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PAŽÍCÍ KONSTRUKCE PRO HLOUBENOU STANICI METRA V KATARU

GROUND RETAINING STRUCTURE FOR CUT & COVER METRO STATION IN QATAR

BAKALÁŘSKÁ PRÁCE
BACHELOR'S THESIS

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Podklady a literatura

Design information:
- Geotechnical Appraisal Report
- Structural and Geotechnical Design Statement
- Concrete Durability Report
- Outline Drawings

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

Software recommendations:
- FINE: GEO5: Pažení posudek
- PLAXIS 2D
- SCIA ENGINEER

Zásady pro vypracování

The objective is to undertake a simplified structural/geotechnical engineering design for the selected elements of the retaining structure for the underground cut & cover metro station box under the new international airport in Qatar.

The work on this Thesis is expected to include the following activities:
1) Assessment and summary of ground conditions;
2) Discussion on durability design (exposure to aggressive ground water);
3) Discussion on and choice of suitable retaining wall and lateral support system;
4) Outline construction sequence (bottom up or top down);
5) Simplified structural analysis and design checks for selected elements of the retaining structure (one characteristic plane strain section only);
6) General arrangement (cross-section) or reinforcement details sketches;

Deliverables:
1), 2) and 3) ... Each one section in a report.
4) Description in a report + 1 drawing.
5) Description in a report + calculation note in appendix.
6) Min 1 drawing.

Baseline information for this Thesis will be available in English. 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).

....

Michal Uhrin
Vedoucí bakalářské práce

Abstrakt

Cílem této práce je zhodnocení geologických poměrů a trvanlivosti hloubené stanice metra v Kataru. Dále se práce zabývá postupem výstavby a výpočtem vybraných prvků konstrukce. Geologické poměry byly vyhodnoceny na základě rozsáhlého geotechnického průzkumu, který byl v místě stavby proveden. Tento průzkum také poskytl nezbytné informace pro návrh trvanlivosti konstrukce. Po vyhodnocení základových poměrů bylo možné zvolit vhodný systém pažících stěn a systém rozepření a dále pokračovat s plánem postupu výstavby. Pro výpočet byla vybrána podzemní stěna a rozpěra ve střední úrovni stavby. U stěny bylo největší namáhání způsobeno ohybovými momenty o značné velikosti, proto byla stěna navržena na obálku těchto momentů získanou z programu Plaxis. Při návrhu rozpěry bylo nutné uvažovat interakci ohybových momentů a normálových sil. Pro oba tyto prvky byla navržena výztuž větších průměrů.

Klíčová slova: opěrné stěny, podzemní stěna, trvanlivost

Abstract

The aim of this thesis is to assess the geological conditions and durability of the cut and cover metro station in Qatar. Furthermore, the thesis deals with the construction sequence and structural checks of selected elements of the structure. The geological conditions were evaluated on the basis of an extensive geotechnical investigation that was conducted at the construction site. This research also gave necessary information for durability design. After the assessment of the ground conditions in the thesis, it was possible to choose suitable retaining walls and lateral support system and continue with a plan of construction sequence. A diaphragm wall and mezzanine level prop were chosen for the structural check. In case of the wall, the main load was caused by bending moments of a big significance, therefore the wall was designed to the envelope of these moments obtained from software Plaxis. Furthermore, it was necessary to consider the interaction of bending moments and normal forces for the design of the prop. For both elements, reinforcement of bigger diameters was designed.

Key words: retaining walls, diaphragm wall, durability

Bibliografická citace VŠKP

Veronika Kočičková *Pažící konstrukce pro hloubenou stanici metra v Kataru.* Brno, 2015. 48 s., 4 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 29.5.2015

………………………………………………………
podpis autora
Veronika Kočičková

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.

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Introduction

The thesis deals with a real structure of a cut and cover metro station in Qatar. The geological and geotechnical conditions of this structure are described in the first chapter. The groundwater is also taken in consideration in this thesis. One of the major concerns for the structure is durability. A design life time for the metro station is 120 years. The structures in Arabian Peninsula are exposed to very aggressive groundwater and environment in general and that might be a problem. It is necessary to choose the suitable retaining walls and lateral support system and to design them with consideration of the aggressive environment. Furthermore the structure has to bear the applied loads. The structural check of diaphragm wall is carried out with help of software Plaxis, according to Eurocodes. For the design of mezzanine level prop softwares Plaxis and SCIA Engineer are used.

1 GEOLOGY

1.2 Qatar Peninsula

Qatar is an extended peninsula on the Arabian Peninsula pointing northwards into the Persian Gulf. Geological composition is mostly made of Made fill, Simsima Limestone, Midra shale and Rus formations.

Simsima limestone forms most of the surface; it is irregularly layered and contains cavities and fissures. The thickness ranges between 30-50 m.

Rus formations are usually composed of soft, dolomitic or chalky limestone, gypsum, anhydrite and shale. The thickness of this layer varies between 42-112 m.

There is a large system of karsts in the Qatar peninsula and they include depressions, sinkholes, caves and solution hollows. These discontinuities may lead to local strong groundwater flows or to a connection between two water sources. All sinkholes of Qatar occur within the Simsima Limestone and are concentrated in the central and northern parts of Qatar. Most depressions are related to the dissolution of gypsum and anhydrite within the Rus formation, resulting in the development of numerous surface-collapse depressions. [1] [2]

Figure 1.1 Location of Qatar [9]

1.3 The New Doha International Airport (NDIA)

NDIA is situated in the East of Doha at Qatar Peninsula. During the design phase there was conducted a large site investigation in the place of the terminal station. This station is situated in the South of the airport, with a man-made lagoon in the South, terminal access roads in the East and West. The site investigation included 24 boreholes, 14 cone penetration tests, standpipe/piezometers and observation well installations, falling head and packer in-situ permeability testing, pumping tests and groundwater sampling. Extensive in-situ and lab sample testing was undertaken.

Figure 1.2 Site investigation location [3]

According to the results of the investigation the ground composition is:

- Made ground / Hydraulic fill - medium to very dense sand
- Silt / Marine deposits – loose / very soft sandy silt
- Caprock – weak to moderately weak calcerenite
- Distinctly weathered Simsima Limestone – dolomitic limestone recovered as granular material during investigation

- Partially weathered Limestone – moderately weak limestone
- Midra Shale – mudstone with variable strength
- Rus Formation – weak chalky limestone

Thickness and top levels of the layers vary across the investigation site.

- Made ground

The top of the Made ground creates the ground surface. The reclamation for the airport ground began in 2005. The original seabed was uneven and that explains the thickness variety of the made ground layer – it varies between 2 m and 6 m.

- Silt / Marine deposits

This unit is not persistent across the investigation site. Where it was found, it had the maximum thickness of 1 m.

- Caprock

This is a geological term for a harder rock overlying a weaker rock. The thickness of the layer varies between 0.65 m and 6.17 m. In many boreholes, Caprock was found with many shell fragments.

- Distinctly weathered Simsima Limestone

The thickness varies between 13.4 m and 22.65 m. It was particularly difficult to obtain a solid core recovery during the investigation. For the most part this stratum was recovered as subangular to subrounded gravel-sized fragments set within sand/silt. The matrix was frequently poorly cemented or indurated and as a result it was washed out during drilling.

- Partially weathered Limestone

The thickness varies between 5.30 m and 12.45 m. The less crystalline nature of this zone suggests that it has not undergone diagenesis to the same degree as the overlying Distinctly weathered Simsima Limestone.

- Midra shale

The thickness varies between 1.5 m and 5.05 m. The borehole records show that this is apparently horizontally bedded mudstone with occasional fossils. The mudstone horizons are generally intercalated with beds of moderately strong chalky and crystalline dolomitic limestone. Layers of gypsum bands and pockets of gypsum were also observed.

- Rus Formation

Chalky limestone belonging to the Rus Formation was encountered until the termination depth of all boreholes. The minimum proven thickness of the Rus Formation is 13.25m.

No cavities were encountered during the site investigation, and it is believed that no cavities have been reported elsewhere across the wider NDIA site. [3]

1.4 Groundwater

Results of the site investigation indicate that the groundwater level is typically at +0.5m QNHD (Qatar National Height Datum) which is approximately 2.5m below the average existing ground level across the site. The lowest and highest recorded ground water level across the site investigation area was -0.88m QNHD and 1.50m QNHD.

The station box site is directly adjacent to the airport lagoons which are connected to the Arabian Gulf. That is why it was expected that there are tidal groundwater variations. The lowest and the highest recorded level in the piezometry installed generally fall into two range of tidal levels for the site, and therefore indicate that the groundwater level rises and falls with the tide.

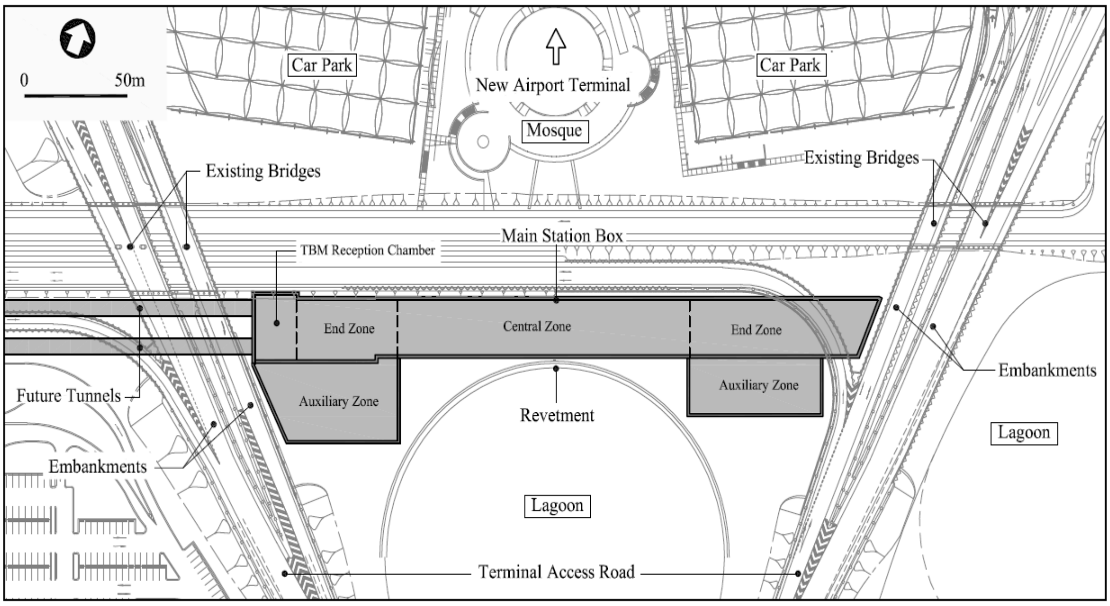


Figure 1.3 Station box and its surroundings [3]

The Caprock and Distinctly Weathered Simsima Limestone are considered to be highly permeable strata. The Partially Weathered Simsima Limestone and the Midra Shale – are considered to be considerably less permeable.

1.4.1 Groundwater aggressiveness

Water aggressiveness for buried structures in the Gulf region is usually elevated. The close distance between the lagoons and station box even more increases the risk because of dissolved salts in the groundwater.

Ground and groundwater aggressiveness testing has been included in the site investigation for the station box site. The results show that pH in the groundwater is between 6.13 and 9.48 (neutral pH is 7, if we more than 7, water is alkaline, less than 7, water is acidic). There are also compounds of sulphur and chlorine at very high level. Furthermore, a high ambient temperature can lead to a faster rate of chemical reactions between these chemicals and concrete.

The aggressiveness tests results are summarised below:

- Sulphur as SO_4 in groundwater samples ranged between 1 625 mg/l and 3 698 mg/l
- pH in soil samples ranged between 8.2 and 9.8
- pH in groundwater samples ranged between 6.13 and 9.48
- Chlorine as Cl in groundwater samples ranged between 23 397 mg/l and 26 977 mg/l (typical Cl level in the Arabian Gulf is approximately 24,000 mg/l) [3]

1.5 Geotechnical parameters

Drained analyses were undertaken for the design. Characteristic values were adopted as stated in Table 1. The appropriate partial factors were applied to these to obtain design values.

Table 1.1: Characteristic Geotechnical Parameters

Strata	γ [kN/m^3]	φ' [^{o}]	c' [kPa]	E'[MPa]
Hydraulic Fill	17.5	36	0	40
Caprock	19.5	36	0	50
Simsima Limestone	19.5 - 20.5	40	5 - 25*	40 - 150*
Midra Shale	17.0	35	100	100
Rus Formation	19.5	40	25	180

* varying with depth/degree of weathering

2 DURABILITY

Durability was one of the main problems, considered that many constructions in the Gulf region do not function until the end of their designed life time.

A design life time for the metro station in Doha is 120 years. The construction is built in a saline environment, next to the lagoon. High levels of both sulphates and chlorides were measured in the groundwater during the site investigation. Exposure conditions according to EN 206-1 [4] vary from XA1 to XA3 for chemical activity, from XC2 to XC4 for corrosion caused by carbonation, depending on the location according the lagoon and tidal zone. These conditions are also different for the internal and external site of the structure.

For underground structures the main factors influencing the durability are water and soil aggressiveness, for deep constructions pressure of the underground water and stray currents. Other factors can be for example aggressive compounds contained in gravel, cement or in mixing water.

2.1 Corrosion

There are different ways of corrosion - freezing cycles, carbonation, reinforcement corrosion, corrosion without the access of air, influence of chlorides, acidic water, sulphur aggressiveness, carbon aggressiveness, alkali reaction of gravel, stray currents.

- Carbonation

It only occurs in the air in the presence of humidity. It cannot be found in the fully saturated environment or in completely dry environment. It lowers the pH value of the concrete and it proceeds from the surface to the deeper parts of the structure. In a certain time it gets to the reinforcement and lowers the pH value of the concrete below 9, then the concrete stops protecting the reinforcement and its corrosion starts.

$Ca(OH)_2 + CO_2 \rightarrow CaCO_3 + H_2O$

Neutralisation - reaction of atmospheric/air CO_2 with binder.

- Influence of chlorides

Chloride ions can be found in the ground (aggressive environment) or directly in concrete aggregates. The ions move in pore water of a concrete matrix. Passive protection of the reinforcement by concrete is destroyed if the level of ions reaches the critical limit. This type of corrosion is very fast and leads to huge losses of the surface area of the reinforcement.

- Sulphur aggressiveness

This type of corrosion can be caused by different types of minerals contained in rocks or water. SO_4^{2-} ions react with the cement paste and usually form plaster or etringite. These products can have 5 times higher volume then the original compound.

- Carbon aggressiveness

The mechanism is almost the same as in case of carbonation but it is more intensive and has higher destructive effects. It is caused by the reaction of CO_2 with Ca contained in cement matrix. The product of the reactions ($Ca(HCO_3)_2$) is dissolved in water and washed out. This type of corrosion leads to porosity of concrete and loss of binder, it lowers the pH value of the concrete.

$$Ca(OH)_2 + CO_2 \rightarrow CaCO_3 + H_2O$$
$$CaCO_3 + CO_2 \rightarrow Ca(HCO_3)_2$$

- Stray currents

It is usually electric current propagated in conductive environment (moisturised soil, underground water or rock). The sources can be electrified railway line, cathodic protection stations, etc. Stray currents occur mostly when there is DC (direct current). It causes intensive electro-chemical reinforcement corrosion.

There are primary, secondary and special measures against corrosion.

- Primary measures

A. Correct way of construction

The quality of the structure depends on many factors. The most obvious ones are mixing the concrete, delivery of the concrete mix from plant to construction site, casting of the concrete mix, compaction of the concrete and treatment of the concrete.

B. Design of the structure, structural principles

Sufficient thickness of the covering layer, limitation of cracks, paying attention to details.

C. Mix design

Concrete class, water-cement ratio, cement type, amount and maximum size of aggregates and fine particles, hydraulic compounds and other additives.

- Secondary measures

Tanking, different types of tiling, coating etc.

- Special measures

Migrating inhibitors of the corrosion, cathodic protection, non-ferrous reinforcement, non-corrodible reinforcement (made of stainless steel, etc.).

Cathodic protection

Electric current is induced to the system in the way that the structure acts as cathode. Corrosion runs only on anode (positive plate). There are two basic systems - system of given anode and system of imposed/forced current.

System of given anode

Elements systematically put to the ground in close proximity of a structure that are unprotected and less resistant to the corrosion than steel reinforcement, electrically connected to the reinforcement through prepared points. The reinforcement should be electrically connected by welding.

System of imposed/forced current

Rectified electrical current is induced to the reinforcement through anode system that is connected to a surface of a concrete.

These systems can be activated after years or decades of a structure life, but it is necessary to have the electrical connections of the reinforcement prepared and to have measure points (indicate the start of the corrosion).

It is possible to distinguish the protection measures as active and passive. The only active method of protection mentioned in this thesis is the cathodic protection.

2.2 NDIA

Primary protection measures are crucial for this structure. It is necessary to have qualified work force and a good quality control on the site and also in the mixing plant. Concrete mix should be designed taking account of the aggressive water and ground environment with high amount of sulphurs and chlorides but also with consideration of CO_2 ions in the air. Also the concrete cover of reinforcement bars should be sufficient to prevent carbonation, corrosion caused by chlorides, sulphurs or $CO_{2.}$

Secondary protection measures are not very practical for the whole structure. It is possible to tank the roof slab and base slab but it would be very expensive and labour- and time-consuming to tank the diaphragm walls, moreover to tank the bored piles. The insulation for the slabs should be adhesive and carried out in a very precise and careful way so that there are no holes for the water to get in between the insulation and structure.

There are many special measures that could be used at every structure but the question is if they are effective. For example, the effectiveness of migrating inhibitors of corrosion is being discussed. They are designed to protect the reinforcement only but they could also affect other qualities of concrete (they could lower the strength, slow down setting of concrete etc.). The non-ferrous reinforcement (for example made of glass or carbon) has good strength qualities but the experience with this type of reinforcement does not give us enough information about the durability and resistance in long term conditions. Good choice would be the non-corrodible reinforcement, but the price is very high when considered the amount of reinforcement that is needed. The best option of special measures is the cathodic protection against stray currents.

Considering all the factors affecting the durability of the structure, following construction measures are to be taken:

The diaphragm walls require a nominal cover of 120 mm on the soil face and 50 mm on the excavated face. The design crack width is 0.15 mm on the soil face and 0.25 mm on the excavated face. [3] The provision for cathodic protection is to be made. Corrosion monitoring is also required and will be used to determine if and when the cathodic protection system needs to be activated. Grade C40/50 concrete has been specified for all structural concrete. The minimum cement content of all structural concrete in contact with the ground is 380kg/m3. A maximum free water/cement ratio of 0.40 is required for all structural concrete except for the bored piles were a maximum ratio of 0.35 is required. Cement types and combinations are to be with double or triple blends of Portland cement, ground granulated blast furnace slag, pulverised fly ash and/or silica fume. The top of the roof slab and underside of the base slab are to be tanked using a proprietary membrane system.

3 CHOICE OF SUITABLE RETAINING WALL AND LATERAL SUPPORT SYSTEM

There are few key factors for the choice of suitable method of construction. One of them is the area on the surface that we can use for the construction. We need space for placing the structure plus the space needed for building machines, construction equipment, etc. That leads us to the second factor – dimensions of the structure (depth and dimensions of the floor plan). The third and very important factor is water – this includes the underground water and the surface water that can be in a close proximity of the construction. The fourth factor is the space we need inside of the structure.

Furthermore, it is necessary to decide whether the retaining walls will be permanent or only temporary. In case that the construction pit is under the groundwater level, feasibility of dewatering should be considered. Moreover, when deep construction pits are made, the lateral support system is usually needed.

There are many possibilities to choose from, but not all of them are suitable for NDIA.

3.1 Construction of retaining walls

3.1.1 Slope work

The excavation pit is made by steps. The sides of the pit are in slope (depends on the type of soil or rock) and at regular depth distances there should be horizontal benches. It is necessary to check the slope stability for pits of a depth bigger than 6 m.

NDIA's base slab is in the depth of approximately 20 m. This system of construction would be very land intensive and there is not much space around the planned structure. Also, the structure is under the groundwater level and it would probably be unfeasible to dewater the construction pit.

3.1.2 Nailing / anchoring

It is possible to nail or anchor the vertical (or almost vertical) construction pits. It is done by horizontal boreholes that are mostly reinforced by bars made of steel and filled with cement grout. The bars are in the head connected to the reinforcing mesh for sprayed concrete. It is necessary to drain the back side of the wall.

It would be necessary to have a very effective drainage system to dewater the NDIA site and also a lot of anchors/nails would have to be made. Nailing would also be risky because of the depth of the structure and because of the water.

3.1.3 Soldier pile wall

It is usually used in cohesive soils. At first, the steel profiles (I, 2xU, HEB) are placed in the ground – they can be placed to boreholes or driven. Then every few meters of excavation lagging (made of wood or concrete panels) is placed. It can also be anchored. This type of retaining walls is only temporary.

These two methods can only be used in case the groundwater level is under the bottom of the pit or just a bit higher than the bottom (so that we can dewater it easily).

3.1.4 Pile wall

- Contiguous (a = distance > d = pile diameter)

Piles are bored in a certain distance (for example by 2 m) and the space between them is secured by walls made of sprayed concrete placed while excavating the pit. It is not possible to make these walls impermeable.

- Tangent piles (a = d)

At first, all the odd piles are made, then the even ones. They are made in a close distance so that they touch each other. They are not impermeable but can be drained and sprayed with concrete.

- Secant (a < d)

At first the odd piles are made, then after hardening of the odd piles the even ones are made. They are partially over – drilled with the odd piles and they are reinforced. They can be made as impermeable and in case they serve as a wall of a structure they are covered with sprayed concrete.

The secant pile wall would be the only possible one for NDIA. It could be considered as an alternative to the monolithic diaphragm walls.

3.1.5 Diaphragm wall

The construction starts after the guide walls are in place. These walls determine the exact position of the future diaphragm walls and stabilize the upper part of the excavation. The width of diaphragm walls is 400 mm up to 1 500 mm, depth can be up to 30 m (or more), panels are of width of approximately 7 m. When the excavation is made, it is secured by bentonite slurry (or other suspension) and then the reinforcement is placed. Concrete is placed from the bottom to the top. Concrete displaces the bentonite mud that has to be drawn off. The adjacent plates are sealed to be waterproof. They can be designed with temporary function or as a permanent bracing structure. The surface of the excavated face of the monolithic wall can be smoothened. They are mostly used for deep structures under the groundwater level.

The system of monolithic diaphragm walls was chosen as the best for NDIA. It is possible to use the walls as permanent and there will not be problems with the underground water.
[5] [6]

3.2 Type of lateral support system

There are two construction possibilities for cut and cover structures: bottom up and top down. In case of the bottom up method, the anchors are usually used as lateral support system for the construction pit that is excavated to its bottom. Inside of this pit, the final structure is built, and then the pit is backfilled. In case of the top down method, the retaining walls are usually part of the permanent structure. The excavation is made to the level of a roof slab of the future structure and then the slab is casted. The excavating works continue under the roof slab (in case of NDIA under the roof slab wallers and props) to the next level. In this case, the structures are mostly braced. This way of construction is faster than the bottom up method and the construction area is smaller. It is not possible to waterproof the whole structure constructed by this method and the spaces for excavation are limited.

Thus, we can anchor or brace the retaining walls of underground structures.

Anchors are made in horizontal or sloped down from horizontal in a distance given by calculation. At NDIA it would be possible to anchor one side of the structure but it might be a problem on the side of the lagoon (neither side, they would be under the groundwater level). Moreover the heads of the anchors would be visible in the finished structure and they would need a cover. Anchors could also cause problems because of land law or future construction at the NDIA site.

In case of bracing system it is necessary to place the props in the way they do not impede further construction. At NDIA there was also an architectonical demand to have a big open space inside of the station box that means the props have to be in a great distance. The system of massive permanent props in a distance of 21.6 m was chosen for NDIA.

4 CONSTRUCTION SEQUENCE

Stage 1: Installation of the diaphragm walls and bored tension piles.
Stage 2: Excavation to -0.9 m while dewatering.
Stage 3: Installation of wallers and props at the ground level (roof slab level). Additional dewatering on the outside of the diaphragm walls necessary.
Stage 4: Excavation to -8.0 m while dewatering inside of the diaphragm walls.
Stage 5: Installation of mezzanine level wallers and props.
Stage 6: Excavation to -12.5 m and installation of temporary props. Dewatering inside of the walls.
Stage 7: Excavation to -17.95 m while dewatering inside of the diaphragm walls.
Stage 8: Installation of waterproofing and casting of the base slab.
Stage 9: Removal of temporary props, installation of temporary cover to the roof openings, installation of waterproofing to the roof, landscaping to +2.83 m.

Water is always dewatered to the level -2.0 m bellow the excavation inside of the construction pit.

The construction sequence scheme was prepared in a form of joined sketches, you can find them in the appendix 1.

5 SIMPLIFIED STRUCTURAL ANALYSIS AND DESIGN CHECKS FOR SELECTED ELEMENTS OF THE RETAINIG STRUCTURE

5.1 Geometry of the structure

It is possible to see different elements of the structure in a section. The diaphragm walls are of the thickness 1.2 m and reach to the depth of -32.00 m under the groundwater level. The bored piles reach to the depth of -32.5 m or more where necessary. Its diameter is 1.2 m. The wallers at the ground level (roof slab level) are of the thickness 1.4 m in a given distance from the corner of the structure and then their thickness is reduced to 0.9 m. The prop at this level is made of reinforced concrete and it is circular with diameter 0.9 m, length is 14.7 m. The wallers at the mezzanine level are of the thickness 1.5 m and are 5.2 m long. The mezzanine level prop is also made of reinforced concrete and has a diameter 1.5 m and its length is 14.7 m. The steel temporary prop is placed at -12.5 m under the groundwater level and its diameter is 0.5 m. The base slab is casted at a depth of -17.5 m and its thickness is 1.8 m.

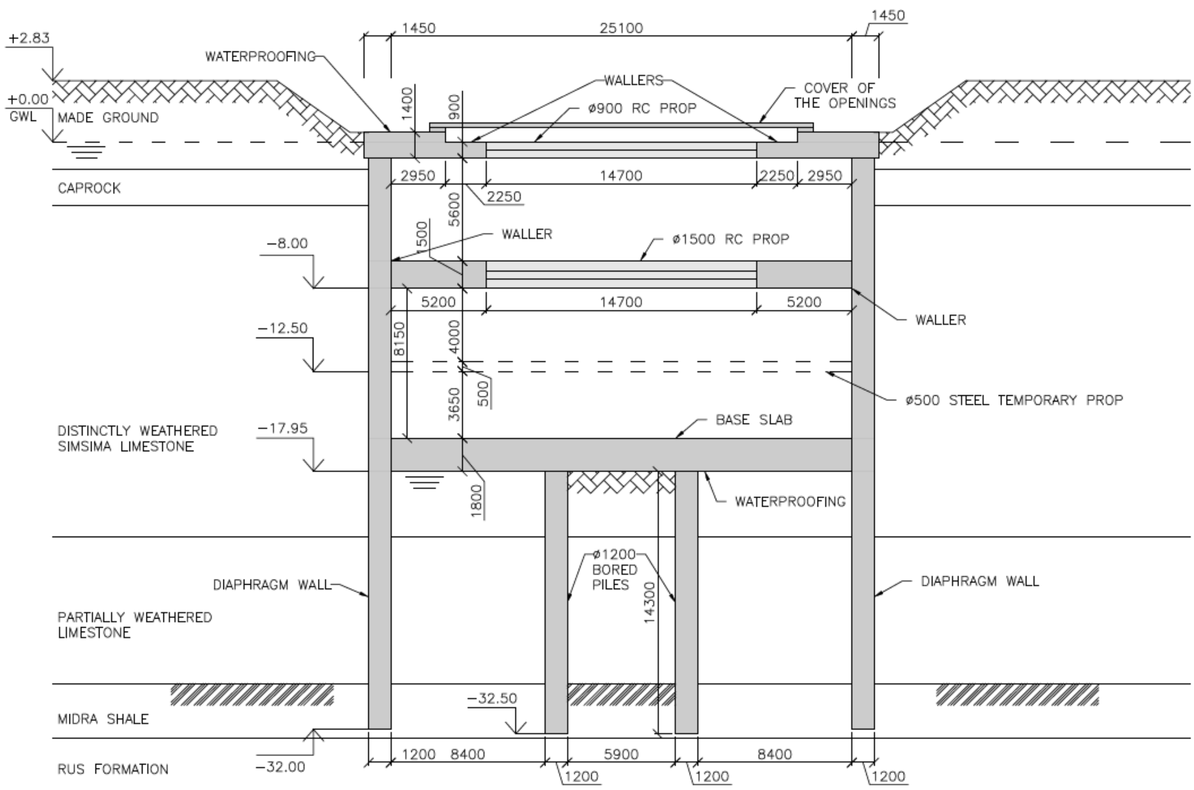


Figure 5.1 Geometry of the structure

Figure 5.1 was prepared as a cross section drawing. Please find it in the appendix 3.

5.2 Plaxis model

In order to obtain the inner forces of the structure a geotechnical model in Plaxis software was made. This programme is based on the finite element method. The Mohr – Coulomb constitutive method was used for the calculation.

5.2.1 Input values

Table 5.1: Geotechnical parameters of soils

	γ [kN/m^3]	E´ [MPa]	c'_{ref} [kPa]	φ'[°]	K_o
Made ground	17.5	40	1	36	0.5
Caprock	19.5	50	1	36	0.75
Distinctly weathered Limestone	19.5	60	7	40	0.75
Partially weathered Limestone	20.5	150	25	40	0.75
Midra shale	17	100	100	35	0.75
Rus Formation	19.5	180	25	40	0.75

Except for Midra shale strata that is set as undrained, the stratas are set as drained.

Table 5.2: Parameters of plates (structure)

	EA [kN/m]	EI [kNm^2/m]	d [m]	w [kN/m/m]	$L_{spacing}$ [m]	E [MPa]	γ [kN/m^3]
Roof slab	22.05 E6	1.488 E6	0.9	15.9	-	-	-
R. s. prop	0.721 E6	36.53 E3	0.9	0.736	-	-	-
Mezzanine level	36.75 E6	6.89 E6	1.5	44.18	-	-	-
M. l. prop	2.004 E6	281.868 E3	1.5	2.045	-	-	-
Base slab	44.1 E6	11.907 E6	1.8	-	-	-	-
Walls	29.4 E6	3.528 E6	1.2	-	-	-	-
Tension piles	-	-	1.2	-	5.4	24.5 E3	25
Temporary prop	4.81 E3	-	0.5	-	21.6		

In order to account for creep and effects of cracking the Young's modulus is reduced for short term calculation to 0.75 of the original value and to 0.5 for long term calculation. The parameters change during the lifetime of the structure – in the model it is changed after a backfill is placed on the roof slab. Parameters in the Table 5.2 are calculated for short term Young's modulus E= 24.5 MPa. All these parameters where also calculated for long term Young's modulus E=17.5 MPa.

The spacing of the roof slab prop and mezzanine level prop was taken in account when EA and EI were calculated. Values of EA and EI for sections of the props were reduced by their spacing L = 21.6 m in order to obtain these values for 1 m.

5.2.2 Model geometry

Type of a model:	Plane-strain (2D model)
Type of finite elements:	15-noded
Model dimensions:	185.1 m x 74.83 m
Number of elements:	1560
Number of nodes:	13 003

Mesh is fined to the coarseness factor 0.25 in the proximity of the construction, elsewhere it is set to coarseness factor 1.00.

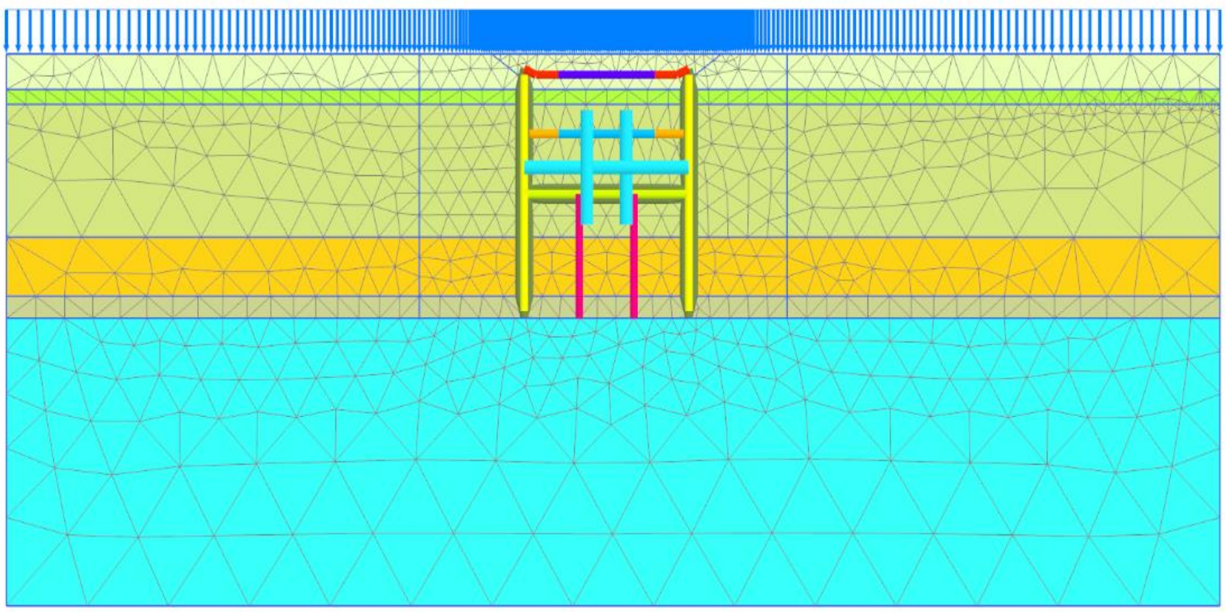


Figure 5.2 Mesh coarseness

Soils and interfaces	Plates	Anchors	Embedded pile row
caprock	base slab Elt	temporary	tension piles
distinctly weathered limestone	base slab Est		
made ground	mezzanine level Elt		
midra shale	mezzanine level Est		
partially weathered limestone	prop - mezzanine level Est		
rus formation	prop - roof slab Elt		
	prop - roof slab Est		
	prop-mezzanine level Elt		
	roof slab Elt		
	roof slab Est		
	walls Elt		
	walls Est		

Figure 5.3 Legend of materials for model geometry

Permanent structural parts where modelled by the element “plate” that ensures the fully fixed connections between parts. The temporary prop was modelled by the element “end to end anchor” – the connection is made by pinned joint.

The model was additionally loaded by surface continuous load 60 kN/m. This is the highest possible estimated constant load on the surface.

5.2.3 Water conditions

The groundwater level was assumed to be hydrostatic from +0.00 m QNHD. As the water level was changing during the construction (because of drainage), the local ground water levels were set for clusters inside of the structure, changing with the phases of construction. After backfilling of the roof slab, water level was set back to the original level in all clusters.

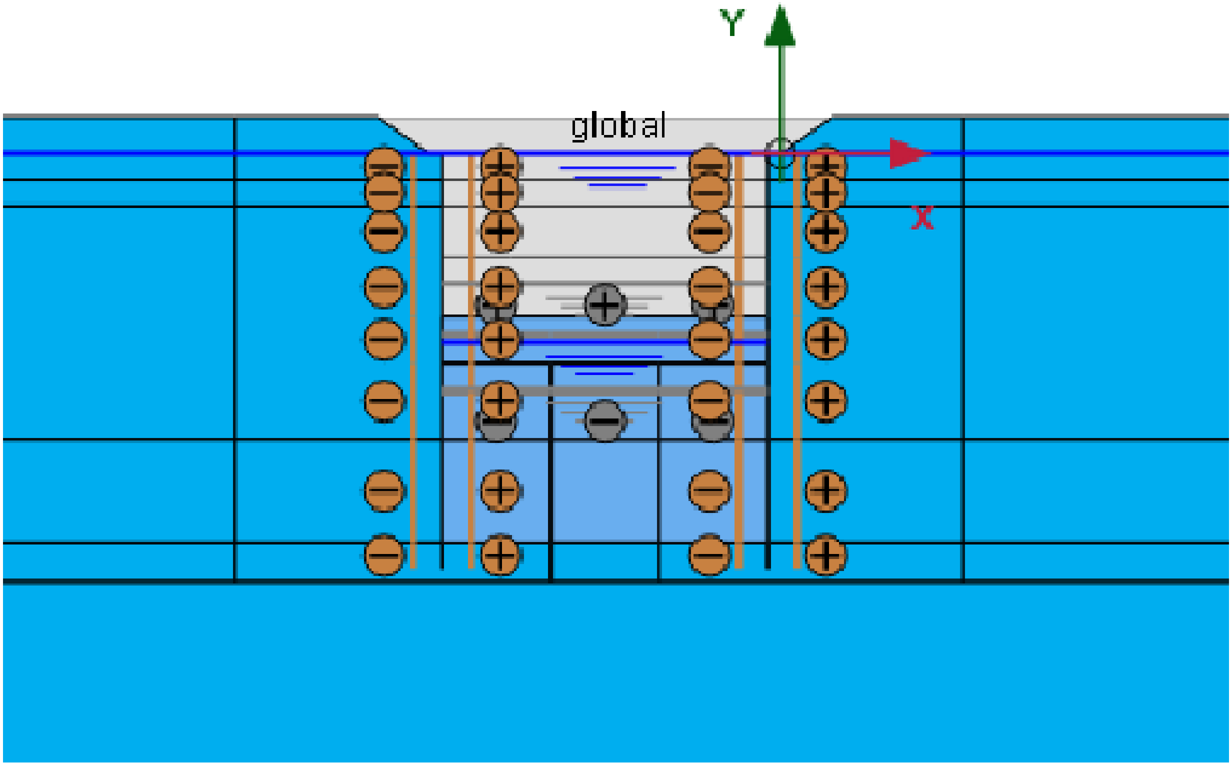


Figure 5.4 Setting of local groundwater level inside of the structure –Stage of construction nr. 6

5.2.4 Stages of construction

Stage 1: Initial phase (K_0 procedure)

Stage 2: Activation of surface load

Stage 3: Activation of diaphragm walls

Stage 4: Excavation to the roof level

Stage 5: Activation of the roof level wallers and prop, excavation to mezzanine level

Stage 6: Activation of mezzanine level wallers and prop, excavation to the level of temporary prop

Stage 7: Activation of temporary prop, excavation to the base level

Stage 8: Activation of base slab and tension piles

Stage 9: Removal (de-activation) of temporary prop

Stage 10: Landscaping

Stage 11: Change of E ($E_{st} \rightarrow E_{lt}$)

Stage 12: One – sided load (surface load is de-activated at the left side of the structure and above the structure)

The structure and the ground were loaded by forces that were changing during the construction. The plaxis model enables us to display the results of these forces on the structure and on the ground.

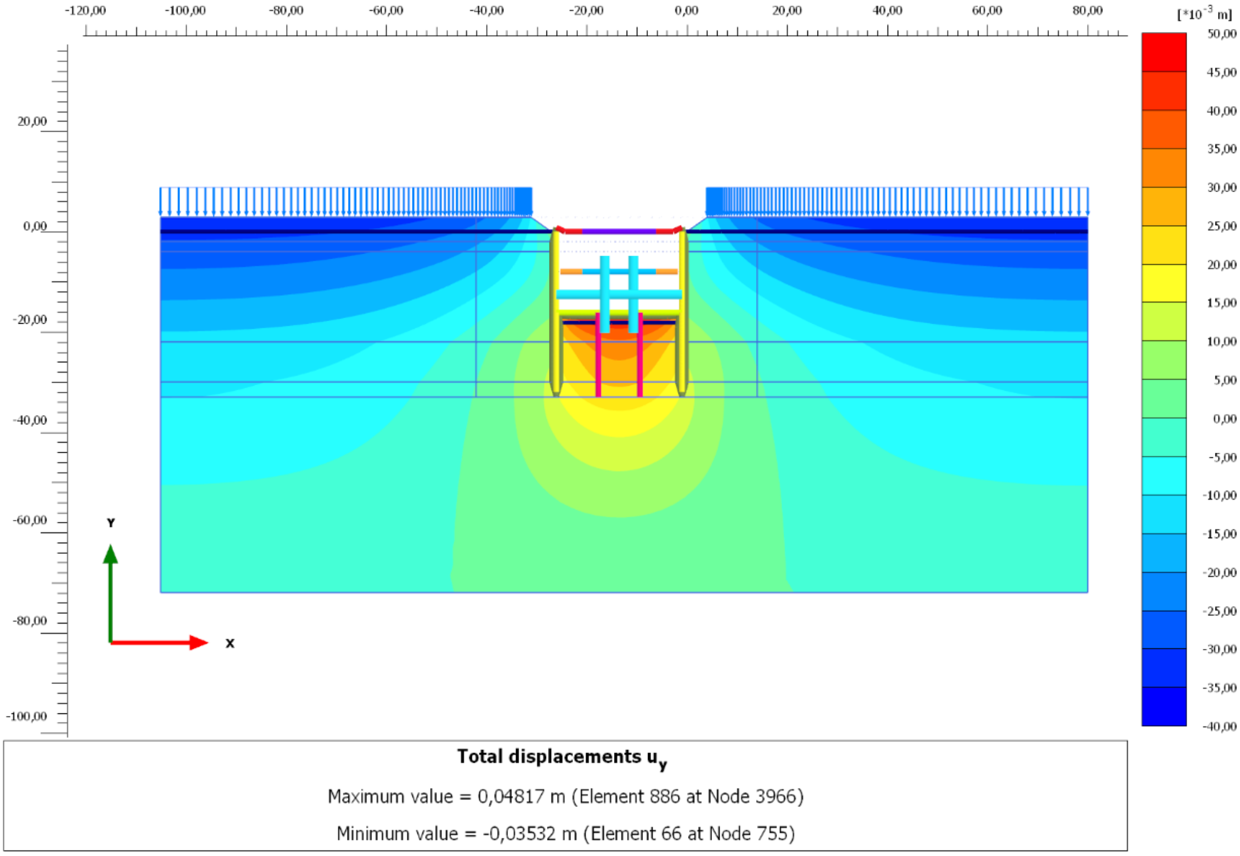


Figure 5.5 Total vertical displacements u_y – Stage 8. The ground under the base slab is pushed upwards by groundwater. This pressure induces great bending moments in the base slab and the base slab is being displaced. The tension piles are constructed to reduce the uplift of the base slab.

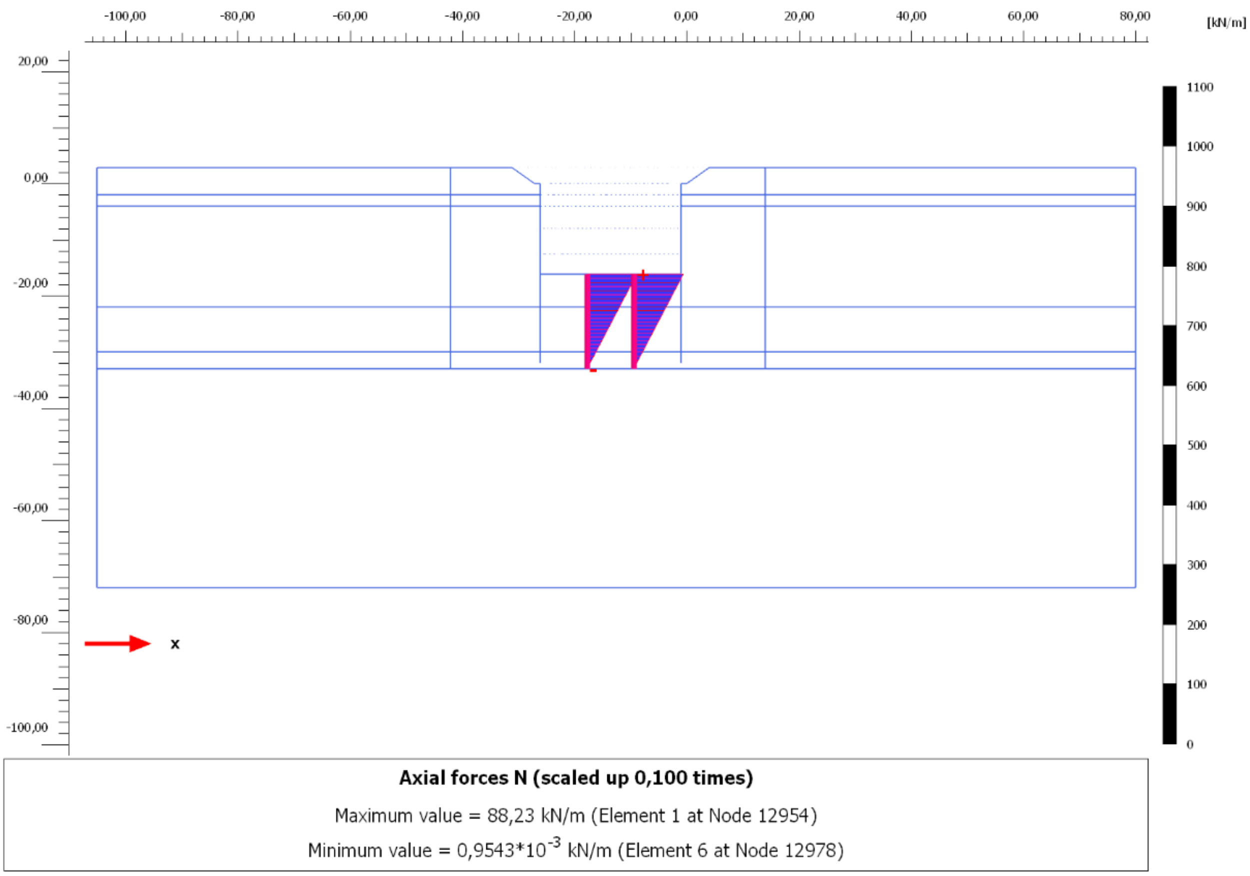


Figure 5.6 Axial forces in tension piles – Stage 9

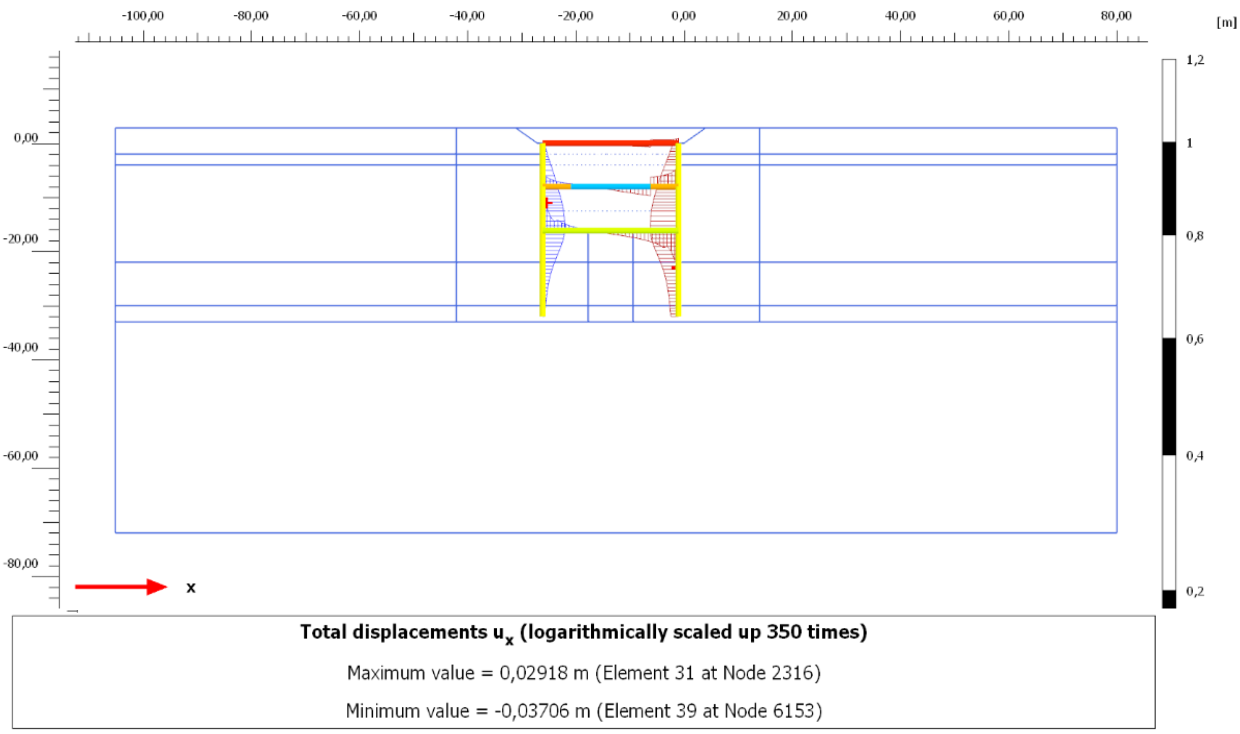


Figure 5.7 Total horizontal displacements of the structure u_x – Stage 12: One - sided load. The displacements of the right wall are only slightly bigger than the displacements of the left wall.

5.3 Design of the diaphragm wall

The design of the diaphragm wall was carried out according to Eurocodes: the actions on the structure were taken according to Eurocode 1, the design of the concrete structural parts was carried out according to Eurocode 2 [7] and the geotechnical parameters were established according to Eurocode 7 – design approach 2 [8].

Concrete class C40/50 was specified for the whole structure and the reinforcement steel B550B was taken for the diaphragm walls.

Concrete class C40/50

$f_{ck} = 40\ MPa$

$f_{cd} = \alpha_{cc} * f_{ck} / \gamma_c = 1.0*40/1.5 = 26.667\ MPa$

$f_{cm} = 48\ MPa$

$E_{cm} = 35\ GPa$

$\lambda = 0.8$

$\eta = 1$

$\varepsilon_{c3} = 1.75$ ‰

$\varepsilon_{cu3} = 3.5$ ‰

Reinforcement - Steel B550B

$f_{yk} = 550\ MPa$

$f_{yd} = f_{yk} / \gamma_s = 550/1.15 = 478.261\ MPa$

$E = 200\ GPa$

$\varepsilon_{yk} = 2.5$ ‰

$\varepsilon_{uk} = 75$ ‰

A bending moment envelope (in characteristic values) was obtained from the Plaxis model – there are moments on the excavated part of the wall and moments on the soil face. According to EC 7 – design approach 2 [8] these values where multiplied by $\gamma_G = 1.35$ to obtain the design values. The reinforcement was designed to the bending moments' envelope in design values. The reinforcement bars of a diameter 28 mm and 40 mm were chosen for the structure with spacing 140 mm, in case of a third row of reinforcement spacing is 280 mm.

The moments on the excavated part of the wall and on the soil face of the wall are quite different in every part. That is why there are six different sections for the placement of reinforcement.

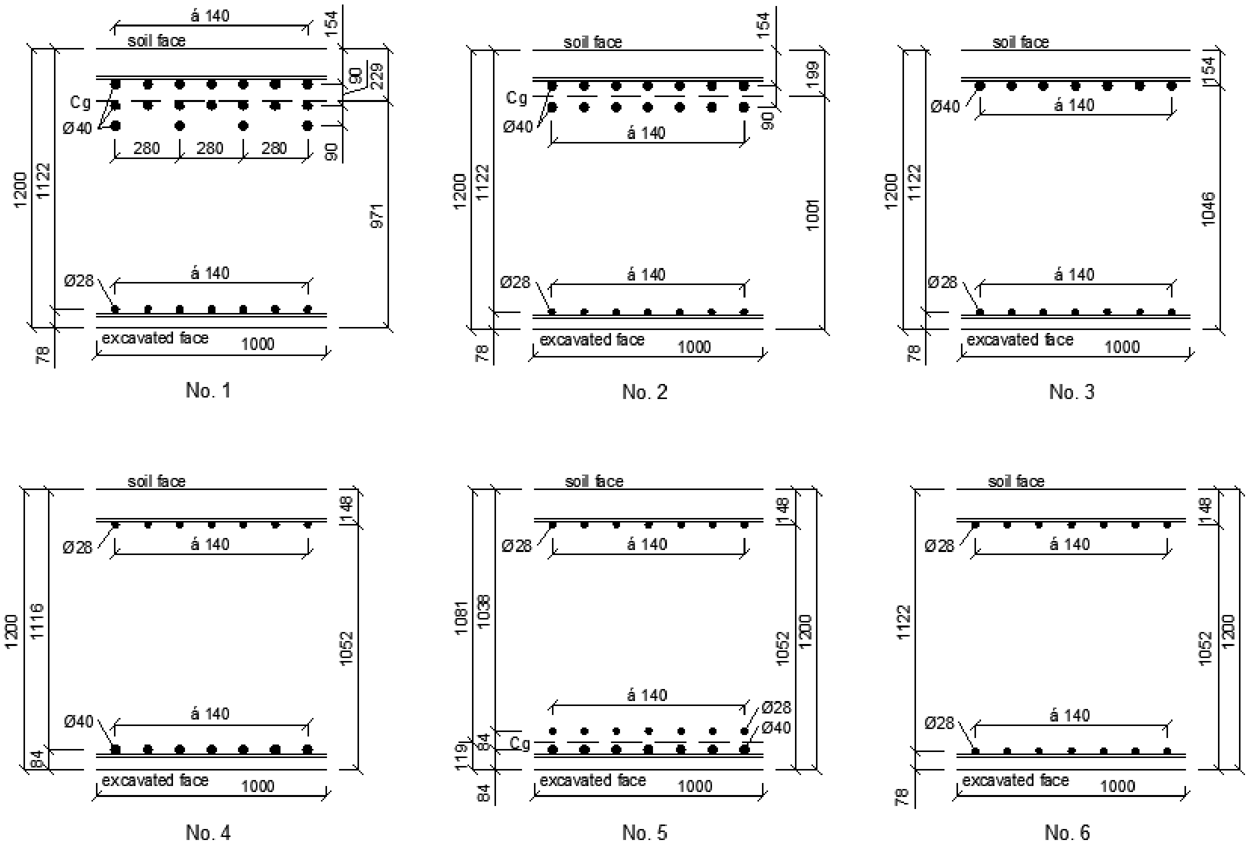


Figure 5.8 Placement of the reinforcement in the wall

Calculation:

$A_{srqd} = b * d * f_{cd} / f_{yd} * \sqrt{1 - 2 * M_{ed} / (b * d^2 * f_{cd})}$

$x = A_{sprov} * f_{yd} / (b * f_{cd} * \lambda)$

$z = d - 0.5 * \lambda * x$

$M_{rd} = A_{sprov} * z * f_{yd}$

$b = 1.0\ m$

$h = 1.2\ m$

$c_{excavated\ face} = 50\ mm$

$c_{soil\ face} = 120\ mm$

Where more layers of reinforcement were used, effective depth "*d*" was taken from the centre of gravity of the reinforcement (calculated by weighted average).

Check of structural principles:

$A_{s,v\,min} = 0.002 * A_c = 0.002 * 1.2 * 1.0 = 24.00 * 10^{-4}\ m^2$ ok

$A_{s,v\,max} = 0.04 * A_c = 0.04 * 1.2 * 1.0 = 480.00\ * 10^{-4}\ m^2$ ok

$s_{max} \leq\ 3 * h\ = 3 * 1200 = 3600\ mm$ ok

$\leq 400\ mm$ ok

$s_{min} \geq max\ \{1.2*\Phi;\ d_g + 5\ mm;\ 20\ mm\}$

$\geq max\ \{1.2*40;\ 16 + 5\ mm;\ 20\ mm\}$

$\geq max\ \{1.2*40;\ 16 + 5\ mm;\ 20\ mm\}$

$\geq 48\ mm$ ok

The biggest moments are on the soil face of the wall. They are created when the structure is backfilled. The jumps in the bending moment envelope are in the level where the structure is joined to the mezzanine level wallers (and prop) and to the base slab. The moments on the excavated part of the wall are the biggest ones in the level where the wall is joined to the base slab.

The aim of the reinforcement design was to use bars of a bigger diameter so that it would cover greater moments and it would not be necessary to change the reinforcement scheme. On the other hand it would not be economical to over reinforce the walls.

This solution is a compromise between these two requirements.

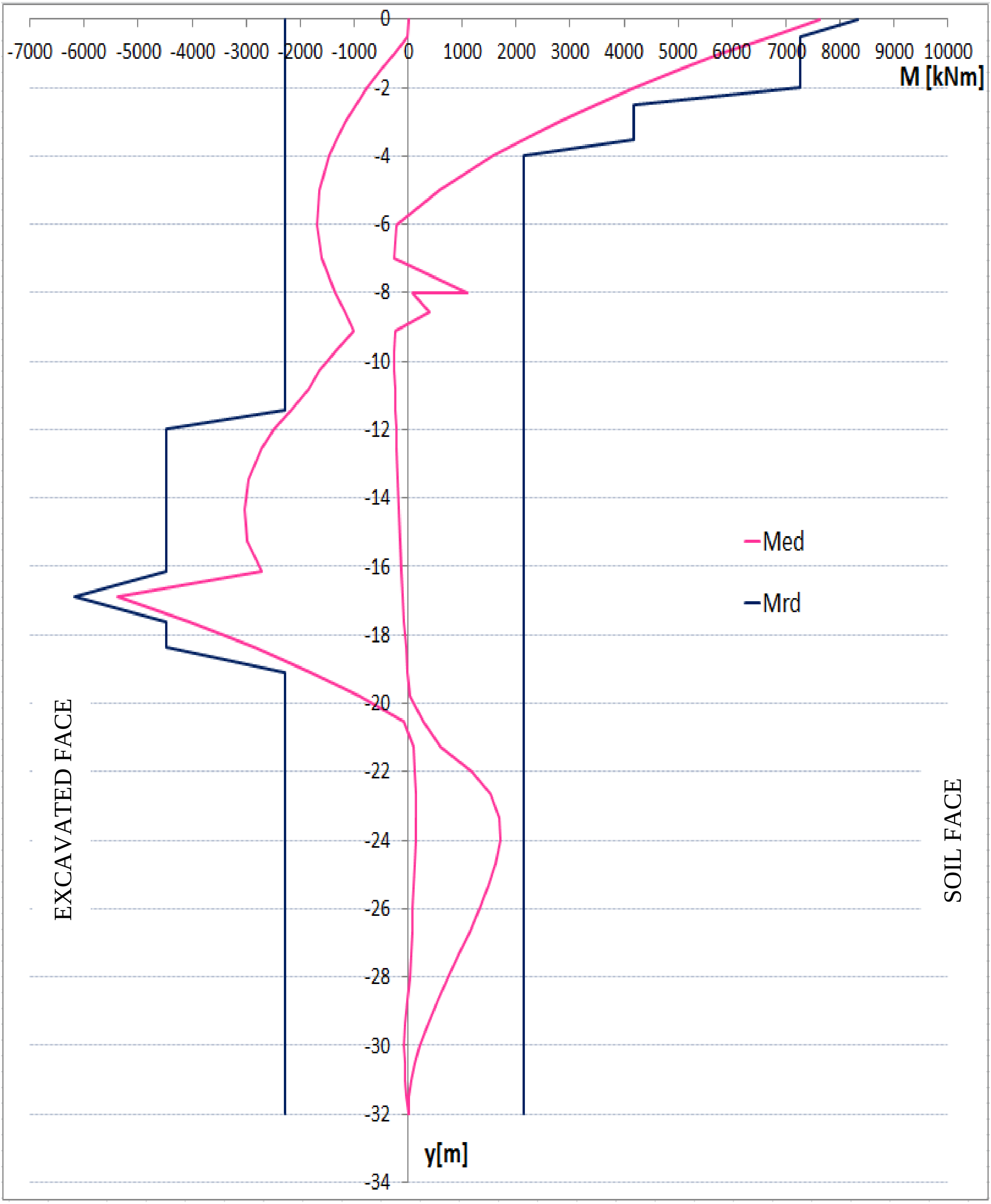


Figure 5.9 Bending moments' envelopes

Embedment length or lap length were not taken in consideration in this calculation.

Find the complete calculation in the appendix.

5.4 Design of the mezzanine level prop

For design of the mezzanine level prop the software SCIA Engineer was used.

5.4.1 Input values

Concrete class C40/50

$f_{ck} = 40\ MPa$

$f_{cd} = \alpha_{cc} * f_{ck} / \gamma_c = 1.0*40/1.5 = 26.667\ MPa$

$f_{cm} = 48\ MPa$

$E_{cm} = 35\ GPa$

$\lambda = 0.8$

$\eta = 1$

$\varepsilon_{c3} = 1.75$ ‰

$\varepsilon_{cu3} = 3.5$ ‰

Reinforcement - Steel B500B

$f_{yk} = 500\ MPa$

$f_{yd} = f_{yk} / \gamma_s = 500/1.15 = 434.78\ MPa$

$E = 200\ GPa$

$\varepsilon_{yk} = 2.5$ ‰

$\varepsilon_{uk} = 75$ ‰

Diameter of the prop	d = 1 500 mm
Length of the prop	l = 14.7 m
Diameter of reinforcement	Φ = 40 mm
Number of reinforcement bars	20 pieces
Reinforcement concrete cover	c = 50 mm
Stirrup s	Φ_s = 14 mm
Surface of the reinforcement	$A_s = 251.3 * 10^{-4}\ m^2$
Surface of the concrete	$A_c = 1.767\ m^2$

Check of structural principles:

$A_{s,\,min} = 0.002 * A_c = 0.002 * 1.767 = 35.34 * 10^{-4}\ m^2$ *ok*

$A_{s,\,max} = 0.04 * A_c = 0.04 * 1.767 = 706.8 * 10^{-4}\ m^2$ *ok*

s_{max}	$\leq$ *400 mm*	*ok*
s_{min}	$\geq$ *max {1.2*Φ; d_g + 5 mm; 20 mm}*	
	$\geq$ *max {1.2*40; 16 + 5 mm; 20 mm}*	
	$\geq$ *max {1.2*40; 16 + 5 mm; 20 mm}*	
	$\geq$ *48 mm*	*ok*
s_s	$\leq$ $20{*}\Phi_l = 20{*}40 = 800$ mm	*ok*
	$\leq$ h = 1500 mm	*ok*
	$\leq$ 300 mm	*ok*
Stirrups Φ ≥ 6 mm		*ok*
	≥ ¼ Φ = ¼ * 40 = 10 mm	*ok*

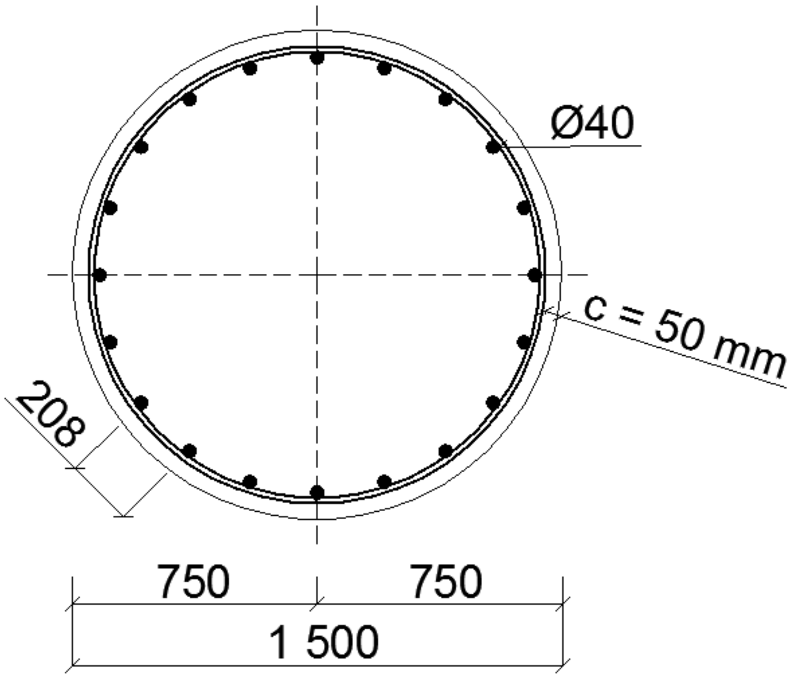


Figure 5.10 Mezzanine level prop geometry

5.4.2 Capacity check

The characteristic values of bending moments and normal forces were taken from the Plaxis model. According to EN 1997 – design approach 2 [8] the coefficient $\gamma_G = 1.35$ was used for constant load to obtain design values of these inner forces.

There are seven stages of construction where the inner forces in the prop change. The first stage (mezzanine level) is the moment when the prop is placed in the structure and the soil is excavated to the level of the placement of temporary props. The second phase (placement of temporary props) is when the temporary props are actually placed and soil is excavated to the level of the base slab. The third phase is depositing of the base slab. The fourth phase is removal of temporary props, the fifth phase is when the roof is landfilled to the original level and the structure becomes an underground box. In the next phase, the Young´s modulus is changed from the short term modulus to the long term modulus. The last phase is for checking the inner forces when the surface continuous load 60 kN/m stays only on one side of the structure.

Table 5.3: Design inner forces for mezzanine level prop from Plaxis

Mezzanine level	Placement of temporary props	Placement of base slab	Removal of temporary props	Backfill of the structure	Change of E ($E_{st} \rightarrow E_{lt}$)	One-sided surface load
M_{ed} [kNm]						
-1 411.7	-2 306.0	-2 358.8	-2 402.9	-2 976.2	-2 976.1	-2 957.3
N_{ed} [kN]						
-24 119.9	-37 220.0	-39 815.3	-40 998.4	-13 785.0	-13 784.6	-20 157.1

The initial imperfections had been taken in account (according to EN 1992-1-1) [7]:

$\theta_i = \theta_0 * \alpha_h * \alpha_m = 1/200 * 2/3 * 1 = 1/300$

$\theta_0 = 1/200$

$\alpha_h = 2/\sqrt{l} = 2/\sqrt{14.7} = 0.52164 \qquad 2/3 \leq \alpha_h \leq 1.0 \qquad \rightarrow \qquad \alpha_h = 2/3$

$\alpha_m = \sqrt{0.5 * (1 + \frac{1}{m})} = 1$

$m = 1$

$e_i = \theta_i * l_0 / 2 = 1/300 * 7.35 / 2 = 0.013\ m$

$l_0 = l/2 = 14.7 / 2 = 7.35\ m$

Additional moments from initial imperfections: $M_{0d} = N_{ed} * e_i$

Total moments: $M_{0ed} = M_{ed} + M_{0d}$

Table 5.4: Bending moments with impact of initial imperfections

Mezzanine level	Placement of temporary props	Placement of base slab	Removal of temporary props	Backfill of the structure	Change of E (E_{st}→E_{lt})	One-sided surface load
M_{0d} [kNm]						
-313.6	-483.9	-517.6	-533.0	-179.2	-179.2	-262.0
M_{0ed} [kNm]						
-1725.2	-2789.8	-2876.4	-2935.9	-3155.4	-3155.3	-3219.3

The effect of slenderness:

$\lambda = l_0 / i = 7.35 / 0.375 = 19.6$

$i = \sqrt{\frac{\pi * r^4}{4 * \pi * r^2}} = \frac{r}{2} = 7.35/2 = 0.375$

$\lambda_{lim} = 20 * A * B * C / \sqrt{n} = 20 * 0.7 * 1.21 * 0.7 / \sqrt{0.87} = 12.71$

$A = 0.7$

$B = \sqrt{1 + 2 * \frac{As * fyd}{Ac * fcd}} = \sqrt{1 + 2 * \frac{251.3 * 10^{-4} * 434.78 * 10^6}{1.767 * 26.67 * 10^6}} = 1.21$

$C = 0.7$

$n = \frac{Ned}{Ac * fcd} = \frac{40\,998.4 * 10^3}{1.767 * 26.67 * 10^6} = 0.87$

$\lambda = 19.6 \geq \lambda_{lim} = 12.71$ → *it is necessary to calculate with slenderness*

Method based on nominal curvature [7]*:*

$M_{ed} = M_{0ed} + M_2$

$M_2 = N_{ed} * e_2$

$e_2 = \frac{1}{r} * l_o^2 / c$

$c = 10$

$\frac{1}{r} = K_r * K_\varphi * 1/r_0$

$1/r_0 = \varepsilon_{yd} / (0.45d) = 0.00217 / (0.45 * 1.416) = 0.0034$

$d = h - c - \Phi_s - \Phi/2 = 1500 - 50 - 14 - 40/2 = 1416\ mm = 1.416\ m$

$\varepsilon_{yd} = f_{yd} / E = 434.78*10^6 / (200*10^9) = 0.00217$

$K_r = (n_u - n) / (n_u - n_{bal}) \qquad \leq 1$

$n_{bal} = 0.4$

$n = N_{ed} / (A_c * f_{cd})$

$n_u = 1 + \omega = 1 + \frac{As*fyd}{Ac*fcd} = 1 + \frac{251.3*10^{-4}*434.78*10^6}{1.767*26.67*10^6} = 1.232$

$K\varphi = 1 + \beta * \varphi_{ef} = 1 + 0.353 * 1.259 = 1.444 \qquad \geq 1 \quad ok$

$\varphi_{ef} = \varphi(\infty, t_0) * M_{0qp} / M_{0ed} = 1.7 * M_0 / (1.35 * M_0) = 1.7 / 1.35 = 1.259$

$\beta = 0.35 + f_{ck}/200 - \lambda/150 = 0.35 + 26.67/200 - 19.6/150 = 0.353$

$t_0 = 28\ days$

$h_0 = 2*A_c / u = 2*A_c / (\pi * d) = 2 * 1\ 767\ 145 / (\pi * 1\ 500) = 750\ mm$

u — perimeter of the part that is exposed to drying

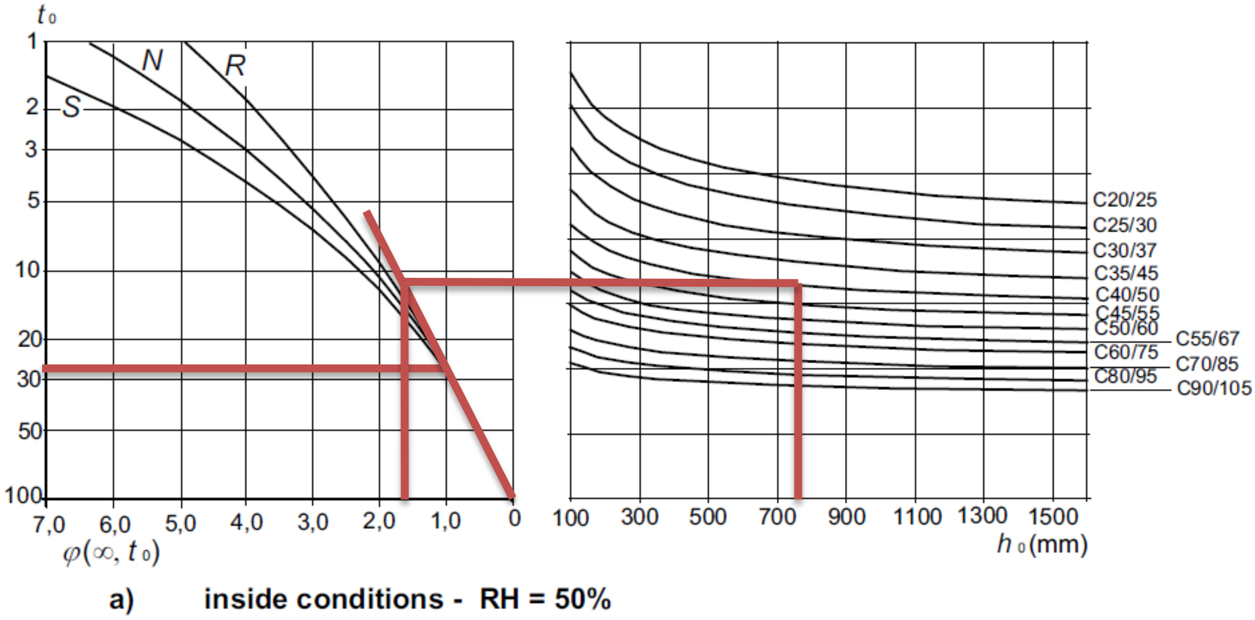


Figure 5.11 Determination of $\varphi(\infty, t_0)$ – curve N [7]

Table 5.5: Calculation of M_2

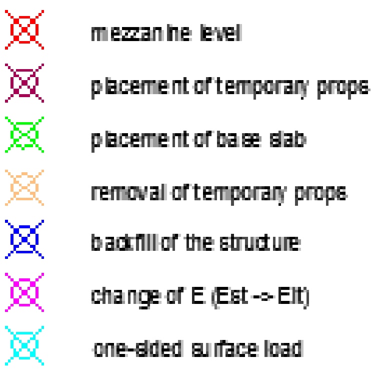
	Mezzanine level	Placement of temporary props	Placement of base slab	Removal of temporary props	Backfill of the structure	Change of E ($E_{st} \rightarrow E_{lt}$)	One-sided surface load
M_2	-553.8	-524.7	-491.4	-473.2	-365.6	-365.6	-516.8
e_2	0.023	0.014	0.012	0.012	0.027	0.027	0.026
1/r	0.004	0.003	0.002	0.002	0.005	0.005	0.005
K_r	0.866	0.531	0.465	0.435	1.000	1.000	0.967
n	0.512	0.790	0.845	0.870	0.293	0.293	0.428

Table 5.6: Final inner forces for capacity check

Mezzanine level	Placement of temporary props	Placement of base slab	Removal of temporary props	Backfill of the structure	Change of E ($E_{st} \rightarrow E_{lt}$)	One-sided surface load
M_{ed} [kNm]						
-2 278.9	-3 314.5	-3 367.8	-3 409.1	-3 521.1	-3 521.0	-3 736.1
N_{ed} [kN]						
-24 119.9	-37 220.0	-39 815.3	-40 998.4	-13 785.0	-13 784.6	-20 157.1

The initial imperfections and slenderness are included in these values of inner forces.

The capacity check was calculated in the extreme section – section in the built-in end.



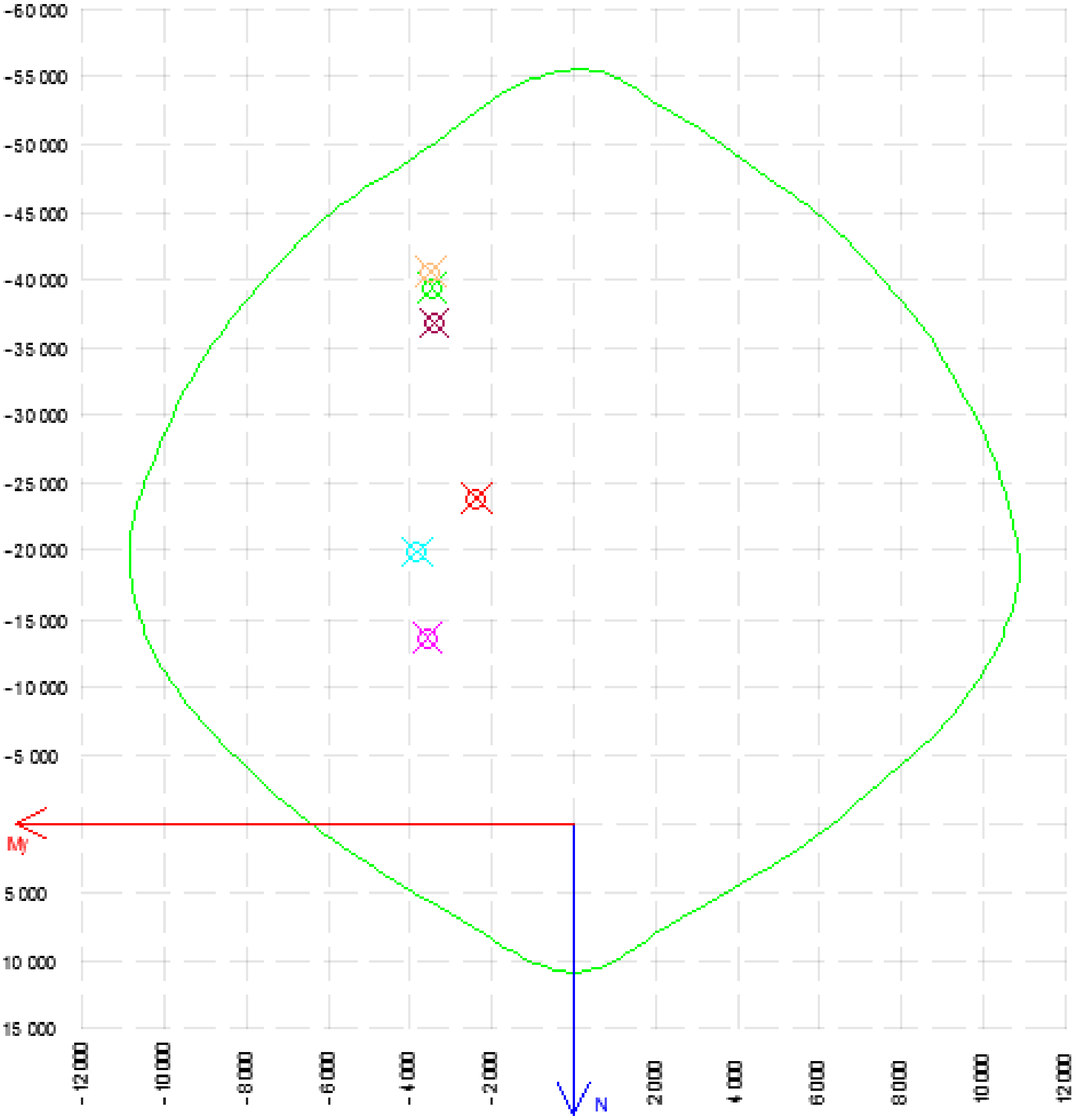


Figure 5.12 Mezzanine level prop capacity check

Stages Backfill of the structure and Change of E ($E_{st} \rightarrow E_{lt}$) have almost the same values and in this diagram they may overlap.

Summary

In the first chapter the geology of Qatar peninsula is described. Then there is the precise geology of the NDIA site that was obtained by the geotechnical investigation. The groundwater conditions and geotechnical parameters are mentioned in this chapter.

The second chapter deals with the durability of the structure and its lifetime. The main aspect affecting the lifetime of the structure is corrosion. It can be caused by many different actions for example by reactions of CO_2 with concrete. The methods of protection can be divided to groups as primary, secondary and special measures but also they can be divided as active and passive.

The third chapter is about a choice of suitable retaining walls and lateral support system. There are many types of lateral systems but only two of them are suitable for NDIA – secant pile walls and diaphragm walls. It is mostly given by the impermeability of these walls. The diaphragm walls were chosen for the structure. As a lateral support system the obvious choice was a system of massive props placed in a great distance ($L_{spacing}$ = 21.6 m).

The fourth chapter describes the Plaxis model made for the calculation of inner forces. Furthermore it deals with the structural checks for ULS of diaphragm wall and mezzanine level prop. The reinforcement of the wall was design in six different sections according to the bending moments' envelope. Two diameters of reinforcement were designed – Φ 28 mm and Φ 40 mm. The bars are placed in one, two or three rows. For the prop software SCIA Engineer was used to obtain the capacity diagram for check of the interaction of bending moments and normal forces. These forces were taken from the Plaxis model (in characteristic values), multiplied by partial safety factor to get the design values and then the additional moments from initial imperfections and slenderness were added. Both designs are satisfactory for the given load.

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List of short cuts and symbols

A_c	cross sectional area of concrete
$A_s = A_{sprov}$	cross sectional area of reinforcement
$A_{s,max}$	maximal cross sectional area of reinforcement
$A_{s,min}$	minimal cross sectional area of reinforcement
A_{srqd}	minimal required cross sectional area of reinforcement
$A_{s,v,max}$	maximal cross sectional area of reinforcement for walls
$A_{s,v,min}$	minimal cross sectional area of reinforcement for walls
b	overall width of a cross-section
c	concrete cover
c	factor depending on the curvature distribution
c'_{ref}	effective ground cohesion
d	diameter
d	effective depth of a cross-section
d_g	largest nominal maximum aggregate size
E	modulus of elasticity
E´	effective modulus of elasticity
E_{cm}	secant modulus of elasticity of concrete
E_{lt}	long term modulus of elasticity
E_{st}	short term modulus of elasticity
EA	axial stiffness
EI	bending stiffness
e_i	eccentricity
e_2	deflection
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
i	radius of gyration
K_0	at-rest earth pressures coefficient
K_r	correction factor depending on axial load
K_φ	factor for taking account of creep

l	length
l_0	effective length
$L_{spacing}$	span between props
M_{ed}	design value of the applied internal bending moment
M_{0d}	first order end moments
M_{0ed}	1st order moment, including the effect of imperfections
M_{0qp}	first order end moments of quasi-permanent combination
M_2	nominal 2nd order moment
m	number of vertical members contributing to the total effect
N_{ed}	design value of the applied axial force (tension or compression)
NDIA	New Doha International Airport
n	relative normal force
n_{bal}	value of n at maximum moment resistance
QNHD	Qatar National Height Datum
s_{max}	maximal spacing of reinforcement bars
s_{min}	minimal spacing of reinforcement bars
s_s	spacing of stirrups
t	time being considered
u	perimeter of the part that is exposed to drying
ULS	ultimate limit state
w	self -weight
x	neutral axis depth
z	lever arm of internal forces
1/r	curvature at a particular section

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
α_h	reduction factor for length or height
α_m	reduction factor for number of members
β	coefficient
γ	unit weight of soil
γ_c	partial factor for concrete
γ_G	partial factor for permanent actions, G
γ_s	partial factor for reinforcing steel
ε_{c3}	compressive strain in the concrete

ε_{cu3}	ultimate compressive strain in the concrete
ε_{uk}	characteristic strain of reinforcement steel at maximum load
ε_{yk}	strain in reinforcing steel
η	factor defining the effective strength
θ_i	inclination
θ_0	basic value of inclination
λ	factor defining the effective height of the compression zone
λ	slenderness ratio
λ_{lim}	slenderness limit ratio
Φ	diameter of a reinforcing bar
Φ_s	diameter of a stirrup
φ'	effective angle of shearing of soil
φ_{ef}	effective creep ratio
$\varphi(\infty, t_0)$	final value of creep coefficient

List of Appendices