Piles for an offshore unloading terminal

Pieux pour une station de déchargement en mer

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SYNOPSIS: A pier was to be built for a coal unloading terminal, about 2 km offshore, together with a connecting jetty. Steel pipe pile foundations were required to carry ultimate loads of as much as 18 MN per pile. The paper describes geotechnical conditions, methods of selecting soil design parameters, full-scale load tests, prediction of pile capacity, and use of pile driving dynamic observations for both design and construction. Concrete and steel plugs were developed to improve pile capacity. Close control of operations was provided to verify achievement of design requirements for the statically indeterminate structures involved.

1 INTRODUCTION

In order to provide a coal unloading terminal for a new power plant in Israel, a pier and connecting jetty were built in the open sea about 2 km off shore, where the water depth was more than 20 meters. This paper deals with various geotechnical aspects of the project, which were guided by the authors as the foundation consultants. Included are problems which arose in the exploration program, the design of the foundations, and the execution of the pile foundations.

2 SITE CONDITIONS

The location of the Coal Unloading Terminal is shown in Figure 1, together with the layout of the borings and test piles utilized in the foundation investigation program. Subsurface investigations were carried out at the potential site by the Soils and Roads Testing Laboratories of the Technion, Israel Institute of Technology, with the borings executed from an elevated tripod platform by TAHAL, Water Planning for Israel, Ltd.

a. Geology

The geological units found in the Israeli Mediterranean coastline belong to the Quaternary age. Dramatic changes in temperature, wind intensity and rainfall occurred at that period in the Middle East simultaneously with tectonic activity. Changes in sea levels were occurring during this period. Faulting and folding caused coastal changes in the form of a general regression, with sporadic limited ingressions. Varying degrees of continental depositional conditions have determined the present coastline. Coarse sediments, often rich in shells and pebbles, were deposited nearest the shoreline. At a later stage these coarse sediments, with the addition of marine matrix materials, lithified and a beach was formed. Sands, silts and clays were heterogeneously deposited at different depths and contain sea fauna, flora, debris and foraminiferal tests. Deposition of silt and clays occurred in existing lagoons and swamps. The following rock and soil units were included in formations extending to a thickness of about 200 meters:

Layer A - Sand Dunes

Layers B & D - Upper Clayey Silty Units

Layer C - Calcarenite Unit, referred to below as a "calcareous unit", and a Biocalcirudite to conglomerate unit.

The carbonaceous components of the above units were composed of shell debris, foraminiferal tests (skeletons) and other microbiological remains. The percent of carbonate grains was higher for deposits which have been formed under a marine environment. Sometimes, marine components were transported by erosional vehicles and then redeposited under a continental regime. Continental deposition in this coastal region may be inferred by features such as laminar cross bedding which were formed by a continental sand dune. Thus, the calcareous unit was composed of layers or concretions of sand bodies and calcareous sand forms (Frydman 1982, Komornik & Hayati 1984). The thickness of this unit varied up to about 35 meters. In the middle of this unit there was a silty and clayey horizon 2-3 meters thick. The calcareous sandstone content varied between 10 to 30% for upper and lower portions. The sand bodies vary in gradation and amount of calcareous content with depth. The upper sand layer (above the upper clay strata) was a poorly graded fine sand (SP) with a uniformity coefficient, Cu, of about 1.5, while the deeper sand was a medium to well graded sand (SW) with a uniformity coefficient of 4-10. The fine and medium size sand contained 60-70% quartz grains and 20-30% calcareous grains of shell debris and foraminiferal tests. The coarse sand size was composed mainly of calcareous sandstone fragments. The silty size material in the sandy layers was found to have high carbonate contents.

In Figure 2, a geological cross-section from Land to Sea (East-West) along the length of the jetty and pier is shown.

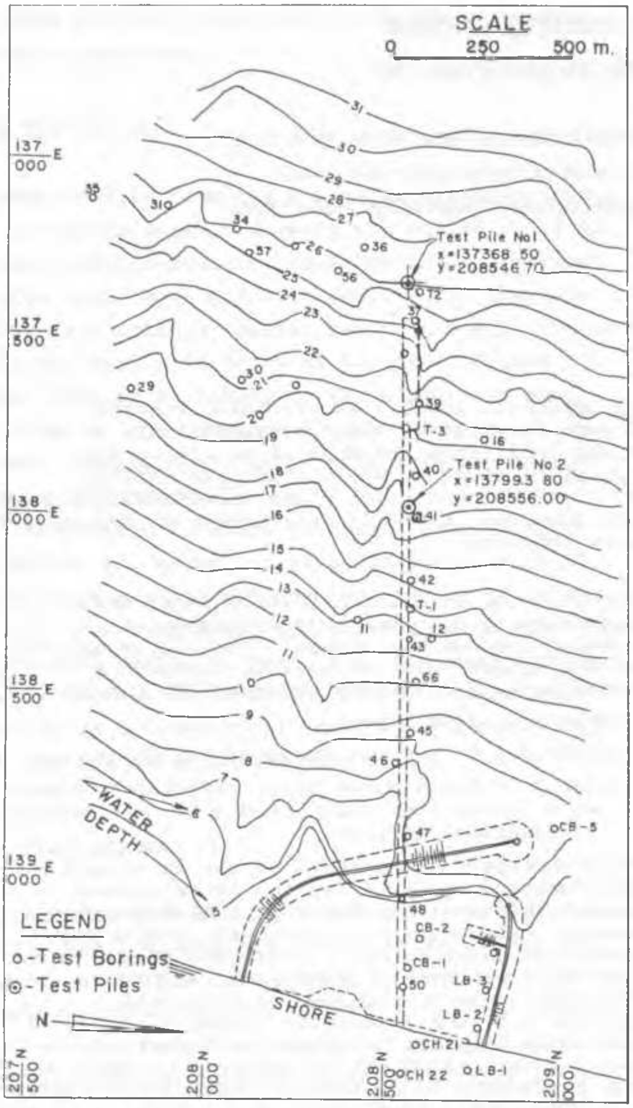


Figure 1. Location of Borings and Test Piles.

b. Typical soil profile

Soil investigations included more than 30 borings. Standard Penetration Tests were made in sandy layers, and shear vane tests in clays. Pressuremeter tests were performed in selected borings. Both disturbed and undisturbed soil samples were taken for an extended serious of laboratory studies. A summarized typical soil profile and the soil parameters selected for design are given in Figure 3.

3 DESCRIPTION OF STRUCTURES

The design for the unloading facility included a finger type open pier of a length of 300 meters and a width of 24 meters to be supported by piles. It was connected to the shore by a pile supported jetty, 1700 meters long and 12 meters wide. The top level of the pier was 14 meters above mean sea level, slightly higher than the

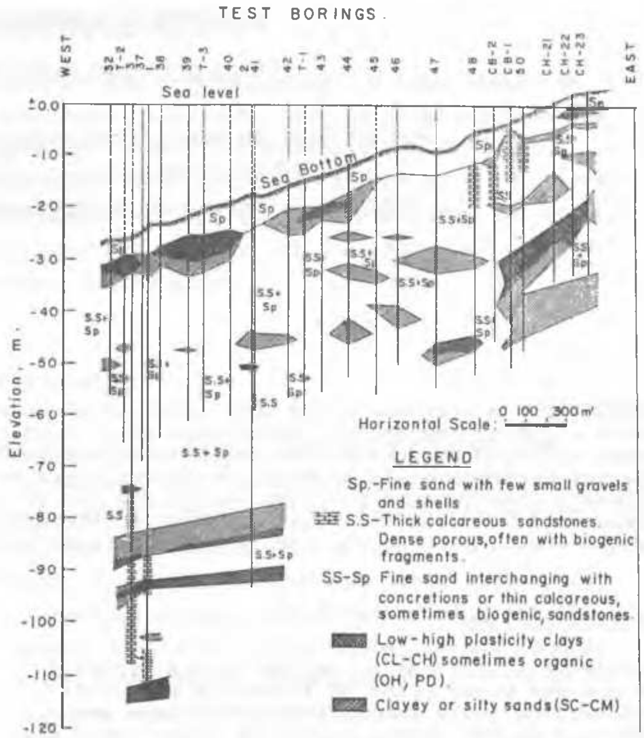


Figure 2. Geotechnical profile along pier and jetty alignment.

anticipated maximum wave height of 13.7 meters in 1:100 years period. The water depth on the land side (east) of the pier was 20 meters and at the sea side (west) 24 meters. The ships were moored to a series of flexible mooring and berthing dolphins not connected to the pier structure. The decks of the pier and jetty were made of reinforced concrete. Precast elements were used as beams, and a deck with concrete cast in place in the joints, formed a monolithic structure as shown in Figure 4. Post tensioning in all directions was performed, as the design of the structural system was based on space frame analysis, fully continuous in all directions, with non-linear interaction between structure and soil. The jetty deck was a composite construction of steel plate girders with precast concrete deck slabs.

4. DESIGN CRITERIA AND CODES

The design was based on the Det Norske Veritas (D.N.V.) "Rules for Design Construction and Inspection of Offshore Structures" (1977); API - RP2A "Recommended Practice for Design, Construction and Inspection of Fixed Offshore Structures" (1980); Israeli standards for design of concrete and prestressed concrete structures; and AISC specifications for steel design. Modifications of the code requirements were made to make them compatible with local conditions.

The pile designs were selected so as to provide a factor of safety of at least 2.0 against permanent loads and at least 1.5 against total loads. Typical piles were checked and found generally to agree with D.N.V. design

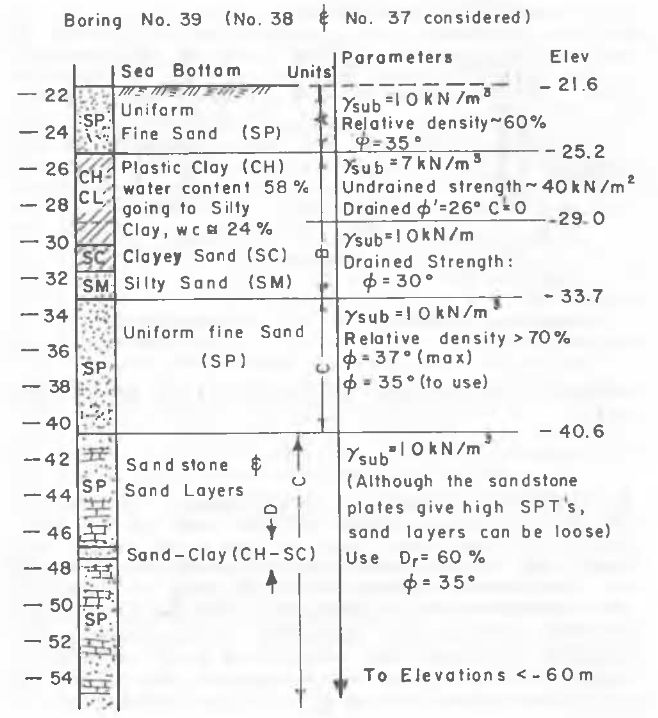


Figure 3. Typical soil profile and soil parameters for design.

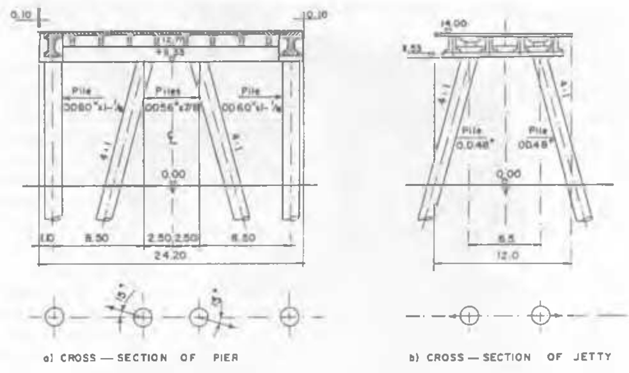


Figure 4. Typical cross-sections of pier and jetty.

requirements, with load coefficients employed to allow a semi-probabilistic limit state design, and with design resistance obtained on the basis of characteristic strengths divided by material coefficients.

Allowable differential settlements were calculated so that a minimum of cracking would occur in the concrete elements. The indeterminate type of structure allowed less differential movements than permitted in the more common statically determinate offshore sea structures. Thus, the allowable differential settlements in the pier were of the order of 10 mm between bents and less than half of that between piles in the same bent.

5. FOUNDATION - DESIGN

The pier foundation was based on 56 and 60 inch diameter open steel pipe piles, driven in bents 20 meters apart, 4 to 6 piles per bent. The piles were designed for various ultimate load capacities, of as much as 18 MN. The approach jetty foundation was based on 48 and 42 inch diameter open steel pipes designed for an ultimate load capacity of up to 10.5 MN, with a bent spacing of 40 meters.

The design required that some of the piles would be able to develop pull out resistances of up to 3.0 MN in the pier area and 6.3 MN in the jetty area.

Open steel pipe piles had been selected to make it possible to drive the piles to the depths necessary for adequate capacity to be achieved. It was specified that monitoring equipment, such as produced by "Pile Dynamics, Inc." be provided to allow for dynamic measurements to be made on the piles during the driving operations, and make it possible to verify that the design resistance had been reached. This approach required that a calibration be possible of the pile driving dynamic characteristics, for use in comparisons with results of loading tests. Load tests were hence included as part of the tender requirements, to allow for the verification of the load-deformation characteristics, with dynamic measurements to be obtained during the driving of the test piles by the same equipment which the contractor would use in the actual construction operations. Criteria were thus established for the supervision of the pile foundation execution. A total of 72 piles were required for the pier structure, and 106 piles for the jetty.

In order to minimize the horizontal loadings on the pier, a series of flexible mooring dolphins (steel pipes of 2.6 and 2.8 meters diameter) were driven to a depth of 25 meters below sea bed. They were located at the two sides of the pier, independent of the pier structure, and designed to carry the full horizontal loads from the wave action on the ships being unloaded. The horizontal load on a dolphin was 3.0 MN acting 26.5 meters above sea bed. Two anchoring mooring berths were built, one at each end of the pier. The pile foundation of the mooring berth at the western end was scheduled to be started at the beginning of the project so that the piles could be first used for the 20 MN load test arrangement, as shown by the photograph, Figure 5. The test pile and the reaction piles were all later incorporated into the final pier construction.

6 BEARING CAPACITY OF OPEN STEEL PIPE PILES

Open steel pipe piles may be considered to transfer their loads to the soil by means of the mechanism shown in Figure 6. The ultimate bearing capacity is provided by the sum of the friction resistance around the outer perimeter of the pipe and by the end bearing resistance, which itself is the sum of the resistance beneath the steel section at the end of the pipe and the end bearing resistance developed in the soil plug.

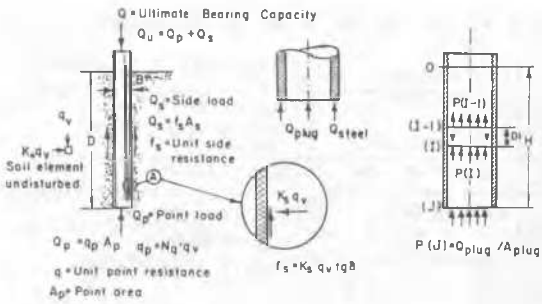
Figure 5. View of loading test arrangement for the 60-inch diameter pipe pile.

The frictional resistance between the soil and a pile is given by the following relationship:

$$f_s = K_h q_v \tan(\delta) \quad (1)$$

where:
- f_s = Frictional resistance at a particular depth, z
- K_h = Horizontal coefficient
- q_v = Effective vertical stresses at depth, z
- δ = friction angle (pile-soil)

This relationship indicates that the frictional resistance should increase with depth, since effective vertical stress would be expected to increase with the weight of the overlying strata. However, various researchers, as Berezantzev et al (1961), Vesic (1977), McClelland (1974), Meyerhof (1976), and Coyle and Castello (1981), found that this increase is limited to a certain depth, depending on the ratio of depth to pile diameter. This critical depth has been suggested to vary between 10 to 20 times the diameter of the pile, B, and is influenced by the effect of soil density on the stress-strain relationship of the sand surrounding the pile. The frictional resistance is dependent also on the normal stresses on the pile surface, and thus on the product of $K_h * q_v$. The coefficient factor K_h may vary between K_a (active) to K_o (at rest) to



(a) Forces Acting on a Pile.

(b) Stresses Developed in a Soil Plug

Figure 6. Forces and stresses acting on a pipe pile.

K_p (passive) depending on the amount of strain and the dilating effect of the sand around the pile. In the upper part of the pile, where the full load is acting on the pile material, there will be also a tendency of the pile to increase its diameter, thus increasing the development of passive pressure in the soil. These phenomena explain the fact that much more load is being taken by frictional resistance at the top of the pile than given by the $f_s = f(q_v)$ relationship. As less deformation develops in the pile at depth, due to load transfer to the upper layers, and the soils are subjected to larger overburden stresses, the horizontal factor K_h reduces to values of K_o (at rest condition). At greater depths, where the soil around the piles is sheared under high lateral pressures, the sand will tend to decrease in volume and the lateral pressure will also tend to decrease, corresponding to a K_a (active) value. Another effect is a tendency for arching to occur within the sandstone layers which are interbedded with soft or loose sand strata, and thus the transfer of the frictional stresses to the adjacent soil layers and then back to the side of the pile does not fully occur. A reduction in friction is also observed when piles are driven into calcareous soils having a high content of carbonates, which are usually in the silt size fraction and have a lower angle of friction than the same sands without the fine fraction. The friction angle will also vary under various lateral loads. High stresses will tend to crush the sand grains and cause a reduction in the friction angle (Vesic 1968, Frydman 1987). The amount of crushing depends on the strength of the grain material, the grain size, the grain shape and the amount of hard grains in a mixture of hard and softer grains. (Fede 1971; Joustre and Debut 1982; Beringen et al. 1982). This phenomena occurs during hard driving conditions and at large depths where the stresses in the soil are high. Driving of open pipe piles through sandstone layers will cause breakage of the sandstone plates and crushing of the sand, thus forming a loose soil envelope around the pile periphery during driving and a reduction in the friction values. Also, due to the crushing effect, the amount of carbonates in the silt size particles, and the vibration of the pile during driving, pore water pressure is developed in the soil during driving and densification of the soil is inhibited. The frictional resistance between the soil and the pile therefore depends on the soil type, on the surface roughness of the pile, on the stress-strain behavior, and the stresses in the soil induced during the driving of the pile and during the loading of the pile under static, repeated or vibratory loads. The type of piles driven, the size and depth, the equipment used to install the pile and the resistance to driving all affect the frictional resistance developed in the soil along the pile as discussed above. Due to all these elements and the heterogeneity of the profiles, the crushing of the soil grains and the effect of various pile driving execution methods used, it can be seen that the final frictional resistance developed around a pile is a varying factor. All of these factors tend to explain the various inconsistencies between a simple formula for predicting frictional resistance distribution with depth and the actual measured stress distribution along an instrumented pile in a load test, or the actual frictional behavior of a loaded pile under working conditions.

(b) End Bearing Resistance

End bearing resistance of the soil at the tip of the pile is usually calculated according to Terzaghi's formula for bearing capacity by simplifying it to take into account only the effective vertical pressure at the tip elevation of the pile:

$$q_p = N_q * q_v \qquad (2)$$

where:

q_p = Ultimate bearing capacity for a unit area.

q_v = Effective vertical overburden pressure at level of pile tip.

N_q = Dimensionless bearing capacity factor, based on ϕ (Vesic 1977).

$$N_q = (1+\tan\phi)e^{\pi\tan\phi} * \tan^2(45+\phi/2)$$

As discussed above in considering side friction, the effective vertical pressure around the pile tip elevation (q_v) is not necessarily equal to overburden pressure. It is limited to an equivalent effective overburden pressure expression which depends on the effective depths - which is taken to vary between 10 to 20 times the diameter of the pile in loose to dense sands respectively. Therefore, long piles (having a length of over 20B in dense sands) do not achieve an end bearing capacity significantly greater than for piles of medium length. The amount of load transmitted to the base of the pile is the difference between the load on the pile and that part of the load which was transferred to the soil through frictional resistance. These loads vary according to the amount of settlement of the pile and the stress-strain characteristics of the soil both in frictional resistance and in end bearing resistance. Disturbance of the soil below the tip of the pile would cause a reduction in the angle of friction (with a reduction in the N_q value) and thus a reduction in the bearing capacity of the pile. The same effect will be noted in cases when increased stresses in the sand cause crushing of the sand grains and tend to increase the amount of fines (e.g. in the form of carbonates). A number of investigators have observed that increasing the depth of a pile in an homogeneous sand would not increase the end bearing capacity of the pile beyond a certain depth (Vesic 1977; Meyerhof 1976; McClelland 1972; Berezantsev 1961; Coyle and Castello 1981).

It also should be noted that because of the rigidity of the steel section at the end of an open pipe pile, there will be a concentration of load transfer on the area of the steel, and less load can be transferred through the soil-plug area. Thus, in the area of the steel section the full bearing capacity of the soil will develop, while in the area of the soil plug a partial load transfer to the underlying soil is occurring.

Open pipes used as piles would allow the penetration of the pile through harder layers above the design bearing layer. Whenever hard driving is expected, the thickness of the steel section at the tip of the pile is increased to minimize damage during driving. The increase in wall thickness may be achieved by increasing the outside diameter of the pile, or by decreasing its inside diameter. Increasing the outside diameter may tend to reduce the outside frictional resistance of the pile above the tip. If the inside diameter of the pipe is reduced, there is a tendency to disturb and loosen the soil entering the pipe during the driving, and reduce the frictional resistance mobilized by the soil plug against the interior wall of the pipe. If the soil is sand and it tends to be recompacted during driving, a "rigid" plug may be formed. The upward movement of such a plug during loading of the pile will be resisted by friction with the pipe and the plug will transfer its load to the soil nearer to the center of the pipe. In order for this type of resistance to develop, the size of the pipe diameter is critical. The bigger the inside diameter the less likely that a "good" soil plug will be formed (Koshide and Isomoto 1977). The height of densified sand required to form the "rigid" part of the plug depends on the soil frictional resistance which is a function of the angle of friction between the sand and the pipe and on the lateral stresses in the sand. These stresses vary between at rest pressures to a maximum value of passive pressures in the area of the thickened shoe and to active values above this area. The height of the rigid part of the soil plug may be taken to be about twice the diameter of the pile, and this is one of the reasons why the thickening of the bottom part of piles is done for a height of about twice the diameter.

The effective weight of the soil above the "rigid" plug area will increase the lateral pressures available to resist upward movement during loading. If, instead of clean sand, the soil contains sands with fines, or sandstone plates interfering with re-densification, the amount of compaction during driving may be very limited. Clayey soils will not form "rigid" plugs and will tend to "slip" in the pipe. For such soils, the resistance of the plug area to load will depend mainly on the effective weight of the soil column. Since the area of the soil plug is usually many times greater than the steel sectional area of the tip of the pile, the potential end bearing capacity of a pile can be fully developed only with the formation of a "good" soil plug which mobilizes the friction of the plug against the interior of the pile.

The factors discussed above show that the transfer of load from an open pipe pile to a foundation soil is a complex subject and any prediction relies on many assumptions that require verification.

In addition, it was necessary to take into account that high tension forces on the piles were required in the design. It was hence recommended to the owner, the Israel Electric Corporation, that two series of loading tests be made at the start of construction, before executing the foundation piles. It was desired to test piles upon which dynamic measurements had been made, with the same construction equipment as would be used in the building of the installation. To save making such tests independently, with high costs of mobilizing equipment, it was agreed that the loading tests be included as part of the contract requirements. This procedure allowed the checking of the characteristics and the performance of the contractor's equipment on the site. The drawback to this decision was that the final design would be dependent upon the results of the loading tests, and some redesign of the foundation and structure might be required. To minimize any delay in the actual execution of the job, the geotechnical advisors were put under considerable pressure to analyze results quickly and form their recommendations as soon as data was available from the field.

7 STATIC PILE DESIGN

Originally, a tentative pile design had been established which would allow a contract to be initiated and steel piles to be ordered. A basis was thus provided for wave equation studies and for the selection of construction equipment.

Each of the piles was required to be designed to be able to transmit the compression, the tension (pull out) and the horizontal loads transmitted from the structure, without exceeding allowable deformations and stresses in the pile material. The first wave equation studies were useful in design to avoid overstressing the steel pile wall during the driving operations, and considering various types of hammers and accessories which may be used in driving the piles to the designed depth. The design calculations for the compression loads were based on the parameters shown in Fig. No. 3 and our experience in driving and testing pipe piles in port construction in Ashdod. Simplified design formulas for friction and end bearing were utilized, taking into account the following concepts:

$$Q = Q_s + Q_p = f_s * A_s + q_p * A_p$$

(a) The equivalent effective overburden pressure was calculated on the basis that the effective depth would be equal to 15 times the diameter of the pile. The choices of parameters took into account the nonhomogeneous nature of the profile and the fact that the driving of the piles would have crushed sand particles and broken the calcareous sandstone plates, thus resulting in lower densities and lesser angles of friction than might have been indicated from the SPT results.

(b) The effective area (A_p) at the end of the pile was considered to be equal to the area of the steel section, plus an equivalent area of an internal soil ring which would be a function of the thickness of the end section of the pile steel. The innermost portion of the pile core or plug would be less effective than a solid pile, and its bearing capacity would be disregarded. As a first assumption, this soil ring was taken to have a thickness of eight times the thickness of the pipe pile end segment. This "effective" area will be that portion of the total pile end area which, when multiplied by the ultimate bearing capacity pressure, will give the estimated end bearing capacity. The resistance of the soil plug area will, thus, not be simply increased in proportion to the end area, as pile diameters become greater.

(c) The friction values along the pile were selected after taking into account an average skin friction based on the VESIC (1977) limit value for skin friction:

$$f_s = 0.08(10)^{1.5D_R{}^4} \quad kg/cm^2$$

where D_R = average relative density over the length of the pile. (Relative Density is also termed "density index", I_D, in ISSMFE symbols.) For this project, in view of correlations with both foreign and local experience, the friction values under compression loading were taken as 1.5 times larger then the values given by the above equation. The side resistance against uplift forces for each pile was assumed to be equal to 70% of the frictional resistance in compression.

The following table sums up the ultimate loads which would be calculated for the loading test piles, adjusted for the actual final penetration depths.

Table 1. Ultimate loads calculated from static soil parameters.

Pile No.	Diameter (inches)	Penetration Depth (m)	Friction in Compression (MN)	Friction in Tension (MN)	Compression End Bearing (MN)	Compression Total Load (MN)
T-1/A	60	16.4	1.0	0.7	10.3	11.3
T-1/B	60	31.1	2.9	2.0	14.0	16.9
T-2/A	46	16.0	0.8	0.6	5.9	6.7
T-2/B	48	55.6	5.8	4.1	6.6	12.4

Note: Calculated values are based on exterior area of pile, and total area of pile end, without evaluating separate action of "plug".

After the results were available for the actual driving and load testing of the test piles which had been planned, such information became part of a complete program to achieve the required pile capacities by coordinating design and execution, as described by the authors (Zeitlen et al. 1987). Figure 7 shows this methodology as a flow diagram.

8 LOADING TESTS

The loading test program was aimed at verifying the soil parameters used in the design of single open steel pipe piles driven to various depths, to check the contractor's equipment and to obtain data for preparing criteria for quality control of the work. Since soil conditions would change along the two kilometer length of the terminal, it was necessary to thoroughly study and analyze the results of the loading tests prior to starting construction. Such studies would not only enable necessary changes in design, but provide a basis

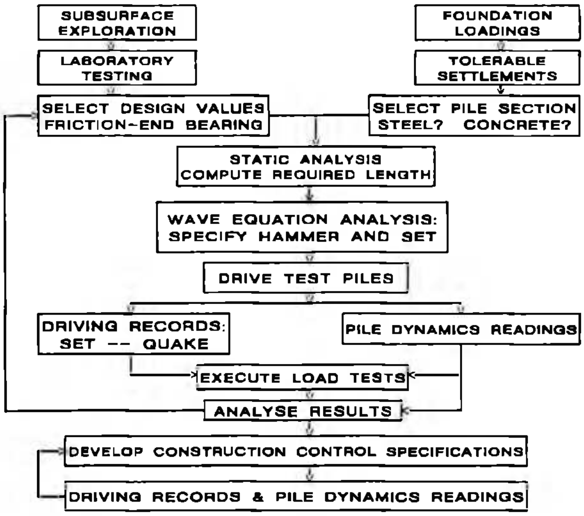


Figure 7. Driven piling - design and control. (after Zeitlen, Komornik, Wiseman, 1987)

for calibrating and controlling pile driving execution to give the required pile resistance at the different locations. Detailed pile driving records were obtained for the two test piles and reaction piles while being driven to various depths. Dynamic behavior of the piles was measured by readings taken of the stresses and accelerations of the pile tops during driving. Necessary equipment, such as strain gauges, electronic equipment, accelerometers, oscilloscope, and a tape recorder were utilized, by means of a system purchased from Pile Dynamics Inc. of Cleveland. The results were interpreted both by use of the Case Method of analysis and by CAPWAP methods proposed by Pile Dynamics Inc. to predict the bearing capacities of the piles. Studies were also made by use of computer programs developed at the Technion (called TEPWAP) which were similar to CAPWAP.

Taking into account that the design loadings necessitated different sizes of pipe piles and the changes found in the soil profile throughout the length of the jetty pier, two series of load tests were performed. One series was on a 60 inch pile at the western end of the pier and the other on a 48 inch pile near the western end of the jetty. A load reaction capacity of 20 MN was provided for the larger test pile, and 10 MN for the smaller test pile. Two driven depths were tested for each pile, with a compressive loading test and a pull out test accomplished at each depth.

Before the driving of the test and reaction piles, pairs of accelerometers and strain gauge devices (the measuring elements of the Pile Dynamics equipment) were installed near the top of each pile. They were connected by cables to the automatic recording boxes. These instruments worked well. Thus, information could be reliably obtained on the stresses in the pipes and the accelerations of the pile tops during the driving of the piles.

The loading sequences were performed in stages, according to Israel Standards No. 940 (Foundations for Buildings). This standard requires that each load be left for a period of at least half an hour and until the rate of settlements is equal or less than 0.25 mm per hour.

Figures 8 and 9 are furnished below to show two examples of the plots obtained from the compression loadings of the 60-inch and 48-inch test piles.

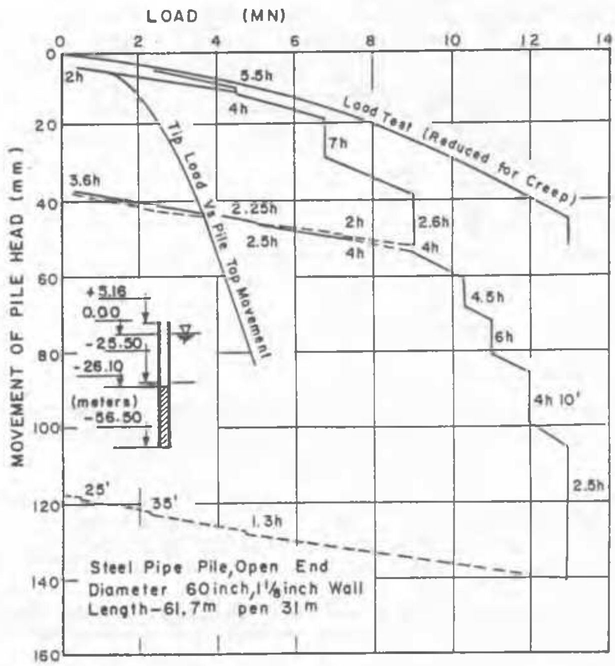


Figure 8. Test results for compression loading of 60-inch pile driven to 31 m penetration.

9 REVIEW OF LOAD TEST RESULTS

The ultimate capacity of the test piles in compression and in tension were estimated from the load-deformation curves and the time-deformation curves at each increment load. Two of the general concepts used to define the ultimate load of a tested pile were considered:

(a) Total Deformation Criteria

The ultimate load capacity of a test pile may be taken as having been achieved when a defined amount of total deformation has been reached. For example, the current Israel Foundation Code defines the pile capacity as the smallest load as found below:

(1) The load that causes the pile to settle 2.5 cm.
(2) The load that causes the pile to settle 10% of its diameter.
(3) The load which is equal to twice the load that causes a total settlement of 1.0 cm.

(b) Load-Deformation Slope Criteria

Various researchers and codes of practice have defined the ultimate load of a pile by analyzing the load-deformation curve. For example, finding the point of change in the slope of the load-deformation curve or deciding on a

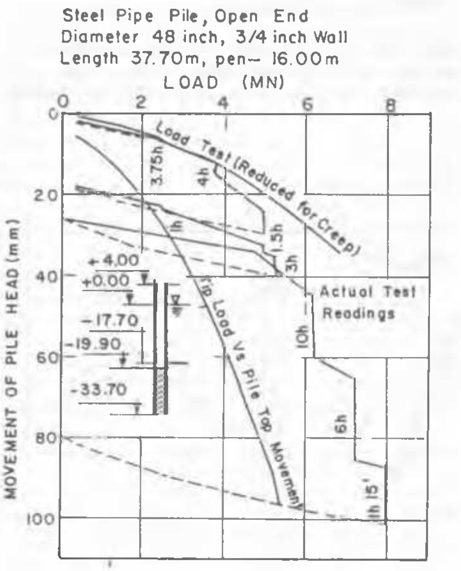


Figure 9. Test results for compression loading of 48-inch pile driven to 16 m penetration.

predetermined slope as a failure slope (e.g., a slope of 1000 kN/7.5 cm), or a combination of the two criteria with the elastic and plastic deformations taken into account (De Beer 1970, Davisson 1972, Chin 1970,1972).

In analyzing the load test results, using the above criteria, ultimate values for compression loads were selected as given in Table No. 2, below. Little differences were found in applying the various criteria for defining the ultimate loads. Table 3 summarizes the results of the pull out testing.

Table 2. Results of compression load testing.

Test No.	Diameter (inches)	Depth (m)	Ultimate Total Load (MN)	Total Friction Load** (MN)	End Bearing Load (MN)	Average Friction Stress** (kN/sq m)	Average End Bearing (kN/sq m)
T-1/A	60"	16.4	9.0	4.5	4.5	29.8	2470
T-1/B	60"	31.1	12.0	7.0	5.0	24.3	2740
T-2/A	48"	16.0	6.0	2.0	4.0	17.2	3420
T-2/B*	48"	55.5	(14.0)*	5.8	8.2	14.4	7010

* No failure was achieved during the test; maximum test loads available were 10.0 MN in compression and 4.2 MN in tension (pull out). Ultimate loads given in the table are estimated from the load deformation curves.

** Includes both friction from soil acting on the outside of pile, and from the soil plug acting on the inside of the pile wall.

Table 3. Results of pull out tests.

Test No.	Diameter (inches)	Depth (m)	Ultimate Pull out Load (MN)	Weight of Pile and Plug (MN)	Net Ultimate Pull out (Outside Side Friction) (MN)	Unit Side Friction (kN/sq m)
T-1/A	60"	16.4	2.9	.8	2.0	25.5
T-1/B	60"	31.1	4.6	1.2	3.4	22.5
T-2/A	48"	16.0	1.2	.4	.8	13.0
T-2/B*	48"	55.5	(6.2)*	.9	5.3	25.0

Each pile had been instrumented by pairs of mechanical tell-tales and pairs of electrical strain gauges to allow the measurement of strains in the pipe pile at three different levels. These gauges were to allow the determination of the distribution of forces along the pile (frictional resistance) and the resistance at the tip of the pile to be calculated for each load increment. Unfortunately, the electrical gauges did not survive the driving and immersion period well, and only the tell-tales could be depended upon to give useful results.

When comparing the results of the load tests in compression with those calculated on the basis of the static formulae, the following general comments can be made:

a. For the 60-inch pile at 16.4 m penetration, the observed total friction of 4.5 MN was appreciably more than twice the predicted external friction load of 1.0 MN, if assuming a comparison on the basis that a plug friction will be developed equal to the external friction. For the same pile driven to 31.1 m, the observed friction of 5.0 MN was not far from twice the predicted outside friction of 2.9 MN. The total loads obtained were 9.0 and 12.0 MN for the two depths, compared to predicted values of 11.3 and 16.9 MN. It was concluded that the end bearing was not able to develop properly because the soil plug slipped and did not resist fully the pressure on the pile tip. For both driving depths, the soil surface measured inside the pipe was only about 0.6 m below the sea bed level, indicating that the plug had risen in relation to the pipe while driving. The pile had acted like a "cookie cutter".

b. Examining results for the 48-inch pile, it may be seen that, at 16.0 m penetration, the observed friction of 2.0 MN was of the order of twice the predicted outside friction of 0.8 MN, but that at the deep penetration of 55.5 m, the observed friction was about the same magnitude as only the external friction prediction. The lesser friction values found in this test may be due to a combination of much lower strains in the pile developed at the greater depths, and lower loads in the upper portions of the soil plug because of much higher resistances developed in the bottom of the soil plug. However, it may be noted that the total pile resistances found in the tests on the smaller pile were close to those which would have been predicted, namely 6.7 and 12.4 MN versus 6.0 and 14.0 MN. Better plug behaviour was observed than that for the larger piles, in that the soil plug was found to be 1.2 m below sea bed when driving to 16.0 m, and 5.25 m below sea bed for the 55.5 m penetration.

c. The results of the pull out tests showed a fairly good correlation between test results and the friction during compression loading which would have been predicted from static soil parameters. The question is open as to whether the friction in tension need be taken as only 70 percent of the compression values. However, in view of the critical nature of tension loadings, and the greater effects of soil variability upon uplift, the design values were left unchanged.

(d) The unit frictional resistance in the pull out tests did show a tendency to decrease with depth (from 25.5 kN/sq m to 22.5 kN/sq m) for the 60 inch piles, while an increase with depth was shown for the smaller pile diameter of 48 inch diameter (from 13.0 kN/sq m to 25.0 kN/sq m). This increase for the smaller pile could be explained by the pile being in a different location and having penetrated much harder layers at depth when being driven to 55 meters.

e. The overall conclusions reached during the driving and loading tests were that the piles, and particularly the larger ones of the pier foundations were not able to develop their end resistances fully, because a strong soil plug was not obtained.

f. The project design would require the piles to be driven much deeper than was originally hoped, unless their capacity could be increased markedly. The contract specifications had been prepared to include provisions for cleaning out the soil plug and pouring a concrete plug, to provide a strong pile tip. This measure should provide a pile of the highest capacity, and was quite suitable for the 60-inch vertical piles. However, it would be difficult to mount a drilling rig to clean out batter piles, so that an alternate solution for such piles was desirable.

10 USE OF PLUGS TO IMPROVE END BEARING

A method was proposed, tested, and found suitable of using a steel plug welded in the pile, at about 13 to 16 meters from the tip. This plug was made in the form of an annulus, to allow the soft upper layers of soil to first flow through a hole in the center, and then for the firmer sand and sandstone layers to arch across the hole and develop resistance against the plug rising further in the pile. Figure 10 shows how concrete and steel plugs were prepared and inserted in the piles. The pile capacities were improved remarkably by this measure, as indicated by their much greater resistance to driving, as shown by Figures 11, 12, 13, and 14. One example, in Figure 13, shows how a 56-inch pile was driven to about 67 meters penetration in order to definitely reach a blow count of more than 32 blows/10 cm. In a nearby bent, another 56-inch pile, containing a steel annulus, developed the required resistance in about 36 m of penetration. These developments resulted in all of the pier vertical 60-inch

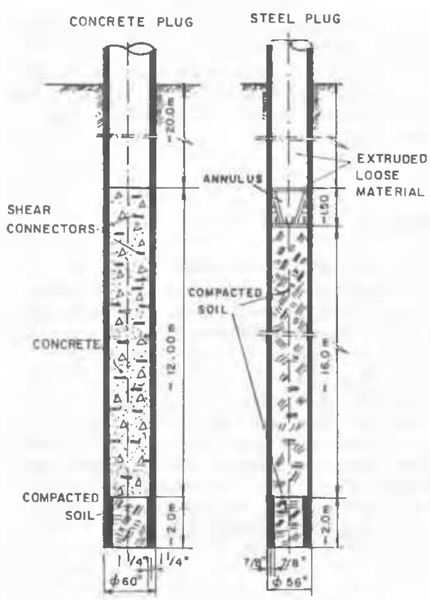
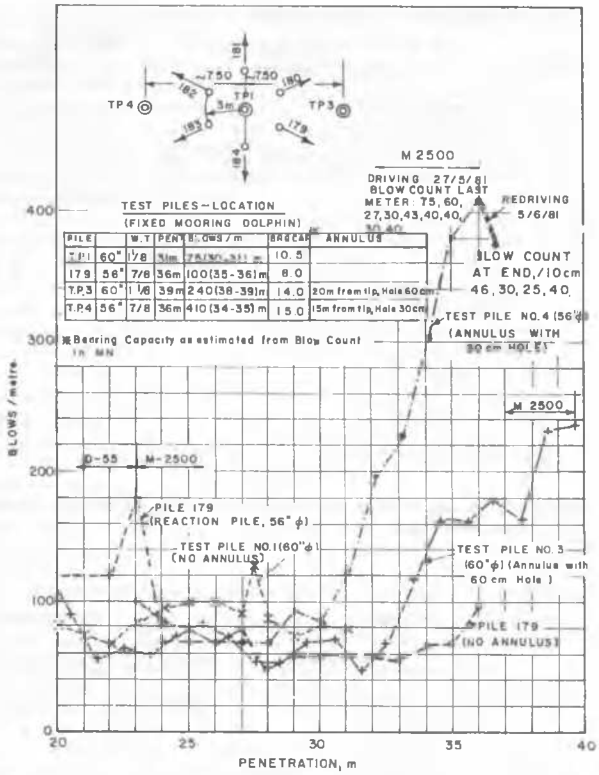


Figure 10. Concrete and steel plugs.



PILE		W.T	PENT	BLOWS / m	BRGCAP	ANNULUS
T.P.1	60"	1 1/8	[illegible]	[illegible]	10.5	
179	56"	7/8	36m	100(35-36)m	8.0	
T.P.3	60"	1 1/8	39m	240(38-39)m	14.0	20m from tip, Hole 60cm
T.P.4	56"	7/8	36m	410(34-35)m	15.0	15m from tip, Hole 30cm

Figure 11. Driving records of test piles, with and without annulus.

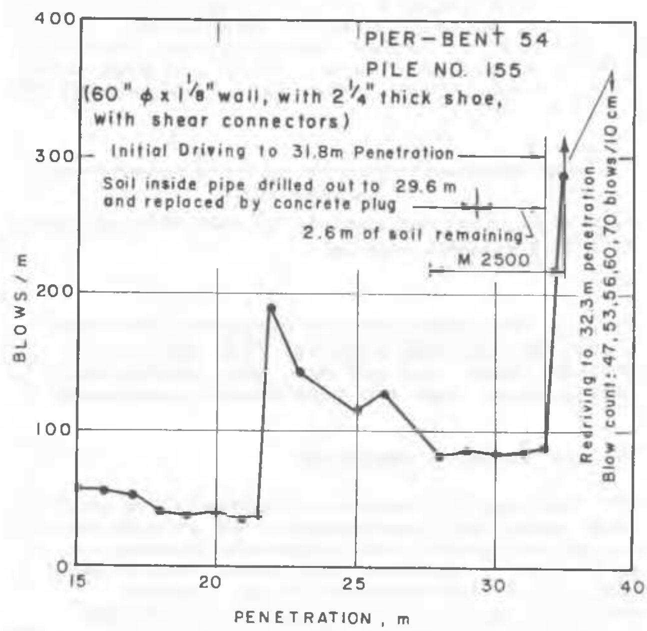


Figure 12. Driving records of a 60-inch pile, before and after installing a concrete plug.

piles being constructed with concrete plugs, and almost all of the pier batter piles with steel annuluses. In driving most of the jetty piles, extensive use was made of the steel plugs.

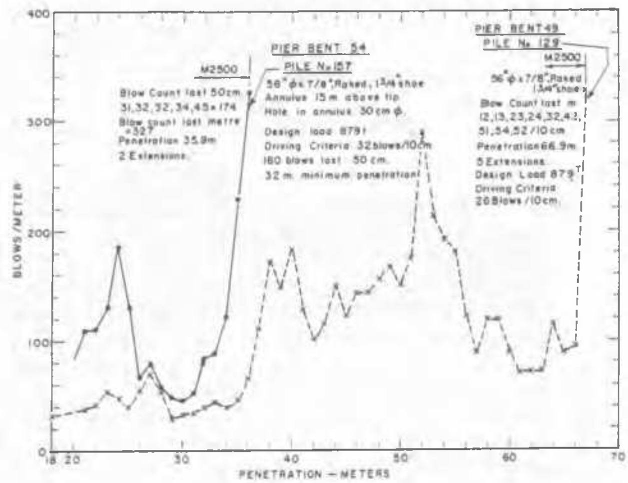


Figure 13. Driving records of two 56-inch piles, with and without annulus.

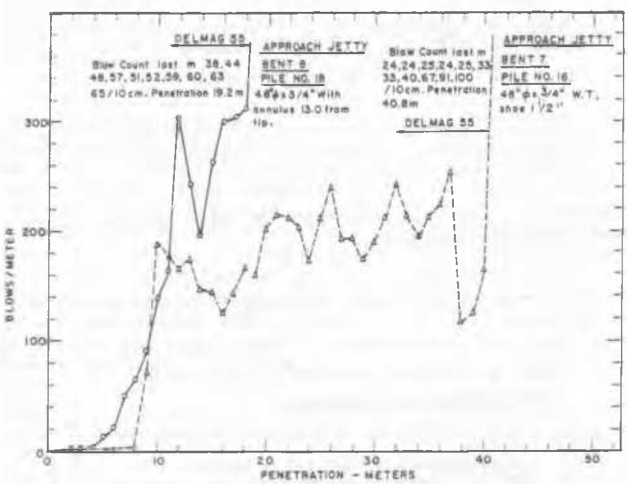


Figure 14. Driving records of two 48-inch piles, with and without annulus.

Despite the added cost of preparing the steel annuluses, savings resulted from the use of shorter piles, and, of even more importance, construction time was considerably shortened.

11 PILE DYNAMICS READINGS

The readings of strain and acceleration which were taken both continuously and intermittently during driving of test piles and production piles were invaluable for proper design and control of the execution of the project.
The data was recorded in terms of load and velocity, in order to compare the functions with the load and velocity per blow which could be predicted from wave equation analyses. Use was made of "CAPWAP", a method developed by Goble et al (1970), which determined the Smith (1960) soil resistance parameters from pile top measurements. This enabled close matching of curves of pile capacity with blow count (number of blows/meter) for particular assumptions of

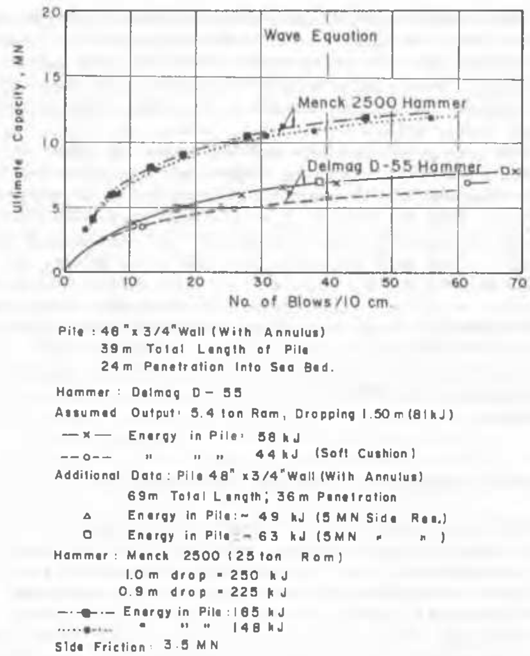


Figure 15. Pile capacity versus blow count, based on wave equation analyses after modifications from load test data.

pile and soil characteristics and hammer energy. A related program, called TEPWAP (TEchnion Pile Wave Analysis Program) and less sophisticated, was used for this project.
Figure 15 shows an example of pile capacity versus blow count curves which were used for control of driving of 48-inch piles for the jetty. The curves resulted from wave equation analyses which were developed after modifying the pile and soil characteristics according to the pile dynamics readings, as well as fit the driving and behaviour of test piles on the project. With such curves on hand, and knowing the energy delivered into the pile from pile dynamics readings taken as controls during the driving, the field inspector was able to require that driving be continued until the necessary pile resistance was obtained.

12 CONCLUDING REMARKS

The description of the geotechnical aspects involved in the design and construction of the Hadera Offshore Coal Unloading Terminal should be recognized as more than simply a case history. An example is shown of an approach which involves coordination of the various phases of field investigations, driving of test piling, loading tests, and advising on the actual execution of the project piling. Follow up observations and measurements of foundation movements have shown that the performance of these structures have been satisfactory in all respects, and well within the tolerances required.

13 ACKNOWLEDGEMENTS

Appreciation is due to the owners, the Israel Electric Corporation, Ltd. for their complete confidence and faith in the consulting advice furnished, as well as their cooperation in supporting the necessary investigations and testing. Particularly responsible were Eng. P. Porath, Chief Civil Engineer, and Eng. D. Zander, Project Engineer.

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