US Army Corps
of Engineers

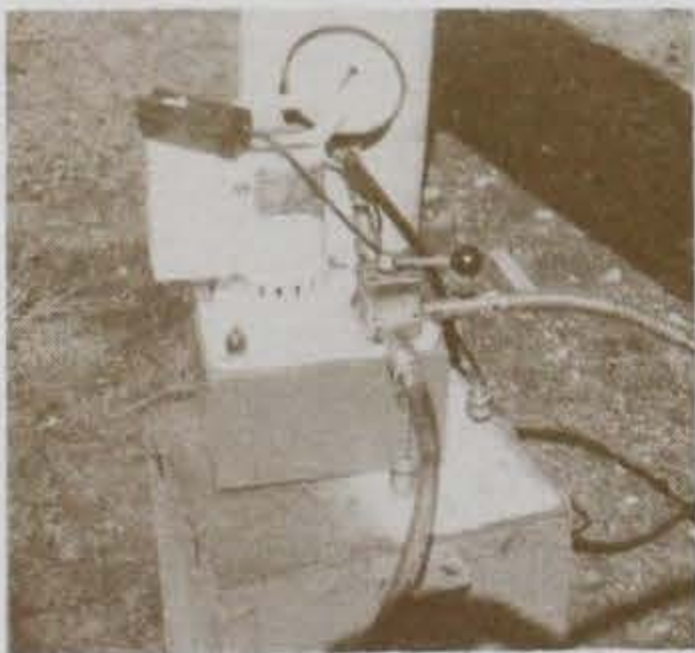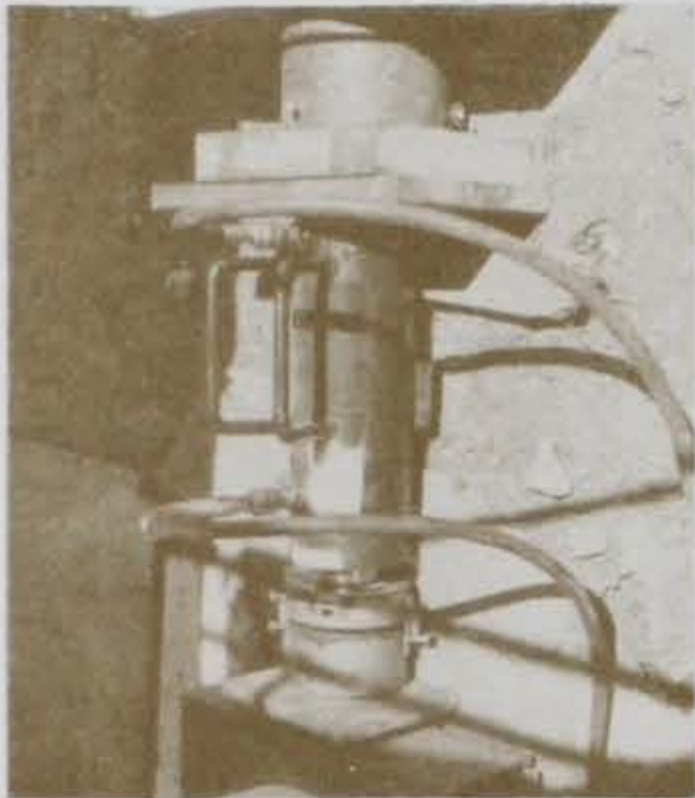# KWAJALEIN DRYDOCK PILE FOUNDATION ANALYSIS

by

Lawrence D. Johnson

Geotechnical Laboratory

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
3909 Halls Ferry Road, Vicksburg, Mississippi 39180-6199

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13. ABSTRACT (Maximum 200 words)

The MCA 9206180 dry dock facility being constructed on Kwajalein Island of the Marshall Islands is to be supported by 12 groups of driven piles, 6 groups supporting each side of the dry dock. Each group contains 12 piles. Soil investigation reports of the calcareous coral sands and static pile load tests conducted near the headwall (landside of the dry dock) indicated that 20 in. by 20 in. by 85 ft long precast prestressed concrete piles embeded to a depth of 53 ft will adequately support the dry dock.

Driving records of the production piles indicated that the penetration resistances of the piles at the final embedment depth became substantially less than expected as piles were driven further toward the lagoon end of the dry dock area. The penetration resistance N required for adequate bearing capacity using a Delmag 46-23 hammer rated at 60 kip-ft was determined to be 12 blows/ft at the final toe (tip) elevation, but the actual N decreased to as low as 2 blows/ft for piles driven near the lagoon end of the dry dock. The lower than expected penetration resistances observed during driving of the production piles were attributed to several mechanisms that include generation of excess pore pressures as a result of driving, encounter of loose (weak) sands or sands

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less dense at the lagoon end compared with those near the headwall, and destruction of cementation bonds in the coral sands.

The capability of the pile foundation to adequately support the dry dock could not be determined from the static load test results performed on piles driven near the headwall and from other existing data. The prudent course of action was to complete a supplemental field investigation of the offshore production piles. This investigation was conducted 7 months after the installation of the pile foundation and included a static load test, the driving of an indicator pile, and the restrikes of 10 of the production piles. The results of this investigation showed that the pile foundation has adequate bearing capacity to support the dry dock. The 7-month delay before initiating the supplemental test program permitted dissipation of any excess pore pressures. Evidence was found indicating that driving of the production piles had densified the sands and could have contributed to generation of excess pore pressures. The delay prior to the supplemental investigation also could have provided time for a recementation mechanism to occur (or to at least begin) in the coral sands.

PREFACE

This study, conducted by the Geotechnical Laboratory (GL) of the US Army Engineer Waterways Experiment Station (WES), confirms that the pile foundation constructed to support the Kwajalein dry dock is adequate and that construction should continue. This report completes Military Interdepartmental Purchase Request Number E87920010 dated 5 Dec 91 from US Army Engineer Division, Pacific Ocean (POD). Other services required to complete this work included a site visit to Kwajalein Atoll 24 February to 5 March 1992 to observe and evaluate the supplemental pile testing program. A briefing of this visit and status report of the work is provided in CEWES-GS-S Memorandum, 30 April 1992, Subject: Trip Report and Status of Kwajalein Dry Dock Construction Project. WES review comments on the A-E report by Frederic R. Harris, Inc., 1992, "Design Analysis Report for Investigation of Load Capacity of Existing Piles at DryDock Facility, US Army Kwajalein Atoll, Marshall Islands," were provided 22 May 1992. The final report by Frederic Harris was completed June 1992.

The Architect-Engineer Design Consultant was Daniel, Mann, Johnson & Mendenhall (DMJM) of Los Angeles, California. The subconsultant for completing the supplemental testing program was Frederic R. Harris, Inc. of New York. Field engineer for Frederic Harris was Mr. Kevin Pierce of San Pedro, California. Dr. D. Michael Holloway, President of InSitu Tech, Inc., Oakland, California, was the pile-driving analyzer consultant. Mr. Olson T. Okada was Project Engineer for POD. This report was prepared by Dr. Lawrence D. Johnson, Research Group (RG), Soil & Rock Mechanics Division (S&RMD), Geotechnical Laboratory (GL), US Army Engineer Waterways Experiment Station (WES).

Many helpful comments were provided by Dr. P. H. Hadala, Assistant Director, GL, Dr. E. B. Perry, RG, S&RMD, GL, Mr. O. T. Okada, Foundations, Materials and Survey Division, POD, and Mr. W. M. Myers, Chief, Soil Mechanics Branch, S&RMD.

This work was performed under the direct supervision of Mr. W. M. Myers and Dr. D. C. Banks, Chief, S&RMD, GL. Dr. William F. Marcuson III was Director, GL.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander and Deputy Director was COL Leonard G. Hassell, EN.

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CONVERSION FACTORS, NON-SI TO SI (METRIC)
UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI
(metric) units as follows:

Multiply	By	To Obtain
cubic feet	0.02831685	cubic metres
cubic yards	0.7645549	cubic metres
Fahrenheit degrees	5/9	Celsius degrees or Kelvins*
feet	0.3048	metres
inches	2.54	centimetres
kips (force)	4.448222	kilonewtons
miles (US statute)	1.609347	kilometres
pounds (force)	4.448222	newtons
pounds (force) per inch	175.1268	newtons per metre
pounds (force) per square inch	6.894757	kilopascals
pounds (force) per square foot	47.88026	pascals
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
pounds (mass) per cubic yard	0.593276	kilograms per cubic metre
square feet	0.09290304	square metres
square inches	6.4516	square centimetres
square yards	0.8361274	square metres
tons (force)	8.896444	kilonewtons
tons (mass) per square foot	9,764.856	kilograms per square metre
tons (2,000 pounds, mass)	907.1847	kilograms
yards	0.9144	metres

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* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use
the following formula: $C = (5/9)(F - 32)$. To obtain Kelvin (K) readings, use
$K = (5/9)(F - 32) + 273.15$.

KWAJALEIN DRYDOCK PILE FOUNDATION ANALYSIS

PART I: INTRODUCTION

Background

1. Military Construction, Army project number 9206180, dry dock facility, is being constructed on Kwajalein Island of the Marshall Islands. This dry dock is to be supported by a series of 12 pile groups, N-1 to N-6 on the northwest and S-1 to S-6 on the southeast side as located in the pile plan, Figure 1. These "production" piles, piles that support the dry dock,
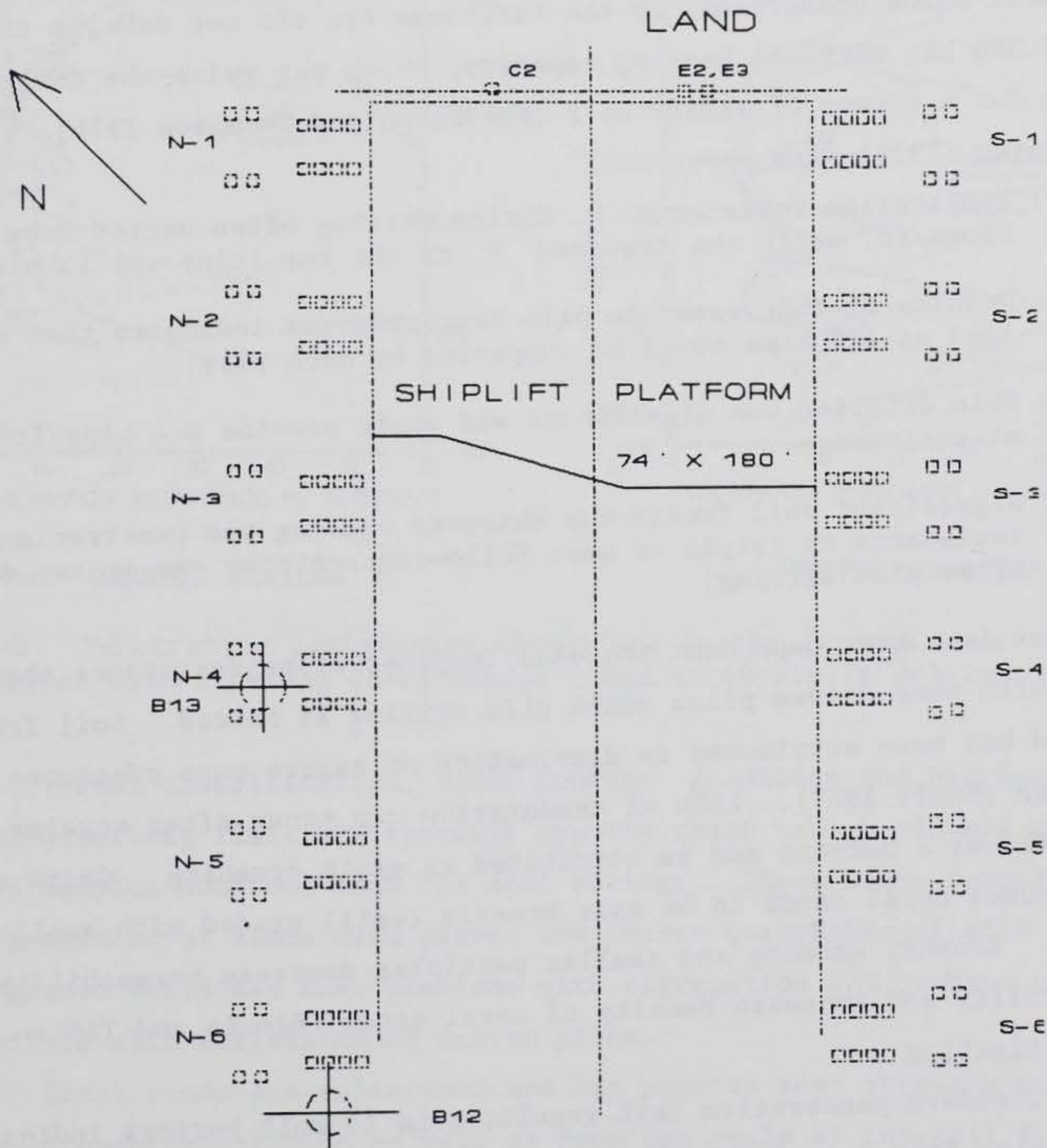
LAND

Figure 1. Pile plan

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were driven into unconsolidated bioclastic limestone (coral) debris with sizes that range from silt to cobble, although the dominant size is sand. The dry dock area was dredged to -26 ft mean sea level (MSL) prior to pile driving. The area was also drilled and blasted during dredging operations.

2.   Previous experience with coral sands indicated that the bearing capacity of driven piles can be uncertain and less than anticipated. A model pile test indicated that coral sand particles can be crushed during pile driving and can cause low skin friction due to reduced lateral pressure on the pile surface (McCarel and Beard 1984). Sixteen inch square precast, prestressed (PCPS) concrete piles driven 80 ft below the seabed into loose to dense coral sands of Barbados in the Caribbean Sea did not develop the required 300 kip ultimate bearing capacity, which was twice the design load of 150 kips for a factor of safety of 2 (Stevenson and Thompson 1978). Stevenson and Thompson (1978) also observed:
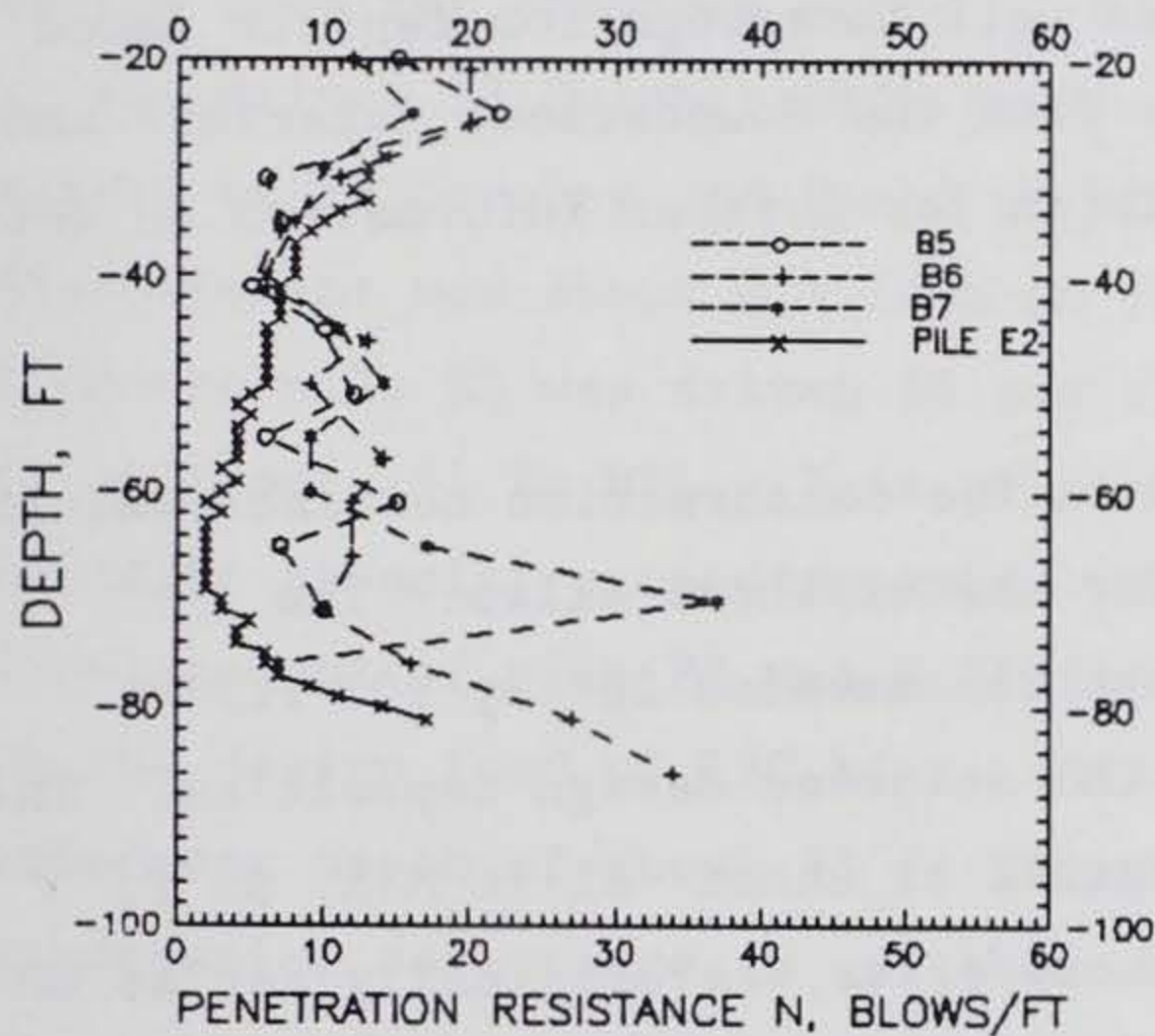
(1) Penetration resistance N during driving often varied from 3 to 6 blows/ft, while the required N at the toe (tip) was 13 blows/ft.

(2) Results of four separate pile test programs indicated that a design load of 120 kips could be supported by each pile.

(3) Skin friction was significant and would provide 0.4 kips/ft^2 (ksf) of resistance.

(4) Significant soil freeze was observed causing the penetration resistance to triple or more following restrike one or two days after pile driving.

Soil freeze is a time-dependent increase in penetration resistance that is observed with some driven piles after pile driving is halted. Soil freeze in coral sand has been attributed to dissipation of excess pore pressures and cementation (Murff 1987). Loss of cementation can occur after strains of only a few tenths of a percent and is attributed to grain crushing. Grain crushing in turn causes coral sands to be more broadly (well) graded with smaller particles. Broader grading and smaller particles decrease permeability and compressibility and increase density of coral sands (Blouin and Timian 1986).

Soil Investigation

3.   Standard penetration test results from 13 soil borings indicated N-values ranging from 3 to 55 blows/ft, which correlate with very loose to very dense sands and gravels with most sands of medium density. The penetration

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resistances of borings B5, B6, and B7 in Figure 2a are representative of the coral sands in the onshore dry dock area. The penetration resistances of offshore borings B2 and B4, located about 100 ft south of the centerline in Figure 1 of the pile plan, and borings B12 and B13 located as shown in Figure 1 are given in Figure 2b. Other borings are not within the area of Figure 1. Depth is relative to MSL. Comparison of Figures 2a and 2b shows that the penetration resistances in the offshore borings are at least as great or greater than those of the onshore borings. This comparison assumes that driving energies delivered to all of the borings were identical.
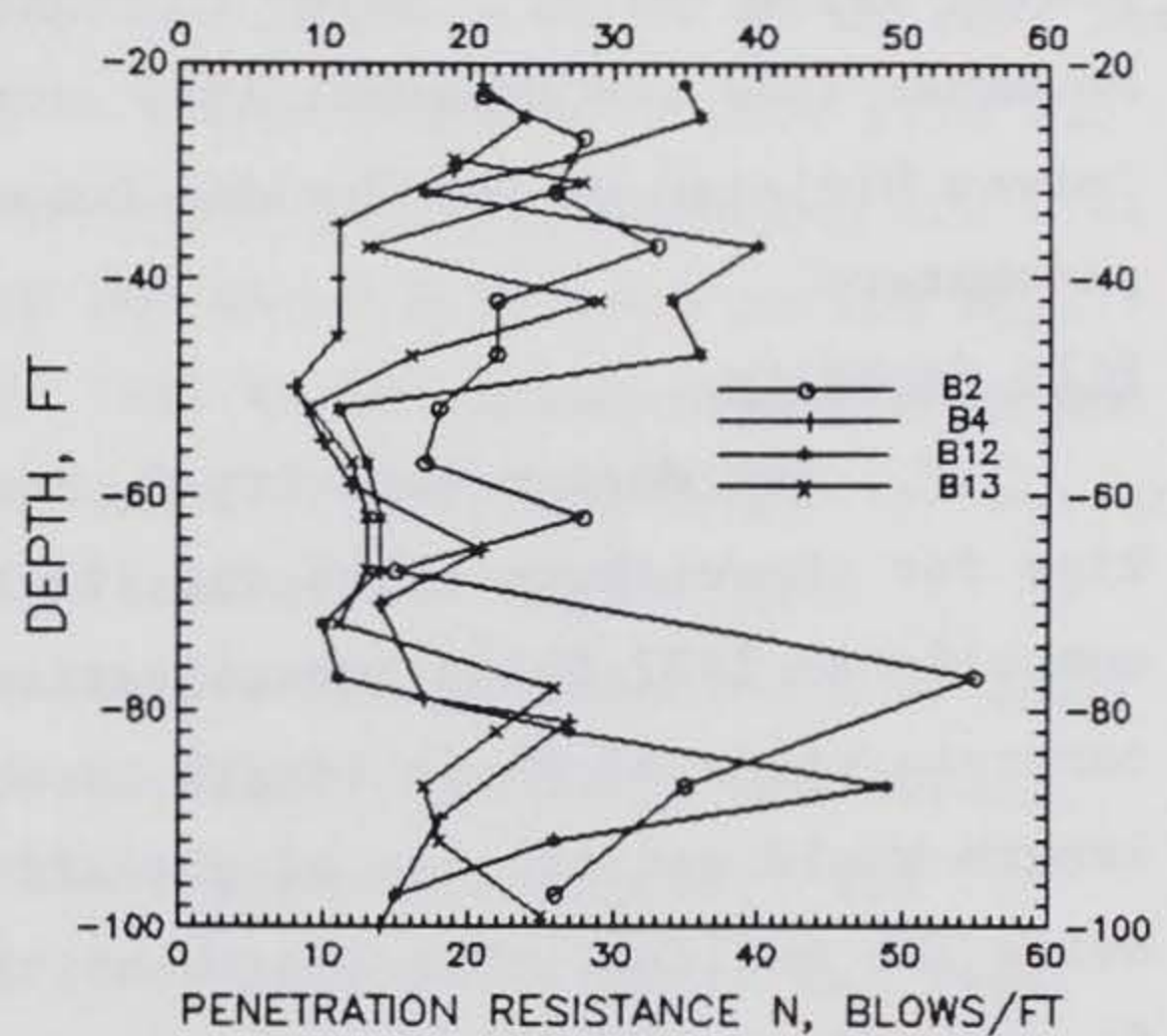
| a. ONSHORE BORINGS | b. OFFSHORE BORINGS |

Figure 2. Penetration resistances of borings in the drydock area. Pile E2 was driven with a Delmag 46-23 hammer rated at 60 kip-ft driving energy.

4. Visual classification, water content, gradation and hydrometer analysis, Atterberg limit and specific gravity tests were performed on disturbed samples obtained from the soil borings. These tests indicate broadly graded coral sands with gravel and lesser quantities of silt and clay. Broadly graded soils may have problems with segregation and perhaps contribute to nonuniform skin resistance of driven piles.

5. Coral sands are calcareous and may provide some strength and bearing capacity through cementation as well as from the angle of internal friction. Bearing capacity contributed by cementation may not be permanent because loads applied to the piles may breakdown cementation between particles and between

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particles and the pile, especially since small strains appear sufficient to break cementation.

6. Penetration resistance of the soil borings in Figure 2 are variable with depth indicating variations in density. Gradation analyses confirmed significant differences in the distribution of gradation with depth. For example, boring B2 contained medium dense silty sand with coral gravel from -23 to -37 ft MSL, medium dense to dense silty coral gravel with fine to coarse grain sands from -37 to -65 ft, medium dense to dense silty, gravelly sand from -65 to -89 ft and dense to very loose silty sand with some coral gravel below -89 ft. Refer to unpublished soil investigation reports dated November 1987 and December 1989 available from the Foundation, Materials and Survey Division of the Pacific Ocean Division for further information on soil parameters.

Pile Capacity

7. The design capacity Q_d required in the construction contract was 160 kips for the onshore piles and 120 kips for the offshore piles. The unpublished 1987 Soils Investigation report recommended 20" by 20" PCPS concrete piles of 85-ft length to obtain the selected design capacities. This length would set the toe of the offshore piles at about -81 ft MSL, 53 ft below the mudline. The length of the onshore piles was originally set at 60 ft and driven with 60-ft lengths in January 1991, but 25-ft sections were spliced and driven in February 1992 to obtain the full 85-ft length. The standard penetration test (SPT) penetration resistance N_{SPT} was expected to exceed 15 blows/ft based on data from the borings in Figure 2b. Past experience, results of load tests, and wave equation analyses indicated that the penetration resistance of the production piles should be at least 12 blows/ft using a Delmag 46-23 hammer rated at 60 kips-ft driving energy.

8. Three proof load tests were conducted on onshore piles E3 and C2 and offshore pile E2, located approximately as shown in Figure 1, according to the ASTM D1143 Quick Load Test procedure. A proof test is one where loads are applied to twice the design load to determine that the piles have adequate bearing capacity, but may not indicate ultimate capacity or the extent of the conservatism in the design. The ASTM Quick Load Test requires loads to be applied in increments of 10 to 15 percent of the proposed design load with a constant time interval between load increments of 2 1/2 minutes or as

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otherwise specified. Quick load tests do not consider long-term effects on pile performance such as consolidation and creep of the foundation soils caused by applied loads. Consolidation and creep in coral sands were not expected to be significant.

9. Onshore pile E3 was driven 10 October 1990 to an embedment depth of 59 ft, using a Delmag 46-23 hammer. The length of this pile was 60 ft consistent with the original design recommendation. This pile was tested on 15 October and found to have an ultimate capacity of 180 kips at a displacement of about 0.35 inch. Displacement of pile E3 was 0.1 inch at the design load for offshore piles of 120 kips. Onshore pile C2 is 85 ft long and it was driven 13 October 1990 to an embedment depth of 81 ft. This pile was tested on 18 October and found to have an ultimate capcity exceeding 320 kips. Displacement was about 0.3 inch at 320 kips and about 0.10 inch at 160 kips. Offshore pile E2 was driven 26 and 27 April 1991 to an embedment depth of 53 ft at about -81 ft MSL. This pile was tested on 27 April and found to have an ultimate capacity exceeding twice the design load of 240 kips. The displacement of pile E2 was 0.25 inch at twice the design load and 0.07 inch at the design load of 120 kips. Pile E2 most closely simulates the actual embedment depth of about 53 ft for the offshore production piles. The penetration resistance N of this pile as it was driven 26 April was less than that of the onshore borings below -40 MSL, as shown in Figure 2a, and as low as 2 blows/ft. Although energy levels during pile driving and boring sampling were probably different so that N-values are not comparable, the penetration resistance of pile E2 was so low that another mechanism such as generation of excess pore pressures could have occurred during the driving of pile E2 to reduce the observed N. Driving was discontinued when pile E2 was 6 ft above the final toe elevation. N increased to 17 blows/ft at the final toe elevation of about -81 ft MSL when driving was continued on 27 April. This soil freeze effect is similar to that observed during installation of the 16" by 16" PCPS concrete piles at Barbados.

10. The static load tests indicated adequate reserve capacity with displacements not exceeding 0.1 inch at the design loads. The entire foundation was expected to have adequate bearing capacity and minimal settlement $\leq$ 0.1 inch if the coral sands supporting the offshore piles are similar to those supporting the onshore test piles.

11. Driving records obtained during installation of the production piles through June 1991 indicated that the penetration resistances of the offshore piles at the final toe elevation decreased significantly from the land side of the dry dock (headwall) toward the lagoon side (far end) of the proposed dry dock; i.e., from 6 to 37 blows/ft near the headwall down to 2 to 12 blows/ft near the far end. Therefore, the capability of the offshore piles to support the dry dock was in question. The bearing capacity of the onshore piles was considered adequate.

Purpose and Scope

12. The purpose of this analysis is to determine the capability of the pile foundation to support the full length of the dry dock. The scope consists of evaluating the bearing capacity and settlement of the piles to determine if the pile foundation can adequately support the dry dock and what redesign is required, if any.

13. A supplemental field test program was performed to obtain data necessary to complete the analyses. This program was completed after a significant time delay to allow dissipation of any excess pore pressures generated during driving of the production piles and to consider the influence of sand densification from drivng of the production piles. The first activity in this program was to perform a static load test on an offshore production pile near the far end of the dry dock to determine the current or "long-term" pile capacity, which excludes the influence of any excess pore pressures. An additional indicator pile was driven offshore near the far end to determine the long-term penetration resistance, which includes influence of any sand densification caused by earlier driving of the production piles. The bearing capacity of this indicator pile was determined with the assistance of wave equation and pile driving analysis (PDA). Ten production piles including the static load test pile, were also restruck to obtain additional information on the long-term bearing capacity. These data were analyzed to gain further information on the cause of the low penetration resistances observed for the offshore production piles driven toward the far end of the dry dock compared to piles driven near the headwall. The data also indicated the ability of the offshore piles to support the dry dock.

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PART II. ANALYSIS OF PRODUCTION PILE PENETRATION RECORDS

14. The production piles were driven using a Delmag 46-23 hammer with an energy variation from 45 to 102 kips-ft for fuel settings 1 to 4. Actual driving energy from the pile load test report on pile E2 was 60.72 kip-ft at a 6-ft stroke. For wave equation analysis, the rated hammer energy E_r was selected as 60 kip-ft with an efficiency $E_h = 0.8$. Pile modulus of elasticity E_p was taken as 3000 ksi and the pile weight as 35 kips. Hammer weight W_r was approximately 20 kips and considered to be the total weight of the striking parts of the ram. Most of the remaining input parameters for wave equation analysis were automatically selected by the wave equation computer program.

15. Computer program GRLWEAP (GRL 1988), licensed to the Corps of Engineers, was used to estimate bearing capacity by wave equation analysis. Bearing capacity was also estimated from pile driving equations and estimates of the angle of internal friction of the coral sands. The production pile driving records were used to obtain the penetration resistances required for evaluation of the bearing capacity by wave and pile driving equations. Settlement of the production piles was estimated by Vesic's method (EM 1110-2-2906) for piles subject to the offshore design load of 120 kips.

Pile Driving Records

Embedment Depth

16. The production pile embedment depths L are given in Table 1. Average embedment depth is about 53 ft. The North group appears to have a slighly smaller average embedment depth than the South group, which could cause the North group to have slighly smaller penetration resistances and bearing capacities than the South group. The embedment depth for one pile in group N-3 was 44 ft and several piles were embeded 49 or 50 ft; otherwise, these piles were driven to an embedment depth from 51 ft to 61 ft (-71 to -81 ft MSL). Matching this depth range with the penetration resistances of the borings in Figure 2 indicates that the boring penetration resistance N_{SPT} was expected to be about 10 blows/ft or greater.

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Table 1
Production Pile Embedment Depths

North Group	Embedment Depth L, Ft			South Group	Embedment Depth L, FT		
	Average	Low	High		Average	Low	High
N-1	55	49	61	S-1	54	51	59
N-2	53	49	58	S-2	53	51	57
N-3	52	44	58	S-3	53	51	57
N-4	52	49	58	S-4	54	52	57
N-5	53	47	57	S-5	55	51	59
N-6	54	50	57	S-6	55	52	58
Ave	53	48	58	Ave	54	51	58

Penetration Resistance

17. The distribution of penetration resistance at the final toe elevation of the offshore piles was plotted with distance from the headwall in Figure 3. Driving was begun in late April 1991 near the headwall and completed by early July at the far end. The penetration resistance decreases
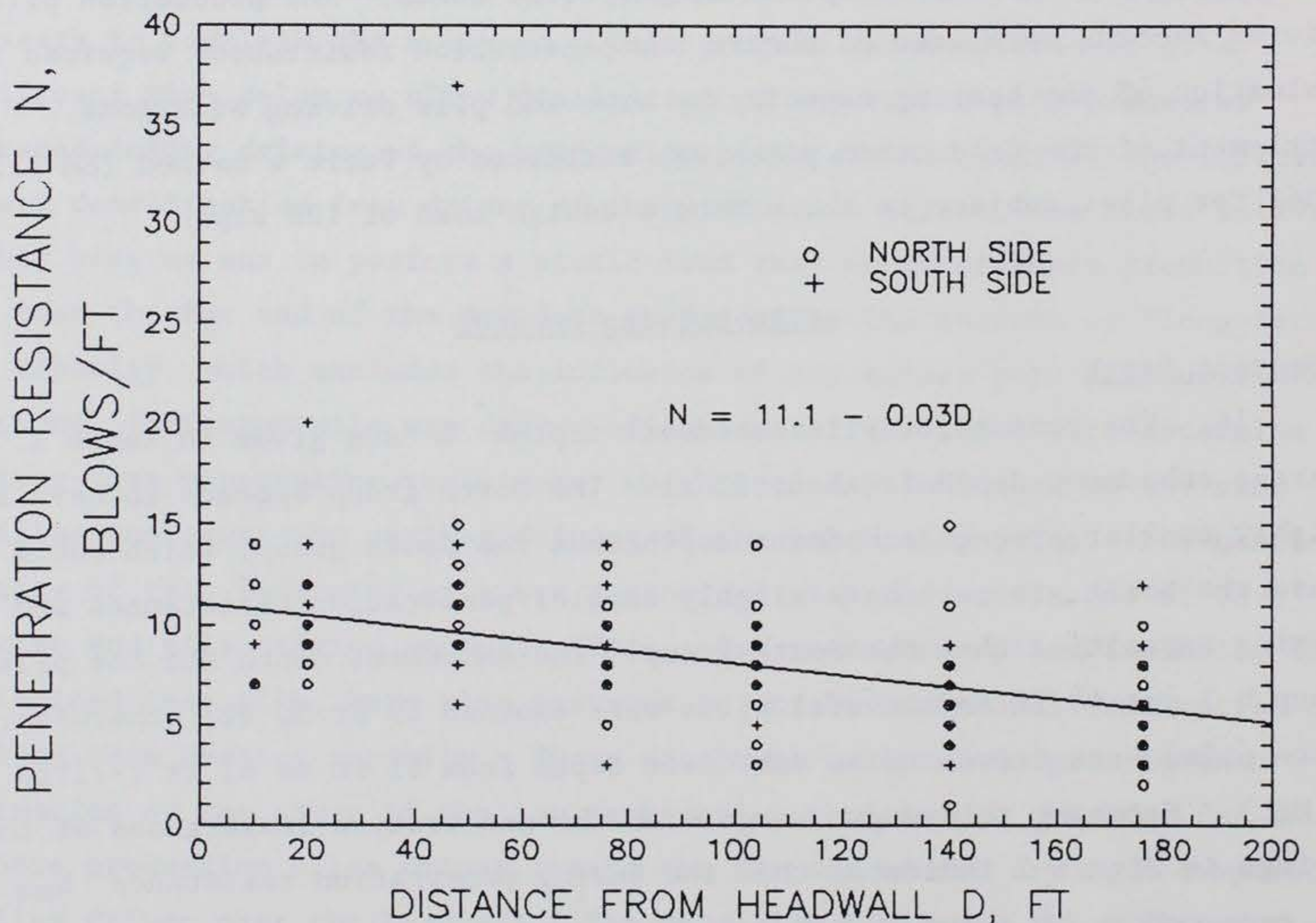

Figure 3. Pile driving record at point bearing depth - 75 MSL

significantly as the distance increases from the headwall. Linear regression
analysis of the penetration resistance data in Figure 3 shows that the
blows/ft value N when the pile is driven to the final toe elevaton may be
approximated by

$$N = 11.1 - 0.03D$$

(1)

where D is distance from the headwall in feet. The correlation coefficient
is $r^2 = 0.32$, which indicates a relatively low correlation. r^2 close to 1.00
indicates an optimum correlation.

18. Average penetration resistances N at the point bearing depth and
at distances 20 and 176 ft from the headwall using Equation 1 are
approximately 11 and 6, respectively. These N values are used to estimate
ultimate bearing capacity and potential for settlement of the offshore piles
near the headwall and the far end of the dry dock.

19. The penetration resistances at the toe of each group of production
piles are shown in Table 2. These data indicate that the penetration
resistance at the toe depth of the North group is slightly less than that for
the South group. The slightly smaller resistances observed for the North
group is consistent with the slightly smaller embedment depth of the North
group compared to that of the South group. Skin resistance and/or additional
end bearing could account for the additional penetration resistance with the
deeper embedment depth of the South group. Table 2 also shows that the
penetration resistance decreases at the toe elevation from 6 to 37 near the
headwall down to 2 to 12 near the far end.

Table 2

Production Pile Toe Penetration Resistance

North Group	Penetration Resistance N, Blows/Ft			South Group	Penetration Resistance N, Blows/Ft		
	Average	Low	High		Average	Low	High
N-1	8.9	7	10	S-1	12.3	9	20
N-2	10.7	9	15	S-2	12.0	6	37
N-3	8.8	5	13	S-3	8.8	3	14
N-4	7.8	3	14	S-4	8.0	5	10
N-5	7.3	4	15	S-5	6.6	3	10
N-6	5.9	2	10	S-6	6.4	3	12
Ave	8.2	5	13	Ave	9.0	5	17

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20. Pile driving records for group S-6 are shown in Figure 4 to
determine how the distribution of penetration resistance may vary with depth.
Figure 4 indicates relatively strong, dense layers near -35 ft, -55 ft and -75
ft MSL. Boring records of B2 and B12, which were closest to group S-6,
indicated medium dense silty sand with gravel near depths with the highest
pile penetration resistance. These strong layers are expected to contribute
to the skin resistance component of bearing capacity. Driving records show
penetration begins at about -22 to -30 ft MSL and that the pile toe is
typically at -81 ft MSL.
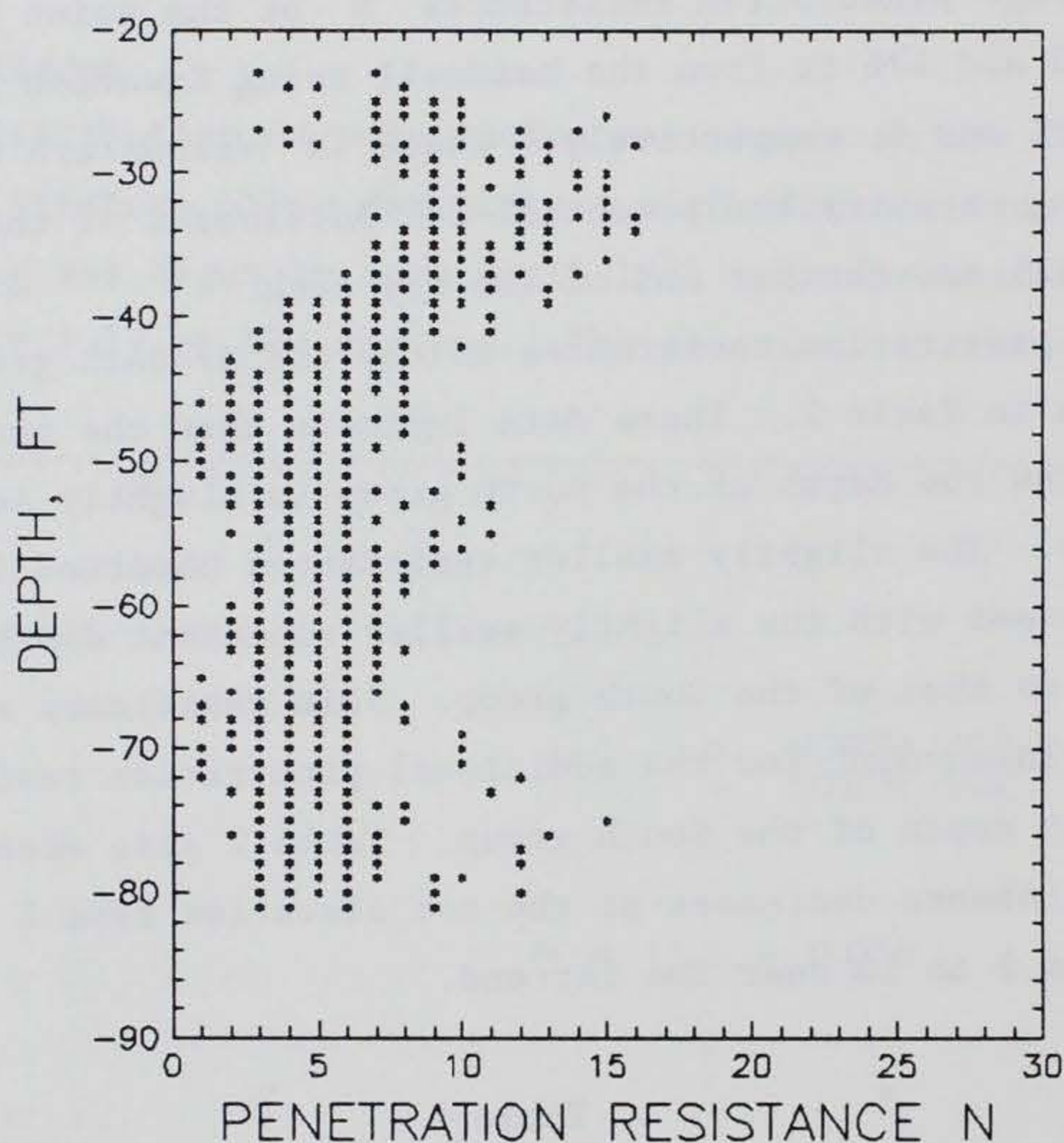
Figure 4. Penetration resistance with depth for group S-6

21. The penetration resistance at the toe depth of - 81 ft varies from
3 to 12 blows/ft, Figure 4, which is less than the 3 to 16 blows/ft observed
when the pile was driven through the strong layers and much less than the > 15
blows/ft expected at - 81 ft MSL on the basis of the penetration resistance of
test pile E2 in Figure 2a. Driving of the offshore production piles
apparently reduces the short-term penetration resistance at the toe elevation

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as driving continued toward the far end. Possible causes for this loss in penetration resistance include generation of excess pore water pressure and/or loss of cementation/chemical bonding between particles of the coral sands. The offshore coral sands may also be less dense than the onshore coral sands as a result of a different depositional environment, thus explaining the reduced penetration resistance of the piles near the far end, but the record of the penetration resistances of the borings in Figure 2 do not support this idea. Excess pore pressure can be generated and contribute to the lower than expected penetration resistance if continuous driving caused the sands to densify. These coral sands contain approximately 20 percent silts and clays which, along with cementation, are expected to contribute to low permeabiities and could support generation of temporary excess pore pressures. Densification of the sands during pile driving is also expected to increase the penetration resistance after the excess pore pressures had dissipated.
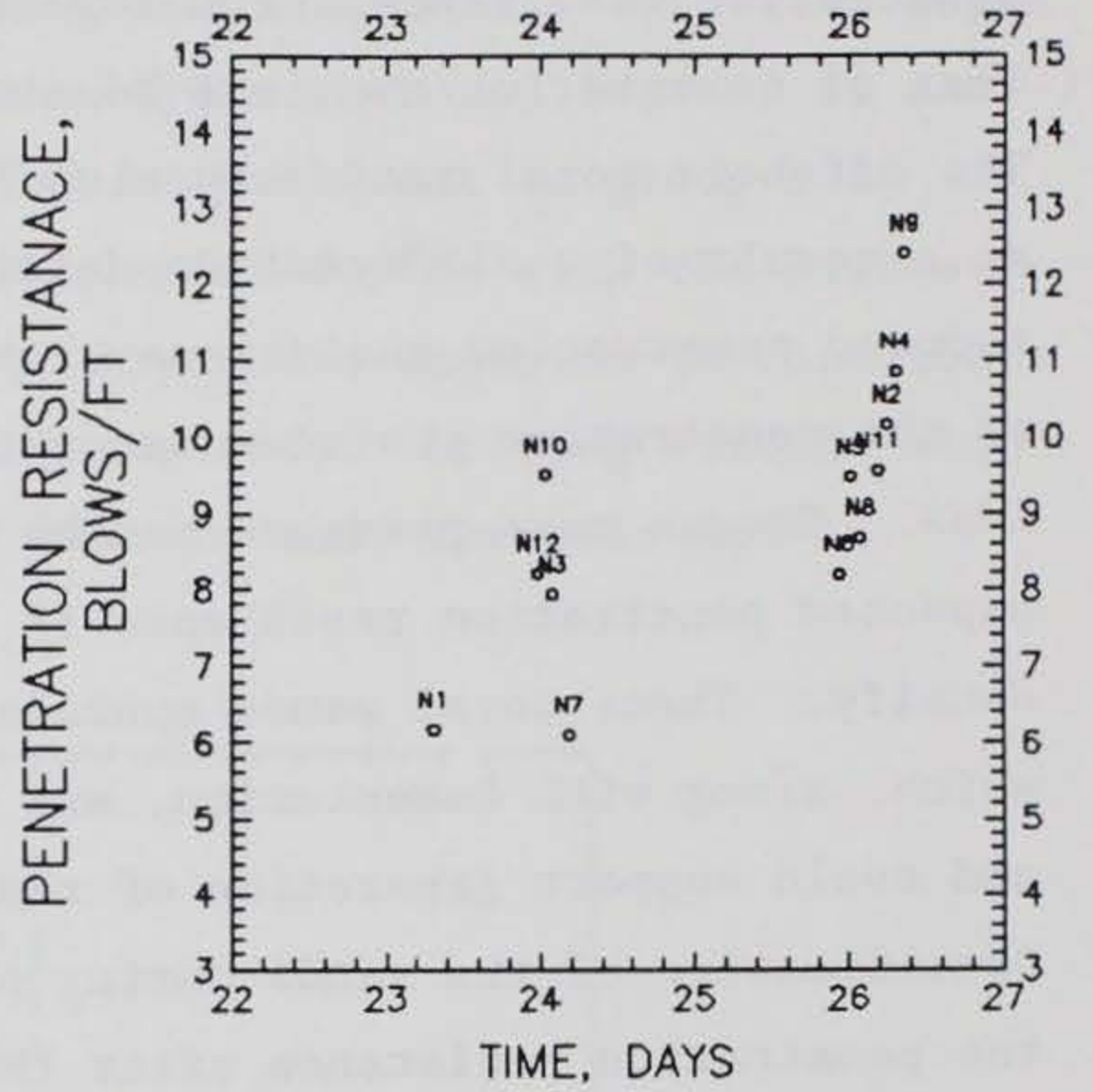
22. The mean penetration resistance N' of each of the piles in the North group was determined and plotted as a function of installation time in Figure 5. N' is the mean blow count of all of the N values recorded for a given pile during its installation. Standard deviations of N are approximately 2 blows/ft. The time of installation is shown relative to the first pile driven on 25 April 1991 at 0910 hours. The piles in a North group indicated in Figure 1 are numbered as shown in Figure 6. Piles N2, N4, N5, N8, N9, and N11 are battered in the directions indicated by the darkened triangles in Figure 6. Half of a group are battered piles.

23. Figure 5 shows that piles driven later in a particular group usually have a higher mean penetration resistance than those driven earlier. The mean penetration resistance N' of the last pile driven in a group is at least twice that of the first pile driven. Battered piles usually have greater resistance than the vertical piles, but the battered piles were driven after the vertical piles in a group. Some batter piles had lower resistance than vertical piles in a group. Batter does not appear to have any discernible difference in behavior relative to the other piles.

24. A probable cause of the increased penetration resistance with time in a group is densification of the sands as a result of driving. Soil densification should also have caused excess pore pressures, but any loss of penetration resistance that could occur from a buildup of excess pore pressure
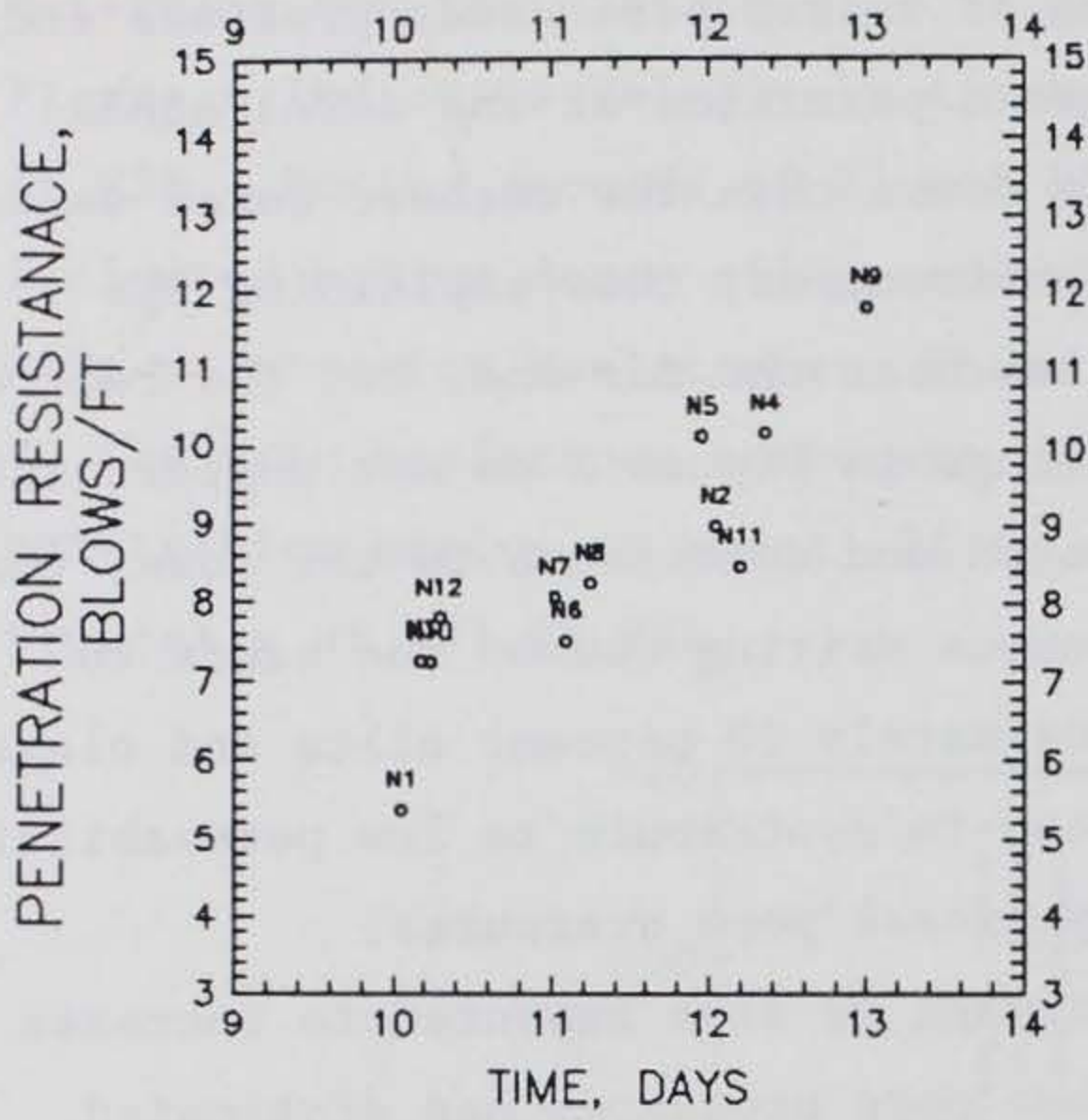
a. GROUP N-1

b. GROUP N-2
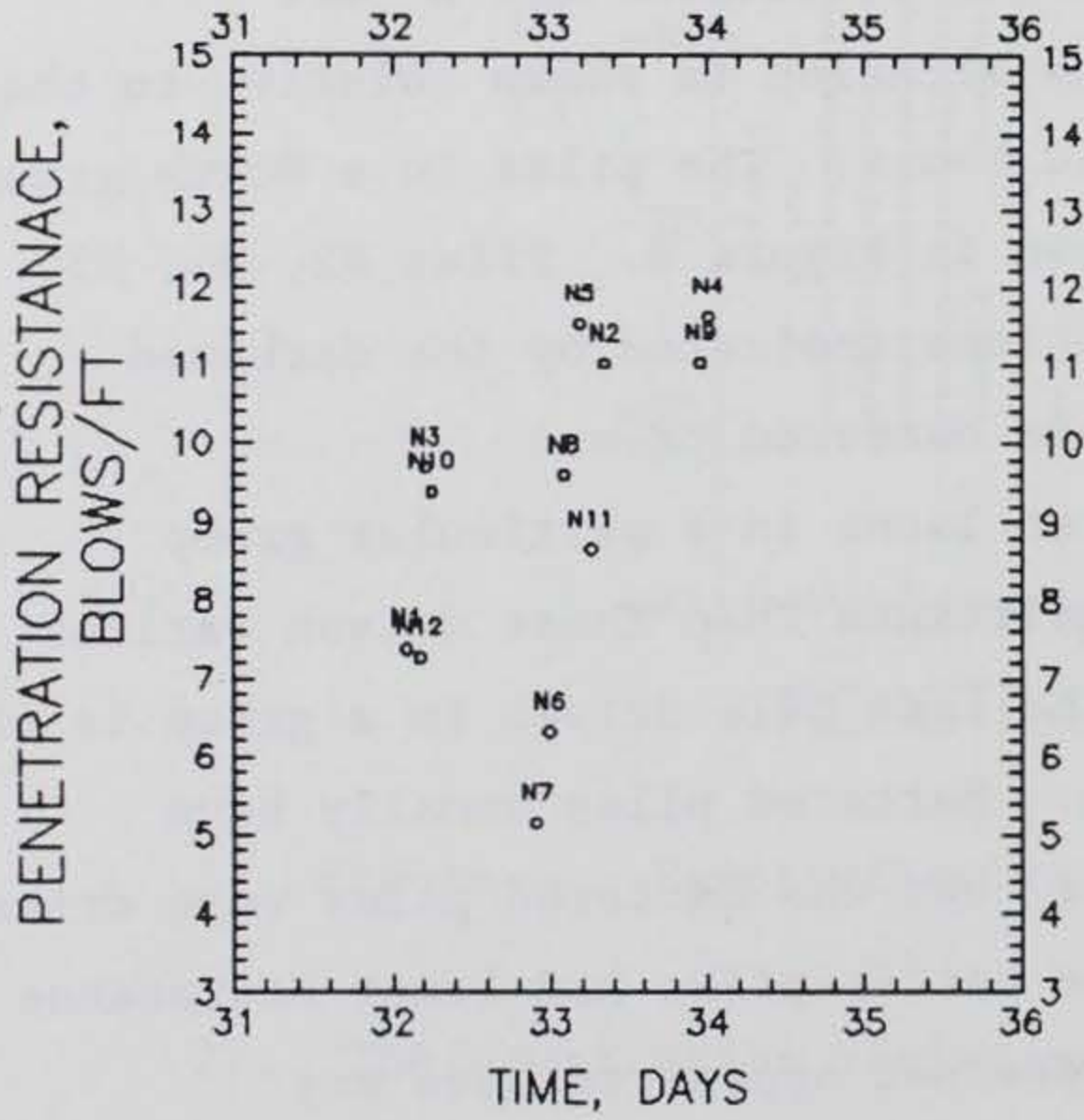
c. GROUP N-3
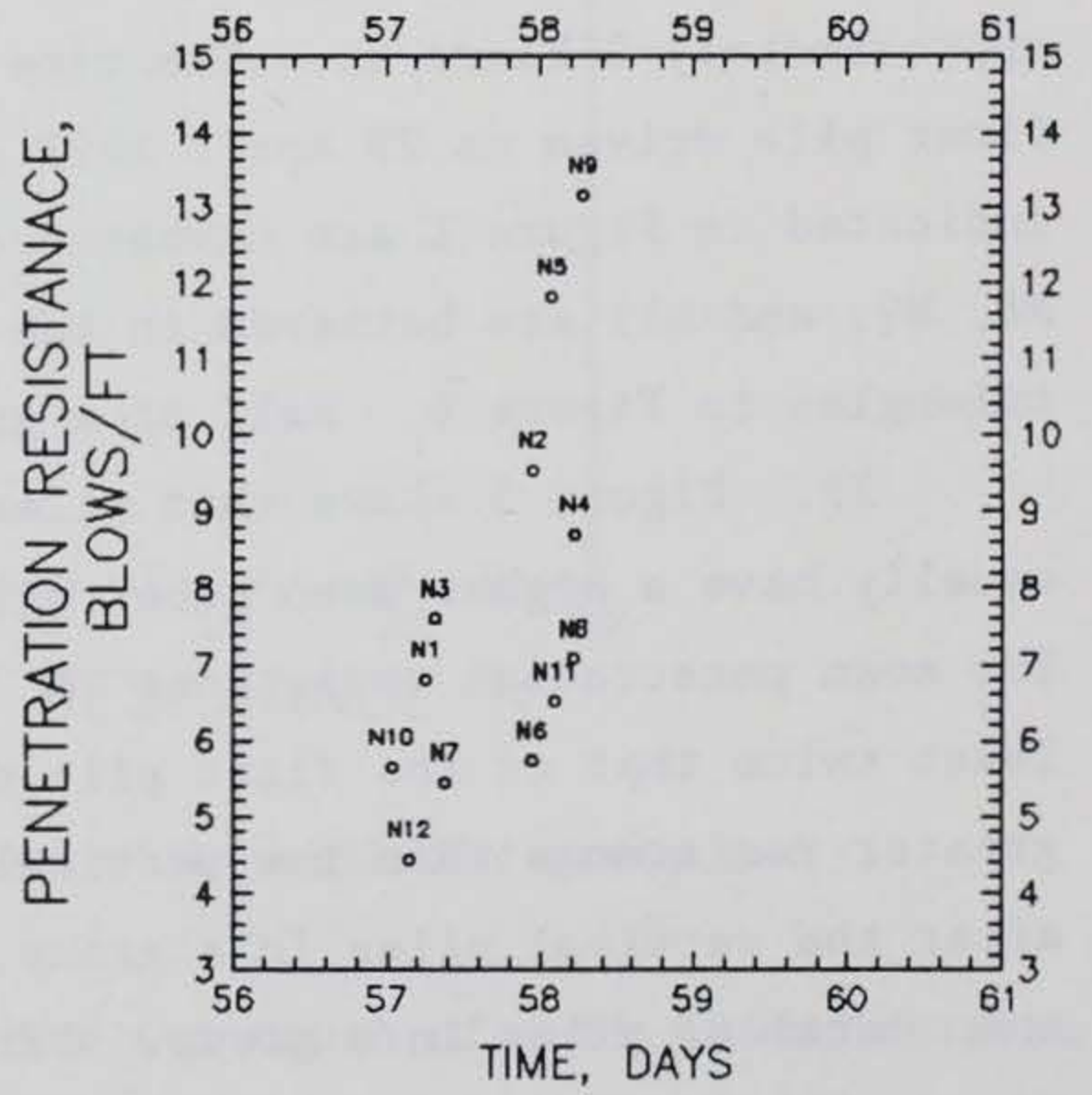
d. GROUP N-4

Figure 5. Mean penetration resistance of the north pile groups. Numbers in the figures are located as shown in Figure 6

12
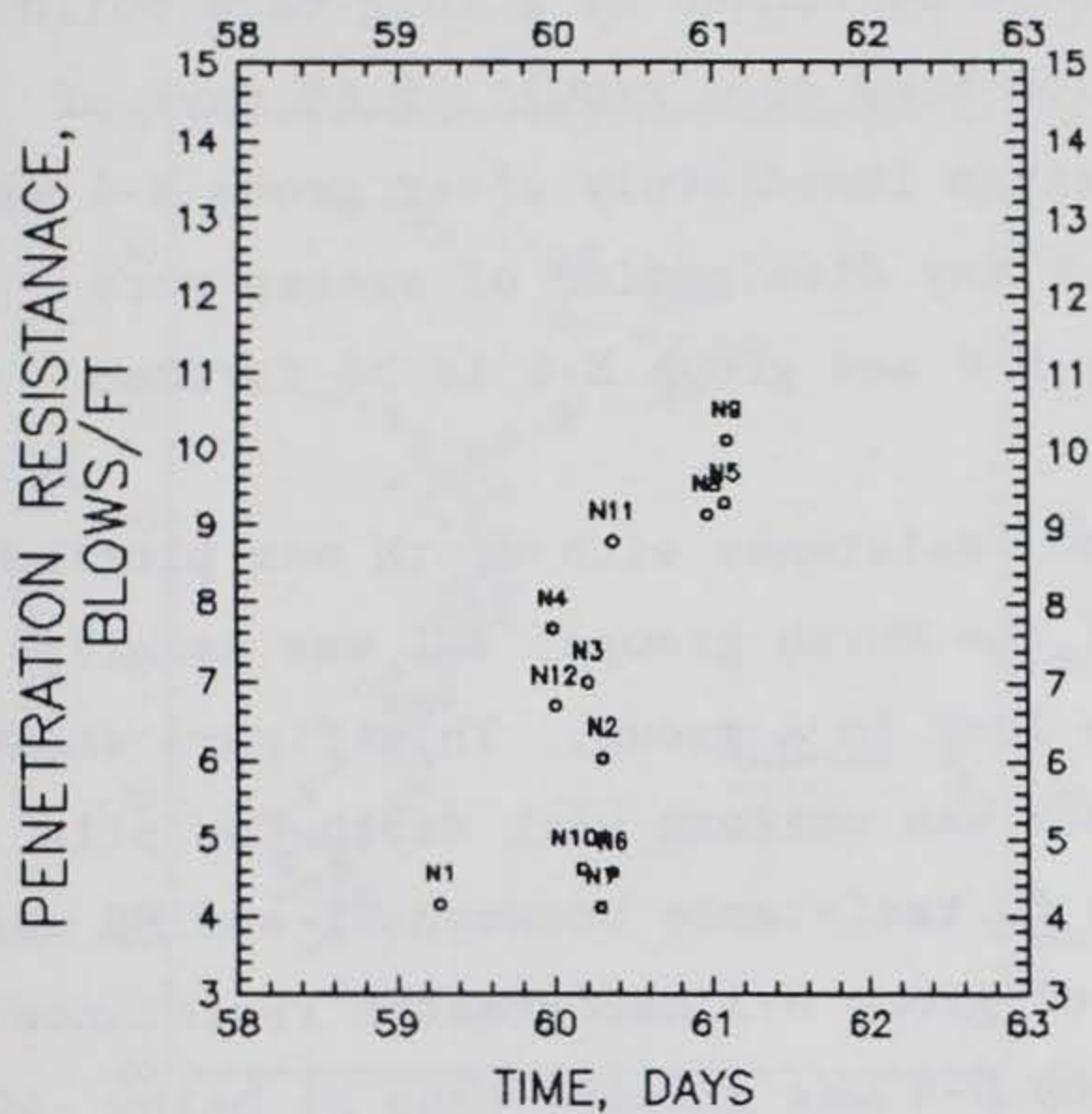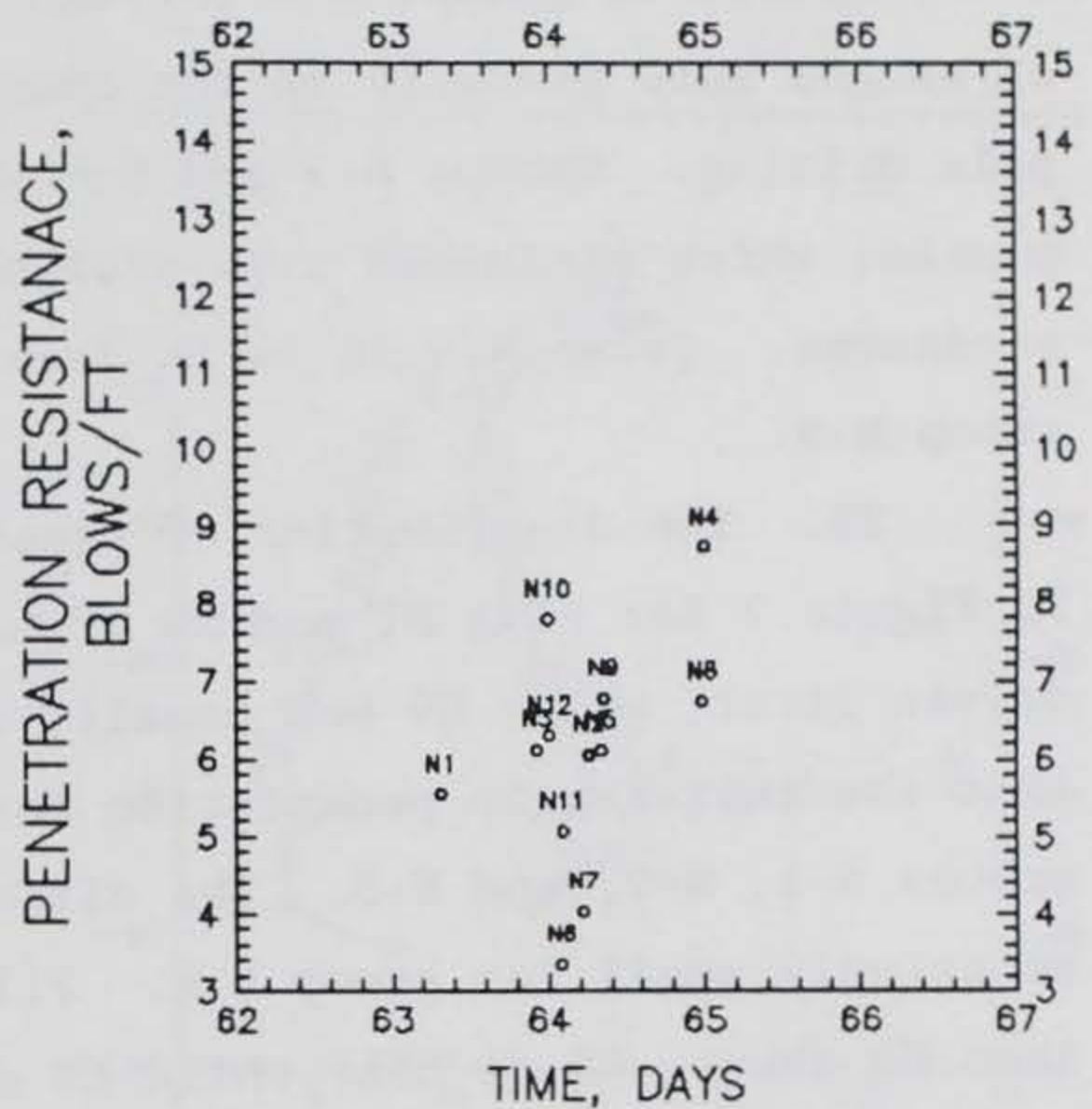
e. GROUP N-5   f. GROUP N-6

Figure 5. (Concluded)
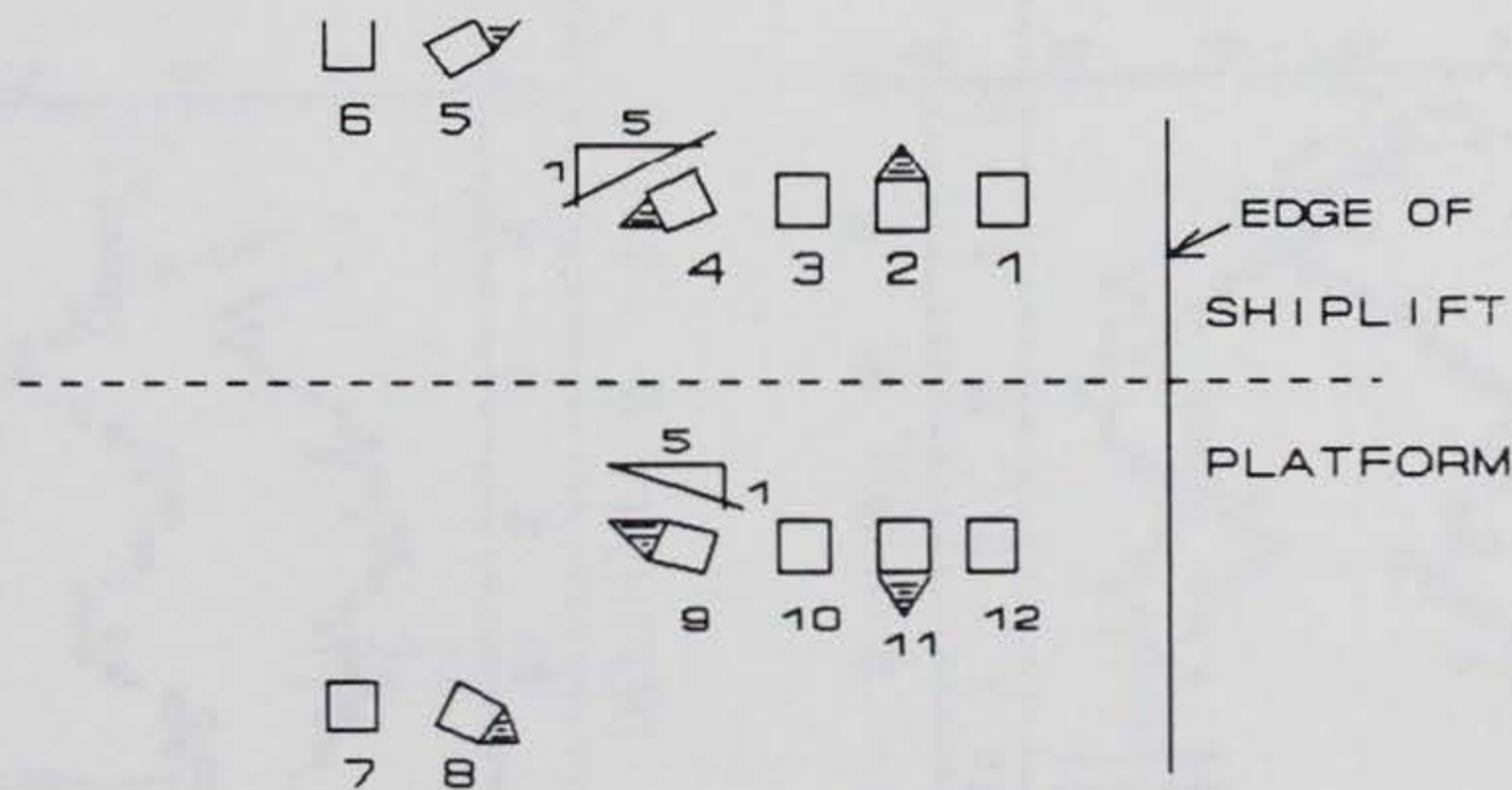
Figure 6. Numbering scheme in a North pile group. Triangles
represent direction of batter

is not readily apparent and appears to be overwhelmed by the gain in
penetration resistance as a result of pile driving.

25. Figure 5 also shows that the smallest and largest mean penetration
resistances of the piles in North groups N-5 and N-6 are less than those in
groups N-1 through N-4. These lower penetration resistances in groups N-5 and

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N-6 compared to groups N-1 through N-4 could be caused by a long-term build up of excess pore pressure in the construction area as a result of 60 days of pile driving. Groups N-5 and N-6 were driven immediately after group N-4 was placed, which minimized time available for any dissipation of excess pore pressures. Group N-5 is 36 ft from group N-4 and group N-6 is 36 ft from group N-5.

26. The distribution of penetration resistance with depth was plotted in Figure 7 for pile N1 and N9 of each of the North groups. N1 was usually driven first, while N9 was usually driven last in a group. This figure shows that the increase in penetration resistance was uniform with depth for pile groups N-1, N-2, and N-5. The difference in resistance between N1 and N9 was relatively small for group N-6. Pile N9 of group N-3 had greater resistance than N1 above -40 ft MSL, while N9 of group N-4 was greater than N1 below -40 ft MSL. The sudden increase in penetration resistance or soil freeze at -76 ft MSL for pile N1 in group N-5 occurred after a time delay of 23 hours in driving. Figure 7 again indicates that batter does not appear to have much influence on penetration resistance.
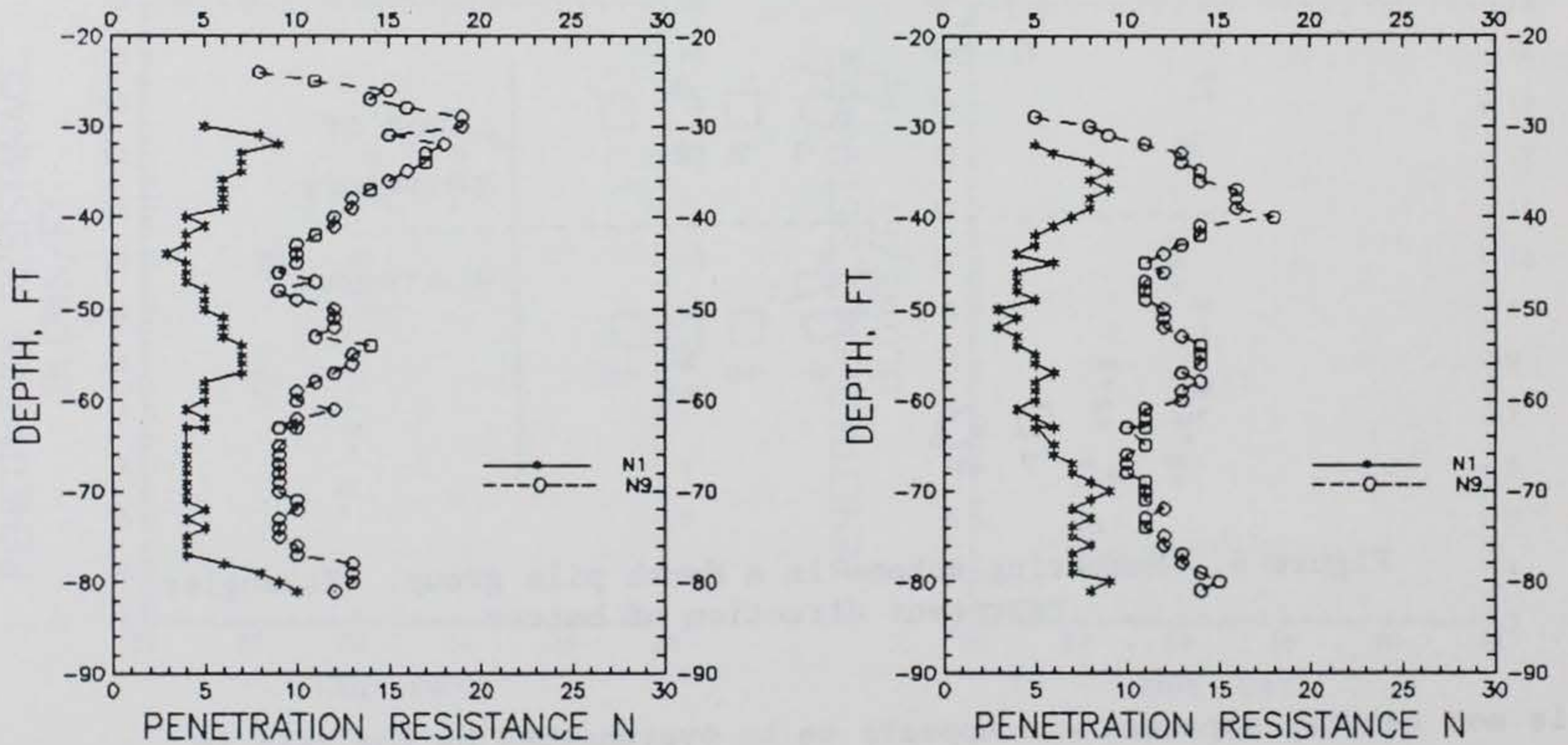
a.   GROUP N-1   b.   GROUP N-2

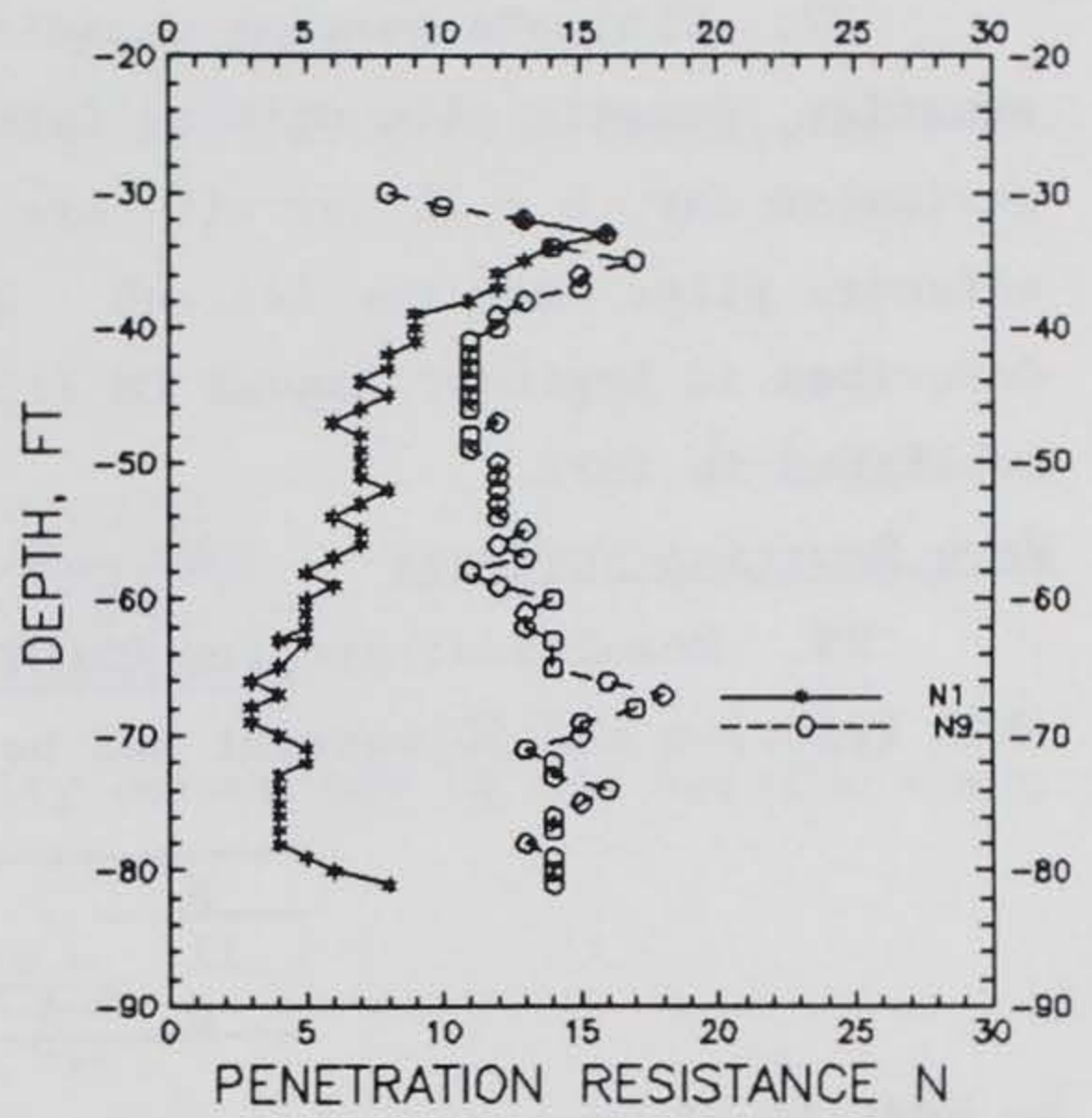Figure 7.   Distribution of penetration resistance with depth
for piles N1 and N9 in the North groups

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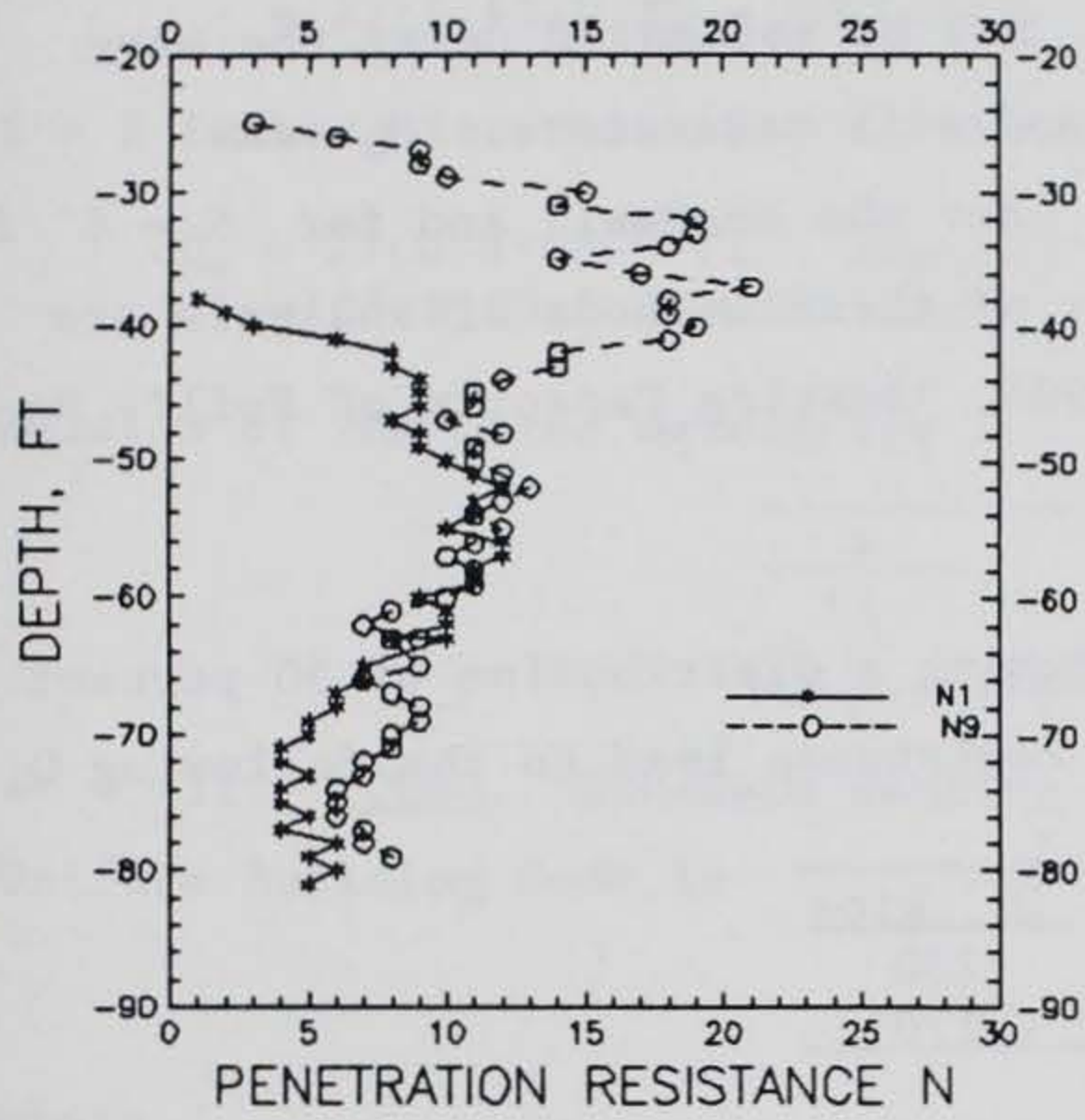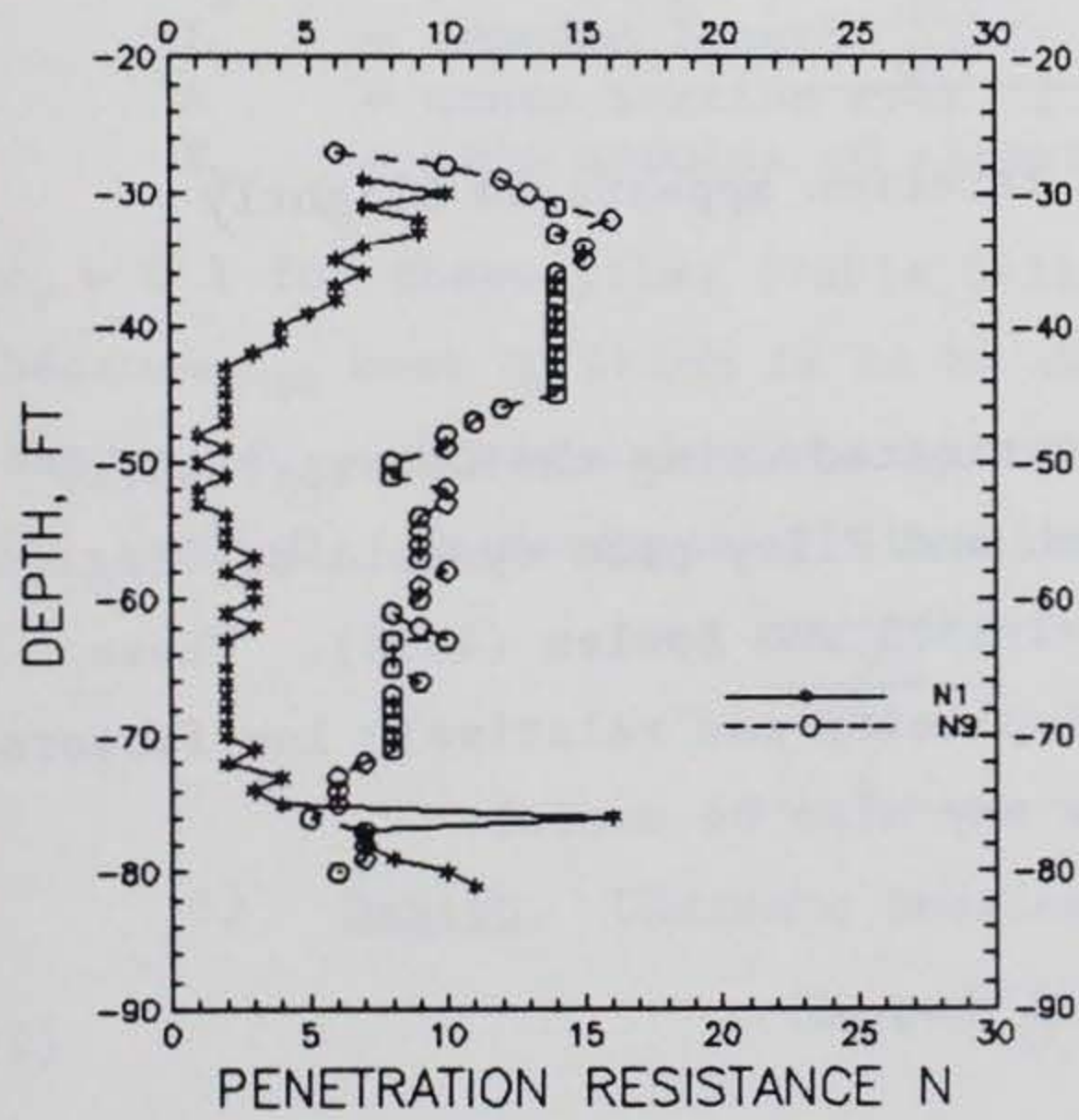
c. GROUP N-3

d. GROUP N-4

e. GROUP N-5
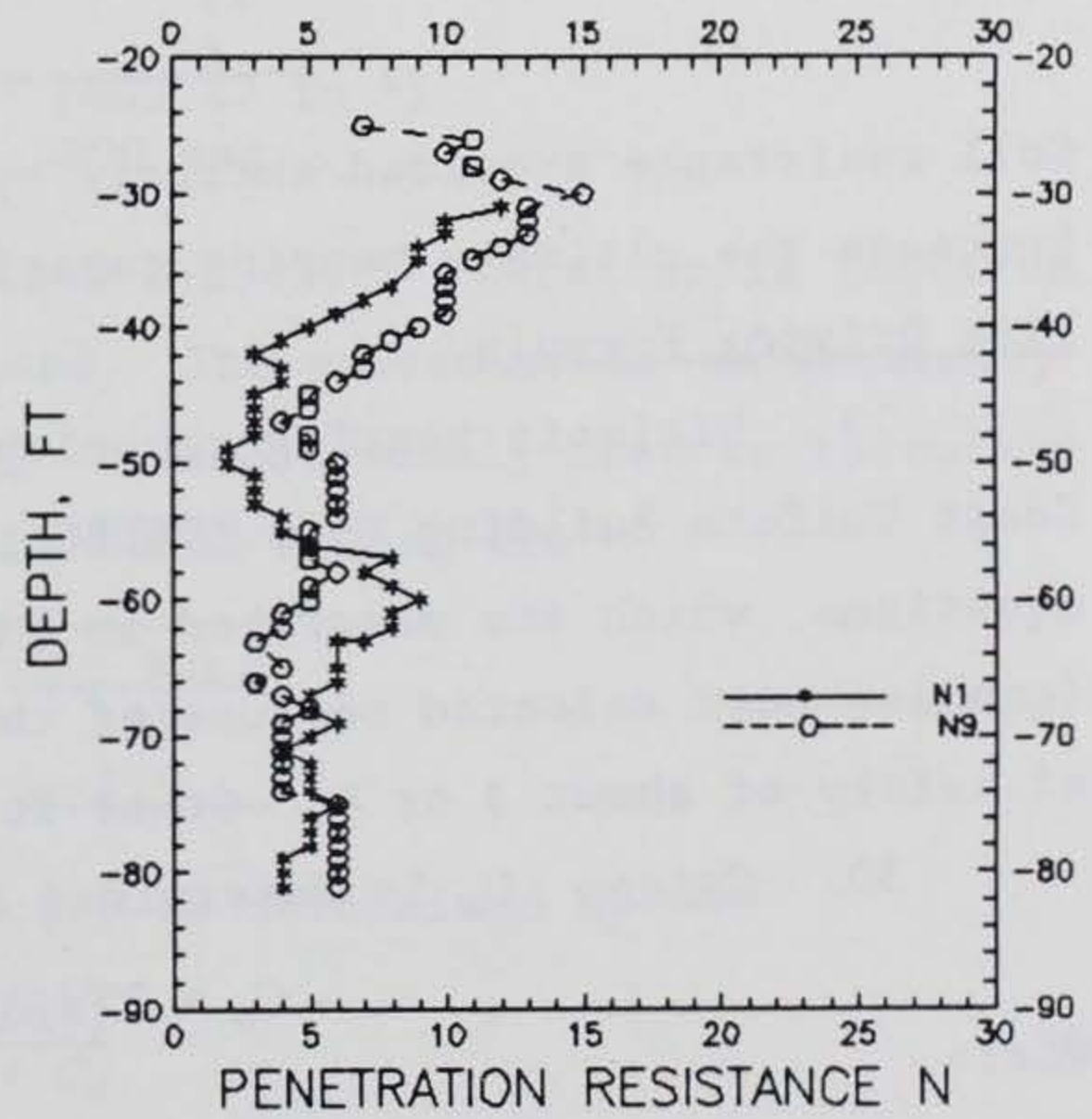
f. GROUP N-6

Figure 7. (Concluded)

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Ultimate Bearing Capacity

27. Ultimate bearing capacity Q_u may be estimated using the wave equation, dynamic pile driving formulas and soil parameters. Q_u was estimated for $N = 11$ for offshore piles near the headwall and for $N = 6$ for offshore piles near the far end. Details of these methods of analysis are described in Engineer Manual EM 1110-1-1905, "Bearing Capacity of Soil", to be published in 1992.

Wave Equation Analysis

28. Results of program GRLWEAP assuming a distribution of 50 percent skin friction and 50 percent end bearing resistance lead to the following Q_u

N	Q_u, kips
11	230
6	170

An additional wave equation analysis repeated assuming 100 percent skin resistance provided the following Q_u

N	Q_u, kips
11	250
6	170

Soil resistance provided entirely by skin friction appears to slightly increase the ultimate bearing capacity.

Pile Driving Formula

29. Ultimate bearing capacity was estimated using the Gates, Pacific Coast Uniform Building Code (PCUBC), Danish and Hiley pile dynamic driving equations, which are described in EM 1110-1-1905 and Bowles (1988). These formulas were selected because of their simplicity and relatively low factors of safety of about 3 or 4. Other formulas may also be useful.

30. _Gates_. Q_u is determined by

$$Q_u = 27 (E_h E_r)^{1/2} (1-\log_{10}S) \tag{2}$$

where

E_h = hammer efficiency, often 0.8
E_r = Manufacturer's hammer energy rating, assumed 60 kips-ft
S = average penetration in inches per blow for the last 5 to 10 blows for drop hammers and 10 to 20 blows for other hammers

$S = 1.09$ inches/blow when $N = 11$ blows/ft. The Gate's formula calculates

$$Q_u = 27(0.8 \cdot 60)^{1/2}(1 - \log_{10} 1.09)$$
$$= 27 \cdot 6.93 \cdot 0.96 = 180 \text{ kips}$$

$S = 2$ inches/blow when $N = 6$ blows/ft. The Gate's formula calculates

$$Q_u = 27(0.8 \cdot 60)^{1/2}(1 - \log_{10} 2)$$
$$= 27 \cdot 6.93 \cdot 0.7 = 131 \text{ kips}$$

Results of the Gates method are

N	Q_u, kips
11	180
6	131

31. __PCUBC__. Ultimate bearing capacity determined by the Pacific Coast Uniform Building Code is

$$Q_u = \frac{12 E_h E_r C_{p1}}{S + C_{p2}} \qquad (3)$$

where

C_{p1}	$= (W_r + c_p W_p)/(W_r + W_p)$, 0.427
C_{p2}	$= 12 Q_u L/(A E_p)$, $0.00053 Q_u$ inches
W_r	= ram weight, 20 kips
W_p	= pile weight, 35 kips
L	= embeded length, 53 ft
A	= cross-section area, 2.78 ft^2 (400.32 in.^{2})
E_p	= pile modulus of elasticity, 3000 ksi

$c_p = 0.1$ for these piles (Table 5-11, EM 1110-1-1905). Iteration is required because C_{p2} uses Q_u which is to be determined. The procedure is to initially assume $C_{p2} = 0$ and calculate Q_u. This Q_u is subsequently used to calculate C_{p2} and Q_u is recomputed. Results of the PCUBC method are

N	Q_u, kips
11	205
6	119

32. __Danish__. Ultimate bearing capacity is determined by

$$Q_u = \frac{12 E_h E_r}{S + C_d} \qquad (4a)$$

$$C_d = \left[\frac{E_h E_r L}{2 A E_p} \right]^{1/2} \qquad (4b)$$

where

L	= embeded length, 53 ft
A	= pile cross section, 2.78 ft^2
E_p	= pile modulus of elasticity, 432,000 ksf

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For this pile, C_d is

$$C_d = \left[\frac{0.8 \cdot 60 \cdot 53}{2 \cdot 2.78 \cdot 432000}\right]^{1/2} = 0.032 \; ft \; or \; 0.39 \; inch$$

Therefore, ultimate bearing capacity for $N = 11$ is

$$Q_u = \frac{12 \cdot 0.8 \cdot 60}{1.09 + 0.39} = 389 \; kips$$

and for $N = 6$ is

$$Q_u = \frac{12 \cdot 0.8 \cdot 60}{2.0 + 0.39} = 241 \; kips$$

Results of the Danish method are

N	Q_u, kips
11	389
6	241

33. <u>Hiley</u>. Ultimate bearing capacity determined by the Hiley equation is (Bowles 1988)

$$Q_u = \frac{12 E_h E_r}{S + \frac{1}{2}(k_1 + k_2 + k_3)} \cdot \frac{W_r + C_r^2 W_p}{W_r + W_p} \quad (5)$$

where

k_1 = elastic compression of capblock and pile cap, 0.2 inch
k_2 = elastic pile compression, 0.1 inch
k_3 = elastic soil compression or quake, 0.1 inch
C_r = coefficient of restitution, 0.4

Minimal k_1, k_2, and k_3 values are selected because blowcounts are low. When $N = 11$ blows/ft, $S = 1.09$ inches/blow and

$$Q_u = \frac{12 \cdot 0.8 \cdot 60}{1.09 + \frac{1}{2}(0.2 + 0.1 + 0.4)} \cdot \frac{20 + 0.4^2 \cdot 35}{20 + 35} = 400 \cdot 0.46 = 186 \; kips$$

When $N = 6$ blows/ft, $S = 2$ inches/blow and

$$Q_u = \frac{12 \cdot 0.8 \cdot 60}{2 + \frac{1}{2}(0.2 + 0.1 + 0.4)} \cdot 0.46 = 110 \; kips$$

Results of the Hiley method are

N	Q_u, kips
11	186
6	110

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Soil Parameter ϕ

34. Ultimate bearing capacity can be estimated using an assumption for the friction angle of the load bearing soil. At the head wall ϕ is estimated at 35 deg ($N \approx 11$) and at the far end 30 deg ($N \approx 6$) based on gradation analysis of the boring samples of the coral sands. $\phi = 35$ deg is reasonable for a medium dense sand and $\phi = 30$ deg for a loose sand using guidance in EM 1110-1-1905. $\phi = 35$ deg was suggested for soils supporting the onshore piles in the November 1987 soils investigation report. Q_u is the sum of point bearing Q_{bu} and skin friction Q_{su} capacities

$$Q_u = Q_{bu} + Q_{su}$$
$$= q_{bu}A_b + C_sLf_s \qquad (6)$$

where

q_{bu} = end bearing resistance, ksf
A_b = point bearing area, 2.78 ft^2
C_s = circumference, 6.67 ft
L = embeded length, 53 ft
f_s = maximum mobilized skin friction, ksf

Meyerhof and Nordlund methods described in EM 1110-1-1905 were used to estimate point bearing and skin friction resistances.

35. **Point Bearing Resistance.** The Meyerhof method determines $q_{bu} = \sigma'_L N_{qp} \zeta_{qp} \leq q_\ell = N_{qp} \tan\phi$ where σ'_L is the effective overburden pressure at the pile toe, N_{qp} is the bearing capacity surcharge factor, and ζ_{qp} is the geometry correction factor. The effective oveburden pressure σ'_L is limited to the overburden pressure at critical depth $L_c = 10B = 10 \cdot 20 = 200$ inches or 16.7 ft. Therefore, $\sigma'_L = (\gamma_{sat} - \gamma_w) \cdot L_c = (115 - 62.5) \cdot 16.7 = 877$ psf or 0.877 ksf. $N_{qp} = 150$ for $\phi = 35$ deg and $N_{qp} = 60$ for $\phi = 30$ deg from Figure 5-15 in EM 1110-1-1905. $\zeta_{qp} = $ unity. Therefore, $q_{bu} = 0.877 \cdot 150 \cdot 1.00 = 131$ ksf at the headwall and $0.877 \cdot 60 \cdot 1.00 = 61$ ksf at the far end. These values exceed q_ℓ; therefore, bearing capacity by the Meyerhof method is

Headwall: $N_{qp} = 150$
($N = 11$) $q_{bu} = 150 \cdot \tan 35 = 105$ ksf
    $Q_{bu} = q_{bu}A_b = 105 \cdot 2.78 = 292$ kips

Far end: $N_{qp} = 60$
($N = 6$) $q_{bu} = 60 \cdot \tan 30 = 34.6$ ksf
    $Q_{bu} = q_{bu}A_b = 34.6 \cdot 2.78 = 96$ kips

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36. q_{bu} calculated by the Nordlund method given in Table 5-8 of EM
1110-1-1905 is $\alpha_f N_{qp} \sigma'_L$ where α_f is the depth-width relationship factor.
Bearing capacity by the Nordlund method is

Headwall: ϕ = 35 deg
(N = 11) α_f = 0.65 from Figure 5-17a in EM 1110-1-1905
           N_{qp} = 70 at the headwall from Figure 5-17b in EM 1110-1-1905
           q_{bu} = 0.65·70·0.877 = 39.9 ksf
           Q_{bu} = 39.9·2.78 = 111 kips

Far end: ϕ = 30 deg
(N = 6) α_f = 0.5
           N_{qp} = 30
           q_{bu} = 0.5·30·0.877 = 13.2 ksf
           Q_{bu} = 36.6 kips

37. <u>Skin Friction Resistance</u>. Skin resistance by the Meyerhof method
is f_s = $\beta_f \sigma'_i$ where β_f is the lateral earth pressure and friction angle
factor, and σ'_i is the average effective overburden pressure along the
embeded length L. σ'_i is approximately σ'_L = 0.877 ksf. The skin
resistance capacity Q_{su} by the Meyerhof method is

Headwall: ϕ = 35 deg
(N = 11) β_f = 0.8 from Figure 5-5 in EM 1110-1-1905
           f_s = 0.8·0.877 = 0.7 ksf
           Q_{su} = $C_s L f_s$ = 6.67·53·0.7 = 247 kips

Far end: ϕ = 30 deg
(N = 6) β_f = 0.2
           f_s = 0.2·0.877 = 0.175 ksf
           Q_{su} = 6.67·53·0.175 = 62 kips

These f_s bracket the 0.4 ksf recommended from the Barbados study for coral
sand (Stevenson and Thompson 1978).

38. The Nordlund method estimates skin resistance Q_{su} = $KC_f \sigma_i' C_s L \sin\delta$
where K is the coefficient of lateral earth pressure, C_f is the correction
factor, and δ is the soil-shaft friction angle. Volume of the soil
displaced per unit length V required to estimate K and δ is V = A_b·1 =
2.78 ft³. Q_{su} is

Headwall: ϕ = 35 deg
(N = 11)   K = 2 from Figure 5-18 in EM 1110-1-1905
           C_f = 1 from Figure 5-20 in EM 1110-1-1905
           δ = 31.5 deg from Figure 5-19 in EM 1110-1-1905
           sin 31.5 = 0.522
           Q_{su} = 2·1·0.877·6.67·53·0.522 = 324 kips

```
Far end:     φ  =  30 deg
(N = 6)      K  =  1.4
             C_f =  1
             δ  =  27 deg
             sin 27 = 0.454
             Q_su = 1.4·1·0.877·6.67·53·0.454 = 197 kips
```

39. <u>Ultimate Capacity</u>. Ultimate capacity is the sum of Q_{bu} and Q_{su} and for the Meyerhof and Nordlund methods is equal to

Method	Q_u, kips	
	N = 11	N = 6
Meyerhof	539	293
Nordlund	435	233

Comparison of Methods

40. A summary of all of the ultimate bearing capacity estimates is given in Table 3.

Table 3

<u>Summary of Ultimate Bearing Capacities</u>

Method	Q_u, kips	
	N = 11	N = 6
Wave Equation	230	140
Pile Driving Formulas		
Gates	180	131
PCUBC	205	119
Danish	389	241
Hiley	186	110
Soil Parameter		
Meyerhof	539	293
Nordlund	435	233
Average	309	181

The average ultimate capacity for N = 11 is 309 kips. The estimated allowable bearing capacity Q_a is about 155 kips using a factor of safety = 2. Q_a appears to be slightly inadequate for a design load Q_d = 160 kips for onshore piles with embeded length L = 53 ft and if the penetration resistance is 11 blows/ft. The allowable bearing capacity for offshore pile near the headwall is also 155 kips and exceeds the design load Q_d = 120 kips. Offshore piles near the headwall are expected to perform adequately.

41. The average ultimate capacity for $N = 6$ blows/ft for the offshore piles near the far end is 181 kips. The allowable bearing capacity Q_a using a factor of safety of 2 is 91 kips, which is not sufficient for the design load $Q_d = 120$ kips.

42. Figure 8 illustrates the range of results of the ultimate bearing capacity versus penetration resistance calculations. The two data points in this figure are the average calculated Q_u for the headwall and the far end. This figure indicates that N should be ≥ 12 to be confident that the required $Q_u = 240$ kips, twice the design load, will be obtained for all of the offshore piles supporting the dry dock. $N \geq 12$ blows/ft supports the previously established design requirement that the penetration resistance of the piles > 12 blows/ft.
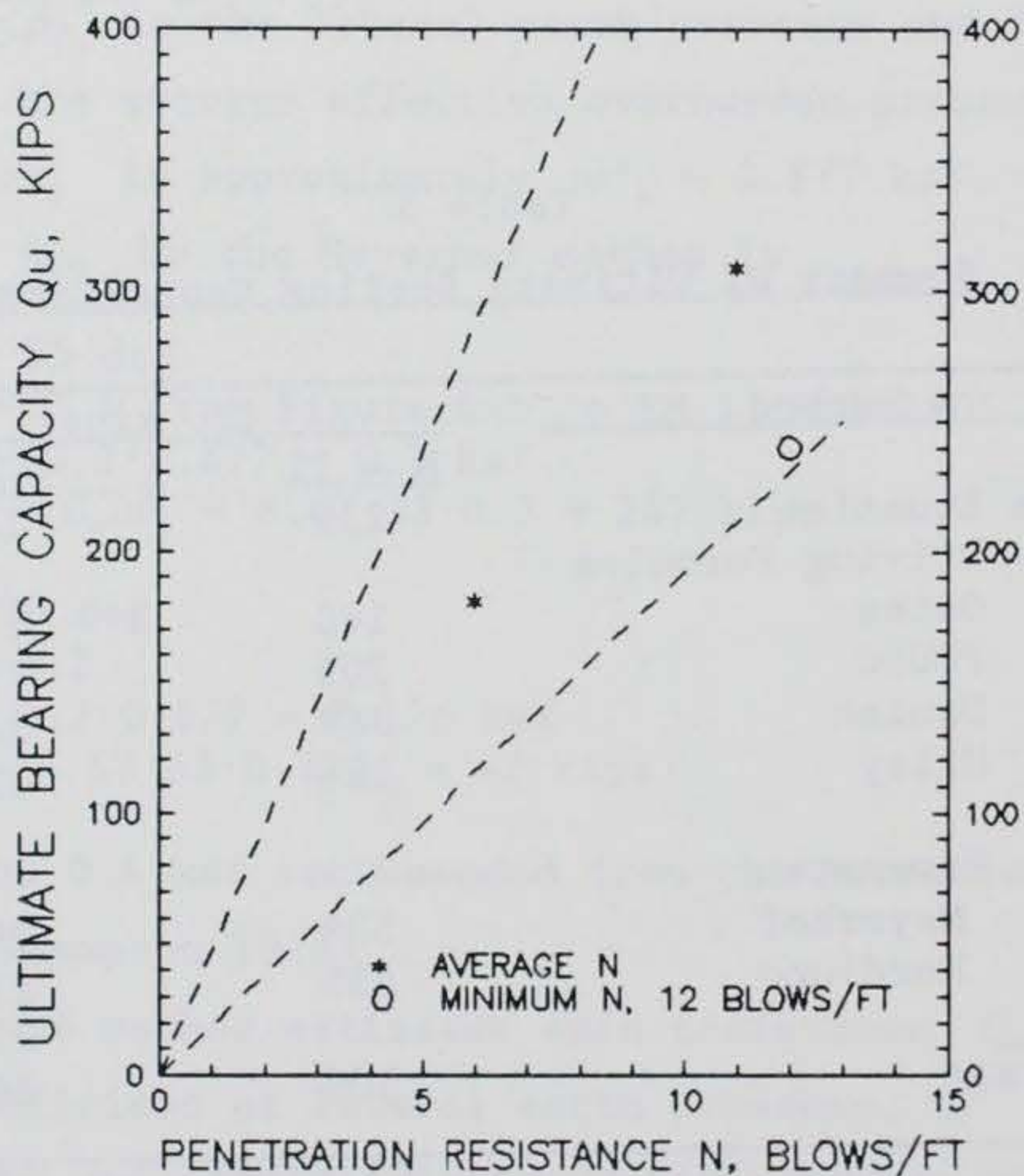

Figure 8. Range of expected ultimate bearing capacities Q_u for various penetration resistance N

Comparison With Load Tests

43. Estimates of Q_u given in Table 3 for offshore piles near the headwall ($N = 11$ blows/ft) are consistent with results of the proof test

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conducted on offshore pile E2. Pile E2 has an embedment length of 53 ft, similar to the production piles. Q_u of this test is greater than 240 kips, Figure 9. The Davisson (1972) failure line is

$$\rho_u = 0.15 + \frac{B}{120} + \frac{L_t}{AE_p} Q_u \tag{7}$$

where

Q_u = failure load at intersection of load test data and Davisson failure line, kips
ρ_u = elastic settlement at failure load, in.
L_t = total pile length, 85 ft
B = pile width, 20 in.
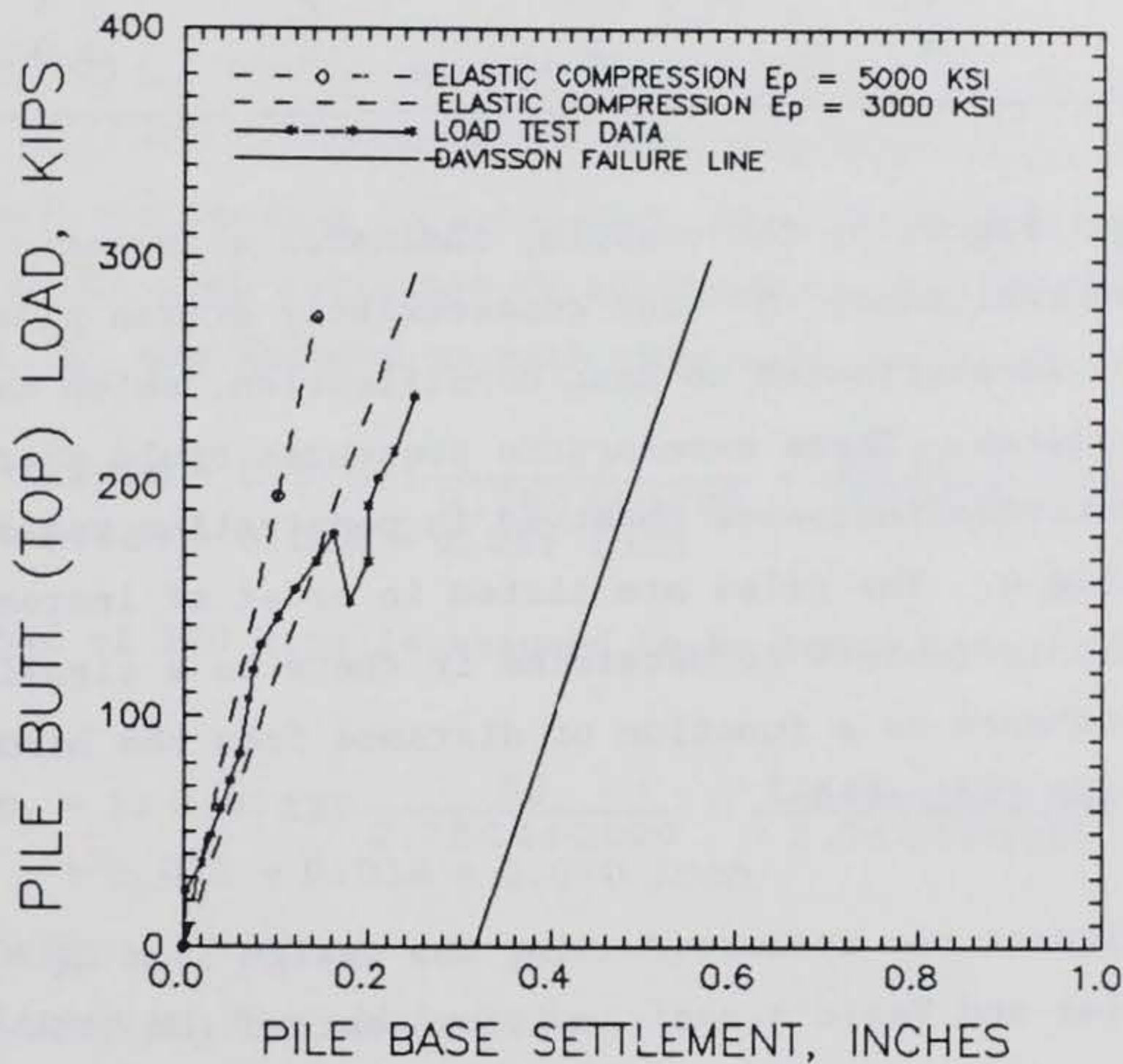
Figure 9. Results of proof test on pile E2 near the headwall

44. Piles E2, E3, and C2 were tested with static loads 1 to 5 days after installation. This delay may have permitted some soil freeze such as from dissipation of excess pore pressure. Table 4 shows that restrikes of the production piles 0.5 to 1 day after installation indicated up to 3 times increase in penetration resistance. Such an increase in penetration resistance from dissipation of excess pore pressures is a logical consequence

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Table 4

Penetration Resistance of Production Piles 0.5 to 1.0 Day After Driving

Pile Number	Installation Resistance, Blows/FT	Restrike Resistance, Blows/Ft	Increase Blows/Ft	Times Increase Factor	Elapsed Time, Days
E2	4	12	8	3.0	1.0
N-2-1	8	8	0	0.0	1.0
S-2-10	8	9	1	1.1	0.5
S-3-6	9	9	0	0.0	0.5
N-3-7	5	6	1	1.2	0.5
N-4-12	3	9	6	3.0	0.5
S-4-6	8	10	2	1.3	0.5
N-5-1	3	10	7	2.3	0.5
N-6-12	8	10	2	1.3	0.5
S-6-1	4	12	8	3.0	0.5

of pile driving. Figure 5, for example, indicates a dramatic increase in the mean penetration resistance N' for consecutively driven piles. This increase in N' is attributed to sand densification, which could lead to excess pore pressures. These excess pore pressures could dissipate rapidly enough to indicate the increases observed in penetration resistance of the restrikes in Table 4. The piles are listed in order of increasing distance from the headwall in Table 4 to determine if there is a significant difference in restrike resistance as a function of distance from the headwall, but such a difference was not indicated.

Settlement

45. Settlement was estimated using the design load Q_d = 120 kips for the offshore piles and Vesic's semi-empirical method (EM 1110-2-2906)

$$\rho = \rho_p + \rho_b + \rho_s \qquad (8)$$

where

ρ = total settlement at the pile top, ft
ρ_p = settlement from axial deformation of the shaft, ft
ρ_s = settlement at tip from load transmitted along the pile shaft, ft
ρ_b = settlement at tip from load transferred at the tip, ft

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Axial Compression

46. Axial compression is

$$\rho_p = 12\,(Q_b + \alpha_s Q_s)\frac{L}{AE_p} + \frac{12\,(L_t - L)\,Q}{AE_p} \tag{9a}$$

where

Q_b	– design load at the pile tip, kips
α_s	– distribution factor for load along pile length, 0.5 to 0.67; normally assume 0.5
Q_s	– design load taken by skin friction, kips
L	= embeded pile length, 53 ft
L_t	– total pile length, 85 ft
A	= cross-section area of pile, 2.78 ft²
E_p	– pile modulus of elasticity, 3000 ksi

A rough estimate of Q_s may be made by assuming that Q_s is the ultimate skin resistance Q_{su}, because nearly all skin resistance will be mobilized before significant end bearing is mobilized, unless the pile is bearing on a hard stratum. Q_b is then estimated by subtracting Q_{su} from the design load Q_d. If Q_b and Q_s are assumed to each take half the load or 60 kips, then

$$\begin{aligned}
\rho_p &= 12\,(60 + 0.5 \cdot 60)\frac{53}{2.78 \cdot 144 \cdot 3000} + \frac{12\,(85 - 53) \cdot 120}{2.78 \cdot 144 \cdot 3000} \\
&= 0.048 + 0.038 = 0.086 \; inch
\end{aligned}$$

If the design load of 120 kips is assumed to be taken totally by skin friction, then

$$\begin{aligned}
\rho_p &= 12 \cdot 0.5 \cdot 120\frac{53}{2.78 \cdot 144 \cdot 3000} + \frac{12\,(85 - 53) \cdot 120}{2.78 \cdot 144 \cdot 3000} \\
&= 0.032 + 0.038 = 0.070 \; inch
\end{aligned}$$

Pile Tip Settlement

47. Settlement at the pile tip is

$$\rho_b = \frac{12\,C_b Q_b}{B q_{bu}} \tag{9b}$$

$$\rho_s = \frac{12\,C_s Q_s}{L q_{bu}} \tag{9c}$$

where

C_b	= empirical coefficient, Table 5
C_s	= coefficient, $[(0.93 + 0.16(L/B_s)^{0.5}]C_b$
Q_b	= load supported by end bearing, kips
Q_s	– load supported by skin friction, kips
q_{bu}	= end bearing resistance, ksf

Table 5

C_b for Long-term Settlement (EM 1110-2-2906)

Soil	Driven Piles	Drilled Shafts
Sand (dense to loose)	0.02 to 0.04	0.09 to 0.18
Clay (stiff to soft)	0.02 to 0.03	0.03 to 0.06
Silt (dense to loose)	0.03 to 0.05	0.09 to 0.12

The bearing stratum is assumed to extend a minimum $10B_b$ beneath the pile tip and the stiffness in this stratum is equal to or higher than the stiffness at the tip elevation. Consolidation settlement should not be significant.

48. <u>Pile settlement near the headwall</u>. Sands are considered dense near the headwall and $C_b = 0.02$ from Table 5. $C_s = [0.93 + 0.16(53/1.67)^{0.5}] \cdot 0.02 = 1.83 \cdot 0.02 = 0.037$. q_{bu} is assumed 40 ksf based on the Nordlund analysis. For skin friction and end bearing resistance evenly divided at 50 percent,

$$\rho_b = \frac{12 \cdot 0.02 \cdot 60}{1.67 \cdot 40} = 0.216 \ inch$$

$$\rho_s = \frac{12 \cdot 0.037 \cdot 60}{53 \cdot 40} = 0.013 \ inch$$

from equations 9. Total settlement $\rho = 0.089 + 0.216 + 0.015 = 0.32$ inch. If the design load $Q_d = 120$ kips is assumed to be taken totally by skin friction so that $Q_b = 0$, then from equation 9c

$$\rho_s = \frac{12 \cdot 0.037 \cdot 120}{53 \cdot 40} = 0.025 \ inch$$

Therefore, $\rho = 0.070 + 0.025 = 0.095$ inch. If N is ≥ 12, which appears to be the penetration resistance required for adequate bearing capacity, then settlement is expected to be about 0.1 inch, but not greater than 0.3 inch.

49. <u>Settlement near the far end</u>. The sand is considered loose at the far end so that $C_b = 0.04$ from Table 5 and $C_s = 1.83 \cdot 0.04 = 0.073$. q_{bu} is 14 ksf based on the Nordlund analysis. For skin friction and end bearing resistance evenly divided at 50 percent,

$$\rho_b = \frac{12 \cdot 0.04 \cdot 60}{1.67 \cdot 14} = 1.23 \ inch$$

$$\rho_s = \frac{12 \cdot 0.073 \cdot 60}{53 \cdot 14} = 0.071 \ inch$$

and total settlement $\rho = 0.086 + 1.23 + 0.071 = 1.38$ inches. This settlement

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is excessive. If the design load is taken entirely by skin friction, then

$$\rho_s = \frac{12 \cdot 0.073 \cdot 120}{53 \cdot 14} = 0.14 \; inch$$

and ρ = 0.07 + 0.14 = 0.21 inch. The design load Q_d = 120 kips applied to a production pile near the far end is expected to cause at least 0.2 inch of settlement, but less than 1.4 inches. This range does not consider soil freeze from dissipation of excess pore pressures.

Comparison With Load Tests

50. Settlement of the proof test conducted on pile E2 near the headwall, Figure 8, shows about 0.07 inch at the design load of 120 kips for offshore piles. This settlement is consistent with the calculated settlement of 0.095 inch calculated for offshore piles near the headwall.

51. The load-displacement behavior of pile E2 is bounded by the elastic compression settlement using a pile elastic modulus of 3000 and 5000 ksi. The slope of the elastic displacement is calculated by

E_p = 3000 KSI:
$$\frac{AE_p}{L_t} = \frac{(20)^2 \cdot 3000}{85 \cdot 12} = 1176.47 \; kips/inch \qquad (10a)$$

E_p = 5000 KSI:
$$\frac{AE_p}{L_t} = \frac{(20)^2 \cdot 5000}{85 \cdot 12} = 1960.78 \; kips/inch \qquad (10b)$$

where

A = pile cross-section area, $(20)^2$ = 400 in.^{2}
E_p = pile modulus of elasticity, ksi
L_t = total pile length, 85 ft

These data show that all settlement for loads up to about 170 kips will be elastic and no permanent settlement is expected for the design load of 120 kips. Settlement at failure ρ_u = 0.00085Q_u + 0.317 inch if E_p = 3000 ksi. Settlement ρ_u is expected to be about 0.6 inch for the average Q_u = 309 kips from Table 3 for N = 11 blows/ft. Skin friction is significant and appears to provide much of the bearing capacity for piles near the headwall.

Discussion of Results

52. Analysis of data taken during the soil investigation indicates that to achieve adequate bearing capacity the selected precast prestressed 20" by

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20" by 85' long piles should have a penetration resistance $N \geq 12$ when driven offshore to a toe depth of -81 ft MSL. Penetration resistances of borings taken in the lagoon near the far end of the dry dock were ≥ 15 blows/ft at the toe depth -81 ft MSL and often greater than penetration resistances of borings taken onshore, Figure 2. Penetration resistance data taken from the boring records and past experience indicate that the proposed pile foundation should have adequate bearing capacity.

53. Driving records of the production piles and results of the onshore load tests indicate that bearing capacity should be adequate for offshore piles near the headwall, but may not be adequate for piles near the far end of the dry dock. The pile toe penetration resistance decreased from 6 to 37 blows/ft near the headwall down to 2 to 12 blows/ft near the far end. Piles near the far end were driven later than those driven near the headwall.

54. Driving records show that the mean penetration resistances N' of each pile in a group tend to increase significantly when driven later in the group, often doubling or tripling in value compared to N' for piles driven earlier in a group. Pile batter does not appear to influence the penetration resistance. The distribution of the increase in penetration resistance of pile N9 compared to pile N1 was uniform for some North groups, but not non-uniform for others. The increases in penetration resistance observed for piles driven later in a group are attributed to densification of the coral sands from pile driving.

55. The mean penetration resistances N' of piles driven in Groups N-5 and N-6 were generally less than those observed for piles driven in groups N-1 through N-4. Groups N-5 and N-6 were driven immediately following group N-4, while 20 days elapsed before group N-4 was driven. Some of the South group piles were driven before group N-4 was driven. The South group is at least 74 ft from any of the North group piles, while the North group N-5 is 36 ft from group N-4 and group N-6 is about 36 ft from group N-5.

56. An explanation for the decrease in penetration resistances observed for piles driven near the far end in groups N-5 and N-6 is excess pore pressures generated during pile driving. Generation of excess pore pressure is a logical consequence of soil densification from pile driving in sands with some silts and clays. The production piles show up to a factor of 3 times increase in penetration resistance when restruck 0.5 to 1 day after

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installation, which would allow time for dissipation of some excess pore pressure. Test pile E2 was driven before the production piles were driven, yet restrike of E2 only one day after installation indicated a 3 fold increase in penetration resistance. If strength gain from dissipation of excess pore pressures generated by a single pile could cause up to a 3 fold increase in penetration resistance after one day, then the possible long-term strength gain from dissipation of excess pore pressures generated by the driving of numerous closely spaced production piles could be much greater. Conversely, the loss in penetration resistance observed in the production piles driven near the far end, which were also driven later than those near the head wall, could be caused by large increases in excess pore pressure.

57. Settlement analysis using Vesic's method indicates that skin resistance is significant in providing bearing capacity to the production piles. Settlement was estimated to be 0.1 inch if the piles were loaded to the design load $Q_d = 120$ kips for the offshore piles. Settlement would be greater if skin resistance was not significant. Results of load tests indicated that settlement should be about 0.1 inch at the design load.

58. Results of data collected before the supplemental test program were not sufficient to determine if the offshore piles have adequate bearing capacity to support the dry dock. Field tests are required on the offshore piles near the far end after a time delay to determine the long-term capability of the pile foundation to support the dry dock and to determine a better understanding of the mechanisms for any long-term strength gain and bearing capacity. Conclusions concerning the influence of cementation on bearing capacity could not be made from analyses of the available data.

PART III. SUPPLEMENTAL TESTING PROGRAM

Background

59. A supplemental pile capacity test program was conducted on offshore piles from 24 February to 5 March 1992, about 7 months after installation of the production piles. One repeated static load test was conducted on pile number 12 of group S-5 near the far end of the foundation, Figure 10. Results of the static load test were used to calibrate wave equation analysis for determination of the bearing capacity of restruck piles. The static load test
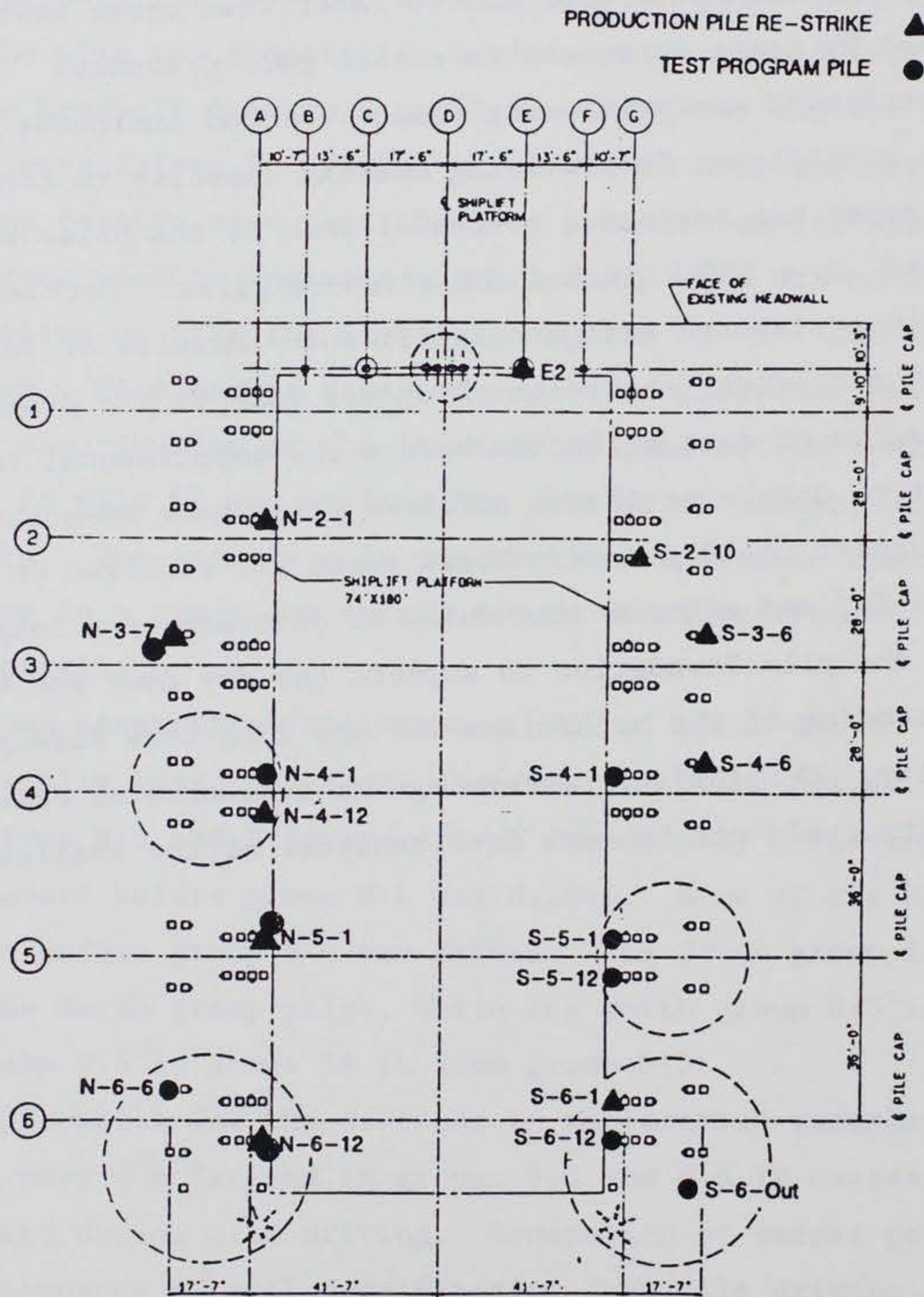

FIGURE 2. TEST PILE LOCATIONS

Figure 10. Test pile locations

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was repeated with greater maximum loads to check for possible breakdown of adhesion between coral particles and the production piles that could reduce bearing capacity. An indicator pile was driven between groups S-4 and S-5 after the repeated static load test to determine the penetration resistance and bearing capacity using the pile driving analyzer and wave equation analysis. Ten restrikes of the production piles were also completed, which included the pile that was load tested. The black circles in Figure 10 indicate test program restruck piles, while the black triangles indicate the production piles that were restruck 0.5 to 1 day after driving. The results of the static load test were combined with field measured data from the pile driving analyzer to predict the static bearing capacity of the indicator pile and restruck piles by the Case method of wave equation analysis (Goble, Likins, and Rausche 1975). The static bearing capacity and resistance distribution of some of the driven piles were also evaluated with the assistance of the CAPWAP wave equation computer program. Full details of the supplemental testing program are provided in "Design Analysis Report for Investigation of Load Capacity of Existing Piles at Drydock Facility, U.S. Army Kwajalein Atoll, Marshall Islands" prepared by Frederic R. Harris, Inc., in June 1992. Appendix B of this report contains the results of the bearing capacity analysis determined by the pile driving analyzer and wave equation.

Repeated Static Load Test

Load Test Procedure

60. The repeated static load test was performed on pile S-5-12 in accordance with ASTM D1143, "Standard Test Method for Piles Under Static Axial Compressive Load". The pile was embeded to a depth of 53 ft and it was tested 261 days after installation. The first cycle loaded the pile to the design load of 120 kips, then reduced the load to zero. The second cycle was conducted to twice the design load, then returned to zero load. The third and final cycle was conducted to three times the design load. The standard loading procedure was used for the first and second load cycles. Loading in excess of the standard load option was used to complete the third cycle.

61. The standard load option requires that each load increment will be 25 percent of the design load or 30 kips for this test and that each increment will be held until the rate of settlement was less than 0.01 in./hr or until 2

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hours had elapsed. Loading in excess of the standard load option requires reloading in increments of 50 percent of the design load allowing 20 minutes between load increments until the previous maximum load is obtained, then the load is increased in 10 percent increments of the design load until failure allowing 20 minutes between load increments.

Results

62. Results of the first cycle conducted to the design load, Figure 11, show total elastic deformation after return to zero load. The second cycle
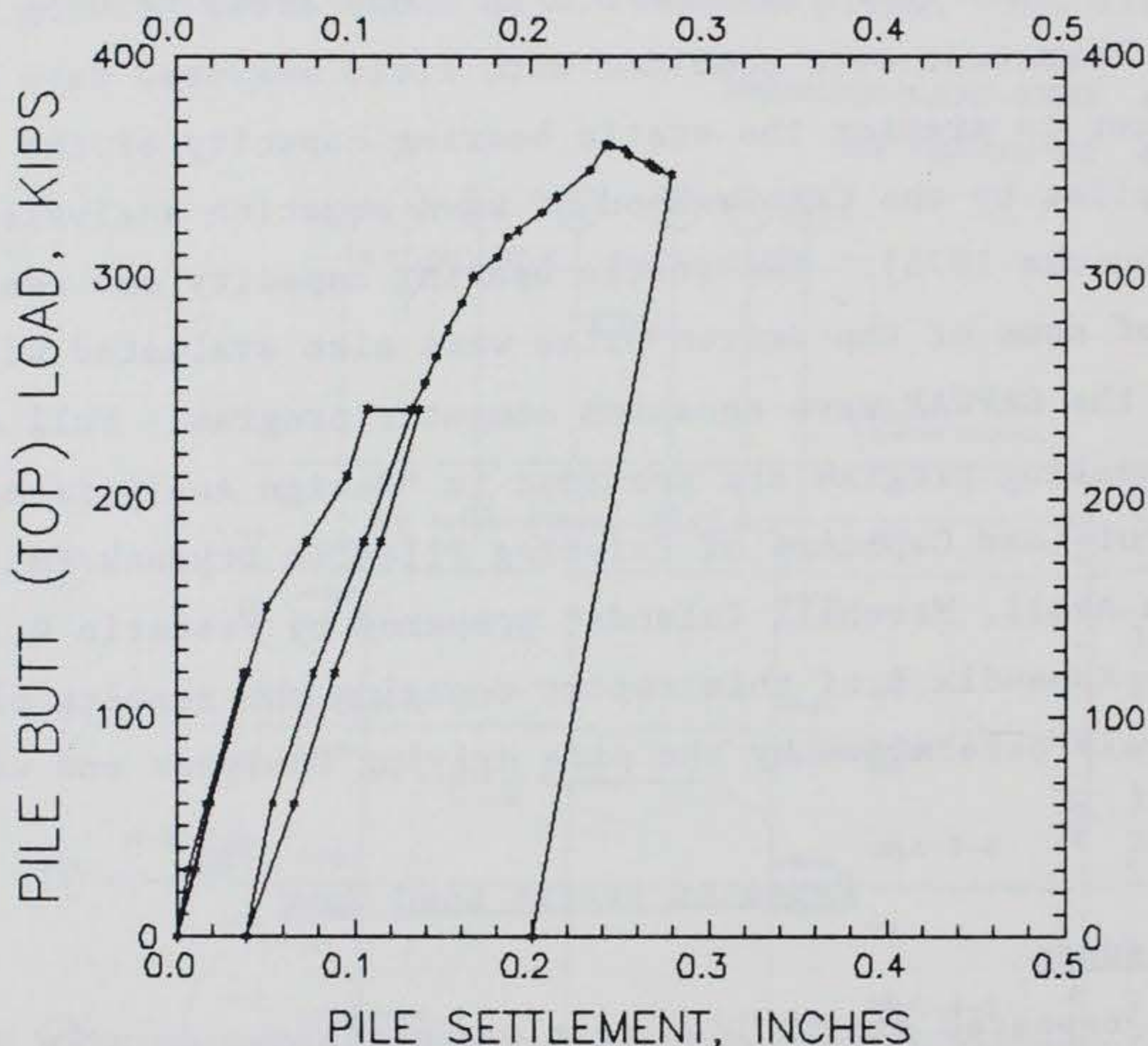

Figure 11. Results of repeated static load test on pile S-5-12

to twice the design load of 240 kips caused approximately 0.03 inch of permanent settlement. Creep recorded during the 12 hour holding period at 240 kips was also 0.03 inch. All of the permanent settlement was caused by creep during the holding period at 240 kips. A third load cycle to three times the design load was attempted, but the hydraulic pump malfunctioned at the 360 kip load. The creep rate observed at the 360 kip load was approximately 0.016 inch/hr. Total permanent settlement of the pile tip at three times the design load was approximately 0.2 inch. A plunging failure was expected near 380 to

400 kips. Wave equation analysis by Insitu Tech using the CAPWAP program and the observed penetration resistance of 34 blows/ft determined from the restrike of this pile indicates a failure load of about 440 kips. Ultimate capacity by the Case method was 298 kips The CAPWAP method evaluates the soil input parameters and distribution of soil resistance from the PDA results and should lead to a better estimate of bearing capacity than the Case method.

61. Comparison of results of pile S-5-12 with those of E2 in Figure 12 shows that pile S-5-12 has a larger stiffness than pile E2 and should also have greater ultimate capacity. The exceptional stiffness and capacity of pile S-5-12 are attributed to a possible increase in the pile elastic modulus, densification of the coral sands from earlier driving of the production piles, dissipation of excess pore pressures since driving of the production piles, and adhesion of the coral sands with the pile concrete since installation.
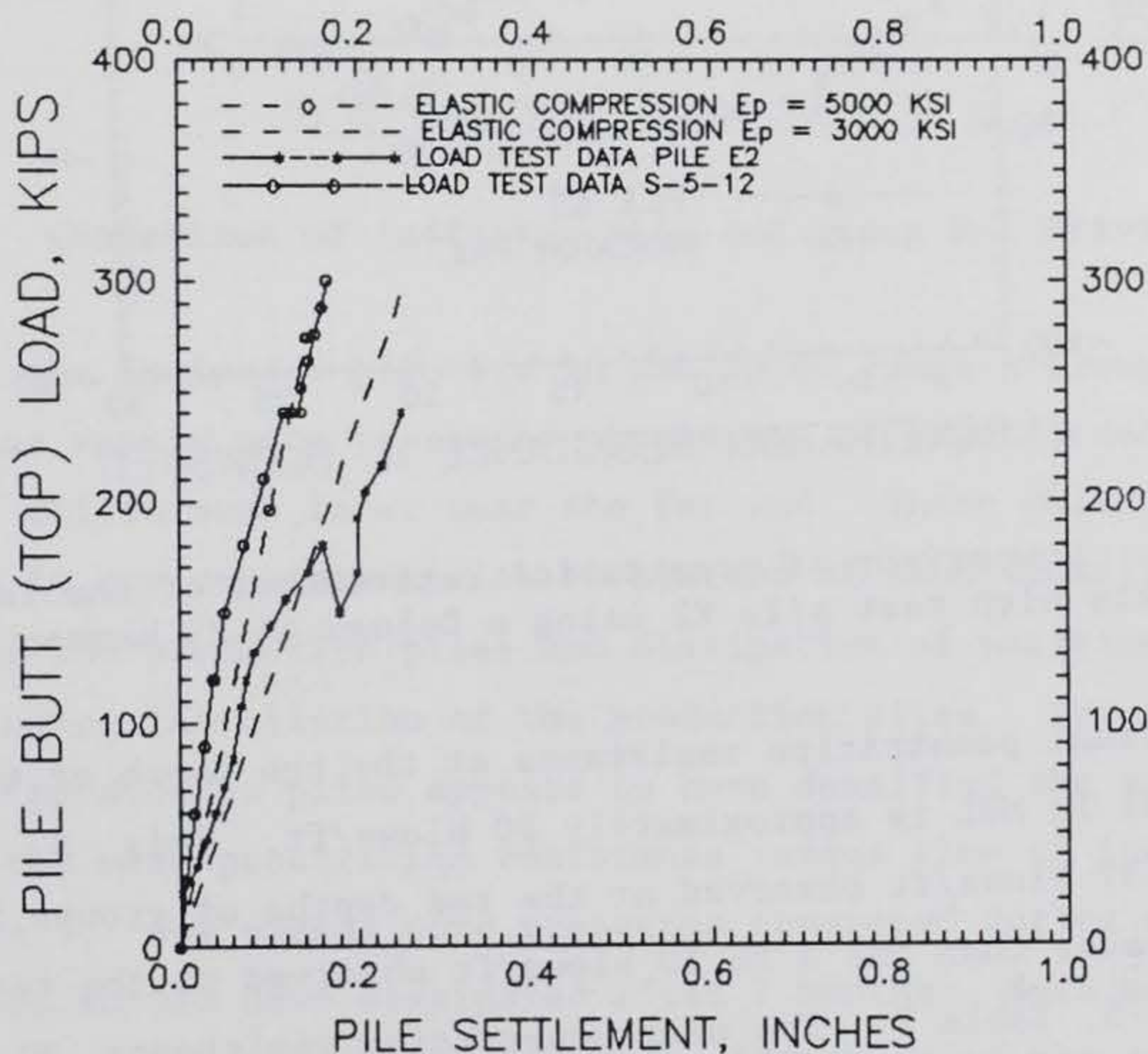

Figure 12. Comparison of load test results

Driving of Indicator Pile

64. An indicator pile of the same dimensions and type as the production piles was driven between groups S-4 and S-5 near pile S-5-1 following the

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repeated static load test. A comparison of the pile driving record of this
indicator pile with the penetration resistance of test pile E2 in Figure 2a is
given in Figure 13. This figure shows that the penetration resistance of the
indicator pile is substantially larger than that of pile E2, especially below
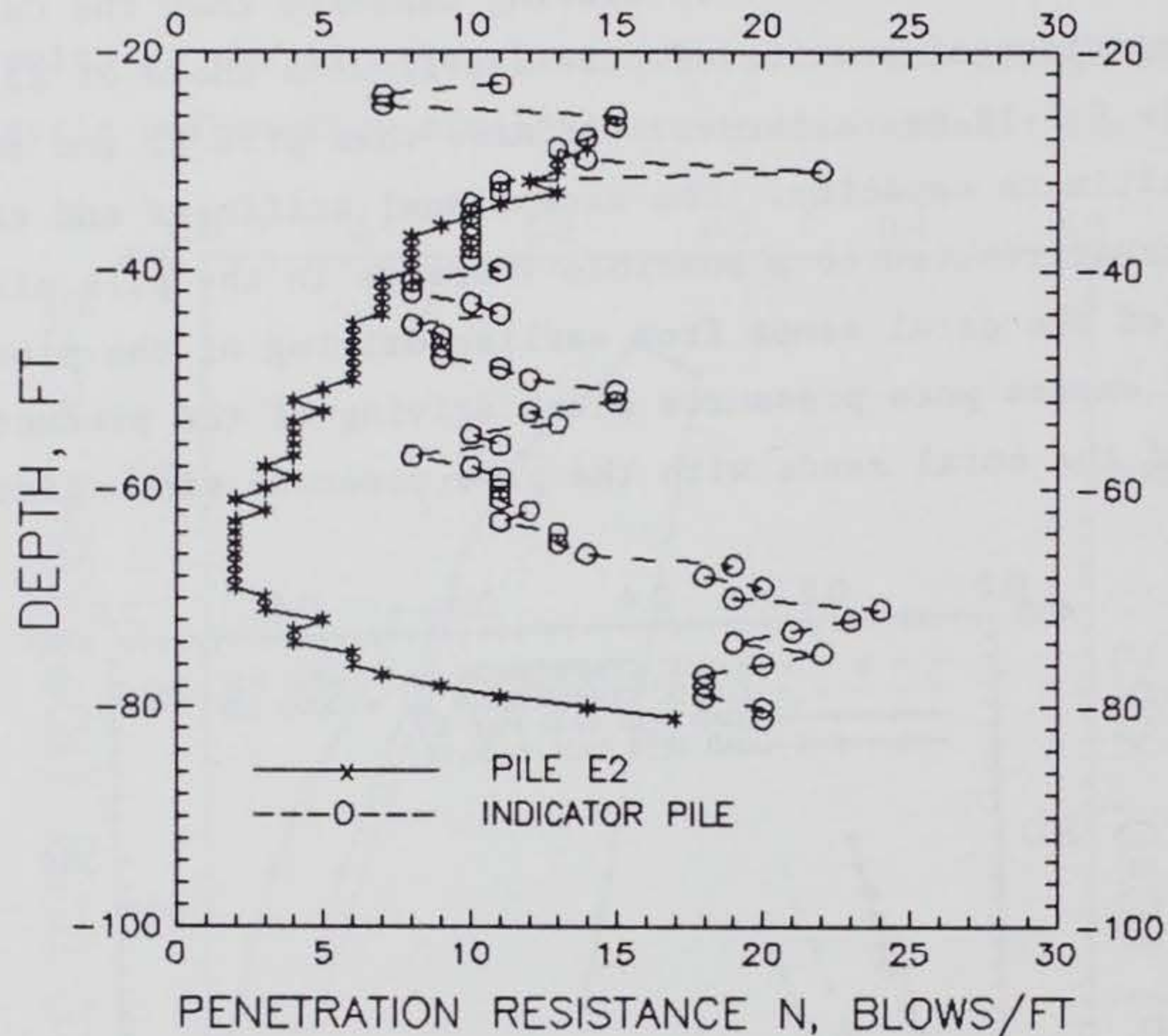-40 ft MSL.

Figure 13. Comparison of penetration resistances of the indicator
pile with test pile E2 using a Delmag 46-23 hammer

65. The final penetration resistance at the toe depth of the indicator
pile at about -81 ft MSL is approximately 20 blows/ft. This N is well
within the 6 to 37 blows/ft observed at the toe depths of groups S-1, S-2, N-1
and N-2, but greater than the 3 to 10 blows/ft observed at the toe depth for
groups S-4 and S-5, Table 2. The mean penetration resistance N' of this
indicator pile is 13.4 ± 4.5 blows/ft, which is the same or exceeds the
largest mean penetration resistance observed during driving of any of the
offshore production piles in the North groups, Figure 5.

66. Comparison of penetration resistances observed during driving for
group S-1 and the indicator pile given in Figure 14 shows that the penetration
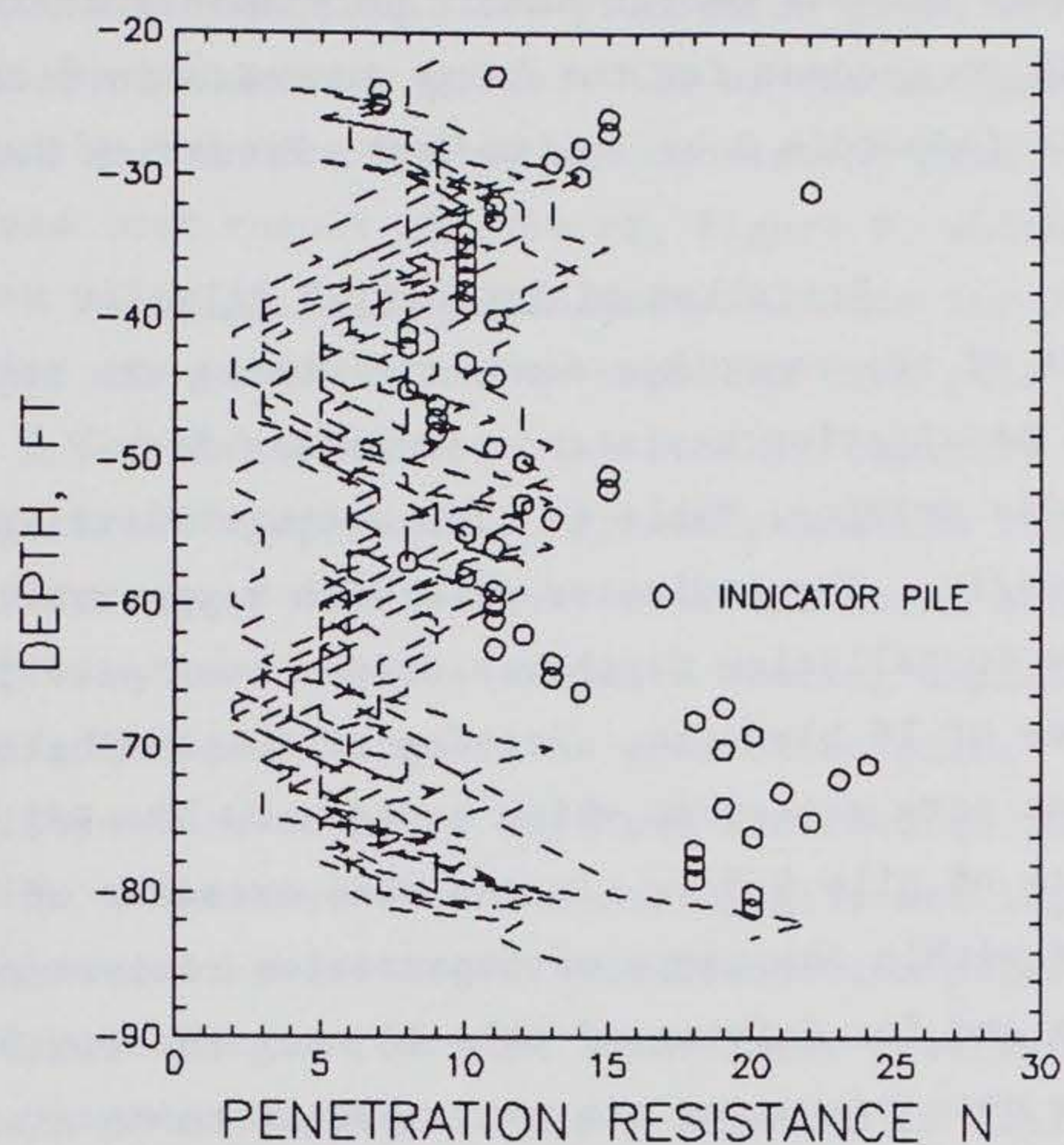
Figure 14. Comparison of indicator pile and group S-1 driving records

resistances of the indicator pile are in the upper range. Group S-1 was selected because excess pore pressures should not be significant compared to those in piles driven much later near the far end. These observations in Figures 13 and 14 are consistent with mechanisms of sand densification during installation of the production piles and dissipation of positive pore water pressures following installation of the production piles. For example, driving of the production piles appears to have densified the sands as observed from the mean penetration resistance versus time of installation data shown in Figure 5. Any excess pore pressures generated during driving of the production piles should have dissipated after 7 months. Such excess pore pressures could have been generated from densification of the sands during pile driving.

67. Another reason for the increased penetration resistance of the indicator pile compared to the resistances observed during driving of the production piles near the far end is that the driving system for the indicator pile was not as efficient as that during installation of the production piles.

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This is not probable because poorer hammer performance at these modest blow counts is unlikely to account for the large increase in final blow counts of the indicator pile (Appendix B by InSitu Tech, Frederick Harris 1992).

Restrikes of Production Piles

68. Records of the restrikes conducted during the supplementary testing program indicated penetration resistance increases from 2.2 to 9.7 times that recorded during pile driving, Table 6. The average restrike resistance varied from 14 to 40 blows/ft. The indicator pile with a penetration resistance of 20 blows/ft at the installation depth was driven near pile S-5-1 with a restrike resistance of 16 blows/ft. Driving of the indicator pile may have caused local excess pore pressures which could have adversely influenced the restrike resistance of pile S-5-1. In any case, results of the production pile restrikes are within the range of penetration resistances observed for the indicator pile and for driving of pile E2 near the final penetration, Figure 13. Excess pore pressures are not expected to be a significant factor in reducing penetration resistance because of the long 7 month delay since installation of the offshore production piles.

Table 6

Penetration Resistance of Restrikes of Production Piles in March 1

Pile Number	Installation Resistance, Blows/FT	Restrike Resistance, Blows/Ft	Increase Blows/Ft	Times Increase Factor	Elapsed Time, Days	Case Method Capacity, kips
N-3-7	8	40	32	4.0	278	310
N-4-1	7	31	24	3.4	255	330
N-5-1	3	26	23	7.7	253	266
N-6-6	3	32	29	9.7	248	240
N-6-12	8	34	26	3.3	248	270
S-4-1	6	32	26	4.3	272	240
S-5-1	5	16	11	2.2	260	340
S-5-12	5	34	29	5.8	261	298
S-6-12	3	24	21	7.0	259	314
S-6-Out	4	14	10	2.5	258	300

69. Results of the Case method of pile driving analysis (PDA) indicate that the restruck production piles have bearing capacities given in Table 6.

These ultimate capacities show that the production piles will be able to adequately support the production piles. All of the ultimate bearing capacities computed by the Case method are consistent with the anticipated capacity of the load test result of pile E2, Figure 9, which exceeds 240 kips. All of the restruck piles in Table 6 had been in place for 248 or more days, yet these capacities are similar to that expected from pile E2 that had been in place for only 1 day and similar to the 240 kip capacity of the indicator pile determined by the PDA (Appendix B, Frederick Harris 1992).

70. These data suggest that adhesion of the coral sands with the pile does not appear necessary to achieve adequate bearing capacity. Excess pore pressures generated during driving of the production piles, especially in groups 5 and 6 of the offshore piles may have contributed to the lower than expected penetration resistance observed during pile driving. Densification of the coral sands caused by driving of the production piles also probably contributed to the much larger restrike resistances observed during the supplemental test program compared to those observed in Table 4.

PART IV: CONCLUSIONS AND RECOMMENDATIONS

Conclusions

71.   The results of the supplemental test program show that the existing pile foundation has adequate capacity to support the dry dock; design specifications for this project led to an adequate foundation.   Ultimate bearing capacity evaluated by the wave equation with input data from the pile driving analyzer (PDA) was consistent with results of static load tests.   The average ultimate bearing capacity evaluated by the wave equation, dynamic pile driving formulas, and soil parameters was also consistent with results of the static load tests.   The lower bound range of bearing capacities evaluated by the wave equation, dynamic pile driving formulas, and soil parameters was useful in determining the minimum acceptable penetration resistance of 12 blows/ft.   Vesic's equation provided reasonable estimates of settlement in these coral sands.

72.   Precast, prestressed concrete piles driven into coral sands typically indicated less than the anticipated penetration resistance during installation, but soil freeze was significant.   Less than the anticipated penetration resistance was attributed to crushing of coral sand particles during driving and generation of excess pore water pressure.   Crushing destroys any cementation between the coral sand particles and can reduce skin friction between the sands and the concrete pile during installation.

73.   Driving of the production piles led to substantial increased mean penetration resistance of piles driven at a later time in a particular group. This increase is attributed to densification of coral sands as a result of pile driving.   The Frederic Harris report on page 14 stated "that if the increased density due to pile driving was the reason for the increased blow counts of the indicator pile, then some increase in blow counts of subsequent production piles in the pile clusters should have been observed."   Such an increase was observed in the mean penetration resistance   N'   of the production piles as shown in Figure 5.   The increased density due to pile driving may therefore explain the increased penetration resistance of the indicator pile at the final toe elevation (N = 20 blows/ft) compared with those of production piles near the far end of the dry dock (N = 1 to 15 blows/ft).

74. Generation of excess pore pressures was expected from sand densification and may have contributed to the decreased penetration resistance as the production piles were installed near the far end of the dry dock. Both densification of the sands as a result of pile driving and dissipation of excess pore pressures after driving the production piles appear to have contributed to the penetration resistances and bearing capacity of the piles tested during the supplemental test program.

75. Cementation of the calcareous sands and/or adhesion between the soil and the concrete piles could have contributed to the penetration resistance and bearing capacity observed during the supplemental test program, but the data were not sufficient to confirm this conclusion. Coral sands near the far end of the dry dock may have originally been less dense and weaker than those onshore, but the available boring data were not sufficient to support this conclusion.

Recommendations

76. Confidence in the design may be improved by evaluating ultimate bearing capacity Q_u by several methods. These methods include wave equation analysis, dynamic pile driving formulas, and analysis using estimates of the angle of internal friction ϕ. None of these methods should be used by themselves to determine Q_u and the design load. At least one static load test, driving of several indicator piles with PDA, and restrike of several piles including the pile selected for the static load test are recommended to confirm calculation of Q_u. A repeated static load test should be performed if operating loads will be cyclic. A factor of safety of 2 with a static load test, indicator piles, and restrikes is adequate. Settlement should be estimated using guidance in EM 1110-2-2906 and results of the static load test. Piles selected for static load tests should be loaded to failure.

77. Program GRLWEAP is recommended for wave equation analysis. Recommended dynamic pile driving formulas include Gates, PCUBC, Danish, and Hiley equations. These formulas are simple to apply and may be used with relatively low factors of safety of about 3 or 4. The ϕ of coral sands should be estimated from results of in situ tests such as the standard penetration (SPT) and cone penetration (CPT) tests. Past correlations of SPT and CPT with ϕ are given in EM 1110-1-1905. Meyerhof and Nordlund methods

are recommended for evaluating bearing capacity from ϕ. The lower bound range of ultimate bearing capacities calculated by these methods may be selected to estimate a reasonably conservative minimum acceptable penetration resistance N_{min}. N_{min} = 12 blows/ft was found for the Kwajalein dry dock as illustrated in Figure 8.

78. The influence of sand densification from pile driving should be determined by plotting the mean penetration resistance N' of each pile as a function of time. Significant increases in N' with time indicate that piles driven earlier should have a greater penetration resistance than indicated by the driving record. An indicator pile may be driven adjacent to a pile group 1 week or more after the group was driven to determine the influence of pile driving on penetration resistance.

79. Construction of driven piles for future projects should include driving of indicator piles at strategic locations such as near the boundaries of the foundation and at interior locations to determine the distribution of the penetration resistance. Weak areas indicated by borings should also be checked with the driving of indicator piles.

80. The influence of soil freeze should be determined if the penetration resistance of some production piles near the final toe elevation is not adequate. Some production piles with excessively low penetration resistances should be driven 1 to 2 ft above the final toe elevation. Driving may then be continued at another location. After approximately 1 week, piles that had not been driven to the final toe elevation should be restruck and driven to the final elevation to determine the influence of soil freeze.

81. Restrike of indicator piles should be performed as a function of time to determine the rate of soil freeze. The rate of soil freeze may assist the differentiation between various strength gain mechanisms such as sand densification, dissipation of excess pore pressure, cementation of coral sand particles, and adhesion of coral sands with the pile. Restrikes, for example, could be performed 1 day, 2 days, 1 week, 1 month, and 2 months after installation depending on the construction schedule and availability of funds.

82. A record of the penetration resistances during restrikes should be kept to determine the variation of the blow counts versus penetration depth. Adhesion of the coral sands with the concrete piles may cause an initially high penetration resistance on restrike, but the penetration resistance may

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drop rapidly when the pile is driven a small distance further into the soil;
e.g., one inch or less.

83. Piles should be inspected for cracking and spalling after
installation if wave equation analyses indicate excessive tensile stresses.
None of the restruck piles exhibited effects of severe structural cracking for
this foundation (Appendix B of the Frederic Harris report).

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