Design of Retaining Walls at Metro Nordhavnen – a Case Story

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ABSTRACT
Zûblin A/S is a part of the MetNord JV who carries out the construction of the Metro Cityringen – Branch off to Nordhavnen. The project consists of the Nordhavn Station, a Cut and Cover tunnel, a Ramp which takes the trains back to the ground level and a bored TBM tunnel. Züblin A/S has been responsible for the design of the temporary structures which mainly is carried out as multiply supported secant pile walls, supported by ground anchors.

The first design was carried out by modelling the secant pile walls with SPOOKS which uses the theory of Brinch Hansen, and commonly used in Denmark, for the ultimate limit state and the finite element program PLAXIS for the serviceability limit state. Due to the limitation of Brinch Hansen's theory and the discussion about stress-strain compatibility, the Employer had doubt that the deformations necessary for obtaining full active and passive earth pressure were sufficient. Therefore the final design ended up being a combination of SPOOKS for the ultimate limit state and a PLAXIS model for ultimate limit state and serviceability limit state to verify the results of Spooks.

The results of the different design methods are compared together with measured anchor loads from load cell installed on site.

Keywords: Case story, retaining walls, PLAXIS

1 INTRODUCTION

As a part of the Metro Cityringen in Copenhagen the Joint Venture (MetNord JV) consisting of Züblin and Hochtief is building the first part of the branch off to Nordhavnen, consisting of a Station box, Cut & Cover tunnel and a Ramp going towards the surface and an elevated track, see Figure 1.

The retaining walls for the Station Box are made of permanent secant piles supported mainly by 3 layers of temporary pre-stressed ground anchors. The retaining walls for the Cut & Cover tunnel are temporary secant piles supported by 1 and 2 layers of pre-stressed ground anchors. The Ramp area is made with permanent sheet piles and supported by 1 layer of permanent pre-stressed ground anchors.

The deepest excavation, where the TBM drive will start, is around 18 m and decreases along the entire alignment towards the end of the ramp section.

This article will focus on the design of the secant pile walls of the Station Box and the Cut & Cover Tunnel in the temporary situation, where two cross sections will be used as examples. The secant piles have a diameter of 1.2 meter and c/c distance of 0.9 – 1 meter and reinforced by reinforcement cages. The wall is used as cut off wall for the ground water and is therefore drilled and casted until level -20 (DVR90).

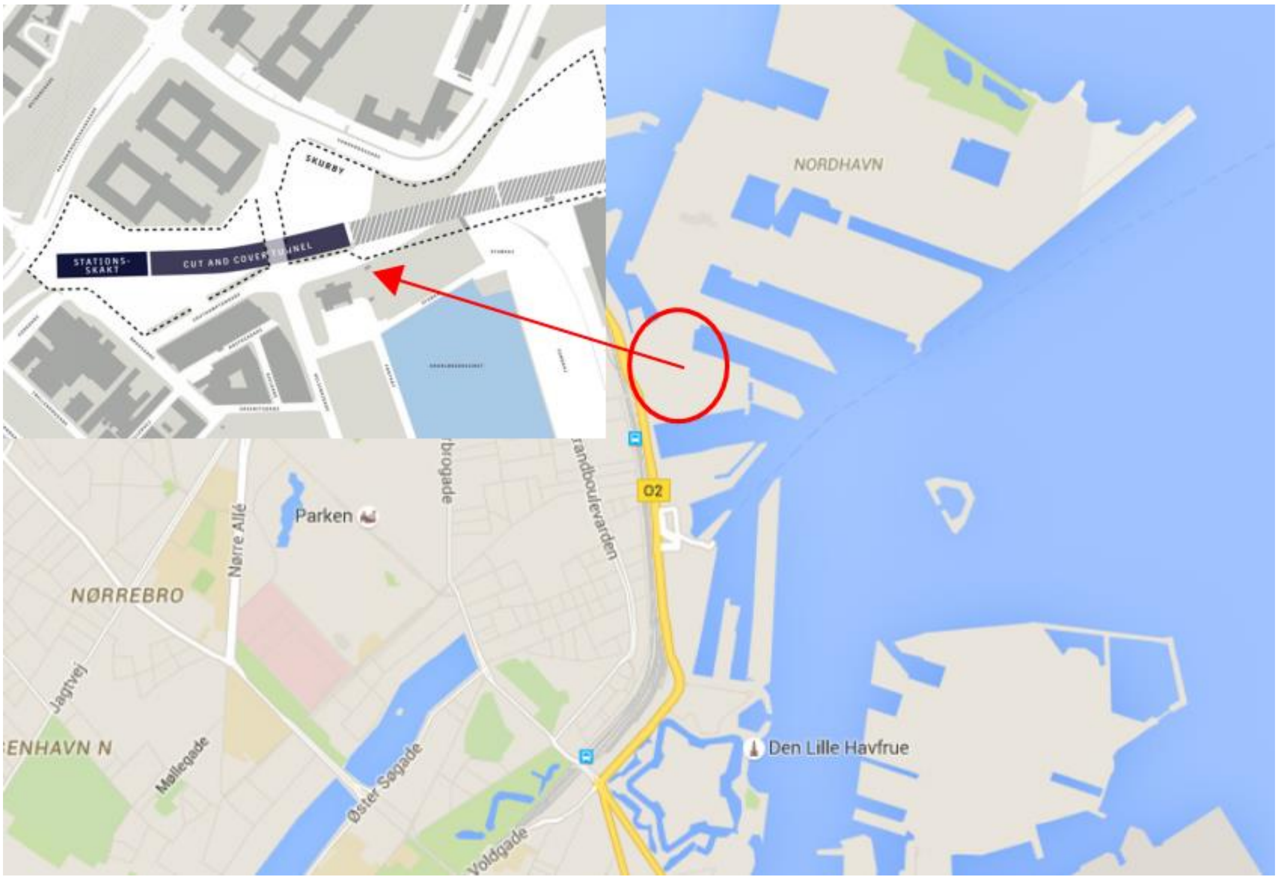

Figure 1: Location of the new Nordhavn Station with Station Box, Cut & Cover tunnel and Ramp (maps.google.dk & m.dk)

2 SOIL AND GROUND WATER CONDITION

2.1 Soil condition

The boreholes made on the location shows fill layers with a thickness variating from 3.5 meter to 11 meter. That matches the fact that it is an old harbour area which is backfilled. Under the fill layer a 2.5 meter to 10 meter thick clay till layer with lenses of sand till and melt water sand is registered. Normally the clay till layer is located on top of the limestone, but in some areas a 6.5 meter thick sand/gravel layer is found. The top level of the limestone is varying throughout the area and has a glacially disturbed zone of around 1 meter. On the safe side a thickness of 2 meter disturbed limestone has been used in the design. The characteristic drained parameters used for the design is shown in Table 2.

2.2 Ground water levels

For the design, a primary and secondary water table is taken into account. The ground water levels used can be seen in Table 1.

Table 1: Ground water level used in the design

Design state	**Primary GWL**	**Secondary GWL**	
		Top level	**Bottom level**
ULS	+1.4	GL	Lower side of fill or clay till layer
SLS	+0.1	GL	

3 DESIGN OF SECANT PILE WALLS

The secant piles are designed in both ultimate limit state (ULS) and serviceability limit state (SLS). The SLS was decisive for amount of reinforcement in the piles as the Employer have strict demands for the crack width at permanent structures. That means that the crack width must not exceed 0.2 mm. In

Table 2: Characteristic strength and stiffness parameters used in the design

Soil layer	Density γ/γ'_m	Plane friction angle φ'	Effective cohesion c'	Oedometer modulus E_{oed}	Poissons ratio ν
	[kN/m3]	[°]	[kN/m2]	[MN/m2]	[-]
Fill	17/19	30	0	3	0.3
Sand	20/20	36	0	15	0.3
Sand till	21/21	40	0	20+1500σ'red[1]	0.25
Clay till	22/22	34	20	12+1500σ'red[1]	0.3
Disturbed Limestone	22/22	45	50	750	0.25-0.30
Limestone	22/22	45	100	900	0.25-0.30

[1] *σ'red [MN/m²] is the vertical stress corresponding to the lowest stress level the soil layer has been subjected to*

many cases the critical part for the crack width was deep under the final excavation level and in that region of the wall which is not necessary for the structural point of view but only as cut off against water.

3.1 SPOOKS

The first design of the secant pile walls was done with the program SPOOKS which uses the earth pressure theory of Brinch Hansen, (Hansen, 1970). The theory is using a combination of zone and line rupture. Furthermore the theory also state, that the necessary displacement to mobilize active and passive earth pressure will be present. From SPOOKS it is possible to get the anchor forces, bending moment and necessary toe level of the retaining wall.

When using SPOOKS it is only possible to add one real anchor layer. When designing multi supported walls, it is necessary to add some of the anchor levels as so called additional pressures. Furthermore it is necessary to check if the anchor force is correct in order to obtain a statically correct solution. The results of the SPOOKS calculations for cross section D and cross section K are shown in Table 3.

Cross section D is a part of the Station box, while Cross section K is a part of the C&C tunnel.

Table 3: Design forces calculated with SPOOKS

	Cross section D	Cross section K
A_1 [kN/m]	230	195
A_2 [kN/m]	636	648
A_3 [kN/m]	388	-
M_{max} [kNm/m]	818	792
Toe level [m]	-16.4	-16.3

The two cross sections are used for comparison as the soil profiles are similar to each other.

3.2 Limitations in SPOOKS

Even though SPOOKS is a well-known program, and has been used in many years, in Denmark, for design of retaining walls, the Employer was concerned about the limitation that it have. The major points for the Employer were:

- Lack of soil-structure interaction, meaning that no stiffness of the system is taken into account.
- Insufficient compatibility between strain and stresses.
- The extra embedment in limestone, which SPOOKS do not take into account.

The Employer was concerned that the deformation necessary to mobilize active earth pressure could not appear as the general system was too stiff. This means that the anchors would be designed for a too low anchor force. Furthermore the Employer would not accept that the extra embedded part was disregarded even though it does not have any statically importance.

Different approaches were used to convince the Employer about the validity of the design. The first approach was a beam calculation with the earth pressures calculated in SPOOKS together with the effective bending stiffness of the wall, which is determined according to DS/EN 1992-1-1. The deformations were then compared with the deformation requirements in DS/EN 1997-1-1 Appendix C.3. As this was not satisfying for the Employer it was agreed to verify the SPOOKS calculation with an ULS calculation in PLAXIS together with the SLS calculation.

3.3 PLAXIS calculation

The PLAXIS calculation is carried out as a 2D plane strain model with a Mohr-Coulomb soil model with drained parameters. The wall is modelled as a plate element and the ground anchors are modelled as node-to-node anchors with geogrids as the bonded length.

The ULS calculations are carried out with the Design Approach function where the partial safety factors are applied to the soil after the SLS calculation phase. This is done for each excavation sequence where the excavation level is 0.5 meter below the anchor level. An example of the cross section with soil stratigraphy is presented in Figure 2.

The pre-stress load for the anchors is based on the anchor forces found from the SPOOKS calculation. The maximum bending moment and anchor force for the ULS and SLS calculation in PLAXIS are shown in Table 4.

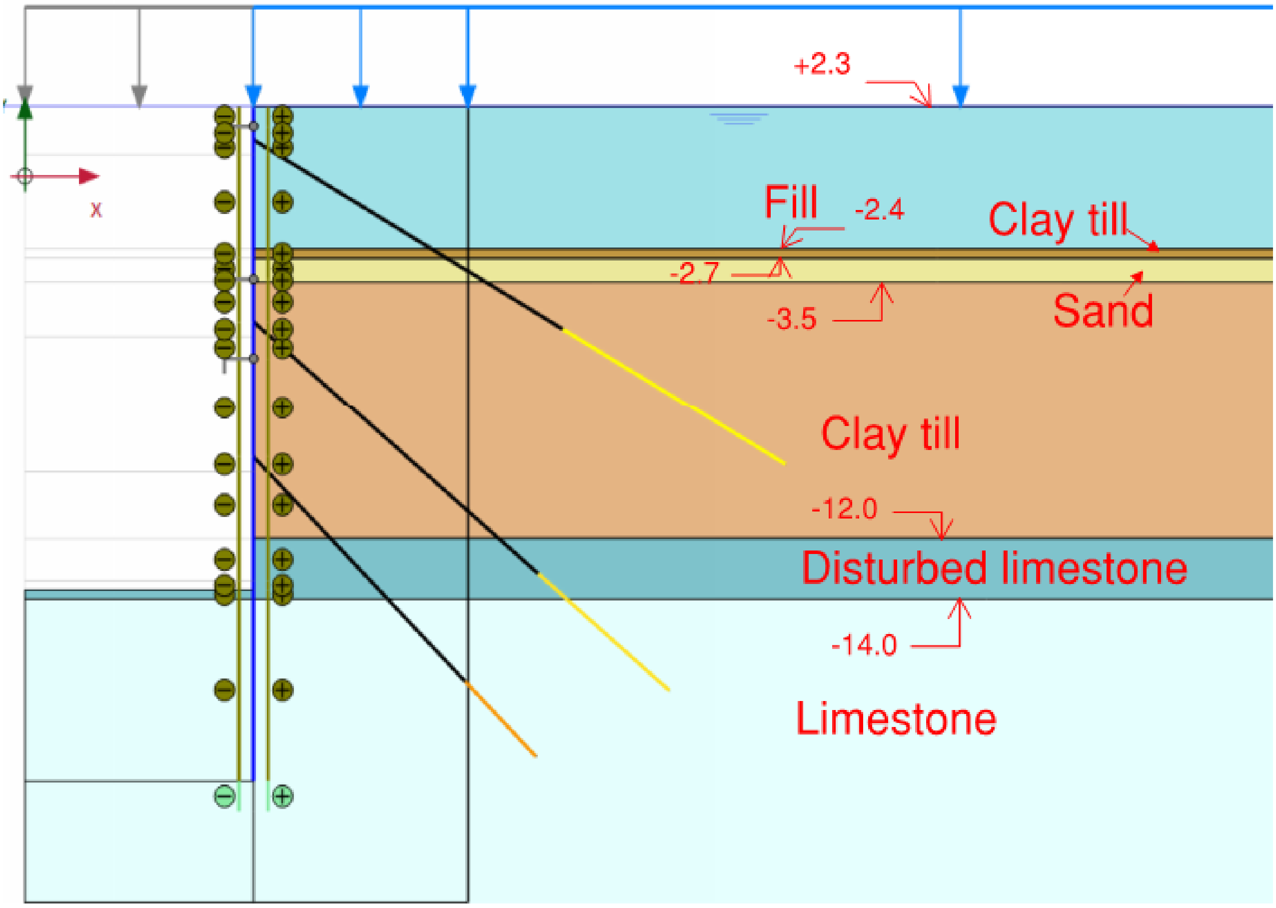

Figure 2: Cross section D with soil layers

Table 4: Anchor forces and bending moments from PLAXIS calculation

	Cross section D	Cross section K
	ULS / SLS	**ULS / SLS**
A_1 [kN/m]	205 / 180	211 / 186
A_2 [kN/m]	688 / 676	661 / 617
A_3 [kN/m]	384 / 367	-
M_{max} [kNm/m]	709 / 721	926 / 781
Toe level [m]	-20	-20

When comparing the results for Cross section D with the SPOOKS calculation, both the ULS and SLS PLAXIS calculation shows higher anchor force for the second anchor layer, while the other anchor forces are within the same range. The bending moment is around 15% higher in the SPOOKS calculation that in the ULS PLAXIS calculation.

In Cross section K is the anchor forces in both ULS PLAXIS and SPOOKS are within the same range, but the bending moment in the ULS PLAXIS is around 17% higher. This is due to the extra embedment in the limestone which makes it possible to carry more loads in the limestone. This is the SPOOKS calculation not able to take into account. The higher load is present in the part which is not statically needed and it can therefore be discussed if it is relevant for the design.

It is clear that the SLS calculations show values close to the ULS values. This indicates that the active and passive earth pressure in ULS and SLS are similar even though the soil parameters are reduced in the ULS calculations. This could be an indication of missing soil-structure interaction, but that will not be investigated further in this article.

4 COMPARISON WITH MONITORING PROGRAM

During the entire construction period a large monitoring program is in place. The monitoring consists of both inclinometers for deformations and load cells on the anchors to measure anchor forces.

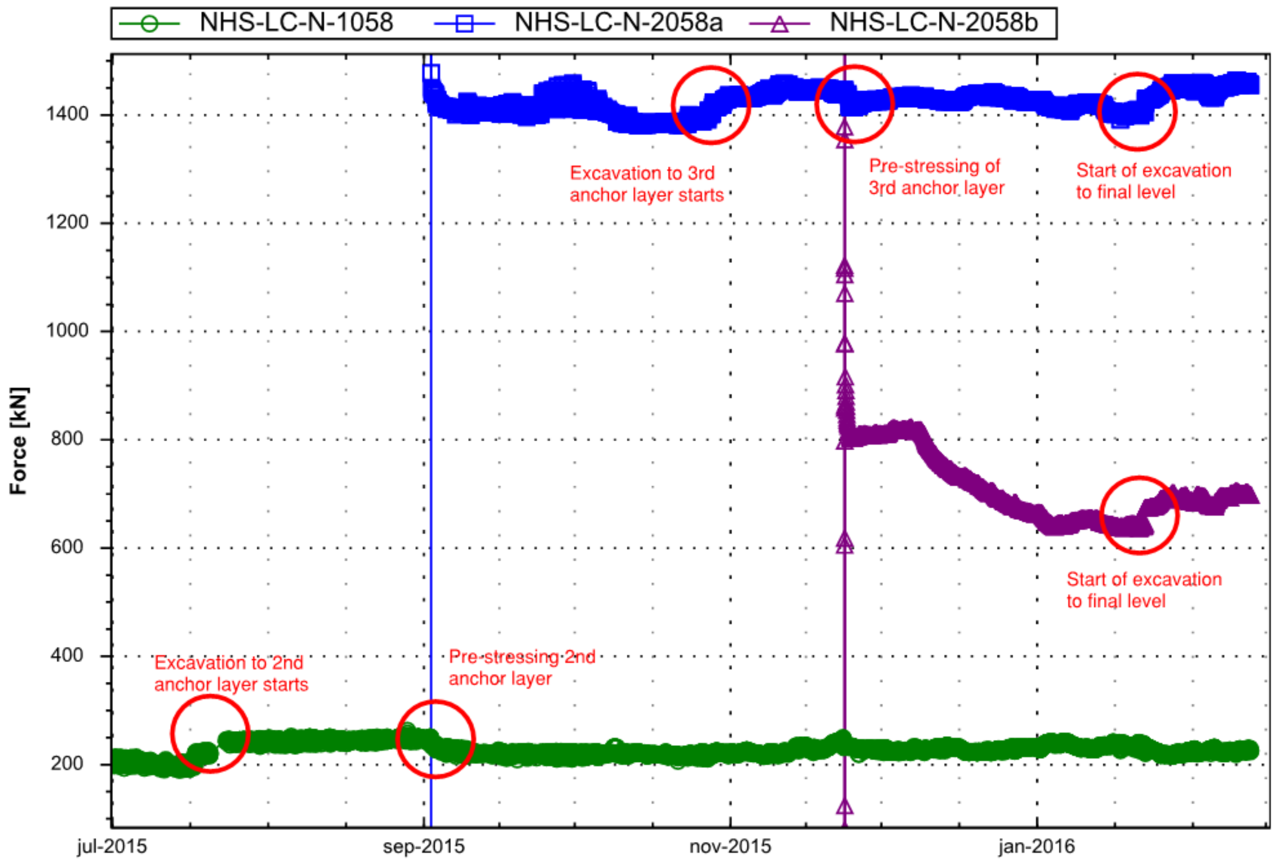

Figure 3: Extract of monitoring program, showing the anchor forces in cross section D for 1st and 2nd anchor layer. The green line is 1st anchor layer, the blue is the 2nd anchor layer and the purple line is 3rd anchor layer.

Looking at the actual anchors forces, see Figure 3, it can be seen that the anchor force in the 1st layer is locked at around 200 kN (87 kN/m). During the excavation to 2nd anchor layer the load increases to around 250 kN, which correspond to 108 kN/m, which is 40% less than calculated in PLAXIS for SLS. When pre-stressing the 2nd anchor the load decreases to around 220 kN (96 kN/m). During the excavation for 3rd anchor layer the load is slightly increased, but decreases again when pre-stressing 3rd anchor layer, meaning the forces is redistributed as expected. For the 2nd anchor layer the pre-stress is around 1400 kN (536 kN/m). As for the 1st layer the anchor load is increasing when the excavation to 3rd anchor layer starts. At the point where the excavation level were reached the anchor force is around 1440 corresponding to 551 kN/m, which is around 18% lower than the calculated SLS value. As the 3rd anchor layer is pre-stressed to around 810 kN (286 kN/m) the anchor force in 2nd anchor layer decreases, as expected, to 1420 kN corresponding to 502 kN/m. During final excavation the load is increasing in both 2nd and 3rd anchor layer and 3rd anchor layer has increases to 700 kN corresponding to 248 kN/m which is 32% lower than estimated in PLAXIS.

It is clear that something is happening in 3rd anchor layer after pre-stressing and it seems that the anchor somehow loses anchor forces even though no activities is going on in the area and therefore no movement of the wall is present. Internally discussions are still ongoing in order to clarify the strange behaviour.

At present there have not been any excavation for Cross section K yet, therefore no measurement can be included in this article.

5 LESSON LEARNED

During the design process of the retaining walls, it has become clear that when designing walls with more than two anchor layers, a kind of Finite Element Method have to be introduced to verify the results founded by the more simple calculation made in SPOOKS. Furthermore the FEM ensure a more correct load distribution to anchors and the wall itself. By introducing FEM in the design it could also reduce the discussion about inadequate compatibility between strains and stresses in the soil as the correct soil-structure interaction is included.
On the other hand one should also keep in mind that the FEM is also just as much a model or method as the SPOOKS calculation and it is therefore difficult to conclude which of the method that is most correct. In the FEM program a lot of different adjustment can be done and it is easy to overlook the consequences when dealing with complex systems. The best way to get an idea of the correct model is to include the monitoring as for example load cells and in that way get an idea of the most correct method and model.

For this project the monitoring shows in general the expected tendencies in respect of increase and decrease of anchor loads during excavation and pre-stressing, but the values of the loads is not in line with the expected values, which could have many explanation, e.g. too conservative soil parameters.

6 REFERENCES

Hansen J.B., "A revised and extended formula for Bearing Capacity", Bulletin 28, the Danish Geotechnical Institute, 1970.