BEHAVIOR OF STEEL SHELLED PILES
AT SINGAPORE MARINA SQUARE

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SUMMARY

The difficulties and technical problems encountered during the installation of 9,349 number of Raymond Step-Taper driven cast-in-situ piles in Marina Square, Singapore are reviewed. The subsoil deposit at the site mainly consists of layers of reclaimed sand, soft marine clays, loose to medium sand, stiff to hard silty clay and very dense sand. Procedures of establishing set criteria for driving the Raymond piles in these soil formation were established on the basis of results of preliminary pile loading tests. The constructional control was made flexible by adopting a set criterion combining terminal driving resistance and aggregate number of blows required to drive a pile in order to accommodate significant variation in the subsoil conditions at the site. The problem of retapping of these driven piles due to heaving was evaluated.

1. INTRODUCTION

Driven cast-in-situ piles have been used successfully in many parts of the world in different soil formations (for example, RAY et al, 1979; WAN et al, 1979). This paper describes the difficulties and problems encountered during the installation of the Raymond Step-Taper driven cast-in-situ piles at the Marina Square in Singapore. A total of 9,349 piles were installed in which 181 were considered as bad piles. This accounts to about 2 per cent of the total number of piles installed. These bad piles were subsequently replaced with driven H-piles.

The subsoil condition of the site was of great variation which was the major factor contributing to the installation difficulties. Set criteria for installation control by combining terminal driving resistance and aggregated number of blows for driving were established in order to cope with the varying soil conditions.

Problems such as derated piles, demudded piles, length limitations, retapping or heaved piles and elapsed time required for soil recovery were encountered. Some of these problems are discussed in the paper.

2. SITE AND SUBSOIL CONDITIONS

2.1 Site Conditions

The Marina Square site is located on a recent reclaimed land along the southeastern seafront and is immediately adjacent to the central business district of Singapore. The site occupies an area of about 92,000 sq.m and was relatively flat with ground elevation varying between approximately 1.96 and 3.20 m above mean sea level. The East Coast Parkway passes through the southeast boundary of the site at a distance of about 500 m away. On the southwestern boundary the site is bounded by the Stamford Canal.

The Marina Square Complex, which is at present under construction, comprises of two 22-storey hotel buildings interconnected by a 2-level shopping mall and another 37-storey hotel linked to the shopping mall by a bridge. The entire site was subdivided into three zones as shown in Fig. 1. The area occupied by the two 22-storey buildings is designated as Zones A and B whilst the 37-storey building is situated in Zone C.

2.2 Subsoil Conditions

A comprehensive subsoil investigation program was carried out to delineate the subsurface conditions. A total of 42 boreholes had been drilled within the site area. In-situ testing including borehole pressuremeter tests and field permeability tests, and laboratory tests on undisturbed samples were performed to determine the soil characteristics. The drilling was carried out in two phases. The first phase consisted of 15 holes which were drilled at locations uniformly distributed over the site area for the purpose of collecting overall soil information for preliminary selection of foundation system. The Phase 2 work was done to define the variation of the soil layers encountered in Phase 1 investigation. Majority of the boreholes was drilled to depths between 25 m and 30 m with one hole advanced to a depth of 70 m.

Overall speaking, the subsoils at the site can be divided into five major layers:

(a) Fill - The top soil, varying in thickness from 1.5 m to 8.5 m, consists of primarily loose coarse to medium sandy fill with occasional pockets of clay. The fill material appeared to be more dense in Zones A and C than that in Zone B.

(b) Marine Clay - Beneath the sandy fill is a layer of very soft to soft, grey color, marine clay with maximum thickness of about 15 m. In Zone C, the thickness of the marine clay was only 0.5 to 3.0 m. A noticeable characteristics of this soil stratum is the presence of a continuous sand layer interbedded with the marine clay over a substantial part of the site. The thickness of this sand layer is mostly less than 5 m with a maximum of about 9 m.

(c) Clayey Sand - Underlying the marine clay is a layer of clayey sand of 3 to 9 m in thickness. Over majority area of the site this sand is in a loose condition. However, in the southern part of Zone A, the sand is relatively dense with Standard Penetration Resistance N value as high as over 40.

(d) Silty Clay - Beneath the clayey sand stratum is a layer of reddish brown silty clay intermixed with a large amount of fine sand pockets. The thickness of the clay layer varies from only 1.5 m to more than 10 m. This layer of soil is very stiff to hard with SPT N values in excess of 40 and the stiffness increases with depth.

(e) Silty Sand - Immediately below the silty clay is a layer of greyish white, slightly cemented, very dense silty sand. Majority of the boreholes was terminated in this stratum. The SPT N values of this soil was often over 100.

Figure 2 presents a typical soil profile and Table 1 summarizes some of the major physical and engineering properties of the various soil layer. Groundwater was found to be at 1.5 m below the ground surface. Piezometric records indicate that the groundwater pressure at the site is practically in static condition, except some excess pore pressures were monitored in the marine clay layer.

3. DESIGN OF FOUNDATION SYSTEM

3.1 Selection of Foundation System

The most significant aspect of the subsoil conditions across the site is the presence of a layer of compressible marine clay formation with variable thickness and located at a shallow depth below the ground surface. This soft clay layer is overlain by a loose sandy fill. In selection of the most suitable foundation system for a building development, the governing factor is the anticipated performance of the superstructures. For the present development, in view of the potential large total settlement and differential settlement, the two 22-storey buildings and the lowrise podium block in Zones A and B must be founded on deep foundations which can derive support from the underlying stiff to hard silty clay and the very dense silty sand, i.e, layers d and e, respectively, either by skin friction or end bearing or their combinations. For the 37-storey building in Zone C, compensated foundation could be adopted if the foundation were placed at a depth of 16 m below the ground surface. Due to the deep excavation required, deep foundation was also adopted for this building.

During the geotechnical investigation stage, driven steel pipe piles, steel H-piles, and cast-in-situ bored piles were evaluated. A number of different piling systems was submitted for consideration by international tenderers. Due to the lowest tender price and shortest proposed construction time, the step-taper piles proposed by Raymond International Builders, Inc. (R.I.) were selected for the project.

The Raymond Step-Taper piles were made of helically corrugated light-steel shells of gauge 14 in sections of 3.66 m (12 ft) length. The outside diameter of the shell increases progressively from 340 mm (No. 3) to 441 mm (No. 7). At the tip of the pile, a No. 2 steel pipe (324 mm OD and 19 mm thick) was attached and the bottom of the pipe was closed with a steel end plate by welding. The piles were driven with an internal steel mandrel that engaged drive rings located at each change in shell diameter and extended to the pile tip. The piles were driven with steam double action (2/O) hammers. After driving, the mandrel was withdrawn and the shells were filled with concrete. The pile makeup and typical driving characteristics are compared to the subsurface profile in Fig. 2.

In addition to the low tender price, Raymond Step-Taper piles have several additional advantages which were also considered during the tender evaluation. This type of piles can be driven into ground quite rapidly, which means the construction time required for pile installation could be shorter than other systems. Records of Raymond indicated that about 15 minutes' time is required to drive one pile. Since the steel shell of the step-taper pile is relatively thin, it can be cut off easily to suit the length requirement after driving and results in low wastage. A further advantage of this type of pile is that the concrete strength can be quite high since the material is poured within the confinement of steel shell and with very little contamination.

In view of the great variation in the thickness of the soft marine clay and the thickness of the stiff to hard soil stratum over the site, difficulties in controlling the pile length and in establishing the driving or setting criteria during the installation of piles, the entire site was subdivided into eight subzones as shown in Fig. 3. Representative soil profiles of each of the subzones are presented in Fig. 4.

3.2 Pile Capacity Analyses

The static bearing capacities of the Raymond Step-Taper piles were analyzed by using the following modified Meyerhof's formula proposed by the Japanese Association for Steel Pipe Piles (JASPP):

$$R_u = 40NA_p + \frac{1}{5}\overline{N}_sA_s + \frac{N_c}{2}A_c \qquad (1)$$

where R = Ultimate bearing capacity of pile, t

A = Area of pile tip, sq.m

N = Standard penetration resistance at pile tip

$\overline{N}_s$ = Average N value of sand layer to the pile tip

$\overline{N}_c$ = Average N value of clay layer to the pile tip

A_s = Total circumferencial area of pile in sandy layer, sq.m

A_c = Total circumferencial area of pile in clay layer.

R.I. had also predicted the static bearing capacities on the basis of combination of Hansen's formula for end bearing (HANSEN, 1968) and Nordlund's formula for skin friction (NORDLUND, 1963). The predicted values of the static bearing capacity of piles in the eight subzones on the basis of representative soil profile for each of the subzone as shown in Fig. 4 are listed in Table 2. Method 1 in the table refers to the Modified Meyerhof formula and Method 2 is the Raymond calculation. The predicted pile bearing capacity values by the two methods were very close in Subzones 1, 2, 3a, 3b and 4, whilst Method 1 gave somewhat higher values than that given by Method 2 in Subzones 5, 6 and C.

A dynamic analysis using the wave equation was performed by R.I. The following set criteria were suggested on the basis of the results of analysis:

For F.S. = 3.0 8 blows per inch
For F.S. = 2.5 6 blows per inch.

It was later found that this analysis was not very useful.

In general, the pile length estimated by the Consultant on the basis of soil boring data was very close to the actual driven length of the test piles. However, several piles in Subzones C and 3a showed larger difference between the predicted and the actual driven pile length as shown in Fig. 5.

3.3 Negative Skin Friction

The very soft compressible clay stratum is of varying thickness across the site. Results of the soil investigation indicate that at the time of investigation, this marine clay was still in the process of consolidation as revealed by the excess pore pressures monitored at various depths. Downdrag or negative skin friction acting on the piles installed in areas where the soft marine clay is relatively thick and is still under consolidation, must be considered in the design. This was of primary importance in Subzones 3a, 4 and 5. The magnitude of the negative skin friction was evaluated. In areas where the estimated settlement rate was less than 2 cm/year, the effect of downdrag was not considered.

For estimating the effect of negative skin friction which could develop along the shaft of steel shelled piles, the Japanese Association for Steel Pipe Piles (1977) proposed that the allowable pile bearing capacity should be the lowest value of the values computed by the following three methods:

$$R_a = \frac{1}{3} R_u \qquad (2)$$

$$R_a = \frac{R_{up} + R'_F}{F.S.} - NF \qquad (3)$$

$$R_a = A_p \cdot f_p \qquad (4)$$

where R_a = Allowable pile capacity

R_u = Ultimate bearing capacity of soil

R_{up} = Ultimate end bearing capacity of pile

R'_F = Positive skin friction

F.S. = Factor of safety

NF = Negative skin friction

A_p = Cross-sectional area of pile

f_p = Allowable stress of pile material.

In the design of pile groups, the magnitude of negative skin friction acting on each pile may be less because of the group action. Reduction factors were applied to the pile groups according to the number of piles and their spacing.

Table 3 lists estimated values of potential negative skin friction which may develop along a single step-taper pile in Subzones 3a, 4 and 5 where a relatively thick layer of the soft marine clay exists by two methods.

Method A is based on the Standard Penetration Resistance N values and the undrained shear strength of the clay (JASPP, 1977) and Method B which was used by Raymond International assumes that the negative friction force is equal to the frictional resistance of the soil that moves downward relative to the pile. Both approaches were considered to be on the conservative side, since (i) the fill has been in place for about seven years; (ii) approximately 2.65 m of the fill would be removed over the site, and (iii) the piles were most likely end-bearing, thus only a small portion of the structural load will be transferred to the compressible soil stratum.

Due to the relatively large values of the potential negative skin friction, it was decided that a factor of safety of 3.0 be used to determine the allowable bearing capacity of piles in Subzones 3a, 4 and 5 and a factor of safety of 2.5 be used in the other subzones where no negative friction was expected.

4. PILE LOAD TESTS

4.1 Preliminary Pile Load Tests

Since it was the first time in Singapore that the Raymond Step-Taper piles were used, a series of preliminary probe tests was carried out to determine the driveup characteristics of this type of piles in the reclaimed land, and to establish driving criteria for installation control. Two types of boot plates at the tip of the pile tube were first tried. One type of boot plate has an external projection beyond the end tube and the other type is a recessed plate. The purpose of the external projection of the boot plate was to serve as a friction cutter during driving and to decrease the penetration depth required to achieve the same driving resistance as a straight shaft pile. This system is generally satisfactory in sandy soil and in soft to medium cohesive soils since the gap created by the extruding boot plate would be closed quickly by the surrounding soil shortly after driving and then to develop skin friction which contributes to a major part of bearing capacity of this type of pile. At the present site, the major subsoil strata which would contribute to the pile bearing capacity are the stiff to hard silty clay and the underlying very dense sand. The stiff clay will be very slow to recover from any large deformation and thus slow in recovering the frictional resistance along the pile shaft. Unless very long time is allowed after driving to elapse, the total bearing capacity of the pile with projected boot plate is likely to be lower than those with a recessed plate. After trying a few probe piles driven with projected boot plate, this makeup was considered to be undesirable and recessed boot plate was adopted for other probe piles as well as all the working piles.

A total of 18 probe piles with recessed boot plate was load tested for the purpose of establishing pile driving criteria at the site. The tests were carried out in accordance with ASTM D 1143 (Method of constant time interval loading with one-hour load interval). Figure 6 shows a typical pile load test record. For interpretation of the ultimate bearing capacity, Davisson's (1975) offset method was adopted. The load carrying capacity of those test piles as determined from the load tests are summarized in Table 2. Among the 18 load tests of probe piles, six of them did not reach the required capacity.

4.2 Development of Set Criteria

On the basis of the driving records of the probe piles and the load tests, the terminal driving set in number of blows per last foot of pile penetration could be determined. However, evaluation of the load test results indicated that the six piles which did not reach the required capacity were all driven with less than 700 aggregated number of blows. This indicated that in addition to the terminal set, the total number of blows which a pile received also played an important role in the pile bearing capacity embedded in the type of soil formation such as the Marina Square. Figure 7 shows a plot of the terminal driving set versus the aggregated driving blow count of the 18 test piles. On the basis of these data, set criteria incorporating the terminal driving set and the aggregated number of blows in driving the pile were established for each of the eight subzones.

5. PROBLEMS ENCOUNTERED DURING THE INSTALLATIONS

Numerous problems were encountered during the pile installation including heaving of piles, mud in shells, out-of-alignment of the flexible shells, etc. Major factors, among many others, contributing to these problems were great variation of the subsoil condition, first experience of the use of step-taper piles, and relatively tight construction schedule. Due to the heterogenity of the subsoil formation the set criteria had to be developed and modified during the pile installation. The final criteria were set only after 1,368 piles were driven. Among these piles, some of them did not meet the criteria of terminal set plus aggregated number of blows as shown in Fig. 7. For those piles which did not meet the criteria but already concreted, they had to be downgraded. For those piles not concreted, they were retapped to the established set criteria. The following sections discuss two of the problems encountered during the installation.

5.1 Heaving and Retapping

The Raymond Step-Taper pile is a displacement pile. For displacement piles, pile heaving is usually a problem during the installation. At the Marina Square site, about 15 per cent of the total number of piles had significant heaving and required retapping. The heaved piles were either reseated to their original position if the set criteria were met or retapped and redriven to the required set.

Heaving of the step-taper pile shell before concreting due to driving of adjacent piles was monitored at selected locations. Figure 8 shows the measured heaves at the butt and tip of a pile due to installation of piles in the same group. The effect of distance of adjacent pile on the amount of heave is illustrated in Fig. 9. From the data collected, it was concluded that the effect of pile driving extends to a distance of about 6 to 8 m away from the pile.

Two pile load tests were carried out in Subzone C to determine whether retapping of heaved piles is necessary. These two piles, No. M/22-1 and M/22-4 were driven into the ground and not retapped for heaving. The piles were allowed to rest for 27 and 28 days before being tested. The total uplift of Pile M/22-1 was measured to be 63 mm and the ultimate bearing capacity determined by load test was 224 tonnes which was 63.5 tonnes less than the required capacity. The second load test on Pile No. M/22-4 revealed an ultimate bearing capacity of 246 tonnes which was also less than the required value. The test result of this pile is shown in Fig. 10. From these test results, it was concluded that pile heaving due to driving of adjacent piles is detrimental to the pile capacity and reseating of the piles is necessary.

Retapping of piles in Subzones C and 6 was carefully monitored in order to establish a suitable criterion for overcoming the effects of pile heave and build-up of excess porewater pressure. These two subzones were selected for monitoring because four of the six test piles failed in the preliminary load testing program were located in these two areas. The subsoil conditions in these two areas are quite similar. It was found that some piles showed a reduced driving resistance when they were retapped a few days after driving. In some instances, piles retapped the second time in 15 days after the first retap showed a gain in the driving resistance. This phenomenon indicates that time is one of the factors affecting the bearing capacity of driven piles in this type of soil formation. Sufficient time is required for the soil to recover and to develop the necessary frictional resistance along the soil-pile interface after being disturbed due to driving. Figure 11 shows the test results of two piles in a same group (Pile Nos. V/31-3 and V/31-2). Both piles were of similar length and set condition but different take-up time before being tested. Pile No. V/31-2 with a 43-day take-up period passed the working load test whilst Pile No. V/31-3 with a 17-day take-up period failed in the load test.

As discussed, there are two factors affecting the pile capacity, that is, pile heave and disturbance to the soil where the driving pile is embedded. Therefore, for the retapping program, piles were left for a period of time to allow the soil to recover before redriving. In general, in the early stage of the retapping program, sufficient time was allowed and retapping was mainly to reseat the piles. This was typically the case in Subzone C. However, during the later part of the pile installation program, time became limited especially in Subzone 6, and retapping was carried out soon after the initial driving. In most cases, the piles penetrated to a depth greater than the initial penetration before heave. Table 6 shows an account of the categories of retapping in Subzones C and 6. There were 1,889 and 549 retaps in the two subzones, C and 6, respectively.

Retapping of piles was carried out either by re-entering of the steel mandrel into the shells before concreting or on the reinforced concreted pile when re-entering of the mandrel was not possible. For the latter, special reinforcements consisted of five numbers of Y16 steel bars extending from the top of the pile to at least 600 mm into the bottom pipe section were installed before concreting. Lateral reinforcement for the top 900 mm of the pile was of R6 loops at 150 mm centers. The concrete in the piles must have achieved at least 5000 psi cylinder strength before they were retapped. Retapping proceeded with a 34 ft mandrel and a 6-inch thick plywood cushion. The retap criterion was that the pile was first given 10 blows with a 2/0 hammer; if the settlement was less than or equal to 15 mm, the pile was considered acceptable. If the settlement was greater than 15 mm, the pile was then given another 10 blows until a penetration not more than 15 mm was achieved.

Two other measures to counter the problem of pile heave were attempted without success. Predrilling before pile driving was tried. This process was not only time consuming but also ineffective. Relief holes were drilled in space between piles to alleviate the problem of heaving. This method was also found to be unsuccessful due to collapse of the sand layer in the holes.

5.2 Mud In Shells

Mud and water leakage were sometimes observed at shell-to-shell or shell-to-pipe joints after driving. When the rate of leakage was low, the water was bailed out before concreting. In cases where the amount of mud was excessive and was anticipated to be destructive to the structural integrity of the pile, the piles were rejected, filled up with sand and replaced.

In one instance, the mud was cleaned out by jetting with compressed air and water. However, mud continued to enter the pile during concreting. This pile was subsequently load tested to 172.5 tonnes (that is, 1.5 x design load). The 172.5 tonnes load was held for only 15 minutes and the gross butt settlement increased from 32 mm (1.26 in.) to 50 mm (1.97 in.) with a residual settlement of 42 mm (1.63 in.). It was retapped after the load test and it went down 290 mm (11.4 in.) for 60 blows. Two other piles of the same category were retapped, one of which settled 112 mm (4.4 in.) for 30 blows. The poor results of the load tests and retapping indicate that the method used for cleaning and concreting of the piles with mud in the shells was unsatisfactory.

6. CONCLUSIONS

For installation of driven cast-in-situ piles it is important to establish "set criteria" for construction control. The set criteria can usually be developed on the basis of preliminary load test results. However, it must be recognized that the "set criteria" is only a tool or guideline for controlling the pile installation to ensure that the piles will meet the necessary bearing capacity and settlement requirements. Due to variations in the subsoil conditions over a site, it is often necessary to modify the established "set criteria" on the basis of driving resistance, pile heave, and pile behavior under load tests.

Continuous review of the set criteria is a necessity for satisfactory control of any large scale pile installation program. The supervising geotechnical engineer must have a thorough understanding of the subsoil conditions at the site, the characteristics of the pile driving equipment and processes, so that he can apply the developed set criteria intelligently. The importance of constant communications between the specialist piling contractor and the foundation consultant during the installation period cannot be over emphazised. It is often the key for solving many construction problems.

This paper describes the case record of installation of Raymond Step-Taper piles at Marina Square in Singapore. During the pile installation stage, working control by set criteria was mutually agreed by the foundation consultant and the specialist contractor from the data of preliminary load tests. These set criteria were constantly reviewed as more test results became available. Necessary remedial work was recommended after careful appraisal of representative test results. The correctness of the required penetration controlled by the set criteria was evaluated by working load test, especially on those piles installed in significantly different subsoil conditions and those of which workmanship was of concern.

7. ACKNOWLEDGEMENTS

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Table 1:
Average Physical and Engineering Properties of Subsoils

Layer	Thick-ness	Soil Description	Soil Properties									
			w_n %	γ_t kN/m^3	e	w_L %	I_p %	q_u kN/m^2	c kN/m^2	ϕ deg	c' kN/m^2	ϕ' deg
1	1.5-8.5	Loose, coarse to medium sandy fill	15.7-21.4	17.6-19.6	0.54-0.85	-	-	-	-	-	-	32.0
2a	0.5-15.0	Very soft to soft marine clay	40.9-52.6	16.5-17.7	1.05-1.94	54-73	32-43	24.0	18.3	7.0-8.2	-	-
2b	0.0-9.0	Loose sand	17.5-24.0	19.9-20.6	0.51-0.79	-	-	-	-	-	-	-
3	3.0-9.0	Loose to medium dense clayey sand	13.9-17.9	20.7-21.5	0.37-0.45	-	-	-	-	-	6.0	34.5
4	1.5-10.0	Silty clay stiff	15.9-22.4	20.0-21.2	0.41-0.52	32-52	19-32	100	35.0	17.5	27.0	23.0
		Hard	11.5-12.8	21.8-22.2	0.30-0.36	33-44	20-26	330	46.0	23.0	23.0	28.0
5		Very dense, slightly cemented silty sand	11.9-16.1	21.1-21.8	0.34-0.43	-	-	-	-	-	-	-

Table 2: Comparison of Pile Load Test Results with Predicted Bearing Capacities

Sub-zone	Predicted Pile Bearing Capacity				Pile Test No.	Pile Length, m	Ultimate Pile Load Capcity from Load Test, t	Aggregate of Driving Blows and Final Set per Foot
	Method 1		Method 2					
	Pile Length	tons	Pile Length	tons				
1	11.0	334	13.3	377	12	18.9	311	753 - 38
2	17.3	389	17.3	>392	4	17.1	345*	358 - 99
					20	21.9	288*	728 - 60
3a	21.5	365	21.2	>392	14	18.3	270	445 - 60
3b	20.3	430	20.3	398	11	26.2	345*	1069 - 60
					18	28.7	345*	1092 - 24
					26	28.3	288*	1233 - 40
4	24.8	417	24.8	739	6	26.8	345*	783 - 105
					10	30.8	345*	654 - 99
					17	24.7	216	414 - 65
5	18.3	538	16.6	344	7	19.3	345*	1084 - 60/6"
					16	18.3	368*	755 - 56
6	16.0	484	15.3	372	19	16.1	185	320 - 60
C	19.5	552	16.3	328	2	20.0	170	460 - 86
					8	20.7	345*	850 - 81
					9	17.7	345*	1145 - 85
					13	18.9	250	636 - 60
					15	15.5	136	382 - 60

* Maximum test load.

Table 3:

Estimated Negative Skin Friction
on a Single Step-Taper Pile

Subzone	Estimated Negative Friction, tons	
	Method A	Method B
3a	24	18.3
4	59	48.9
5	17	7.9

Table 4:
Categories in Retapping in Zones C and 6

Subzone	Percentage of Total Retaps		
	Good Retap	Reseat for Heave	Retap for Heave and Soil Disturbance
C	72.7	19.6	7.7
6	54.0	18.0	28.0

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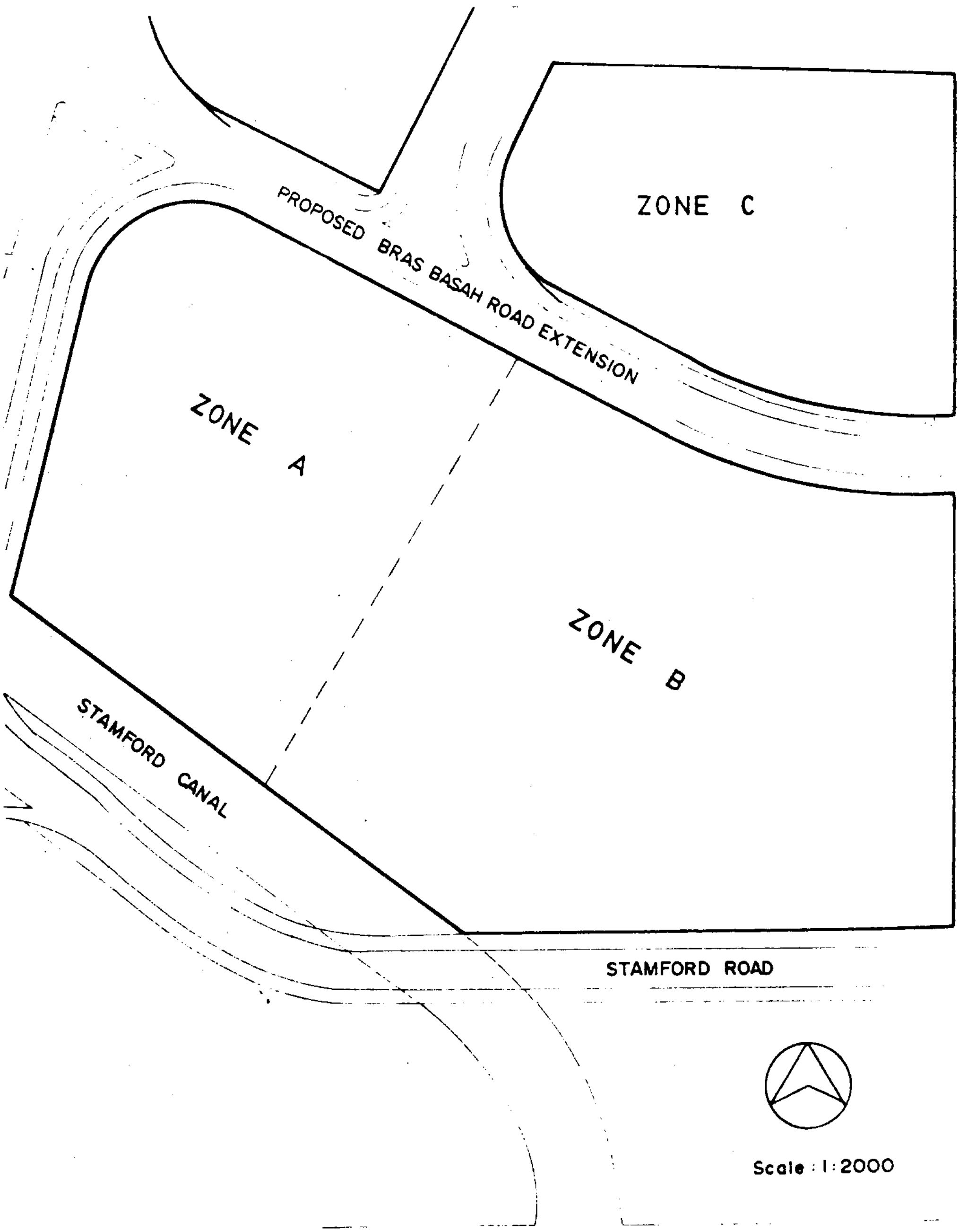

Fig. 1 Site Plan of Marina Square

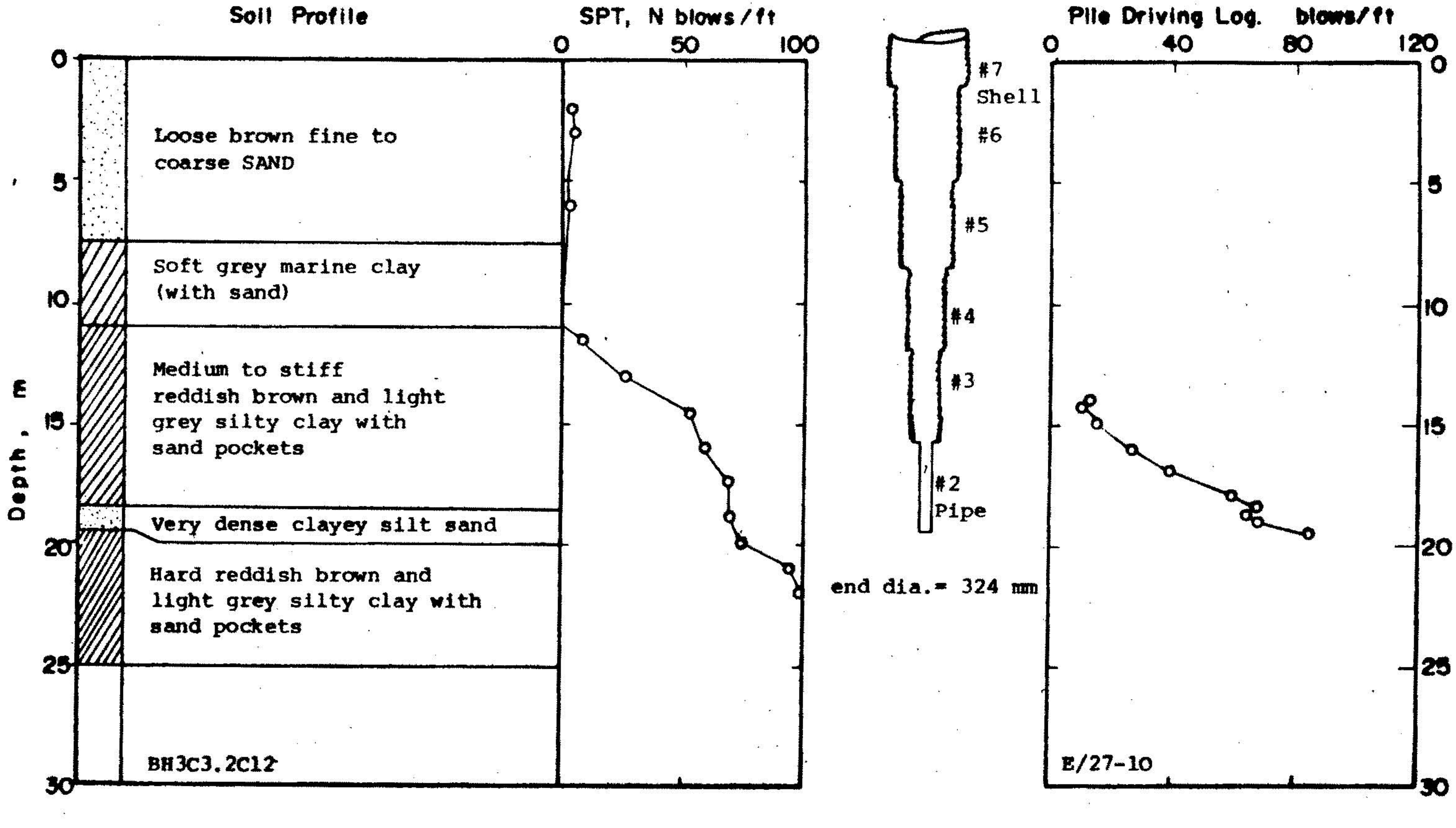

Fig. 2 Typical Soil Profile and Pile Driving Log

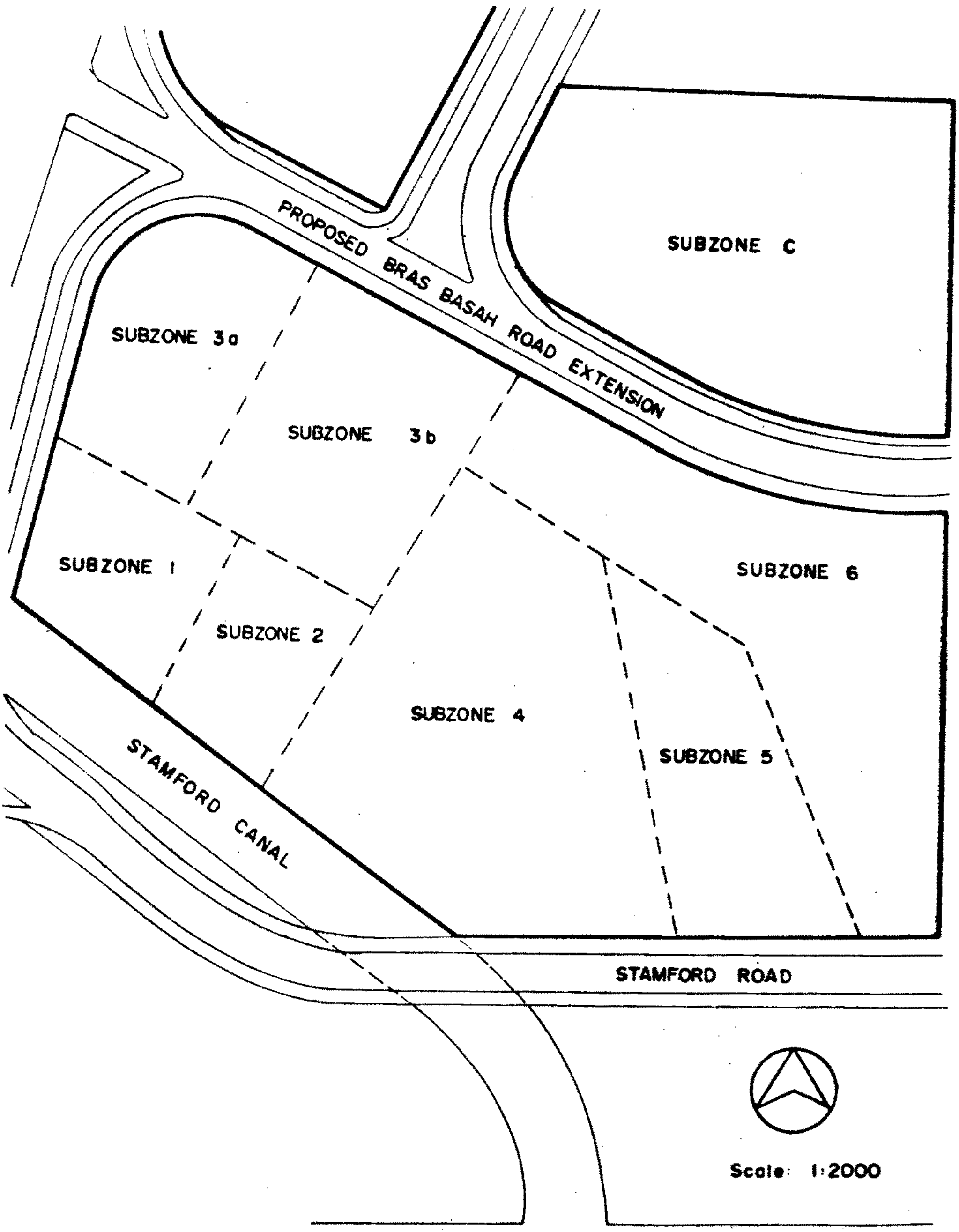

Fig. 3 Subdivision of Site for Pile Installation

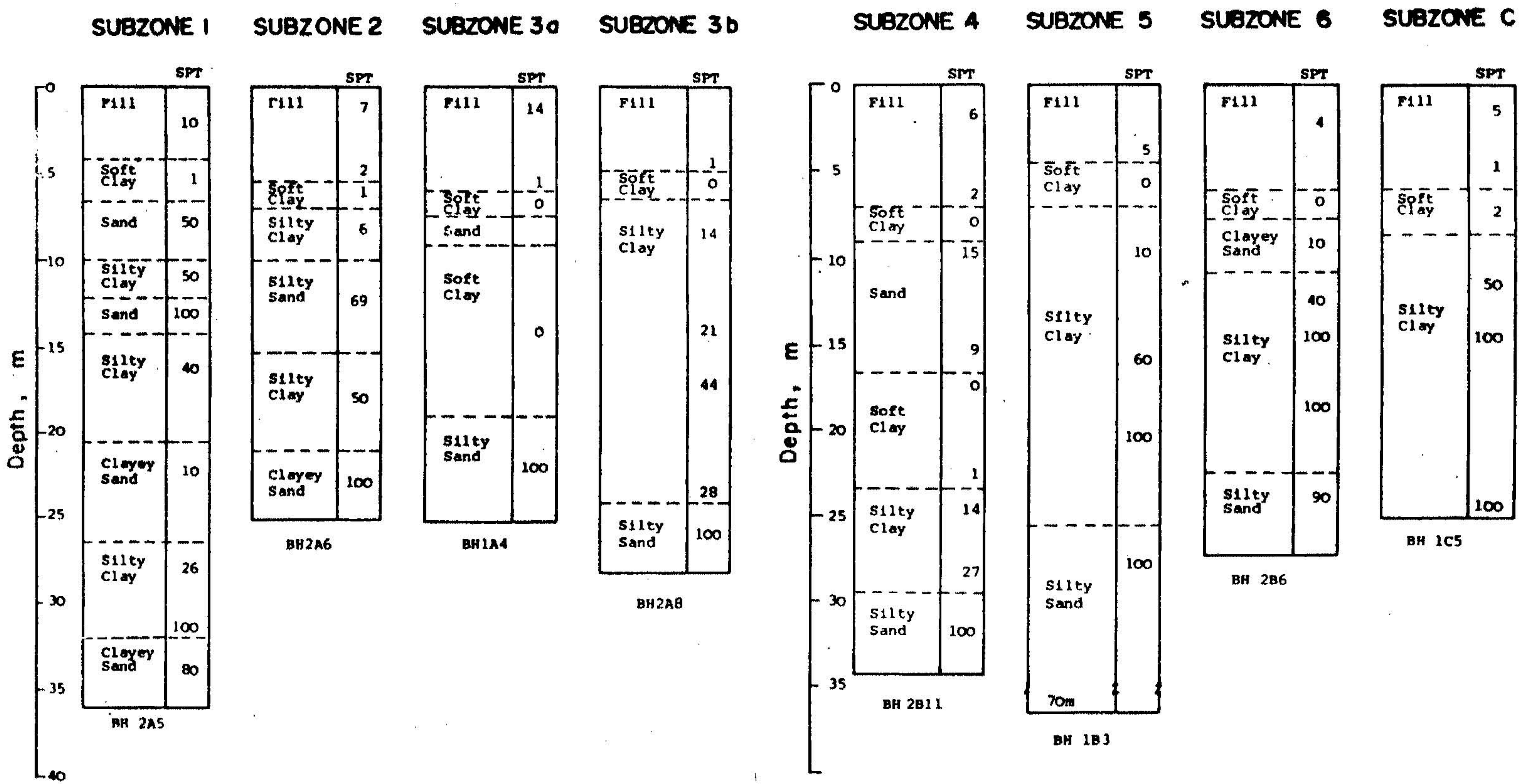

Fig. 4 Representative Soil Profiles in the Subzones

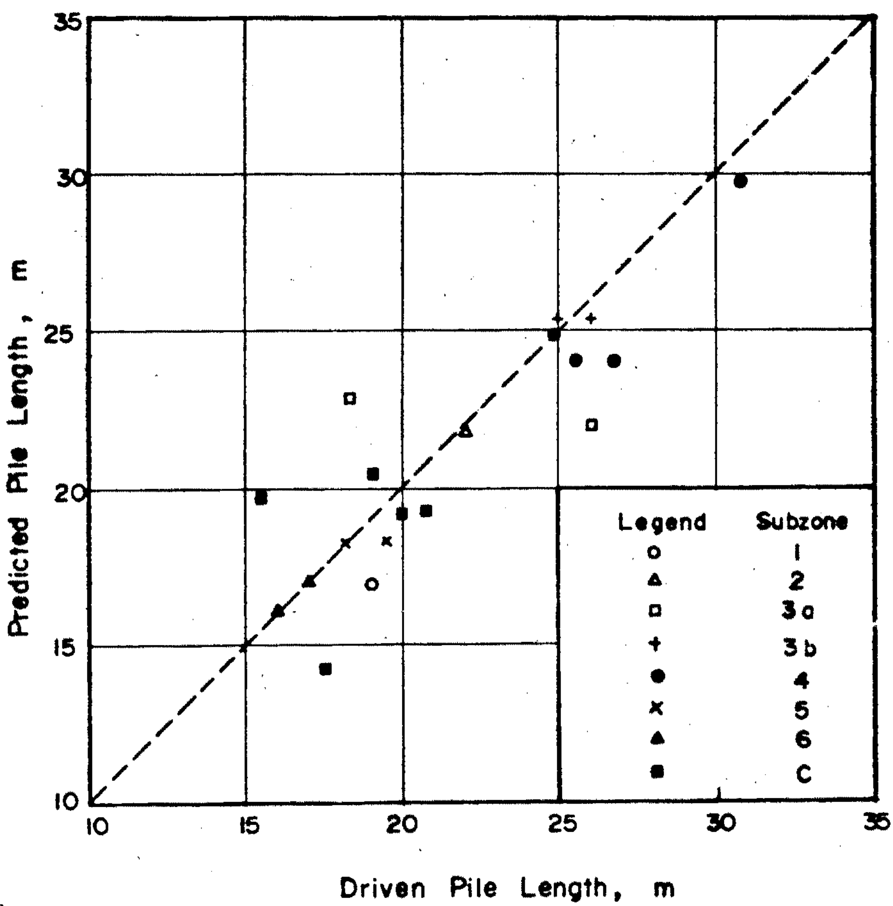

Fig. 5 Predicted Pile Length versus Driven Pile Length

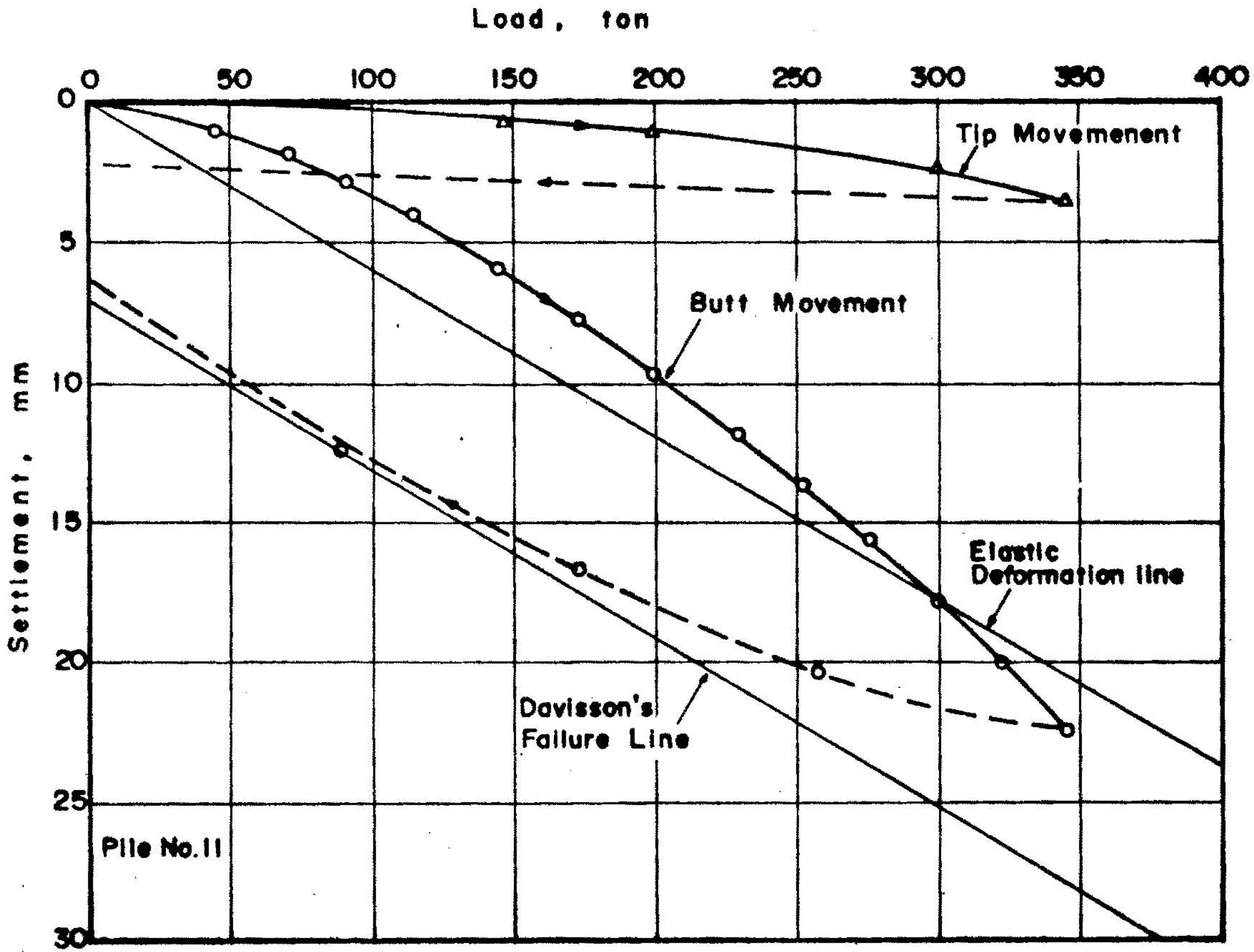

Fig. 6 Typical Load-Settlement Curve of a Test Pile

449

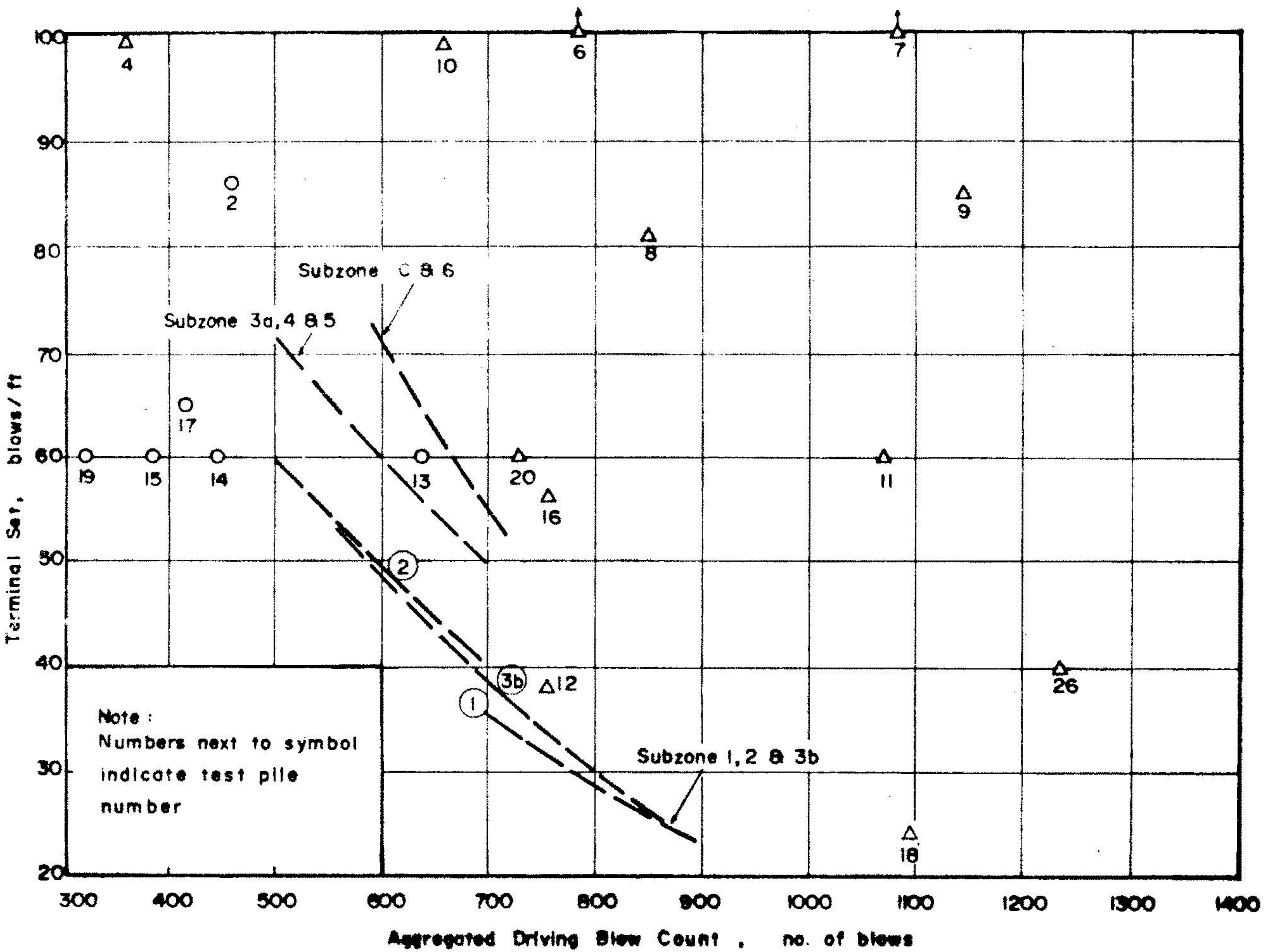

Fig. 7 Terminal Driving Set versus Aggregated Driving Blow Counts

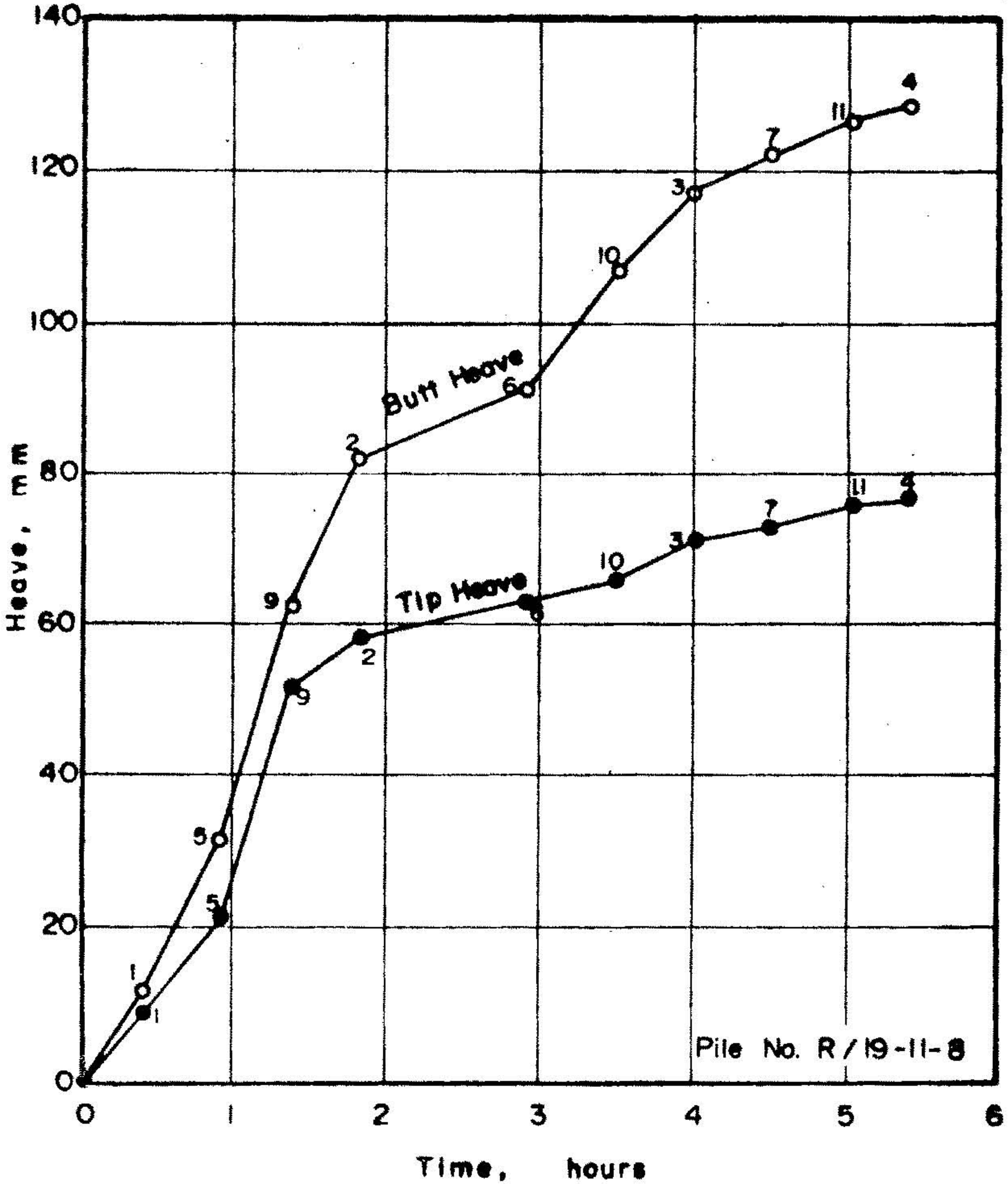

Fig. 8 Measured Heave of a Pile due to Driving of Adjacent Piles

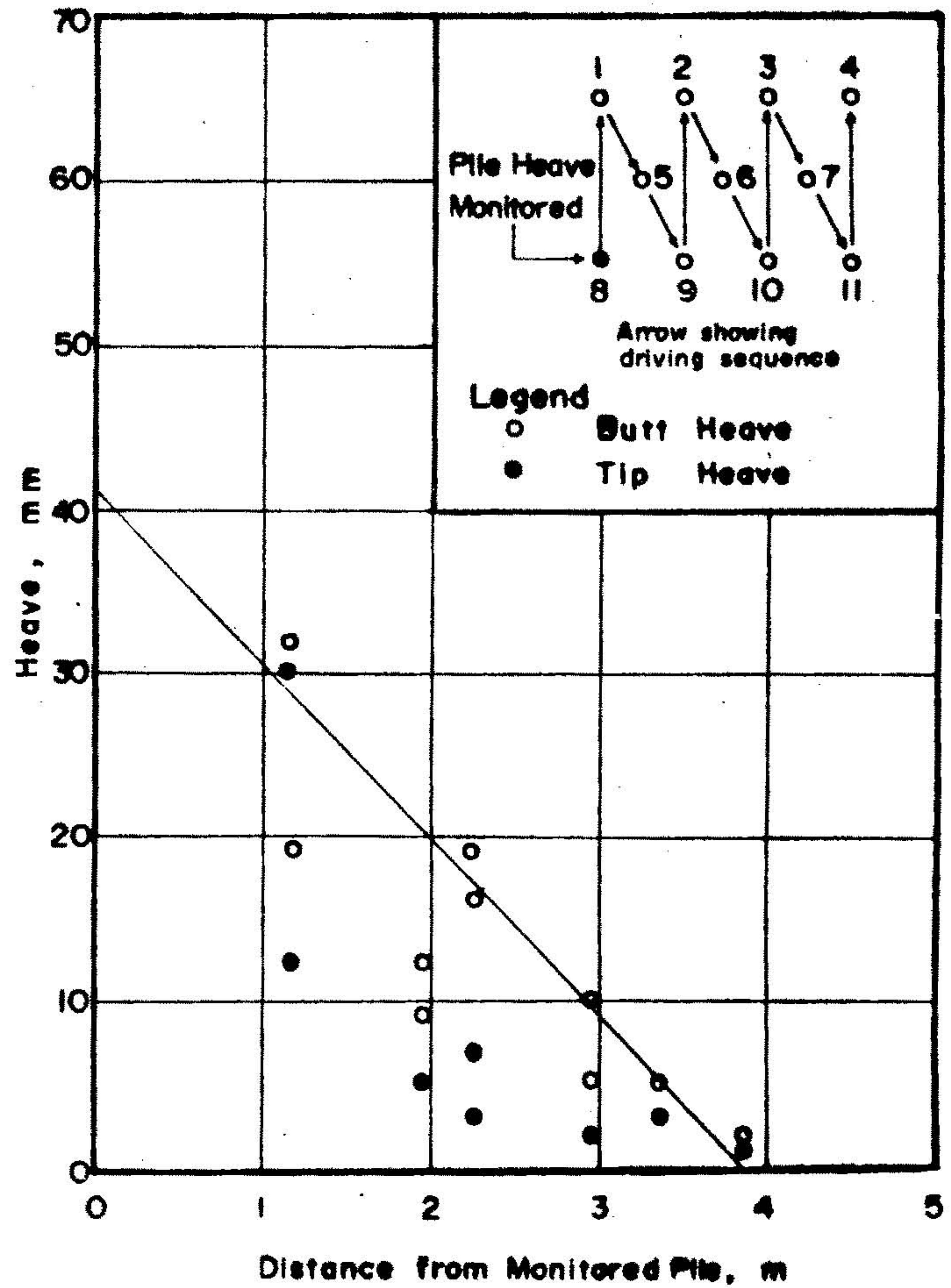

Fig. 9 Heave versus Distance between Piles

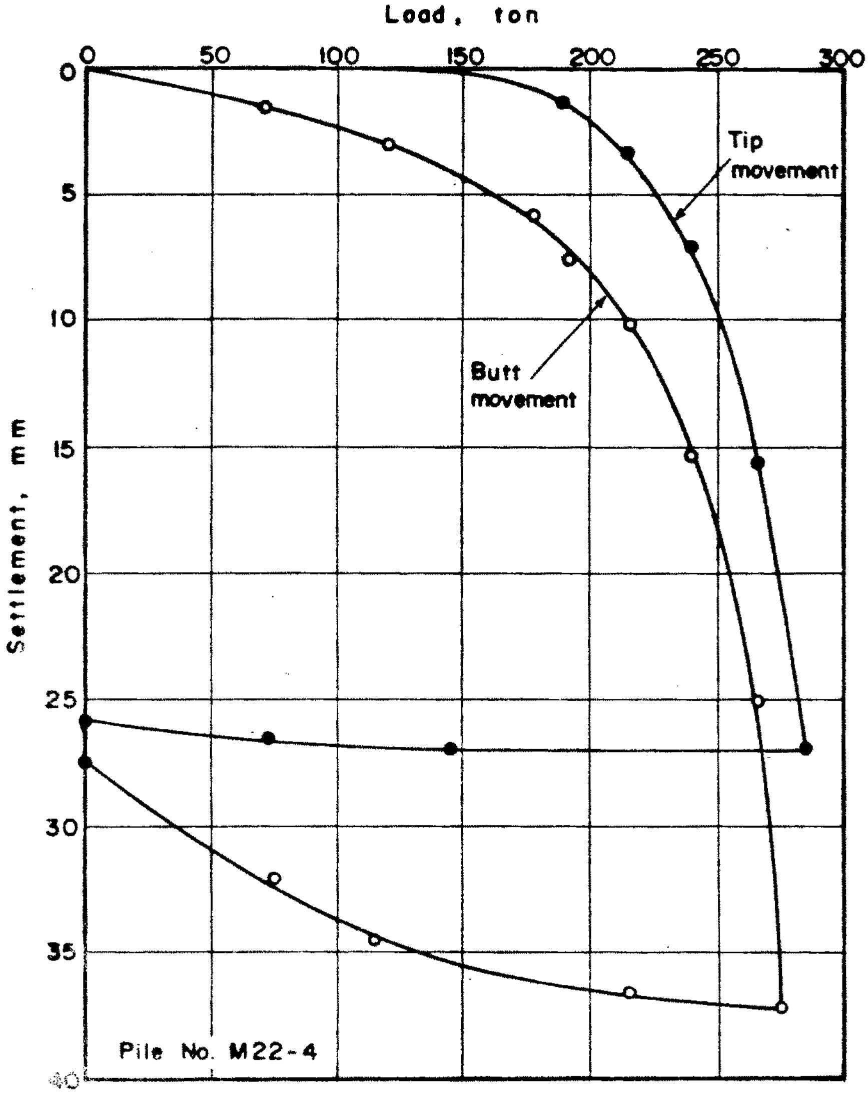

Fig. 10 Load - Settlement Curve of a Heaved Pile

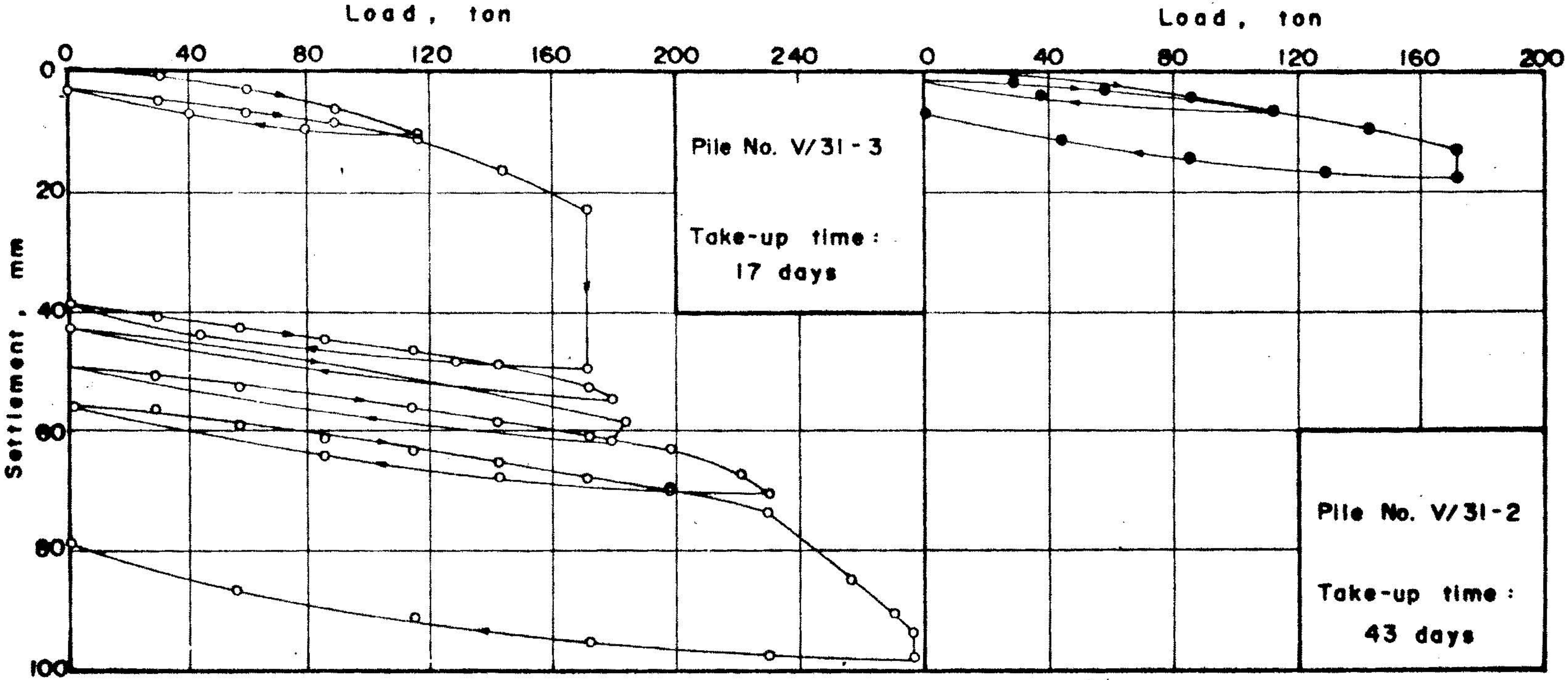

Fig. 11 Effect of Take-up Time on Pile Loading Test