Proceedings of the XVIII ECSMGE 2024
GEOTECHNICAL ENGINEERING CHALLENGES
TO MEET CURRENT AND EMERGING NEEDS OF SOCIETY
© 2024 the Authors
ISBN 978-1-032-54816-6
DOI 10.1201/9781003431749-484
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Finite element analyses of slurry trench stability and deformations – back-calculation of field tests in soft clay

Analyses par éléments finis de la stabilité et des déformations des tranchées à coulis - Rétro-calcul des tests de terrain en argile molle

E. Nilsen*, K. Karlsrud, E.J. Lande
NGI, Oslo, Norway

**eirik.nilsen@ngi.no*

ABSTRACT: The slurry trench method has, since the early 1970s, been used in Norwegian soft clays to construct diaphragm walls to support deep excavations. The technique involves excavation of a deep trench stabilized by a fluid/slurry that exerts a pressure on the trench walls. When the method was used for construction of diaphragm walls for a subway and railway tunnel through central Oslo in the 1970s there was little experience with the method in soft clays. Therefore, a field test program with several test trenches was initiated which included monitoring of ground settlements, pore pressures, and horizontal deformations at depth. The present study analyses results from two slurry trenches at Vaterland, one of size 1×5 m and the other of size 1×1.8 m. These trenches were deliberately brought to failure by drawdown of the supporting slurry. Finite element analysis (FEA) is used to back-calculate the observed behaviour with respect to safety and deformations. The NGI-ADP model is used for the clay at the test site. The results are compared with the observed behaviour and the factor of safety calculated using a simplified analytical approach proposed earlier. The analyses show that the NGI-ADP model can predict failure with satisfying accuracy, and that effect of the trench shape on the stability is well captured.

RÉSUMÉ: La méthode des tranchées à coulis est utilisée depuis le début des années 1970 dans les argiles molles norvégiennes pour construire des parois moulées destinées à soutenir des excavations profondes. La technique consiste à creuser une tranchée profonde qui est stabilisé par un fluide/boue, en exerçant une pression sur les parois de la tranchée. Lorsque cette méthode a été utilisée dans les années 1970 pour la construction de parois moulées pour un tunnel de métro et de chemin de fer traversant le centre d'Oslo, il y avait peu d'expérience dans son utilisation avec les argiles molles. Un programme de tests sur le terrain impliquant plusieurs tranchées d'essai a donc été lancé. Le programme comprenait des mesures de tassement, des pressions interstitielles et des déformations. La présente étude analyse les résultats de deux tranchées à coulis à Vaterland, l'une de dimension 1×5 m et l'autre de dimension 1×1,8 m. Ces tranchées ont été délibérément poussées jusqu'à la rupture en abaissant le niveau du coulis de support. L'analyse par éléments finis (FEA) permet de rétro-calculer le comportement observé relatif à la sécurité et les déformations. Le modèle NGI-ADP est utilisé pour l'argile sur le site de test. Les résultats sont comparés au comportement observé, et le facteur de sécurité est calculé à l'aide d'une approche analytique simplifiée proposée précédemment. Les analyses montrent que le modèle NGI-ADP peut prédire la rupture, et que l'effet de la forme de la tranchée sur la stabilité est bien capturé.

Keywords: Slurry trench; stability; FEA.

1 INTRODUCTION

The slurry trench method for constructing, for example, diaphragm walls is a technique that has been used in Norwegian soft clays in the past and is seeing an increased relevancy today. The technique involves excavation of a deep trench that is stabilized with a suitable fluid, called slurry.

In Norway, the slurry trench method was used during the construction of the *Fellestunnelen* rail- and subway tunnel in central Oslo in the 1970s, involving construction of diaphragm walls to depths up to 28 m. At the time there was little experience with this method in soft clays typical to Norway.

Therefore, test sites were established at three locations in central Oslo. The tests included trenches of different sizes with extensive instrumentation and monitoring. The collected data include ground surface settlements, pore pressures, trench wall deformations, and horizontal deformations measured by inclinometers, collected from slurry trenches in Kongens gate, Vaterland (DiBiagio *et al.*, 2020), and Studenterlunden (DiBiagio and Myrvoll, 1972). Schäfer and Triantafyllidis (2004) and Grandas-Tavera and Triantafyllidis (2012) have previously analysed slurry trenches at the Studenterlunden test site using a visco-hypoplastic constitutive model.

In this study, the data collected at the Vaterland test site is revisited. The site includes two trenches, one 1×5 m and the other 1×1.8 m. Finite element analyses (FEA) is used to back-calculate the observed behaviour using the NGI-ADP constitutive model. The aim to investigate the ability of the NGI-ADP model to predict the safety and deformations of slurry trenches through 3D modelling of the field tests.

2 BACKGROUND

2.1 Soil conditions

The terrain at the Vaterland test site is approximately level and situated at elevation +2 m above sea level. The bedrock surface forms a gulley (Figure 1). The trenches are situated roughly at its centre, where the depth to bedrock is 18–22 m. In the northwest and southeast directions, the depth to bedrock decreases.

The soil consists of a 1–2 m thick layer of fill material over a deposit of normally consolidated marine clay. The top part of the clay has been weathered to form a dry crust extending to a depth of 2–4 m. Underneath, is a soft marine clay with liquid limit of 40–45 %, plastic limit of about 25 %, and water content of 35–40 %. The ground water table is located at a depth of about 1 m.

The undrained shear strength (c_u) of the clay is estimated based on vane tests and triaxial tests on piston samples from the site, and CPTUs from a nearby location (Figure 2). For the vane tests, the correction factor proposed by Aas (1986) is used to estimate the average field c_u. The active shear strength c_{uA} is obtained using a ratio of direct to active strength calculated as proposed by Karlsrud and Hernandez-Martinez (2013). The CPTUs were interpreted using the correlations presented by Karlsrud et al., (2005). An overall assessment of both the N_{kt} correlation and the $N_{\Delta u}$ correlation for six different CPTUs is the basis for the interpreted profile in Figure 2.

The final input profile for the FEA is determined by an overall assessment of the different data, considering the likely effect of sample disturbance on the anisotropically consolidated undrained compression test (CAUC) results (Karlsrud and Hernandez-Martinez, 2013).

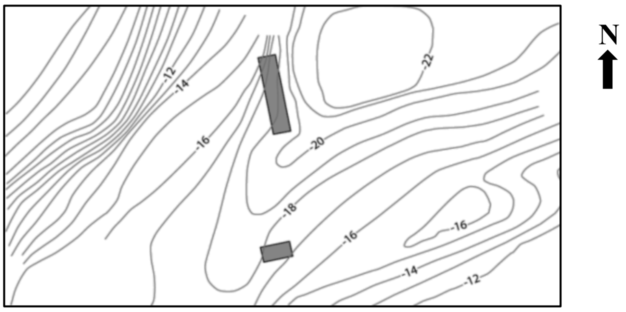

Figure 1. Map of bedrock elevation at the slurry trench test site and the location of the two trenches (shaded).

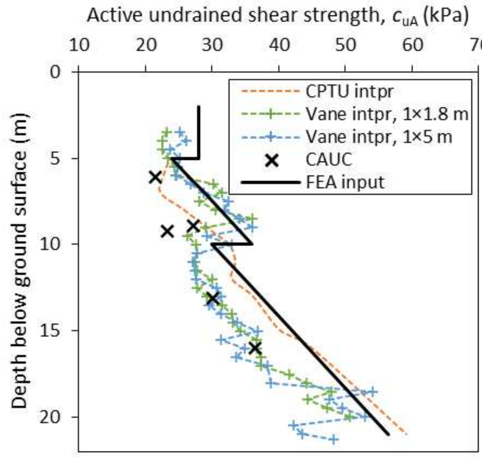

Figure 2. Active undrained shear strength vs. depth.

2.2 Instrumentation and test timeline

The trenches were instrumented and monitored with hydraulic deformation gauges, settlement anchors inclinometers, and pore pressure. The instrumentation layout is described by DiBiagio et al., (2020).

Excavation started February 26, 1970, by excavation of the 1.8 m trench. Then, the 5 m trench was excavated, followed by a series of tests involving replacement and drawdown of the stabilising fluids (Table 1). The slurry had an average specific gravity of 1.17, while the oil had a specific gravity of 0.85. Failure was initiated in the 1.8 m trench during drawdown to 8.3 m depth, while failure of the 5 m trench was initiated by drawdown to 4.7 m. Drawdown was performed at a rate of 1 m every 30 min.

Table 1. Timeline for the slurry trench test at Vaterland, reflected in the FEA calculation phases.

Phase	Date
Excavation of 1.8 m trench with slurry	26 Feb.
Excavation of 5 m trench with slurry	2 March
Replacement with water in 1.8 m trench	4 March
Lowering of water to 4.3 m in 1.8 m trench	5 March
Refilling of water in 1.8 m trench	5 March
Lowering of water to 8.3 m in 1.8 m trench	6 March
Refilling of water in 1.8 m trench	6 March
Replacement with slurry in 1.8 m trench	9 March
Replacement with water in 5 m trench	9 March
Replacement with oil in 5 m trench	11 March
Replacement with water in 5 m trench	14 March
Lowering of water to 4.7 m in 5 m trench	19 March

3 FEA MODEL

A 3D FEA model was established to analyse the slurry trench tests. The soil was divided into five layers. Down to a depth of 1.5 m is a layer of fill material. Then comes a layer of dry crust above clay which is divided into three sublayers. The geometry of the model is shown in Figure 3.

The model has normal constraints on vertical boundaries and the bottom boundary is fully fixed. The top surface is free, while distributed loads are applied on the inside surfaces of the trenches representing the pressure from the stabilising fluids. The calculation phases reflects the timeline s in Table 1. «

3.1 Soil parameters

3.1.1 Clay

Undrained behaviour is assumed for the clay, using the NGI-ADP material model Grimstad et al. (2012). The input parameters are based on the field and laboratory tests and are summarized in Table 2.

3.1.2 Fill material and dry crust

For the fill material and dry crust, the HS-small model (Benz, 2007) is used. The material properties are summarised in Table 3, and are mostly typical values.

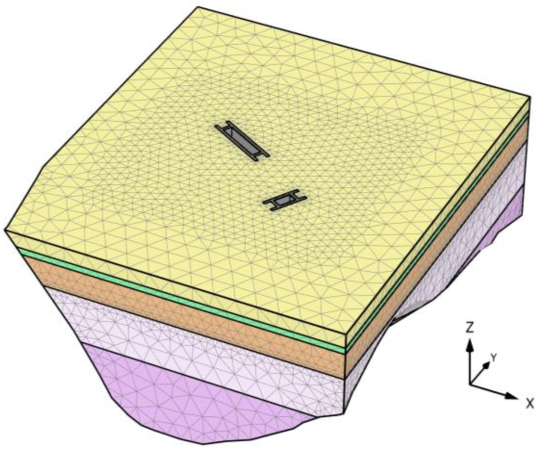

Figure 3. 3D perspective view of the FEA geometry.

Table 2. NGI-ADP material properties for the clay in the FEA model.

Parameter	Value
Unit weight, γ (kN/m^3)	18.5
Relative unloading modulus, $G_{\mathrm{ur}}/c_{\mathrm{uA}}$ (-)	700
Failure strain in compression, $\gamma_{\mathrm{f}}^{\mathrm{C}}$ (%)	0.8
Failure strain in extension, $\gamma_{\mathrm{f}}^{\mathrm{E}}$ (%)	6
Failure strain in direct shear, $\gamma_{\mathrm{f}}^{\mathrm{DSS}}$ (%)	3
Active shear strength, c_{uA} (kPa)	Fig. 2
Relative passive strength, $c_{\mathrm{uP}}/c_{\mathrm{uA}}$ (-)	0.35
Initial mobilization, τ_0/c_{uA} (-)	0.5
Relative DSS strength, $c_{\mathrm{uD}}/c_{\mathrm{uA}}$ (-)	0.67

Table 3. HS-small material properties for the fill material and the dry crust in the FEA model.

Parameter	Fill / dry crust
Unit weight, γ (kN/m^3)	20 / 17
Reference Young's modulus, E_{50}^{ref} (kPa)	3.0×10^4
Reference oedometer modulus, $E_{\mathrm{oed}}^{\mathrm{ref}}$ (kPa)	3.0×10^4
Reference unloading modulus, $E_{\mathrm{ur}}^{\mathrm{ref}}$ (kPa)	9.0×10^4
Poisson ratio for unloading, ν_{ur} (-)	0.20
Power, m (-)	0.50
Reference shear strain, $\gamma_{0.7}$ (-)	1.0×10^{-4}
Small strain shear modulus, G_0^{ref} (kPa)	1.0×10^5
Cohesion, c'_{ref} (kPa)	0
Friction angle, φ' (°)	32
Dilation angle, ψ (°)	0
Over-consolidation ratio, OCR (-)	1.0
Failure ratio, R_{f} (-)	0.90
Normal consolidation stress ratio, K_0^{NC} (-)	0.47

4 RESULTS

The FEA results show reasonable agreement with the observed/measured behaviour. As an example, Figure 4 compares the calculated reduction in trench width for the 1.8 m trench during drawdown to 4.3 m depth to the measured values. The agreement is generally good, particularly along the upper part. The difference seen at large depths can be due to inaccuracy in the geometry of the bedrock surface in the FEA and the effect the bedrock has on the stresses, and/or the shear strength being underestimated at large depths.

Furthermore, the FEA showed that the displacement in the 1.8 m trench increased rapidly when drawdown was continued to 8.3 m, reaching values of decimetres. The maximum displacement during this phase occurs at a depth of 8.5–9 m, that is, just below the top level of the water in the trench (Figure 5). Similarly, large, displacements were calculated for the 5 m trench during drawdown to 4.7 m depth. Consequently, these two drawdown phases can be considered close to failure. This is in line with the observed behaviour of both trenches.

Using the simplified analytical approach described by Aas (1976) the following factors of safety can be calculated for the two drawdown phases described above (DiBiagio *et al.*, 2020):

- Drawdown in 1.8 m trench to 8.3 m: 0.99
- Drawdown in 5 m trench to 4.7 m: 0.70

It is, thus, seen that the FEA using the NGI-ADP model is in line with the analytical method for the 1.8 m trench as both indicate failure for drawdown to 8.3 m. For the 5 m trench the analytical method results in a factor of safety of 0.7. Actually, a factor of safety of less than 1 is obtained for a drawdown of only 1 m.

This indicates that the analytical method underestimates the safety for this trench size as the field observations (and the FEA) shows that the 5 m trench is stable at this drawdown. The analytical method, thus, seems to underestimate the geometric effects for trenches with larger width to depth ratios.

5 CONCLUSIONS

The FEA using the NGI-ADP model can predict displacements with reasonable accuracy as well as accurately capture safety with respect to failure of the slurry trenches. The simplified analytical approach presented by Aas (1976) agrees well with the FEA and the measurements for the 1.8 m trench but seems to underestimate the safety for the 5 m trench.

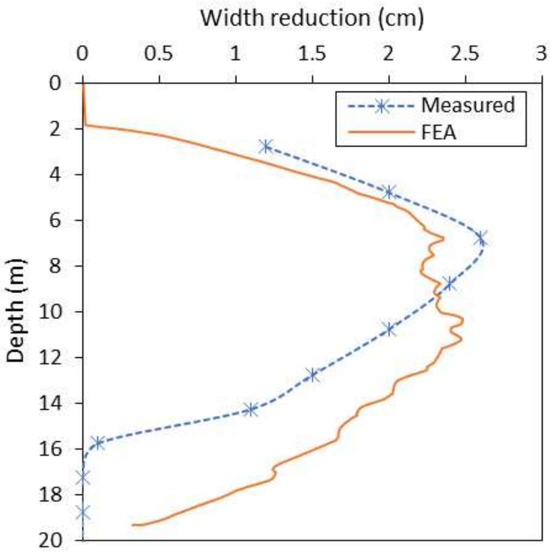

Figure 4. Comparison between measured and calculated trench width reduction during drawdown to 4.3 m in the 1.8 m trench.

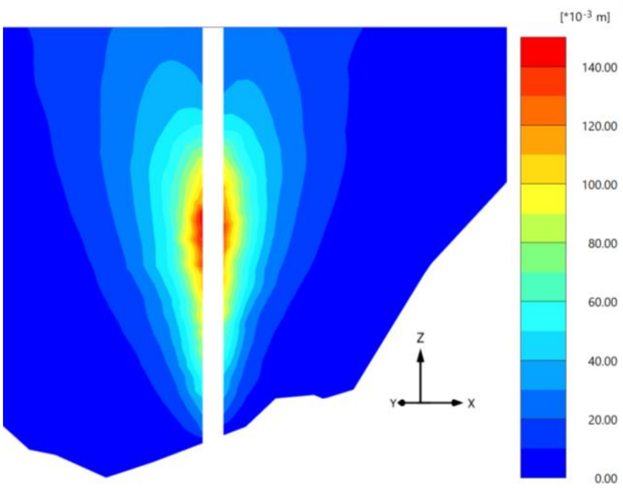

Figure 5. Contours of total displacement calculated in a cross section perpendicular to the 1.8 m trench after drawdown to 8.3 m depth.

ACKNOWLEDGEMENTS

The authors are grateful for the financial support provided by the Research Council of Norway through the public basic funding for research institutes.

REFERENCES

Aas, G. (1976) Stability of Slurry Trench Excavations in Soft Clay, In: Norwegian Geotechnical Institute Publication 111, NGI, Oslo, Norway, pp. 1–7.

Aas, G. (1986) In situ investigation techniques and interpretation for offshore practice. Recommended interpretation of vane tests. Final report, NGI, Oslo, Norway.

Benz, T. (2007) Small-Strain Stiffness of Soils and its Numerical Consequences, Doctoral dissertation. Universität Stuttgart. Available at: https://d-nb.info/984776931/04, accessed: 27/10/23

DiBiagio, E. et al. (2020) Deliberate Failure of Slurry Trench Excavations in Soft Clay, Geotechnical Engineering, 51(4), pp. 209–223. Available at: https://www.researchgate.net/publication/352166347_Deliberate_Failure_of_Slurry_Trench_Excavations_in_Soft_Clay, accessed: 27/10/23.

DiBiagio, E. and Myrvoll, F. (1972) Full Scale Field Tests with a Slurry Trench Excavation in Soft . Clay, In: Proceedings of the 5th European Conference on Soil Mechanics and Foundation Engineering, Madrid, Spain, pp. 461–471.

Grandas-Tavera, C.E. and Triantafyllidis, T. (2012) Simulation of a corner slurry trench failure in clay, Computers and Geotechnics, 45, pp. 107–117. https://doi.org/10.1016/j.compgeo.2012.05.007.

Grimstad, G., Andresen, L. and Jostad, H.P. (2012) NGI-ADP: Anisotropic shear strength model for clay, International Journal for Numerical and Analytical Methods in Geomechanics, 36(4), pp. 483–497. https://doi.org/10.1002/nag.1016.

Karlsrud, K. et al. (2005) CPTU correlations for clays, In: Proceedings of the 16th International Conference on Soil Mechanics and Geotechnical Engineering, Osaka, Japan, pp. 693–702. https://doi.org/10.3233/978-1-61499-656-9-693.

Karlsrud, K. and Hernandez-Martinez, F.G. (2013) Strength and deformation properties of Norwegian clays from laboratory tests on high-quality block samples, Canadian Geotechnical Journal, 50(12), https://doi.org/10.1139/cgj-2013-0298

Schäfer, R. and Triantafyllidis, T. (2004) Modelling of earth and water pressure development during diaphragm wall construction in soft clay, International Journal for Numerical and Analytical Methods in Geomechanics, 28(13), pp. 1305–1326. https://doi.org/10.1002/nag.388.

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The paper was published in the proceedings of the 18th European Conference on Soil Mechanics and Geotechnical Engineering and was edited by Nuno Guerra. The conference was held from August 26th to August 30th 2024 in Lisbon, Portugal.