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PROGRESSIVE FAILURE ANALYSIS OF WINDROW INDUCED SLOPE MOVEMENTS AT THE SYNCRUDE LIMITED MINE SITE

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ABSTRACT

The presence of Marine Sediments below the operating bench in the Syncrude Canada Limited oil sand mine (NE Alberta, Canada) has negatively affected dragline mining operation. Any material that exhibits strain softening behaviour is prone to progressive failure. The Marine Clays at the Syncrude site exhibit distinct strain softening behaviour and numerous windrow-induced failures have occurred along these layers. Some of these failures have resulted in highwall failures but no highwall failures have been reported in the absence of windrow or dragline loads. Therefore, the possibility of progressive failure of an idealized highwall due to external soil loading (windrows) was of interest to study. Elastoplastic finite element analyses were carried out to investigate the influence of several variables (oilsand stiffness, shear strength parameters of marine clay, pore water pressure coefficients, depth of embedment of clay layer, insitu stress field, windrow construction mode etc.) on highwall stability, and enable us to develop a better understanding of the progressive failure mechanism. The results of the numerical analysis revealed that windrow construction results in undrained loading of the clay layer and can induce progressive failure along the brittle marine clay layers. The progressive failure process is amenable to analysis by the finite element method. The analysis indicated the importance of identifying the location and the strength properties of any weak inclusion in an otherwise strong material.

KEYWORDS: Oilsands, Progressive Failure Analysis, Slope Stability, Finite Element Analysis, Syncrude Mine.

INTRODUCTION

Syncrude Canada Limited operates one of the largest open pit oil sand mining operations in the world. The ore is initially exposed by a truck and shovel operation, which removes the overburden and then four 70 m^3 draglines excavate the oil sand from below their operating bench, placing it in windrows for future handling by bucket wheel reclaimers. The ore is then conveyed to the plant site where it yields synthetic crude oil (McKenna and List, 1990). The presence of Marine Sediments below the operating bench has negatively affected the dragline operation due to limited short-term production restrictions and potential development of highwall failures which may incorporate the dragline [Morgenstern et al (1988)]. List and McKenna (1990) present an overview of the behaviour of the Marine Sediments, while McKenna and List (1990) report the impact of the Marine Sediments on Syncrude draglines. The geology of the Athabasca Oil Sands has been thoroughly discussed by Mossop (1980).

GEOTECHNICAL PARAMETERS OF MARINE SEDIMENTS

The Marine Sediments are divided into two geotechnical subunits, Marine Sands and Marine Clays. The Marine Sands are composed of dense to very dense, brown, angular subhorizontally bedded silty sands and generally have low bitumen contents. The Marine Clays are dark gray to black, soft to stiff (typically firm), highly bioturbated silty clays of medium to high plasticity. Marine Clay layers greater than 1 cm thick are generally continuous over tens to hundreds of meters, are flat lying and show fissility along bedding planes.

The residual shear strength curve for the Marine Clays shows an apparent cohesion which suggests a bilinear strength envelope (Cameron and Carr, 1988). Skempton (1985) suggests that for design purposes it is often useful to take a "best-fit" linear envelope over the range of normal effective pressures involved, using both the effective cohesion and the effective angle of shearing resistance. This approach is used by Syncrude for slope stability analyses involving Marine Clays and is supported by back-analysis of numerous failures. Effective strength parameters of $\phi'^{r}= 13°$ and $c'^{r}=0$ kPa is typical for many design applications, given that the typical average effective vertical stress is about 140 kPa for most designs. Although Marine Clays are easily polished by hand when inspecting core, pre-sheared failure planes are extremely difficult to detect in the field although evidence of disturbance (upthrust blocks of clay and sand, overhang of Marine Sediments on the highwall and shear planes detected by slope inclinometers) is often seen. Back-analysis of numerous failures including loading and unloading cases indicate that at the time of failure, the Marine Clays are at or close to residual strength (McKenna and List, 1990). Due to the uncertainty of detecting pre-sheared planes in core, designs at the Syncrude Mine site are performed using residual strengths for all significant Marine Clay layers (i.e greater than 1 cm thick) even if presheared planes are not observed in the core.

STATEMENT OF THE PROBLEM

Since Marine Clays exhibit strain weakening behaviour and numerous windrow induced failures have occurred along these layers, the possibility of progressive failure of an idealized highwall due to external soil loading was of interest to study. The assumed slope stratigraphy is composed of 40 m of oilsand with a clay inclusion 0.25 m thick at 4.75 m below the top of the slope and a slope angle of 50°. The windrow has a base width of 100 m, 30 m height and its center line is located 65 m away from the edge of the slope. The ground water table is 3 m below the bench level. The magnitude of the vertical pressures applied due to the windrow loading, in combination with the geological conditions indicate that a progressive failure analysis should be carried out. The assumed slope stratigraphy is similar to conditions encountered at the Syncrude mining site, where 40-50 m high oil sand slopes with interbedded continuous and/or discontinuous clay layers are overlying the bedrock. Because of its strength (Dusseault and Morgenstern, 1978), the oilsand is unlikely to fail under the windrow loading. On the other hand, excess pore water pressures are measured in the Marine clay layers during the windrow construction, which however dissipate quickly. The fast dissipation rate of the excess pore water pressures supports the observation that failures associated with windrow construction occur within a few days of the waste piles placement. Therefore, the undrained behaviour of the Marine clays during the windrow construction is likely to be a critical design condition.

OILSAND

Agar (1984), reported that curve fitting of triaxial compression experimental data indicated that the hyperbolic model [Duncan and Chang (1970)] is a useful empirical technique for modeling the nonlinear stress-strain behaviour of Athabasca oilsands up to 85% of the peak deviatoric stress level. The following parameters were used to model the drained behaviour of the oilsand deposit:

$K=25000$, $K_{ur}=30000$, $n=0.9$, $G=0.2$, $R_f=0.55$, $P_{atm}=101.4$.

MARINE CLAY

List and McKenna (1990) report typical strength parameters of the Marine Clay layers, as obtained from direct shear tests. In this study, it was assumed that the clay layer behaves in a linearly elastic manner up to the peak strength and then displays brittle perfectly plastic behaviour in the post peak range of deformation (strain weakening). The Drucker-Prager failure criterion with an associated flow rule was used to model both the drained and undrained behaviour of the clay layer. Since the difference between the peak and residual friction angles is not that high, the use of an associated flow rule will not result in an erroneous overprediction of the dilative behaviour of the clay layer.(Mathioudakis, 1992). The following model parameters were initially used to describe the material behaviour during the loading history (List and McKenna, 1990):

E=50000 kPa, ϕ^p=17.5°, c^p=50 kPa, ϕ^r=11.0°, c^r=18 kPa, A_p=-0.1 and B_p=0.98

BEDROCK

It was assumed that the bedrock was encountered at the bottom of the open pit, as is usually the case in most of the areas on the Syncrude site. Since no failure was expected on the pit floor, it was assumed that the bedrock behaved in a linearly elastic manner. A Poisson's ratio v=0.3 and a value of E=2000000 kPa for the elastic modulus were used for the Devonian limestone as indicated by Dusseault (1977).

WINDROW

The windrow material is oil sand excavated from below the operating bench and is therefore expected to behave as a loose granular material. Since the purpose of this study is not to model failures that are located within the windrow, but rather to study the response of the Marine Clay layer due to the windrow construction loading, it was assumed that this material behaves in a linearly elastic manner. A Poisson's ratio v=0.33 and a value of E=20000 kPa for the elastic modulus were used.

IN-SITU STRESS PROFILE

As a result of the complex geological history of the region (Mossop, 1980) with repeated glaciation and subsequent scouring and erosion events, the McMurray Formation consists of heavily overconsolidated sediments and consequently a high horizontal in-situ stress field is locked in. This is confirmed by the results of various field and analytical studies conducted in the past (Morgenstern et al, 1988). Therefore, a high value of the coefficient of earth pressures at rest, K_o, ($1.5 < K_o < 2.0$) was adopted for this study (Mathioudakis, 1992).

FINITE ELEMENT ANALYSIS

Elastoplastic finite element analyses were carried out to investigate the influence of several variables on the stability of the highwall and to develop a better understanding of the progressive failure mechanism. The analyzed domain is 350 m deep and 1500 m long in order to minimize any boundary effects on the solution (Mathioudakis et al, 1992). The upper 40 m is the idealized stratigraphy as described earlier, while bedrock is encountered from elevation +310 m and below.

Finite element meshes combining 8- and 6- noded isoparametric two dimensional elements were employed in the analyses and to investigate the influence of several input parameters on the extent of the yielding process. A 3*3 Gauss integration scheme was used with the modified Newton-Raphson solution algorithm for equilibrium iteration. A "closest point integration algorithm" (Borja et al, 1989) was employed for the stress integration, which preserves the asymptotic rate of convergence of the Newton-Raphson method (Simo and Taylor, 1985). The excavation algorithm scheme as proposed by Ghaboussi and Pecknold (1984) was employed to simulate the open pit excavation as well as the windrow construction. Although some site specific cases may require a three dimensional simulation, plain strain conditions were considered for purposes of illustration. An in-situ stress field with K_o higher than 1.0 was established in two steps by invoking the principle of superposition, which is justified since no yielding occurs at the end of the first step. (For more details on this phase of the analysis, see Mathioudakis, 1992.)

In the second phase of the analysis (steps 3 to 7), the real material properties were assigned to the respective soil layers and the excavation of the open pit was simulated. Since there is no significant pore water pressure response due to the slope excavation (List and McKenna, 1990), all soil layers can be considered to behave in a drained manner during this stage of the simulation. The hyperbolic and the Drucker-Prager models with an associated flow rule were used to simulate the behaviour of the oil sand and the Marine Clay layer respectively. Linear elastic behaviour was assumed for the bedrock.

In the third phase of the analysis (steps 10 to 22), the stage construction process of the windrow was simulated and its effect on the extent of yielding and the pore pressure response of the clay layer was closely monitored. Again, the hyperbolic and the Drucker-Prager models were used to simulate the behaviour of the oil sand and the Marine Clay layer. Since during windrow construction no significant excess pore water pressures are recorded in the oil sand deposit, it was assumed that the oil sand behaves in a drained manner during this stage of the simulation. However, significant windrow induced excess pore water pressures are recorded along the Marine Clay layers (List and McKenna, 1990). These construction induced excess pore water pressures, show a rapid rate of dissipation which can be attributed to sand burrows and lenses within and/or adjacent to the Marine Clay layer and/or pore water movement along pre-existing shear planes, but may have also been affected by the neighbouring piezometer installation trench. The length of the drainage path through these burrows and lenses is unknown but is likely critical to the ability of the clay layer to transport moisture to and from failure planes (List and McKenna, 1990). Values of A_p=-0.1 and B_p=0.98 were initially thought to be appropriate for Skempton's pore water pressure coefficients for plane strain, considering the nature of the clay (stiff, heavily overconsolidated) and that it lies below the groundwater table (saturated), (Skempton, 1964). It was assumed that the bedrock and the windrow material behave in a linear elastic manner.

DISCUSSION OF THE RESULTS

IN-SITU STRESS FIELD GENERATION

By switching on gravity and generating the desirable in-situ stress field, all nodes move vertically downwards, with all nodes at the same elevation settling by the same amount. As a result, no shear stresses are generated during this step and all vertical force components on each Gaussian point are compressive and of equal magnitude for points lying on the same elevation.

OPEN PIT EXCAVATION

This stage of the analysis was carried out in eight loading steps. In each step a row of elements representing the open-pit floor was removed from the mesh to create the new pit floor. During each unloading step, all nodes lying on the new floor pit heaved as expected. Nodes lying on the upper half face of the high wall settled down and in to the pit, while nodes lying on the lower half were moving up, creating a compressive stress state in the vicinity of the highwall face. Insignificant tensile stresses were developed locally on the operating bench as a result of the unloading process and they caused no tensile yielding. No yielding along the clay layer occurred during this stage of the analysis, due mainly to the high shear strength of the clay layer and the stiffness of the surrounding oilsand.

WINDROW CONSTRUCTION STEP

The stage-construction of the windrow was simulated in 12 loading steps and its influence on the extent of yielding along the clay layer was assessed. By activating elements representing layers of the windrow material, the incremental displacements of nodes lying along the clay layer and symmetrical around the windrow center line are of equal mangitude but of opposite direction as expected. This results in a net increase of the horizontal displacements of the nodes lying between the windrow center line and the face of the highwall, while the opposite holds true for the nodes lying beyond the windrow center line. Also, the generated excess pore water pressures are increasing after every loading increment, which results in an effective stress state along the clay layer closer to yielding. At the end of this stage of the simulation, minimal yielding was observed along the clay layer while the calculated construction induced pore pressures were much higher (by 3 times) than those measured in the field. This led us to believe that one or a combination of the following was occuring:

(1) The assumed shear strength parameters of the clay layer were overestimated.
(2) The assumed stiffness of the surrounding oilsand material was overestimated.
(3) The clay layer was not fully saturated.

A series of analyses were carried out, where the stiffness of the oilsand, the shear strength of the clay layer and Skempton's pore water pressures parameters (A_p, B_p) were reduced until the numerically calculated pore water pressures matched those measured in the field and reported by List and McKenna (1990). These analyses revealed the following:

(1) As long as the clay layer possesses a value of peak cohesion c^P=50 kPa and a value of peak friction angle ϕ^P=17.5°, even a 30 m windrow height will cause no yielding of the clay layer for a value of K_o= 1.5.

(2) A combination of 25% reduction of the oilsand stiffness and an 80% reduction in the value of peak cohesion of the clay layer will cause some yielding along the clay layer (K_o=1.5).

(3) In order to match the measured field pore water pressures due to the windrow loading, a value of B_p=0.65 has to be assumed, while A_p can vary between 0.0 and -0.25.

The above conclusions led us to revise the initially assumed "appropriate" values of the stiffness, strength and pore water pressure parameters of the various soil layers to:

Oilsand K=18500, K_{ur}=22500, n=0.9, G=0.2, R_f=0.55, P_{atm} =101.4.

Clay E=50000 kPa, ϕ^P=17.5°, c^P=5 kPa, ϕ^r=11.0°, c^r=5 kPa, A_p=-0.1, B_p=0.65.

Note that the initially assumed strength and stiffness parameters for the limestone remained unchanged. The revised set of parameters resulted in a 140 m long yielding zone along the clay layer and a pore pressure response within 3% of the one observed in the field. The revised set of parameters was also considered to be the "benchmark set" for the sensitivity analysis that was carried out.

SENSITIVITY ANALYSIS

The influence of the following variables on the extent of yielding and the measured construction induced excess pore water pressures have been investigated:

1. Stiffness of the surrounding oilsand material.
2. Shear strength parameters of the clay layer.
3. Pore water pressure parameters.
4. Depth of embedment of the clay layer.
5. Insitu stress field.
6. Construction process of the windrow.

STIFFNESS OF THE SURROUNDING OILSAND MATERIAL

The influence of the stiffness of the surrounding oilsand material on the extent of the yielding along the clay layer was investigated. Three series of analyses were carried out. The first one was the "benchmark", while in the second one the stiffness of the oilsand was increased by 33% and that resulted in a reduction of the yielding zone by almost 50%, when the windrow reached full height (30 m). In the third set, the stiffness of the oilsand was decreased by 33% and resulted in the yielding of the entire clay layer (~ 1000 m) and subsequent failure when the windrow reached the height of 22 m, i.e. 8 m before full height. The results of the analysis are illustrated in Fig. 1 and confirm the findings reported by Chan and Morgenstern (1987), that the stiffness of the surrounding materials can have a significant effect on the yielding behaviour of weak layers.

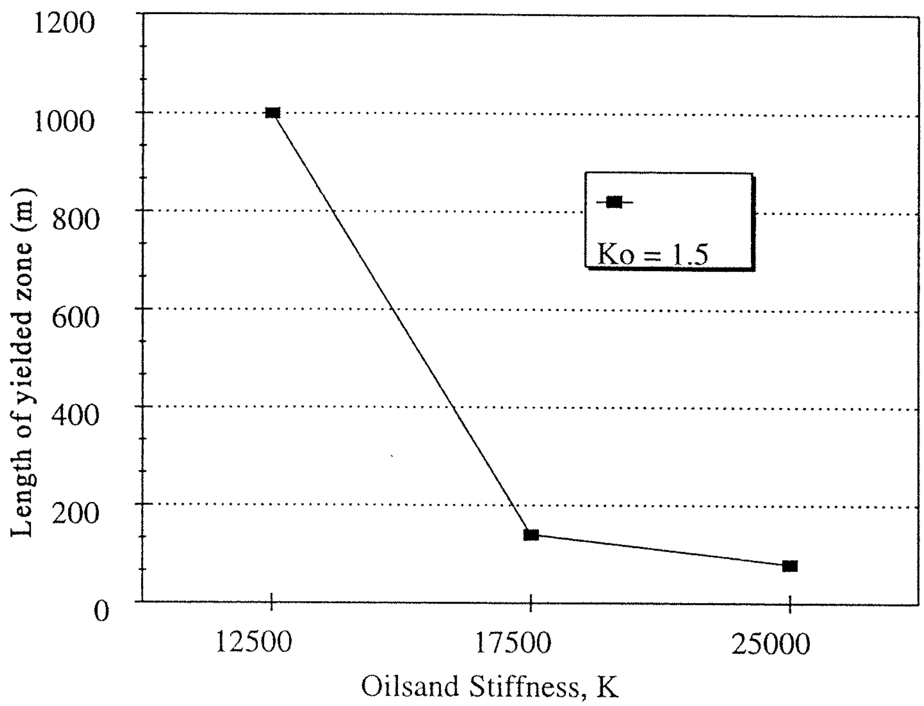

Fig. 1 Influence of oilsand stiffness on the extend of yielding along the clay layer

STRENGTH PARAMETERS OF THE CLAY LAYER

The influence of the shear strength of the clay layer on the extent of yielding was investigated. By varying the value of the peak cohesion, c^P, from 5 to 12.5 and 25 kPa, the length of the yielding zone was reduced from 140 m, to 6 m and 4 m respectively after the application of the full windrow height. Hence a small decrease to the value of the peak cohesion, from 12.5 to 5 kPa, results in a dramatic rate of increase in the total length of the yielding zone along the clay layer. The rate of extent of yielding is even more dramatic for higher values of K_o. Therefore, if for any reason softening of an already weakened clay layer will occur, it can cause extensive yielding, which in turn can lead to the development of a shallow block slide failure mechanism along the clay layer with serious consequences for the dragline safety. The results of the analysis are illustrated in Fig. 2. Note, that even the value of 25 kPa is 50% less than the value of the peak cohesion reported by List and McKenna (1990) for the clay layer. This led us to believe that the clay layers have experienced a considerable degree of softening prior to the windrow construction, due to a mechanism not necessarily related to the excavation of the pit. List and McKenna (1990) list some possible mechanisms that can cause softening of the clays. The extent of the yielding appeared to be insensitive to the value of the Poisson's ratio of the clay layer.

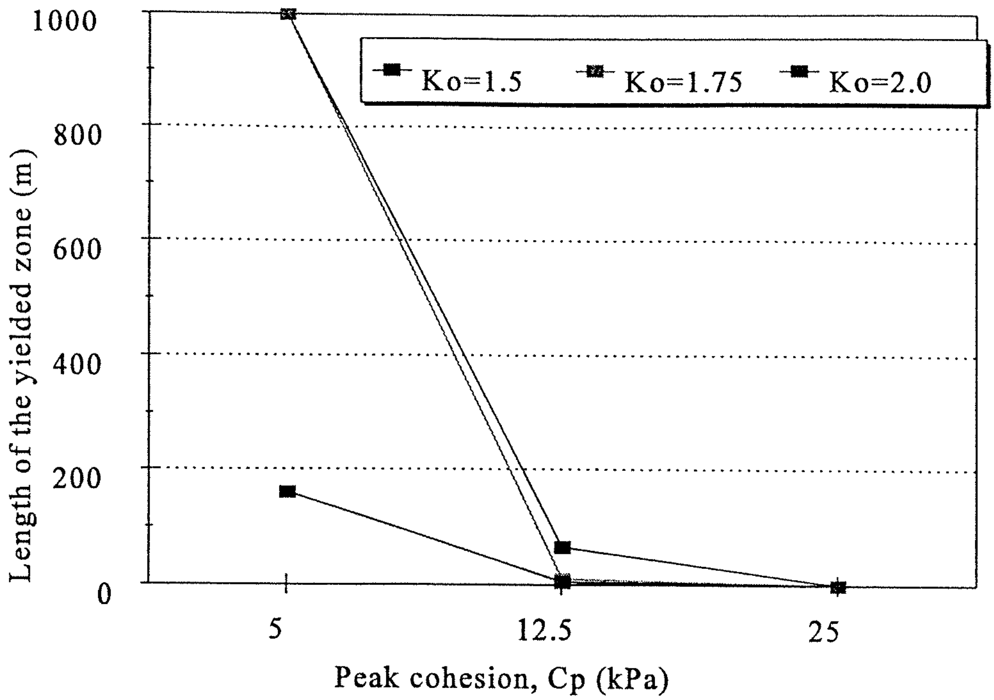

Fig. 2 Influence of shear strength parameters of clay layer on the extend of yielding along the clay layer

PORE WATER PRESSURE PARAMETERS

It was initially thought that the most appropriate values for Skempton's pore pressure parameters for plane strain conditions along the clay layer were A_p=-0.2 and B_p=0.98. These parameters failed to predict the measured pore water pressure response due to the windrow loading. In this section of the sensitivity analysis, the relative influence of A_p and B_p on the calculated pore water pressure response of the clay layer was undertaken.

By definition $\Delta u = B_p[\Delta\sigma_3 + A_p(\Delta\sigma_1 - \Delta\sigma_3)]$ (Skempton, 1954)
where Δu = increment of pore water pressure
$\Delta\sigma_1$, $\Delta\sigma_3$ = changes of principal stresses σ_1 and σ_3 respectively
A_p, B_p = Skempton's pore pressure coefficients for plane strain

Evaluating the changes of principal stresses σ_1 and σ_3, i.e $\Delta\sigma_1$ and $\Delta\sigma_3$ along the clay layer for any two consecutive loading steps, one can see that the term $(\Delta\sigma_1 - \Delta\sigma_3)$ is very small. It can then be concluded that regardless of the value of A_p ($-0.25 < A_p < 0.0$, since the clay is heavily overconsolidated), the dominant parameter in the accurate prediction of the pore pressure is B_p. This is confirmed by the sensitivity analysis, where by varying the value of A_p in the above range, insignificant changes occur in the magnitude of the calculated pore water pressures. By varying the value of B_p from 0.98, to 0.7 and to 0.65, the excess pore water pressure is being reduced by 2 and 3 times respectively. The last is in excellent agreement with the excess pore water pressure response measured in the field.

One may argue that a clay layer that lies underneath the water table cannot have an unsaturated response, i.e the value of B_p ought to be close to 1.0. The Marine Sediments respond to undrained loading very similar to partly saturated soils due to the presence of gas, which results in considerably greater compressibility of the fluids. This response can also be attributed to sand burrows and lenses within and/or adjacent to the Marine Clay and/or pore water movement along pre-existing shear planes, but may have also been affected by the neighbouring piezometer installation trench (List and McKenna, 1990).

DEPTH OF EMBEDMENT OF THE CLAY LAYER

The influence of the depth of embedment of the clay layer on the extent of yielding was investigated. Three analyses were carried out, varying the depth of embedment from 4.75, to 9.75 and 19.75 m. The results show a decreasing length of the yielded zone with increasing depth of embedment. This can be explained by the fact that the deeper the clay layer is located, the lesser is the influence of the windrow load on the clay layer, which results in a lesser length of the yielded zone along the clay layer. Consequently, the potential for progressive failure ef embedment on This iillustrated from the results of the finite element analysis in Fig. 3, where the length of the yielded zone along the clay layer versus its depth of embedment is plotted.

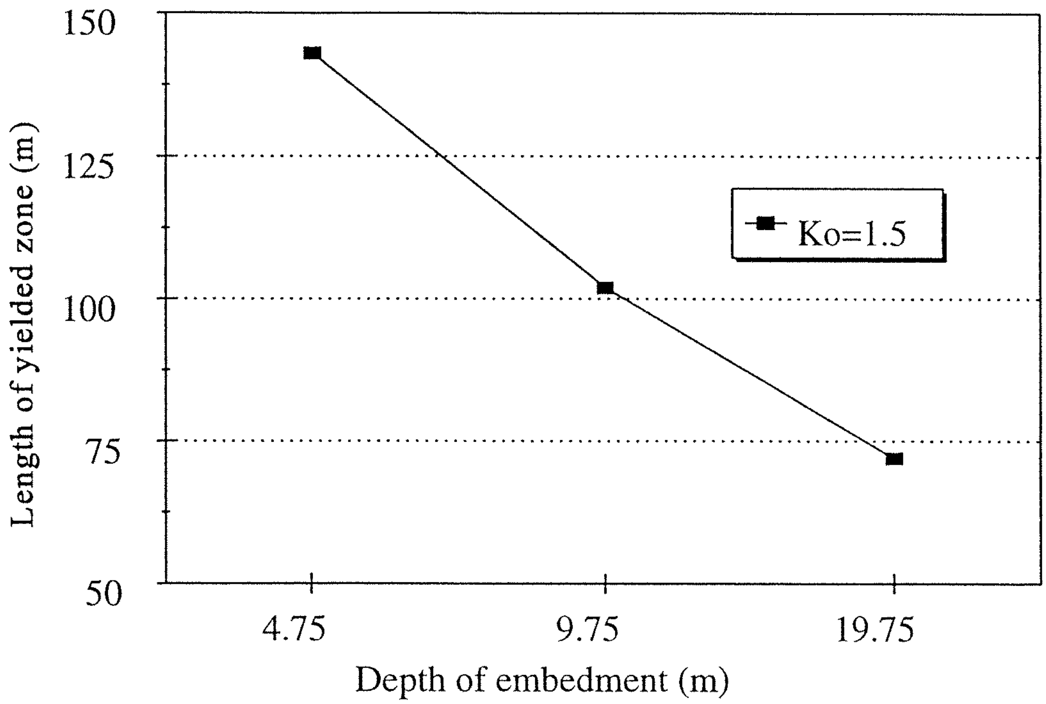

Fig. 3 Influence of depth of embedment on the extend of yielding along the clay layer

IN-SITU STRESS FIELD

Bjerrum (1967) recognized the importance of the magnitude of the in-situ stress field on the extent and the rate of propagation of progressive failure in overconsolidated clays. To assess quantitatively the influence of K_o on the extent of yielding along the clay layer due to the windrow loading, three analyses were carried out. The value of K_o was varied from 1.5 ("benchmark analysis"), to 1.75 and 2.0. By increasing the value of K_o from 1.5 to 1.75, the length of the yielding zone along the clay layer increased from 140 m to 900 m at the end of the windrow construction. I.e. increasing the insitu stress field by 15%, results in an increase of the yielding zone by more than 600%! Increasing the value of K_o to 2.0, results in the spread of yielding along the entire length of the clay layer at the end of the windrow construction. For higher values of the peak cohesion c^P, the extent of yielding is much less. The results of the analysis, Fig. 4, show how the softening of the clay layer in combination with the high in-situ stress field can result in an uncontrolled rate of the propagation of yielding.

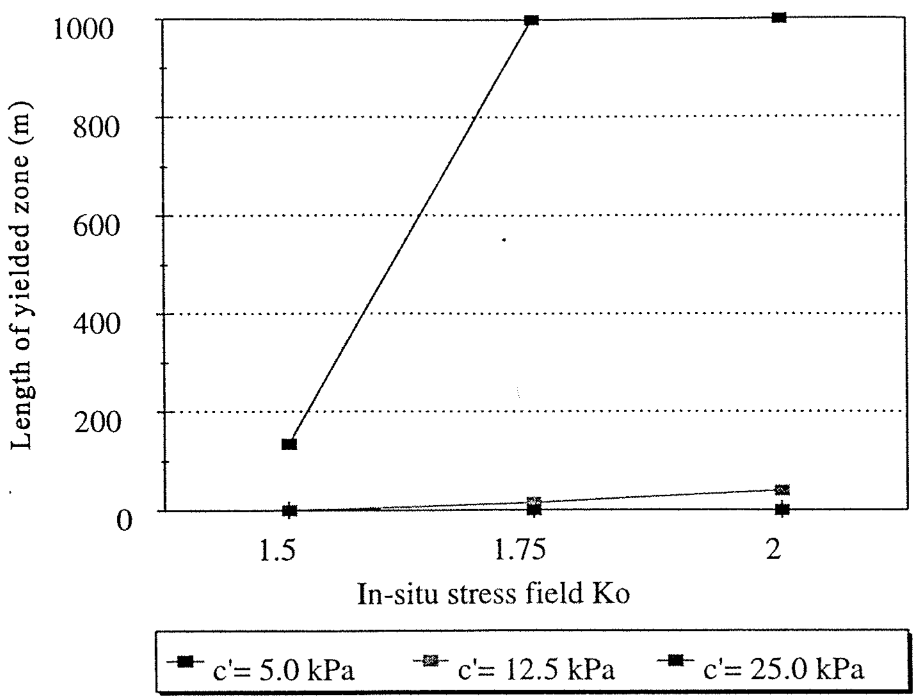

Fig. 4 Influence of in-situ stress field K_o on the extend of yielding along the clay layer

CONSTRUCTION METHOD

The influence of the construction procedure of the windrow on the extent of yielding and the generated pore water pressures was investigated. Two different construction methods were adopted in this sensitivity analysis, which will be called "center line method" and "double pyramid method", and which from now on will be referred to as "CLM" and "DPM" respectively. In CLM, the construction proceeds by building up and sideways from the centerline of the windrow until the desirable height is achieved. In DPM, the construction starts by building two twin pyramids on both sides of the windrow center line and proceeds by filling up the gap, until again the desirable height of the windrow is reached. The horizontal weak layer was assumed to be 4.75 m below the working bench. Although the number of elements employed in the simulation of the two construction modes was not the same, an equal number of steps was employed in each simulation. Since it was thought that the weight of the windrow material per construction step would dominate the extent and the propagation of yielding, a considerable effort was made to keep this variable almost equal per corresponding step of the two analyses. Although it was expected that the construction procedure would have an effect on the extent and the propagation of yielding along the clay layer, the analysis showed that for all practical purposes the total length of the yielding zone along the clay layer is the same for the two modes of construction investigated in the sensitivity analysis.

CONCLUSIONS

From the results of the analysis, the following conclusions can be drawn:

1. The windrow construction is an undrained loading for the clay layer and can induce progressive failure along the brittle marine clay layers.
2. This process is amenable to analysis by the finite element method.
3. The extent of progressive failure is primarily a function of the shear strength of the clay layer and the insitu stress field. Combinations of softened shear strengths of the clay layer and high insitu stress fields can lead to dramatic rates of propagation of yielding.
4. The extent of progressive failure is also a function of the depth of embedment of the clay layer and the stiffness of the surrounding oilsand material.

5. The progressive failure process is not much influenced by the windrow construction method for the two modes examined in this study.
6. The construction induced pore pressures along the clay layer can be predicted accurately using the finite element method. However, an accurate knowledge of the insitu value of the B_p parameter, rather than both A_p and B_p, is crucial for any reliable prediction.

Once more, the analysis carried out indicated the importance of identifying the location and the strength properties of any weak inclusion in an otherwise strong material. A great deal of effort should be spent during the site investigation phase in order to locate materials that can lead to a progressive mode of failure.

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