Comparison between pore water pressures measured 'in-situ' and theoretical predictions

Comparaison de pression de l'eau interstitielle mesurée 'in-situ' avec des prévisions théoriques

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SYNOPSIS

The present paper gives an account of the acquired experience with field instrumentation concerning the measurement of excess pore water pressure relative to hydrostatic pore water pressure in a soft clay deposit by means of Casagrande-type open stand pipe, and Bishop-type closed double standpipe hydraulic piezometers, as well as a comparison with the excess pore water pressure predicted by the formulas given by Skempton and Henkel based upon special triaxial tests CIEL (Isotropically Consolidated – Lateral Extension) and CKeEL (Anisotropically Consolidated – Lateral Extension) type.

INTRODUCTION

This paper is part of a series of research projects on the determination of "in situ" undrained shear strength of the soft clay deposit, more specifically on the dessicated superficial crust which is covered with vegetation and therefore presenting difficulties on sampling, which occurs in variable thicknesses in Baixada Fluminense along Washington Luiz Highway (Rio-Petrópolis Highway).

The procedures of site works consisted basically in "in situ" measurements of excess pore water pressures generated by the mechanical excavation operations (dragline type) of several depths of the deposits after Stage I of the excavation sequence which has developed during approximately 5 hours reaching failure in the beginning of Stage IV.

Values of au (change in pore water pressure) were determined at the laboratory by means of special tests performed on samples obtained from the same depths where the piezometers were installed, using fixed piston and Shelby type samplers with nominal diameters of 50mm and 100mm.

The results obtained from the field investigations were compared with the changes in total pressures estimated by the Elastic Theory.

GEOLOGICAL AND GEOTECHNICAL ASPECTS OF THE STUDIED SITE

The soft clay deposit which is the object of these studies, is situated at km 7,5 of BR-040, Rio-Petrópolis Highway – Rio de Janeiro-Brazil, which according Antunes (1978) has its origins in fluvial and marine sediments (nearly 6000 years old) and may be classified as thiomorphic soils.

The borings executed for subsurface exploration, execution of field tests, undisturbed sampling, and SPT, presented the following geological-geotechnical profiles: a layer of sandy, silty, well-graded fill with mica and gravels, with thicknesses between 1,0 and 1,4m, over a layer of organic soft grey clay with thicknesses between 3,2 and 4,6m. Then follows a layer of clay with coarse to medium size sand and undernet it, a layer of sandy, hard, light grey clay. The depth reached by excavation involves, mainly, the two first layers.

The distribution of the natural moisture content (w), Atterberg limits (\(\gamma_L\), \(\gamma_H\)) and natural unit weight (\(\gamma_n\)) with depth is presented in Fig. 1.

The hydraulic conditions at the site, some months before the execution of excavation were hydrostatic, what can be seen in Fig. 2, indicating that the primary consolidation process under the weight of the fill executed in 1977 (2,5 years before the Experimental Excavation) had been completed. These conditions were confirmed later by the results of consolidation tests performed by Sayão (1980) which have yielded values of OCR (over consolidation ratio) between 1,0 and 1,3.

LABORATORY TESTS

A program of tests performed on undisturbed samples obtained by conventional Shelby tube and thin-walled samplers of fixed tube (Sayão, 1980) were executed. The program consisted of the following types of tests:

- standard consolidation
- UU triaxial tests with axial loading with time till failure of 5 min.
- CU triaxial tests with axial loading with time till failure of 5 min.
- CK0U triaxial lateral decrease of pressure with time till failure of 13 hours.
- CIU triaxial lateral decrease of pressure with time till failure of 30 hours.

The consolidated tests yielded the following average values \(C_v = 4 \times 10^{-4}\) cm²/s, \(m_v = 0,25\) a 0,40 cm²/kg, \(C_c = 1,80\), OCR = 0,8 a 1,20. The triaxial tests showed undrained shear strength within the range of 1,2 and 2,8 tf/m². As for the effective strength parameters, average values of \(c' = 0\) and \(\varphi' = 24\) were found.

DESCRIPTION OF THE EXECUTION OF THE EXPERIMENTAL EXCAVATION

The Experimental Excavation was executed by a Bucyrus-22B digger equipped with a clam-shell bucket in approximately 5 hours and 45 minutes, in 4 stages, with an interval between stages of approximately 30 minutes, so as to allow readings of all instruments. The execution followed the sequence shown on Fig. 3. The bench-mark was used as the reference point for the optical measurements at the initial levelling of all the instruments. The reference was fixed in a layer of hard sandy clay at a depth of 12m, where it can be considered as immovable. For greater rapidity and precision of the levellings, two...
which should occur during excavation would only mark so as to give precise information on the depth of excavation after each stage.

In order to control of operating of the Experimental Excavation, the total length to be excavated was divided in six sections. After each stage of excavation the bottom of each section was levelled in relation with the benchmark so as to give precise information on the depth of excavation after each stage.

RESULTS OBTAINED WITH THE INSTRUMENTATION RELATIVE TO PORE WATER PRESSURE

Water level measures - the lowering of water level caused by excavation was approximately 1.6 cm at the end of Stage I, 3.0 cm at the end of Stage II, 6.2 cm at the Stage III and 6.8 cm after failure. These values were calculated considering the water level as the average of the water levels inside the measures.

Casagrande piezometers – due to the existence of a canal very near the excavation site, an attempt was made to establish it the daily changes in the canal water level had any influence on the piezometers readings, by successive readings made from all piezometers throughout the day. After that, it was verified that the changes in the readings did not exceed the precision of the water level measures used (0.5 cm).

Although the experiments of the successive readings could not be considered conclusive on the actual influence of the changes in water level in the canal on the water level in the piezometers, they were very useful in the sense that they made it clear that any significant change which should occur during excavation would be due only to the excavation itself.

During the execution of the excavation, the piezometers recorded increasing decrements of the pore water pressure after each stage, as it can be seen in the following table.

<table>
<thead>
<tr>
<th>EXCAVATION SEQUENCE</th>
<th>PZ NO</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth (m)</td>
<td></td>
<td>1,670</td>
<td>3,200</td>
<td>4,136</td>
<td>1,020</td>
</tr>
<tr>
<td>After Stage I</td>
<td></td>
<td>-0,013</td>
<td>-0,002</td>
<td>-0,005</td>
<td>-0,002</td>
</tr>
<tr>
<td>After Stage II</td>
<td></td>
<td>-0,016</td>
<td>-0,005</td>
<td>-0,005</td>
<td>-0,001</td>
</tr>
<tr>
<td>After Stage III</td>
<td></td>
<td>-0,013</td>
<td>-0,005</td>
<td>-0,030</td>
<td>-0,022</td>
</tr>
<tr>
<td>After Failure</td>
<td></td>
<td>-0,121</td>
<td>-0,018</td>
<td>-0,017</td>
<td>-0,036</td>
</tr>
<tr>
<td>After Failure</td>
<td></td>
<td>-0,135</td>
<td>-0,014</td>
<td>-0,019</td>
<td>-0,042</td>
</tr>
</tbody>
</table>

PREDICTION OF EXCESSES OF PORE WATER PRESSURE

In order to make a reasonable prediction of the excess pore water pressures it is necessary to have a very correct knowledge of values of the increases and decreases of total stresses (Δσ). Once is known, the following step is to determine the porepressure parameter A, by means of laboratory tests which try to reproduce the stress conditions present in the field.

Due to practical difficulties in the measurement of total stresses on the foundation during loading or unloading stages, theoretical methods have been applied for their determination (elastic theory, numerical methods) using geotechnical parameters obtained from laboratory tests performed in good quality samples or field tests.

Change in total stresses in the sites were hydraulic piezometers were installed was estimated by elastic theory after Stage I of excavation. These estimate values are presented in Fig. 5.

As for the pore pressure parameter A, since triaxial consolidated undrained tests with pore pressure measurement were made, following several stress path on undisturbed samples, by Sayao (1980) (C1U, C1EL, C0EL) the results were used to discuss for soft Sarapul clay, the influence of anisotropy and stress path on the prediction of pore pressures.

Since the values of A and Δσ were known, pore water pressure change (Δσ) was calculated by Henkel’s and Bishop’s equations, care being taken that the results used would be the ones obtained for samples collected at the same depths as those of the hydraulic piezometers. The results for calculations of Δu values are presented in Fig. 6, together with values observed at the end of Stage I.

Quantification of the parcel of pore water pressures generated by shear in the triaxial tests performed.

This quantification can easily be determined by sample calculation of the total normal octahedral stress for each test. The difference between pore water pressures measured in the tests at failure (Δuf) and the change in octahedral normal stress (Δσ0c) yields the parcel of pore water pressure generated by shear (Δu shear) according to the following equation:

\[ Δu\ Shear = Δuf - Δu\ oct \]

The results of the calculations of Δu shear are presented in Fig. 7 and analysing that figure it can be verified that in C1U, C1EL and C0EL tests the parcel of pore water pressure due to shear was positive, with values during failure ranging from 0.25 tf/m² to 4.50 tf/m² and in C1EL and C1EA tests, this parcel was negative with values during failure ranging from -0.09 tf/m² to -0.78 tf/m².
tf/m² except for test which presented a positive pararel of 0.27 tf/m². Thus it can be observed that specimen and enlarge its breadth have generated positive values of \( \Delta u \) shear whereas tests which tend to increase the sample length and decrease its breadth have generated negative values of \( \Delta u \) shear.

CONCLUSIONS

From the analysis of the results presented throughout the paper, referring to excess pore water pressures generated by the execution of excavation it can be verified that in spite of a slight discrepancy between the predicted and observed values, the changes in pore water pressures predicted by Henkel's formula were very much nearer to the hydraulic piezometers than those predicted by Skempton's formula. This may indicate that Henkel's equation is more suitable to represent the various types of solicitation.

The main reasons for the discrepancy between values may be of various kinds, since there are many factors which may influence Skempton's pore water pressure parameters and, beginning by the fact that the soft clay studied was considered to be saturated in the calculations for the prediction of \( \Delta u \) (\( B = 1 \)), which was not confirmed during tests performed by Sayão (1980) where the values obtained were \( B = 0.98 \pm 0.01 \).

The information available was not enough as to make it possible to take into account, separately the influence of factors like: type of solicitation (axi - symmetrical and plane strain) and rotation of the direction of principal stresses axis.

Observing Fig. 6 it can be noted that tests with different stress paths were performed near depths of 2.00 and 3.00m (next to piezometers 3 and 7) and predicted. Values of \( B \) and \( A \), obtained were 0.98 ± 0.01, for all tests performed by Sayão (1980) where the values obtained were \( B = 0.98 \pm 0.01 \).

The information available was not enough as to make it possible to take into account, separately the influence of factors like: type of solicitation (axi - symmetrical and plane strain) and rotation of the direction of principal stresses axis.

Anisotropy and remoulding were two other factors which would be analysed in addition to the stress path based both on other author's experience, other types of clay and test results performed on Sarapuí soft clay (Sayão 1980 and Leite and Sayão 1981).

The similar values of \( A \) obtained from CI - EL and CKC - EL tests may be an indication that anisotropy is not a predominant factor in the difference between predicted and measured values of pore water pressure.

Effort was made in the sense of keeping remoulding influence to a minimum degree, since samples tested by Sayão (1980) and Leite and Sayão (1981) were trimmed to diameters much smaller than the diameters of Shelby tubes used for sampling. In addition, Shelby tubes were reamed with paraffin wax and wrapped in plastic bags, after each test and returned to the wet chamber.

Even if the tests performed could different stress conditions taking into account the rotation of principal stress axis the results of predicted \( \Delta u \) would still be different from values of \( \Delta u \) observed that is due to the influence of the stress level because the \( A \) value used for all the predictions was calculated for all tests from the excess in pore water pressure developed failure (\( \Delta u_p \)) whereas the increase in total stress (\( \Delta \sigma \)) used were calculated after the conclusion of Stage I of excavation, that is when the relief of loading corresponded just to a layer of 10m from the fill. The quantification of this aspect becomes unpeasible since, in that Stage, the mass of soil was certainly far from reaching failure conditions.

REFERENCES


ACKNOWLEDGMENTS

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**Fig. 3** - Execution Sequence of Experimental Excavation.

**Fig. 4** - Location of Fissures Caused by Failure of Excavation.

**Fig. 5** - Calculations of Values of $A_r$ in the Sites of Hydraulic Piezometers by Elastic Theory.

**Fig. 6** - Comparison Between Predicted and Observed Values of AU in the Hydraulic Piezometers at the End of Stage I.

**Fig. 7** - Qualification of the Parcel of Pore Water Pressure Generated by Shear.