INTERIM REPORT

DESIGN OF PILE FOUNDATIONS ARCO DOCK COMPLETION PROJECT
CHERRY POINT REFINERY
CHERRY POINT, WASHINGTON

a report to

Anvil Corporation
1675 W. Bakerview Rd.
Bellingham, WA 98226

by

Lymon C. Reese

and

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November, 1993

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INTRODUCTION

The soil investigation was carried out by Shannon & Wilson, Inc., during the time period of September 13 and September 24, 1993. The report on the subsurface conditions was completed in November 1993, and the information presented herein is based on the November report provided by the consultants.

The proposed construction will consist of a trestle, extending from an existing trestle, and a second pier or shiploader. The soil borings were arranged to interfere minimally with the activity of the existing pier; it is presumed that the same requirement will exist for the new construction.

With respect to the design of the piles for the structure, it is anticipated that LCR&A, in cooperation with Anvil and other firms, will be principally responsible for interpreting the response of the soil to the axial and lateral loading of the supporting piles. This brief presentation is aimed at giving some guidance toward the steps that will be necessary in design.

GENERAL DESCRIPTION OF SOIL CONDITIONS

The subsurface conditions were investigated by 13 over-water borings, which were drilled to depths below the mudline ranging from 74 ft at Borehole B-13 to 151 ft at Borehole B-4. The soils at the top 5 ft are loose to medium-dense, silty sand with shells and organic material. The blow counts from the Standard Penetration Test (SPT) range from 2 near the mudline to about 20 at the depth of 5 ft below the mudline. At depths from 5 to 30 ft is a stratum of medium-dense to dense, gray, clean to slightly silty, fine to medium sand with abundant shell fragments is at depths from 5 to 30 feet.. The average values of the SPT blow counts for this section range from 20 at Borehole B-7 to over 60 at Borehole B-1.

The soils at depths of 30 ft to 60 ft show an increase in the content of gravel and silt. The soil can be characterized as dense to very dense, fine to coarse sand with gravel. The percentage of gravel is significantly higher for the bore holes closer to the shore. Thin layers of silty clay or

clayey silt were interbedded within this layer. The SPT blow counts range from 20 at the silty clay layer to 100 at the gravely sand layer.

Interbedded, hard, slightly sandy clayey silt and very dense, slightly silty to clean, fine to medium sand was encountered at depths of 60 ft to 150 feet. The SPT blow counts are generally over 100.

While the gravels found from the SPT samplers are about 0.5 to 1.0 in. in diameter, gravels with a larger size of 1.0 to 2.0 in. were discovered in a thin-walled tube. There may be cobbles in some of the boreholes; however, any such cobbles failed to impede the drilling. The content of fines is about 10% for soils near the mudline and 20 to 30% for soils at depths of 40 to 60 feet. An undisturbed thin-wall tube taken at depths of 35 to 37 ft at Boring B-9 was used for a direct shear test. The test results indicated that the silty sand has an internal friction angle of 36.5° and 2.5 tsf of cohesion.

It is common practice that the internal friction angle of granular soil can be correlated to the SPT blow counts. The data on SPT blow counts supplied by Shannon & Wilson, Inc., were converted into angles of internal friction using a chart from the 1984 *Handbook on Design of Piles and Drilled Shafts Under Lateral Load* by the United States Department of Transportation. This chart takes both SPT values and confining pressure into account. The computed values of friction angles compared well with values recommended in sources such as *Foundation Analysis and Design* by Bowles (1977), *Principles of Geotechnical Engineering* by Das (1985), and *Essentials of Soil Mechanics and Foundations* by McCarthy (1977).

Plots of depth versus angle of internal friction for each boring are attached in the Appendix. The converted internal friction angles are generally equal to or higher than 40 degrees except the soil within the top 10 feet. The borings that lay along the projected locations of the mooring dolphins were analyzed, and a soil profile was defined.

NATURE OF LOADINGS

The specific loadings will await the development of further details on the design; however, the loadings on the trestle and shiploader will be principally compressive with only minor amounts of lateral loads. The compressive loadings will have a steady component and a minor component that will be cyclic. There will be no tensile loads of the piles for the trestle and ship loader.

The loadings on the mooring and breasting dolphins will be lateral and cyclic. Furthermore, these loads will come generally from the same direction.

CONCEPTS FOR THE PILING

The subsurface conditions favor driven rather than drilled-in piling. Drilling would have a role if it were necessary to achieve a considerable amount of tension because it may be impossible to drive piles to a great depth into the sand. The sand, however, will provide an ample amount of compressive resistance, particularly in end bearing, to sustain any axial load that is anticipated.

For the compressive loading, a number of small-diameter piles in clusters or a single, large-diameter pile can be used. The penetrations to sustain the axial loadings, in either case, are expected to be in the order of about 60 feet or less. If the large-diameter pile is selected, an open-ended steel tube is to be preferred. The driving of the piles with a vibratory hammer can be given consideration.

For the breasting dolphins, the design must be made to absorb the energy of the docking vessel. Two approaches are viable for each of the dolphins: (1) a cluster of piles, some on a batter, can be driven with the energy being absorbed principally by a secondary system; or (2) a large-diameter steel pipe can be used and a portion of the energy, perhaps a major portion, can be absorbed by the area under a lateral load-deflection curve. Similar concepts can be used for the design of the mooring dolphins.

PRELIMINARY DESIGN CURVES FOR THE SITE

Figure 1 shows the preliminary curves for side resistance (skin friction), end bearing, and total resistance for an open-ended, steel-pipe pile with a diameter of 4 feet. The curves will need further study because of the depth that was selected at which the pile will plug and behave as a closed-ended pile.

Figure 2 shows a preliminary lateral load-deflection curve for a pipe pile with a diameter of 10 ft, a wall thickness of 2 inches, and 80 ft of penetration. The load is assumed to be applied at 70 ft above the mudline. Figures 3 and 4 are the predicted deflection and moment curves for the piles. The curves will need further study because the loadings are from a single direction with the possibility of locked-in deflection.

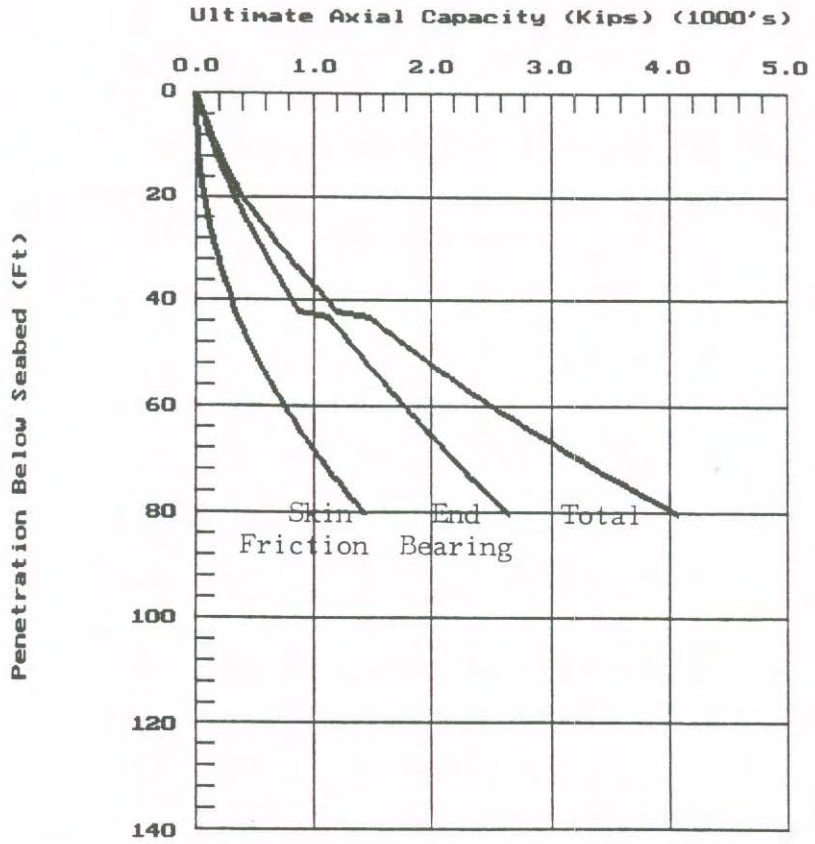


Fig. 1. Predicted axial capacity for an open-ended steel-pipe pile with 4-ft diameter.

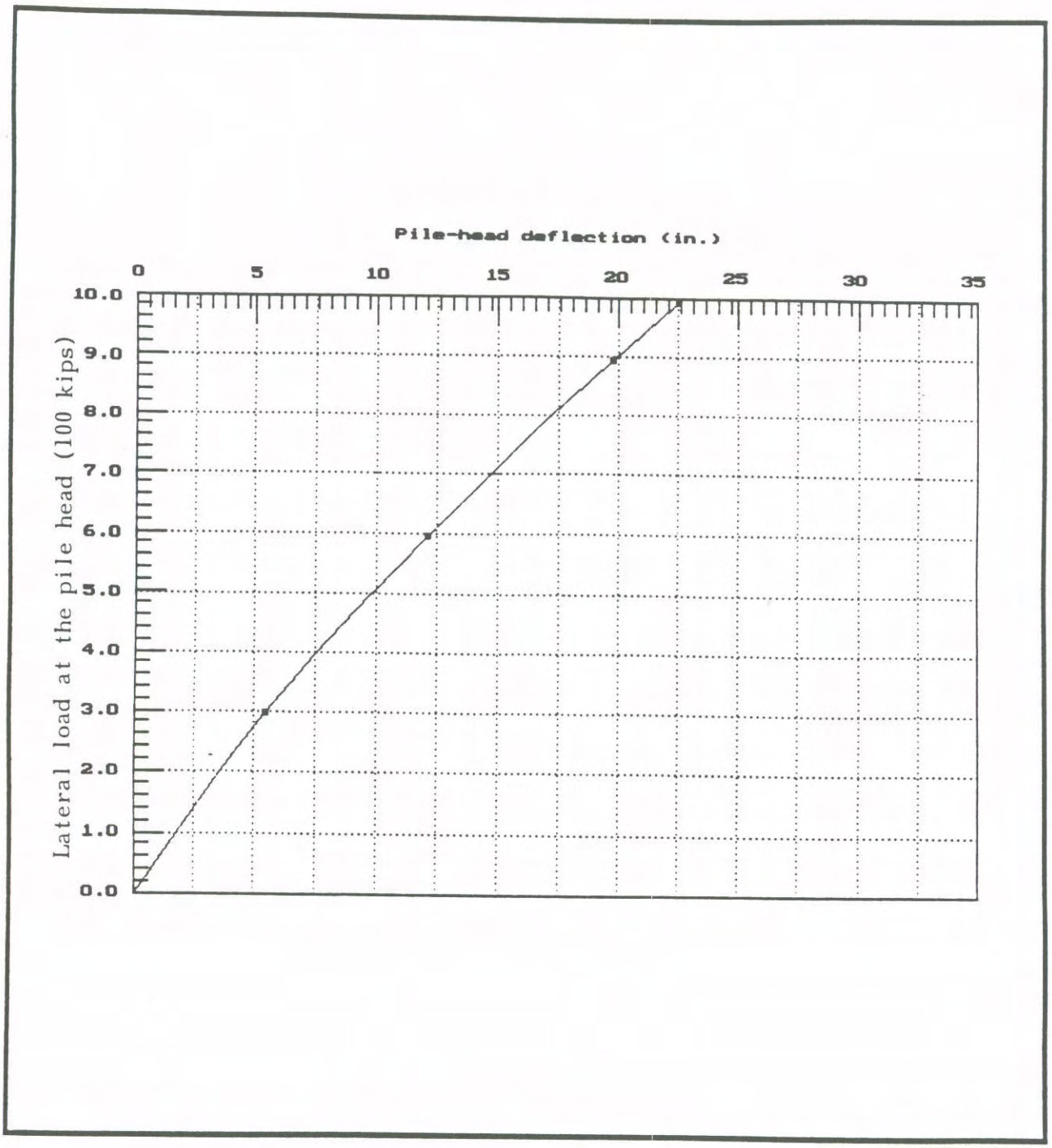


Fig. 2. Load-deflection curves at the pile head
(diameter = 10 ft.)

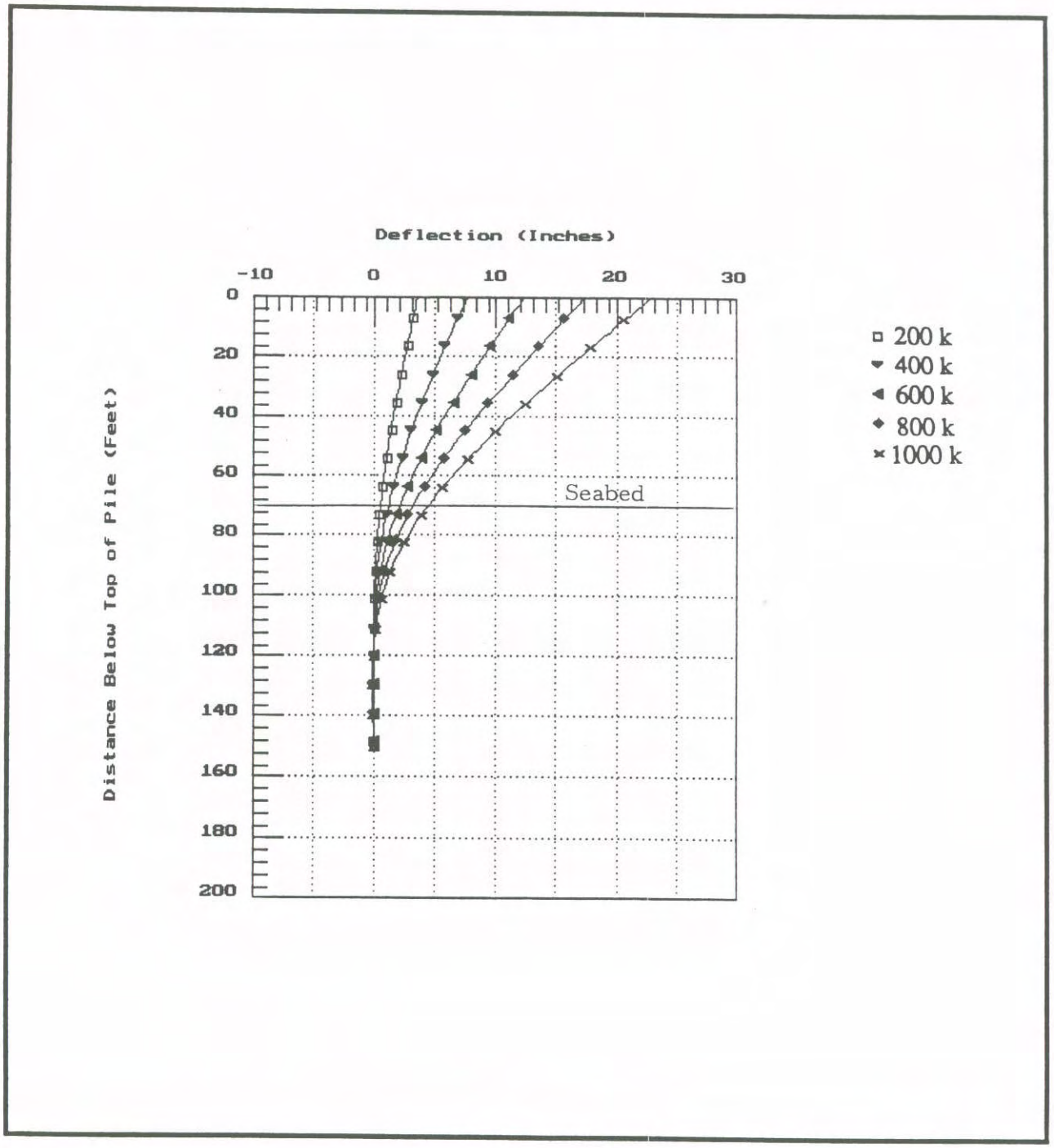


Fig. 3. Deflection curves for 10' ϕ piles under 80-ft penetration.

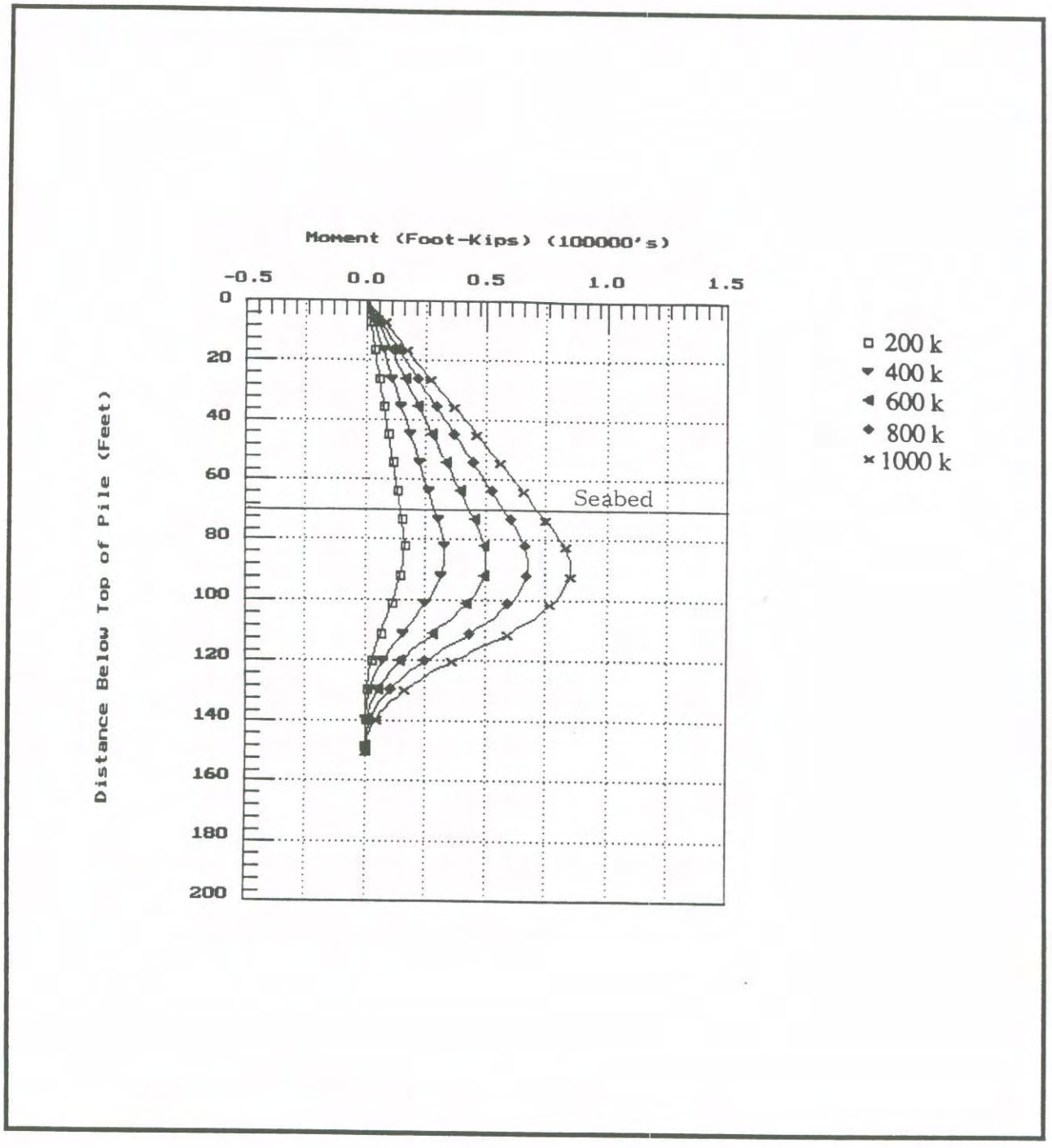


Fig. 4. Moment distribution curves for 10' ϕ pile with 80-ft penetration.

STEPS NEEDED FOR PROCEEDING TOWARD A DESIGN OF FOUNDATION PILES

The following steps are presented in order to outline the kinds of information that will be required in completing the designs for the piles.

- Development of preliminary concept for structure.

- Selection of axial and lateral loading for the components of the structure (factors to be considered are wave and current forces, dead loads, deck loadings, and loadings from ship docking and berthing).

- Review of the performance of the current platform to include information on possible corrosion, scour around the piling, and the permanent deflection of the piles, axially and laterally.

- Selection of the scheme for the piling for the elements of the structure.

- Consideration of methods of construction of piles to include a consideration of the construction capabilities available in the area.

- Preliminary design of piles.

- Evaluation of desirability of field loading tests.

- Comprehensive review of design and construction.

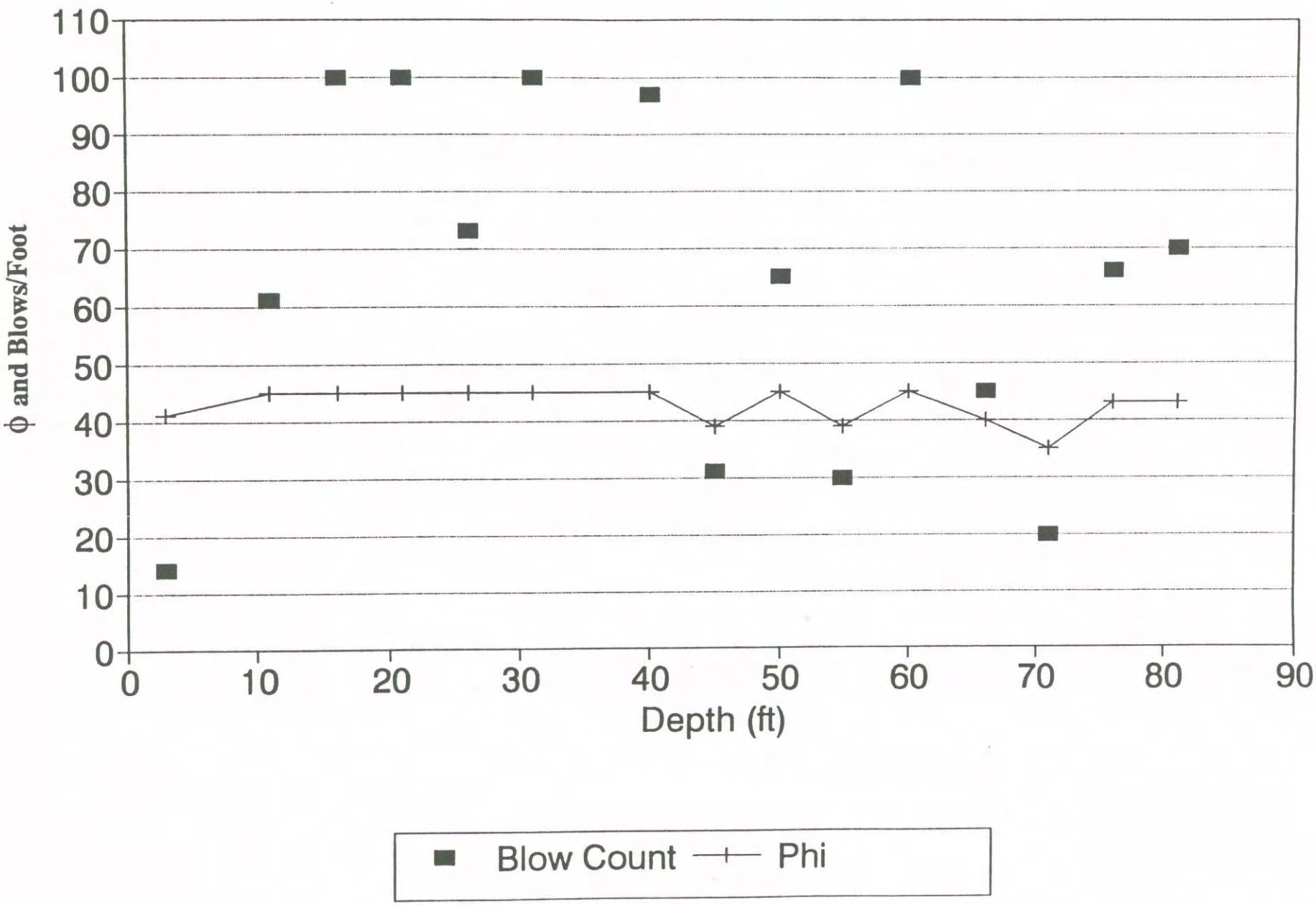
- Making necessary modifications and preparation of final designs, to include plans and specifications for construction.

LCR&A will expect to take a leading role in some of the steps outlined above and to consult with other firms in the performing of other steps.

APPENDIX

Soil Profile for SPT Blow Counts and Correlated Internal Friction Angles

Boring B-1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ϕ and Blows/Foot
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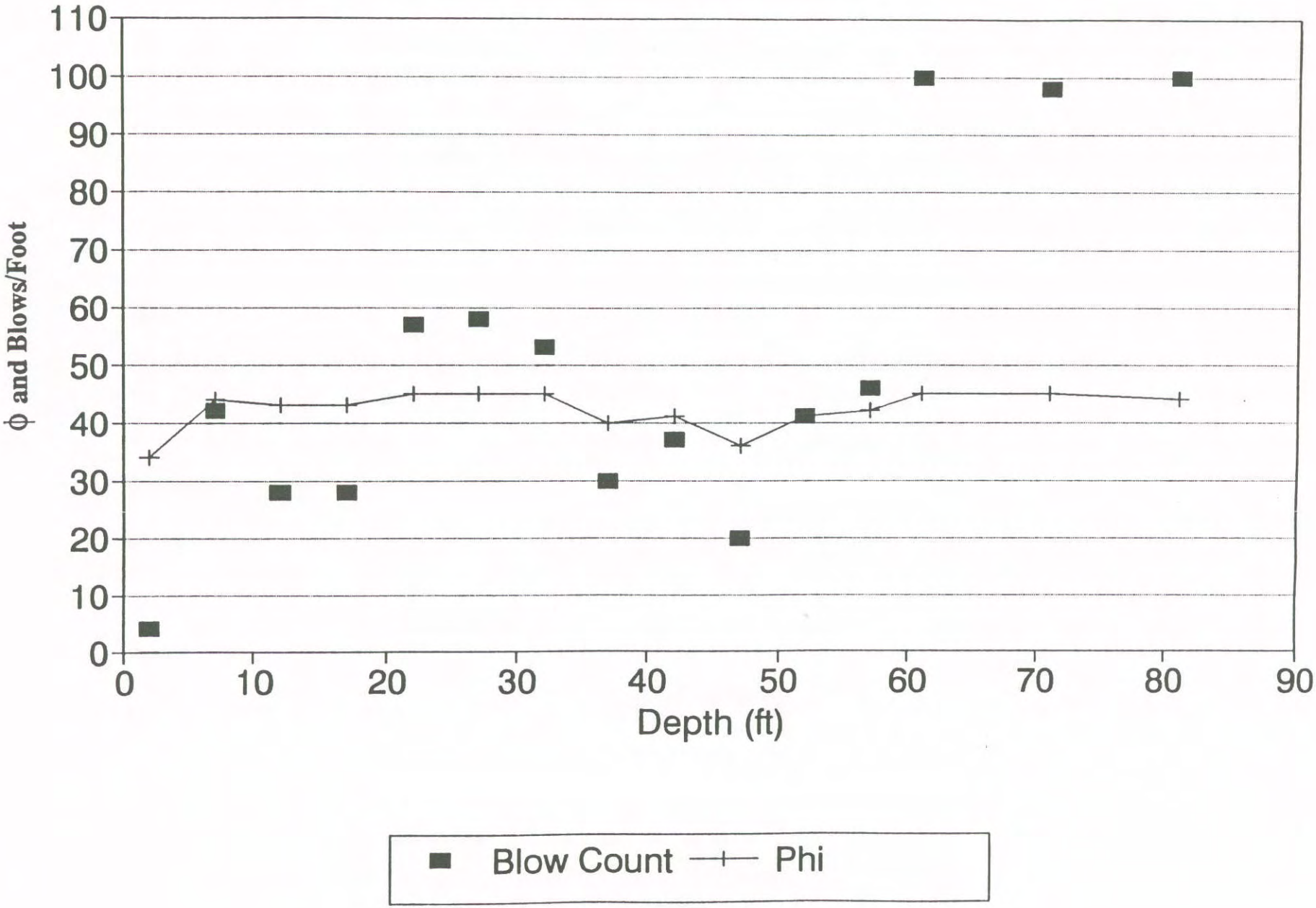
90
Depth (ft)
Blow Count
Phi

Boring B-2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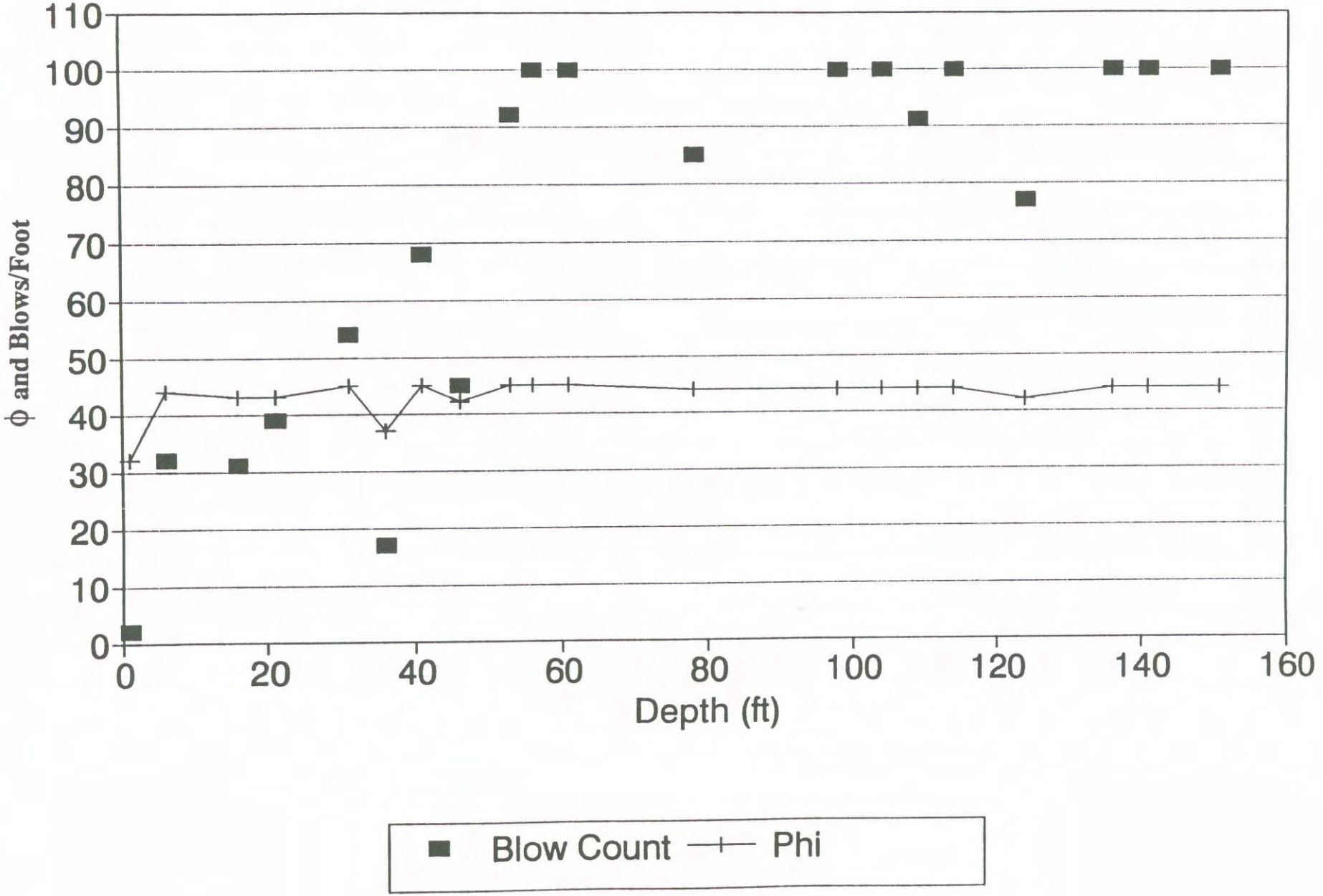
Depth (ft)
Blow Count
Phi

Boring B-4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ϕ and Blows/Foot
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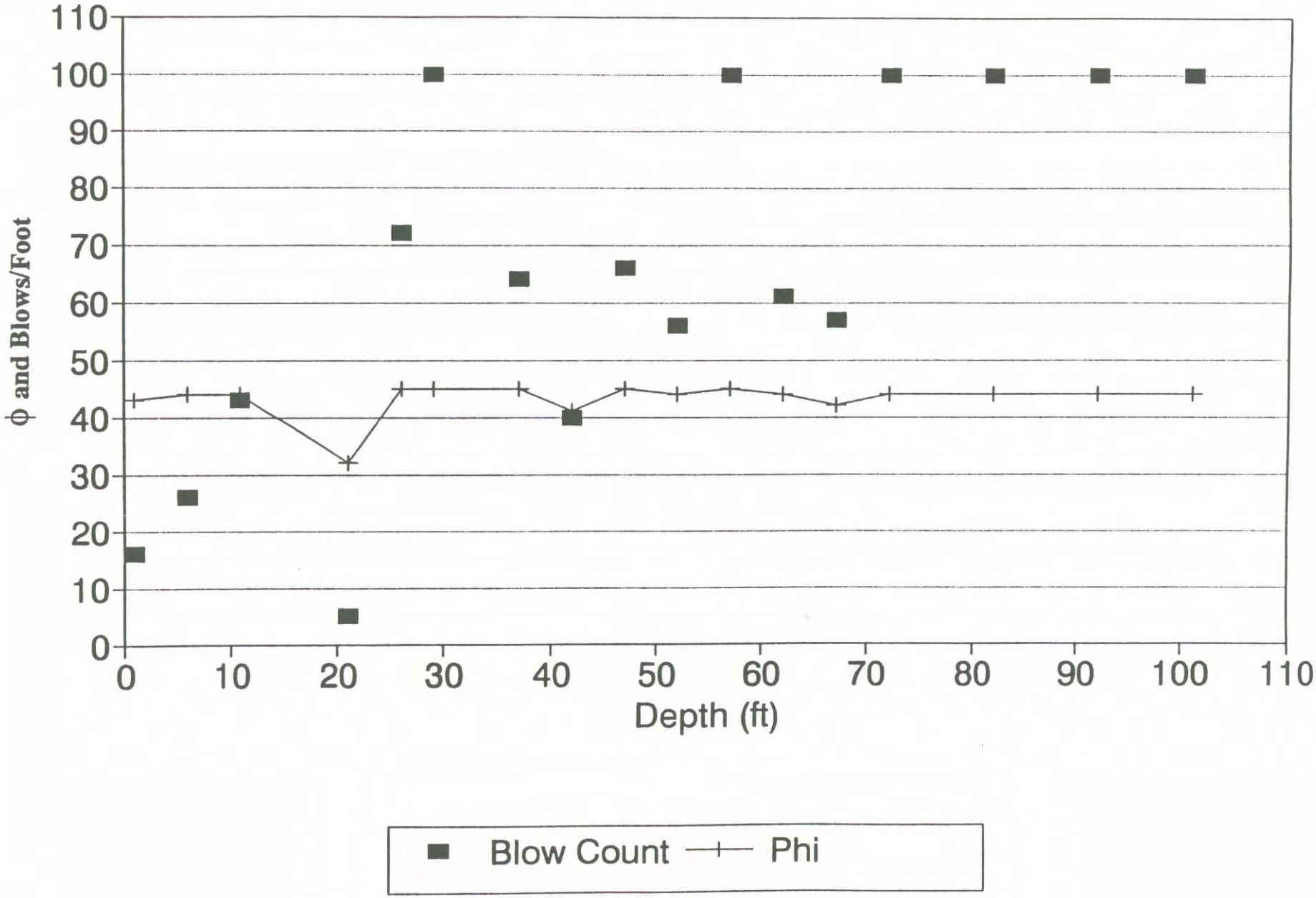
100
120
140
160
Depth (ft)
Blow Count
Phi

Boring B-6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ϕ and Blows/Foot
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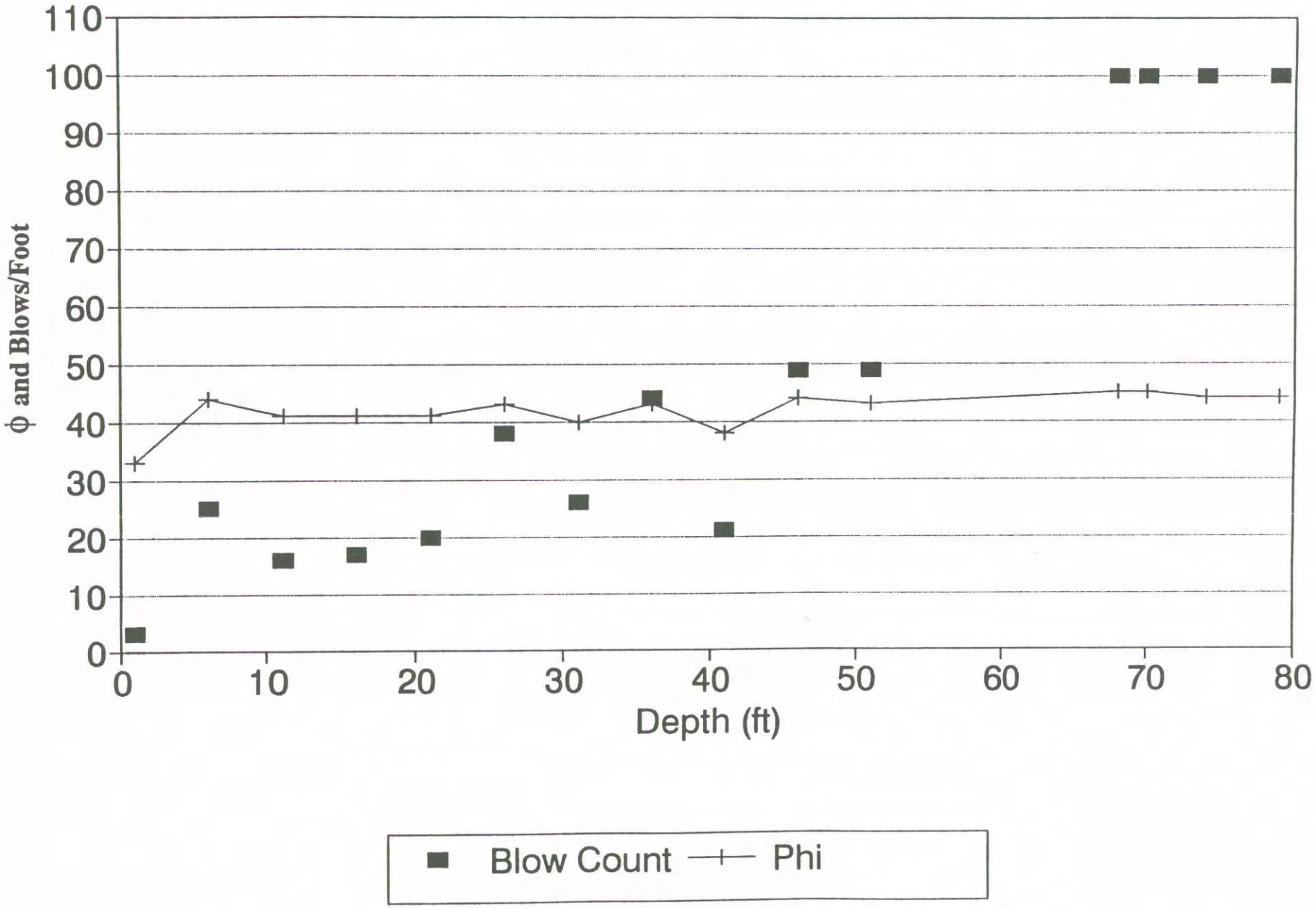
90
100
110
Depth (ft)
Blow Count
Phi

Boring B-7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ϕ and Blows/Foot
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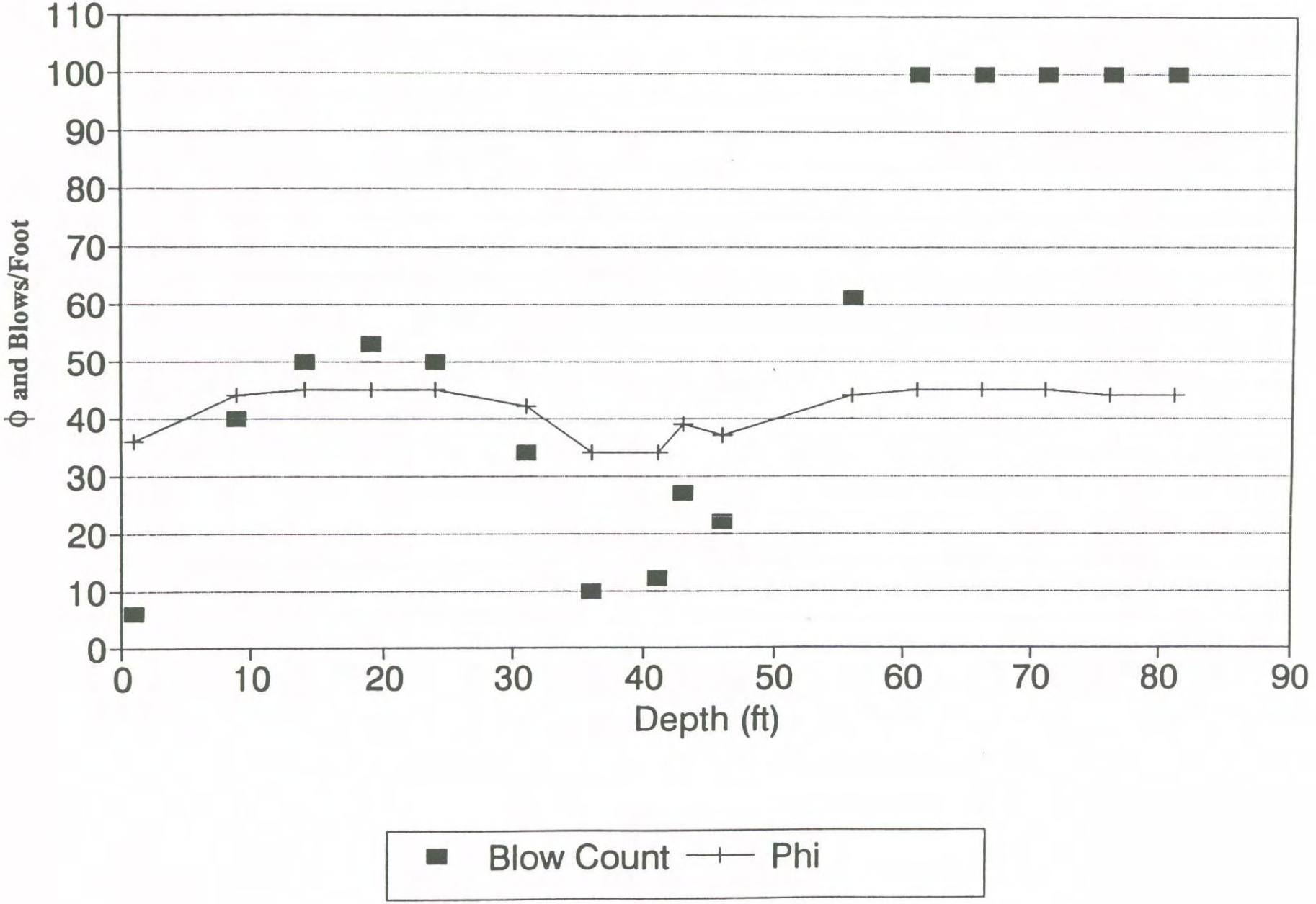
80
Depth (ft)
Blow Count
Phi

Boring B-8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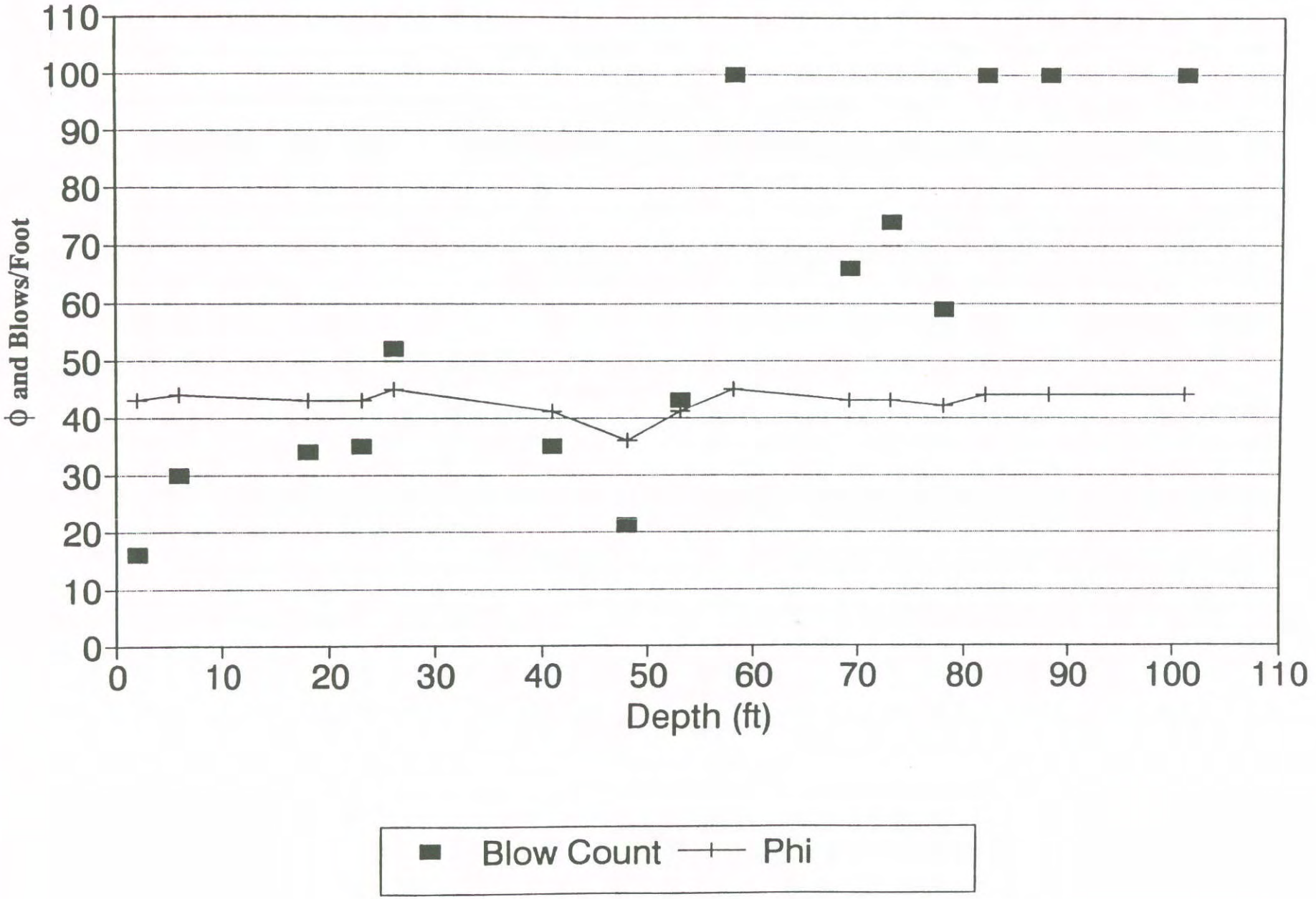
90
Depth (ft)
Blow Count
Phi

Boring B-9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ϕ and Blows/Foot
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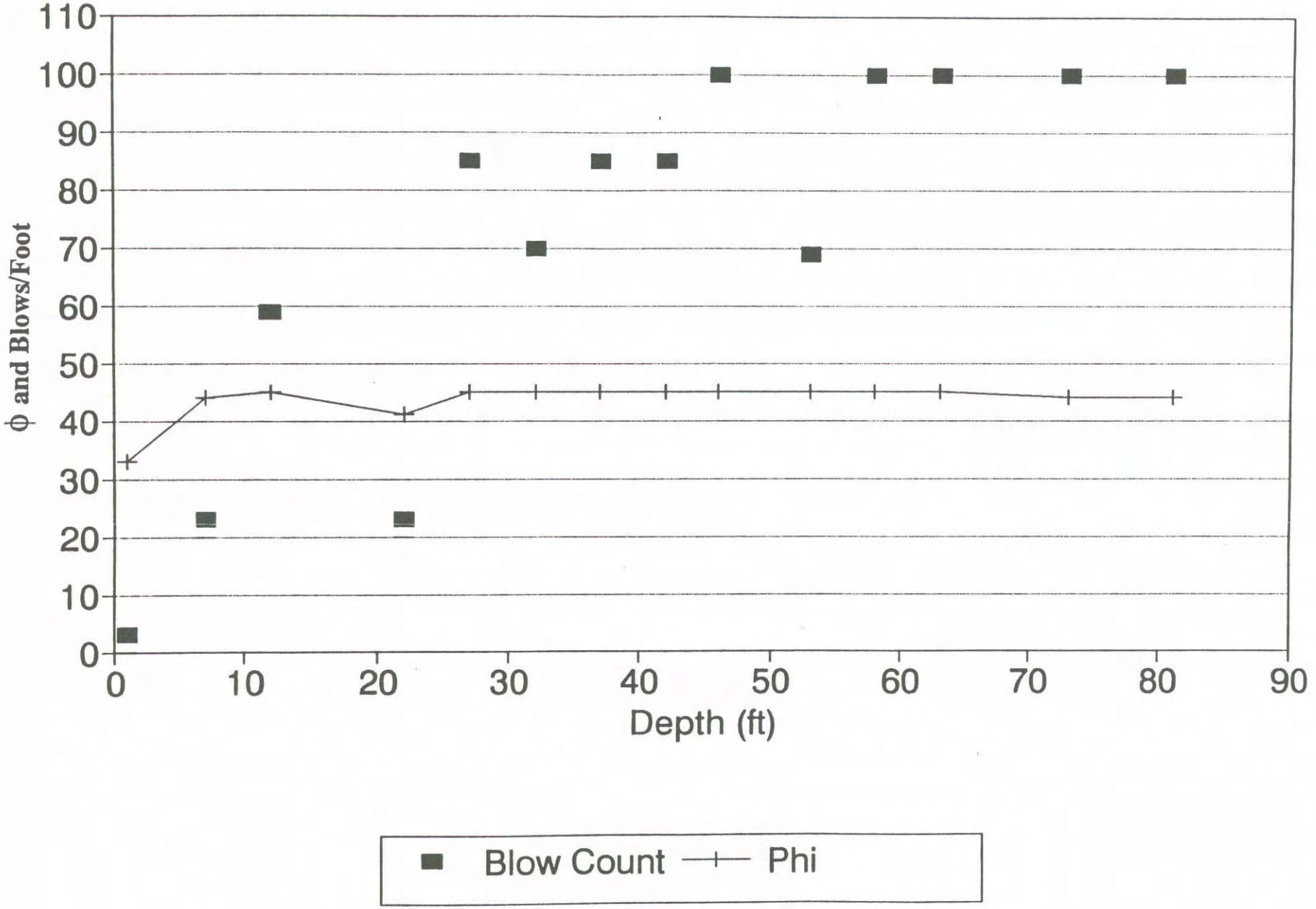
Depth (ft)
Blow Count
Phi

Boring B-10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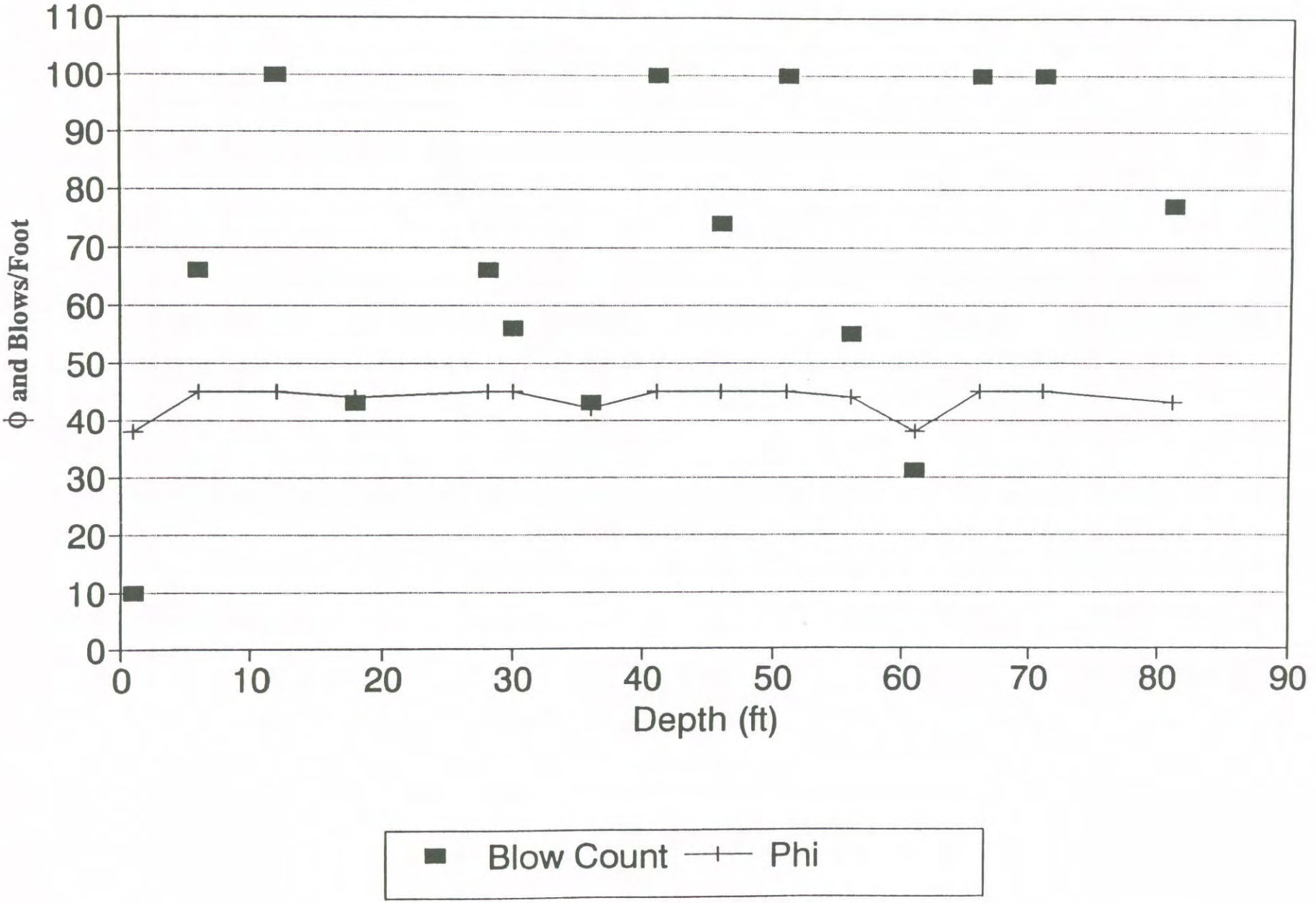
Depth (ft)
Blow Count
Phi

Boring B-11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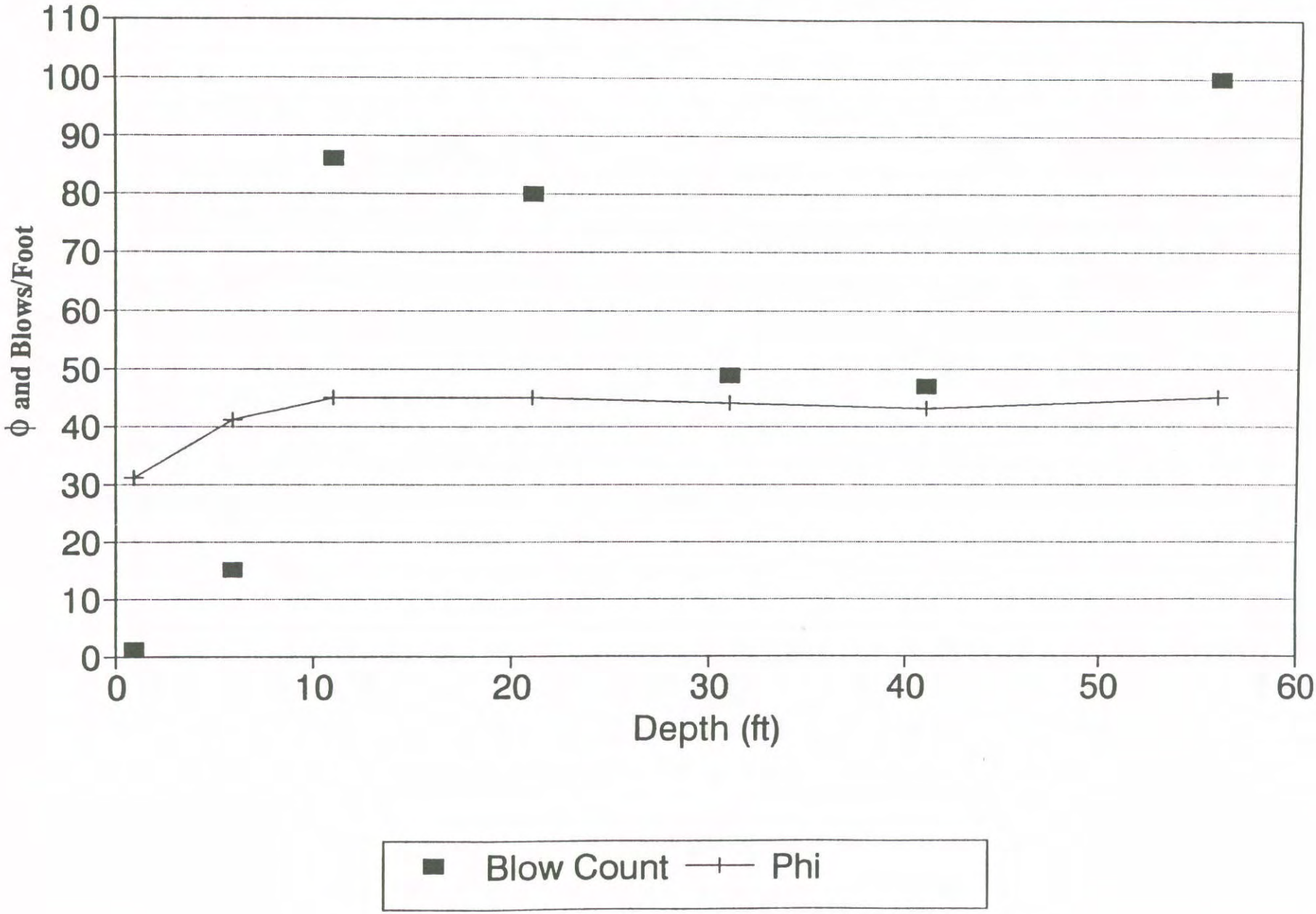
Depth (ft)
Blow Count
Phi

Boring B-12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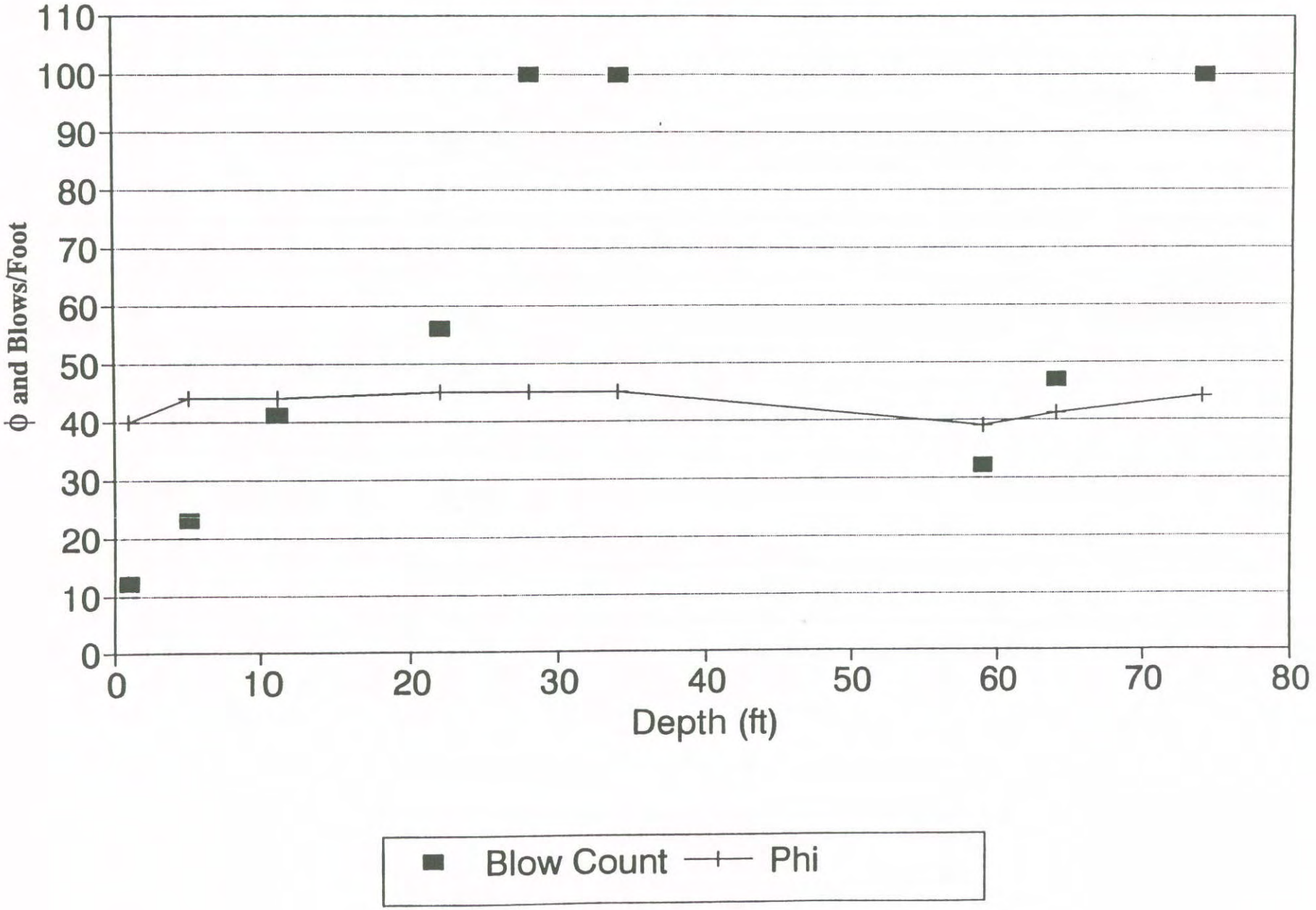
Depth (ft)
Blow Count
Phi

Boring B-13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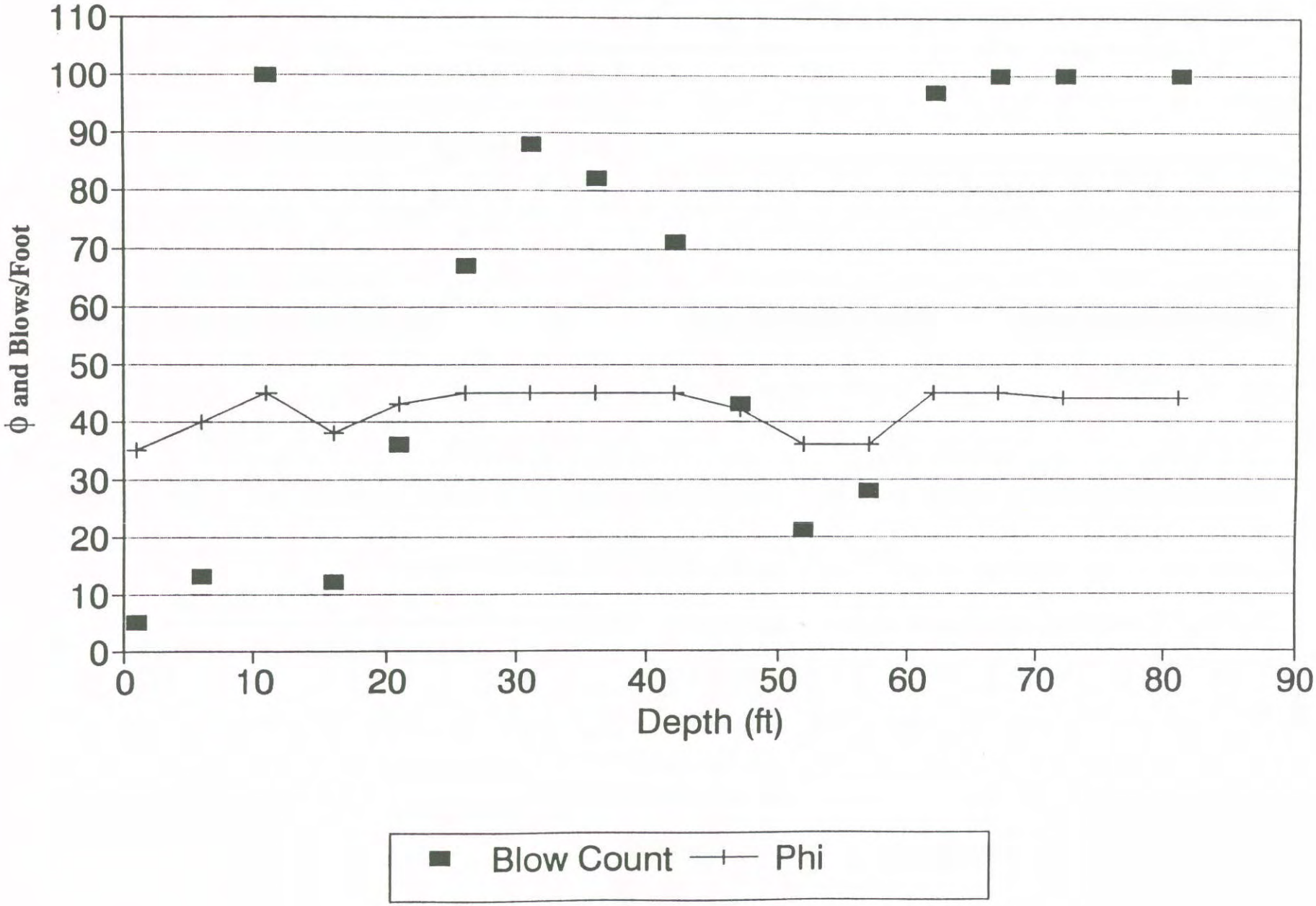
Depth (ft)
Blow Count
Phi

Boring B-14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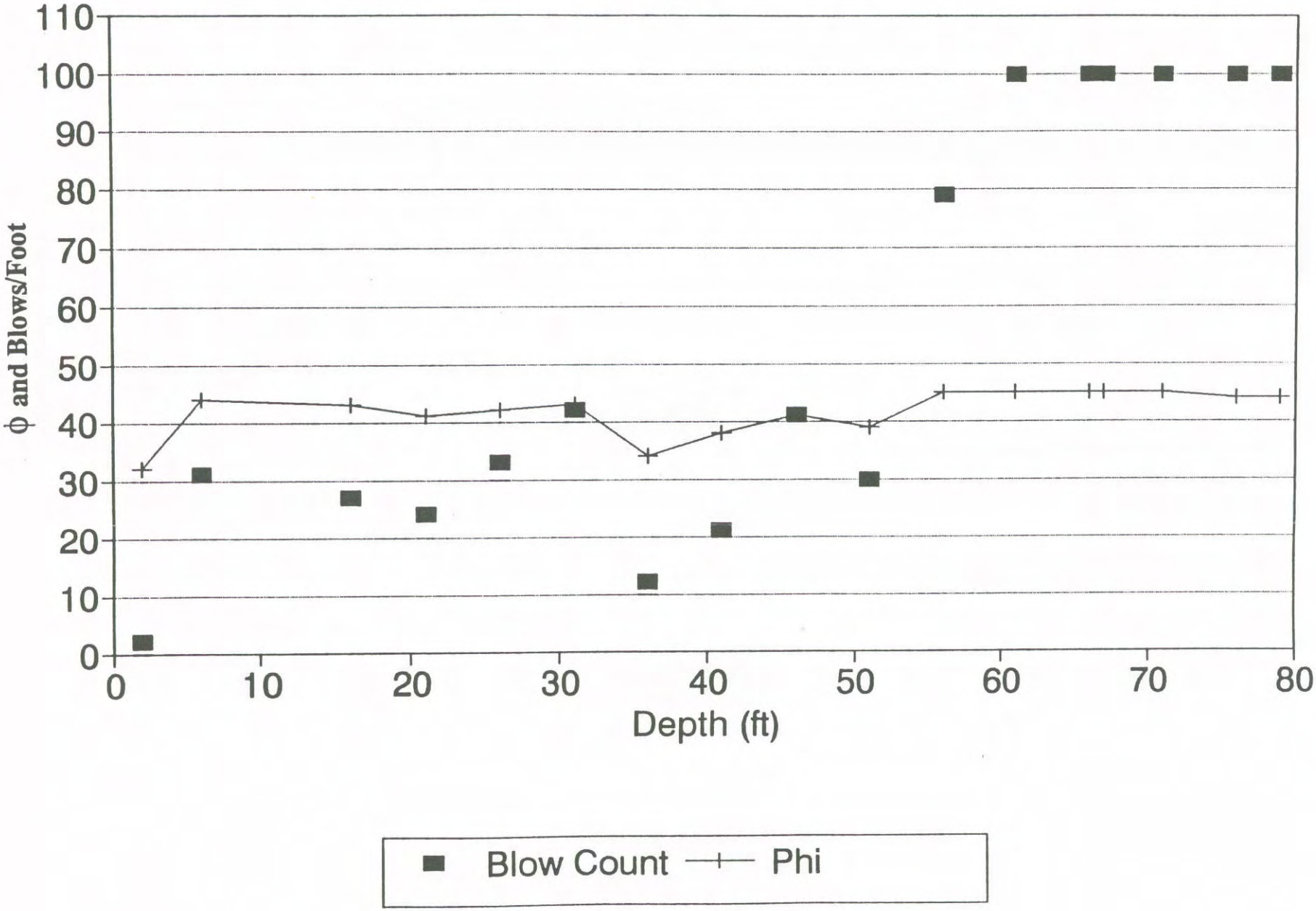
Depth (ft)
Blow Count
Phi

Boring B-15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ϕ and Blows/Foot
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40
50
60
70
80
Depth (ft)
Blow Count
Phi

INTERIM REPORT NO. 2

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INSTALLATION OF PILE FOUNDATIONS
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CHERRY POINT REFINERY
CHERRY POINT, WASHINGTON

INTRODUCTION

A previous interim report on the project presented a discussion of the design of the piles. Included in that report was an analysis of the soil-boring report from Shannon & Wilson* with the view of characterizing the soil from the standpoint of design. A brief discussion of the installation of the piles was included previously; a more complete discussion is included herein. All of the piles are assumed to consist of open, steel tubes. The wall thickness may vary with the computed value of combined stress and is assumed to be constant for these studies.

The material presented herein should be useful even though the loading and environmental conditions may change after Anvil gains additional information on being authorized to make further studies.

The principal thrust of the work reported herein is to develop information on the size of hammer necessary to install the piles at the site. The wave-equation method was used as the primary tool, taking into consideration a number of special conditions at the site. The general technique was (1) to select representative piles and the appropriate stratigraphy and soil properties; (2) to compute by static methods the penetration of the pile required to sustain the expected loading; and (3) to use the computer code to select a particular impact hammer to install the pile. The large-diameter breasting dolphins constituted a special case.

SOIL CONDITIONS

The soil borings revealed that the soils that will support the foundations are almost altogether granular. Only a few samples could be taken with push tubes and no significant strata of cohesive soils were found to exist. The granular soils consist predominantly of sands with some gravel. The relative density of the soil is in the range of loose to medium near the mudline with increasing density with depth. At a depth of about 60 to 80 feet below the mudline, the granular soils become very dense where refusal may be expected in the driving of piles.

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* Shannon and Wilson, Inc. *Geotechnical Data Report ARCO Dock Completion Project Cherry Point, Washington.* Seattle, Washington, October 1993.

The split spoon used in the Standard Penetration Test has an inside diameter of 1.375 inches; therefore, only gravel of relatively small sizes could be recovered. However, there was no indication of thick beds of gravel or cobbles that could have prevented the penetration of piles during impact driving.

The specific soil conditions, selected for the studies of the required pile hammers, are given in the following table.

Depth below mudline, ft	N-value, blows/ft	Description
5	10	medium dense silty sand
20	30	medium dense to dense silty sand
38	40	very dense fine to coarse sand with gravels
52	60~100	very dense silty sand with gravels
80	>100	very dense silty sand with gravels

LOADINGS OF TYPICAL PILES IN STRUCTURE

The loadings that were used in the studies were provided by Anvil and are shown below.

Anticipated rage of ultimate loadings on piles:

Axial		
	compressive	300k to 500K
	uplift	nominal
Lateral		40K to 100K

Anticipated range of ultimate loading on caissons (large diameter piles):

Axial		
	compressive	700K to 2700K
	uplift	nominal
Lateral		200K to 400K

Mooring Dolphins

Axial		
	compressive	nominal
	uplift	nominal
Lateral		200K to 600K

Breasting Dolphins

Axial		
	compressive	nominal
	uplift	200K
Lateral		500K to 1000K

It is of interest to note that none of the piles is subjected to significant uplift loading; therefore, the penetration for capacity in compression will be fairly nominal because the unit values in end bearing will be large.

RESULTS OF STUDIES FOR INSTALLATION OF PILES

As will be noted in the presentations that follow, the analysis of driveability are based on the use of impact hammers. A computer code, using the wave-equation method, was employed in the analyses.

Large-sized vibratory hammers are now available and were investigated for the installation of breasting dolphins. However, no currently acceptable analytical technique is available for analyzing the behavior of the piles driven by a vibratory hammer. Dependence was placed on the experience of the suppliers of the hammers. While it was judged that the largest vibratory hammers owned by U.S. companies would not be able to install the 10-foot diameter breasting dolphins, these hammers should be able to install the piles for the trestle and the ship-loader.

In regard to vibratory hammers, which should perform well in the granular soil at the site, there is a possibility that even larger hammers than the ones from U.S. companies may be available from foreign suppliers. The possibility will continue to be investigated.

The wall thicknesses selected for these studies were only for preliminary analysis, principally to investigate the driveability of the piles. The wall thicknesses can be selected for the final design to conform to computed stresses. For a given pile, three or four different thicknesses can be specified for the section along the length of the pile with the outside diameter remaining uniform. The computations on driveability will be affected, of course, but the results presented herein are not expected to change much.

Trestle Piles

The pile analyzed had a diameter of 30 in. and a wall thickness of 1 inch. The free weight of the 120-ft pile was computed to be 37 kips. The penetration necessary to sustain a load of 500 kips was computed to be 50 feet.

The wave-equation studies showed that a Vulcan 520 hammer was needed to drive the pile to the indicated depth. The blows per foot at the end of the driving was computed to be 20. The hammer has a rated energy of 100,000 ft-lbs and is assumed to operate at an efficiency of 63 percent.

The efficiency of 63 percent may be somewhat conservative. However, that value is in the range of many observation in the field and is suitable for the selection of a hammer. If the efficiency during the final driving of a pile is higher, the blows per foot of driving will be smaller than given here.

Ship-Loader Piles (with clarification by LCR & A)

Based on the preliminary load ranges, the pile analyzed had a diameter of 48 inches and a wall thickness of 1 inch. The free weight of the 150-ft pile was computed to be 75 kips. The penetration necessary to sustain a load of 2700 kips was computed to be 70 feet. It should be noted that the ship-loader piles may need to be increased in diameter and penetration of when the final design conditions are established.

The wave-equation studies showed that a Vulcan 560 hammer was needed to drive the pile to the indicated depth. The blows per foot at the end of the driving was computed to be 60. The hammer has a rated energy of 300,000 ft-lbs and is assumed to operate at an efficiency of 63 percent.

Breasting Dolphins

As noted in our earlier interim report, the design of a breasting dolphin is usually done by considering the energy of the docking vessel. The data for such an approach is currently unavailable; therefore, the loadings given earlier were used in the studies.

For a lateral load of 1000 kips, the studies in our interim report showed that the 10-foot-diameter dolphin would need a penetration below the mudline of 80 feet or more. Taking into account the depth of the water at the front of the pier, the stick-up of the dolphin, and the penetration, the pile should have a total length of 170 feet. Assuming a wall thickness of 2 inches, the weight of the pile in air would be approximately 429 kips.

Similar studies were also conducted for a lateral load of 500 kips. The pile with a diameter of 8 ft and wall thickness of 2 inches will still have 19-in predicted deflection at the top

of the pile and 2.5-in predicted deflection at the seabed. The penetration below the mudline was computed to be 80 feet.

Installation of 10-foot-diameter Pile by Impact Hammer

The wave-equation studies showed that a Vulcan 6300 hammer will be needed to drive the pile to the indicated depth. The blows per foot at the end of the driving was computed to be 25. The hammer has a rated energy of 1,800,000 ft-lbs and operating efficiency was assumed to be 63 per cent.

The Vulcan hammer in the above studies is one that is used for driving piles at offshore locations and that, or a similar hammer, can be brought to the site by a barge. However, the expense could be considerable.

Installation of 10-foot-diameter Pile by Impact Hammer and Internal Jetting

With some initial fabrication and rigging, the jetting can be done simultaneously with pile driving. A properly-designed jet ring with a series of jets can be installed near the tip of the pile and water supplied by pipes running down the inside of the breasting dolphins. Then, water can be pumped to these lines. A swivel in the system could allow the necessary flexibility.

The tip resistance for the pile at 80-ft below the seabed is approximately 4700 kips. With jetting as noted, only 10% of the 4700 kips is assumed to be retained during jetting. The total penetration resistance will be about 4168 kips which is the sum of the side resistance and the retained tip resistance. The Vulcan 560 hammer which has a rated energy of 300 ft-kips seems to be able to drive the pile to the designated depth with the help of the inside jetting. The blows per foot at the end of the driving was computed to be 49.

Installation of 8-ft-diameter Pile by Impact Hammer

The wave equation studies showed that a Vulcan 5150 hammer was needed to drive the pile into the indicated depth. The blows per foot at the end of the driving was computed to be 43. The hammer has a rated energy of 750,000 ft-lbs and is assumed to operate at an efficiency of 63 percent.

Installation of 8-ft-diameter Pile by Impact Hammer and Internal Jetting

The tip resistance for the 8-ft diameter pile at 80-ft below the seabed is approximately 3740 kips. If only 10% of the 3740 kips will be retained during jetting, the total penetration resistance will be about 3330 kips which is the sum of the side resistance and the retained tip resistance. Computations with the Vulcan 560 hammer, with a rated energy of 300 ft-kips, showed that the pile could be driven to the designated depth with the help from the inside jetting. The blows per foot at the end of the driving was computed to be 38.

Installation of the 10-foot-diameter Pile with Vibratory Hammer

Vibratory hammers have been used successfully for the installation of piles, particularly when driving into sands. Some large vibratory hammers have appeared recently in the industry and letters were written to two companies.

Mississippi Valley Equipment Company
St. Louis, Missouri
Telephone: 800-325-8001
Telefax: 314-869-6862
Reference: Mr. Karl Bogle
Hammer: MKT V-140

International Construction Equipment Company
Matthews, North Carolina
Telephone: 800-438-9281
Telefax: 704-521-6448
Reference: Mr. Scott Morris
Hammer: ICE 1412B

We included in our letters a soil profile, the estimated size of the pile, and the depth of penetration. Both companies thought that the pile was too heavy to be driven by their largest vibratory hammer. However, as noted above, larger hammers may be available form foreign companies.

Alternate Methods

Predrilling and grouting. This is a technique used for the installation of offshore piles and can be used at the site. The expense of bringing a drilling machine and the other equipment to the site will be significant, however.

Re-design. If the final design computations reveal a confirmation of the diameter and penetration in these initial studies, consideration can be given to other configurations of the dolphins. The use of a group of smaller sized piles could be considered, driven vertically to take advantage of the energy absorption due to deflection, and spaced in plan to accommodate the ship operations. The penetration will be less because of the smaller loads and diameters.

CONCLUDING COMMENTS

The computations reported herein are principally for the purpose of planning. It is understood that Anvil will be making additional studies of a number of features concerning the final design. The pile loadings, and their resulting dimensions, can well be affected by these further studies.

The earlier report noted that a large deflection at the mudline of a breasting dolphin will cause particles of uncemented sand to collapse and prevent the return to the original position. This phenomenon was observed during the performance of full-scale field studies. Thus, there will be some permanent deflection as a result of repeated dockings. The "locking-over" of a pile will be more severe for a large diameter pile because the deflection, for a given percentage of the total load, is a function of the diameter.

A detailed survey of the existing structure to learn if some of the piles have undergone permanent deformation will be useful. Such studies should also make observations about the existing scour, because the penetration of a pile to sustain load must consider the expected mudline scour over the life of the structure.