



Evaluation of Pile Driving Effects on Slope Stability in Clay

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Abstract Pile driving can trigger slope failure, specifically in sensitive soil deposits or marginally stable slopes. Cases of slope failure during pile installation have been reported in countries such as Norway, Sweden and Canada where soft sensitive clay deposits are prevalent. Although a few approaches have been suggested to assess the factor of safety of a slope during pile driving, there is currently no standard method recognized by the geotechnical society to account for the reduction of safety factor of a slope as an effect of pile installation. Some approaches suggest using the excess pore pressure generated during pile driving to account for the reduction in soil strength (by reducing effective stresses), neglecting the change in total stresses. This paper discusses the accuracy of stability analysis methods currently in use for this problem by looking at stress changes in the soil during pile driving as well as using numerical analysis results. Importantly, the paper identifies the shortcomings regarding these approaches, thus allowing geo-engineers to understand the limitations of their analysis and the uncertainties in the models.

Keywords Piles & piling · Slopes · Pore pressures

Abbreviations

OCR	Over consolidation ratio of the soil
d	Pile diameter
l	Centre-to-centre distance between piles
Δu	Excess pore pressure
Δu_{\max}	Maximum generated excess pore pressure
u_0	Initial hydrostatic pressure
R	T outer radius of the pile
r	Radial distance from the centre of the pile
G	Shear modulus of the soil
S	Undrained shear strength ratio in normally consolidated state
σ'_0	In-situ vertical effective stress
S_u	Undrained shear strength before and after pile driving
S_{u1}	Undrained shear strength after pile driving
P'	Mean effective stress after pile driving
p'_0	Initial effective stress (before pile driving)
m	SHANSEP constant dependent on type of clay
I_p	Plasticity index of soil
K_h	Coefficient of horizontal pressure
q	Shear stress
p	Mean stress
M	Slope of the failure line in triaxial compression
φ'	Effective friction angle of soil
σ_v	Vertical stress in the soil
σ_h	Horizontal stress in the soil

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γ_w	Unit weight of water
z	Depth from the ground surface
γ_M	Required partial factors for shear strength in slope stability analysis

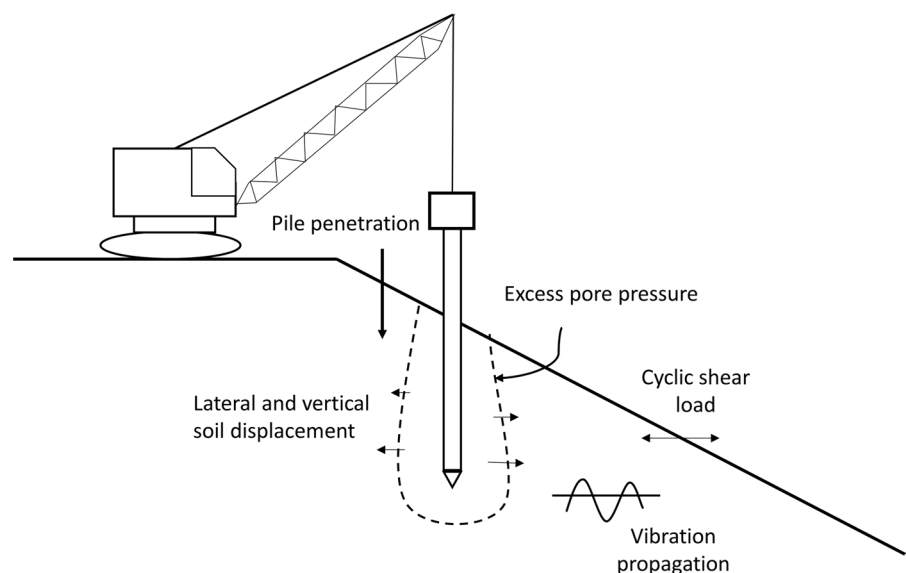
1 Introduction

Piles are a common foundation solution to support buildings, bridge abutments and embankments in soft soil deposits to transfer structural loads to deeper more stable soil layers or bedrock. Driving piles into the ground changes the stress situation in the field surrounding the piles, induces deformations in the soil and leads to remoulding of the adjacent soil (Flaate 1972). As a result of compression and/or shear, excess pore pressure is generated, and soil may experience reduction in its shear strength. This might result in loss of stability when pile driving is conducted near marginally stable slopes (Bjerrum and Johannessen 1960). Stability issues are also encountered in submarine slopes during port expansions or in onshore slopes during pile installations for bridge abutments. Lamens (2017) discusses the effect of offshore pile driving on a sandy submarine slope by investigating the risk of liquefaction and slope failure during pile installations for construction of a sea lock in the Netherlands. Similarly, stability challenges regarding pile driving for expansion of the port of Seattle in

silty deposits are covered in detail by Nykamp et al. (2004) and Fellenius et al. (1982) and uncertainties regarding pile driving for several bridge foundations in soft clay in Norway are discussed by Aas (1975) and Tefera et al. (2013) among others. Due to these uncertainties, close monitoring of soil movements and excess pore pressures is usually advised or obligated in such instances. In some of these cases, stability of the slope had to be improved prior to pile driving or driven piles had to be replaced with bored piles due to stability concerns (Tefera et al. 2013). However, changing the design, improving ground conditions prior to pile driving and constant on-site measurements during installation can greatly increase project expenses. Furthermore, limits applied on the in-situ measurements during pile driving might result in pauses in construction procedure, increase construction time and costs (Tefera et al. 2013). Hence, more accurate predictions of the risks involved during pile driving in the vicinity of slopes is important with regards to both safety and costs.

Figure 1 illustrates certain impacts of pile driving that might trigger slope failure. Even though pile driving in slopes has previously resulted in hazardous events in the past, there is currently no accepted method for estimating the reduction in the factor of safety of a slope due to pile driving (Langford et al. 2021).

Fig. 1 Effects of pile installation in a slope, remade with reference to (Lamens 2017)



2 Review of Literature

Pile driving in clay occurs rapidly, causing an undrained soil behaviour at failure due to the low permeability of clay (Randolph and Gourvenec 2011). Seed and Reese (1957) were among the first to address excess pore pressures around piles driven in clay. Later, Bjerrum and Johannessen (1960) reported that a rise in pore pressure due to pile driving and its following dissipation is accompanied by a temporary drop in the undrained shear strength and a reduction in the factor of safety of adjacent slopes. Safety of a slope during or directly after pile driving may, according to Massarsch and Broms (1981), be evaluated based on the generated excess pore pressures. Cyclic loading due to pile driving can also generate excess pore pressures and thereby cause degradation of strength (Lamens and Askarinejad 2020). Pile driving also generates vibrations, however, stability problems caused by pile driving are most often caused by mass displacement and pore pressure build-up around the pile and not due to vibrations (L'Heureux and Johansson 2016).

During and after pile driving, properties of the soil in the vicinity of the pile wall are affected resulting in elastic and plastic zones of disturbance around the pile (Karlsrud and Nadim 1990). The extent of the zone with high shear-induced pore pressures (zone I, as shown in Fig. 2) is limited to a region close to the pile. While the range of the zone of excess pore pressure due to displacement-only (elastic zone) is larger (exponentially decaying with increasing distance). This area itself can be divided into zone II and III. In zone II, pore pressure is reduced after driving stops, while dissipation of the excess pore pressure leads to additional (delayed)

increase in pore pressure and thereby a delayed reduction of effective stress in zone III which has not been majorly affected by pile driving activities (see Fig. 2).

generation and dissipation.

Challenges during pile driving have in some cases resulted in landslides in different magnitudes, with minor or in some cases immense consequences. Table 1 provides a list of slope failure incidents due to pile driving. In some of these cases, noticeable rise in excess pore pressure was observed prior or after the landslide. The following section provides an overview of methods for estimating excess pore pressure and how they are commonly used to assess slope stability during pile installation.

2.1 Estimation of Excess Pore Pressure Due to Pile Driving

Excess pore pressure due to pile driving comes from (1) increased total stress due to the soil volume displaced by the pile and (2) shear deformation of the soil (Massarsch 1976). In-situ pore pressure (discussed in this paper) are a sum of these two components. The first component of pore pressure is always positive, while shear-induced pore pressure can be negative or positive depending on dilatancy properties of soil. Normal to slightly over-consolidated clays (Over-Consolidation Ratio (OCR) < 3.0) show a contractive behaviour during shearing causing an increase in pore pressures (Burns and Mayne 1999). While highly over-consolidated soils exhibit a dilative behaviour, resulting in a temporary reduction in pore pressure during shearing.

Fig. 2 Affected zones around the pile wall due to pile driving, with regards to pore pressure

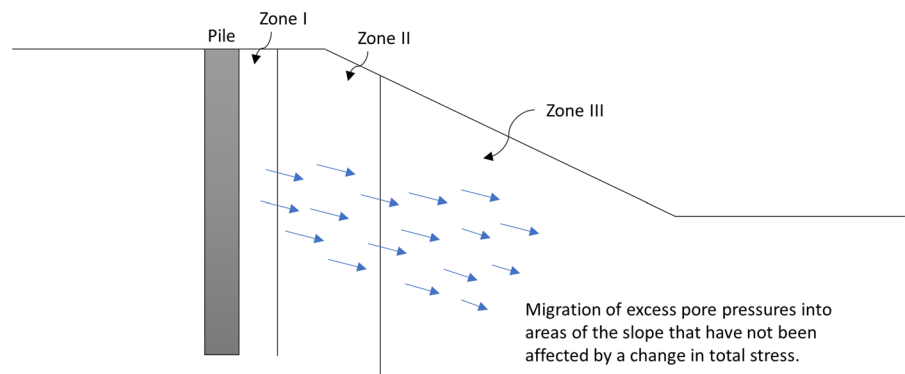


Table 1 Cases of slope failure due to pile driving

Location	Description	References
Drammen, Norway	Piling activity for a factory building triggered a large landslide at which approximately 1000 m ² of the site subsided and slid out into the nearby River	(Aas 1975)
Fredrikstad, Norway	Pile driving for a bridge foundation (Værstebrua) caused excessive ground movements and resulted in misaligning of already-driven piles and disruption in construction activities	Norwegian project report by Geovita AS (2013) 'Værstebrua, bysiden rapport'
Sandvika, Norway	Foundation installation for an embankment during construction of E18 road in Mustadjordet which included 150 number of concrete piles, resulted in a landslide volume of 5,000m ³	Norwegian project report by Veglaboratoriet (1963)
Quebec, Canada	A landslide occurred due to driving of three friction piles which resulted in a loss of life	(Carson 2011)
Portland, USA	During construction of a wharf facility, an underwater slope failed during dredging and subsequent driving of piles through the slope	(LaGatta and Whiteside 1984)
Trøndelag, Norway	In a harbour projects, a slide occurred in an engineered embankment during its construction, which was attributed to the movements induced by pile driving	(Thakur et al. 2008)
Shanghai, China	A riverbank dike was damaged by pile driving in very soft Shanghai clay. Severe deformation and cracks of bank slope occurred during pile driving for the reinforcement of Shanghai Bailianjing port	(Shen et al. 2005)
Gothenburg, Sweden	A slide took place due to pile driving for a family house, which slid away 31 houses in the area	(Bernander 2011)
Rollsbo Kungälv, Sweden	Pile driving for a family house resulted in a slope failure	(Bernander 2011)
Rävekärr, Mölndal, Sweden	Driving of pipes for sand drains resulted in a landslide	(Bernander 2011)

2.1.1 Changes in Pore Water Pressure due to Single Pile Installation

Measurements of magnitude and extent of pore pressure generation due to pile driving show that the main influencing parameters are the undrained shear strength, the shear modulus, OCR, pile type (open or closed) and soil sensitivity (Lo and Stermac 1965). Empirical methods or analytical solutions based on cavity expansion theory combined with the critical state soil mechanics or the Mohr-Coulomb failure criterion can be used to calculate excess pore pressures (Burns and Mayne 1999). According to Gibson and Anderson (1961), the generated excess pore pressure can be estimated based

on the shear strength (S_u) and the equivalent linear shear modulus of clay (G) during undrained installation by using Eq. 1. R is the pile (outer) radius and r is the radial distance from the pile center:

$$\frac{\Delta u}{S_u} = \ln \frac{G}{S_u} - \frac{r}{R} \quad (1)$$

Although G is a strain dependent parameter, the maximum pore pressure generated due to pile driving in lightly over-consolidated clays is in the order of 4 to 6 times S_u (Randolph 2003). Suites of instrumented pile testing such as those conducted by Doherty (2010) and McCabe (2002) confirm this estimate.

2.1.2 Changes in Pore Water Pressure During Installation of Pile Groups

In practice, piles are mostly driven in groups. Therefore, the effect of driving multiple piles is of main interest for practical applications. Measurements from various pile driving projects show that excess pore pressure changes irregularly in a pile group. If a constant speed of driving is assumed, the pore pressure increases due to driving a pile, then partly drops before driving of the next pile. This increase in excess pore pressure is more rapid at the beginning of driving the pile groups. Eventually, the high peaks of the excess pore pressure over initial pore pressure reaches an upper boundary, after which driving more piles does not increase pore pressures (Hoem 1975). Hence, adding up the generated pore pressure calculated for a certain number of single piles cannot predict the resulting excess pore pressures generated in the soil due to pile group installations.

Svanø (1978) developed a formula based on mass displacement of piles using undrained tangent shear modulus of clay. It considers the material behaviour being contractive or dilative and provides a range for average excess pore pressure generation between piles. Another method was developed for determining excess pore pressure in connection with a piling project in Drammen which provides a curve for

upper and lower bound of pore pressure generation based on a combination of measurements reported in the literature and field measurements on that site (Hoem 1975). Hoem (1975) studied seven sites in Norway where pile driving was conducted in clay. Based on these results, an empirical method was developed to predict the excess pore pressure, in the middle of the pile group, due to driving of pile groups based on pile size and centre to centre distances. These pore pressures were measured one day after completion of driving and are plotted in Fig. 3.

A trend line that fits these points can be formulated as Eq. 2:

$$\frac{\Delta u_{\max}}{u_0} = 0.5 + 30\left(\frac{d}{l}\right)^2 \tag{2}$$

where Δu_{\max} is the maximum generated pore pressure due to pile driving, d is pile diameter, l is distance between centre to centre of piles and u_0 is the initial hydrostatic pressure (if water table is at ground level, $u_0 = z \cdot \gamma_w$ may be assumed). Hoem (1975) plotted excess pore pressure generations (Δu) (normalized by maximum generated excess pore pressure predicted in the previous step) against distance from the centre of the pile group (see Fig. 4). This approach is currently used to predict excess pore pressure for many pile driving projects as a basis for slope stability analysis approaches explained in the following section.

Fig. 3 Pore pressures recorded from pile driving suites in seven sites in Norway against pile size, (Hoem 1975)

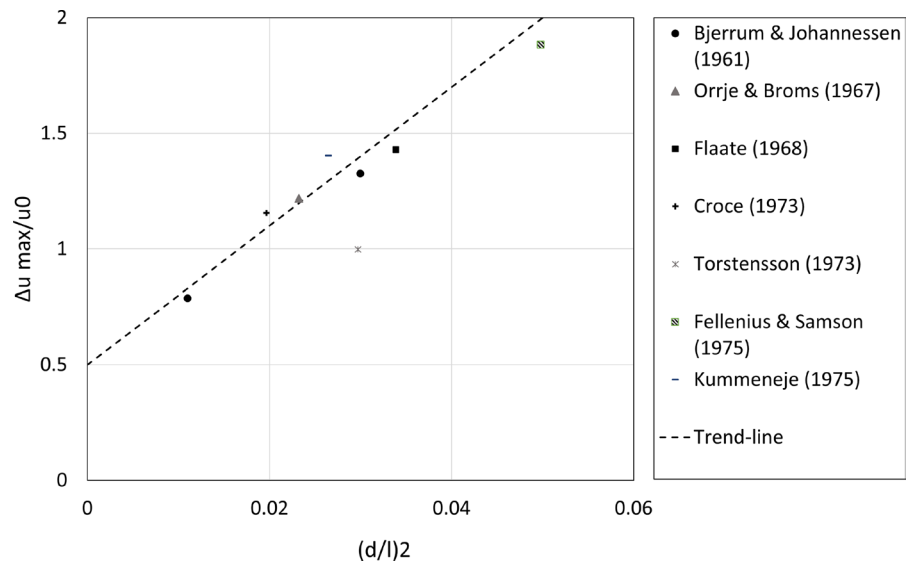
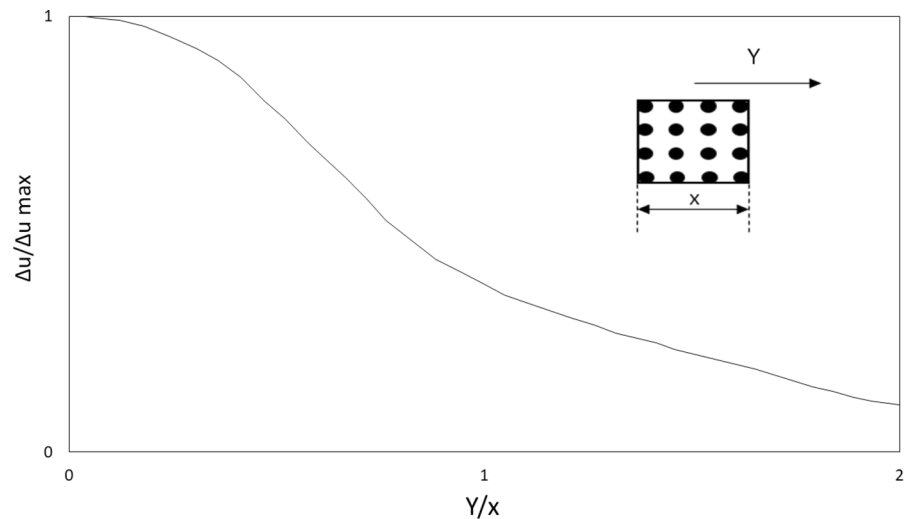


Fig. 4 Excess pore pressure generation as a function of distance to the centre of the pile group, (Hoem 1975)



3 Slope Stability Analysis with Respect to Pile Driving

There is currently no standard method recognized by geotechnical engineers to account for the reduced safety of the slope due to pile driving. However, three methods, commonly used to assess slope stability during pile driving in clay are summarized below.

3.1 Methods of Analysis

3.1.1 Method 1—Effective-Stress-Based LEM

Method 1 is an effective-stress-based analysis. Pile driving, which has an undrained rapid loading mechanism in cohesive soils, results in generation of excess pore pressures. This method considers the generated pore pressures to reduce the effective stresses, assuming no change in total stresses due to pile driving. The excess pore pressure profiles are introduced into an effective-stress-based Limit Equilibrium Method (LEM) which result in a reduced factor of safety. LEM is the conventional method of evaluating slope stability problems. Limit equilibrium analyses identify critical failure surfaces and provide the minimum factor of safety for the slope. The factor of safety is defined as the ratio of the available shear strength along the critical failure surface to the shear stress along this surface.

3.1.2 Method 2 – Total-Stress-Based Analysis with Reduced Undrained Shear Strength Using SHANSEP

This approach also uses estimations of excess pore pressures due to pile driving. If no change in total stresses is assumed, this increase results in reduction of vertical effective stresses. The reduced values of effective stresses are then applied in a SHANSEP (Stress History and Normalized Soil Engineering Properties) formula to reduce the undrained shear strength of soil. SHANSEP technique which assesses the effect of stress history on the shear strength of soil was developed by Ladd and Foott (1974). This concept is based on a series of laboratory tests which establish undrained shear strength as a function of OCR and accordingly, calculates the undrained shear strength using the in-situ vertical effective stress, using Eq. 3.

$$S_u = S(OCR^m)(P_0') \quad (3)$$

where S_u is the undrained shear strength, P_0' is the in-situ vertical effective stress. S is undrained shear strength ratio in normally consolidated state ($OCR=1$) and m is a constant dependent on the type and Plasticity Index (I_p) of clay. Using the SHANSEP principle in a pile driving scenario will provide reduced shear strength values (Kirkebo et al., 2008), (see Eq. 4). Using these reduced strength values in an undrained slope stability analysis will then result in a lower factor of safety during pile driving.

$$\begin{aligned}
 (S_{u1}) &= \left(\frac{S_u}{P_0'}\right) OCR^m (P_1') \\
 &= \left(\frac{S_u}{P_0'}\right) \left(\frac{P_0'}{P_0' - \Delta u}\right)^m (P_0' - \Delta u)
 \end{aligned}
 \tag{4}$$

where S_u and S_{u1} are the undrained shear strength before and after pile driving, respectively. P_0' and P_1' are mean effective stresses before and after pile driving and Δu is pore pressure generated during pile driving.

3.1.3 Method 3 – Change in Horizontal Earth Pressure Coefficient

A method developed by Aas (1975) takes into account the effect of mass displacement due to pile driving by applying a horizontal load equal to the vertical pressure of clay in a vertical plane along the pile length. An increase in horizontal stresses corresponding to the coefficient of horizontal pressure (K_h) equal to 1 is assumed in this method. This increase can be applied as a driving force in a total or effective stress-based LEM analysis to provide a reduced factor of safety due to pile driving. However, it shall be noted that these horizontal stresses will reduce as soon as lateral deformations occur in the slope.

3.2 Resulting Stress Paths

Stress paths for a soil element along the possible failure mechanism in all three methods are illustrated

in Fig. 5, in a p-q plane. In method 1, a reduction in effective stress moves the stress condition closer to the failure line. In method 2, the undrained shear strength is reduced due to an effective stress change, while method 3 assumes a change in shear stresses.

3.3 Requirements According to Design Codes

In Norway, a strict construction control is applied when piling is conducted adjacent to slopes. If method 1 and 2 are used, based on slope stability analysis results, criteria are applied on excess pore pressure developed during and at a certain time after pile driving by installing piezometers on site. In case of further increase of excess pore pressures above the advised limit, pile driving should be stopped or continued on a different part of the site. In extreme cases, piling equipment should be removed from site completely. Design codes note that ground conditions containing sensitive or quick clay may cause a progressive (brittle) failure which necessitate factor of safety between 1.4 to 1.6 during pile driving, according to consequence class (see Table 2).

4 Numerical Analyses

The effect of pile driving on slope stability was examined using the finite element package Plaxis 2D. A

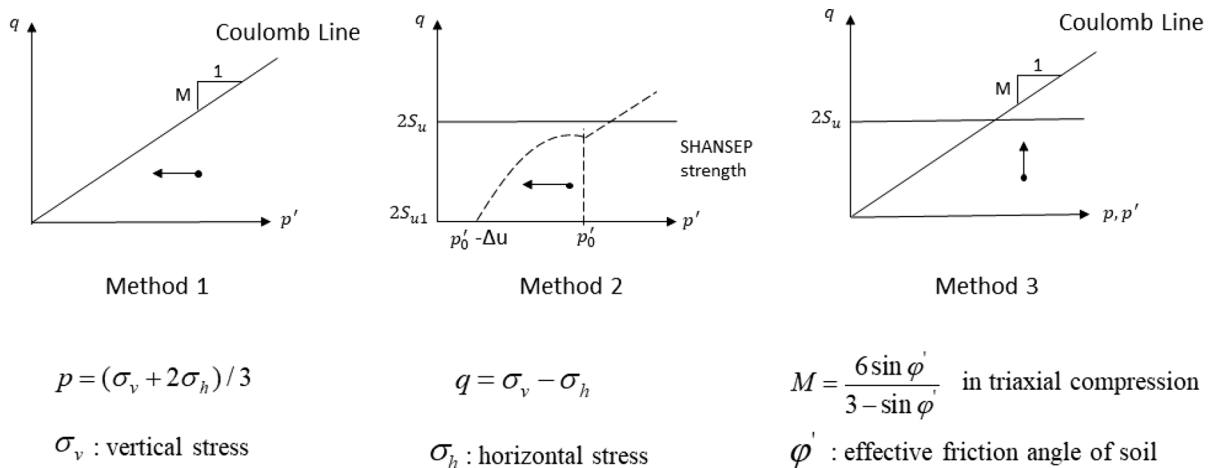


Fig. 5 Stress paths related to a soil element in the presented methods

Table 2 Required factor of safety in effective and total stress analysis from Norwegian Road Authorities Handbook N200: “Geotechnics in road design” in accordance with Eurocode 7

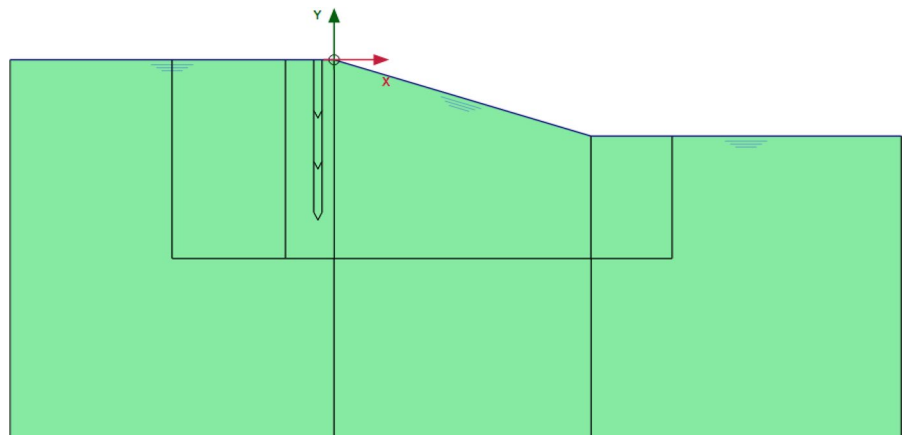
Severity	Failure mechanism		
	Ductile dilatant failure	Neutral failure	Brittle contracting failure
Low	1.25	1.3	1.4
Medium	1.3	1.4	1.5
High	1.4	1.5	1.6

series of finite element analyses using three different constitutive soil models were conducted. The purpose of these analyses was to show the limitations of the methods discussed in Sect. 3 and provide a better understanding of the failure mechanisms that pile driving may initiate in a slope. In some cases, layers of silt can be present between layers of clay. This was taken into account by applying a higher horizontal permeability in the soil material to show the effect of quicker pore pressure dissipation into the slope.

4.1 Model Geometry

An arbitrary slope was modelled in Plaxis 2D. Figure 6 shows the model geometry, comprising of a slope and a pile row at the top of the slope, as is common for a bridge abutment.

Fig. 6 Model geometry in Plaxis 2D



4.2 Constitutive Models and Parameters

First, an elastic-perfectly plastic Mohr–Coulomb soil model was used. Secondly, more advanced soil models were used: Soft Soil that takes into account the excess pore pressure generated by the contraction of clay and NGI-ADPsoft that can simulate the strain softening in the soil. The soil parameters were selected in an arbitrary way to represent soft and brittle clay deposits.

4.2.1 Mohr–Coulomb

A numerical analysis was conducted using the elastoplastic Mohr–Coulomb constitutive soil model. This was done using both Undrained A analysis, where soil strength is defined using effective stress properties. Table 3 shows the soil parameters used in this model.

4.2.2 Soft Soil

Soft Soil is a constitutive soil model based on Modified Cam Clay. The model has a stress-dependent stiffness while the failure criterion is handled in the same way as Mohr–Coulomb. This soil model can take into account the excess pore pressure generated due to shear deformation of clay. The required parameters include compression and swelling indexes as summarised in Table 4. An effective stress analysis was done in this case while the pile driving was conducted in a consolidation phase where pore pressures are also being calculated. To facilitate dissipation, the

Table 3 Parameters selected for the Mohr-Coulomb model (Undrained A) in Plaxis

Parameter	Undrained A model
Unit weight (kN/m ³)	18
Poisson's ratio	0.3
Young's Modulus (kN/m ²)	10,000
Effective cohesion (kPa)	5
Friction angle (deg)	25
Dilatancy angle (deg)	0

Table 4 Parameters for the Soft Soil model in Plaxis

Parameter	Value
Unit weight (kN/m ³)	18
Initial void ratio	0.5
Modified compression index	0.1
Modified swelling index	0.02
Effective cohesion (kPa)	3
Friction angle (deg)	30
Dilatancy angle (deg)	0
Coefficient of lateral stress in normal consolidation	0.637
Slope of the critical state line	1.2
Vertical permeability (m/day)	1e-5
Horizontal permeability (m/day)	1e-4

Table 5 Parameters for the NGI-ADPSoft model in Plaxis

Parameter	Value
Unit weight (kN/m ³)	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 active strength (%)	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 (m)	0.1

horizontal permeability of the soil was set higher than the vertical permeability in this case.

4.2.3 NGI-ADPsoft

Sensitive clays show a reduction in their undrained shear strength after the peak strength is reached in undrained conditions. This characteristic is referred to as strain-softening. The NGI-ADPsoft is an advanced constitutive soil model that includes both anisotropic undrained shear strength and post peak strain softening (where shear strength reduces as function of shear strain until the residual shear strength is reached, as illustrated in Fig. 12). To obtain a representative factor of safety in a strain-softening material, the gravitational loading applied to the soil is gradually increased until the slope becomes unstable. The representative factor of safety of the slope in this case is the ratio between the increased gravitational acceleration when slope failure occurs to the actual gravitational acceleration. More information about this soil model can be found in Jostad et al. (2014) and Grimstad and Jostad (2012). A total stress analysis was conducted to investigate the effects of pile driving on slope stability using NGI-ADPsoft. Table 5 provides the input parameters used for this analysis.

4.3 Analysis Sequences

For the purpose of this study, it was deemed suitable to only simulate the lateral push from pile driving on the slope stability, assuming the row of piles are installed simultaneously to full depth. For this reason, pile driving was simulated using the prescribed volumetric strain feature in Plaxis, where in a 2D setting, lateral strains are applied to a certain area. This function expands or contracts a soil cluster equal to the applied strain while maintaining the same stress levels (Brinkgreve et al. 2017). This change in strains will then affect the surrounding soil until an equilibrium is reached between all clusters. Application of prescribed displacements on the pile cluster, generates lateral displacements in the soil similar to expansion of a cylindrical cavity in a soil medium, which is widely used in combination with a constitutive soil model to simulate the effects of pile driving in the soil (Hunt 2000).

All the analyses start with a Gravity Loading phase as a slope stability problem is dealt with. This is followed by a phase where lateral strains are applied to the defined pile row based on pile size and centre to centre spacing into the plane. In the Mohr–Coulomb analysis a phi-c reduction 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 analysis, pile driving was followed by a consolidation phase where excess pore pressure generated during pile driving is partially dissipating.

4.4 Results

4.4.1 Mohr-Coulomb Analysis

The resulting excess pore pressure contour from the numerical analysis using Mohr-Coulomb (Undrained A) is illustrated in Fig. 7. The analysis using Mohr–Coulomb constitutive model cannot represent the excess pore pressure contour around the pile accurately as the model does not take into account the excess pore pressure that results from shearing of clay. It shall be noted that excess pore pressure is shown with a negative sign in Plaxis. Furthermore, the phi-c reduction analysis showed no change in the factor of safety after pile driving, as illustrated in Fig. 11.

4.4.2 Soft Soil Analysis

Figure 8 show the contours of positive excess pore pressures generated using the Soft Soil model.

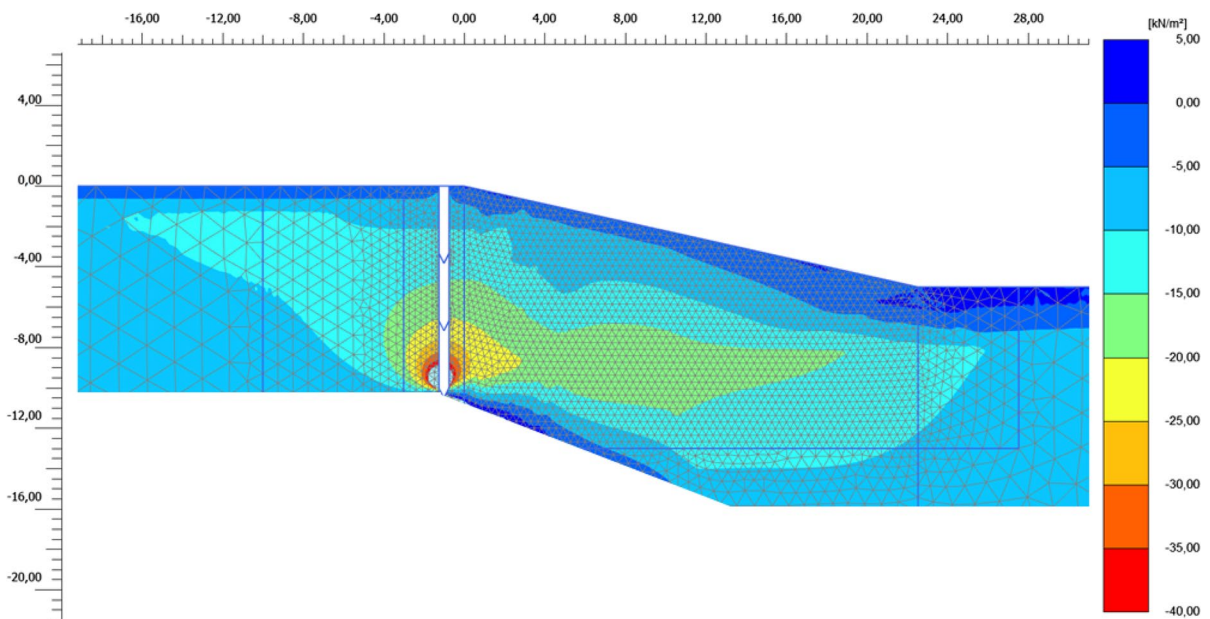


Fig. 7 Excess pore pressure field due to pile driving (Mohr-Coulomb constitutive soil model, undrained A analysis results)

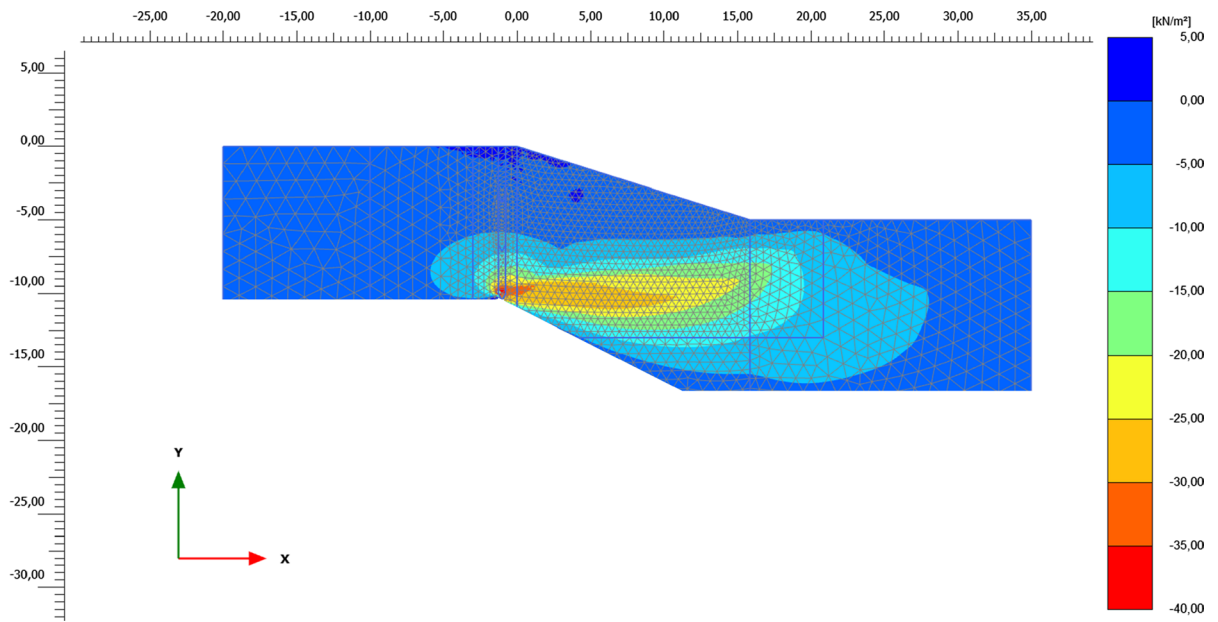


Fig. 8 Excess pore pressure field due to pile driving (soft soil)

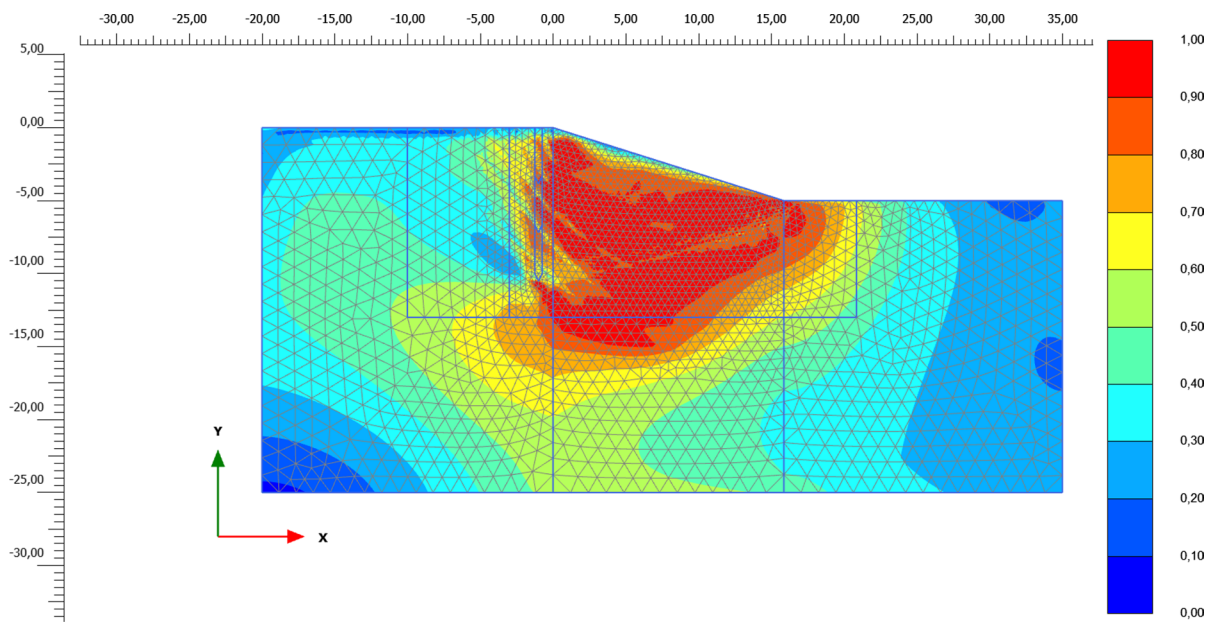


Fig. 9 Shear strength mobilisation in the slope after pile driving

The phi-c reduction phase is not representative in a soft soil model, but Fig. 9 which depicts the relative mobilisation of shear strength in the slope after pile driving shows a situation very close to failure, where

a significant part of the soil in the slope shows 90% mobilisation and higher.

Furthermore, slope movements were assessed during the dissipation phase. Figure 10 shows the horizontal movements of the slope during the

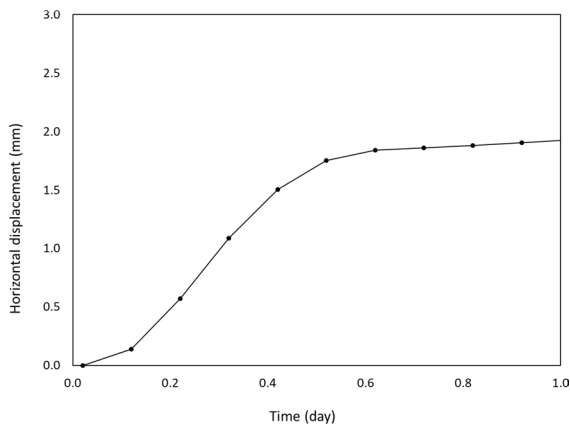


Fig. 10 Horizontal displacements of the slope during excess pore pressure dissipation

dissipation phase versus time. It can be seen that dissipation of pore pressures can be associated with increasing displacements in the slope. The graph in Fig. 10 shows a critical time of highest velocity of pore pressure dissipation of 0.4 days in this arbitrary case. In this case, the effective stress is reduced at the pile toe during dissipation of excess pore pressures as the hydraulic conductivities are anisotropic. While the effective stresses at the pile side are not increased significantly during dissipation as the total stress increase does not significantly transfer to effective stresses due to the volume reduction in the soil (horizontal relaxation). Hence, there is a moment during the dissipation process that the critical mechanism needs to increase shear mobilization to take the shear and therefore increases the horizontal displacement. Although the displacements can be minimal, it can have a negative effect on the slope stability, especially in sensitive clay deposits, where small displacements can result in progressive types of failure.

4.4.3 NGI-ADPsoft Analysis

The results of the gravity increase phase before and after pile driving phase, using NGI-ADPsoft constitutive soil model in Plaxis showed a reduction in the factor of safety of slope due to pile driving (see Fig. 11).

4.5 Numerical Analysis Overview

According to the provided results, an analysis using the Mohr–Coulomb soil model cannot simulate the reduction of the safety factor of the slope, as the excess pore pressures generated are (1) does not include shear induced pore pressure and a reduction in the mean effective stress, and (2) do not have an effect on soil strength in undrained stability analysis. This proves the shortcoming of method 1 and 2 according to which the excess pore pressures generated due to pile driving incorrectly reduce the effective stresses in the surrounding soil and hence, reduce shear strength and factor of safety of the slope. However, the total stress increase due to pile driving does not affect the effective stresses and does not change the soil strength. With no change in soil strength, such simulations do not provide a reduction in the factor of safety of the slope during pile driving and cannot represent a triggering mechanism.

The analysis using Soft Soil model is more representative in terms of replicating the excess pore pressure field generated around the pile. This analysis also showed that stability of the slope can worsen when these pore pressures migrate to other parts of the slope. In other words, the slope can be in a more critical situation in terms of stability during the dissipation phase of excess pore pressure (when lower values are reached in the piezometers during on-site monitoring) can put the slope in a more critical situation.

5 Discussion

5.1 Limitations of the Current Methods in use

The three methods presented in Sect. 3.1. have clear limitations:

- Field data from instrumented sites shows generation of excess pore pressure which depend on OCR of soil (Burns and Mayne 1999). Method 1 and 2 disregard this and relate pore pressure changes only to size of pile group, regardless of soil properties and stress history.
- Sensitive clays lose strength under undrained loading conditions (Thakur et al. 2014). Figure 12 illustrates the difference between a strain-softening material with a peak and residual shear

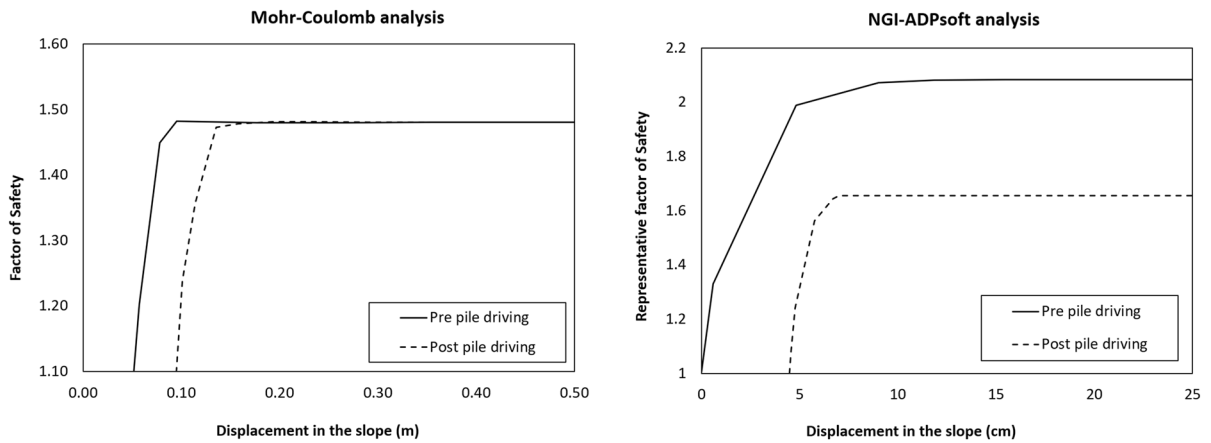


Fig. 11 Comparison of factor of safety using different constitutive soil models in Plaxis

strength, and an elastic-perfectly plastic material. LEM assumes an elastic-perfectly plastic soil response which fails to capture strain-softening behaviour of sensitive clays (Bernander 2011). Increased displacements in soil due to pile driving may cause a gradual progression of strain-softening behaviour throughout the slope that could ultimately lead to a global failure. This is known as a progressive failure mechanism. Considering the possibility of a progressive failure mechanism in such deposits, pile driving in sensitive soils poses higher stability risks. Strain-softening of the soil material can be simulated using more advanced soil models such as NGI-ADPsoft. Employing this material model could represent a drop in the factor

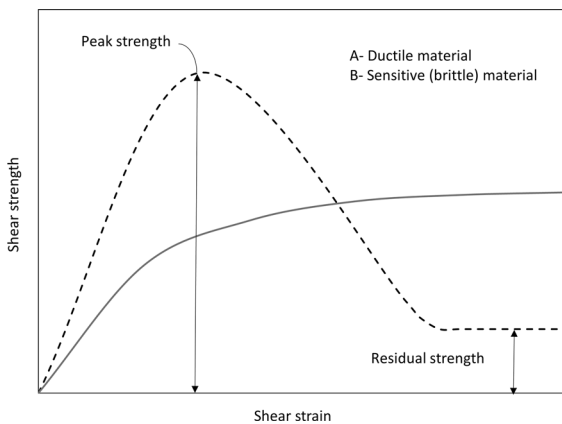


Fig. 12 Illustration of shear stress vs. shear strain behaviour in different types of soil, based on a figure by Bernander (2011)

of safety of the slope due to pile driving, as illustrated in Fig. 11.

- Method 1 and 2 assume that generated excess pore pressures due to pile driving reduce effective stresses in the soil (as illustrated in Fig. 6) and consequently also reduce shear strength. However, this assumption is conceptually wrong as pile driving also increases total stresses in the soil. Hence, excess pore pressures that originate from an increase in total stresses may not affect the effective stresses nor the undrained shear strength in short-term for cohesive soils.
- Method 3 calculates a reduced factor of safety due to pile driving by inducing a shear stress increase in a slope stability analysis from calculated increase in horizontal earth pressure. However, these horizontal stresses will reduce as soon as lateral deformations occur in the slope.
- Based on the use of in-site pore pressure measurements during pile driving (and using method 1 or 2 to assess slope stability after pile driving) it is assumed that dissipation of the excess pore pressure and accordingly, reduction of the recorded pore pressure in the piezometers mean that the slope is safe for continuation of pile driving. However, the slope can be in a more critical condition during dissipation of excess pore pressures, as observed by numerical analysis results in Fig. 10. The presence of several silt layers can expedite dissipation of the generated excess pore pressure into the slope and cause areas of reduced strength (see Fig. 2). This could trigger stability in

other parts of the slope. Hence, dropping of excess pore pressures in the piezometers does not guarantee that piling can resume without further risks to slope stability.

- According to Sect. 3.1., the excess pore pressure due to pile driving is a function of the shear strengths of the soil where higher soil strength results in higher excess pore pressure generation. According to method 1 and 2, higher excess pore pressure has a higher negative effect on the slope's short-term factor of safety. However, in a slope stability problem, the factor of safety of the slope is also a function of the shear strength where higher strength results in a higher factor of safety.

5.2 Reliability

Despite these limitations, engineers have developed a confidence towards these methods. This can be attributed to the fact that the excess pore pressure generated due to pile driving is simple to measure and control on site, while earth pressure and total stress changes may not be.

6 Conclusions

Pile driving can affect soil strength and cause slope failure in soft sensitive soils, as observed in case examples mentioned in Sect. 2. Methods currently used to calculate reduction in the safety factor of slope during pile driving have certain limitations which might have risks specifically in the presence of sensitive soils. The following can be concluded:

- Method 1 and 2 estimate strength reduction due to increased pore pressure assuming no change in total stresses. This assumption is not correct as pile installation displaces the soil radially outwards which increases the total horizontal stresses in soils.
- Method 3 does not provide a realistic reduction in the factor of safety, since it does not consider that the horizontal stresses generated in the soil by pile driving is reduced when the soil start to move downwards at failure.

- Sensitivity and brittleness behaviour of soil are not considered in any of the methods. Hence, the possibility of a progressive failure which might occur in sensitive soil deposits is not considered.
- Applying a limit on measured pore pressure changes due to pile driving will not necessarily guarantee that failure cannot occur as excess pore water pressure can dissipate to other parts of slope that could trigger instability. The slope can be in a more critical situation during dissipation of the excess pore pressure, as seen in the results of the numerical analysis.
- This evaluation proves a need for further research to better understand the mechanism of failures during pile driving in soft and sensitive clays in order to improve the methods or introduce substitute approaches.
- An alternative to assess slope stability by means of LEM, is to use numerical methods such as the Finite Element Method (FEM), especially in the case of a brittle or quick clay deposit which increases the risk of a progressive type of failure.
- The effect of pile driving on slope stability cannot be simulated using a simple soil model such as Mohr–Coulomb. Reduced stability or failure may be predicted if a more advanced soil models that can simulate strain softening and shear induced pore pressure of the actual clay are used. In the presence of comprehensive information about the sensitivity and brittleness of the soil material, such constitutive soil models might be able to provide an accurate prediction of the failure.

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Data availability All the data that was used are provided in references.

Code availability Commercial software Plaxis 2D was used where the university has license for. No code was developed by authors for this publication.

Declarations

Conflict of interest All the authors declared that they have no conflict of interest.

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