STABILITY ANALYSIS OF EMBANKMENT ON SOFT GROUND  
(A CASE STUDY)

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ABSTRACT

The effects of partial drainage and construction settlement on stability analysis of embankment on soft ground have been investigated by analyzing a built to failure embankment on Muar clay deposit, Malaysian. Although for design, ignoring the partial drainage and construction settlement effects in stability analysis is on safe side, to understand the failure mechanism and to back-analyze the mobilized strength of soft ground at failure, these effects need to be considered. For the embankment analyzed, undrained strength increase and settlement of the ground during construction can cause a increase of factor of safety by about 0.2, or each of the effect by 0.1. The contour of shear stress level (shear stress/shear strength) of the soft ground has been analyzed from finite element analysis results. It has been illustrated that the contour of shear stress level of soft ground can indicate the stability condition of the embankment. At the embankment failure, a large area beneath the embankment and around the embankment toe, the shear stress level is 1.0. However, finite element analysis yields a poor simulation of the foundation deformation ratio (horizontal/vertical).

Key words: embankment, settlement, shear strength, soft soil, stability (IGC: E2/E13)

INTRODUCTION

For design of the embankment on soft ground, stability analysis is an important design step. In addition, the stability analysis of built to failure embankment on soft ground can verify the strength theory of the soft soil. Therefore, extensive research works have been done on this topic (e.g. Bjerrum, 1972, 1973; Tavenas and Leroueil, 1980; Brand and Shen, 1984; Ladd, 1991).

Several stability analysis methods have been developed under the limit equilibrium theory (Fredlund and Krahn, 1977; Espinoza et al., 1992). However, for embankment on soft ground, simplified Bishop’s method (Bishop, 1955) which satisfies the moment equilibrium is commonly used. Although more rigorous methods which satisfy both force and moment equilibriums, such as Morgenstern and Price’s method (1965), have been developed, this kind of refinement only has marginal effect on the factor of safety (Espinoza et al., 1992). The most difficult thing in stability analysis is to determine the strength of the soil mass. Another factor, for embankment on soft ground, is the ground settlement during construction which will cause the actual embankment height different from the fill thickness.

In this paper, the undrained strength determination methods for soft soil and the possible construction settlement of embankment on soft ground are discussed first. The influence of partial drainage and ground settlement of soft ground on embankment factor of safety is investigated by analyzing a built to failure embankment on Muar clay deposit, Malaysian. The contour of shear stress level (shear stress/shear strength) of soft ground from finite element analysis and the ratio of maximum lateral to vertical foundation deformation are also analyzed.

UNDRAINED SHEAR STRENGTH OF CLAY

According to the strength parameters used, the stability analysis may be classified as total stress, undrained strength, and effective stress analyses (Ladd, 1991). For an embankment on soft ground, the undrained strength analysis is closer to field condition because the embankment failure occurs in a very short time, which equivalent to an undrained shear, although the undrained shear strength of soft soil can increase due to partial drainage before the embankment failure (Ladd, 1991). Several methods for determining the undrained shear strength of
soft soil have been developed (Bjerrum, 1972, 1973; Ladd, 1991).

**Field Vane Shear Test**

The field vane shear strength is most commonly used in practice. The vane shear test can minimize the disturbance on natural ground, and conducted under in-situ stress condition, but it has limitations to simulate the actual embankment failure process, such as strength anisotropy, strain rate, etc. Therefore, the values of the field vane strength used for slope Stability analysis is by applying a correction factor, such as that proposed by Bjerrum (1972).

**Laboratory Test**

The triaxial unconsolidated undrained (UU) and triaxial isotropically consolidated undrained (CIU) tests are commonly used in practice to determine the undrained shear strength of soil. However, the tests cannot simulate the actual field condition well. More sophisticated laboratory test programs have been proposed by considering the stress history, anisotropy, and stress path effects. The well known methods are recompression technique recommended by Norwegian Geotechnical Institute (Bjerrum, 1973) and SHANSEP technique established by Ladd and Foot (1977). The application of both methods needs high quality soil samples and detailed information of stress history of the samples. However, the SHANSEP technique cannot be applied to highly structured clay because the compression of a soil sample beyond its post-maximum consolidation pressure well destroy the structure of the sample.

**Empirical Equation**

The undrained shear strength, $S_u$, of soft soil is the function of internal friction angle, $\phi$, and mean effective stress of the soil at failure, $p_f$. Since the $p_f$ is generally not known, in empirical equations, it is related to the vertical effective pressure, $\sigma_v$, by overconsolidation ratio, OCR (Wroth, 1984). The equation proposed by Ladd (1991) is as follows:

$$\frac{S_u}{\sigma_v} = S(OCR)^m$$

(1)

Where $S$ and $m$ are constants. For the soils investigated by Ladd (1991), the value of $S$ is from 0.162 to 0.25, and $m$ is from 0.75 to 1.0.

**From Constitutive Model**

The undrained strength of the soft soil can also be derived from constitutive model. The modified Cam clay model (Roscoe and Burland, 1968) is one of the most widely used soil models for soft clay. This model yields a undrained shear strength of soil as follows:

$$S_u = \frac{1}{2^{\lambda+\eta}} M \phi (OCR)^\eta$$

(2)

$$p_0 = \left(\frac{M^2 + \eta^2}{M^2}\right) p^\prime$$

(3)

where $p^\prime$ is the effective mean stress, $\eta$ is the stress ratio $q/p^\prime$ ($q$ is deviator stress), $p_0$ is the corresponding effective mean stress on isotropic consolidation line, $M$ is the slope of the failure line in $(q, p^\prime)$ plot, OCR represents the overconsolidation ratio, and $\lambda$ equals $(1 - \kappa/\lambda)$, $\kappa$ and $\lambda$ are the slopes at void ratio versus logarithm mean effective stress plot during reloading and virgin loading, respectively.

**CONSTRUCTION SETTLEMENT**

It is well known that during an embankment construction there is a difference in fill thickness and embankment height because of the ground settlement. The effects of the ground settlement on embankment stability are: (1) reducing the surcharge loading, and (2) partially replacing the soft soil (weaker) by the fill material (stronger). In stability analysis, the net driving moment is from embankment fill above the ground surface. Therefore, the settlement will cause the reduction of net driving moment. For estimation, the percentage reduction in net driving moment can be the same as the percentage reduction in embankment height. In this case, from the definition of the factor of safety, the approximately same percentage increase in the embankment factor of safety can be expected. If the resistance from fill material can be ignored, it can have roughly the same effect of increasing the undrained shear strength of soft ground by the same percentage. Therefore, in back-analysis, if the embankment fill thickness is used as surcharge loading, the mobilized undrained strength of soft ground will be overestimated.

The construction settlement may be calculated by one of the following methods: (1) elastic theory, (2) empirical equation, and (3) finite element analysis.

The elastic theory usually assumes that during construction the foundation behaves undrained. Using the elastic theory to determine the construction settlement, the key point is to determine a proper foundation modulus. The ratio of undrained modulus to undrained shear strength is varied from 200 to 2000 (Ladd, 1991).

From investigating the behavior of several field test embankments, Tavenas and Leroueil (1980) pointed out that the clay response to the embankment construction loading is not truly undrained and a significant consolidation develops initially in the overconsolidated natural clay deposit, and only after clay becomes normally consolidated, the undrained condition can be assumed. According to this assumption, a method of estimating the construction settlement has been proposed by separating the consolidation settlement and undrained distortion.

The construction settlement due to undrained distortion, $S_u$, is empirically estimated as a linear function of embankment height (Tavenas and Leroueil, 1980).

$$S_u = (0.07 \pm 0.03)(H - H_m)$$

(4)

where $H$ is the embankment height, and $H_m$ is the threshold height at which the foundation soil becomes normally consolidated. The Equation (4) indicates that
the settlement due to undrained distortion is 4 to 10% of corresponding embankment height.

Another method of calculating the construction settlement is using finite element analysis. In finite element analysis, the construction process, and the partial drainage effect of the foundation soil can be systematically considered. Although, the results of finite element analysis depend on soil models and parameters used, the acceptable results can be obtained by experienced user. In following section, the finite element method is used to obtain deformation and pore pressure information for analyzing the stability of the Malaysian embankment.

STABILITY ANALYSIS OF THE MALAYSIAN EMBANKMENT

For quantitatively illustrating the influence of undrained shear strength increase and ground settlement on the factor of safety of an embankment on soft ground, one of the Malaysian test embankments (MHA, 1989) which was rapidly built to failure has been analyzed. The slip circle analysis (simplified Bishop method) is used to define the factor of safety. The undrained shear strength of the soft clay determined by different method and corresponding embankment factor of safety are compared. The contour of shear stress level of soft ground and foundation deformation ratio have been also analyzed.

Brief Description of the Embankment

The rapidly built to failure embankment was constructed directly on the natural ground. The fill was compacted in 0.2 m layers at a nominal rate of 0.4 m per week until failure occurred. The fill material was decomposed granite (sand 50%, clay 38%). The embankment was constructed with a base dimension of 55 m wide and 90 m long initially to a fill thickness of 2.5 m. Then, a 15 m wide berm was left on three sides, and the embankment was built to failure with a fill thickness of 5.4 m (Brand, 1991). Index properties of Muar clay deposit at test site are listed in Table 1 (AIT, 1988, 1989). Detailed description of this test embankment was given by Brand and Premchitt (1989) and Brand (1991).

Finite Element Analysis

The finite element mesh is shown in Fig. 1, and the key field instrumentation points and the construction schedule are also shown in the figure. The 8 node rectangular or 6 node triangular plane strain elements are used.

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Type of Soil</th>
<th>Water Content ( Wc ) %</th>
<th>Liquid Limit ( WL ) %</th>
<th>Plasticity Index ( Lu ) %</th>
<th>( C_v ) ( 1 + e_0 )</th>
<th>Clay Content %</th>
<th>Silt Content %</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-2</td>
<td>Crust</td>
<td>60-80</td>
<td>—</td>
<td>—</td>
<td>0.30</td>
<td>62</td>
<td>35</td>
</tr>
<tr>
<td>2-7</td>
<td>Very soft silt clay</td>
<td>80-110</td>
<td>65-90</td>
<td>40-50</td>
<td>0.50</td>
<td>45</td>
<td>52</td>
</tr>
<tr>
<td>7-12</td>
<td>Soft silt</td>
<td>70-100</td>
<td>75-90</td>
<td>40-50</td>
<td>0.30</td>
<td>50</td>
<td>47</td>
</tr>
<tr>
<td>12-18</td>
<td>clay</td>
<td>55-65</td>
<td>58-65</td>
<td>30-40</td>
<td>0.24</td>
<td>50</td>
<td>47</td>
</tr>
<tr>
<td>18-22</td>
<td>Sandy clay</td>
<td></td>
<td></td>
<td></td>
<td>0.10</td>
<td>20</td>
<td>36</td>
</tr>
</tbody>
</table>

![Fig. 1 Finite element mesh with key field instrumentation points](image-url)
The behavior of the foundation soil is simulated by modified Cam clay model (Roscoe and Burland, 1968). For compacted fill material, the hyperbolic nonlinear elastic model (Duncan et al., 1980) is used. The corresponding input parameters for foundation soil and fill material are given in Table 2 and Table 3, respectively. The initial stresses of foundation soil are listed in Table 4. The modified Cam clay parameters are derived based on test results (AIT, 1988, 1989). The values of vertical permeability are taken as two times of laboratory test values and horizontal values are two times of the corresponding vertical values. The strength parameters of the fill material are from unconsolidated undrained triaxial test results (Brand and Premchitt, 1989) and other parameters are selected from the parameters collected by Duncan et al. (1980). For this embankment, the fill thickness was specified. For ensuring the applied fill thickness is the same as the field value, and also for approximately simulating the large deformation phenomenon, the node coordinates including those above current construction level are updated at the end of each incremental analysis.

The comparison of finite element results with observed data is presented in Figs. 2 and 3 for excess pore pressures and displacements, respectively. All the measured data for this embankment are from Malaysian Highway Authority (MHA, 1989). It can be seen that the finite element analysis simulates the embankment behavior reasonably well.

**Effect of Partial Drainage**

The undrained shear strength of the foundation soil determined by different method is listed in Table 5. The field vane shear strength is from Brand and Premchitt (1989). The variation of the strength with fill thickness is only calculated by Cam clay theory (Eqs. (2) and (3)) with finite element analysis results. The Cam clay strength is the average value of the soil elements from middle of the embankment to 20 m away from the embankment toe. The stability analyses by using the initial

<table>
<thead>
<tr>
<th>Table 2. Cam clay parameters for Muar clay deposit</th>
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<tbody>
<tr>
<td>Depth m</td>
</tr>
<tr>
<td>----------</td>
</tr>
<tr>
<td>0-2</td>
</tr>
<tr>
<td>2-7</td>
</tr>
<tr>
<td>7-12</td>
</tr>
<tr>
<td>12-18</td>
</tr>
<tr>
<td>18-22</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 3. Soil parameters for fill material</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cohesion $c$ (kPa)</td>
</tr>
<tr>
<td>-------------------</td>
</tr>
<tr>
<td>19</td>
</tr>
</tbody>
</table>

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**Table 4. Initial stress of Muar clay deposit**

<table>
<thead>
<tr>
<th>Depth m</th>
<th>$\sigma_{ho}$ (kPa)</th>
<th>$\sigma_{ho}$ (kPa)</th>
<th>Water Pressure $\psi$ (kPa)</th>
<th>Yield Locus $P_y$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>92.7</td>
</tr>
<tr>
<td>0.8</td>
<td>12.4</td>
<td>12.4</td>
<td>0</td>
<td>92.7</td>
</tr>
<tr>
<td>2.0</td>
<td>15.2</td>
<td>19.0</td>
<td>12.0</td>
<td>40.0</td>
</tr>
<tr>
<td>3.0</td>
<td>16.2</td>
<td>23.5</td>
<td>22.0</td>
<td>40.0</td>
</tr>
<tr>
<td>6.0</td>
<td>22.6</td>
<td>37.0</td>
<td>52.0</td>
<td>47.6</td>
</tr>
<tr>
<td>11.0</td>
<td>37.8</td>
<td>62.0</td>
<td>102.0</td>
<td>73.4</td>
</tr>
<tr>
<td>18.0</td>
<td>61.0</td>
<td>100.0</td>
<td>172.0</td>
<td>110.0</td>
</tr>
<tr>
<td>22.0</td>
<td>78.2</td>
<td>128.0</td>
<td>212.0</td>
<td>130.0</td>
</tr>
</tbody>
</table>

**Fig. 2.** Comparison of measured and calculated (FEM) excess pore pressure

**Fig. 3.** Comparison of measured and calculated (FEM) settlement

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Table 5. Undrained shear strength of Muar clay deposit

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Initial Strength, kPa</th>
<th>Strength from Cam Clay Theory, (Varied), kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Vane Shear</td>
<td>Empirical</td>
</tr>
<tr>
<td>0-2.0</td>
<td>20.0</td>
<td>14.5</td>
</tr>
<tr>
<td>2.0-4.5</td>
<td>9.5</td>
<td>8.7</td>
</tr>
<tr>
<td>4.5-7.0</td>
<td>13.5</td>
<td>10.6</td>
</tr>
<tr>
<td>7.0-9.5</td>
<td>16.5</td>
<td>13.2</td>
</tr>
<tr>
<td>9.5-12.0</td>
<td>20.0</td>
<td>16.5</td>
</tr>
</tbody>
</table>

\[ S_u = 0.22 \cdot (OCR)^{0.8} \sigma_v' \] (Ladd, 1991)

*Strength parameters of fill material: \( c = 19 \) kPa, and \( \phi = 26^\circ \)

strength are total stress analyses. It can be seen that at the end of the embankment construction, the undrained shear strength of the foundation soil can increase about 20% near the ground surface (0-4.5 m) and about 10% in the deeper layers. The relationship between embankment fill thickness and the factor of safety is shown in Fig. 4.

As discussed by Brand (1991), for this embankment, the factor of safety is influenced by the strength of the fill material significantly. If the strength parameters used for fill material are the correct values, using the undrained shear strength from Cam clay theory yields a factor of safety very close to 1.0 at the embankment failure. The difference of the factor of safety of using in-situ undrained strength (Cam clay) and that of at embankment failure (Cam clay) indicates that the partial drainage effect can increase the embankment factor of safety by about 0.1. As pointed out by Brand (1991), rather than a single failure surface, there is a failure band. Several slip circle failure surfaces can yield nearly the same factor of safety. The comparison of a typical slip circle failure surface with the actual one is shown in Fig. 5. The actual failure surface is reproduced from Brand (1991).

Effect of Ground Settlement

Figure 6 is the comparison of the factor of safety calculated by using the fill thickness and the actual embankment height as surcharge height. For this comparison, the undrained shear strength from Cam clay theory is used. The construction settlement is 0.7 m, and it is about 13% of the total fill thickness. It can be seen that the foundation settlement increased the factor of safety by about 0.1.

Although ignoring the foundation settlement in stability analysis is on safe side, for understanding the mechanism of embankment failure, and for back-analyzing the mobilized strength of soft soil, this factor should not be ignored. For this embankment, the back-analyzed average undrained shear strength of the soft ground by using the fill thickness as surcharge is about 15% higher than that of using the actual embankment height at the failure. In practice, the vane shear strength correction factor is derived from the back-analysis of the embankments built to failure. If the correction factor is determined by considering the construction settlement, it should be applied in the same condition.

Shear Stress Level and Deformation Ratio

Except the slip circle analysis, the (1) contour of shear stress level of soft ground from finite element results, and (2) displacement ratio defined as the ratio of maximum lateral to vertical foundation displacement are also analyzed. Figure 7 shows the contours of shear stress level of a zone of foundation soil at different fill thickness. Figure 7(a) shows at the embankment fill thickness of 2.5 m, a
Fig. 7. Calculated (FEM) contour of shear stress level a) at 2.5 m fill thickness b) at 3.5 m fill thickness c) at 4.5 m fill thickness d) at 5.4 m fill thickness

confined relatively higher shear stress level zone is formed under the shoulder of the embankment. When the fill thickness reaches 3.5 m, this zone is expanded to about 25 m (horizontal) by 10 m (vertical) (Fig. 7(b)). At the fill thickness of 4.5 m, a limited failure zone with the shear stress level of 1.0 (10 m horizontal by 6 m vertical)
under the toe and the shoulder of the embankment is developed (Fig. 7(c)). Finally, at the fill thickness of 5.4 m (embankment failure), the failure zone is formed beneath embankment and around embankment toe (Fig. 7(d)). This analysis indicates that the contour of shear stress level can be used to indicate the embankment stability condition. However, the contour does not clearly indicate a potential failure surface. Therefore, a value of the factor of safety is not easy to be directly defined.

The variation of deformation ratio with fill thickness for this embankment is shown in Fig. 8. The field data show that at embankment failure, the deformation ratio was about 0.6, and after the fill thickness reached 5 m, the field deformation ratio showed a rapid increase with the increase of the fill thickness. Although at the embankment failure, the deformation ratio from finite element results is close to the field value, for fill thickness less than 4 m, the finite element results yield more than twice of the field values. However, the finite element results also indicate a decreased deformation ratio when the embankment close to failure. At present, finite element analysis still has limitations to systematically and successfully predict both settlement and lateral displacement under embankment loading. Usually, the lateral displacement under the toe of embankment at early stage of construction is overestimated (Tavenas and Leroueil, 1980; Chai and Bergado, 1993). Several reasons have been cited for the poor lateral displacement prediction, such as effect of anisotropic behavior of natural clay, rotation of principal stress direction under the toe of embankment, and the value of Poisson's ratio (Poulos, 1972). The anisotropy and principal stress rotation effects have not been included into the soil model used (modified Cam clay theory). As mentioned previously, base on the field data, Tavenas and Leroueil (1980) proposed that initial response (before soil yielding) of the natural ground to the embankment loading is drained. If such an analysis is used for this case, the predicted lateral displacement and excess pore pressure can be improved, but the settlement at early stage will be overpredicted. The direct comparison of measured and calculated (FEM) maximum lateral displacements is shown by a insert figure of Fig. 8.

CONCLUSION

For stability analysis of an embankment on soft ground, the most important thing is to determine the undrained shear strength of soft soil. For the embankment analyzed, the increase of the soft ground undrained shear strength during construction is about 10 to 20% for lower and upper clay layers, respectively.

The construction settlement of soft ground also plays a role in embankment stability analysis. A lower safety factor will be obtained by using the fill thickness as surcharge loading than that of using the actual embankment height. However, in the case of back-analysis, a higher mobilized soft ground strength will be obtained with fill thickness as surcharge loading. For this particular embankment, the construction settlement increased the factor of safety by about 0.1.

It has been shown that the contour of shear stress level of the soft soil from finite element analysis can indicate the stability condition of the embankment. However, the finite element analysis yields a poor prediction on the foundation deformation ratio.

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