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Stability analysis of a slope during pile driving

Analyse de la stabilité des pentes pendant le battage de pieux

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ABSTRACT: In geotechnical practice, there is no standard method recognized by the industry to account for the reduction of safety factor of a slope related to pile installations. Pile driving causes remolding of the soil and mass displacement which may result in large strains and generates excess pore pressures in a zone around the pile, resulting in a decrease of the shear strength in the surrounding soil. Moreover, dissipation of excess pore pressure may cause weakening of areas outside the volume of soil remoulded during installation. This phenomenon may cause slope failure. Changes in mean and shearing stress make it challenging to predict installation induced pore pressure response. Furthermore, it is a complex task to follow the rate and path of pore pressure dissipation in order to analyze the effect on slope stability. In sensitive soils, it is in addition, necessary to implement soil models that account for strain softening in the analyzing tools. Up to now, experience based semi-empirical methods have been suggested to predict the effect of pile driving on slope stability. However, this is still not well understood or agreed upon. In Norway, approaches applied by geotechnical engineers for this problem are based on pore pressure monitoring with possible remediation actions to avoid excessive pore pressure build up. This paper looks at this problem by studying a case of slope failure due to pile driving occurred in Norway.

RÉSUMÉ : Dans la pratique géotechnique, il n'existe pas de méthode standard reconnue par l'industrie pour tenir compte de la réduction du facteur de sécurité d'une pente lors de l'installation de pieux. Le battage des pieux provoque de grandes contraintes et génère des pressions interstitielles excessives dans une zone autour du pieu, entraînant une diminution de la résistance au cisaillement dans le sol environnant. De plus, la dissipation de l'excès de pression interstitielle mis en place peut provoquer un affaiblissement des zones en dehors du volume de sol remanié lors de l'installation. Ce phénomène peut provoquer une rupture de pente. Les changements de contrainte moyenne et de cisaillement rendent difficile la prédiction de la réponse de la pression interstitielle induite par l'installation. De plus, c'est une tâche complexe que de suivre le taux et le chemin de dissipation de la pression interstitielle afin d'analyser la stabilité des pentes. Dans les sols sensibles, il est en outre nécessaire de mettre en œuvre des modèles de sol prenant en compte l'adoucissement des déformations dans les outils d'analyse. Jusqu'à présent, des méthodes semi-empiriques basées sur l'expérience ont été suggérées pour prédire l'effet du battage de pieux sur la stabilité des pentes. Cependant, cela n'est toujours pas bien compris ou accepté. En Norvège, les approches générales appliquées par les ingénieurs géotechniciens pour ce problème sont basées sur la surveillance de la pression interstitielle. Cet article examine ce problème en étudiant un cas de rupture de pente due au battage de pieux survenu en Norvège.

KEYWORDS: Pile driving, Slope stability, Excess pore pressure

1 INTRODUCTION.

Displacement piles are a common foundation type to support structures, bridge abutments and embankments, specifically in soft soil deposits where they transfer structural loads to deeper more stable soil layers or bedrock. This makes them a common solution in Scandinavian soft soil conditions. Driving piles into the ground results in remolding of the soil, mass displacement and hence induces stresses and strains in the adjacent soil and generates excess pore pressure (Cummings et al., 1950). These can affect the strength and stiffness of the adjacent soil. Pile driving in or near a slope may trigger instability, specifically in situations where stability of the slope is marginal before pile installation (Massarsch and Broms 1981). A number of slope failures in Norway, Sweden, Canada and China in soft soil deposits lie in this category, such as those explained in (Carson 1979), (Aas 1975) and (LaGatta and Whiteside 1984) among others. This paper attempts to study the failure mechanisms involved in this phenomenon by considering a case of slope failure during construction of a bridge in Norway.

2 SLOPE FAILURE DURING CONSTRUCTION OF V ÆRSTEBRUA.

During construction of a bridge in Fredrikstad, Norway, a slope failure occurred after driving a number of piles planned for the foundation. Værstebrua Bridge (see Figure 1) crosses over the river Vesterelva in Fredrikstad (Southeast of Norway) which connects the island Kråkerøy to the mainland. The bridge has three spans and a total length of approximately 100 m.

Figure 1 Overview of the Værstebrua (Photo by: Terje Løchen)

The bridge was planned with four foundation sections of end bearing piles driven to bedrock. Figure 2 shows a sketch of the planned foundation. The pile group delineated in Figure 2 is the topic of this paper. The foundation consisted of two rows of piles, containing four close-ended piles in each row with a diameter of 813 mm. This section consists of two inclinations of approximately 38 and 22 degrees (see Figure 2).

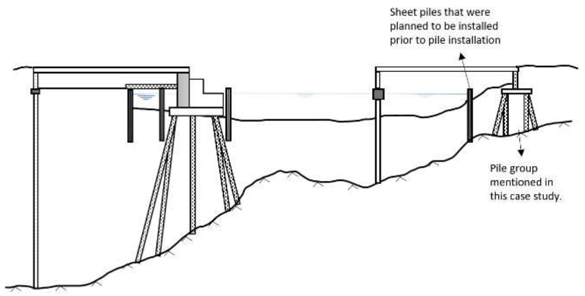

Figure 2 Cross section of the foundation for the entire bridge

2.1 *Soil conditions*

Ground investigation was conducted prior to construction of the bridge. The tests included several total soundings, 4 CPTUs and 6 soil samples for lab tests. The lab tests included Triaxial Compression tests and Oedometer testing. The geotechnical investigations report states that the soil is a relatively homogeneous silty clay. The soil had a medium to low sensitivity (top 10 m of the soil shows sensitivity ranging from 9 to 15 and 4 to 6 in deeper parts of the soil). Depth of bedrock was ranging from 7.5 to 20m deep. The water content of the soil ranges between 30 to 50 percent. Test results show an increase in both stiffness and strength with depth.

2.2 *Planning and construction*

A strict construction control program was planned for the installation of piles of Værstebrua. It was originally planned that permanent sheet piles are installed to bedrock prior to the pile driving. This would have provided more stability to the slope and piling could possibly have been conducted without stability issues. However, due to late delivery of sheet piles, the piles were driven before the sheet piles were installed (Geovita Aas 2013).

The development of excess pore pressures during pile driving were closely monitored by three piezometers installed in a location 6.5m away from the center of the pile group and at depths (4, 8 and 12m deep). Based on the stability analysis prior to pile driving and estimations of excess pore pressures due to piling, the geotechnical engineers advised allowable levels of excess pore pressure generation for each piezometer.

If the piezometers logged a pore pressure value above the limit, the engineers would be notified with a SMS-warning and piling needed to stop until further notice. Despite these measures, a slope failure occurred during the installation of the fifth pile of the total of eight piles that was planned for Axis 4. The failure was described as settlements up to 70 cm and development of cracks in the surface. The piezometer logs showed that the pore pressure had risen above the boundary that was set by the geotechnical engineers, but this was not communicated due to an error in the SMS-warning system (Geovita Aas 2013).

3 TRIGGERING MECHANISM

This section discusses the possible triggering mechanisms in this slope failure.

3.1 *Excess pore pressure generation*

Pile driving displaces the surrounding soil and generates excess pore pressures. Conventional methods assume that this increase in pore pressure result in a decrease in the effective stresses which in turn, reduce the shear strength of the soil. However, this assumption is only correct if small changes in total stress occurs in the soil. However, pile installation will increase both the total mean stresses and the pore pressure in the soil resulting in smaller changes in effective stresses. The total pressure increase and the excess pore pressure caused by volume displacement may in addition reduce if small global slope deformation occurs. These issues are discussed in more detail in Attari et al. (2021). Hence, pore pressure increase due to pile driving cannot explain the failure. However, it can have a detrimental effect on stability, as described in section 3.2.

3.2 *Excess pore pressure dissipation*

The soil in this area is described as clay containing high percentage of silt. The permeability of this material is higher than normal clay which facilitates dissipation of excess pore pressures throughout the slope. If this excess pore pressure migrates to parts of the slope with no increase in total stresses due to pile driving, this can reduce the effective stresses in this area and hence reduce the shear strength of the soil. This can result in a reduction of the slope stability and cause slope failure.

3.3 *Excessive movements of the slope*

Driving of displacement piles results in lateral and vertical movements in the surrounding soil. The slope failure at Værstebrua was described as 70 cm vertical deformation and generation of cracks in the ground as well as outward movement of the top masses. It is possible that there was no real failure in the slope, but excessive movements were generated due to insertion of piles. However, the magnitude of this deformations was problematic for continuation of the construction work.

3.3 *Progressive failure*

Understanding the behavior of soft and sensitive clay is an important topic in Scandinavia due to the existence of large deposits of soft and quick clays in this region (Rosenqvist 1953).

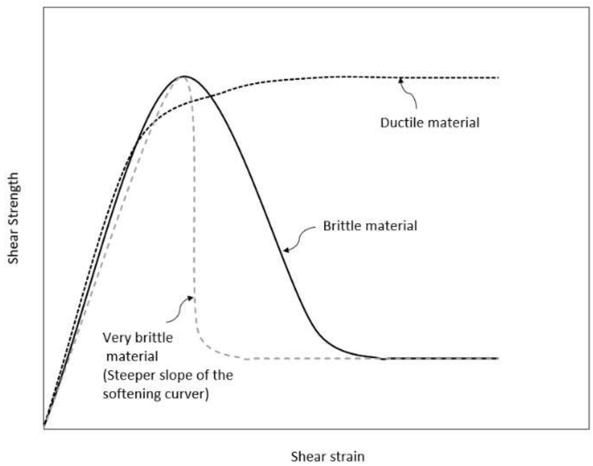

Figure 3 Ductile vs brittle material behavior

Sensitivity of a clay material is described in terms of the ratio between undisturbed shear strength and the shear strength of the soil in a remolded state, while brittleness of the material is defined by the slope of the softening curve (see Figure 3).

A strain softening behavior causes the soil to have a shear strength that decreases with increasing shear strain. It can be inferred that increased displacements in the soil due to pile driving may cause a local failure with a gradual progression of strain-softening behavior throughout the slope which could ultimately lead to a global failure (Bernander 2011). This type of failure is defined as progressive failure in sensitive deposits. Since the soil in this site is described as medium to low sensitive, a progressive failure could have occurred here as well.

4 NUMERICAL ANALYSIS

A set of finite element analyses was conducted to get a better understanding of the failure in this case study.

4.1 *Geometry*

The slope geometry was modelled in Plaxis 2D finite element geotechnical suite. The failure occurred after driving of 5 piles (out of the total 8), which includes 4 piles close to river on the right side and one pile on the left side. Hence, the inclined pile on the back side of the foundation is not included in the analysis. Figure 4 shows the geometry modelled in Plaxis 2D.

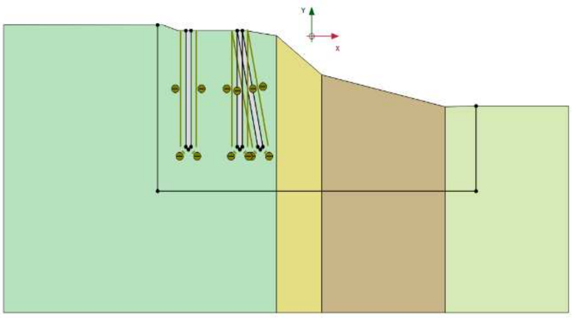

Figure 4 Model geometry in Plaxis 2D

4.2 *Constitutive models and soil parameters*

The case study is analyzed using three different constitutive soil models. Firstly, the Elastic-perfectly plastic soil Mohr-Coulomb model was used. Furthermore, the advanced Soft Soil model which can simulate the shear-induced pore pressure in clay and lastly the NGI-ADPsoft soil model which can replicate the strain softening behavior of the soil is used. The latter is used as a user-defined model (dll-file).

4.2.1 *Mohr-Coulomb analysis*

This material model includes both anisotropic undrained shear strength and post peak strain softening. Softening is modelled as a reduction in the undrained shear strength for shear strains larger than the strain at the peak undrained shear strength. The shear strength reduces as function of shear strain until the residual shear strength is reached. This material model can simulate progressive failures in clay. More information about this soil model can be found in (Jostad et al., 2014), (Grimstad and Jostad 2012) and (D'Ignazio 2016). Table 1 provides the input parameters used for this analysis.

Table 1. Parameters selected for the Mohr Coulomb model in Plaxis.

Parameter	*Undrained A model*	*Undrained B model*
Unit weight (kN/m3)	18	18
Young's Modulus (kPa)	8000+400z	8000+400z
Effective Poisson's ratio	0.3	0.3
Undrained shear strength (kPa)	-	18+1z
Friction angle (deg)	23	-
Cohesion (kPa)	4	-

4.4.2 *Soft soil analysis*

Soft Soil is a constitutive soil model based on Modified Cam Clay. This soil model can to some extent simulate the excess pore pressure generated due to shearing of clay, and hence, will provide a more realistic estimate of generated excess pore pressures. The required parameters for this model are summarized in Table 2, where the compression index was estimated based on Oedometer test results from the soil investigation suite. An effective stress analysis was done in this case and the pile driving was conducted in a consolidation phase with a duration of a day.

Table 2. Parameters selected for the Soft Soil model in Plaxis

Parameter	*Value*
Unit weight (kN/m3)	18
Initial void ratio	0.5
Modified compression index	0.065
Modified swelling index	0.02
Effective cohesion (kPa)	4
Friction angle (deg)	23
Dilatancy angle (deg)	0
Coefficient of lateral stress in normal consolidation	0.625
Slope of the critical state line	1.2

4.2.3 *NGI-ADPsoft Analysis*

This material model includes both anisotropic undrained shear strength and post peak strain softening. Softening is modelled as a reduction in the undrained shear strength for shear strains larger than the strain at the peak undrained shear strength. The shear strength reduces as function of shear strain until the residual shear strength is reached. This material model can simulate progressive failures in clay. More information about this soil model can be found in (Jostad et al., 2014), (Grimstad and Jostad 2012) and (D'Ignazio 2016). Table 3 provides the input parameters used for this analysis.

Table 3. Table caption numbered consecutively. Tables placed below caption. Use Times New Roman 8 for the caption, text and numbers in the table.

Parameter	*value*
Unit weight (kN/m3)	18
Initial Undrained shear modulus/Undrained shear strength	500
Peak undrained active shear strength (kPa)	18+1z
Normalized undrained DSS ”peak” shear strength by the peak undrained active shear strength	0.7
Normalized undrained passive ”peak” shear strength by the peak undrained active shear strength	0.5
Initial shear mobilization	0
Normalized undrained active residual shear strength by the peak undrained active shear strength	0.1
Normalized undrained DSS residual shear strength by the peak undrained active shear strength	0.1
Normalized undrained residual passive shear strength by the peak undrained active shear strength	0.1
Shear strain at ”peak” undrained active strength	1.5 %
Shear strain at ”residual” undrained activestrength	15 %
Softening parameter 1*	1.0
Softening parameter 2*	1.0
Drained Poisson's ratio	0.25
Undrained Poisson's ratio	0.495
A Parameter for non-local strain	2
Internal length	0.1 m

*The softening parameters and other parameters required for NGI-ADPsoft soil model are explained in detail in the report NGI (2016).

4.3 *Analysis sequence*

To simulate pile driving, the Volumetric Expansion feature in Plaxis was used. This feature can be used to expand or contract a cluster of soil, as specified in Plaxis (2021). Each analysis starts with gravity loading, followed by applying lateral expansion to the clusters modelled as piles, based on pile diameter and center to center distance. The expansion clusters are given the same stiffness and strength properties as the surrounding soil. In the Mohr-Coulomb analysis a phi-c reduction analysis is conducted before and after pile driving simulation which shows the factor of safety of the slope in the two phases. In the Soft Soil model, pile driving was followed by a consolidation phase where excess pore pressure generated during piling is dissipating with time (one day).

The phi-c reduction analysis cannot be used together with the NGI-ADPsoft constitutive soil model. In order to examine the effect of pile driving in this analysis, the unit weight of the soil material was increased until failure was reached before and after pile driving. The increased gravity when the soil becomes unstable is corresponding to the reduction of factor of safety in this analysis.

5 RESULTS AND DISCUSSION

This section summarizes the results of the numerical analy sis using different constitutive soil models and provides a discussion.

5.1 *Mohr-Coulomb analysis*

As illustrated in Figure 5, the phi-c reduction analysis showed no change in the factor of safety after pile driving in both analyses using the Mohr-Coulomb material model (undrained A and B). It shall be noted that the undrained shear strength by these two approaches (A and B) is different, because the drained friction angle is not corrected in order to account for the shear induced pore pressure not described by this model.

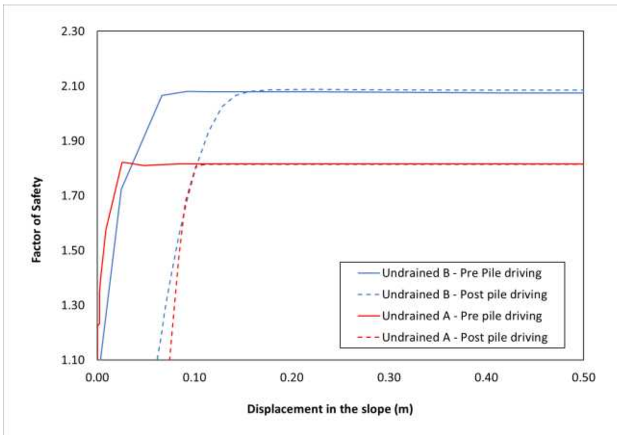

Figure 5 Results of the phi-c reduction phase before and after pile driving phase, using Mohr Coulomb constitutive soil model in Plaxis.

5.2 *Soft Soil analysis*

Figure 6 shows the contours of excess pore pressures generated using the Soft Soil material model. This analysis also showed an increase in the slope movement during dissipation of excess pore pressures. Although the analysis did not indicate a slope failure in this phase, it showed a very high mobilization of undrained shear strength which noted the slope was close to failure state. Figures 7 and 8 show the shear strength mobilization after pile driving, one before and the other after dissipation of generated excess pore pressures, respectively. These figures show areas of full mobilization which could be the initiation of progressive failure.

5.3 *NGI-ADPsoft analysis*

The results of the NGI-ADPsoft analysis showed a reduction in the factor of safety of slope due to pile driving, changing from 1.8 to 1.4. Although there is some information about the sensitivity of the soil, none of the available test results provide the brittleness of the material which can affect the decrease in the factor of safety (multiple analyzes show the more brittle material, the higher is the reduction in the factor of safety of the slope after pile driving). The pile driving in this analysis induces 70mm total displacements.

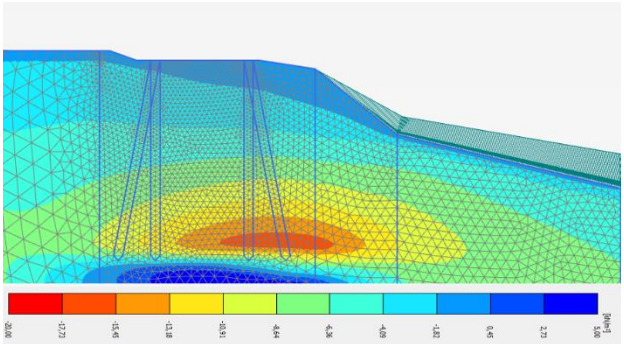

Figure 6 Excess pore pressure due to pile driving, predicted using Soft Soil constitutive soil model.

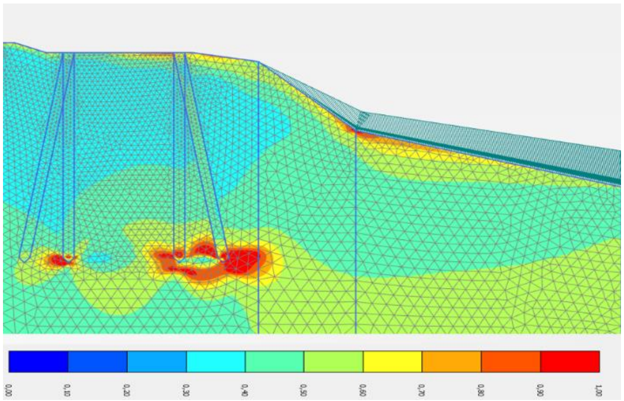

Figure 7 Shear strength mobilization after pile driving, before dissipation of excess pore pressures

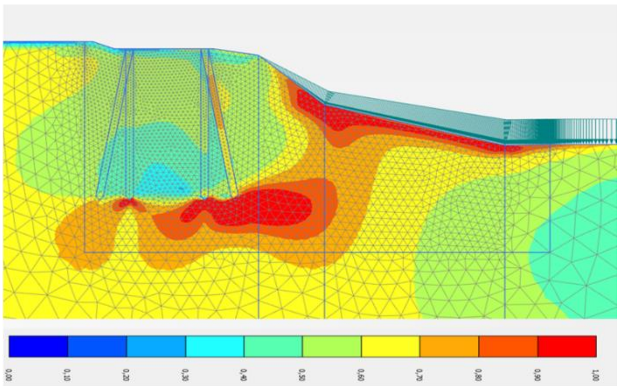

Figure 8 Shear strength mobilization after pile driving and following dissipation of excess pore pressures

5.4. *Discussion*

A number of finite element analyses were conducted to examine the failure mechanism involved in this slope failure during pile driving in sensitive clay. According to these results, a Mohr-Coulomb elasto-plastic analysis cannot replicate the reduction in the slope's factor of safety due to pile driving. The soft soil model can provide a more realistic prediction of the excess pore pressures generated due to pile driving since it can simulate shear-induced excess pore pressures. However, it is difficult to control the magnitude and the pore pressure after the peak undrained shear strength. Dissipation of these excess pore pressures into parts of the slope that have not been affected by pile driving may reduce the effective stress and hence reduce the strength. This may result in a slope failure provided the shear strength is fully mobilized in significant areas of the slope. The analysis conducted using NGI-ADPsoft soil model shows a significant decrease in the factor of safety of the slope due to pile driving. In the presence of comprehensive information about the sensitivity and brittleness of the soil material, this model might be able to provide an accurate prediction of the failure. The failure may also be a combination of large shear induced pore pressure during softening and pore pressure dissipation. This requires an effective stress-based model that can model destructuration of soil, which lies outside the scope of this paper.

6 CONCLUSIONS

A slope failure occurred in clay soil deposits during pile d riving at the top of the slope in Fredrikstad , Norway. Thi s incident was evaluated, and possible triggering mechanis ms involved were examined using finite element modelling using PLAXIS 2D. It is shown that in a conventional elas to-plastic analysis (using Mohr-Coulomb soil model) where the pile installation is simulated as volume expansion, no change in factor of safety of the slope due to pile driving can be found.

Another possible failure mechanism investigated is progr essive failure of the slope. The shear strength of the soil could also have been reduced due to strain softening of th e clay triggered by the pile driving. This reduces the stabi lizing forces and results in a drop in the factor of safety of the slope. Another cause of this incident could be attrib uted to migration of excess pore pressure to parts of the s lope that have not been directly affected by pile driving. Hence, applying a limit on piezometers does not always m ean the slope is safe for continuation of pile driving. It is required to use advanced material models that describe the strain softening and shear induced pore pressure of the act ual clay in order to define these limits.

7 ACKNOWLEDGEMENTS

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