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BEARING CAPACITY OF TAPER PILES AT MAADI SATELLITE PROJECT, CAIRO

BY

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ABSTRACT:

One of the largest pile driving jobs on which high capacity Raymond piles have been used in Egypt, the Maadi Satellite project, Maadi, Cairo. More than 4000 piles of step taper Raymond piles were driven by Mier-Raymond company, to support high rise towers.

This paper analyzes the methods of prediction of load capacity compared to full scale tests and gives a useful recommendation.

INTRODUCTION:

The basic problem of computation of load capacity of a deep foundation can be formulated as follows: A cylindrical shaft is placed by some means to a depth inside a soil mass of known physical properties. A static, vertical, central load is applied at the top and increased until a shear failure in the soil is produced. To be determined is the ultimate load which this foundation can support.

If the surrounding soil is cohesionless silt or sand, pile driving may cause soil densification, which is most pronounced in the immediate vicinity of pile shaft and extends in gradually diminishing intensity over a zone extending between one to two pile diameters around the pile shaft. The driving process is also accompanied by increase in horizontal ground stress, and change in vertical stress in pile vicinity. In very dense cohesionless soils, such as sand or gravel, loosening may take place in some zones, along with substantial grain crushing and densification in the immediate vicinity of the pile. (According to Karwai (1962) some of the test piles in dense sand were excavated and pulled out with a hull of highly densified, crushed material that resembled a fine-grained sandstone). In such soils there are permanent changes in horizontal as well as in vertical ground stress that can be very pronounced. Hard driving can leave large residual stresses in both the pile and the soil, consideration of which may be essential for understanding the behaviour of the pile-soil system. As the piles are often made in groups the picture is further complicated by the complex and not always well understood effect of placing of adjacent piles. For these and other reasons the problem under consideration poses difficulties unparalleled in other common soil mechanics problems. A general solution of the problem is not yet available and will be hard to formulate and solve.

Soil Conditions:

The location of Maadi Satellite Project is about 10 km south Cairo, along the Nile river. At that site the soil con-
sists of medium clayey silt from the ground surface down to a depth of 1.5 m followed by loess, fine to medium sand to 14 m from surface then dense medium to coarse sand up to the end of borings 30 m from surface, ground water table was located about 1.5 m from surface. Fig. 1 shows the general soil profile.

Bearing Capacity Analysis by Wave Equation:

A thorough review of analytical solutions to the one-dimensional wave equation and their applications to impact pile driving problems is given elsewhere (2).

The general wave equation for a pile surrounded by a medium providing side and end resistance to the acceleration:

$$\frac{\partial^2 u}{\partial t^2} - \frac{E}{\rho} \frac{\partial^2 u}{\partial y^2} + R = 0$$

Initial conditions at $t = t_0$:

$u(x,t) = u_0(x)$

$\frac{\partial u}{\partial t}(x,t) = v_0(x)$, $R(x,t) = R_0(x)$

Boundary conditions:

$u(x,t)$ or $\frac{\partial u}{\partial x}(x,t)$ at $x = 0$

$u(x,t)$ or $\frac{\partial u}{\partial x}(x,t)$ at $x = L$.

The exact solution of such a system of equations is available only for a few special cases.

The development of high-speed digital computers made solutions of large systems easily accomplished.

The greatest success in wave equation applications may be gained in the selection of the most effective driving system.

The major uncertainties that have been found are in usage of the method to determine pile bearing capacities and the related question of whether or not penetration is possible under very hard driving. The inaccuracies found in applying the wave equation solutions are generally attributed to the inadequacy of the assumed rheological model of soil resistance. The nature of pile-soil interaction behaviour is extremely complex. As the pile penetrates, discontinuous shear deformations develop at the pile tip and along the shaft. Simultaneously the material beneath the tip is compressed and displaced as the region adjacent to the tip is sheared and deformed in extension. As the pile penetrates further the soil along the shaft is severely sheared. Soil exhibit an extremely complex deformation and failure behaviour in the laboratory under well-controlled stress deformation conditions. Soil behaviour may be a function of stress/deformation history,
stress level, stress path, and (particularly important for impact driving behavior) deformation rate. The development and dissipation of excess pore fluid pressures affects the effective stress, deformation and failure response of the soil. It is essential, therefore, to recognize that the determination of representative soil parameters as input to a pile driving analysis is truly a crude exercise of engineering judgement.

Soil disturbance due to pile installation may drastically affect the resistance to penetration during driving and the static behavior, on well. For sensitive clays the remolded strength may be substantially less than undisturbed strength such that a driven pile may show very little resistance during driving or immediately after driving. A strength regain and, therefore, an increase pile capacity may develop as reconsolidation (excess pore pressure dissipation) of the disturbed soil proceeds with time. Strength regain may continue in some sensitive cohesive soils over several years as a result of consolidation and thixotropic effects. This strength regain in commonly described as pile set-up or freeze. Dense deposits of fine cohesionless soils may develop negative pore pressures during pile driving, giving high transient strengths.

A pile load test or subsequent redriving after excess pore pressures dissipate often reveals a much lower resistance to penetration than the initial driving resistance would indicate.

The development of high resistance during high rate of deformation loading that do not prevail under static loading condition is commonly termed "relaxation". In many cases the actual soil resistance to penetration during driving bears little resemblance to the resistance observed during a load test performed after transient phenomena have passed.

Bearing Capacity For Soil Data:

The ultimate capacity \( P_u \) of the pile consists of skin friction \( P_s \) and point bearing \( P_p \).

Consequently:

\[ P_u = P_s + P_p \]

For calculation purposes it is generally assumed that the skin friction resistance and the point resistance can be determined separately and that these two factors do not affect each other. Test results reported by Cambefort (1955), Kazdi (1957) and Stuart, Hanne and Naylor (1960) show however that the skin friction resistance affects the point resistance for piles which have been driven through cohesionless soils. However this influence is in most cases small and can be neglected. Very small axial deformations are generally necessary to mobilize completely the skin friction resistance along a pile as observed, among others, by Muller (1959), Schenck (1951),

In contrast relatively large deformations are required to mobilize the maximum point resistance of piles which are driven into cohesionless soils. Therefore the largest part of the applied loads while at high load levels the largest part is carried by point resistance (Mansur & Kaufman, 1968, Mohan, Jain & Kumar, 1963).

Skin resistance:

The skin resistance has been evaluated, since the turn of this century, similar to the resistance to sliding of a rigid body in contact with soil. For sand this implies an assumption that should be proportional to the average overburden pressure along the skin.

\[ \sigma_s = A_s \cdot K_s \cdot \tan \phi \cdot q_b \]

\( \sigma_s \) = friction angle pile/soil
\( q_b \) = overburden pressure.

In which \( K_s \) is dimensionless factor represents the ratio of effective horizontal to overburden stress, and ranging between active to passive pressure.

Point resistance:

It has been suggested by Caquot (1934) and Buisman (1935) that the point resistance of piles in sand should be proportional to initial overburden pressure at the level of the pile point:

\[ P_p = A_p \cdot q \cdot N_q \]

\( A_p \) = area of pile point
\( q \) = overburden at tip.

In which \( N_q \) represents the bearing capacity factor for a deep foundation. Numerous theoretical and semi-empirical curves for \( N_q \) as a unique function of the angle of internal friction \( \phi \) have been proposed.

Bearing Capacity Analysis by Nordlund Equation:

Nordlund (1963) has developed an empirical method which takes into account the volume of soil displaced by the pile, the material of the pile, and the shape of the pile. This equation not valid for a pile for which jetting or predrilling as in this study.

\[ P_u = N_q A_p D + \sum_{d=D}^{d=0} K_s p_d \sin \phi \cdot C_d \cdot d \]
in which \( P_u \) = ultimate bearing capacity, \( N_q \) = dimensionless factor
\( A \) = Area of pile point, \( P_0 \) = overburden pressure to depth \( D \)
\( K_0 \) = dimensionless factor, \( \phi \) = friction angle pile/soil
\( C \) = perimeter encompassing the pile.

Load Tests:
A serial of load tests (15 tests) were conducted; lengths of piles from 11 m. to 21 m. driven to refusal according to wave equation some of load tests were done on redriven and reset piles. All tests were done according to ASTM designation.

Ultimate Load Criterion:
Various ultimate load criteria, all empirical in nature, have been proposed and used by different researchers and design organizations (see, for example Cheilla, 1964); a survey is presented in Table 2.

The author suggested to use criterion lb in the following form: Unless the load-settlement curve of a pile shows a definite peak load, the ultimate load is defined as the load causing total settlement of the pile point equal to 10\% of the point diameter.

Computation of Ultimate Loads:
Ultimate loads of all tests were computed by all methods except Norlund and listed in Table 1.

Interpretation of Test Results and Conclusions:

By using a safety factor 3 to get the working load from soil data and a safety factor 1.5 to get the working load from load test, and comparing all results with the wave equation load, it observed that the working load from soil data is conservative in general, and the results of wave equation are nearer to observed.

A considerable amount of research including well instrumented observation on full size tests are needed for safer and more economical design in future.
REFERENCES:

   Refer to IRMS 1966 "Methods of calculating the bearing capacity of piles" Sola Soils No. 10-19, 1966.
<table>
<thead>
<tr>
<th>Test No.</th>
<th>Wave equ. working load</th>
<th>Ultimate load from soil data</th>
<th>Observed ultimate load from soil data</th>
<th>Working load</th>
<th>Observed working load</th>
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<td>2</td>
<td>60</td>
<td>192</td>
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### General Soil Profile

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<tr>
<th>Depth (ft)</th>
<th>Strata</th>
<th>Description</th>
<th>Water Table</th>
<th>Remarks</th>
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<td>8</td>
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<td>Dark gray med. clayey silt.</td>
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<tr>
<td>8</td>
<td>8</td>
<td>Brownish gray loose, fine to med. sand.</td>
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<td>6</td>
<td>10</td>
<td>Grayish br. loose, f. to med. sand.</td>
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<tr>
<td>9</td>
<td>10</td>
<td>Br. gray loose, med. to coarse sand, tr. of f. gravel.</td>
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<td></td>
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<tr>
<td>8</td>
<td>12</td>
<td>Grayish br. med. to c. sand.</td>
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<td></td>
</tr>
<tr>
<td>13</td>
<td>14</td>
<td>Gray med. to f. sand.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>15</td>
<td>Gr. br. med. to c. sand.</td>
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<td>16</td>
<td>Br. gr. med. to c. sand.</td>
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<tr>
<td>17</td>
<td>18</td>
<td>Br. gr. dense, med. to c. sand.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
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LOAD (Metric Tons)

20  40  60  80  100  120

TEST NO. 2
L = 13.5 m

DEFORMATION (mm)

2  4  6  8  10

TEST NO. 3
L = 17 m

LOAD (Metric Tons)

20  40  60  80  100  120

TEST NO. 4
L = 15 m

DEFORMATION (mm)

2  4  6  8  10  12