EXPERIENCE WITH THE DRIVING AND LOAD TESTING OF PRE-STRESSED CONCRETE PILING AT THE PORT OF ASHDOD

EXPERIENCE AVEC LE BATTAGE ET L'ESSAI DE RESISTANCE DES PIEUX EN BETON PRECONTRAINT AU PORT D'ASHDOD

ОПЫТ ЗАБИВКИ И СТАТИЧЕСКИХ ИСПЫТАНИЙ ПРЕДВАРИТЕЛЬНО-НАПРЯЖЕННЫХ СВАЙ В ПОРТУ АШДОД

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SYNOPSIS: At Ashdod Port, Israel, about 3000 piles are being driven for construction of additional piers for container cargoes. The piling is from 18 to 37 m long, of 50 cm octagonal cross-section, designed to carry loads from 60 to 100 tons. Pile hammers used are a BSP of 6 ton meters energy and a DELMAG 44, rated at 12 ton meters, which are compared to determine the set required during driving. Variable subsurface conditions result in some piles being end-bearing in sand and some as friction piling in stiff clayey soils. Pile loading tests, to 200 tons, were made for the various conditions and compared by analyses of expected performance based both on energy and on wave equations. In a portion of the site where the piles terminate in an upper sand layer, the proximity of clay to the pile tip presented a problem which was studied theoretically and checked by loading tests.

INTRODUCTION

The Port of Ashdod is an off-shore deep sea port protected by means of breakwaters, with berthing facilities arranged along finger piers projecting into the protected water basin (Kaplan, 1971). Prior to the first phase of construction, an extensive subsurface investigation was carried out (Komornik and Rohrlich, 1966). The soil profile off-shore generally consists of uniformly graded dense sand underlain by a layer of varying thickness of stiff to medium highly plastic clay and clayey sands. This clayey layer is not continuous over the entire area. Under the clayey stratum is a further stratum of dense sands and loosely cemented calcareous sandstone.

Those piers constructed in 1964 and 1966 were founded on precast and prestressed concrete piling jetted and driven into the upper sand layer.

During this first phase of construction ten pile loading tests were carried out (Komornik and Wiseman, 1971). Though some upper thin lenses of clay were penetrated during the construction of the piers, the piles were all jetted and driven to end bearing in dense sand. The clay stratum, when existing, was always well below the bottom of the piles and therefore did not influence the performance of the piles.

At the present time (1972) additional finger piers for container cargo are being constructed: Pier No. 5 with a dredge depth of 10.5 m and Pier No. 7 with a dredge depth of 12 m. At the location of these piers the clay stratum is continuous and its top is close enough to the dredge depth to have strongly influenced the design of the pile foundations for these two piers. A pile loading test program was established which included checking the behavior of piles in conjunction with the clay stratum.

SOIL EXPLORATION

Though the general soil profile was known from the subsurface exploration program executed prior to the first phase of construction, additional prickings and wash borings for better definition of strata depths were done along the alignment of the proposed construction. At each location where it was anticipated that a pile loading test would be required a special boring was executed. Standard Penetration Tests were used to determine relative density of the granular layers and field vane tests for shear strength of the clays. Undisturbed samples were taken in the clay stratum. The sands were dense, sub-angular and uniformly graded. The blow count in the Standard Penetration Test ranged from 60 to 100 blows for 30 cm of penetration. The clays were generally highly plastic. The field vane shear strengths ranged between 1.0 to 2.0 kg/cm^2 above elev. - 25m and from 2.0 to 4.0

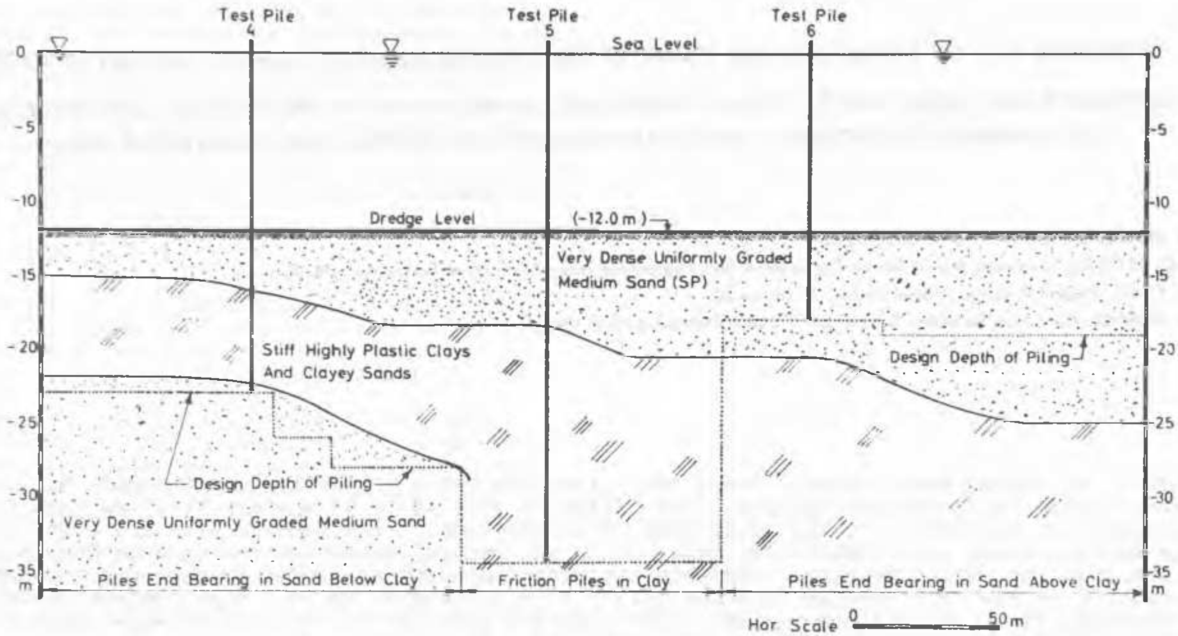

Fig. 1 Longitudinal Section Through Pier No. 7

kg/cm^2 below elevation - 25m. The clays were all preconsolidated. The minimum preconsolidation pressure was 2.6 kg/cm^2 for a sample from elevation - 19.0m. Sea bottom at the time of sampling was at elevation - 3.5 m. A typical soil profile is shown in Fig. 1 for Pier No. 7.

PILE FOUNDATIONS

The new piers are designed for heavy container cargo. The dead load is about 25 tons per pile but various possible combinations of live load can bring the total loading on the piles to a range from 60 to 100 tons. A typical pile bent for Pier No. 7 is shown in Fig. 2.

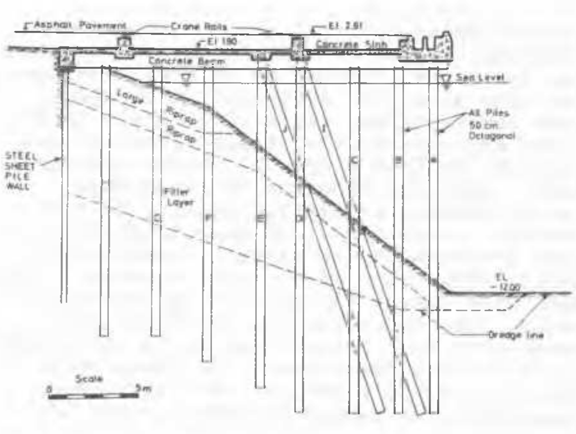

Fig. 2 Typical Cross-Section of Pier No. 7

The bents are at 3.0 m spacing, with two batter piles at 1:3 slope located midway between the bents. A typical bent in Pier No. 5 is similar to that shown for Pier No. 7, but has 7 vertical piles. The total length of new construction for Pier No. 5 is 340 m and for Pier No. 7 is 380 m. Approximately 3000 piles are being driven during the present phase of the construction project. Shown on Fig. 1 is the design depth of piling for Pier 7. Both the depth of embedment and the distance above the clay layer were chosen as design criteria for ordering piling. The main purpose of the pile load test program was to check these two criteria and modify them if necessary. In addition it was felt that the driving records obtained for the test piles would be useful to use as a calibration for specifying driving resistances during execution. The profile along Pier No. 5 was similar to that along Pier No. 7 except that, since the dredge depth was only 10.5 m as against 12.0 m for Pier No. 7, it was possible to either penetrate to the sand below the clay stratum or to remain in the sand sufficiently above the clay stratum. Thus, all piles for Pier No. 5 were end-bearing in sand.

The piles are cast off the job site and transported by trailer truck to the port. The minimum concrete cube strengths specified was 500 kg/cm^2 and the piles have been prestressed to a residual stress of 50 kg/cm^2.

The piles are 50 cm, octagonal in section, and the maximum single length cast is 28 m. Where greater lengths are required a special Herkules splice connector, manufactured in Sweden by Scanpile AB Gothenburg, is cast integrally with the piles at the casting yard. The upper extension piece is added in the pile driving rig just before the actual driving. A special rig was built locally by the contractor for the job. The rig consists of a platform which is supported on the sea bottom during driving, but is easily floated to a new position. The tower frame is moveable on the platform and all the vertical piles of one bent and one pair of batter piles between bents can be driven from one positioning of the platform. Two external jet pipes are used alongside the piles to assist achieving the required depth of penetration in the sand.

A single acting BSP hammer having a rated energy of 6.0 ton meters has been used for driving the shorter piles on Pier No. 5 (up to 25 m long). A diesel Delmag 44 hammer having a rated energy of 12.0 ton meters is being used for driving the longer piles on Pier No. 7. Piles up to 40 meters long (15 m + 25 m) have been driven without difficulty with this hammer. For a portion of Pier No. 7, where the piles are driven through clay to end bearing in sand, the question of uplift from the driving of adjacent piles was investigated. It was found that though there was measurable uplift it was not cumulative and the maximum total measured uplift was less than 0.5 cm. A loading test on a production pile showed no adverse effects due to the driving of the adjacent piles.

PILE LOADING TESTS

The pile loading tests were performed by jacking against dead weight of more than 200 tons, consisting of concrete piling lying horizontally on a platform 3.0 m above sea level. A wooden working platform 1 m above sea level was provided for the technicians who took the readings of the pile settlement and controlled the load. The platform was supported on 4 steel pipe piles 50 cm in diameter forming a square 6.5 m by 6.5 m. Since tests were performed in up to 12 m of water the pipe piles were suitably braced against buckling and sidesway. The bracing system consisted of two prefabricated units 5.7 m high with sleeves at the corners which fitted over the 50 cm pipe piles. Two additional piles were driven outside of the platform to act as fixed supports for a horizontal reference beam to which three dial gauges (reading to 1/100 mm) for measuring the settlement of the pile head were attached. At the date of the writing of this paper (July 1972) six pile loading tests have been carried out (Table I).
Load settlement curves are shown in Fig. 3 and 4.

Table I. Bearing Capacity of Piles Tested

LOAD TEST NO.	PIER NO.	DRIVEN LENGTH OF PILE (m)	DREDGE ELEV (m)	HAMMER	ELEV. TIP OF PILE (m)	DISTANCE FROM TIP OF PILE TO CLAY (m)	$\frac{W_H}{W_H + W_P}$	SET (s) (cm)	QUAKE (c) (cm)	BEARING CAPACITY (TONS) COMPUTED DELMAG FORMULA	WAVE EQUATION	MEASURED LOAD TEST
2	5	18	- 10.7	BSP	- 17.15	1.7	0.36	0.60	1.5	130	145	90
2 A	5	20	- 10.5	BSP	- 16.40	2.4	0.33	0.30	1.1	185	190	120
4	7	27	- 11.5	BSP	-22.65	THROUGH CLAY INTO SAND	0.27	0.20	0.5	290	280	180
5	7	40	- 12.0	DELMAG	-35.00	FRICTION PILE IN CLAY	0.18	0.80	0.35	CONSIDERED NOT APPLICABLE		> 200
6	7	21	- 12.2	BSP	- 18.05	2.5	0.32	0.30	0.75	230	250	150
x 20 A	7	27	- 6.0	DELMAG	- 24.50	THROUGH CLAY INTO SAND	0.24	0.75	0.5	240	290	>200

x PRODUCTION PILE

All pile loading tests were carried out to failure or until a load of 200 tons was reached, whichever occurred first. The load was applied in increments of less than 25 per cent of the design load, with each increment being maintained for a minimum of 2 hours as recommended by the Israeli Building Code. If towards the end of the 2 hour period the rate of movement of the pile head was more than 1/100 mm in 10 min. the load was maintained until this criteria was achieved. The 100 ton load was maintained for 24 hours and the pile was considered to have reached a stable condition if towards the end of this period the rate of movement of the pile head was less than 1/100 mm in 20 minutes. If failure was not achieved at an earlier stage the 200 ton load was also maintained for 24 hours.

For pile loading test No. 6 the pile had not quite reached stability at 150 ton, the rate of movement being 2/100 mm in 20 min. at the end of 30 hours, with a total settlement of about 17 mm. The pile was then unloaded to zero load and reloaded to 100 ton and beyond this load in increments of 12.5 ton, each load being maintained for one hour. When the load was at 200 tons the rate of movement at the end of an hour was 50/100 mm in 10 minutes. This rate is appreciably less than is commonly suggested for "constant rate of penetration" (C.R.P.) tests. Hence a C.R.P. test, if employed in this case, would have seriously overestimated the capacity of the pile.

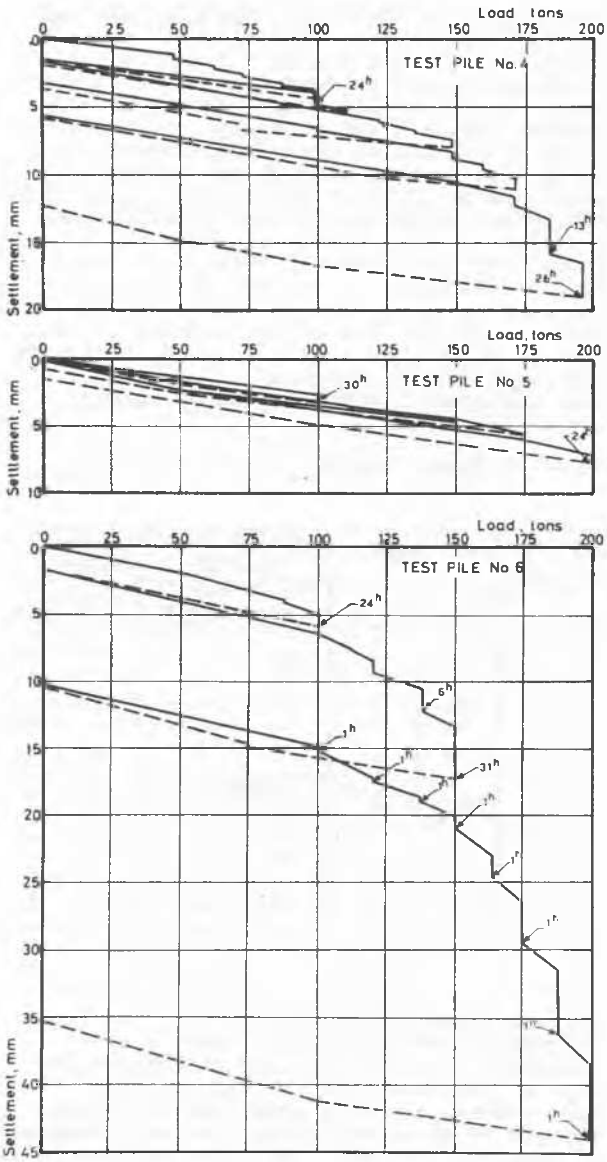

Fig. 3 Loading Test Results for Pier No. 7

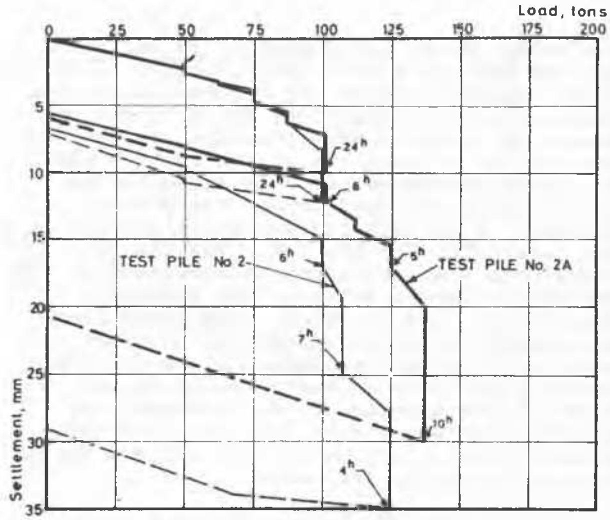

Fig. 4 Loading Test Results for Pier No. 5

CONTROL OF PILE DRIVING

The length of piling required to carry the design loads was estimated for various sections of the piers, based on the soil profile and the measured strength parameters. For each typical section a test pile was driven and then test loaded to failure or a maximum of 200 tons. If the loading test was judged to be satisfactory the set obtained for the last 20 blows was then presumed to be the required set for all piles in that section, should both test piles and production piles be driven with the same pile hammer. Pile driving was then controlled by specifying bottom elevation of pile and set per blow. For friction piles in clay only bottom elevation was specified.

All test piles (except for No. 5) were driven with a B.S.P. compressed air single acting hammer having a manufacturers' rated energy of 6.0 ton meters and a weight of ram of 5.0 tons. For pier No. 7 it was decided to drive the production piles with a diesel hammer (Delmag 44) having a maximum rated energy of 12.0 ton meters and a weight of ram of 4.3 tons. It therefore became necessary to convert the sets per blow for the BSP hammer to required sets per blow for the Delmag 44, which has double the BSP rated energy.

A comparison of the required sets can be made using a pile driving formula based on energy considerations. All such formulae, of which there are many, can be written in the following form:

$$E\eta = Q_u\ (s + c/2) \quad \ldots\ldots\ldots (1)$$

$$\text{or } Q_u = \frac{E}{s+c/2} \cdot \eta \quad \ldots\ldots\ldots (2)$$

where

E = rated energy of hammer (ton cm)

η = efficiency of transfer of energy from hammer through dolly to pile

Q_u = ultimate bearing capacity (tons)

s = set per blow (cm)

c = elastic compression of pile and soil during driving (cm)

It may be observed that the Delmag formula is identical with eq. (2) with $\eta = \frac{W_h}{W_P+W_H}$ where W_H = weight of ram, and W_P = weight of pile. Since the comparison is to be made for the same ultimate bearing capacity for piles of the same length, and for hammers having about the same weight, we may assume as a first approximation, that Q_u, η, and c are the same for both cases.

It can then be shown from Eq. 1 that for a ratio of rated energies of two,

$$s_{\text{Delmag 44}} = 2s_{BSP} + 0.5\ c. \qquad (3)$$

Some comparative driving tests were made at Pier No. 7 with interchanging of BSP and DELMAG hammers. Results of measured and computed values are shown in Table II. The measured c values for all these piles was about 0.4 cm.

Table II: Comparison of Hammers

Pile No	Set per blow, s, in cm			
	BSP	DELMAG 44		
	measured	measured	computed from	
			Eq.(1)	wave equation
1 A	0.50	1.00	1.20	1.22
1 G	0.23	0.65	0.66	0.75
1 H	0.21	0.56	0.62	0.61

Computations were based on the above energy equation and on the wave equations as discussed below. Both methods of computation give results which are in close accord with the measured results.

The wave equations were used for predicting driving stresses (both tension and compression) and set versus pile capacity (Smith, 1962). The piles were assumed to carry 85 percent of their load in end bearing with the remainder being carried by friction. The static pile capacity was assumed to be related to the dynamic resistance in the following manner (Coyle and Gibson, 1970).

$Q_{dy} = Q_{st}\ (1 + JV^{0.2})$ where

Q_{dy} = the resistance of the soil to dynamic penetration of the pile,

Q_{st} = the static bearing capacity of the pile,

V = the velocity of the tip of the pile at the time of driving (cm/sec),

J = a factor indicating the rate dependent resistance of the soil (assumed = 0.5 for end bearing and 0.17 for friction).

The above equation, with the assumed values for J, gives a dynamic resistance of about double the static resistance.

It should be pointed out that the static bearing capacity computed by the wave equation is analogous to the bearing capacity as measured in the C.R.P. loading test (Whitaker and Cooke 1961) and would be expected to be higher than the bearing capacity as determined by maintained load (M.L.) pile testing.

THE INFLUENCE OF THE CLAY LAYER BENEATH THE TIP OF THE PILES

The clay layer which exists for part of the site several meters below the bottom of the piles influenced the performance of the piles both during driving and in subsequent load testing. The piles terminating in the sand above the clay were all driven with the BSP hammer. The elastic compression of the pile and soil (quake) was measured by moving a pencil horizontally across a piece of paper attached to the pile while driving proceeded. The quake was found to depend on the distance between the tip of the pile and the top of the clay layer (Table III).

TABLE NO. III

Quake Measurements			
PIER NO.	PILE NO.	DIST. TIP OF PILE ABOVE CLAY (m)	QUAKE c (cm)
5	113C	4.0	0.60
	79C	2.5	1.50
	2A	2.4	1.10
	2	1.7	1.50
7	6	3.5	0.65
		2.5	0.75

Where no clay layer was present beneath the tip of the pile, the measured quakes for piles driven to end bearing in sand were about 0.4 cm.

The large variation in quake values made it essential to take them into consideration in estimating the bearing capacity from the pile sets. For example, pile No. 2A and 6 were of the same length, the same depth of penetration in the sand, their tips were the same distance above a clay layer and both had the same sets. The quake measured for Pile No. 6 was two thirds of that measured for No. 2A. (Vane tests performed in the clay layer adjacent to these two piles indicated that the shear strength of the clay at Pile No. 6 was somewhat greater than that at Pile No. 2A. It may be noted from Table I that the values computed by both the Delmag and wave equations, as well as the actual load test data, indicated a higher capacity for Pile No. 6 than for No. 2A.

The distance between the tip of the pile and the clay directly influenced the bearing capacity. Thus, in Table I, it may be noted that Pile No. 2 had a slightly greater depth of embedment than Pile No. 2A, and hence the bearing capacity of the sand would have been expected to be greater rather than less. The lower bearing capacity measured is no doubt due to the proximity of the clay layer.

CONCLUSIONS

The clay layer beneath the sand strongly influenced the design and the construction phases of the pier foundations through its effect on length of piling, load test program and procedure, bearing capacity, quake, and interpretation of sets.

Due to the existence of the clay layer beneath the tip of the piles for part of the site, C.R.P. loading tests would have overestimated the permissible design loads on the piles.

It was found that the wave equation could be used with a fair degree of accuracy to compare the required sets for different pile hammers.

For this site both the wave equations and equations based on energy considerations consistently overestimated the failure load as determined by maintained load pile testing by about 50 percent.

In predicting pile capacity from measured sets it is of utmost importance to include realistic elastic deformations of the pile and soil. This was found to be true for both energy type pile formulas and for wave equation analysis.

This site provided an excellent example of the need for continuous exchange of information between the project engineer, the designer and the foundation consultants throughout all phases of design and construction so that adjustments could be made to both the construction procedures and the design as the job progressed.

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