FRICITION CAPACITY OF AXIALLY LOADED MODEL PILE IN SAND

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ABSTRACT

This paper presents an experimental program to investigate the effect of radial strain on the friction capacity of an instrumented model pipe pile.

The model was buried in a dry INAGI sand. Loads were then applied in four different ways with and without applied surcharge pressures, one compression (push-down) and three tensions (pull-out). The first pull-out load was applied at the top of the model, the second was applied at the tip and the last was applied at the tip for an axially prestressed condition.

The use of these different types of loading had a significant effect on the friction capacity, that is, the friction capacity decreases in the following order; push-down, pull-out under prestress, pull-out at the tip and pull-out at the top.

These decreases were interpreted to be the results of radial strain and due to the differences in arching in the sand.

Key words: load test, model test, pile, radial strain, sand, uplift (IGC: E4)

INTRODUCTION

There is very little data available on shaft friction for tensile “pull-out” loading, and the available data is to some extent conflicting. Tests reported by Ireland (1957) for example on piles driven into fine sand suggested that the average skin friction values ($f_s$) for uplift loading from the top were equal to those for vertical (compression) ($f_c$), however, data summarized by Sowa (1970) and Downs and Chieuretti (1966) indicated considerable variations in the average shaft friction between different tests, although there is a tendency for the friction values to be lower for uplift loading than for vertical (downward) loading especially for cast-in-situ piles. The same tendency was reported in most of the model pile tests performed in sand, such as by Rao and Venkatesh (1985), Levacher and Sieffert (1984), Baghdadi et al. (1991). The average ($f_s/f_c$) ratio was approximately 50%.

The amount of reduction in shaft friction capacity was attributed to some parameters and phenomena which are related to the soil, the pile type or the method of installation. These parameters presumably vary between the uplift process and the vertical (down-loading) process.

Investigation of the possible effect of Poisson’s ratio in particular has been mentioned by many researchers. In an elastic analysis, ensuring compatibility of displacement in the vertical and radial direction, in order to study the working load response of an axially loaded elastic pile embedded in a linearly non-homogeneous incompressible elastic (Gibson) soil, Rajapakse (1990) found that the influence of radial compatibility on the shaft friction resistance does not exceed 6% of the estimated total shaft friction resistance. In a discussion given by Fleming et al. (1985) for silicious soils, they explained that this reduction was the result of lower lateral stress in the soil due to the tendency of the pile to contract radially under tensile load.

On the contrary, Kulhawy et al. (1983) found these effects to be negligible. Examination of available load test data by Stas and Kulhawy (1984) has not shown any discernible difference. The same conclusion was made by Hirayama (1993).

Other phenomena which may contribute to the reduction of the friction capacity, during the uplift process, is the absence of arching action in the soil. The arching action phenomenon was considered by Vesic (1967) and Kerisel (1961) in relation to shaft and base resistance, by Terzaghi (1943) in relation to tunnels and by Ono and Yamada (1993) in relation to stability of trapdoors.

We have not been able to study some papers in order to

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Manuscript was received for review on March 7, 1994.

Written discussions on this paper should be submitted before October 1, 1995 to the Japanese Society of Soil Mechanics and Foundation Engineering, Sugayama Bldg. 4 F, Kanda Awaji-cho 2-23, Chiyoda-ku, Tokyo 101, Japan. Upon request the closing date may be extended one month.
determine a quantitative evaluation concerning the effect of Poisson’s ratio based on experimental investigation.

In this paper, a new pipe pile model for which an uplift load could be applied from the tip was designed to quantify the effect of Poisson's ratio on the shaft friction resistance. According to the test results, it was concluded that about 30% of the 50% reduction in frictional capacity between down-loading tests and uplift-loading tests (applied at the top of the model) was a result of the effects of radial strain and the remainder was the result of the effect of another mechanism such as arching which differs between downward and upward loading.

EXPERIMENTAL PROGRAM

Model Description

The test model (Fig. 1) is an acrylic tube 25 mm in diameter with Young's modulus $E_p$ of $2.7 \times 10^4$ kPa, Poisson's ratio $\nu_p$ of 0.38 and a wall thickness of 2 mm. A steel cone is bonded to the end of the tube. The model top is fitted with a force transducer and strain gauges are mounted on both sides of the pipe, in the axial direction (L1 – L6) and circumferential direction (P1 – P3). In a cylindrical coordinate $(r, \theta, z)$, (L1 – L6) give the values of $\varepsilon_z$ and (P1 – P3) give the values of $\varepsilon_\theta$ which is equal to $\varepsilon_r = -\nu_p \varepsilon_z$ in the case of a thin wall pipe. For all the tests, the model was buried in the soil to an embedded depth of 400 mm so that the center of the uppermost gauges correspond to the sand surface.

Laboratory Soil

The soil used in the test model is a clean, poorly graded INAGI sand which is sieved to 840 $\mu$m. The grain size distribution curve is presented in Fig. 2. The maximum and minimum densities are 1.755 Mg/m$^3$ and 1.435 Mg/m$^3$ respectively. The angle of internal friction obtained from direct shear tests is $\phi = 42.1^\circ$ with a relative density of 85% used as the testing condition. The sand was maintained at a water content of less than 0.2%. The angle of friction between the sand and model shaft is $\phi_f = 29.4^\circ$, obtained by direct shear tests between acryl and sand. In direct shear tests, an acrylic plate, made of the same material as the model pipe, was roughened to a similar condition to the model shaft and then placed in the lower part of the direct shear box. The soil was poured in the upper part, then the lower box was moved under different normal stress at a displacement rate of 0.5 mm/min.

![Fig. 1. Model design, and the location of strain gauges (Unit: mm)](image)

Figure 3 presents a plot of shear stress versus normal stress.

Experimental Setup

All the tests were carried out in the laboratory. A schematic of the model pile test setup is shown in Fig. 4. It is composed of a rectangular container (600 $\times$ 400 mm) with a depth of 600 mm. The load is applied with a threaded rod. In the case of tests with surcharge, the surcharge was applied to the sand surface by means of pressurized air bag. The strain data acquisition was obtained using a strain meter and stored on floppy disk for further study.

The model was placed in the center of the container and then sand was poured into it to a relative density of 85%.

Testing Methodology

The same model pile was used in the course of the test-
Table 1. Different type of loading

<table>
<thead>
<tr>
<th>Type</th>
<th>Loading Type</th>
<th>Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a)</td>
<td>Down-load test</td>
<td>Applied at the top of the model</td>
</tr>
<tr>
<td>(b)</td>
<td>Top uplift test</td>
<td>Applied at the top of the model</td>
</tr>
<tr>
<td>(c)</td>
<td>Tip uplift test</td>
<td>Applied at the tip of the model by a steel rod inside the model</td>
</tr>
<tr>
<td>(d)</td>
<td>Tip uplift test</td>
<td>Applied prestress to the model; prior to the loading and after application of surcharge pressure, the model was prestressed by a screw to ensure, throughout the loading process, the inverted pattern of radial strain mobilized in tests type (a) with approximately the same maximum and minimum values of strain (cf. Fig. 5a and Fig. 5d)</td>
</tr>
</tbody>
</table>

Behavior. Surcharge values of 49, 98, 147 and 196 kPa were used. The model in all the cases was pushed-down (or pulled out) at a constant top displacement rate of 0.5 mm/min. The values of all strain gauges were calibrated initially just before loads were applied except for load type (d) in which the strain gauges were calibrated initially before the model was prestressed. The types of loads and the strain distribution along the model are illustrated in Fig. 5. Type (a) and type (b) respectively represent conventional down-loading and uplift-loading tests. In type (c) the load was transmitted to the tip of the model through a steel rod mounted inside the pipe model pile. The top of the model was free of any stress. In test type (d) the steel rod was connected to the tip and screwed from the top until a certain amount of uniform radial strain was mobilized in the model. The different types of loading are summarized in Table 1.

EXPERIMENTAL RESULTS

Loads Transfer in Model

Figures 6(a) and 6(b) represent an example of axial load transfer with depth, where the nondimensional parameter means the top settlement divided by model diameter ($S/D$). In Fig. 6(a), the load and top displacements were positive for down-load tests and negative for uplift tests.

In tests type (c) and type (d) the largest portion of the loads was carried by the lowest part of the model which can be explained by the large amount of radial strain mobilized near the model tip due to the application of load at that point.

Total Shaft Friction Resistance

The total shaft friction resistance for down-loads tests was estimated as the difference between total load and tip resistance. Figures 7(a) to 7(d) present the curves of total shaft friction related to the top nondimensional displacement for surcharges of 49, 98, 147, and 196 kPa. It can be seen, that the mobilization of ultimate total shaft friction resistance requires a pile movement between 5 and 15% of the model diameter.
Fig. 6(a). Down- and uplift-load transfer for an applied surcharge of 98 kPa

Fig. 6(b). Tip uplift-load transfer along the model for a surcharge pressure of 98 kPa

Fig. 7. Total shaft friction resistance versus top nondimensional settlement for the different types of loading and surcharge pressures.
Table 2 shows a comparative summary of the results from Fig. 7 in which the ultimate total shaft frictions \( F_n \), \( F_r \) and \( F_d \), corresponding to load types (b), (c) and (d) respectively, were divided by the ultimate total shaft friction \( F_e \) corresponding to load type (a). It shows a reduction of the ultimate total friction resistance for the uplift test type (b) compared to that of the down-load test type (a). The average ratio \( F_e/F_n \) for the different cases of surcharge pressures is 0.48. This is in agreement with results reported in most of the model pile tests, some examples of which were mentioned in the introduction of this paper.

By applying the axial uplift-load at the tip (test type (c)), the model diameter expanded radially resulting in an average recovery of about 10% of the ultimate total friction capacity mobilized in load type (a). The ratio of total friction for test type (d) to the total friction for test type (a) is approximately 0.80. Therefore, by applying to the model the same inverted pattern of radial strain as mobilized in test type (a), about 30% of the ultimate shaft friction resistance was recovered. This means that 30% of the 50% reduction between the ultimate total shaft friction in down-load type (a) and that of uplift-load type (b) results from the difference in radial strain distribution. In test types (a) and (d), the opposite direction of loading, down-load for test type (a) and uplift-load for test type (d), could mobilize other phenomenon such as arching in the sand which is absent throughout the uplift process.

For the down-load tests, Fig. 8 indicates the variation of tip and shaft friction resistance with surcharge pressures at the non-dimensional settlement of 10%. Without applied surcharge pressure, the tip resistance is slightly higher than the shaft resistance. As the surcharge increases however, the contribution from the shaft friction becomes larger than the tip resistance. As stated above, the high surcharge pressures in the soil were assumed to simulate the field stresses at a greater depth. A long pile presenting the same characteristics should behave as a friction pile. In these model tests, however, the tip resistance remains constant for a surcharge pressure larger than 98 kPa, which may be explained by the presence of the smooth sharp cone mounted at the tip of the model and the remolded sand (under the tip) due to the method of installation.

**Unit Shaft Friction Resistance**

The unit shaft friction resistance \( f_s \) was estimated using the following equation and the experimental results shown in Figs. 6(a) and (b):

\[
f_s = \frac{1}{U} \frac{\Delta N}{\Delta z}
\]

where \( \Delta N \) is the increment of axial force, \( \Delta z \) is depth between two cross sections and \( U \) is the perimeter of the model cross section.

Figure 9 represents an example of unit shaft friction distribution varying with depth for the case with a surcharge pressure of 98 kPa.

For test type (a) the unit shaft friction decreases linearly from the top to the tip of model. Whereas in test type (c) the unit shaft friction resistance decreases hyperbolically from the tip to the top of model, which determines

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**Table 2. Ratio of total ultimate shaft friction resistance to the shaft friction resistance of the down-load tests for different surcharge pressures**

<table>
<thead>
<tr>
<th>Surcharge (kPa)</th>
<th>49</th>
<th>98</th>
<th>147</th>
<th>196</th>
<th>Average Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>( F_n/F_e )</td>
<td>0.49</td>
<td>0.50</td>
<td>0.47</td>
<td>0.44</td>
<td>0.48</td>
</tr>
<tr>
<td>( F_r/F_e )</td>
<td>0.65</td>
<td>0.61</td>
<td>0.58</td>
<td>0.54</td>
<td>0.60</td>
</tr>
<tr>
<td>( F_d/F_e )</td>
<td>0.79</td>
<td>0.86</td>
<td>0.71</td>
<td>0.83</td>
<td>0.80</td>
</tr>
</tbody>
</table>

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**Fig. 8. Distribution of ultimate load components for test type (a)**

**Fig. 9. Distribution of \( (f_s/\sigma_v) \) with depth**
a unit shaft friction for test type (c) at a depth of 320 mm slightly higher than that for test type (a) at a depth of 80 mm. These two cross sections are located at the same distance from the application point of the corresponding type of loading and they are not affected by the boundary conditions. The total shaft friction resistance in the test type (a) however is larger than that of test type (c) because the integrated area of unit shaft friction of Fig. 9 of the former type of loading is larger than the latter.

The basic concept of unit shaft resistance is explained quantitatively as follows:

\[ f_s = \mu \sigma_v = \mu K \sigma_v \]  

(2)

where \( \mu \) is frictional coefficient, \( \sigma_v \), \( \sigma_r \) are the vertical and lateral earth pressures respectively and \( K \) is the coefficient of lateral earth pressure.

Dividing both sides of Eq. (2) by \( \sigma_v \), we have

\[ f_s / \sigma_v = \mu K \]  

(3)

In Eq. (3) we may consider that \( \mu \) is a function of tangential relative displacement \( d \) of pile-soil interface and \( K \) is a function of normal displacement of the pile-soil interface. The normal displacement may be substituted with radial strain \( \varepsilon_r \) of the pile.

Therefore, as the first step, the values of unit shaft resistance divided by the vertical earth pressure \( f_s / \sigma_v \) are plotted versus corresponding relative vertical displacement \( d/D \) between sand and model shaft expressed as a percentage of model diameter, as shown in Fig. 10. The unit shaft friction resistances were computed for every type of loading at the same distance from the point of application of load which imply a depth of 80 mm for test types (a) and (b) and 320 mm of depth for test types (c) and (d). Figures 10(a) to 10(d) show that \( f_s / \sigma_v \) increases as \( d/D \) increases and remains constant for a relative displacement larger than 5% of model diameter. It also indicates that test type (d) gives the highest values of \( f_s / \sigma_v \) and test type (b) gives the lowest.

From these figures, it can also be clearly seen that the relationship between \( f_s / \sigma_v \) and relative displacements changes considerably according to the type of tests. This implies that there are factors other than the vertical earth pressure and relative displacement such as radial strain that affect the unit shaft friction resistance.

**Effect of Radial Strain**

As was stated before, 3 pairs of strain gauges were mounted at depths of 80, 160 and 320 mm for the measurement of radial strain. For a second step, as shown in Fig. 11, \( f_s / \sigma_v \) was plotted versus the radial strain at the depth corresponding to those used in Fig. 10. For each type of loading, the values of \( f_s / \sigma_v \) and \( \varepsilon_r \) correspond to the relative displacement \( d/D \) of 5%, 10% and 15%.

These figures indicate that the frictional ratio \( f_s / \sigma_v \) increases linearly with the radial strain of the model-soil interface with references to load types (a), (b) and (d),

![Fig. 10a](image1.png)  
![Fig. 10b](image2.png)  
![Fig. 10c](image3.png)  
![Fig. 10d](image4.png)

**Fig. 10.** Plot of \( (f_s / \sigma_v) \) vs relative displacement at a depth of 80 mm for load types (a and b) and 320 mm for load types (c and d)
while for load type (c), $f_r/\sigma_v$ increases with a slight decrease in radial strain. The latest phenomenon can be attributed to the testing methodology. Prior to the loading process and after application of surcharge pressure, some amount of strain is mobilized in the model due to the settlement of the sand. The distribution of the absolute values of the axial load was observed as the radial strain increases from zero at the surface to a maximum value at a depth of 320 mm and decreases as they reach the model tip. When the loading process takes place the strain distribution decreases from a new maximum located at the tip to zero strain at the top and the value of strain at 320 mm represents a slight reduction compared to the maximum mobilized by surcharge pressure. The analysis of Fig. 6(b) shows that the unit friction defined in Eq. (1) and represented as the slope of the tangent line of the curve of load-transfer at a depth of 320 mm increases as the load capacity increases.

It was also observed that the interpolation of Fig. 11 of $f_r/\sigma_v$ and radial strain $\varepsilon_r$ follows the Rankine active and passive condition. For a radial strain $\varepsilon_r$ equal to zero, the value of $K$ approximates the value of $K_0=0.33$ given by Jaky’s equation. From the same figures, it can be seen that the value of $f_r/\sigma_v$ corresponding to $\varepsilon_r=0$ is about 0.2 for all the cases. Replacing in Eq. (3) $K$ with its corresponding value $K_0$ and $f_r/\sigma_v$ by the value corresponding to $\varepsilon_r=0$, for a model displacement larger than 5%, $\mu$ is estimated to be 0.60, which is slightly higher than 0.56=\tan \phi_v=f_r/K \sigma_v$ obtained experimentally from direct shear tests between acrylic and sand.

**CONCLUSIONS**

1. The total shaft friction resistances during uplift tests with respect to down-load tests are 50% for test type (b), 60% for test type (c) and 80% for test type (d).

2. For these model tests, the effect of radial strain of the model on the reduction of the total shaft friction resistance between down-load and uplift-load tests does not exceed 30% of the total shaft friction mobilized in down-load tests. This effect is presented as the difference between the load type (b) and the load type (d). The remaining 20% may be attributed to other phenomena such as arching in sand.

3. For these model tests, the mobilization of the ultimate total shaft friction resistance requires a pile movement between 5 and 15% of the model diameter.

4. The values of unit shaft friction resistance divided by the corresponding vertical stress $f_r/\sigma_v$ remains constant for a relative displacement larger than 5% of the model diameter.

5. The unit shaft friction divided by vertical stress $f_r/\sigma_v$ is related uniquely to the radial strain without any reference to the type of loading and a value of relative displacement larger than 5% of the model diameter.
REFERENCES


