LONG TERM GEOTECHNICAL STUDIES AT THE HSIN-TA POWER PLANT SITE

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SYNOPSIS: The Hsin-Ta Fossil Power Plant is constructed on a piece of reclaimed land in 1978. All the major structures are supported by piles. Due to the presence of deep seated compressible soil strata, the entire site including the piled structures has been progressively settling. In 1982, the settlement rate has accelerated because of increase of pumping of groundwater for fishery farms in areas adjacent to the Plant. In 1987, instrumented test piles were installed to verify the development of negative skin friction due to subsidence. This Paper describes the geotechnical conditions of the site, foundation considerations in the early stage, settlements of the existing structures, long term groundwater monitoring results and subsidence of the area. Discussions also cover the negative skin friction monitoring of the test piles.

1. INTRODUCTION

The Hsin-Ta Fossil Plant is one of the important power plants located in the southern part of Taiwan. The Plant is planned to have eight power generating units and many associated facilities including oil tanks, stacks, circulating water ducts and water storage reservoir, etc. The project site is a piece of reclaimed land with 2-5 m of hydraulic fill placed on top of soft deposits. Since the construction in 1978, 4 units of power generators, 2 high stacks and associated facilities have been completed. Power generating units 5 and 6 are being planned for construction. The entire project is divided into 5 phases of development.

This Paper describes the long term geotechnical studies at the site since 1978. Many of these studies are still on-going. The results of study obtained so far illustrate the unique characteristics of the project.

2. GEOTECHNICAL CONDITIONS OF THE SITE

The Plant has an area of 2,000 m by 750 m. The original ground surface after reclamation was 1 m above the mean sea level. The subsoil condition at the site is relatively uniform. From ground surface (El.0-2 m) downward, it can be divided into several layers as follows:

(1) Hydraulic Fill - The hydraulic fill consists of 2 to 5 m thick medium to fine loose sand containing 10 to 25 percent fine. The SPT N-values vary between 3 and 10 giving relative density of about 40 percent.

(2) Soft Silty Clay - The soft silty clay layer is about 14 m thick . This layer has high water content of 40-60 percent and medium to high plasticity. A layer of 2 m medium dense fine sand was found intermixed within the clay layer. Some sea shells, organic matter and silt lenses were found indicating that this layer is a marine deposit.

(3) Silty Sand - Below the silty clay is a layer of medium dense to dense silty sand of 70 m thick. Within this layer, thin layers or seams of fine sandy silt, silt or clayey silt were found at various depths.

(4) Stiff Silty Clay - Underlying the silty sand stratum, a layer of silty clay extends from depth of 90 m to 150 m. It is stiff to very stiff, low to medium plasticity, stratified with thin layers of silt and clayey silt.

The general characteristics of the subsoils are presented in Table 1.

Depth, m		Description	N Value	γ_t, t/m³	W_n, %	S_u, t/m²	$\bar{c}$, t/m²	Φ, deg.
		Silty fine sand	10	1.89	25	-	0	33
10		Silty soft clay	2	1.84	38	2.2	-	-
		Silty fine sand	12	1.93	25	-	0	32.5
		Soft silty clay	4	1.88	34	3.1	-	-
20		Silty fine sand	27	1.91	26	-	0	34
		Silty clay	12	1.90	32	4.7	-	-
30		Silty fine sand	24	1.95	27	-	0	34
40			49	1.99	23	-	0	35
50			45	2.00	30	-	-	-
60		Sandy silt or clayey silt	30	1.98	26	16.9	0	38.5
70, 80		Silty sand	50	2.00	24	-	-	-
90		Sandy silt	35	1.92	28	-	-	-
		Stiff silty clay	20	2.00	27	18.5	0	39.5

Table 1 General characteristics of the subsoils

3. FOUNDATION CONSIDERATIONS IN THE EARLY STAGE

In early 1979, shortly after the completion of hydraulic filling, exploration work covered the entire site was carried out. More than 100 boreholes were sunk to 40 m depth with a few boreholes drilled down to 50-70 m. It was generally believed then that the thick silty sand layer is a feasible bearing layer for the foundation piles.

There were few major geotechnical concerns for design of foundation at that time and the following solutions were adopted:

(1) Densification of the top hydraulic fill layer to overcome liquefaction problem - Compaction sand piles were used to improve the surface sand deposit. Compaction sand piles of 70 cm diameter, 7.5 m long with sand volume of 5 cubic meter per pile were installed at 1.8 m center to center in a triangular pattern. The relative density of this layer after improvement was increased to over 75 percent. (Woo et al, 1980; Moh et al, 1981)

(2) Preconsolidation of site for oil storage tanks- for supporting the oil storage tanks, the site area was improved by using preloading with surcharge and sand drains.

(3) Selection of type and length of piles - Thirty four (34) m long closed- end steel pipe piles of 508 mm diameter were driven to support the generator units 1 and 2, and the associated stack (Soo et al, 1980). For generator units 3 and 4, thirty four (34) m long and 60 cm diameter prestressed concrete piles were used.

(4) Negative skin friction on pile due to compression of the soft silty clay layer loaded by the hydraulic fill - Ninety (90) tons of negative skin friction per pile was designed to account for potential downdrag force of the top 20 m of the compressible soils.

4. SETTLEMENT STUDIES OF STRUCTURES (1979-1983)

For the purpose of monitoring general ground settlement and construction survey control, a total of 32 numbers of 10 m long and 9 numbers of 34 m long concrete piles were driven into the ground around the site as control bench marks. Surface settlement points were also installed in the main structures.

Figures 1 and 2 present the settlement contours of the control piles in the period of July 1979 to May 1983. The settlements of piles were mainly due to the consolidation of subsoils under the hydraulic fill surcharge. The piles especially the 10 m short piles settled in direct proportion to the height of fill (Fig. 3). The 34 m long piles, though resting in the dense sand layer, also settled, mostly in the order of 5 cm.

The settlements of those structures supported by 34 m long piles and their settlement rates are shown in Table 2. Comparing the settlements of the structures with that of the control piles, loads from the structures also caused significant amount of settlement of the piles. Among all the structures, the tilting of the 250 m high stack is the most serious. Possible causes of structure settlements were investigated, these included:

(1) insufficient bearing capacity of the 34 m long piles,

(2) ground vibration induced from operation equipments,

(3) variation of groundwater pressure,

(4) deep-seated sliding or plastic flow of subsoil toward the sea under hydraulic fill, and

(5) existence of compressible soils layers below the pile tip, in this case 34 m.

Deep check borings were sunk down to 150 m deep for the first time at this site. Results indicated that there is a stiff silty clay layer existing below the depth of 90 m. Settlement of structures could be due to the compression of compressible layers below pile tip. Based on the consolidation characteristics of the deep seated soil strata, settlement rates of structures were back figured as presented in Fig. 4. As before 1982, the monitored data matched well with the calculated settlement values. However, after 1982, an abnormal phenomenon was noticed, the settlement rate of structures started to increase. Factors other than structure loading alone must have contributed to the increasing rate of settlement.

5. GEOTECHNICAL STUDIES (1983 - 1988)

The causes responsible for the increasing settlement rate after 1982 were not studied until 1986, when the total settlement of the stacks had reached 23 cm with a differential settlement of 8 cm. (Fig. 5) It was alarmed that the progressive settlement and tilting of

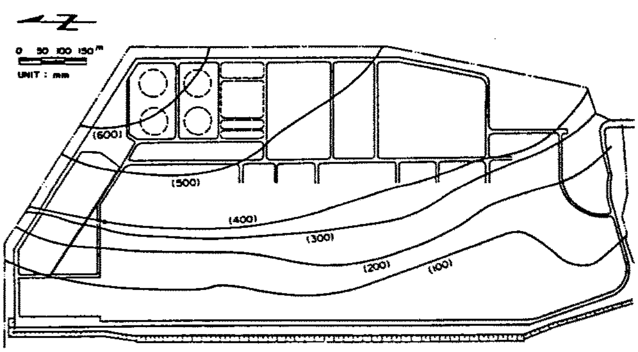

Figure 1 Settlement contours of the 10m piles (July 1979 to May 1983)

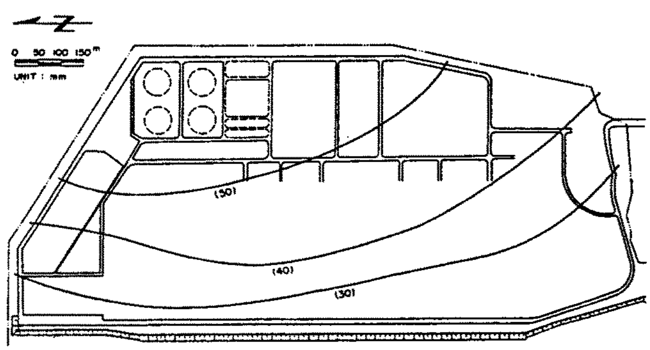

Figure 2 Settlement contours of the 34m piles (July 1980 to May 1983)

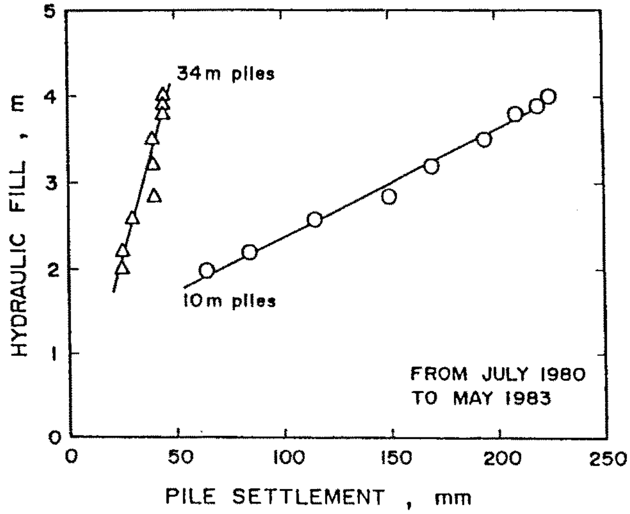

Figure 3 Piles settlement in proportional to height of fill

Settlement Points on Foundation	Starting Date of Monitoring	Settlement Rate, mm/month				Total Settlement as up to July 1983 mm
		Jan 80	Jan 81-Jan 82	Jan 82-Jan 83	Jan 83-July 83	
Building, Unit No.1	'80.11.17	9.5	6.2	5.3	5.2	195
Building, Unit No.2	'80, 8.11	3.5-6.2	4.2-4.4	4.5-5.5	5.8	164-194
Furnace, Unit No.1	'79.10, 6	2.7-3	3.4-4	5-5.5	5.2-5.7	184-200
Furnace, Unit No.2	'80, 5, 8	1.8-2.3	2.2-3.3	3.5-5	6-6.3	128-163
Turbine, Unit No.1	'80, 8.25	1.8-2.2	3.8-4	5.3-5.6	5.2-5.3	158-181
Turbine, Unit No.2	'81, 3.15	-	2.3-3.2	4.3-5	5.8-6.2	117-135
Stack, Units 1 & 2	'80, 5.27	0.9-1.6	0.4-0.8	1.5-3.5	1.5-3.7	40-93

Table 2 Settlement of foundations

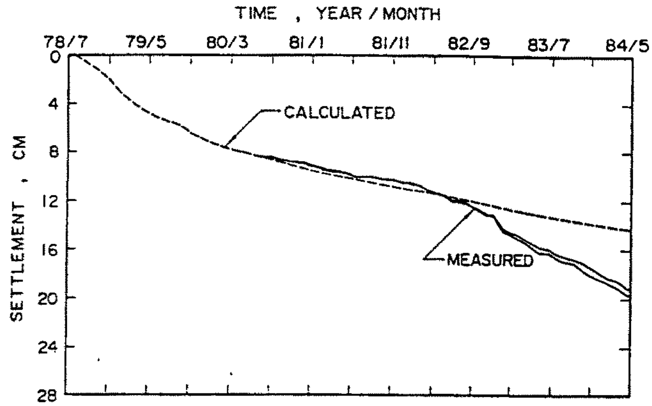

Figure 4 Comparison of back figured settlement rate with measured results of the stack foundation

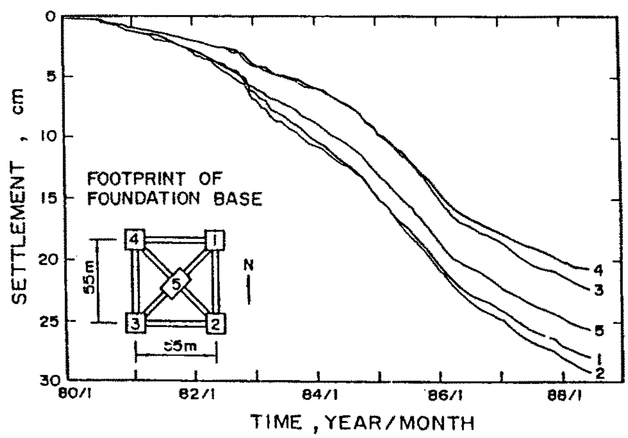

Figure 5 Settlement of stack foundation of units 1 & 2

the high stacks might endanger the operation of the Plant. A comprehensive study was then called for to investigate the safety of the stacks and the causes of the accelerating settlement. Fortunately, results of structural analysis indicated that the stacks were still safe. (Moh and Associates, 1988)

5.1 Groundwater Study

The groundwater conditions within the site, and for the first time, in the adjacent areas were monitored. Past records of water wells were also collected for study. From the collected data, it was concluded that the increased settlement of the structures since 1982 was caused by increase of pumping of groundwater for fishery farms in areas adjacent to the power plant.

In the early 1980's, numerous fish ponds were made in the nearby area of the power plant for fishfarming. About half of these ponds need fresh water throughout the year. The highest consumption of groundwater is normally in the late winter season when "warm" groundwater is needed to raise the water temperature in a pond (Fig. 6). In order to meet the large consumption of fresh water, hundreds of pumping wells of 30-90 m deep were sunk to extract groundwater from the underlying sand layers. As almost all these pumping wells were drilled illegally, no pumping records could be found. Therefore the exact quantity of water pumped out from the ground was unknown. However a study by Academic Sinica (1987) estimates that 67 million tons of groundwater per year is needed for this area which is about 3 km by 5.5 km in total area. Due to the over-extraction of groundwater, the water level in the subsoil has been lowered significantly. Fig. 7 presents observation results from three wells which are located within 5 km from the Plant. A clear drop of the groundwater surface is seen starting from 1982. Similar variations of the piezometric pressures (Fig.8) were monitored within the Plant area indicating that the groundwater regime in the Plant area has been greatly affected by the well pumpings. Generally, the groundwater table in the Plant area appears to be lower towards the location where highest density of pumping wells were drilled (Fig. 9). Similar inclination of groundwater table was also observed in the surrounding areas. In Fig.8 the piezometric pressure head measured in the silty clay layers (OT5-16 and OT5-10) in the Plant are at about 4.5 tons per sq m higher

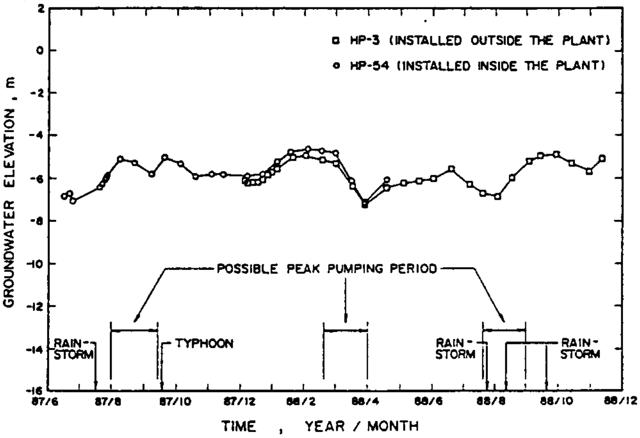

Figure 6 Fluctuation of groundwater elevation from piezometers at 30m depth

than that in the sand layer (HP-16) in 1987. This indicates that the silty clay layers above 20 m depth were still undergoing consolidation. At depth below 20 m, the groundwater pressure was found lower than hydrostatic (Fig. 10).

5.2 Measurements of Subsidence

Settlement measurements of the 10 m and 34 m long control piles mentioned in Sec. 4 were continued monthly to keep track of the site subsidence. Fig. 11 presents the settlement-time relationships of the two types of piles. The measured results for a period of eight years indicate that the site was still settling. The subsidence appears to be more serious after 1982 due to the effect of uncontrolled well pumping. Subsidence rate of about 1 mm - 4 mm per month was recorded. The progressive subsidence of the site may pose serious downdrag forces to the piles installed. An accurate estimate of the magnitude of the downdrag forces (or negative skin friction) becomes essential for evaluation of foundation safety.

5.3 Measurement of Negative Skin Friction

Two instrumented steel pipe piles were driven

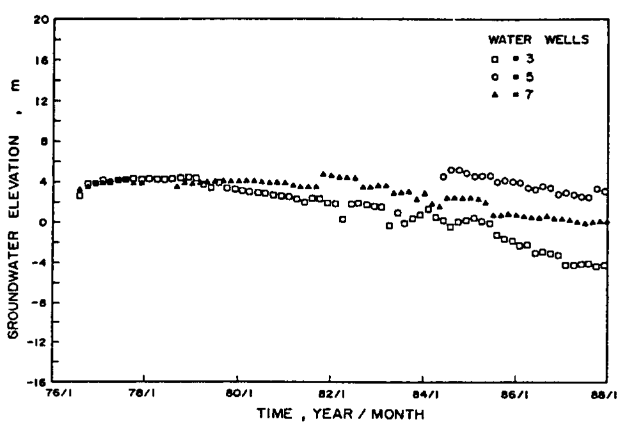

Figure 7 Monitored results of water wells

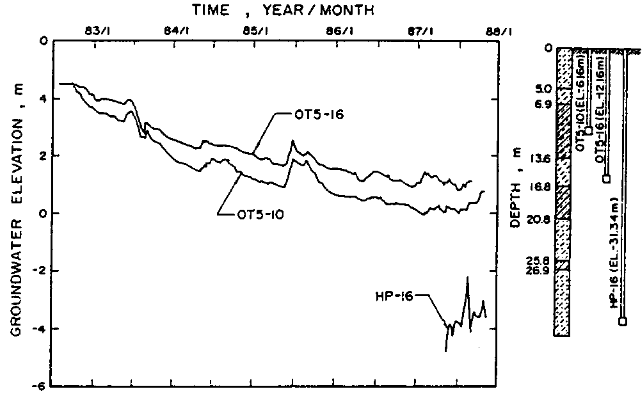

Figure 8 Measured groundwater elevation inside the Plant

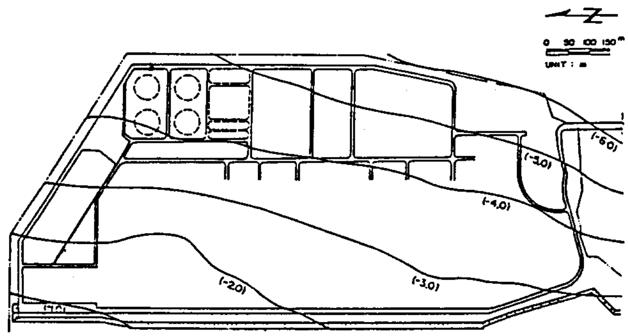

Figure 9 Groundwater elevation as measured at 35m depth on July 15 1988

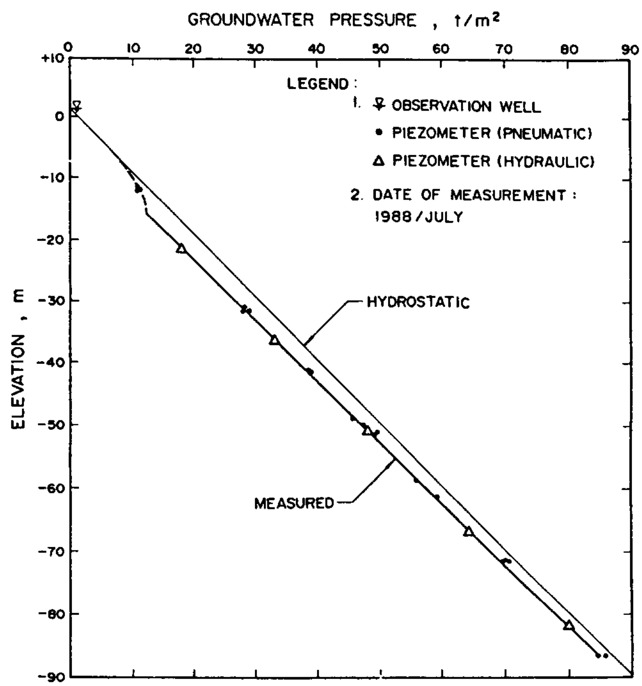

Figure 10 Distribution of groundwater pressure in the Plant

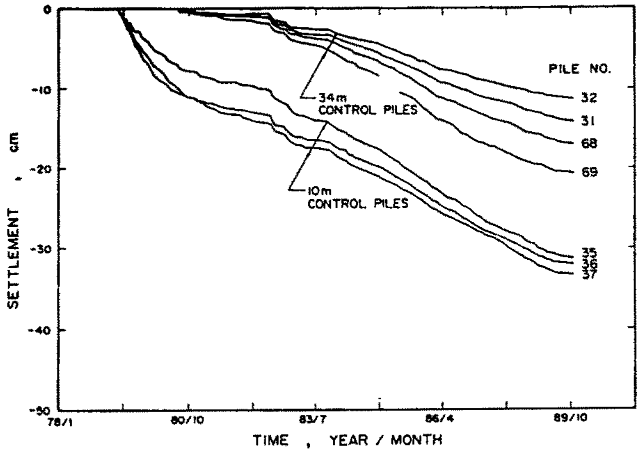

Figure 11 Settlement of the control piles

in May 1987 for study of negative skin friction. The closed-end pipe piles designated as TP4 and TP6 were 36 m long, 609 mm outside diameter with wall thickness of 12 mm. The piles were driven 34.25 m into soil and were about 70 m apart. Six pairs of strain gauge (OYO HS-10 type) were installed inside each pile at various elevations. The average readings of each pair of gauges were converted to axial stress of the pile. A pair of tell-tale was also installed at the pile tip extending through the pile to the ground surface. Dial gauges were used to measure the relative movements between the pile tip and the pile top.

Test pile TP4 was loaded to 430 tons and Test pile TP6 to 446 tons before unloaded to zero. Because of the slight variation of the subsoil conditions at the tips of the two piles, the end bearing resistances were found to be very much different. TP4 had an ultimate bearing capacity of 423 tons whilst TP6 did not fail at 446 tons, having an estimated capacity of about 600 tons.

After the load tests, the piles were left in the ground with no loading for observation of downdrag from July 1987 till February 1989. Figure 12 & 13 present the increase of axial force in the pile at various depths during the monitoring period. The axis force of the test pile measured 3 days after the load test was taken as the initial base for long term downdrag monitoring. The following observations were made from the data of the two test piles:

(1) The axial force of the pile increased with time. The increase was more rapid in the first six months. After one year, only small fluctuations were observed.

(2) The maximum load as interpreted due to downdrag was about 65 tons. Maximum load occurred at depth of about 20 m, indicating that the neutral point was located below the thick silty clay layers.

(3) Between the depth range of 20-26 m, the pile had positive friction along the shaft in the sand layers and had developed maximum friction in the first month. The later progressive increase of pile stress was mainly attributed to the increase of end bearing, and increase of pile friction in the clay layers below 26 m depth.

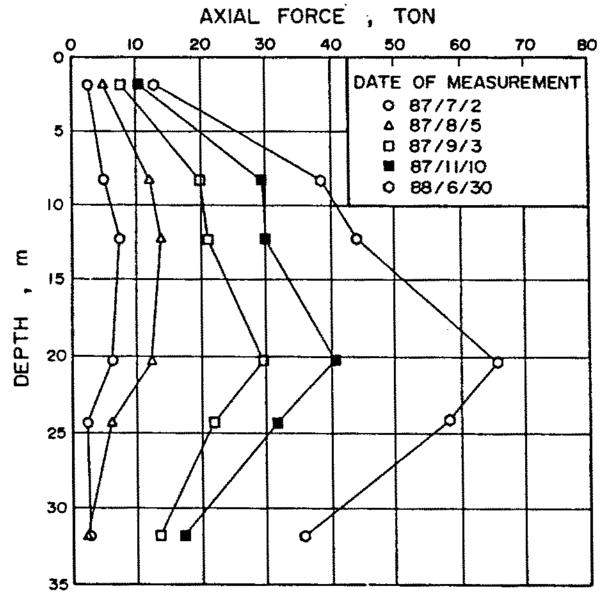

Figure 12 Increase of axial force in pile TP4 due to negative skin friction

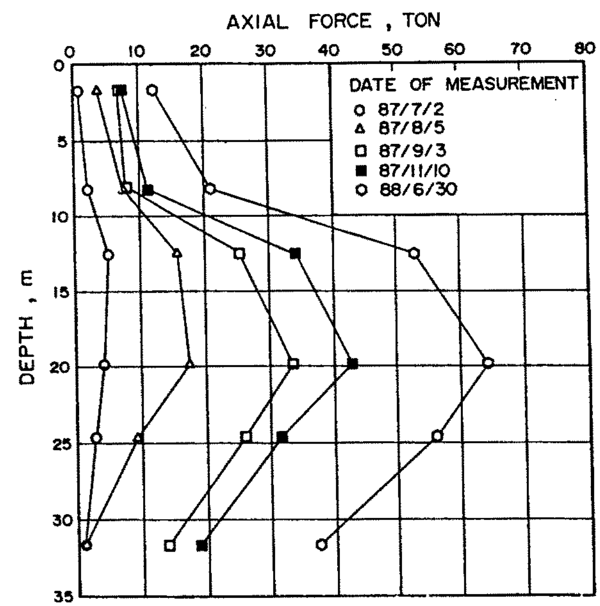

Figure 13 Increase of axial force in pile TP6 due to negative skin friction

(4) The stress of the pile increased with lowering of groundwater pressure, even if it was a slight fluctuation.

(5) As at downdrag of 65 tons, shortening of pile was found to be 1.2 to 2.0 mm. During this period the ground surface settled about 2 to 3 cm.

Based on the effective stress analysis method (NACFAC, 1982), downdrag or negative skin friction varies in proportional to the effective overburden pressure.

$$f_n = \beta \, \overline{P}_o$$

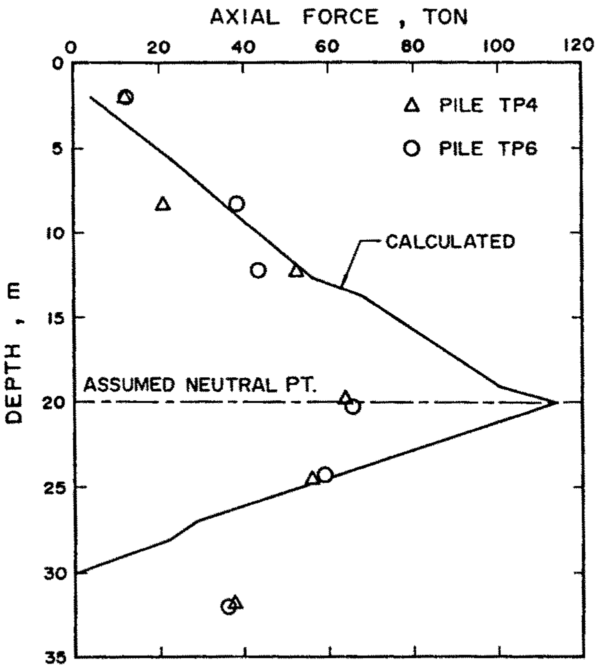

Figure 14 Comparison of analytical load distribution with measured results

where $\bar{P}_o$ = effective overburden pressure, assumed constant at depth below 20 times of pile diameter,

β = 0.5 for sand, 0.25 for clay.

Assuming the neutral point is at 20 m depth, Fig. 14 compares the analytical distribution of load along the piles with the measured data on 88/6/30. It is seen that full negative skin friction has been developed at top 12 m. The portion adjacent to the neutral point and below has only developed partly. This is probably due to the fact that the pile/soil relative movement was too small to induce full friction at lower depth.

From the above downdrag study, it can be concluded that the piles originally designed to take 90 ton negative skin friction are still adequate.

6. CURRENT STUDIES

Several studies are continuing at the Plant site, including:

(1) Continuing subsidence monitoring for the whole Plant.

(2) Settlement survey of major structures, especially the foundation settlement of the high stacks.

(3) Long term observation of groundwater pressure in the Plant as well as in the adjacent areas.

(4) Long term settlement monitoring of the adjacent areas.

It is anticipated that these studies will continue for sometime, possibly 3 to 5 years, in order to gather more meaningful information. As up to now, the fishfarming industry is still rapidly growing, there is no sign of reduction of the rate of subsidence.

7. CONCLUSIONS

The Hsin-Ta Fossil Power Plant has given a special opportunity to carry out complicated geotechnical case studies at various stages of construction. Inherently, the subsoils which consist of a layer of hydraulic fill sand on top of a thick layer of soft clay are difficult to handle. Methods of soil treatment, such as compaction sand piles, sand drains with preloading were used to improve the weak subsoils. Steel pipe piles and concrete piles were used to support major structures. Although the piles have given sufficient supports to all the structures, large pile settlements were observed because of the existence of silt/clay layers below the pile tip which extends to great depth. Deep well pumping from adjacent areas starting in 1982 has caused serious subsidence to the Plant area. This leads to studies of groundwater pressure changes and settlement of structures. Because of the site subsidence, considerable amount of downdrag forces are induced onto the piles. A detailed investigation was carried out to evaluate the safety of the structures. It was found that the amount of negative skin friction induced by ground subsidence was within the estimated limit of the original design. Monitoring of the long term regional subsidence is being continued till the year of 1990.

From this study it can be concluded that during the life span of a complex project, different geotechnical problems may arise at any time. Many of the influence factors are foreseenable, whilst many of them are not. Hence a comprehensive site investigation and long term geotechnical monitoring studies are essential for the success of such a project.

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