VYSOKÉ UČENÍ TECHNICKÉ V BRNĚ
BRNO UNIVERSITY OF TECHNOLOGY

FAKULTA STAVEBNÍ
ÚSTAV GEOTECHNIKY

FACULTY OF CIVIL ENGINEERING

INSTITUTE OF GEOTECHNICS

HLOUBENÝ TRAMVAJOVÝ TUNEL V NORSKU
CUT & COVER LRT TUNNEL IN NORWAY

BAKALÁŘSKÁ PRÁCE
BACHELOR'S THESIS

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VYSOKÉ UČENÍ TECHNICKÉ V BRNĚ
FAKULTA STAVEBNÍ

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ZADÁNÍ BAKALÁŘSKÉ PRÁCE

Student	Simona Zetková
Název	Hloubený tramvajový tunel v Norsku
Vedoucí bakalářské práce	Michal Uhrin
Datum zadání bakalářské práce	30. 11. 2014
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V Brně dne 30. 11. 2014

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doc. Ing. Lumír Miča, Ph.D. Vedoucí ústavu	prof. Ing. Rostislav Drochytka, CSc., MBA Děkan Fakulty stavební VUT

Podklady a literatura

Design information:
- GIR + factual data;
- Design Statement;
- Summary Report;
- Outline Drawings;

Standards:
- Eurocodes, especially EN 1990, 1991, 1992, 1993, 1997 and 1998;

Software recommendations:
- SCIA ENGINEER;

Zásady pro vypracování

The objective is to undertake a simplified structural engineering design for an urban light rail (LRT) tunnel in Norway.

The work on this Thesis is expected to include the following activities:
1) Summary description of ground conditions;
2) Construction phasing plan;
3) Outline construction sequence schemes;
4) Comparison of drained and undrained tunnel design alternatives;
5) Simplified structural analysis and design checks for selected section;
6) Production of general arrangement or reinforcement details sketches for selected elements;

Deliverables:
1) Section in a report;
2) and 3) Description in a report + drawing;
4) and 5) Description in a report + calculation note in Appendix;
6) Min 1 drawing;

The baseline data will be provided in English and Norwegian. The Thesis shall be carried out in English.

Struktura bakalářské/diplomové práce

VŠKP vypracujte a rozčleňte podle dále uvedené struktury:

1. Textová část VŠKP zpracovaná podle Směrnice rektora "Úprava, odevzdávání, zveřejňování a uchovávání vysokoškolských kvalifikačních prací" a Směrnice děkana "Úprava, odevzdávání, zveřejňování a uchovávání vysokoškolských kvalifikačních prací na FAST VUT" (povinná součást VŠKP).
2. Přílohy textové části VŠKP zpracované podle Směrnice rektora "Úprava, odevzdávání, zveřejňování a uchovávání vysokoškolských kvalifikačních prací" a Směrnice děkana "Úprava, odevzdávání, zveřejňování a uchovávání vysokoškolských kvalifikačních prací na FAST VUT" (nepovinná součást VŠKP v případě, že přílohy nejsou součástí textové části VŠKP, ale textovou část doplňují).

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Michal Uhrin
Vedoucí bakalářské práce

Abstract

The aim of the thesis is to design and assess a Cut and Cover tunnel for light rail traffic underpassing a busy road. The tunnel is a part of route extension to the airport. As a part of the design it is required to assess ground conditions, design phasing of constructions, demonstrate construction sequence, perform simplified structural analysis, compare drained and undrained tunnel design alternatives and draw reinforcement schemes. All mentioned requirements were successfully accomplished with help of corresponding Eurocodes, books and technical advice. According to structural analysis it was possible to design a structure which meets the requirements of rail traffic and local geological conditions. The phasing was designed such that the impact on traffic on the road is minimal. Using the high quality concrete and reinforcement bars enabled designing sustainable and safe structure for the planned traffic. The comparison of the alternatives demonstrated that both design alternatives are feasible and realistic. When internal forces are compared, it is obvious that higher values occurred on undrained alternative, and therefore cross-sections would have to be enlarged or the structure would have to be more reinforced, which would increase its price. Important advantage of drained alternative is easier construction. On the other hand, the impact on the environment would be much lower with the undrained tunnel.

Key words

cut&cover, drained, undrained,

Abstrakt

Cílem této práce je navrhnout a posoudit hloubený tramvajový tunel, který je součástí plánovaného prodloužení tramvajové linky na letiště. Mezi požadované části patří posouzení a zhodnocení geologických podmínek, naplánování fází výstavby, schéma výstavby příčného řezu, zjednodušená statická analýza konstrukce, porovnání odvodněného a neodvodněného návrhu tunelu a schéma vyztužení příčného řezu. Všechny zmíněné části práce byly úspěšně zpracovány za pomoci potřebné literatury. Podle statické analýzy bylo možné navrhnout konstrukci, která vyhovuje požadavkům kolejové dopravy i místním geologickým podmínkám. Fázování výstavby bylo navrženo tak, aby minimálně ovlivnilo dopravu na podcházené komunikaci. Použití kvalitního betonu a betonářské výztuže umožnilo navrhnout bezpečnou konstrukci pro plánovanou dopravu. Porovnání zmíněných návrhů ukázalo, že obě možnosti jsou pro tuto konstrukci vhodné. Při porovnání vnitřních sil je možné pozorovat vyšší hodnoty u tunelu neodvodněného, bylo by tedy nutné zvětšit průřezy nebo zvýšit vyztužení konstrukce, což by vedlo k vyšší ceně konstrukce. Na druhou stranu neodvodněný tunel by měl mnohem menší dopad na životní prostředí.

Klíčová slova

Metoda hloubení, odvodněný, neodvodněný

Bibliografická citace VŠKP

Simona Zetková *Hloubený tramvajový tunel v Norsku.* Brno, 2015. 60 s., 19 s. příl. Bakalářská práce. Vysoké učení technické v Brně, Fakulta stavební, Ústav geotechniky. Vedoucí práce Michal Uhrin.

Prohlášení:

Prohlašuji, že jsem bakalářskou práci zpracovala samostatně a že jsem uvedla všechny použité informační zdroje.

V Brně dne 25.5.2015

…………………………………………………………

podpis autora

Simona Zetková

Poděkování:

Na tomto místě bych chtěla poděkovat panu Ing. Michalovi Uhrinovi za odborné vedení práce, cenné rady a připomínky, ochotu a vstřícný přístup v průběhu zpracování této bakalářské práce. Dále bych chtěla poděkovat paní Ing. Věře Glisníkové, CSc. za vstřícný přístup ústavu geotechniky.

List of Content

Introduction

The thesis is dealing with a design of a Cut and Cover tunnel in specific geological conditions underpassing a busy road.

Geology of the site is very important for the design of the tunnel. Therefore, by means of a geotechnical investigation report, geotechnical conditions are summarized and the procedure of geotechnical investigation is described.

As for the general planning it is necessary to design a plan of construction phases. Phasing needs to be designed in order to minimize the impact of the construction on road traffic and they must satisfy the site constraints. Construction sequence of the chosen cross-section is designed according to local geological conditions.

The geometry of the cross-section is simplified and suitable analytical models are created. For analytical models, the Scia Engineer software is used. It is necessary to take into account all loads that might occur and affect the structure. Load cases and their possible combinations are calculated according to corresponding Eurocodes.

Then design checks for M+N interaction and shear at critical sections are calculated and the reinforcement sketch is drawn according to these designs.

Finally, the comparison of drained and undrained tunnel design alternatives is carried out.

1. GEOLOGY

1.1. Geology of Norway

Norway is part of Scandinavia, but the geologic term for this area is the Fennoscandian Shield (or Baltic Shield). This includes Norway, Sweden, Finland and the north-western part of Russia. The rocks of Norway are very old, the oldest rocks are 3.5 billion years old, and they are generally very much alterated. Typical rocks are crystallines and metamorphites.

1.2. Geotechnical Investigation

An important part of site investigation in Norway is monitoring the rock exposures and recording their discontinuities in the rock mass. Nevertheless many other things have to be determined. Mostly geophysical testing is applied, accompanied by necessary probing or borehole drilling. The aim of the investigation is to determine the ground water level, the depth to rock surface and the character of soil on top.

1.3. Location of the Tunnel

The site is near the city of Bergen in Norway. The cut&cover tunnel is located on a light rail train route approximately 7.1 km long, which leads to the Bergen Airport Flessland. The tunnel is about 250 meters long tunnel underneath the busy road that leads to the airport.

Figure 1.1 Location of the Site [1]

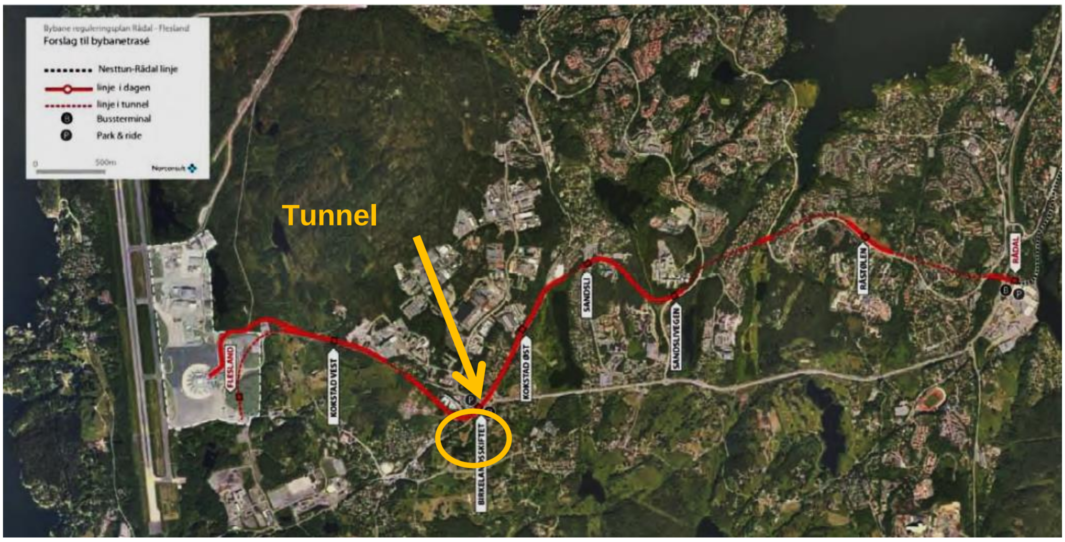

Figure 1.2 Location Plan [2]

1.4. Topography

In the topographical map we can estimate that the tunnel is approximately 50 metres above sea level. There are several natural rock exposures and excavated rock cuttings which cross the route and therefore the topography can change significantly over a short distance both parallel and perpendicular to the route.

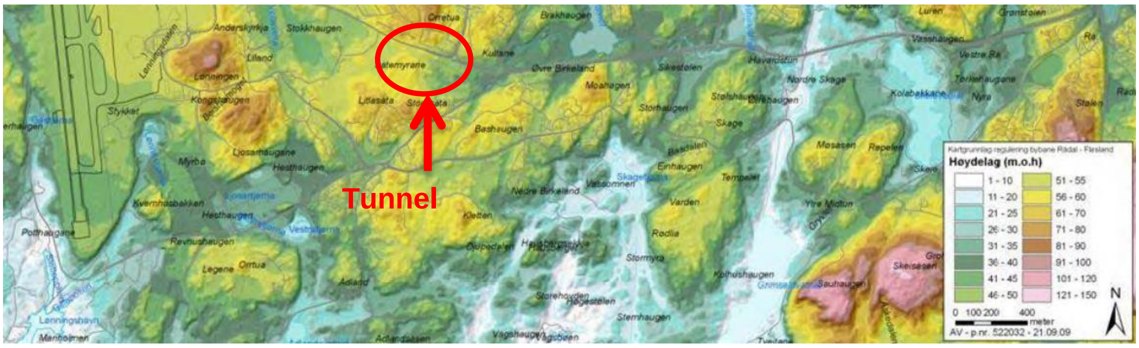

Figure 1.3 Topographical Map [2]

1.5. Geology

1.5.1. Geological Map

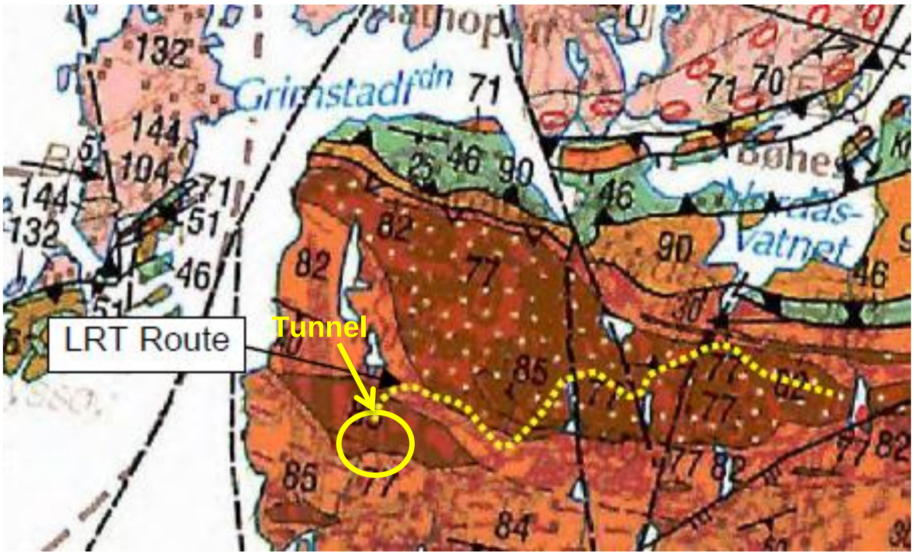

Figure 1.4 Geological Map of Route [3]

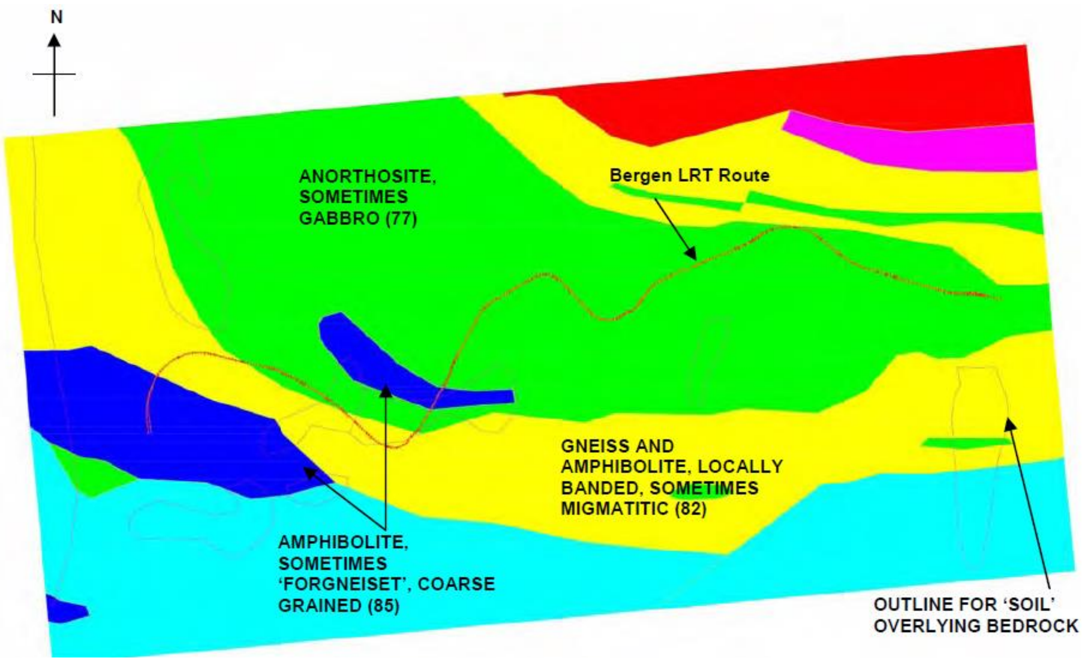

Figure 1.5 Detailed Geological Map of Route [3]

Rock description	Key in the map
Gneiss and amphibolite, locally banded, sometimes migmatitic	82
Anorthosite, sometimes also gabbro	77

1.5.2. Solid Geology

The geology of the Bergen area is dominated by the Bergen Arc which forms an arcuate structure of Caledonian nappes, centred on the city of Bergen.

During the Caledonian orogeny, the western margin of Scandinavia was subducted below Greenland. Subduction and subsequent collision created high-pressure metamorphic rocks that were subsequently exhumed. During exhumation, high-pressure rocks of the Bergen Arcs were thrusted towards the southeast, onto the Scandinavia margin.

The Bergen Arc consists of five tectonic units (from west to east):

- Øygarden Gneiss Complex
- Minor Bergen Arc
- Ulriken Gneiss Complex (Blåmanen Nappe)
- Anorthosite Complex (Lindås Nappe)
- Major Bergen Arc

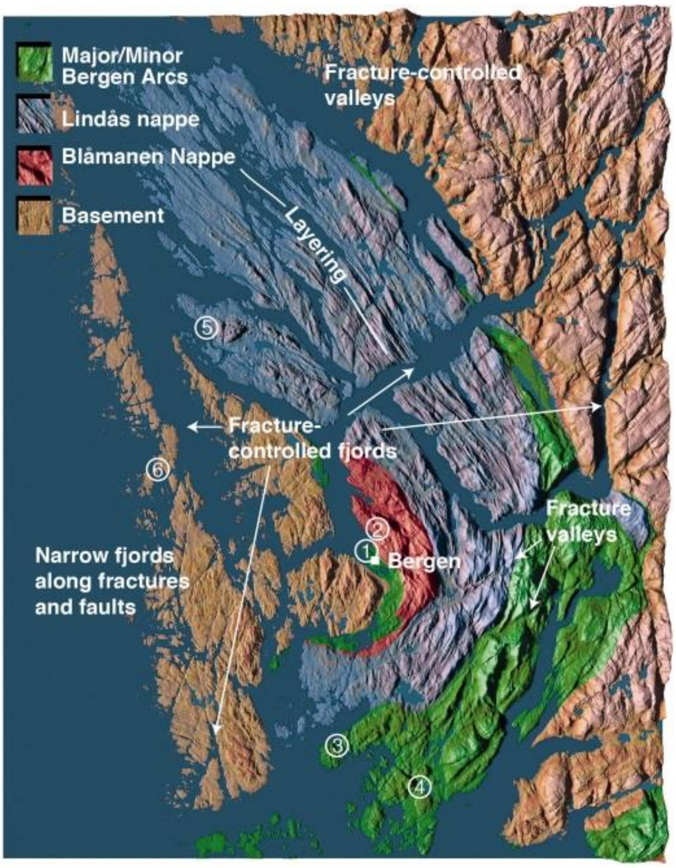

Figure 1.6 Bergen Arcs [4]

Lindås Nappe

The LRT route is dominantly within the Anorthosite Complex of the Lindås Nappe. The Lindås nappe is interpreted as reworked continental lower crust.

Regional Discontinuity Direction

The geological map indicates that the strike of the rock foliation in the area is usually in the direction east to west, but it may be locally variable. The dip of the foliation is also shown on the geological map to the range between 40° to the south and 90° (vertical).

Glaciation

The historical and existing ice sheets have had a profound impact on the Norwegian landscape. Many deep fjords, long U-shaped valleys, cirques and thousands of lakes in over-deepened bedrock basins are the result of glacial activity.

In a glaciated valley environment there are several characteristic ground conditions:

- Layers of cobbles and boulders at varying depths.
- Coarse granular tills which are often hard or very dense.

- Varying depth to rockhead.

The glaciers have deposited some of the material within the valleys and these can be referred to as moraines. The material within moraines usually shows no bedding and is not sorted or compacted. The material sizes range from sand to boulders.

Superficial Materials

The route is overlain by the following superficial materials:

- Peat bog
- Thin Moraine
- Bare rock, sometimes thin loose cover

Hydrogeology

As the tunnel is located in bog area the ground water is assumed to be on terrain.

1.5.3. Fieldwork and Laboratory Testing

Ground Investigation Techniques

- Wing or Vane Boring - rods are used to penetrate the ground with a large or small vane. It can be used to measure the undrained shear strength in cohesive soils.
- Total Probing - a combination of rock control drilling and modified rotary pressure probing.
- Marsh Probing - rods are inserted manually into ground and stop on hard ground
- Seismic Geophysics - seismic refraction

Laboratory Testing

In areas where it was impossible to access with a rig, samples were collected with a handheld auger drill.

Seismic Refraction Geophysics

The surveys were carried out in areas planned for tunnelling and portals. The precision of calculations of loose mass thickness for refraction seismology is indicated to be 2 *m* or 15%.

Deviations from the specified accuracy can occur with unfavourable geometry, side refraction and in conjunction with low speed zones and blind layers in the loose masses (layer with a lower speed under layer with higher speed).

Rock Exposures

The location of rock exposures at the ground surface have been identified and provided.

1.5.4. Summary from investigation

Using mentioned techniques ground sections were defined. The depth of rock varies between 0-3.5 m along the tunnel. The overlaying material is peat, soft clay, firm clay or cohesive and granular glacial moraine or their combination. The rock exposures along the tunnels are mainly composed of gneiss (very strong) and Mica Schist. Mica Schist varies from very weak to medium strong rock that easily split into the flakes or slabs due to the well-developed preferred orientation of the minerals. Mica Schist appearing along the tunnel can cause instability of the excavation. It is therefore necessary to define its occurrence and direction to prevent any failure during construction. The characteristics of these are described further. The ground water level is assumed to be on ground surface.

Peat and Organic Clay

It is anticipated that all peat material will be removed from beneath the route and therefore parameters will not be required for foundation design.

Glacial Cohesive and Granular (Moraine)

No laboratory tests have been carried out.

1.5.5. Superficial Materials Geotechnical Parameters

Geotechnical characteristic values are given in Table 6.2.

Table 1.1 Superficial Material Geotechnical Characteristic Values [2]

Material Description	Abbreviation	Unit Weight γ (kN/m3)	Critical Angle of Friction φ'crit (º)	Effective Cohesion c' (kPa)	Undrained Shear Strength cu (kPa)
Peat and Organic Clay	PEAT	14	0	0	5
Firm Clay and Sand	CLSA	19	24	0	40
Granular material	GLAC	18 moist 20	34	0	granular
Imported Granular Fill	n/a	19 moist 20	38	0	granular

1.5.6. Rock Geotechnical Parameters

Geotechnical parameters are given in Table 6.7.

Table 1.2 Rock Geotechnical Parameters [2]

Material	Anorthosite, Gneiss, Gabbro, Dolerite	Mica Schist
Unit Weight (kN/m3)	27	26
Porosity (%)	0.5 to 2.0	0.55 to 0.84
Poisson's Ratio	0.2	0.2
Basic Friction Angle (º)	27	20
Minimum Strain Modulus (MPa)	7500	2000

1.5.7. Rock Discontinuities

Generally, three to four significant discontinuity sets were noted at each rock exposure during the 2012 rock exposure site visits. The rock can be described as typically moderately fractured with 5 to 15 joints per m^3 of rock.

Foliation

Foliation or gneissose banding is produced by parallel layering of different composition. The foliation recorded at this site is due to layering of felsic (light plagioclase feldspar) and mafic (dark possibly olivine or pyroxene) materials. Additional schistose foliation (alignment of mica platy minerals) has been observed in rock exposures along the tunnel.

1.6. Summary

- Shallow rock cover with possible localization of deeper areas
- Strong rock interbedded with weaker Micha Schist rock
- Conventional excavation in soils in slope 1:2
- Retaining walls in soils are used where the construction site is limited
- Blasting in rock in slope 5:1
- Footings of the tunnel are based on rock
- No stable ground water level – for design is assumed on ground level

2. PHASE OF CONSTRUCTION

2.1. Local Point of View

2.1.1. Construction Sequence

Construction sequence of general cross-section is depicted in Annex A.1

- The slopes of excavation in soils are 1:2
- At soil-rock interface is a bench created
- Slopes in rock are 5:1

2.2. Global Point of View

The tunnel will pass under the existing road. The crossing will be at a very acute angle. A temporary diversion of the road traffic will be required during the cut and cover construction of the tunnel across the road. Therefore, a staged construction is envisaged for the tunnel as follows:

1. The section of the tunnel (approximately 1/3 of its total length) south of the road will be constructed and backfilled while the road remains in operation. Temporary retaining walls up to 5 *m* high will be required to minimize the extent of excavation for the tunnel above the rock level in areas close to the road.

2. A temporary road bypass will be constructed over the completed section of the tunnel. Road traffic will be diverted via the bypass.

3. The remaining section of the tunnel will be constructed and backfilled. Due to limited land available for the bypass, temporary retaining walls up to 5m high will be required to minimize the extent of excavation for the tunnel above rock level where the tunnel cut runs parallel to the road bypass.

4. The road will be restored into its original position, road traffic diverted back and the bypass removed. The tunnel will be constructed in an open cut and backfilled.

2.2.1. Phase plan

Phase plan is designed in order to minimize the impact on traffic on the road the individual steps are depicted in Annex A.2.

3. STRUCTURAL CALCULATION

3.1. Standards for Calculation

Eurocode 7 is used for the calculation of load cases in which design approach three is applied as far as Norway prefers this approach. In the design approach 3(DA-3) soil parameters and static parameters are reduced according to table below. For dimensioning of reinforced concrete frame Eurocode 2 is used. [5] For load combinations Eurocode 0 is used.

3.1.1. Design Approach 3 – Eurocode 7 [5]

DA-3: (A1 or A2)* + M2 + R3

*A1 is for structural actions and A2 is for geotechnical actions

Table 3.1 Partial factors on actions or effects of actions

Action		Symbol	Set	
			A1	**A2**
Permanent	Unfavourable	γ_G	1,35	1,0
	Favourable		1,0	1,0
Variable	Unfavourable	γ_Q	1,5	1,3
	Favourable		0	0

Table 3.2 Partial factors for soil parameters (γ_M)

Soil parameter	Symbol	Value	
		M1	**M2**
Shearing resistance	$\gamma_{\varphi}{}^{1}$	1,0	1,25
Effective cohesion	γ_c	1,0	1,25
Undrained strength	γ_{cu}	1,0	1,4
Unconfined strength	γ_{qu}	1,0	1,4
Effective cohesion	γ_c	1,0	1,4
Weight density	γ_{γ}	1,0	1,0
[1] This factor is applied to tan φ'			

3.2. Material

3.2.1. Concrete

Class of concrete: C45/55 at 28 days

$$f_{ck} = 45\ MPa$$
$$f_{ck\ cube} = 55\ Mpa$$
$$f_{cm} = 53\ MPa$$
$$E_{cm} = 36\ GPa$$
$$\nu = 0.2$$
$$\alpha_T = 1.00 \times 10^{-5}\ C^{\circ -1}$$
$$\alpha_{cc} = 1.0$$
$$\alpha_{ct} = 1.0$$
$$\lambda = 0.8$$
$$\eta = 1$$
$$\varepsilon_{c3} = 1.75\ ‰$$
$$\varepsilon_{cu3} = 3.5\ ‰$$
$$\gamma_c = 1.5$$

3.2.2. Reinforcement

Class of Reinforcement: B500

Ductility: C

$$f_{yk} = 500\ MPa$$
$$E_s = 200\ GPa$$
$$\varepsilon_{yk} = 2.5\ ‰$$
$$\varepsilon_{uk} = 75\ ‰$$
$$\gamma_s = 1.15$$
$$f_{yd} = \frac{f_{yk}}{\gamma_s} = \frac{500}{1.15} = 434.78\ MPa$$

3.2.3. Subsoil

Stiffness of the springs

The subsoil of tunnel is formed by rock. For the calculation I assume the minimal strain modulus of Mica Schist as far as it is the weakest rock along the route.

$E_{min} = 2000\ MPa$ …strain modulus of Mica Schist

$B = 2.2\ m$ …width of footing

$$k_z \cong \frac{E}{B} = \frac{2000\ MPa}{2.2\ m} = 909.09\ MNm^{-3}$$

$$k_x \cong 0.1 \times k_z = 0.1 * 909.09 \cong 90.91\ MNm^{-3}$$

3.2.4. Backfill material

Imported granular fill – well graded will be used as a backfill material.

Design properties

$\gamma = 19\ kNm^{-3}$ …unit weight – natural moisture content

$\varphi' = 38°$ …effective internal friction angle

$c' = 0\ kPa$ …effective cohesion

3.3. Cut and Cover Geometry

3.3.1. Geometry

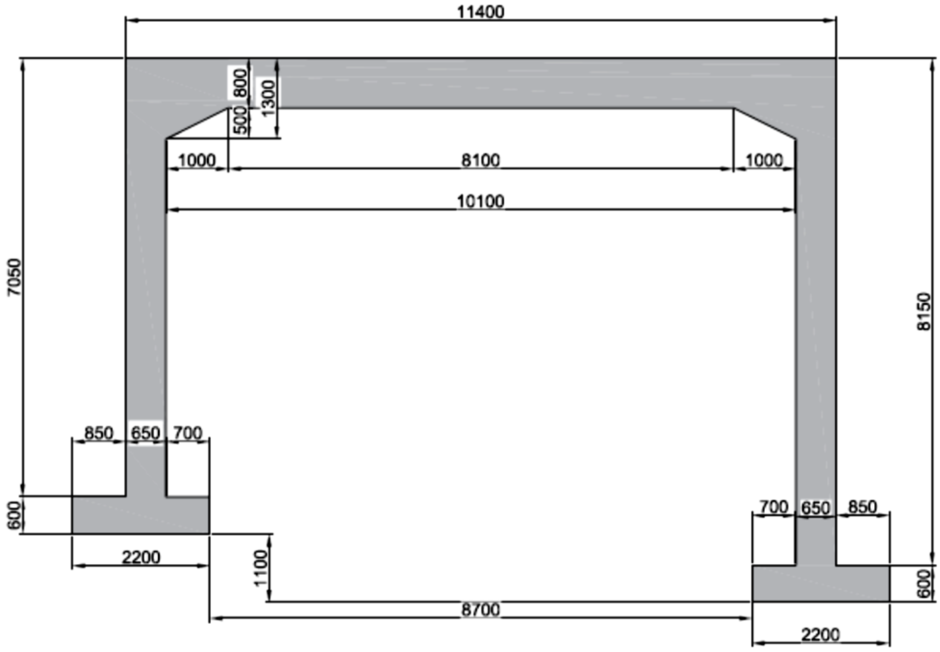

Figure 3.1

3.3.2. Scia Model

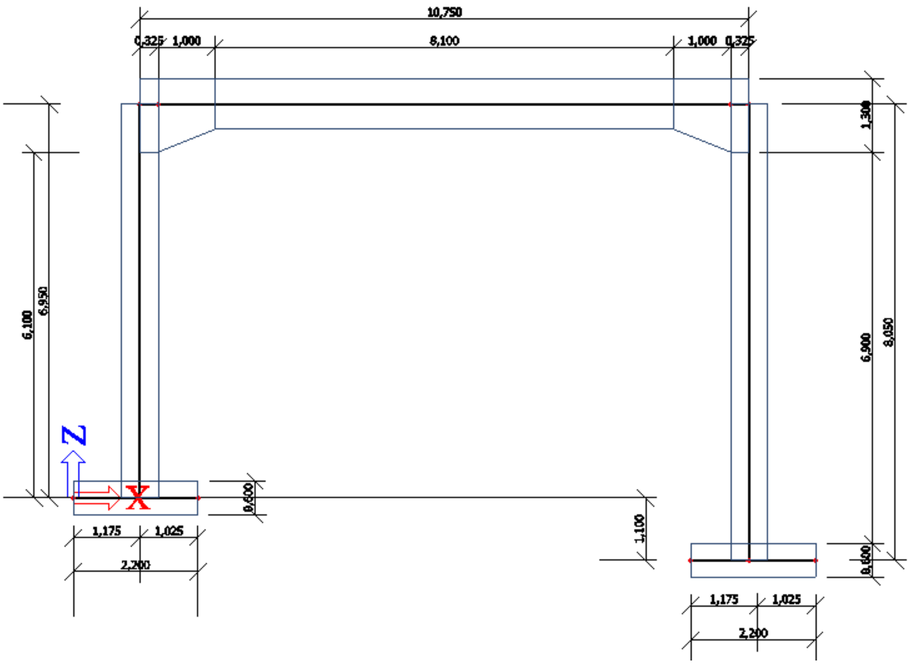

Figure 3.2

3.3.3. Version A - Drained Tunnel

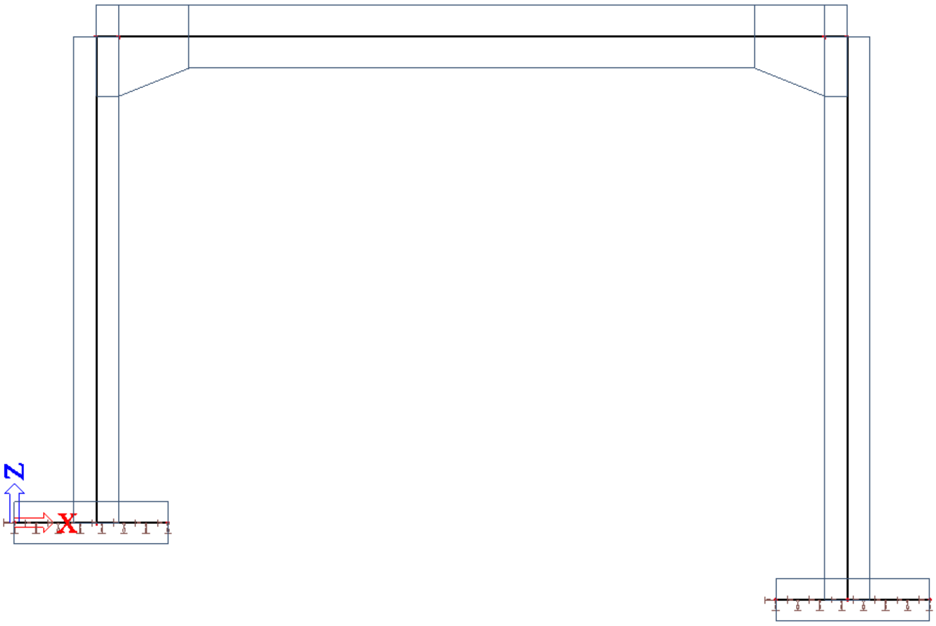

Figure 3.3

3.3.4. Version B – Unrained Tunnel

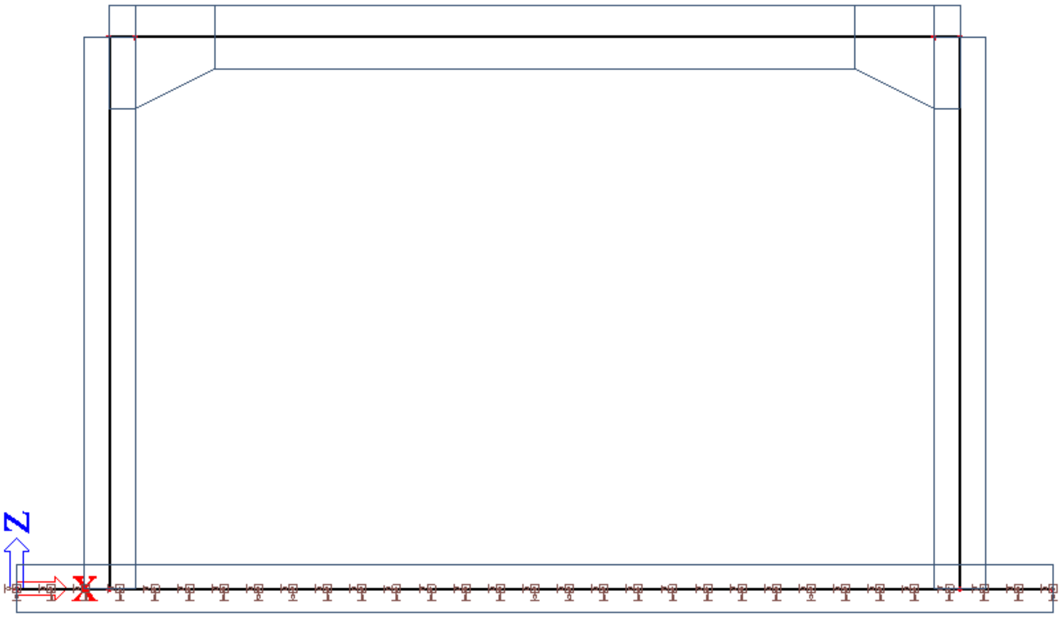

Figure 3.4

3.4. Load Cases

3.4.1. LC 1 – Self weight of RC structure

3.4.2. LC2 – Lateral backfill pressure to roof level

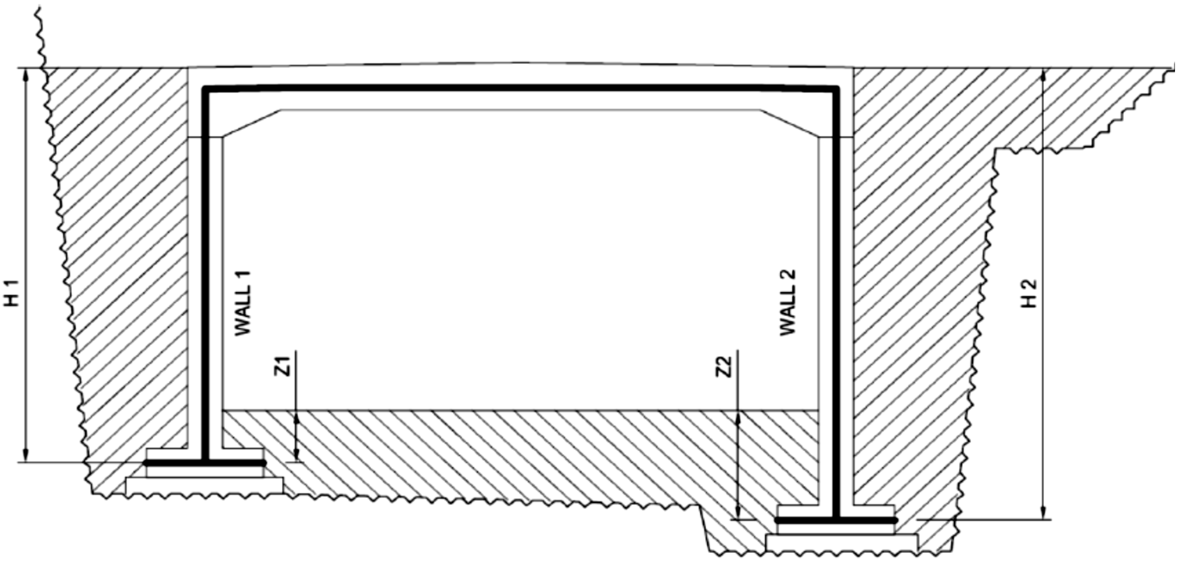

Figure 3.5 Version A

$\gamma = 19\ kNm^{-3}$

$\varphi' = 38°$

$\gamma_M = 1.25$ …partial material factor (EC7 - DA3)

$\gamma_g = 1.00$ …partial action factor (EC7 - DA3)

$H_1 = 7.35\ m$

$H_2 = 8.45\ m$

$Z_1 = 1.00\ m$

$Z_2 = 2.10\ m$

$\varphi'_d = \tan^{-1}\left[\frac{(\tan\ \varphi')}{\gamma_M}\right] = \tan^{-1}\left[\frac{(\tan 38°)}{1.25}\right] = 32°$ …design value

$K_0 = 1 - \sin\ \varphi'_d = 1 - \sin 32° = 0.47$ …ground pressure coefficient

Lateral backfill pressure on the side walls

Left wall:

$$\sigma = \gamma \times H_1 \times K_0 \times \gamma_g = 19 \times 7.35 \times 0.47 \times 1.0 = 65.64\ kPa$$

$$\sigma = \gamma \times Z_1 \times K_0 \times \gamma_g = 19 \times 1.00 \times 0.47 \times 1.0 = 8.93\ kPa$$

Right wall:

$$\sigma = \gamma \times H_2 \times K_0 \times \gamma_g = 19 \times 8.45 \times 0.47 \times 1.0 = 75.46\ kPa$$

$$\sigma = \gamma \times Z_2 \times K_0 \times \gamma_g = 19 \times 2.10 \times 0.47 \times 1.0 = 18.75\ kPa$$

For version B the load is symmetrical and height of lateral pressure is the same as for the left wall in Version A. A detailed calculation will not be mentioned here. For results see figures.

Figure 3.6

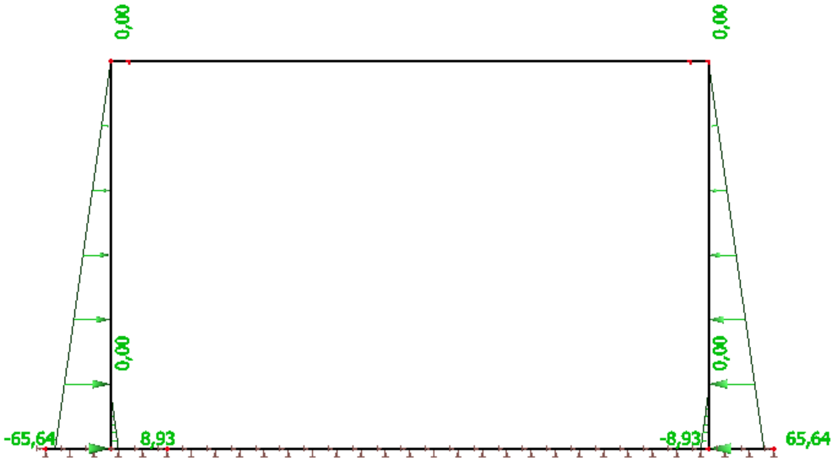

Figure 3.7

3.4.3. LC3 – Lateral backfill pressure to ground level

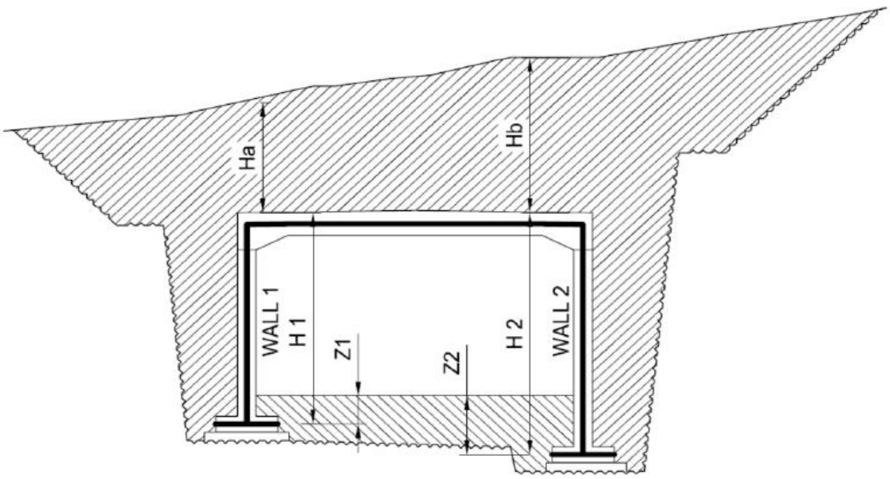

Figure 3.8

$H_a = H_b = 6.50\ m$

Lateral backfill pressure on the side walls

Left wall:

$$\sigma_{top} = \gamma \times H_a \times K_0 \times \gamma_g = 19 \times 6.5 \times 0.47 \times 1.0 = 58.05\ kPa$$

$$\sigma_{bottom} = \gamma \times (H_a + H_1) \times K_0 \times \gamma_g = 19 \times 13.85 \times 0.47 \times 1.0 = 123.68\ kPa$$

$$\sigma = \gamma \times Z_1 \times K_0 \times \gamma_g = 19 \times 1.00 \times 0.47 \times 1.0 = 8.93\ kPa$$

Right wall:

$$\sigma_{top} = \gamma \times H_a \times K_0 \times \gamma_g = 19 \times 6.5 \times 0.47 \times 1.0 = 58.05\ kPa$$

$$\sigma_{bottom} = \gamma \times (H_a + H_2) \times K_0 \times \gamma_g = 19 \times 14.95 \times 0.47 \times 1.0 = 133.50\ kPa$$

$$\sigma = \gamma \times Z_2 \times K_0 \times \gamma_g = 19 \times 2.10 \times 0.47 \times 1.0 = 18.75\ kPa$$

For version B the load is symmetrical and height of lateral pressure is the same as for the left wall in Version A. A detailed calculation will not be mentioned here. For results see figures.

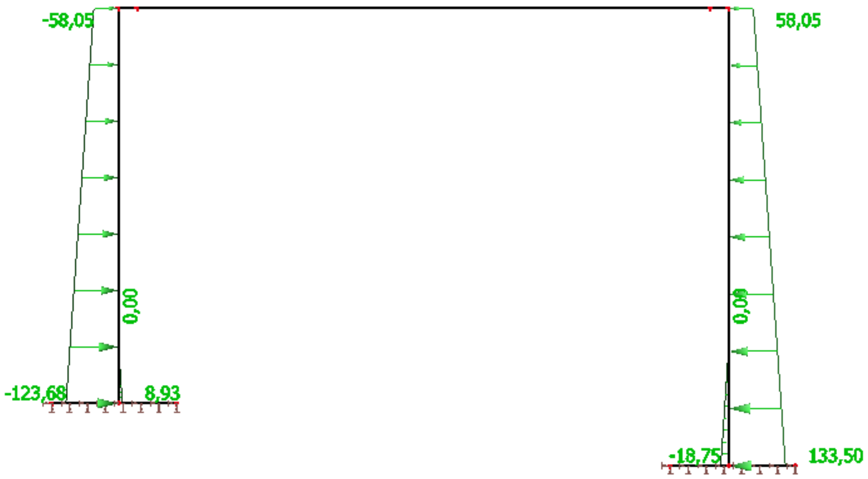

Figure 3.9

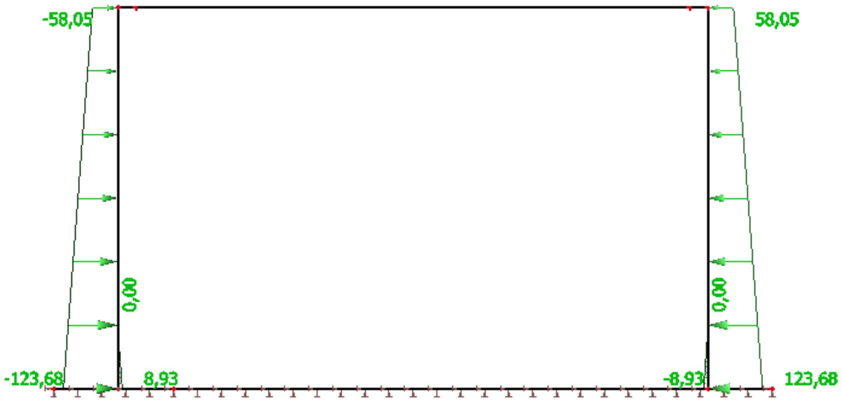

Figure 3.10

3.4.4. LC4 – Vertical load caused by backfill pressure, protective concrete etc.

Vertical load from protective concrete C25/30:

$$\gamma_2 = 23\ kNm^{-3}$$

$$H_c = 0.05\ m$$

$$q = 23 \times 0.05 = 1.15\ kPa$$

$$\gamma_{gv} = 1.35$$

Vertical load from backfill above the box:

On the roof: $q = \gamma \times H_a \times \gamma_{gv} = 19 \times 6.5 \times 1.35 = 166.725\ kPa$

Left footing $q_1 = \gamma \times (H_1 + H_a) \times \gamma_{gv} = 19 \times 13.85 \times 1.35 = 355.253\ kPa$

$q_2 = \gamma \times Z_1 \times \gamma_{gv} = 19 \times 1.00 \times 1.35 = 25.65\ kPa$

Right footing $q_3 = \gamma \times (H_2 + H_a) \times \gamma_{gv} = 19 \times 14.95 \times 1.35 = 383.468\ kPa$

$q_4 = \gamma \times Z_2 \times \gamma_{gv} = 19 \times 2.1 \times 1.35 = 53.865\ kPa$

For version B the load is symmetrical with values according to left side of the section. A detailed calculation will not be mentioned here. For results see figures.

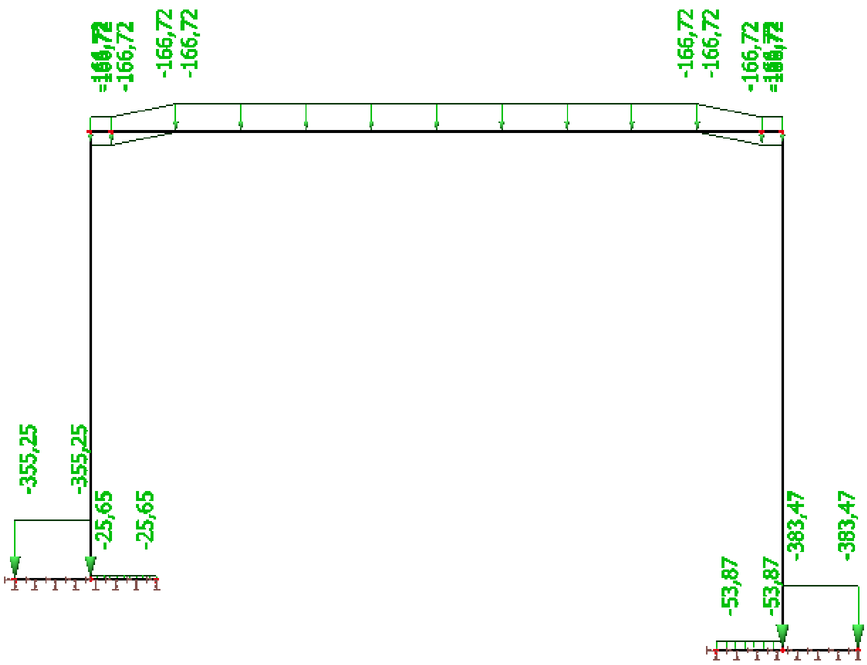

Figure 3.11

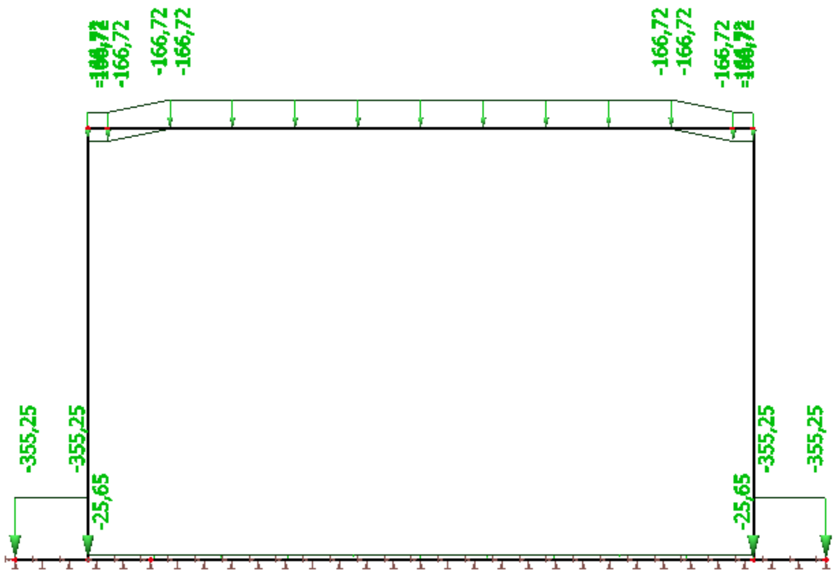

Figure 3.12

3.4.5. LC5 – Road traffic

The load from traffic is solved according to NS-EN 1991-2 [6]. The tandem system (TS) and the uniformly distributed load (UDL) are assumed. The vertical load dispersion till the centreline is calculated in slope of 1:1.

Tandem system

$Q_{TS} = 300\ kN$ …axial load

Uniformly distributed load

$q_{UDL} = 9\ kN/m^2$

Area of load

$b = 1.6\ m$

$l = 4.9\ m$

$A = 7.84\ m^2$

$\gamma_{qv} = 1.5$

$\gamma_{qh} = 1.3$

Area of load H_3 metres above the roof slab

$$A_{H3} = (1.6 + 2 \times H_3) \times (4.9 + 2 \times H_3) = (1.6 + 2 \times 6.5) \times (4.9 + 2 \times 6.5) = 261.3\ \mathrm{m}^2$$

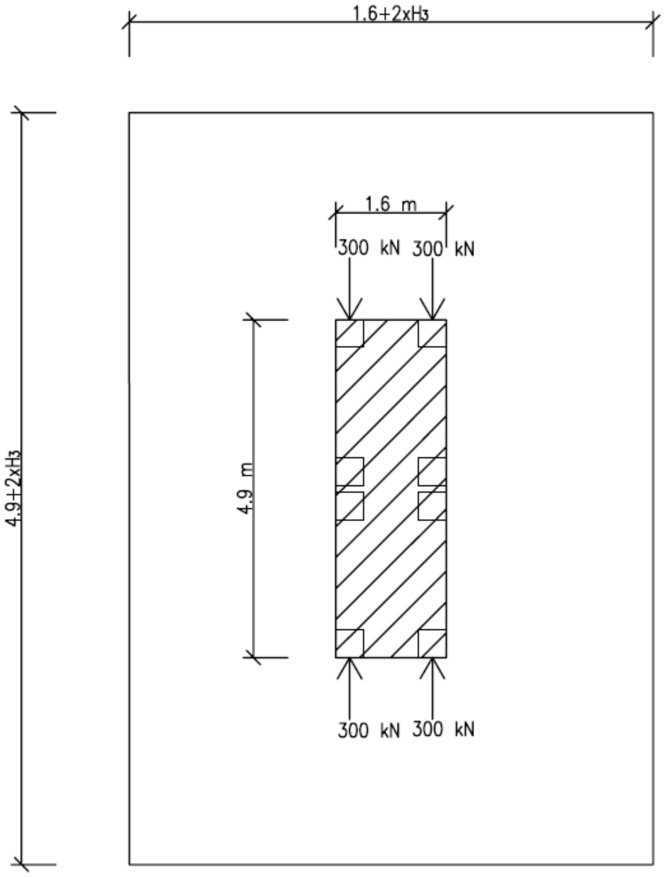

Figure 3.13 Local Verification [6]

Pressure on the roof slab

$$q = \left(\frac{Q_{TS} \times 4}{A_{H3}} + q_{UDL}\right) \times \gamma_{qv} = \left(\frac{300 \times 4}{261.3} + 9\right) \times 1.5 = 20.4\ kN/m^2$$

Pressure on side walls

$$q = q \times K_0 \times \gamma_{qh} = 13.6 \times 0.47 \times 1.3 = 8.3\ kN/m^2$$

The load from traffic can be applied in several variations, either only pressure from left with or without pressure on the roof slab or from right respectively. The other possibility is the pressure from both sides again with or without pressure on the roof slab.

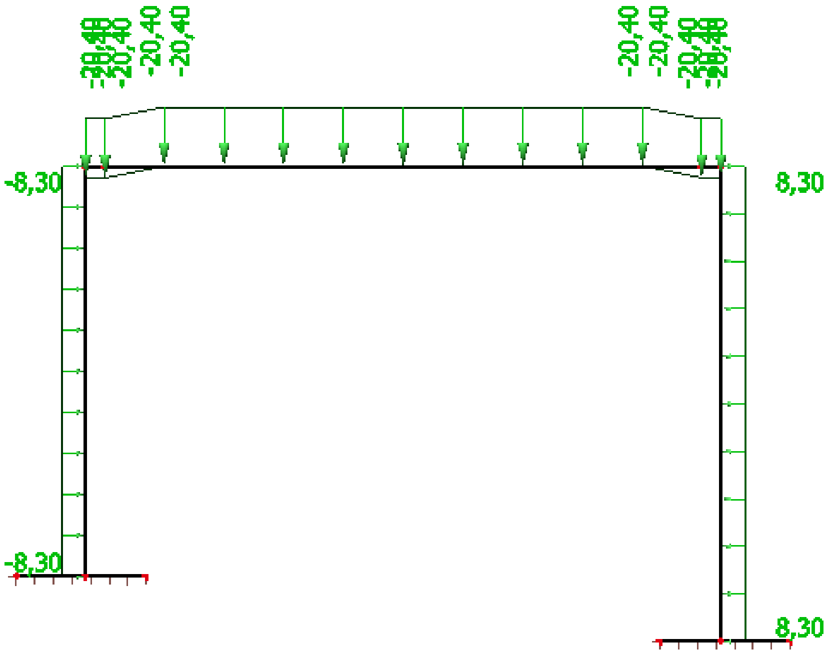

Figure 3.14

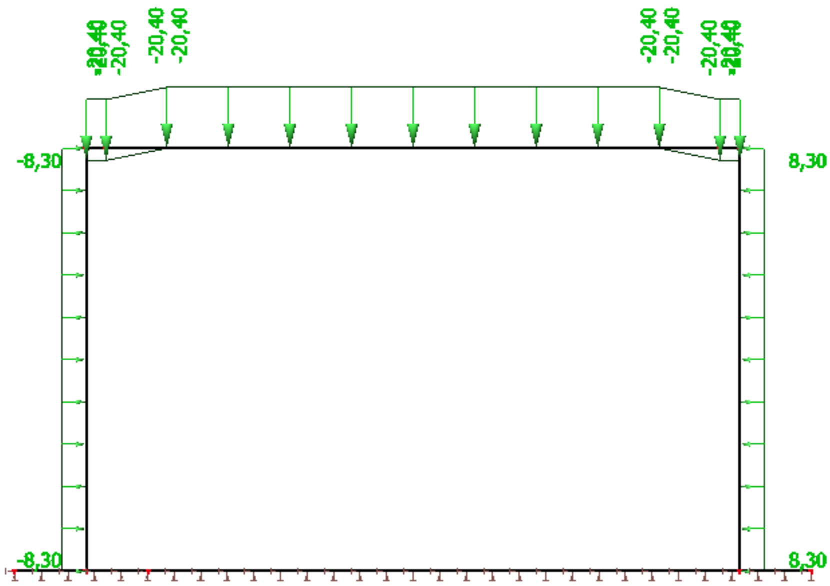

Figure 3.15

3.4.6. LC6 – Surface surcharge

Pressure on the roof slab

$q = 20\ kPa$ …uniformly distributed load

$q_v = 20 \times \gamma_{qv} = 20 \times 1.5 = 30\ kPa$

Pressure on side walls

$q_h = q \times K_0 \times \gamma_{qh} = 20 \times 0.47 \times 1.3 = 12.22\ kPa$

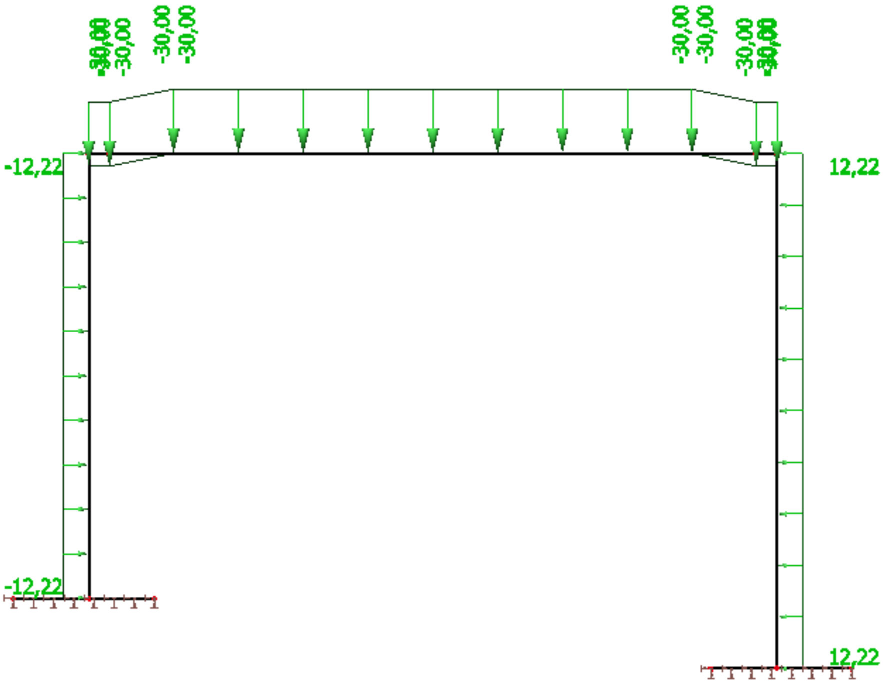

Figure 3.16

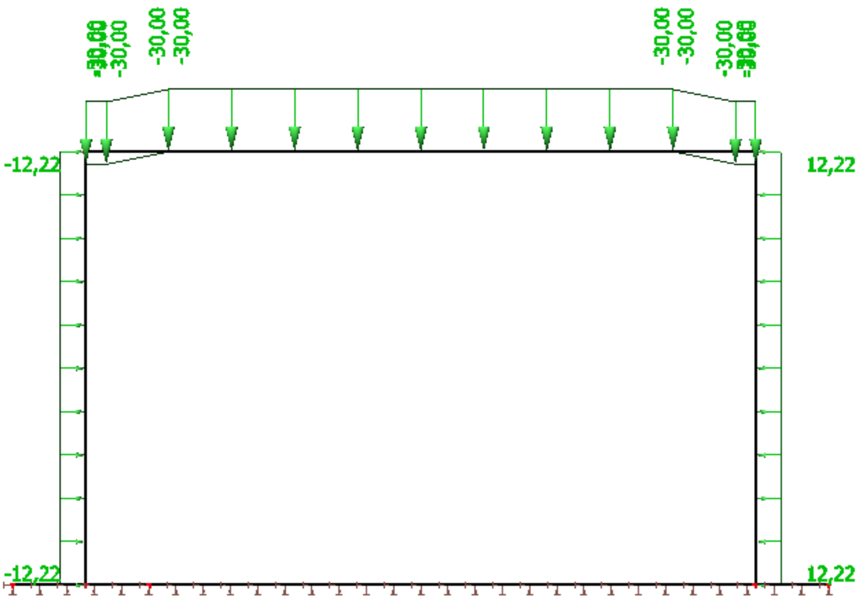

Figure 3.17

3.4.7. LC7 – Suction from the rail traffic

$q = 3\ kPa$

$q_d = q \times \gamma_q = 3 \times 1.5 = 4.5\ kPa$

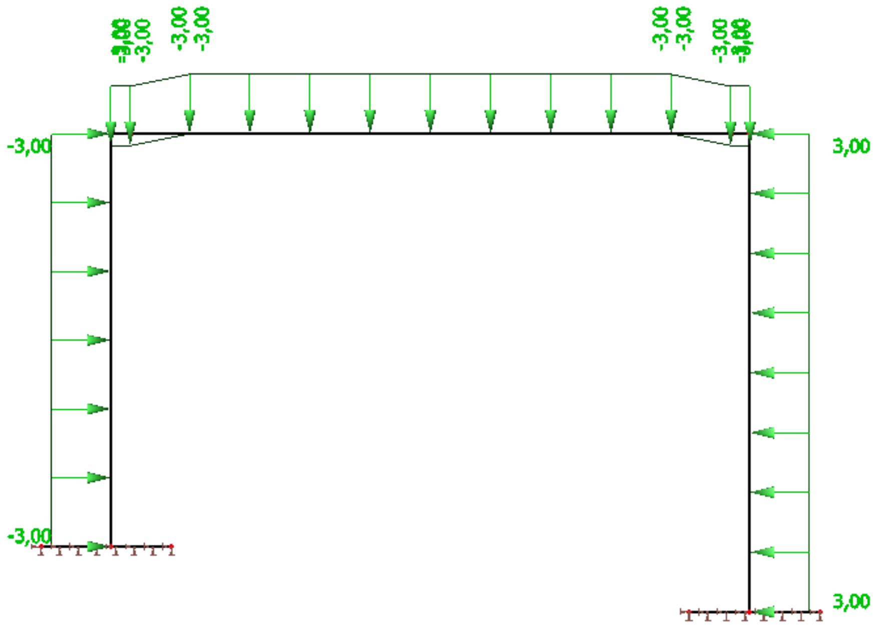

Figure 3.18

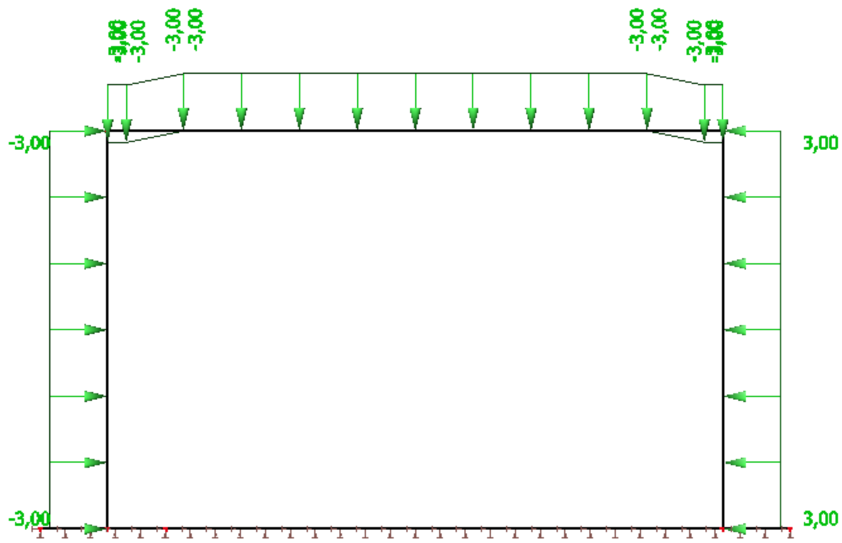

Figure 3.19

3.4.8. LC8 – Accidental load – seismic

According to NS-EN 1998 and Norwegian national annex [5]

$a_g = 0.85\ ms^{-2}$ …ground acceleration from Figure NA.3(901)

$g = 9.81\ ms^{-2}$ …gravity acceleration

$$\alpha = \frac{a_g}{g} = \frac{0.85}{9.81} = 0.09$$

$S = 1.0$ …ground factor, ground type A, Table 3.2

$\gamma = 19\ kN/m^3$ …ground unit weight

$H_2 = 8.45\ m$ …height of the structure for drained version (A)

$H_1 = 7.35\ m$ …height of the structure for undrained version (B)

$\Delta P_d = \alpha \times S \times \gamma \times H^2$ …lateral ground pressure NS-En 1998-5, E.9 [7]

$$\Delta P_{dA} = 0.09 \times 1.0 \times 19 \times 8.45^2 = 117.41\ kN$$
$$\Delta P_{dB} = 0.09 \times 1.0 \times 19 \times 7.35^2 = 92.38\ kN$$
$$\Delta p_{0A} = 0.5 \times \alpha \times S \times \gamma \times H = 0.5 \times 0.09 \times 1 \times 19 \times 8.45 = 7.22\ kPa$$
$$\Delta p_{hA} = 1.5 \times \alpha \times S \times \gamma \times H = 1.5 \times 0.09 \times 1 \times 19 \times 8.45 = 21.67\ kPa$$
$$\Delta p_{0B} = 0.5 \times \alpha \times S \times \gamma \times H = 0.5 \times 0.09 \times 1 \times 19 \times 7.35 = 6.28\ kPa$$
$$\Delta p_{hB} = 1.5 \times \alpha \times S \times \gamma \times H = 1.5 \times 0.09 \times 1 \times 19 \times 7.35 = 18.85\ kPa$$

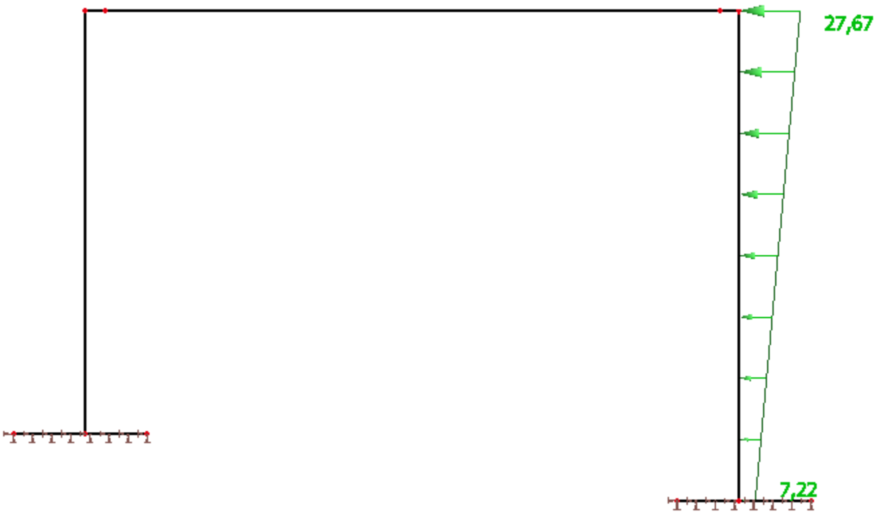

Figure 3.20

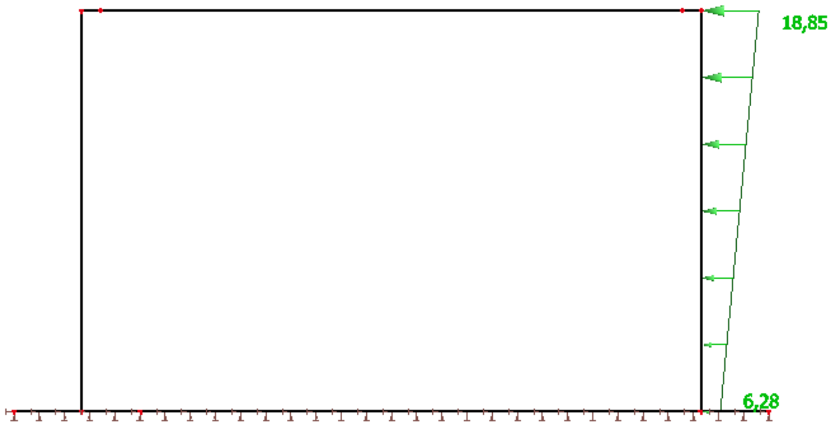

Figure 3.21

3.4.9. LC9 – Water pressure (only for undrained tunnel calculation)

Pressure on the roof slab

$q_A = \gamma_w \times z = 10 \times 6.5 = 65\ kPa$

Pressure on the side walls

$q_{top} = q_A = 65\ kPa$

$q_{bottom} = \gamma_w \times z = 10 \times 13.85 = 138.5\ kPa$

Pressure on footing

$q_B = q_{bottom} = 138.5\ kPa$

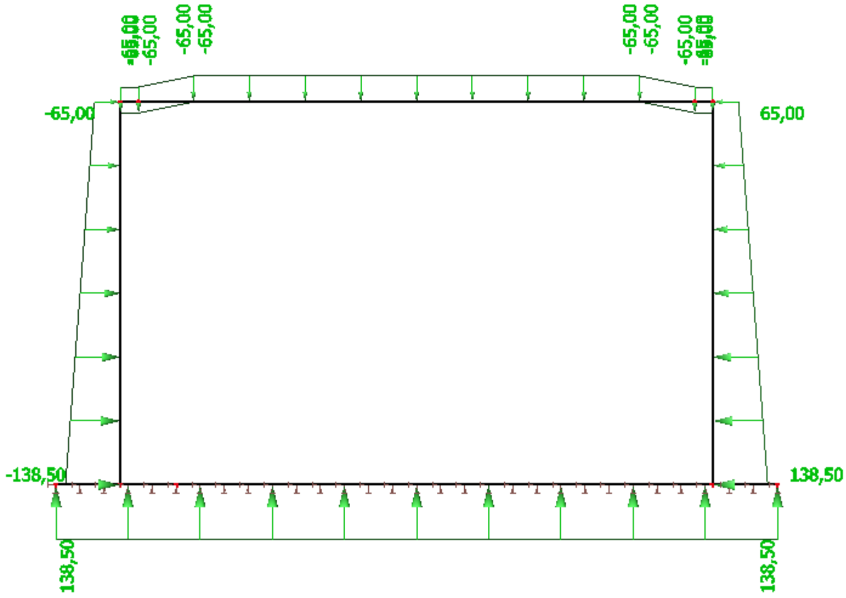

Figure 3.22

3.4.10. LC10 – Construction load

$q = 20\ kPa$

$q_d = \gamma \times q = 1.5 \times 20 = 30\ kPa$

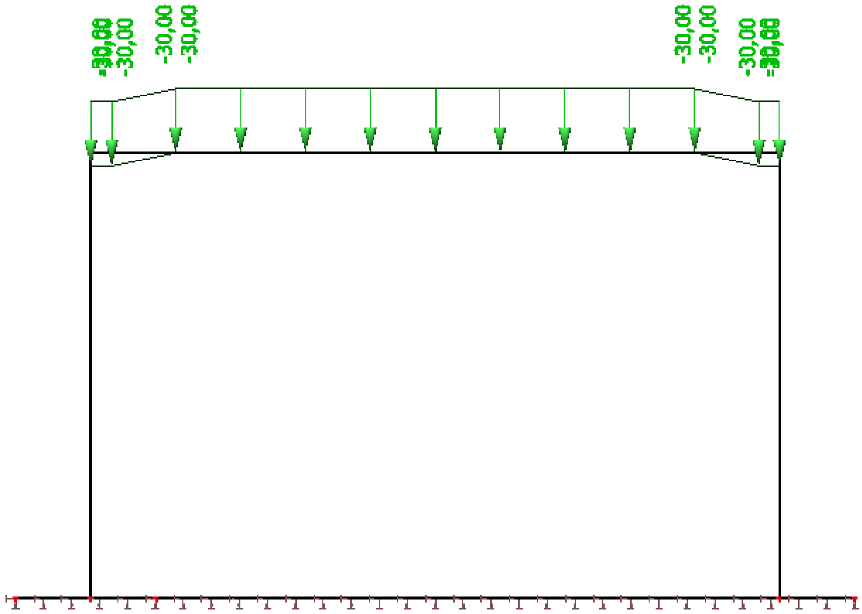

Figure 3.23

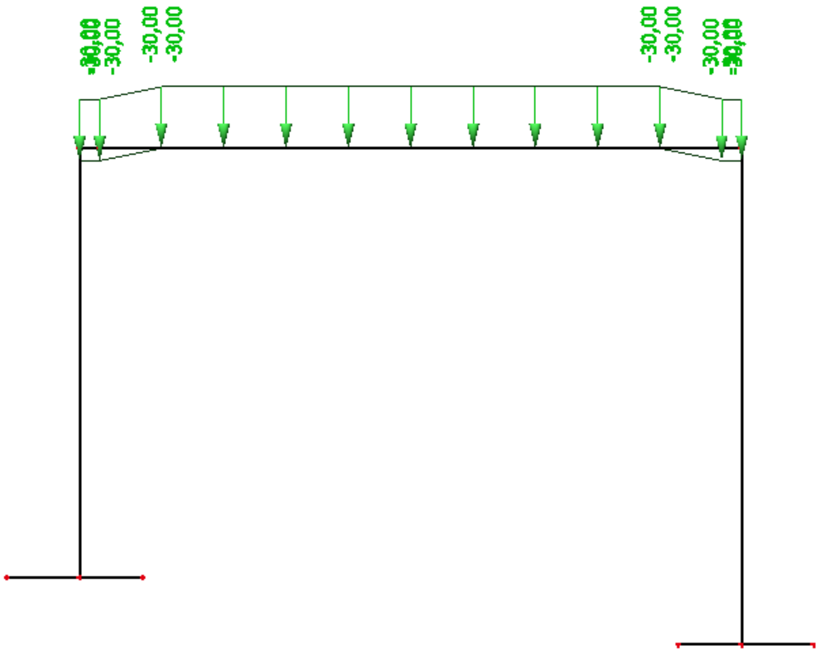

Figure 3.24

3.5. Combinations

The combinations were created according to Eurocode 0.

Table 3.3 Combinations [6] [5]

NONLINEAR COMBINATIONS FOR ULTIMATE LIMIT STATE (Drained Tunnel)

COMBINATION	TYPE	LC1	LC2	LC3	LC4	LC6a	LC6b	LC6c	LC6d	LC7	LC8	LC10
NC1	6.10.a	1.35										0.70
NC2	6.10.a	1.35	1.00									0.70
NC3	6.10.a	1.35		1.00	1.00	0.70				1.05		
NC4	6.10.a	1.35		1.00	1.00	0.70	0.70			1.05		
NC5	6.10.a	1.35		1.00	1.00	0.70		0.70		1.05		
NC6	6.10.a	1.35		1.00	1.00	0.70			0.70	1.05		
NC7	6.10.b	1.15										1.00
NC8	6.10.b	1.15	1.00									1.00
NC9	6.10.b	1.15		1.00	0.89	1.00				1.05		
NC10	6.10.b	1.15		1.00	0.89	1.00	1.00			1.05		
NC11	6.10.b	1.15		1.00	0.89	1.00		1.00		1.05		
NC12	6.10.b	1.15		1.00	0.89	1.00			1.00	1.05		
NC13	6.12.	1.00		1.00	0.74	0.67	0.77				1.00	
NC14	6.12.	1.00		1.00	0.74	0.67		0.77			1.00	
NC15	6.12.	1.00		1.00	0.74	0.67			0.77		1.00	

Load case 5 – the load from traffic is not considered for combinations, because load case 6–surface surcharge has a greater impact.

3.6. Internal Forces

3.6.1. Axial Forces Envelope

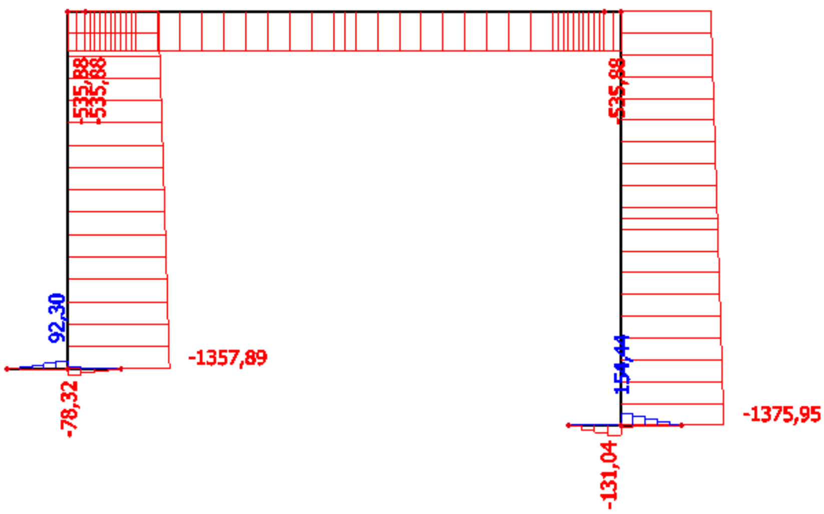

Figure 3.25

3.6.2. Shear Forces Envelope

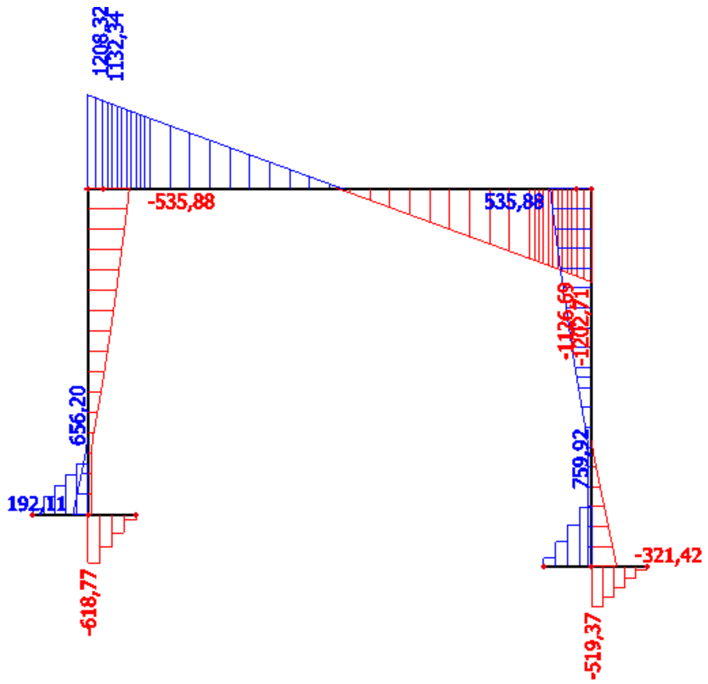

Figure 3.26

3.6.3. Bending Moment Envelope

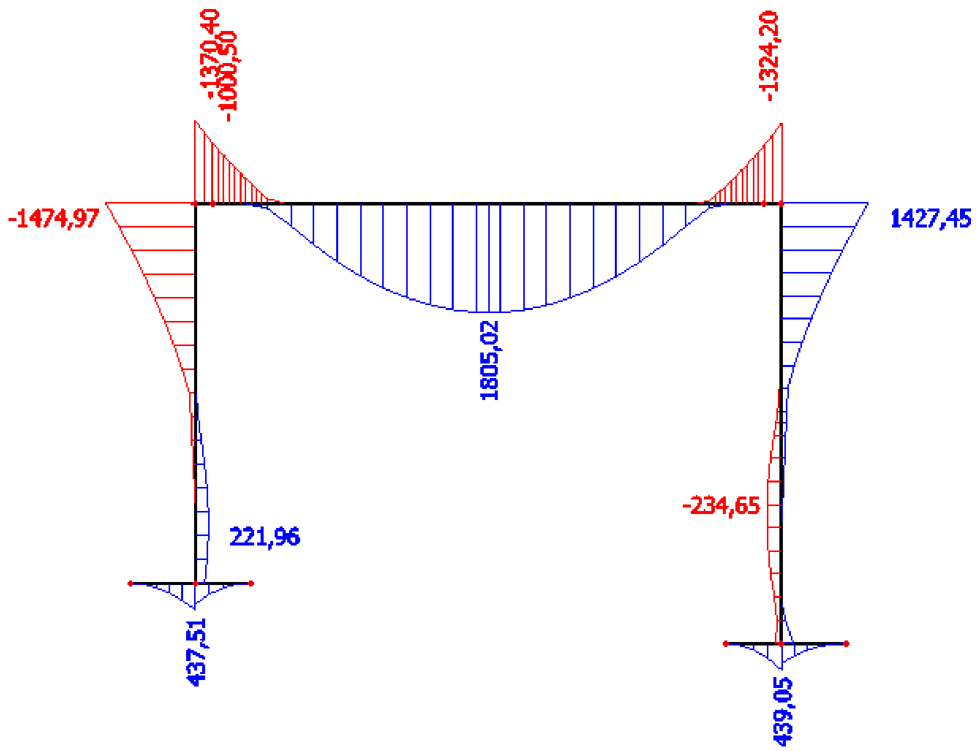

Figure 3.27

3.7. Reinforcement Design – Bending with Axial Force

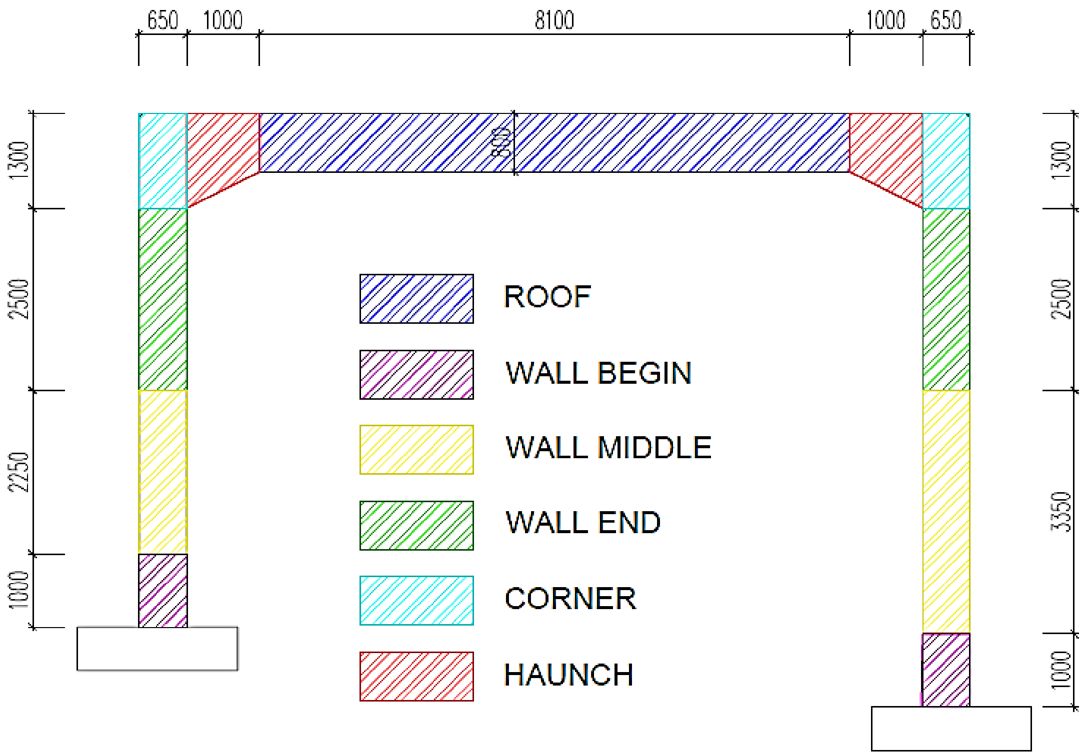

Figure 3.28 Check Sections

Design is presented by M+N Interaction Diagrams.

Table 3.4 Material Characteristics

Concrete

Compressive strength	f_{ck}	45	MPa
Partial safety factor	γ_c	1.5	
Concrete factor	α_{cc}	1	
Design compressive strength	f_{cd}	30	MPa

Steel

Yield strength	f_{yk}	550	MPa
Partial safety factor	γ_s	1.15	
Elastic modulus	E_s	200	GPa
Design yield strength	f_{yd}	478.2609	MPa
Factored yield strain	ε_{yd}	2.39	‰
Maximum compressive strain	ε_{cu2}	3.5	‰
Strain at reaching maximum strength	ε_{c2}	2	‰

3.7.1. Roof

The decisive section of the roof is in the middle of the span, where bottom fibres are tensioned.

Table 3.5 Roof - Interaction Diagram characteristics

Sections characteristics

Width of the section	b	1	m
Depth of the section	h	0.8	m

Bars

	Tensile			Compressive		
Diameter	Φ	40	mm	Φ	20	mm
Spacing	s	200	mm	s	200	mm
Cover	c	75	mm	c	75	mm
Steel Area	A_{st}	0.006283	m^2	A_{sc}	0.001571	m^2

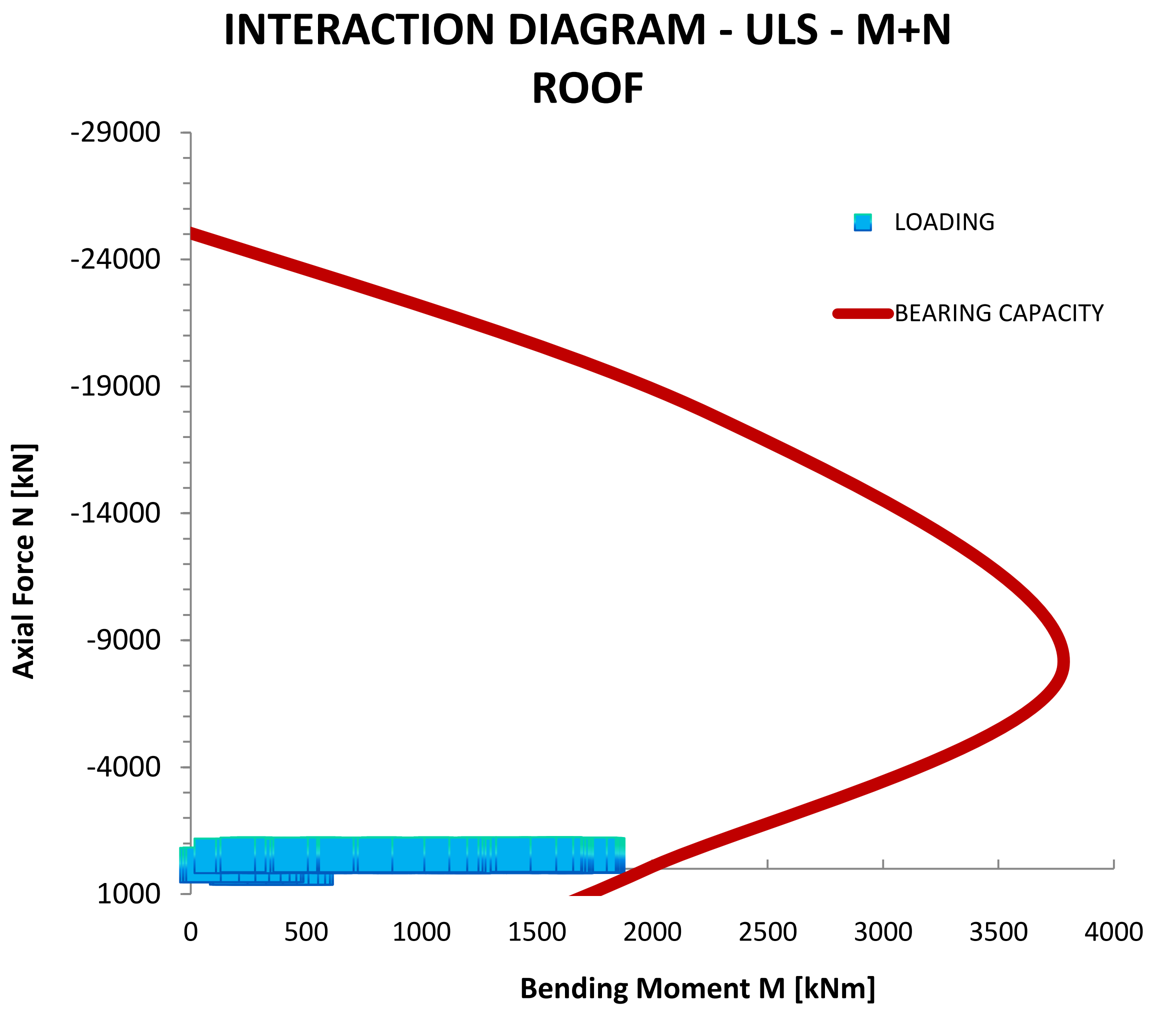

Figure 3.29 Interaction Diagram - Roof

3.7.2. Wall bottom

Table 3.6 Wall bottom - Interaction Diagram Characteristics

Sections characteristics

Width of the section	b	1	m
Depth of the section	h	0.65	m

Bars

	Tensile			Compressive		
Diameter	Φ	20	mm	Φ	20	mm
Spacing	s	200	mm	s	200	mm
Cover	c	75	mm	c	75	mm
Steel Area	A_{st}	0.001571	m^2	A_{sc}	0.001571	m^2

For interaction diagram see Annex B.

3.7.3. Wall Middle

Table 3.7 Wall Middle - Interaction Diagram characteristics

Sections characteristics

Width of the section	b	1	m
Depth of the section	h	0.65	m

Bars

	Tensile			Compressive		
Diameter	Φ	20	mm	Φ	20	mm
Spacing	s	200	mm	s	200	mm
Cover	c	75	mm	c	75	mm
Steel Area	A_{st}	0.001571	m^2	A_{sc}	0.001571	m^2

For interaction diagram see Annex B.

3.7.1. Wall End

Table 3.8 Wall End - Interaction Diagram characteristics

Sections characteristics

Width of the section	b	1	m
Depth of the section	h	0.65	m

Bars

		Tensile			Compressive	
Diameter	Φ	40	mm	Φ	20	mm
Spacing	s	200	mm	s	200	mm
Cover	c	75	mm	c	75	mm
Steel Area	A_{st}	0.006283	m^2	A_{sc}	0.001571	m^2

For interaction diagram see Annex B.

3.7.2. Corner

Table 3.9 Corner - Interaction Diagram characteristics

Sections characteristics

Width of the section	b	1	m
Depth of the section	h	1.3	m

Bars

		Tensile			Compressive	
Diameter	Φ	40	mm	Φ	20	mm
Spacing	s	200	mm	s	200	mm
Cover	c	75	mm	c	75	mm
Steel Area	A_{st}	0.006283	m^2	A_{sc}	0.001571	m^2

For interaction diagram see Annex B.

3.7.3. Haunch

Table 3.10 Haunch - Interaction Diagram characteristics

Sections characteristics

Width of the section	b	1	m
Depth of the section	h	0.8	m

Bars

		Tensile			Compressive	
Diameter	Φ	40	mm	Φ	20	mm
Spacing	s	200	mm	s	200	mm
Cover	c	75	mm	c	75	mm
Steel Area	A_{st}	0.006283	m^2	A_{sc}	0.001571	m^2

For interaction diagram see Annex B.

3.8. Reinforcement Design – Shear

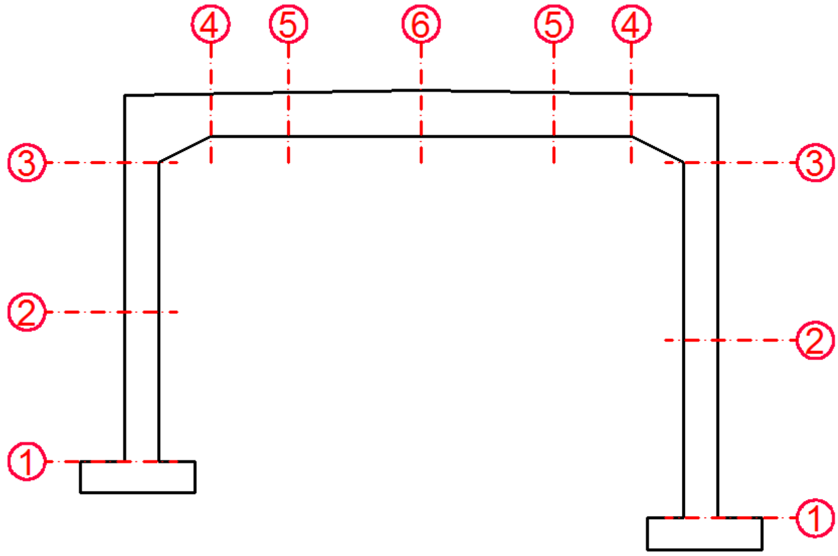

Figure 3.30 Critical Sections

Shear is calculated according to [8] and Eurode 2. [9]

Table 3.11 Concrete Characteristics

Concrete			
Concrete strength	f_{ck}	45	MPa
Concrete factor	γ_c	1.5	
Partial safety factor	α_{cc}	1	
Design Concrete srength	f_{cd}	30	MPa
Max contributing component	σ_{cp}	6	MPa

3.8.1. Section 1 – Wall bottom

Table 3.12 Wall bottom - Shear-thrust characteristics

Sections characteristics		
Depth of the section	h	0.65
Width of the section	b	1

Concrete only - shear resistance

coefficient	C_{rdc}	0.12	
coefficient	k	1.59	
coefficient	k_1	0.15	
Min Shear resistance	v_{min}	0.473	MPa
Tensile reinforcement ratio	ϱ	0.0024	
Concrete only - shear resistance	V_{rdcm}	239.58	kN
$V_{rdcm} \geq v_{min}$*b*d	=	**267.245**	kN

Shear reinforcement

Diameter	Φ	10	mm
Number of links in one row	n	5	
Spacing of rows	s	300	mm
Yield strength	f_{yw}	500	MPa
Partial safety factor	γ_s	1.15	
Design strength of shear-links	f_{ywd}	434.7826	MPa
Area of shear reinforcement	A_{sw}	392.6991	mm^2
Shear reinforcement ratio	ϱ_w	0.001309	
Minimum Shear reinf. ratio	ϱ_{wmin}	0.001073	**OK**
Strength reduction factor	v	0.492	
Inclination of compression strut	cot θ	2.5	

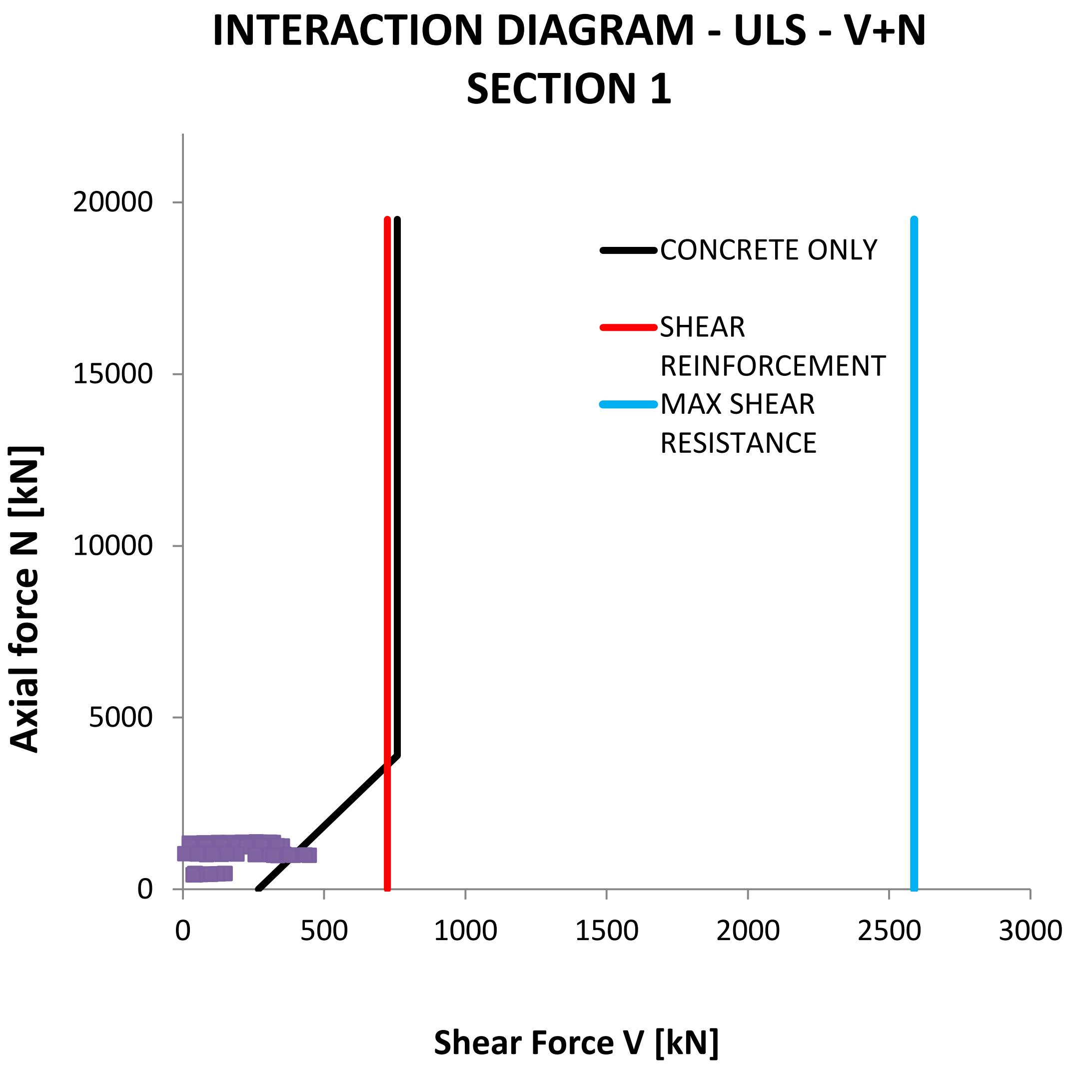

Figure 3.31 Shear-thrust Interaction Diagram- Section 1

3.8.2. Section 2 – Wall Middle

Table 3.13 Wall Middle - Shear-thrust characteristics

Sections characteristics		
Depth of the section	h	0.65
Width of the section	b	1

Concrete only - shear resistance			
coefficient	C_{rdc}	0.12	
coefficient	k	1.59	
coefficient	k_1	0.15	
Min Shear resistance	v_{min}	0.473	MPa
	ϱ	0.0024	
Concrete only - shear resistance	V_{rdcm}	239.58	kN
$V_{rdcm} \geq v_{min}*b*d$	=	**267.245**	kN

For interaction diagram see Annex B.

3.8.3. Section 3 – Wall Top

Table 3.14 Wall Top - Shear-thrust characteristics

Sections characteristics		
Depth of the section	h	0.65
Width of the section	b	1

Concrete only - shear resistance			
coefficient	C_{rdc}	0.12	
coefficient	k	1.59	
coefficient	k_1	0.15	
Min Shear resistance	v_{min}	0.473	MPa
	ϱ	0.0097	
Concrete only - shear resistance	V_{rdcm}	**373.58**	kN
$V_{rdcm} \geq v_{min}*b*d$	=	262.515	kN

Shear reinforcement			
Diameter	Φ	10	mm
Number of links in one row	n	5	
Spacing of rows	s	300	mm
Yield strength	f_{yw}	500	MPa
Partial safety factor	γ_s	1.15	
Design strength of shear-links	f_{ywd}	434.7826	MPa
Area of shear reinforcement	A_{sw}	392.6991	mm^2
Shear reinforcement ratio	ϱ_w	0.001309	
Minimum Shear reinf. ratio	ϱ_{wmin}	0.001073	**OK**
Strength reduction factor	v	0.492	
Inclination of compression strut	cot θ	2.5	

For interaction diagram see Annex B.

3.8.4. Section 4 – Roof Ends

Table 3.15 Roof Ends - Shear-thrust characteristics

Sections characteristics

Depth of the section	h	0.8
Width of the section	b	1

Concrete only - shear resistance

coefficient	C_{rdc}	0.12	
coefficient	k	1.53	
coefficient	k_1	0.15	
Min Shear resistance	v_{min}	0.444	MPa
	ϱ	0.0079	
Concrete only - shear resistance	V_{rdcm}	**424.47**	kN
$V_{rdcm} \geq v_{min}$*b*d	=	313.02	kN

Shear reinforcement

Diameter	Φ	12	mm
Number of links in one row	n	5	
Spacing of rows	s	300	mm
Yield strength	f_{yw}	500	MPa
Partial safety factor	γ_s	1.15	
Design strength of shear-links	f_{ywd}	434.7826	MPa
Area of shear reinforcement	A_{sw}	565.4867	mm^2
Shear reinforcement ratio	ϱ_w	0.001885	
Minimum Shear reinf. ratio	ϱ_{wmin}	0.001073	**OK**
Strength reduction factor	v	0.492	
Inclination of compression strut	cot θ	2.5	

For interaction diagram see Annex B.

3.8.5. Section 5 and 6 – Roof Middle

Table 3.16 Roof Middle - Shear-thrust characteristics

Sections characteristics

Depth of the section	h	0.8
Width of the section	b	1

Concrete only - shear resistance

coefficient	C_{rdc}	0.12	
coefficient	k	1.53	
coefficient	k_1	0.15	
Min Shear resistance	v_{min}	0.444	MPa
	ϱ	0.0079	
Concrete only - shear resistance	V_{rdcm}	**424.47**	kN
$V_{rdcm} \geq v_{min}*b*d$	=	313.02	kN

For interaction diagram see Annex B.

Section 5 is calculated according to anchorage length 2.95 m from the beginning of the roof.

4. COMPARISON OF DRAINED AND UNDRAINED TUNNEL DESIGN ALTERNATIVES

4.1. Comparison of Internal Forces

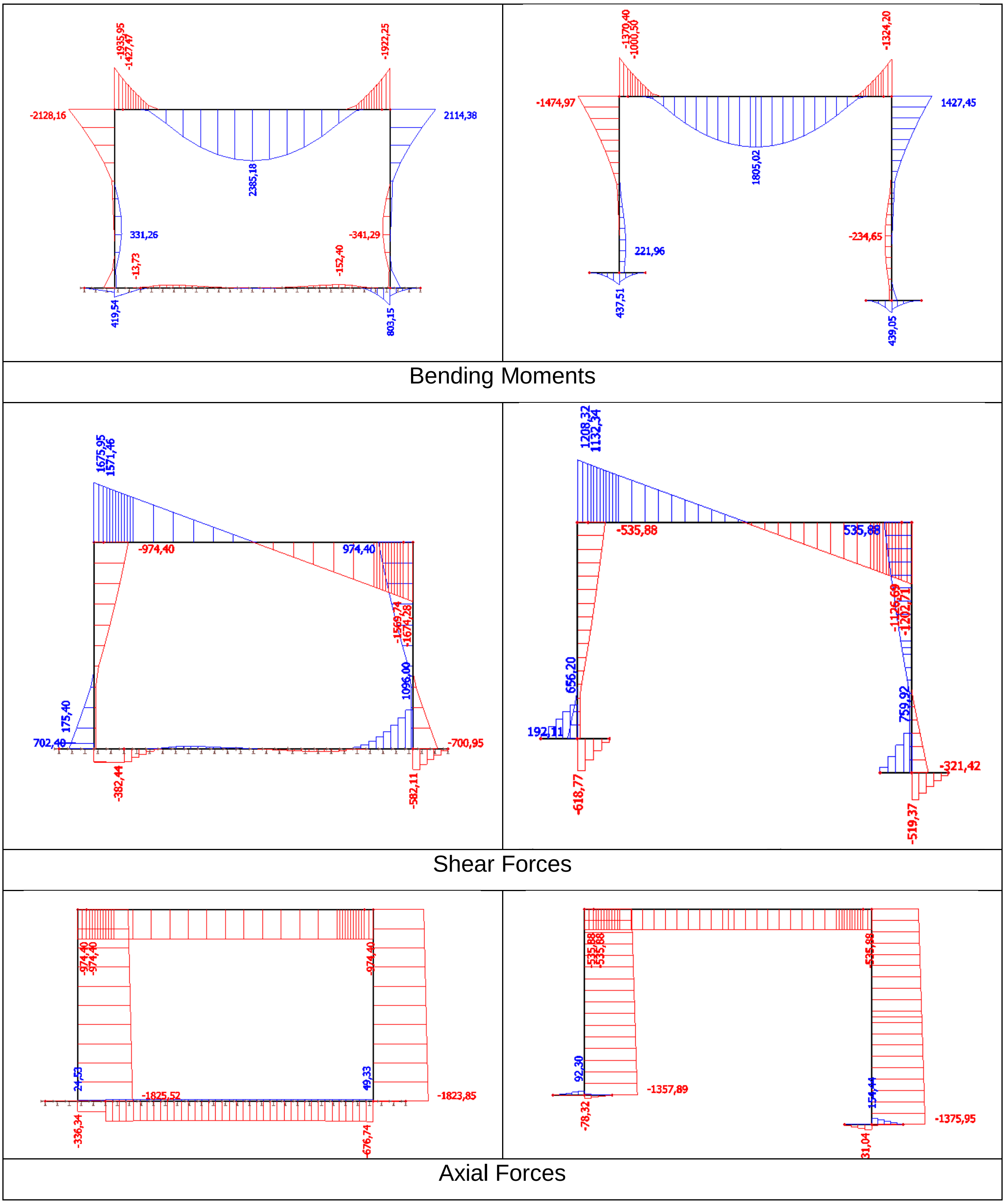

Figure 4.1 Comparison of Internal Forces

4.2. Comparison by Interaction Diagrams

4.2.1. Roof

Desired bearing capacity for undrained design is reached with same design characteristics by lowering the spacing of the reinforcement from 200 mm to 150 mm.

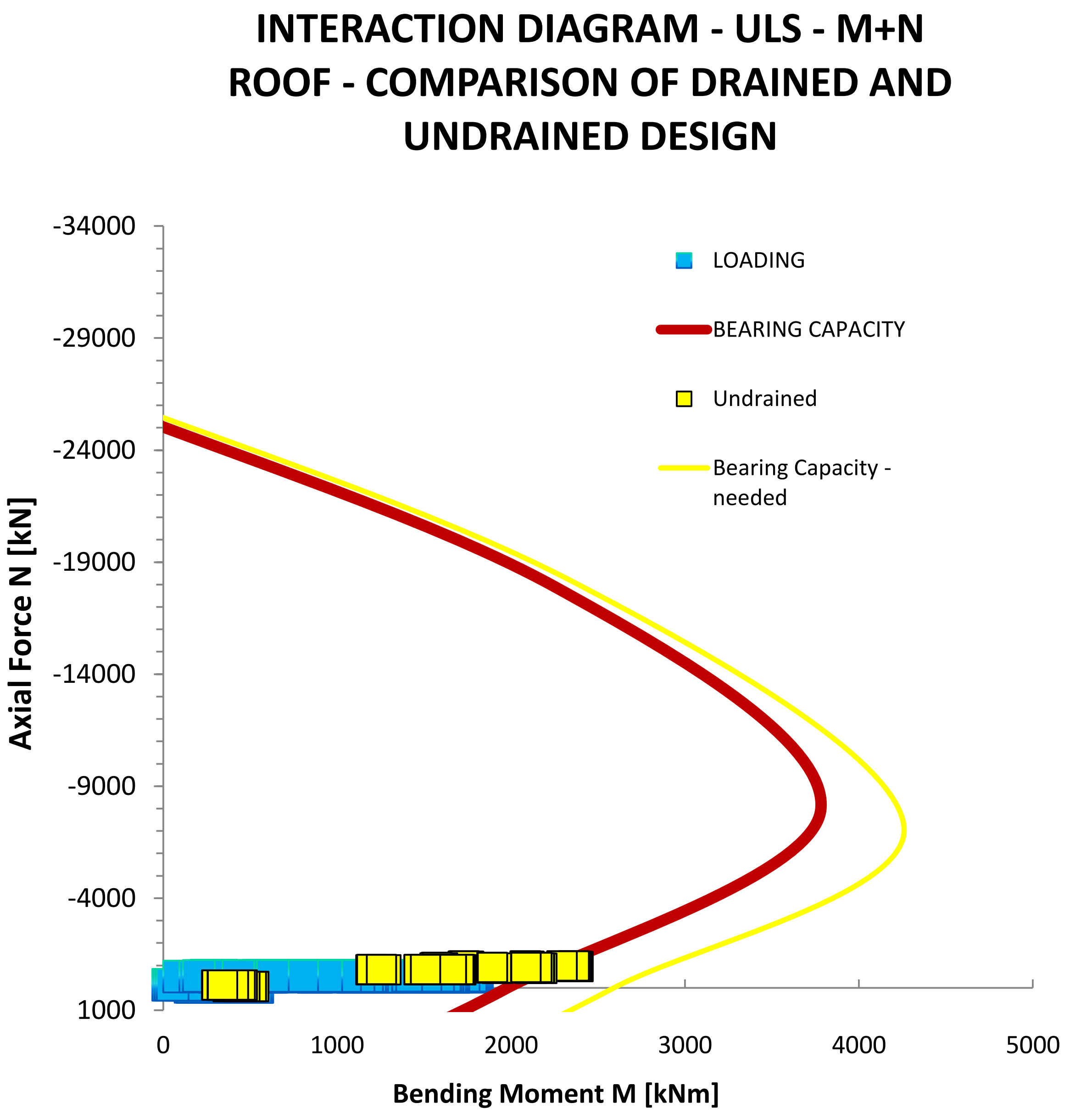

Figure 4.2 Interaction Diagram - Roof - Comparison

As expected the undrained tunnel would need more reinforcement or greater cross-sections. However the values are not unrealistic in terms of the required reinforcement.

5. REINFORCEMENT SCHEMES

For reinforcement scheme see Annex D.1.

Summary

Several final comments to my work are as follows. At first the calculation showed that the geological conditions are suitable for such a construction and that it was possible to design structure with reasonable dimensions. Secondly, the design of construction phasing affects the traffic on the road at minimum as requested and enables smooth process of tunnel erection. Furthermore, when analysing the internal forces, it could be noted that axial forces and bending moments on all members reach significant values, which results in the assessment by their interaction. Moreover, the shear reinforcement design was accomplished by the shear-thrust interaction in order to perform an economic design. The comparison showed us the possibility to protect the environment and keep the original natural habitat in the area, even though the construction would be more expensive and feasibility would be more complicated. The undrained alternative is an unusual solution in Norway and that is why the classical drained tunnel is built. However the calculation demonstrated that the design of the cross-section or reinforcement would not be unrealistic or unfeasible.

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List of short cuts and symbols

A_c	cross-sectional area of concrete
A_{st}	area of steel
c	concrete cover
c	effective ground cohesion
d	effective depth of a cross-section
E	modulus of elasticity
E_{cm}	secant modulus of elasticity of concrete
EI	bending stiffness
f_{ck}	characteristic compressive cylinder strength of concrete at 28 days
f_{cd}	design value of concrete compressive strength
f_{yk}	characteristic yield strength of reinforcement
f_{yd}	design yield strength of reinforcement
h	overall depth of a cross-section
K_0	at-rest earth pressures coefficient
α_{cc}	coefficient taking account of long term effects on the compressive strength and of unfavourable effects resulting from the way the load is applied
γ	unit weight of soil
γ_c	partial safety factor for concrete
γ_G	partial safety factor for permanent actions, *G*
γ_s	partial safety factor for reinforcing steel
ε_{c3}	compressive strain in the concrete
ε_{cu3}	ultimate compressive strain in the concrete
η	factor defining the effective strength
θ	inclination of compression strut
Φ	diameter of a reinforcing bar
v_{min}	min. shear resistance
s	spacing
LC	load case
LRT	light rail train

List of Annexes