Design of a deep tunnel in a layer of a normally consolidated clay

Dimensionnement d'un tunnel profond dans l'argile normalement consolidée

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ABSTRACT

The paper describes the procedure which was used to design a 400 m deep section of the tunnel in weak normally consolidated clays. A short description of geological conditions typical for a particular location is followed by the explanation of the geotechnical model used, including the design material parameters. The geotechnical model was based on the behaviour of normally consolidated clays when subjected to unloading, in this case caused by the excavation of the tunnel. The role of the rise of the resulting negative excess pore pressures on strength and stiffness of clays is discussed for undrained, consolidation and fully drained conditions. The full sequence of the construction of a 5 m diameter tunnel was modelled using both the 3D and 2D finite element analyses. The influence of the suction limit posed for the clays on the results of the finite elements analyses are discussed in some detail. Finally, the results of the analyses are presented and the conclusions are discussed with an emphasis to their relevance to the final design solution.

RÉSUMÉ

L'article décrit la procédure qui a été utilisée pour concevoir une section de 400 m de profondeur dans une couche de l'argile faible, normalement consolidée. Une brève description des conditions géologiques typiques d'un lieu particulier est suivi par l'explication de la modèle géotechnique utilisée, y compris les paramètres matériels. Le modèle géotechnique est fondé sur le comportement des argiles normalement consolidées lorsqu'elles sont soumises au déchargement pendant l'excavation du tunnel. L'inluence des surpressions interstitielles négatives sur la résistance et la rigidité de l'argile est examinée pour les conditions nondrainées, conditions de la consolidation et les coditions drainées. La séquence complète de la construction d'un tunnel de 5 m de diamètre a été modélée en utilisant à la fois le calcul 2D et 3D par éléments finis . L'influence de la valeur limite de succion imposée à l'argile sur les résultats de calcul par éléments finis est discutée en quelque détail. Enfin, les résultats des analyses numériques sont présentés et les conclusions sont examinées avec une attention particulière à leur pertinence pour la solution finale de la conception.

Keywords : normally consolidated clays, suction, numerical analyses, tunnel design

1 INTRODUCTION

The analyses presented in the paper were based on a particular geological sequence which is characterised by several mutually bound formations. A particular clay formation found in this sequence is a structurally complex clayey-marly material, which is embedded in the intermittent limestone and flysch formations. This tectonically reworked material can be found at substantial depths of up to 400 m. For the purpose of geotechnical modelling this material stratum is best described as normally consolidated clay.

The paper presents the results of the analyses of the stability of a small diameter deep tunnel in this type of the material. The numerical model was based on a hydro-technical tunnel with an outer diameter of 5.0m and inner diameter of 3.4 m. The paper describes the design philosophy behind the analyses and presents the interpretation of the results. The results and the conclusion of the analyses should not be regarded as solely bound for the particular location or the type of the tunnel, as they present a general assessment of this type of a boundary value problem.

2 MATERIAL PARAMETERS

A comprehensive laboratory testing including index tests, oedometer tests, direct shear and drained and undrained triaxial tests were considered to form a geotechnical model. At high depths the samples were classified as fully saturated clay of high plasticity, with liquid limits LL ranging between 60-80% for the corresponding index of plasticity PI lying within the 30-50% range.

For the samples at high depths of up to 300m the oedometer tests revealed normally consolidated material while triaxial and direct shear tests provided with the strength parameters including apparent cohesion of 0kPa and the peak angle of shear resistance ranging between 18° and 22° degrees. The material parameters used in analyses are summarised in Table 1. The categories of the lower and the upper bound parameters are denoted for the ˝very poor˝ and the ˝poor˝ type of clay, respectively.

Table 1. Material parameters for ˝very poor˝ and ˝poor˝ clay material used in the analyses

Parameter	Very poor	Poor	Units	Description
c' =	0	0	kPa	Apparent cohesion
φ' =	18	22	deg	Angle of shear resistance
OCR =	1.0	1.0		Overconsolidation ratio at high depths
κ =	0.025	0.025		The gradient of unloading-reloading line
λ =	0.085	0.035		The gradient of the normal compression line
ν =	0.3	0.3		Poisson coefficient
γ =	22.0	22.0	kN/m^3	Unit weight

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The undrained shear strength c_u was not reliably evaluated in the laboratory or in-situ. Some theoretical evaluations were carried out in order to make the best estimate of the appropriate value using the constituent equations of Cam-Clay model (Schofield and Wroth, 1968), according to which the value of the undrained shear strength can be estimated from the values given in Table 1. For the given model and the parameters the relationship between the vertical effective stress and the undrained shear strength can be calculated for the ˝very poor˝ material as:

$$c_u = 0.18\sigma_v^{'} \quad (1)$$

where $\sigma`_v$ is the in situ vertical effective stress. Slightly higher value is expected for the material denoted as ˝poor˝.

D. M. Wood (1990) gives the empirical results for several clays of different plasticity index and demonstrates from the laboratory investigations that the empirical correlation of:

$$c_u = 0.25\sigma_v^{'} \quad (2)$$

was found to be the lower bound for most of the soils. The undrained shear strength was therefore estimated to range between $c_u=(0.18\text{-}0.25)\sigma`_v$. For example, for the 400 m overburden and with 100m of the height of the water table the value of the undrained shear strength can be expected to vary between 1.4 MPa and 1.9 MPa.

3 TUNNEL DESIGN

The design philosophy behind the proposed solution for the executive phase of the tunnel construction is based on the evaluation of the soil properties and on the experience of the tunnelling in the soft ground and high depths. There are several sources of the relevant case histories presented in the literature such as Peck (1969) and Mair et al. (1992).

The design philosophy is based on the established knowledge that the response of the fully saturated, normally consolidated clays of low permeability on the excavation of the deep tunnel is predominantly undrained. Due to inability to change the volume the soil responds with an increase of the negative excess pore pressures, which consequently increases the effective stresses and thus the shear strength of the soil. This mechanism enables even the relatively weak materials to have so called a ˝stand up time˝ that allows for the installation of the primary lining, with an immediate task to take some of the load from the ground.

For the relatively weak material it is then of utmost importance to disallow swelling while the negative excess pore pressure dissipation starts to take place around the perimeter of the cavity after the excavation. This is the reasoning behind insisting that the primary lining ring must be closed as soon as possible or, more precisely, as soon as the technological sequence allows for it. For the same purpose of reducing the displacement and further unloading and consequent swelling of the ground the supporting of the face of the excavation is predicted using the anchors with the high capacity in tension.

Due to the room constrains at the face of the excavation the primary lining ring is fully closed some 4.4 m far for the excavation face. In turn, the primary lining is relatively quickly supported by the secondary lining, which must be installed not further than 50m behind the excavation face.

The consolidation phase that follows as a result of the distribution of the negative excess pore pressures is taking place within the contained boundary conditions. Due to impermeability of the tunnel lining the dissipation takes place solely within the body of the surrounding soil, thus extruding some extra capacity of the ground in the long term.

The characteristic cross-section of the tunnel is shown in Figure 1. The lining of the tunnel is divided into the primary lining and the secondary lining. Between the primary and the secondary lining there is the watertight membrane, which is protected by the geo-textile. The technical details for all the materials used in the lining of the tunnel are shown in the same drawing.

The excavation of the tunnel is carried out in steps. Each excavation step is followed by and immediate installation of the support element of the primary lining. The length of the step of the excavation of the tunnel varies between 0.5 and 1.2 m, depending on the height of the overburden. Every six meters of the excavation the reinforcement of the excavation face is carried out using the 25.5 m long anchors. The primary lining comprises the following elements:

1. microfibre reinforced shotcrete CP30 of nominal thickness varying between 25 cm and 30 cm, depending on the height of the overburden and the quality of the rock mass
2. 2 IPE 180 or 2 IPE160 steel profiles, placed at the distance varying between 0.5 and 1.2 m depending on the height of the overburden and the quality of the rock mass
3. Reinforcement mesh ∅6 mm/15 cm at the inner side of the primary lining

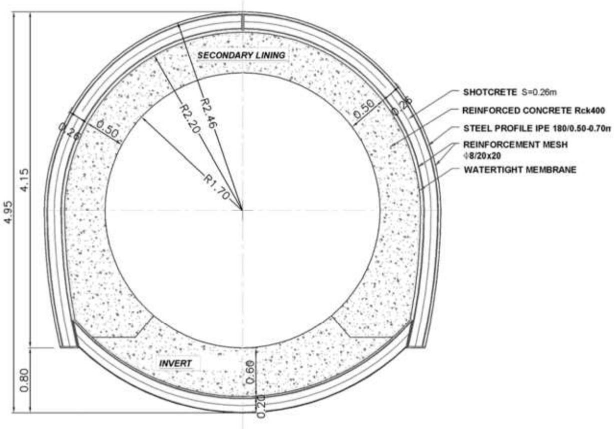

Figure 1. The characteristic cross section of the tunnel

The secondary lining is made of the reinforced concrete Rck40MPa, which is cast in place in two phases. In the first phase the reinforced concrete is cast in the invert following the two steps of the excavation and the support using the primary lining. Secondary lining is then cast in full not further than 50 m behind the excavation face.

4 NUMERICAL ANALYSES

The numerical analyses were carried out using the finite element program PLAXIS. The analyses were carried out both as 3D analyses and as plane strain 2D analyses. Generally, the 3D analyses were used to model the short term response that takes place during the construction of the tunnel while 2D analyses were used to model the long term conditions. The analyses are related as the results of the 3D analyses were used as the input to 2D analyses in terms of the share of the load attributed to the ground, primary lining and the secondary lining, respectively.

Soft soil model, as defined in PLAXIS, was chosen to model the soil behaviour. This elastic-plastic model that takes into account isotropic hardening and features a plastic cap in volumetric compression is based on the Modified Cam Clay model. The difference between the two models is that the Hvorslev surface modelled as part of the ellipse in Modified Cam Clay Model is replaced by the linear Mohr-Coulomb envelope. This model is widely accepted as the non-linear model that is appropriate for normally consolidated clays (D. M. Wood, 1990).

Numerical analyses were carried out with an aim to demonstrate: short term stability of the tunnel during the construction, long term stability of the tunnel, and serviceability of the tunnel in the long term.

4.1 *3D Analyses: Short term stability of the tunnel during the construction*

The 3D analyses were carried out only for the ˝very poor˝ type of material, adopting the conservative approach. The purpose of the 3D analyses was to demonstrate that is possible to construct the tunnel for a given excavation and supporting sequence and to evaluate the distribution of the load to the surrounding ground, primary lining and the secondary lining.

The analyses were carried out for the most onerous case for which 400 m of overburden was accounted for. The water table was assumed to be 100 m above the axis of the tunnel which was a reasonable assumption for given hydro-geological conditions. Finite element mesh had 4082 elements and was modelling 54.6 m long block of material composed of thirteen 4.2 m thick slices, which was chosen for the convenience of better representing the technological sequence. In the cross section the FE mesh is 20 m wide, 35 m high (15 m below and 20 m above the tunnel). The symmetric profile is assumed and only half of the profile is calculated. The overburden is modelled by 1 m thick layer with appropriate unit weight in order to represent the weight of 400 m high rock mass.

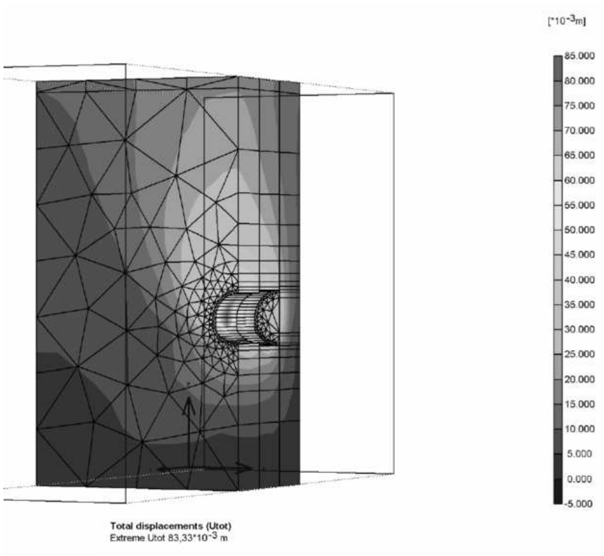

Figure 2. 3D analyses – calculated total displacements

The tunnel primary and secondary lining are modelled by the volume elements in order to represent accurately the position and thickness of both linings in comparison with plate elements that have zero thickness. The overburden and tunnel linings are assumed as the linear elastic materials. The ground, normally consolidated clay is modelled as the soft soil creep model and as an undrained material. No creep was modelled. The effective stress parameters for the soft soil model were used for ˝very poor˝ material, as shown in Table 1. The secondary lining is installed in one stage following the 50 m of the tunnel supported only by the primary lining.

The construction of the tunnel was modelled in 10 short term steps: excavation of the 4.2 m long section, consequent installation of the primary lining in 4.2 m long section. The installation of the secondary lining was modelled in a single stage. At the end of the short term stages the consolidation analyses was carried out in order to model the long term conditions. In this way it is assumed that the load due to transition from undrained to drained conditions (short to long term) is taken mostly by the ground and the secondary lining, which will take the full hydrostatic pressures. Before the consolidation stage the stiffness of the primary lining was reduced 80% in order to model the partial plastic behaviour of the lining, which was at this stage at the end of the capacity range.

The effect of the aging of the shotcrete was modelled by the gradual increase of the elastic stiffness Young´s modulus. Immediately after the installation of the primary lining, the modulus is set to be 9 GPa, two steps behind the face 20 GPa and the four steps behind face 25 GPa.

Initial ground water pore pressures are linearly distributed with depth assuming the water table 100 m above the tunnel. Initial effective stresses are obtained from the soil unit weight and the k_0 value taken according to Jaky (1944) as k_0=1-sin18°=0.69 for the ˝very poor˝ type of material.

The construction sequence was modelled in following steps: 1. step: excavation of the one 4.2 m long tunnel section, 40% of the geostatic stresses released. One slice ahead of face is "extra reinforced", further two slices ahead of face are reinforced; 2. step: primary lining (E=9 GPa) is installed in previously excavated section and full geostatic stresses are released. In the 3rd slice behind face the primary lining is attributed the modulus of E=20 GPa and for the 5th slice behind the excavation face the modulus of the primary lining is set to 25 GPa. Steps 1 and 2 are repeated along the model up to the end of the excavation stage. The next steps were carried out as follows: 3. step: installation of the full 50m of the secondary lining in one stage; 4. step: reduction of the stiffness of the primary lining for 80%; 5. step: consolidation stage

The results of the 3D analyses show that the maximum total displacement prior to installation of the secondary lining are expected to be lower than 10 cm, with the distribution as shown in Figure 2. The calculation showed that the full geostatic pressures were approximately distributed as follows: 70% to the ground and 30% to the tunnel linings. This is in good agreement with the cases that were documented in the literature for deep tunnels in normally consolidated clays, e.g. Mair et all (1992). Further on, the corresponding load of the tunnel lining was found to be approximately equally divided to the primary and the secondary lining, that is, that each of them takes approximately 15% of the full geostatic load. These results were used as an input data to perform the 2D analyses for the long term behaviour assuming the same distribution of the forces within the ground and the tunnel linings.

4.2 *2D Analyses: Short term and long term stability of the tunnel*

The 2D numerical analyses were carried out as the continuation of the 3D analyses. The model was set using the same material parameters, the same 400 m overburden and the same 100 m elevation of the water table.

The amount of the redistribution of the stresses, which is the input parameter for 2D analyses, was derived from the results of the 3D analyses following the criterion of the equal displacement magnitude caused by the tunnel excavation for the two boundary value problems. Following this criterion, the inner forces in the primary and the secondary lining were also checked and were found to be in reasonable agreement. Given the fact that 2D analyses are much faster and more economical to carry out the continuation of the analyses was carried our solely using the 2D analyses and for different amount of overburden, that is: for 300 m, 200 m and 120 m. In all the cases the level of the water table was kept at 100 m above the axis of the tunnel. The aim of the 2D analyses was as follows: a) to check the structural integrity of the supporting elements for both primary and secondary linings and b) to check the stability of the tunnel in the long term.

Numerical analyses were carried out using finite element program PLAXIS 2D- version 8.2. The analyses were carried out assuming the plane strain boundary condition and using effective stress parameters for the two types of materials: ˝very poor˝ and ˝poor˝, as shown in Table 1. The chosen material model was Soft soil model, which was the same as for 3D analyses. Finite element mesh had 1061 elements and was modelling a section that is 130 m high and 100 m wide. The tunnel model was some 30 m above the lower boundary and some 90 m below the upper boundary. The boundary value problem was treated as symmetrical.

The ground is modelled as the soft soil model and as an undrained material in the short term. As previously emphasised, the generation of the negative excess pore pressures (i.e. suction) gives to the material a temporary "cohesion" in order to be stable during excavation. In the case of the 2D analyses the negative excess pore pressures were limited by the value of ´´cavitation´´, which is essentially the suction limit, that can be taken by the material. This suction limit was fixed at 1000 kPa, which was found to be appropriate for similar materials. Initial stresses were set up in the same manner as for the 3D analyses.

General approach to the analyses was the modelling of the excavation of the tunnel as partial reduction of the confinement pressure that originates from the in-situ stresses, as provided by the programme to allow for the modelling of the 3D effect. The sequence that was modelled was taken from the results of the 3D analyses, as explained before.

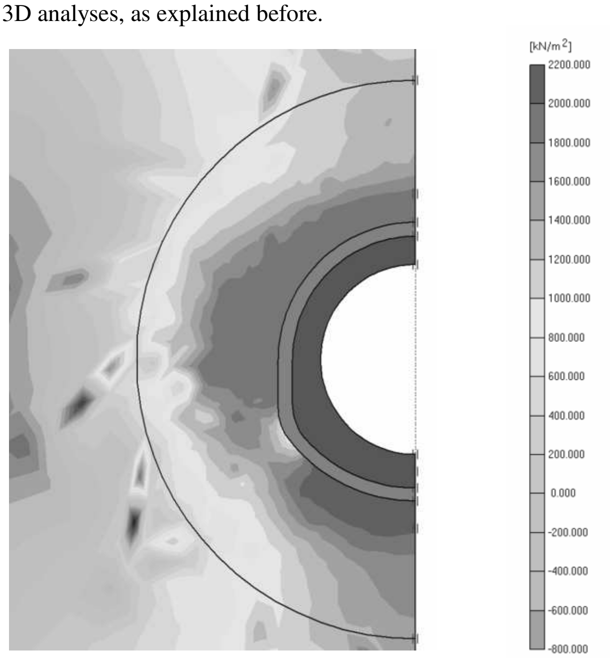

Figure 3. The distribution of the total negative excess pore pressures for the 400m overburden

For the material denoted as ˝very poor˝, the summary of the results is as follows. The initial convergence movement for the unsupported tunnel are calculated to be between 80 mm for 120 m overburden and about 135 mm for 300 m and 400 m overburden. Slightly higher value of 190mm was observed for the 200 m overburden, which can be attributed to the influence of the arbitrary suction limit value. Maximum cumulative long-term convergence displacement varies between 94 mm for 120 m overburden and 150 mm for 300 m and 400 m overburden. Again slightly higher value of 202 mm was observed for the 200 m overburden, which can be attributed to the influence of the arbitrary suction limit value. The total negative excess pore pressures vary from around 1.3 MPa for 120 m overburden to 2.2 MPa for 200 m, 300 m and 400 m overburden. These are the suction values that are believed to be appropriate for the normally consolidated clays and are easily contained within the clay matrix. The distribution of the total negative excess pore pressures for the 400 m overburden is shown in Figure 3.

Maximum mobilised deviatoric stresses vary from 0.76 MPa for 120m overburden to 3.5 MPa for 400 m overburden. This implies maximum mobilised shear stress in undrained conditions of 0.37 MPa for 120 m overburden to 1.75 MPa for 400 m overburden. This is in good agreement with the expected values of the undrained shear strength, which were discussed in Section 2.

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Maximum inner forces in the primary and the secondary lining were found to be below the required capacity. The required capacity of the secondary lining is achieved with the relatively light reinforcement.

5 CONCLUSIONS

The 3D numerical analyses of the stability of the tunnel using finite element method showed that is possible to construct the tunnel for a given technological sequence and the choice of the supporting elements. Four typical load cases have been analysed using the 2D finite element analyses for two sets of material parameters: a) ˝very poor˝ set of parameters that correspond to the lower limit value of the expected stiffness and strength of varicoloured clays and b) ˝poor˝ set of parameters that correspond to the upper limit value of the expected stiffness and strength of varicoloured clays. These were carried out for four different values of overburden: 400m, 300m, 200m and 120m, eight analyses in total. The level of water table was assumed to be 100m above the tunnel axis in all cases. The amount of the redistribution of the stresses, which is the input parameter for 2D analyses, was derived from the results of the 3D analyses following the criterion of the equal displacement magnitude caused by the tunnel excavation.

The stability of the tunnel in the long term was checked following the consolidation stage after the predominantly undrained event of tunnel construction. Results from the analyses confirmed numerically that the concept of building a deep tunnel in saturated normally consolidated clays is possible (feasible?). This finding was used to design the primary and secondary support of the tunnel. The care has been taken that the both sets of analyses were based on the reasonable and conservative assumptions, which ware derived from the available geological and geomechanical data and the theoretical knowledge and the experience documented in the literature.

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