A numerical study of a deep excavation in soft clay in Norway–comparison of 2D and 3D analyses

Etude nummerique d'une excavation profonde en argile mouse-comparaison entre analyses deux- et tredimmensionelles

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ABSTRACT
The Norwegian Public Roads Administration is planning a main road northbound from the city of Trondheim in Norway. This comprises a tunnel through soft sensitive clay (partly quick clay) based on the cut-and-cover method. To limit deformations, the excavation is designed with steel sheet pile walls, ribs of jet grouted piles and steel struts. Comparison of 2D and 3D FEM analyses has been performed. The paper comprises evaluations of all construction sequences down to the final excavation level on short term loading. Soil-structure interaction and bottom heave failure of the excavation pit is given emphasize.

RÉSUMÉ
L'Administration Publique des Routes et Autoroutes en Norvege planifie la construction d'une autoroute a partir de la ville de Trondheim vers le Nord. Le tracée demande la construction d'un tunnel en argiles sensitives, molles (partiellement « quick clay ») en utilisant la methode "cut-and-cover". Pour limiter les deformations, l'excavation est soutenue des rideaux de palplanches métaliques boutonnées et soutenues, sous le niveau d'excavation, avec parois transversales de pieux injectées (jet grouted piles). Des comparaisons sont effectuées entre les resultats des analyses en elements finis en deux et trois dimensions. L'article presente evaluations des resultats pour toutes les phases de construction jusqu'à la phase finale d'excavation, en etat non-drainée. L'interaction sol-structure et la sécurité vis à vis du soulèvement du fond de fouille sont egalement etudiées.

Keywords : Deep excavation, FEM analysis, 2D, 3D, sensitive clay

1 INTRODUCTION

The Norwegian Public Roads Administration is planning a main road northbound from the city of Trondheim in Norway. Multiconsult has been responsible for the geotechnical design in an early phase of one section of the new road. This comprises a tunnel through soft sensitive clay (partly quick clay) based on the cut-and-cover method, which ends up into the bedrock entrance. The maximum excavation depth in soft, sensitive clay is approximately 20 m. The tunnel is planned through a populated area in the city. The solution presented herein is a design from an early stage of the project. Design optimizing is ongoing as this paper is being written, and the final solution might thus be different from the presented one.

To limit deformations, the excavation is designed to be supported by steel sheet pile walls, ribs of jet-grouted piles below excavation level and steel struts at maximum 4 levels. The supported excavation, with complex soil-structure interaction, is designed by use of the finite element method (FEM) code PLAXIS 2D and hand calculations. In addition, advanced analyses are performed with PLAXIS 3D Tunnel to verify the design with respect to structural integrity and soil body collapse for the critical stage. In particular, the structural behavior of the jet-grouted ribs due to the bottom heave failure mode has been important to verify.

2 SOIL CONDITIONS

As commonly found in Norway, the soil conditions consist of approximately normally consolidated, Quaternary deposits overlying hard bedrock.

An extensive soil investigation has been performed along the cut-and-cover tunnel, both by Multiconsult and other companies. The soil investigation includes high quality sampling using the Sherbrooke block sampler. High quality triaxial and direct shear testing forming local correlations with CPTU over the area, have been the basis for the interpretation of the design parameters. For further details of the soil investigation and of design parameters selection, see Tørum et al. 2008 and Rønning et al. 2009. The latter paper is also presented at this conference.

A top layer of manmade fill and marine shore deposits of sand is found from terrain level and down to approximately 2-4 m. Further down, silt and clay and a veneer of bottom moraine over bedrock are present. For the lower support heights, the bedrock surface is located very deep and is sloping up to the rock tunnel entrance. At a section of the tunnel with the largest support height the clay is classified as quick clay. According to Norwegian classification rules, having a remolded undrained shear strength less than 0.5 kPa. The clay is lightly overconsolidated, probably due to aging processes, and is of a very lean nature. This is typical for (quick) clays found in the Trøndelag region. A distinct feature of these clays is also the large variations in compression- (s_u^C), direct- (s_u^{DSS}) and extension (s_u^E) shear strengths. Table 1 shows some basic engineering parameters for the different layers.

Table 1. Basic soil parameters for the critical cross section.

Layer	w [%]	γ [kN/m^3]	Ip [%]	OCR [-]	St [-]
Silt	25-35	19.5	5-7	2.0-3.0	5-10
Clay	25-35	19.5	10-18	1.7-2.0	3-9
Quick clay	30-45	19.5	5-10	1.1-1.8	70-255

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In Table 2, the strength and deformation parameters included in the FEM calculations are presented. It should be noted that the G_{50} values are very high. This is due to the very low strain values at the peak shear stress of the collapsible quick clay, typically 0.5 % axial strain or less. For the standard 54 mm piston samples obtained in this project, sample disturbances and hence lower peak strength values and higher axial failure strains were obtained. Hence, the G_{50} values would be considerable less if they were based on standard investigation methods.

Table 2. Strength and deformation parameters for critical cross section.

Layer	s_u^C [kPa]	G_{50}/ s_{u_m} [-]	s_u^{DSS}/s_u^C [-]	s_u^E/s_u^C [-]
Silt	~25-35[1)]	290	0.76	0.42
Clay	~35-40[1)]	370	0.56	0.29
Quick clay	~40-70[1)]	350	0.60	0.23

[1)] Linearly increasing.

3 2D ANALYSIS

3.1 *Geometry*

The case study presented herein is based on the critical cross section with respect to bottom heave failure along the cut-and-cover tunnel. This cross section has a total support height of 15.4 m, width between the sheet pile walls of 22.5 m, 3 levels of steel struts, jet-grouted ribs with c/c of 5.2 m and a total depth of 9 m below the excavation depth. In addition it is included qeo-concrete above the excavation level in order to minimize the deformations and ease the earthworks. The geo-concrete will be removed sequential as the excavation is constructed. A terrain load of 33.75 kPa in the ULS condition (and conservatively the same also in SLS) was included.

In order to have directly comparable models, the effect of sectional excavation has not been included in this paper. However, this effect can be significant, and has been included in the design solution.

3.2 *Soil model and structural elements*

PLAXIS 8.6 with plane strain conditions has been used in the 2D calculations. The linear elastic-perfectly plastic Mohr Coulomb soil model was applied. A total stress analysis with the 'drained' material model and a Poisson's ratio of $\nu = 0.495$ was used. The cohesion was set equal to the average shear strength, i.e. $s_{u\text{-}m} = 1/3\ (s_u^C+s_u^{DSS}+s_u^E)$, and the friction angle set to $\varphi = 0°$. No ground water is included in the model. The ratio between $G_{50}/\ s_{u\_m}$ in Table 2 were included in the MC model.

Structural plate elements were modeled with elastoplastic plates. The sheet pile wall was modeled with one particular type with a first moment of area of approximately 8000 cm^3/m. The actual weight and axial stiffness for this type was also included. The shear reduction factor at the interface (R_{inter} in PLAXIS) is of great importance to the failure mode. The value has been evaluated based on the actual geometry of the analyzed sheet pile wall and its critical failure mode for vertical shear failure. Also, the reduction in shear strength as compared to the average shear strength for a vertical oriented plane has been given consideration. A value of R_{inter} = 0.97 was included on the outside and R_{inter} = 0.67 on the excavation side of the wall.

The struts were modeled with elastoplastic node-to-node anchors. Dimensions of Ø711x10 mm and Ø813x12.5 mm steel pipes were used, the latter one applicable for the second strut level.

3.3 *Modeling of 3D effect*

In order to account for the positive 3D effect of the jet-grouted ribs below excavation level; several methods were investigated. The method giving the highest level of confidence was to include node-to-node anchors with equivalent stiffness, equally distributed over the depth of the jet-grouted ribs. Hence, it would not influence on a shear surface through the soil beneath excavation level in the c-phi reduction. The 3D effect from the jet-grouted ribs was included by applying a distributed load at the mid-depth of the ribs, which represents the end bearing and side friction distributed into the paper plane. As a reaction to this force, an equal line load of opposite direction was applied at the sheet pile walls. In this manner, both the traditional bottom heave evaluation and the vertical equilibrium of the system as a whole were ensured. From the hand calculations, the 3D effect in the model is considered as follows:

$$\Delta p = \frac{Q_{end\_bearing} + Q_{side\_friction}}{c/c \cdot B} \tag{1}$$

where,

$Q_{\text{end bearing}}$ = undrained end bearing capacity per rib
$Q_{\text{side friction}}$ = undrained side friction per rib
c/c = center to center distance between the ribs
B = width of excavation pit

3.4 *Discretization and staged construction sequence*

A width of 140 m and a height of 46.6 m have been used in the 2D model, which includes both support sides. A total of 1694 triangular 15-node elements are applied. Interface elements are introduced at each side of the sheet pile walls. Standard boundary conditions are used, i.e. fixed vertically and horizontally at the bottom line, and fixed horizontally at the vertical boundaries.

The analysis is performed as a staged construction analysis, taking into account the actual planned construction sequence down to the final excavation depth, see Table 3.

Table 3. Staged construction sequence

Phase	Description
1	Activation of sheet pile walls, jet grouted ribs and load
2	Excavation to 2 m depth
3	Struts at 1.2 m depth activated and prestressed to 50 kN/m
4	Excavation to 6.2 m depth
5	Struts at 5.5 m depth activated and prestressed to 50 kN/m
6	Excavation to 10.7 m depth
7	Struts at 10.2 m depth activated and prestressed to 50 kN/m
8	Excavation to 15.4 m depth
9	c-phi reduction for phase 8
10[1)]	Bottom concrete plate installed
11[1)]	Removal of lower strut level
12-16[1)]	c-phi reduction for phases 2, 4, 6, 10 and 11

[1)] Only performed in 2D analysis

A preload of 50 kN/m has been included for all struts. After the last excavation stage to 15.4 m depth, the bottom concrete plate was activated and the lower strut level removed. The results of these sequences are however not presented in this paper. For each of the stages, a c-phi reduction phase has been performed. The c-phi reduction is stopped at ΣMSF = 1.4.

3.5 *Results*

Figure 1 shows the obtained critical shear surface after c-phi reduction. The failure mode, overlapping from both sides, would not have been accounted for by simplified hand calculations, which are often used for such excavations. Maximum total displacements of 53 mm have been calculated. The deformed element mesh is shown in Figure 2.

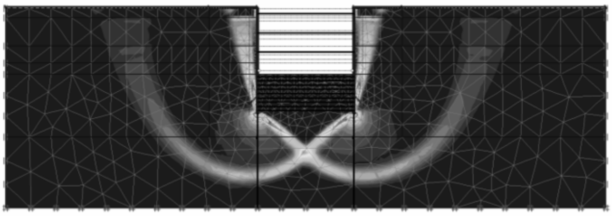

Figure 1. Critical shear surface (incremental shear strains) after c-phi reduction in 2D analysis.

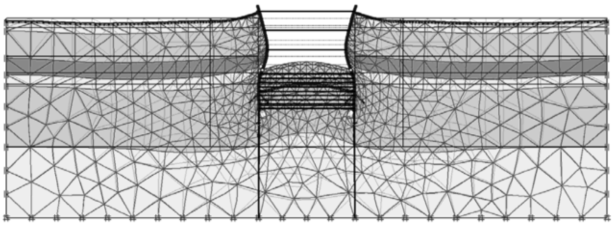

Figure 2. Total displacements (scaled 100 times) 2D analysis

4 3D ANALYSIS

4.1 *PLAXIS 3D Tunnel model*

Additionally, the 3D-effects were evaluated by a more advanced approach, using the FEM-program PLAXIS 3D Tunnel 2.0. The same cross section, with the same geometry, as described for the 2D analysis was analyzed. The model comprises one half of a jet-grouted rib, the entire next rib and the half of the third rib. The 3D-analysis is described in detail by Kirkebø et al. 2008. Solely, a short description is given herein to compare the results with those from the 2D-analysis.

The soil and sheet pile were modeled identically in both approaches. However, the sheet pile walls were modeled with the same bending stiffness about two axes in 3D. The struts and jet-grouted ribs were modeled with a c/c distance of 5.2 m in the longitudinal direction, to capture stress concentrations due to variations in the stiffness of the structure in this direction. The ribs were given an equivalent width of 1.5 m in the longitudinal direction. The behavior was modeled with volume elements with stiffness and strength by use of the Mohr-Coulomb model. Tension was not allowed. Interface elements have been included on these ribs. Also, the reduction in shear strength on a vertical oriented plane has been considered. This is the same principle as used in the hand calculations of the bottom heave, and in the calculation of the equivalent load for the 3D effect in the 2D analysis. In the 3D-analysis, the sheet pile walls were connected to bedrock by use of tension piles with a certain c/c distance.

4.2 *Results*

The 3D-model is showed in Figure 3, with deformations at the final excavation stage.

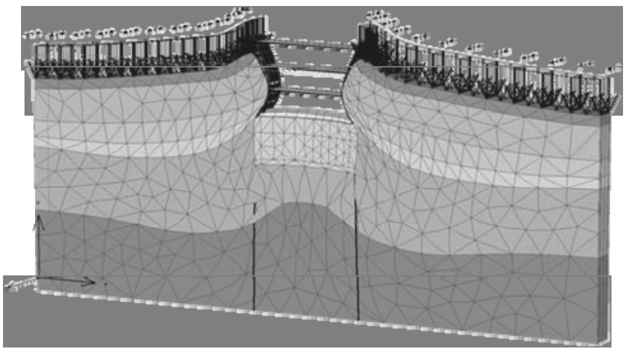

Figure 3. Total displacements (scaled 100 times)

The computed maximum displacement is 41 mm. The 3D analysis has revealed approximately the same critical shear surface as the 2D analysis, see Figure 4. The computed material factor against bottom heave is 1.48 which is somewhat below the required safety factor in the project. Effects of sectional excavation and casting of the bottom plate is however not accounted for in this analysis.

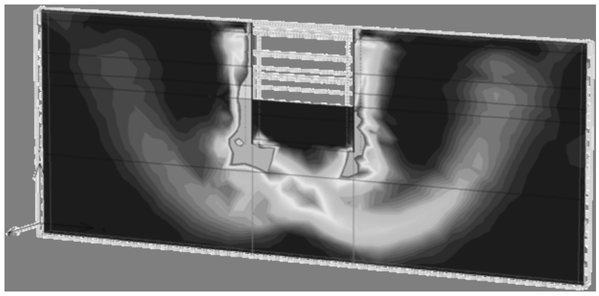

Figure 4. Critical shear surface after c-phi reduction 3D Tunnel

Further results from both 2D and 3D analyses are presented in the next section.

5 COMPARISON OF RESULTS

Table 4 show the extreme values of the bending moments and deformations in the sheet pile wall at the Serviceability Limit State (SLS), for phases 1-8. Table 5 show the bending moments in the sheet pile wall at the final excavation stage at soil body collapse with a factor of safety of approximately 1.4 and 1.5 for the 2D and 3D analysis, respectively.

Table 4. Comparison of results 2D/ 3D analyses SLS, phases 1-9

		2D	3D
Max moment M_{h1} in sheet pile	[kNm/m]	1110[1)]	1659
Max moment M_{h2} in sheet pile	[kNm/m]	n/a	1123
Max total deformations	[mm]	53	41

[1)] For phase 16 this value is larger (1330 kNm/m)

Table 5. Comparison of results 2D/ 3D analyses at soil failure, phase 9

		2D	3D
Max moment M_{h1} in sheet pile	[kNm/m]	1270	2137
Max moment M_{h2} in sheet pile	[kNm/m]	n/a	1432
Resulting safety factor, final excavation depth, F_s	[-]	1.4	1.5

In general, there are two different methods to deal with the ULS condition in FEM analyses. The strength reduction may be applied on the soil strength through a partial material factor (γ_M). This can be done by performing a c-phi reduction in PLAXIS. Alternatively, the internal forces in the structure from an analysis of the SLS condition, can be multiplied with an equivalent load factor (γ_E). According to the European regulations in Eurocode 7, both methods should be applied. An assessment of the two different methods has been performed for the 2D analysis. In this study, both a strength reduction of γ_M = 1.4 (method A) and an equivalent load factor of γ_E = 1.35 (method B) have been used. The resulting extreme values of the internal forcesare presented in Table 6. It should be emphasized that these values are not the safety levels required in the project, but rather values used for the comparisons in this case study.

Table 6. Resulting forces in ULS condition in 2D, phases 1 - 16

		A, γ_M = 1.4	B, γ_E = 1.35	Ratio A/B
Max moment M_{h1}	[kNm/m]	1470	1796	0.82
Max strut force, level 1	[kN/m]	254	307	0.83
Max strut force, level 2	[kN/m]	908	1079	0.84
Max strut force, level 3	[kN/m]	686	481	1.42
Total force in jet-grouted ribs	[kN/m]	3245	5019	0.65

6 DISCUSSION

6.1 *Comparison of 2D- and 3D-analyses*

The calculated factor of safety factor from the 3D analysis is 8 % larger than obtained in the 2D analysis. Resulting safety factors from hand calculations are about the same as from the 2D plane strain calculations.

For both FEM analyses, the critical failure mode is a combined failure mode with a progressing bottom heave failure beneath the excavation level, combined with a vertical uplift failure, i.e. beginning monolithically uplift of the whole excavation pit. For the 2D analysis, the 3D effect will thus be different, depending on whether the soil inside the ribs is moving relatively to the ribs or not. However, with respect to bottom heave failure, the 3D analysis has revealed that the applied stabilizing force in the 2D-analysis is on the conservative side. The 3D analysis has thus verified the simplified calculations with respect to bottom heave failure.

From Table 4 it can be seen that the sheet pile bending moment is underestimated in the 2D analysis. Almost 50 % larger bending moment has been calculated in the SLS condition in the 3D analysis. The deviation between the two methods is believed to be caused by the effect of the jet grouted ribs and struts. The maximum moment in the 3D analysis is located at the front plane of the model, just below the excavation level, i.e. at the location of one jet-grouted rib. This value is more than twice the value midway between two ribs. The 2D model has analyzed the average situation in the longitudinal direction, while the 3D analyses reveal peak values due to variable stiffness of the structure in the longitudinal direction. It is, however, better agreement when it comes to calculated deformations, since these are more governed by the average behavior.

6.2 *ULS method PLAXIS 2D*

From Table 6 it can be seen that the overall safety in general is significantly lower using the partial factor on the shear strength (A) than applying an equivalent load factor on the structural forces (B). However, for the maximum strut force in the lower level, this is vice versa. In the c-phi reduction the maximum strut force occurs at phase 9 with excavation to the final level. At this phase, the soil is at failure, and this might explain the large deviation. All other strut forces in Table 6 occure in other construction phases.

The support system for this excavation pit is a very rigid and strong structure. Hence it is believed that the reduction in the shear strength of the soil is of minor importance for the structural forces, as long as there is some margin to soil body collapse. A redistribution of forces into the jet-grouted ribs occurs as the strength of the soil is reduced in the FEM analysis. Experience from other projects indicate that the method being more on the safe side is depending on the flexibility and capacity of the structure. In cases with a flexible structure, method A seems to be on the safe side compared to method B.

It has previously been argued by Karlsrud & Andresen 2009 that method A should not be used for the design of the structural system, since the soil material factor and structural forces are actually non-linear. This has also been demonstrated in this project, with results rather in the opposite direction of those obtained with more flexible support systems.

6.3 *General Principals*

The analyzed quick clay is of a highly anisotropic nature. It also has a very distinct strain softening behavior, and it could be argued that a more sophisticated soil model should be used to take into account these characteristics of the soil.

In this case, the major part of the failure surface of the bottom heave failure runs through the active zone. Hence, the applied isotropic soil model with a mean value of the shear strength gives results on the safe side with respect to this failure mode. The passive earth pressure inside the excavation is of minor importance, since the jet grouted ribs limit the deformations.

Advanced soil models, taking into account anisotropy and strain softening are now commercially available. However, the models are still very time consuming to use, and are known to involve numerical difficulties, particularly for complex geometry models as in this case. Based on this, and the fact that little gain was expected, it was decided to use the isotropic soil model, since the average shear strength is reasonably representative. This is also concluded in the work by Karlsrud & Andresen 2007.

For this problem, the calculated strains are very small and well below the peak shear strength. In order to account for the stress dependant soil stiffness for the post-peak behavior it is possible to model the problem with the hardening soil model in PLAXIS. With such an approach, fictitious values of c and φ must be included in order to represent the undrained shear strength. For this project, this exercise has been performed, and the results are comparable to the elastic-perfectly plastic MC model. The resulting stresses and displacements are in this model considered more reliable. Due to space limitations, these analyses will however not be discussed further.

7 CONCLUSIONS

The 3D analysis has verified the more simplified 2D analysis with respect to bottom heave failure. The 2D analysis underpredicts the maximum sheet pile wall bending moment with approximately 50 %. In the 3D analyses, the stress concentrations in the longitudinal direction are captured. Thus, utmost care should be shown when designing truly 3D problems using plane strain conditions. However, 3D models are time consuming, and thus somewhat impractical to use in traditional design.

Two different methods of handeling the ULS condition have been evaluated. It has been demonstrated that applying the partial material factor on the shear strength (A) in this case is less safe than applying an equivalent load factor on the structural forces (B) when the soil is not at failure.

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