Failure of an embankment treated with vacuum preloading method

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ABSTRACT: During the construction of an embankment in one of the expressway in Malaysia employing the vacuum preloading method with prefabricated vertical drains, one of the embankment failed with large cracks and settlement in the final stage of filling to a fill height of 5.5m. The embankment was constructed on a very soft silty Clay of 4.5m thick and follows by a layer of soft sandy Clay to a depth of about 12m. The embankment was constructed in five stages and suction of 40kPa was applied through the vacuum system. This paper presents the findings by the Authors who have been engaged to investigate the failure.

1 INTRODUCTION

The vacuum preloading method implies the use of an airtight membrane placed over the ground to be improved and sealed to the low-permeable soil along the edges. Suction tubes are placed through the sealed membrane and connected to vacuum pumps. In order to ensure the uniform distribution of the suction pressure, a sand layer is placed on the ground before laying of fill. The suction (negative pressure) generated by the vacuum system causes the water in the pores of the soil to move towards the surface because of the hydraulic gradient set up. The flow of water in the subsoil is improved with the use of vertical drains. The effectiveness of the method is dependent on many factors like the pump capacity, the airtight seal between the edge of the geomembrane and the subsoil and integrity of the geomembrane at the ground surface and effectiveness of the vertical drains, etc. This method requires close monitoring of the pore water pressures in the subsoil during filling to prevent failure. Therefore, observational method (Peck, 1969) is usually employed as an effective way of construction control.

Two highway embankments namely Embankment A and Embankment B treated with vacuum preloading method at the same site in Peninsular Malaysia were investigated by the Authors after Embankment A failed after reaching the final thickness of 5.5m but Embankment B, which is not far from Embankment A and employed the same ground treatment did not fail. Figure 1 shows the typical cross-section of the embankment. Other than measuring the settlements, piezometers were also installed in the subsoil beneath the embankment at three different depths of 3m, 6m and 8m respectively in the very soft cohesive soil. The measurements of these piezometers were taken during construction of the embankment and during resting period.

2 THE SITE

The embankments were constructed on very soft silty Clay of 4.5m thick and follows by a layer of soft sandy Clay to a depth of about 12m. Beneath these very soft to soft cohesive soils is a layer of loose clayey Sand follows by layers of medium to stiff silty Clay and sandy Clay. The subsoil profile is also shown in Figure 1. Figure 2 shows the undrained shear strength (s_u) profile of the subsoil obtained from the field vane tests together with the adopted design values. The sensitivity of the soft clay generally ranges from about 2 to about 10 and can be categorized as sensitive to extra sensitive clays according to definition of sensitivity by Skempton and Northey (1952).

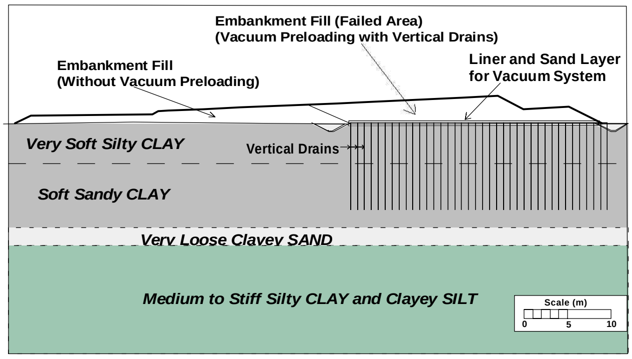

Figure 1. Cross-section of embankment treated with vacuum preloading method.

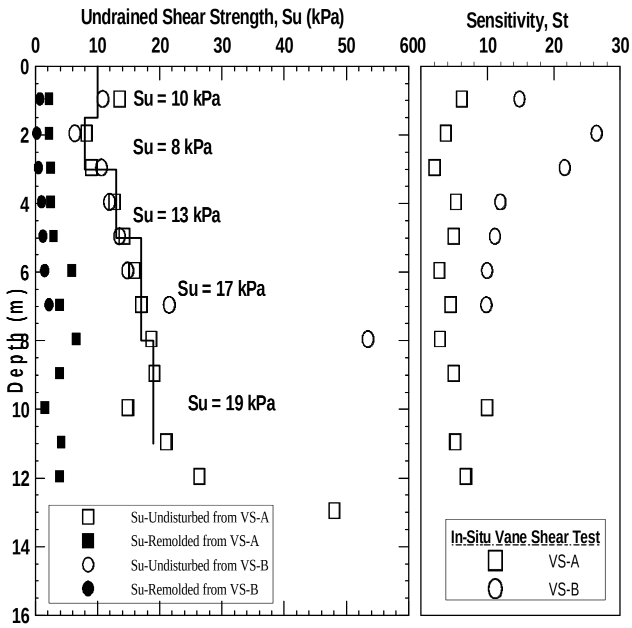

Figure 2. Undrained shear strength profile.

3 CONSTRUCTION AND MONITORING

Instruments like piezometers, settlement gauges and vacuum meters have been installed with the intention to monitor the performance of the subsoils treated using vacuum preloading as filling progresses. For this case history investigated by the Authors, only the results of the piezometers showing the response of pore water pressure will be discussed in this paper as the results of the settlement monitoring and vacuum meters do not show any trend to indicate signs of adverse condition.

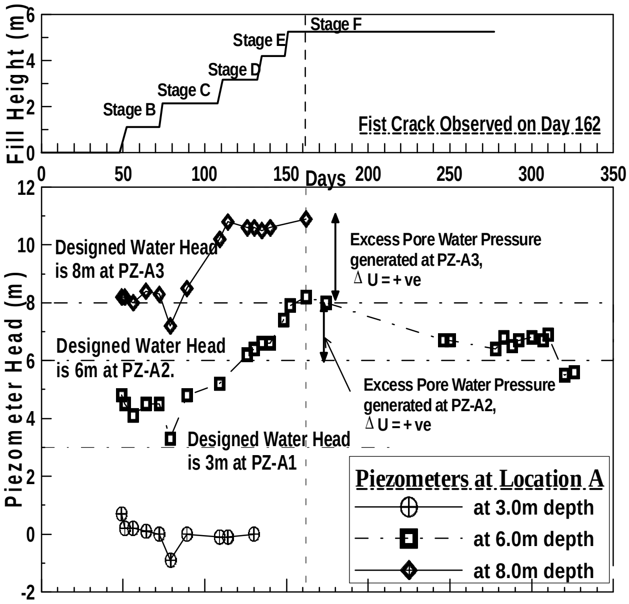

Figure 3. Construction sequence and monitored pore water pressure changes of failed Embankment A.

The construction sequence of Embankment A and changes of pore water pressure of the piezometers in the subsoil at depths 3m, 6m and 8m throughout the construction are shown in Figure 3. Embankment A failed not long after reaching the final fill height of 5.5m.

As shown in Figure 3, from Stage C filling onwards, the pore water pressure measured from piezometers PZ-A2 and PZ-A3 at depths of 6m and 8m respectively increased beyond the design pore pressure until failure at Day162 after reaching the final fill height. Piezometer PZ-A1 at 3m deep did not show increase in pore water pressure until it was out of order after Day 130. In brief, the measurement from piezometers PZ-A2 and PZ-A3 at Embankment A had indicated that the vacuum suction at these depths were not functioning properly to prevent increase in pore water pressures in the cohesive subsoil with respect to the filling.

The trend of increase in pore water pressures have been observed for more than one month but no contingency action was taken by the Contractor, who was also responsible for the design, to investigate the causes and to stop filling until the pore water pressure in the subsoil drops below the allowable design values.

Embankment B, which was not far away from Embankment A, also employed the same vacuum preloading ground treatment, was successfully constructed. Figure 4 shows the changes of pore pressures in the piezometers at different depths throughout the construction of Embankment B. The filling sequence is also presented in the same figure for easy reference. The pore pressures in all the piezometers installed were within the designed range indicating the vacuum suction performed as per design.

The observations from two embankments clearly show the importance of observational approach when employing vacuum preloading method for embankment construction. It also shows the effectiveness of the observational approach in identifying problems well before failure provided that the design engineer constantly reviews the monitoring results obtained from the site. In brief, the failure of Embankment A would have been prevented if engineers had observed the changes of pore water pressure in PZ-A2 and PZ-A3 and take the necessary action like stop further filling or remove of fill materials.

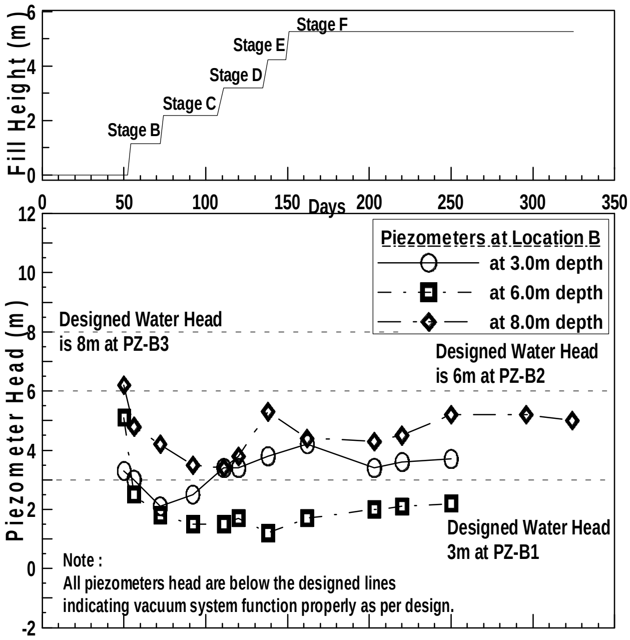

Figure 4. Construction sequence and monitored pore water pressure changes of successful Embankment B.

4 ANALYSIS

4.1 *Limit Equilibrium Stability Analyses*

Undrained Strength Method (Ladd, 1991) was used in the limit equilibrium slope stability analyses to check the adequacy of the embankment design of this case history with normalized strength increment ratio, $Äs_u / Äó'_v$, of 0.22 and with design suction of 40kPa. Stability analyses were carried out according to the construction sequence proposed by the contractor for construction. The adjacent subsoil outside the treatment area, excess pore water pressure will still be generated and has to based on the in-situ undrained shear strength (s_u). The stability analyses carried out indicate that the factor of safety against failure (FOS) is higher than 1.2 as required for temporary stage up to the final fill height of 5.5m indicating the design was acceptable if the vacuum system had performed as designed. This finding is in line with the success of Embankment B, which the vacuum system has performed properly.

4.2 *Finite Element Method*

Finite element method (FEM) utilizing computer program "PLAXIS", was also used to simulate the construction of the embankment employing vacuum preloading. Figure 5 shows the plane strain condition FEM model of the embankment utilising 6-node elements. Two stress-stain models have been used and they are the elastic-plastic Mohr-Coulomb model for the embankment fill materials and the Soft Soil model (Vermeer & Brinkgreve, 1998) for the soft cohesive subsoil. The Soft Soil model resembles the Modified Cam-Clay model and is an isotropic-hardening model.

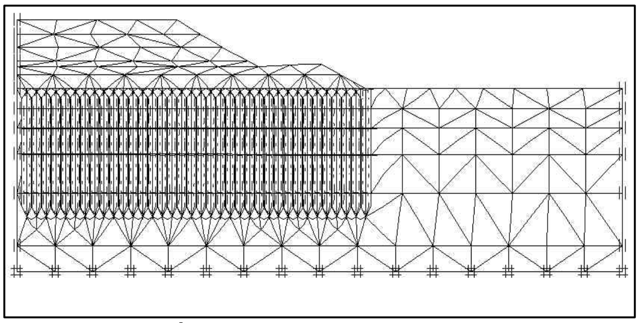

Figure 5. FEM Mesh.

The actual three dimensional axisymmetric behaviour of vertical drains of 10m deep installed at 0.8m triangular spacing at site were converted to an equivalent two dimensional plane strain behaviour using the method proposed by Bergado et al. (1994). The vertical drain system was transformed into continuous drainage wall with same spacing as that of the actual case using interface elements with permeability equivalent to 100 times the permeability of the subsoil. The converted horizontal permeability of the subsoil, $k_{m,}$ has included smear effect based on the condition of the equal discharge rate with the assumption that the coefficient of permeability is independent of state of seepage flow (Bergado et al., 1994). In this approach, the permeability of the soil between drain walls in plain strain model was adjusted to made the same discharge as the actual three-dimensional case at site. There is no change in the vertical permeability of the subsoil, k_v in the analysis.

In order to simulate the pumping of water during vaccum preloading with suction of 40kPa, an equivalent drop in groundwater head of 4m is defined along the ground surface within the vertical drains treatment and at the drain-soil interface. The stage construction was idealized as shown in Figure 2 with the first stage construction after vertical drain installation and with application of 40kPa suction.

Three different conditions are simulated in the FEM analyses of embankment A and they are :

- Condition 1 : with vertical drains and constant suction of 40kPa.
- Condition 2 : with vertical drains in the subsoil and free drainage blanket on top of the original ground surface (No Suction).
- Condition 3 : without vertical drains and no suction.

For each stage of filling and resting period (duration for consolidation), the pore water pressures in the subsoil at the centre of the embankment were obtained at depths of 3m, 6m and 8m for comparison between the three different conditions and measured values. The factors of safety (FOS) of the embankment against slip failure were also calculated by reducing the strength parameters of the soil (Brinkgreve & Bakker, 1991). When calculating the FOS, all soil models will be replaced with a standard Mohr-Coulomb model since stress-dependent stiffness behaviour and hardening effects are excluded from the analysis. The advantage of this method in determining the FOS is that it can cope with most complex kind of geotechnical constructions and does not have to predefine the failure mechanism. Figure 6 shows the calculated pore water pressures and the FOS for different conditions at each loading stage for Embankment A.

The FEM analyses indicate that if the vacuum suction has functioned effectively at site, the pore water pressure will not exceed the initial hydrostatic pore water pressure of the subsoil for all stages of construction except at 1st stage (Stage B) where positive excess pore water pressures of about 20kPa were generated in the subsoil. Even if there is no application of suction and with only vertical drains functioning properly in the subsoil, positive excess pore water pressure of about 30kPa is generated in the subsoil during loading. For both conditions, the FOS obtained from the FEM analysis are still higher than 1.2 as shown in Figure 6. Therefore the embankment will not fail if vertical drains and the drainage blanket are effective with or without vacuum suction. However, in the actual condition at site, the drainage blanket is sealed by membrane connecting to suction pump. As a result, effective free drainage to drain off the excess pore water collected in the vertical drains is unlikely in the case of breakdown of suction pump and the stability of the embankment is thus not warranted.

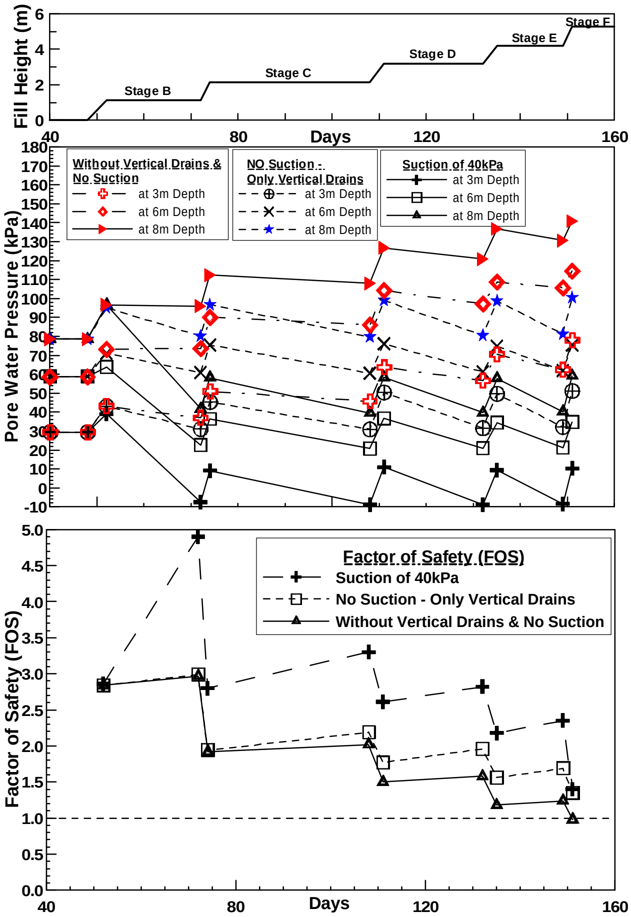

Figure 6 : Calculated pore water pressures and factor of safety for each stage of loading for Embankment A.

The positive excess pore water pressures in the subsoil will be higher than 50kPa if there is no suction and the vertical drains are not effective. The FOS will also drop from higher than 2.0 in the 1st stage (Stage B) of loading to less than 1.0 in the final stage (Stage F) of loading to a fill height of 5.5m.

5 CONCLUSION

The investigation by the Authors indicates that the failure of Embankment A could have been avoided if observational method was employed properly. This is because the pore water pressure measured at site will be able to indicate whether the vacuum system and the vertical drain are functioning properly. Through the conventional limit equilibrium stability analyses using Undrained Strength Method (Ladd, 1991) and finite element analyses, it is evident that Embankment A failed because the vacuum preloading was not functioning effectively at site. FEM analyses further indicate that even if suction was not applied, the embankment would not have failed if the vertical drains and free drainage of vertical drains are effective at the site.

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