INTERPRETATION OF LOAD TESTS ON PILES IN ROCK

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SYNOPSIS

The technique of load-testing piles in rock with an embedded jack and tell-tales is described. Some difficulties which arise in the analysis of the tests are: a. The fact that the displacements are not measured at the point of application of the load, b. Due to the small displacements, the resulting load-settlement curves are often irregualr and c. Because of high capacity, many tests are discontinued before failure is reached. The proposed method of analysis, which is based on both the hyperbolic approximation and a Winkler-type spring model, overcomes the above difficulties and may serve as a design tool.

INTRODUCTION

During the last decade, piling in rock has been increasingly utilized in Israel. The piles are drilled using a pneumatic percussion drill, with typical diameters 300 to 600 mm and depths up to 20 m. Piles are designed for skin friction only, end-bearing being disregarded because of the small diameter and the inability to clean the bottom. Allowable skin friction values are mainly based on local experience, which, provides no clue as to the expected settlemets. To obtain these, a method of load-testing the piles with an embedded jack was developed. This paper describes the test procedure, shows how its results may be analysed and discusses the results of a large number of tests.

EQUIPMENT AND PROCEDURE

The concept of the test is somewhat similar to that reported by Gibson and Devenny (1973), the main difference being that a certain length of pile is also cast under the jack (Fig. 1). The downward displacements of this portion are measured via steel-rod tell-tales, attached to a steel plate at the bottom of the pile and passing thorough the jack. Thus, two load tests are performed simultaneously: One upwards and the other downwards. A calibrated piston-type jack is normally preferred, after flat jacks were found to have insufficient stroke and inadequate accuracy. The load is applied in 6 to 10 stages, procedure closely following ASTM Standard D-1143.

LOAD-TEST ANALYSIS

In the interpretation of the tests, two different methods were employed: One phenomenological, using the hyperbolic approximation, and the other fundamental, based on elastic properties: The following sections describe these approaches:

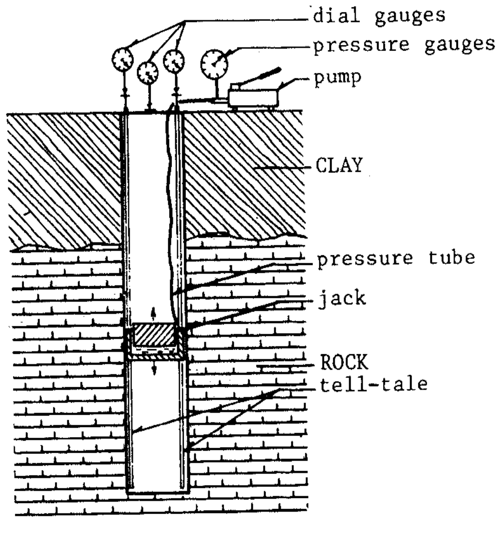

Fig. 1: Load-testing with an embedded piston

The hyperbolic approximation

Rocks are basically a work-softening material, so that a non-linear load-settlement bahavior is to be expected. This can be expressed by a hyperbolic approximation (Kondner 1962):

$$s = \frac{P}{A + B.s} \quad (1)$$

where A and B are constants, s is the displacement and P the load. For any load-test, A and B can be evaluated by a linear regression of s/P versus s (Fig. 2), taking the intercept and the slope of the resulting straight line, respectively. Both parameters also have a physical significance: 1/A is the tangent spring modulus, and 1/B equals the failure load. Thus,, once A and B are known, the complete non-linear behavior of the pile is determined. As this method is not based on fundamental properties of the pile-rock system, the resulting parameters are inapplicable to piles of other lengths or diameters.

The Winkler-Type Model

Assuming a linear relationship between skin friction and displacement (a Winkler-type model), the displacement s at any depth z may be expressed by the following equation (Scott 1981):

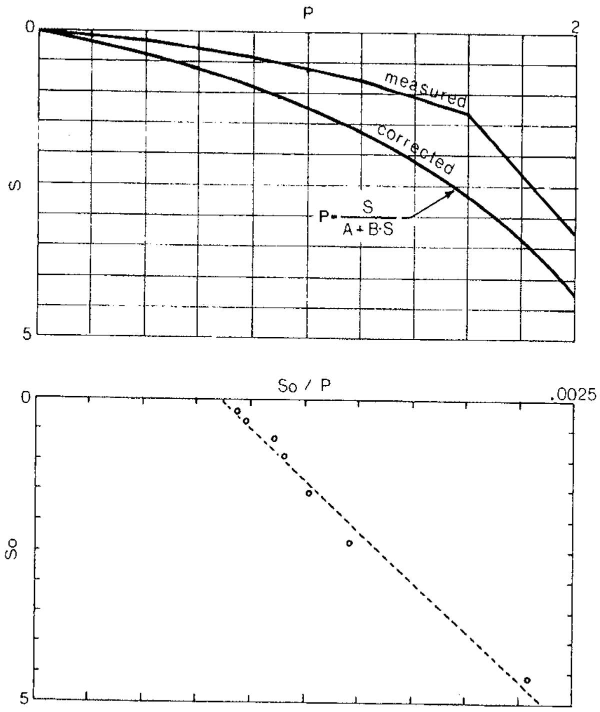

Fig. 2: Hyperbolic approximation for load-settlement curves

$$\frac{\partial^2 s}{\partial z^2} - \lambda^2 s = 0 \qquad (2)$$

the general solution of whch is:

$$s = C_1 e^{-\lambda z} + C_2 e^{\lambda z} \qquad (3)$$

where:

$$\lambda = 2\sqrt{\frac{k}{E_p D}} \qquad (4)$$

and k = spring constant
E_p = Young's Modulus of the pile
D = pile diameter

The axial force at any depth is given by:

$$F = -E_p A \, (\partial s/\partial z) \qquad (5)$$

in which A is the area of the pile cross-section. The load test as described above differs from conventional pile tests in that the load is applied at one end of the pile, while the displacements are measured at its other end. Substituing the appropriate boundary conditions (the load P at z = o and a zero force at the free end z = l), one gets the following solution for the displacement at the free end:

$$s = \frac{P}{\lambda E_p A} \frac{1}{\sinh(\lambda l)} = \frac{\varepsilon}{\lambda l \sinh(\lambda l)} \qquad (6)$$

where ε is the elastic shortening of a free-standing column with identical midensions.

As λ is the only unknown term in (6), it can be readily calculated using a programmble calculator, or found graphicaly from Fig. 3. The unknown displacement at the loaded end z = o is further given by:

$$s_0 = \frac{\varepsilon}{\lambda l \tanh(\lambda l)} \qquad (7)$$

To adjust the measured displacements one should, therefore, multiply them by the ratio:

$$\frac{s_0}{s} = \cosh(\lambda l) \qquad (8)$$

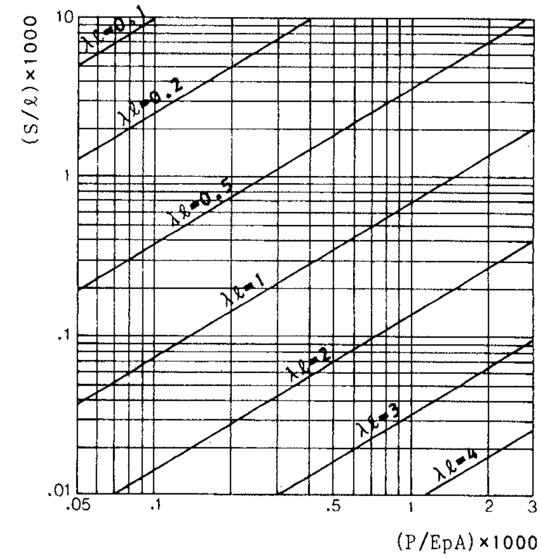

Fig. 3: λl as a function of load and deflection

For relatively rigid systems, where l is more than 3, the ratio s_0/s exceeds 10. Using the test results without a suitable correction may thus introduce large, unconservative Errors. Once the value of λ is established for a given site, the behavior of piles of different diameters or lengths can be predicted.

LOAD-TEST RESULTS

Available data

Altogether, the results of 51 tests have been studied (Table 1). Most of these were on 300 mm piles.

TABLE 1: Details of Load-Tests

Rock Type	Direction of Testing Up	Down	No. of Sites
Chalk & marl	11	9	6
Limestone	7	2	4
Chert	6	4	3
Dolomite (0 600 mm)	4	-	1
Cemented boulders & conglomerates	5	3	3

Two difficulties which became immediately apparent were:

a. In certain tests, the measured settlements were in the order of accuracy of the measuring system (a few hundredths of a millimeter), giving the load-settlement curves an irregular "dog leg" shape.

b. In many cases the load carrying capacity of the pile exceed the limitations of the equipment so that the tests had to be abandoned before failure could be reached.

Analysis Procedure

In view of the above considerations, the procedure which was adopted to analyse the results was as follows:

a. For each loading stage, λ and s_0 were calculated using Eqs. (6) and (7).

b. The resulting P - s_0 curve was normalized using Eq. (1). In process of the linear regression, it was found that in many tests the first few points deviated from the straight line (probably due to joint closure). These points were duly disregarded.

Load Carrying Capacity

As the piles tested were of different lengths, their ultimate capacities were expressed in terms of unit skin friction, using Eq. (1). Although this equation tends to over-estimate pile capacity in certain cases (Leonards & Lovell 1978), the resulting B-parameters were found to give reasonably good results for most of the tests.

As the tests were carried-out on a number of different sites, a large scatter in the results was expected (Table 2). Generally, it is possible to distinguish between the capacity in the upward direction and that in the downward direction, the ratio varying between 1.59 (chalk and marl) and 2 (chert).

TABLE 2: ULTIMATE SKIN FRICTION VALUES

Rock Type	Direction of Loading	No. of Tests	Ultimate skin friction (MPa) Mean	Coefficient of variation (%)
Chalk & marl	Up	11	0.61	39
	Down	9	0.97	64
Limestone	Both	9	1.45	95
Chert	Up	6	1.45	46
	Down		2.89	28
Dolomite	Up	2	2.56	18
Cemented boulders & conglomerates	Up	5	0.58	66
	Down	3	1.02	67

Displacements

As expected, piles with higher capacity (1/B) also possessed a higher regidity (1/A), although the large scatter caused the correlation to be relatively weak (Fig. 4). A strong correlation enables the complete load-settlement curve of a pile to be expressed by a single parameter.

When the piles were analysed according to the procedure outlined above, the resulting λ values for most piles tended to decrease for each pile as settlement increased. Values obtained for loads equal to one third of the ultimate pile capacity, were compiled in Table 3.

TABLE 3: RELATIVE STIFFNESSES

Rock Type	Direction of Loading	No. of Tests	λ (m^{-1}) Mean	Coefficient of Variation (%)
Chalk & marl	Up	11	0.31	30
	Down	7	0.41	36
Limestone	Up	8	1.02	72
	Down	1	0.53	--
Chert	Up	6	0.48	50
	Down	3	0.86	49
Dolomite*	Up	4	1.34	34
Cemented boulders & Conglomerates	Up	5	0.48	32
	Down	3	0.96	34

* Corrected for the different diameters

Again the values in the downward direction were larger than those in the upward direction, the ratio being 1.32 to 2 - similar to those obtained for ultimate skin friction values.

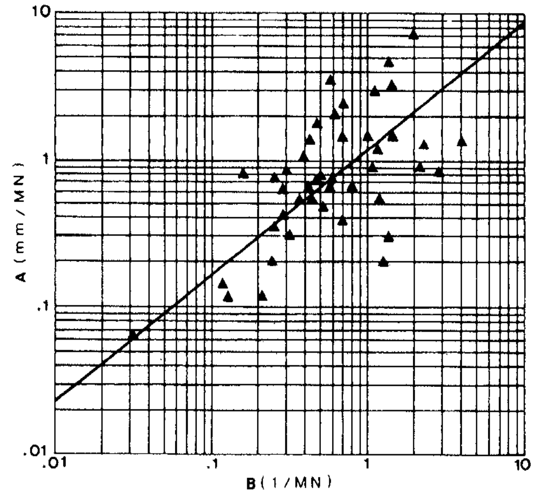

Fig. 4: Parameters A vs. B

According to Scott (1981), the spring constant k is given by:

$$k = \frac{E_R}{2D} \qquad (9)$$

where E_R is the Young's Modulus of the rock. Combining Eqs. (9) and (4) gives:

$$E_R/E_p = (\lambda D)^2 \qquad (10)$$

Eq. 10 may serve to obtain the mass modulus of the surrounding rock. Assuming E_p = 30 GPa, the resulting rock moduli were between 40 and 680 MPa.

CONCLUSIONS

1. Load-testing by the embedded jack method is a feasible method to study the load-settlement behavior of piles in rock.
2. The test results should be corrected for boundary conditions.
3. The hyperbolic approximation is useful for normalizing irregular test results and for predicting the bahavior of piles with the same length and diameter as the test piles.
4. By assuming a linear spring behavior of the skin friction, results of a load test may serve to obtain the rock mass modulus and to predict the performance of pile of different geometries.
5. Piles in rock can bobilize high skin friction (0.6 to 3.0 MPa), but with a considerable scatter. Therefore, when a new site is investigated, at least three tests should be performed to permit statistical evaluation of the results.
6. Both ultimate skin friction and rigidity are larger downward than upwards, by a factor of up to 2.

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