Fracturing of Soil Caused by Pile Driving in Clay

Fracturation du Sol Causé par le Battage de Pieux dans l'Argile

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SYNOPSIS The effect of pile driving on the stress distribution in cohesive soil has been analyzed and compared with field observations. The analysis indicates that the increase of pore pressure during driving can cause fractures or cracks in the plastic zone around the pile. The cracks increase the rate of pore pressure dissipation and reconsolidation of the clay after the driving.

INTRODUCTION

Driving of piles in normally consolidated clay increases the pore pressure in the soil. The dissipation of the resulting excess pore pressures starts immediately after the driving. This reconsolidation has been analyzed theoretically by e.g. Rees and Seed (1955), Soderberg (1962) and by Krize & Krugmann (1972). A comparison between calculated reconsolidation rates and results from field measurements indicates that for normally consolidated clays the theoretical solutions underestimate considerably the reconsolidation rate immediately after driving, see e.g. Cummings, Kerkhoff & Peck (1950), Reese and Seed (1955), Lambe and Horn (1965), Orrje & Broms (1967), Flaate (1972) and Massarsch (1976). A significant part of the reconsolidation (up to 50 percent of the excess pore pressure) can occur within a few days (Hagerty and Garlanger, 1972). The pore pressure reduction close to a pile increases also the bearing capacity of the pile and the shear strength of the surrounding soil (Seed and Reese 1955, Reese and Seed 1957, Koizumi and Ito 1967, Flaate 1968, Torstensson 1973).

EXPANSION OF A CYLINDRICAL CAVITY

The change of the stress conditions in the soil during the driving of a pile can be studied theoretically by investigating the expansion of a cylindrical cavity. The soil along the shaft of a pile will mainly be displaced radially away from the pile during the driving similar as for an expanding cylindrical cavity (Massarsch 1976). With cylindrical coordinates and the stress conditions shown in Fig. 1, the following parameters are defined, Table 1.

It is assumed in the following analysis that the expansion of the cavity takes place under initially isotropic stress conditions (K_0 = 1.0) in a homogeneous, isotropic, ideal elasto-plastic material.

Table 1. Definition of parameters

$\Delta\sigma_r$	Total radial stress change
$\Delta\sigma_t$	Total tangential stress change
p_u	Expansion pressure at the cavity wall
r	Radius of the expanding cavity
r_{pl}	Radius of the plastic zone around the cavity
ρ	Radial displacement

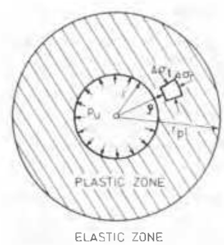

Fig. 1 Definition of parameters describing the expansion of a cavity

The yield strength of the material is assumed to be unaffected by the hydrostatic pressure. For the sake of simplicity, the undrained case is investigated first. Then the effect of pore pressure is taken into account.

The extension of the plastic zone around an expanding cylindrical cavity of infinite length under plain strain conditions has been analyzed by Vesic (1972).

$$\frac{r_{pl}}{r} = \left(\frac{E}{2\,\tau_f\,(1+\nu)}\right)^{1/2} \quad (1)$$

The radial displacement is thus a function of the soil rigidity ratio (E/τ_f) and the Poisson ratio (ν).

The lateral pressure distribution around a cavity (pile) can be estimated from the following equation given by Vesic (1972)

$$\frac{p_u}{\tau_f} = \ln\left(\frac{E}{2\,\tau_f\,(1+\nu)}\right) + 1 \quad (2)$$

This equation indicates that the expansion pressure is independent of the diameter of the cavity. The distribution of the total stresses within the plastic zone around a cylindrical cavity at a distance ρ can be expressed by the following equations (Vesic 1972)

$$\frac{\Delta\sigma_r}{\tau_f} = 2\,\ln\left(\frac{r_{pl}}{\rho}\right) + 1 \quad (3)$$

$$\frac{\Delta\sigma_t}{\tau_f} = 2\,\ln\left(\frac{r_{pl}}{\rho}\right) - 1 \quad (4)$$

$$\frac{\Delta\sigma_z}{\tau_f} = 2\,\ln\left(\frac{r_{pl}}{\rho}\right) \quad (5)$$

If the pore water pressures within the plastic zone for a cavity expanding under undrained conditions can be taken into account then the distribution of the effective stresses can be calculated. The excess pore water pressure Δu can be expressed in terms of the mean normal stress change

$$\Delta\sigma = 1/3\,(\Delta\sigma_z + \Delta\sigma_r + \Delta\sigma_t) \quad (6)$$

and the mean shear stress change

$$\Delta\tau = 1/3\left[(\Delta\sigma_r-\Delta\sigma_z)^2+(\Delta\sigma_z-\Delta\sigma_t)^2+(\Delta\sigma_t-\Delta\sigma_r)^2\right]^{1/2} \quad (7)$$

since

$$\Delta u = \beta\,\Delta\,\sigma + \alpha\,\Delta\,\tau \quad (8)$$

where α and β are pore pressure parameters (Henkel 1960). The excess pore pressure at any point in the plastic zone around the expanding cavity can be obtained from Eqs (3) through (8) (Torstensson 1973). The pore pressure parameter $\beta = 1$ for saturated clay. The Henkel pore pressure parameter α_f at failure is related to the Skempton parameter A_f by the relationship

$$\alpha_f = 0.707\,(3\,A_f - 1) \quad (9)$$

The effective stress can then be computed from the following relationships (Massarsch 1976)

$$\frac{\Delta\sigma'_r}{\tau_f} = 1 - 0.577\,(3\,A_f - 1) \quad (10)$$

$$\frac{\Delta\sigma'_t}{\tau_f} = -\left[1 + 0.577\,(3\,A_f - 1)\right] \quad (11)$$

$$\frac{\Delta\sigma'_z}{\tau_f} = -\,0.577\,(3\,A_f - 1) \quad (12)$$

The effect of the pore pressure coefficient A_f on the distribution of the effective stresses in the plastic zone around a cylindrical cavity is shown in Fig. 2. It is interesting to note that the effective stress theoretically is independent of the location within the plastic zone.

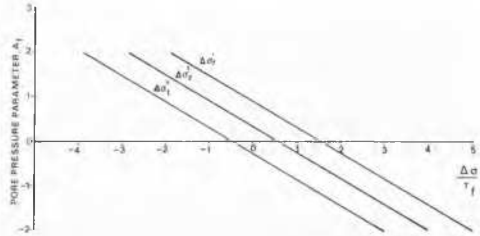

Fig. 2 Variation of effective stresses with respect to pore pressure parameter A_f

It can be seen that the tangential effective stress is decreased and can be negative. Consequently the tensile strength of the soil can be exceeded. The tangential effective stress is equal to zero when

$$K_0\,\sigma'_v + \Delta\sigma'_t = 0 \quad (13)$$

The initial effective stress before pile driving is equal to

$$\sigma'_h = K_0\,\sigma'_v = \sigma'_r = \sigma'_t \quad (14)$$

By inserting Eq. (11) into Eq. (13) the stress conditions when the tangential stress is equal to zero and vertical cracks develop in the soil can be calculated. When the tensile strength of the soil is equal to zero then

$$\frac{K_0\,\sigma'_v}{\tau_f} = 1.73\,A_f + 0.42 \quad (15)$$

This relationship is shown in Fig. 3.

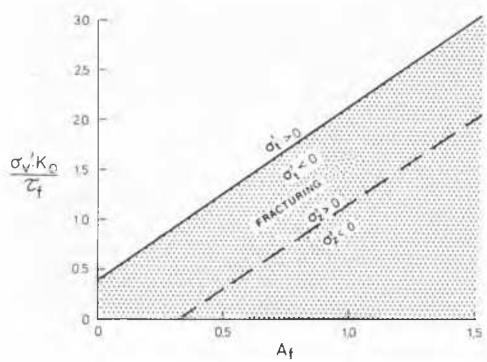

Fig. 3 Idealized stress conditions at which fracturing of the clay can occur

A similar critical value σ'_z can be determined for the case when horizontal cracks develop

$$\frac{K_o\,\sigma'_v}{\tau_f} < 1.73\,A_f - 0.577 \qquad (16)$$

The solutions given above are applicable on single piles and on soils with ideal properties and without tensile strength. Therefore only a range of values can be obtained from the solution given above. Also the direction of fractures can vary and will not necessarily be vertical or horizontal. Fracturing of the soil can thus occur when the pore pressure parameter A_f is about 0.85 as is the case for most soft normally consolidated clays.

Hydraulic fracturing from excess water pressures (Bjerrum & Andersen, 1972) is similar to the soil fracturing described herein. Bjerrum and Andersen have presented comprehensive data from field and laboratory tests which illustrate the development of tensile cracks in soils.

FIELD TESTS

The increase of pore pressures during pile driving in soft clay has been studied at two sites as well as hydraulic fracturing. The results have been reported by Massarsch (1976). The soils at both sites consisted of normally consolidated clay with an undrained shear strength of 10 to 20 kPa, and a sensitivity of 10 to 40. Pore pressures were measured by vibrating wire piezometers during the driving of timber piles with an average diameter of 22 cm.

Tests with hydraulic fracturing were also carried out at the two sites. The excess pore pressures which were measured during the falling head permeability tests are shown in Fig. 4.

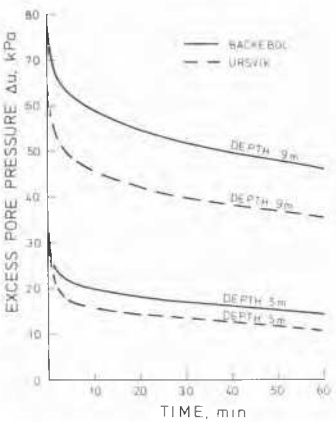

Fig. 4 Dissipation of applied pore water pressure from hydraulic fracturing tests

It can be seen that the pore water pressure decreased rapidly for the first five minutes during the hydraulic fracturing. Then the flow rate suddenly decreased. This effect is similar to the reduction of the excess pore water pressures observed after the driving of the piles. In Fig. 5 the variation of excess pore pressure at test site Ursvik is shown.

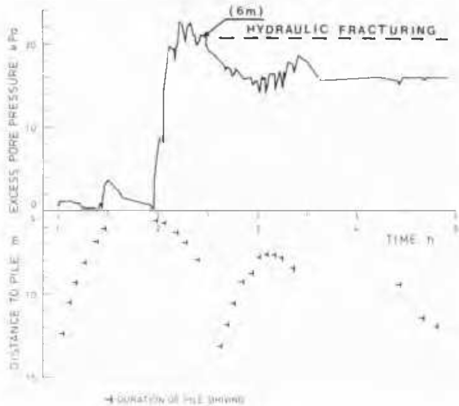

Fig. 5 Variation of excess pore water pressure during pile driving, test site Ursvik

The dissipation of excess pore pressure after the driving of two piles at test site Bäckebol is shown in Fig. 6

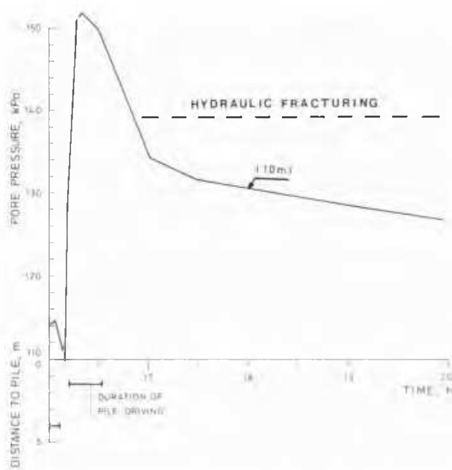

Fig. 6 Variation of pore water pressure during pile driving, test site Bäckebol

The decrease of the pore water pressure during the hydraulic fracturing tests was similar. A sudden reduction in pore water flow occurred. The "close-up" pressure from the hydraulic fracturing tests corresponded to the total stress in the direction perpendicular to the crack, (Bjerrum and Andersen 1972). The in situ stresses are changed by the driving as well as the close-up pressure. The close-up pressure for a pile may thus be different from that determined by hydraulic fracturing. The results from the hydraulic fracturing tests are also indicated in Fig. 5 and 6. It can be seen that the close-up pressures for the hydraulic fracturing tests are slightly higher than those at the pile driving.

CONCLUSIONS

The theoretical analysis in this article suggests that cracks can develop in normally consolidated clays during pile driving. The open cracks in the plastic zone around the pile increase temporarily the permeability of the soil and cause a rapid dissipation of the excess pore water pressures. Field tests indicate that the mechanism of soil fracturing from pile driving is similar to that at hydraulic fracturing during falling head permeability tests in the field.

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