EMBANKMENTS OVER SOFT CLAY DEPOSITS: THE CONTRIBUTION OF BASAL REINFORCEMENT AND SURFACE SAND LAYER TO STABILITY

By Henrique Magnani Oliveira,¹; Mauricio Ehrlich, M.,² Member, ASCE, and Marcio S. S. Almeida,³ Member, ASCE.

ABSTRACT: This paper evaluates the significance of basal reinforcement and the presence of the surface sand layer in the stability. This evaluation is carried out by means of field measurements and stability analyses of three test embankments on soft clay taken to failure. Two of the test embankments were reinforced and one was unreinforced. Stability analyses were carried out taking into account measured values of reinforcement tension forces during construction. The set of analyses have shown that the top sand layer was more important to the stability of the embankments than the basal reinforcement. The cases studied have also shown that the conventional design practice that assumes for the reinforcement a fixed tension contribution may lead to unrealistic higher factor of safety.

Keywords: soft clay, surface sand layer, reinforcement, test embankments, monitoring, stability analyses.

¹Department of Civil Engineering, Federal University of Santa Catarina, Brazil, henriquemo@tcu.gov.br
²Graduate School of Engineering, Federal University of Rio de Janeiro, Brazil, me@coc.ufrj.br
³Graduate School of Engineering, Federal University of Rio de Janeiro, Brazil, mssal@globo.com
INTRODUCTION

In the mid-nineties, a reclamation hydraulic sand fill (working platform) was built over a soft clay deposit as the first construction phase of a motorway near shore in the city of Florianopolis in southern Brazil. This working platform had the purpose of allowing traffic of heavy equipments and also to raise the ground surface above the flooding level so that construction could be carried out using standard fill construction methods.

In order to provide quality data for the geotechnical design and construction control for the future motorway project three test embankments - two reinforced and one unreinforced - were performed (Magnani, 2006). Reinforced mobilized tension forces were monitored during construction using load cells. The test embankments allowed verify the significance of reinforcement in the stability of the embankments against basal failure.

This paper accesses the significance of basal reinforcement and the presence of the surface sand layer (working platform) in the stability of the test embankments by means of field measurements and stability analyses.

TEST EMBANKMENTS

Soils, reinforcement and drains

Table 1 presents a summary of the main geotechnical characteristics of Florianopolis soft clay. The soft clay under the test embankments presents the behavior of normally consolidated clay. Note that this normally consolidated behavior was promoted by the stress increase due to the construction of the hydraulic fill (working platform) six years before the construction of the test embankments.
The test embankments were conceived with different features in order to yield relevant data for the motorway construction. Test embankment 1, TE1, was conceived with vertical drains and basal reinforcement, as is generally adopted in motorways, and test embankment 2, TE2, was conceived just with basal reinforcement. Test embankment 3, TE3, without drains and reinforcement, was also built. Table 2 presents the main features of the reinforced test embankments. Reinforcement was provided by a single layer of a bidirectional polyester woven geotextile with 200 kN/m by 45kN/m ultimate strength.

Undrained strength profiles at the centre of each embankment are presented in Figure 1. These continuous profiles were computed from piezocone tests performed immediately before construction of the test embankment using equation \( S_u = \frac{(q_T - \sigma_v)}{N_{kt}} \) where \( S_u \) is the undrained strength, \( q_T \) is the corrected cone resistance, \( \sigma_v \) is the total vertical stress and \( N_{kt} \) is the empirical cone factor. \( N_{kt} \) value equal to 12 was adopted based on correlations between a number of vane and piezocone tests performed at the test site (Magnani and Almeida, 2008).

It is observed that, although the embankment locations were close, the thicknesses of the soft clay under each embankment are quite different. Further details about these embankments are found in Magnani et al (2008) and Magnani and Almeida (2008).

**Geometry and instrumentation**

The three test embankments had essentially the same geometry and instrumentation. In Figure 2 the cross section of embankment TE1 (before and after failure) with slopes 1(V): 1.5(H) is presented. The embankment platform in plan view has 20 m width and 30 m length. In addition, longitudinal berms in the other direction were also provided. The direction of the failure was naturally induced by the gentle inclination of the
embankment base which resulted from the process of construction of the working platform – hydraulic sand deposition. The test embankments were constructed in layers of 0.3m thickness and were built to fail in about two months.

Reinforcement forces have been measured through load cells located at four points in the reinforcement (see Figure 2). Note that actual reinforcement loads were not determined indirectly through strain measurements in the polymeric reinforcement. The load cells used for that (Magnani, 2006) have been used in a number of previous studies carried in recent years (e.g. Saramago, 2002; Saramago & Ehrlich, 2005). The test embankments were also instrumented for the monitoring of vertical and horizontal displacements and pore pressure. Analyses of these measurements are available in Magnani and Almeida (2008).

**STABILITY ANALYSES OF THE TEST EMBANKMENTS**

**Parameters and hypotheses**

Stability analyses of the two test embankments were carried out taking into consideration soil parameters and reinforcements loads shown in Table 3 and assuming that failure took place in the 9th layer of the embankment construction (for embankments TE1 and TE2) and 12th layer for embankment TE3 in accordance with monitoring data. At that stage of construction it was not possible to insert the inclinometer sensor inside the inclinometer tube located at the toe and cracks at the top of the embankments appeared. Lateral movements measured by inclinometers were especially important for definition of the failure moment (Indraratna et al., 1992, Magnani et al, 2009). The stability analyses were performed using values of the reinforcement forces T measured in the load cells shown in Figure 3. In these analyses the actual embankments geometry measured by instrumentation was considered.
For the test embankment TE1, conceived with vertical drains, Magnani (2006) estimated that the gain in strength $\Delta S_u$ before the last construction phase (~2 months) was about 15% of the original measured $S_u$ values shown in Figure 1. However, conventional procedures for the stability analyses calculation of short-term constructed embankments do not consider the gain in clay strength, thus the same procedure was adopted herein for the three embankments.

**Computed factors of safety**

Stability analyses (Magnani, 2006; Magnani et al 2008) were carried out using modified Bishop Method, as the monitoring data and field evidence indicated circular failure surfaces. This assumption is in accordance with the conventional analysis procedure (Low et al., 1990, Kaniraj and Abdullah, 1992). Field evidence also indicated that the embankment failures were three-dimensional (3D), thus the correction for 3D condition proposed by Azzouz et al. (1983) was considered

$$\frac{FS_{3D}}{FS_{2D}} = \left( 1 + 0.7 \cdot \frac{DR}{2L} \right)$$

where $FS_{2D}$ is the 2D standard factor of safety, $DR$ is the thickness of the failed region, and $2L$ is the width of the failed region. Values of the ratio $FS_{3D}/FS_{2D}$ (Magnani, 2006) for the three embankments were in the range 1.10 - 1.14.

Through these analyses the Bjerrum’s correction factor (Bjerrum, 1972) $\mu$ was computed to be close to 0.60 for embankments TE1 and TE2 and close to 0.7 for the embankment TE3. Figure 4 shows the critical circular failure surfaces obtained for embankment TE1 that match very well with inclinometer measurements. Additional information regarding variation of the factor of safety with the embankment construction and the monitoring data are shown in Magnani et al (2008).
Significance of reinforcement and the layer of sand on the stability

The significance of both reinforcement and the working platform (top sand layer) on the stability of the test embankments are evaluated here. Stability analyses were performed assuming the corresponding heights of failure and undrained shear stress resistance for the soft clay soil in embankments foundation. Analyses were carried out considering or not the presences of reinforcement (using measured T values) and surface sand layer in order to assess the contribution of each of these. Similar to previous analyses factors of safety were calculated for 3D conditions using previous determined Bjerrum’s correction factors. Table 4 summarizes the results of these studies.

It can be noted in Table 4 that the contribution of the reinforcement to the stability of the two embankments is relatively small, 2.4% and 3.6% for embankments TE1 and TE2, respectively. The low contribution of reinforcement is directly related to the low values of reinforcement tension forces T measured. The ranges of the values of T measured (and considered in the stability analyses) at the 9th layer of construction were 14 kN - 39kN/m and 8 kN/m - 33 kN/m, respectively for TE1 and TE2, as shown in Figure 3. The low reinforcement tension mobilization is related to the presence of the thick and continuous top sand layer (working platform) at the embankment foundation. Compare to the soft clay layer below the top sand layer has high strength and stiffness and represents restriction to lateral foundation movements. This restriction increases embankment stability and also reduces the mobilized tension in the reinforcement.

The relative greater influence of reinforcement in the stability for TE2 is related to its smaller soft clay thickness. Note that for both test embankments (TE1 and TE2), in general, the soft clay soil in the embankments foundation show similar shear strength resistance according the performed piezocone and vane tests (see Figure 1).
Analyses demonstrated (see Table 4) that the importance of the surface sand layer (working platform) on stability is quite high, 59% for embankment TE1 and 56% for embankment TE2. The significant contribution on embankments stability of the sand layer is mostly due to its greater strength compared to the soft clay layer below. For unreinforced embankment TE3 the contribution of the working platform was around 43%, i.e., proportionally lower than verified for the two reinforced embankments. The clay layer under the unreinforced embankment, TE3, has higher shear strength resistance, thus it reached greater height at failure. That explains the difference of the determined results, since the higher the values of soft clay strength resistance the lower the relative importance of the sand layer to the global resisting forces.

Thus, it be concluded that the importance of the surface sand layer (working platform) on the two experimental embankments was much higher than the contribution of the reinforcement. This mechanism and behavior is due to the subsoil profile at the site area, besides to the geometry of the embankments. The subsoil at the embankment foundation is composed by a thick and continuous top sand layer over a clay layer of relative low shear resistance. The top sand layer promoted restriction to foundation lateral movements and reinforcement load mobilization, as previously discussed. Note that this condition may be a common situation in engineering practice.

**Stability analyses for a constant and varying T values**

Reinforcement tension mobilization depends on the foundation lateral deformation due to increasing embankment height. Nevertheless, conventional stability analyses for reinforced embankments often assume on calculations constant T value for mobilized reinforcement force no matter this force T may or may not be actually verified in situ. The fixed T value adopted is in general a fraction of the ultimate resistance $T_{\text{max}}$ of the
reinforcement computed by applying reduction coefficients. In the case study under consideration $T_{\text{max}}$ is equal to 200kN/m. Considering standard reduction coefficients for creep and installation, as well as environmental damage, the value obtained for conventional stability calculations would be $T = 120\text{kN/m}$.

Two sets of factors of safety $F_s$ were computed, both taking into account 3D effects and Bjerrum factor $\mu = 0.60$. In one group of analyses $F_s$ were computed using the measured $T$ values varying with embankment height during construction. In the other group $F_s$ were computed using $T = 120\text{kN/m}$ constant and independent of the embankment height. The two sets of determined $F_s$ values are compared in Figure 5 for embankment TE1. Similar results were also obtained for embankment TE2. It is common to admit at service condition a factor of safety $F_s$ equal to 1.40, which would correspond for the conventional analysis with $T = 120\text{kN/m}$, to the seventh layer of construction of embankment TE1. However, the actual factor of safety determined considering measured $T$ values would be equal to 1.15, as shown in Figure 5. Thus, these analyses clearly demonstrate that unsafe results may be obtained using the conventional procedure that indiscriminately adopt fixed values of $T$ without any consideration of the actual $T$ value mobilized in situ.

This case study clear exemplify that conventional design practice may lead to unrealistic higher factor of safety. The performed analyses also indicate that under similar conditions adoption of reinforcement with a high tensile resistance would not necessary lead to an increasing safety factor.

**CONCLUSIONS**

Stability analyses were carried out for three test embankments, one of which was unreinforced and two reinforced. Stability analyses of the reinforced embankments were
performed taking into account values of measured tension in the reinforcements. These analyses used Bjerrum’s correction factors applied to the vane undrained resistance and Azzouz and Baligh’s correction for three-dimensional effects. The critical rupture surfaces obtained were close to the failure surfaces observed in situ.

For the case studies under consideration analyses demonstrated that the mobilized tension values in the basal reinforcement increase the factor of safety in the range 2.4 – 3.6%. The low contribution of reinforcement is directly related to the low values of tension mobilization in the basal reinforcement, which maximum measured values were about 30kN/m. In the other hand, analyses also point out that the working platform was responsible for increment of factors of safety by 43% to 59%. Thus, considering the performed analyses the importance of the top sand layer (working platform) on stability of the experimental embankments was much higher than the contribution of the basal reinforcement. This mechanism and behavior is due to the thick and continuous top sand layer over a clay layer of relative low shear resistance. The sand layer promoted lateral restriction to foundation lateral movements and low tension mobilization in the basal reinforcement, which may be a common situation in engineering practice.

Stability analyses assuming a fixed reinforcement tension contribution based on the ultimate tension force may lead to factors of safety substantially higher than actual ones determined using measured mobilized tension value. In the cases studies under consideration the difference of values was about 20%. Thus, this case study clear exemplifies that conventional design practice that assumes for reinforcement a fixed tension contribution may lead to unrealistic higher factor of safety to basal failure.

REFERENCES


### Table 1 – Geotechnical parameters of Florianopolis soft clay

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water content w (%)</td>
<td>100–170</td>
</tr>
<tr>
<td>Liquidity index wL (%)</td>
<td>105–165</td>
</tr>
<tr>
<td>Average plasticity index IP (%)</td>
<td>80</td>
</tr>
<tr>
<td>Bulk weight γ (kN/m³)</td>
<td>13.2–14.2</td>
</tr>
<tr>
<td>Voids ratio e</td>
<td>2.9–4.5</td>
</tr>
<tr>
<td>Compression ratio Cc / (1 + e₀)</td>
<td>0.30–0.45</td>
</tr>
<tr>
<td>Coefficient of vertical consolidation cᵥ – normally consolidated (m²/s)</td>
<td>0.7–1.0 × 10⁻⁸</td>
</tr>
<tr>
<td>Sensitivity (vane)</td>
<td>3–6</td>
</tr>
</tbody>
</table>

### Table 2 – Main features of the three test embankments

<table>
<thead>
<tr>
<th>Embankment</th>
<th>TE1</th>
<th>TE2</th>
<th>TE3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforcement</td>
<td>Polyester Stabilenka -200×45 kN/m; J=1700 kN/m</td>
<td>No reinforcement</td>
<td>No reinforcement</td>
</tr>
<tr>
<td>Vertical drains</td>
<td>Colbondrain CX 1000, 10 cm × 0.5 cm, triangular array, 1.30 m spacing</td>
<td>No drains</td>
<td>No drains</td>
</tr>
<tr>
<td>Clay thickness (m)</td>
<td>8.2</td>
<td>5.6</td>
<td>4.5 (sand layer from 2.8 - 3.5 m)</td>
</tr>
<tr>
<td>Working platform thickness (sand hydraulic fill) (m)</td>
<td>1.7</td>
<td>1.8</td>
<td>2.1</td>
</tr>
</tbody>
</table>

### Table 3 – Soil parameters and reinforcements loads adopted in stability analyses

<table>
<thead>
<tr>
<th>Embankment and sand layer working platform</th>
<th>Soft clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bulk weight, kN/m³</td>
<td>17.5</td>
</tr>
<tr>
<td>Strength parameters</td>
<td>c = 0 and φ’= 33.8°</td>
</tr>
<tr>
<td>Reinforcements forces</td>
<td>Values measured in each load cell in each loading stage (see Figure 3)</td>
</tr>
</tbody>
</table>

### Table 4 – Contribution of the reinforcement and working platform on the stability for failure conditions

<table>
<thead>
<tr>
<th>Test embankment</th>
<th>Values of F₃SD</th>
<th>Increment on the values of F₃SD</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>With working platform</td>
<td>Without sand layer and reinforcement</td>
</tr>
<tr>
<td>TE1</td>
<td>1.058</td>
<td>1.034</td>
</tr>
<tr>
<td>TE2</td>
<td>1.098</td>
<td>1.059</td>
</tr>
<tr>
<td>TE3</td>
<td>-</td>
<td>1.054</td>
</tr>
</tbody>
</table>
Figure 1 – Undrained strength profiles at test embankments TE1, TE 2 and TE3
Figure 2 – Embankment geometry and instrumentation (TE1)

- Dec 09 ⇔ 10th layer of construction (significant cracks appeared)
- Position of the load cells in the reinforcement
Figures 3 – Measured mobilized tension in the reinforcement for each embankment layer of construction
Figure 4– Failure surface determined through stability analyses and measured vertical deviation ($\theta$) by inclinometers (embankment TE2)

Dec 09 $\leftrightarrow$ 10th layer of construction (significant cracks appeared)
Figure 5 – Determined factors of safety for embankment TE1 assuming for tension in the reinforcement: (a) measured values and; (b) constant independent of the embankment height.