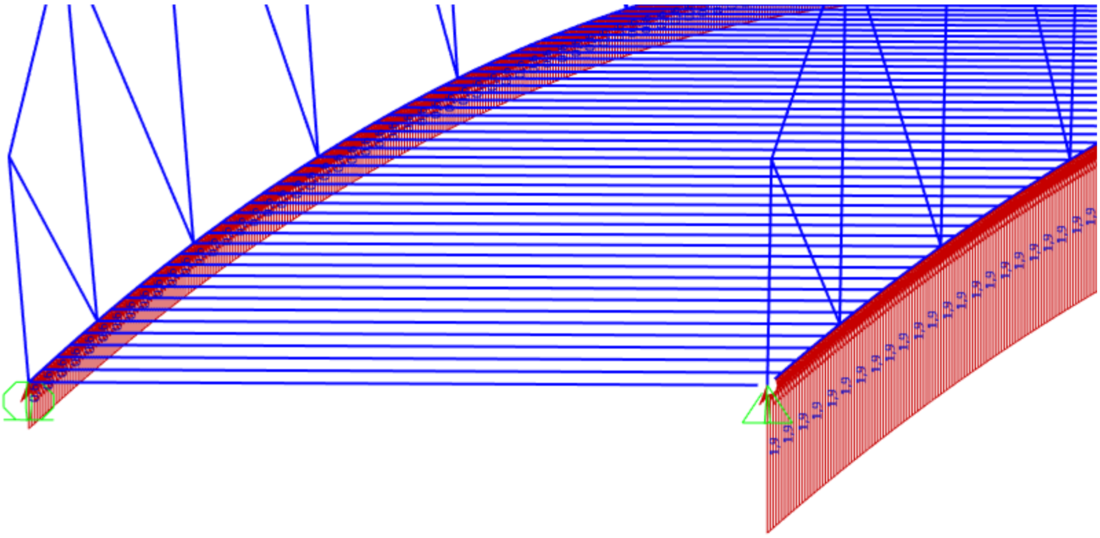

Figure 12-5 upward lower chord forces due to northward wind

Longitudinal wind is applied directly on the chord and is always causing compression.

The described forces are then combined to the following load cases.

Loadings are consistent with the wind calculations annex (see annex 1).

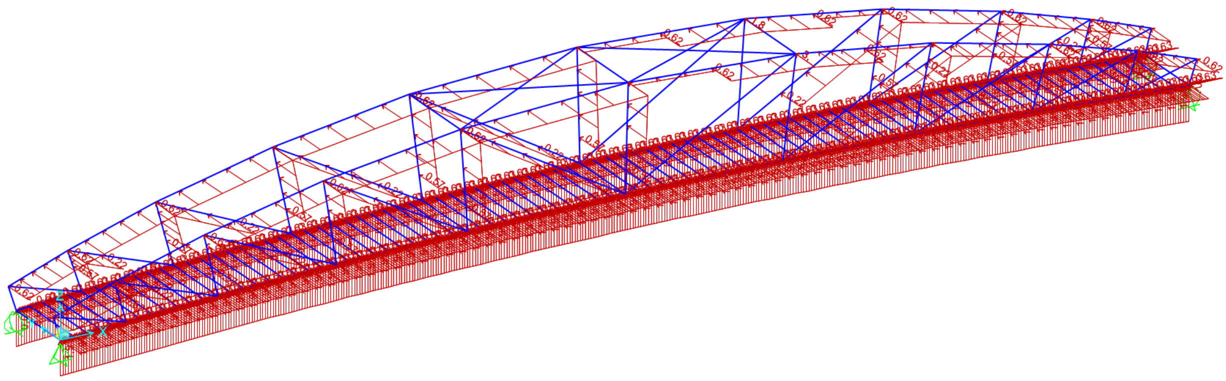

Figure 12-6 WINDYPOSZPOS northward upward wind*

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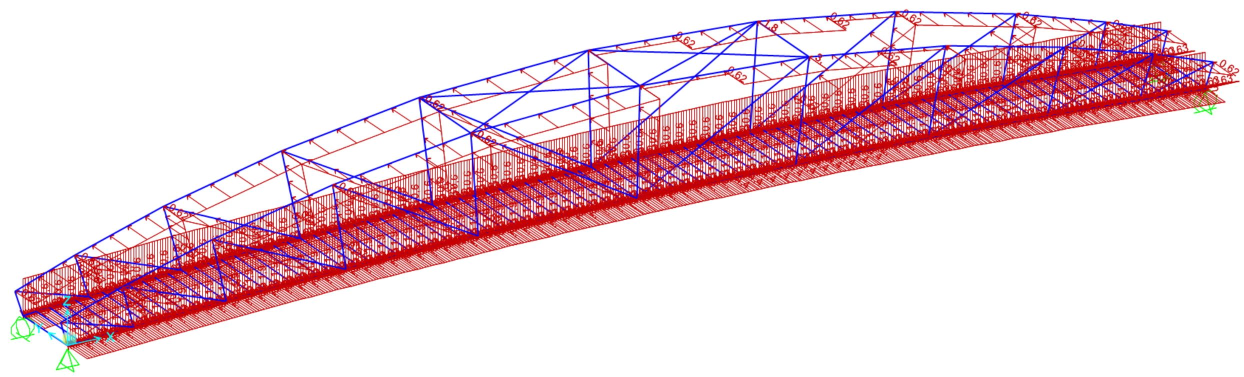

Figure 12-7 WINDYPOSZNEG northward downward wind*

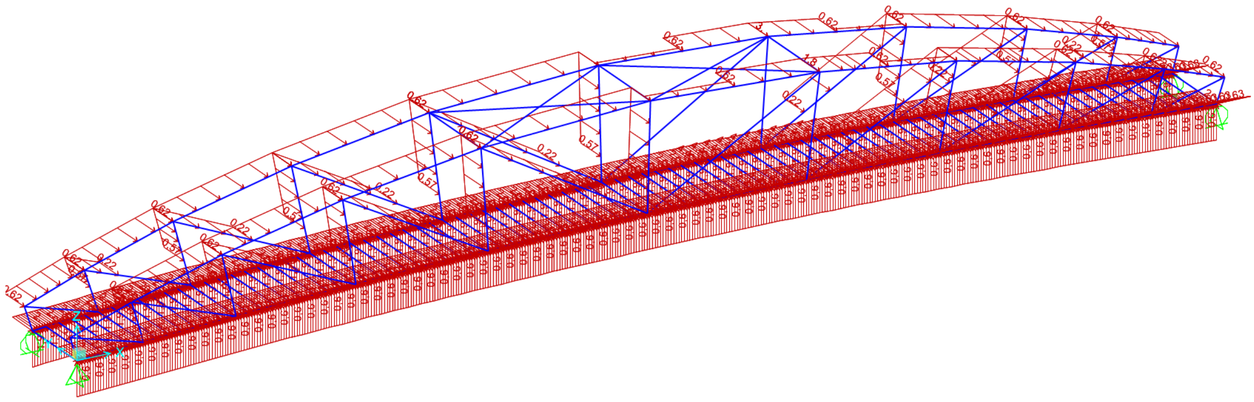

Figure 12-8 WINDYNEGZPOS southward upward wind*

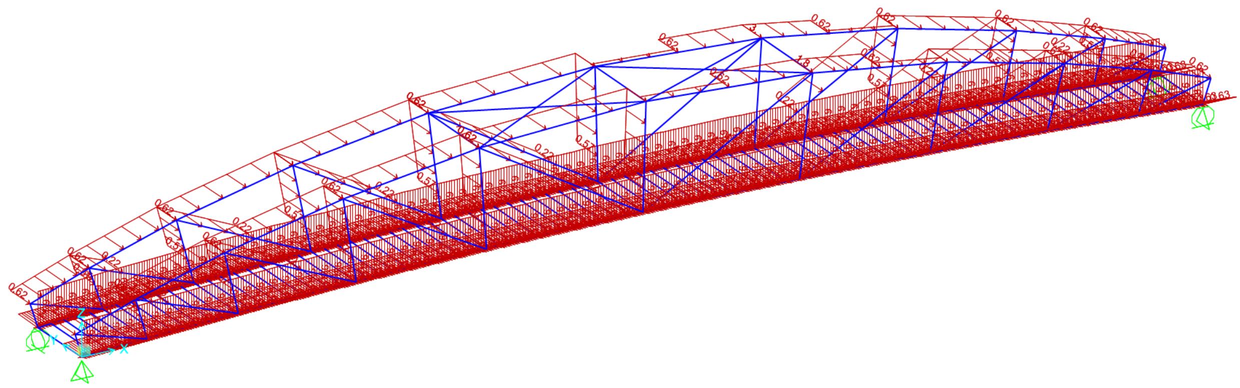

Figure 12-9 WINDYNEGZNEG southward downward wind*

* Loadings are scaled per element so size can be misleading

12.3 VARIABLE LOADS – TEMPERATURE

Three cases are considered. Differential temperature is unlikely as the bridge is oriented east-west and even if it happened, it would have no effect as the bridge is free to move. Temperature difference however could be relevant.

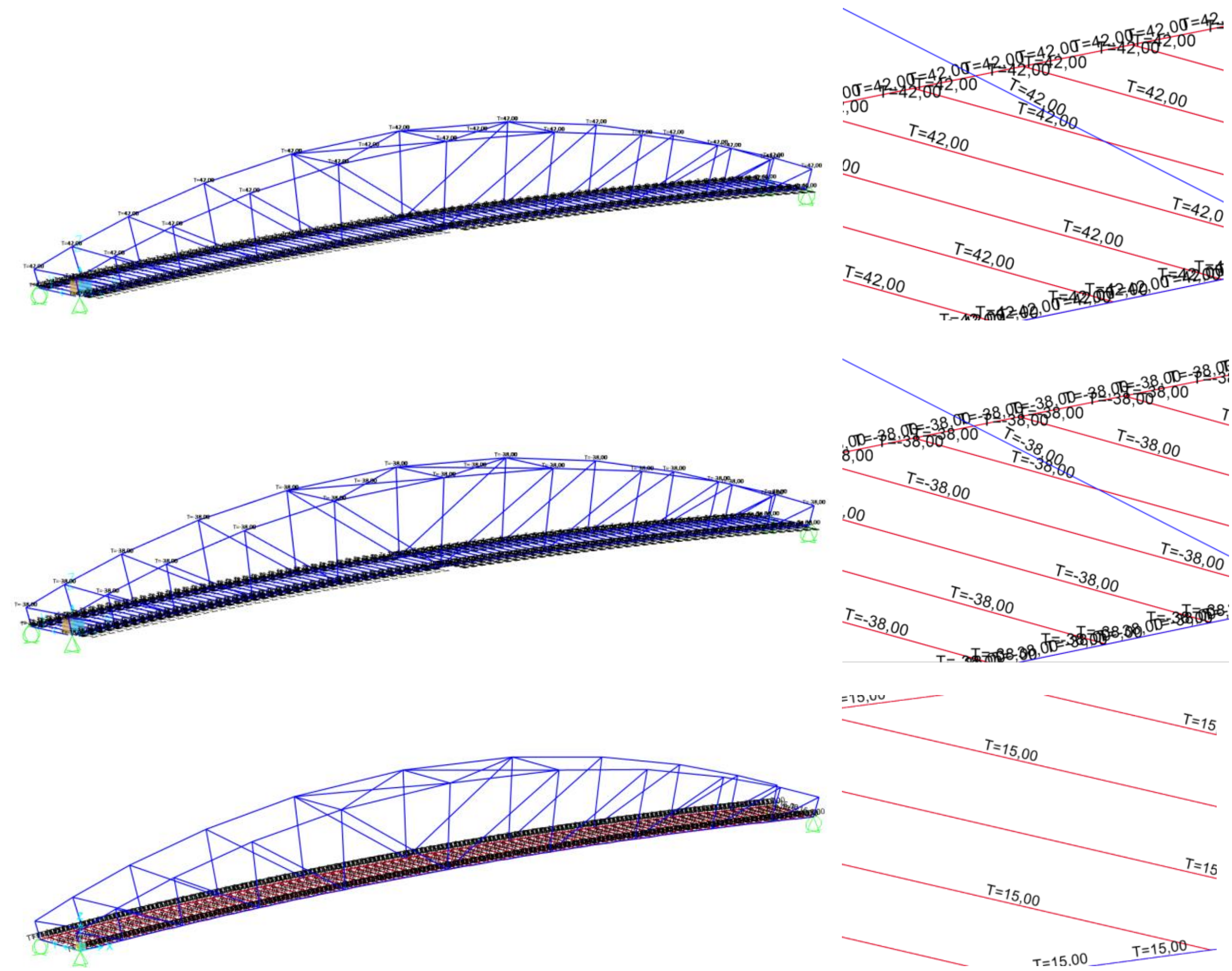

Figure 12-10 Temperature load cases

A couple of additional temperature load cases are considered, in order to evalaute the extra temperature for bearings (±20C) and joints design.

12.4 VARIABLE LOADS – SNOW

Snow loads always give lower loads than traffic. They are therefore not used in the combinations as it cannot be combined with traffic.

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12.5 VARIABLE LOADS – FRICTION FORCES AT SLIDING BEARINGS

Friction loads at sliding bearings are obtained as a 6% of the permanent loads.

The reactions with permanent loads are:

TABLE: Joint Reactions								
Joint	OutputCase	CaseType	F1	F2	F3	M1	M2	M3
Text	Text	Text	KN	KN	KN	KN-m	KN-m	KN-m
UN_H_1	PERM	LinStatic	0	0	139,883	0	0	0
UN_V_1	PERM	LinStatic	0	0	139,883	0	0	0
UN_H_87	PERM	LinStatic	0	0	139,883	0	0	0
UN_V_87	PERM	LinStatic	0	0	139,883	0	0	0

The value for the bearings on the directions sliding are therefore:

$$139,88\ \text{kN} \cdot 6\ \% = 8,3928\ \text{kN}$$

The loads are applied in the worst direction possible creating the maximum compressions.

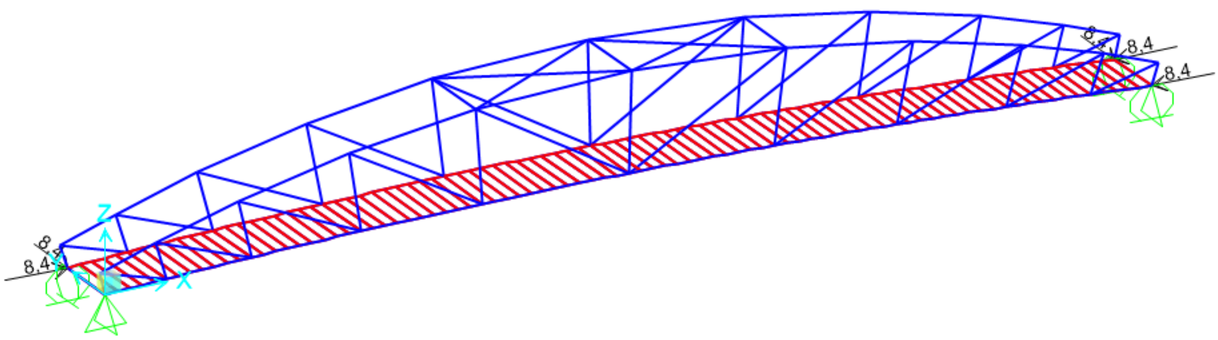

Figure 12-11 Bearing friction forces

12.6 VARIABLE LOADS – TRAFFIC LOADS

This section is divided in two groups, the ones for crowd and the service vehicle loading. Values are according to design basis.

12.6.1 Distributed loads

Vertical load is considered distributed along the whole length of the bridge

Horizontal forces are uniformly distributed along the pavement according to NS-EN 1991-2 4.4.1 (4)

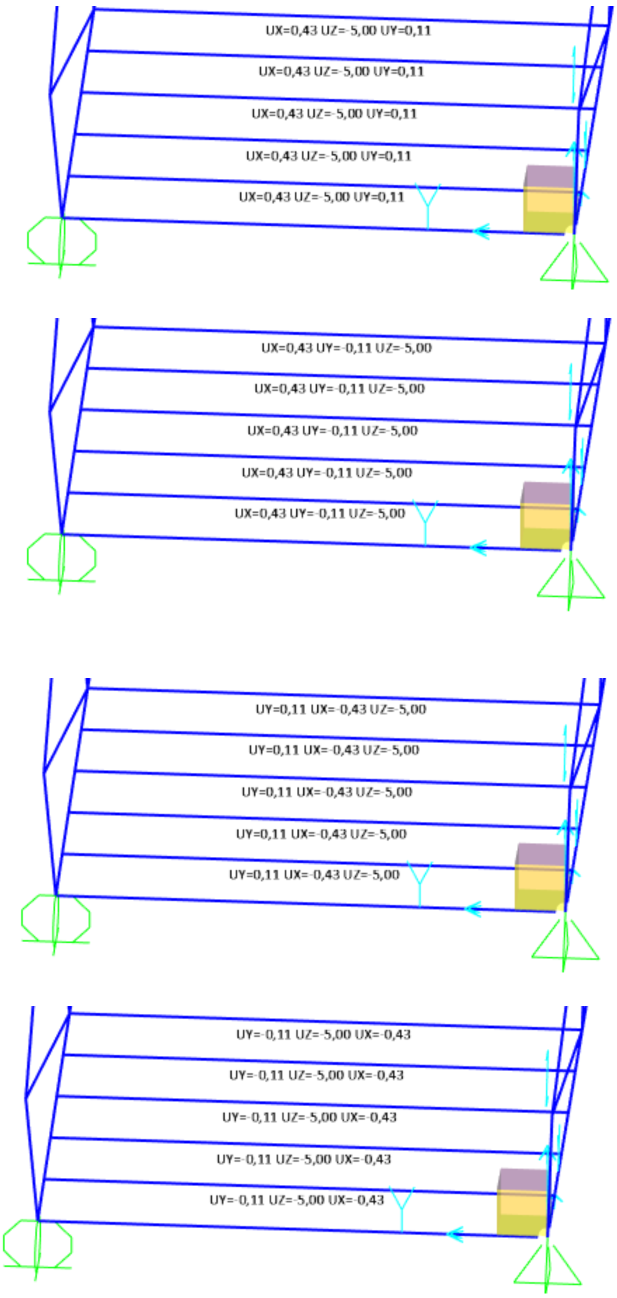

Figure 12-12 Load cases for uniformly distributed traffic

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12.6.2 Concentrated loads: Service vehicle

A service vehicle of 43kN is considered. This is based on the Volvo L20 F vehicle taken as a reference. The load is therefore 10,75kN per wheel.

In order to taken into account the worst cases, three options are considered: centric, eccentric to the left with one wheel in the middle and eccentric to the right with one wheel in the middle.

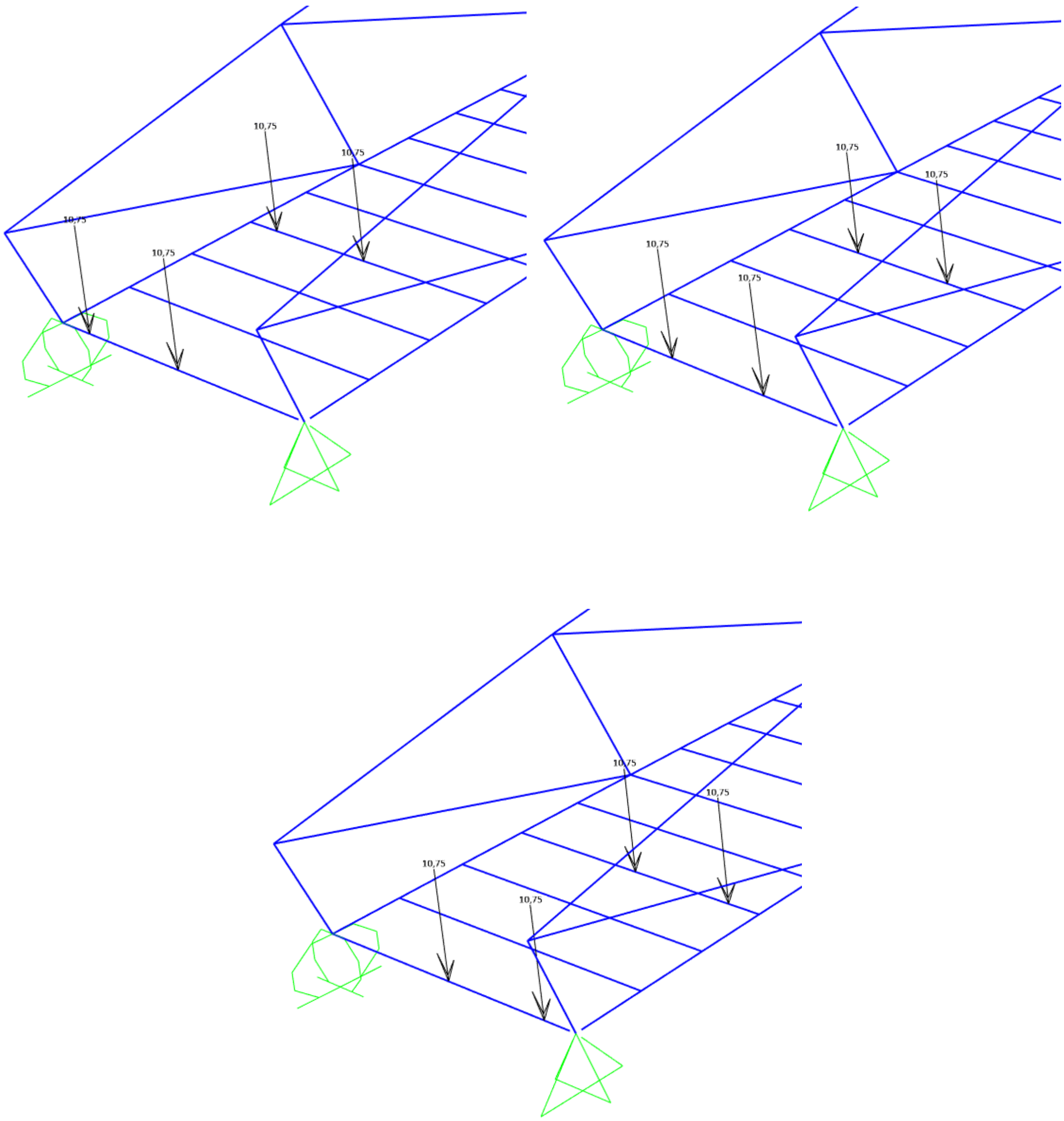

Figure 12-13 Concentrated loads on three different lanes

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Load cases with truck partly on the bridge were also considered.

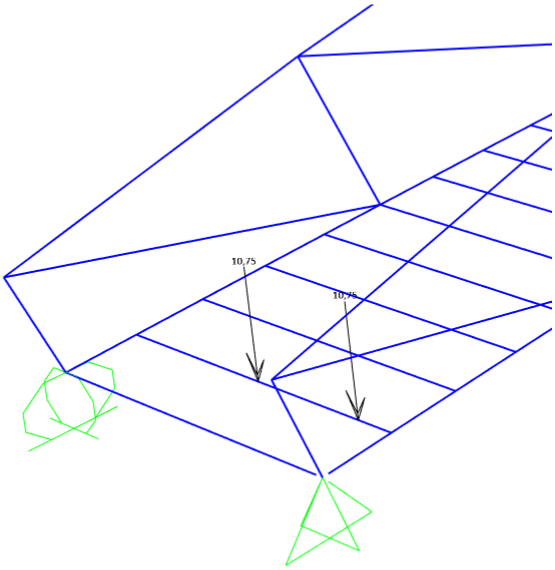

Figure 12-14 Concentrated loads with truck partly on the bridge

We defined 90 load cases for each lane on SAP2000 and then combined them in one envelope combination in order to simplify the load combination matrix.

For load case definition, concentrated loads were combined with horizontal traffic loads.

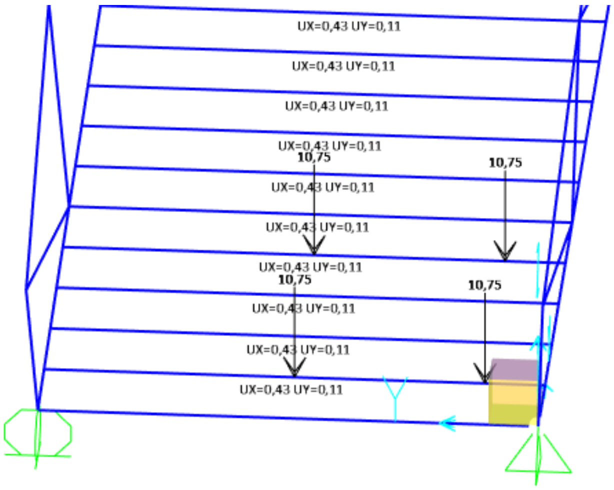

Figure 12-15 Load case definition with both vertical and horizontal loads

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12.7 IMPERFECTION

In order to account for global buckling, and imperfection load is used in accordance with chapter 5.2.2 in NS-EN 1993-1-1.

With rectangular hollow sections the buckling curve will be “a”. Therefore the imperfection e0 will be:

$$L := 51{,}5\ \mathrm{m} \qquad e_0 := \frac{L}{300} = 171{,}6667\ \mathrm{mm}$$

The maximum load considering Figure 5.4 of 1993-1-1 will therefore be:

$$N_{Ed} := 1600\ \mathrm{kN} \qquad \frac{8 \cdot N_{Ed} \cdot e_0}{L^2} = 0{,}8285\ \frac{\mathrm{kN}}{\mathrm{m}}$$

The distribution along the top chord will vary along with the design axial force:

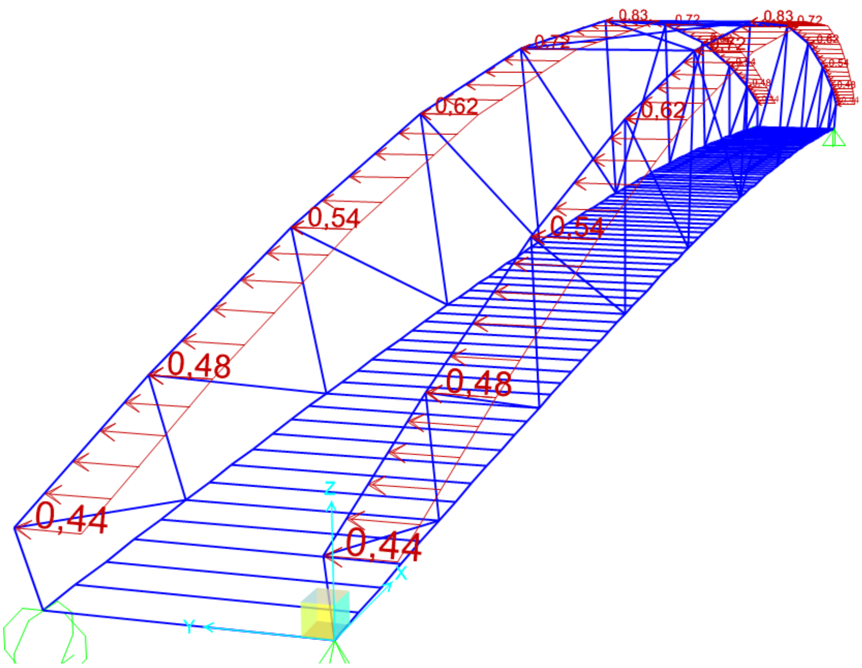

Figure 12-16 Imperfection load distribution.

12.8 DYNAMIC LOAD

In order to calculate the forces for dynamic loading we have used the guidelines of Design of lightweight footbridges for human induced vibrations from the European Commission as the Eurocodes don’t give force values to be used for pedestrian bridges.

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Input data

$\xi := 0,4\ \%$ Steel damping from DB

$S := 50\ m \cdot 3\ m = 150\ m^2$ Walkable surface

$f_{sv} := 4,47\ Hz$ $f_{sh} := 1,33\ Hz$ Eigenfrequencies from FEM

Loads

Reference: EUR 23984 EN, Design of lightweight footbridges for human induces vibrations (Table 4.8)

EUR 23984 EN 4.3.1

$n_{service} := 15$ 15 people - very light traffic

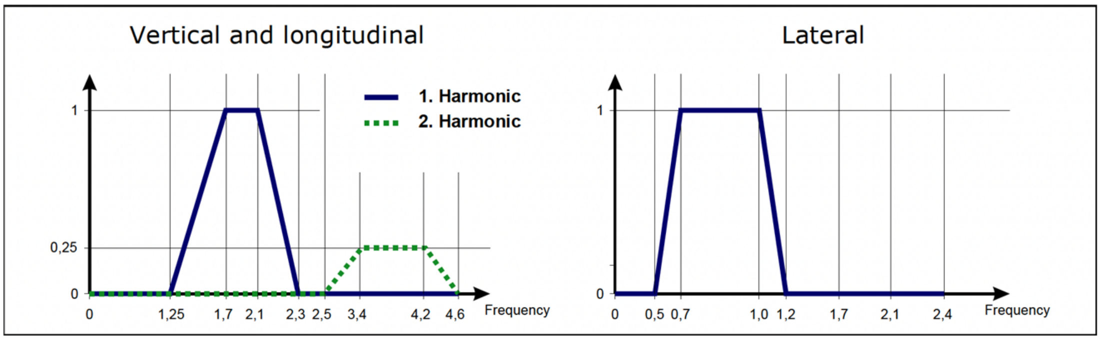
$d_{crowd} := \frac{1}{m^2}$

EUR 23984 EN Table 4.8

$P_v := 280\ N$ Vertical force, mulitply by two to get amplitude

$P_h := 35\ N$ Horizontal force, mulitply by two to get amplitude

Reduction coefficient



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$$\psi_v := \text{linterp}\left(\begin{bmatrix}4,2\\4,6\end{bmatrix}; \begin{bmatrix}0,25\\0\end{bmatrix}; \frac{f_{sv}}{\text{Hz}}\right) = 0,0812 \qquad \psi_h := 0$$

Normal use (table 4.8)

$$n := n_{service}$$

$$n_p := \frac{10,8 \cdot \sqrt{\xi \cdot n}}{S} = 0,018 \cdot \frac{1}{\text{m}^2}$$

Equivalent forces $P_v \cdot n_p \cdot \psi_v = 0,401 \frac{\text{N}}{\text{m}^2}$ $P_h \cdot 2 \cdot n_p \cdot \psi_h = 0 \frac{\text{kN}}{\text{m}^2}$

Crowded

$$n := d_{crowd} \cdot S = 150$$

$$n_p := \frac{1,85 \cdot \sqrt{n}}{S} = 0,15 \cdot \frac{1}{\text{m}^2}$$

Equivalent forces $P_v \cdot n_p \cdot \psi_v = 3,44 \frac{\text{N}}{\text{m}^2}$ $P_h \cdot 2 \cdot n_p \cdot \psi_h = 0 \frac{\text{kN}}{\text{m}^2}$

The dynamic loads are modelled with a uniformly distributed load on the plates.

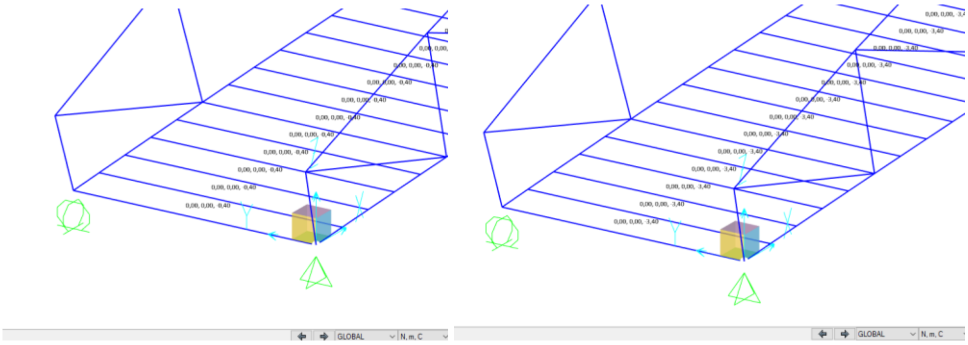

Figure 12-17 Vertical dynamic loads in service and full bridge in N/m².

The loads were applied with a sinusoidal shape in a time-history analysis.

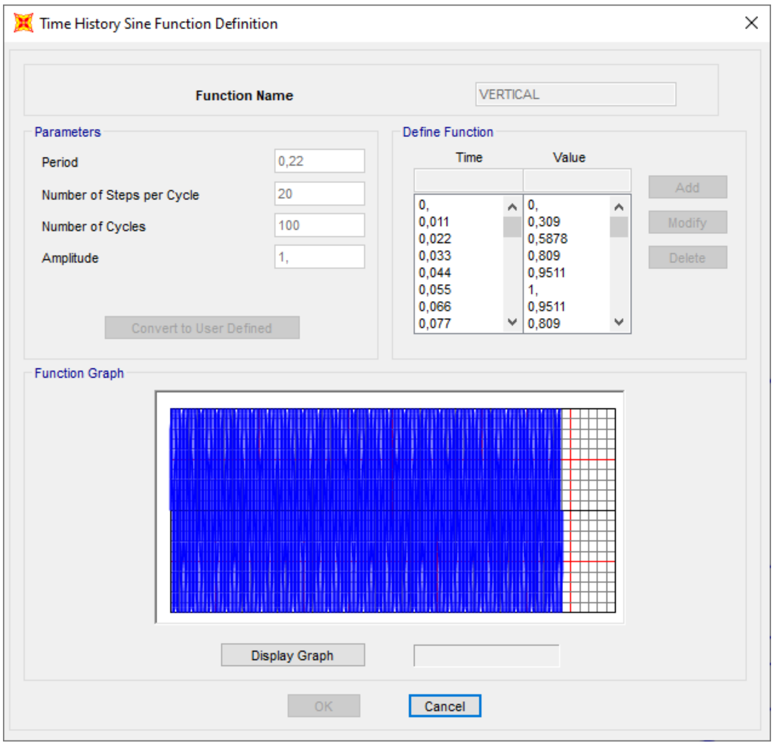

Figure 12-18 Time history definition for vertical forces.

The results are as follows:

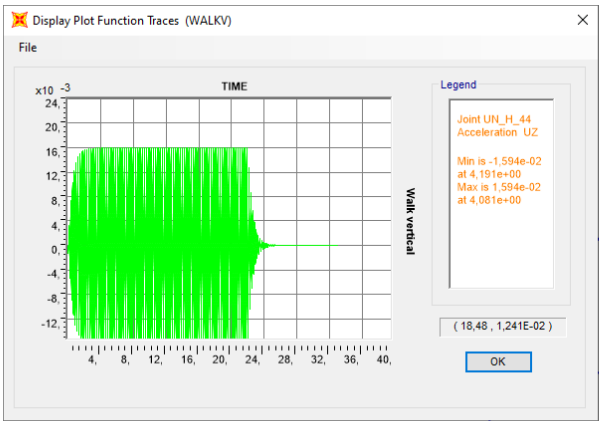

Figure 12-19 Vertical accelerations in service.

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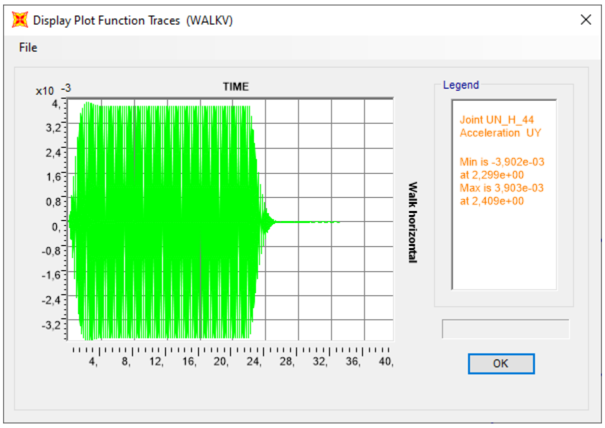

Figure 12-20 Horizontal accelerations in service.

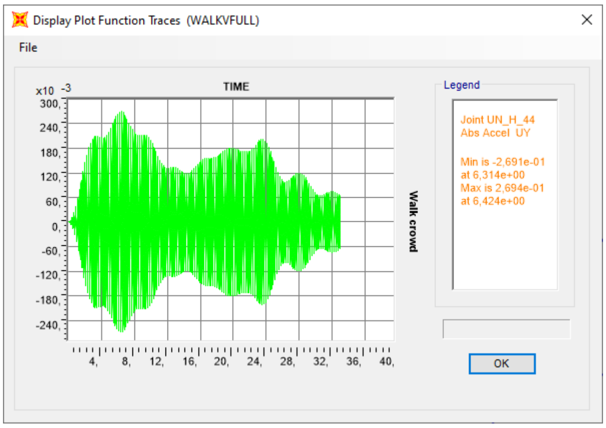

Figure 12-21 Horizontal accelerations with a crowd.

Limits

Reference 1991 -2 A2.4.3.2

$a_{vRd} := 0,7 \ \frac{m}{s^2}$ $a_{hRd} := 0,2 \ \frac{m}{s^2}$ $a_{crowdRd} := 0,4 \ \frac{m}{s^2}$

Calculated accelerations

Results from SAP2000 model.

$a_{vEd} := (0,015) \ \frac{m}{s^2}$ $a_{hEd} := (0,003) \ \frac{m}{s^2}$ $a_{crowdEd} := (0,27) \ \frac{m}{s^2}$

$\frac{a_{vEd}}{a_{vRd}} = 0,021$ $\frac{a_{hEd}}{a_{hRd}} = 0,015$ $\frac{a_{crowdEd}}{a_{crowdRd}} = 0,68$

12.9 LOAD COMBINATIONS

12.9.1 General design combinations

Load combinations are done by means of an Excel sheet. The criterion has been testing all possible combinations of actions. The loads have been defined in a way that allows this to be implemented.

Combo	TYPE	DEAD	SDL	W	T	TR	BE
ULS_01	ULS	1,20	1,20				
ULS_02	ULS	1,00	1,00				
ULS_03	ULS	1,20	1,20	1,60	0,84	0,95	0,95
ULS_04	ULS	1,00	1,00	1,60	0,84	0,95	0,95
ULS_05	ULS	1,20	1,20	1,12	1,20	0,95	0,95
ULS_06	ULS	1,00	1,00	1,12	1,20	0,95	0,95
ULS_07	ULS	1,20	1,20	1,12	0,84	1,35	0,95
ULS_08	ULS	1,00	1,00	1,12	0,84	1,35	0,95
ULS_09	ULS	1,20	1,20	1,12	0,84	0,95	1,35
ULS_10	ULS	1,00	1,00	1,12	0,84	0,95	1,35
ULS_11	ULS	1,35	1,35	1,12	0,84	0,95	0,95
ULS_12	ULS	1,00	1,00	1,12	0,84	0,95	0,95
SLSR_01	SLS	1,00	1,00				
SLSR_03	SLS	1,00	1,00	1,00	0,70	0,70	0,70
SLSR_05	SLS	1,00	1,00	0,70	1,00	0,70	0,70
SLSR_07	SLS	1,00	1,00	0,70	0,70	1,00	0,70
SLSR_09	SLS	1,00	1,00	0,70	0,70	0,70	1,00
SLSFQ_01	SLS	1,00	1,00				
SLSFQ_03	SLS	1,00	1,00	0,60	0,50	0,50	0,50
SLSFQ_05	SLS	1,00	1,00	0,50	0,60	0,50	0,50
SLSFQ_07	SLS	1,00	1,00	0,50	0,50	0,70	0,50
SLSFQ_09	SLS	1,00	1,00	0,50	0,50	0,50	0,60
SLSQP_01	SLS	1,00	1,00				
SLSQP_03	SLS	1,00	1,00	0,50	0,50	0,50	0,50

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An example of some combinations is shown below:

Combo	DEAD	SDL	ND_YposZ	ND_YposZ	ND_YnegZ	ND_YnegZ	BEARFR	TEMPGRAD	TEMP_HOT	TEMP_COLD
ULS_01_1	1,20	1,20								
ULS_02_1	1,00	1,00								
ULS_03_1	1,20	1,20								
ULS_03_2	1,20	1,20						0,84		
ULS_03_3	1,20	1,20							0,84	
ULS_03_4	1,20	1,20								0,84
ULS_03_5	1,20	1,20					0,95			
ULS_03_6	1,20	1,20					0,95	0,84		
ULS_03_7	1,20	1,20					0,95		0,84	
ULS_03_8	1,20	1,20					0,95			0,84
ULS_03_9	1,20	1,20	1,60							
ULS_03_10	1,20	1,20	1,60					0,84		
ULS_03_11	1,20	1,20	1,60						0,84	
ULS_03_12	1,20	1,20	1,60							0,84
ULS_03_13	1,20	1,20	1,60				0,95			
ULS_03_14	1,20	1,20	1,60				0,95	0,84		
ULS_03_15	1,20	1,20	1,60				0,95		0,84	
ULS_03_16	1,20	1,20	1,60				0,95			0,84
ULS_03_17	1,20	1,20		1,60						
ULS_03_18	1,20	1,20		1,60				0,84		
ULS_03_19	1,20	1,20		1,60					0,84	
ULS_03_20	1,20	1,20		1,60						0,84

12.9.2 Second order analysis combinations

Additionally, the combinations with the highest axial load on the top chord were picked for a second order analysis including imperfections. Partial factors were copied, and imperfection was added. The list is shown below.

Combo	DEAD	SDL	TRAF_Xpos	WIND_YposZneg	BEARFR	TEMPGRAD	TEMP_HOT	TEMP_COLD	IMP
BUCKL_07_57	1,20	1,20	1,35	1,12					1,00
BUCKL_07_58	1,20	1,20	1,35	1,12		0,84			1,00
BUCKL_07_59	1,20	1,20	1,35	1,12			0,84		1,00
BUCKL_07_60	1,20	1,20	1,35	1,12				0,84	1,00
BUCKL_07_61	1,20	1,20	1,35	1,12	0,95				1,00
BUCKL_07_62	1,20	1,20	1,35	1,12	0,95	0,84			1,00

Both the imperfection, the relevant combinations and the relevant buckling mode all show a torsion of the structure in the same direction. The imperfection is therefore considered adequate.

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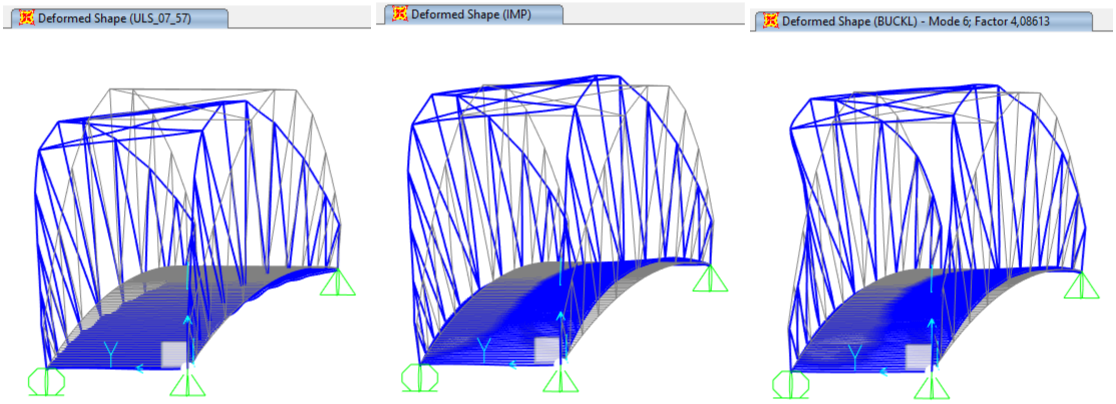
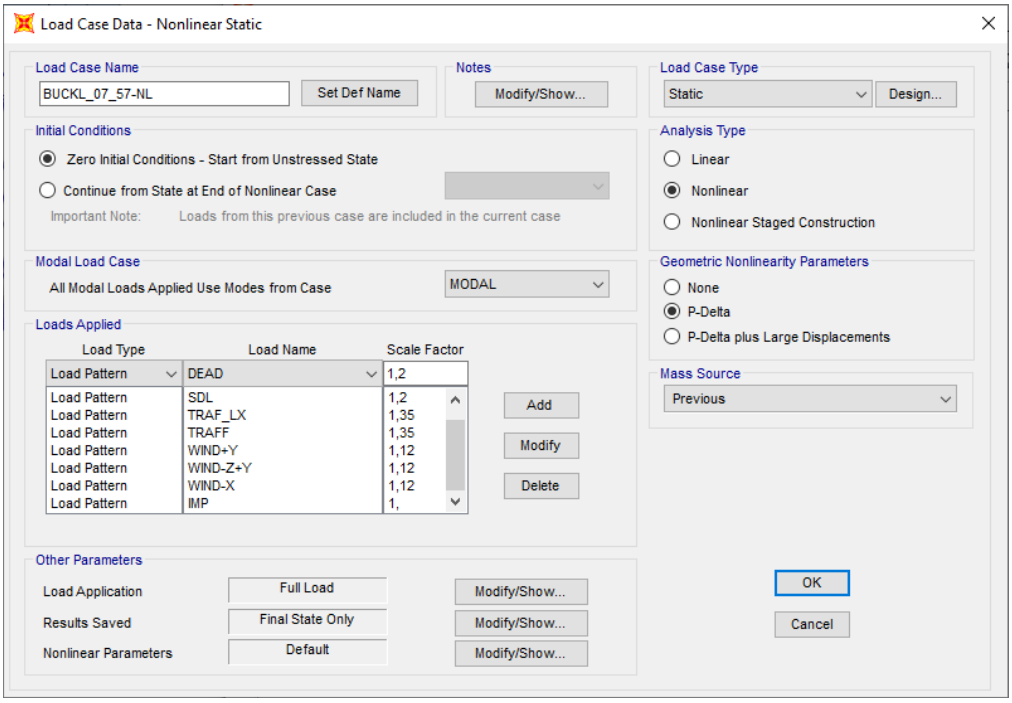


Figure 12-22 Most unfavorable load combination for upper chord, imperfection loadcase and first global buckling mode.

Since second order effects are nonlinear, the combinations are actually defined as a load cases.



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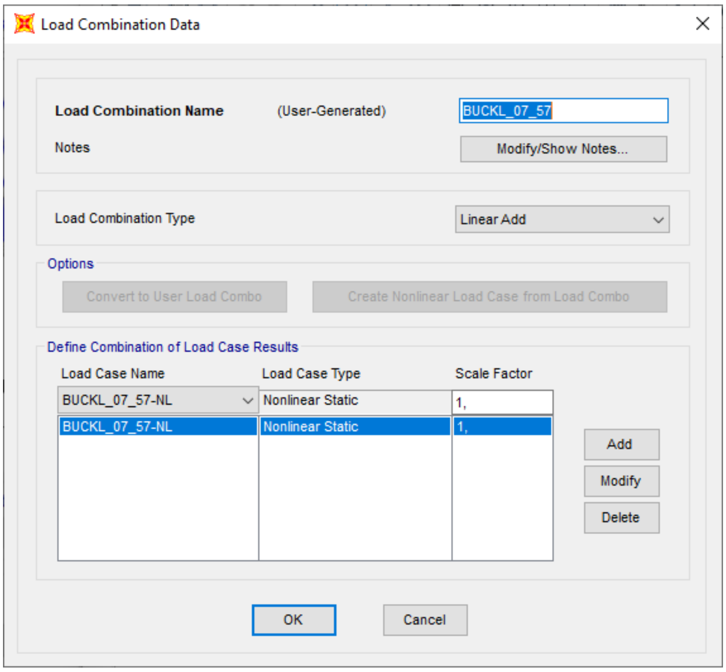

Figure 12-23 Example of a nonlinear load combination definition.

13 VEDLEGG 4 – GLOBAL OPPFØRSEL

13.1 GLOBAL STABILITY

We performed a global buckling analysis considering the combination that gives the highest compression forces on the upper chord.

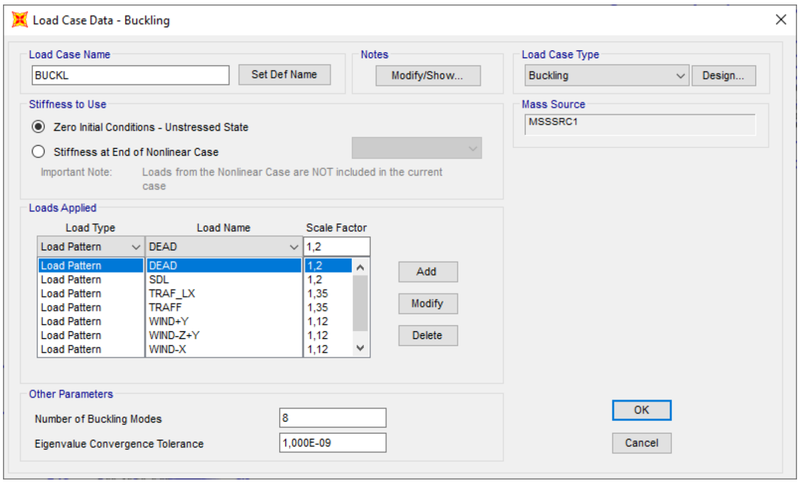

Figure 13-1 Buckling load case definition

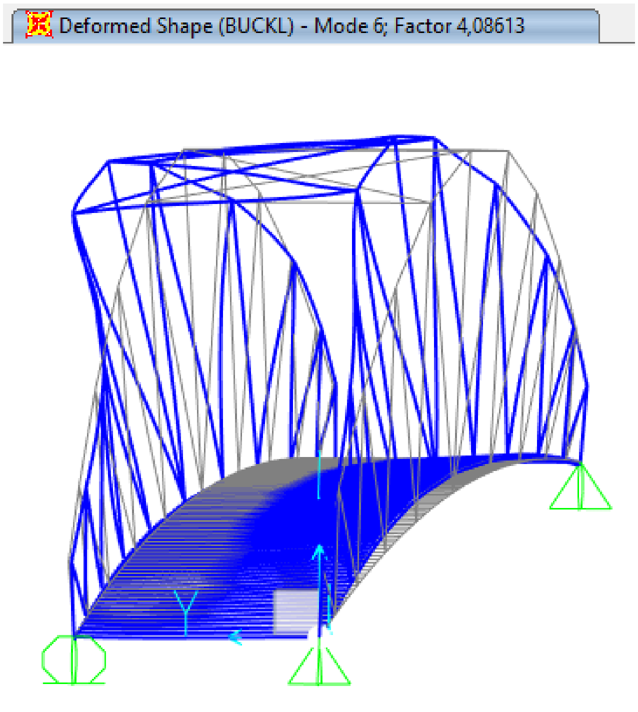

Figure 13-2 First relevant buckling mode

The amplification value is larger than 3 but close. We have performed nonlinear analyses with the worst load combinations.

13.2 MODAL ANALYSIS AND VIBRATION

We carried out a model analysis to analyze the vibration modes of the bridge.

The first mode has a period of less than 2s, so dynamic wind effects can be disregarded. Be aware that this mode is excited by upper lateral loading (wind).

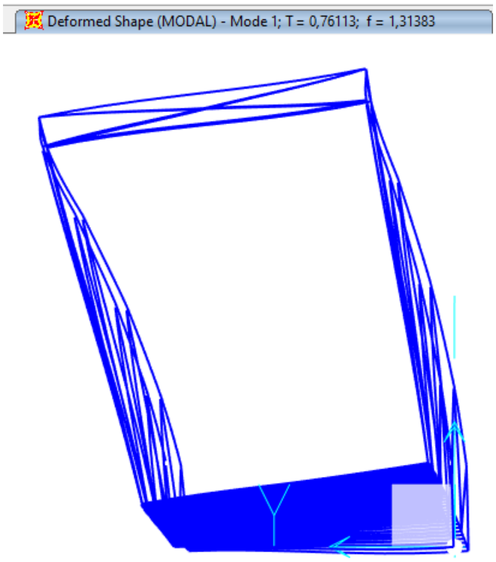

Figure 13-3 First vibration mode (torsion)

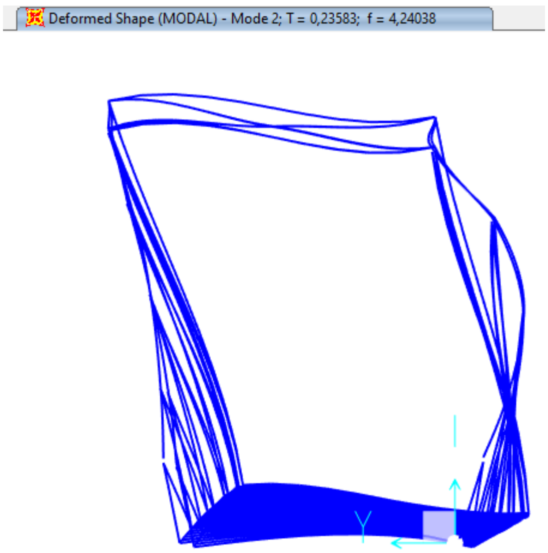

Figure 13-4 Second vibration mode (sway)

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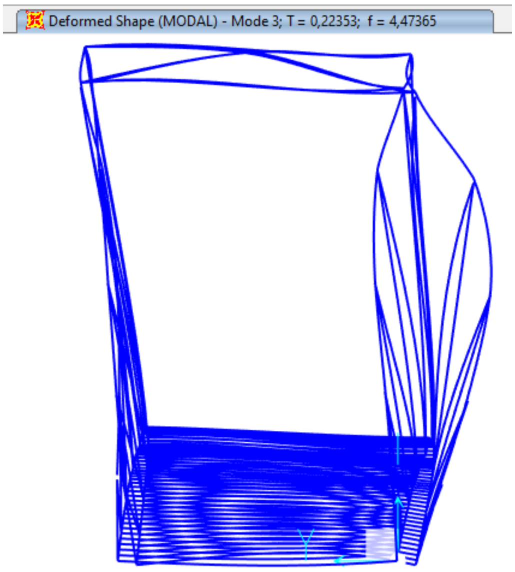


Figure 13-5 Third vibration mode (vertical)

As there were several modes, a time history analysis was carried out. The highest acceleration values were found in the lower node in the middle and are presented below.

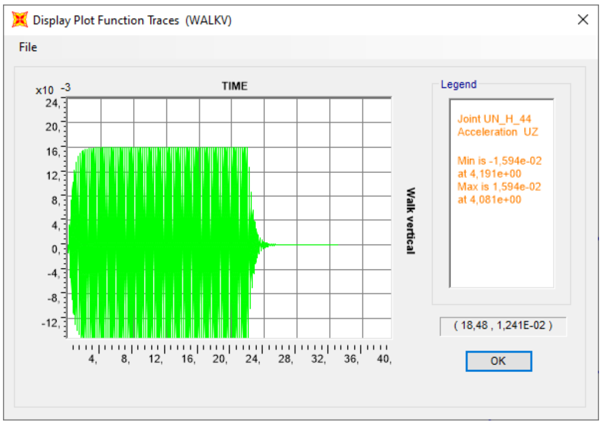


Figure 13-6 Vertical accelerations in service.

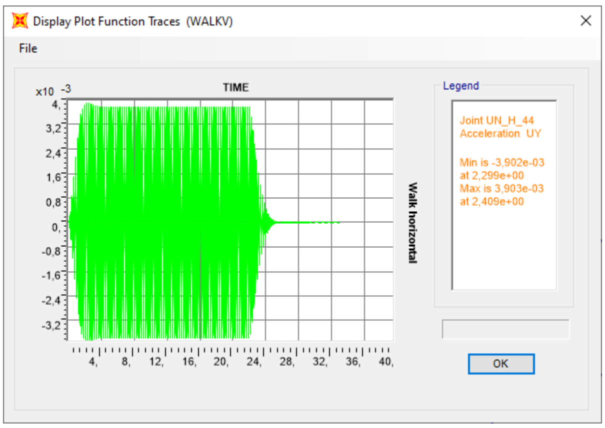


Figure 13-7 Horizontal accelerations in service.

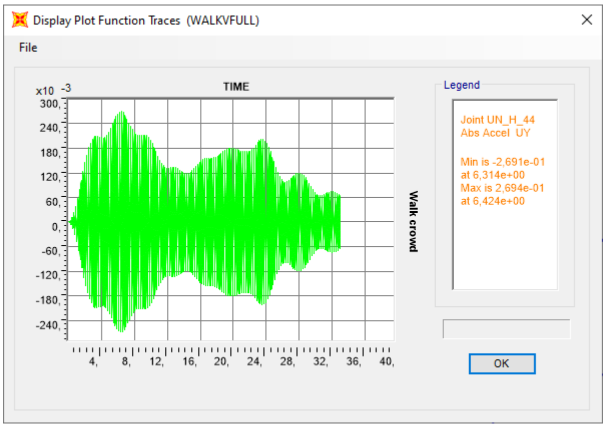


Figure 13-8 Horizontal accelerations with a crowd.

Limits

Reference 1991 -2 A2.4.3.2

$a_{vRd} := 0,7 \frac{m}{s^2}$ $a_{hRd} := 0,2 \frac{m}{s^2}$ $a_{crowdRd} := 0,4 \frac{m}{s^2}$

Calculated accelerations

Results from SAP2000 model.

$a_{vEd} := (0,015) \frac{m}{s^2}$ $a_{hEd} := (0,003) \frac{m}{s^2}$ $a_{crowdEd} := (0,27) \frac{m}{s^2}$

$\frac{a_{vEd}}{a_{vRd}} = 0,021$ $\frac{a_{hEd}}{a_{hRd}} = 0,015$ $\frac{a_{crowdEd}}{a_{crowdRd}} = 0,68$

14 VEDLEGG 5 – FEM RESULTATER

14.1 FORCES

This chapter shows the envelope of forces at main girders and piers for any combination.

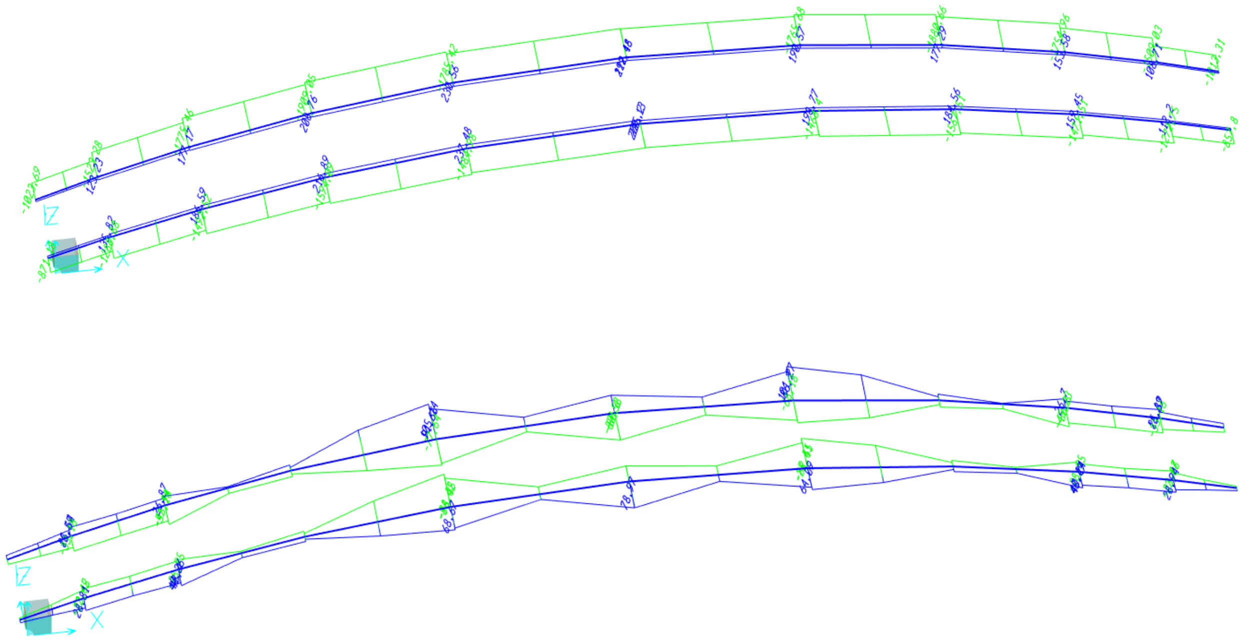
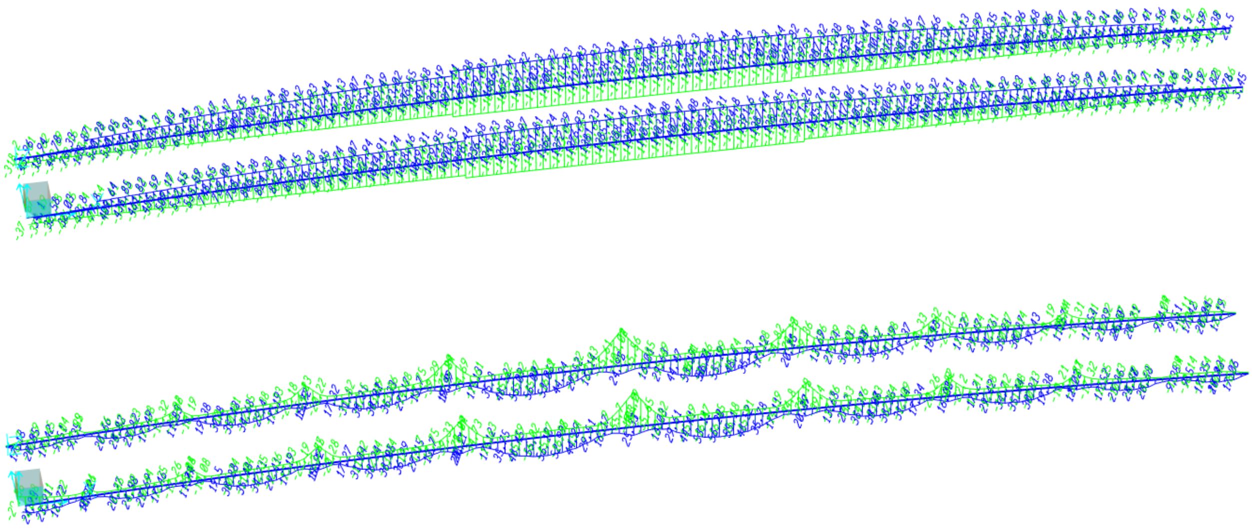


Figure 14-1 ULS axial forces and moments on upper chord



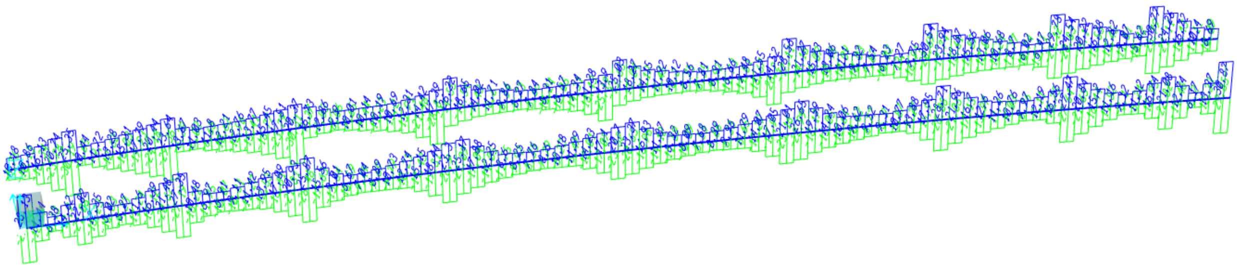


Figure 14-2 ULS axial forces, main moments and torsion on lower chord

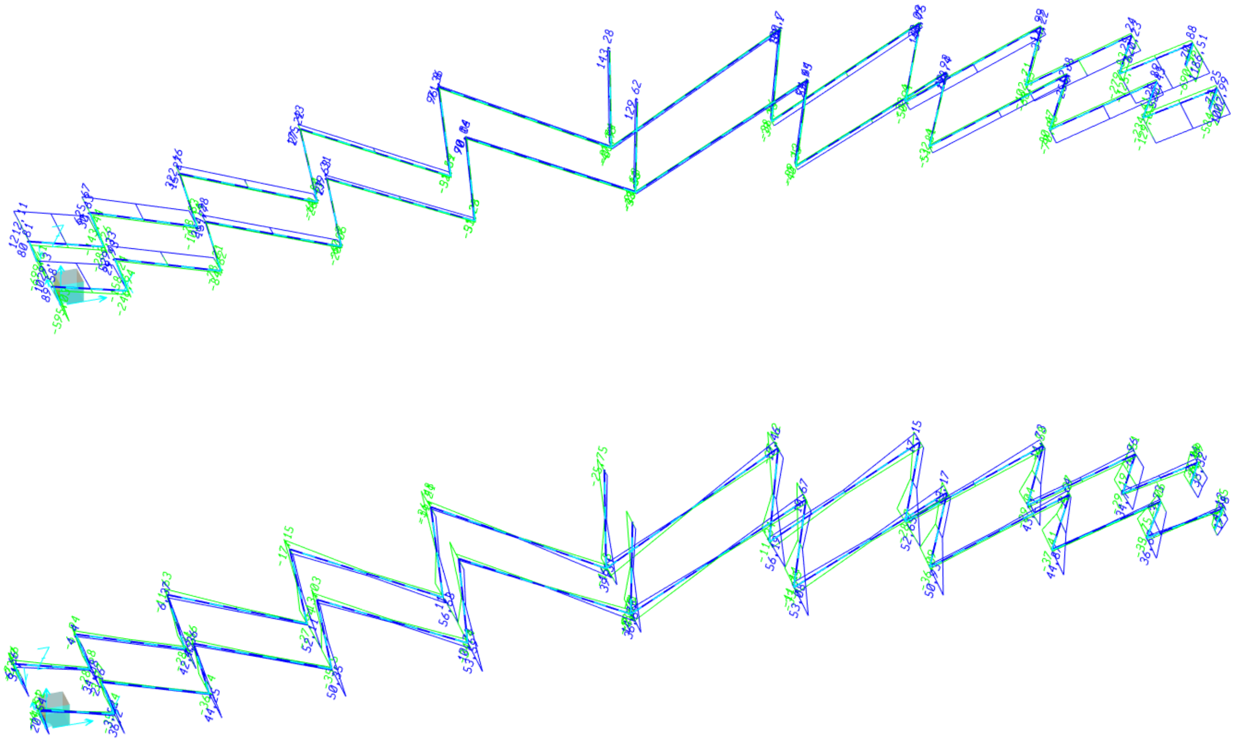
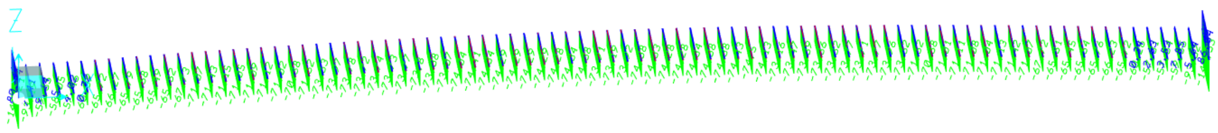


Figure 14-3 ULS axial forces and moments on main bracing



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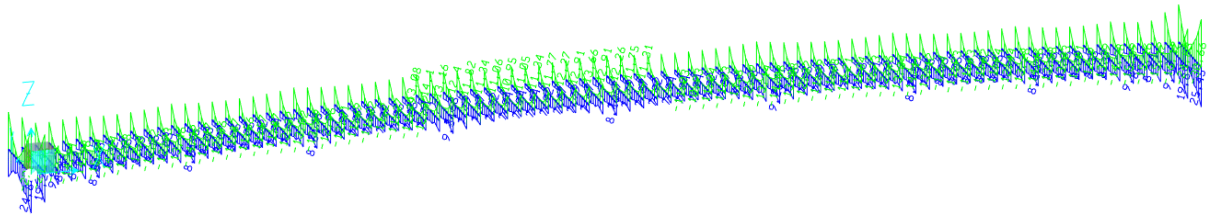
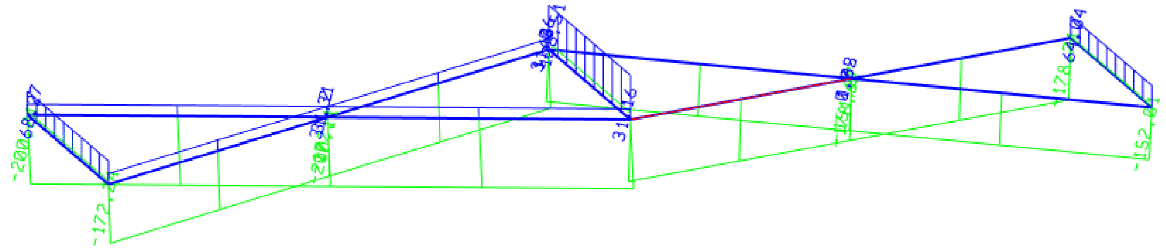


Figure 14-4 ULS axial forces and moments on lower horizontal beams



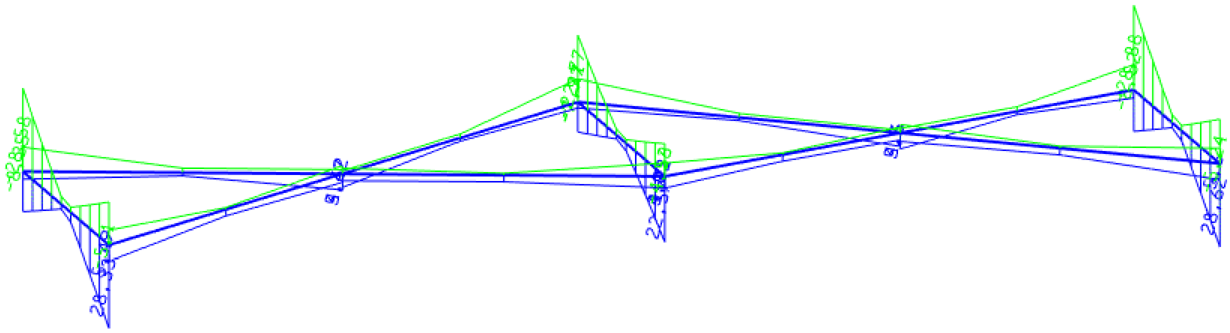


Figure 14-5 ULS axial forces and moments on wind bracing

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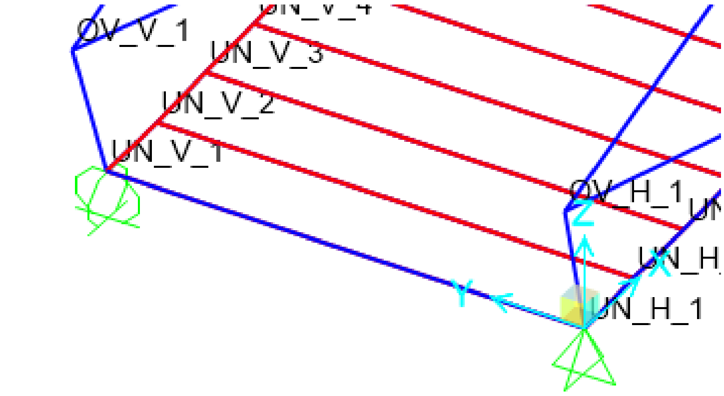
14.2 REACTIONS

Reaction points are shown in the following figures and summarized in the table below



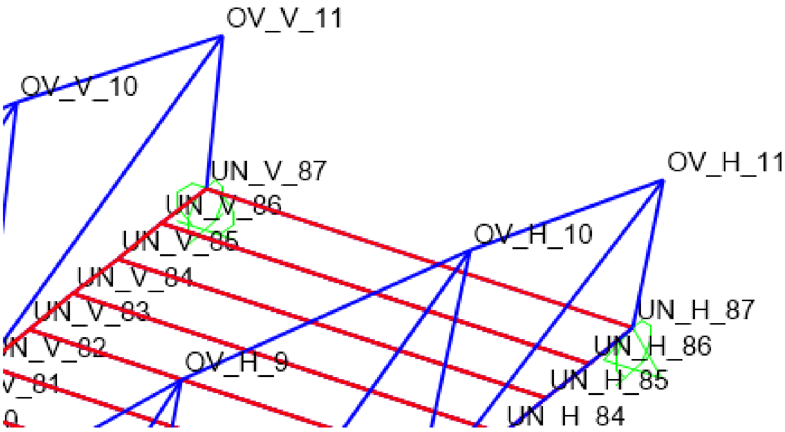


Figure 14-6 Node names.

TABLE: Joint Reactions						
Joint	OutputCase	CaseType	StepType	F1	F2	F3
Text	Text	Text	Text	KN	KN	KN
UN_H_1	DEAD	LinStatic		0	0	107
UN_H_1	SDL	LinStatic		0	0	34
UN_H_1	TEMP_HOT	LinStatic		0	0	0
UN_H_1	TEMP_COLD	LinStatic		0	0	0
UN_H_1	TEMPGRAD	LinStatic		0	0	0
UN_H_1	BEARFR	LinStatic		8	8	0
UN_H_1	ULS	Combination	Max	195	296	612
UN_H_1	ULS	Combination	Min	-107	-276	-98
UN_H_1	Vehicle	Combination	Max	-79	-3	27
UN_H_1	Vehicle	Combination	Min	-79	-6	-3

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UN_H_1	SLS	Combination	Max	136	187	453
UN_H_1	SLS	Combination	Min	-79	-173	-9
UN_V_1	DEAD	LinStatic		0	0	107
UN_V_1	SDL	LinStatic		0	0	34
UN_V_1	TEMP_HOT	LinStatic		0	0	0
UN_V_1	TEMP_COLD	LinStatic		0	0	0
UN_V_1	TEMPGRAD	LinStatic		0	0	0
UN_V_1	BEARFR	LinStatic		0	0	0
UN_V_1	ULS	Combination	Max	0	0	607
UN_V_1	ULS	Combination	Min	0	0	-100
UN_V_1	Vehicle	Combination	Max	0	0	31
UN_V_1	Vehicle	Combination	Min	0	0	1
UN_V_1	SLS	Combination	Max	0	0	451
UN_V_1	SLS	Combination	Min	0	0	-10
UN_H_87	DEAD	LinStatic		0	0	107
UN_H_87	SDL	LinStatic		0	0	34
UN_H_87	TEMP_HOT	LinStatic		0	0	0
UN_H_87	TEMP_COLD	LinStatic		0	0	0
UN_H_87	TEMPGRAD	LinStatic		0	0	0
UN_H_87	BEARFR	LinStatic		0	8	0
UN_H_87	ULS	Combination	Max	0	307	603
UN_H_87	ULS	Combination	Min	0	-298	-94
UN_H_87	Vehicle	Combination	Max	0	3	30
UN_H_87	Vehicle	Combination	Min	0	-1	1
UN_H_87	SLS	Combination	Max	0	194	448
UN_H_87	SLS	Combination	Min	0	-187	-6
UN_V_87	DEAD	LinStatic		0	0	107
UN_V_87	SDL	LinStatic		0	0	34
UN_V_87	TEMP_HOT	LinStatic		0	0	0
UN_V_87	TEMP_COLD	LinStatic		0	0	0
UN_V_87	TEMPGRAD	LinStatic		0	0	0
UN_V_87	BEARFR	LinStatic		0	0	0
UN_V_87	ULS	Combination	Max	0	0	604
UN_V_87	ULS	Combination	Min	0	0	-91
UN_V_87	Vehicle	Combination	Max	0	0	29
UN_V_87	Vehicle	Combination	Min	0	0	0
UN_V_87	SLS	Combination	Max	0	0	448
UN_V_87	SLS	Combination	Min	0	0	-4

15 VEDLEGG 6 – STEEL DESIGN VERIFICATIONS

15.1 FRAME DESIGN

Steel frames have been designed with SAP2000 internal code. The results have been verified, especially when it comes to buckling lengths.

15.1.1 Ultimate Limit State verifications

15.1.1.1 Classification of cross-sections

The section classification has been done considering all profiles welded. This is not taking into account the radius as NS-EN 1993-1-1 allows.

Section	c/t	Class
RHS 350x250x14	25	1
SHS 200x200x10	20	1
SHS 200x200x14	12	1
RHS 200x100x10	20	1
SHS 120x120x5	24	1
SHS 120x120x6	20	1

15.1.1.2 Bending and shear resistance

All steel sections have been designed to be class 1. Therefore local plate buckling is not a concern except at supports (handled as special detail). The joint plate buckling is taken into account with Eurocodes formulae.

Analyses are performed according to NS-EN 1993-1-1.

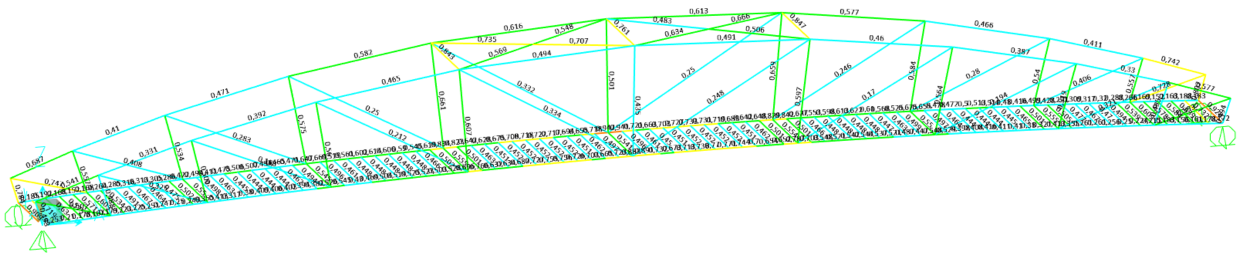

Figure 15-1 Bending and shear utilization in color codes. Red is over the limit.

The worst cases are shown below:

Revisjon	Dato	Utf.	Kontr.	Godkj.
04	30-01-2024	FA	FI	BO

15.1.1.3 Lower chord

The worst case for the lower chord is the nonlinear case with the wind forces blowing and making it compress.

```
Eurocode 3-2005 STEEL SECTION CHECK  (Flexural Details for Combo and Station)
Units  : KN, m, C

Frame  : UN_V_44    X Mid: 26,085     Combo: ULSBUCKL          Design Type: Beam
Length: 0,600       Y Mid: 3,543      Shape: UN_CEN_200x200x14Frame Type: DCH-MRF
Loc    : 0,600      Z Mid: 0,814      Class: Class 1           Rolled : No

Country=CEN Default                   Combination=Eq. 6.10                Reliability=Class 2
Interaction=Method 2 (Annex B)        MultiResponse=Envelopes             P-Delta Done? No
Consider Torsion? No

GammaM0=1,10        GammaM1=1,10      GammaM2=1,25
An/Ag=1,00          RLLF=1,000        PLLF=0,750         D/C Lim=1,000

Aeff=0,010          eNy=0,000         eNz=0,000
A=0,010             Iyy=6,040E-05     iyy=0,076          Wel,yy=6,040E-04    Weff,yy=6,040E-04
It=9,009E-05        Izz=6,040E-05     izz=0,076          Wel,zz=6,040E-04    Weff,zz=6,040E-04
Iw=0,000            Iyz=0,000         h=0,200            Wpl,yy=7,279E-04    Av,y=0,006
E=210000000,0       fy=355000,000     fu=510000,000      Wpl,zz=7,279E-04    Av,z=0,006


STRESS CHECK FORCES & MOMENTS
    Location              Ned       Med,yy       Med,zz       Ved,z        Ved,y          Ted
    0,600            1705,616     -101,948       -4,161     -17,104       -1,090       21,326

PMM DEMAND/CAPACITY RATIO   (Governing Equation EC3 6.2.1(7))
    D/C Ratio:    0,959 = 0,507 + 0,434 + 0,018     <         1,000         OK
                        = (NEd/NRd) + (My,Ed/My,Rd) + (Mz,Ed/Mz,Rd)         (EC3 6.2.1(7))

BASIC FACTORS
    Buckling Mode   K Factor    L Factor       Lcr/i
    Major (y-y)        1,000       1,000       7,879
    Major Braced       1,000       1,000       7,879
    Minor (z-z)        1,000       1,000       7,879
    Minor Braced       1,000       1,000       7,879
    LTB                1,000       1,000       7,879

AXIAL FORCE DESIGN
                          Ned        Nc,Rd        Nt,Rd
                        Force     Capacity     Capacity
    Axial            1705,616     3361,527     3361,527

                        Npl,Rd        Nu,Rd       Ncr,T      Ncr,TF         An/Ag
                      3361,527     3824,755  627414,679  347734,928         1,000

                 Curve    Alpha          Ncr   LambdaBar         Phi          Chi        Nb,Rd
    Major (y-y)     c     0,490   347734,928       0,103       0,482        1,000     3361,527
    MajorB(y-y)     c     0,490   347734,928       0,103       0,482        1,000     3361,527
    Minor (z-z)     c     0,490   347734,928       0,103       0,482        1,000     3361,527
    MinorB(z-z)     c     0,490   347734,928       0,103       0,482        1,000     3361,527
    Torsional TF    c     0,490   347734,928       0,103       0,482        1,000     3361,527
```

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15.1.1.4 Upper chord

Nonlinear combination is governing as expected.

```
Eurocode 3-2005 STEEL SECTION CHECK  (Flexural Details for Combo and Station)
Units  : KN, m, C

Frame : OV_V_5      X Mid: 21,954      Combo: BUCKL_07_58      Design Type: Brace
Length: 7,673       Y Mid: 3,789       Shape: OV_350x250x14    Frame Type: DCH-MRF
Loc   : 7,673       Z Mid: 5,510       Class: Class 1          Rolled : Yes

Country=CEN Default                    Combination=Eq. 6.10                  Reliability=Class 2
Interaction=Method 2 (Annex B)         MultiResponse=Envelopes               P-Delta Done? No
Consider Torsion? No

GammaM0=1,10       GammaM1=1,10       GammaM2=1,25
An/Ag=1,00         RLLF=1,000         PLLF=0,750         D/C Lim=0,950

Aeff=0,016         eNy=0,000          eNz=0,000
A=0,016            Iyy=2,790E-04      iyy=0,131          Wel,yy=0,002        Weff,yy=0,002
It=3,115E-04       Izz=1,641E-04      izz=0,101          Wel,zz=0,001        Weff,zz=0,001
Iw=0,000           Iyz=0,000          h=0,350            Wpl,yy=0,002        Av,y=0,007
E=210000000,0      fy=355000,000      fu=510000,000      Wpl,zz=0,002        Av,z=0,010


STRESS CHECK FORCES & MOMENTS
    Location             Ned        Med,yy       Med,zz        Ved,z        Ved,y          Ted
    7,673            -1821,409     -77,469       46,008       33,386      -22,687        6,723

PMM DEMAND/CAPACITY RATIO   (Governing Equation EC3 6.3.3(4)-6.62)
    D/C Ratio:   0,616 = 0,522 + 0,041 + 0,053   <      0,950          OK
                       = NEd/(Chi_z NRk/GammaM1) + kzy (My,Ed+NEd eNy)/(Chi_LT My,Rk/GammaM1)
                           + kzz (Mz,Ed+NEd eNz)/(Mz,Rk/GammaM1)        (EC3 6.3.3(4)-6.62)

BASIC FACTORS
    Buckling Mode    K Factor    L Factor      Lcr/i
    Major (y-y)         1,000       1,000     58,533
    Major Braced        1,000       1,000     58,533
    Minor (z-z)         1,000       1,000     76,323
    Minor Braced        1,000       1,000     76,323
    LTB                 1,000       1,000     76,323

AXIAL FORCE DESIGN
                         Ned        Nc,Rd        Nt,Rd
                       Force     Capacity     Capacity
    Axial          -1821,409     5238,974     5238,974

                      Npl,Rd        Nu,Rd        Ncr,T       Ncr,TF        An/Ag
                    5238,974     5960,919   921822,015     5775,954        1,000

                 Curve   Alpha          Ncr    LambdaBar          Phi          Chi        Nb,Rd
    Major (y-y)     a    0,210     9820,397        0,766        0,853        0,815     4267,264
    MajorB(y-y)     a    0,210     9820,397        0,766        0,853        0,815     4267,264
    Minor (z-z)     a    0,210     5775,954        0,999        1,083        0,666     3491,226
    MinorB(z-z)     a    0,210     5775,954        0,999        1,083        0,666     3491,226
    Torsional TF    a    0,210     5775,954        0,999        1,083        0,666     3491,226
```

15.1.1.5 Main bracing (diagonal members)

The small diagonals at the end are the most loaded ones.

```
Eurocode 3-2005 STEEL SECTION CHECK  (Flexural Details for Combo and Station)
Units  : KN, m, C

Frame : DI_V_2     X Mid: 1,332       Combo: BUCKL_07_60       Design Type: Brace
Length: 3,446      Y Mid: 3,533       Shape: DI_200x100x10     Frame Type: DCH-MRF
Loc   : 3,446      Z Mid: 0,627       Class: Class 1           Rolled : No

Country=CEN Default                   Combination=Eq. 6.10                   Reliability=Class 2
Interaction=Method 2 (Annex B)        MultiResponse=Envelopes                P-Delta Done? No
Consider Torsion? No

GammaM0=1,10       GammaM1=1,10       GammaM2=1,25
An/Ag=1,00         RLLF=1,000         PLLF=0,750         D/C Lim=0,950

Aeff=0,006         eNy=0,000          eNz=0,000
A=0,006            Iyy=2,779E-05      iyy=0,070          Wel,yy=2,779E-04    Weff,yy=2,779E-04
It=2,089E-05       Izz=8,987E-06      izz=0,040          Wel,zz=1,797E-04    Weff,zz=1,797E-04
Iw=0,000           Iyz=0,000          h=0,200            Wpl,yy=3,520E-04    Av,y=0,002
E=210000000,0      fy=355000,000      fu=510000,000      Wpl,zz=2,120E-04    Av,z=0,004


STRESS CHECK FORCES & MOMENTS
    Location               Ned       Med,yy      Med,zz       Ved,z       Ved,y         Ted
    3,446             1239,690      -6,908       0,000      -3,264     -11,315       0,797

PMM DEMAND/CAPACITY RATIO   (Governing Equation EC3 6.2.1(7))
    D/C Ratio:    0,747 = 0,686 + 0,061 + 0,000    <        0,950           OK
                        = (NEd/NRd) + (My,Ed/My,Rd) + (Mz,Ed/Mz,Rd)        (EC3 6.2.1(7))

BASIC FACTORS
    Buckling Mode    K Factor    L Factor      Lcr/i
    Major (y-y)         1,000       1,000     48,924
    Major Braced        1,000       1,000     48,924
    Minor (z-z)         1,000       1,000     86,028
    Minor Braced        1,000       1,000     86,028
    LTB                 1,000       1,000     86,028

AXIAL FORCE DESIGN
                           Ned       Nc,Rd       Nt,Rd
                         Force    Capacity    Capacity
    Axial             1239,690    1807,273    1807,273

                        Npl,Rd       Nu,Rd      Ncr,T      Ncr,TF       An/Ag
                      1807,273    2056,320  256900,625    1568,295       1,000

               Curve     Alpha         Ncr   LambdaBar         Phi         Chi       Nb,Rd
    Major (y-y)    c     0,490    4849,148       0,640       0,813       0,761    1375,797
    MajorB(y-y)    c     0,490    4849,148       0,640       0,813       0,761    1375,797
    Minor (z-z)    c     0,490    1568,295       1,126       1,361       0,471     850,606
    MinorB(z-z)    c     0,490    1568,295       1,126       1,361       0,471     850,606
    Torsional TF   c     0,490    1568,295       1,126       1,361       0,471     850,606
```

15.1.1.6 Main bracing (vertical members)

The worst combination is the one with the wind blowing directly which results in a very small axial force but a considerable moment.

```
Eurocode 3-2005 STEEL SECTION CHECK  (Flexural Details for Combo and Station)
Units  : KN, m, C

Frame : DI_H_21    X Mid: 51,737     Combo: ULSBUCKL         Design Type: Brace
Length: 1,127      Y Mid: -0,028     Shape: DI_VERT_200x200x1Frame Type: DCH-MRF
Loc   : 1,127      Z Mid: 0,538      Class: Class 1          Rolled : No

Country=CEN Default                  Combination=Eq. 6.10                Reliability=Class 2
Interaction=Method 2 (Annex B)       MultiResponse=Envelopes             P-Delta Done? No
Consider Torsion? No

GammaM0=1,10        GammaM1=1,10        GammaM2=1,25
An/Ag=1,00          RLLF=1,000          PLLF=0,750          D/C Lim=1,000

Aeff=0,008          eNy=0,000           eNz=0,000
A=0,008             Iyy=4,585E-05       iyy=0,078           Wel,yy=4,585E-04    Weff,yy=4,585E-04
It=6,859E-05        Izz=4,585E-05       izz=0,078           Wel,zz=4,585E-04    Weff,zz=4,585E-04
Iw=0,000            Iyz=0,000           h=0,200             Wpl,yy=5,420E-04    Av,y=0,004
E=210000000,0       fy=355000,000       fu=510000,000       Wpl,zz=5,420E-04    Av,z=0,004


STRESS CHECK FORCES & MOMENTS
    Location                 Ned        Med,yy       Med,zz         Ved,z        Ved,y          Ted
    1,127               -607,084        13,556        0,000       -11,087        0,316       -4,701

PMM DEMAND/CAPACITY RATIO    (Governing Equation EC3 6.2.1(7))
    D/C Ratio:    0,325 = 0,248 + 0,077 + 0,000     <          1,000           OK
                          = (NEd/NRd) + (My,Ed/My,Rd) + (Mz,Ed/Mz,Rd)         (EC3 6.2.1(7))

BASIC FACTORS
    Buckling Mode    K Factor     L Factor        Lcr/i
    Major (y-y)         1,000        1,000       14,505
    Major Braced        1,000        1,000       14,505
    Minor (z-z)         1,000        1,000       14,505
    Minor Braced        1,000        1,000       14,505
    LTB                 1,000        1,000       14,505

AXIAL FORCE DESIGN
                             Ned        Nc,Rd        Nt,Rd
                           Force     Capacity     Capacity
    Axial               -607,084     2452,727     2452,727

                          Npl,Rd        Nu,Rd        Ncr,T       Ncr,TF        An/Ag
                        2452,727     2790,720   459112,835    74872,150        1,000

               Curve     Alpha          Ncr    LambdaBar          Phi          Chi        Nb,Rd
    Major (y-y)     c     0,490    74872,150        0,190        0,516        1,000     2452,727
    MajorB(y-y)     c     0,490    74872,150        0,190        0,516        1,000     2452,727
    Minor (z-z)     c     0,490    74872,150        0,190        0,516        1,000     2452,727
    MinorB(z-z)     c     0,490    74872,150        0,190        0,516        1,000     2452,727
    Torsional TF    c     0,490    74872,150        0,190        0,516        1,000     2452,727
```

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15.1.1.7 Wind bracing

Imperfection loads force the upper horizontal members to try to keep the bridge shape creating important secondary moments.

```
Eurocode 3-2005 STEEL SECTION CHECK  (Flexural Details for Combo and Station)
Units  : KN, m, C

Frame : VA_1       X Mid: 18,122     Combo: BUCKL_07_58      Design Type: Beam
Length: 4,057      Y Mid: 1,750      Shape: VA_120x120x5     Frame Type: DCH-MRF
Loc   : 0,000      Z Mid: 5,316      Class: Class 1          Rolled : No

Country=CEN Default                  Combination=Eq. 6.10                Reliability=Class 2
Interaction=Method 2 (Annex B)       MultiResponse=Envelopes             P-Delta Done? No
Consider Torsion? No

GammaM0=1,10       GammaM1=1,10      GammaM2=1,25
An/Ag=1,00         RLLF=1,000        PLLF=0,750        D/C Lim=0,950

Aeff=0,002         eNy=0,000         eNz=0,000
A=0,002            Iyy=5,079E-06     iyy=0,047         Wel,yy=8,465E-05    Weff,yy=8,465E-05
It=7,604E-06       Izz=5,079E-06     izz=0,047         Wel,zz=8,465E-05    Weff,zz=8,465E-05
Iw=0,000           Iyz=0,000         h=0,120           Wpl,yy=9,925E-05    Av,y=0,001
E=210000000,0      fy=355000,000     fu=510000,000     Wpl,zz=9,925E-05    Av,z=0,001


STRESS CHECK FORCES & MOMENTS
    Location              Ned       Med,yy       Med,zz       Ved,z        Ved,y          Ted
    0,000               63,651     -24,259       0,000      -10,404        0,030        0,299

PMM DEMAND/CAPACITY RATIO   (Governing Equation EC3 6.2.1(7))
    D/C Ratio:    0,843 = 0,086 + 0,757 + 0,000    <          0,950         OK
                        = (NEd/NRd) + (My,Ed/My,Rd) + (Mz,Ed/Mz,Rd)        (EC3 6.2.1(7))

BASIC FACTORS
    Buckling Mode    K Factor    L Factor       Lcr/i
    Major (y-y)         1,000       1,000      86,338
    Major Braced        1,000       1,000      86,338
    Minor (z-z)         1,000       1,000      86,338
    Minor Braced        1,000       1,000      86,338
    LTB                 1,000       1,000      86,338

AXIAL FORCE DESIGN
                          Ned        Nc,Rd       Nt,Rd
                        Force     Capacity    Capacity
    Axial              63,651      742,273     742,273

                        Npl,Rd       Nu,Rd       Ncr,T       Ncr,TF        An/Ag
                       742,273     844,560  139064,042      639,510        1,000

                  Curve    Alpha         Ncr   LambdaBar          Phi          Chi        Nb,Rd
    Major (y-y)     c      0,490     639,510       1,130        1,366        0,469      347,801
    MajorB(y-y)     c      0,490     639,510       1,130        1,366        0,469      347,801
    Minor (z-z)     c      0,490     639,510       1,130        1,366        0,469      347,801
    MinorB(z-z)     c      0,490     639,510       1,130        1,366        0,469      347,801
    Torsional TF    c      0,490     639,510       1,130        1,366        0,469      347,801
```

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15.1.1.8 Horizontal floor beams

Design is governed by service vehicle as expected.

```
Eurocode 3-2005 STEEL SECTION CHECK  (Flexural Details for Combo and Station)
Units  : KN, m, C

Frame : HO_87      X Mid: 51,571      Combo: ULS_07_150       Design Type: Beam
Length: 3,500      Y Mid: 1,750       Shape: HO_END_120X120X5 Frame Type: DCH-MRF
Loc   : 0,000      Z Mid: 0,000       Class: Class 1          Rolled : No

Country=CEN Default                   Combination=Eq. 6.10                 Reliability=Class 2
Interaction=Method 2 (Annex B)        MultiResponse=Envelopes              P-Delta Done? No
Consider Torsion? No

GammaM0=1,10       GammaM1=1,10       GammaM2=1,25
An/Ag=1,00         RLLF=1,000         PLLF=0,750         D/C Lim=0,950

Aeff=0,002         eNy=0,000          eNz=0,000
A=0,002            Iyy=5,079E-06      iyy=0,047          Wel,yy=8,465E-05    Weff,yy=8,465E-05
It=7,604E-06       Izz=5,079E-06      izz=0,047          Wel,zz=8,465E-05    Weff,zz=8,465E-05
Iw=0,000           Iyz=0,000          h=0,120            Wpl,yy=9,925E-05    Av,y=0,001
E=210000000,0      fy=355000,000      fu=510000,000      Wpl,zz=9,925E-05    Av,z=0,001


STRESS CHECK FORCES & MOMENTS
    Location             Ned        Med,yy       Med,zz        Ved,z        Ved,y          Ted
    0,000           -105,030       -24,595       -0,607      -28,495       -0,397        1,005

PMM DEMAND/CAPACITY RATIO   (Governing Equation EC3 6.2.1(7))
    D/C Ratio:    0,928 = 0,141 + 0,768 + 0,019    <       0,950          OK
                        = (NEd/NRd) + (My,Ed/My,Rd) + (Mz,Ed/Mz,Rd)       (EC3 6.2.1(7))

BASIC FACTORS
    Buckling Mode    K Factor     L Factor        Lcr/i
    Major (y-y)         1,000        1,000       74,479
    Major Braced        1,000        1,000       74,479
    Minor (z-z)         1,000        1,000       74,479
    Minor Braced        1,000        1,000       74,479
    LTB                 1,000        1,000       74,479

AXIAL FORCE DESIGN
                         Ned         Nc,Rd        Nt,Rd
                       Force      Capacity     Capacity
    Axial           -105,030       742,273      742,273

                      Npl,Rd         Nu,Rd        Ncr,T       Ncr,TF        An/Ag
                     742,273       844,560   139064,042      859,361        1,000

               Curve    Alpha          Ncr    LambdaBar          Phi          Chi        Nb,Rd
    Major (y-y)    c    0,490      859,361        0,975        1,165        0,555      411,758
    MajorB(y-y)    c    0,490      859,361        0,975        1,165        0,555      411,758
    Minor (z-z)    c    0,490      859,361        0,975        1,165        0,555      411,758
    MinorB(z-z)    c    0,490      859,361        0,975        1,165        0,555      411,758
    Torsional TF   c    0,490      859,361        0,975        1,165        0,555      411,758
```

15.2 SERVICEABILITY LIMIT STATE VERIFICATIONS

15.2.1 Displacements

There are two criteria limiting the maximum displacement. The distances to the road and the train need to be kept. The distances to the train need to be kept in the frequent combination. The closest distance in the frequent combination is 1,64m:

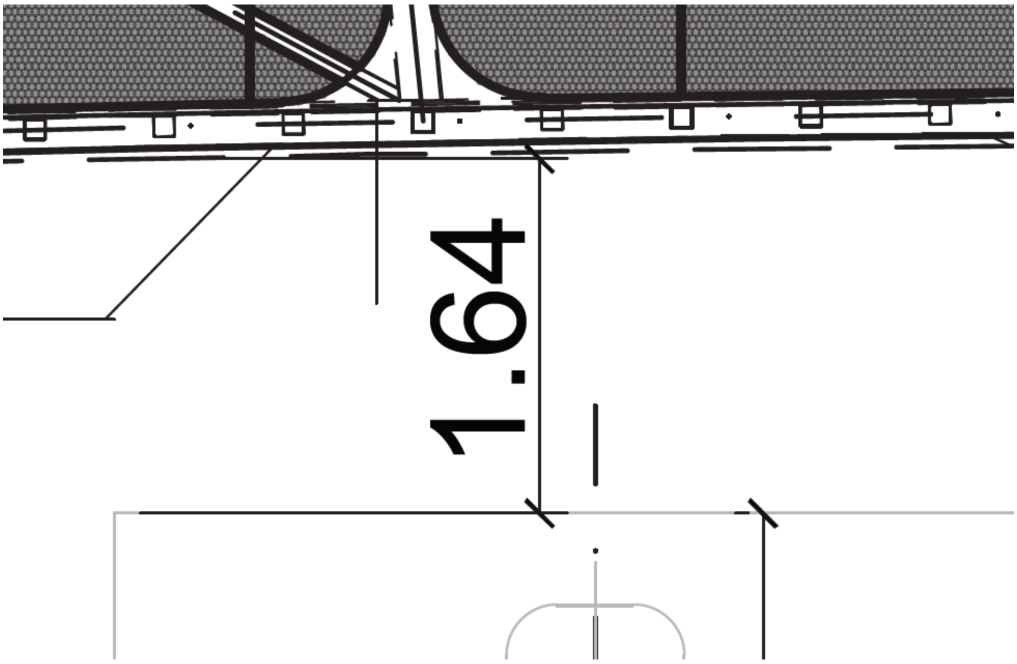

Figure 15-2: Minimum clearance for the almost permanent deformed shape

The other limit is for the almost permanent combination and is the bridge span divided by 350. This results in the following limit:

$$\frac{50,5\ \text{m}}{350} = 144,2857\ \text{mm}$$

The displacements even for the characteristic combination (worse than both frequent and almost permanent) are much smaller. The largest 3 displacements in mm are:

TABLE: Joint Displacements (mm)				
Joint	Output Case	U1	U2	U3
UN_H_38	SLSR_07_74	10,3	-36,9	-82,3
UN_H_50	SLSR_07_74	13,3	-37,0	-82,3
UN_H_38	SLSR_07_98	4,2	38,8	-82,1

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15.2.2 Stress limits

For the characteristic SLS combination of actions, the criteria given in cl. 7.2.2(5), NS-EN 1994-2 which refers to cl. 7.3, NS-EN 1993-2, for the normal and shear stresses in the structural steel should be verified:

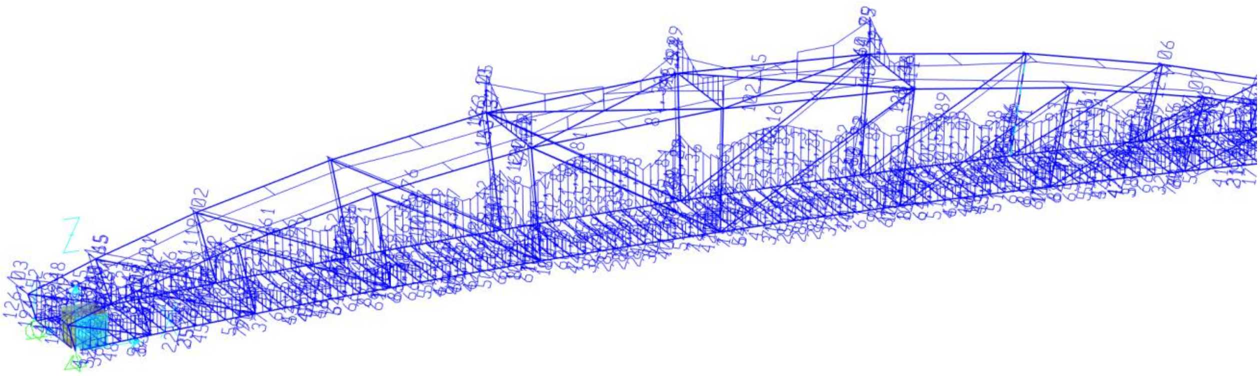
$$\sigma_{Ed,ser} \leq \frac{f_y}{\gamma_{M,ser}}$$

$$\tau_{Ed,ser} \leq \frac{f_y}{\sqrt{3}.\gamma_{M,ser}}$$

$$\sqrt{\sigma_{Ed,ser}{}^2 + 3\tau_{Ed,ser}{}^2} \leq \frac{f_y}{\gamma_{M,ser}}$$

Bending stresses are obtained from SAP2000 directly. This can result in conservative estimates as SAP2000 assumes that both the maximum longitudinal, vertical shear and horizontal shear tension happen simultaneously.

For the worst case, stresses are below 266MPa, comfortably below the limit.



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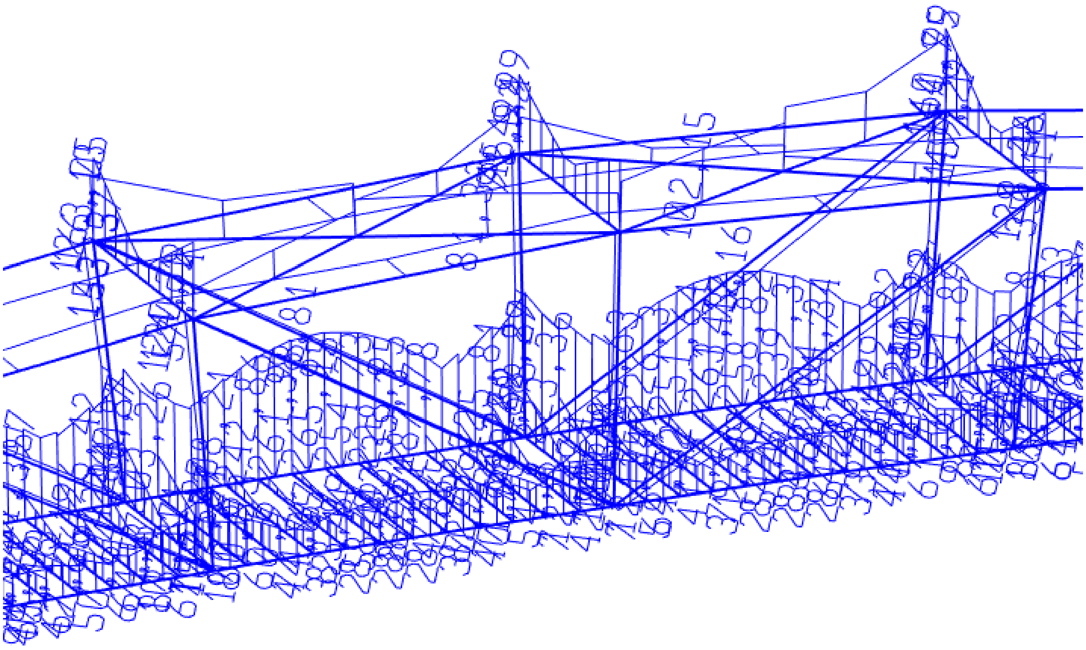

Figure 15-3: Maximum Von Mises stresses for characteristic SLS

15.3 JOINT DESIGN

Joints have been designed with hand calculations according to the criteria in NS-EN 1993-1-8.

All joints show enough capacity.

Placement	Joint type	Design criterion	Actual value	Resisted value
Main truss lower	K joint overlap	Axial force	631 kN	1263 kN
Main truss lower	T joint	M out of plane	3,7	76
Main truss upper	K joint overlap	Axial force	1214 kN	1320 kN
Main truss upper	T joint	M out of plane	3,6	49
Lower horizontals	T joint	M out of plane	29,5 kN·m	47,1 kN·m
Upper wind truss (horizontal)	K joint gap	Axial force	118 kN	888 kN
Upper wind truss (horizontal	T joint	M out of plane	28,3	40,6
Upper wind truss (diagonal)	K joint gap	Axial force	191 kN	1776 kN
Upper wind truss (diagonal)	T joint	M out of plane	8,3	40,6

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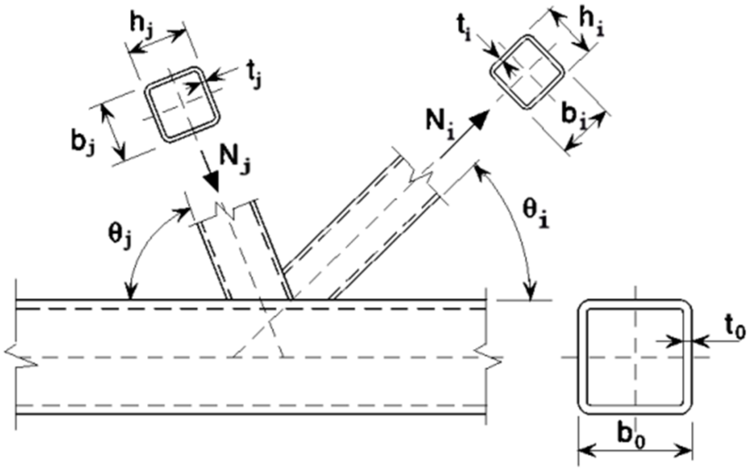
Lower chord central part overlap

According to NS-EN 1993-1-8 Table 7.10



$f_y := 355\ MPa$

$\gamma_{M1} := 1,1$

$\gamma_{M5} := 1,0$

$b_0 := 200\ mm$ $\quad b_1 := 200\ mm$ $\quad b_2 := 200\ mm$ $\quad \lambda_{ov} := 25\ \%$

$h_0 := 200\ mm$ $\quad h_1 := 100\ mm$ $\quad h_2 := 100\ mm$

$t_0 := 14\ mm$ $\quad t_1 := 10\ mm$ $\quad t_2 := 10\ mm$

$\theta_1 := 90\ deg$ $\quad \theta_2 := 30\ deg$

$$b_{eff} := \min\left(\left[\frac{10}{\frac{b_0}{t_0}} \cdot \frac{t_0}{t_2} \cdot b_2 \quad b_2\right]\right) = 0,196\ m$$

$$b_{e,ov} := \min\left(\left[\frac{10}{\frac{b_1}{t_1}} \cdot \frac{t_1}{t_2} \cdot b_2 \quad b_2\right]\right) = 0,1\ m$$

$$N_{iRd} := f_y \cdot t_2 \cdot \left(b_{eff} + b_{e,ov} + 2 \cdot h_2 \cdot \min\left(\left[\frac{\lambda_{ov}}{50\ \%} \quad 1\right]\right) - 4 \cdot t_2\right) \cdot \frac{1}{\gamma_{M5}} = 1263,8\ kN$$

$N_{2,Ed} := 631\ kN$

$$\frac{N_{2,Ed}}{N_{iRd}} = 0,4993$$

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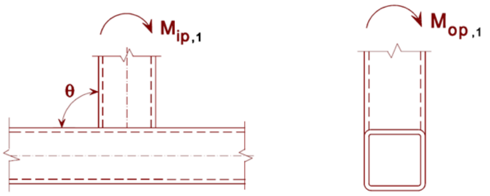
Out of plane moment



$b_0 := 200\ mm$ $\quad b_1 := 200\ mm$

$h_0 := 200\ mm$ $\quad h_1 := 100\ mm$

$t_0 := 14\ mm$ $\quad t_1 := 10\ mm$

$f_y := 355\ MPa$ $\quad N_{Ed,0} := 620\ kN$ $\quad \gamma_{M1} := 1,1$ $\quad \gamma_{M5} := 1,0$

$$b_{eff} := \frac{10}{\frac{b_0}{t_0}} \cdot \frac{t_0}{t_1} \cdot b_1 = 0,196\ m \qquad \beta := \frac{b_1 + h_1}{2 \cdot b_0} = 0,75$$

$$A_0 := 2 \cdot \left(h_0 + b_0\right) \cdot t_0 \qquad W_{pl,1} := \frac{b_1^{\ 2} \cdot h_1}{4} - \left(h_1 - 2 \cdot t_1\right) \cdot \left(\frac{b_1}{2} - t_1\right)^2$$

$$\sigma_{Ed} := \frac{-N_{Ed,0}}{A_0} \qquad n := \frac{\sigma_{Ed}}{f_y} = -0,1559$$

$$k_n := \text{if } n > 0: \min\left(\left[1,3 - \frac{0,4 \cdot n}{\beta} \quad 1\right]\right) \text{ else } 1 \quad = 1$$

$$M_{op,Rd,1} := \text{if } \beta \le 0,85: k_n \cdot f_y \cdot t_0^{\ 2} \cdot \left(\frac{h_1 \cdot (1+\beta)}{2 \cdot (1-\beta)} + \sqrt{\frac{2 \cdot b_0 \cdot b_1 \cdot (1+\beta)}{1-\beta}}\right) \cdot \frac{1}{\gamma_{M5}} \text{ else } f_y \cdot t_0 \cdot \left(b_0 - t_0\right) \cdot \left(h_1 + 5 \cdot t\right) \cdot \frac{1}{\gamma_{M5}}$$

$$M_{op,Rd,2} := 2 \cdot f_y \cdot t_0 \cdot \left(h_1 \cdot t_0 + \sqrt{b_0 \cdot h_0 \cdot t_0 \cdot \left(b_0 + h_0\right)}\right) \cdot \frac{1}{\gamma_{M5}}$$

$$M_{op,Rd,3} := \text{if } \beta \le 0,85: 10000\ MN\ m \text{ else } f_y \cdot \left(W_{pl,1} - 0,5 \cdot \left(1 - \frac{b_{eff}}{b_1}\right)^2 \cdot b_1^{\ 2} \cdot t_1\right) \cdot \frac{1}{\gamma_{M5}}$$

$$M_{op,Rd} := \min\left(\left[M_{op,Rd,1} \quad M_{op,Rd,2} \quad M_{op,Rd,3}\right]\right) = 76,4219\ kN\ m$$

$$M_{Ed} := 3,7\ kN\ m$$

$$\frac{M_{Ed}}{M_{op,Rd}} = 0,0484 \qquad \frac{N_{2,Ed}}{N_{iRd}} + \frac{M_{Ed}}{M_{op,Rd}} = 0,5477$$

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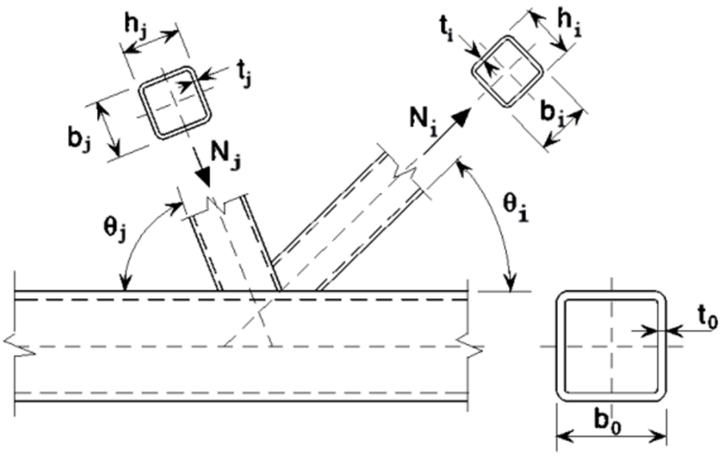
Upper chord overlap

According to NS-EN 1993-1-8 Table 7.10



$f_y := 355\ \text{MPa}$

$\gamma_{M1} := 1,1$

$\gamma_{M5} := 1,0$

$b_0 := 350\ \text{mm}$ $\quad b_1 := 200\ \text{mm}$ $\quad b_2 := 200\ \text{mm}$ $\quad \lambda_{ov} := 50\ \%$

$h_0 := 250\ \text{mm}$ $\quad h_1 := 200\ \text{mm}$ $\quad h_2 := 100\ \text{mm}$

$t_0 := 14\ \text{mm}$ $\quad t_1 := 10\ \text{mm}$ $\quad t_2 := 10\ \text{mm}$

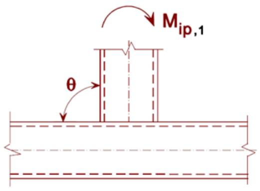
$\theta_1 := 90\ \text{deg}$ $\quad \theta_2 := 30\ \text{deg}$

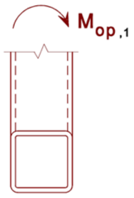
$$b_{eff} := \min\left(\left[\frac{10}{\frac{b_0}{t_0}} \cdot \frac{t_0}{t_2} \cdot b_2 \quad b_2\right]\right) = 0,112\ \text{m} \qquad b_{e,ov} := \min\left(\left[\frac{10}{\frac{b_1}{t_1}} \cdot \frac{t_1}{t_2} \cdot b_2 \quad b_2\right]\right) = 0,1\ \text{m}$$

$$N_{iRd} := f_y \cdot t_2 \cdot \left(b_{eff} + b_{e,ov} + 2 \cdot h_2 \cdot \min\left(\left[\frac{\lambda_{ov}}{50\ \%} \quad 1\right]\right) - 4 \cdot t_2\right) \cdot \frac{1}{\gamma_{M5}} = 1320,6\ \text{kN} \qquad N_{2,Ed} := 1214\ \text{kN}$$

$$\frac{N_{2,Ed}}{N_{iRd}} = 0,9193$$

Out of plane moment





$b_0 := 350\ \text{mm}$ $\quad b_1 := 200\ \text{mm}$

$h_0 := 250\ \text{mm}$ $\quad h_1 := 100\ \text{mm}$

$t_0 := 14\ \text{mm}$ $\quad t_1 := 10\ \text{mm}$

$f_y := 355\ \text{MPa}$ $\quad N_{Ed,0} := 1211\ \text{kN}$ $\quad \gamma_{M1} := 1,1$ $\quad \gamma_{M5} := 1,0$

$$b_{eff} := \frac{10}{\frac{b_0}{t_0}} \cdot \frac{t_0}{t_1} \cdot b_1 = 0,112\ \text{m} \qquad \beta := \frac{b_1 + h_1}{2 \cdot b_0} = 0,4286$$

$$A_0 := 2 \cdot (h_0 + b_0) \cdot t_0 \qquad W_{pl,1} := \frac{b_1^{\,2} \cdot h_1}{4} - (h_1 - 2 \cdot t_1) \cdot \left(\frac{b_1}{2} - t_1\right)^2$$

$$\sigma_{Ed} := \frac{-N_{Ed,0}}{A_0} \qquad n := \frac{\sigma_{Ed}}{f_y} = -0,2031$$

$$k_n := \text{if } n > 0 \;\; \min\left(\left[1,3 - \frac{0,4 \cdot n}{\beta} \;\; 1\right]\right) \;\; \text{else} \;\; 1 \quad = 1$$

$$M_{op,Rd,1} := \text{if } \beta \le 0,85 \;\; k_n \cdot f_y \cdot t_0^{\,2} \cdot \left(\frac{h_1 \cdot (1+\beta)}{2 \cdot (1-\beta)} + \sqrt{\frac{2 \cdot b_0 \cdot b_1 \cdot (1+\beta)}{1-\beta}}\right) \cdot \frac{1}{\gamma_{M5}} \;\; \text{else} \;\; f_y \cdot t_0 \cdot (b_0 - t_0) \cdot (h_1 + 5 \cdot t) \cdot \frac{1}{\gamma_{M5}}$$

$$M_{op,Rd,2} := 2 \cdot f_y \cdot t_0 \cdot \left(h_1 \cdot t_0 + \sqrt{b_0 \cdot h_0 \cdot t_0 \cdot (b_0 + h_0)}\right) \cdot \frac{1}{\gamma_{M5}}$$

$$M_{op,Rd,3} := \text{if } \beta \le 0,85 \;\; 10000\ \text{MN m} \;\; \text{else} \;\; f_y \cdot \left(W_{pl,1} - 0,5 \cdot \left(1 - \frac{b_{eff}}{b_1}\right)^2 \cdot b_1^{\,2} \cdot t_1\right) \cdot \frac{1}{\gamma_{M5}}$$

$$M_{op,Rd} := \min\left(\left[M_{op,Rd,1} \;\; M_{op,Rd,2} \;\; M_{op,Rd,3}\right]\right) = 49,8616\ \text{kN m}$$

$M_{Ed} := 3,6\ \text{kN m}$

$$\frac{M_{Ed}}{M_{op,Rd}} = 0,0722 \qquad \frac{N_{2,Ed}}{N_{iRd}} + \frac{M_{Ed}}{M_{op,Rd}} = 0,9915$$

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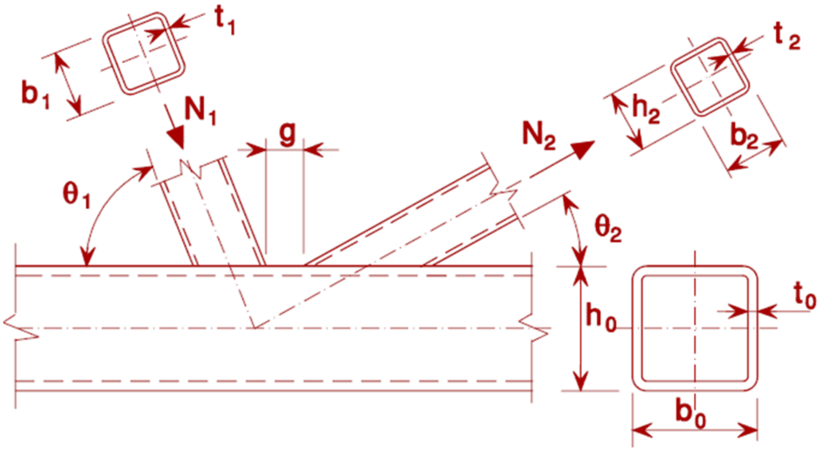
Wind bracing

According to NS-EN 1993-1-8 Table 7.10



$f_y := 355 \text{ MPa}$

$\gamma_{M1} := 1,1$

$\gamma_{M5} := 1,0$

$N_{Ed,0} := (-1181) \text{ kN}$

$b_0 := 250 \text{ mm}$ $\quad b_1 := 120 \text{ mm}$ $\quad b_2 := 120 \text{ mm}$

$h_0 := 350 \text{ mm}$ $\quad h_1 := 120 \text{ mm}$ $\quad h_2 := 120 \text{ mm}$

$t_0 := 14 \text{ mm}$ $\quad t_1 := 5 \text{ mm}$ $\quad t_2 := 5 \text{ mm}$

$\theta_1 := 90 \text{ deg}$ $\quad \theta_2 := 30 \text{ deg}$

$A_0 := 2 \cdot (h_0 + b_0) \cdot t_0$

$$\beta := \frac{b_1 + b_2 + h_1 + h_2}{4 \cdot b_0} = 0,48$$

Gap limits: $0,5 \cdot (1-\beta) \cdot b_0 = 65 \text{ mm}$

$1,5 \cdot (1-\beta) \cdot b_0 = 195 \text{ mm}$

$$\sigma_{Ed} := \frac{-N_{Ed,0}}{A_0}$$

$$n := \frac{\sigma_{Ed}}{f_y} = 0,198$$

$$\gamma := \frac{b_0}{2 \cdot t_0} = 8,9286$$

$$k_n := \text{if } n > 0 \;\; \min\left(\left[1,3 - \frac{0,4 \cdot n}{\beta} \;\; 1\right]\right) \;\text{else}\; 1 \;\; = 1$$

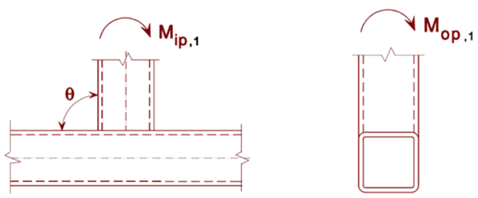
$$N_{1,Rd} := \frac{8,9 \cdot \gamma^{0,5} \cdot k_n \cdot f_y \cdot t_0^{\,2}}{\sin(\theta_1)} \cdot \left(\frac{b_1 + b_2}{2 \cdot b_0}\right) \cdot \frac{1}{\gamma_{M5}} = 888,1916 \text{ kN}$$

$N_{1,Ed} := 118 \text{ kN}$

$$N_{2,Rd} := \frac{8,9 \cdot \gamma^{0,5} \cdot k_n \cdot f_y \cdot t_0^{\,2}}{\sin(\theta_2)} \cdot \left(\frac{b_1 + b_2}{2 \cdot b_0}\right) \cdot \frac{1}{\gamma_{M5}} = 1776,3832 \text{ kN}$$

$N_{2,Ed} := 191 \text{ kN}$

Out of plane moment



$b_0 := 250\ \text{mm}$ $\quad b_1 := 120\ \text{mm}$

$h_0 := 350\ \text{mm}$ $\quad h_1 := 120\ \text{mm}$

$t_0 := 14\ \text{mm}$ $\quad t_1 := 5\ \text{mm}$

$f_y := 355\ \text{MPa}$ $\quad N_{Ed,0} := 134\ \text{kN}$ $\quad \gamma_{M1} := 1,1$ $\quad \gamma_{M5} := 1,0$

$$b_{eff} := \frac{10}{\frac{b_0}{t_0}} \cdot \frac{t_0}{t_1} \cdot b_1 = 0,1882\ \text{m} \qquad \beta := \frac{b_1 + h_1}{2 \cdot b_0} = 0,48$$

$$A_0 := 2 \cdot (h_0 + b_0) \cdot t_0 \qquad W_{pl,1} := \frac{b_1^{\ 2} \cdot h_1}{4} - (h_1 - 2 \cdot t_1) \cdot \left(\frac{b_1}{2} - t_1\right)^2$$

$$\sigma_{Ed} := \frac{-N_{Ed,0}}{A_0} \qquad n := \frac{\sigma_{Ed}}{f_y} = -0,0225$$

$$k_n := \text{if } n > 0 \quad \min\left(\left[1,3 - \frac{0,4 \cdot n}{\beta} \quad 1\right]\right) \quad \text{else} \quad 1 \qquad = 1$$

$$M_{op,Rd,1} := \text{if } \beta \le 0,85 \quad k_n \cdot f_y \cdot t_0^{\ 2} \cdot \left(\frac{h_1 \cdot (1+\beta)}{2 \cdot (1-\beta)} + \sqrt{\frac{2 \cdot b_0 \cdot b_1 \cdot (1+\beta)}{1-\beta}}\right) \cdot \frac{1}{\gamma_{M5}} \quad \text{else} \quad f_y \cdot t_0 \cdot (b_0 - t_0) \cdot (h_1 + 5 \cdot t) \cdot \frac{1}{\gamma_{M5}}$$

$$M_{op,Rd,2} := 2 \cdot f_y \cdot t_0 \cdot \left(h_1 \cdot t_0 + \sqrt{b_0 \cdot h_0 \cdot t_0 \cdot (b_0 + h_0)}\right) \cdot \frac{1}{\gamma_{M5}}$$

$$M_{op,Rd,3} := \text{if } \beta \le 0,85 \quad 10000\ \text{MN m} \quad \text{else} \quad f_y \cdot \left(W_{pl,1} - 0,5 \cdot \left(1 - \frac{b_{eff}}{b_1}\right)^2 \cdot b_1^{\ 2} \cdot t_1\right) \cdot \frac{1}{\gamma_{M5}}$$

$$M_{op,Rd} := \min\left(\left[M_{op,Rd,1} \quad M_{op,Rd,2} \quad M_{op,Rd,3}\right]\right) = 40,6355\ \text{kN m}$$

$$M_{Ed} := 28,3\ \text{kN m}$$

$$\frac{M_{Ed}}{M_{op,Rd}} = 0,6964 \qquad \frac{N_{2,Ed}}{N_{iRd}} + \frac{M_{Ed}}{M_{op,Rd}} = 0,8411$$

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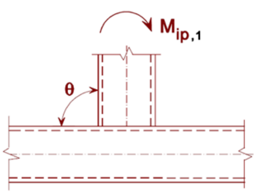
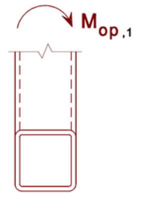
Floor joints





$b_0 := 200\ mm$ $\quad b_1 := 120\ mm$

$h_0 := 200\ mm$ $\quad h_1 := 120\ mm$

$t_0 := 14\ mm$ $\quad t_1 := 5\ mm$

$f_y := 355\ MPa$ $\quad N_{Ed,0} := 1000\ kN$ $\quad \gamma_{M1} := 1,1$ $\quad \gamma_{M5} := 1,0$

$$b_{eff} := \frac{10}{\frac{b_0}{t_0}} \cdot \frac{t_0}{t_1} \cdot b_1 = 0,2352\ m \qquad \beta := \frac{b_1 + h_1}{2 \cdot b_0} = 0,6$$

$$A_0 := 2 \cdot \left(h_0 + b_0\right) \cdot t_0 \qquad W_{pl,1} := \frac{b_1^{\ 2} \cdot h_1}{4} - \left(h_1 - 2 \cdot t_1\right) \cdot \left(\frac{b_1}{2} - t_1\right)^2$$

$$\sigma_{Ed} := \frac{-N_{Ed,0}}{A_0} \qquad n := \frac{\sigma_{Ed}}{f_y} = -0,2515$$

$$k_n := \text{if } n > 0 \;\; \min\left(\left[1,3 - \frac{0,4 \cdot n}{\beta} \;\; 1\right]\right) \;\; \text{else } 1 \;\; = 1$$

$$M_{op,Rd,1} := \text{if } \beta \le 0,85 \;\; k_n \cdot f_y \cdot t_0^{\ 2} \cdot \left(\frac{h_1 \cdot (1 + \beta)}{2 \cdot (1 - \beta)} + \sqrt{\frac{2 \cdot b_0 \cdot b_1 \cdot (1 + \beta)}{1 - \beta}}\right) \cdot \frac{1}{\gamma_{M5}} \;\; \text{else } f_y \cdot t_0 \cdot \left(b_0 - t_0\right) \cdot \left(h_1 + 5 \cdot t\right) \cdot \frac{1}{\gamma_{M5}}$$

$$M_{op,Rd,2} := 2 \cdot f_y \cdot t_0 \cdot \left(h_1 \cdot t_0 + \sqrt{b_0 \cdot h_0 \cdot t_0 \cdot \left(b_0 + h_0\right)}\right) \cdot \frac{1}{\gamma_{M5}}$$

$$M_{op,Rd,3} := \text{if } \beta \le 0,85 \;\; 10000\ MN\ m \;\; \text{else } f_y \cdot \left(W_{pl,1} - 0,5 \cdot \left(1 - \frac{b_{eff}}{b_1}\right)^2 \cdot b_1^{\ 2} \cdot t_1\right) \cdot \frac{1}{\gamma_{M5}}$$

$$M_{op,Rd} := \min\left(\left[M_{op,Rd,1} \;\; M_{op,Rd,2} \;\; M_{op,Rd,3}\right]\right) = 47,1876\ kN\ m$$

$$M_{Ed} := 28,3\ kN\ m$$

$$\frac{M_{Ed}}{M_{op,Rd}} = 0,5997$$

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15.4 BOTTOM STEEL PLATE

In order to verify the results from the FE model, hand calculations were performed first. The plate will be somewhere between simply supported and clamped, the formulae for that configuration considering the plate geometry are:

$$\frac{1,39 \cdot 1,35 \cdot P}{t^2} = 315,1934 \text{ MPa} \quad > fy > \quad \frac{1,008 \cdot P \cdot 1,35}{t^2} = 228,5719 \text{ MPa}$$

So the stresses from the SAP2000 model should be in that range.

In order to calculate the plate with a load in the middle, the plate of the global model was divided in 4 subplates, this was done previously to the meshing to guarantee that the force was being transferred properly.

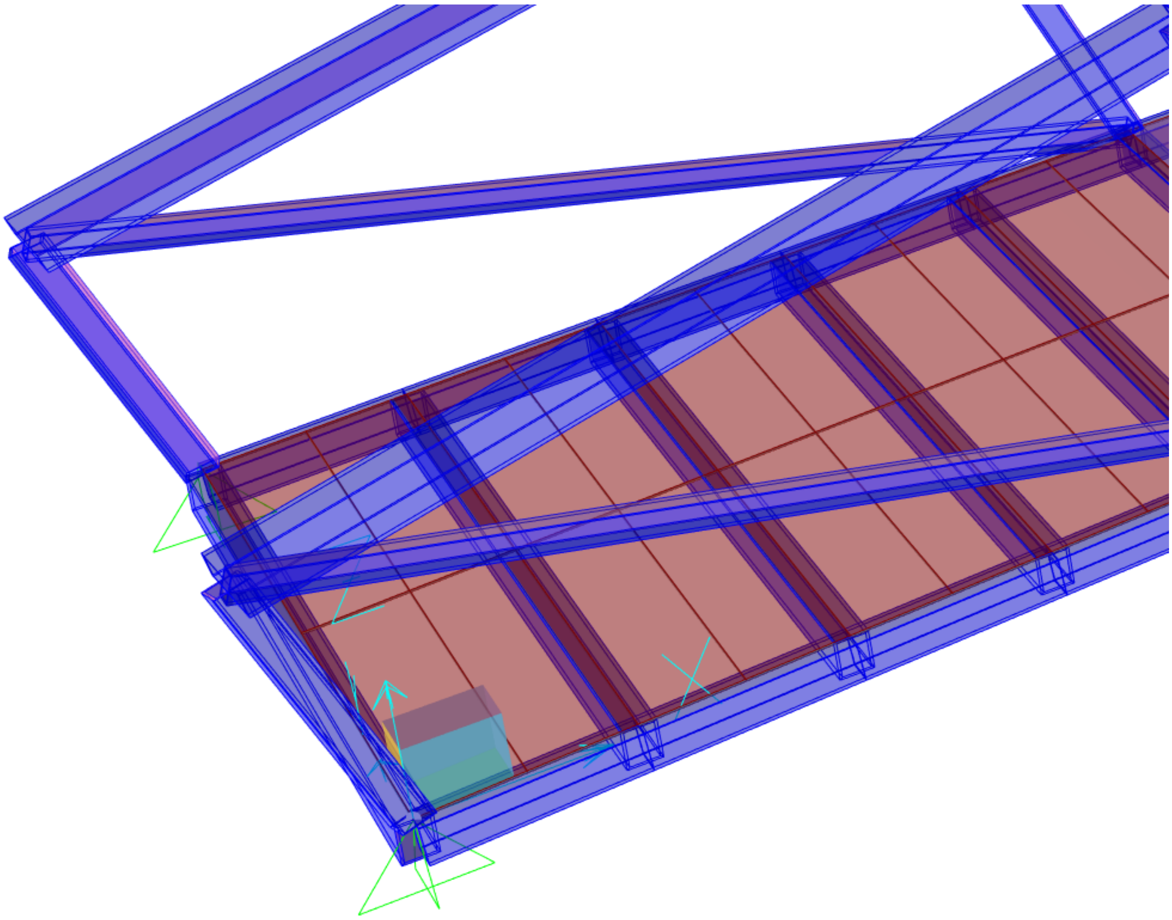

Figure 15-4: Plate analytical model.

The cases shown to be the most critical include the point load at three different locations:

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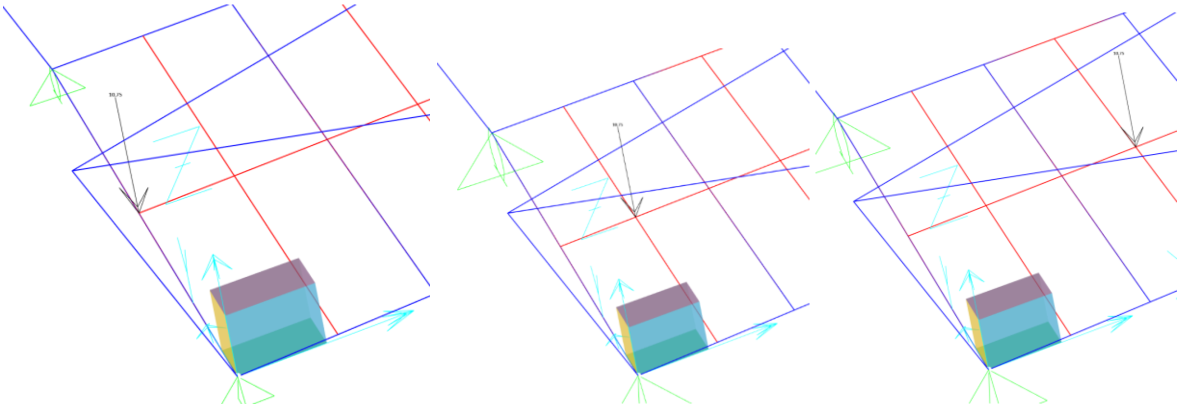

Figure 15-5: Plate force locations.

The most critical case was the one with the load centered in the first plate showing the following Von Mises stresses at ULS (1,35 dead loads+1,35 traffic)

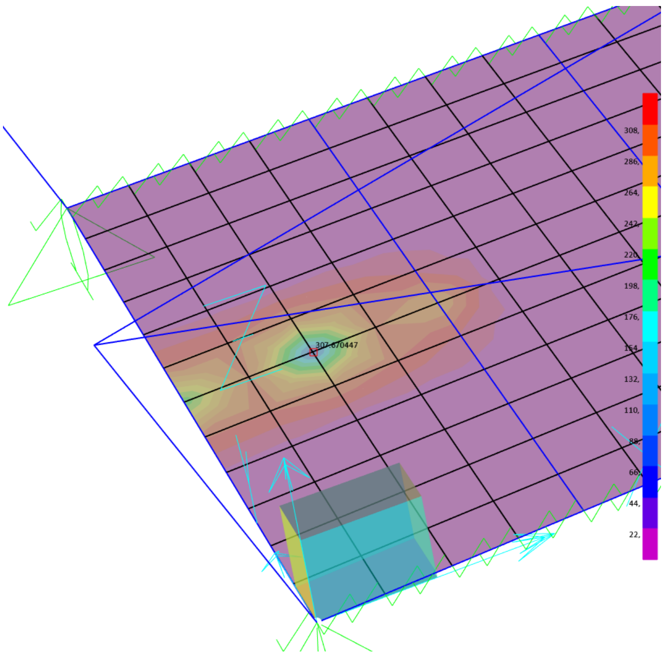

Figure 15-6: Largest Von Mises stresses at the plate.

The stresses of 307Mpa are within the range given in the hand calculations and considered valid. Even with the conservative approach of a point load the stresses are within limits.

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16 VEDLEGG 7 – CONCRETE DESIGN VERIFICATIONS

Abutments are designed to be symmetrical. The analysis presented here is therefore for both axes. The case with the highest load is always picked for design.

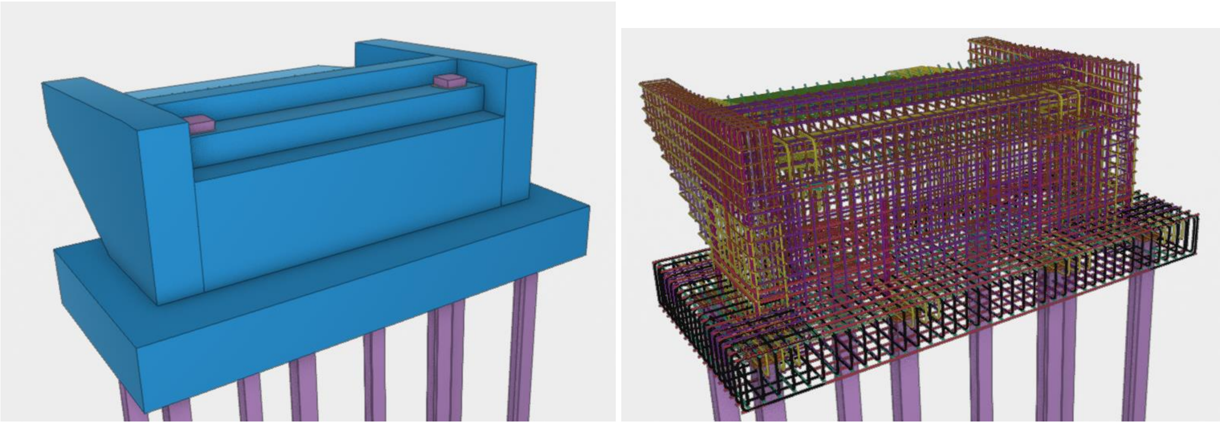

Figure 16-1: Abutment model with reinforcement.

16.1 ABUTMENT WALL

The minimum reinforcement is satisfied with 20mm rebars at 150mm distance.

Abutment wall

$$A_c := 1450\ \text{mm}$$

$$A_{sVmin} := 0{,}002 \cdot A_c = 29\ \frac{\text{cm}^2}{\text{m}}$$ NS EN 1992-1-1 NA.9.6.2

$$A_{sV} := 2 \cdot \frac{\frac{\pi \cdot (20\ \text{mm})^2}{4}}{150\ \text{mm}} = 41{,}8879\ \frac{\text{cm}^2}{\text{m}}$$

$$\frac{A_{sVmin}}{A_{sV}} = 0{,}6923$$

$$A_{sHmin} := 0{,}3 \cdot A_c \cdot \frac{f_{ctm}}{f_{yk}} = 33{,}0204\ \frac{\text{cm}^2}{\text{m}}$$ NS EN 1992-1-1 NA.9.6.3

$$A_{sH} := 2 \cdot \frac{\frac{\pi \cdot (20\ \text{mm})^2}{4}}{150\ \text{mm}} = 41{,}8879\ \frac{\text{cm}^2}{\text{m}}$$

$$\frac{A_{sHmin}}{A_{sH}} = 0{,}7883$$

16.1.1 Strut and tie model

Based on the loads applied and the expected stress distribution, we created a model with both abutments. The stiff connection between both sides of the abutment is expected to take horizontal loads. This is modelled as a stiff connection. We also placed specific reinforcement to take that force.

Earth pressure is 27kN and therefore an order of magnitude smaller. It can be neglected if we are not too close to 100% utilization.

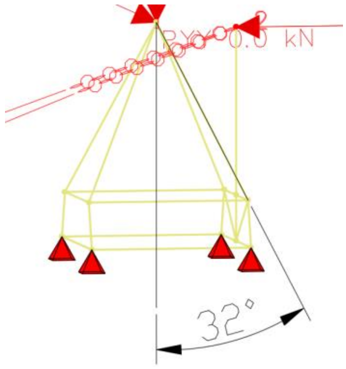

Figure 16-2: Strut angle larger than 30 degrees.

Several load cases were attempte with both models. Below we show one with both vertical, longitudinal and transversal forces and earth pressure.

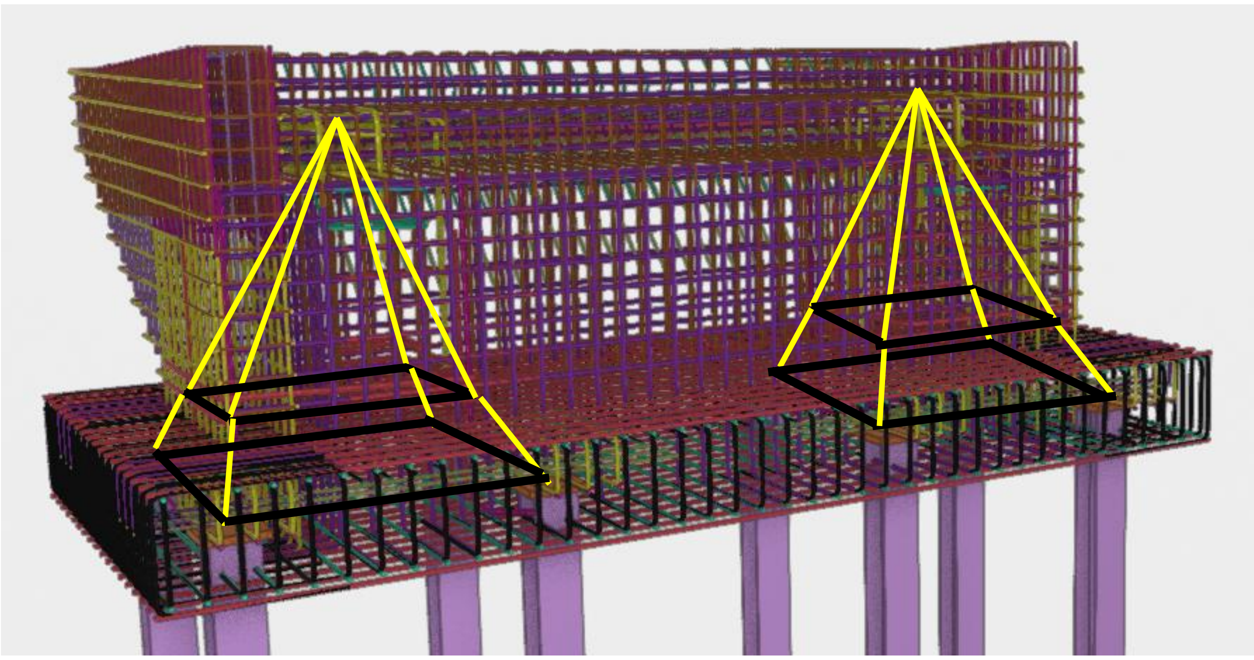

Figure 16-3: Strut and tie models with the actual reinforcement.

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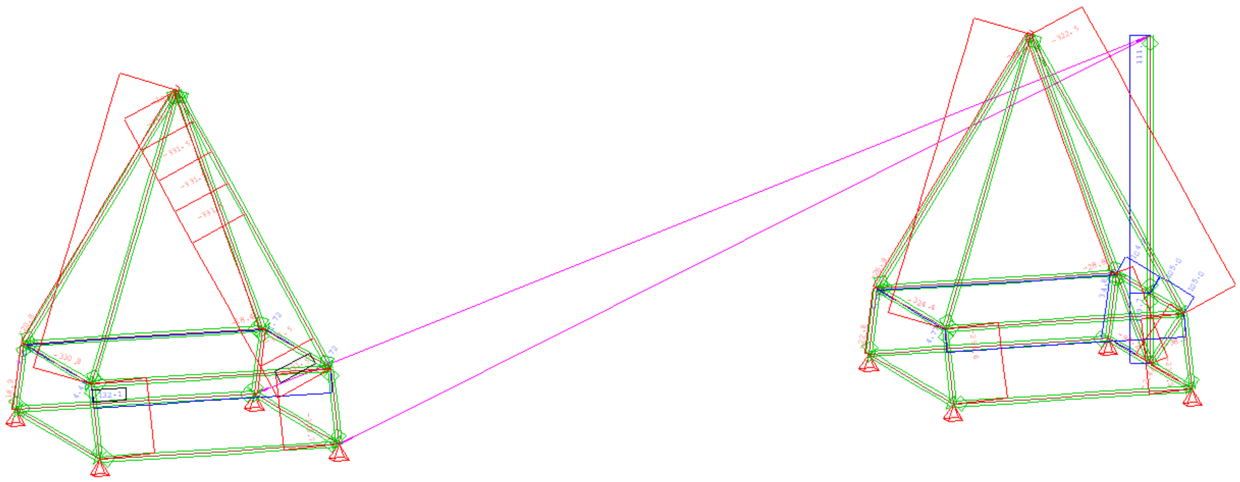

Figure 16-4: Strut and tie analytical model.

The following results were obtained.

Strut	Max load kN	Tie	Max load kN
Upper pyramid	331.5	Vertical	111.7
Lower vertical	303.7	Upper horizontal	132.1
Lower diagonal	94.2	Lower horizontal	0.2

The minimum area needs are:

Strut	Side (mm)	Tie	Rebar
Upper pyramid	120	Vertical	1Ø20
Lower vertical	110	Upper horizontal	2ø16
Lower diagonal	60	Lower horizontal	-

All the minimum strut dimensions are comfortably reached just considering the concrete cover (75mm to one side).

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16.1.2 Bearing local reinforcement

The bearing will be designed locally according to horizontal forces and splitting. Cracking stress is also to be considered when designing reinforcement to resist horizontal forces in serviceability limit state.

Input

$\gamma_c := 25 \frac{kN}{m^3}$ $\gamma_{soil} := 19 \frac{kN}{m^3}$ $f_{ck} := 45\ MPa$ $f_{yk} := 500\ MPa$

$f_{cd} := \frac{0,85 \cdot f_{ck}}{1,5}$ $f_{yd} := \frac{f_{yk}}{1,15}$ $\sigma_s := 200\ MPa$

$R_{ULS} := 600\ kN$ From FE model, rounded up

$R_{SLS} := 450\ kN$

$A_{up} := \frac{R_{ULS}}{f_{ck}}$ $D := \sqrt{A_{up}} = 115,4701\ mm$ Minimum possible support dimension (conservative)

$B := 550\ mm$ $A_{lo} := \frac{\pi \cdot B^2}{4}$

$B - D = 0,4345\ m$

Horizontal forces

$F_{HULS} := 300\ kN$ $F_{HSLS} := 200\ kN$

$A_{ULS} := \frac{F_{HULS}}{f_{yd}} = 6,9\ cm^2$ $A_{SLS} := \frac{F_{HSLS}}{\sigma_s} = 10\ cm^2$

$$A_s := 2 \cdot \left(\pi \cdot \frac{(16\ mm)^2}{4} \right) + 2 \cdot \left(\pi \cdot \frac{(25\ mm)^2}{4} \right) = 13,8387\ cm^2$$

$\frac{A_{ULS}}{A_s} = 0,4986$ $\frac{A_{SLS}}{A_s} = 0,7226$

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Spliting

NS EN 1992 -1 -1 6.7

$$A_{up} \cdot f_{cd} \cdot \sqrt{\frac{A_{1o}}{A_{up}}} = 1435,2157 \text{ kN}$$

$$3 \cdot f_{cd} \cdot A_{up} = 1020 \text{ kN}$$

$$F_{Rdu} := \min\left(\left[A_{up} \cdot f_{cd} \cdot \sqrt{\frac{A_{1o}}{A_{up}}} \quad 3 \cdot f_{cd} \cdot A_{up} \right]\right) = 1020 \text{ kN}$$

$$\frac{R_{ULS}}{F_{Rdu}} = 0,5882$$

Transverse tie NS EN 1992 -1 -1 6.5.3

$$T_{ULS} := \frac{1}{4} \cdot \frac{B - D}{B} \cdot R_{ULS} = 118,5082 \text{ kN}$$

$$T_{SLS} := \frac{1}{4} \cdot \frac{B - D}{B} \cdot R_{SLS} = 88,8811 \text{ kN}$$

$$A_{ULS} := \frac{T_{ULS}}{f_{yd}} = 2,7257 \text{ cm}^2$$

$$A_{SLS} := \frac{T_{SLS}}{\sigma_s} = 4,4441 \text{ cm}^2$$

$$A_s := 2 \cdot \left(\pi \cdot \frac{(20 \text{ mm})^2}{4} \right) = 6,2832 \text{ cm}^2$$

20 bars at 150, minimum 2 bars intersected

$$\frac{A_{ULS}}{A_s} = 0,4338$$

$$\frac{A_{SLS}}{A_s} = 0,7073$$

16.1.3 Jacking point local reinforcement

The jacking point is designed to take only vertical forces.

Input

$\gamma_c := 25 \frac{kN}{m^3}$ $\gamma_{soil} := 19 \frac{kN}{m^3}$ $f_{ck} := 45\ MPa$ $f_{yk} := 500\ MPa$

$f_{cd} := \frac{0,85 \cdot f_{ck}}{1,5}$ $f_{yd} := \frac{f_{yk}}{1,15}$

$R_{SLS} := 132\ kN + 72\ kN = 204\ kN$ $R_{ULS} := 1,35 \cdot 132\ kN + 1,6 \cdot 72\ kN = 293,4\ kN$

$A_{up} := \frac{R_{ULS}}{f_{ck}}$ $D := \sqrt{\frac{4 \cdot A_{up}}{\pi}} = 91,1127\ mm$ Minimum possible jack diameter (conservative)

$B := 400\ mm$ $A_{1o} := \frac{\pi \cdot B^2}{4}$ $B - D = 0,3089\ m$

Verification

NS EN 1992 -1 -1 6.7

$A_{up} \cdot f_{cd} \cdot \sqrt{\frac{A_{1o}}{A_{up}}} = 729,9093\ kN$ $3 \cdot f_{cd} \cdot A_{up} = 498,78\ kN$

$F_{Rdu} := \min\left(\left[A_{up} \cdot f_{cd} \cdot \sqrt{\frac{A_{1o}}{A_{up}}} \quad 3 \cdot f_{cd} \cdot A_{up}\right]\right) = 498,78\ kN$

$\frac{R_{ULS}}{F_{Rdu}} = 0,5882$

Transverse tie NS EN 1992 -1 -1 6.5.3

$T_{ULS} := \frac{1}{4} \cdot \frac{B - D}{B} \cdot R_{ULS} = 56,6422\ kN$ $T_{SLS} := \frac{1}{4} \cdot \frac{B - D}{B} \cdot R_{SLS} = 39,3831\ kN$

$A_{ULS} := \frac{T_{ULS}}{f_{yd}} = 1,3028\ cm^2$ $A_{SLS} := \frac{T_{SLS}}{f_{yd}} = 0,9058\ cm^2$

$A_s := 1 \cdot \left(\pi \cdot \frac{(16\ mm)^2}{4}\right) = 2,0106\ cm^2$ Ø16 bars at 150, minimum 2 bars intersected

$\frac{A_{ULS}}{A_s} = 0,6479$

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16.2 ABUTMENT WINGS

The minimum reinforcement on the wings is 16 rebars at 150mm:

$A_c := 500\ \text{mm}$ $\quad h := 500\ \text{mm}$

$\sigma_s := 200\ \text{MPa}$ $\quad \phi_{sp} := 32\ \text{mm}$ NS EN 1992-1-1 Table 7.2N

$$A_{sVmin} := 0{,}002 \cdot A_c = 10\ \frac{\text{cm}^2}{\text{m}}$$ NS EN 1992-1-1 NA.9.6.2

$$A_{sHmin} := 0{,}3 \cdot A_c \cdot \frac{f_{ctm}}{f_{yk}} = 11{,}3863\ \frac{\text{cm}^2}{\text{m}}$$ NS EN 1992-1-1 NA.9.6.3

$$d := h - c - \frac{\phi}{2} = 0{,}417\ \text{m}$$

$w_{max} := 0{,}375\ \text{mm}$ Reference Design Basis

$h_{cr} := 0{,}5 \cdot h$ NS EN 1992-1-1 7.3.2

$k := 0{,}65$

$k_c := 0{,}4$

$$A_{s,min,cr} := \frac{k_c \cdot k \cdot f_{ctm} \cdot \frac{h}{2}}{\sigma_s} = 12{,}3352\ \frac{\text{cm}^2}{\text{m}}$$

$$\phi_s := \phi_{sp} \cdot \left(\frac{f_{ct,eff}}{2{,}9\ \text{MPa}}\right) \cdot \frac{k_c \cdot h_{cr}}{2 \cdot (h - d)} = 25{,}23\ \text{mm}$$ NS EN 1992-1-1 (7.6N)

$$A_s := \frac{\frac{\pi \cdot (16\ \text{mm})^2}{4}}{150\ \text{mm}} = 13{,}4041\ \frac{\text{cm}^2}{\text{m}}$$

$$\frac{\max\left(\left[A_{s,min,cr}\ A_{sVmin}\ A_{sHmin}\right]\right)}{A_s} = 0{,}9203$$ Ok

Due to the low forces on the wings, the minimum reinforcement is sufficient as shown in the following calculations.

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Wings reinforcement calculation

Reference: NS-EN 1992-1-1

Input data

$f_{yk} := 500\ \text{MPa}$ $f_{ck} := 45\ \text{MPa}$

$f_{yd} := \frac{f_{yk}}{1,15} = 434,7826\ \text{MPa}$ $f_{cd} := \frac{0,85 \cdot f_{ck}}{1,5} = 25,5\ \text{MPa}$

$\sigma_s := 200\ \text{MPa}$

$h := 1,81\ \text{m}$ $L := 1,94\ \text{m}$ $t := 500\ \text{mm}$ Height and length

$\gamma_{earth} := 19\ \frac{\text{kN}}{\text{m}^3}$ Reference Design Basis

$c := 75\ \text{mm}$ Reference Design Basis

$K_0 := 0,5$ Reference Design Basis

$\phi := 16\ \text{mm}$

$d := t - c - \frac{\phi}{2} = 0,417\ \text{m}$

Loads

$q_{fk} := 5\ \frac{\text{kN}}{\text{m}^2}$ Reference Design Basis

$q_{Gtop} := 0 = 0\ \text{kPa}$ $q_{Gbot} := K_0 \cdot \gamma_{earth} \cdot h = 17,195\ \text{kPa}$ Earth loads

$q_{Qtop} := q_{fk} \cdot K_0 = 2,5\ \text{kPa}$ $q_{Qbot} := q_{fk} \cdot K_0 = 2,5\ \text{kPa}$ Traffic loads

Moment design

$q_{Gtop} + 1,35 \cdot q_{Qtop} = 3375\ \text{Pa}$

$M_{SLS} := 0,5 \cdot \left(q_{Gbot} + 0,5 \cdot q_{Qbot}\right) \cdot h^2 = 30,2138\ \frac{\text{kN m}}{\text{m}}$

$M_{ULS} := 0,5 \cdot \left(q_{Gbot} + 1,35 \cdot q_{Qbot}\right) \cdot h^2 = 33,6947\ \frac{\text{kN m}}{\text{m}}$

$q_{Gbot} + 1,35 \cdot q_{Qbot} = 20570\ \text{Pa}$

$A_{SLSbot} := \frac{M_{SLS}}{0,9 \cdot \sigma_s \cdot d} = 4,0253\ \frac{\text{cm}^2}{\text{m}}$

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$$A_{ULSbot} := \frac{M_{ULS}}{0,9 \cdot f_{yd} \cdot d} = 2,065 \frac{cm^2}{m}$$

$$A_s := \frac{\frac{\pi \cdot (16\ mm)^2}{4}}{150\ mm} = 13,4041 \frac{cm^2}{m} \qquad \frac{\pi \cdot (16\ mm)^2}{4} \cdot f_{yd} = 87,4182\ kN$$

$$\frac{A_{SLSbot}}{A_s} = 0,3003 \qquad \frac{A_{ULSbot}}{A_s} = 0,1541$$

Shear design

$$V_{ULS} := \left(q_{Gbot} + 1,35 \cdot q_{Qbot}\right) \cdot h = 37,2317 \frac{kN}{m}$$

$$k_2 := 0,18 \qquad \alpha := 90\ deg$$

$$C_{Rdc} := \frac{k_2}{1,5} = 0,12 \qquad \rho := \min\left(\left[\frac{A_s}{d}\ \ 0,02\right]\right) = 0,0032 \qquad k := \min\left(\left[1 + \sqrt{\frac{200\ mm}{d}}\ \ 2\right]\right) = 1,6925$$

$$v_{min} := 0,035 \cdot k^{\frac{3}{2}} \cdot \sqrt{f_{ck}\ MPa} = 0,517\ MPa \qquad v_{Rdc} := C_{Rdc} \cdot k \cdot \left(100 \cdot \rho \cdot f_{ck}\ MPa^2\right)^{\frac{1}{3}} = 0,4949\ MPa$$

$$V_{Rdc} := \max\left(\left[v_{min}\ \ v_{Rdc}\right]\right) \cdot d = 215,5858 \frac{kN}{m}$$

$$\frac{V_{ULS}}{V_{Rdc}} = 0,1727$$

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16.3 FOOTING

16.3.1 Minimum reinforcement

Foundation slab

$h := 700\ \text{mm}$

$\sigma_s := 200\ \text{MPa}$ $\phi_{sp} := 32\ \text{mm}$ NS EN 1992-1-1 Table 7.2N

$d := h - c - \frac{\phi}{2} = 0{,}617\ \text{m}$

$w_{max} := 0{,}375\ \text{mm}$ Reference Design Basis

$h_{cr} := 0{,}5 \cdot h$ NS EN 1992-1-1 7.3.2

$k := 0{,}65$

$k_c := 0{,}4$

$$A_{s,min,cr} := \frac{k_c \cdot k \cdot f_{ctm} \cdot \frac{h}{2}}{\sigma_s} = 17{,}2693\ \frac{\text{cm}^2}{\text{m}}$$

$$A_{s,min,1} := \max\left(\left[\begin{matrix} 0{,}26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot d & 0{,}0013 \cdot d \end{matrix}\right]\right) = 12{,}1773\ \frac{\text{cm}^2}{\text{m}}$$ NS EN 1992-1-1 NA 9.2.1.1

$$\phi_s := \phi_{sp} \cdot \left(\frac{f_{ct,eff}}{2{,}9\ \text{MPa}}\right) \cdot \frac{k_c \cdot h_{cr}}{2 \cdot (h - d)} = 35{,}32\ \text{mm}$$ NS EN 1992-1-1 (7.6N)

$$A_s := \frac{\frac{\pi \cdot (20\ \text{mm})^2}{4}}{150\ \text{mm}} = 20{,}944\ \frac{\text{cm}^2}{\text{m}}$$

$$\frac{\max\left(\left[\begin{matrix} A_{s,min,cr} & A_{s,min,1} \end{matrix}\right]\right)}{A_s} = 0{,}8245$$ Ok

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16.3.2 Pile caps

Punching shear cannot occur with the piles in compression as loads will transfer through a strut and tie mechanism.

It can however be an issue if the pile is in tension. Notice that the pile tensions will probably disappear if we consider the weight of the concrete (not included in the strut and tie model). However, we choose to include its effect.

PUNCHING RESISTANCE TO CL. 6.2.2.5 EN 1994-2 AND TO CL. 6.4.4 NS-EN 1992-2

Reference documents

NS-EN 1992-1-1:2004 Design of concrete structures - Part 1-1: General rules and rules for buildings

Input

Section	Rectangular	
Ø	-	m
a	0,30	m
b	0,30	m
u0	1,20	m
u1	3,27	m
β	1,50	
h	0,260	m
r	0,075	m
fck	45	MPa
γc	1,50	

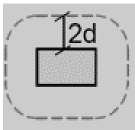

Fig. 1: u1

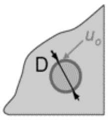

Fig. 2: u0

$F_{ed,uo}$	33	kN
$F_{ed,u1}$	33	kN

UPPER/LOWER REINFORCEMENT LAYER

X-dir		
ø	20	mm
spa	150	mm
Y-dir		
ø	20	mm
spa	150	mm

PUNCHING SHEAR REINFORCEMENT

ø	0	mm
spa sr	248	mm
Nr. Links / ~~perimenter~~ total	0	*Total number of shear links*

Output

Ed (u0)	0,25	MPa	OK
Rd,max	7,38	MPa	
Ed (u1)	0,092	MPa	OK
Rd,c (u1)	0,771	MPa	
Rd,cs (u1)	0,58	MPa	NO NEED

Calculations

N(u0)	33	kN	ρx	1,20E-02	
N(u1)	33	kN	ρy	1,35E-02	
			ρ	1,27E-02	
dx	0,175	m	k	2,000	-
dy	0,155	m	vmin	**0,664**	MPa
d	0,165	m	vRd,c	0,771	MPa
			vRd,max	7,380	MPa
uout	0,389	m	fywd	400	MPa
			fywd,ef	291	MPa
			α	90	º
			vRd,cs	0,578	MPa

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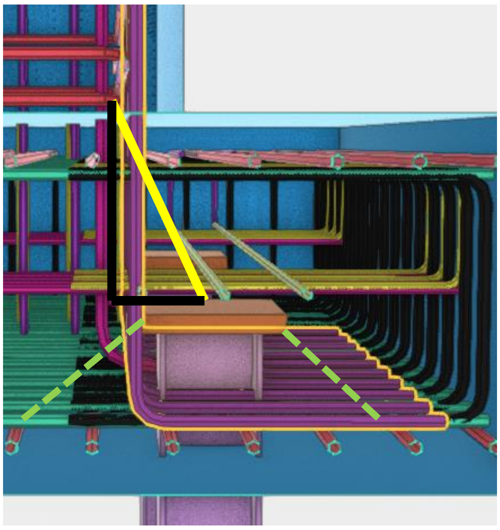

Figure 16-5: Strut and tie model for downward forces and punching shear for uplift forces.

16.4 TRANSITION SLAB AND CORBEL

Standard design according to SVV brudetaljer for footbridges.

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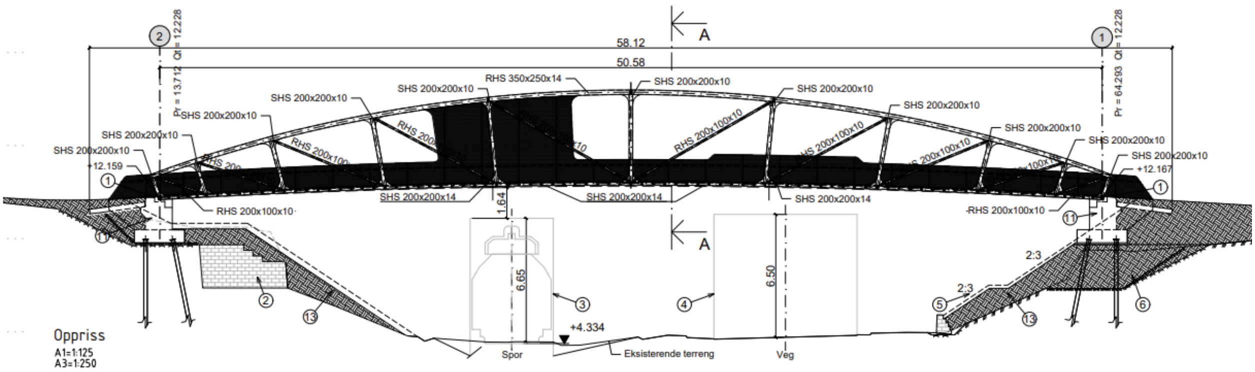
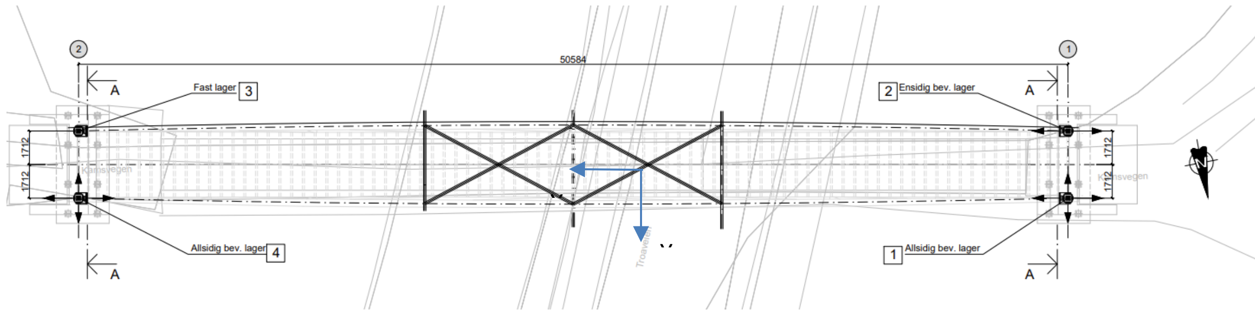
17 VEDLEGG 8 – REACTION FOR GEO DESIGN

In the new revised layout (REV 4) the axis 2 abutment is the longitudinal fix point, while the axis 1 is free.





ULS LOADS ENVELOPE

Lager	type	Fz	Fx (long.)	Fy (transv)	Zone
-	-	kN	kN	kN	-
Lager 4	Free	607	±12.6*	±12.6*	North-west
Lager 3	Fix	612	195 / -107	296 / -276	South-west
Lager 2	Unidir X	603	±12.6*	307 / -298	South-east
Lager 1	Free	604	±12.6*	±12.6*	North-east

* Values due to friction at bearing, friction need to be added to the overall load acting on the foundation.

DEGREE OF FREEDOM

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SLS LOADS ENVELOPE

Lager	type	Fz	Fx (long.)	Fy (transv)	Zone
-	-	kN	kN	kN	-
Lager 4	Free	451	±8.4*	±8.4*	North-west
Lager 3	Fix	453	136/ -79	187 / -173	South-west
Lager 2	Unidir X	448	±8.4*	194 / -187	South-east
Lager 1	Free	448	±8.4*	±8.4*	North-east

* Values due to friction at bearing, friction need to be added to the overall load acting on the foundation.

REFERENCE SCHEME FOR LOAD APPLICATION TO BE CONSIDERED IN THE FOUNDATION DESIGN: