Finite element investigation of vertical stabilisation piles in a stiff clay excavated slope using a nonlocal strain softening model

Etude par la méthode des éléments finis de pieux stabilisateurs verticaux sur une pente excavée dans des sols argileux rigides à l'aide d'une modélisation non-local de l'amollissement

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ABSTRACT Slopes excavated in stiff clay are prone to delayed and brittle failure. These slopes are widespread across the rail and road networks in the United Kingdom. The use of a row of discrete vertical piles is an established method, sucessfully used to remediate failure of existing slopes and to stabilise potentially unstable slopes created by widening transport corridors. This paper will challenge the assumptions made in current design procedures for these piles, which treat the pile only as an additional force or moment and simplify soil/pile interaction. Two dimensional plane-strain finite element analyses were performed to simulate the excavation of the slope in an overconsolidated clay and the interaction of vertical piles within the slope. A nonlocal strain softening model was employed for the stiff clay to reduce the mesh dependency of the solution. This model controls the development of strain by relating the surrounding strains to the calculation of strain at that point, using a weighting function. A variety of different failure mechanisms developed depending on pile location and length. The variability of the pile and slope interaction that was modelled suggests that an oversimplification during design could miss the critical failure mechanism or provide a conservative stabilisation solution. Given the prevalence of stiff clay in the UK transport infrastructure, increased capacity requirements and the age of slopes in this material, an informed and more realistic design of stabilisation piles will become increasingly necessary.

RÉSUMÉ Les pentes excavées dans des sols argileux rigides sont susceptibles de produire une rupture cassante et différée. Ce type de pentes est fréquemment rencontré aux abords des réseaux ferroviaires et routiers du Royaume-Uni. L'implantation d'une rangée de pieux verticaux distincts est une méthode couramment utilisée pour parer à l'instabilité de pentes existantes et pour stabiliser des pentes potentiellement déstabilisées par l'élargissement d'une voie de transport. Cet article conteste les hypothèses de base communément employées lors de la mise en place d'une telle solution, qui simplifient le modèle du pieu, le considérant comme une force ou un moment additionnel en négligeant toute interaction entre le sol et le pieu. Des analyses éléments finis de déformation plane bidimensionnelles ont été réalisées pour modéliser l'excavation de la pente dans un sol argileux sur-consolidé ainsi que l'interaction des pieux verticaux sur la pente. Dans ces analyses, on a utilisé un processus d'amollissement de type non-local pour reproduire le comportement de l'argile dans le but de réduire la dépendance des résultats à la finesse du maillage. Ce modèle prend en compte la progression des déformations autour du point d'étude en utilisant une fonction de pondération. Des mécanismes de rupture différents sont apparus selon l'emplacement et la longueur des pieux. La variabilité des interactions entre pieux et pentes modélisés indique qu'une simplification excessive lors de la conception peut conduire à ne pas identifier le mécanisme de rupture critique ou à sur-dimensionner l'ouvrage. Compte tenu de la prévalence de ce type de talus de sol argileux aux abords des infrastructures de transport au Royaume-Uni, de l'augmentation des charges sur ces voies de transport et de l'âge des pentes, une optimisation des méthodes de dimensionnement des pieux stabilisateurs sera de plus en plus nécessaire.

1 INTRODUCTION

Slopes excavated in stiff clay are prone to delayed and brittle failure (Potts et al. 1997). These slopes are widespread across the rail and road networks in the UK (Wilkinson et al. 2011). The use of a row of discrete vertical piles is an established stabilisation method, successfully used to remediate failure of an existing slope and to stabilize potentially unstable slopes created by widening transport corridors (Carder 2009; Ellis et al. 2010).

The current design procedures for horizontally loaded vertical stabilisation piles employ the displacements and critical slip surface of the unstablised slope. The p-y method uses the expected soil displacements to calculate pile reaction (Baguelin et al. 1977). In a limit equilibrium or limit analysis design procedure, the pile is treated only as an additional force or moment located where the critical slip surface and pile coincide (Hassiotis et al. 1997).

These methods assume that the insertion of a pile will not affect the failure mechanism and the stabilizing effect of the pile will not be significantly affected by its position or length. The finite element method can model the pile and soil interaction in an unstable slope without a predetermined location for the slip surface. The influence of pile location and length on the slope failure mechanism can therefore be assessed.

2 FINITE ELEMENT ANALYSES

Two dimensional plane-strain finite element analyses were performed to simulate the excavation of the slope in overconsolidated clay and the interaction of vertical piles within the slope. The slope is not a specific case study, but a generic slope with dimensions known to be unstable in London Clay (Potts et al. 1997; Ellis & O'Brien, 2007). The slope is 10m in height with a 1 in 3 vertical to horizontal slope angle (Figure 1). Soil properties for London Clay are employed (Table 1).

Table 1. Soil properties for London clay excavated slope analyses.

Property	Value
Bulk unit weight	$\gamma = 18.8\text{kN/m}^3$
Peak strength	$c'_p = 7\text{kPa}$, $\varphi'_p = 20°$
Residual strength	$c'_r = 2\text{kPa}$, $\varphi'_r = 13°$
Nonlocal plastic strain limits	$\varepsilon_p^{p*} = 5\%$, $\varepsilon_r^{p*} = 20\%$
Voids ratio	$\mu = 0.2$
Stiffness, Young's Modulus	$E = 25\ (p'+100)$ min 4000kPa
Coefficient of Earth Pressure at rest	$K_0 = 2.0$
Permeability	$k_0 = 5\text{x}10^{-10}\text{m/s}$ $b = 0.003\text{m}^2/\text{kN}$
Angle of Dilation	$\psi = 0°$

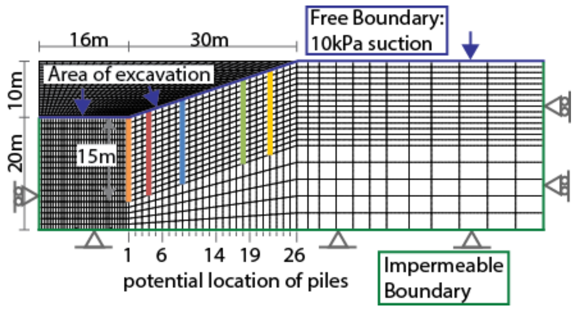

Figure 1. Finite element mesh with excavation dimensions, boundary conditions and potential pile locations.

2.1 Boundary conditions

Coupled consolidation analyses were performed using the Imperial College Finite Element Program (ICFEP). Plane strain eight–noded isoparametric elements with reduced integration were used. An accelerated modified Newton-Raphson scheme with a sub-stepping stress point algorithm was employed to solve the nonlinear finite element equations (Potts & Zdravkovic 1999). No horizontal displacement was allowed on the vertical boundaries, whereas the bottom boundary was fixed in both horizontal and vertical directions (Figure 1).

Before excavation of the slope, initial stresses are specified in the soil using a bulk unit weight of $\gamma = 18.8\text{kN/m}^3$ and a uniform coefficient of lateral earth pressure $K_0 = 2$. The pore water pressures are hydrostatic with 10kPa suction specified at the soil surface, following the average height expected for the phreatic surface in the UK (Vaughan & Walbanke 1973). Seasonal fluctuations are not modeled. The bottom and side boundaries are impermeable. The permeability, k of the soil is modeled as isotropic and linked to the mean effective stress, p' using the non-linear relationship in Equation 1 (Vaughan 1994).

$$k = k_0\, e^{-bp'} \quad (1)$$

The slope was excavated in horizontal layers over 0.25 years. This unloads the soil surrounding the excavation and the low permeability of the soil creates negative pore water pressures. After excavation 10kPa suction is applied at the free boundary (Figure 1). Time and consolidation allow these excess pore water pressures to slowly dissipate. The changes in pore water pressures and strain softening behaviour of the stiff clay eventually lead to failure of the slope. The point of failure is defined as the last increment of the analysis that will converge with a time step of 0.01years. Initially time steps of 1 year are employed and the size of the incremental step is reduced as slope failure is approached.

2.2 *Nonlocal strain softening soil model*

A nonlocal elasto-plastic constitutive soil model is employed to simulate strain softening soil behaviour. A Mohr-Coloumb failure surface is adopted. The soil strength properties, the angle of shearing resistance, φ' and cohesion, c' vary with the nonlocal strain ε^{p*}. Peak and residual values are applied before and after the specified nonlocal plastic strain limits respectively (Table 1), with a linear progression between the limits. The nonlocal strain is employed to reduce the mesh dependency of the strain softening calculations (Summersgill et al. 2014). It regulates the reduction in soil strength by referencing a nonlocal strain, which is calculated by relating the surrounding values of local deviatoric plastic strain $\varepsilon^p(x_n^{'})$ to strain at the calculation point, $\varepsilon^p(x_n)$ using a weighting function, $\omega(x^{'}_n)$ (Equations 2, 3 and 4). The weighting function uses the G&S modifications (Galavi & Schweiger 2010).

$$\varepsilon^{p*}\left(x_n\right)=\frac{1}{V_w}\iiint\left(\omega\left(x_n^{'}\right)\varepsilon^p\left(x_n+x_n^{'}\right)\right)dx_1^{'}\,dx_2^{'}\,dx_3^{'} \qquad (2)$$

$$\omega\left(x_n^{'}\right)=\frac{\sqrt{\left(x_n^{'}-x_n\right)^T\left(x_n^{'}-x_n\right)}}{l^2}\exp\left[-\frac{\left(x_n^{'}-x_n\right)^T\left(x_n^{'}-x_n\right)}{l^2}\right] \qquad (3)$$

$$V_w=\iiint\omega\left(x_n^{'}\right)dx_1^{'}\,dx_2^{'}\,dx_3^{'} \qquad (4)$$

The nonlocal length parameter, l controls the shape of the weighting function. It also affects the softening rate of the soil. A value of l = 1m was used to create an appropriate softening rate with the strain limits of 5% and 20%. An additional nonlocal parameter, the radius of influence, was used to restrict the area of the reference space for the nonlocal calculations and increase numerical efficiency. The radius of influence was set at 3m. With a nonlocal length parameter of 1m and 3m radius of influence the analyses only required a 30% increase in computational time compared to the equivalent analyses employing a local strain softening method (Summersgill 2015).

2.3 *Pile Simulation*

The mesh has been designed to allow the placement of vertical piles in 26 different locations between the toe and crest of the slope (Figure 1). The length of the pile can be varied at 1m intervals up to 15 meters. In these analyses the pile is wished in place immediately after excavation of the slope.

The pile is modelled using a single column of beam elements placed between the solid quadrilateral elements. These elements are of zero thickness and model the bending behaviour of the pile using the specified stiffness, density, cross sectional area, A and second moment of inertia, I. The simulated pile diameter is 0.9m with a spacing of 2.7m or three diameters. The calculated A and I were divided by the pile spacing to account for the total quantity of soil that would be supported by a discrete pile in a row. A Young's modulus of 14GPa and a density of 2400kg/m^3 were specified. A linear elastic constitutive soil model is employed and the maximum bending moment is monitored to identify potential plastic hinge formation.

3 RESULTS

The slope failure mechanism for each analysis can be identified from the contours of accumulated plastic strain or the incremental displacement vectors for the final increment of the analysis. The improvement in the stability of the slope is indicated by the time to slope failure for each analysis, as well as the change in failure mechanism.

For a slope without any stabilisation piles, failure occurred 40.46 years after excavation was complete. The contours of strain showed the development of two potential slip surfaces initiating below the toe of the slope and extending into and towards the crest of the slope. The shallower slip surface became critical. Inserting stabilisation piles that interact with either of the two slip surfaces changed the failure mechanism and time to slope failure. The two sets of analyses presented investigate the influence of pile position and length.

3.1 Pile Position

A 15m long pile was placed in each of the 26 locations between the toe and crest of the slope in Figure 1. These locations are spaced 1.2m apart. The position of the pile was found to have a large influence on the pile and slope failure mechanism. Five failure mechanisms were identified by the pattern of slope and pile movements. The vectors of incremental displacement for the final increment give an example of each mechanism in Figure 2. The sizes of the arrows are relative to the largest incremental displacement for each analysis, but not proportional between the analyses due to the large difference in the size of displacements depending on the mechanism.

The positions of the five analyses shown in Figure 2 are identified in Figure 1 by the thicker coloured lines. The numbers in Figure 1 identify the last pile position for each mechanism type. This is reinforced by the different colours of the bars in Figure 3, comparing the variation in time to failure due to the position of the pile.

Two of the mechanisms, (a) and (e), did not interact with the pile. With a pile at the very toe of the slope, failure occurred above the pile and the time to failure was 3 years less than without a pile. This is likely due to the supressed movements at the base of the excavation that created non-critical slip surfaces in the no-pile analysis.

The failure of the slope downslope of the pile without pile interaction, mechanism (e), provides a small increase in time to failure of 5 to 23 years. The pile effectively reduces the height of the slope by up to 2.4m, but sufficient height remains for failure of the slope to occur without the contribution of the mass behind the pile to increase the destabilising force. There was therefore very little displacement or bending of the piles in these analyses.

For mechanism (d) the piles did undergo bending and displacement, as it can be seen by the horizontal arrows at the location of the pile and small movements of the soil upslope, Figure 1(d). This resulted in an increase in the time to failure from 40 years to between 57 and 98 years after excavation. This is still likely to be an inadequate improvement in stability for the required lifetime of transport slopes.

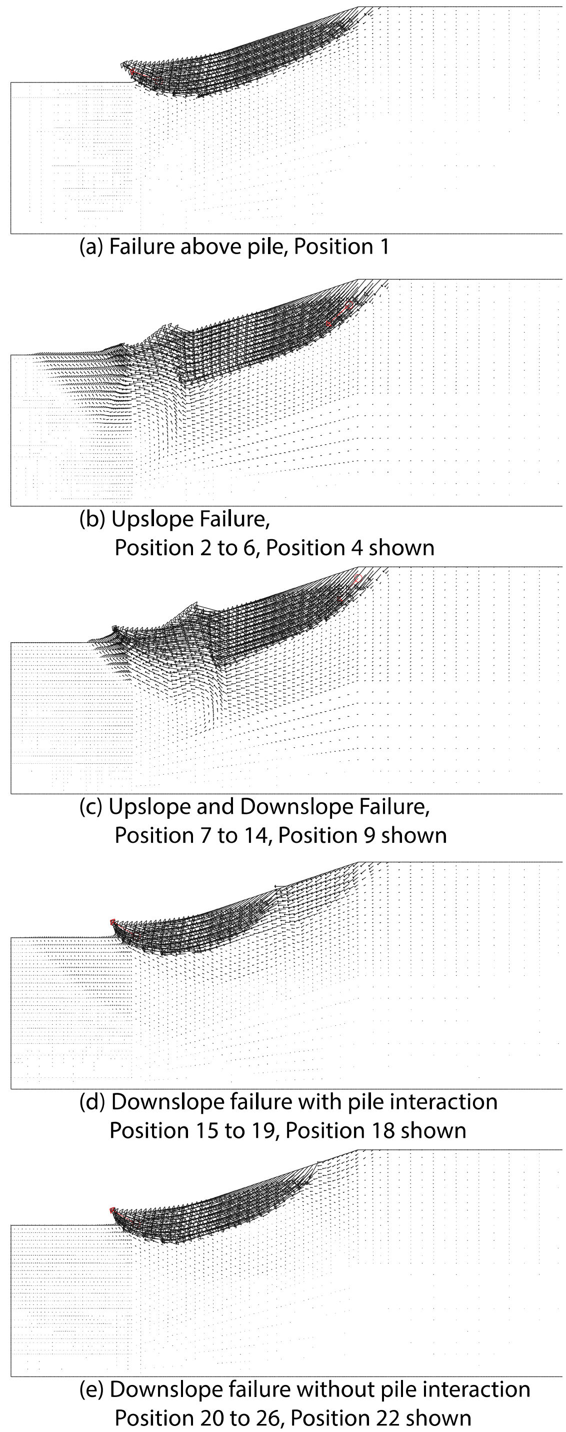

Figure 2. Pile and slope failure mechanisms, shown by the incremental displacement vectors for the final increment of an analysis.

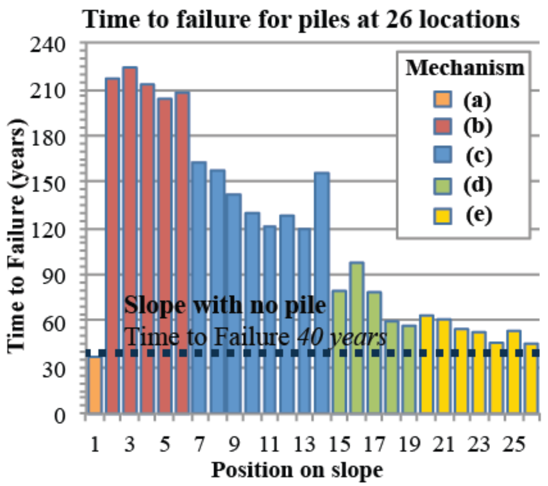

Figure 3. Comparison of time to failure depending on the position of the pile between the toe and crest of the excavated slope.

Two mechanisms (b) and (c) do provide a significant improvement in time to failure, with ranges of 203 to 224 and 120 to 163 years respectively prior to slope failure. The movement of the pile in these analyses is an integral part of the failure mechanism. The pile movement occurs due to the force from the soil upslope or the movement of soil downslope reducing support in front of the pile.

In mechanism (c), it is a combination of these two movements causing upslope and downslope failure. Slip surfaces have formed downslope of the pile in the same area as the no-pile analysis, Figure 4(a). Movement on these slip surfaces contributes to slope movement and is the reason for a smaller time to failure than mechanism (b). In mechanism (b), sufficient soil is present behind these locations to move the pile downslope and form a slip surface upslope. The soil downslope in mechanism (b) positions is pushed into the excavation, but has not formed its own slip surface.

The presence of the pile does not prevent soil movement, which occurs as a reaction to slope excavation and high lateral soil stresses. The pile is most effective when it interacts with the development of the slip surface and affects the failure mechanism. For these slope dimensions, this occurs in the bottom half of the slope, but not at the very toe of the slope. The piles placed in the top half of the slope create only marginal improvements.

3.2 Pile Length

The variation of pile length had a significant influence on the stabilizing effect of the pile. The analyses for a pile in position 9 with different lengths are presented in this section. The analysis for the 15m length pile is the same analysis as shown in Figure 2(c). Additional analyses varying the pile length from 3m to 12m at 3m intervals are presented with the 15m results in Figure 4. The accumulated strain contours show the position and relative development of slip surfaces, both critical and non-critical. The change in strain development for these analyses is directly compared to the analysis without a pile, Figure 4(a).

The time to failure for each analysis is stated on the label. Constructing a pile 3m or 6m in length immediately after excavation reduced the stability of the slope by a quarter compared to not constructing a pile. A 3m pile does not intersect the shallow slip surface and a 6m pile barely intersects this surface. The critical slip surface forms beneath the base of the pile, translating it within the unstable mass. Furthermore, the presence of the pile encourages the development of the critical surface, reducing the time to slope failure and development of other slip surfaces.

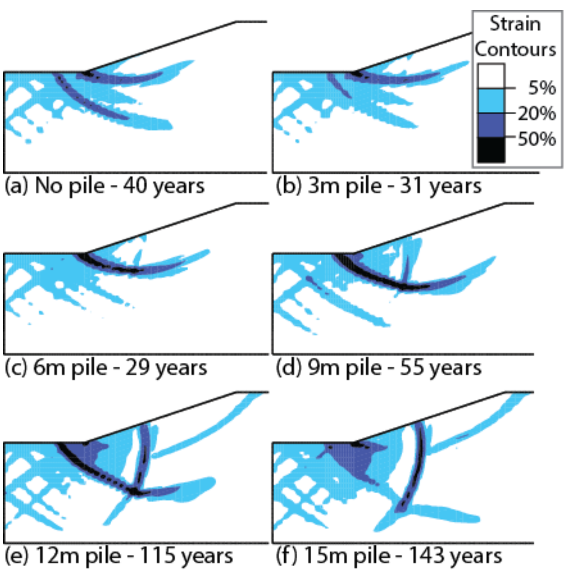

Figure 4. Accumulated plastic strain contours for the last increment of the analysis, showing the influence of pile length on mechanism and time to failure.

The 9m pile provides some improvement by increasing the depth of the critical slip surface, but it can still pass beneath the pile (Figure 4(d)). With a 12m pile, a mechanically viable slip surface cannot form underneath the pile. The slip surface formed down-slope of the pile and pressure from the soil behind the pile eventually causes sufficient bending of the pile for a slip surface to form upslope. This requires a longer period for development, extending the stability of the slope to 115 years. The further increase in pile length to 15m requires even more time. This is likely due to the reduced movements downslope of the pile because the base of the 15m pile is too deep to interact with soil movement at the toe of the slope in the same way as the 12m pile.

The behaviour with pile length discussed here is only valid for position 9 in the slope with less than 10m of soil below the base of the pile. It would be expected for the impact of pile length to vary with location and depth to bedrock, reflecting the changing interaction of the pile and mechanically viable slip surfaces.

4 CONCLUSION

The interaction of a vertical stabilisation pile and slope is complex. Construction of a pile does not provide a single stabilizing action at the intersection with the critical slip surface of the unstabilised slope. Moreover, the pile is most effective in extending the stability of the slope when the failure mechanism is significantly altered by the presence of the pile. An oversimplification during design could miss the critical failure mechanism or provide a conservative stabilisation solution.

The pile position and length have a large influence on the stabilising effect of the pile. The pile should be designed to interact with all potentially critical slip surfaces. These analyses demonstrated that for stiff clay excavated slopes the pile should be placed between the midslope and the toe of the slope, although not exactly at the toe of the slope. For the presented example of a 10m high, 1 in 3 angled slope, a pile placed one third in from the toe of the slope should be more than 9m long to provide a reasonable improvement in stability.

These analyses indicate the sensitivity of the discrete pile row and slope interaction to pile design. In addition to pile length and position, further factors to consider would include the pile diameter, spacing, stiffness, time of pile construction and 3D analyses modelling arching between piles. Without an understanding of these factors, a simplified design method could provide misleading results.

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