The paper was published in the proceedings of the 25th European Young Geotechnical Engineers Conference and was edited by Ernest Olinic and Sanda Manea. The conference was held in Sibiu, Romania 21-24 June 2016.

25th European Young Geotechnical Engineers Conference
21st – 24th of June 2016, Sibiu, Romania

Dry Dock at the Merwede River Banks

Leon SCHADEE*

Fugro GeoServices Geo-Consultancy, the NETHERLANDS

ABSTRACT

The market perspective for the Dutch shipbuilding market is positive. To be competitive and for future growth Neptune Shipyards wants to build a new dry dock for large river cruise ships. The location for this dry dock is situated at the river bank of the important shipping route Merwede, which is a logistically beneficial location. The project location is located in a densely built area. At a 20 m distance a dike is present, as primary flood defence. Due to the dimensions of the dry dock and the densely built location many boundary conditions have to be taken into account. These boundary conditions lead to many geotechnical challenges.

This paper highlights the predesign of the dry dock and discusses the key value of the Geotechnical Engineer in this project.

Keywords: dry dock, sheet piles, deep excavation, PLAXIS, flood defence

1. INTRODUCTION

Often construction and large maintenance of ships are performed in a dry environment. A conventional way for dry construction is a ship ramp. A more modern and delicate method is the use of a dry dock. The geotechnical engineer plays an important role in the design of this dry dock. Many critical aspects are related to soil conditions and soil-structure interaction. The role and influence of the geotechnical engineer is crucial in this predesign phase, i.e. in designing the walls of the dock, resisting the uplift forces and analysing the stability of the dike.

2. PROJECT LOCATION

The project is located on the banks of the river Merwede. This river is one of the important shipping routes in the Netherlands and is part of the Rhine–Meuse–Scheldt delta. The Merwede connects the river Rhine with the river Meuse. These rivers serve as major transportation routes to Germany, Belgium, France and the rest of Europe.

Due to the beneficial location many companies are located on the banks of the river Merwede. One of the shipbuilders wants to build a dry dock for constructing and repairing ships. Figure 1 shows an aerial photo of the project location.

* presenting author

Figure 1. Project location with the dry dock marked in green, surrounding structures in yellow and the primary flood defence in red Bing Maps (2016).

2.1. Site Characterisation

Throughout the centuries the Merwede river meandered at this location and deposited sands and clays. The soil profile consists of soft soils with a dense sand layer at depth. In the past this terrain was created by reclamation. In the “man-made” top soil layer rubble was encountered.

The current surface level is around NAP +3.0 m, where NAP is the general reference level in the Netherlands. Table 1 shows the soil stratigraphy. These soil conditions are commonly found in this part of the Netherlands. A typical CPT result is shown in Figure 2. The future grade level is at NAP +4.0 m.

Table 1. General Soil Stratigraphy

Top of layer [rel. to NAP]	Soil Type
+3.0 m	Sand, with some rubble / man-made soil
-1.6 to -2.0 m	Sand, medium dense
-4.8 to -6.8 m	Peat and Clay
-7.2 to -10.3 m	Sand, dense
-36.5 to -40.7 m	Clay, silty (overconsolidated)
-38.3 to -42.9 m	Sand, dense
-45 m	Maximum explored depth

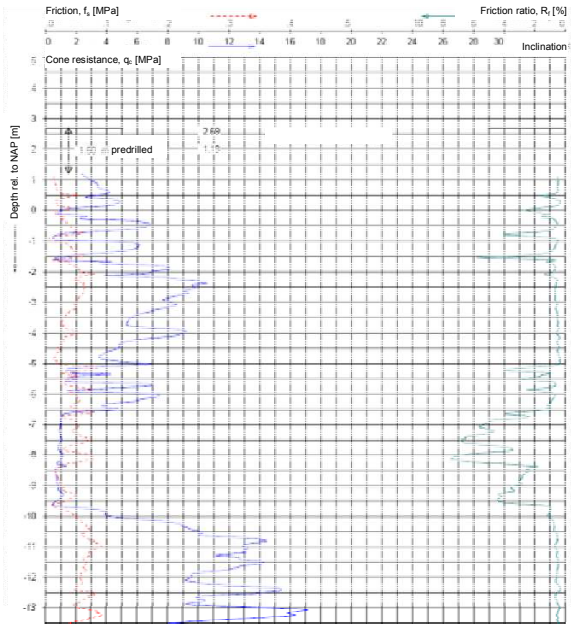

Figure 2. Typical CPT at the project location.

From piezometers at ca. 400 m and 800 m behind the dike the groundwater table and the hydraulic head of the first water bearing strata are determined. The groundwater table is at NAP +0.9 m. This is the average water level of the Merwede. At depth a water bearing sand layer is encountered, between approximately NAP -7 m to NAP -40 m. The hydraulic head in this sand layer is connected to the water level in the Merwede. High water levels in the river can rise up to NAP +4.15 m. During these high water levels the head in first water bearing layer

layer can reach a level up to NAP +3.25 m. This head corresponds to a once per 50 years condition.

2.2. Soil parameters

The soil investigation is limited as a predesign is made. Soil strength and stiffness parameters are derived from the CPT results. Table 2 presents the effective strength parameters of this project. For the final design additional soil investigation will be performed, i.e. CPTs, borings and lab tests.

2.3. Geometry

The dimensions of the dry dock will be 30 m wide, 130 m long and 12 m deep. The entrance, to float ships into the dock, will be at the south west side of the dock. The walls of the dock consist of permanent steel sheet piles. Due to the position of the dock, one wall is retaining soil and water, whereas the other wall retains only water. Consequently the lateral load on the dock is not symmetrical, which is a disadvantage in the design.

On top of the dry dock a steel superstructure will be constructed. The steel superstructure is 20 m high and contains a gantry crane for handling large and heavy objects.

Two excavation methods were considered, dry and wet. Section 2.5 explains the excavation methods in more detail. In both excavation methods a strut is applied at 3.2 m depth. For the dry excavation a second strut is used at 7 m depth below surface level. The use of the second strut during the wet excavation is not necessary due to the beneficial water pressure inside the building pit during excavation.

The sheet pile walls of the dock have a different construction. The wall at the riverside has to retain only water, where the wall at the landside retains both water and soil. In the cross section of the excavation the asymmetrical loading can be seen, see Figure 3 and Figure 4. The sheet piles on the landside will be anchored using grout anchors. This is a commonly used anchoring system in the Netherlands. At the riverside wall no anchoring is possible, due to lack of a soil body adjacent to the retaining wall. The lateral loads on the riverside wall are somewhat lower compared to the landside of the dock, as the retaining height of the soil is much lower.

Table 2. Effective strength parameters for the predesign of the dry dock, based on DKM4

Top of layer in m depth*	**Soil Type**	**γ_{unsat} / γ_{sat} in kN/m^3**	**φ' (phi) in deg.[°]**	**c' in kPa**
0	Sand, with traces of rumble/ manmade	18 / 20	32,5	-
-4.6	Sand, medium dense	18 / 20	32,5	-
-8.1	Clay, sandy	18 / 18	22,5	5
-8.6	Sand, medium dense	18 / 20	32,5	-
-9.6	Peat	12 / 12	15,0	2,5
-11.4	Clay, silty	17 / 17	17,5	5
-12.7	Sand, dense	18 / 20	32,5	-

γ_{unsat} / γ_{sat} are the dry and wet unit wheight of the soil.

φ' (phi) is the internal friction angle of the soil.

c' is the effective cohesion, i.e. shear strength, of the soil.

*surface level equals NAP +4.0 m.

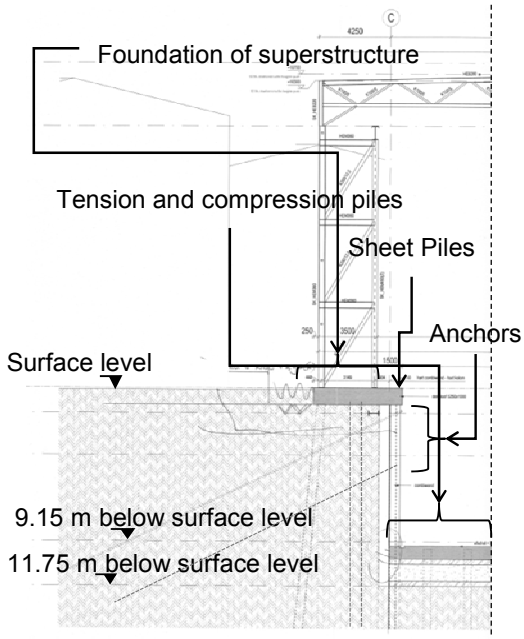

Figure 3. Cross section at the landside of the dry dock, with superstructure, foundation elements and underwater concrete.

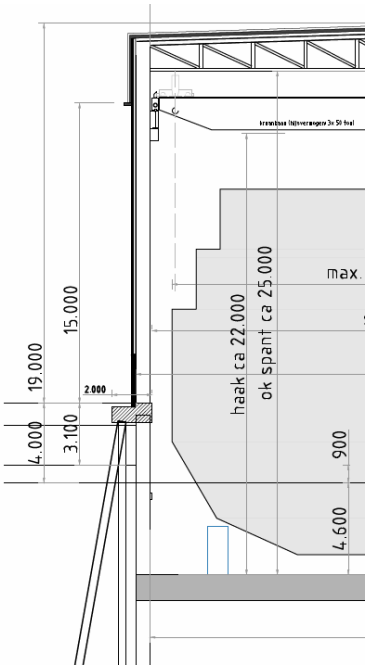

Figure 4. Cross section at the riverside of the dry dock.

2.4. Special Conditions/Topics

At the Northern part of the project a primary flood defence is present, i.e. dike. The Merwede may not be obstructed by the dry dock. Due to the dry dock's length and the nearby waterway a part of the dock will intersect with this dike. The safety of the polder protected by this dike may not be impaired. Therefore special attention is given to the interaction between the structure and dike. This challenge is discussed in more detail in Section 3.

Currently there are several structures present at the location of the future dry dock, see Figure 1. Most of the existing structures will be demolished. The nearby structures which will be maintained are marked yellow in Figure 1. The Southern structures are founded on shallow foundations. The office at the Northern side is constructed on a piled foundation. The buildings on a shallow foundation are sensitive to deformations and vibrations. Deformations have to be limited as the existing structure may not be damaged by the execution of the dry dock.

2.5. Construction Method

In the predesign phase two construction methods are considered. One of these methods is a wet excavation. After the final depth is reached, underwater concrete is used to construct a water tight sealing. The excavation will then be pumped dry and the dry dock construction can be finalized.

At the project site no natural watertight layer was encountered. In order to perform a dry excavation a horizontal soil injection technique is proposed to create a water tight sealing. After the sheet piles are installed the soil is injected at a minimum depth of 22,5 m (eq. NAP-18.5 m). This artificial horizontal waterthight layer is created from surface level. The horizontal injection is located between the sheet piles to create a confined space. Therefore the sheet piles have to reach below the minimum injection depth. Using this artificial layer it is possible to perform a dry excavation.

Underwater concrete is a large amount of the construction cost. Compared to underwater concrete the horizontal soil injection technique is very competitive, if the soil conditions meet certain criteria. A horizontal soil injection can be applied in

sands with sufficient permeability to inject the water glass mixture which creates a watertight layer. The sheet piles need to have a minimum depth to reach beyond the depth of the horizontal soil injection.

3. CHALLANGES AND OPPERTUNITIES

One of the important geotechnical challenges is the asymmetrical loading condition of the dry dock. In Figure 1 it can been seen that the dock replaces an existing quay wall. Figure 3 and Figure 4 show the cross section of the dock. The different load conditions by the soil and water are clearly shown.

To limit deformations it is preferable to apply anchors. The anchors transfer tension loads into the deep sand layer. However, only the retaining walls at the landside can be supplied with anchors. At the riverside anchors are not possible. The only lateral support in the final situation of the retaining wall at the riverside is through the dock's floor at 10 m depth.

Besides the asymmetrical loading conditions other high loads act on the structure as well. These high loads are caused by the inclined anchors, the gantry crane and alternating wind loads acting on the superstructure. These loads are transferred to the retaining walls and to foundation elements at short distance from the retaining walls. The loads cause a complex load distribution in the structure.

Another challenge is the interaction with the primary flood defence. In 2000 the dike was reinforced by a cofferdam of sheet piles. The cofferdam consists of two rows of AZ26 sheet piles. At the outer dike, i.e. south side, the sheet piles reach down to NAP -13,0 m. At the inner dike section the sheet piles reach to NAP -15,0 m.

Due to the hinterland which is an urban area, the safety of this dike must always be guaranteed. In addition large lateral loads on the retaining walls are caused by the dike. Furthermore, in the design a future dike reinforcement has to be included without additional measures for the dry dock during the reinforcement. Also, the installation of grout anchors for the retaining wall underneath the dike, may not impair the dike's safety.

In the design process there is a lot of interaction with the stakeholders, who all have different interests. In this project there are four main stakeholders. The shipbuilder, i.e. future owner of the dock, wants to build the dry dock as beneficial as possible. The river Merwede is controlled and maintained by Rijkswaterstaat (Dutch Ministry of Infrastructure and the Environment). The Water Board is responsible for the maintenance and safety of the primary flood defence. The last stakeholder is the local municipality of Gorichem. The municipality is responsible for the urban environment and special planning of the area. In the end they all have to agree with the design and construction method of the dry dock. During the predesign phase of the project these parties were already involved, to resolve as many difficulties as possible in an early stage.

4. DESIGN

This section discusses not all design aspects, but highlights the most interesting topics. Besides these topics the foundation, consisting of compression and tension piles, is calculated using the Eurocode 7, the sheet piles are designed according the Dutch design guideline for sheet piles (CUR166) and the hydrologic study is performed using the software package Watex.

4.1. Retaining Walls

To simulate the interaction due to the asymmetrical excavation an interaction calculation was performed. This study is performed using the software package D-Sheetpiling of Deltares, which is based on a spring model to simulate the soil behaviour. In a later stage the calculations are verified using FEM-software PLAXIS 2D.

Iterative Procedure

The sheet pile walls on the landside and riverside have been modelled in a separate model. On the landside anchors are applied. The supports, i.e. dock floor and struts if applicable, will transfer the loads from the riverside to the landside.

To investigate the interaction between both walls in the cross section the following iterative procedure is used:

1. Both side walls are analysed without interaction, using the same constructive phasing. Supports are modelled with springs.
2. The loads in the supports on the riverside are derived and applied on the landside in the corresponding phase. This is the first iteration step.
3. From this calculation the deformation at the level of the support is derived at the landside. On the riverside the support's stiffness is reduced to meet the deformation of the support on the landside.
4. The model on the riverside is calculated with the less stiff support. From this new calculation the forces in the support are derived and again applied on the landside. This is the second iteration step.
5. If the difference between the deformations is sufficiently small the iteration is stopped. Otherwise the steps from no. 3. to no. 5. are repeated until the displacements meet the criteria.

In this case the displacements on the riverside should be within 5% of the previous iteration. This exercise is performed in the serviceability limit state (SLS) as displacements are considered. Figure 5 shows the final model with the spring supports and corresponding loads.

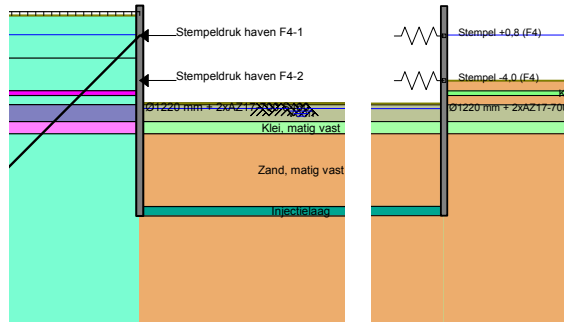

Figure 5. Model across the dock, with spring supports on the riverside (right) and loads on the landside (left).

Verification with Finite Element Model

The Finite Element Model (FEM) and calculation is made using PLAXIS 2D 2015. The geometry, loads and phasing are equal to the calculation in the spring model with D-Sheetpiling. In PLAXIS the soil is modelled using the Hardening Soil Small Strain Stiffness model. The geometry is shown is Figure 6.

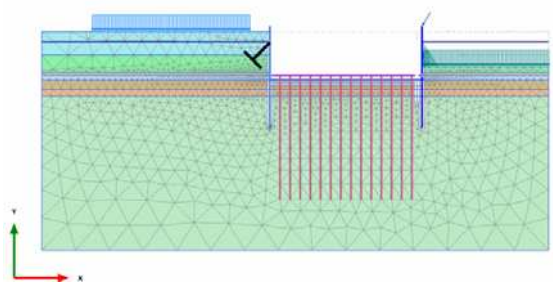

Figure 6. Plaxis model in final stage, with structure of dry dock, surface loads and loads from superstructure.

Table 3 shows the comparison of the results of both calculations. The spring model matches or gives an underestimate of the forces and deformations in the FEM. The benefit of the FEM is the prediction of soil deformations behind next to the excavation as shown in Figure 7

Table 3. Comparison of results Springmodel and FEM in final stage for wet excavation.

	Spring model	FEM	FEM / Spring.
$M_{max;riverside}$ in kNm/m[1]	478	474	0.99
$u_{max;riverside}$ in mm	29	17	0.60
$M_{max;landside}$ in kNm/m[1]	503	287	0.57
$u_{max;landside}$ in mm	77	57	0.74

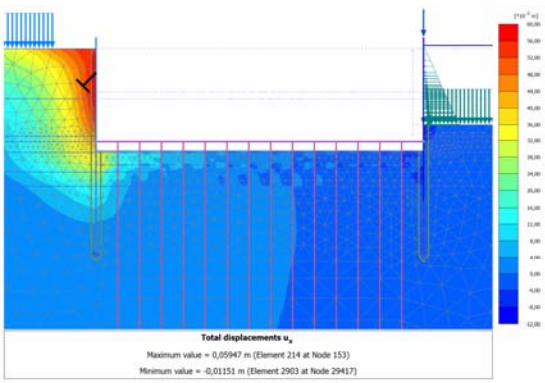

Figure 7. Final result of the finite element calculation.

4.2. Dike

The use of a free sheet pile wall is not feasible at the landside or next to the dike. Using anchoring the deformations, sheet pile length and size, and thereby the hindrance due to installation effects, are reduced by a considerable amount.

As anchoring system a grout anchor was chosen. This type of anchor can be installed with non to low relaxation of the subsoil. Using a specific casing the grouted anchor body can be installed with a high overpressure, to avoid leakage through the dike along the anchor rod. Properly installed anchors may improve the stability and reduces the risk of a leakage. To generate enough tensile strength the anchoring body is located approx. 9 m in the Pleistocene sand layer (15 m below surface level, i.e. NAP -40 m).

In the final design a stability and interaction study will be performed using FEM. This study has to prove the safety of the dike during the different construction stages and in the final situation. During this design stage the Water Board will be involved to be certain the design is save and all safety aspects are taken into account.

During the construction phase of the dry dock sufficient monitoring has to be done. The aim of the monitoring is to control the safety level of the construction works and the dike.

Sustainability

The anchors will reach into the stability core of the dike. During future dike reinforcements the anchors can be an obstacle. As mentioned before it shall be possible to reinforce the dike without severe mitigating measures. The following aspects are considered in the design:

- If the dike is reinforced by soil, the anchors do not obstruct the construction method regarding the reinforcement.
- If the dike is reinforced by a new cofferdam construction or retaining wall the anchor do not obstruct this construction method as they are located to deep.

The “design” dike reinforcement is excpecte to consist of an additional table geidht of the dike by approximately a meter.

5. OUTCOME AND CONCLUSION

From this predesign the feasibility is proven and a general design basis is made. Both excavation methods are technically possible to execute. The decision whether the horizontal injection or the underwater concrete is applied will be based on financial aspects and (geotechnical) risks.

This paper shows a case study where complex interactions between structural elements are modelled in a relatively simplified manner. This approach shows that the results are comparable with the

more advanced finite element models. However the results from the finite element model are more comprehensive, though the simplified method is sufficient for a predesign.

In this project the geotechnical engineer was involved in an early stage. Due to the early involvement of the geotechnical engineer a number of engineering topics could be based on proper design assumptions. Engineering topics are for example the sheet piles, the foundation design, the hydrologic conditions of the area, which are very important regarding the risk of uplift and the soil-structure interaction. In this early stage less conventional building methods could be investigated and geotechnical risks could be identified. This all contributed to a more optimised predesign and an optimal cost estimate of the building costs. The key value of the geotechnical engineer during this project is the insight of geotechnical risks, mitigation options and proper assumptions where needed. Which results in a realistic predesign given the available information.

ACKNOWLEDGEMENTS

The financial support offered by the Geotechnical Department of the KIVI Engineering Society, i.e. Netherlands Society for Soil Mechanics and Geotechnical Engineering, is gracefully acknowledged

REFERENCES

Bing Maps (2016), http://www.bing.com/maps

Eurocode 7 (EC7), NEN9997-1:2012 Geotechnical

CUR166, Sheet Pile Retaining Structures, 6th edition 2012, CUR SBR.