May 6th, 12:00 AM

Field observation of pore pressure in soft clay

R.L. Wei

Follow this and additional works at: http://scholarsmine.mst.edu/icchge

Recommended Citation
http://scholarsmine.mst.edu/icchge/1icchge/1icchge-theme1/20

This Article - Conference proceedings is brought to you for free and open access by the Geosciences and Geological and Petroleum Engineering at Scholars’ Mine. It has been accepted for inclusion in International Conference on Case Histories in Geotechnical Engineering by an authorized administrator of Scholars’ Mine. For more information, please contact weaverjr@mst.edu.
Field Observation of Pore Pressure in Soft Clay

R. L. Wei
Senior Engineer, Deputy Head of Geotechnical Engrg. Div., Nanjing Hydraulic Research Institute, Nanjing, China

SYNOPSIS
Pore pressure measurements made during the preloading of a large oil tank on a deep deposit of soft clay are reported and analyzed. The piezometer data show that the excess pore pressure at a point in the subsoil increases linearly with the increasing surface load until a certain critical load is reached. Then the induced shear stress at that point reaches the shearing strength of the soil, and local yield occurs. Henceforth there is a pronounced increase in the observed rate of pore pressure built-up.

INTRODUCTION
According to the principle of effective stress, the strength and deformation behaviour of a soil are mainly dependent on the effective stress acting on the soil. Therefore, it is necessary first to know the changes in pore pressure in order to evaluate the performance of soil foundation during loading. In the process of preloading a large oil tank (~ 20,000 m³) built on a deep deposit of soft clay, the pore pressures were measured and analyzed.

Altogether, 14 piezometers were installed under the tank (Figure 1). The subsoil was very soft (see Table I). The design and operation of the tank were based on the condition that the bearing capacity of the soft clay foundation could be increased by means of preloading. During the filling of the tank with water in the preloading process, the increasing pore pressures in the subsoil were observed. After a period of preloading, the tank was put in service successfully under a load of almost twice the originally estimated bearing capacity with no problem other than about 1.5m of settlement under the center of the tank.

OBSERVED RESULTS OF IN-SITU PORE PRESSURE
The observed pore pressure changes in the subsoil during the first and second preloadings are shown in Figures 2 and 3, respectively. It was noted that the pore pressures in the subsoil increased with external load. The accumulated increments of pore pressure (the dissipated pore pressure was also noted) under different loading steps were plotted versus the external load in Figure 4 and 5. It is shown that when the dissipation effect is not considered, the relationship between the pore pressure and load can be expressed by two straight lines. Initially, the pore pressure increased slowly in proportion with external load. As soon as the load had attained some critical value, the pore pressure would increase linearly at a higher rate with the load. This trend of pore pressure increase at an increasing rate at a critical state was very noticeable both in the first and second preloading periods. However, if the net readings of pore pressure after dissipation were plotted versus the external load no such distinct trend could be observed as shown by the fine dotted lines in Figure 5.

When the observed data of in-situ pore pressure

Table I -- The soil layers and their physico-mechanical properties.

<table>
<thead>
<tr>
<th>No.</th>
<th>Layer</th>
<th>Thickness (m)</th>
<th>Water content (%)</th>
<th>Unit weight (g/cm³)</th>
<th>Void ratio</th>
<th>Consolidated strength (C/(1+e))</th>
<th>Shearing strength (kg/cm²)</th>
<th>Unconfined modulus</th>
<th>Drained (kg/cm²)</th>
<th>Coefficient of permeability</th>
<th>BBS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Hydraulic fill</td>
<td>4.0</td>
<td>38.4</td>
<td>1.79</td>
<td>1.09</td>
<td>21.5</td>
<td>0.05</td>
<td>32</td>
<td>0</td>
<td>0.10</td>
<td>2.53x10⁻⁶</td>
</tr>
<tr>
<td>2</td>
<td>Soft silty clay</td>
<td>10.2</td>
<td>37.4</td>
<td>1.80</td>
<td>1.07</td>
<td>22.6</td>
<td>0.06</td>
<td>32</td>
<td>0</td>
<td>0.10</td>
<td>2.35x10⁻⁵</td>
</tr>
<tr>
<td>3</td>
<td>Soft clay</td>
<td>12.0</td>
<td>48.8</td>
<td>1.71</td>
<td>1.39</td>
<td>10</td>
<td>0.10</td>
<td>28</td>
<td>0</td>
<td>0.20</td>
<td>1.02x10⁻⁷</td>
</tr>
<tr>
<td>4a</td>
<td>Grey sandy clay</td>
<td>3.1</td>
<td>24.5</td>
<td>1.98</td>
<td>0.72</td>
<td>4.2</td>
<td>0.03</td>
<td>24</td>
<td>0</td>
<td>0.06</td>
<td>0.06</td>
</tr>
<tr>
<td>4b</td>
<td>Brown sandy clay</td>
<td>2.5</td>
<td>33.5</td>
<td>1.85</td>
<td>0.96</td>
<td>24.2</td>
<td>0.03</td>
<td>18</td>
<td>0</td>
<td>0.14</td>
<td>0.14</td>
</tr>
</tbody>
</table>
Fig. 1 - The soil layers under the tank and the layout of the piezometers.

Fig. 2 - The observed pore pressures during the first preloading.

Fig. 3 - The observed pore pressures during the second preloading.

was interpreted and analyzed, the accumulated increments \( \Delta u \) were plotted rather than the actual reading \( u \) taken immediately after each step of loading to eliminate the dissipation effect, and a more regular relationship could be obtained. However, it is not implied that the curves in Figures 4 and 5 represent the completely undrained condition.

Figures 4 and 5 show that when each step of loading was maintained for a period of time so that the subsoil might be consolidated to a certain degree, the above mentioned critical load increased and the rate of increase of pore pressure beyond critical load decreased with the repetition of loading.

After many repetitions of loading and unloading during the service operation of the tank, the critical load on the subsoil would approach the maximum service load and the rate of increase of pore pressure with external load approached a constant value which coincided with that under loads less than critical during previous loading.

DISCUSSION

Some characteristic values of Figures 4 and 5 are summarized in Table II. Of special interest are the critical loads at which the rate of increase of pore pressure changes abruptly and the rates of increase of pore pressure with load before and after the critical load. These values vary with soil properties and the changes of stress at the location which the piezometers are installed.
Fig. 4 - The variation of pore pressure in soft silty clay with surface load.

Legend:
- $Z(\Delta u)$ during first preloading.
- $Z(\Delta u)$ during second preloading.
- $Z(\Delta u)$ during loading after 10 months of service.
- $Z(\Delta u)$ during loading after 4 years of service.
- The net pore pressure readings after loading during first preloading.

Fig. 5 - The variation of pore pressure in soft clay with surface load.

Table II - Rate of pore pressure increase and critical load at various points.

<table>
<thead>
<tr>
<th>Layer</th>
<th>Critical Rate of increase</th>
<th>1st</th>
<th>2nd</th>
<th>Before critical load</th>
<th>1st</th>
<th>2nd</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft</td>
<td>$U_1$</td>
<td>2.3</td>
<td>7.4</td>
<td>0.16</td>
<td>0.67</td>
<td>0.39</td>
</tr>
<tr>
<td></td>
<td>$U_2$</td>
<td>4.2</td>
<td>5.5</td>
<td>0.43</td>
<td>0.94</td>
<td>0.82</td>
</tr>
<tr>
<td>silty clay</td>
<td>$U_3$</td>
<td>2.5</td>
<td>8.3</td>
<td>0.12</td>
<td>0.55</td>
<td>0.28</td>
</tr>
<tr>
<td></td>
<td>$U_4$</td>
<td>2.7</td>
<td>8.6</td>
<td>0.11</td>
<td>0.44</td>
<td>0.26</td>
</tr>
<tr>
<td></td>
<td>$U_5$</td>
<td>4.0</td>
<td>4.8</td>
<td>0.40</td>
<td>0.90</td>
<td>0.80</td>
</tr>
<tr>
<td></td>
<td>$U_6$</td>
<td>3.4</td>
<td>4.8</td>
<td>0.36</td>
<td>0.77</td>
<td>0.73</td>
</tr>
<tr>
<td>clay</td>
<td>$U_7$</td>
<td>3.9</td>
<td>4.6</td>
<td>0.36</td>
<td>0.75</td>
<td>0.72</td>
</tr>
<tr>
<td></td>
<td>$U_8$</td>
<td>5.4</td>
<td>-</td>
<td>0.40</td>
<td>0.79</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>$U_9$</td>
<td>5.0</td>
<td>-</td>
<td>0.28</td>
<td>0.59</td>
<td>-</td>
</tr>
</tbody>
</table>

If some minor fluctuation are ignored, the mean values can be estimated roughly as follows:

In the soft silty clay:
- $(\Delta u/\Delta p)_0 = 0.15, (\Delta u/\Delta p)_1 = 0.55, (\Delta u/\Delta p)_2 = 0.30$

In the soft clay:
- $(\Delta u/\Delta p)_0 = 0.40, (\Delta u/\Delta p)_1 = 0.80, (\Delta u/\Delta p)_2 = 0.75$

where the subscripts 0, 1 and 2 denote the value obtained before the critical load is attained and the value beyond the critical load during the first and second time of preloading, respectively.

The stress induced by the external load at various points can be calculated according to Boussinesq's Theory. The rate of increase of pore pressure with stress in these points may be predicted as shown in Table III. It is seen that the trend of variation of pore pressure with the principal stress is most evident. If $B = \Delta u/\Delta \sigma$, is defined as the coefficient of in-situ pore pressure, then the mean values in the upper two main layers are as follows:

In the soft silty clay:
- $B_0 = 0.15, B_1 = 0.65, B_2 = 0.40$

In the soft clay:
- $B_0 = 0.65, B_1 = 1.40, B_2 = 1.30$

where the subscripts 0, 1 and 2 are the same as before.
Table III - Rate of increase for pore pressure with stress at various points (μ = 0.5).

<table>
<thead>
<tr>
<th>R/R</th>
<th>0.25</th>
<th>0.50</th>
<th>0.85</th>
<th>0.25</th>
<th>0.50</th>
<th>0.85</th>
<th>0.25</th>
<th>0.50</th>
<th>0.85</th>
</tr>
</thead>
<tbody>
<tr>
<td>z/R</td>
<td>(au/Δσ)₀</td>
<td>(au/Δσ)₁</td>
<td>(au/Δσ)₂</td>
<td>(au/Δσ)₀</td>
<td>(au/Δσ)₁</td>
<td>(au/Δσ)₂</td>
<td>(au/Δσ)₀</td>
<td>(au/Δσ)₁</td>
<td>(au/Δσ)₂</td>
</tr>
<tr>
<td>0.25</td>
<td>0.15</td>
<td>-</td>
<td>0.70</td>
<td>-</td>
<td>0.40</td>
<td>-</td>
<td>0.05</td>
<td>-</td>
<td>0.25</td>
</tr>
<tr>
<td>0.60</td>
<td>0.25</td>
<td>0.15</td>
<td>0.20</td>
<td>0.70</td>
<td>0.70</td>
<td>0.80</td>
<td>0.45</td>
<td>0.35</td>
<td>0.50</td>
</tr>
<tr>
<td>0.75</td>
<td>-</td>
<td>-</td>
<td>0.85</td>
<td>-</td>
<td>1.65</td>
<td>-</td>
<td>1.20</td>
<td>1.30</td>
<td>1.65</td>
</tr>
<tr>
<td>0.90</td>
<td>0.65</td>
<td>0.65</td>
<td>0.80</td>
<td>1.40</td>
<td>1.50</td>
<td>1.75</td>
<td>1.20</td>
<td>1.30</td>
<td>1.65</td>
</tr>
<tr>
<td>1.20</td>
<td>0.70</td>
<td>-</td>
<td>0.80</td>
<td>1.40</td>
<td>-</td>
<td>1.65</td>
<td>1.35</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

The regularity of au/Δp and au/Δσ, obtained in the above is of practical interest. The pore pressure in the subsoil may be predicted according to the designed load or the stress induced in the soil by means of the above mentioned empirical coefficients as an application of effective stress analysis.

In the soft clay which is less permeable and less influenced by the dissipation during loading, the effect of shear stress on the built-up of pore pressure may be differentiated and analyzed separately from that of the normal stress. The diagram of Σ(au - Δσ) versus Σ(Δσ) and Σ(au - Δσ) versus Σ(Δσ - Δσ) are plotted and their shapes are similar to those in Figure 5. The Skempton's (1954) and Henkel's (1960) pore pressure coefficients A = (au - Δσ)/(Δσ - Δσ) and Table IV - The pore pressure coefficients in soft clay.

<table>
<thead>
<tr>
<th>R/R</th>
<th>0.25</th>
<th>0.50</th>
<th>0.85</th>
<th>0.25</th>
<th>0.50</th>
<th>0.85</th>
<th>0.25</th>
<th>0.50</th>
<th>0.85</th>
</tr>
</thead>
<tbody>
<tr>
<td>z/R</td>
<td>(au/Δσ)₀</td>
<td>(au/Δσ)₁</td>
<td>(au/Δσ)₂</td>
<td>(au/Δσ)₀</td>
<td>(au/Δσ)₁</td>
<td>(au/Δσ)₂</td>
<td>(au/Δσ)₀</td>
<td>(au/Δσ)₁</td>
<td>(au/Δσ)₂</td>
</tr>
<tr>
<td>0.75</td>
<td>-</td>
<td>0.5</td>
<td>-</td>
<td>0.6</td>
<td>-</td>
<td>0.2</td>
<td>-</td>
<td>0.1</td>
<td>-</td>
</tr>
<tr>
<td>0.90</td>
<td>0.5</td>
<td>0.5</td>
<td>0.6</td>
<td>2.3</td>
<td>2.3</td>
<td>2.3</td>
<td>2.0</td>
<td>2.2</td>
<td>2.1</td>
</tr>
<tr>
<td>1.20</td>
<td>0.6</td>
<td>-</td>
<td>0.6</td>
<td>2.3</td>
<td>-</td>
<td>2.3</td>
<td>2.2</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Legend:

- --- Principal increments of stresses due to overburden (μ = 0.33).
  (Δσ)₀ = γz, (Δσ)₁ = k₀γz, θ₀ = 0
- --- Principal increments of stress due to weight of structure (μ = 0.33).
  (Δσ)₀ = 0.560, (Δσ)₁ = 0.120, θ₁ = 26
- --- Principal increments of stress due to preloading (μ = 0.5).
  (Δσ)₀ = 0.12 p, (Δσ)₁ = 0.12 p, θ₁ = 28.9

Fig. 6 - The principal increments of stresses and their directions at point U₁₀ under different loads.

The regularity of these coefficients is even better than the above mentioned coefficient B. Their mean values are as follows:

A₀ = 0.6, A₁ = 1.5, A₂ = 1.4

The regularity of the critical load, the magnitudes and directions of the principal increments of stress induced by the overburden by the weight of tank (including
Fig. 7 - The vectorial summation of principal increments of stress due to various loads by means of Mohr's circles.

its foundation) and by critical load applied during preloading are calculated (Figure 6). The vectorial sum of these stress increments is obtained graphically with Mohr's stress circles (Figure 7). The effective stress ratios \( \frac{\sigma' - \sigma_f}{\sigma'' - \sigma_f} \) thus obtained in various points corresponding to critical load are plotted in the effective shear strength diagram obtained by laboratory tests. It shows that these field data points all fall in the vicinity of the laboratory shear strength envelope (Figure 8). The shear stress induced by the critical load reached the effective shear strength, hence local shear failure must have taken place in the considered points.

CONCLUSION

The piezometer data shows that the excess pore pressure at a point increases linearly with the increasing surface load until a certain critical load is reached. Then the induced shear stress at that point reaches the shearing strength of the soil and local shear failure occurs. Henceforth, there is a pronounced increase in the observed rate of pore pressure built-up. Prior to local shear failure, the excess pore pressure is caused mainly by the increase of mean normal stress. The influence of increase of shear stress is a minor factor. After local shear failure, the influence of shear stress increases abruptly and the value of coefficient of pore pressure becomes several times as large as before. For instance, in the soft clay mentioned above, the coefficient A and \( \alpha \) increases to 2.5 to 4 times after local shear failure.

The in-situ effective shearing strength indicated by local shear failure is backfigured using the critical load and the corresponding piezometer data. These values agree fairly well with the laboratory test results.

REFERENCES


Henkel, D. J. (1960), "The Shear Strength of Saturated Remolded Clay", Proc. of Research Conf. on Shear Strength of Cohesive Soil, ASCE.


