RC open frame bridge

TABLE OF CONTENT

Part A : CALCULATION ASSUMPTIONS

RC open frame bridge

CONTENTS

1. GENERAL / MEASUREMENT

1.1 CONSTRUCTION TYPE

Open frame orthogoanal constructed using reinforced contrete.

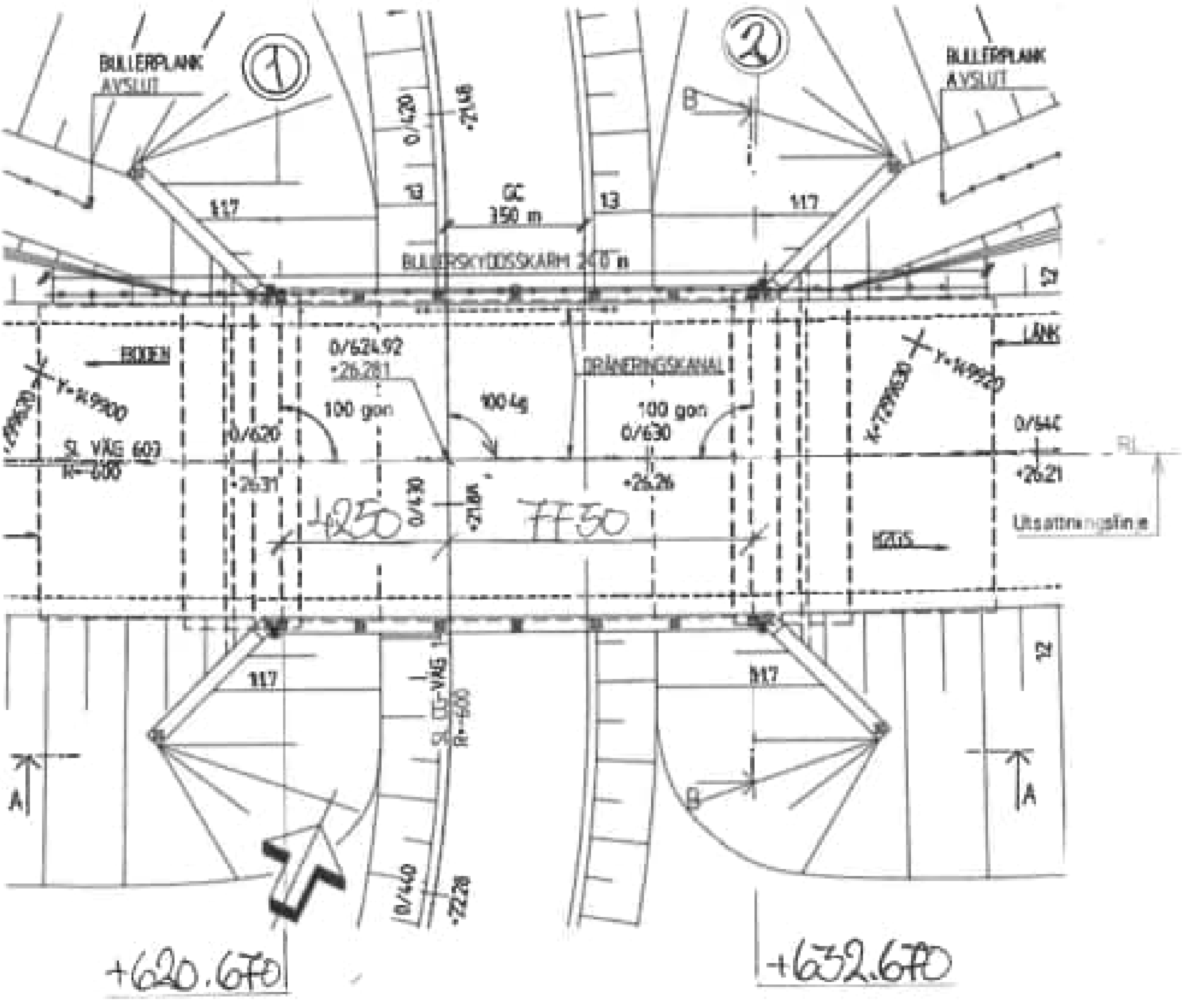
Foundation on compacted gravel.

At each abutement link slabs are constructed.

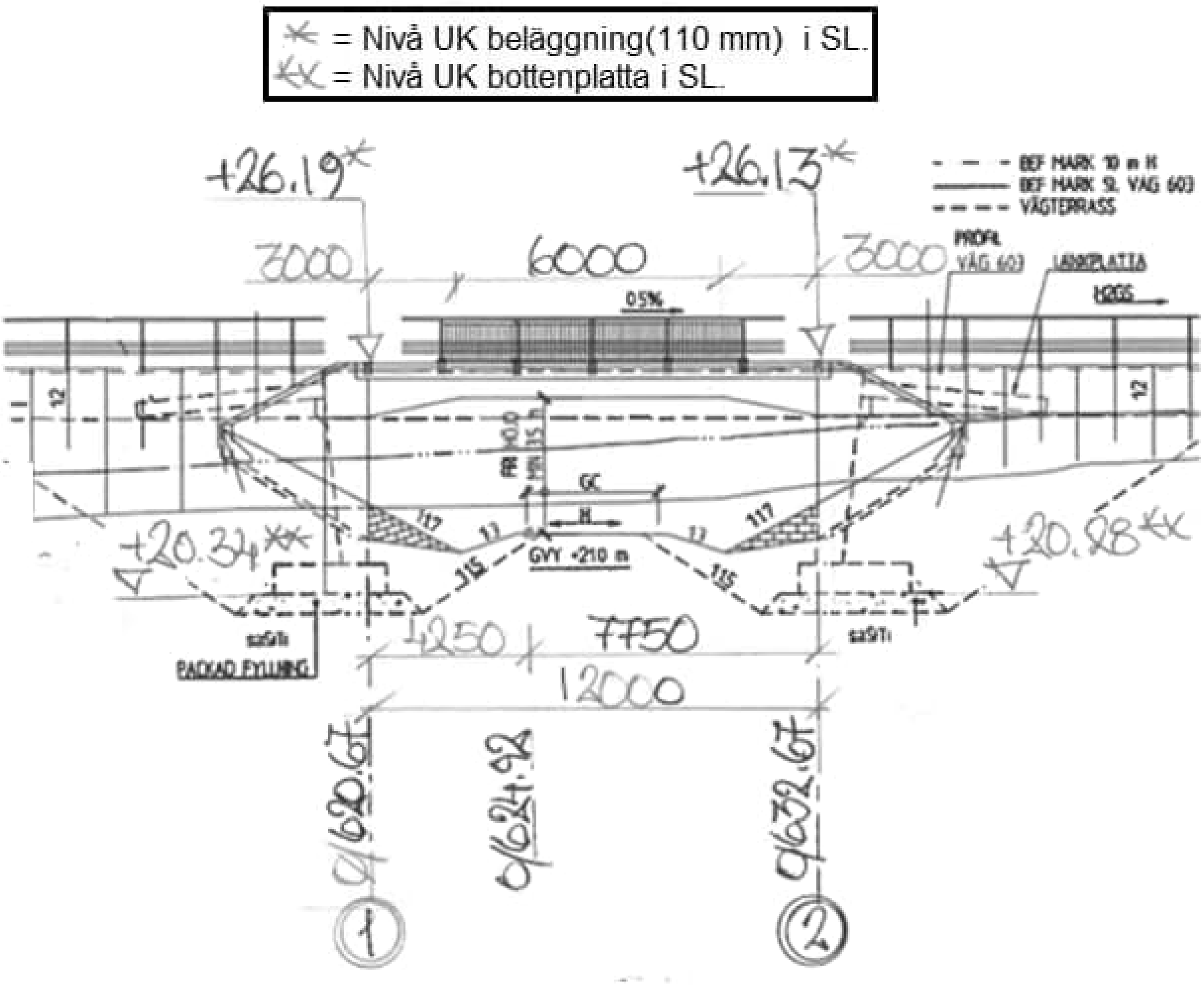
The bridges abutments, wingwall and bridge deck is constructed during one casting stage.

1.2 MEASUREMENTS

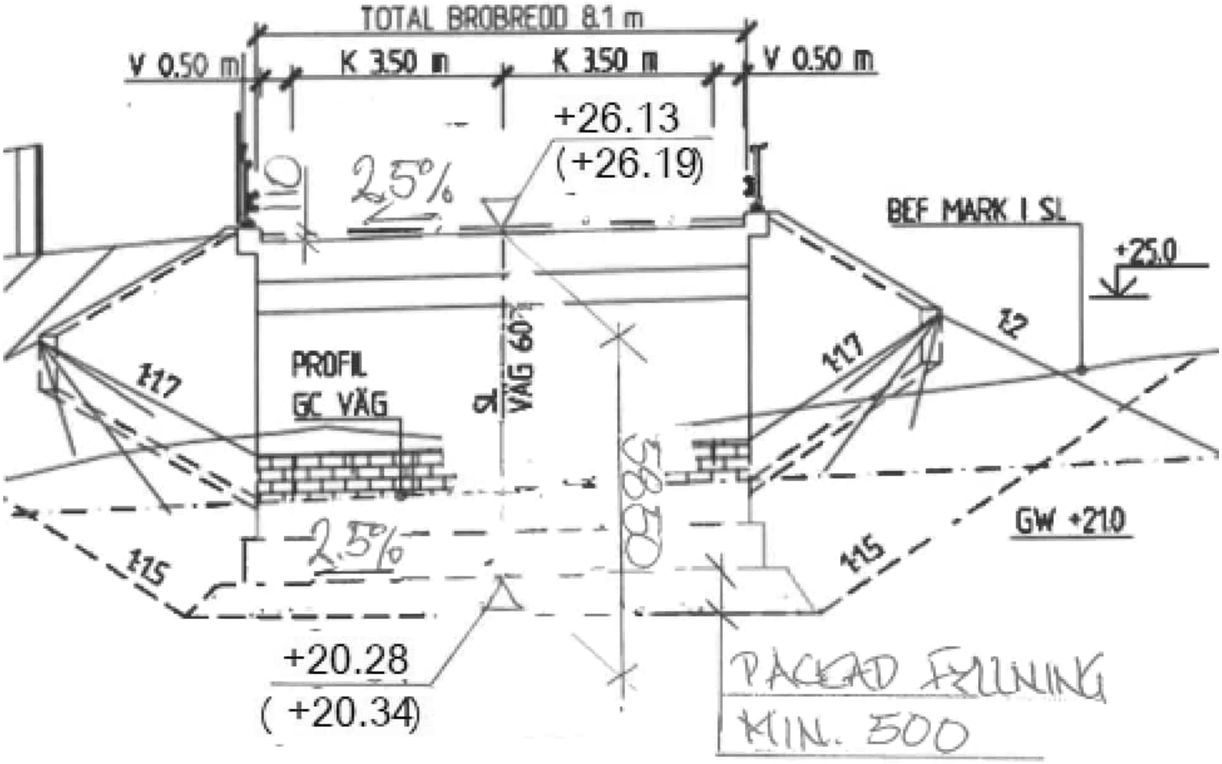
1.2.1 Theoretical geometry



PLAN
Översikt



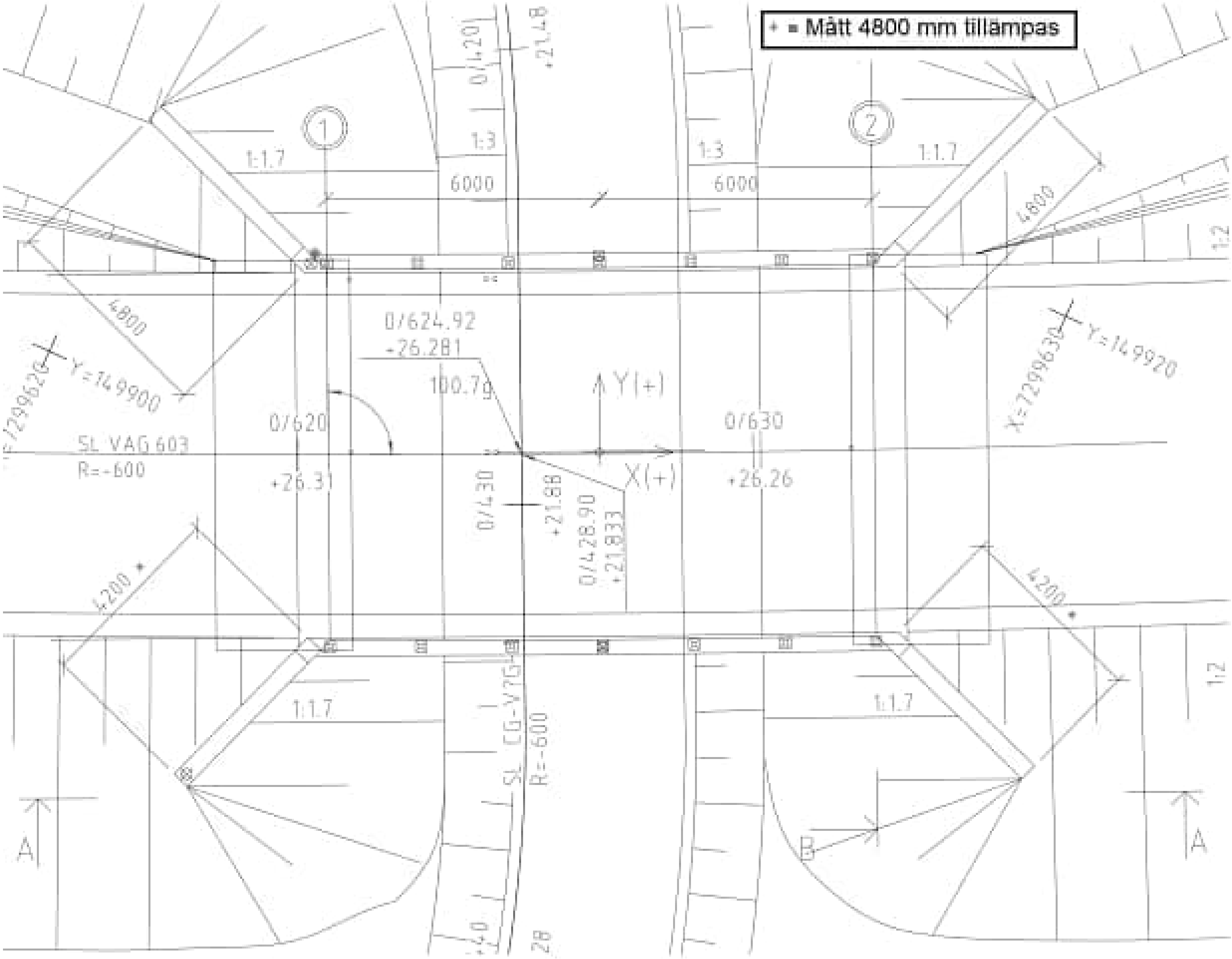
SECTION A-A
Elevation



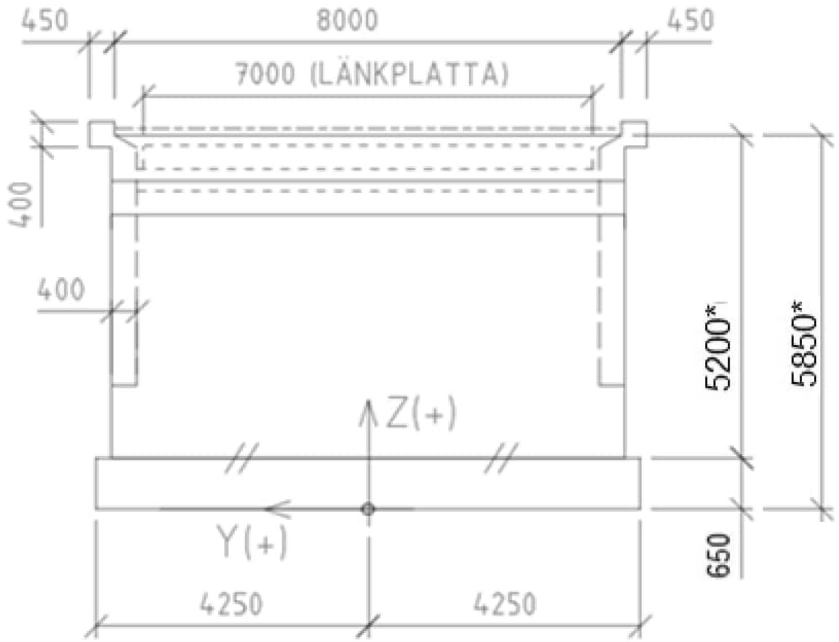
SECTION B-B

Abutement 2. Values inside parentensis are at abutement 1.

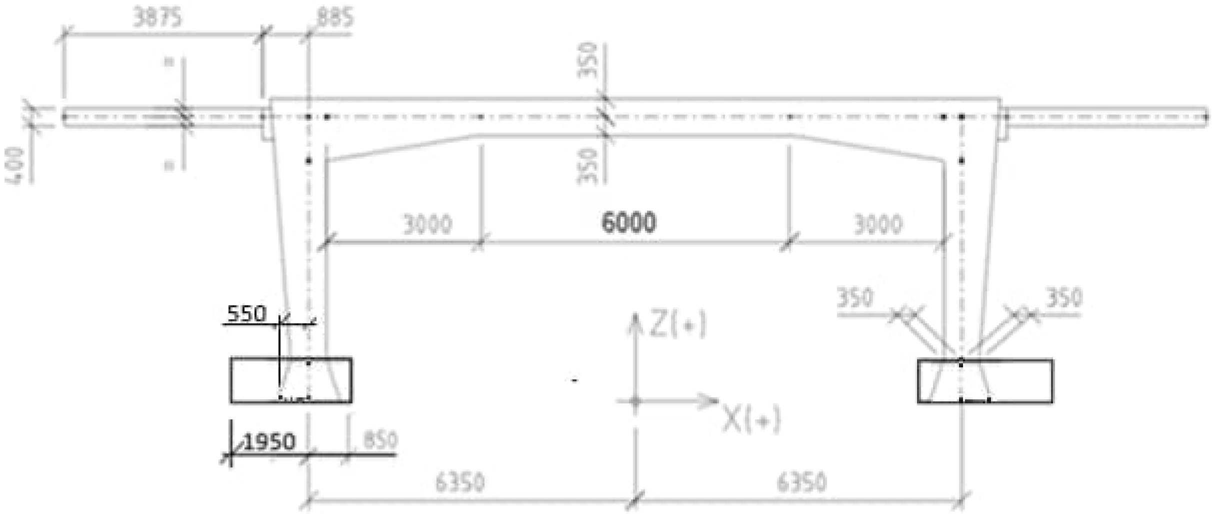
1.2.2 Geometry calcualtions

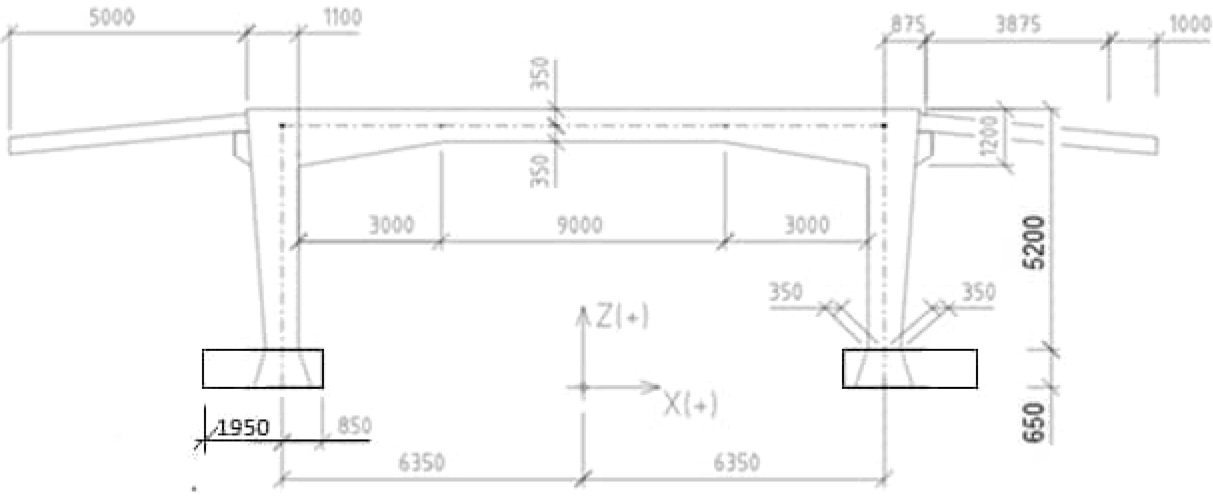


PLAN

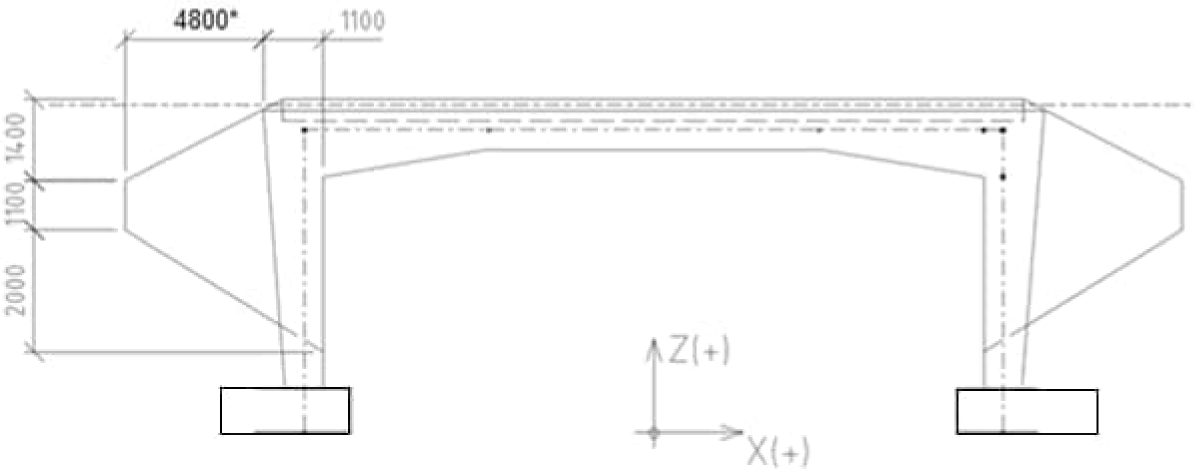


SECTION A-A
Transversal direction.





<u>SECTION B-B</u>
Longitudinal section through link slabs.Längsled genom länkplattor



<u>SECTION B-B</u>
Wingwalls.

1.3 FOUNDATION

Both bottom slabs are founded on 0.5 m compacted gravel on rock.

Abutement 1:
Foundation level +20.34 (SL road)
Level bottom of compacted gravel +19.70.

Abutement 2:
Foundation level +20.28 (SL road)
Level bottom of compacted gravel +19.64.

Design waterlevel:
Highest level +22.0.

1.4 CODE OCH TENDER DOCUMENTS

Documents	Version	Name
SS-EN 1990-1997	-	Svensk Standard Eurokod 1-7
TRVINFRA-00226	2.0	KRAV, Bro och broliknande konstruktion, Allmänna krav
TRVINFRA-00227	2.0	KRAV, Bro och broliknande konstruktion, Byggande
TRVINFRA-00228	2.0	KRAV, Bro och broliknande konstruktion, Brounderhåll
TRVINFRA-00331	2.0	KRAV, Bro och broliknande konstruktion, Bärighetsberäkning
TSFS 2018:57		Transportstyrelsens föreskrifter och allmänna råd om tillämpning av eurokoder
TDOK 2013:0667	2.0	Trafikverkets tekniska krav för geokonstruktioner. TK Geo 13
TDOK 2013:0668	2.0	Trafikverkets tekniska råd för geokonstruktioner. TR Geo 13
AMA Anläggning 23		AMA, Svensk Byggtjänst
TDOK 2023:0125	2.0	TRVAMA Anläggning 23
SS 137006:2015	-	Betongkonstruktioner – Utförande – Tillämpning av SS-EN 13670:2009 i Sverige

1.5 TECHNICAL SERVICE LIFE

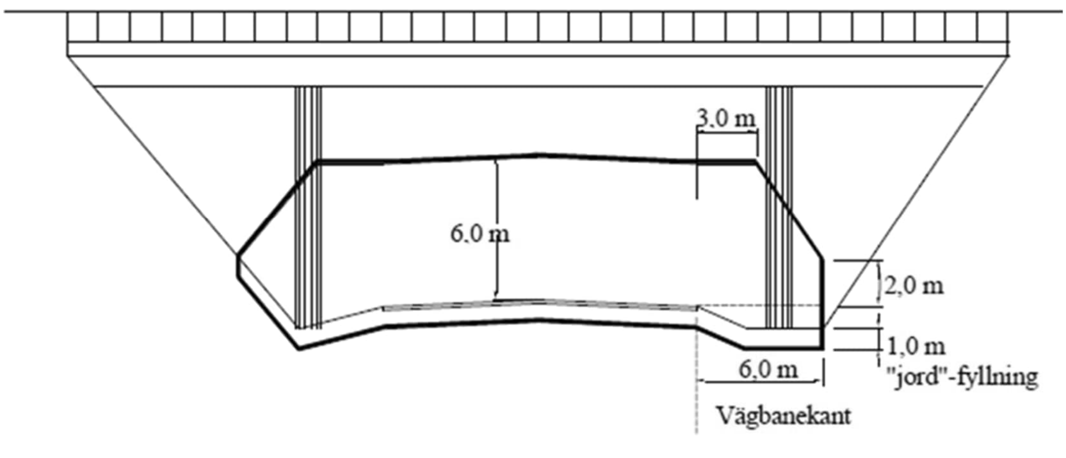
Technical life span 20 years (L20) for railing.

Technical life span 120 years (L100) for all else.

1.6 ENVIROMENT

Exposure class accoring to TSFS 2018:57 section 5.3.2.3 and SS-EN 206-1.

In TSFS 2021:57 figure 1.1, the road environment is defined according to the figure below. Which means that only edge beams in road environment.



1.7 MATERIAL

Concrete (see SS-EN 1992-1-1, Table 3.1):
C30/37 (f_{ck} = 30 MPa) - bottom slab
C35/45 (f_{ck} = 35 MPa) - other stuctures

Reinforcement (SS-EN 10080 och SS 212540):
K500CT (f_{yk} = 500MPa)

Backfill:
"Grovkrossad sprängsten" according to AMA CEB.524.
TRVINFRA-00230 table A1 gives material properties $\phi_k = 45°$, $\gamma = 20\frac{kN}{m^3}$, $\gamma' = 13\frac{kN}{m^3}$ and E_k = 50 MPa.

Compacted fill:
"Krossad sprängsten" 500 mm according to AMA CEB.411.
TRVINFRA-00230 table A1 gives material properties $\phi_k = 45°$, $\gamma = 18\frac{kN}{m^3}$, $\gamma' = 11\frac{kN}{m^3}$ and E_k = 50 MPa.

1.8 GEOTECHNICAL CLASS

Geotechnical class GK2.

1.9 SAFETY CLASS

Geotechnical resistance: SK 2

Linkplate & wingwall: SK 2

Bridge structure: SK 2 safety class 2 (∴ spann < 15 m)

Safety class 2 according to TSFS chapater 2 table 2.1.

1.10 CONCRETE COVER AND CRACK CRITERIA

Class identification bridge components :

Construction part	Exposure class [1.)]	Life spann	max vct_{ekv} [2.)]	ζ [3.)]
Substructure incl. linkplate :				
▫ Wingwall towards filling	XD1/XF4	L100	0.40	1.5
▫ Wingwall from filling	XD1/XF4	L100	0.40	1.5
▫ Abutment towards filling (level 1 m under surfacing)	XC2/XF3	L100	0.50	1.0
▫ Abutment from filling (above ground)	XC4/XF3	L100	0.50	1.2
▫ Abutment from filling (under ground)	XC2/XF3	L100	0.50	1.0
▫ Bottom slab in general	XC2/XF3	L100	0.50	1.0
▫ Bottom slab underside	XC2/XF3	L100	0.50	1.0
▫ Linkplate in general	XD1/XF2	L100	0.40	1.5
▫ Linkplate underside	XD1/XF2	L100	0.40	1.5
Superstructure:				
▫ Edge beam	XD3/XF3	L100	0.40	1.8
▫ Bridge deck	XD1/XF4	L100	0.40	1.5

Footnotes:

1.) TRVINRA-00227 section 5.3.2.3

2.) TSFS table 12.1

3.) TSFS table 12.3

Design parameters low corrosion sensitive reinforcement (rebars):

$c_{min.dur}$: minimum cover with regard to environmental impact

$c_{min.b}$: minimum cover with regard to adhesion requirements
Δc_{dev} : execution tolerance
c_{min} = max ($c_{min.b}$; $c_{min.dur}$; $10mm$) : SS-EN 1992-1-1 eq. 4.2
c_{nom} = c_{min} + Δc_{dev} : SS-EN 1992-1-1 eq. 4.1, noted as BM on the drawing

Construction part	$c_{min.dur}$ 1.)	$c_{min.b}$ 2.)	c_{min}	Δc_{dev} 3.)	c_{nom} 6.)	$w_{k,till}$ 4.)
Substructure incl. linkplate :						
▫ Wingwall towards filling	25	20	25	10	40	0.20
▫ Wingwall from filling	25	20	25	10	40	0.20
▫ Abutment towards filling (level 1 m under surfacing)	15	20	20	10	40	0.40
▫ Abutment from filling (above ground)	15	20	20	10	40	0.30
▫ Abutment from filling (under ground)	15	20	20	10	40	0.40
▫ Bottom slab in general	15	20	20	10	40	0.40
▫ Bottom slab underside	15	20	35 5.)	10	45	0.40
▫ Linkplate in general	25	20	25	10	40	0.20
▫ Linkplate underside	25	20	40 5.)	10	50	0.20
Superstructure:	45	20	45	10	55	0.15
▫ Edge beam	25	20	25	10	40	0.20
	mm	mm	mm	mm	mm	mm

Fotnotes:
1.) TSFS table 12.1
2.) SS-EN 1992-1-1 section 4.4.1.2 table 4.2
3.) SS-EN 1992-1-1 section 4.4.1.3
4.) TSFS table 12.2
5.) TSFS chapter 12 section 2§ $k_1 = c_{min}$ + 15 mm when casting against building foil.
6.) General Swedish practice min. c_{nom} = 40 mm (∴ ϕ32 + 5 mm)

2. SYSTEM ANALYSIS

2.1 GENERAL

The bridge is built using reinforced concrete.

The computational geometry is simplified so that the bridge deck is completely horizontal both longitudinally and vertically.

Abutments, bridge deck and link plate are modelled using high-node shell elements (QTS8). These shell elements are modelled with isotropic material properties corresponding to uncracked concrete.

The system calculation is performed using FEM- anlysis.

Wing walls are not modelled in the structural calculation as they are considered statically inactive in the vertical plane. Only minimum reinforcement will be introduced in the vertical plane to limit crack widths.

In the structural model, the wing length corresponding to the longest wing wall (4800 mm) is applied to all wing walls.

Edge beams are not modelled in the structural calculation on the safe side to allow future replacement without reduced load-bearing capacity.

The bottom of the girder will be connected to a fictitious "rigid beam" with negligible axial stiffness.

The centre of the "rigid beam" will be connected to a "super node” through a rigid joint (JSH4). This order to handle that bottom slabs are place excentric to abutments.

Link slabs are connected to top of abutments through a moment-free joint. These are designated as line joint element JNT4. Line supports are introduced 1.0 m from its end, which gives a slab with a free span of 4.0 m, considered to be on the safe side.

According to Swedish general practice a stiffness increase of 1:3 is applied within structural parts at the nodes.

Traffic load evaluation is carried out using Vehicle Load Optimiser (VLO) function for Swedish traffic loads.

Attachments:

Attachment	Name
1	Input receipt
2	Results reactions
3	Results abutments
4	Results bridge deck

2.2 SKETCH SYSTEM ANALYSIS

To describe geometry first POINTS are defined.

Beam elements are defined by applying attributes to LINES.

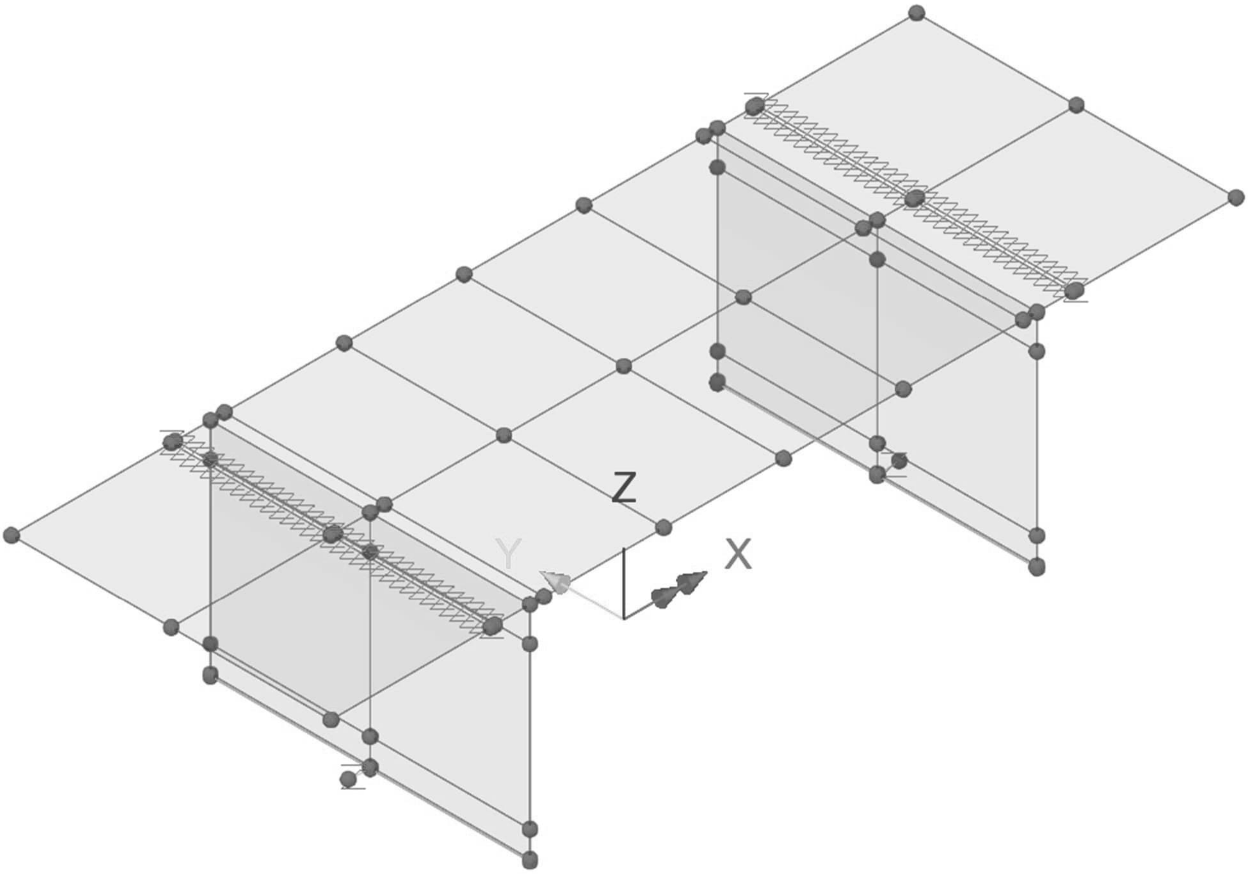
Shell elements are defined by applying attributes to SURFACES.

Attached pictures are retrieved from graphical sketches generated by FEM-program of POINTS, LINES and SURFACES.

All coordinates needed to describe POINTS are found in attachment 1.

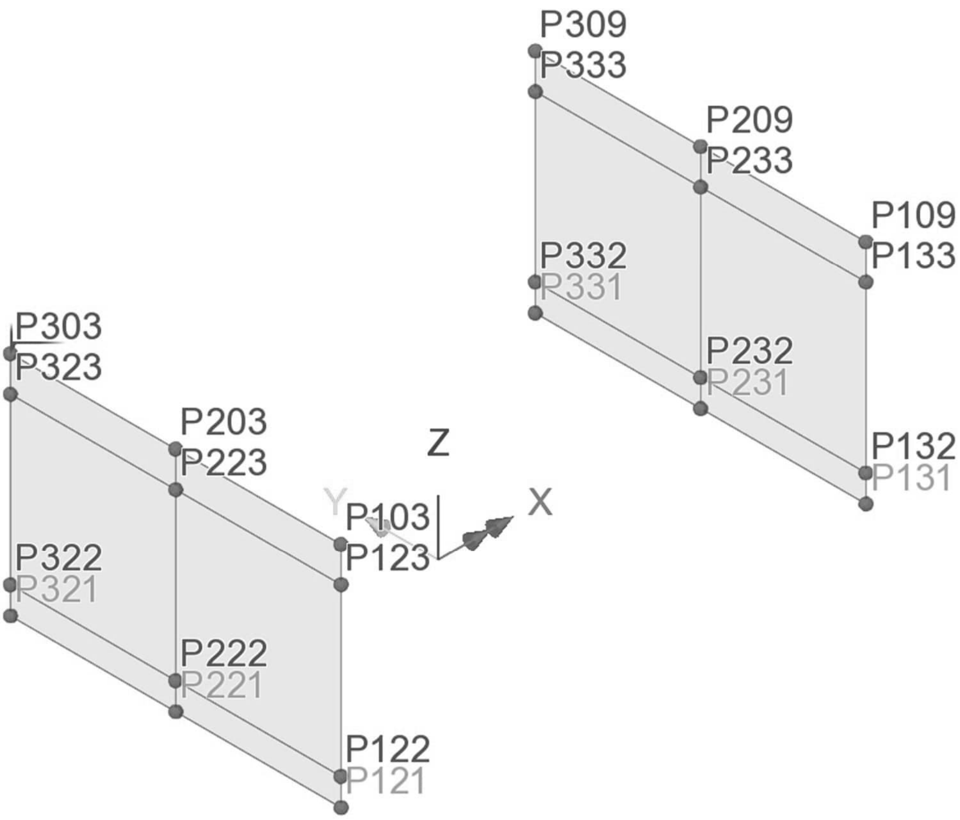
All POINTS needed to describe LINES are found in attachment 1.

All LINES need to describe SURFACE are found in attachment 1.

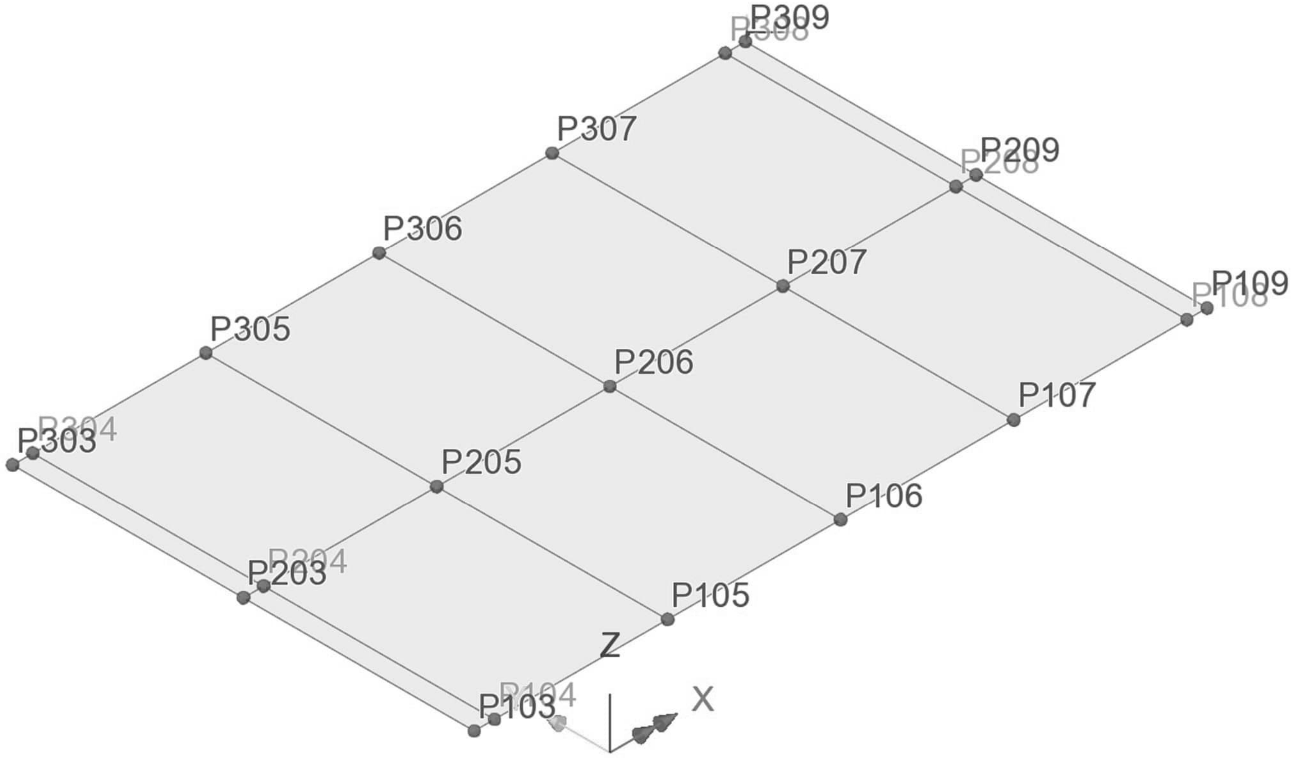


Overview 3D
Geometry

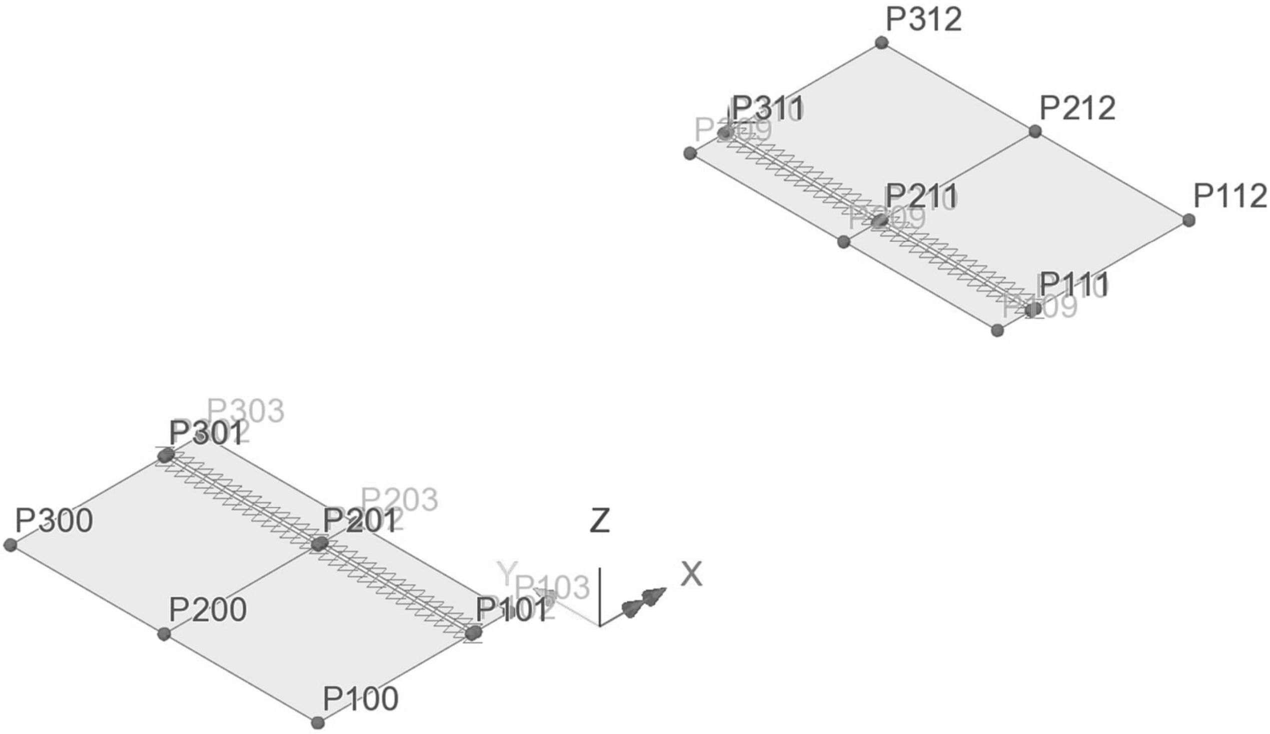
2.2.1 Geometry : POINTS



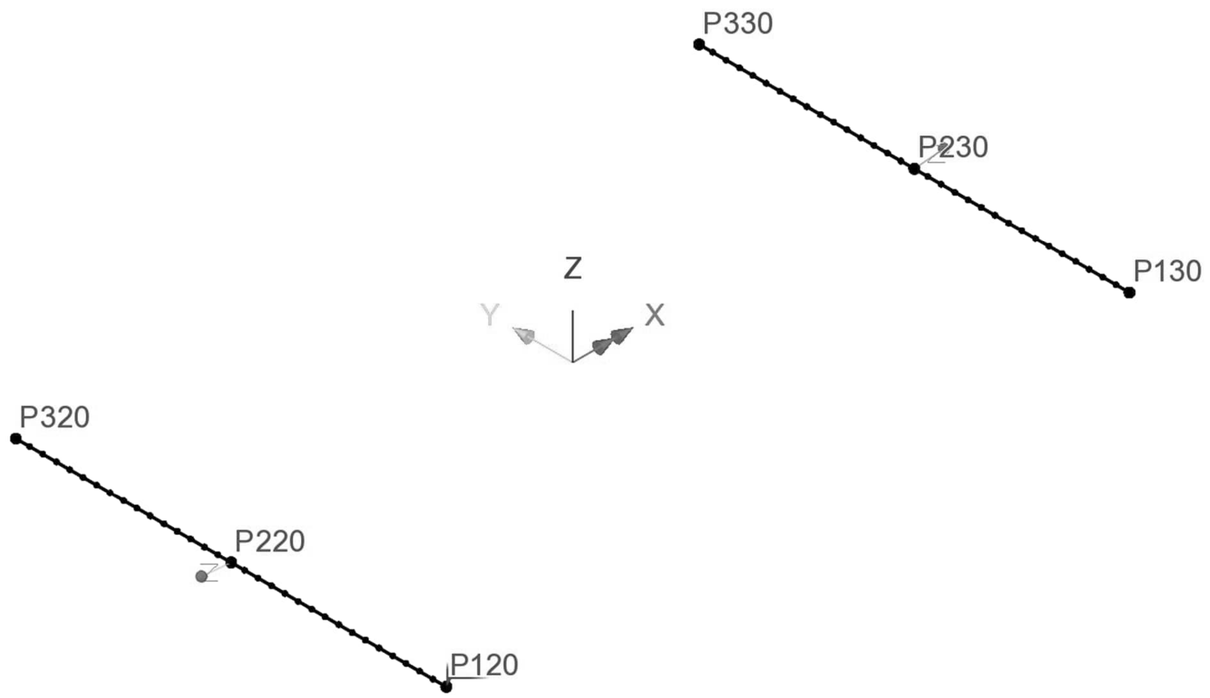
Abutment 1 & 2



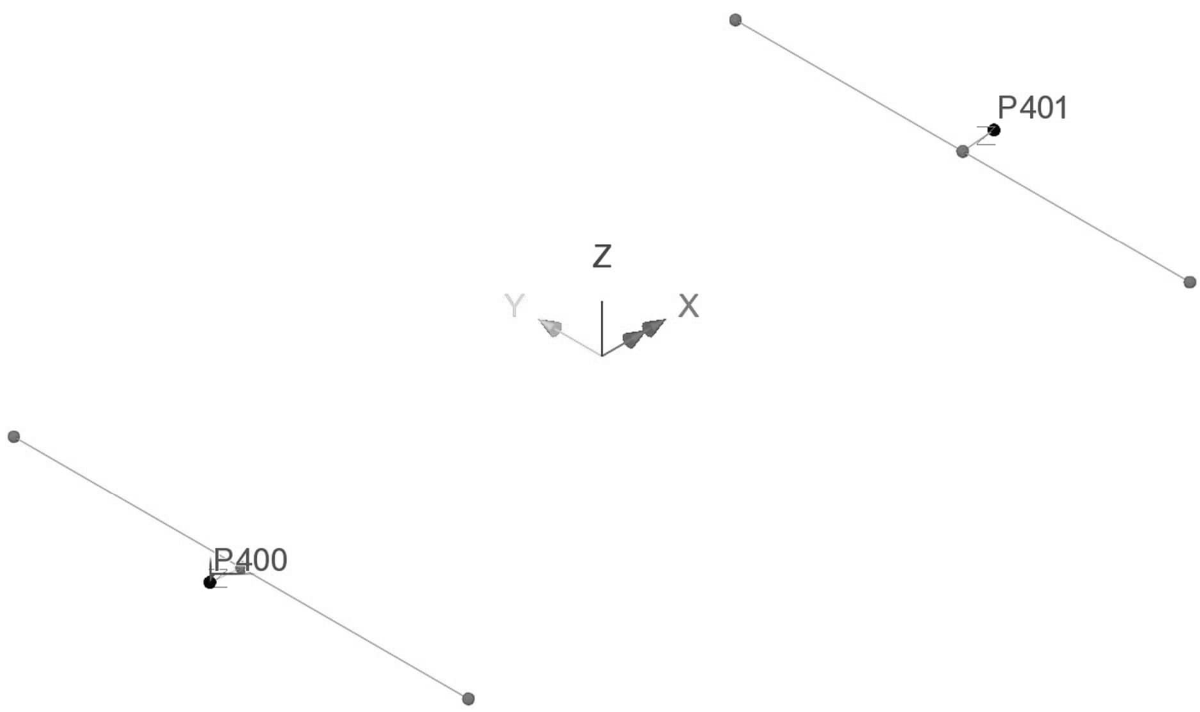
Bridge deck



Link plate

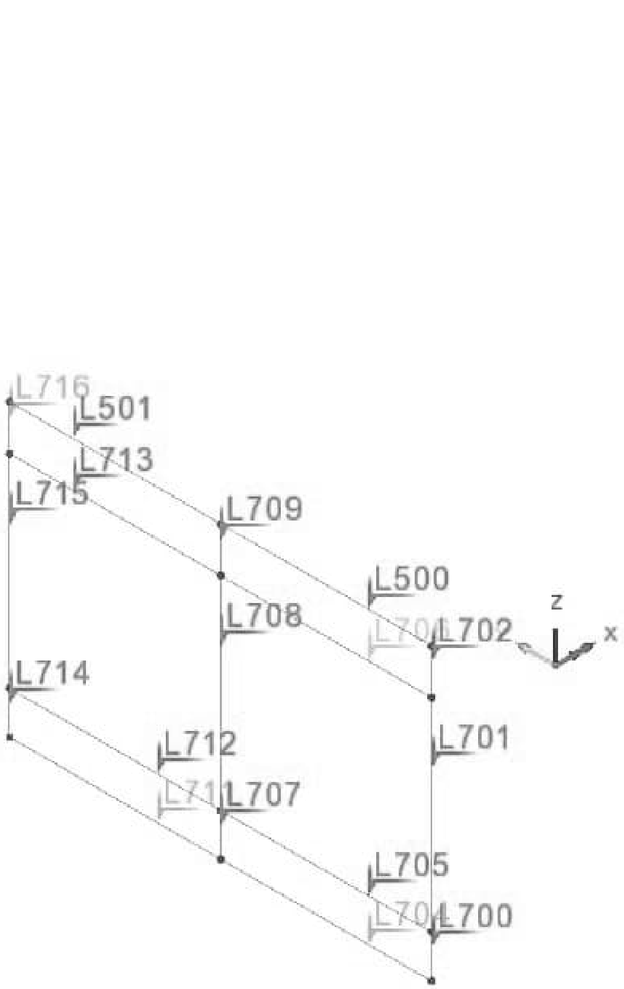


Fictious "rigid beam"

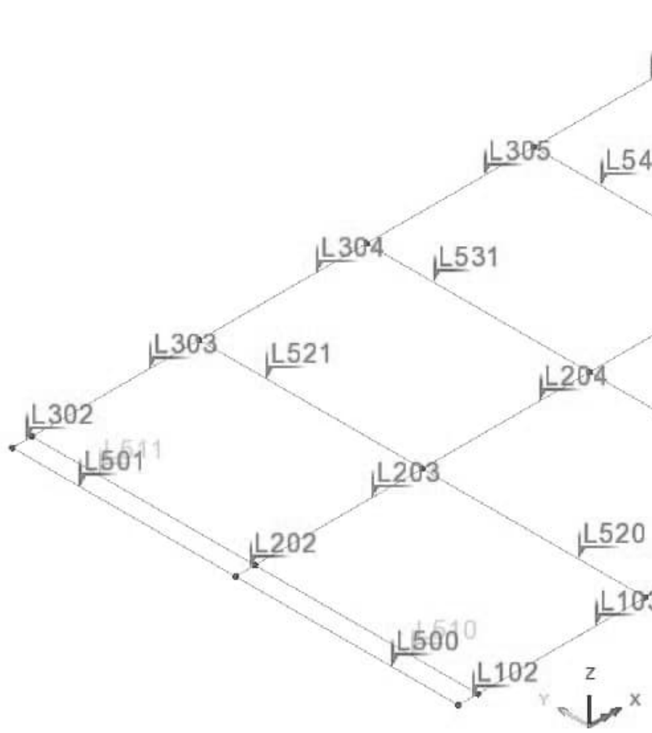


Fictious "super nodes"

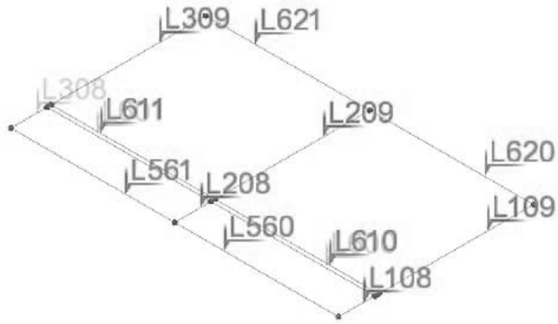
2.2.2 Geometry : LINES

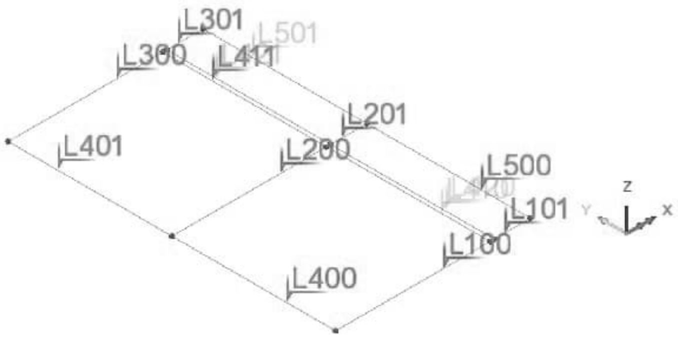


Abutment 1 & 2

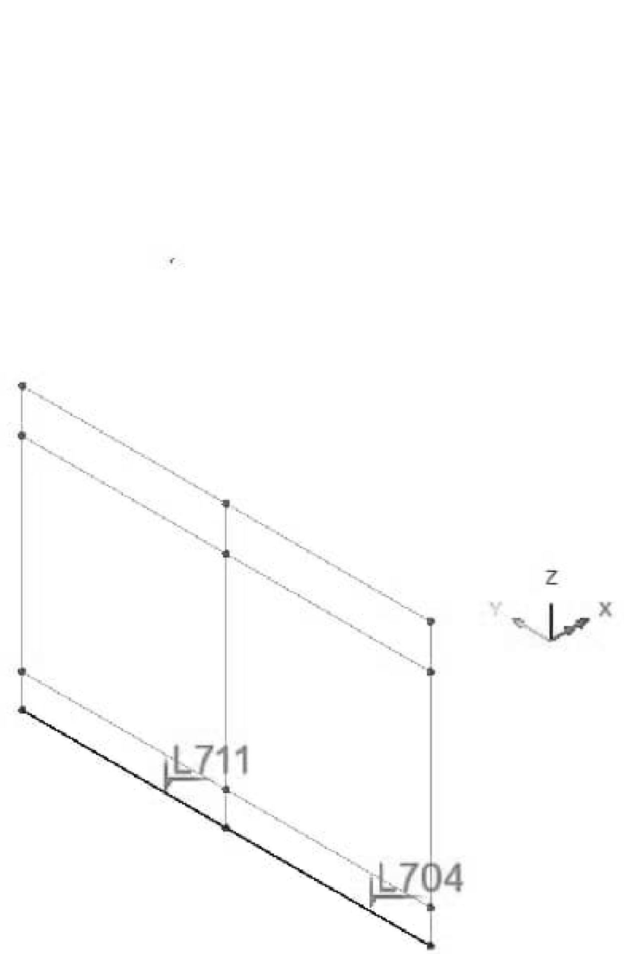


Bridge deck



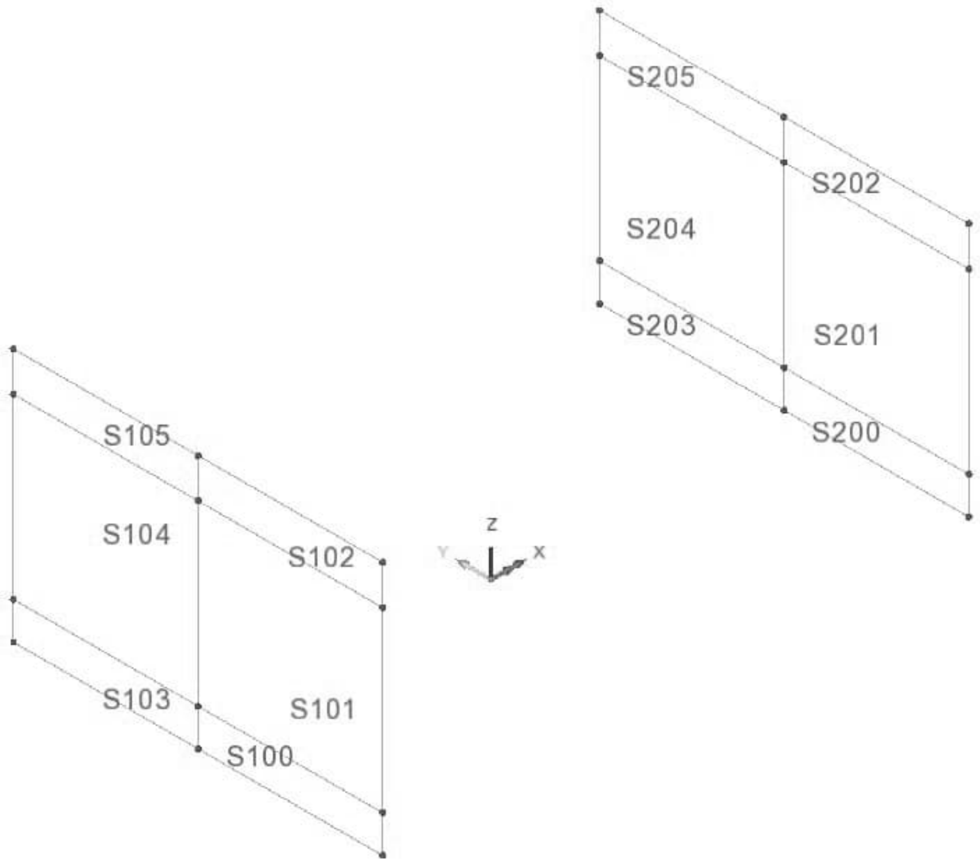


Link plates

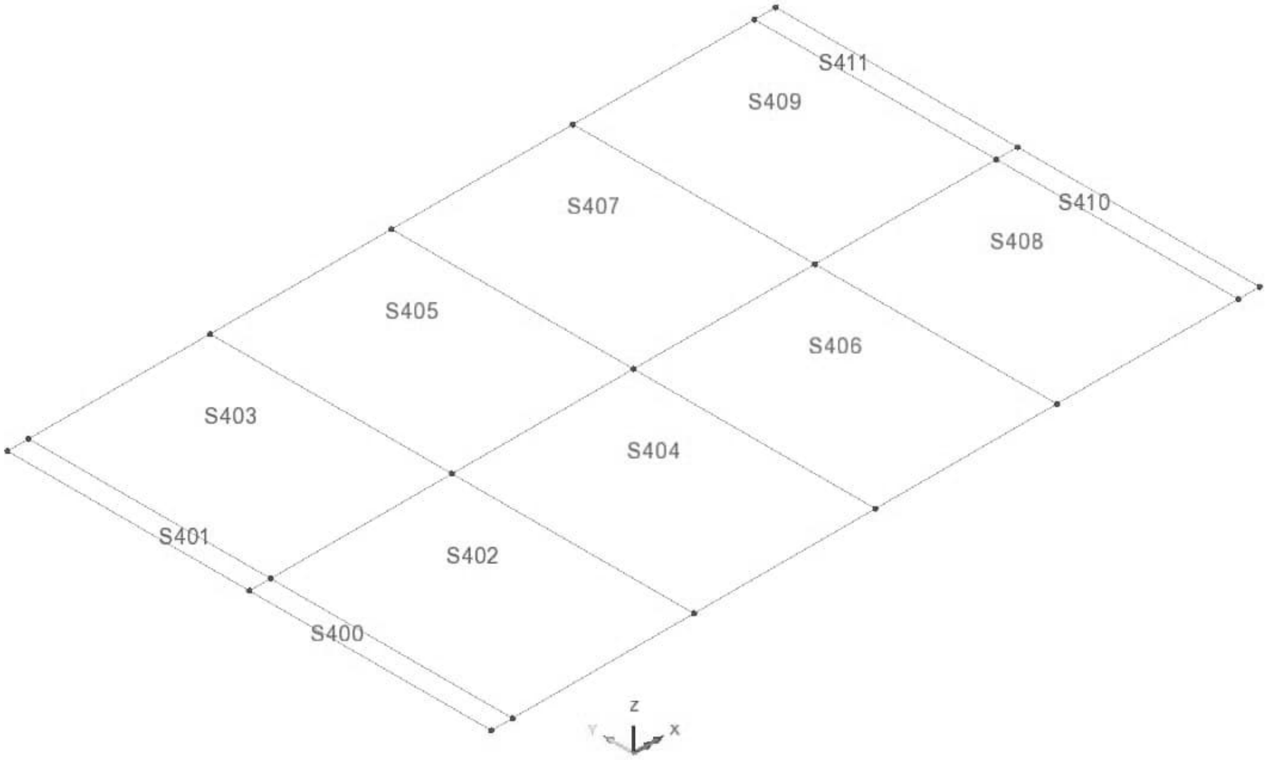


Fictious "rigid beam"

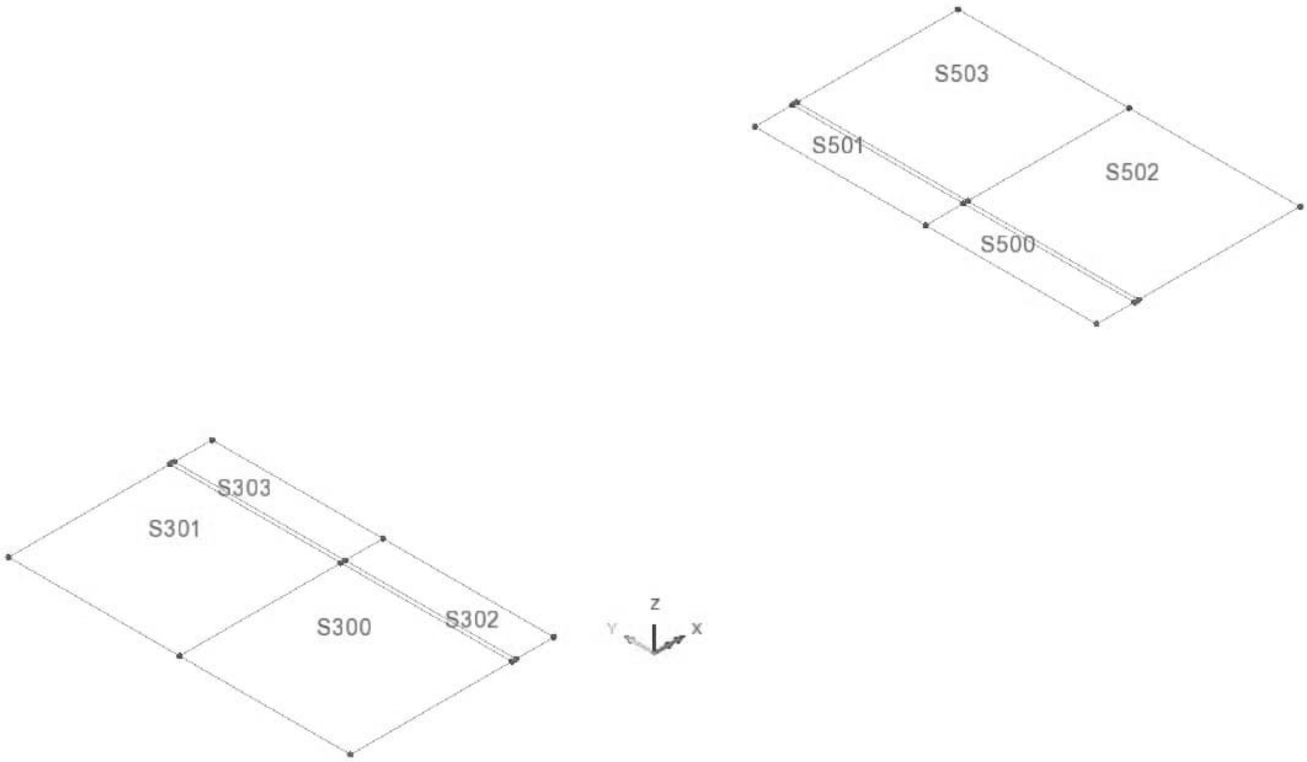
2.2.3 Geometri : SURFACES



Abutemnent 1/2


S411
S409
S410
S407
S408
S405
S406
S403
S404
S401
S402
S400
Z
Y
X

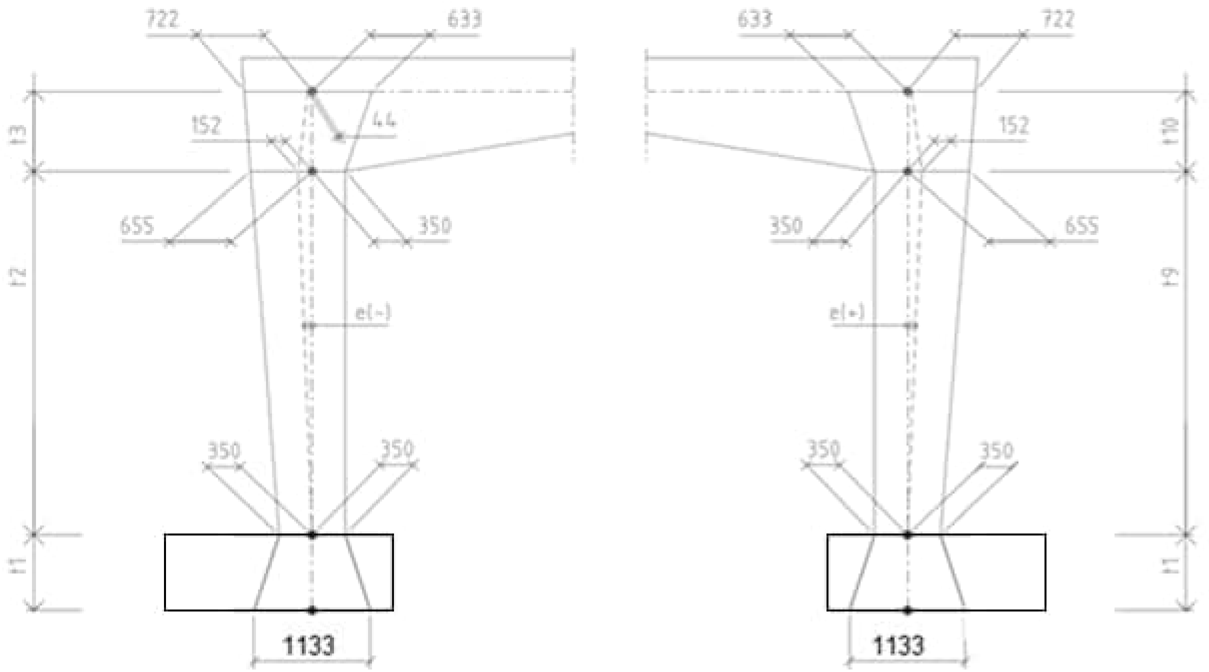
Bridge deck



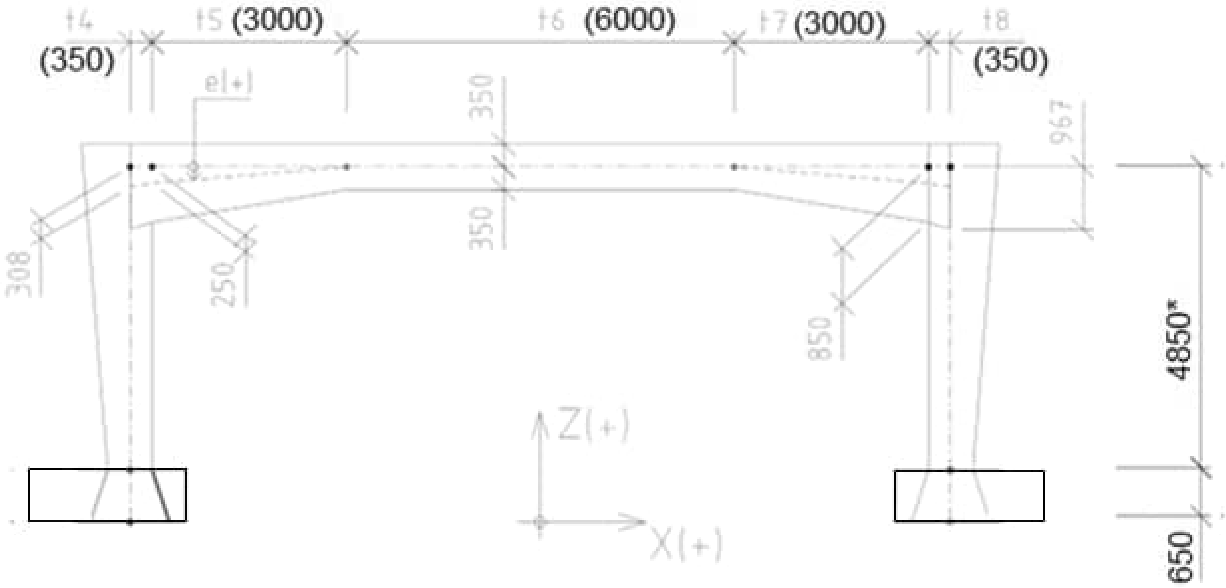
Link plate

2.3 CROSS SECTION PROPERTIES

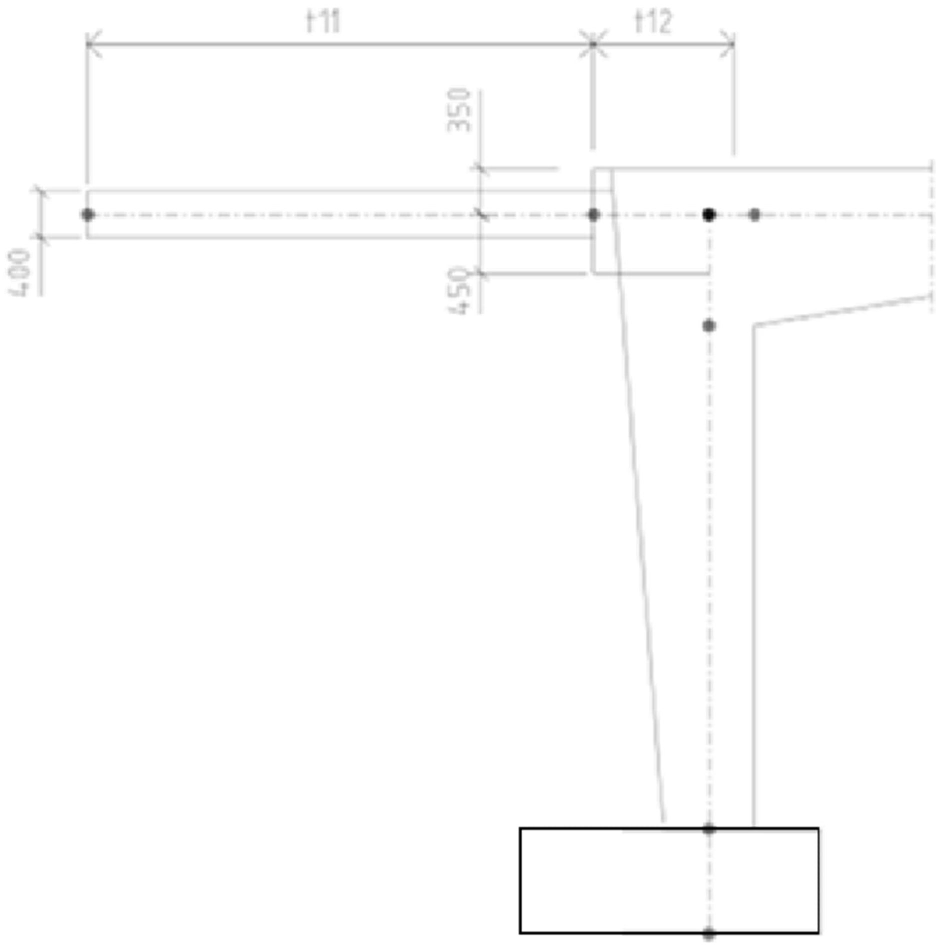
By experience stiffness increases by 1:3 at all joints as seen below.



Stöd 1/2



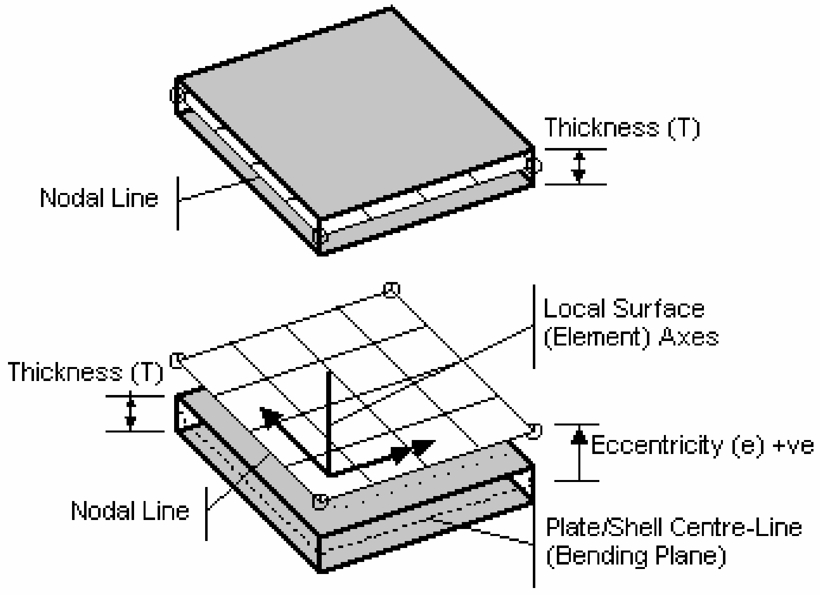
Bridge deck



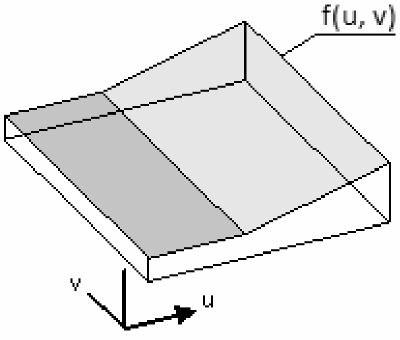
Link plate

2.3.1 Shell element

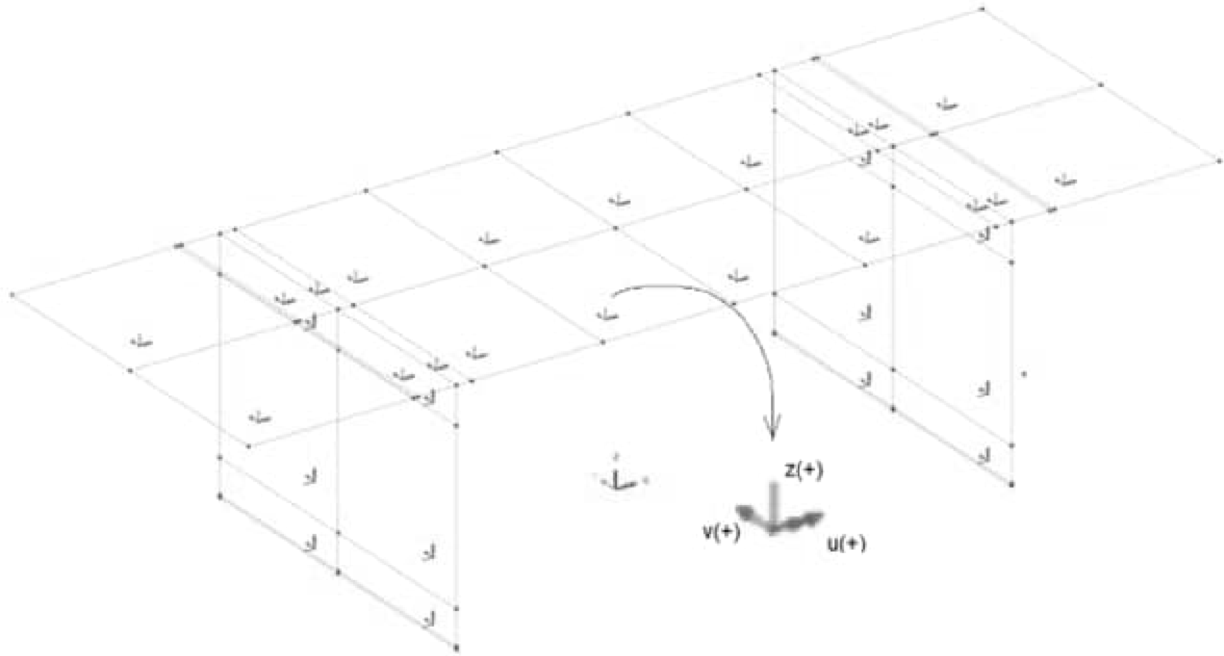
Principle figures of geometry associated to shell elements (”Thick shell” / QTS4) are seen below.



Varying thickness in shell element is handled using ”Function variation”. This makes it possible to create a function f(u,v) as seen below.



Local coordinate system (u,v) according to sketch below:



Surface function thickness :

Variation	Function(u,v)	Remark
t1	1.133-0.433·u	Bottom slab at abutement 1 & 212
t2	0.700+0.305·u	Abutement 1
t3	1.005+0.350·u	Abutement 1 conection superstructure
t4	1.317-0.117·u	Superstructure connection abutment 1
t5	1.200-0.500·u	Superstructure inclination area left side
t7	0.700+0.500·u	Superstructure inclination area right side
t8	1.200+0.117·u	Superstructure connection abutment 2
t9	0.700+0.305·u	Abutment 2
t10	1.005+0.350·u	Abutment 2 connection superstructure
-	m	-

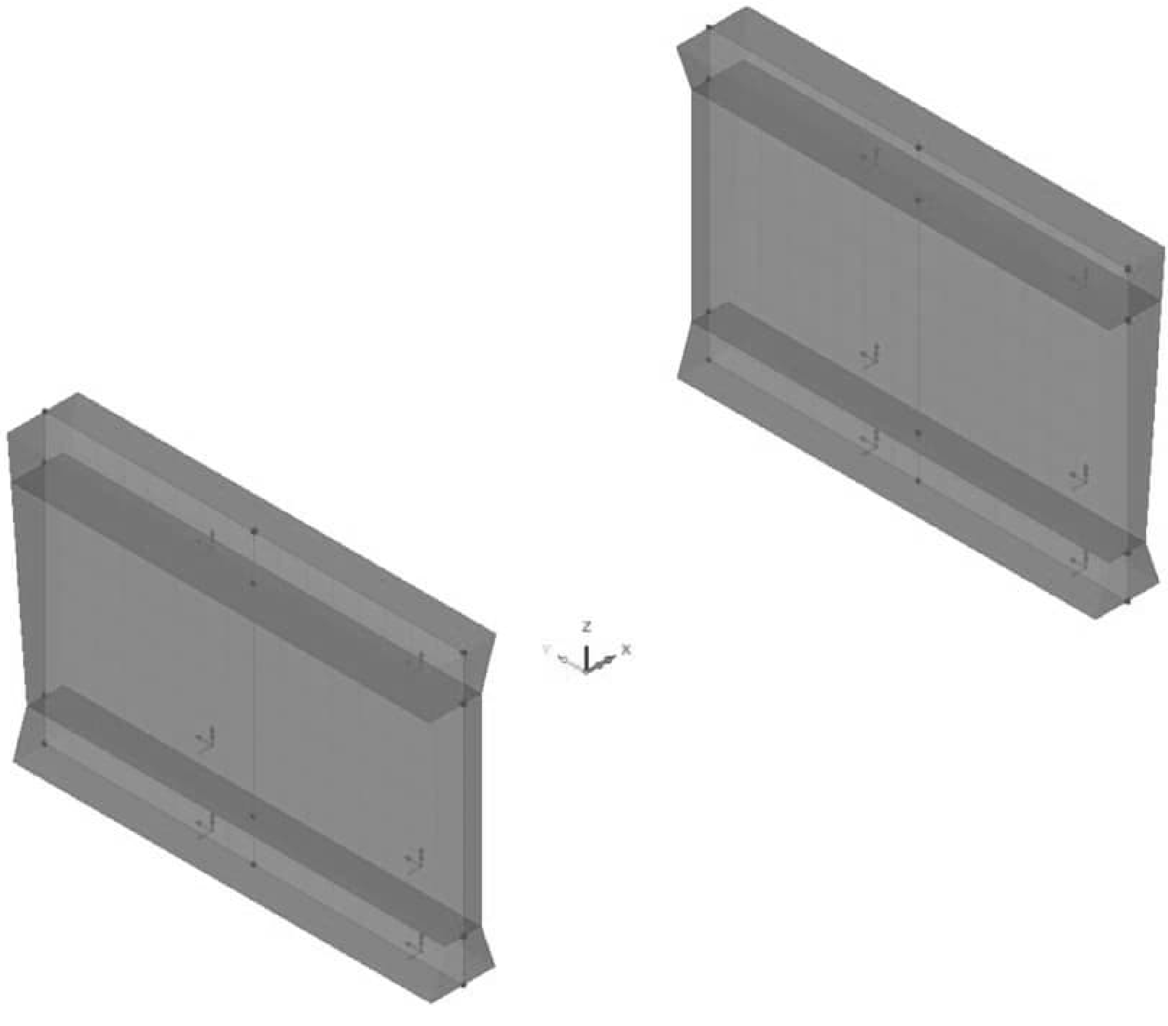
Surface function excentrisity :

Variation	Function(u,v)	Remark
e2	-0.152·u	Abutement 1
e3	-0.152+0.108·u	Abutement 1 conection superstructure
e4	0.308-0.058·u	Superstructure connection abutment 1
e5	0.250-0.250·u	Superstructure inclination area left side
e7	0.250·u	Superstructure inclination area right side
e8	0.250+0.058·u	Superstructure connection abutment 2
e9	0.152·u	Abutment 2
e10	0.152-0.108·u	Abutment 2 connection superstructure
-	m	-

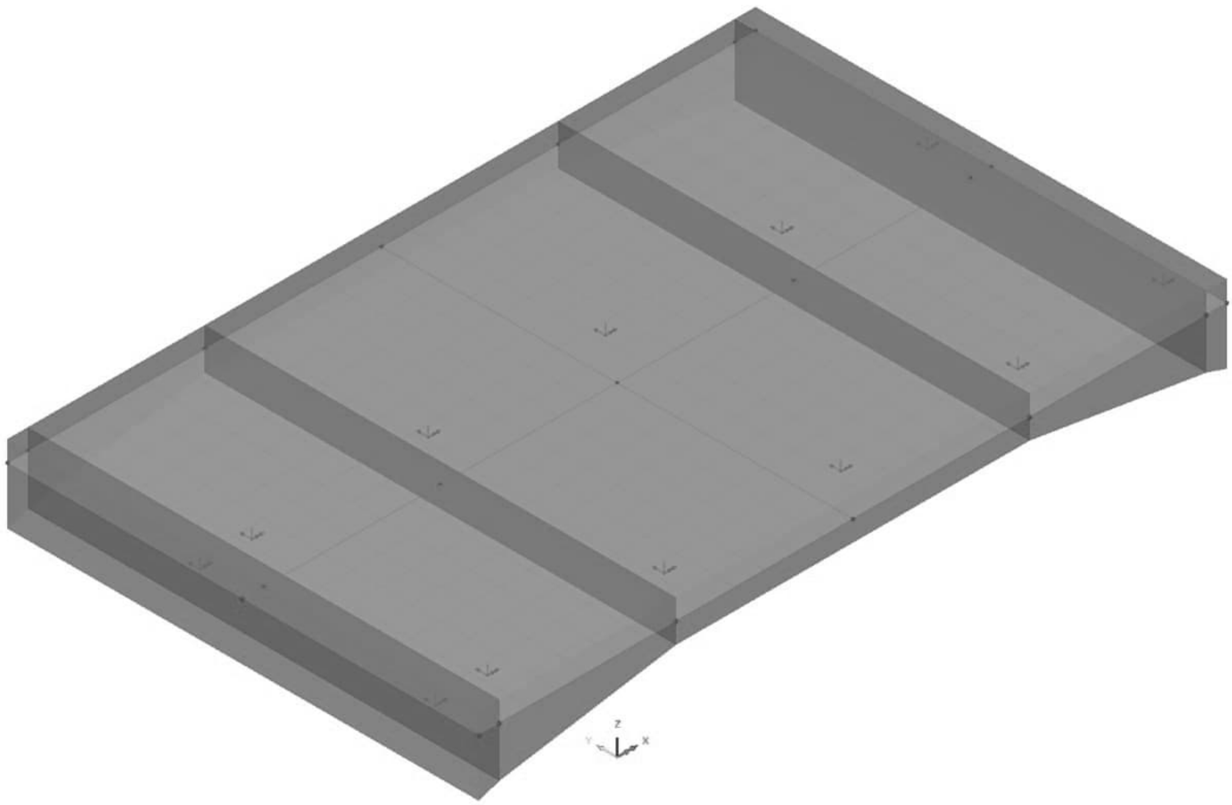
Surface geometry :

Attribute	t	ez	Remark
t1	*t1*	0	Bottomslab
t2	*t2*	e2	Abutement 1
t3	*t3*	e3	Abutement 1 conection superstructure
t4	*t4*	*e4*	Superstructure connection abutment 1
t5	*t5*	e5	Superstructure inclination area left side
t6	*t6*	*0*	Superstructure
t7	*t7*	e7	Superstructure inclination area left side
t8	*t8*	*e8*	Superstructure connection abutment 2
t9	*t9*	e9	Abutment 2
t10	*t10*	*e10*	Abutment 2 connection superstructure
t11	*0.400*	*0*	Linkplate
t12	*0.800*	*0*	Support area linkplate
-	m	m	-

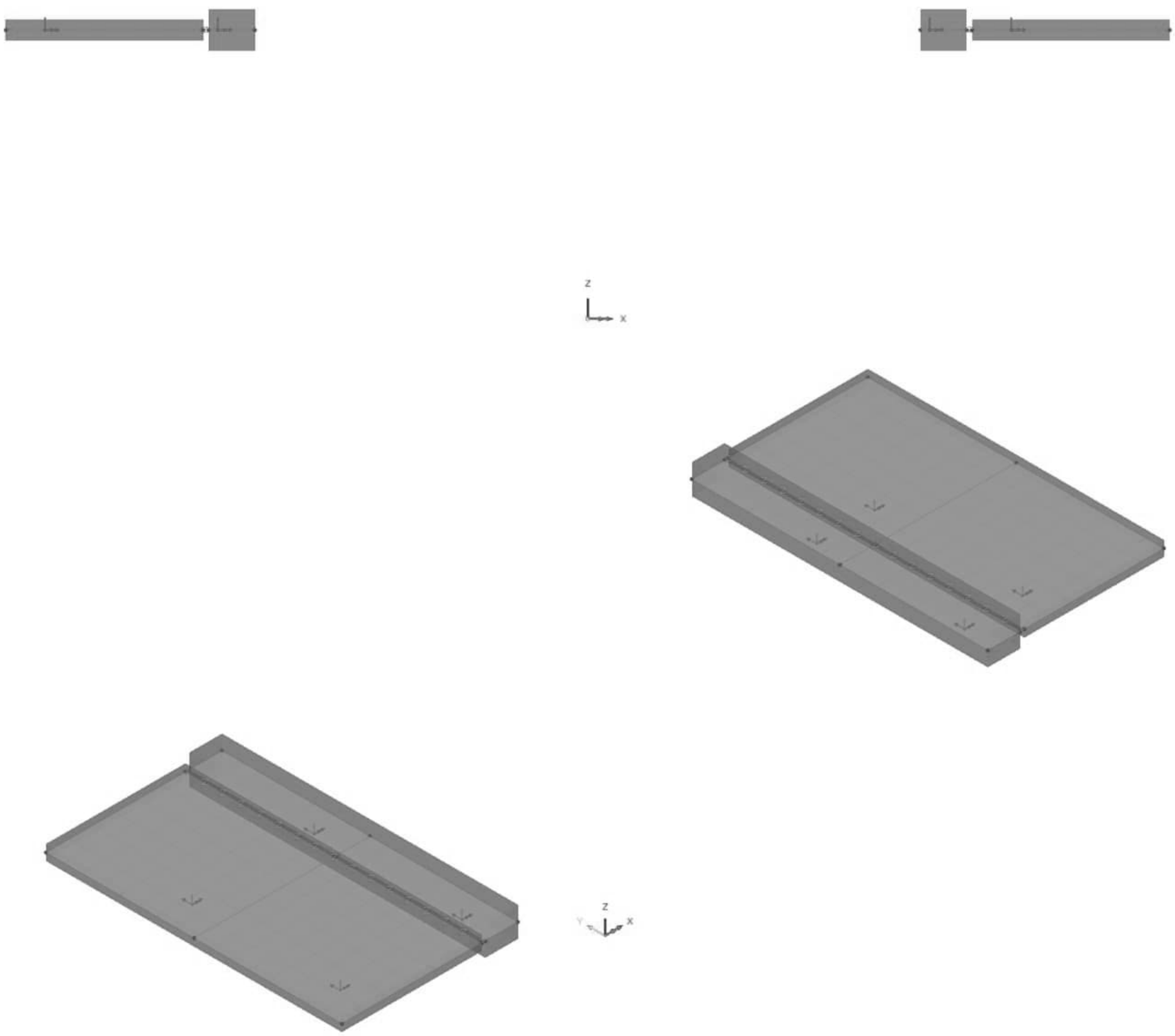
Abutment 1/2 :



Superstructure:

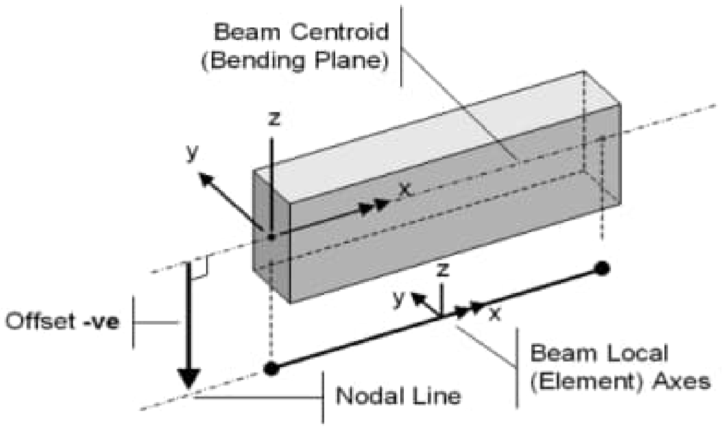


Link plates:

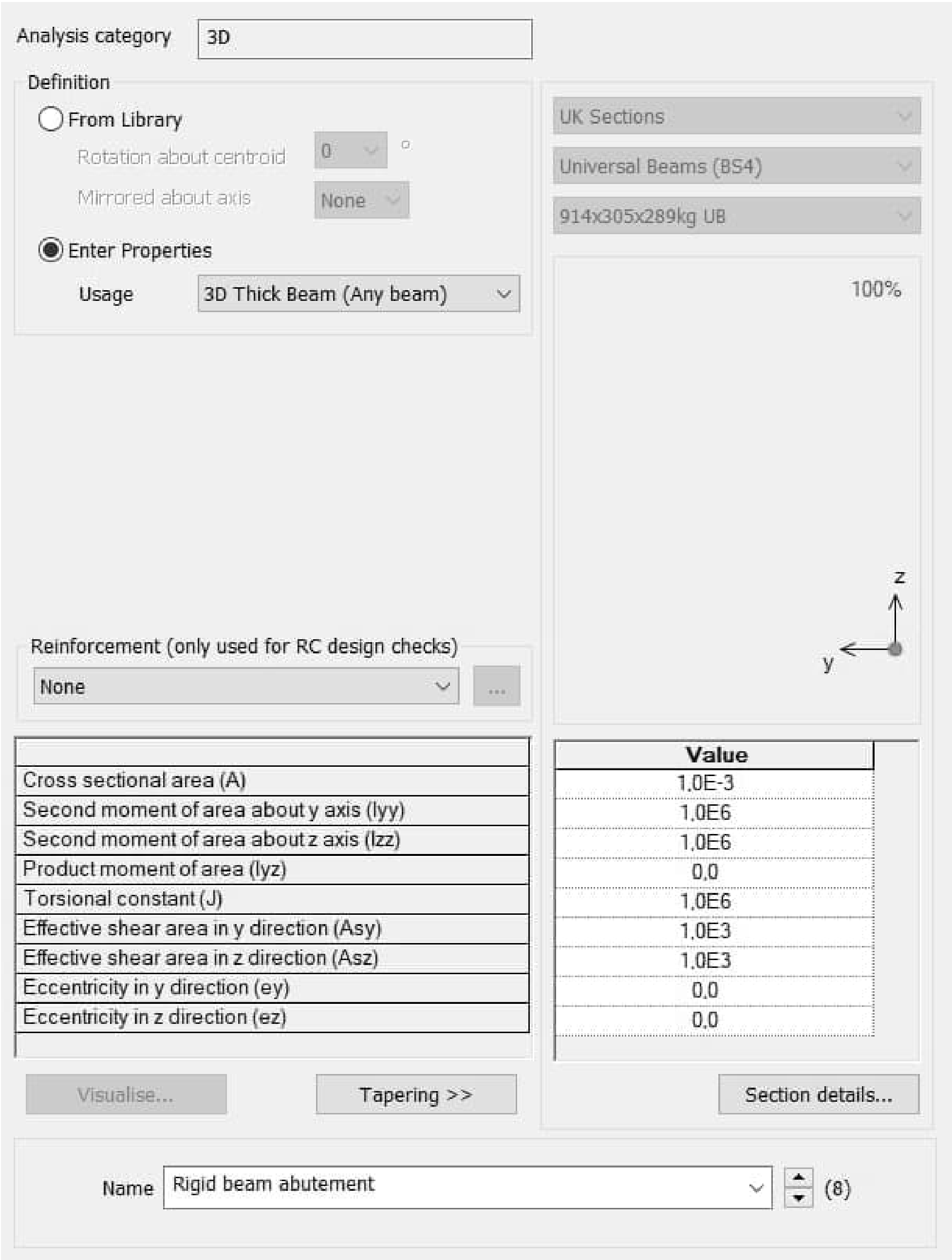


2.3.2 3D-beams (”Thick beam” / BMS3)

Principal sketch of geometry associated to 3D beam elements are seen below.



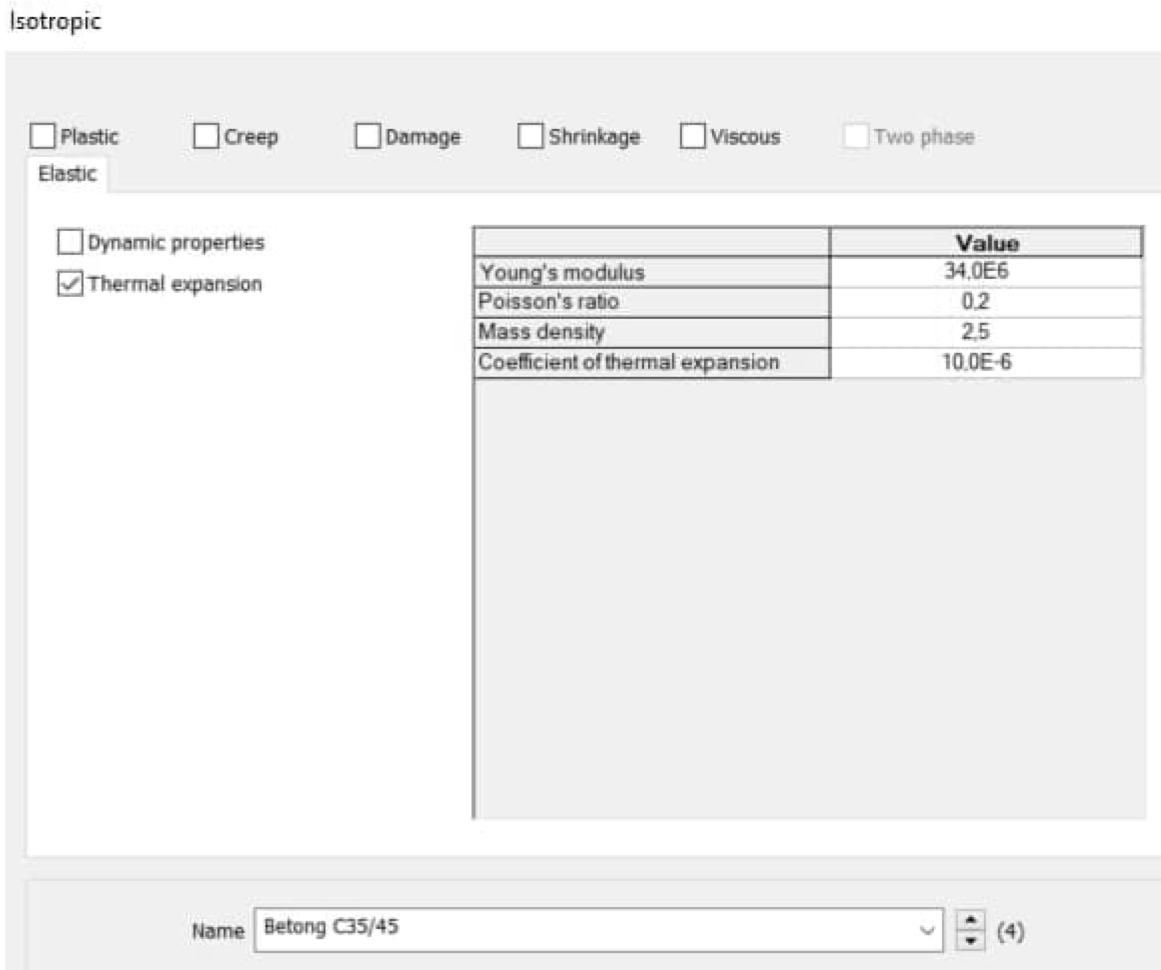
A fictive rigid beam is introduced at bottom of each abutment. The beam has infinite stiffness in all direction apart from axial direction. In this direction stiffness is negligible.



2.4 MATERIAL

Material properties seen below are to be used for all parts.

Substructure C35/45 : E_{cm} = 34 GPa

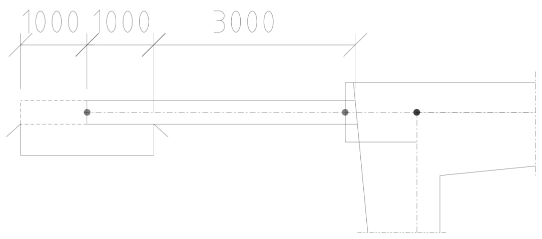


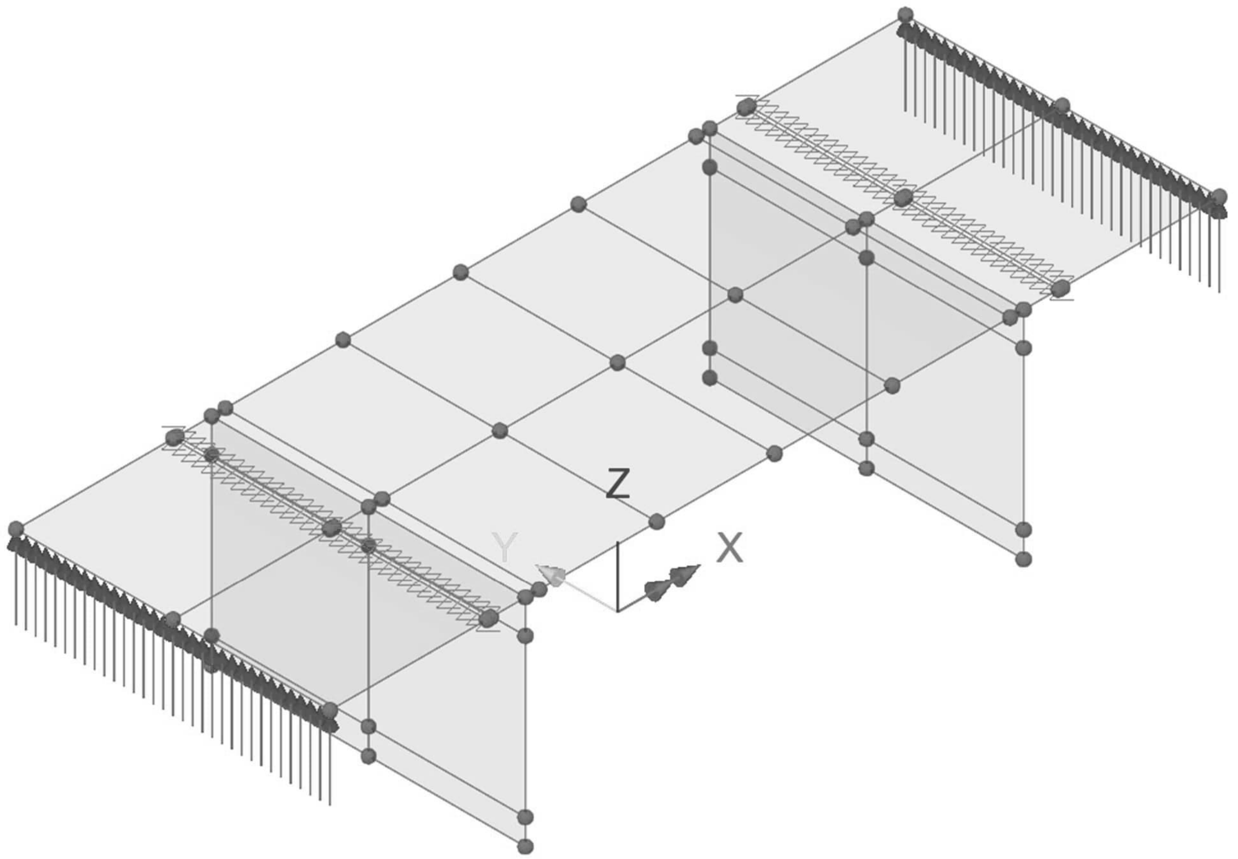
-

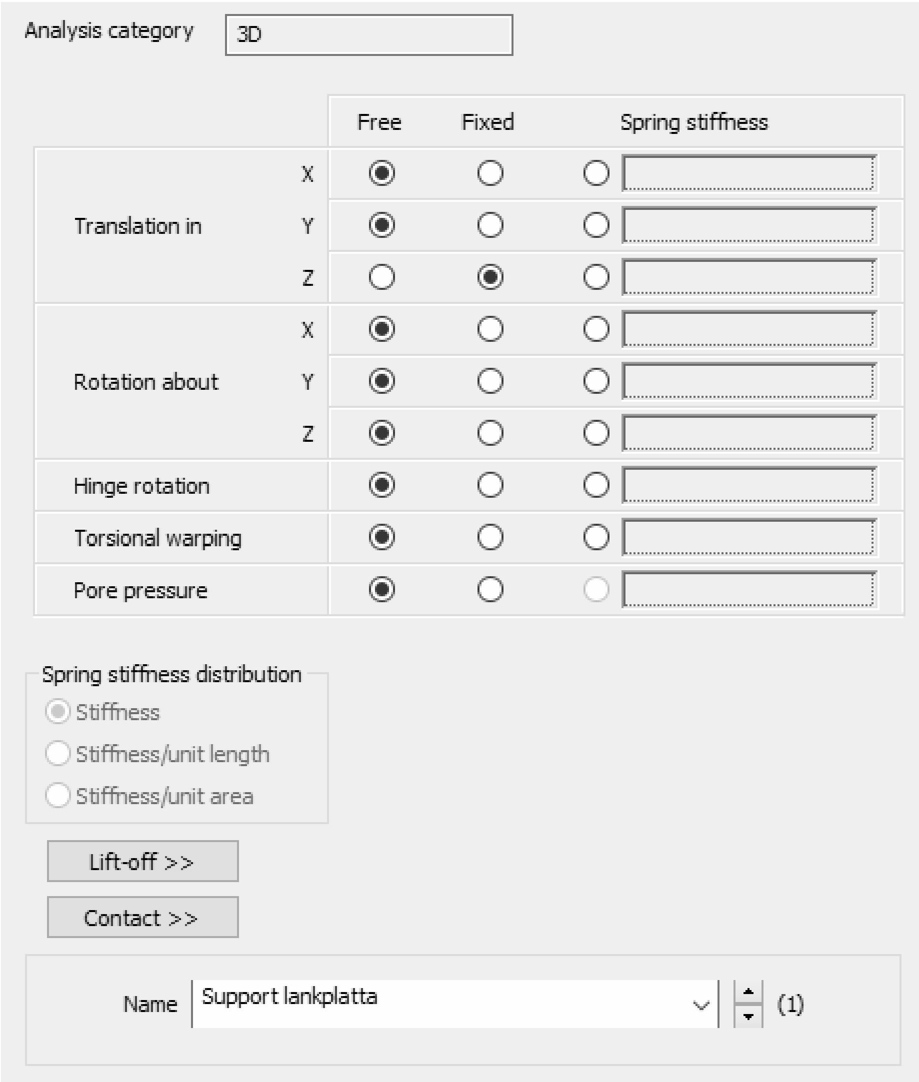
2.5 BOUNDARY CONDITIONS

2.5.1 Boundary conditions link slab

At a distance 1 m from edge of link slab att fictive line support is added in z-direction.





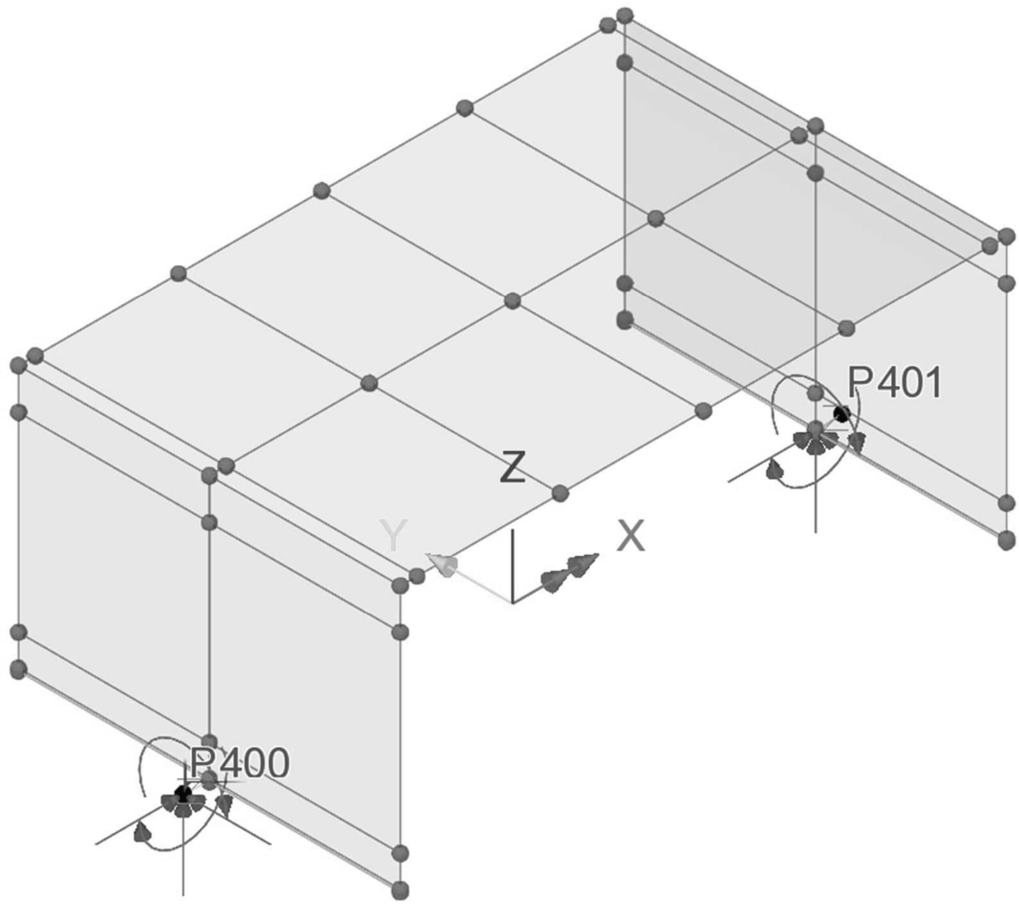


Analysis category
3D
Free
Fixed
Spring stiffness
Translation in
X
Y
Z
Rotation about
X
Y
Z
Hinge rotation
Torsional warping
Pore pressure
Spring stiffness distribution
Stiffness
Stiffness/unit length
Stiffness/unit area
Lift-off >>
Contact >>
Name
Support lankplatta
(1)

2.5.2 Boundary conditions abutments

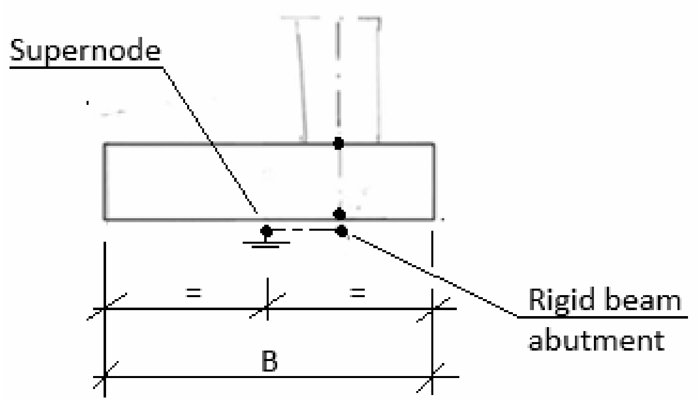
Boundary conditions for each support is modelled using super nodes.

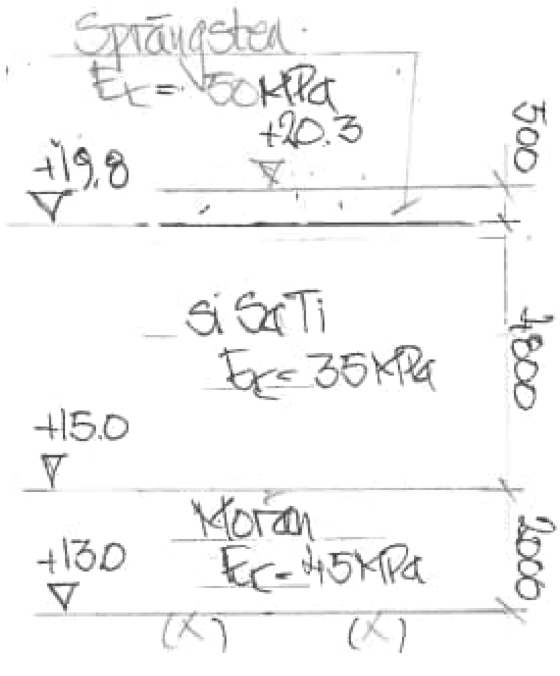
The super nodes are location at centre of rigid beam abutment, see sketches below.



Rotational stiffness of foundation is determined using software PROG G3.005.

Since the distance to solid ground (H) is less than twice the width of the bottom plate (2B), the method according to TRVINFRA-00227 appendix B5.1 since H > 2B.





Stiffness transversal direction (Rotation X-X) :

$K_{Rx} = 678000 \frac{kNm}{rad}$: page A2:33

Stiffness longitudinal direction (∴ Rotation Y-Y) :

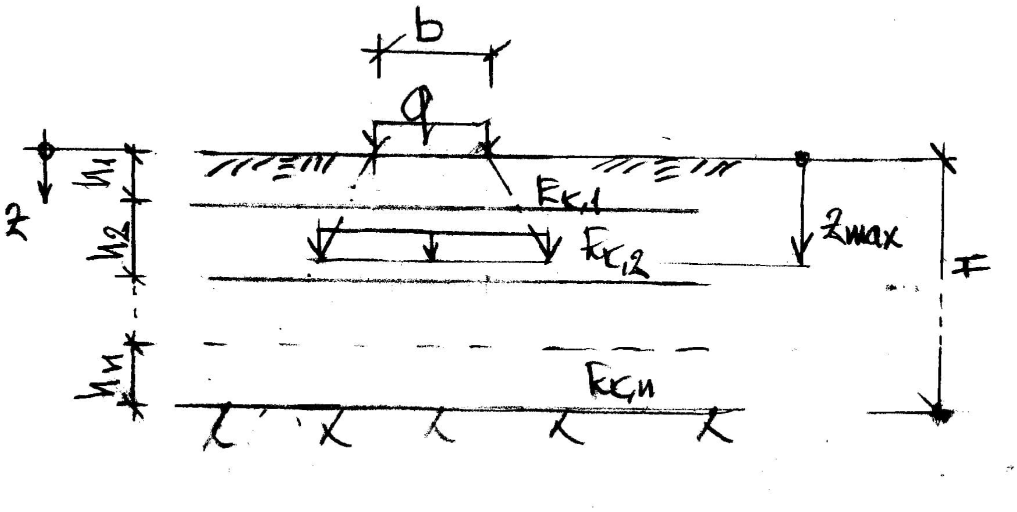
$K_{Ry} = 136000 \frac{kNm}{rad}$: page A2:33

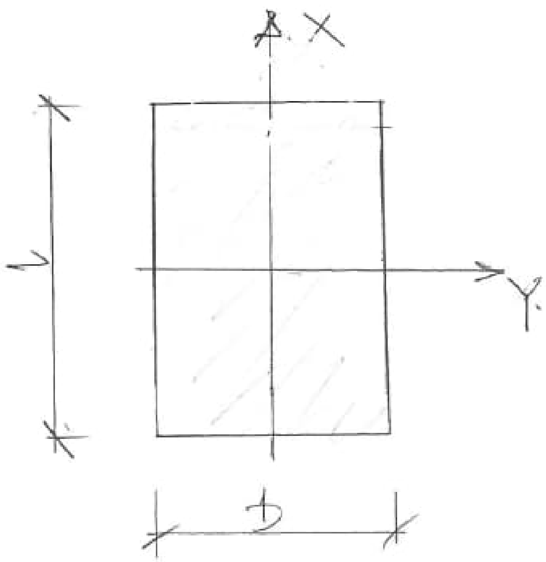
Object : Support 1/2

PRINCIPFIGUR

Geometri och undergrund

Beräkningen av ekivalent styvhet i grund är utförd enligt Råd Brobyggande bilaga 106. Den ekvivalenta E-modulen förutsätter lastspridning 2:1. Bestämningen har skett för en fiktiv lasten motsvarande q (= 100 kPa) .

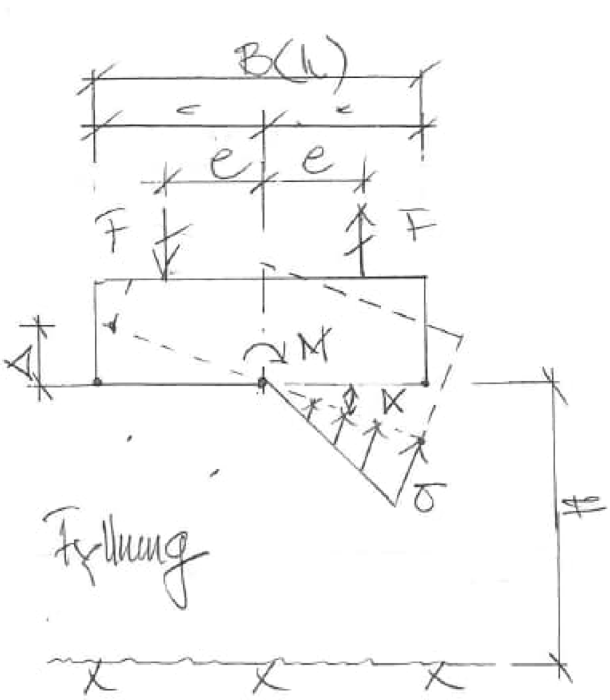




TEORI

När avståndet till fast botten är mindre än 2B så tilllämpas härled formel nedan som även förekommer i BH sida 594 (avsnitt 6.4:22 Beräkningsmodell).

När avstånd till fast botten är större än 2B tillämpas Råd Brobyggnade bilaga 107.



$$\Delta = \alpha \cdot \frac{B}{2} \;; \qquad \varepsilon = \frac{\Delta}{H} = \frac{\alpha \cdot \frac{B}{2}}{H}$$

$$\sigma = \varepsilon \cdot E'_k = \frac{E'_k \cdot \alpha \cdot B}{2 \cdot H}$$

$$F = \sigma \cdot \frac{B}{2} \cdot \frac{1}{2} \cdot L = \frac{E'_k \cdot \alpha \cdot B}{2 \cdot H} \cdot \frac{B \cdot L}{4} = \frac{E'_k \cdot \alpha \cdot B^2 \cdot L}{8 \cdot H}$$

$$M = F \cdot 2e \;; \qquad e = \frac{B}{3} \;;$$

$$M = F \cdot \frac{2 \cdot B}{3} = \alpha \cdot \frac{L \cdot B^3 \cdot E'_k}{12 \cdot H}$$

$$\Rightarrow \frac{\alpha}{M} = \frac{12 \cdot H}{L \cdot B^3 \cdot E'_k}$$

INDATA

Geometri

Bottenplatta : $b = 2.8m$ $l = 8.5m$

Underliggande jordmaterial

Antal skikt (minst 2 skickt erfordras): $n = 3st$

Skikt	E_k	h
1	50	0,50
2	35	4,80
3	45	2,00
-	MPa	m

BERÄKNINGAR

Total skikttjocklek

$$H = \sum_{i=1}^{n} h_i$$

$$H = 7.3\,m$$

Sättningsområde

$$z_{max} = \min(2 \cdot b, H)$$

$$z_{max} = 3.4\,m$$

Nivåer för respektive skikt

$$z_s = \left| \begin{array}{l} \text{for } i \in 1..n \\ \quad \left| \begin{array}{l} z_{2 \cdot i} \leftarrow z_{2 \cdot i - 1} + h_i - 1mm \\ z_{2 \cdot i + 1} \leftarrow z_{2 \cdot i} + 1mm \end{array} \right. \\ z \end{array} \right.$$

$$z_s = (0.000 \quad 0.500 \quad 0.501 \quad 5.300 \quad 5.301 \quad 7.300 \quad 7.301)\,m$$

Funktion för sättningsmodul

$$E_{sk} := \left| \begin{array}{l} \text{for } i \in 1..n-1 \\ \quad \left| \begin{array}{l} E_{2\cdot i} \leftarrow E_{k_i} \\ E_{2\cdot i+1} \leftarrow E_{k_{i+1}} \end{array} \right. \\ E \end{array} \right.$$

$$E_{sk} = (50 \;\; 50 \;\; 35 \;\; 35 \;\; 45 \;\; 45 \;\; 1000)\cdot MPa$$

$$E_k := \text{linterp}\left(z_s, E_{sk}, z\right)$$

Påkänningar enligt 2:1

$$q = 100kPa$$

$$\Delta\sigma_v = q\cdot\frac{b\cdot l}{(b+z)\cdot(l+z)}$$

Karakteristisk sättning

$$s_k = \int_{0m}^{H} \frac{\Delta\sigma_v}{E_k}\,dz \qquad s_k = 5.9\cdot mm$$

Ekvivalent sättningsmodul

$$E'_k = \frac{\int_{0m}^{z_{max}} \Delta\sigma_v\,dz}{s_k} \qquad E'_k = 28\cdot MPa$$

Funktion för styvhet grunden när H < 2B
(Se härledning avsnitt TEORI)

$$k_{\theta k}(B, L) = \frac{L\cdot B^3\cdot E'_k}{12H}$$

RESULTAT

Resultat då H < 2B :

Rotation kring plattans korta riktning (x-x riktning):

$$k_{\theta k}(b,l) = 13167 \cdot \frac{kNm}{rad}$$

$$C_{\phi} = \frac{1}{k_{\theta k}(b,l)} \qquad 10^9 \cdot C_{\phi} = 75950 \cdot \frac{rad}{kNm}$$

Rotation kring plattans långa riktning (y-y riktning):

$$k_{\theta k}(l,b) = 329163 \cdot \frac{kNm}{rad}$$

$$C_{\eta} = \frac{1}{k_{\theta k}(l,b)} \qquad 10^9 \cdot C_{\eta} = 3038 \cdot \frac{rad}{kNm}$$

Resultat då H > 2B :

Rotation kring plattans korta riktning (x-x riktning):

$$K_{\phi} = E'_k \cdot \frac{b^2 \cdot l}{5} = 135692 \cdot \frac{kNm}{rad}$$

$$C_{\phi} = \frac{1}{K_{\phi}} \qquad 10^6 \cdot C_{\phi} = 7.370 \cdot \frac{rad}{kNm}$$

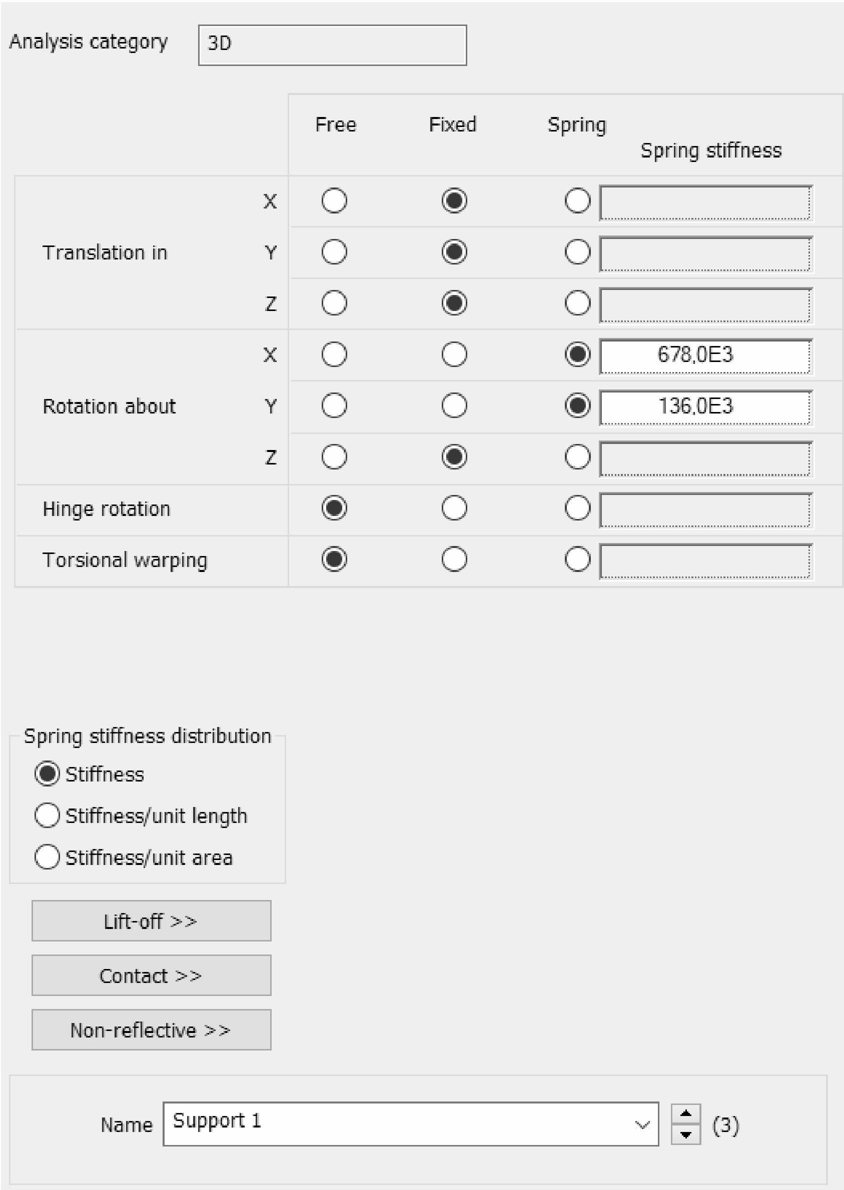
Rotation kring plattans långa riktning (y-y riktning):

$$K_{\eta} = E'_k \cdot \frac{l^2 \cdot b}{5} = 678462 \cdot \frac{kNm}{rad}$$

$$C_{\eta} = \frac{1}{K_{\eta}} \qquad 10^6 \cdot C_{\eta} = 1.474 \cdot \frac{rad}{kNm}$$

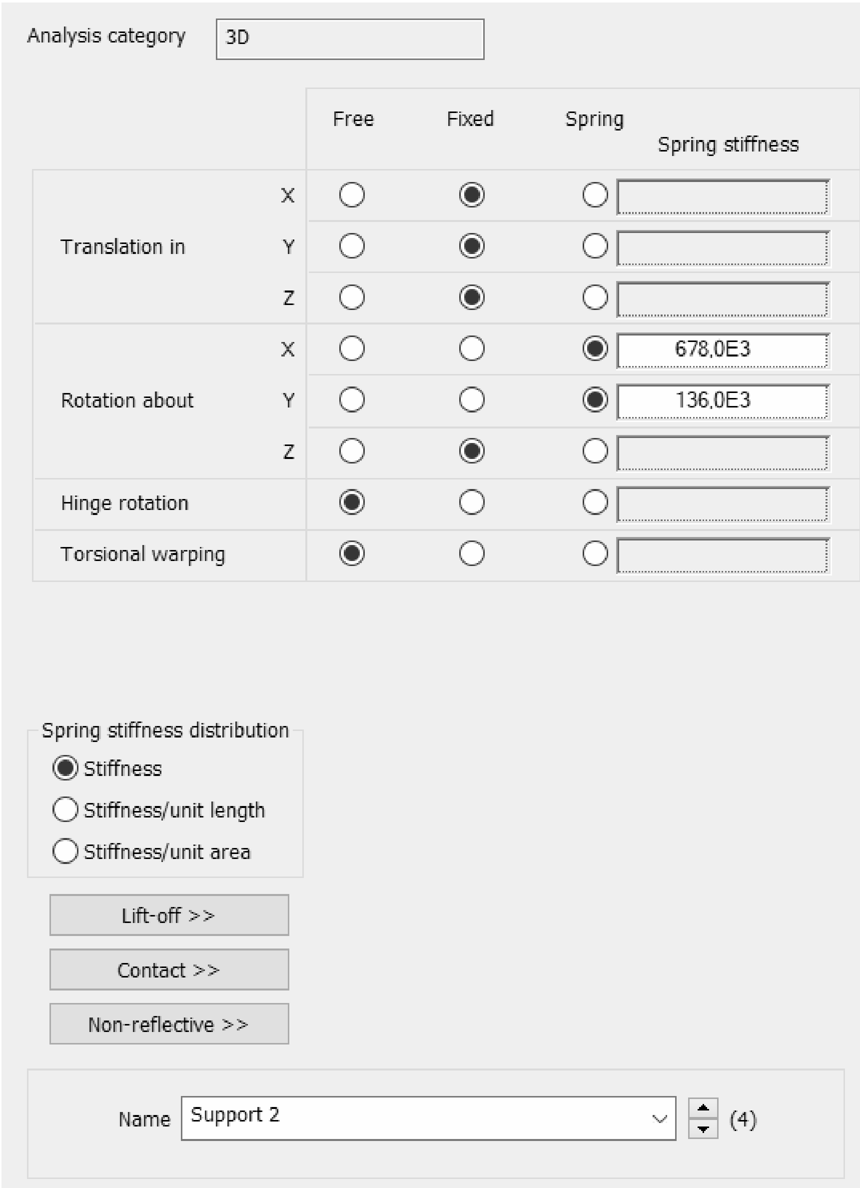
2.5.2.1 Boundary support 1

“Super node” at support 1 is modelled as seen below. This super node is termed point P221 in model of system analysis.



2.5.2.2 Boundary support 2

“Super node” at support 2 is modelled as seen below. This super node is termed point P231 in model of system analysis.



2.6 MESH

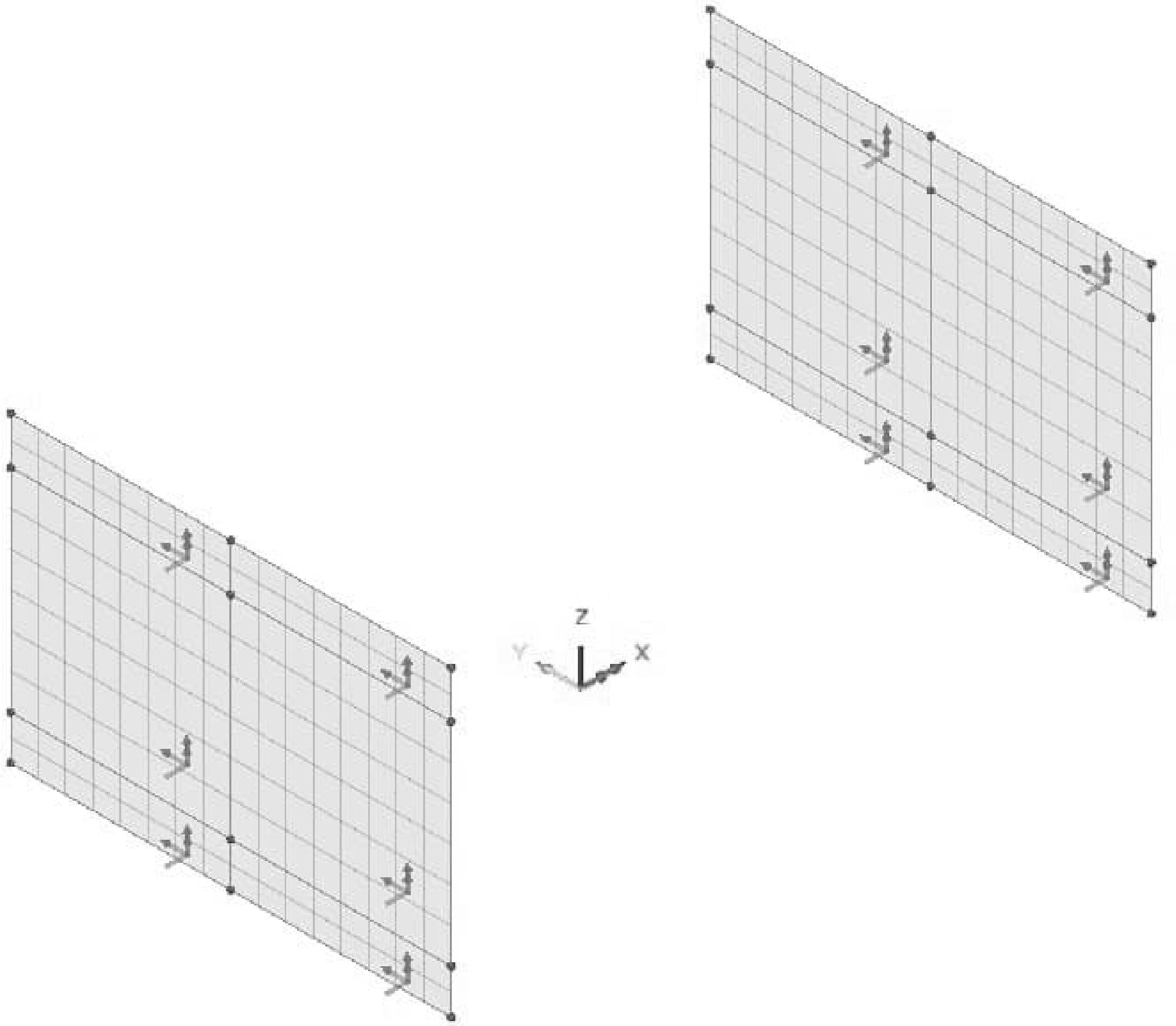
2.6.1 Shell element (QTS8): linear

Bridge is modelled using shell elements.

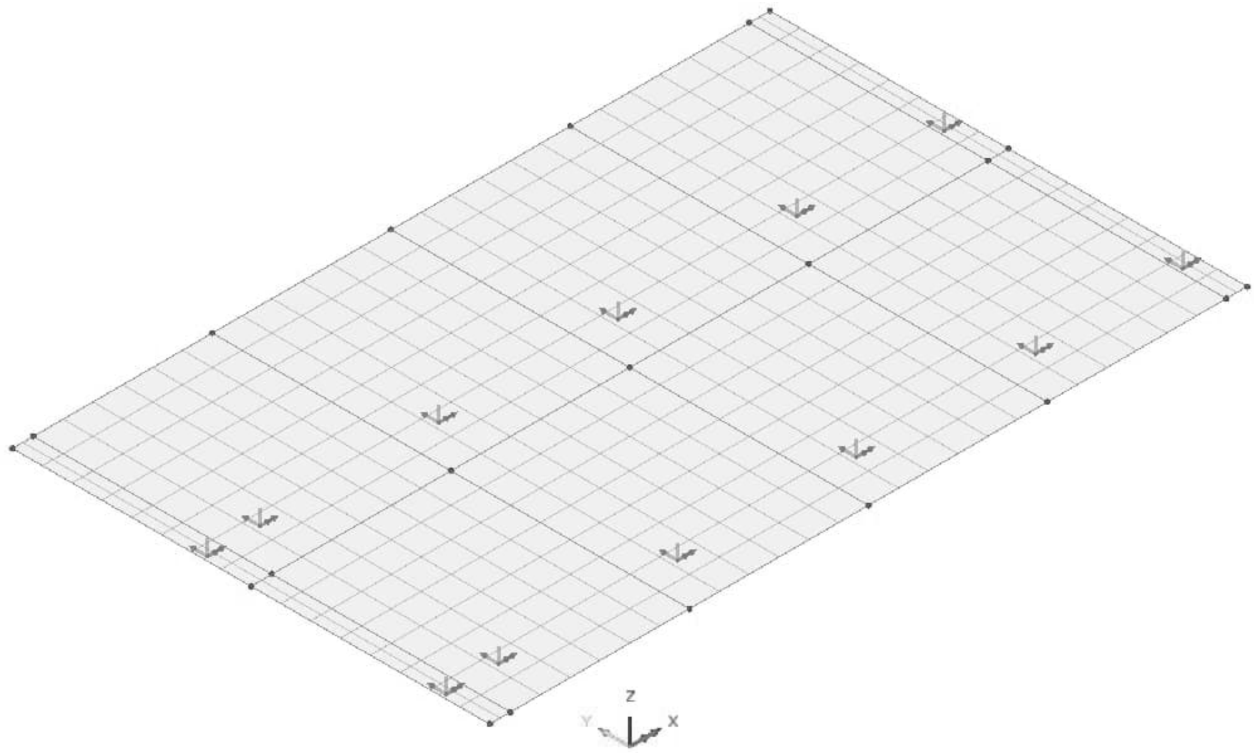
High node elements are chosen (”Thick shell” / QTS8) to limit the number of elements while maintaining accuracy.

Type	x-divisions	y-divisions
Element 2 x 8	2	8
Element 6 x 8	6	8

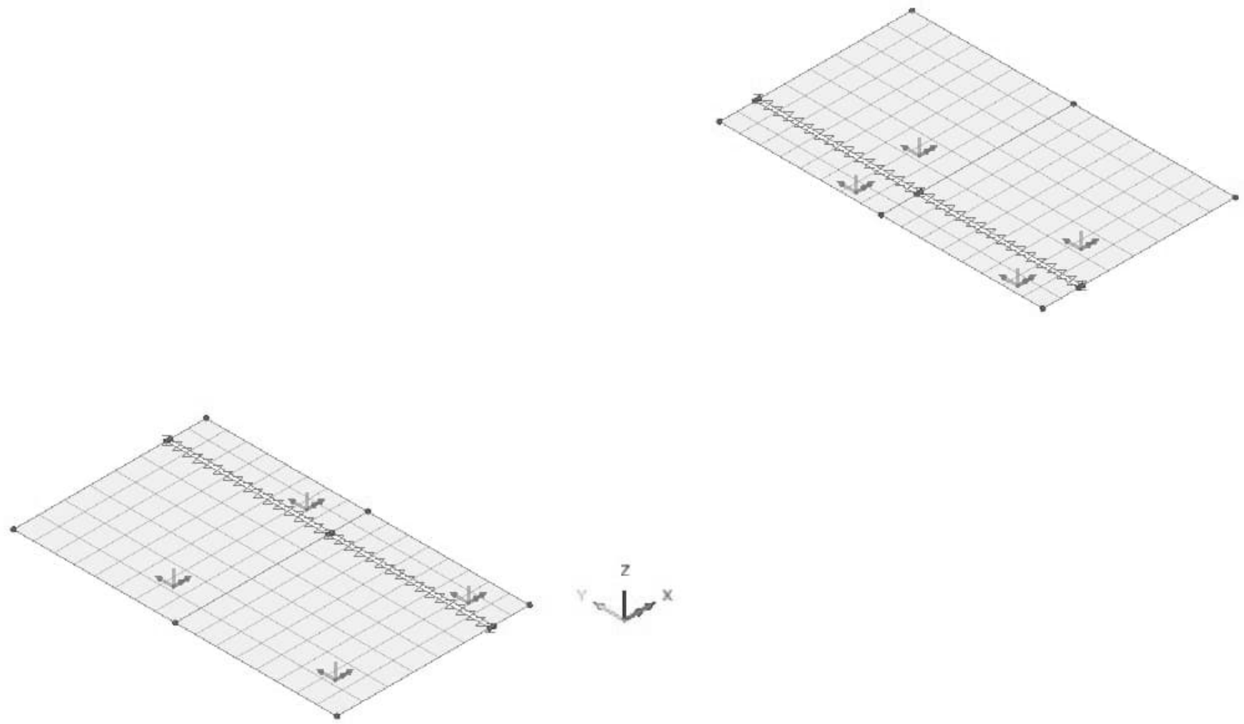
Abutment 1 & 2:



Bridge deck:



Linkplates :

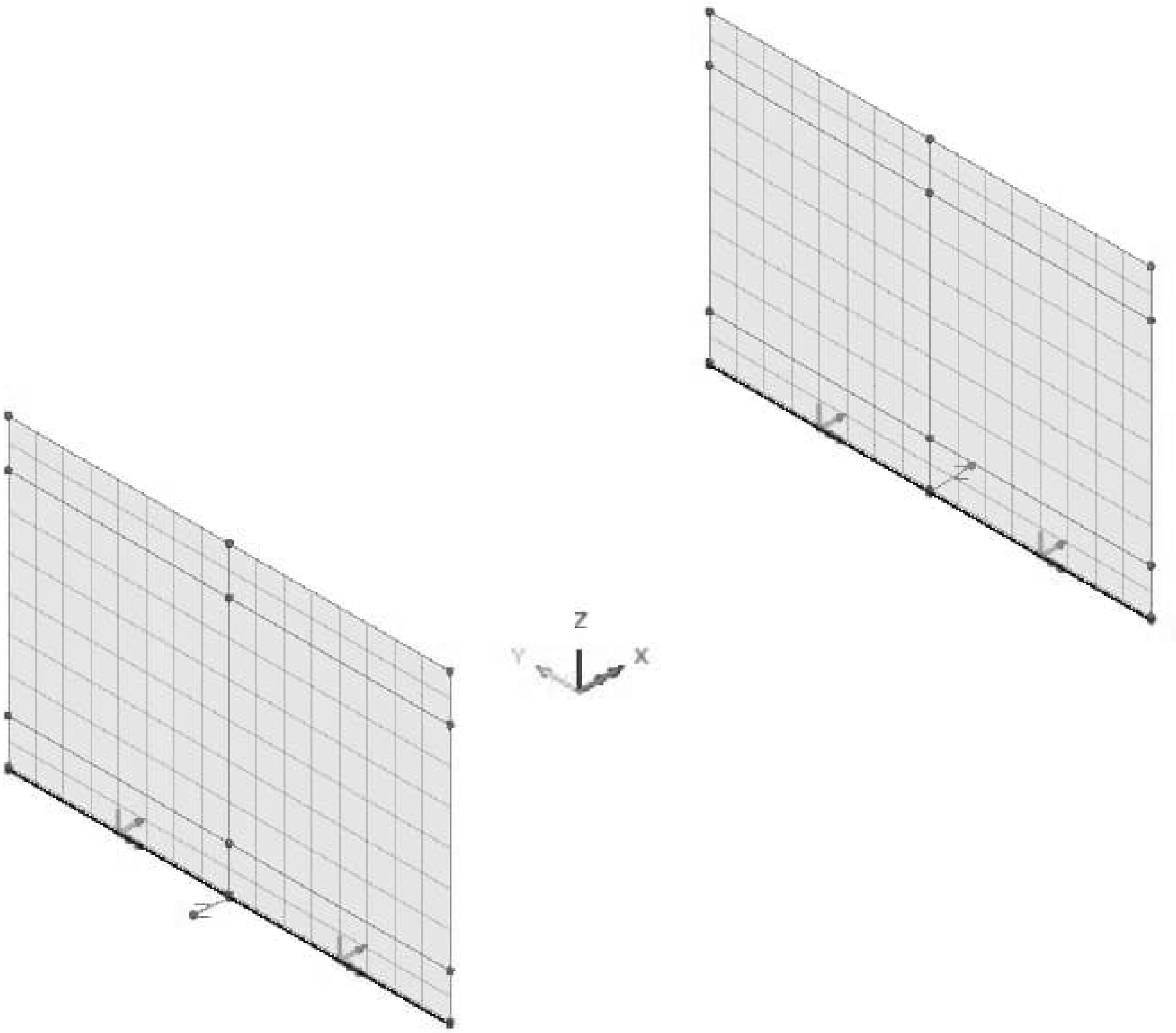


2.6.2 Beam element (BMI21) : linear

Studied bridge uses beam elements (” Beam element” / BMI21) for support beam.

Type	Divisions	End release: Start	End release: End	Structure
Element 8	8	None	None	Rigid support beam

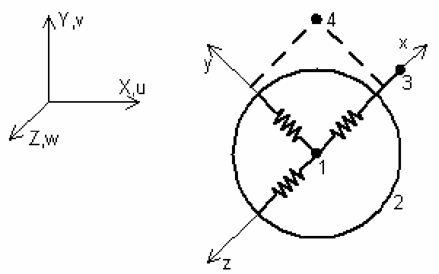
Rigid beam :



2.6.3 Joint element for between features (JNT4) : No rotational stiffness

Connection between link slab & superstructure is of type joint element, see presentation below. The joint uses no rotational stiffness in order achieve a hinge.

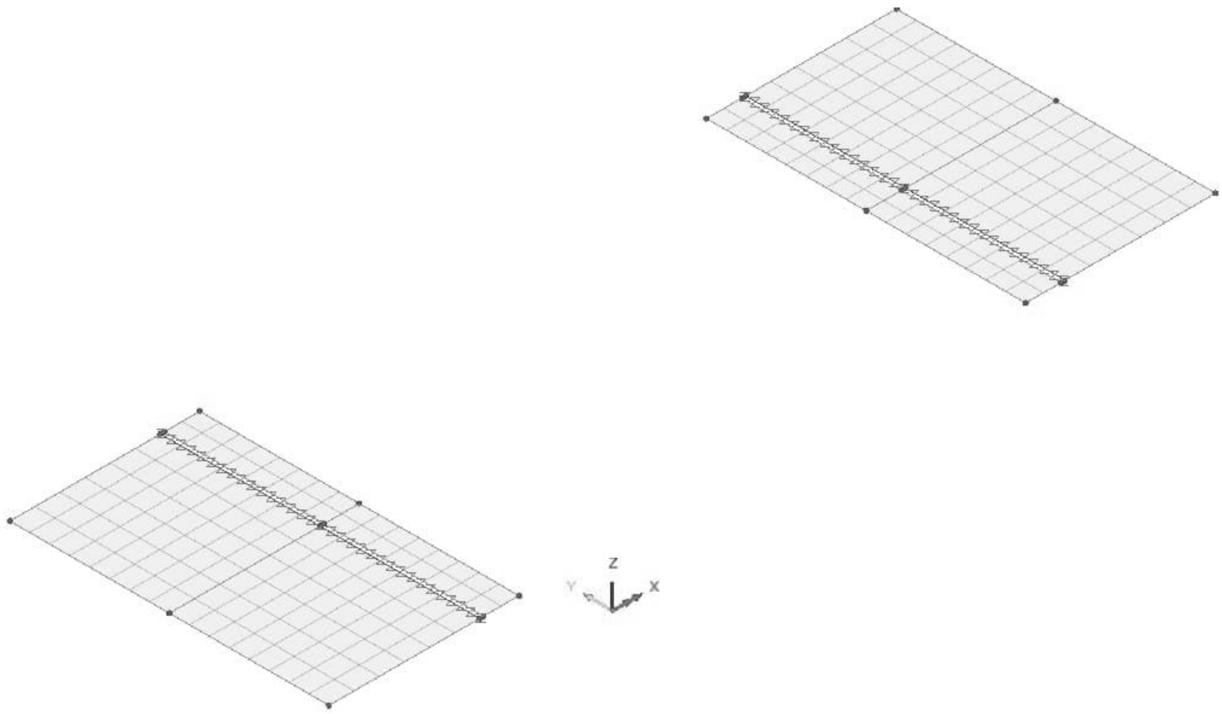
Element Name	**JNT4**



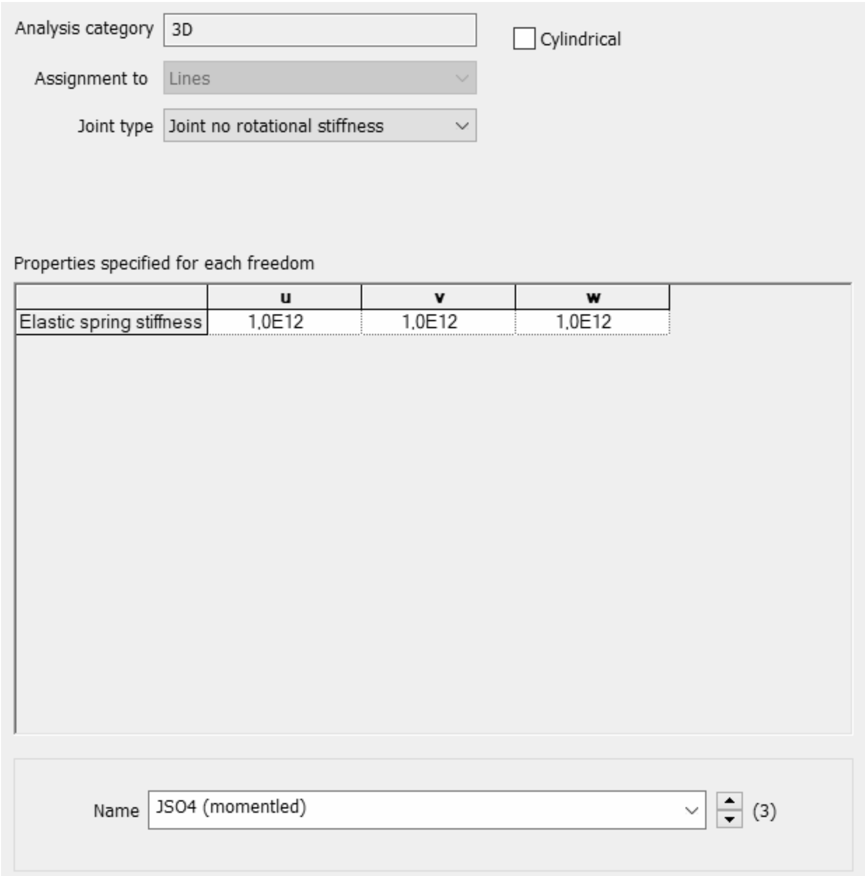
Element Group	Joints
Element Subgroup	3D Joints
Element Description	A 3D joint element which connects two nodes by three springs in the local x, y and z-directions.
Number Of Nodes	4. The 3rd and 4th nodes are used to define the local x-axis and local xy-plane.
Freedoms	U, V, W: at nodes 1 and 2 (active nodes).
Node Coordinates	X, Y, Z: at each node.

Link plate is termed *Master* while ridge deck is termed *Slave.*

Overview:



Joint material:

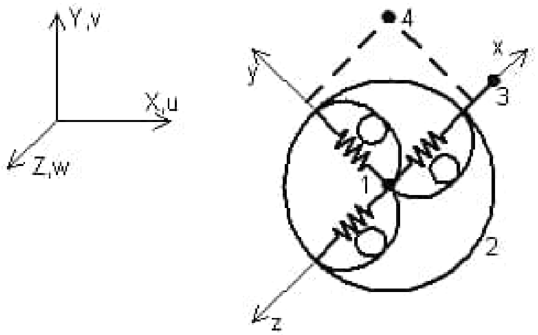


	u	v	w
Elastic spring stiffness	1,0E12	1,0E12	1,0E12

2.6.4 Joint point element for beams (JSH4)

Connection between link slab & superstructure is of type joint element, see presentation below. The joint uses no rotational stiffness in order achieve a hinge.

Element Name	**JSH4, JL46**



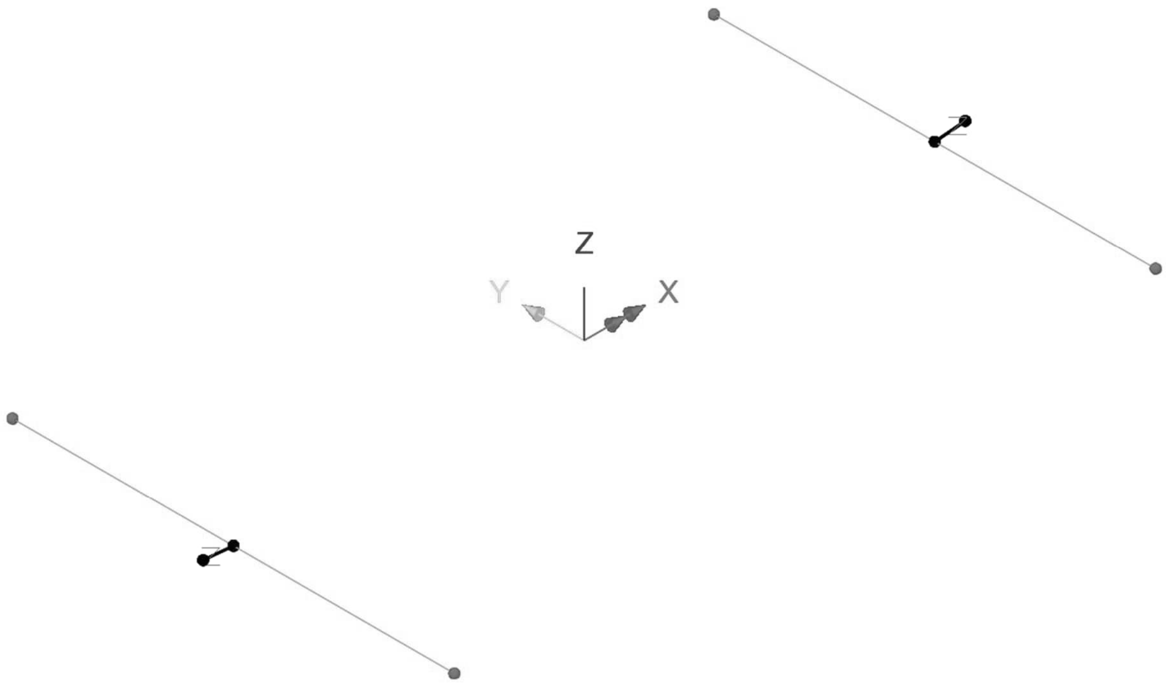
Element Group	Joints
Element Subgroup	3D Joints
Element Description	3D joint elements which connects two nodes by six springs in the local x, y and z-directions.
Number Of Nodes	4. The 3rd and 4th nodes are used to define the local x-axis and local xy-plane respectively.
Freedoms	U, V, W, θx, θy, θz: at nodes 1 and 2 (active nodes).
Node Coordinates	X, Y, Z: at each node.

Geometric Properties

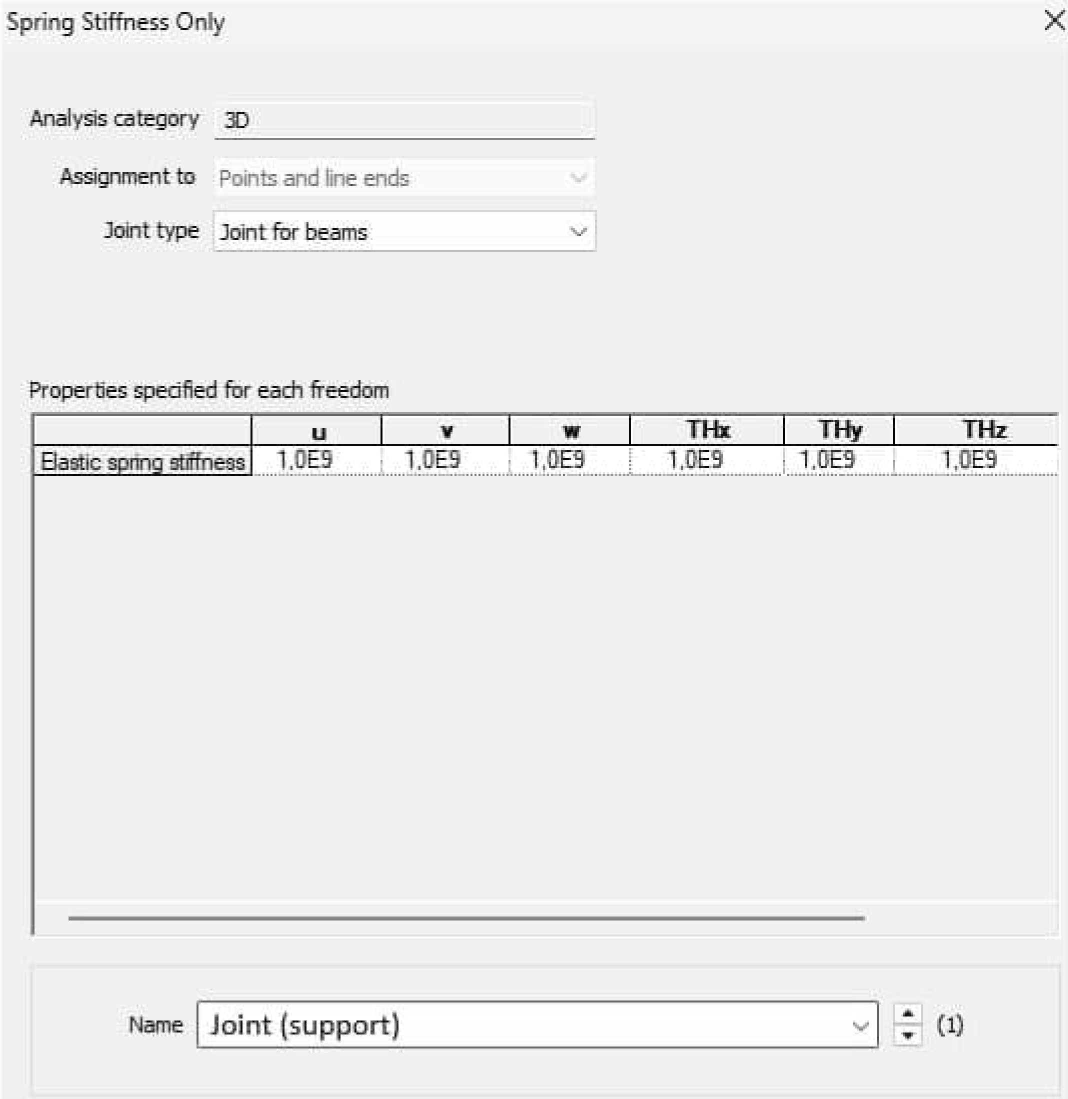
ez	Eccentricity measured from the joint xy-plane to the nodal line.
dy	Parametric distance factor (between 0.0 and 1.0), which defines the position of the shear spring for the local y direction between nodes 1 and 2. It is measured from node 1 (dy=0) along the local x direction.
dz	Parametric distance factor (between 0.0 and 1.0), which defines the position of the shear spring for the local z direction between nodes 1 and 2. It is measured from node 1 (dz=0) along the local x direction.

“Super nodes” are termed *Master* while centre of rigid beam is termed *Slave*.

Overview:



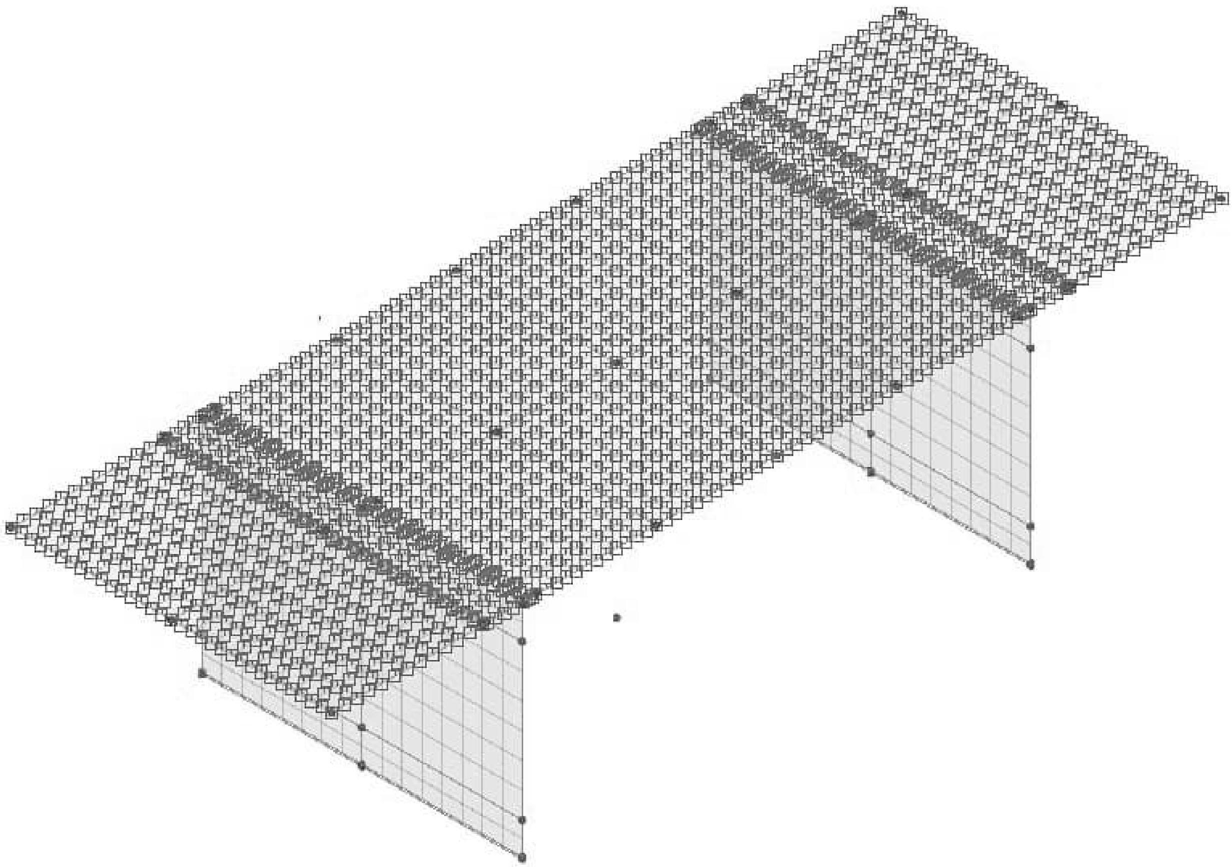
Joint material:



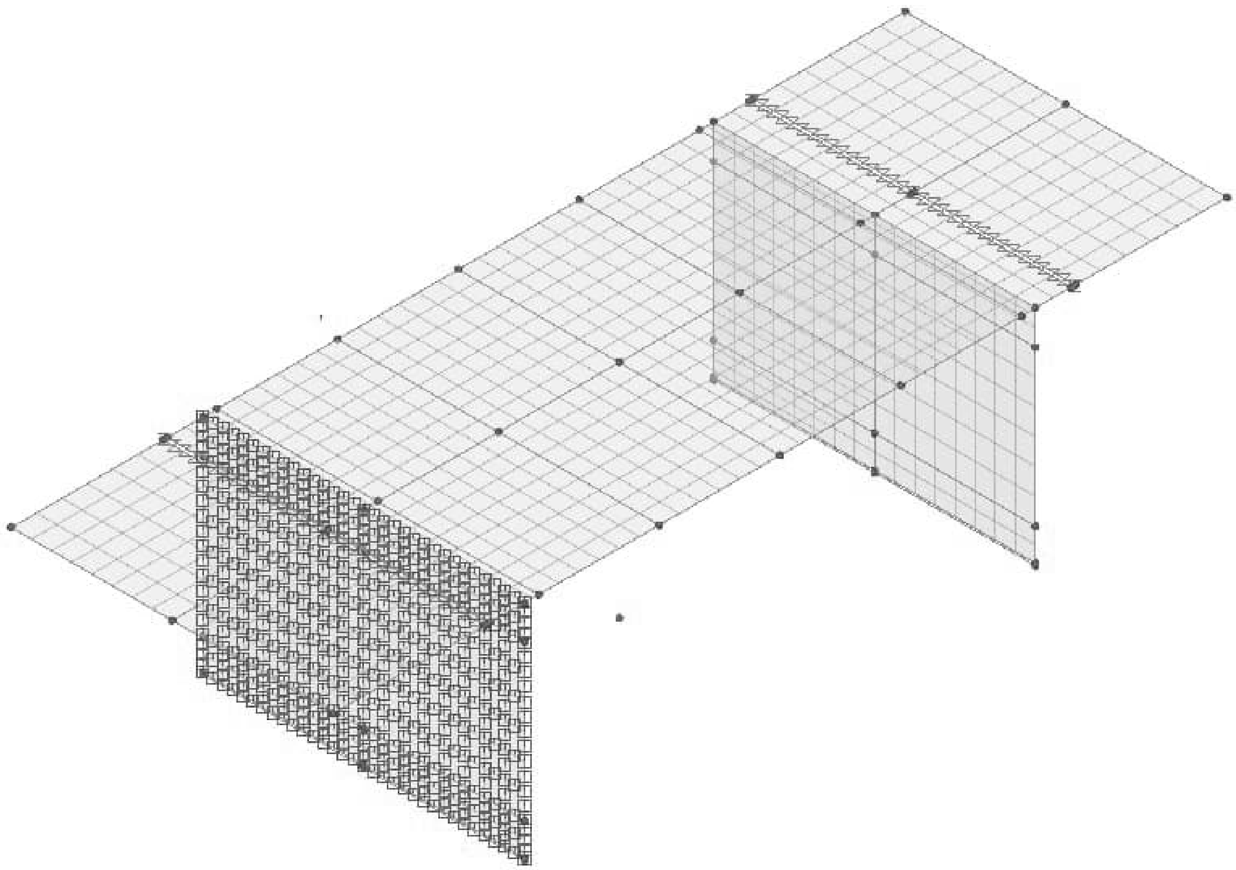
2.7 SEARCH AREA

Discrete load can be applied to structure as geometrical load areas. In FEM-program load areas are termed Search Area.

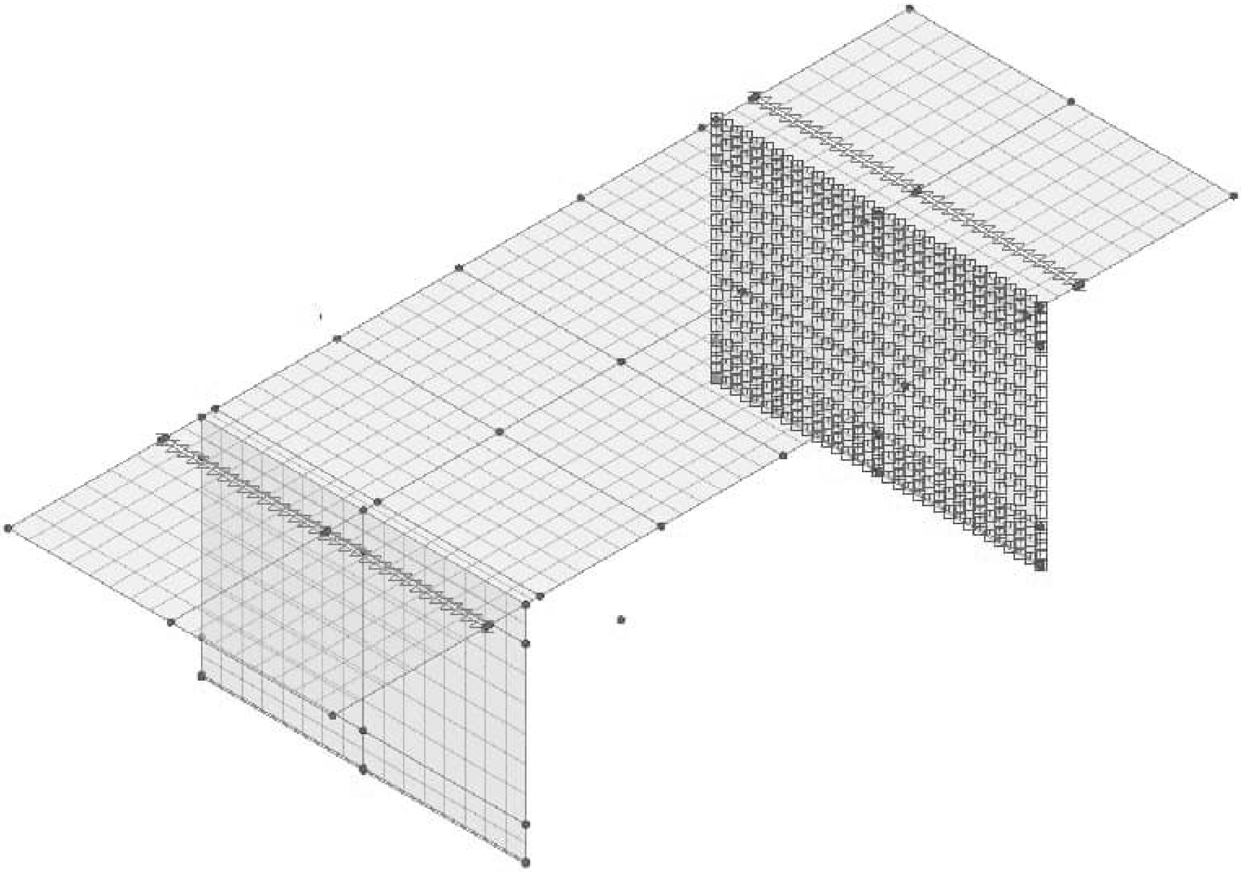
2.7.1 Search area : Superstructure



2.7.2 Search area : Abutment 1



2.7.3 Search area : Abutment 2



3. LASTER

3.1 DEAD WEIGHT

$\gamma_c = 25 \cdot \frac{kN}{m^3}$: concrete

3.1.1 Abutments

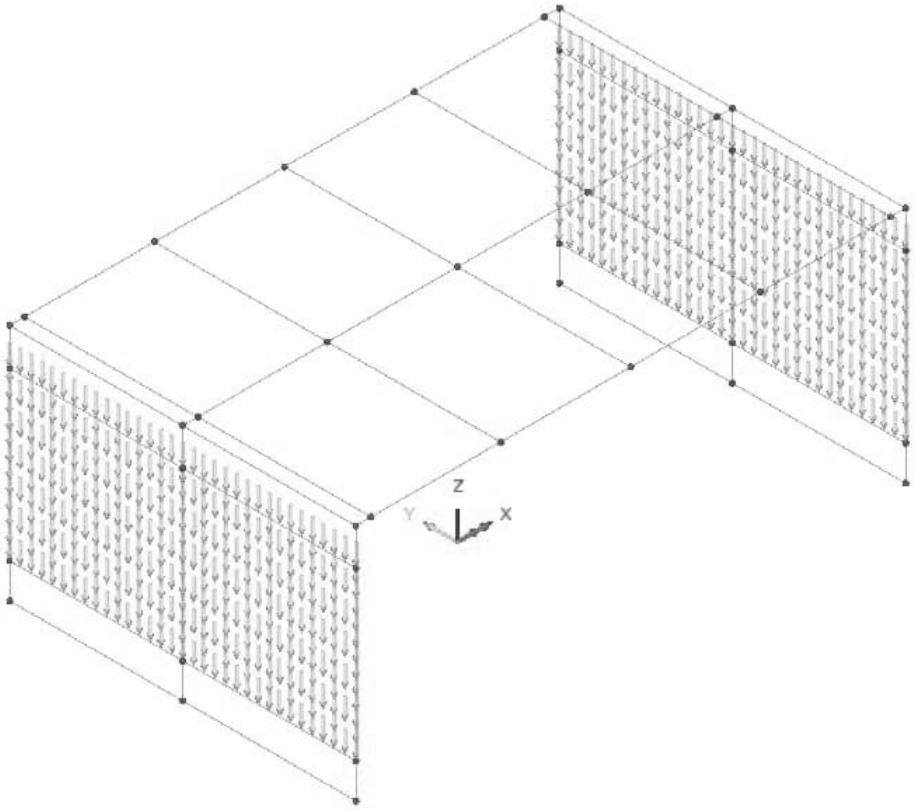
Load case : EGEN 1

Structural loading : Body force

Linear acceleration in Z (a_z) : -10 $\frac{m}{s^2}$

Component	Value
Linear acceleration in X	0,0
Linear acceleration in Y	0,0
Linear acceleration in Z	-10,0
Angular velocity about X axis	0,0
Angular velocity about Y axis	0,0
Angular velocity about Z axis	0,0
Angular acceleration about X axis	0,0
Angular acceleration about Y axis	0,0
Angular acceleration about Z axis	0,0

Name EGEN 1 (2)



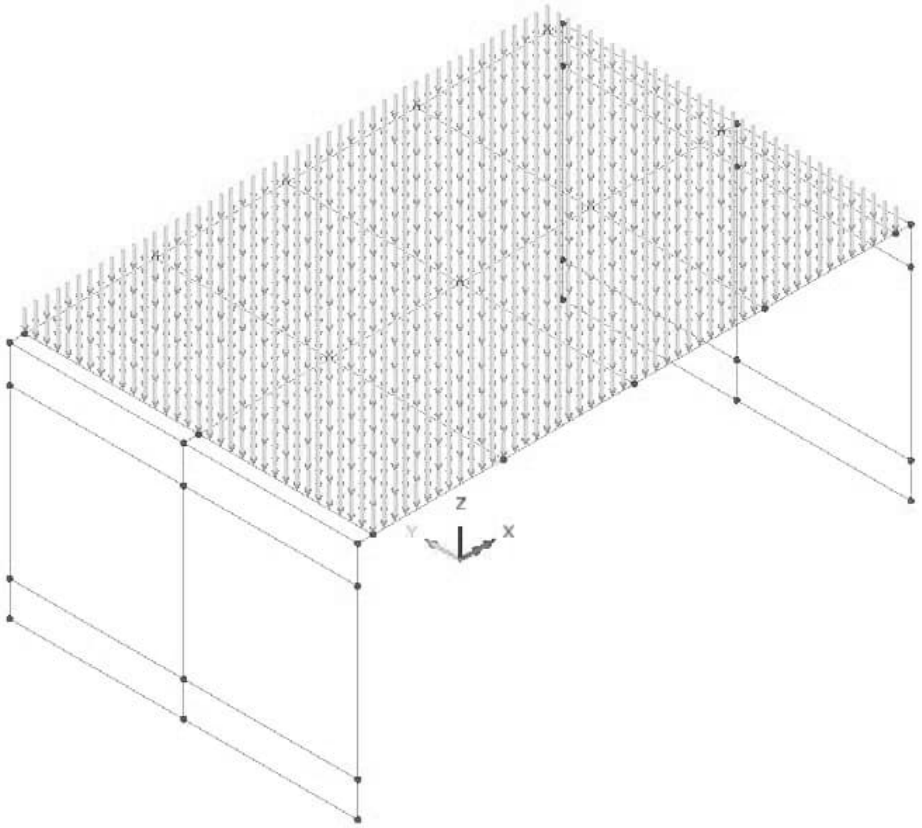
Overview 3D

3.1.2 Bridge deck

Load case : EGEN 2

Structural loading : Body force

Linear acceleration in Z (a_z) : -10 $\frac{m}{s^2}$



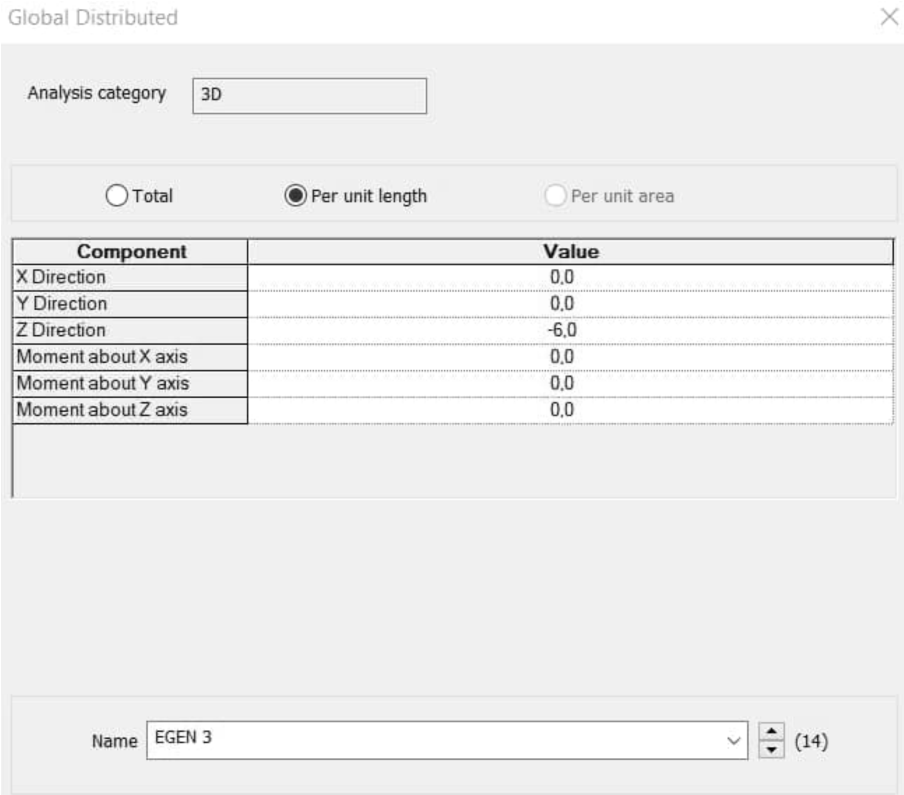
Overview 3D

3.1.3 Edge beams including railing

Along each edge bream a line load is introduced. The load includes weight of edge beam and railing.

$p_{räcke} = 0.7\frac{kN}{m}$: weight railing

$\rightarrow p_z = p_{räcke} + p_{KB} = 0.7\frac{kN}{m} + 0.40m \cdot 0.45m \cdot 25\frac{kN}{m^3} = -6\frac{kN}{m}$

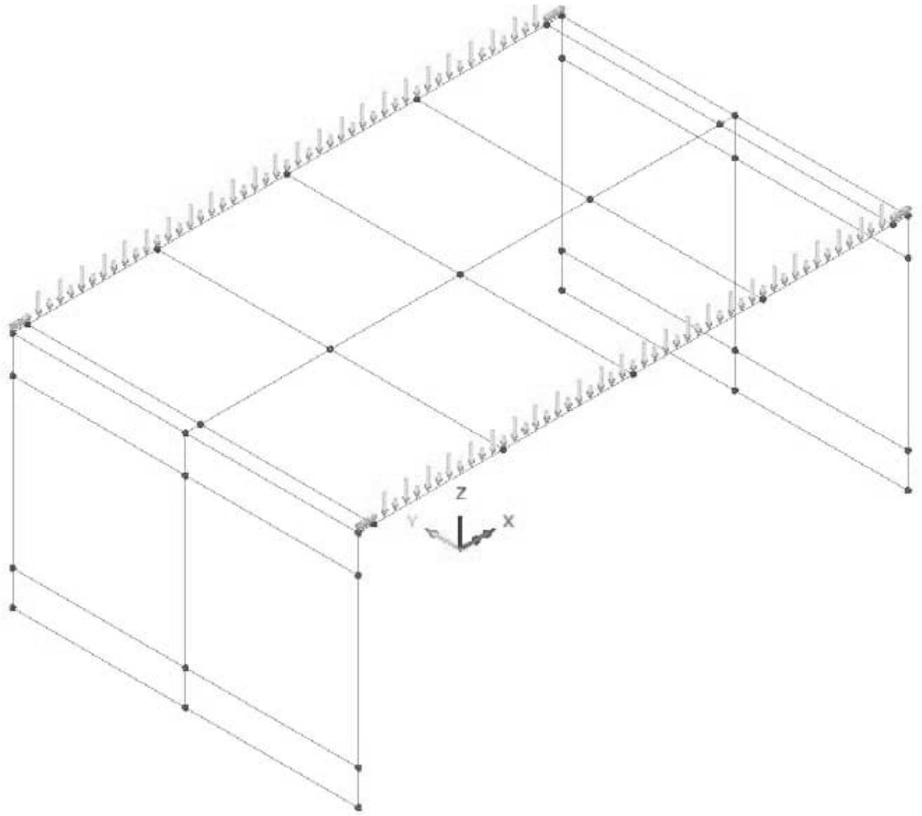

Global Distributed

Analysis category: 3D

Total / Per unit length (selected) / Per unit area

Component	Value
X Direction	0,0
Y Direction	0,0
Z Direction	-6,0
Moment about X axis	0,0
Moment about Y axis	0,0
Moment about Z axis	0,0

Name: EGEN 3 (14)



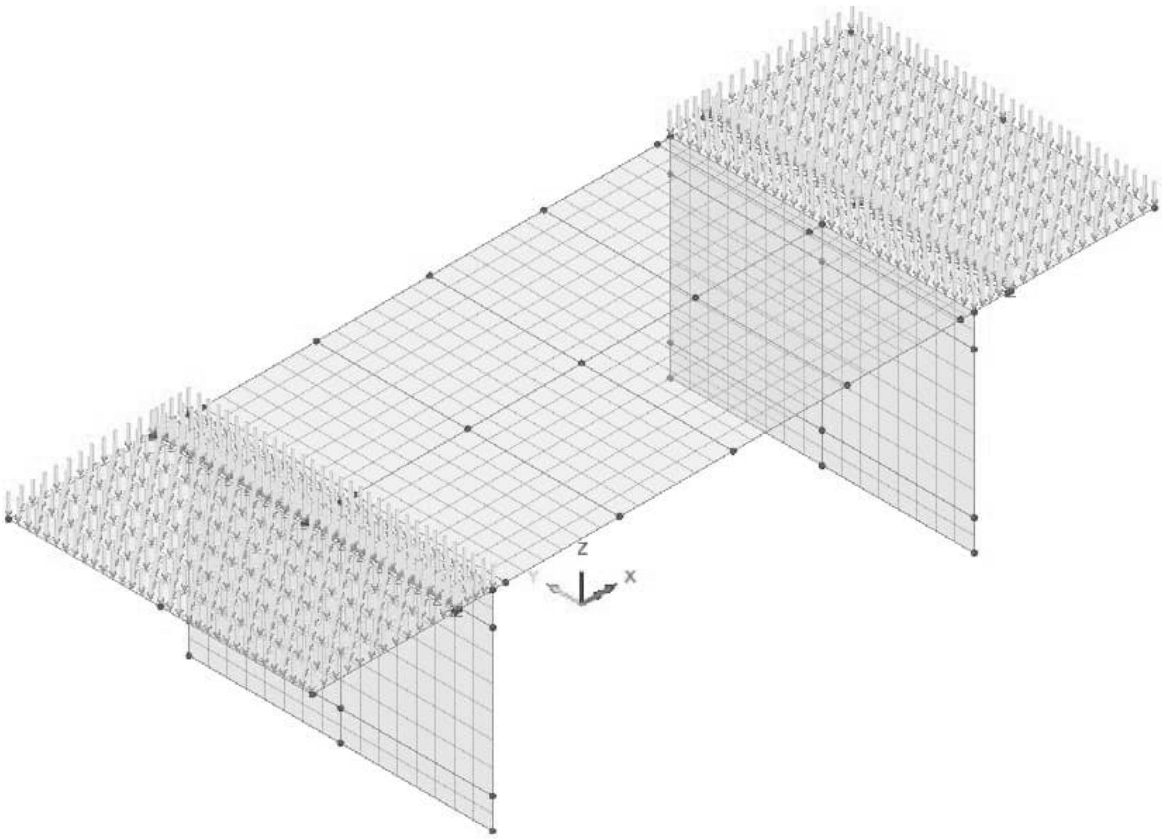
Overview 3D

3.1.4 Link plate

Load case : EGEN 4

Structural loading : Body force

Linear acceleration in Z (a_z) : -10 $\frac{m}{s^2}$



Ovierview 3D

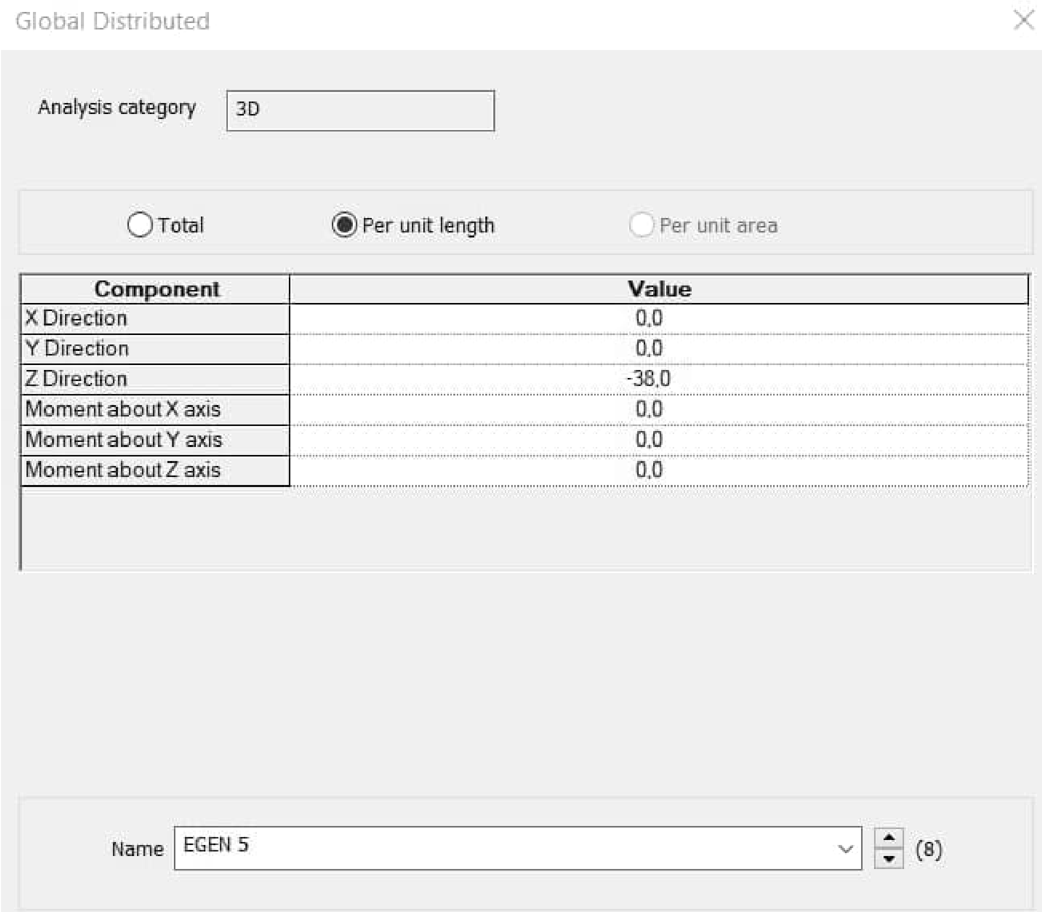
3.1.5 Wingwalls

All wingwalls are alike (∴ L = 4.8 m).

$P_z = -145kN$: page A3:33

Load is distributed along edge of abutments from bottom of superstructure and distance 3.85 m downward.

$$p_z = \frac{P_z}{h} = -\frac{145kN}{3.85m} = -38\frac{kN}{m}$$



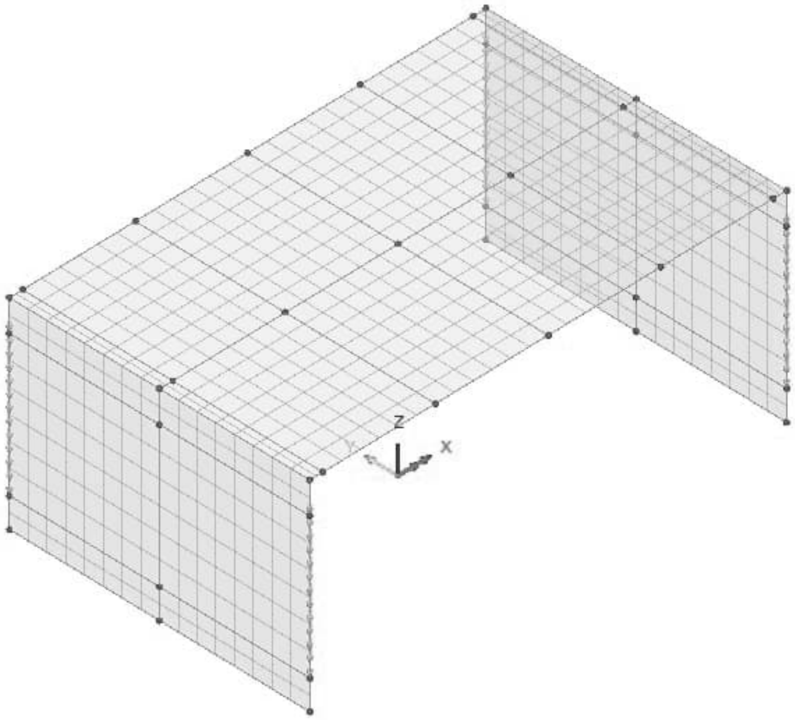
Global Distributed

Analysis category: 3D

○ Total ◉ Per unit length ○ Per unit area

Component	Value
X Direction	0,0
Y Direction	0,0
Z Direction	-38,0
Moment about X axis	0,0
Moment about Y axis	0,0
Moment about Z axis	0,0

Name: EGEN 5 (8)



Ovierview 3D

3.1.6 Load combination dead weight: EGEN

Basic loadcomination EGEN :

Load case	Factor
EGEN 1	1.00
EGEN 2	1.00
EGEN 3	1.00
EGEN 4	1.00
EGEN 5	1.00

3.2 SURFACING

Thickness pavement with a thickness of 110 mm built as follows:

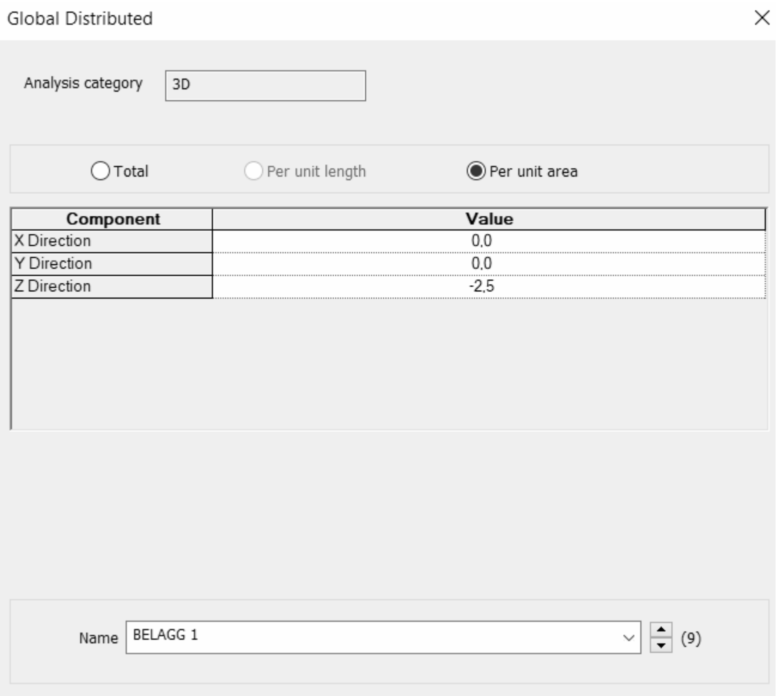
- Wearing course ABT 40 — 40 mm
- Binde course ABb — 40 mm
- Protective course ABT — 25 mm
- Waterproofing layer — 5 mm

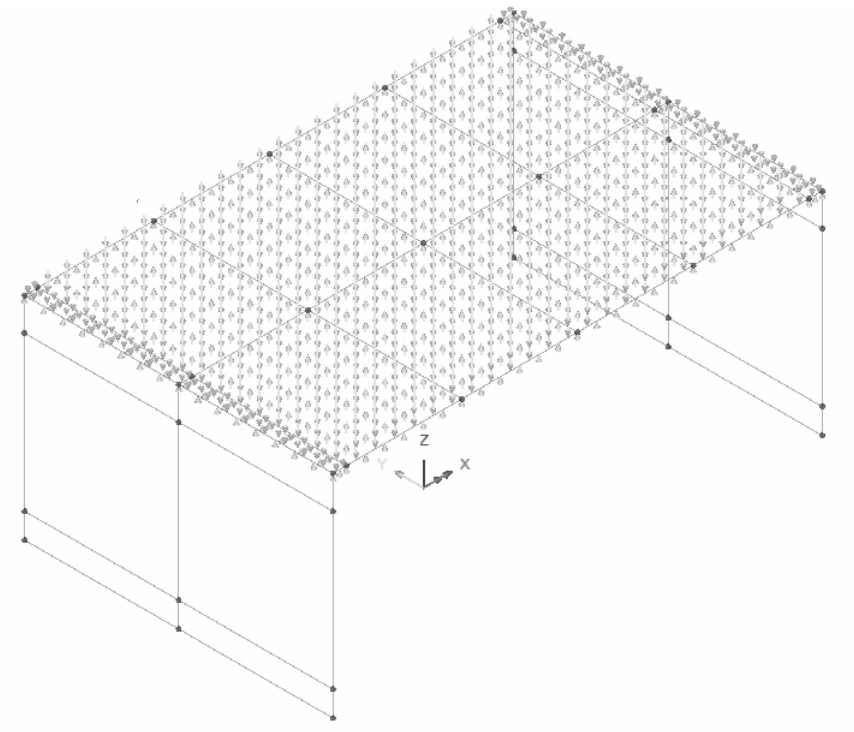
$\gamma = 23\frac{kN}{m^3}$: ABT and ABb

$\gamma = 23\frac{kN}{m^3}$: course

$\rightarrow p_{belägg} = 0.11kPa + 23\frac{kN}{m^3} \cdot 0.105m = 2.5kPa$

3.2.1 Load on bridge deck





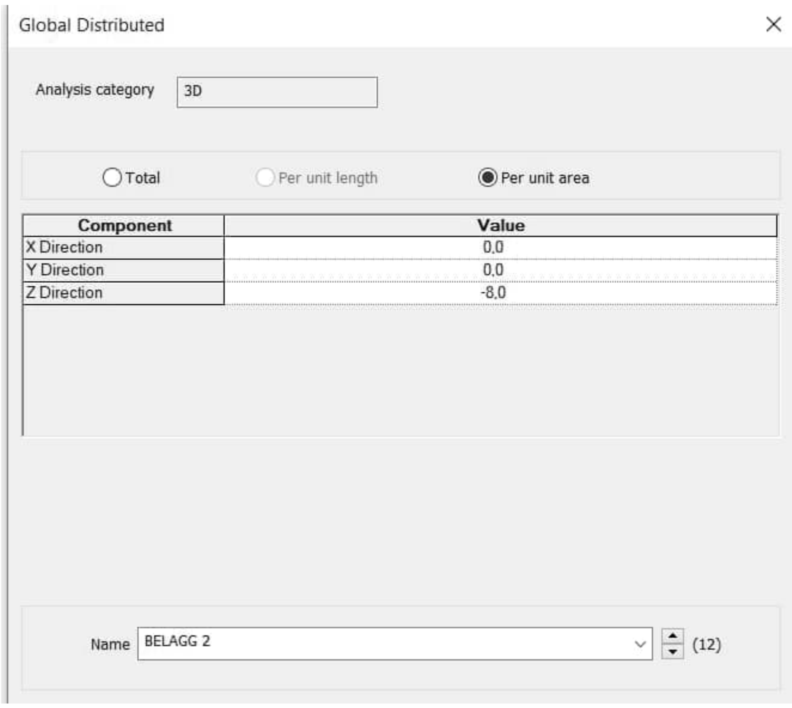
Overview 3D

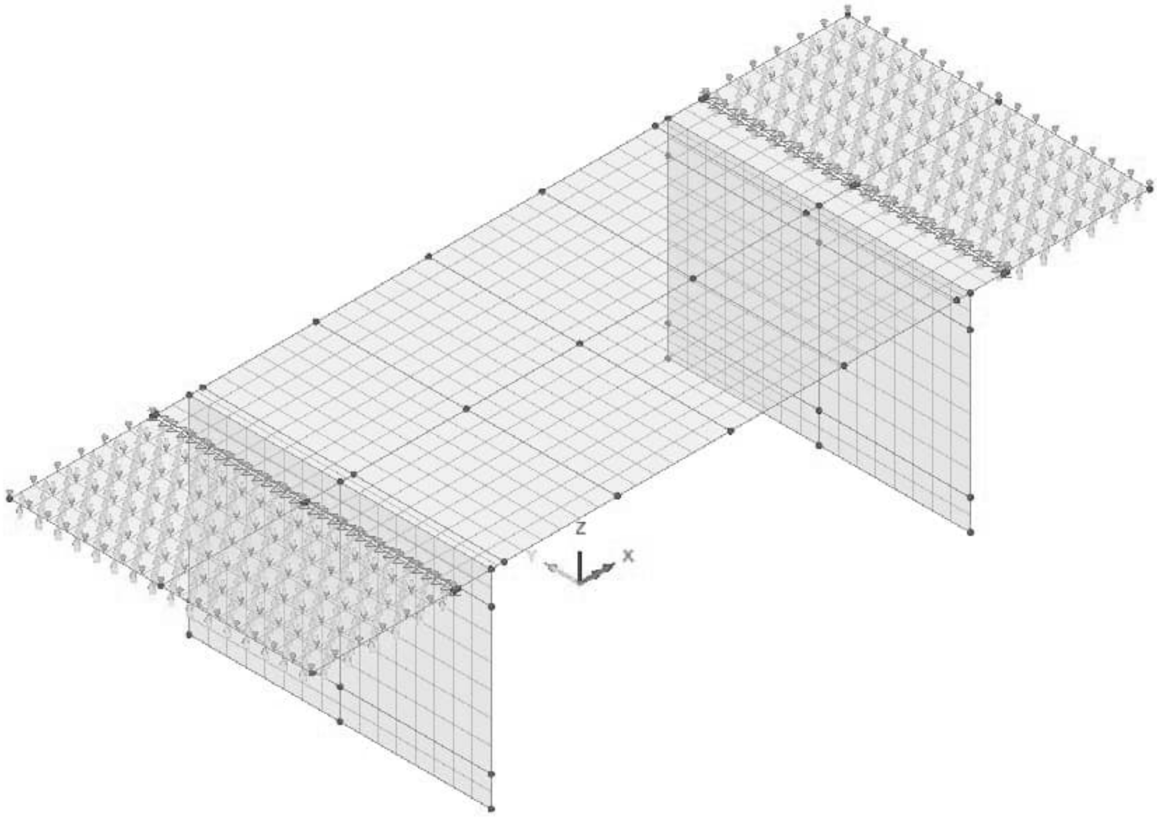
3.2.2 Load on link plate

On the upper side of the link plate, there is a 95 mm pavement and an overfill with varying thickness (100-400 mm).

The overfill is considered equivalent to the base layer. In the static model, a fictitious load corresponding to the weights for the pavement and overfill is introduced (an average thickness of 250 mm is applied.

$$q_{belägg} = 2.5kPa + 22\frac{kN}{m^3} \cdot 0.25m = 8.0kPa$$





Overview 3D

3.2.3 Load combination surfacing: BELAGG

Basic loadcomination BELAGG :

Load case	Factor
BELAGG 1	1.00
BELAGG 2	1.00

3.3 EARTH PRESSURE

Earth pressure in filling corresponds to coarse crushed blasted rock (AMA CEB.524).

φ_k = 45°

$\gamma = 20 \frac{kN}{m^3}$

$\gamma' = 13 \frac{kN}{m^3}$

$$X_d = \frac{1}{\gamma_m} \cdot \eta \cdot \overline{X} \equiv \frac{1}{\gamma_m} \cdot X_k$$

Earth pressure coefficient for design method 2 (D2) :
Design coefficients associated to *A1 + M1 + R2* according to SS-EN 1997-1 section 2.4.7.3.4.3 is applied.

$\gamma_{m.D2}$ = 1.0 : see TSFS chapter 38 table 38.3 for M1

$\rightarrow \varphi_d = artctan\left(\frac{tan\varphi_k}{\gamma_{m.D2}}\right) = arctan\left(\frac{tan45°}{1.0}\right) = 45°$

$K_0 = 1 - \sin(\varphi_d) = 0.29$

$K_a = tan^2\left(45° - \frac{\varphi_d}{2}\right) = 0.17$

$K_p = tan^2\left(45° + \frac{\varphi_d}{2}\right) = 5.82$

Earth pressure for design method 3 (D3) :
Design coefficients associated to *A1(design loads) + A2(geotechnical loads) + M2 + R3* according to SS-EN 1997-1 section 2.4.7.3.4.4 is applied.

$\gamma_{m.D3}$ = 1.3 : see TSFS chapter 8 table 38.3 för M2

$\rightarrow \varphi_d = artctan\left(\frac{tan\varphi_k}{\gamma_m}\right) = arctan\left(\frac{tan45°}{1.3}\right) = 38°$

Remark
These are not used in FEM-analysis. This is done by adjusting load coefficients.

Earth pressure in FEM-analysis:

During design earth press coefficients associated to method D2 will used applied, however the load coefficients are adjusted according verification, see section 3.13.1.

$$K_o = 1 - sin(\varphi_d)$$

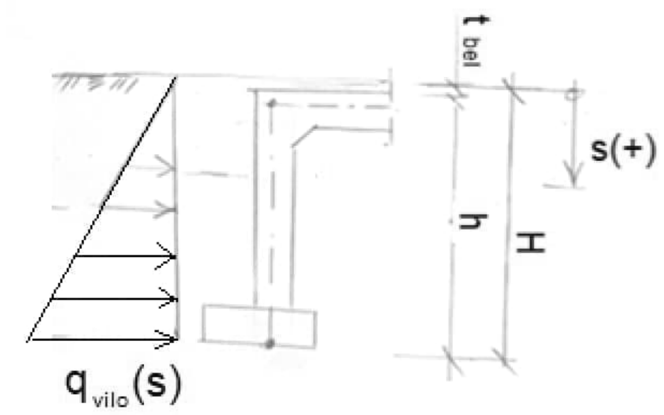
$$K_a = tan^2(45° - \frac{\varphi_d}{2})$$

$$K_p = tan^2(45° + \frac{\varphi_d}{2})$$

φ_d	K_a	K_0	K_p	Metod
(38°)	(0.24)	(0.38)	(4.20)	D3
45°	0.17	0.29	5.82	D2

$$H = 0.11m + 5.85m = 5.96m$$

$$q_{vilo}(s) = K_0 \cdot \gamma \cdot s = 0.29 \cdot 20 \frac{kN}{m^3} \cdot s(+) = s(+) \cdot 5.8kPa$$



Remark
Load width B = 8.0 m is used for the bottom slab even though width B = 8.5 m. This simplification is possible since favourable effect of passive earth pressure has not been considered on safe side.

3.3.1 Load against abutment 1

$q_{vilo}\ (0m) = 0kPa$: top surfacing

$q_{vilo}\ (5.96m) = 5.96m \cdot 5.8\frac{kN}{m^3} = 35kPa$: undersida bottom slab

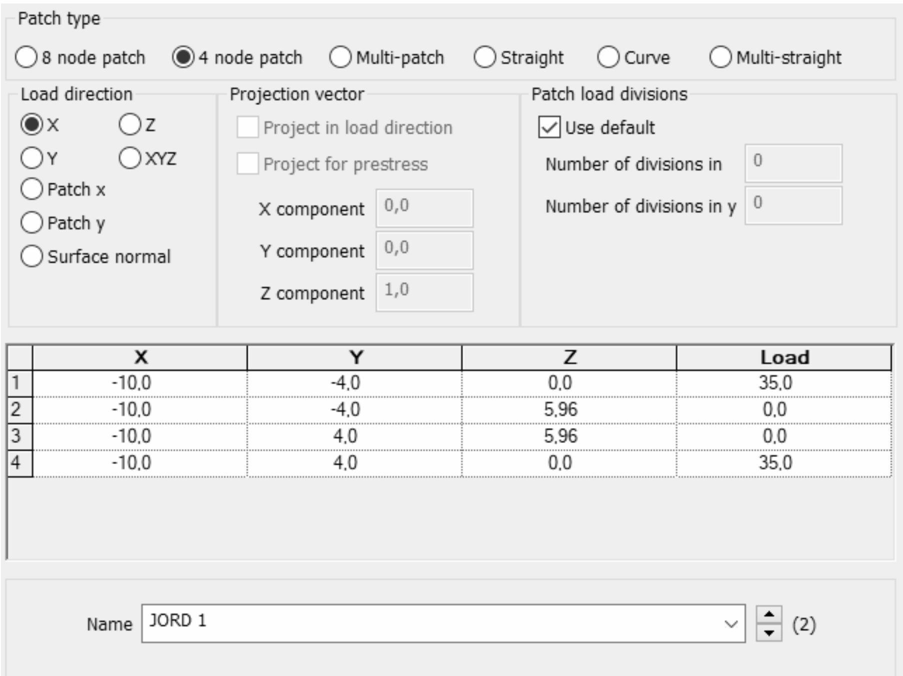
Load case : JORD 1

Structural loading : Discrete 4 node patch

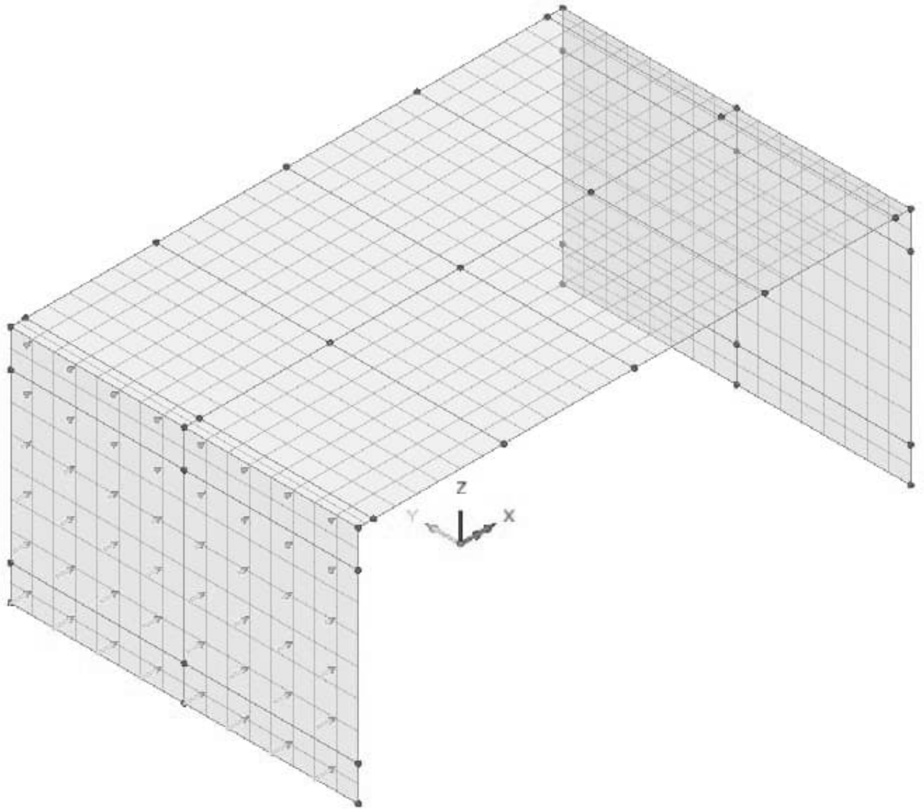
Surface load (q_x) : 0 kPa → +35 kPa

Search Area : Abutment 1

Loads outside search area : Include full load



	X	Y	Z	Load
1	-10,0	-4,0	0,0	35,0
2	-10,0	-4,0	5,96	0,0
3	-10,0	4,0	5,96	0,0
4	-10,0	4,0	0,0	35,0



Overview 3D

3.3.2 Load against abutment 2

$q_{vilo}\ (0m) = 0kPa$: top surfacing

$q_{vilo}\ (5.96m) = 5.96m \cdot 5.8\frac{kN}{m^3} = 35kPa$: undersida bottom slab

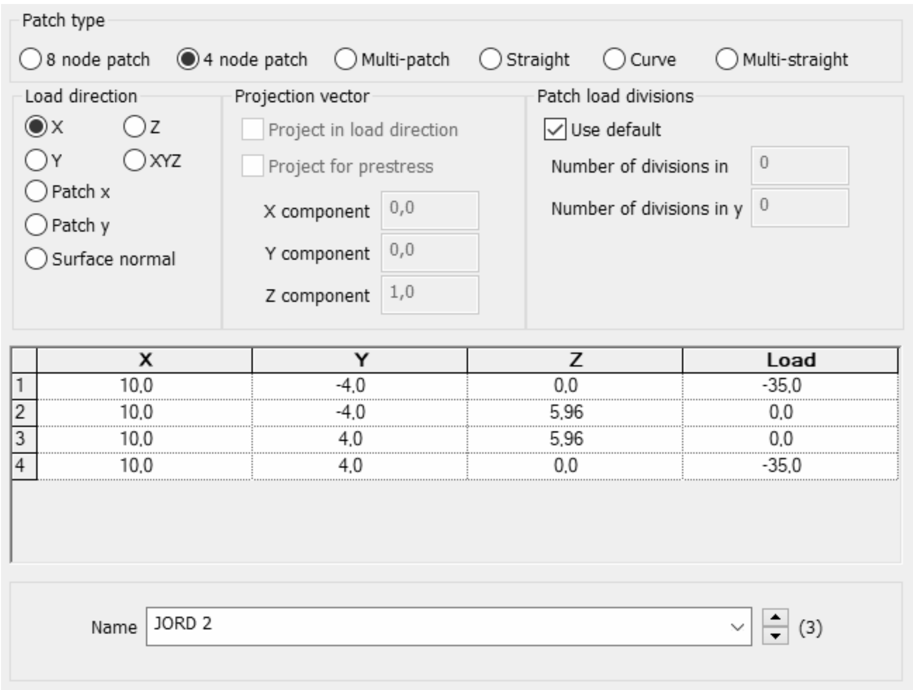
Loadcase : JORD 2

Structural loading : Discrete 4 node patch

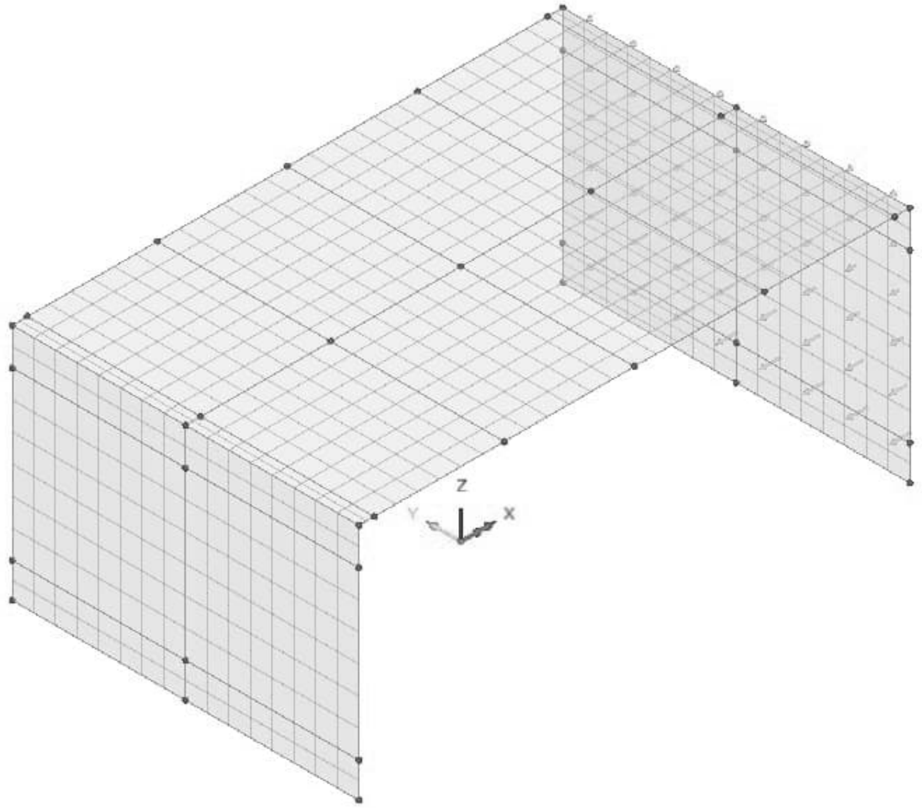
Surface load (q_x) : 0 kPa → -35 kPa

Search Area : Abutment 2

Loads outside search area : Include full load



	X	Y	Z	Load
1	10,0	-4,0	0,0	-35,0
2	10,0	-4,0	5,96	0,0
3	10,0	4,0	5,96	0,0
4	10,0	4,0	0,0	-35,0



Overview 3D

3.3.3 Load against wingwalls

Design software K2.002 is used to determine earth pressure against wingwalls according to Culmans´s theory. All wingwalls are assumed to have same length (L = 4.8 m).

Load is distributed along edge of abutments from bottom of superstructure and distance 3.85 m. This assumption is on safe side.

Effective height along edge abutment:

$H_{ef} = 3.85m$: see page A3:33

Load at abutment edge quasi-permanent load combination (SLS-Q):

$N_{SLS-Q} = +39 \frac{kNm}{m}$: see page A3:33

$M_{SLS-Q} = 104 \frac{kNm}{m}$: see page A3:33

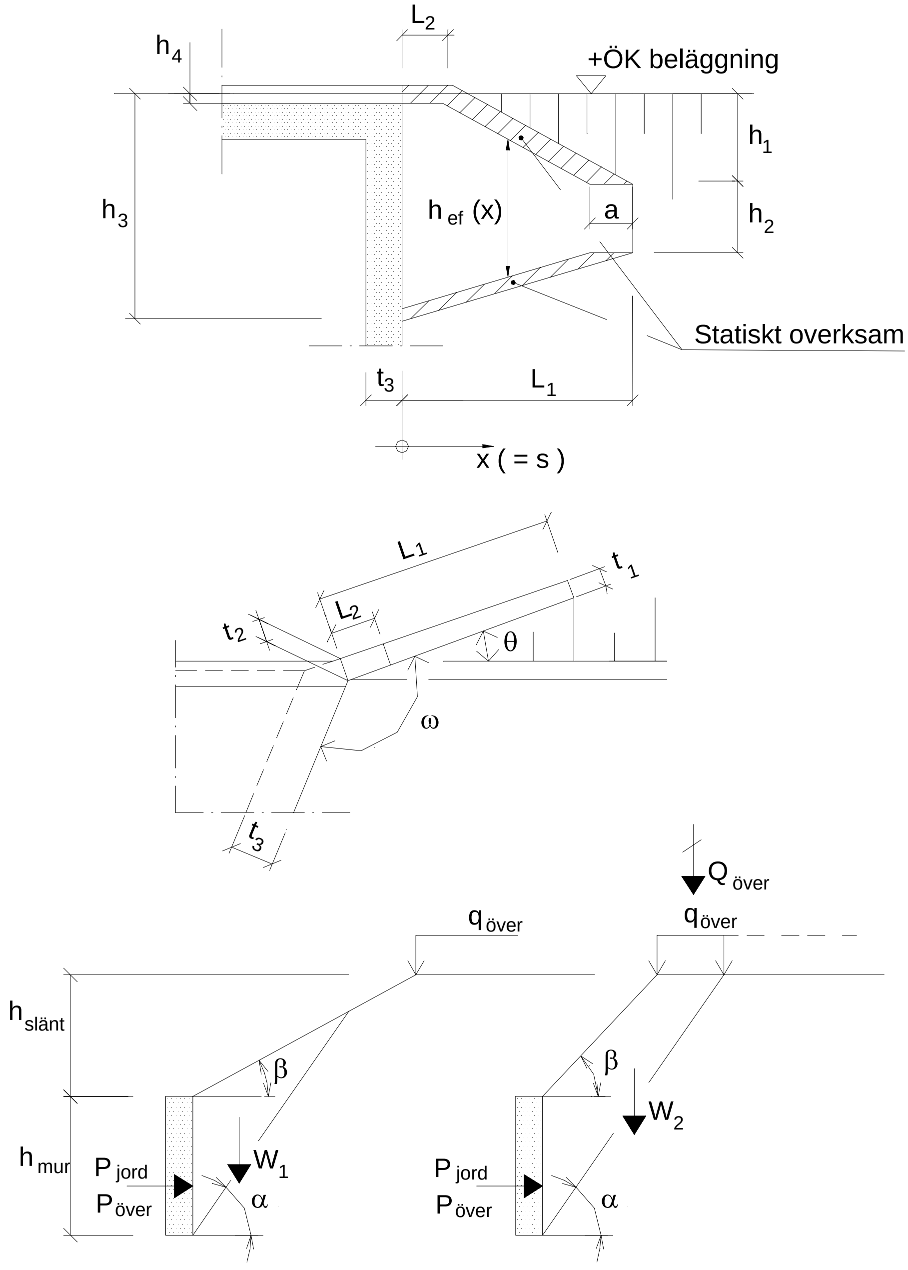
Characteristic earth pressure at abutment edge:

$$N_{jord} = +52 \frac{kNm}{m} \cdot \frac{1}{1.35} = 29 \frac{kN}{m}$$

$$M_{jord} = 104 \frac{kNm}{m} \cdot \frac{1}{1.35} = 77 \frac{kNm}{m}$$

Objekt : Vingmur L = 4.8 m

PRINCIPFIGUR



Typ 1 ($Q_{över}$ = 0) Typ 2 ($Q_{över} \neq 0$)

Jordtryck enligt Culmans metod

INDATA

Geometri :

$L_1 = 4.8m$ $L_2 = 0.5m$

$h_1 = 1.4m$ $h_2 = 1.1m$ $h_3 = 4.8m$ $h_4 = 0.2m$

$t_1 = 0.40m$ $t_2 = 0.40m$ $t_3 = \frac{0.7m + 1.1m}{2} = 0.9\,m$

Vinkel vinge-ramben på baksidan : $\omega = 135°$

Vinkel vinge-vägbankskrön på baksidan : $\theta = 45°$

Avstånd till brytpunkt för effektiv höjd : $a = 1.56m$

Material :

Jordmaterial : $\gamma_{jord} = 20\frac{kN}{m^3}$ $K_o = 0.29$ $K_a = 0.17$

Betong : $\gamma_{btg} = 25\frac{kN}{m^3}$

Laster :

Överlast : $q_{över} = 20kPa$

Lastkoefficienter :

Jordtryck

$\psi\gamma_{ULS.1} = 1.45$ $\psi\gamma_{SLS.1} = 1.35$

Överlast

$\psi\gamma_{ULS.2} = 1.70$ $\psi\gamma_{SLS.2} = 0$

BERÄKNING

Jordtryck enligt Culmans metod :

Nivå överkant vingmur : $mur_{ök} = \mathrm{linterp}\left[\left(0m \;\; L_2 \;\; L_1\right), \left(h_3 \;\; h_3 \;\; h_3 - h_1\right), s\right]$

Nivå underkant vingmur : $mur_{uk} = \mathrm{linterp}\left[\left(0m \;\; L_1\right), \left(0m \;\; h_3 - h_1 - h_2\right), s\right]$

Vingmurens höjd : $h_{mur} = mur_{ök} - mur_{uk}$

Släntens höjd : $h_{slänt} = \mathrm{linterp}\left[\left(0m \;\; L_2 \;\; L_1\right), \left(0m \;\; 0m \;\; h_1\right), s\right]$

Friktionsvinkel: $\varphi = \mathrm{asin}\left(1 - K_o\right)$

Lutning hos slänten ned till överkant vingmur mätt vinkelrätt mot vingen :

$$\beta = \mathrm{atan}\left[\frac{h_1}{\left(L_1 - L_2\right)\tan(\theta)}\right]$$

Vertikallast för brottfigur typ 1 (= brottlinje i slänt, sålunda inget tillskott av överlast) :

$$W_1 = h_{mur}\cdot\sin\left(\frac{\pi}{2} - \alpha\right)\cdot\left(h_{mur}\cdot\cos\left(\frac{\pi}{2} - \alpha\right) + \frac{h_{mur}\cdot\sin\left(\frac{\pi}{2} - \alpha\right)}{\tan(\alpha - \beta)}\right)\cdot\frac{\gamma_{jord}}{2}$$

Vertikallaster för brottfigur typ 2 (= brottlinje hamnar ovanför slänt vilket ger ett bidrag från överlast) :

$$W_2 = \left[\left(h_{mur} + h_{slänt}\right)^2\cdot\tan\left(\frac{\pi}{2} - \alpha\right) - \frac{h_{slänt}^{\,2}}{\tan(\beta)}\right]\cdot\frac{\gamma_{jord}}{2}$$

$$Q_{över}(q) = q\cdot\left[\left(h_{mur} + h_{slänt}\right)\cdot\tan\left(\frac{\pi}{2} - \alpha\right) - \frac{h_{slänt}(s)}{\tan(\beta)}\right]$$

Vilojordtrycksresultant enligt Culmann under inverkan av jordlast + överlast :

$$p_o(q) = \left| \begin{array}{l} W_{jord} \leftarrow W_1 \ \text{ if } \left(h_{mur} + h_{slänt}\right) \cdot \tan(90° - \alpha) < \dfrac{h_{slänt}}{\tan(\beta)} \\ W_{jord} \leftarrow W_2 + Q_{över}(q) \ \text{ otherwise} \\ p_{aktiv} \leftarrow W_{jord} \cdot \tan(\alpha - \varphi) \\ p_{aktiv} \cdot \dfrac{K_o}{K_a} \end{array} \right.$$

Utvärdera största last av jordtryck och överlast genom att kontrollera N_a antal vinklar mellan φ och 90°. Överlastens lasteffekt fås som skillnaden mellan jordtrycksresultant med och utan överlast.

$$p_{jord}(s) = \left| \begin{array}{l} N_\alpha \leftarrow 20st \\ \Delta\alpha \leftarrow \dfrac{90° - \varphi}{N_\alpha - 1} \\ \alpha \leftarrow \varphi \\ p_{max} \leftarrow p_o(0kPa) \\ \text{for } i \in 2 .. N_\alpha \\ \quad \left| \begin{array}{l} \alpha \leftarrow \alpha + \Delta\alpha \\ p_{vilo} \leftarrow p_o(0kPa) \\ \text{if } p_{vilo} > p_{max} \\ \quad \left| \begin{array}{l} p_{max} \leftarrow p_{vilo} \\ \alpha_{max} \leftarrow \alpha \end{array} \right. \end{array} \right. \end{array} \right.$$

$$p_{över}(s) = \left| \begin{array}{l} N_\alpha \leftarrow 20st \\ \Delta\alpha \leftarrow \dfrac{90° - \varphi}{N_\alpha - 1} \\ \alpha \leftarrow \varphi \\ p_{max} \leftarrow p_o\left(q_{över}\right) - p_o(0kPa) \\ \text{for } i \in 2 .. N_\alpha \\ \quad \left| \begin{array}{l} \alpha \leftarrow \alpha + \Delta\alpha \\ p_{över} \leftarrow p_o\left(q_{över}\right) - p_o(0kPa) \\ \text{if } p_{över} > p_{max} \\ \quad \left| \begin{array}{l} p_{max} \leftarrow p_{över} \\ \alpha_{max} \leftarrow \alpha \end{array} \right. \end{array} \right. \end{array} \right.$$

Snittkrafter jordtryck + överlast :

$$H'_{jord}\left(x_s\right) = \int_{x_s}^{L_1} p_{jord}(s)\, ds \qquad M'_{jord}\left(x_s\right) = \int_{x_s}^{L_1} \left(s - x_s\right) \cdot p_{jord}(s)\, ds$$

$$H'_{över}\left(x_s\right) = \int_{x_s}^{L_1} p_{över}(s)\, ds \qquad M'_{över}\left(x_s\right) = \int_{x_s}^{L_1} \left(s - x_s\right) \cdot p_{över}(s)\, ds$$

Lastkombinering - Lk ULS och Lk SLS :

Snittkraft i frontmur för inspänningssnitt :

$$N'_{ULS.front} = \left(\psi\gamma_{ULS.1} \cdot H'_{jord}(0 \cdot m) + \psi\gamma_{ULS.2} \cdot H'_{över}(0 \cdot m)\right) \cdot \sin(\omega)$$

$$M'_{ULS.front} = \psi\gamma_{ULS.1} \cdot M'_{jord}(0 \cdot m) + \psi\gamma_{ULS.2} \cdot M'_{över}(0 \cdot m) + N'_{ULS.front} \cdot \frac{t_3}{2}$$

$$N'_{SLS.front} = \left(\psi\gamma_{SLS.1} \cdot H'_{jord}(0 \cdot m) + \psi\gamma_{SLS.2} \cdot H'_{över}(0 \cdot m)\right) \cdot \sin(\omega)$$

$$M'_{SLS.front} = \psi\gamma_{SLS.1} \cdot M'_{jord}(0 \cdot m) + \psi\gamma_{SLS.2} \cdot M'_{över}(0 \cdot m) + N'_{SLS.front} \cdot \frac{t_3}{2}$$

Snittkrafter i vingmur :

$$Q'_{ULS}(s) = \psi\gamma_{ULS.1} \cdot H'_{jord}(s) + \psi\gamma_{ULS.2} \cdot H'_{över}(s)$$

$$M'_{ULS}(s) = \psi\gamma_{ULS.1} \cdot M'_{jord}(s) + \psi\gamma_{ULS.2} \cdot M'_{över}(s)$$

$$M'_{SLS}(s) = \psi\gamma_{SLS.1} \cdot M'_{jord}(s) + \psi\gamma_{SLS.2} \cdot M'_{över}(s)$$

Beräkning av effektiv höjd :

$$\Delta h = h_3 - h_2 - h_1 \qquad \Delta h = 2.300 \cdot m$$

Nivå överkant effektiv vingmur :

$$Nivå_{ök} = \text{linterp}\left[\left(0m \;\; L_2 \;\; L_1 - a \;\; L_1\right), \left(h_3 - h_4 \;\; h_3 - h_4 \;\; h_3 - h_1 \;\; h_3 - h_1\right), s\right]$$

Nivå underkant effektiv vingmur :

$$Nivå_{uk} = \text{linterp}\left[\left(0m \;\; L_1 - L_2 \;\; L_1\right), \left(\frac{a}{L} \cdot \Delta h \;\; \Delta h \;\; \Delta h\right), s\right]$$

Effektiv höjd vingmur :

$$h_{ef}(s) = Nivå_{ök} - Nivå_{uk}$$

Dimensionerande snittkrafter (Lk ULS och Lk SLS) fördelade på effektiv höjd :

Snittkraft i frontmur för inspänningssnitt :

$$H_{ef} = h_{ef}(0m)$$

$$N_{ULS.front} = \frac{N'_{ULS.front}}{H_{ef}}$$

$$M_{ULS.front} = \frac{M'_{ULS.front}}{H_{ef}}$$

$$N_{SLS.front} = \frac{N'_{SLS.front}}{H_{ef}}$$

$$M_{SLS.front} = \frac{M'_{SLS.front}}{H_{ef}}$$

Snittkrafter i vingmur :

$$Q_{ULS.ving} = \frac{Q'_{ULS}(s)}{h_{ef}(s)}$$

$$M_{ULS.ving} = \frac{M'_{ULS}(s)}{h_{ef}(s)}$$

$$M_{SLS.ving} = \frac{M'_{SLS}(s)}{h_{ef}(s)}$$

Egenvikt vingmur :

$$t = t_2 - \frac{t_2 - t_1}{L_1} \cdot s$$

$$A(s) = h_{mur} \cdot t$$

$$V_{egen} = \gamma_{btg} \cdot \int_0^{L_1} A(s)\, ds$$

$$M_{egen} = \gamma_{btg} \cdot \int_0^{L_1} A(s) \cdot s\, ds$$

RESULTAT

Mellanresultat

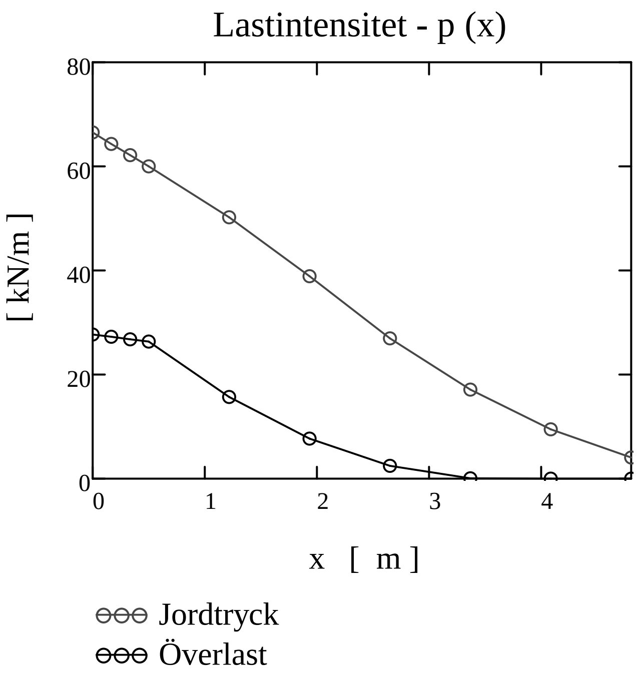
$\varphi = 45 \cdot °$: Dimensionerande friktionsvinkel tillhörande K_o

$\beta = 18 \cdot °$: Lutning hos slänten ned till överkant vingmur mätt vinkelrätt mot vingen

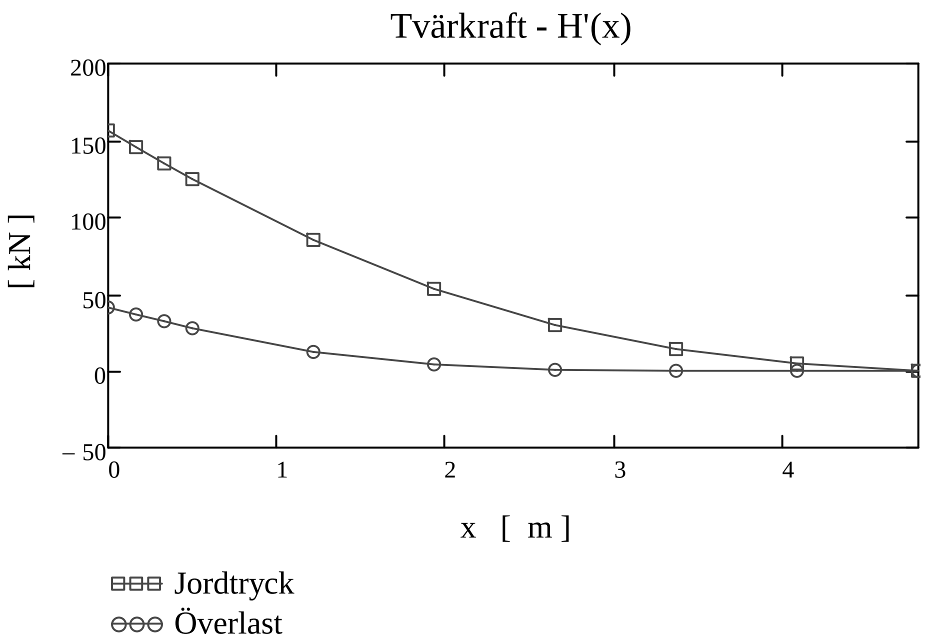
Utvärdering av jordtryck + överlast samt tilhörande farligaste brottvinkel redovisad i tabellform :

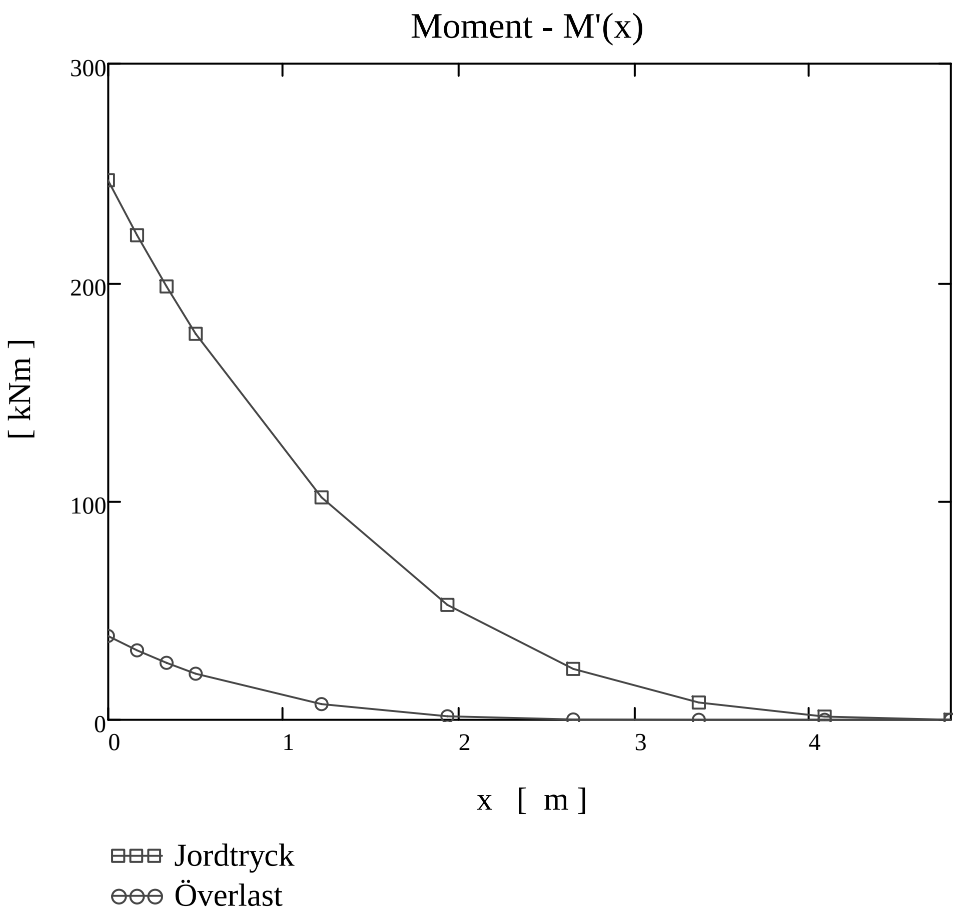
x	p_{jord}	α_{tillh}	$p_{över}$	α_{tillh}
0	66,5	66	27,7	66
0,167	64,3	66	27,2	66
0,333	62,1	66	26,8	66
0,500	60,0	66	26,3	66
1,22	50,2	66	15,7	62
1,93	38,9	66	7,7	57
2,65	26,9	66	2,4	52
3,37	17,1	66	0,1	48
4,08	9,5	66	0,0	0
4,80	4,1	66	0,0	0
m	kN/m	grader	kN/m	grader

Utvärdering av jordtryck + överlast i diagramform :

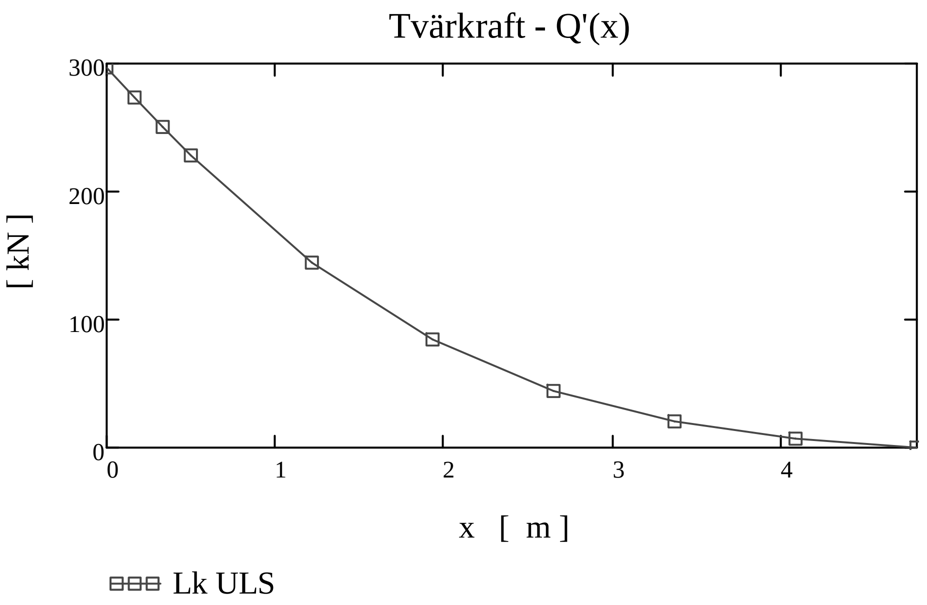


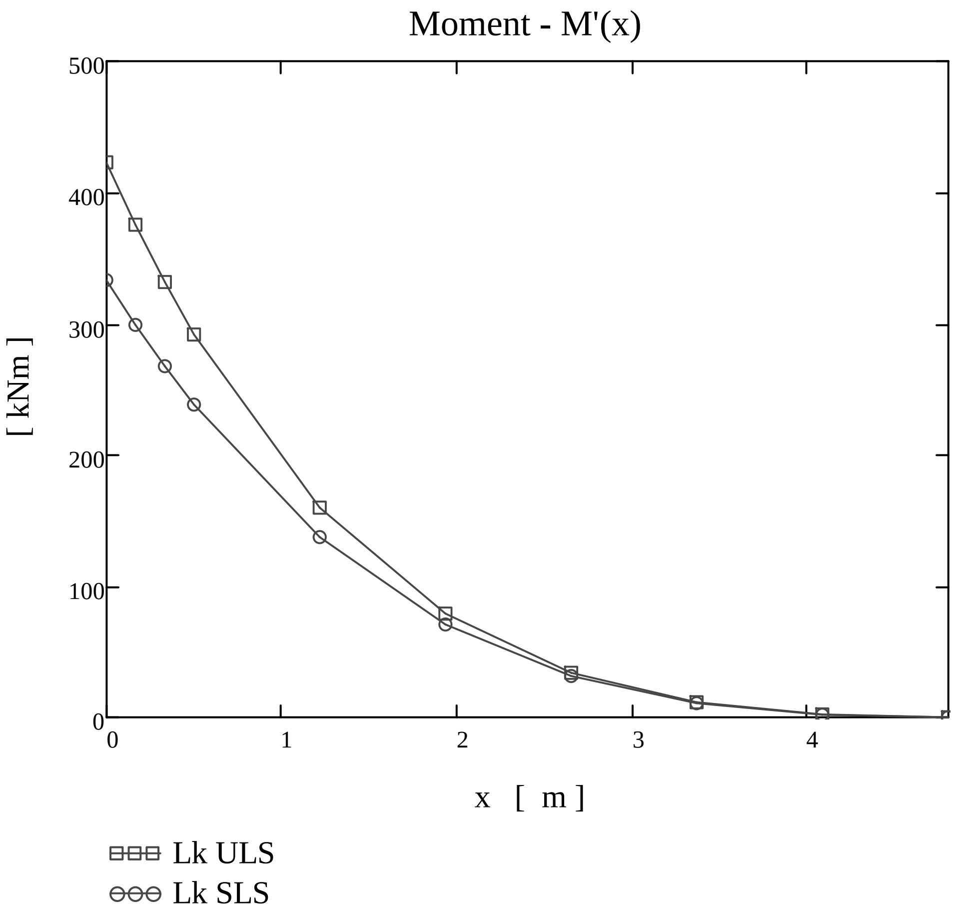
Uvärdering av snittkrafter tillhörande jordtryck + överlast i diagramform :



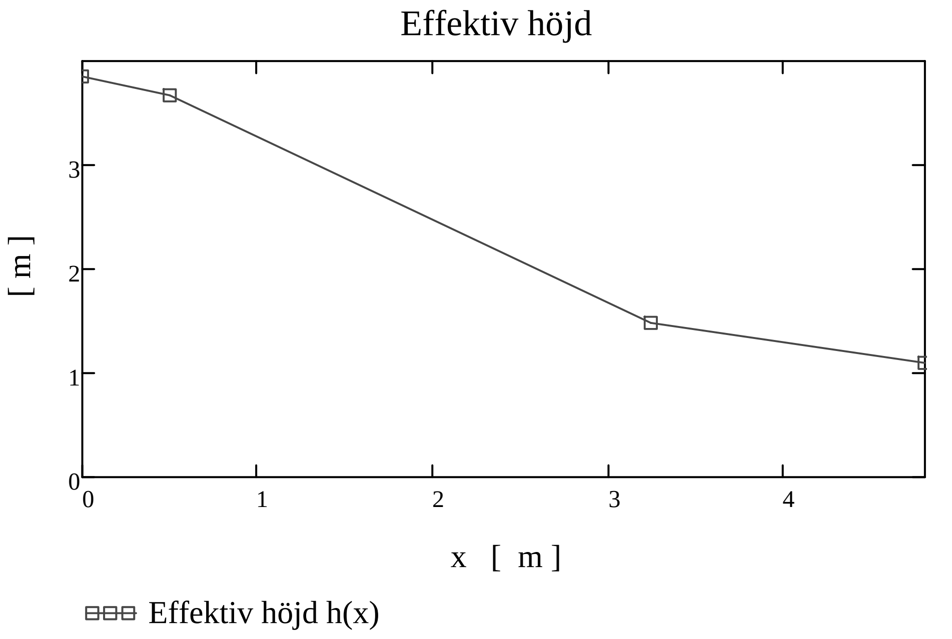


Uvärdering av dimensionerande snittkrafter för Lk ULS och Lk SLS i diagramform :





Uvärdering av effektivhöjd i diagramform :



Resultat

Dimensionerande snittkrafter i frontmur för inspänningssnitt

Effektiv höjd i inspänningssnitt : $H_{ef} = h_{ef}(0m)$ $H_{ef} = 3.85 \cdot m$

$N_{ULS.front}$	$M_{ULS.front}$	$N_{SLS.front}$	$M_{SLS.front}$
55	134	39	104
kN/m	kNm/m	kN/m	kNm/m

Dimensionerande snittkrafter i vingmur

x	$Q_{ULS.ving}$	$M_{ULS.ving}$	$M_{SLS.ving}$	t (x)
0	77	110	86	0,400
0,167	72	99	79	0,400
0,333	67	89	72	0,400
0,500	62	79	65	0,400
1,22	47	52	44	0,400
1,93	33	31	28	0,400
2,65	23	17	16	0,400
3,37	14	8	7	0,400
4,08	6	2	2	0,400
4,80	0	0	0	0,400
m	kN/m	kNm/m	kNm/m	m

Egenvikt vingmur

Egenvikt ger upphov till en triangulär linjelast med största intensiteten enligt nedan :

$V_{egen} = 145 \cdot kN$ $M_{egen} = 275 \cdot kNm$

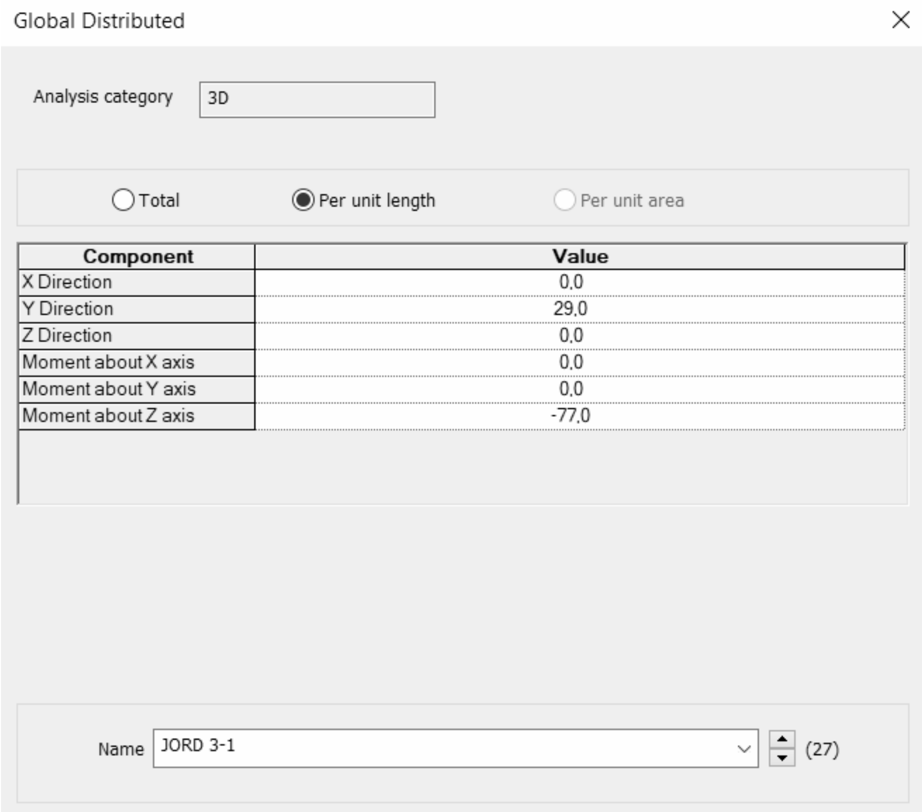
$$x_{tp} = \frac{M_{egen}}{V_{egen}}$$

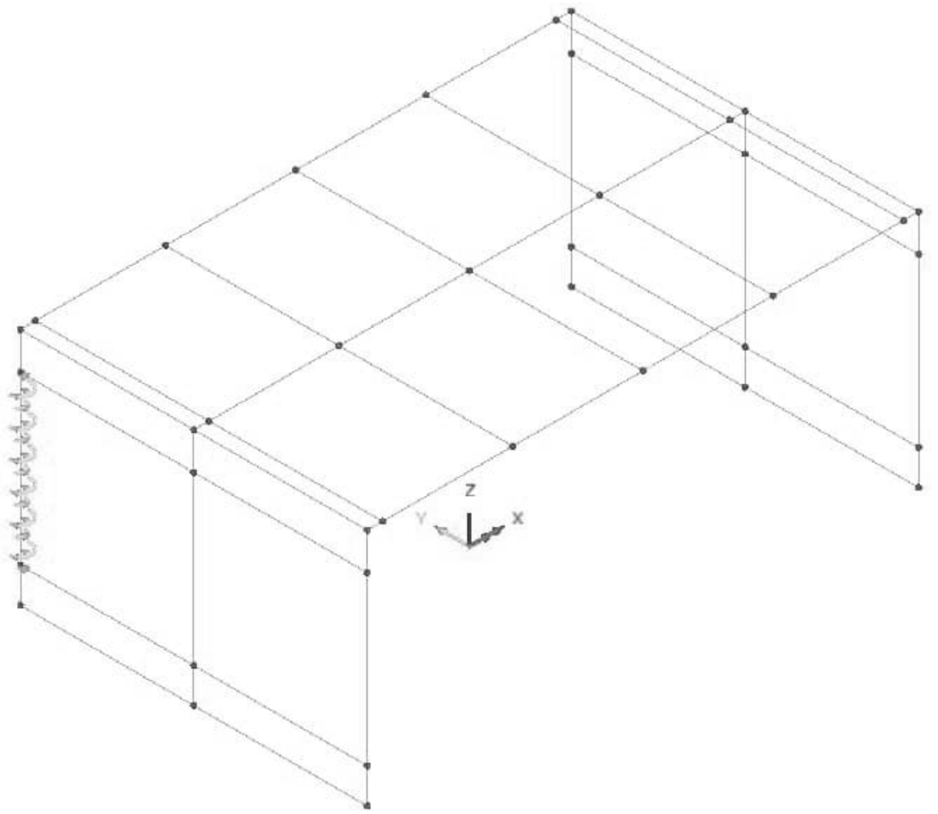
$x_{tp} = 1.9\,m$

Load case: JORD 3-1
(Northern wingwall abutment 1)

$$p_y = +29\frac{kN}{m}$$

$$m_z = -77\frac{kNm}{m}$$



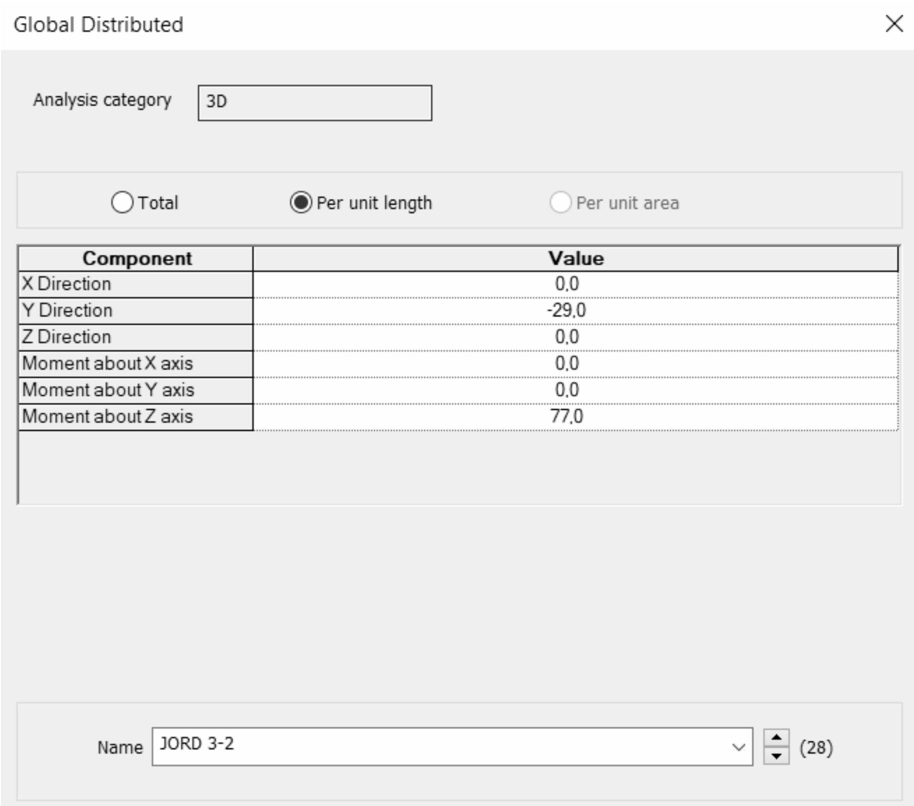


Overview 3D

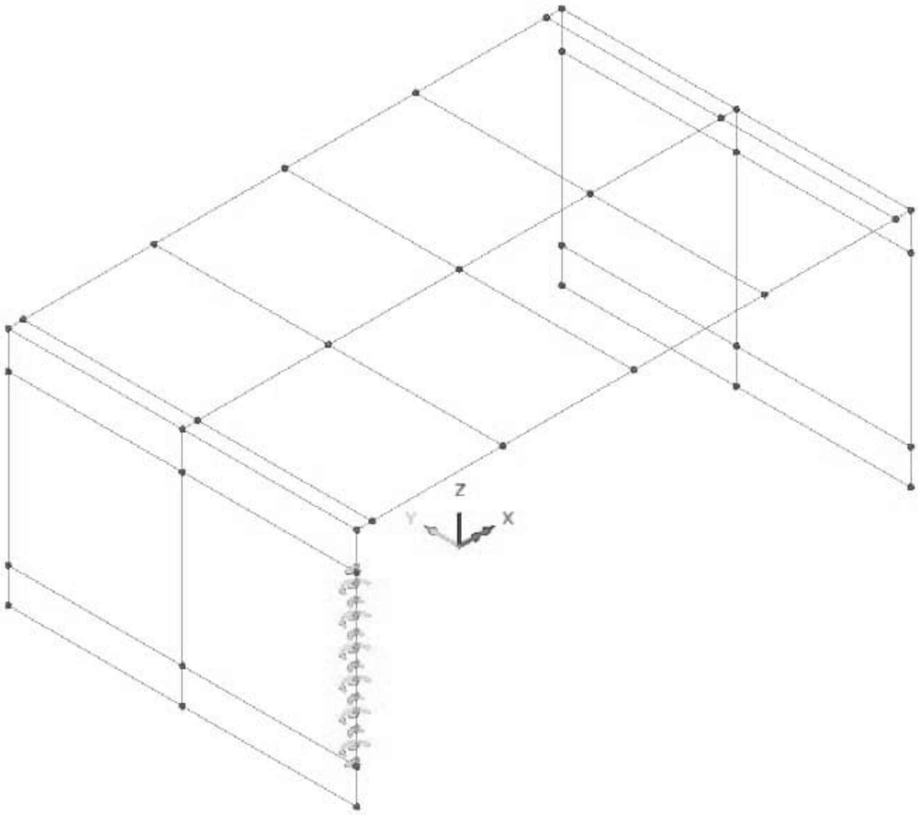
Load case : JORD 3-2
(Southern wingwall abutment 1)

$$p_y = -29\frac{kN}{m}$$

$$m_z = +77\frac{kNm}{m}$$



Component	Value
X Direction	0,0
Y Direction	-29,0
Z Direction	0,0
Moment about X axis	0,0
Moment about Y axis	0,0
Moment about Z axis	77,0

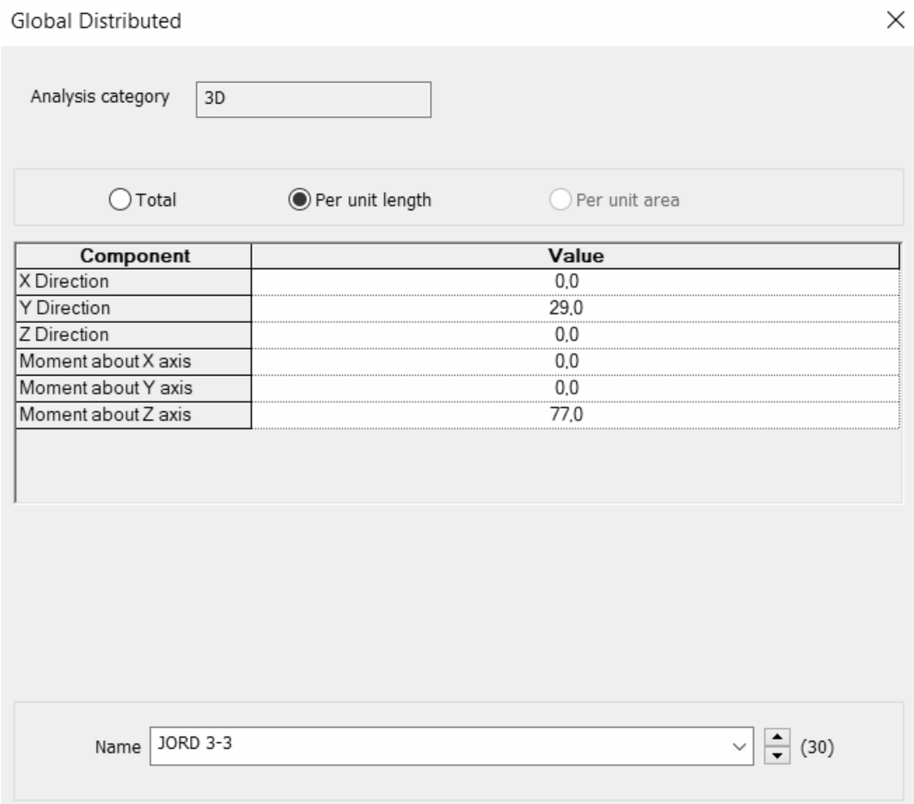


Oveview 3D

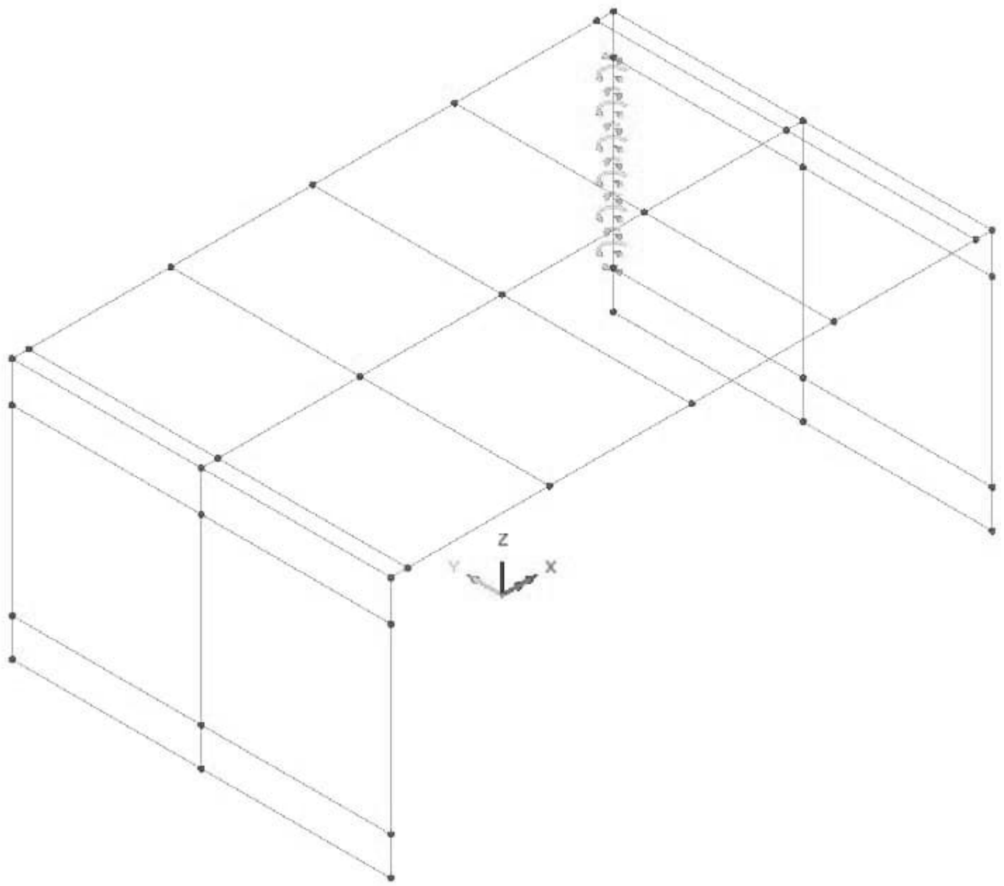
Load case : JORD 3-3
(Northern wingwall abutment 2)

$$p_y = +29\frac{kN}{m}$$

$$m_z = +77\frac{kNm}{m}$$



Component	Value
X Direction	0,0
Y Direction	29,0
Z Direction	0,0
Moment about X axis	0,0
Moment about Y axis	0,0
Moment about Z axis	77,0

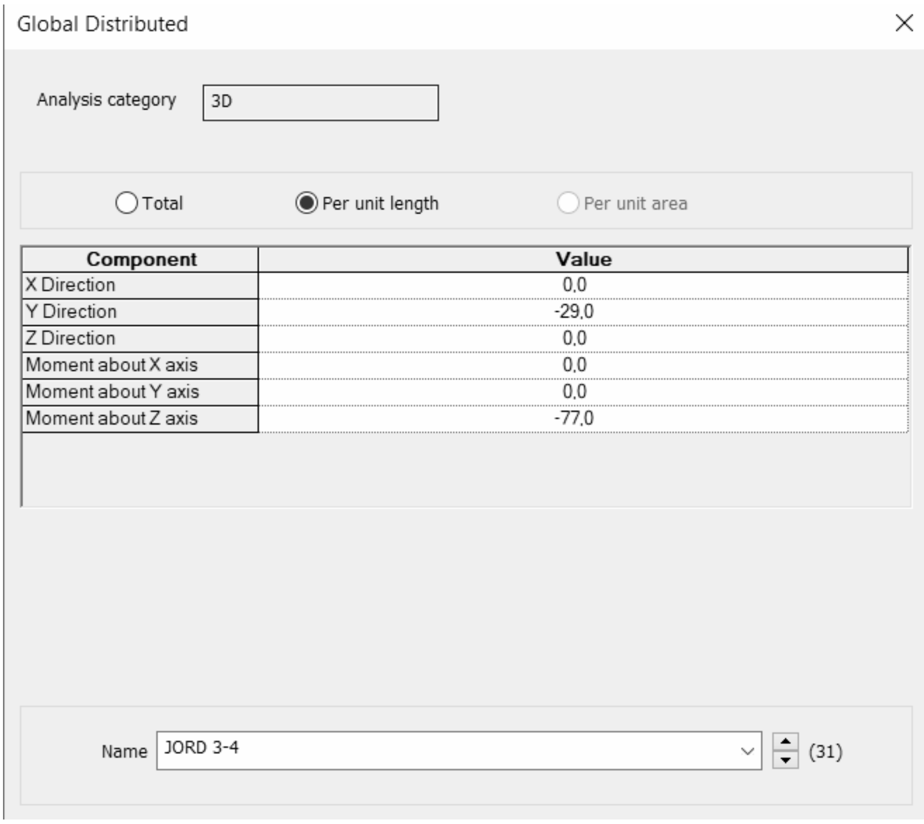


Overview 3D

Load case : JORD 3-4
(Southern wingwall abutment 2)

$$p_y = +29 \frac{kN}{m}$$

$$m_z = +77 \frac{kNm}{m}$$

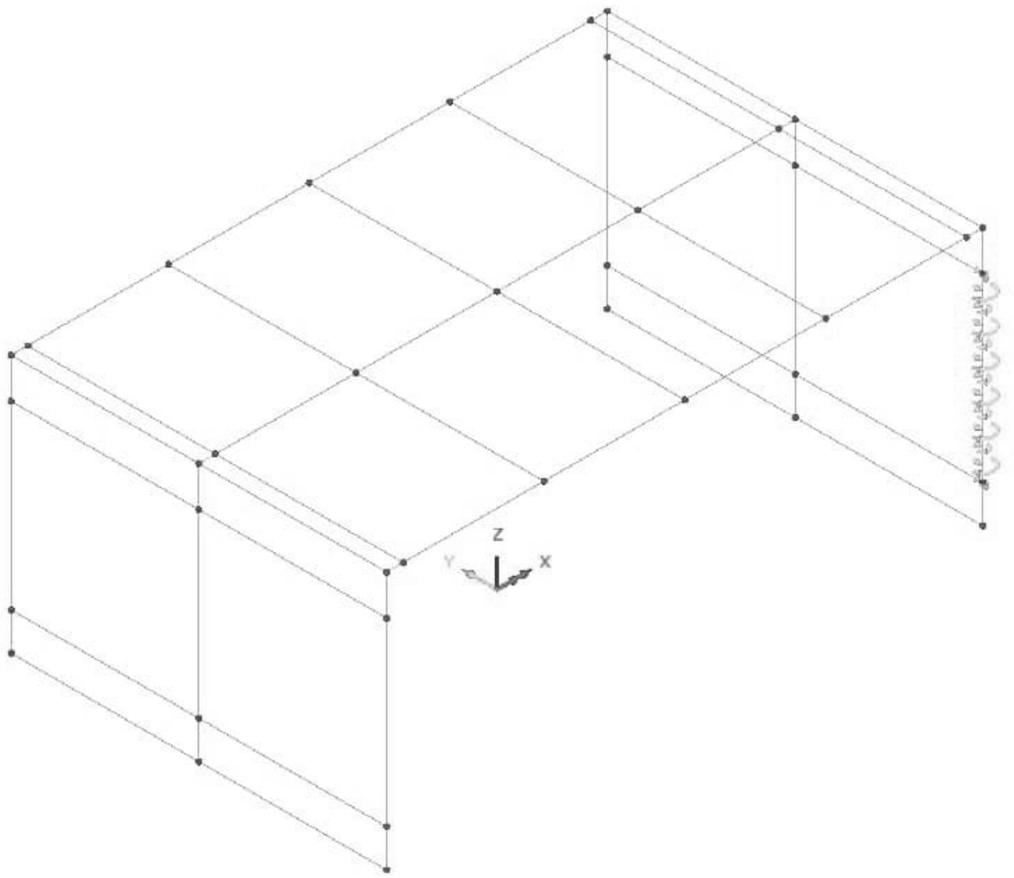

Global Distributed

Analysis category: 3D

Total / Per unit length / Per unit area

Component	Value
X Direction	0,0
Y Direction	-29,0
Z Direction	0,0
Moment about X axis	0,0
Moment about Y axis	0,0
Moment about Z axis	-77,0

Name: JORD 3-4 (31)



Overview 3D

3.3.4 Last combination earth pressure: JORD

Basic load combination JORD :

Load case	Factor
JORD 1	1
JORD 2	1
JORD 3-1	1
JORD 3-2	1
JORD 3-3	1
JORD 3-4	1

3.4 SUPPORT SETTLEMENT

Load effect of support settlement shall be considered in TRVINFRA-00227 section 7.2.1.1.1.1.

Only horizontal support displacement in the longitudinal direction of the bridge needs to be considered. Additionally, it is stated that horizontal and vertical support displacements do not need to be combined.

A horizontal support displacement in the longitudinal direction of the bridge (x-direction) of ±5 mm for each support is applied.

Vertical settlement difference (Z-direction) corresponding to support settlement of 10 mm is assumed to occur for all supports.

A verification will be performed to demonstrate that this is on the safe side.

Horizontal displacement (X-direction) amounts to ±10 mm.

When determining associated load effects, reduction is carried out with consideration to creep and cracking.

Note:
The impact of support settlement in serviceability limit state (SLS) according to SS-EN 1992-1-1 §2.3.1.3. If this occurs, a gradual crack development should be applied according to SS-EN 1992-1-1 §5.4(3). Reduction is carried out with consideration to creep and cracking.

The impact of support settlement is not considered in the ultimate limit state (ULS) according to SS-EN 1992-1-1 §2.3.1.3 for this type of bridge.

3.4.1 Vertical settlement

3.4.1.1 Support 1

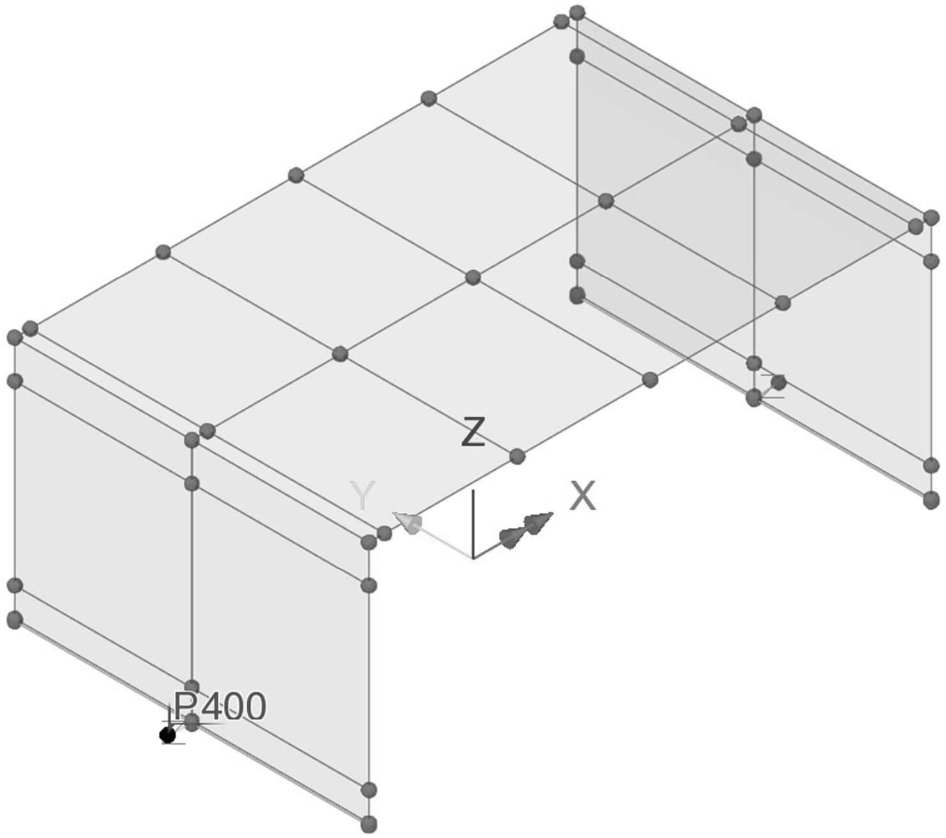
Load: STOD_1Z

Structural loading : Prescribed Displacement

Translation at point in Z direction : -0.010 m

Load case : STOD_1Z

Point : P400



Overview 3D

3.4.1.2 Support 2

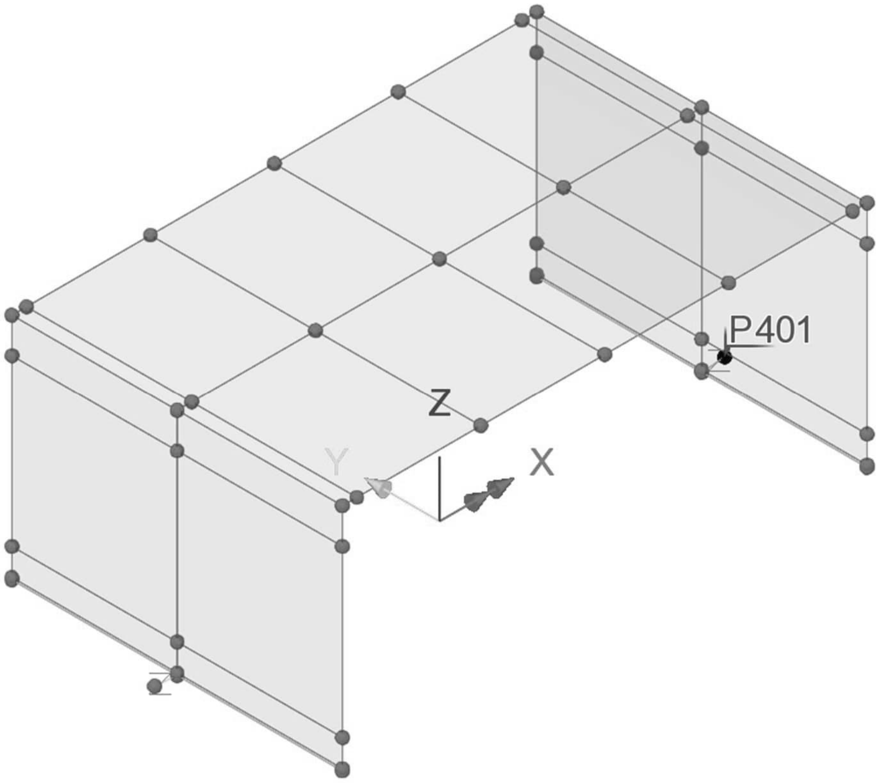
Load: STOD_2Z

Structural loading : Prescribed Displacement

Translation at point in Z direction : -0.010 m

Loadcase : STOD_2Z

Point : P401



Overview_3D

3.4.2 Horizontal settlement

3.4.2.1 Support 1

Load : STOD_1X

Structural loading : Prescribed Displacement

Translation at point in X direction : 0.010 m

Load case : STOD_1X+

Point : P400

3.4.2.2 Support 2

Load: STOD_2X

Structural loading: Prescribed Displacement

Translation at point in X direction: 0.010 m

Load case : STOD_2X+

Point : P401

3.4.3 Load combination settlement: STOD

Basic loadcases :

Loadcase	Loadcase	Factor
STOD_1X-	STOD_1X+	-1
STOD_2X-	STOD_2X+	-1

Envelope STOD-X :

Loadcase
STOD_1X+
STOD_2X+
STOD_1X-
STOD_2X-

Smart loadcombination STOD-Z :

Loadcase	Permanent factor	Variable factor
STOD_1Z	0	1.0
STOD_2Z	0	1.0

Envelope STOD :

Loadcase
STOD-X
STOD-Z

3.5 CREEP

Total creep is determined according to SS-EN 1992-1-1 §3.1.4 and TRVINFRA-00227 section 7.1.6.4 for RH 80% at time t_1.

Time for first loading (= time when formwork was removed) is termed t_0.

$t_0 = 5\ days$

$t_1 = 120\ years$

Bridge consists of parts with different thicknesses as seen below.

Creep is determine using Mathcad program PROG A001.

Structure (b = 8.0 m; C35/45):
For t = 0.80 m → $\phi(t_1, t_0) = 1.97$: see page A3:52
For t = 1.20 m → $\phi(t_1, t_0) = 1.91$: see page A3:52

Creep $\phi(t_1, t_0) = 1.9$ is used for the entire bridge on safe side since reduces stiffness and associated constraint forces (∴ support settlement, shrinkage and temperature).

$$\varepsilon_{cc}(t_1, t_0) = \varphi(t_1, t_0) \cdot \frac{\sigma_c}{E_c}$$

To study the effect concrete stiffness according to SS-EN 1992-1-1 5.8.7 creep values seen below are used.

Load cases	φ
Permanent	1.9
Variable excluding temperature	0
Temperature	0.3*

* = According to Swedish work practice

$$E^{system} = \frac{E_{cm}}{1+\varphi}$$

Instead of adjusting E-modulus the load coefficients are adjusted.

$$f_{KRYMP} = \frac{1}{1+\varphi_{ef}} = \frac{1}{1+1.70} = 0.37$$

$$f_{STÖD} = \frac{1}{1+\varphi_{ef}} = \frac{1}{1+1.70} = 0.37$$

$$f_{JTEMP} = \frac{1}{1+\varphi_{ef}} = \frac{1}{1+0.3} = 0.77$$

Note:
According to TRVINFRA-00227 section 7.2.1.1.2.4, no reduction is permitted for uneven temperature across the cross-section. This is because this temperature variation is considered to have a very short duration (only over the day).

Objekt: Plattram

Betong (C30/37, C35/45, C40/50 och C45/55): $BTG = "C35/45"$

Relativ fuktighet : $RH = 80\%$

Tvärsnittetsbredd : $b = 8.0m$

Tvärsnittetshöjd : $h = 0.7m$ (1.20 m)

Tvärsnittsarea : $A_c = b \cdot h = 5.6 \cdot m^2$ (9.60 m²)

Omkrets i kontakt med "luft" : $u = 2 \cdot b = 16\,m$

Bärverkets ekvivalenta tjocklek : $h_0 = \frac{2 \cdot A_c}{u} = 0.7\,m$

Studerad tidpunkt för bestämning av krypning : $t_1 = 120år$ $t_1 = 43800 \cdot dag$

Tidpunkt för pålastning (= formrivning): $t_0 = 5dag$

Indatakvitto

$f_{cm} = 43 \cdot MPa$

BERÄKNING

Uttryck för bestämning av kryptalet är hämtat från SS-EN 1992-1-1 Bilaga B.1.

$$\alpha_1 = \left(\frac{35\text{MPa}}{f_{cm}}\right)^{0.7} = 0.87$$

$$\alpha_2 = \left(\frac{35\text{MPa}}{f_{cm}}\right)^{0.2} = 0.96$$

$$\alpha_3 = \left(\frac{35\text{MPa}}{f_{cm}}\right)^{0.5} = 0.9$$

$$\varphi_{RH} = \begin{cases} 1 + \dfrac{1 - RH}{0.1\sqrt[3]{\dfrac{h_0}{mm}}} & \text{if } f_{cm} \le 35\text{MPa} \\ \left(1 + \dfrac{1 - RH}{0.1\sqrt[3]{\dfrac{h_0}{mm}}}\cdot\alpha_1\right)\cdot\alpha_2 & \text{otherwise} \end{cases} = 1.15$$

$$\beta_0 = \frac{1}{0.1 + t_0^{0.20}} = 0.68$$

$$\beta_{fcm} = \frac{16.8}{\sqrt{\dfrac{f_{cm}}{MPa}}} = 2.56$$

$$\beta_H = \begin{cases} \text{if } f_{cm} \le 35MPa \\ \quad \beta_{H.max} \leftarrow 1500 \\ \quad \beta_H \leftarrow \beta_{H.max} \text{ if } 1.5\cdot\left[1 + (0.012\cdot 100RH)^{18}\right]\cdot\frac{h_0}{mm} + 250 > \beta_{H.max} \\ \quad \beta_H \leftarrow 1.5\cdot\left[1 + (0.012\cdot 100RH)^{18}\right]\cdot\frac{h_0}{mm} + 250 \text{ otherwise} \\ \text{if } f_{cm} > 35MPa \\ \quad \beta_{H.max} \leftarrow 1500\cdot\alpha_3 \\ \quad \beta_H \leftarrow \beta_{H.max} \text{ if } 1.5\cdot\left[1 + (0.012\cdot 100RH)^{18}\right]\cdot\frac{h_0}{mm} + 250 > \beta_{H.max} \\ \quad \beta_H \leftarrow 1.5\cdot\left[1 + (0.012\cdot 100RH)^{18}\right]\cdot\frac{h_0}{mm} + 250\cdot\alpha_3 \text{ otherwise} \\ \beta_H \end{cases}$$

$\beta_H = 1353$

$$\beta_c = \left(\frac{t_1 - t_0}{\beta_H + t_1 - t_0}\right)^{0.3} = 0.99$$

$\varphi_{t0} = \varphi_{RH}\cdot\beta_{fcm}\cdot\beta_0 = 1.99$ (1.93)

$\varphi_{t1} = \varphi_{t0}\cdot\beta_c = 1.97$ (1.91)

3.6 SHRINKAGE

Total shrinkage according to SS-EN 1992-1-1 §3.1.4 and TRVINFRA-00227 section 7.1.6.4 for RH 80% at time t_1.

Determination of load effect from shrinkage should consider the reduced concrete stiffness from creep.

$t_s = 0\ days$

$t_1 = 120\ years$

Shrinkage is determined using Mathcad program PROG A002 after time t_1.

Structure (b = 8.0 m; C35/45):
For t = 0.80 m → $\varepsilon_{cs}\ (t_1) = 0.024\%$: see page A3:53
For t = 1.20 m → $\varepsilon_{cs}\ (t_1) = 0.023\%$: see page A3:53

Shrinkage $\varepsilon_{cs} = 0.024\%$ is applied to all construction parts for safety. The movement corresponds to that which occurs due to an imaginary temperature load ∴ T = -24°C.

Remark
Shrinkage must be considered for service state (SLS) see SS-EN 1992-1-1 §2.3.2.2(1).

Shrinkage does not have to be used for ultimate state (ULS) see SS-EN 1992-1-1 §2.3.2.2(2).

Objekt: Plattram

Betong (C30/37, C35/45, C40/50 och C45/55): $BTG := "C35/45"$

Relativ fuktighet (se KBB avsnitt B.3.3.6): $RH := 80\%$

Tvärsnittetsbredd : $b := 8.0m$

Tvärsnittetshöjd : $h := 0.7m$ (1.2m)

Tvärsnittsarea : $A_c := b \cdot h = 5.60 \cdot m^2$

Omkrets i kontakt med "luft" : $u := 2 \cdot b = 16\,m$

Bärverkets ekvivalenta tjocklek : $h_0 := \frac{2 \cdot A_c}{u} = 0.7\,m$

Studerad tidpunkt för bestämning av krympning : $t_1 := 120år$ $t_1 = 43800 \cdot dag$

Tidpunkt för pålastning (= formrivning): $t_0 := 5dag$

Cementklass (S, N, R) : $Klass := "N"$

Betongens ålder då uttorkningskrympning påbörjas : $t_s := 0dag$

Indatakvitto

$f_{cm} = 43 \cdot MPa$ $f_{ck} = 35 \cdot MPa$ $f_{ck.kub} = 45 \cdot MPa$

BERÄKNING

Grundvärdet för krympning från uttorkning (SS-EN 1992-1-1 Bilaga B.2)

$$\alpha_{ds1} = \begin{vmatrix} 3.0 \text{ if Klass} = \text{"S"} \\ 4.0 \text{ if Klass} = \text{"N"} \\ 6.0 \text{ if Klass} = \text{"R"} \end{vmatrix} = 4.00$$

$$\alpha_{ds2} = \begin{vmatrix} 0.13 \text{ if Klass} = \text{"S"} \\ 0.12 \text{ if Klass} = \text{"N"} \\ 0.11 \text{ if Klass} = \text{"R"} \end{vmatrix} = 0.12$$

$$RH_o = 100 \cdot \%$$

$$\beta_{RH} = 1.55 \cdot \left[1 - \left(\frac{RH}{RH_o}\right)^3\right] = 0.76$$

$$\varepsilon_{cd.0} = 0.85 \left[(220 + 110 \cdot \alpha_{ds1}) \cdot e^{-\alpha_{ds2} \cdot \frac{f_{cm}}{f_{cmo}}}\right] \cdot 10^{-6} \cdot \beta_{RH} = 0.025 \cdot \%$$

Grundvärdet för krympning från uttorkning (SS-EN 1992-1-1 avsnitt 3.1.4 ekv. 3.9 och 3.10)

$$k_h = \text{linterp}\left[\left(0 \;\; 100 \;\; 200 \;\; 300 \;\; 500 \;\; 10^4\right)\text{mm}^T, (1.00 \;\; 1.00 \;\; 0.85 \;\; 0.75 \;\; 0.70 \;\; 0.70)^T, h_0\right] = 0.70$$

$$\beta_{ds} = \frac{t_1 - t_s}{t_1 - t_s + 0.04\sqrt{\left(\frac{h_0}{mm}\right)^3}} = 0.98$$

$$\varepsilon_{cd} = \beta_{ds} \cdot k_h \cdot \varepsilon_{cd.0} = 0.017 \cdot \%$$

Autogen krympning (SS-EN 1992-1-1 avsnitt 3.1.4 ekv. 3.11, 3.12 och 3.13)

$$k_h = \text{linterp}\left[\begin{pmatrix}0 & 100 & 200 & 300 & 500 & 10^4\end{pmatrix}\text{mm}^T, \begin{pmatrix}1.00 & 1.00 & 0.85 & 0.75 & 0.70 & 0.70\end{pmatrix}^T, h_0\right] = 0.70$$

$$\beta_{as} = 1 - e^{-0.2\cdot\sqrt{t_1}} = 1.00$$

$$\varepsilon_{ca.\propto} = 2.5\cdot\left(\frac{f_{ck}}{MPa} - 10\right)\cdot 10^{-6} = 0.006\cdot\%$$

$$\varepsilon_{ca} = \beta_{as}\cdot\varepsilon_{ca.\propto} = 0.006\cdot\%$$

Total krympning (SS-EN 1992-1-1 avsnitt 3.1.4 ekv. 3.8)

$$\varepsilon_{cs} = \varepsilon_{cd} + \varepsilon_{ca} = 0.024\cdot\%$$ (0.023%)

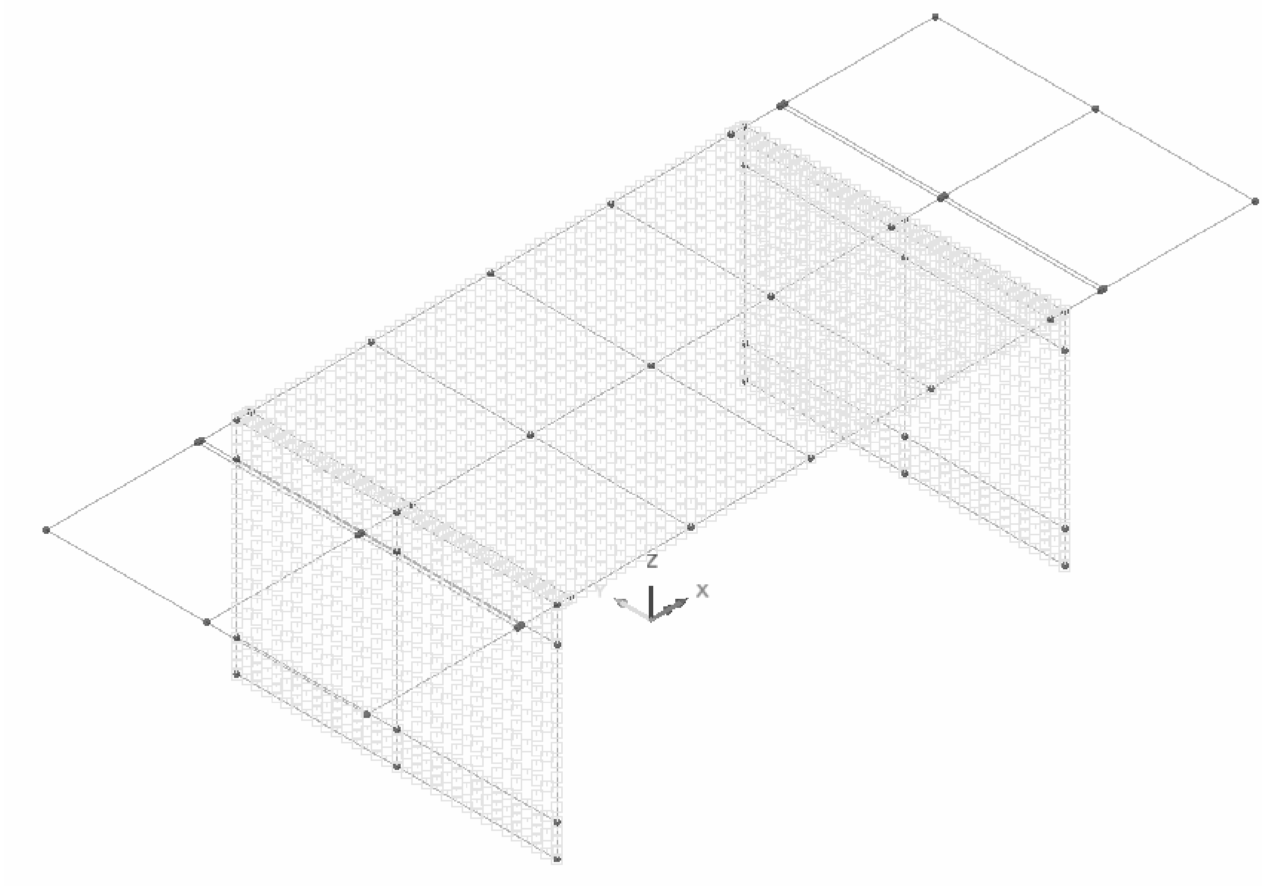
Load case : KRYMP

Structural loading : Temperature

Definition : Nodal

Initial temperature : 0 °C

Final temperature : -24 °C



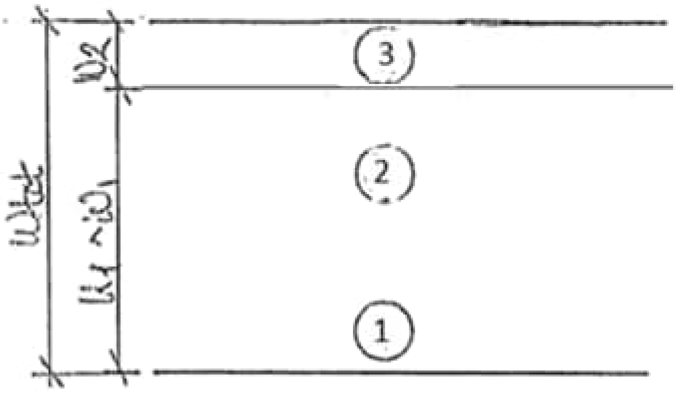
Overview 3D

3.7 TRAFFIC LOAD

Evaluation of vertical traffic is performed for LM 1 and LM 2 according to SS-EN 1991-2 section 4.3.

Evaluation will also be performed EG A/B = 180kN/300 kN according to TRVFS 2011:12 chapter 6 point 3§.

3.7.1 Traffic lane division



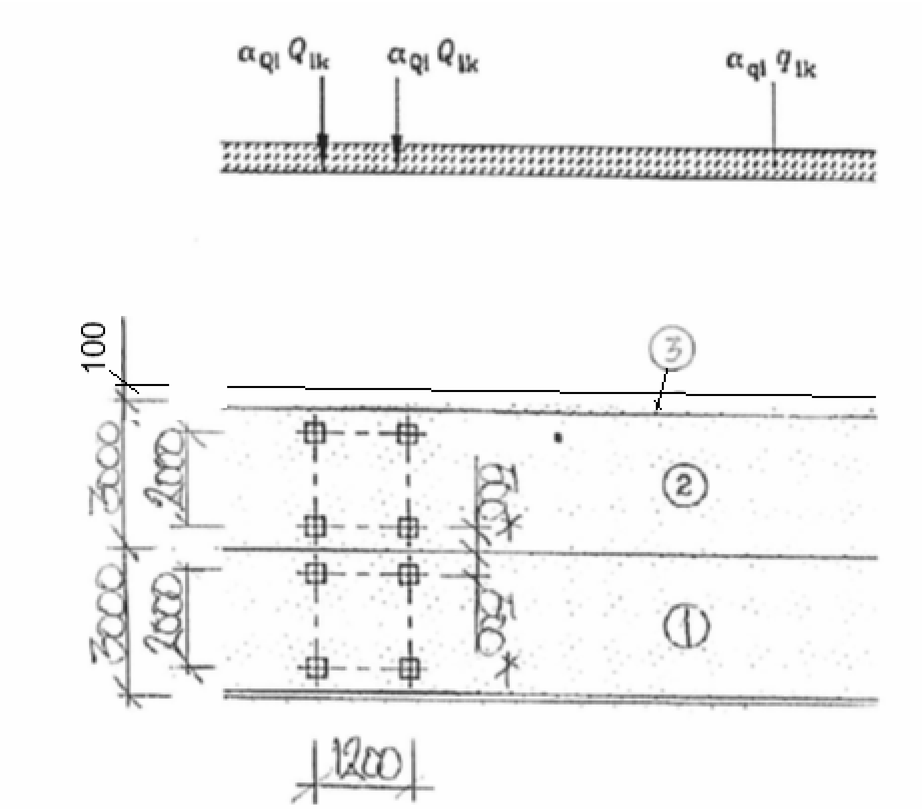
-

Total traffic width : $w_{tot} = 8.0\,m$

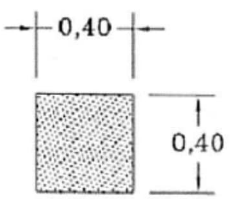
Number of traffic lanes : $n_1 = Integer\left[\frac{w_{tot}}{3.0m}\right] = 2\,lanes$

Full traffic width : $w_1 = 3.0m$

Remaining width : $w_2 = 2.0m$



* = When studying local effects 250 mm is to be assumed.



Axle loads:

α_Q : national adaptation factor according to TRVFS 2011:12 table 7.1

$Q'_k = \alpha_Q \cdot Q_k$: characteristic value including national adaptation factor

Traffic lane	Q_k	α_Q	Q'_k	Remark
1	300	0,9	270	LM1- 2 x 270 kN
2	200	0,9	180	LM1- 2 x 180 kN
3	100	0	0	No load
-	kN	-	kN	-

Surface loads:

α_q : national adaptation factor according to TRVFS 2011:12 table 7.1

$q'_k = \alpha_q \cdot q_k$: characteristic value including national adaptation factor

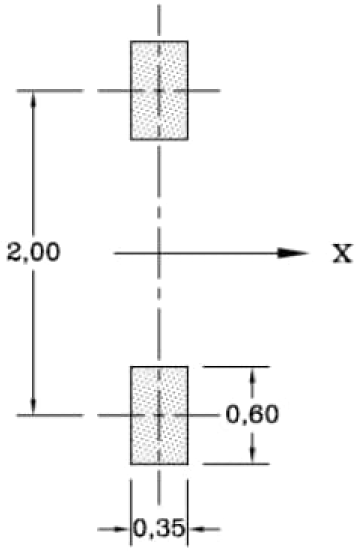
Traffic lande	q_k	α_q	q'_k
1	9.0	0.8	7.2
2-3	2.5	1.0	2.5
-	kPa	-	kPa

Remark

Evaluation is performed using Vehicle Load Optimisation (VLO), see section 3.7.4.

3.7.3 Load model 2 (LM 2)

Characteristic vertical load according to SS-EN 1991-2 §4.3.3.



$\beta_Q = \alpha_Q = 0.90$: national adaptation factor

$Q_k = 400$ kN : characteristic value

$Q'_k = \beta_k \cdot Q_k = 360$ kN : characteristic value including national adaptation factor

Remark

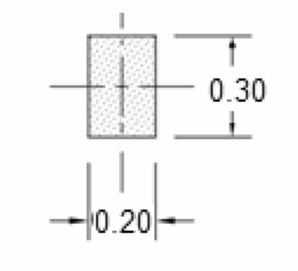
Not considered as critical for this bridge type, thus not included in FEM-analysis.

3.7.4 Load model EG A/B

Calculation is performed using traffic load EG A/B = 180 kN/300 kN excluding dynamic factor.

Traffic load EG A/B are applied to two traffic lanes. Traffic on first lane is multiplied by 1.00 while second lane is multiplied 0.80.

The center distance between the wheel pressures is 2.0 meters according to TSFS chapter 11 §2.



Wheel pressure

$\varepsilon_{dyn} = 25$ % : dynamic factor [1.)]

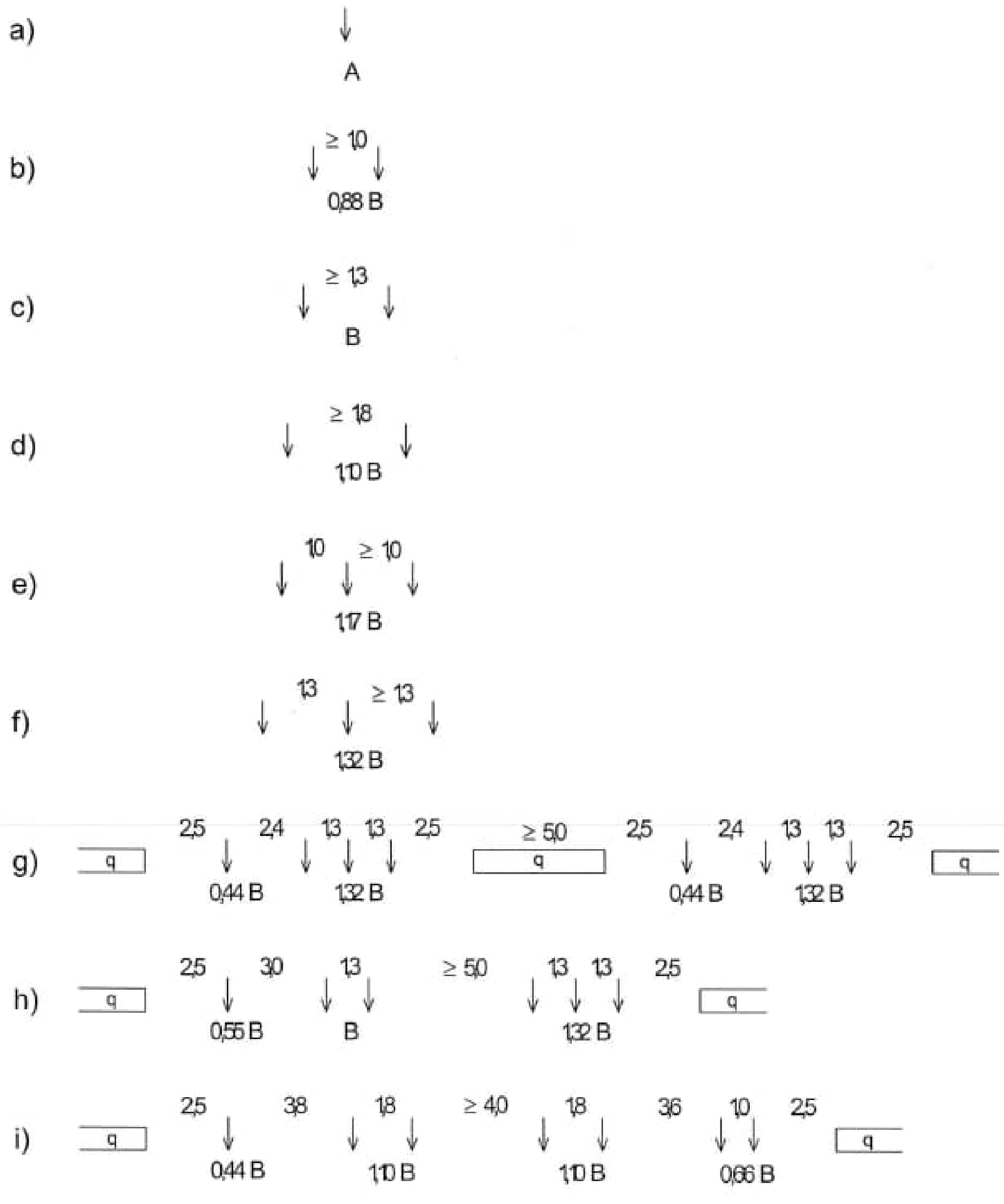
$A' = A \cdot (1 + \varepsilon_{dyn}) = 180$ kN$\cdot(1+0.25) = 225$ kN : single load including dynamic factor

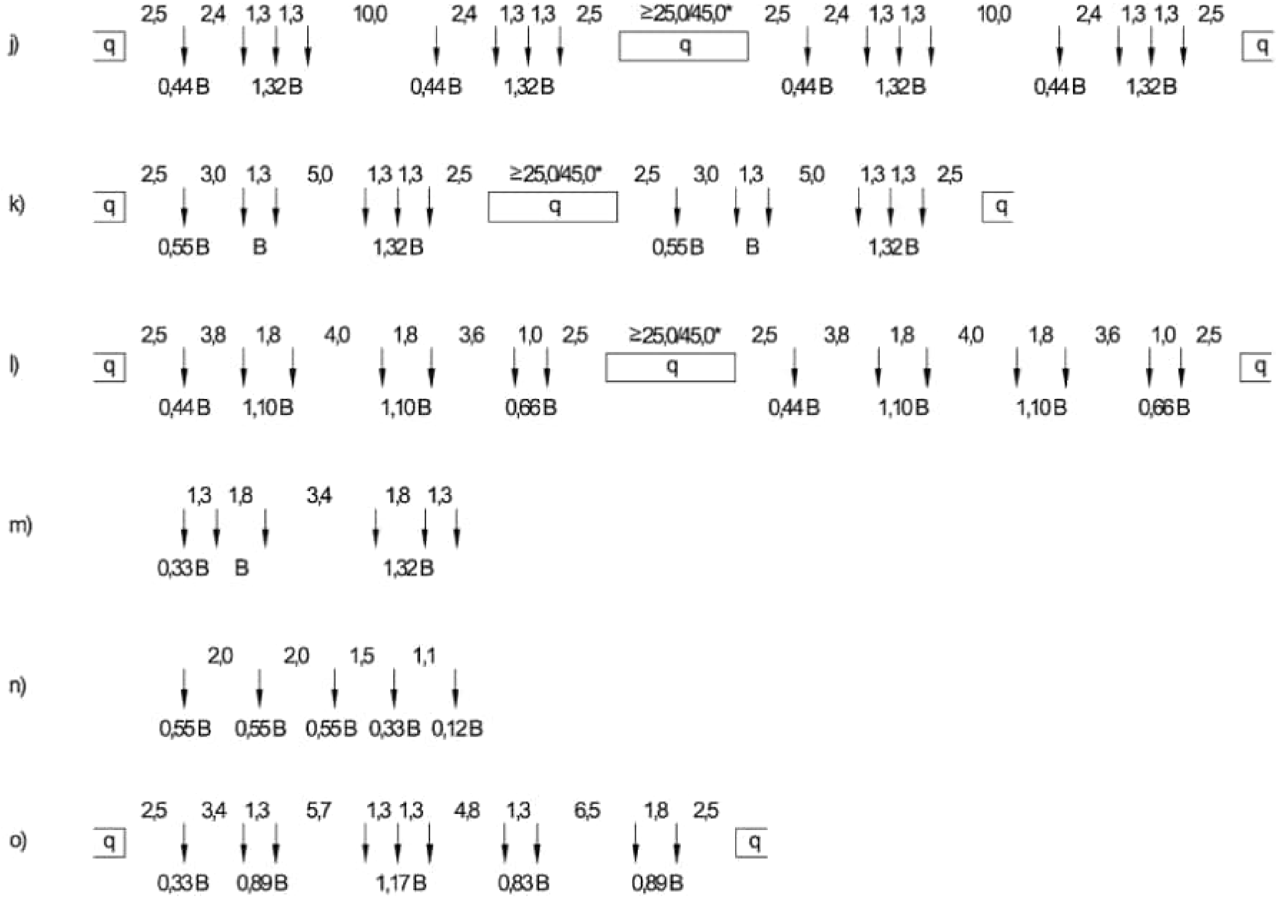
$B' = B \cdot (1 + \varepsilon_{dyn}) = 300$ kN$\cdot(1+0.25) = 375$ kN : tandem load including dynamic factor

$p = 5 \frac{kN}{m}$: surface load

Footnote:
[1.)] TRVINFRA-00227 table 7.1-5 section 4.2.1(1) states apply 25 %.

Graphic presentation of common vehicle types:
(Vehicle types according to TRVINFRA-00331 Appendix 1)



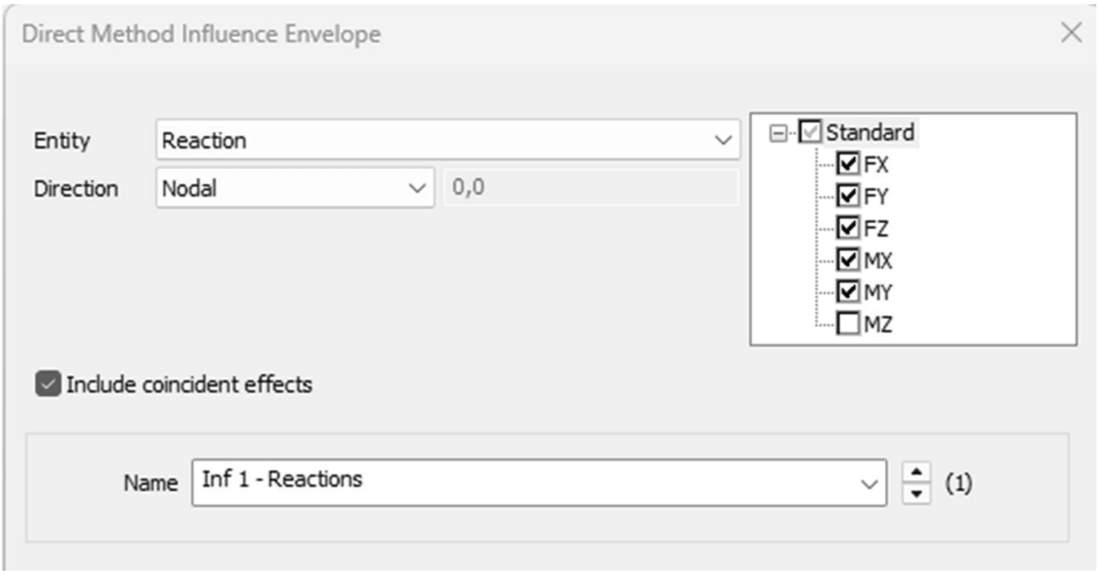


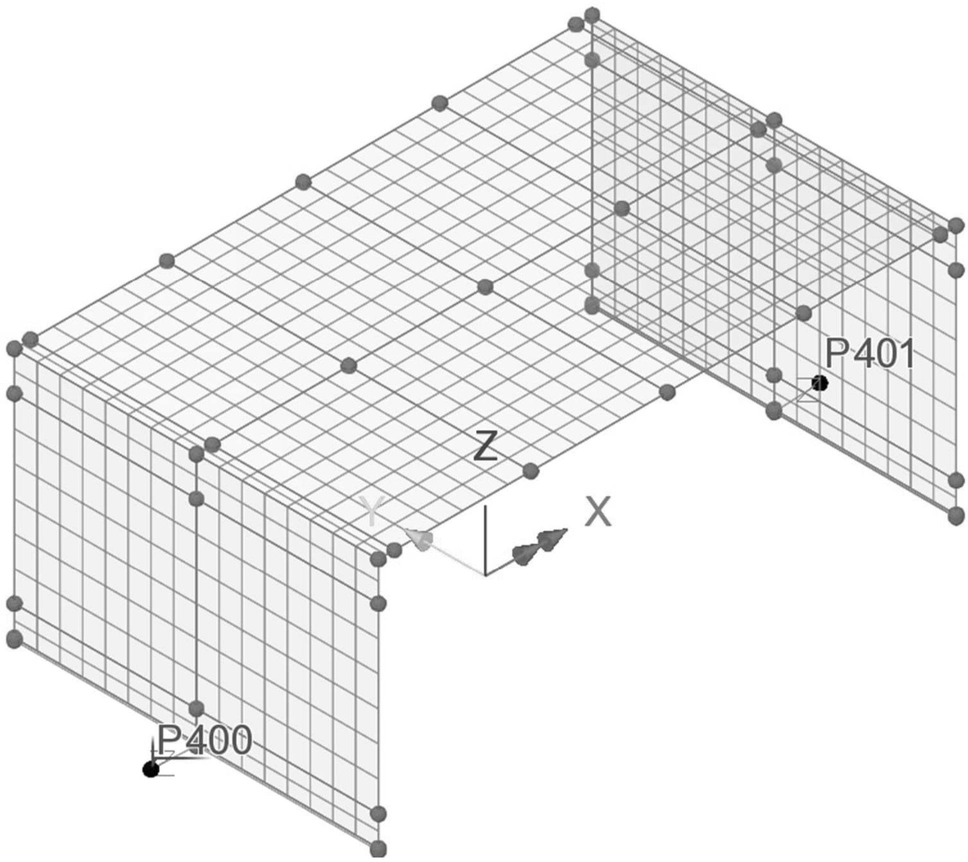
Note:
Evaluation is carried out with the script Vehicle Load Optimization (VLO), see sections 3.5.3 and 3.5.4.
Since there is no motorway, * = 45 m is applied according to TRVINFRA-00331 section 8.3.2.2.1 for vehicle types j, k, and l.

3.7.5 Vehicle Load Optimization (VLO)

Influence surfaces are created using *Direct Method Influence Envelope*. This is done by applying *Influence components* seen below.

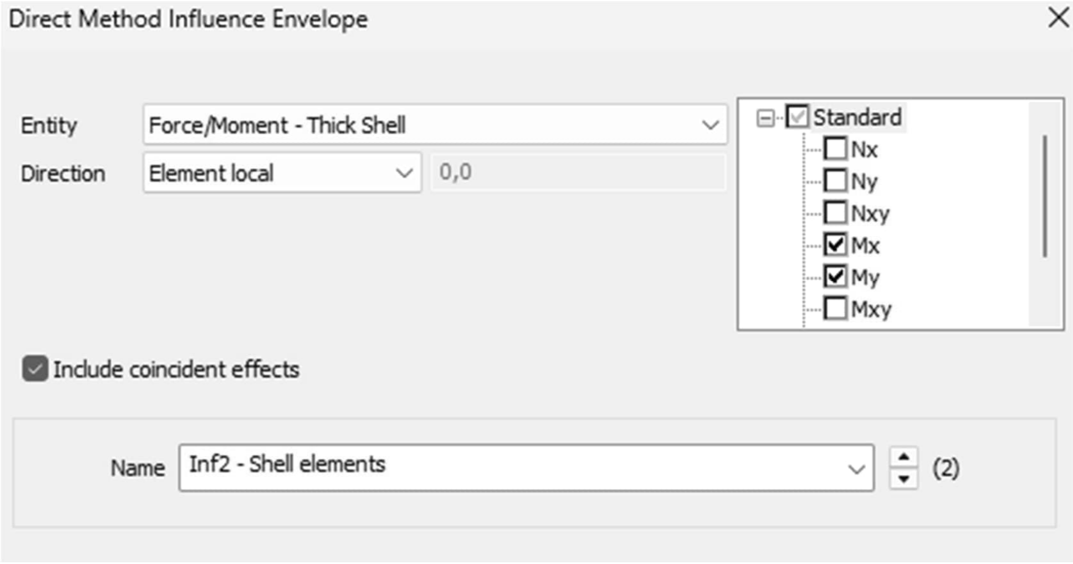
Inf1 – Reactions :

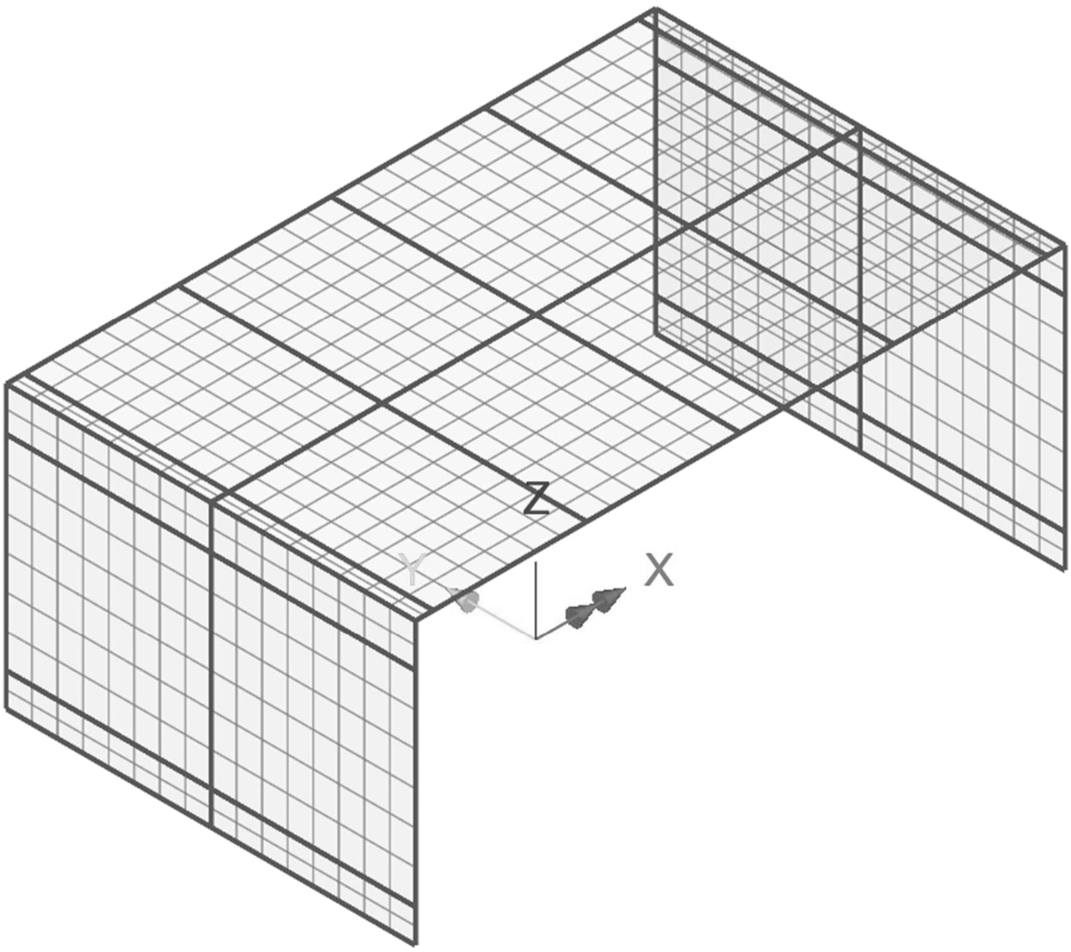




Overview 3D

Inf2 – Thick shells :
(Abutements & bridge deck)





Overview 3D

Influence surfaces :

Search area: Superstructure

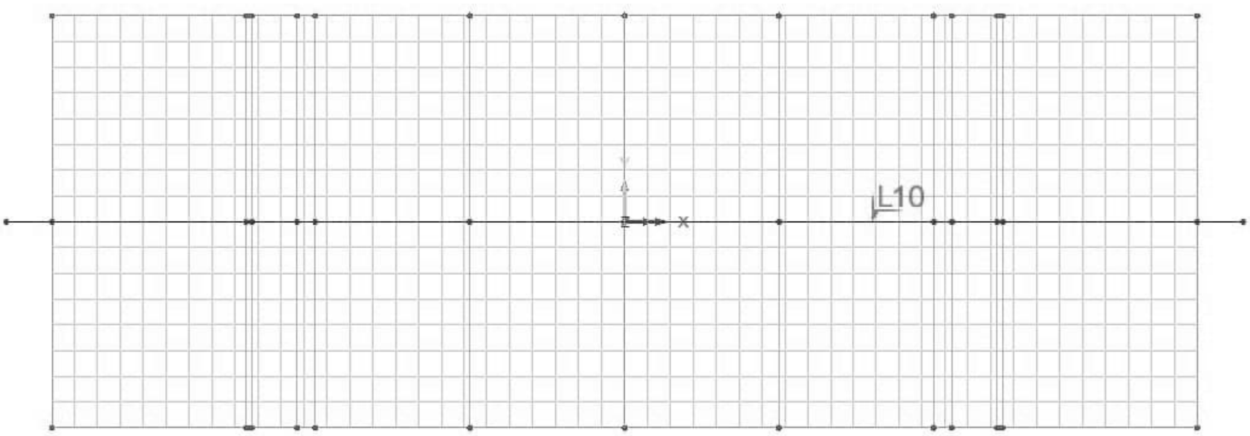
Definition type: Grid

Centerline (path): L10

Transverse width: 8.0 m

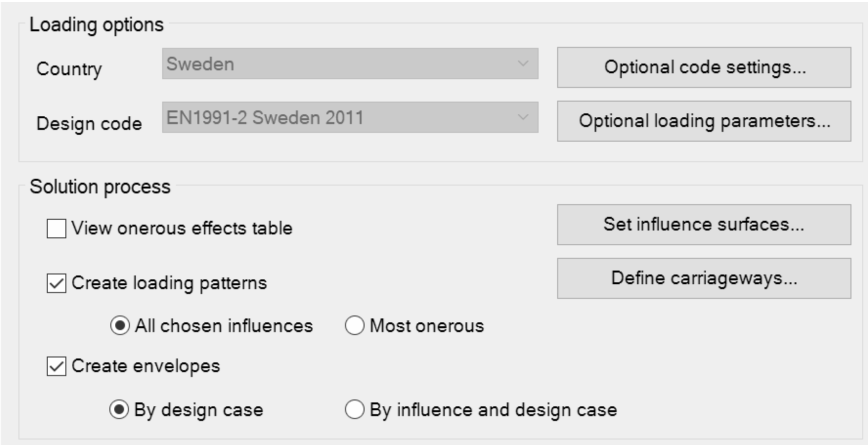
Longitudinal spacing: 0.5 m

Transversal spacing: 0.5 m

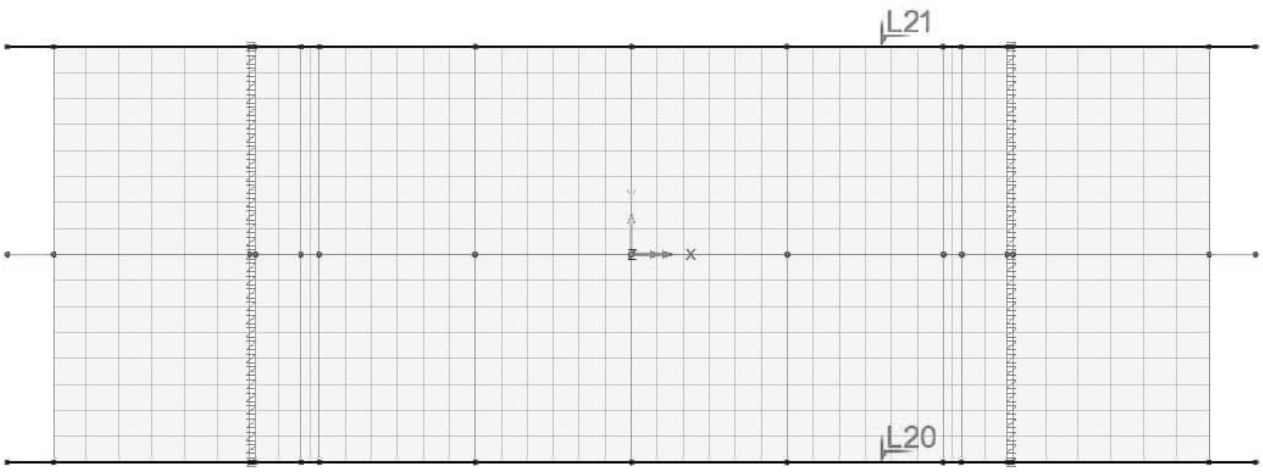


PLAN

Vehicle load optimisation options:



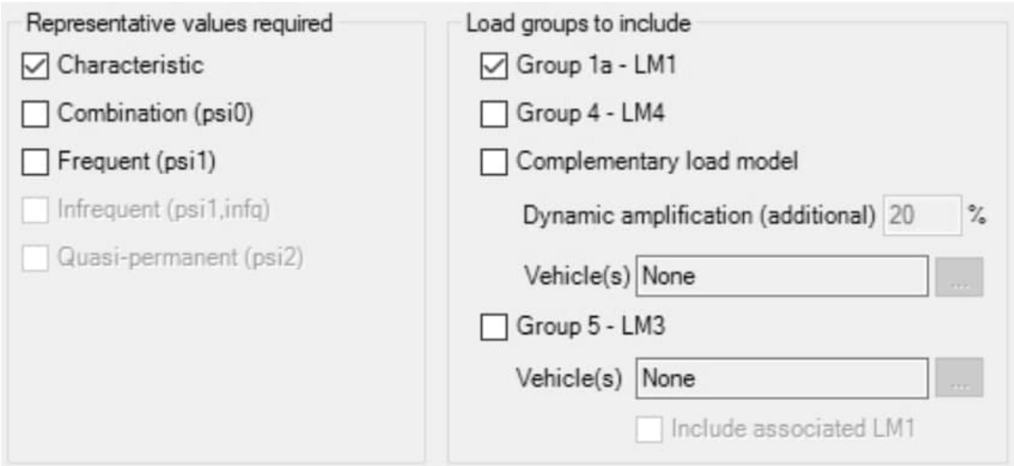
Vehicle longitudinal incremental movement:	0.25 m
Vehicle transverse incremental movement:	0.50 m
Vehicle direction:	both
Definition of carriageway (kerbs):	L20 & L21
Influence surfaces:	Include all (positive & negative)



PLAN

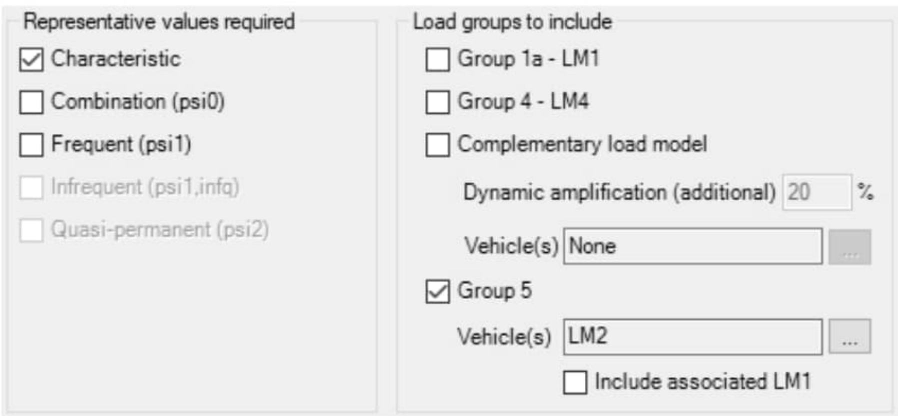
3.7.5.1 Envelope : LM 1

Lastmodell 1 (LM1) är definierad enligt SS-EN 1991-2 avsnitt 4.3.2.



3.7.5.2 Envelope : LM 2

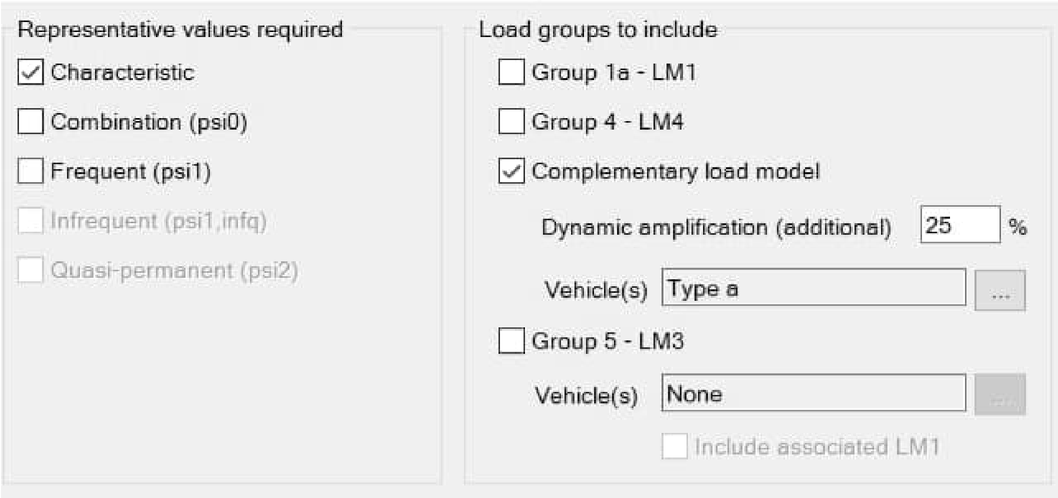
Lastmodell 2 (LM2) är definierad enligt SS-EN 1991-2 avsnitt 4.3.3. Lasten är införd i Group 5 (special vehicle) eftersom Group 1b saknas i nuvarande version.



Anm.
Detta lastfall anses inte dimensionerade för studerade brotyp. Lastfall utgår i statisk modell.

3.7.5.3 Envelope : EG A

Typfordon EG A är definierad enligt nedan.

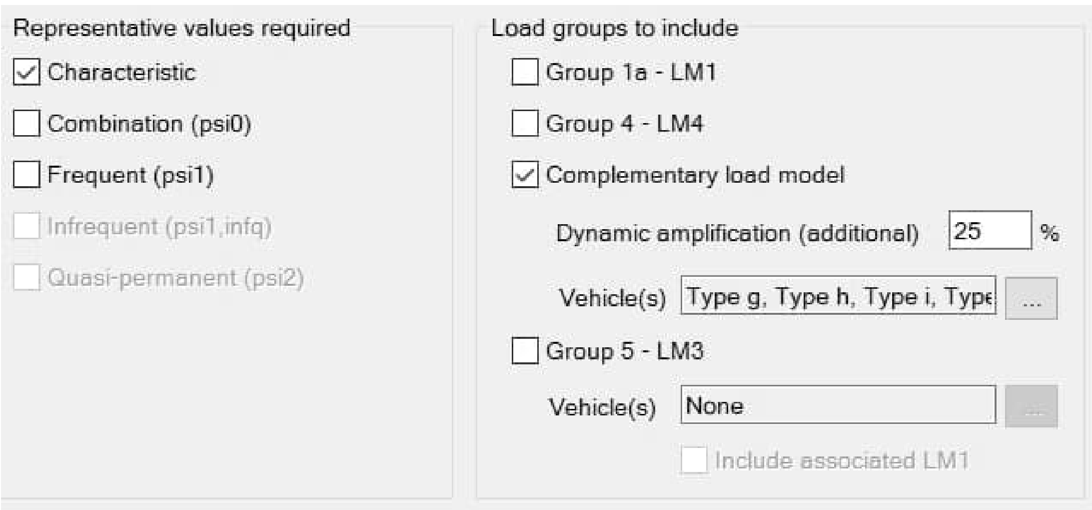


Dynamic amplification (additional): 25 %

Vehicle selection: Type a

3.7.5.4 Envelope : EG B

Typfordon EG B är definierad enligt nedan.



Dynamic amplification (additional): 25 %

Vehicle selection: Type b → o

3.7.5.5 Combined traffic load (TRAFIK)

There are a total 4 different traffic loads termed LM 1, LM2, EG A and EG B.

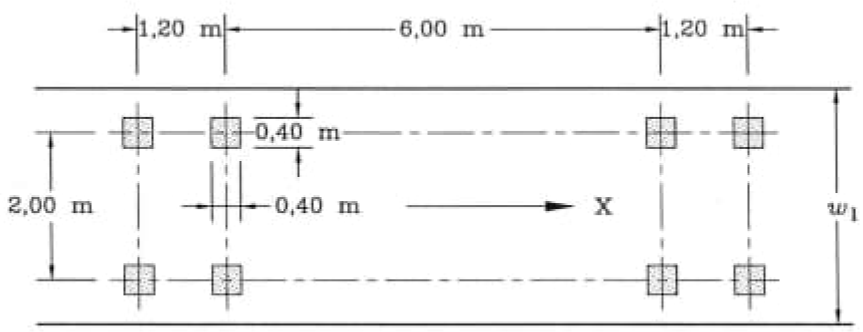
The envelope is used to identify the most onerous load effect.

Envelope TRAFIK :

Envelope
LM 1
LM 2
EG A
EG B

3.7.5.6 Fatigue model

Fatigue model 3 (UTM3) defined in SS-EN 1991-2 section 4.6.4. The load is defined in Group 5 (special vehicle) in present version of software.



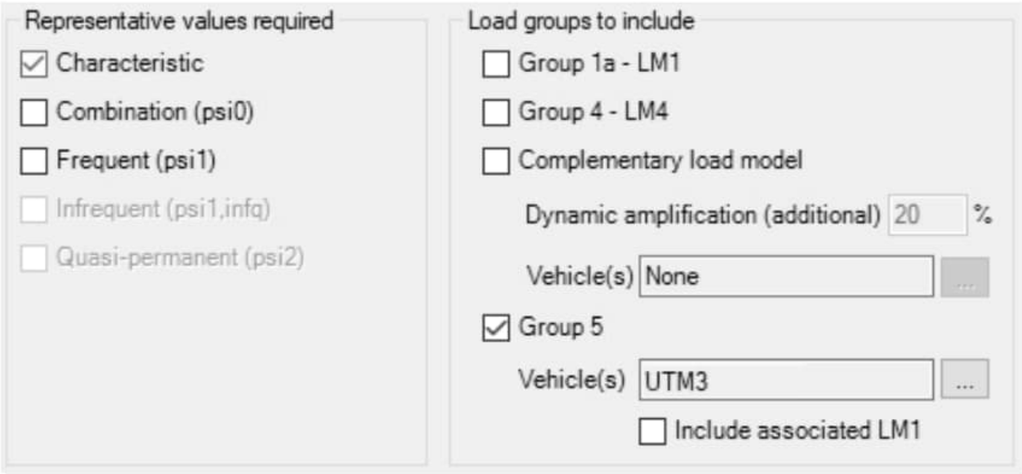
Q_k = 120 kN : characteristic value including nation adaptation factors.

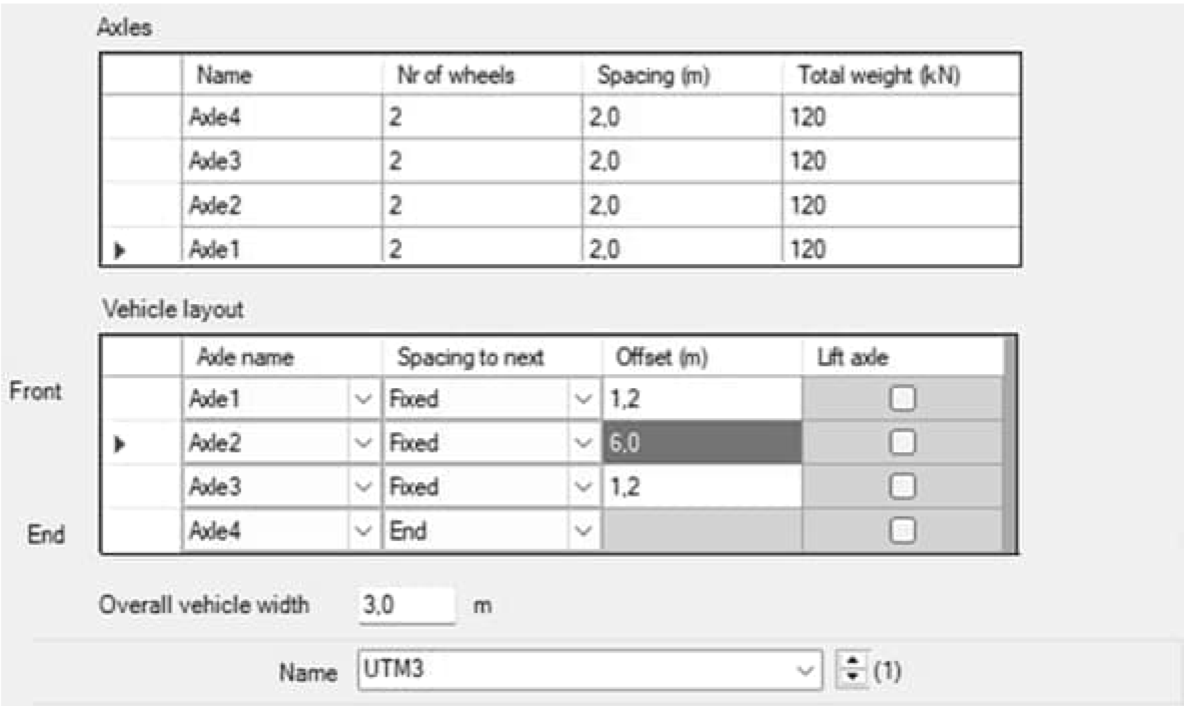
Traffic category:
TRVINFRA-0027 table 7.1-5(h) gives traffic category 3

Reference values for the number of heavy vehicles:
According to SS-EN 1991-2 section 4.6.1 table 4.5(n), Category 3 is obtained

→ N_{obs} = 125,000 vehicles/year

The load definition:
The load UTM3 is defined as a special vehicle in "load group 5."





Axles

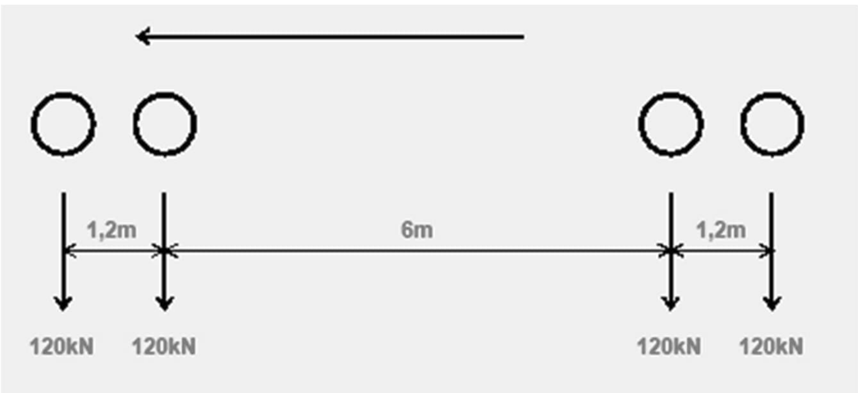
	Name	Nr of wheels	Spacing (m)	Total weight (kN)
	Axle4	2	2,0	120
	Axle3	2	2,0	120
	Axle2	2	2,0	120
▸	Axle1	2	2,0	120

Vehicle layout

		Axle name	Spacing to next	Offset (m)	Lift axle
Front		Axle1	Fixed	1,2	☐
	▸	Axle2	Fixed	6,0	☐
		Axle3	Fixed	1,2	☐
End		Axle4	End		☐

Overall vehicle width 3,0 m

Name UTM3 (1)



3.8 BRAKING LOAD

Braking load is defined by SS-EN 1991-2 §4.4.1.

Load acts at level of surfacing.

L = 0.25 m + 1.2 m + 12.0m + 1.2 m + 0.25 m = 14.4 m

Load modell LM 1 :

$Q_{lk} = 0.6\alpha_{Q1} \cdot (2Q_{ik}) + 0.1\alpha_{q1} \cdot q_{1k} \cdot w_1 \cdot L$

$180kN \cdot \alpha_{Q1} \leq Q_{lk} \leq 900kN$

$Q_{broms} = 0.6 \cdot (2 \cdot 270kN) + 0.1 \cdot 7.2kPa \cdot 3.0m \cdot 14.4m = 324kN + 31kN = 355kN$

Load model EG B = 300 kN (see TSFS chapter 11 §2) :

Type o is dimensioning.

$Q_{lk} = 0.35 \cdot \sum Q_{EG\ B} + 0.1 \cdot p \cdot L_q$

$Q_{lk} \leq 500kN$

$Q_{broms} = 0.35 \cdot (0.44 + 1.10 + 1.10) \cdot B = 0.35 \cdot 2.64 \cdot 300kN = 277kN$

Note:
The braking force associated with LM 1 is applied on the safe side in the system calculation.

The impact of the resisting earth pressure against the frame legs is neglected on the safe side.

Load definition:

The load is introduced as a surface load in the bridge deck's system line, located 0.46 meters below the pavement level. In the static model, this is disregarded since the effect of load my is small.

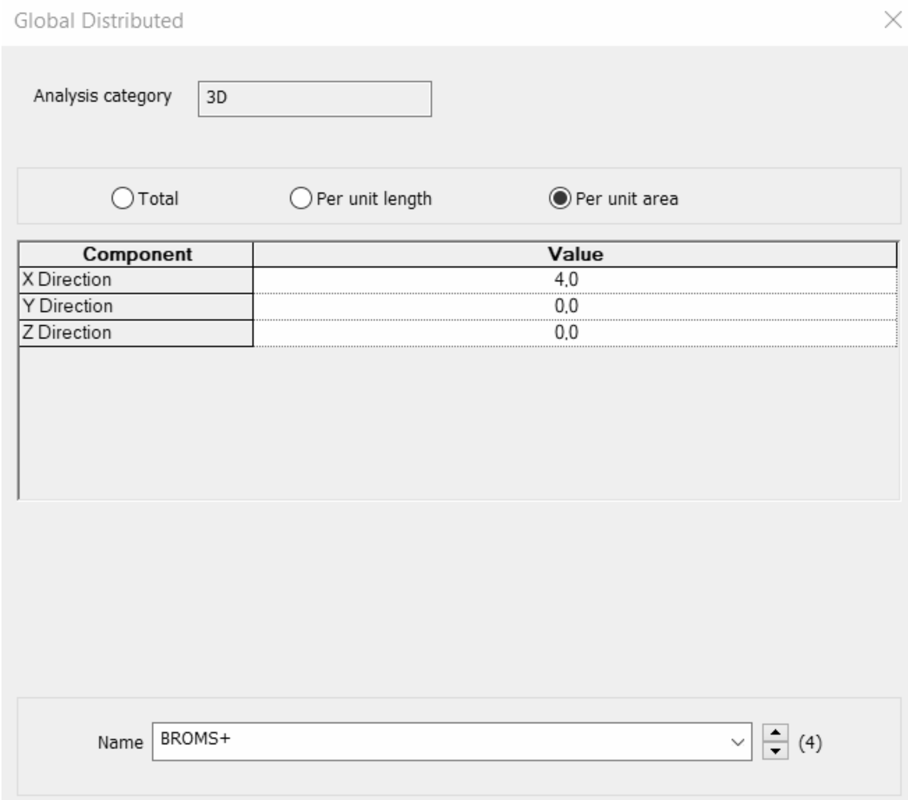
To avoid considering varying load placement laterally, it is assumed that two braking forces occur symmetrically on the bridge deck. This simplification of braking forces is considered safe.

$$q_x = \frac{Q_{broms}}{w_{tot} \cdot L} = \frac{355kN}{8.0m \cdot 12.0m} = 4\frac{kN}{m^2}$$

$$m_y = q_x \cdot (0.35m + t_{bel}) = 4\frac{kN}{m} \cdot (0.35m + 0.11m) = 1.7\frac{kNm}{m^2}$$

.

Load case : BROMS +

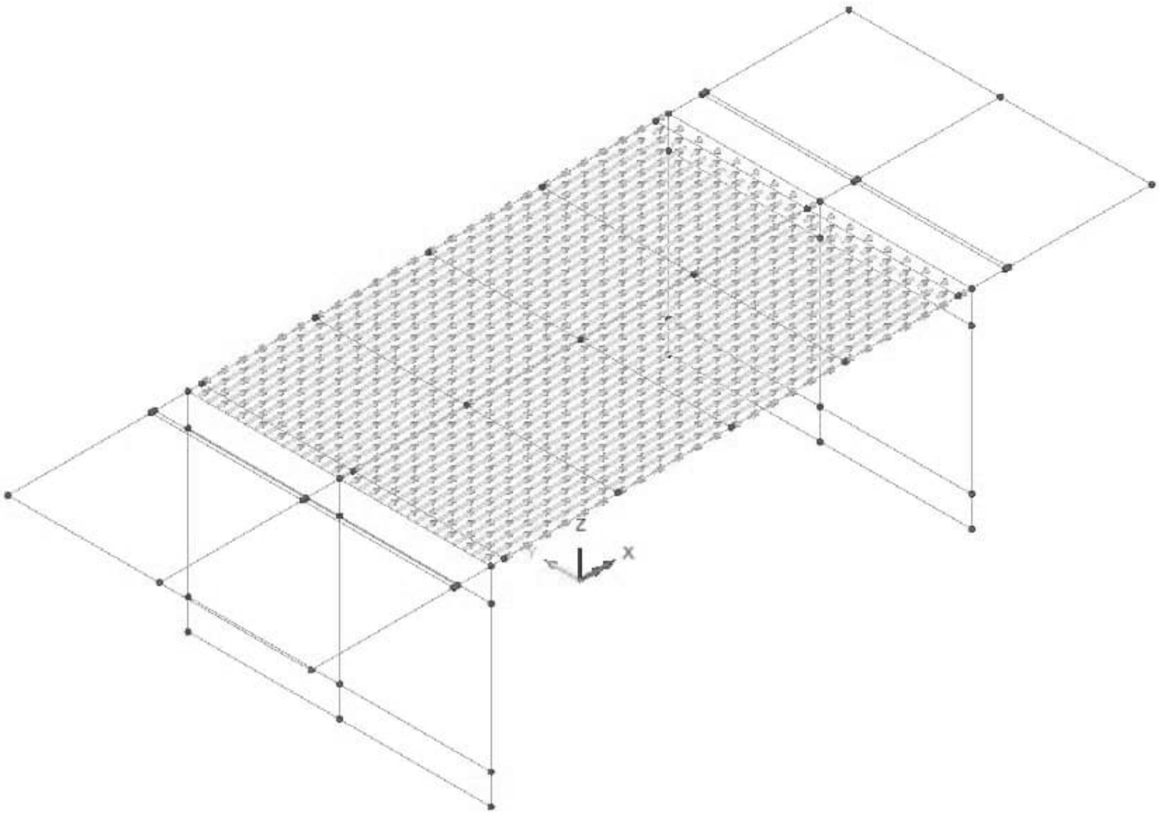

Global Distributed

Analysis category: 3D

( ) Total ( ) Per unit length (•) Per unit area

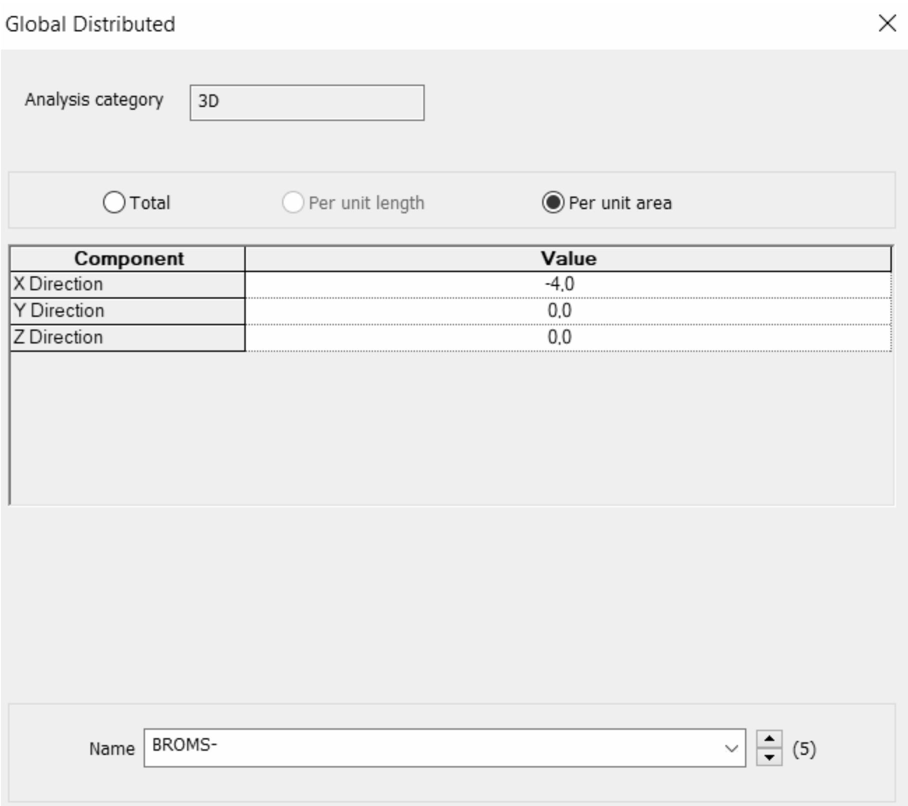
Component	Value
X Direction	4,0
Y Direction	0,0
Z Direction	0,0

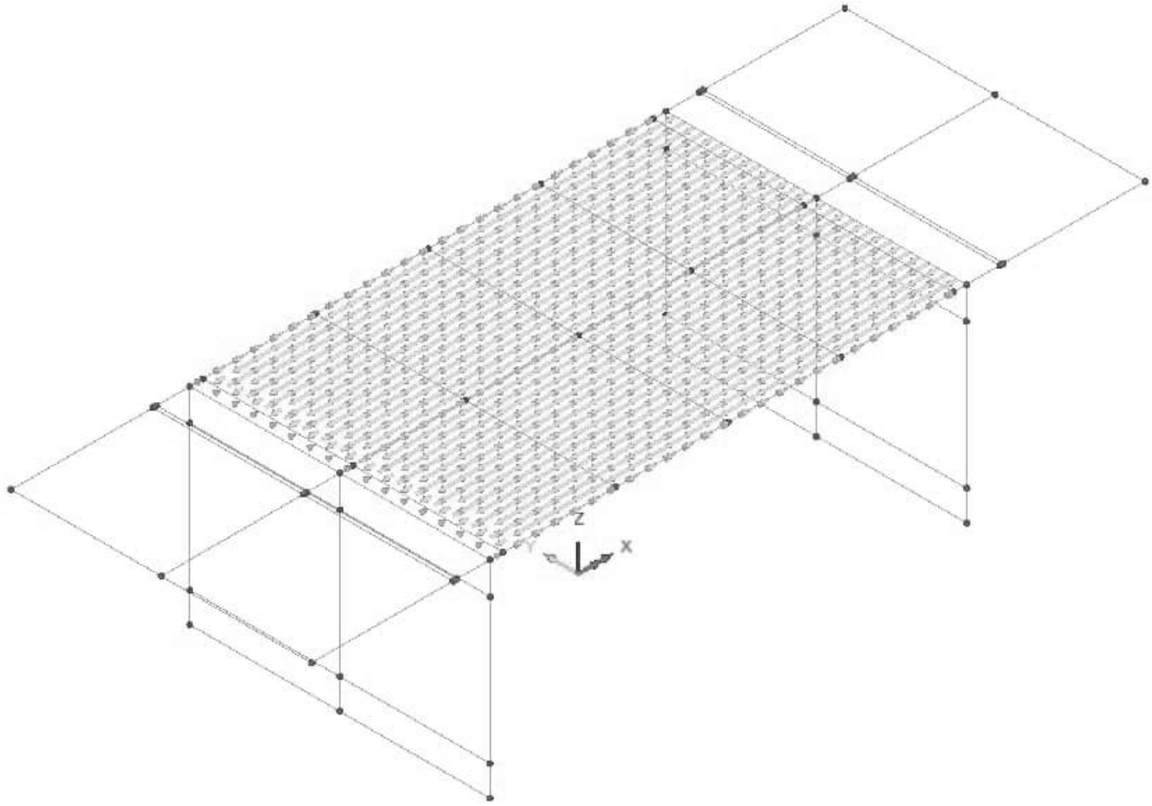
Name: BROMS+ (4)



Overview 3D

Load case : BROMS-





Overview 3D

3.8.2.2 Load combination (BROMS)

Envelope BROMS :

Load case
BROMS +
BROMS -

3.9 LATERAL FORCE

Lateral force is defined by SS-EN 1991-2 §4.4.2.

The load is orthogonal to braking force and acts due to skewed braking or centrifugal forces (R = 600 m). The largest of these are used.

The acting load acts at the level of the pavement and evenly distributed over the load length.

Load model LM 1 :

$Q_{tk.1} = 0.25Q_{lk} = 0.25 \cdot 355kN = 89kN$: skewed braking

$Q_{tk.2} = \frac{40m}{R}\sum \alpha_{Qi} \cdot (2Q_{ik}) = \frac{40m}{600m} \cdot 2 \cdot (270kN + 180kN) = 60kN$: centrifugal force

Load model EG B = 300 kN (see TSFS chapter 11 §2) :
Type i is considered dimensioning.

$Q_{tk.1} = 0.25Q_{lk} = 0.25 \cdot 277kN = 69kN$: skewed braking

$Q_{tk.2} = \frac{40m}{R} \cdot Q_v = \frac{40m}{600m} \cdot (2.64 \cdot 300kN) = 53kN$: centrifugalkraft

-

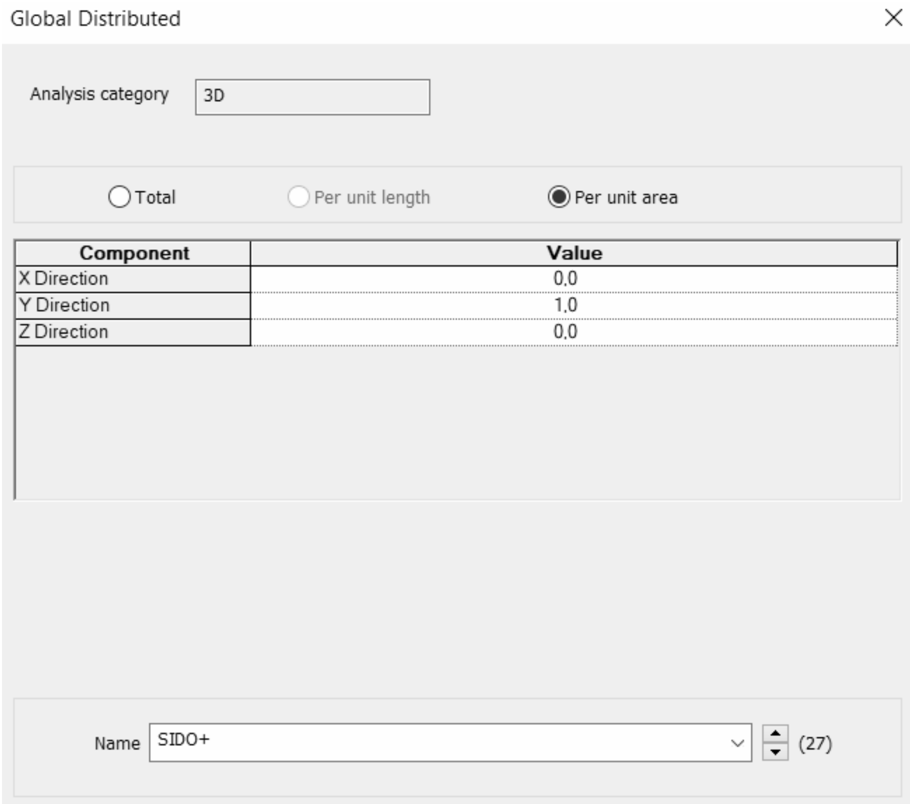
Load definition:
The load is applied as a surface load on the system line of the bridge deck, which is located 0.46 m below the pavement level. In the static model, the impact of m_x is not considered as it is deemed negligible.

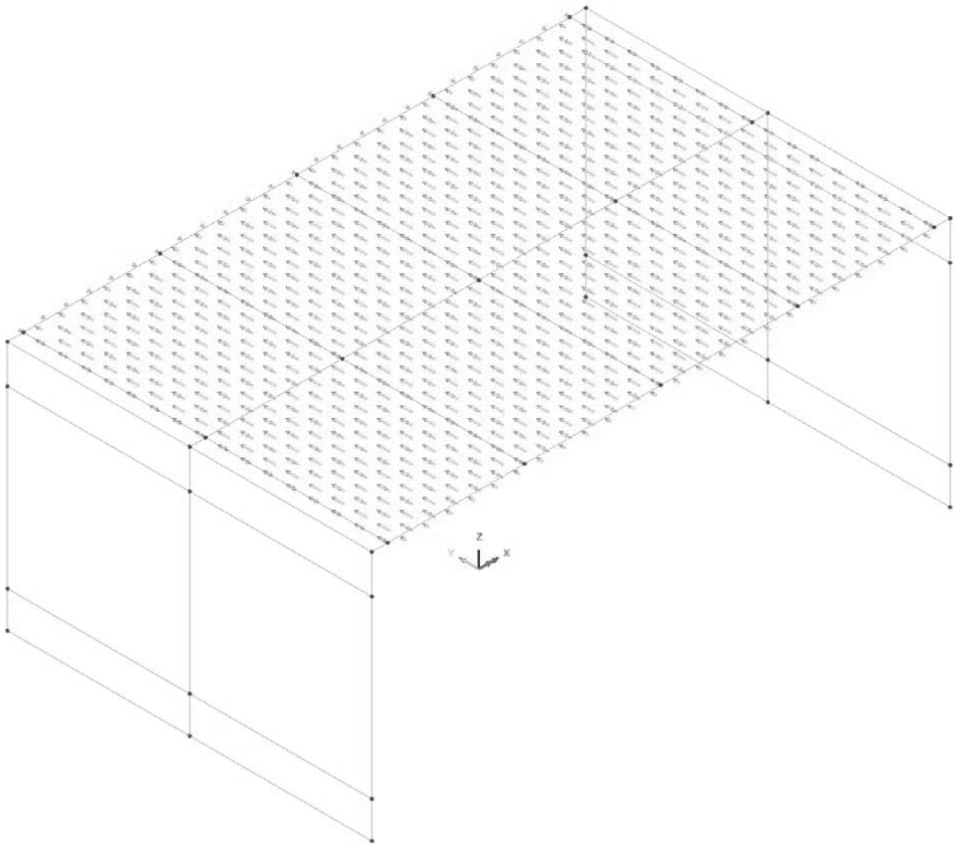
$$q_y = \frac{Q_{sido}}{w_{tot} \cdot L} = \frac{89kN}{8.0m \cdot 12.0m} = 1\frac{kN}{m^2}$$

$$m_x = q_y \cdot (0.35m + t_{bel}) = 1\frac{kN}{m} \cdot (0.35m + 0.11m) = 0.4\frac{kNm}{m^2}$$

.

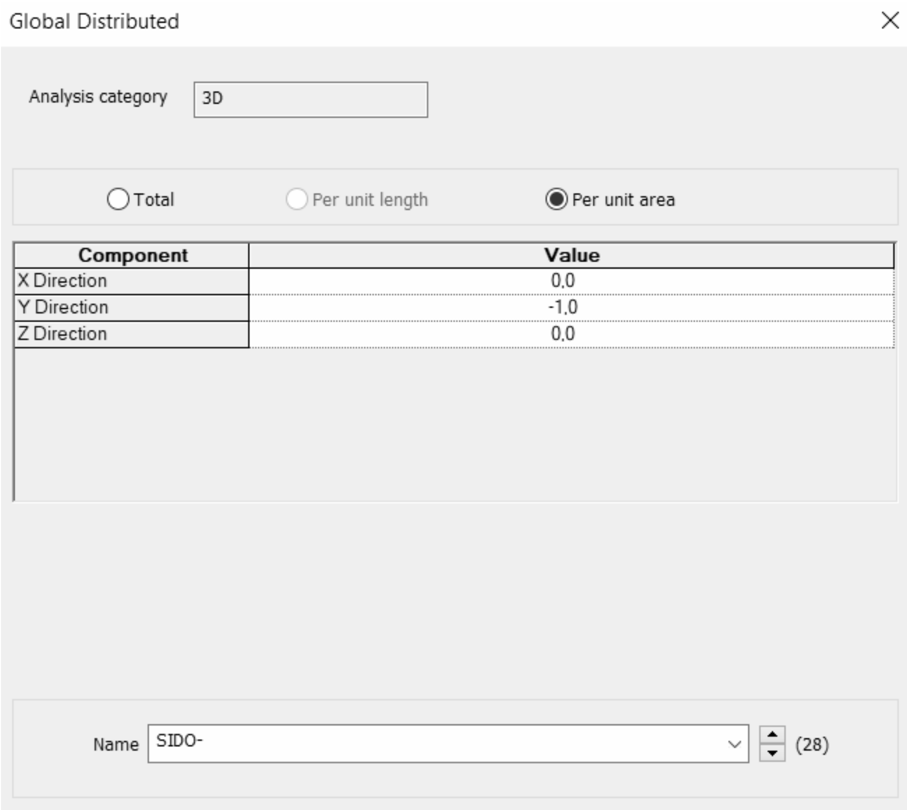
Load case : SIDO +

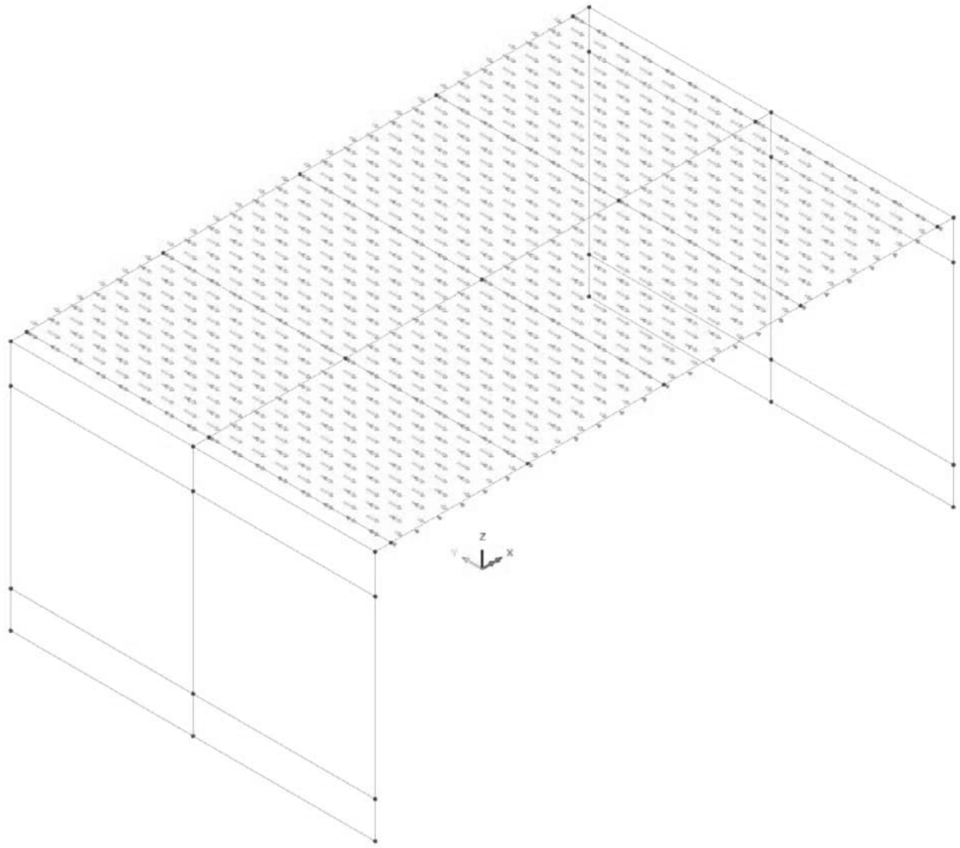




Overview 3D

Load case : SIDO-





Overview 3D

3.9.2.2 Load combination

Envelope SIDO :

Load case
SIDO+
SIDO-

3.10 WIND LOAD

Wind load on bridges is defined by EN 1991-1-4 chapter 8.

The effect of wind is considered as negligible for this bridge type; thus, wind load is not applied to model.

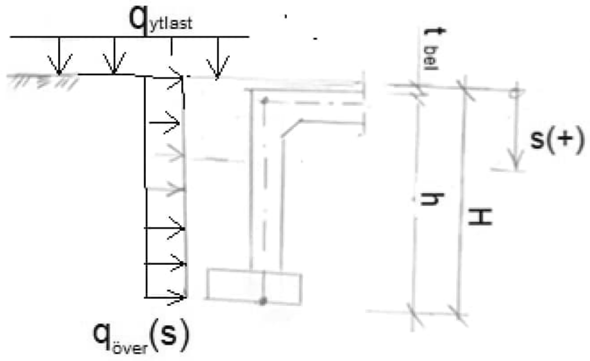
3.11 SURCHARGE

TSFS chapter 11 section §8 describes load seen below.

$q_{ytlast.1} = 20kPa$: road width 6.0 m

$q_{ytlast.2} = 10kPa$: remaining width

$q_{över}(s) = K_0 \cdot q_{ytlast}$



$q_{ytlast} = 0.29 \cdot 20kPa = 6kPa$

Note:
Since the bridge has a link plate with a fictitious bearing located 4 meters from the back edge of the frame beam, no overload theoretically occurs towards the frame beam or wing walls. On the safe side, this favourable effect is not considered.

On the safe side, a surface load of 20 kPa is applied to the entire bridge width of 8 meters.

The favourable impact of the counteracting earth pressure due to movement is not accounted for on the safe side.

3.11.1 Load abutment 1

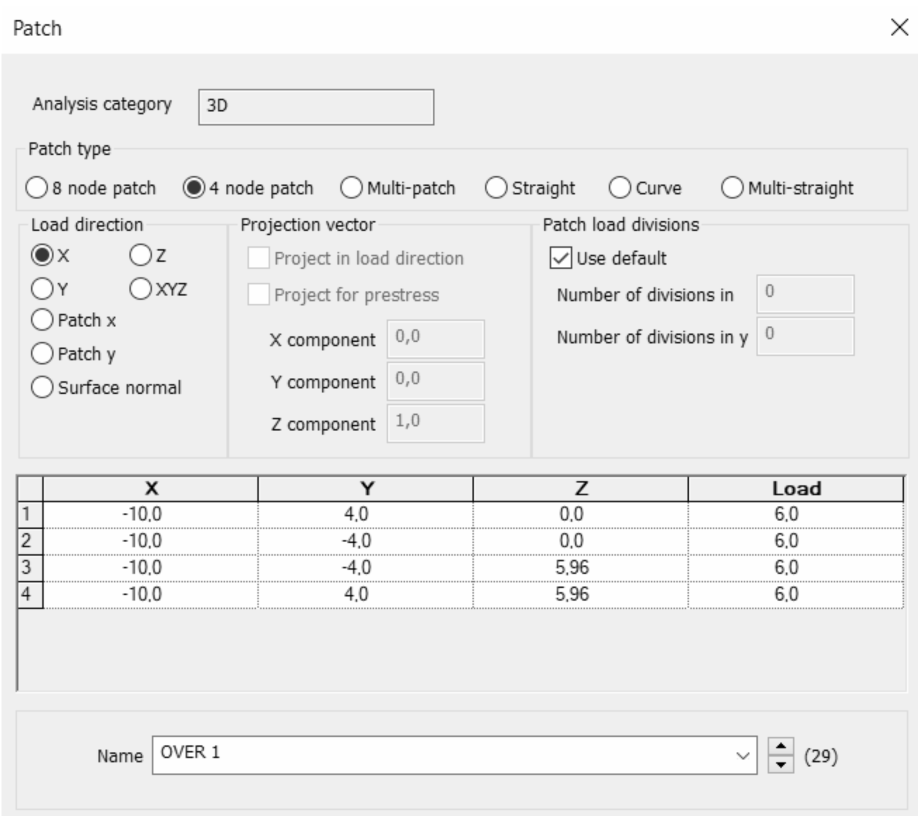
Load case : OVER 1

Structural loading : Discrete 4 node patch load

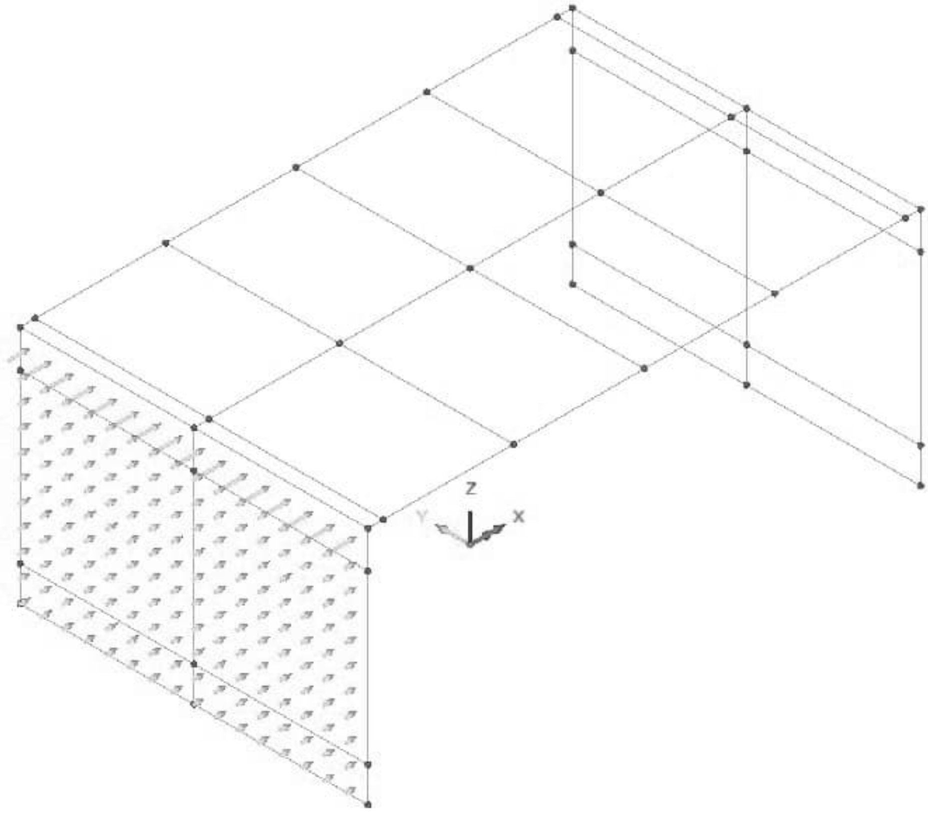
Surface load (q_x) : 6 kPa

Search Area : Abutment 1

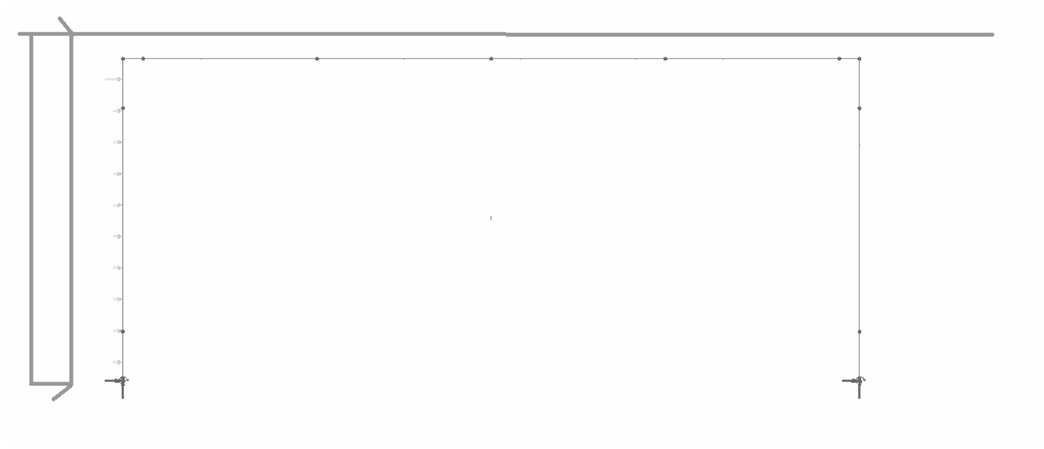
Loads outside search area : Include full load



	X	Y	Z	Load
1	-10,0	4,0	0,0	6,0
2	-10,0	-4,0	0,0	6,0
3	-10,0	-4,0	5,96	6,0
4	-10,0	4,0	5,96	6,0



Overview 3D



Elevation

The vector for load intensity in the figure appears to be higher at the top of the frame leg. This is because the load surface 'Abutment 1' is lower than the load surface."

3.11.2 Load abutment 2

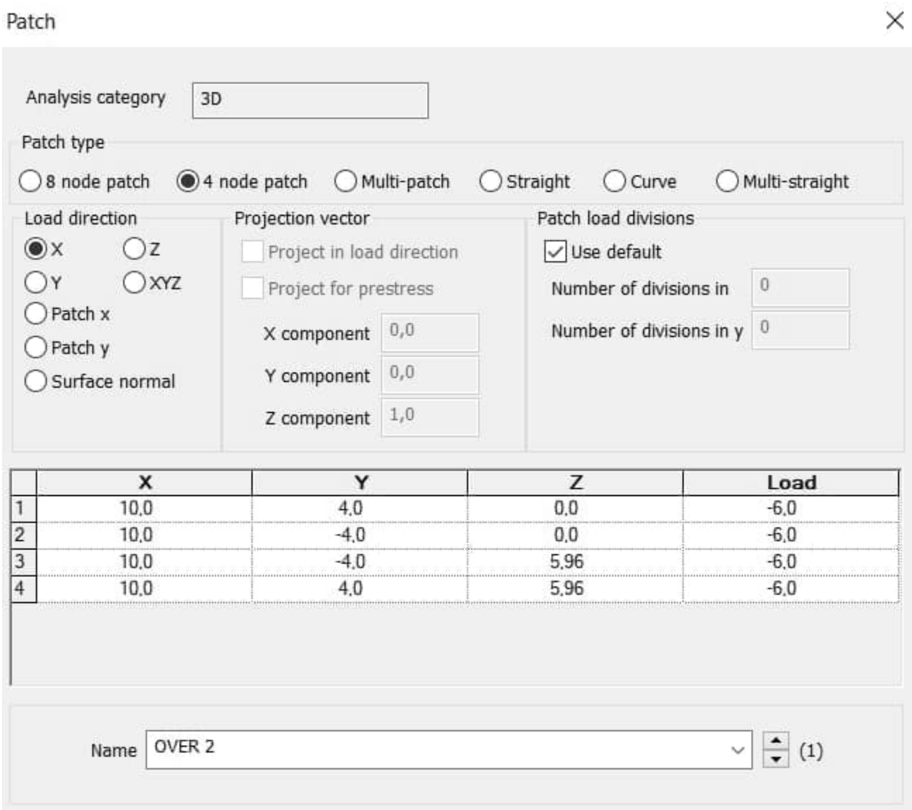
Load case : OVER 2

Structural loading : Discrete 4 node patch load

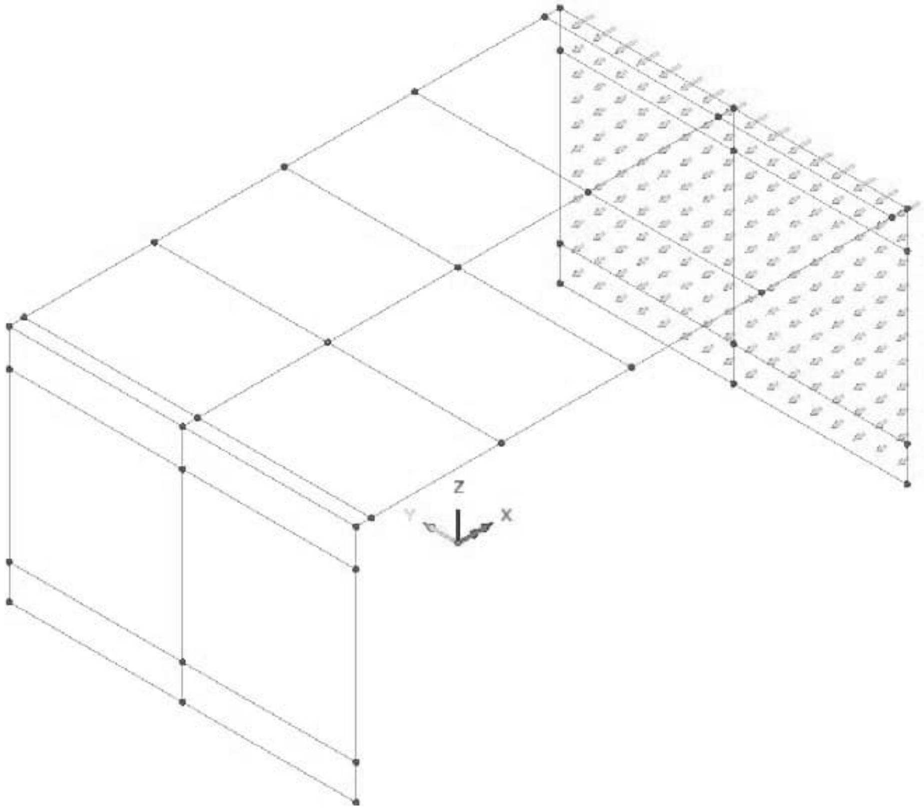
Surface load (q_x) : -6 kPa

Search Area : Ramben 2

Loads outside search area : Include full load



	X	Y	Z	Load
1	10,0	4,0	0,0	-6,0
2	10,0	-4,0	0,0	-6,0
3	10,0	-4,0	5,96	-6,0
4	10,0	4,0	5,96	-6,0



Overview 3D

3.11.3 Load wingwalls

Calculation software K2.002 is used to determine the earth pressure against wing walls according to Culman's method. All wing walls are assumed to have the same length (L = 4.8 m).

Load is distributed along edge of abutments from bottom of superstructure and distance 3.85 m downward. This assumption is on safe side.

Effective height at edge abutment:

$H_{ef} = 3.85m$: see page A3:33

Forces at edge abutment in limit state (ULS):

$N_{ULS} = +55\frac{kNm}{m}$: see page A3:33

$M_{ULS} = 134\frac{kNm}{m}$: see page A3:33

Characteristic earth pressure at edge abutment:

$N_{jord} = 29\frac{kN}{m}$: see page A3:21

$M_{jord} = 77\frac{kNm}{m}$: see page A3:21

Characteristic surcharge at edge abutment:

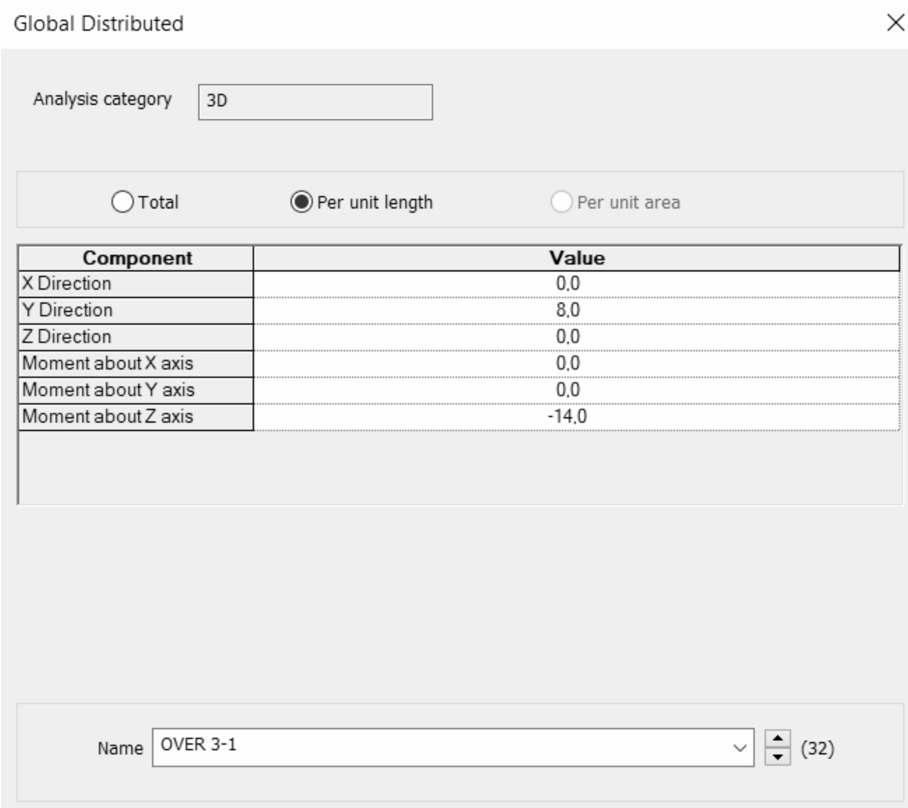
$$N_{\ddot{o}ver} = \left(52\frac{kN}{m} - 29\frac{kN}{m}\cdot 1.45\right)\cdot\frac{1}{1.70} = 8\frac{kN}{m}$$

$$M_{\ddot{o}ver} = \left(127\frac{kNm}{m} - 77\frac{kNm}{m}\cdot 1.45\right)\cdot\frac{1}{1.70} = 14\frac{kN}{m}$$

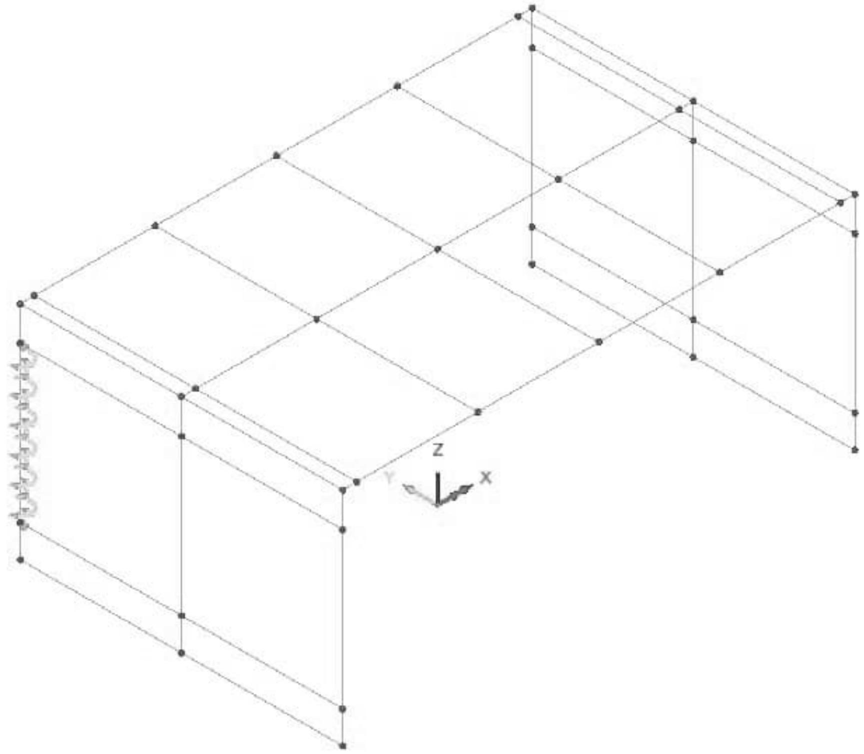
Load case : JORD 3-1
(Northern wing wall abutment 1)

$$p_y = +8\frac{kN}{m}$$

$$m_z = -14\frac{kNm}{m}$$



Component	Value
X Direction	0,0
Y Direction	8,0
Z Direction	0,0
Moment about X axis	0,0
Moment about Y axis	0,0
Moment about Z axis	-14,0

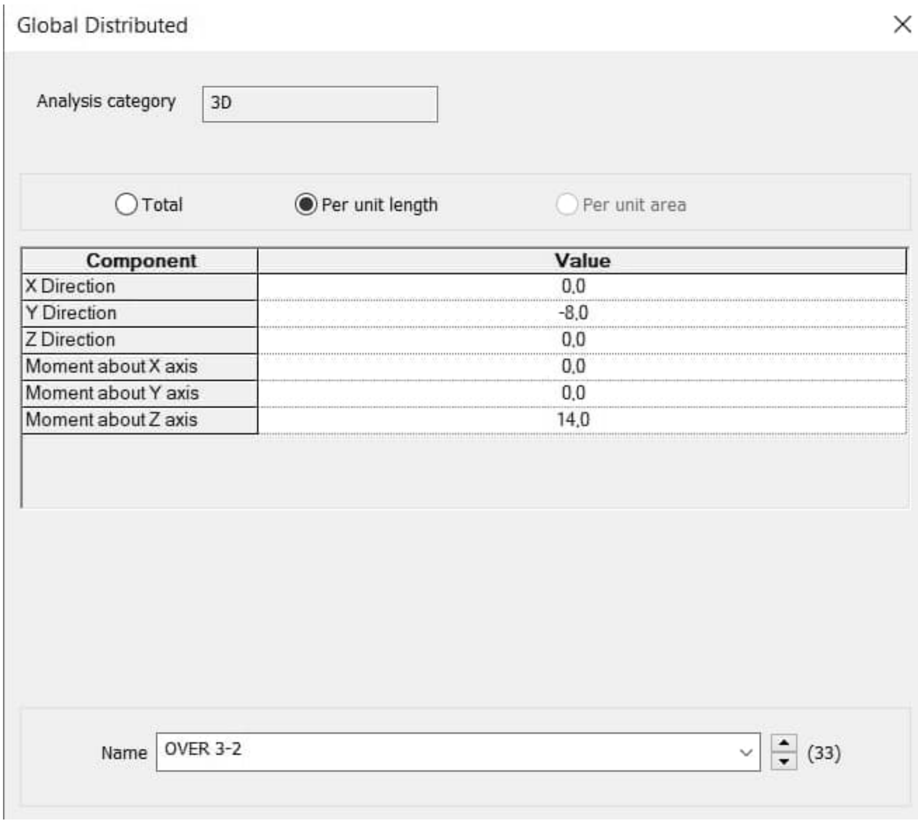


Overview 3D

Load case : OVER 3-2
(Southern wing wall abutment 1)

$$p_y = -8\frac{kN}{m}$$

$$m_z = +14\frac{kNm}{m}$$

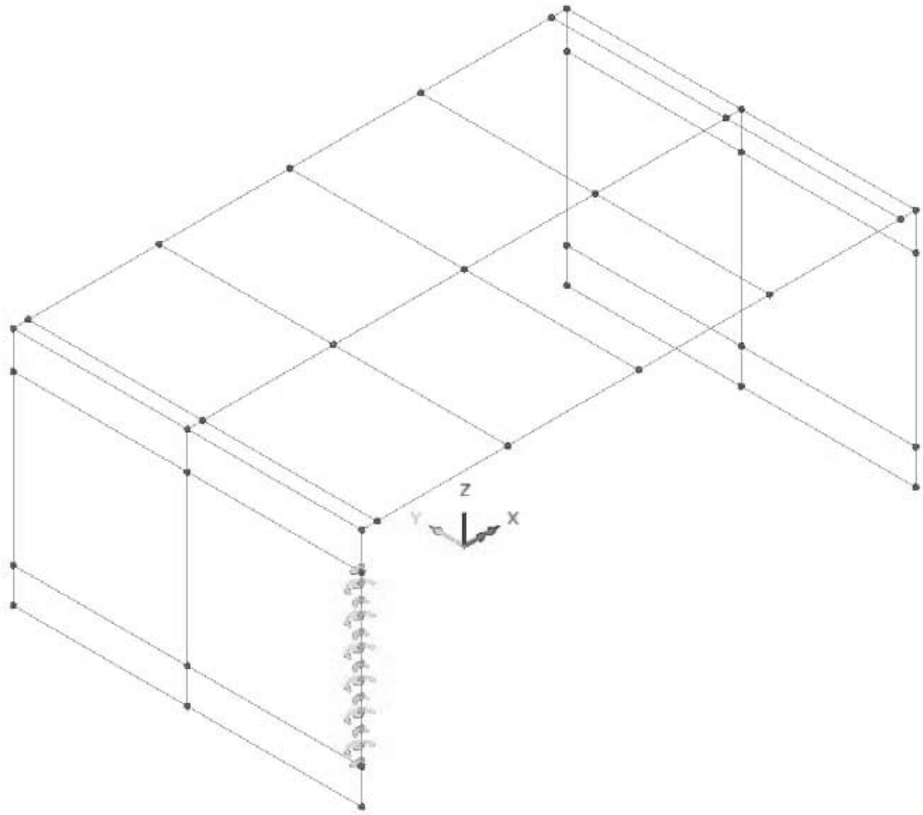

Global Distributed

Analysis category: 3D

Total / Per unit length (selected) / Per unit area

Component	Value
X Direction	0,0
Y Direction	-8,0
Z Direction	0,0
Moment about X axis	0,0
Moment about Y axis	0,0
Moment about Z axis	14,0

Name: OVER 3-2 (33)

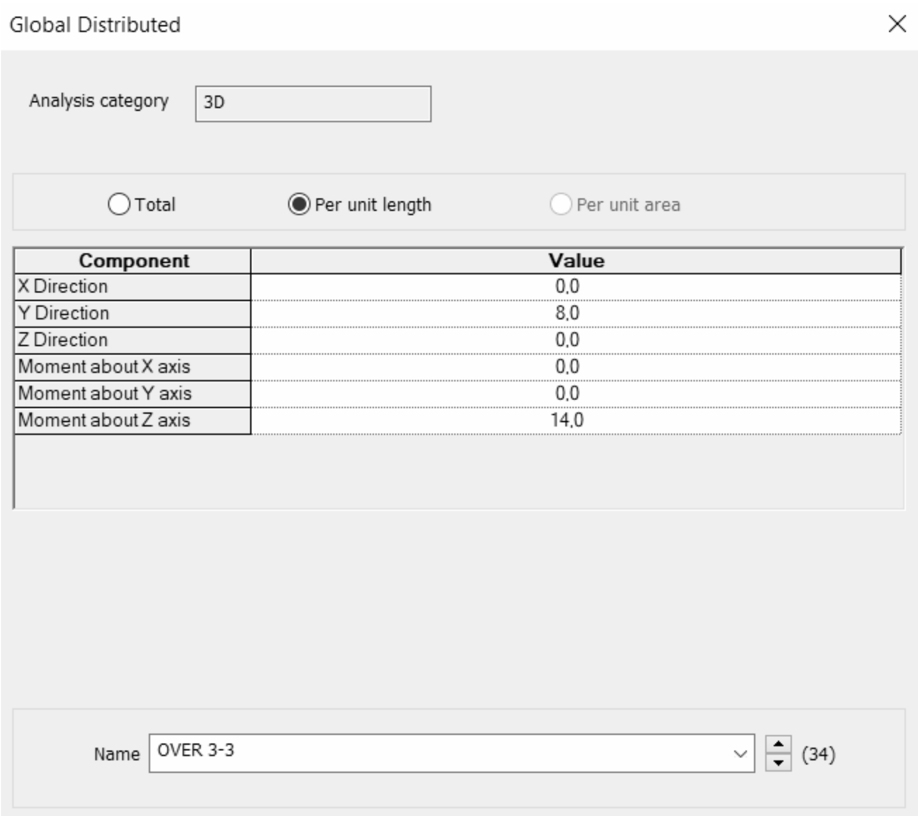


Overview 3D

Load case : OVER 3-3
(Northern wing wall abutment 2)

$$p_y = +8\frac{kN}{m}$$

$$m_z = +14\frac{kNm}{m}$$

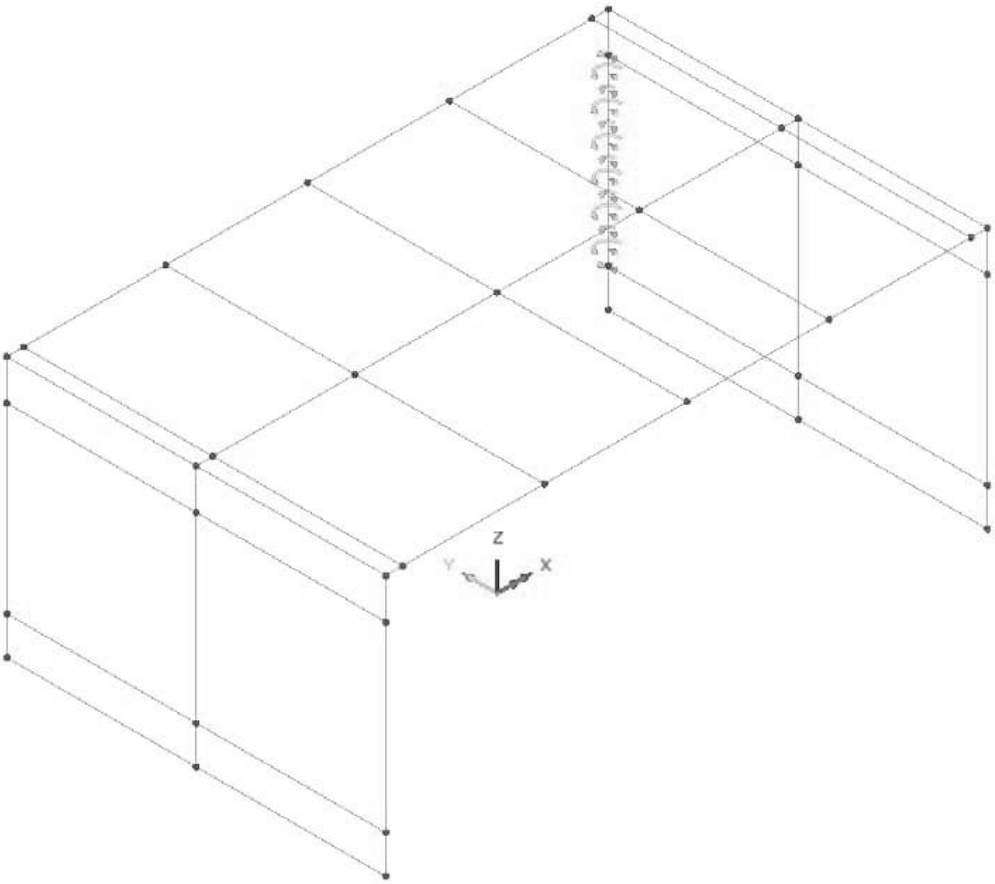

Global Distributed

Analysis category: 3D

Total / Per unit length (selected) / Per unit area

Component	Value
X Direction	0,0
Y Direction	8,0
Z Direction	0,0
Moment about X axis	0,0
Moment about Y axis	0,0
Moment about Z axis	14,0

Name: OVER 3-3 (34)

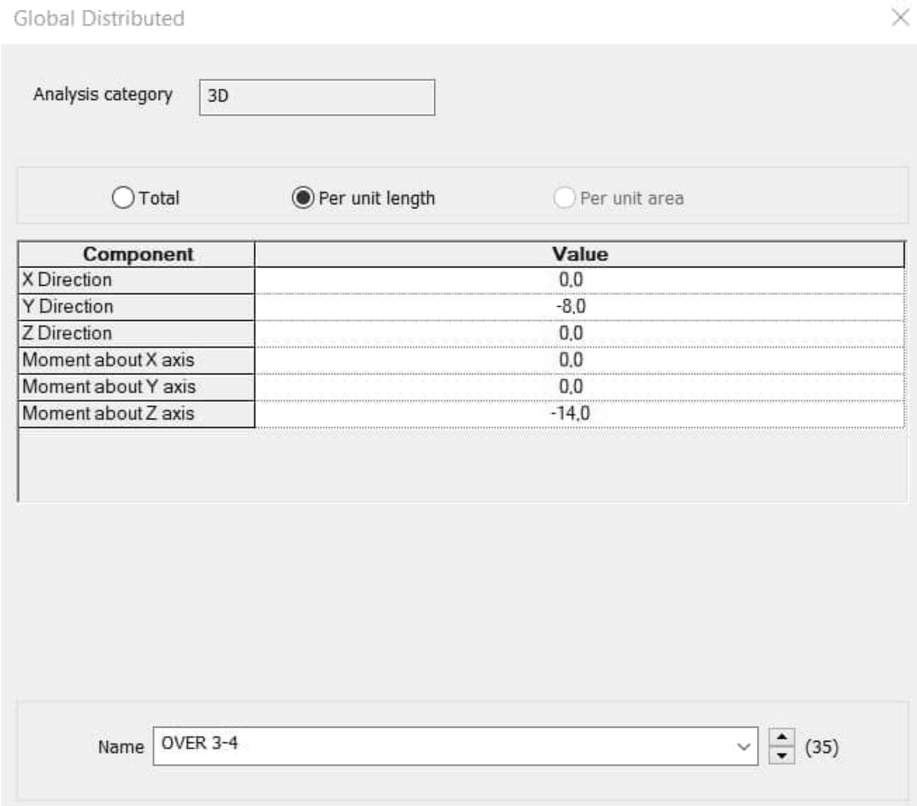


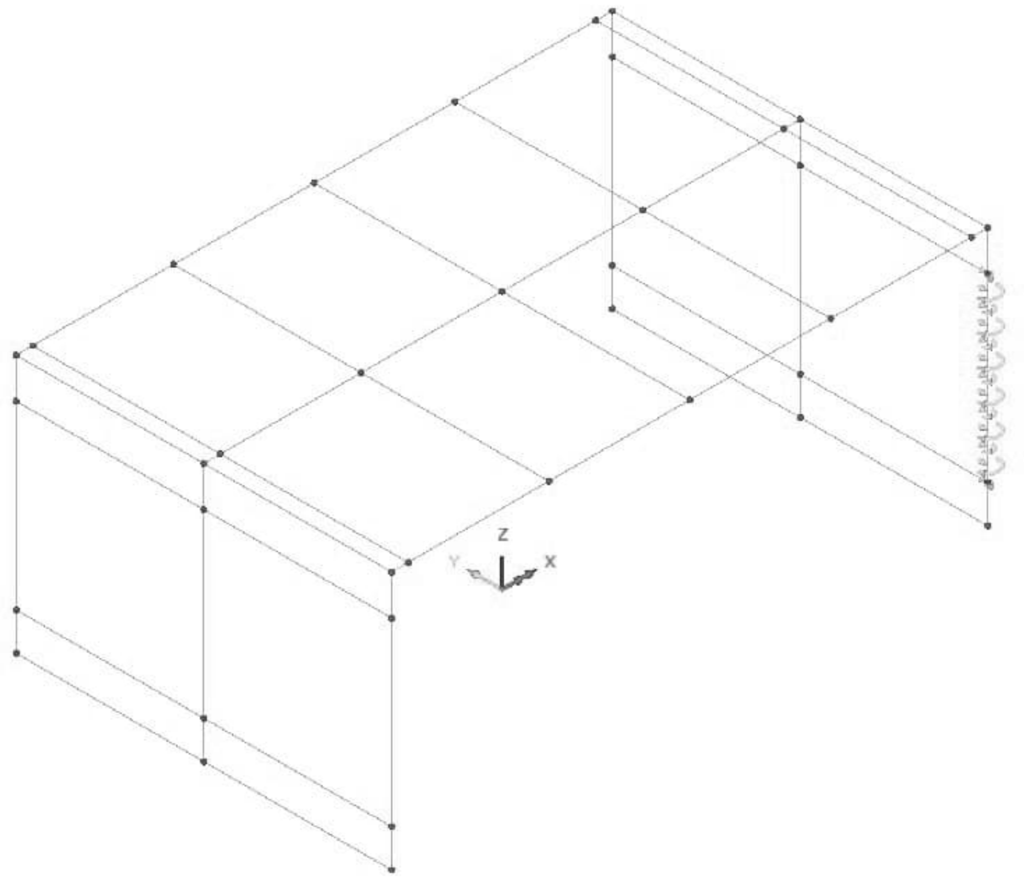
Overview 3D

Load case : OVER 3-4
(Southern wing wall abutment 2)

$$p_y = +8\frac{kN}{m}$$

$$m_z = +14\frac{kNm}{m}$$





Overview 3D

3.11.5 Load combination

Load combination smart OVER *:*

Load case	Permanent factor	Variable factor
OVER 1	0	1
OVER 2	0	1
OVER 3-1	0	1
OVER 3-2	0	1
OVER 3-3	0	1
OVER 3-4	0	1

3.12 TEMPERATURE

Temperaturpåverkan på broar anges i TSFS 2018:57 avsnitt B.3.2.5 och EN 1991-1-5 kapitel 6.

Inverkan av gradvis sprickutveckling medges enligt SS-EN 1992-1-1 §5.4(3) vid bestämning av lasteffekter. Detta hanteras genom tillämpning av reducerad styvhet.

Temperature effect bridges according to TSFS section B.3.2.5 and EN 1991-1-5 chapter 6.

Effect in service state see SS-EN 1992-1-1 §2.3.1.2. If used then apply effect of gradual cracking according to SS-EN 1992-1-1 §5.4(3).

Effect in ultimate state is not required according to SS-EN 1992-1-1 §2.3.1.2. If used apply reduced stiffness according to SS-EN 1992-1-1 §5.4(3).

Casting temperature, T_{mont} = +10° C : EN 1991-1-5A.1(3)

Expansion coefficient, $\alpha = 12 \cdot 10^{-6}$

Concrete slab $\Rightarrow$ typ 3

Location : Boden

$T_{max} = +32℃$: TSFS chapter 8 sketch 8.1
$T_{min} = -42℃$: TSFS chapter 8 sketch 8.2

Duration coefficients :

Coefficients according to SS-EN 1990/A1 table A2.3

$\psi_0 = 0.60$
$\psi_1 = 0.60$
$\psi_2 = 0.50$

3.12.1 Effect of concrete stiffness due to cracking

Impact is considered in the serviceability limit state according to SS-EN 1992-1-1 §2.3.1.2 (1). If this is done, a gradual development of cracking may be applied according to SS-EN 1992-1-1 §5.4(3).

In the studied bridge, all concrete is assumed to be cracked for the load cases of temperature, support settlement, and creep.

This assumption will be verified to ensure that it is accurate. If it is not, the calculation model will be adjusted accordingly.

Verification of whether the concrete is cracked will be done according to SS-EN 1992.2 section 7.2 (102) under the condition $\sigma_{ct}^{SLS-K} > f_{ctm}$.

A review of the bending stiffness according to SS-EN 1992-1-1 shows that completely uncracked reinforced concrete has a stiffness that is 16% of the cracked concrete according to SS-EN 1992-1-1 section 7.4.3 when applied to a slab with a thickness of 700 mm, see page A3:107.

Selected bending stiffness in the calculation model:

In the chosen calculation, standard Swedish calculation practice is applied with stiffness of 60% for cracked compared to uncracked concrete.

Uncracked section (stage I): $EI_{osprucket} = EI_I = E_{cm} \cdot I_c$

Cracked section (stage II): $EI_{sprucket} = EI_{II} = 0.6E_{cm} \cdot I_c$

Evaluation of bending stiffnesses according to SS-EN 1992-1-1 :
The control is performed for a section corresponding to b x h = 1000 mm x 1200 mm and reinforcement ϕ16s250 (∴ 804 mm^2/m).

The evaluation of stiffness for stage II (cracked section) is done using the calculation program caeEc205.

$$I_I = \frac{b \cdot h^3}{12} = \frac{1.0m \cdot (0.70m)^3}{12} = 286 \cdot 10^{-4} m^4$$: uncracked crossection

$$I_{II} = 42 \cdot 10^{-3} m^4$$: cracked crossection, see page A3: 108

$\rightarrow \eta = \frac{I_{II}}{I_I} = 16\% < 60\,\%$ thus, on safe side !

```
Projekt: Bro 2
Position: Kontroll böjstyvhet stadium II
caeEc205────────────────────────────────────────────────────────Version 2.2.2
```

Stadium I och II

```
Materialparametrar bruksstadie──────────────────────────────────────────────

Betong ..........  fcm        fctk0,05     Ecm ────────────────────────────
                   MPa          MPa        GPa
C30/37               2,9        2,00        33,0

Materialindata──────────────────────────────────────────────────────────────

Spricksäkerhetsfaktor, Zeta..........................   1,00
Effektivt kryptal, FIeff ............................   2,00
Betongens slutkrympning, ecs ........................   0,00 ‰
Elasticitetsmodul armering, Es....................... 200,00 GPa
Töjning i förespänd armering, ep ...................    0,00 ‰

Krafter + armering──────────────────────────────────────────────────────────

Moment, MEd (Positivt dragen underkant)............   200,0 kNm
Normalkraft, NEd (Positiv draget tvärsnitt)........     0,0 kN
Normalkraftens excentricitet, e (Pos från ÖK uppåt)    0 mm
Effektiv höjd underkantsarmering, d................  620 mm
Armeringsarea underkantsarmering, As...............  804 mm2

Rektangulärt tvärsnitt, mått i mm───────────────────────────────────────────

    h      bw     bök    tök   tsök    buk    tuk   tsuk
    700   1000      0      0      0      0      0      0

Beräkningsresultat stadium II───────────────────────────────────────────────

Neutrallagrets läge .................................  0,120 m
Ideelt tröghetsmoment ............................... 42,304*10^-4 m4
Betongspänning tryckt kant ..........................  -5,71 MPa
Stålspänning tryckt kant ............................  -60,9 MPa
Stålspänning dragen kant ............................  429,1 MPa
```

3.12.2 Even temperature over entire bridge (JTEMP)

Even temperature over entire bridge according to EN 1991-1-5 section 6.1.3.3. This temperature change is seasonal.

Uniform temperature change across the entire bridge is given by EN 1991-1-5, section 6.1.3.3. This temperature change is seasonal and primarily causes translation from the bridge's movement center towards the respective supports. This movement is considered to give rise to increased earth pressure due to the movement.

Function according to SS EN 1991-1-5 sketch 6.1 (bridge type 3):

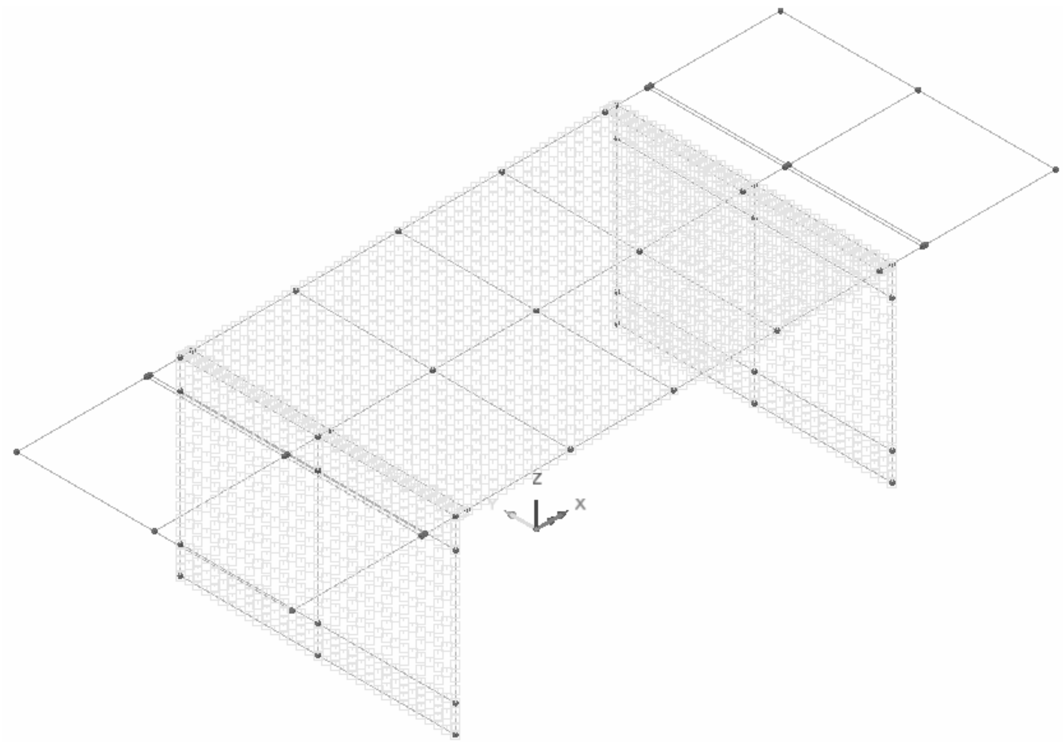
$$T_e(T) = \text{linterp}\left[(-50 \;\; 0 \;\; 30 \;\; 50)^T \cdot {}^\circ C, (-42 \;\; 7 \;\; 32 \;\; 52)^T \cdot {}^\circ C, T\right]$$

$$T_{e.max} = T_e(T_{max}) = 34\ {}^\circ C$$

$$T_{e.min} = T_e(T_{min}) = -34\ {}^\circ C$$

$$T^+ = T_{e.max} - T_0 = +34{}^\circ C - 10{}^\circ C = +24{}^\circ C$$

$$T^- = T_{e.min} - T_0 = -34{}^\circ C - 10{}^\circ C = -44{}^\circ C$$



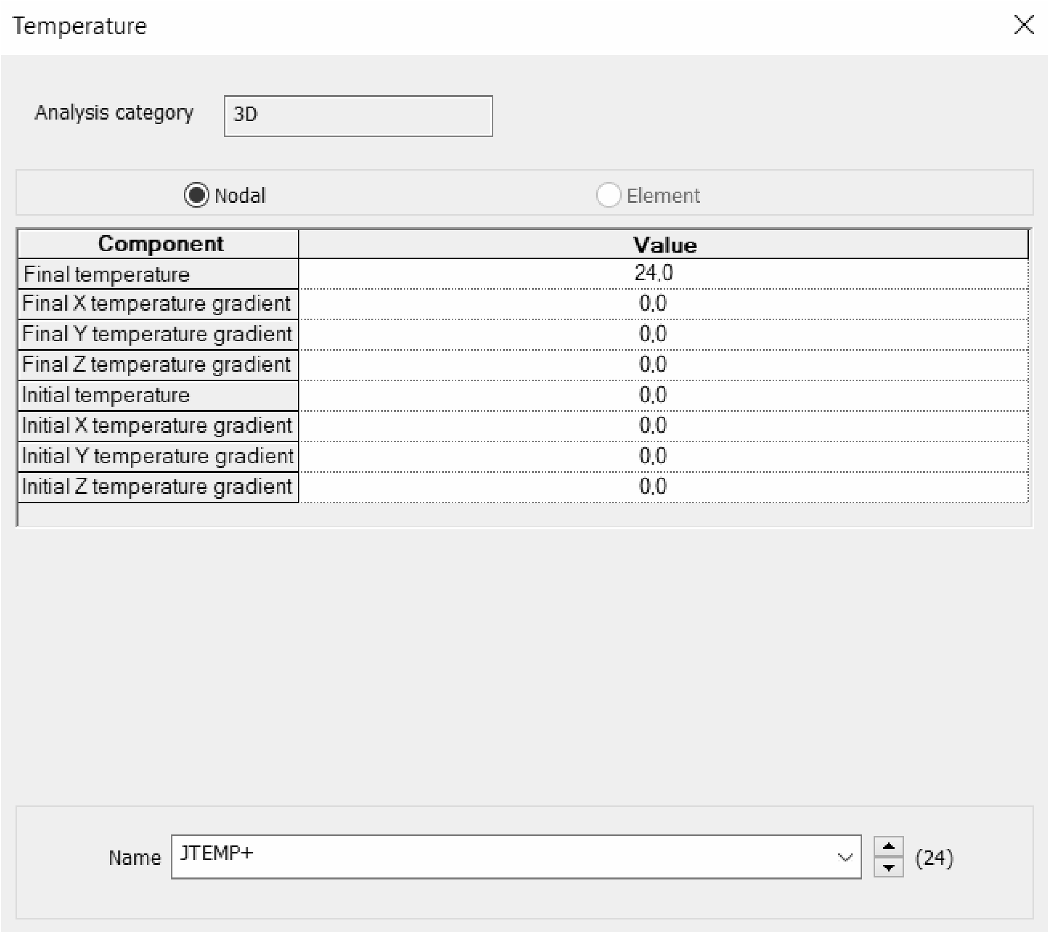
Load : JTEMP+

Structural loading : Temperature

Final temperature : +24C

Initial temperature : ±0 C

Load case : JTEMP+



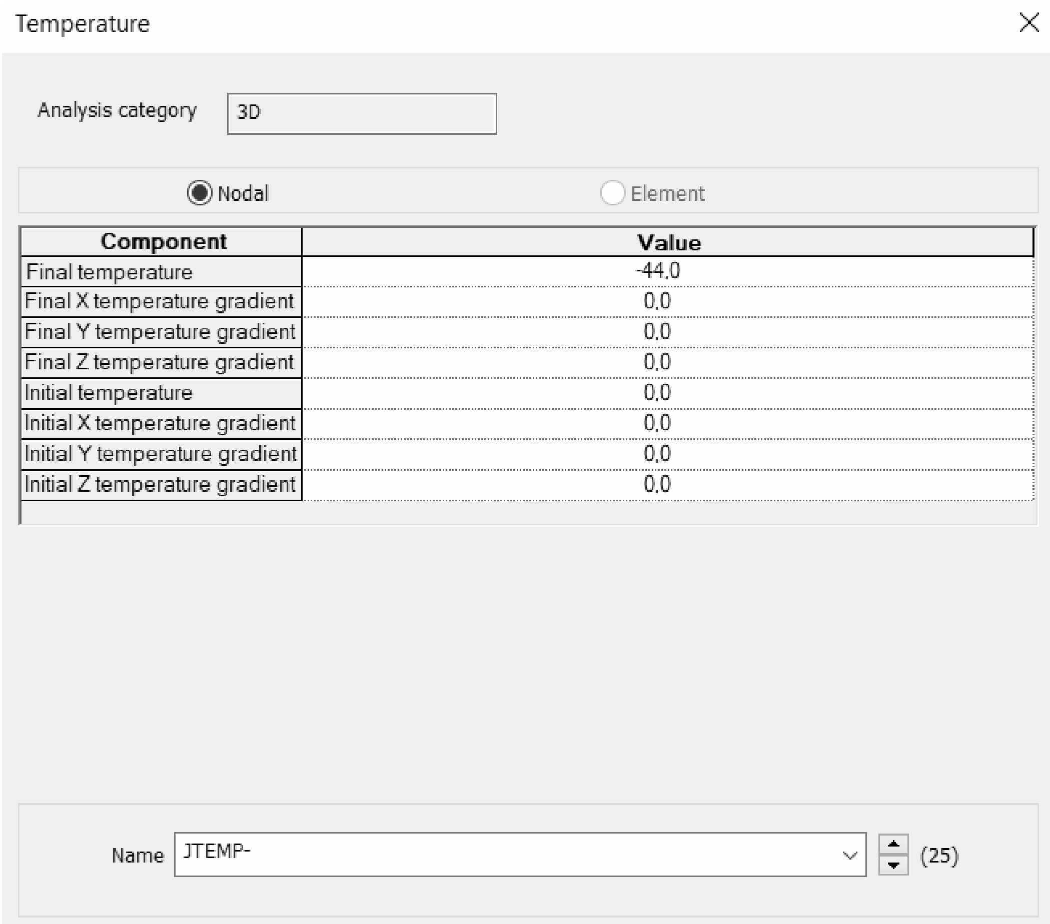
Load : JTEMP-

Structural loading : Temperature

Final temperature : -44C

Initial temperature : ±0 C

Load case : JTEMP-


Temperature

Analysis category: 3D

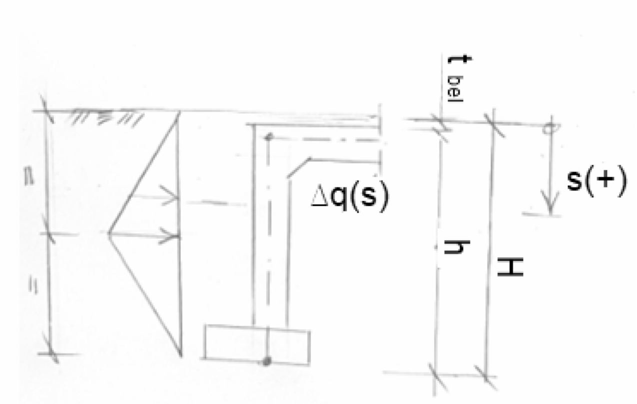
Nodal / Element

Component	Value
Final temperature	-44,0
Final X temperature gradient	0,0
Final Y temperature gradient	0,0
Final Z temperature gradient	0,0
Initial temperature	0,0
Initial X temperature gradient	0,0
Initial Y temperature gradient	0,0
Initial Z temperature gradient	0,0

Name: JTEMP- (25)

3.12.3 Increased earth pressure due to movement (DELTA P)

The increased earth pressure caused by the movement of the approach slab towards the backfill is calculated according to TRVINFRA-00227, section 7.2.1..2.1. This load corresponds to what is stated in SS-EN 1997-1, section C.3. This section is used to ensure that movements for Δq(s) do not exceed the limit for passive earth pressure. During this check, "firm ground" and "wall movement type" b are applied.



$$\rightarrow \delta = (T^{+} - T^{-}) \cdot \alpha \cdot \frac{L_{bro}}{2} = (24℃ + 44℃) \cdot 1.2 \cdot 10^{-5} \cdot \frac{14200mm}{2} = 6mm$$

$$\Delta q(s) = c \cdot \gamma \cdot s \cdot \frac{\delta}{H} = 600 \cdot 20 \frac{kN}{m^3} \cdot s \cdot \frac{\delta}{H}$$

$$H = 0.11m + 5.85m = 5.96m$$

$$\Delta q(s) = c \cdot \gamma \cdot s \cdot \frac{\delta}{h} = 600 \cdot 22 \frac{kN}{m^3} \cdot s \cdot \frac{13mm}{6975mm}$$

$$\Delta q_{max} = 600 \cdot 20 \frac{kN}{m^3} \cdot \frac{5960mm}{2} \cdot \frac{6mm}{5960mm} = 36kPa$$

Note:
No reduction is made considering creep or cracking, as this is not an internal constraint load. This applies to both ultimate limit state (ULS) and serviceability limit state (SLS).

3.12.3.1 Load abutment 1

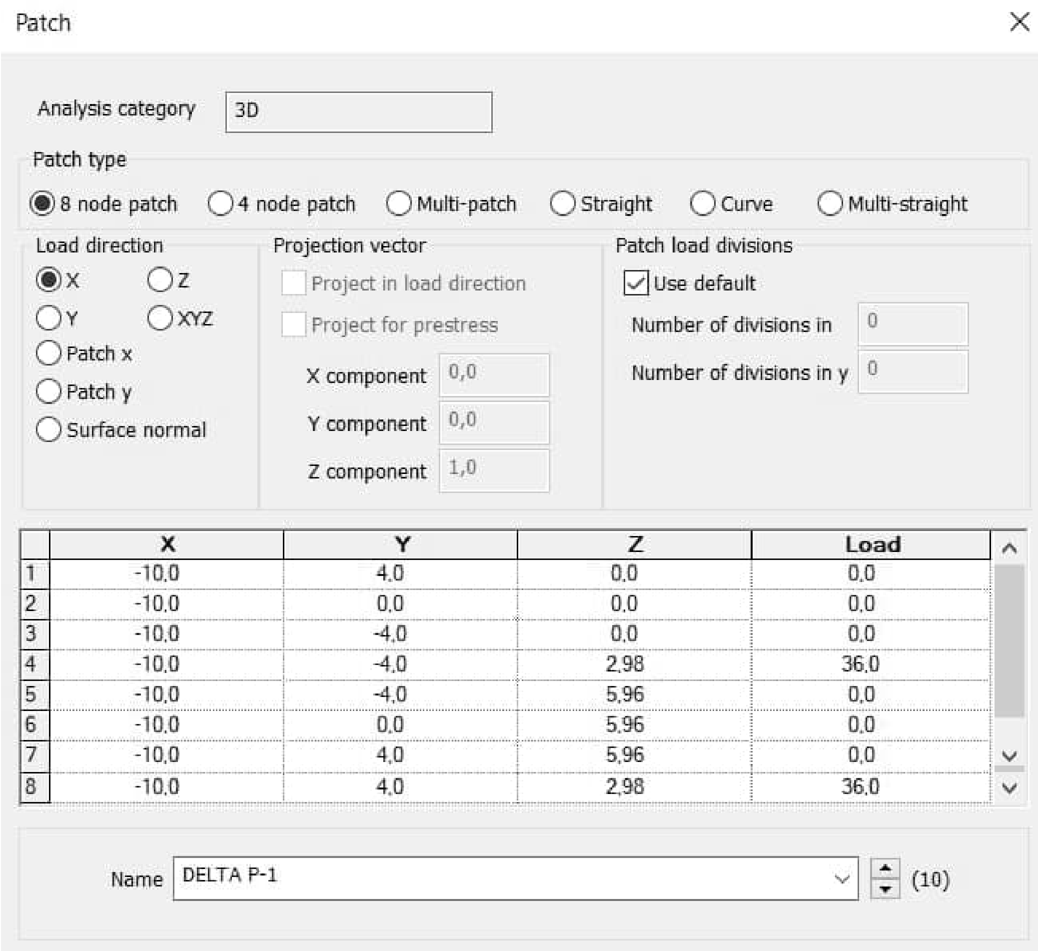
Discrete patch load : DELTA P-1

Structural loading : Discrete 8 node patch

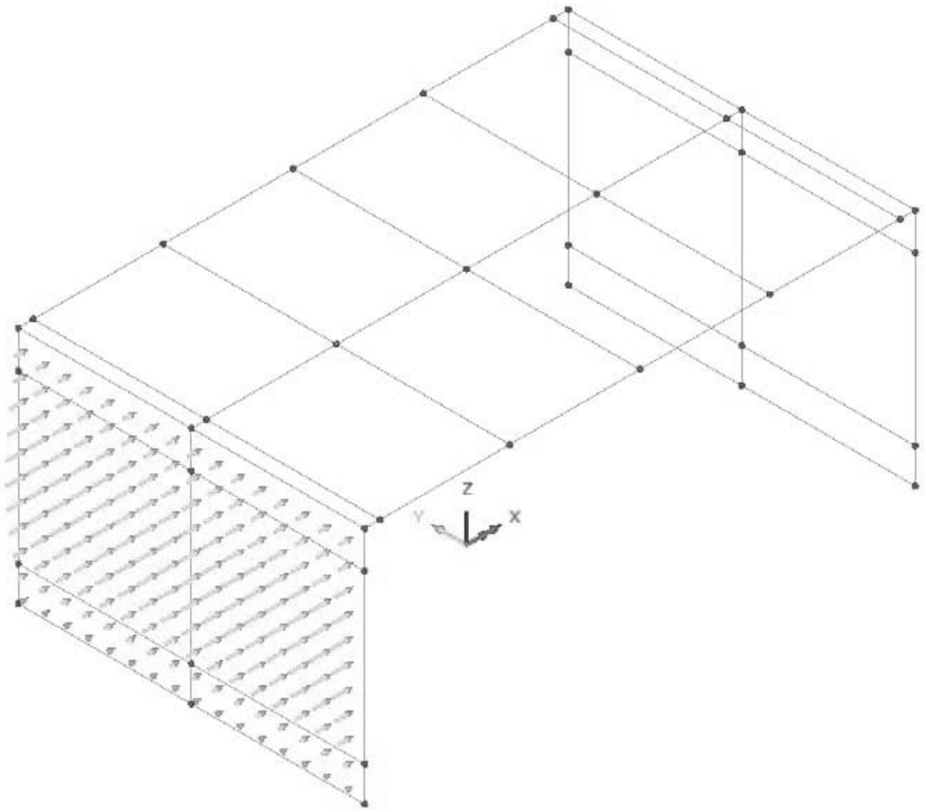
Surface load (q_x) : 0 kPa $\rightarrow$ +36 kPa

Search Area : Abutment 1

Loads outside search area : Include full load



	X	Y	Z	Load
1	-10,0	4,0	0,0	0,0
2	-10,0	0,0	0,0	0,0
3	-10,0	-4,0	0,0	0,0
4	-10,0	-4,0	2,98	36,0
5	-10,0	-4,0	5,96	0,0
6	-10,0	0,0	5,96	0,0
7	-10,0	4,0	5,96	0,0
8	-10,0	4,0	2,98	36,0



Overview 3D

3.12.3.2 Load abutment 2

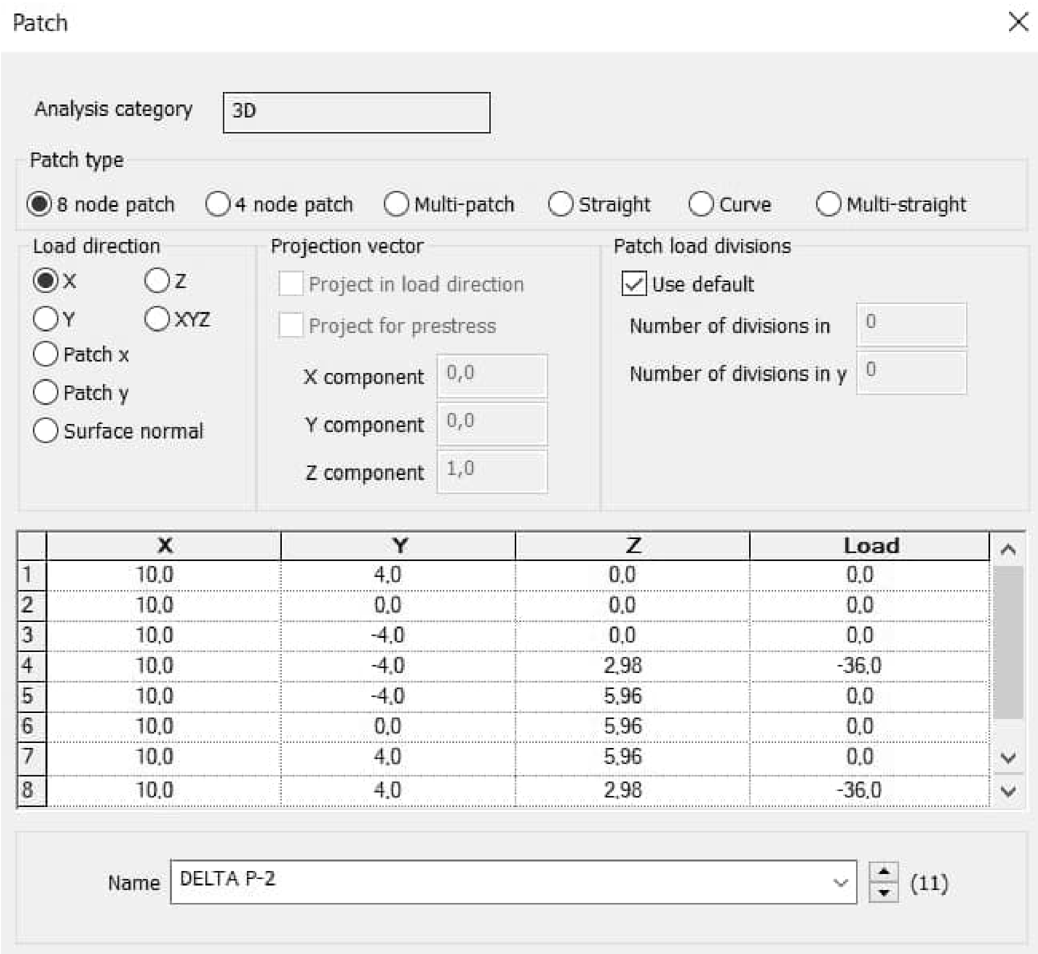
Discrete patch load : DELTA P-2

Structural loading : Discrete 8 node patch

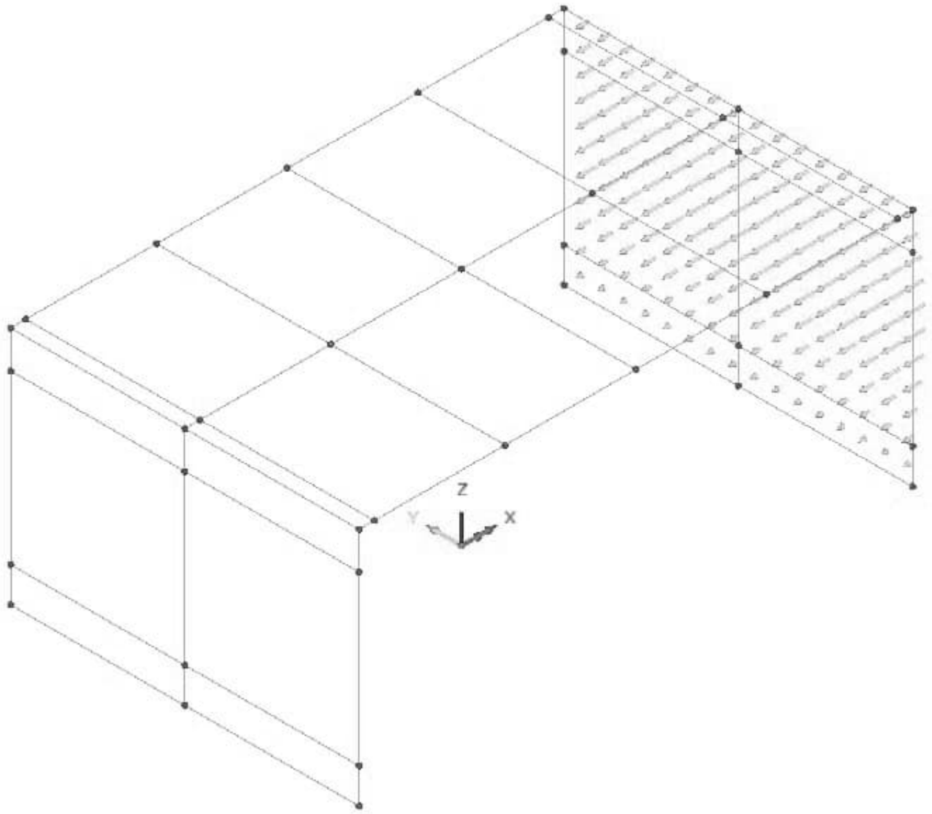
Surface load (q_x) : 0 kPa → -36 kPa

Search Area : Abutement 2

Loads outside search area : Include full load



	X	Y	Z	Load
1	10,0	4,0	0,0	0,0
2	10,0	0,0	0,0	0,0
3	10,0	-4,0	0,0	0,0
4	10,0	-4,0	2,98	-36,0
5	10,0	-4,0	5,96	0,0
6	10,0	0,0	5,96	0,0
7	10,0	4,0	5,96	0,0
8	10,0	4,0	2,98	-36,0



Overview 3D

3.12.3.3 Load combination

Load combination basic DELTA P :

Load case	Factor
DELTA P-1	1
DELTA P-2	1

3.12.4 Ojämn temperatur över tvärsnittet (OJTEMP 1)

Determined according to EN 1991-1-5 § 6.1.4.1. When assessing the impact, a coating with a thickness of 110 mm is applied on the safe side.

$k_{1.sur}(t = 100mm) = 0.7$

$k_{1.sur}(t = 150mm) = 0.5$ $\rightarrow$ $k_{1.sur}(t = 110mm) = 0.66$

$k_{2.sur}(t = 110mm) = 1.0$

$\Delta T_{max} = +15℃ \cdot k_{1.sur} = +10℃$: : upper surface warmer

$\Delta T_{min} = -8℃ \cdot k_{2.sur} = -8℃$: : lower surface warmer

The occurring temperature change ΔT refers to the linear difference between the temperature at the top and bottom of the bridge deck slab.

Uneven temperature is indicated as a temperature gradient $\frac{\delta T}{\delta Z}$ when defined in FEM-program.

On safe side δZ = 0.7 m is assumed.

$\frac{\delta T^{max}}{\delta Z} = \frac{+10℃}{0.7m} = +14\frac{℃}{m}$: maximal temperature gradient

$\frac{\delta T^{min}}{\delta Z} = \frac{-8℃}{0.70m} = -11\frac{℃}{m}$: maximal temperature gradient

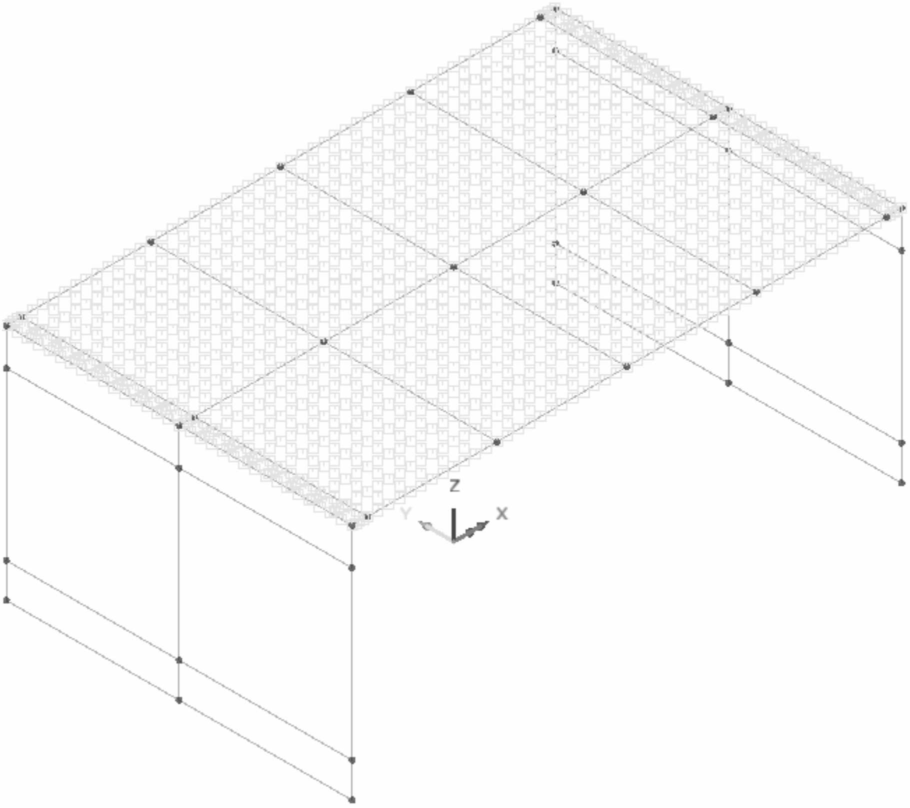
Load case : OJTEMP 1+

Structural loading : Temperature

Definition : Element

Final Z temperature gradient : +14 °C/m

Initial Z temperature gradient : 0°C/m



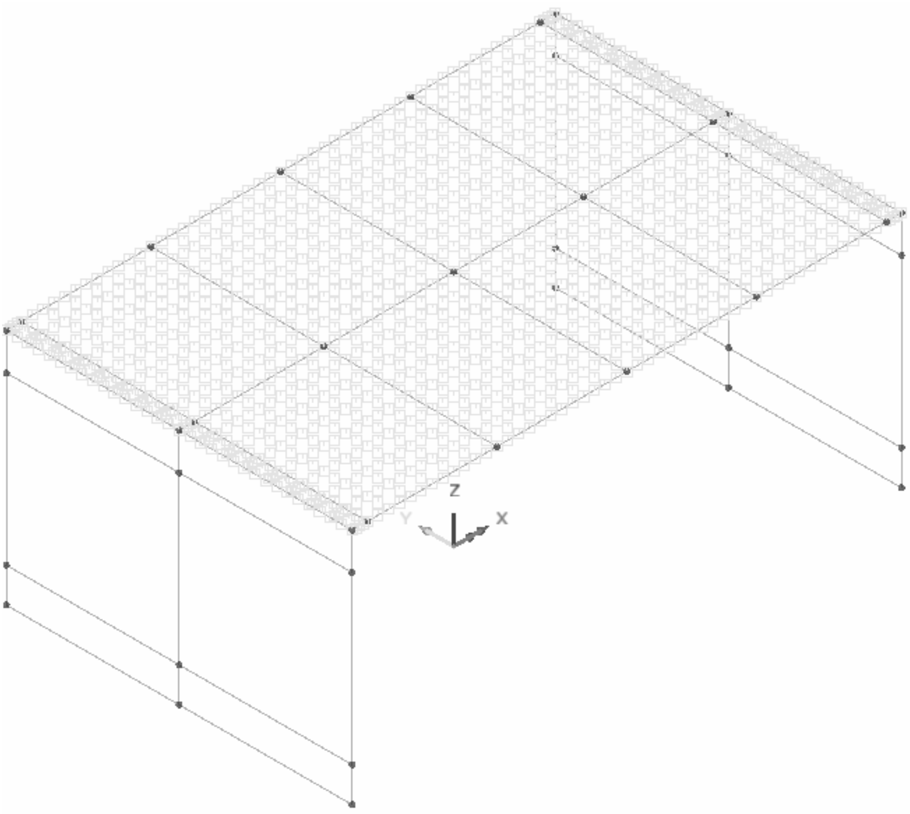
Load case : OJTEMP 1-

Structural loading : Temperature

Definition : Element

Final Z temperature gradient : -11 °C/m

Initial Z temperature gradient : 0°C/m



Envelope OJTEMP 1 :

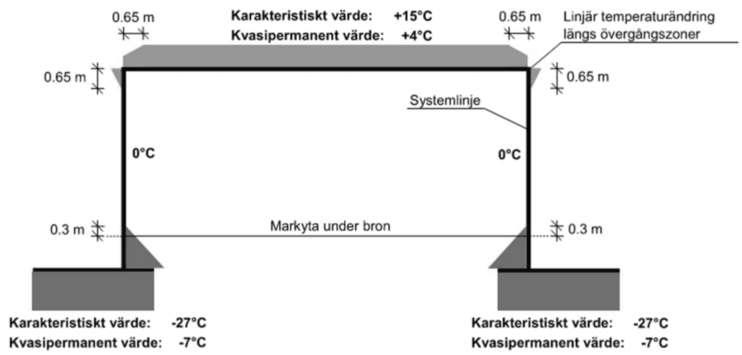
Load case
OJTEMP 1+
OJTEMP 1-

3.12.5 Uneven temperature differences between different construction parts (OJTEMP 2)

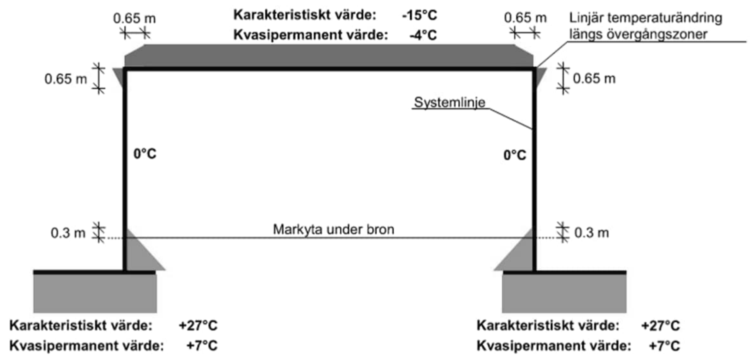
This section states that these effects should be combined with those caused by uniform temperature across the entire bridge (JTEMP).

Recommended values should, according to TRVINFRA-00227 section 2.1.1.2.4, be obtained from TVBK-0373. See extract below.

In TRVINFRA-00227 section 2.1.1.2.4, it is stated that no reduction should be made considering creep, but consideration should be made for cracking.



Principal sketch T(+)



Principal sketch T(-)

Applied temperature differences between superstructure and abutments:

$\Delta T = \pm 4°C$: SLS-Q

$\Delta T = \pm 15°C$: SLS-K, SLS-F and ULS

Remark

Since temperature load is not considered in the ultimate limit state (ULS) only temperature variation $\Delta T = \pm 4°C$ is considered.

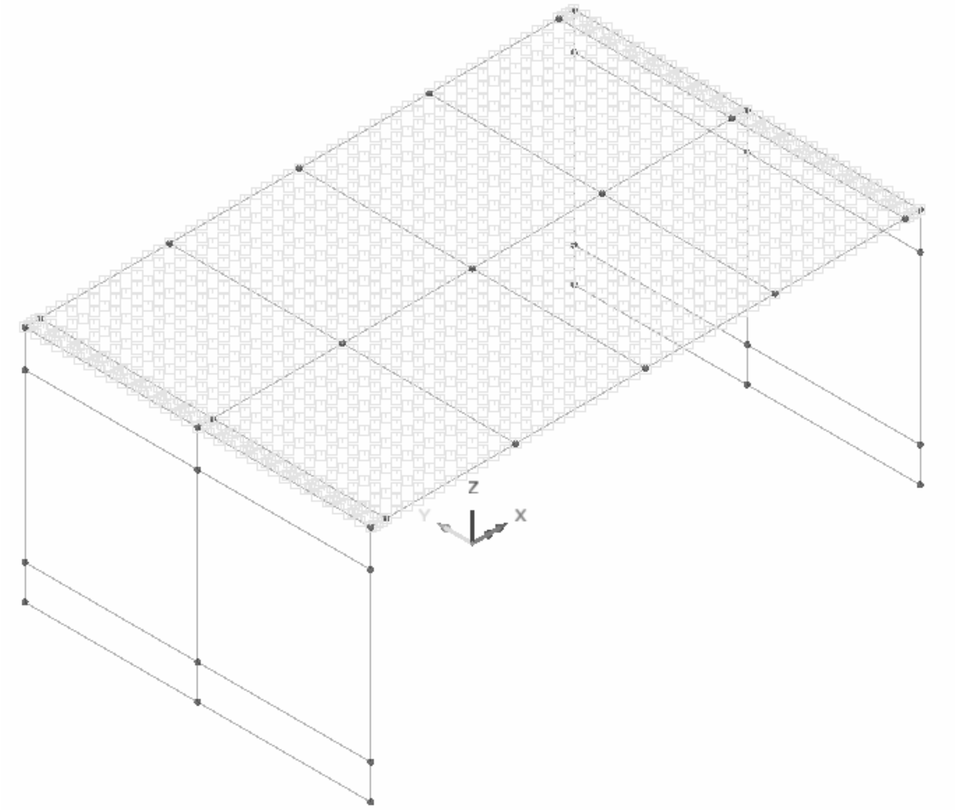
Load : OJTEMP 2

Structural loading : Temperature

Final temperature : +4 C

Initial temperature : ±0 C

Loadcase : OJTEMP 2+



Basic load cases :

Loadcase	Load	Factor
OJTEMP 2-	OJTEMP 2+	-1

Envelope OJTEMP *2:*

Loadcase
OTJEMP 2+
OJTEMP 2-

3.12.6 Impact of uniform temperature change

Load combination basic JTEMP MAX :

Load case	Factor
JTEMP+	1.00

Load combination basic JTEMP MIN :

Load case	Factor
JTEMP-	1.00

Envelope JTEMP :

Load case
JTEMP MAX
JTEMP MIN

3.12.6.1 Combining load case JTEMP and OJTEMP 1

Load combination is conducted according to SS-EN 1991-1-5, section 6.1.5. For such a combination, $\omega_M = 0.75$ och $\omega_N = 0.35$ shall be applied as shown below.

Alternative 1 ($\omega_M = 0.75$) : $T + \omega_M \cdot \Delta T$

Alternative 2 ($\omega_N = 0.35$) : $\omega_N \cdot T + \Delta T$

Load combination smart TEMP-1 :

Load case	Permanent factor	Variable factor
JTEMP	0	0.47 (= 0.77[1.)] x 0.6[2.)] x 1.00)
OJTEMP 1	0	0.45 (= 1.00[1.)] x 0.6[2.)] x 0.75)
DELTA-P	0	1.0

Load combination smart TEMP-2 :

Load case	Permanent factor	Variable factor
JTEMP	0	0.16 (= 0.77[1.)] x 0.6[2.)] x 0.35)
OJTEMP 1	0	0.60 (= 1.00[1.)] x 0.6[2.)] x 1.00)
DELTA-P	0	1.0

Note:
1.) Impact of creep results in reduced rigidity, see page A3:49.
2.) Impact of cracking results in reduced rigidity, see page A3:105.

3.12.6.2 Combining load case JTEMP and OJTEMP 2

Load combination smart TEMP-3 (SLS-F & SLS-K) :

Load case	Permanent factor	Variable factor
JTEMP	0	0.46 (= 0.77[1.)] x 0.6[2.)])
OJTEMP 2	0	0.60 (= 1.00 x 0.6[2.)])
DELTA-P	0	1.0

Note:
[1.)] Impact of creep results in reduced rigidity, see page A3:49.
[2.)] Impact of cracking results in reduced rigidity, see page A3:105.

3.12.6.3 Summary load combination limit service state (SLS)

Envelope TEMP:

Loadcase
TEMP-1
TEMP-2
TEMP-3

3.12.6.4 Summary load combination ultimate limit state (ULS)

Basic loadcomination TEMP-ULS:

Load case
DELTA P

3.13 LOAD COMBINATIONS

Verification of load capacity shall be carried out for several limit states as detailed in this section.

Fatigue Limit State:
The risk of fatigue according to the partial factor method is checked using equation 6.69 provided in document SS-EN 1992-1-1.

Other Limit States:
For other limit states, section 6.4.3 of EN-1990 is applied.

3.13.1 Ultimate Limit States (ULS)

When checking the ultimate limit state, the load factors vary depending on the type of failure as detailed below:

STR: Verification of structural bearing capacity

GEO: Verification of geotechnical bearing capacity

For checking the ultimate limit state, TRVNFRA-00227 section 7.1.6.3 specifies requirements for load combinations as follows.

Design Method D2 (Set B):
Design Method D2 (Set B) according to TSFS 2018:57 Table 4.4 shall be applied for the structural bearing capacity of the construction (STR; SK 3).

Design Method is defined according to EN-1990 equations 6.10a and 6.10b as detailed below.

$$E_{Sd}^{10a} = \sum_{j\geq 1} \gamma_{G.j} \cdot G_{k,j} + \gamma_{Q.1} \cdot \psi_{0,1} \cdot Q_{k.1} + \sum_{i>1} \gamma_{Q.i} \cdot \psi_{0,i} \cdot Q_{k,i} = \psi\gamma_{ULS-A} \cdot (\sum_{j>1} G_{k,j} + \sum_{i>1} Q_{k,i})$$

$$E_{sd}^{10b} = \sum_{j\geq 1} \xi_j \cdot \gamma_{G.j} \cdot G_{k,j} + \gamma_{Q.1} \cdot Q_{k.1} + \sum_{i>1} \gamma_{Q.i} \cdot \psi_{0,i} \cdot Q_{k,i} = \psi\gamma_{ULS-B} \cdot (\sum_{j>1} G_{k,j} + \sum_{i>1} Q_{k,i})$$

Equation 6.10a refers to the (ULS-A) case where the permanent loads are dominant, usually during the construction phase.

Equation 6.10b refers to the (ULS-B) case where the variable loads are dominant.
Design method 2 (set B) according to TSFS 2018:57 table 4.4 shall be applied for the structural capacity (STR; SK3).

A1 (construction loads)
All load factors are greater than set C.

A2 (geotechnical loads)

- Load coefficient earth pressure:
  $\psi\gamma_{ULS-A} = \gamma_d \cdot 1.35 \cdot \eta_{sup.G} = 0.91 \cdot 0.89 \cdot 1.35 \cdot 1.1 = 1.21$ ← dimensioning
  $\psi\gamma_{ULS-B} = \gamma_d \cdot 0.89 \cdot 1.35 \cdot \eta_{sup.G} = 0.91 \cdot 0.89 \cdot 1.35 \cdot 1.1 = 1.20$

- Load coefficient surcharge:
  $\psi\gamma_{ULS-A} = \gamma_d \cdot \psi_0 \cdot 1.50 = 0.91 \cdot 0.75 \cdot 1.50 = 1.02$
  $\psi\gamma_{ULS-B} = \gamma_d \cdot 1.50 = 0.91 \cdot 1.50 = 1.37$ ← dimensioning

Design method D3 (set C):

Design method D3 (set C) according to TSFS 2018:57 table 4.5 shall be applied for determining geotechnical bearing capacity (GEO; SK 2).

The design method is defined according to EN-1990 equation 6.10a and 6.10b as presented below.

$$E_{Sd}^{10a} = \sum_{j\geq 1} \gamma_{G.j} \cdot G_{k,j} + \gamma_{Q.1} \cdot \psi_{0,1} \cdot Q_{k.1} + \sum_{i>1} \gamma_{Q.i} \cdot \psi_{0,i} \cdot Q_{k,i} = \psi\gamma_{ULS-GA} \cdot (\sum_{j>1} G_{k,j} + \sum_{i>1} Q_{k,i})$$

$$E_{sd}^{10b} = \sum_{j\geq 1} \xi_j \cdot \gamma_{G.j} \cdot G_{k,j} + \gamma_{Q.1} \cdot Q_{k.1} + \sum_{i>1} \gamma_{Q.i} \cdot \psi_{0,i} \cdot Q_{k,i} = \psi\gamma_{ULS-GB} \cdot (\sum_{j>1} G_{k,j} + \sum_{i>1} Q_{k,i})$$

Equation 6.10a refers to the (ULS-A) case where the permanent loads are dominant, usually during the construction phase.

Equation 6.10b (ULS-B) refers to the case where the variable loads are dominant.

Design method 3 (set C) according to TSFS 2018:57 table 4.5 shall be applied for determining geotechnical bearing capacity (GEO).

A1 (construction loads)
All load factors are less than set B.

A2 (geotechnical loads)

- Load coefficient earth pressure: $\psi\gamma_{jord} = \gamma_d \cdot 1.1 \cdot \eta_{sup.G} = 0.91 \cdot 1.1 \cdot 1.1 = 1.10$

- Load coefficient surcharge: $\psi\gamma_{\text{över}} = \gamma_d \cdot 1.40 = 0.91 \cdot 1.40 = 1.27$

Simplified Design Method ULS:

To limit the number of load combinations, design method D2 (STR) is also applied for checking geotechnical bearing capacity (GEO). This is done by adjusting load coefficients associated with the geotechnical loads.
When applying the geotechnical loads, the earth pressure coefficient corresponding to D2 is applied.

Check load coefficients associated with the geotechnical loads

$K_o(D2) = 1 - sin(\varphi_d) = 1 - sin45^\circ = 0.29$

$K_o(D3) = 1 - sin(\varphi_d) = 1 - sin38^\circ = 0.38$

Earth pressure→ 1.45[1.)] $\cdot K_0(D2) = 0.42 \equiv 1.10 \cdot K_0(D3) = 0.42$ i.e. OK!

Surcharge → 1.70[2.)] $\cdot K_0(D2) = 0.49 \equiv 1.27 \cdot K_0(D3) = 0.49$ i.e. OK!

Footnotes

[1.)] Last coefficient $\psi\gamma_{ULS} = 1.45$ is applied instead of 1.21.
[2.)] Last coefficient $\psi\gamma_{ULS} = 1.70$ is applied instead of 1.37.

Permanent loads:

Nr	Load		$\psi\gamma_{ULS-A}$	$\psi\gamma_{ULS-B}$		$\psi\gamma_{ULS}$
1	Egentyngd	max	1,23	1,09		1,09
		min	1,00	1,00		1,00
2	Beläggning	max	1,36	1,21		1,21
		min	0,90	0.90		0,90
3	Överfyllnad	max	1,36	1,21		1,21
		min	0,90	0,90		1,00
4	Jordtryck	max	1,36	1,21	⇒	1,45[1.)]
		min	0.90	0,90		0.90
5	Vattentryck	max	1,23	1,09		1,09
		min	1,00	1,00		1,00
6	Stödförskjutning	max	1,23	1,09		1,09
		min	1,00	1,00		1,00
7	Krympning	max	1,23	1,09		1,09
		min	1,00	1,00		1,00

Footnote:

[1.)] Load coefficient according to page A3:131 is applied.

Remark

Equation ULS-B is considered dominant; thus ULS-A is not considered.

Variable loads:

Nr	Load	$\psi\gamma_{ULS-A}$	$\psi\gamma_{ULS-B}$		$\psi\gamma_{ULS}$
	Lastmodell LM 1 :				
9	Boggiesystem	1.03	1.03/1.37		1.03/1.37
10	Utbredd last	0.55	0.55/1.37		0.55/1.37
11	Bromskraft	0.76	0.76/1.03		0.76/1.03
12	Sidokraft	0.76	0.76/1.03		0.76/1.03
13	Centrifugalkraft	0.76	0.76/1.03		0.76/1.03
	Lastmodell LM 2 :				
14	Enstaka axellast	0	0/1.37		0/1.37
	Typfordon EG A/B :				
15	Typfordon EG A/B	1.03	1.03/1.37		1.03/1.37
16	Bromskraft	0.76	0.76/1.03		0.76/1.03
17	Sidokraft	0.76	0.76/1.03		0.76/1.03
18	Centrifugalkraft	0.76	0.76/1.03	⇒	0.76/1.03
19	Temperatur	0.82	0.82/1.37		0.82/1.37
	Vindlaster:				
20	Vindlast mot bro	0.41	0.41/1.37		0.41/1.37
21	Vindlast mot trafik	0.45	0.41		0.45/1.50
22	Överlast	1.03	1.03/1.37		1.03/1.70 2)

Footnote:

2.) Load coefficient according to page A3:131 is applied.

Loadcombination smart *ULS-PERM :*

Load case	Permanent factor	Variable factor
EGEN	1.00	0.09
BELÄGG	0.90	0.31
JORD	0.90	0.55
STOD	0	0.22 = (1.09 x 0.34.[1)] x 0.6[2.)])
KRYMP	0	0.22 = (1.09 x 0.34.[1)] x 0.6[2.)])

Footnotes:
[1.)] The effect of creep results in reduced stiffness; see page A3:49.
[2.)] The effect of cracking results in reduced stiffness; see page A3:105.

Load combination smart *ULS-VAR :*
(Load cases to consider: 5 / Variable load cases : 1)

Load case	Permanent factor	Variable factor
TRAFIK	1.03	0.34
BROMS	0.76	0.27
SIDO	0.76	0.27
TEMP-ULS	0.82	0.55
OVER	1.03	0.67

Load combination smart *ULS :*

Load case	Permanent factor	Variable factor
ULS-PERM	1	0
ULS-VAR	0	1

3.16.2 Service limit state (SLS)

The service limit state is divided into 3 load combinations based on their duration. The load combinations are presented below.

Load combination	Duration
SLS:K	Characteristic
SLS:F	Frequent
SLS:Q	Quasi-permanent

Load Combination SLS:K according to EN 1990 equation 6.14b is presented below.

$$E_{Sd} = \sum_{j\geq 1} G_{k,j} + Q_{k.1} + \sum_{i>1} \psi_{0,i} \cdot Q_{k,i} = \psi\gamma_{SLS.K} \cdot (\sum_{j>1} G_{k,j} + \sum_{i>1} Q_{k,i})$$

Load Combination SLS:F according to EN 1990 equation 6.15b is presented below.

$$E_{Sd} = \sum_{j\geq 1} G_{k,j} + \psi_1 \cdot Q_{k.1} + \sum_{i>1} \psi_{2,i} \cdot Q_{k,i} = \psi\gamma_{SLS.2} \cdot (\sum_{j>1} G_{k,j} + \sum_{i>1} Q_{k,i})$$

Load Combination SLS:Q according to EN 1990 equation 6.16b is presented below.

$$E_{Sd} = \sum_{j\geq 1} G_{k,j} + \sum_{i>0} \psi_{2,i} \cdot Q_{k,i} = \psi\gamma_{SLS.Q} \cdot (\sum_{j>1} G_{k,j} + \sum_{i>1} Q_{k,i})$$

When designing, load coefficients according to equations 6.14a, 6.15b, and 6.16b are applied.

Adjustment of load coefficients for geotechnical loads:

$K_o(D2) = 1 - sin(\varphi_d) = 1 - sin45^{\circ} = 0.29$

$K_o(D3) = 1 - sin(\varphi_d) = 1 - sin38^{\circ} = 0.39$

LC	Earth pressue
SLS-K	$\frac{K0(D3)}{K0(D2)} \cdot 1.1 = \frac{0.39}{0.29} \cdot 1.1 = 1.48$
SLS-F	$\frac{K0(D3)}{K0(D2)} \cdot 1.1 = \frac{0.39}{0.29} \cdot 1.1 = 1.48$
SLS-Q	$\frac{K0(D3)}{K0(D2)} \cdot 1.0 = \frac{0.39}{0.29} \cdot 1.0 = 1.34$

LC	Temperature
SLS-K	$\frac{K0(D3)}{K0(D2)} \cdot 1.0 = \frac{0.39}{0.29} \cdot 1.00 = 1.34$
SLS-F	$\frac{K0(D3)}{K0(D2)} \cdot 0.60 = \frac{0.39}{0.29} \cdot 0.60 = 0.81$
SLS-Q	0

LC	Surcharge
SLS-K	$\frac{K0(D3)}{K0(D2)} \cdot 1.0 = \frac{0.39}{0.29} \cdot 1.00 = 1.34$
SLS-F	$\frac{K0(D3)}{K0(D2)} \cdot 0.75 = \frac{0.39}{0.29} \cdot 0.75 = 1.01$
SLS-Q	0

Permanent loads:

Nr	Load		$\psi\gamma_{SLS-K}$	$\psi\gamma_{SLS-F}$	$\psi\gamma_{SLS-Q}$
1	Egentyngd	max	1.00	1.00	1.00
		min	1.00	1.00	1.00
2	Beläggning	max	1.10	1.10	1.00
		min	0.90	0.90	1.00
3	Överfyllnad	max	1.10	1.10	1.00
		min	0.90	0.90	1.00
4	Jordtryck	max	1.48 [3.)]	1.48 [3.)]	1.34 [3.)]
		min	0.90	0.90	1.00
5	Vattentryck	max	1.00	1.00	1,00
		min	1.00	1.00	1.00
6	Stödförskjutning	max	1.00	1.00	1.00
		min	1.00	1.00	1.00
7	Krympning	max	1.00	1.00	1.00
		min	1.00	1.00	1.00
8	Spännkraft	max	1.00	1.00	1.00
		min	1.00	1.00	1.00

Footnote:
[3.)] Load coefficient page A3:136 is applied.

Variable loads:

Nr	Load	$\psi\gamma_{SLS-K}$	$\psi\gamma_{SLS-F}$	$\psi\gamma_{SLS-Q}$
	Lastmodell LM 1 :			
9	Boggiesystem	0.75/1.00	0/0.75	0
10	Utbredd last	0.40/1.00	0/0.40	0
11	Bromskraft	0.56/0.75	0/0.56	0
12	Sidokraft	0.56/0.75	0/0.56	0
13	Centrifugalkraft	0.56/0.75	0/0.56	0
	Lastmodell LM 2 :			
14	Enstaka axellast	0.75/1.00	0/0.75	0
	Typfordon EG A/B :			
15	Typfordon EG A/B	0.75/1.00	0/0.75	0
16	Bromskraft	0.56/0.75	0/0.56	0
17	Sidokraft	0.56/0.75	0/0.56	0
18	Centrifugalkraft	0.56/0.75	0/0.56	0
19	Temperatur	0.60/1.00	0.50/0.60	0.50
	Vindlaster:			
20	Vindlast mot bro	0.30/1.00	0/0.30	0
21	Vindlast mot trafik	0.30/1.00	0/0.30	0
22	Överlast	0.75/1.34[4.)]	0/1.01[4.)]	0

Footnote:

[4.)] Load coefficients according to page A3:136 is applied.

Load combination smart *SLS-PERM :*

Loadcase	Permanent factor	Variable factor
EGEN	1.00	0
BELÄGG	0.90	0.20
JORD	0.90	0.58
STOD	0	0.22 = (1.0 x 0.37[1.)] x 0.6[2.)])
KRYMP	0	0.22 = (1.0 x 0.37[1).] x 0.6[2.)])

Footnotes:
[1.)] The effect of creep results in reduced stiffness; see page A3:49
[2.)] The effect of cracking results in reduced stiffness; see page A3:105

Load combination smart *SLS-K-VAR :*
(Load cases to consider : 5 / Variable load cases : 1)

Load case	Permanent factor	Variable factor
TRAFIK	0.75	0.25
BROMS	0.56	0.19
SIDO	0.56	0.19
TEMP	0.60	0.40
OVER	0.75	0.50

Load combination smart *SLS-F-VAR :*

Load case	Permanent factor	Variable factor
TRAFIK	0	0.75
BROMS	0	0.56
SIDO	0	0.56
TEMP	0	0.60
OVER	0	0.95

Load combination smart *SLS-K :*

Load case	Permanent factor	Variable factor
SLS-PERM	1	0
SLS-K-VAR	0	1

Load combination smart *SLS-F :*

Load case	Permanent factor	Variable factor
SLS-PERM	1	0
SLS-F-VAR	0	1

Load combination smart *SLS-Q :*

Load case	Permanent factor	Variable factor
SLS-PERM	1	0
TEMP	0	0.50

3.16.3 Accidental load combination

No accidental load cases are assumed.

3.14.4 Fatigue load combination

Fatigue is considered according to SS EN 1992-1-1, 6.8.4 and 6.8.6, and SS EN 1992-2, 6.8 and Appendix NN.

The risk of fatigue is checked using a simplified method, denoted as the λ-method.
Load combination according to equation SS-EN 1992-1-1 section 6.8.3 equation 6.69.

In this load combination, the traffic load is considered to consist of UTM, whereby other traffic loads are excluded.

$$E_{Sd} = \sum_{j\geq 1} G_{k,j} + P + \psi_{1,1} \cdot Q_{k,1} + \sum_{i>1} \psi_{2,i} \cdot Q_{k,i} + Q_{fat} = \psi\gamma_{UTM} \cdot (\sum_{j\geq 1} G_{k,j} + P + \sum_{i\geq 1} Q_{k,i} + Q_{fat})$$

Permanent loads:

Nr	Load		$\psi\gamma_{UTM}$
1	Egentyngd	max	1.00
		min	1.00
2	Beläggning	max	1.10
		min	0.90
3	Överfyllnad	max	1.10
		min	0.90
4	Jordtryck	max	1.48
		min	0.90
5	Vattentryck	max	1.00
		min	1.00
6	Stödförskjutning	max	1.00
		min	1.00
7	Krympning	max	1.00
		min	1.00

Variable loads:

Nr	Load	$\psi\gamma_{UTM}$
	Lastmodell LM 1 :	
9	Boggiesystem	-
10	Utbredd last	-
11	Bromskraft	-
12	Sidokraft	-
13	Centrifugalkraft	-
	Lastmodell LM 2 :	
14	Enstaka axellast	-
	Typfordon EG A/B :	
15	Typfordon EG A/B	-
16	Bromskraft	-
17	Sidokraft	-
18	Centrifugalkraft	-
19	Temperatur	0.60
	Vindlaster:	
20	Vindlast mot bro	0.30
21	Vindlast mot trafik	0.30
22	Överlast	1.01
23	UTM3	1.00

Load combination smart FAT :

Load case	Permanent factor	Variable factor
EGEN	1.00	0
BELÄGG	1.00	0
JORD	1.35	0
STOD	-	-
KRYMP	-	-
VIND	-	-
UTM	-	1.00
OVER	-	-
TEMP	-	-

Load case JORD is not a fatigue load, thus load coefficient highest load coefficient is applied.

Load cases TEMP, VIND and OVER are not fatigue loads, thus load is not considered.

During verification STR, the load case TEMP can be neglected according to SS-EN 1992-1-1 section 2.3.1.2(2).