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INTERPRETATION OF INSTRUMENTED DRIVEN STEEL PIPE PILES

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ABSTRACT: This paper presents the results of a series of studies on the behavior of driven close-ended steel pipe piles. Extensive field explorations and geotechnical analyses have been carried out. Load tests were conducted, and strain gages and tell-tales were installed on tested piles. The behavior of the instrumented piles has been monitored for 14 months since pile driving. One emphasis of this paper is upon the magnitude and distribution of residual stress after pile driving and load testing. This paper also illustrates the setup of pile capacities after driving, the difference in the ultimate skin friction of pile under compression and under tension, and the development of negative skin friction after pile load testing.

INTRODUCTION

The study site, Hsin-Ta Power Plant, reclaimed with 3 to 4 m of hydraulic fill in 1978, is situated on the southern coast of Taiwan. Four power generator units of 50 to 55 megawatts and two stacks 250 m in height have already been installed; in addition, two generator units and one stack are being planned for construction. All the major structures completed so far are supported by close-ended steel pipe piles or prestressed concrete piles approximately 34 m in length. Including the piled structures, the entire plant site has been progressively settling at a rate of 0.1 to 0.4 mm per month for the last 5 years due to excessive groundwater pumping resulting from fish farming in adjacent areas. As an engineering effort to ensure the safety of the structures, especially of the high stacks, a long-term monitoring program for the ground settlement and groundwater conditions has been conducted since May 1987. Furthermore, an elaborate pile load test and instrumentation program has been

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carried out since July 1987 at the proposed site of the generators and stack in order to evaluate the pile capacities as well as the development of negative skin friction due to subsidence. A total of three prestressed concrete piles, designated TP1 to TP3, and three close-ended steel pipe piles, designated TP4 to TP6, are included in the pile test program. Two of the steel pipe piles, TP4 and TP6, were instrumented with strain gages and tell-tales. In this paper, focus is placed on the behavior of the three steel pipe piles, particularly the two instrumented pipe piles.

SUBSOIL CONDITIONS

According to the results of field explorations conducted adjacent to the test piles in January 1987, three soil profiles delineating the specific subsoil conditions of TP4, TP5, and TP6 are shown in Fig. 1. The distance between each of the three piles is about 55 to 70 m. Within the depths of primary concern for pile design, the subsoils can be generally divided into three main layers. The top layer consists of hydraulic sand fill and natural sand, with a total thickness of approximately 6 m. Underlying that layer is a clay layer of about 13 m thickness with a thin interlayer of sandy soils. Immediately below that clay layer is a sand layer which is generally as thick as 35 m, interbedded at depths of approximately 28 to 36 m with clay sublayers 1 to 2 m in thickness. These interbedded clay sublayers may significantly affect the point resistance of piles if encountered in the vicinity of the pile tips. Figure 1 indicates

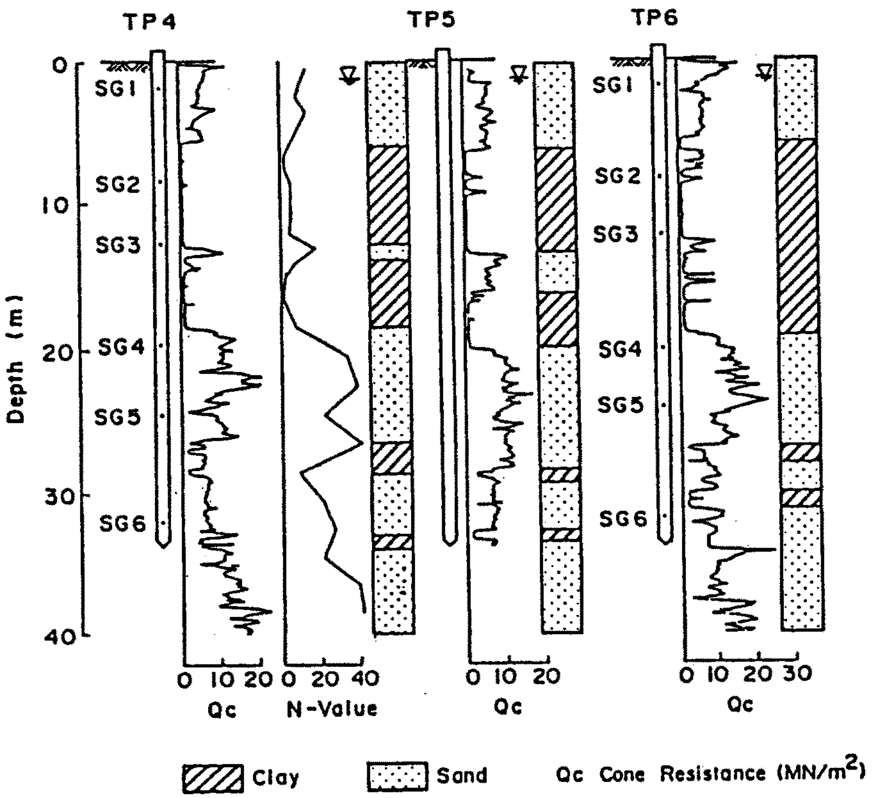

Figure 1. Subsoil Conditions and Strain Gage Locations of Test Piles

that the soil deposits above the three pile tips have no significant differences.

PILE LOAD TEST AND INSTRUMENTATION PROGRAM

Pile Load Test. The three steel pipe piles (TP4, TP5, TP6) are all close-ended and of the same size: 36 m in length, 0.609 m in outside diameter, and 12 mm in wall thickness. All the test piles were driven with a Delmag D62-22 diesel hammer. The pile driving data for the three test piles are summarized in Table 1. After the piles were driven to depths of 26 m, a Pile Driving Analyzer (PDA) was used at an interval of 2 m installation length to collect the dynamic records of the force and velocity at the top of piles.

Compression load tests were cyclically performed on TP4 and TP6 using the quick load method in accordance with ASTM D1143-81. The maximum number of loading cycles was 4 if the pile did not test to failure. As for TP5, the tension load test was conducted, and the slow load method complying with ASTM D3689-83 was adopted. It was loaded to a maximum uplift load of 2500 kN after 4 cycles. Table 2 summarizes the test data of the three test piles.

Pile Instrumentation. Two independent instrumentation systems were installed to monitor the load transfer behavior along the pile shaft. The first system consists of 12 anti-shock electric-resistance type strain gages, OYO Model HS-10, installed on each of the test piles TP4 and TP6. The strain gages, fixed on steel mounts welded to the piles, were symmetrically installed in pairs at 6 locations along the internal face of the steel pipe piles. The representative axial force corresponding to each pair of gages was calculated on the basis of elastic theory. Before being installed in

Table 1. Summary of Pile Driving Data

Test Pile No.	Installation Date (m/d/y)	Final Resistance (blows/ft)	Embedded Length (m)
TP4	5/21/87	15	34.25
TP5	5/22/87	15	34.25
TP6	5/22/87	16	34.25

Table 2. Summary of Pile Load Test

Test Pile No.	Type of Test	Test Date (m/d/y)	Test Cycles	Maximum Test Load (kN)
TP4	Compression	6/22/87 - 6/23/87	3	4330
TP6	Compression	6/17/87 - 6/19/87	1	4460
TP5	Tension	6/19/87 - 6/22/87	4	2500

the field, two pairs of strain gages installed on H-beams had been calibrated on a universal machine and compressed to 150 MN/m2. The calibration indicates that the strain gages have a linear stress-strain relationship and behave very reliably even after many cycles of loading and unloading.

Another monitoring system consists of 2 aluminum tell-tales symmetrically installed on each TP4 and TP6. Attached to the bottom of the test piles, the tell-tale rods of 3 mm O.D. are completely separate from the test piles in order to measure the relative shaft deformation between the bottom and top of the piles.

PILE LOAD TEST RESULTS

Figure 2 presents the compression test load versus pile head displacement curves for test piles TP4 and TP6; Fig. 3 shows the tension load versus pile displacement curve for TP5. The rupture load of TP4 clearly defines its ultimate compression capacity to be about 4150 kN. However, pile capacities of TP5 and TP6 could not be as easily defined as TP4. Therefore, various interpretation methods as discussed by Fellenius (4) were applied to evaluate the ultimate capacities of all these test piles. Table 3 summarizes the interpretation results and indicates the following: (i) The ultimate pile capacity varies considerably with interpretation method. (ii)

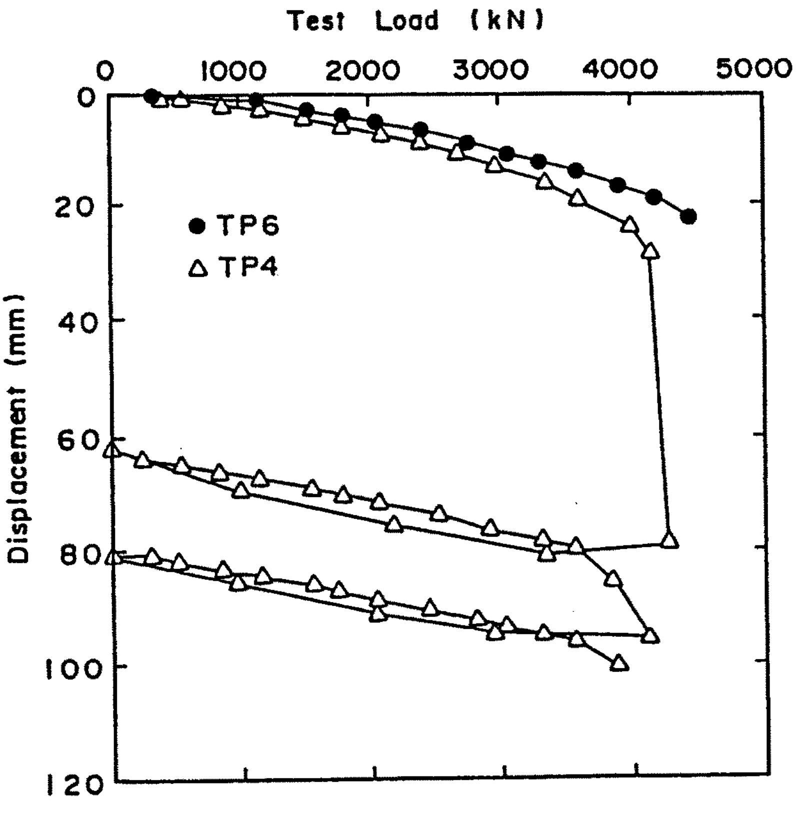

Figure 2. Load Versus Pile Head Displacement Curves for Compression Test on TP4 and TP6

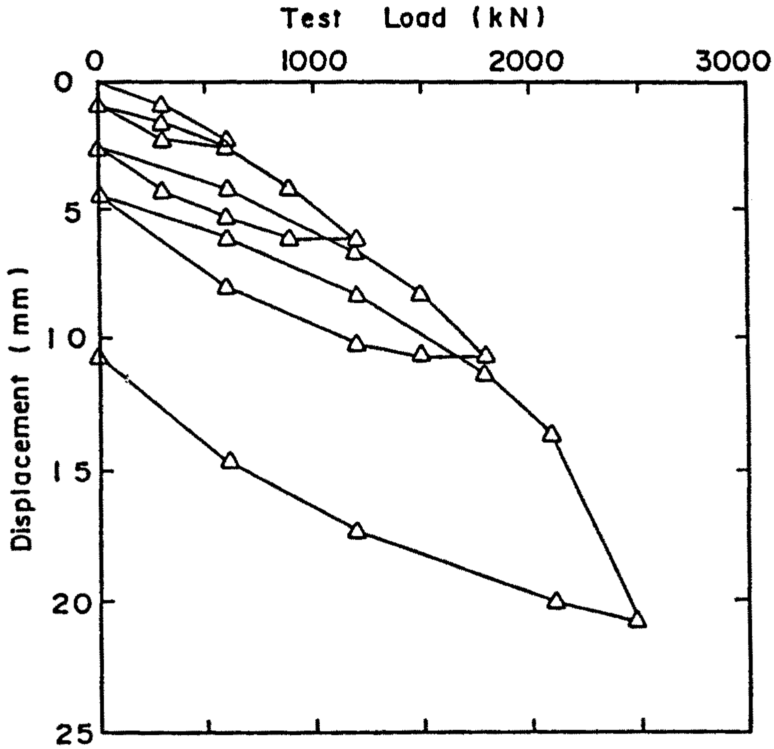

Figure 3. Load Versus Pile Head Displacement for Tension Test on TP5

Table 3. Pile Capacities Interpreted from Pile Load Test Results

Test Pile No.	Ultimate Pile Capacity (kN)						Used in this Study (kN)
	Chin	Davi-sson	Fuller& Hoy	Butler& Hoy	Hansen	Terzaghi	
TP4[a]	4800	4200	4250	3900	3850	4240	4150[c]
TP6[a]	6120	4920	5440	5010	4910	5290	4910-6120
TP5[b]	3240	2410	2780	2490	2600	2860	2630

a - compression pile load test
b - tension pile load test
c - according to rupture load

Chin's method always gives the highest values for the three test piles. (iii) For TP4, compared with the rupture load from the test results, the ultimate capacity interpreted by Chin's method is much higher than the rupture load. On the other hand, those interpreted by Butler & Hoy's method and by Hansen's method are in general more conservative. Davisson's method can give a very good assessment of rupture load. (iv) For the tension load test performed on TP5, except for that interpreted from Chin's method, the ultimate pile capacities interpreted from the other methods fall in a narrow range from 2410 to 2860 kN. The average value 2630 kN is used for further analysis. (v) Since test pile TP6 has penetrated 3 m into the dense sand bearing stratum, the pile head displacement under the maximum test load was significantly less than TP4. Consequently, the ultimate capacity interpreted by various methods varies significantly from 4910 to 6120 kN. This range of pile capacity will be used in this study.

RESIDUAL STRESS

During the downward movement of a pile due to pile driving, skin friction of soils acts upward, i.e. positive, on the pile to resist the pile penetration (Fig. 4a). At that time, the point resistance at the pile tip also acts upward. As the driving is completed, the soil skin friction will be reversed (Fig. 4b), i.e. become negative, which is similar to the conventionally-defined negative skin friction caused by ground subsidence, due to the rebound effect caused not only by the pushing of the point resistance at the pile tip, but also by the elastic decompression of the pile itself. After that, the piles will remain still, and the compressive stresses will be locked into the soil and pile. That compressive stress is called residual stress due to soil-pile rebound after pile driving (1,2,3, 8, 9,10,). Depending on the extent of rebound, the residual stress may exist along the entire pile shaft if rebound is great enough. In addition to those stresses resulting from rebound, stresses are also induced in piles due to local permanent deformation of pile shaft incurred during severe impact driving. These additional stresses will make the total residual stresses much higher.

BEHAVIOR OF INSTRUMENTED PILES AFTER DRIVING

In studying the behavior of driven piles, it is very important to

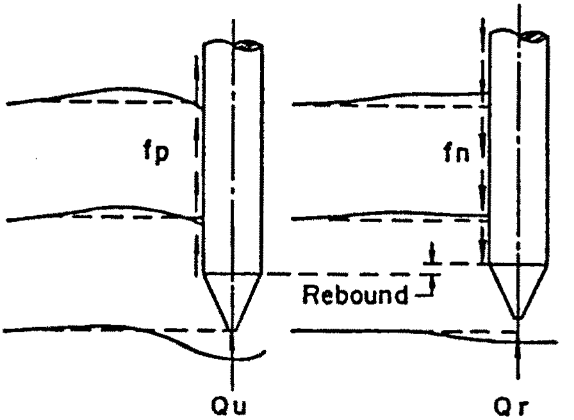

Figure 4. Illustration of Probable Distribution of Skin Friction and Point Resistance on Pile (Adapted from Cooke, 1979, p. 235)

make a clear distinction between the concept of true axial force and apparent axial force of the piles. It is noted that residual stress will develop due to pile driving. Therefore, the strain gage readings taken prior to pile driving should be considered as the initial readings to calculate the true axial forces that a pile will experience during its entire loading history. Because the predriving data are considered as the initial basis, any residual stresses due to pile driving are accounted for. Readings at any other stage can be used as reference readings; however, the axial forces calculated on the basis of the reference readings should only be regarded as apparent axial forces.

Residual Forces Due to Pile Driving. The monitored pile forces at TP4 and TP6 immediately after pile driving are presented in Fig. 5. The monitoring results indicate that residual forces indeed locked in the driven piles due to pile driving. The extremely high forces of SG3, SG4, and SG5 at TP4, as well as SG2 and SG4 at TP6 were most likely due to the local buckling or neutral-axis shifting of the instrumented steel pipe piles caused by the severe driving impact. The other factor that probably introduced the high axial

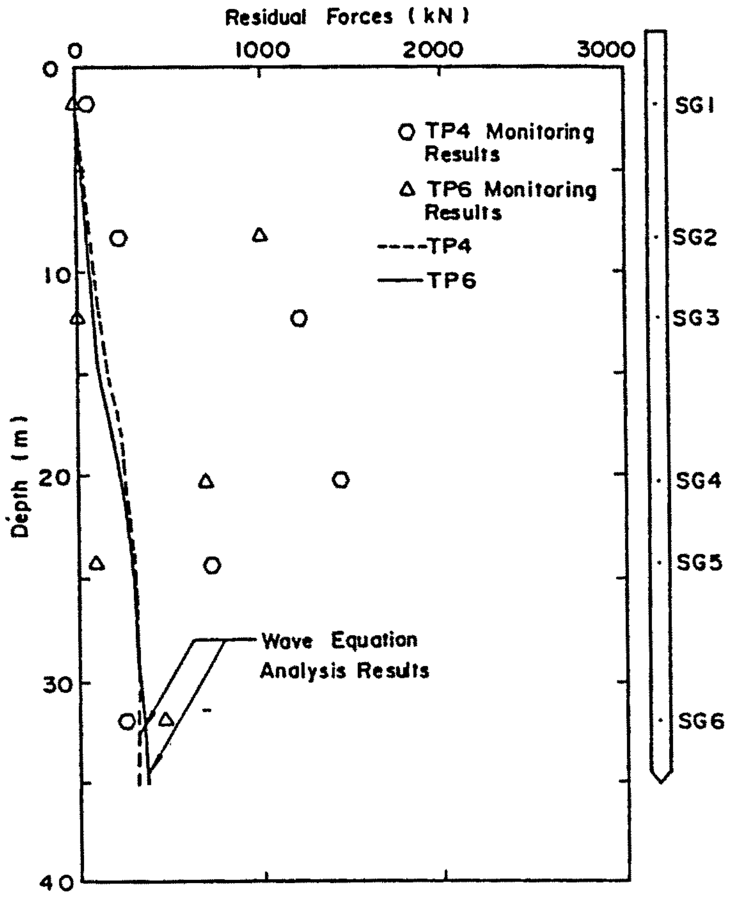

Figure 5. Monitored and Estimated Residual Forces Due to Pile Driving

forces is the great stiffness of the steel mounts where the strain gages were attached, which could probably lead to a greater amount of impact forces being transmitted to the transducer and thus introducing high axial forces (6). Excluding the extremely high data in Figs. 5, the maximum residual forces due to pile-soil rebound effect are about 260 and 470 kN for SG6 of TP4 and SG6 of TP6, respectively. The larger forces at TP6 are most likely the result of TP6's firmer bearing stratum making the rebound effect at the pile tip more significant than that of TP4.

For comparison with the monitoring results, wave equation analyses by means of WEAP87 program (7) using the soil quake and Smith's viscous damping recommended in the program manual were carried out. In the analyses, the analytical results of WEAP87 were adjusted to match the maximum transferred energy and axial forces at the pile head from PDA as closely as possible. The analysis results are also presented in Fig. 5. In spite of the lack of well-correlated soil parameters in residual stress analysis, Fig. 5 still indicates that the wave equation analyses can show fair agreement with the monitoring results.

Axial Forces before Pile Load Test. Figures 6 and 7 present the long-term monitoring results of the true axial forces of TP4 and TP6. They indicate that the axial forces have increased significantly between the period of pile driving and pile load test which was conducted about 25 to 30 days after pile installation. In addition to the effect of the ground subsidence of the entire site, the increase of the axial force is most likely due mainly to the

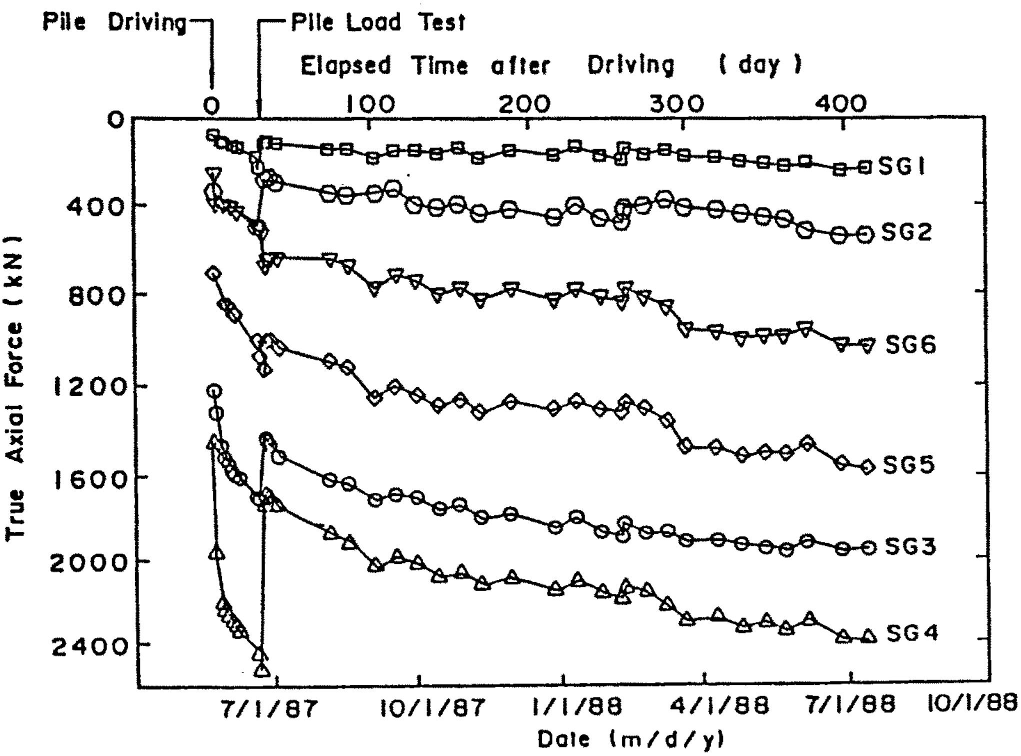

Figure 6. Long-term Monitoring Results of Axial Forces of TP4

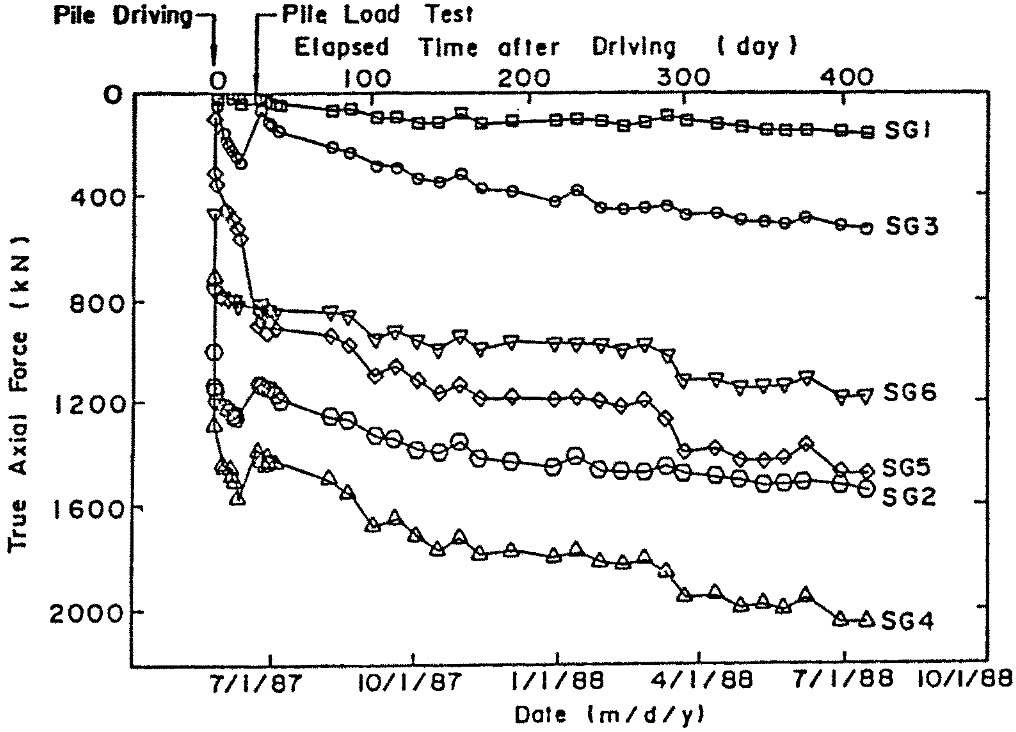

Figure 7. Long-term Monitoring Results of Axial Forces of TP6

reconsolidation of the clay soils, about 13 m thick, caused by pile driving. As indicated in Fig. 6, the maximum increment occurring at SG4 of TP4, located at about 20 m depth, is as high as about 1000 kN.

Axial Forces During Pile Load Test. By referring to the strain gage data obtained prior to driving, the true axial forces of TP4 and TP6 as tested to the maximum load are presented in Figs. 8 and 9, respectively. However, conventionally used in the analysis is the apparent axial force calculated by considering the gage data prior to pile load test as reference reading. For comparison purposes, apparent axial forces are also shown in these figures. Figure 10 presents the distributions of the ultimate true and apparent skin friction of the two piles. Since the soil strata above the tips of TP4 and TP6 are quite similar, there is only negligible difference in the ultimate skin friction of both piles. The interpreted ultimate true skin friction for TP4 and TP6 is about 3300 kN. The value is about 670 - kN more than the ultimate skin friction determined from the tension test on TP5 (2630 kN). This increase of the skin friction in compression test is most likely due to the Poisson's ratio effect and the increase of effective overburden pressure (5). Subtracting the ultimate true skin friction from the ultimate pile capacities in Table 2, the true point resistance can be determined as equal to 850 kN for TP4, and 1610 to 2820 kN for TP6. As indicated in Fig. 10, the apparent skin friction for both of the piles is approximately 700 kN greater than the true skin friction, which subsequently leads to an apparent point resistance

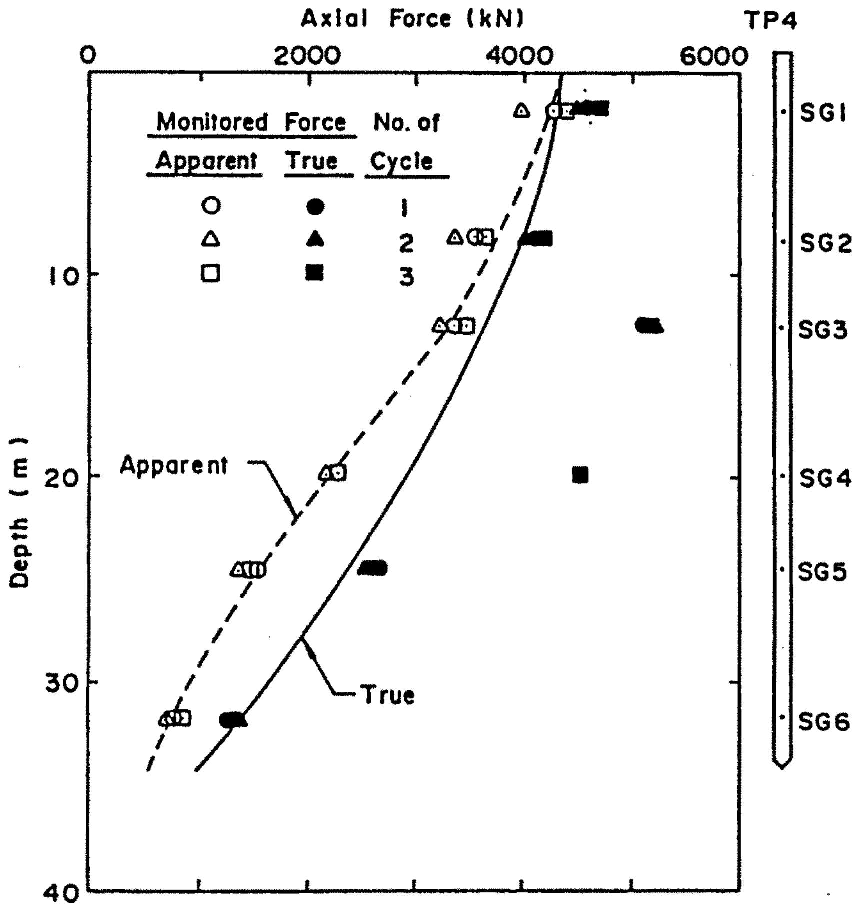

Figure 8. Axial Forces along TP4 as Tested to Maximum Test Load

less than the true one by 700 kN. Therefore, the interpretation results of the apparent pile axial forces will lead to an apparent point resistance which is lower than the true point resistance, and to an apparent skin friction which is greater than the true skin friction. That is consistent with the study results of other investigations (1, 2,3, 8, 9, 10,).

<u>Residual Forces after Pile Load Test</u>. Existing pile axial forces before pile load test will be redistributed during the test. Similar to the pile-soil rebound effect occurring at driving, additional residual forces are locked in pile shafts after they are tested under great test loads. Figure 11 presents the true pile axial forces before and after pile load tests. As indicated in this figure, the forces had significantly decreased after load test and the decrease at TP4 is generally more than that at TP6. This might be attributed to the different loading procedures used in the pile load tests. The forces already locked in the piles would be released after the tests. The greater the number of loading and unloading cycles, the more significant the release of the locked forces.

The compression load versus pile shaft deformation curve from the

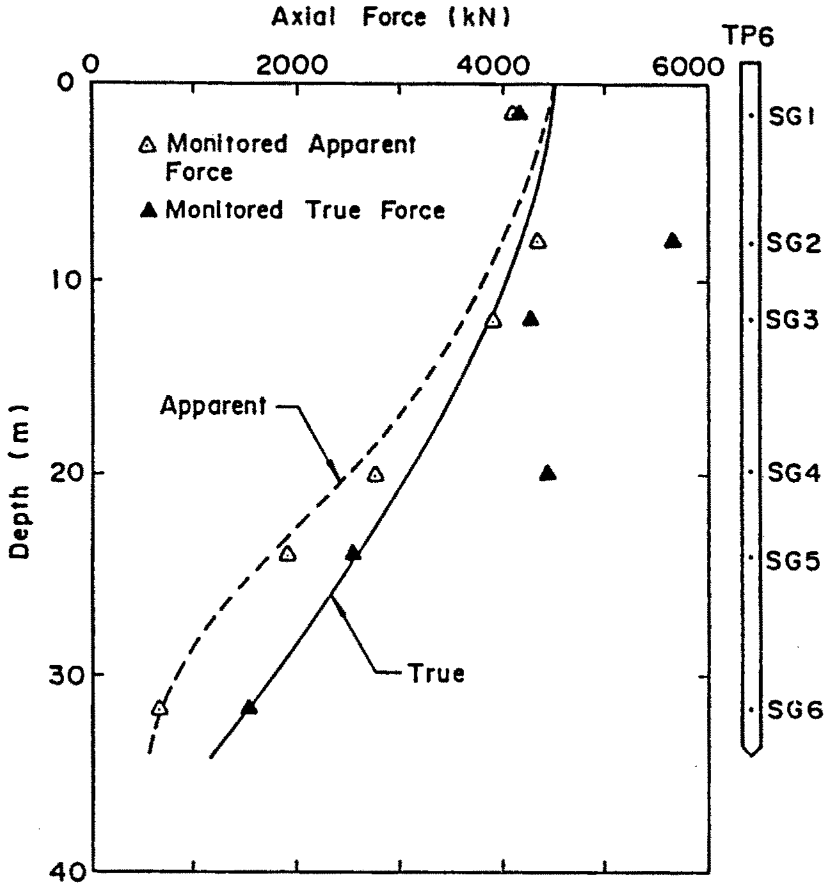

Figure 9. Axial Forces along TP6 as Tested to Maximum Test Load

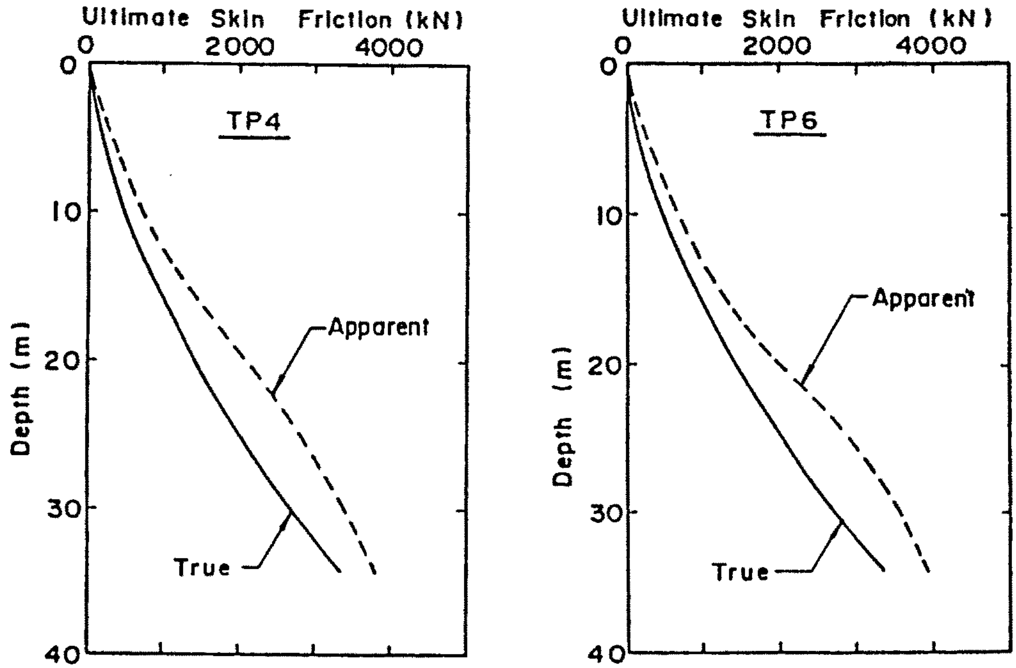

Figure 10. Interpreted Ultimate Skin Friction along Test Piles

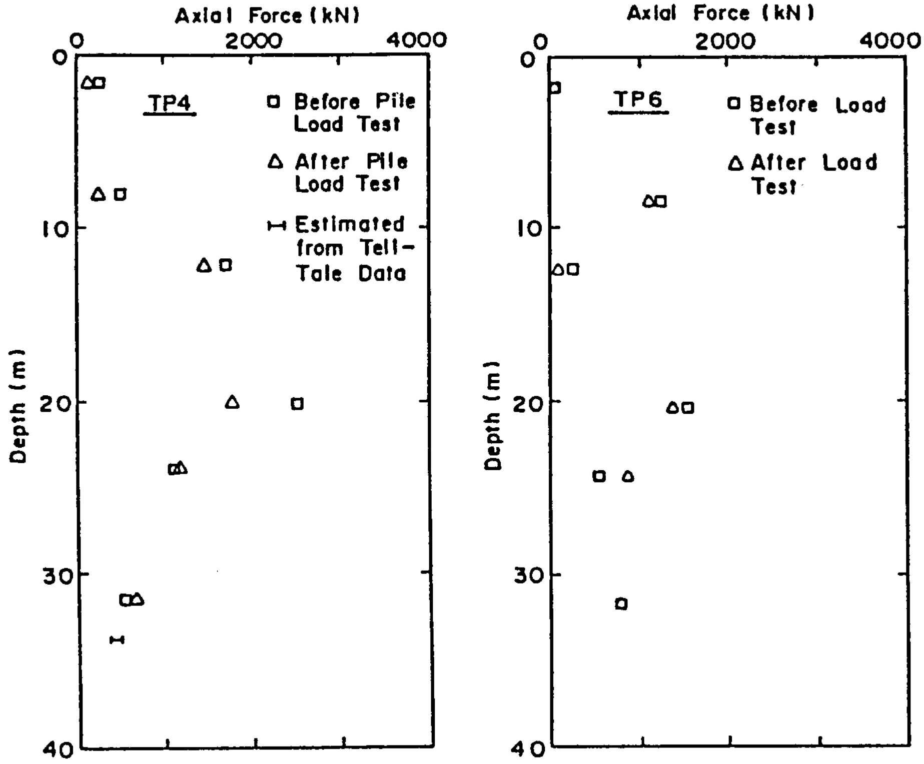

Figure 11. Axial Force Distribution along TP4 and TP6 before and after Pile Load Test

tell-tale of TP4 is presented in Fig. 12, which indicates that the residual shaft deformation was 0.7 and 2.3 mm after the test pile was completely unloaded. Since the material of the steel pipe pile did not yield under the maximum test load, the deformation is most likely caused by the stresses locked between the test pile and surrounding soils after the test load was completely removed from the pile head. Therefore, in addition to the strain gage data, this residual shaft deformation from tell-tale monitoring data can also be used to evaluate the existence and estimate the magnitude of pile residual forces due to pile load test. The following assumptions were made: (i) The entire pile length is acted upon by downward skin friction of soils. (ii) The skin friction has uniform distribution or triangular distribution with zero at pile head and a maximum value at pile tip. The tip residual force (Qtr) can then be calculated from the following equation:

$$Qtr = CAES/L \quad (1)$$

in which C= constant, equal to 2.0 and 1.5 for uniform and triangular distribution of skin friction distribution, respectively; A= cross-sectional area of pile; E= elastic modulus of the material of pile; S= residual deformation of pile from tell-tale; and L= total pile length. According to the average residual deformation of TP4, the calculated residual force at pile tip ranges from 360 to 480 kN, which is fairly close to the monitoring results near the

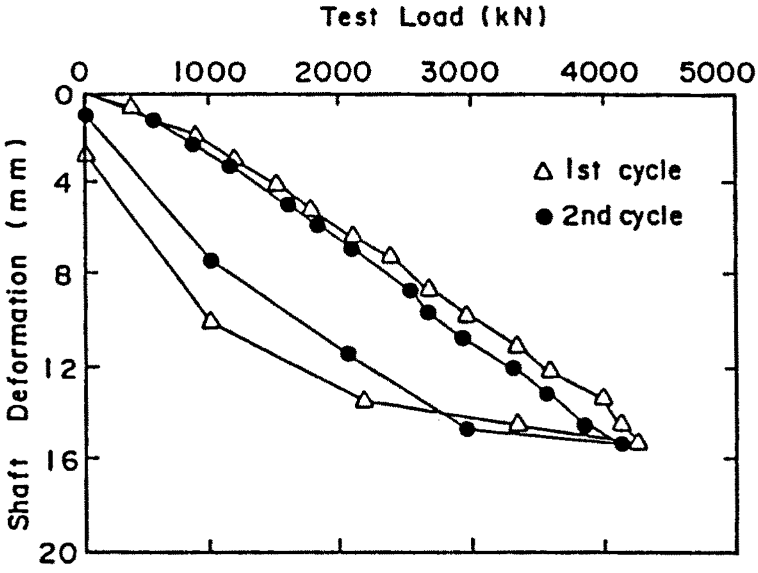

Figure 12. Load Versus Pile Shaft Deformation Curves from Tell-tales at TP4

pile tip as indicated in Fig. 11. Although based on the above simplified assumption, the estimated results can serve as additional evidence for the existence of the residual force locked at pile tip after pile load test.

<u>Long-term Monitoring Results after Pile Load Test</u>. Figure 13 presents the variation in axial force of TP4 after pile load test as well as long-term monitoring results of tell-tale and piezometer. Located adjacent to the instrumented pile, the piezometer was installed at a depth of about 25 m. These monitoring results indicate: (i) After the pile load test, the pile force has increased gradually. In other words, down-drag force on the test pile has developed. This phenomenon is consistent with the occurrence of ground subsidence in the study site at a rate of 1 to 4 mm per month. (ii) The variation in the axial force is closely related to the variation in the piezometric level caused by the groundwater pumping in adjacent areas. The force generally increased with the lowering of the piezometric level, which resulted in the increase of the effective stresses of the soils surrounding the test pile. On the other hand, the pile force decreased slightly, or even remained the same, with the rising of the piezometric level. The rapid response between the increasing axial forces and the lowering of piezometric level is most likely due to the increase in effective overburden pressure which caused the immediate settlement of the sandy soils above pile tip, and the increase of skin friction acting on the pile shaft. (iii) The monitored pile shaft deformation from the tell-tale increased with the rise in pile axial forces. This is consistent with the measurement results of the pile force from strain gages.

The down-drag force occurring after pile load test in TP4 and TP6 are presented in Fig. 14. As indicated in this figure, the maximum down-drag force, approximately 700 kN, occurred at a depth of about

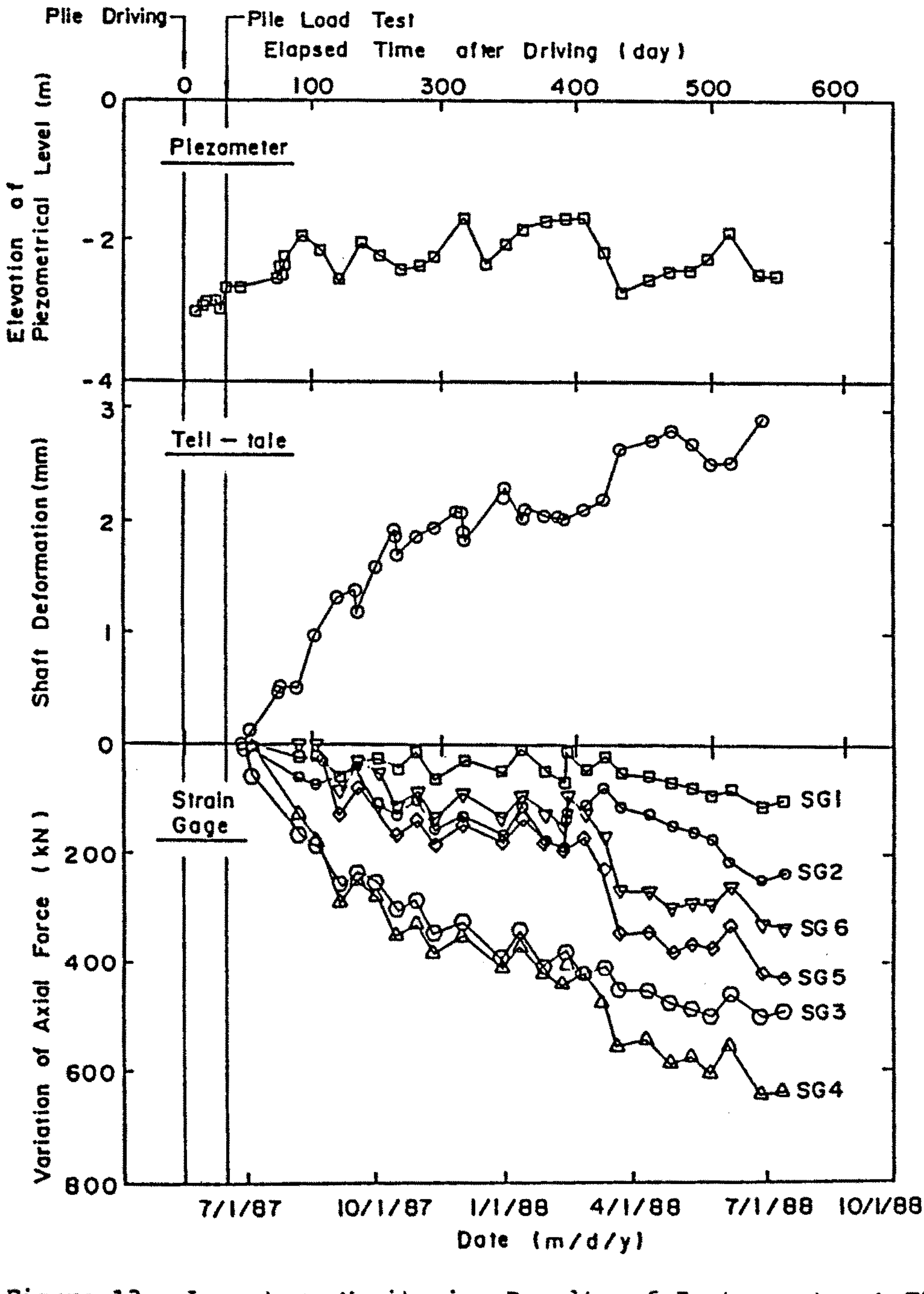

Figure 13. Long-term Monitoring Results of Instruments at TP4

20 m. It is noted that the true axial forces existing in the piles is greater than those indicated in Fig. 14 because of the fact that after load test some residual forces were locked in the piles. The distribution of the true axial forces at the test piles TP4 and TP6 are presented in Fig. 15. Neglecting the extremely high measurement data (SG3, SG4, and SG5 of TP4, and SG2, SG4 of TP6) most likely caused by the local permanent deformation of the piles due to pile driving, the maximum axial force 14 months after driving is as high as 1500 kN (SG5 of TP6), approximately 30 % of the ultimate compression capacity.

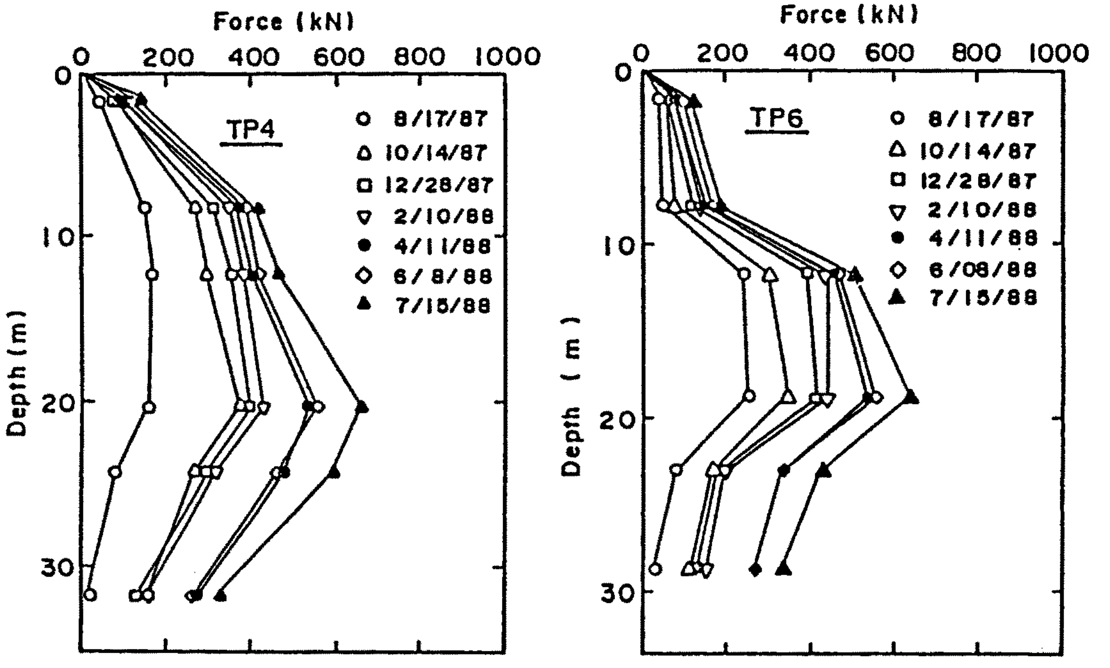

Figure 14. Down-drag Force Occuring after Pile Load Test

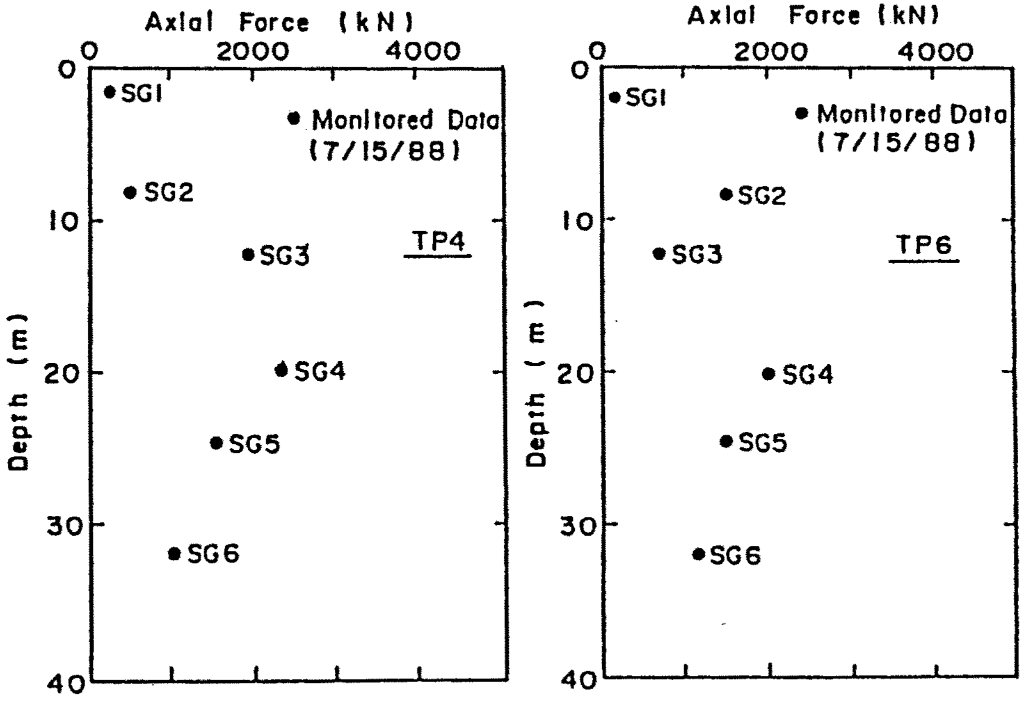

Figure 15. Axial Force Distribution along Instrumented Piles

CONCLUSION

According to the pile load tests and long-term instrumentation results presented in this paper, the following conclusions can be drawn:

1. Significant residual stresses exist in piles due to pile driving and subsequent pile load tests.
2. Both results obtained from wave equation analysis and from the interpretation of tell-tale monitoring data support the existence of residual stresses due to pile driving and pile load test.
3. The conventional pile interpretation method, which takes the strain gage data prior to pile load test as zero, overestimates the ultimate skin friction and subsequently underestimates the point resistance by the same amount. The overestimation of ultimate skin friction is about 20%.
4. The residual stress induced by soil rebound at pile tip is more significant and should be properly taken into consideration for piles having greater point resistance from a firm bearing stratum.
5. Pile axial forces increase significantly due to the reconsolidation of clayey soils after driving. The maximum increment in this study reached 1000 kN about 30 days after driving.
6. Based on the limited data base, this study indicates that the ultimate skin friction from a tension pile test is about 20% less than from compression pile test for similar soil deposits above the tip of test piles.
7. Pile forces existing in piles before pile compression load tests were considerably released after the loading and unloading cycles conducted during the pile load test.
8. During the monitoring period, the maximum existing pile force has reached about 1500 kN, which is equivalent to approximately 30% of the ultimate pile capacity.
9. The variation in the axial pile forces is closely related to the variation in the groundwater pressure. The lower the groundwater pressure, the greater the axial forces.

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