SOFT SOIL RESPONSE AND BEHAVIOUR OF PILES UNDER A GEOTEXTILE REINFORCED EMBANKMENT

Prasun Halder\textsuperscript{1}\thanks{M.Tech. Student, Department of Civil Engineering, Indian Institute of Technology Guwahati, Guwahati – 781039. p.halder@iitg.ernet.in and baleshwar@iitg.ernet.in}
Baleshwar Singh\textsuperscript{2}
\textsuperscript{1}\textsuperscript{2}Professor, Department of Civil Engineering, Indian Institute of Technology Guwahati, Guwahati – 781039. p.halder@iitg.ernet.in and baleshwar@iitg.ernet.in

ABSTRACT: It is always problematic to design and construct road embankments in soft soils having low shear strength and high compressibility characteristics due to the risk of excessive settlement, bearing failure and, lateral movement. But the inclusion of rigid piles in the soft soil and a geotextile layer at the base of the embankment can efficiently transfer the superstructure load to competent strata beneath. In this paper, a road embankment is first modeled using finite element software and its overall response is studied. Thereafter, piles and a single geotextile layer are incorporated in the system as reinforcement to investigate the change in the response of the foundation soil. The behavior of the rigid piles is also analyzed at different locations under the embankment base. Numerical results indicate that there is considerable reduction in subsoil settlement, lateral displacement of soil, vertical pressure on soft foundation soil and excess pore water pressure due to reinforcement action. The ground reaction modulus is found to decrease with the increase of ground settlement. The frictional resistance of a driven pile in clay soil after a month is found to be considerably higher than its value immediately after installation due to the thixotropic hardening phenomenon.

Keywords: Pile-supported embankment, geotextile reinforcement, settlement, ground reaction modulus

1 INTRODUCTION

It is very risky to construct road embankment in soft soils owing to its susceptibility to failure due to excessive settlement, differential settlement, inadequate bearing capacity and lateral displacement of soils under the toe of the embankment. To address such problems embankments can be supported by piles which appear to be a useful ground improvement method. In this technique vertical stiff piles are driven through the soft layers and embedded in a competent substratum beneath to support the granular earth embankment above. The partial load transfer onto the piles takes place due to the arching effect in the embankment fill. Addition of geosynthetic layer over piles enhances the load transfer mechanism resulting stress reduction on the subsoil and consequent surface settlement reduction.

Many researchers have studied this problem over the years and come up with solutions either analytically or, numerically. Terzaghi (1936) described the concept of soil arching through trap-door experiment. The concept of arching with a semi-spherical dome of arching shell was proposed by Hewlett and Randolph (1988). Low et al. (1994) made some refinements by introducing a factor ‘$\alpha$’ for the unreinforced case to incorporate the non-uniform vertical stress on subsoil and then given the formula for stress concentration ratio, efficiency. Abusharar et al. (2009) came up with some refinements like introduction of uniform surcharge load over the embankment and consideration of the skin friction mechanism to address the soil-geotextile interaction problem. Through a numerical study Han and Garb (2002) concluded that GRPS system reduces settlement and larger stiffness of piles promotes higher soil arching effect. A 3-D finite element analysis was performed by Liu et al. (2007) of a road embankment which was originally constructed in Shanghai, China. They compared the computed 3-D results with the field data available and found it to be in close agreement with the field study. Rowe and Liu (2015) performed a fully coupled and fully three dimensional finite element analyses for an embankment to study the behavior of the same under different ground improvement techniques. In this paper an embankment is modelled in PLAXIS 2D and the response of soft soil under the embankment is evaluated. The behavior of piles is also studied.

2 MODELING DETAILS

The numerical analysis is performed using the finite-element software PLAXIS 2D. The finite-element mesh used in the analysis is shown in Fig. 1. The embankment is 1.8 m high having a base width of 15.6 m. The groundwater table lies 1 m below the ground surface. The soft foundation soil is 13 m deep and consists of three layers. The subsoil profile is modelled with a rough rigid bottom boundary. To minimize boundary effects, the lateral boundary of the finite-element mesh is extended 40 m horizontally either side of the embankment center. The planes along $x = -$
40 and \( x = +40 \) are smooth and rigid i.e. Zero displacement in the \( x \)-direction. At the bottom of the mesh the imperious boundary condition has been applied also. First only the embankment is modeled to see its overall performance. Then the combination of piles and one layer of geotextile (4000 kN/m) is used to study the improvement in response. The geotextile layer is placed in between two gravel layers, each having the thickness of 250 mm. 8 m long and 300 mm diameter piles are used at 1.3 m spacing center to center for support.

Mohr-Coulomb and soft soil models have been used to simulate the behavior of embankment fill and soft foundation soil respectively. Here the 15-node triangular soil element is used. The stiff pile is modelled with 5-node embedded pile element which is isotropic linear elastic material. The three layered foundation soil is constructed in one step in the first phase. Then installation of piles and embankment construction are done in the next phases one by one. The whole system is kept for 567 days of monitoring period after the end of embankment construction. The properties of embankment fill, gravel and pile are given in Table 1 and the properties of soft foundation soil are indicated in Table 2.

### Table 1 Properties of Embankment Fill, Gravel and Pile used in FE Analysis

<table>
<thead>
<tr>
<th>Material</th>
<th>Unit weight (kN/m³)</th>
<th>Friction angle (degrees)</th>
<th>Cohesion (kPa)</th>
<th>Young’s modulus (MPa)</th>
<th>Poisson’s ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Embankment Fill</td>
<td>18.60</td>
<td>33.8</td>
<td>11.5</td>
<td>20</td>
<td>0.3</td>
</tr>
<tr>
<td>Gravel</td>
<td>20</td>
<td>36</td>
<td>60</td>
<td>70</td>
<td>0.3</td>
</tr>
<tr>
<td>Pile</td>
<td>24</td>
<td>-</td>
<td>-</td>
<td>20000</td>
<td>0.2</td>
</tr>
</tbody>
</table>

3  RESULTS AND DISCUSSION

Point ‘A’ (19.80, 0) and ‘K’ (19.73, -0.02) are chosen on the top of the subsoil layer for the calculation of vertical settlement of sub soil and vertical stress. For the calculation of excess pore pressure three points have been marked on soft soil along the centerline of embankment as ‘C’ (19.80, -1), ‘D’ (19.80, -5), and ‘E’ (19.80, -10.5).

3.1 Settlement of Foundation Soil

This is almost 67% reduction in maximum settlement due to the inclusion of piles and geotextile. Figure 2 indicates the variation of vertical settlement at different depth with elapsed time. The neutral point comes down as the consolidation process progresses.

Table 2 Soft Foundation Soil Properties used in FE Analysis

<table>
<thead>
<tr>
<th>Layers</th>
<th>Sandy silt</th>
<th>Soft clay</th>
<th>Clayey silt</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness (m)</td>
<td>2</td>
<td>6</td>
<td>5</td>
</tr>
<tr>
<td>( \lambda )</td>
<td>0.092</td>
<td>0.116</td>
<td>0.027</td>
</tr>
<tr>
<td>( k )</td>
<td>0.014</td>
<td>0.017</td>
<td>0.004</td>
</tr>
<tr>
<td>( C_c )</td>
<td>0.212</td>
<td>0.267</td>
<td>0.062</td>
</tr>
<tr>
<td>( C_r )</td>
<td>0.032</td>
<td>0.040</td>
<td>0.009</td>
</tr>
<tr>
<td>( \Phi ) (deg)</td>
<td>30.6</td>
<td>27</td>
<td>34</td>
</tr>
<tr>
<td>( C ) (kPa)</td>
<td>4</td>
<td>13</td>
<td>0</td>
</tr>
<tr>
<td>( K_v ) (m/day)</td>
<td>6.91*10⁻⁵</td>
<td>8.3*10⁻⁸</td>
<td>8.9*10⁻⁶</td>
</tr>
<tr>
<td>( K_o )</td>
<td>1.668</td>
<td>0.662</td>
<td>0.52</td>
</tr>
</tbody>
</table>

Figure 3 shows the stress acting on the soft soil is 37.5 kPa for the unimproved case and 12.5 kPa for the improved case which is 33.3% of the previous case. This is due to the arching action.
3.3 Ground Reaction Curves

The ground modulus \((k)\) has been calculated by dividing the vertical stress acting on the natural subsoil under embankment centerline with the ground surface settlement measured between two adjacent piles. Figure 4 shows the plot between the ground modulus and the settlement. The values of ground modulus obtained from the improved section (i.e. with piles) are lower than those for the unimproved section (no pile), which is a consequence of soil arching. Actually for such soil it was found that the increase of stress (due to the absence of piles) is larger than the increase of settlement, resulting in a larger ground modulus value.

3.4 Excess Pore Pressure Generation and Dissipation

Figure 5 shows that at points C and E, the excess pore water pressure increases during construction and starts dissipating during the consolidation period. But at point D, this excess pore pressure increases during dissipation. Huang et al. (2009) explained that due to longer drainage path and low permeability of silty clay such increase in pore pressure takes place. This is called the Mandel-Cryer effect. Maximum excess pore pressure parameter ‘\(B_{max}\)’ is defined as the ratio of the maximum excess pore pressure to the change in total vertical stress of the embankment. Rowe and Soderman (1985) suggested that this value should be less than 0.34 for the safety of embankment against bearing failure. Here the estimated ‘\(B_{max}\)’ is 0.15. It can be well understood that for all the depths the ratio is less than the limiting value. Therefore, the embankment is safe against bearing capacity failure state.

3.5 Lateral Displacement of Soil below the Toe of the Embankment

Figure 6 shows the lateral displacement profile of soft soil calculated at 1 m distance from the toe of the embankment for both unimproved and improved cases. It can be seen that the maximum displacement occurs at depth of 3.5 m below the ground level due to higher compressibility.
(2002) showed that for the embankment safety, this ratio must be less than 0.5. In this case the ratio is 0.07 which is on the safer side in terms of the stability of the road embankment considered.

3.6 Axial Load and Skin Friction Distribution in Piles

Figure 7 and 8 show the axial load distribution and skin friction variation along the length of the piles. The values are directly obtained from the numerical analysis. The maximum load at the pile head is 37 kN/m and 18 kN/m for the central and corner pile respectively.

![Fig. 7 Comparison of axial load distribution between the central and corner piles](image1)

![Fig. 8 Comparison of skin friction resistances between the central and corner piles](image2)

Due to the high compressibility characteristics of the clay soils, they move more than the piles in the downward direction resulting negative skin friction upto some depth in the soil as shown in Figure 8.

4 CONCLUSIONS

From the work presented in this study the following conclusions can be drawn:

1. The subsoil settlement and the vertical pressure acting on the soft foundation soil get reduced by 67% and 65% respectively.
2. The excess pore water pressure increases during dissipation in clayey soil having very low permeability away from the drainage boundary.
3. For the considered embankment, the ratio ‘BM’ is less than the limiting value of 0.34. Therefore, the embankment is safe against bearing capacity failure state.
4. The ratio of the maximum lateral displacement to maximum central settlement has been calculated as 0.07 which the stability of the road embankment considered.

References


