# GENERAL FOUNDATION INVESTIGATION FOR THE PROPOSED NEW MILLER JUNIOR HIGH SCHOOL ABERDEEN SCHOOL DISTRICT #5 ABERDEEN, WASHINGTON

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J-304

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# GENERAL FOUNDATION INVESTIGATION FOR THE PROPOSED NEW MILLER JUNIOR HIGH SCHOOL ABERDEEN SCHOOL DISTRICT #5 ABERDEEN, WASHINGTON

## INTRODUCTION

This report presents the results of a general foundation investigation for the proposed new Miller Junior High School to be constructed south of Pioneer Park in Aberdeen, Washington. The purpose of this study was to establish design criteria and general foundation recommendations relating to construction of the proposed facility at the subject site. In addition, laboratory analyses have been made of groundwater samples obtained at the site. Scope of the work included field explorations, laboratory tests, and engineering studies.

This investigation has been conducted as part of a general foundation investigation for the proposed new Miller Junior High School as outlined in our proposal dated April 9, 1976. Results of the portion of the general foundation investigation regarding combustible gas emissions have been presented in a separate report dated June 11, 1976.

Field explorations consisted of five Dutch cone penetrometer probes, two borings, and eight shallow gas and groundwater sampling probes located as shown on the Site Plan, Figure 1. Field procedures and results for the combustible gas measurement and sampling have been presented in our previous report. Field exploration procedures and results of the borings, probes and groundwater sampling are presented in Appendix A and interpretive boring logs and penetrometer logs are presented on Figures A-1 through A-8.

Laboratory tests consisted of an analysis of the groundwater samples obtained, water content determinations, Atterberg limits, unconfined compresion, direct shear, and consolidation tests on selected samples retrieved from the borings. The results of the laboratory tests are presented in Appendix B.

Engineering studies were made to arrive at conclusions and recommendations for foundation design and construction, including the allowable load and estimated settlement of timber pile foundations, lateral load resistance of foundations, pile installation and other construction considerations anticipated as a result of this study.

## PROJECT AND SITE DESCRIPTION

The site of the proposed Miller Junior High School is located immediately south of the existing Pioneer Park and railroad tracks, east of Evans Street, and north of Harriman Street in Aberdeen, Washington, as shown on the Site Plan, Figure 1.

Based on discussion and preliminary site plans provided by Mr. Robert F. Street of Street & Lundgren, A.I.A., and Mr. Ray Chalker of Chalker Engineers, Inc., it is our understanding that the proposed Miller Junior High School will be a one-story wood structure having the approximate limits shown on the Site Plan, Figure 1, and a timber pile foundation with sturcturally supported floor. It is anticipated that the total dead load and long term live load per square foot of floor space will be approximately 120 pounds per square foot if a concrete floor is utilized or 50 pounds per square foot if a wooden floor is selected. To support these loads, timber pile foundations will be spaced on a grid having dimensions ranging from approximately 10 to 25 feet. It is also our understanding that the only significant new loads on the subsurface soils will be those resulting from the structure itself and that no significant amount of additional fill materials will be placed at the site.

The site consists of an essentially flat, open field with only grass or weed vegetation. Results of the two test borings and

five Dutch cone penetrometer probes at the site indicate the presence of the following subsurface materials which are described in their general order of occurrence with depth below the ground surface:

WOOD WASTE FILL

Loose sawdust, wood chips, wood fragments, rock fragments and other debris. This material was encountered to depths of 6½ feet in Boring B-1 and 8½ feet in Boring B-2 and has been interpreted to exist to depths ranging from 5 to 10 feet in the five Dutch cone probes. The loose and variable nature and anticipated continuing decomposition of this material indicates that it is unsuitable for support of foundation loads without excessive and nonuniform settlements. As discussed in our report dated June 11, this material is also the source of combustible methane gas emissions observed at the site. In most areas of the site, this WOOD WASTE FILL is overlain by a thin cap of silty, fine sand with a thickness of approximately 1 foot.

ORGANIC SILT

Very soft to stiff, saturated, gray, clayey silt (OH-MH) to (OL-ML) with organic material and occasional fine sandy zones. This material was encountered in both borings and is interpreted from the results of all five Dutch cone probes. These materials were observed to extend from the base of the WOOD WASTE FILL to the top of the underlying DENSE SANDS which occur at depths ranging from 89 to 124 feet. These silt soils are relatively weak and quite compressible, even under relatively light loadings. The silts tend to increase slightly in strength and decrease in compressibility with depth with the most significant change occuring at a depth of approximately 43 feet. Because these materials extend to typical depths on the order of 100 feet, the use of pile foundations extended below these compressible soils to minimize settlements would be inordinately expensive for the relatively light structural loads involved. Consequently, our engineering studies have been

oriented primarily toward the use of timber pile foundations to transfer loads to sufficient depth within these ORGANIC SILT soils to reduce settlements to acceptable values.

VERY DENSE SAND

Very dense, saturated, gray, silty, gravelly, fine to coarse sand (SM). These soils were encountered at a depth of approximately 89 feet in Boring B-2 and were interpreted from the Dutch cone probes to exist below depths ranging from 95 to 124 feet. Boring B-1 was completed to a depth of 70½ feet and did not encounter these DENSE SAND soils. The results of the Dutch cone probes indicate that these DENSE SANDS may be overlain by up to six feet of sand of lower density. Although these soils would provide good support for foundations, the long piles that would be required to reach these materials may not be economical for the light loads anticipated at the site.

Observations in the eight gas probes at this site indicate a groundwater level ranging from 1.3 to 3.6 feet below the ground surface. It is anticipated that some variation in groundwater level may result from tidal fluctuation.

ENGINEERING STUDIES: CONCLUSIONS AND RECOMMENDATIONS

Results of our studies at the subject site indicate that the proposed building location is underlain by WOOD WASTE FILL having a typical thickness of approximately 6 or 7 feet overlying compressible ORGANIC SILT soils which extend to depths of 89 feet or more. Although DENSE SANDS underlying the deep compressible deposits could provide good foundation support with negligible settlement, adequate foundation support with moderate settlements for the light building loads anticipated could also be provided by 50- to 75-foot long friction piles in the ORGANIC SILT. The placement of even small amounts of additional fill materials at the site would

result in significant additional settlement with detrimental downdrag loads on pile foundations. Detailed conclusions and recommendations regarding foundation design and construction are discussed in the following sections.

## Shallow Foundations

Preliminary evaluation of suitable foundation types for the proposed building included use of shallow support such as spread footing or raft foundations. The high compressibility and large thickness of the ORGANIC SILT soils, however, would result in excessively large settlements even for the relatively light building loads involved. Precompression of the soft soils by placement of a preload and surcharge fill for periods on the order of 4 to 6 months prior to construction would reduce settlement in the ORGANIC SILT to acceptable values.

Use of a preload fill, however, would not have much effect in reducing long-term settlements resulting from organic decomposition of the WOOD WASTE FILL existing at the site. In addition, settlements of these materials are likely to be highly variable across the site due to variations in the character and rate of decomposition of the fill. Excavation below the fill for placement of spread footings or structural backfill to support footings could be performed, but the cost of this type of operation below ground water level and the additional need for preloading suggest that pile foundations should provide a more economical solution.

## Pile Foundations and Settlements

Recommended allowable vertical load capacity for downward loads for 8-inch minimum tip diameter timber piles are presented on Figure 2. Vertical pile capacities have been derived from static equations using a factor of safety equal to 2.5, generally accepted engineering practice. These allowable vertical capacities are for total dead plus live load and may be increased by up to 1/3 to

accommodate the transient portion only of loads that include wind or seismic forces. A minimum penetration depth of 50 feet below the existing ground surface is recommended for all piles.

Because of the large anticipated pile spacings on the order of 10 to 25 feet, we have analyzed settlements for the proposed construction based on the total dead load plus long-term live load per square foot of building area that will be transferred to the pile foundations. It is our understanding that such building loads for the proposed structure with either wood or concrete flooring will be on the order of 50 and 120 psf, respectively.

Using this approach estimated settlement of foundations for the anticipated loads are related on Figure 2 to the penetration depth of the piles. From this figure it can be seen that, although individual pile capacities increase with penetration depth, the estimated settlement decreases as long as the building load per square foot remains the same and is uniformly distributed to all piles. The estimated settlements relate, therefore, to the long term building load per square foot and the depth to which those loads are transferred by the piles. The total dead load plus live load on each pile and the pile spacings do not affect the settlement estimates significantly provided the allowable (DL + LL) pile capacities of Figure 2 are not exceeded and piles are spaced no closer than 8 feet, center-to-center.

Resistance to Lateral Loads

Although a concrete floor slab or pile caps may be placed in direct contact with the subgrade soils at the site, anticipated future settlement of the WOOD WASTE FILL underlying the building may result in a loss of contact between these pile-supported structural elements and the ground surface. For this reason, it is recommended that design for lateral loads be based on the lateral resistance of the pile foundations rather than a coefficient of friction between concrete and the surface soils.

Lateral resistance and deflection of pile foundations are governed primarily by materials existing near the ground surface. Consequently, the high variability and relatively low stiffness of the existing WOOD WASTE FILL at the site will generally govern the deflection characteristics of the timber piles under lateral loads. Selection of a lateral modulus for this type of material without lateral bearing tests in the field is difficult and somewhat speculative. Using a modulus value characteristic of loose soils, however, estimated horizontal deflection at the top of the pile and bending moments for laterally loaded, 12-inch butt diameter, timber piles fixed from rotation at the ground surface by structural restraints are as follows:

Horizontal Deflection (per kip of lateral load)

1) At top of pile
   0.08 inches/kip
2) At depth of approximately 15 feet
   None

Bending Moments (per kip of lateral load)

1) At top of pile (i.e., point of fixity by structure at ground surface)
   -4 kip-ft/kip
2) At approximately 10-foot depth
   +1.2 kip-ft/kip

These deflection and moment values are our best estimate of the values that are likely to develop under applied long-term or transient loads. No modification for factor of safety or transient loading conditions have been made. The actual deflections and bending moments that will occur could vary significantly from the above estimates as a result of variations in the WOOD WASTE FILL and the limitations associated with our assumption of a lateral modulus for this material.

Pile Installation

Because of the large pile spacing, it is not anticipated that heave of soil or piles during driving will be a significant problem. None

of the wood fragments or other debris encountered within the WOOD WASTE FILL in the borings or probes was large enough to cause major problems in pile driving; however, larger fragments may be encountered locally during construction requiring removal so that driving can proceed without damage to the piles.

We recommend that timber piles be driven with a steam or diesel hammer having a rated energy of not less than 15,000 foot-pounds or more than 26,000 foot-pounds.

Appropriate pile driving formulas commonly used to determine pile capacity based on driving resistance have been correlated for use in granular soils. Engineering experience and research have shown that fine-grained soils, such as the ORGANIC SILT at the subject site, often become remolded at the pile surface during driving resulting in a temporary strength decrease. As time passes after remolding, however, a fine-grained soil usually regains most of its original shear strength. Because of this remolding effect in fine-grained soils, pile resistance during re-driving after some time has elapsed is usually greater than that observed during initial driving.

It is anticipated, therefore, that initial driving resistance of piles at the site may not indicate a pile bearing capacity at a given penetration depth equal to the values represented on Figure 2. In spite of these limitations, we recommend that all pile driving operations be inspected and continous driving resistance records be kept under the supervision of a qualified soils engineer familiar with the site conditions in order that some means of verifying design capacities and construction control be available. Piles should be driven initially to the design tip elevation based on Figure 2 and the capacity determined using the penetration resistance at that elevation and the following $S_o$ formula.

$$Q = \frac{a\ E_r}{S + \frac{1}{2}S_o}$$

where

$$S_o = \left[\frac{2 \cdot a \cdot E_r \cdot L}{A \cdot E}\right]^{\frac{1}{2}}$$

and $Q$ = ultimate pile capacity, pounds,

$E_r$ = rated hammer energy, foot-pounds,

$a$ = hammer efficiency = $\frac{\text{delivered energy}}{\text{rated energy}}$ (about 0.8),

$A$ = cross sectional area of pile tip, inches$^2$,

$L$ = length of pile, feet,

$E$ = modulus of elasticity of pile material, psi,

$S$ = final set, penetration per blow, feet.

Should the pile capacity computed on this basis indicate a factor of safety less than 2.0 for the first few piles driven, pile capacities should be evaluated further by re-driving these piles after a reasonable set-up time has elapsed. If re-driving does not indicate an increase in pile driving resistance to the factor of safety of 2.0, we should be advised immediately in order that additional evaluation and recommendations can be provided. To avoid unnecessary construction costs, piles not meeting the driving resistance criteria and factor of safety should <u>not</u> be driven below design tip elevation unless our evaluation indicates that such additional pile length is required for adequate foundation support.

## Other Settlement Considerations

Settlement estimates presented above do not include any settlement or downdrag effects on piles that would result from the placement of additional fill materials or re-grading of existing materials at the site. It should be emphasized that the placement of any additional weight on the ground surface at the site, even apparently minor amounts of additional fill, could result in a significant increase in settlement of both pile foundations due to downdrag and the ground surface.

For example, a one-foot thickness of additional fill would impose an additional load on the foundation soils on the order of 120 psf

which is as great as the entire dead load of the building for the heavier concrete floor construction. Settlement resulting from one foot of additional fill would be even greater than that resulting from the building load since the fill load would not be entirely transmitted to the greater depth like the structural loads.

If any additional loads greater than 40 psf are to be placed within 100 feet horizontally of the proposed building, we should be advised in order that we may evaluate the effects of such loads and make additional recommendations, as required.

Groundwater and Leachate Considerations

As stated previously, groundwater was observed at levels ranging from 1.3 to 3.6 feet below the surface of the wood chip fill. Additionally, seepage was noted along the sides of the fill emptying into perimeter ditches filled with water. These observations suggest that low groundwater gradients exist from the center of the fill (elevation +15 feet) towards the outer edges (elevation +13 feet). In our opinion, incident rainfall percolates into the fill and moves laterally down gradient to the outer edges where seepage was observed. The amount of seepage probably varies seasonally with rainfall.

The production of leachate is dependent on the reactions between rainwater, groundwater and the material contained in the fill. Because of the inferred existing flow system, leachate could be expected to follow the same flow paths to the outer edges of the fill into the ditches. At the time of our investigation seepage was observed along the north and west sides of the fill but no quantities were estimated in that in some places seepage appeared as wet portions below a seepage line. Water was flowing in the ditch to the north at probably less than one gallon per minute.

Samples of groundwater were taken from the gas probes (described in Appendix A) and analyzed for bacterial and chemical content by

Laucks Laboratories Incorporated, Seattle, Washington. Their report is contained in total within Appendix B. All samples were tested for pH, specific conductance which is a qualitative indicator of dissolved solids, C.O.D. and B.O.D.

Bacteriological analyses were performed on probes N.E. and S.W. for total bacterial, total coliform, fecal coliform, and fecal strep.

The test results are compared to a recently completed study in the Grays Harbor area on the production and analysis of wood waste leachate.[1] The study employed laboratory and field analysis to determine the production and chemical characteristics of aqueous solution of wood extractions commonly referred to as leachate. Briefly, they have summarized the characteristic effects of the woodwaste environments and leachate as having observable growth of bacteria apparently feeding upon the wood sugars that are contained in the leachate. The leachate has an oxygen demand illustrated by high C.O.D. and B.O.D. values, a depression in the D.O. of the receiving water and the reduction of iron (III) in rocks on a stream bottom to green iron (II). Carboxylic and phenolic compounds in the leachate cause an increased acidity (low pH) in receiving waters and the leachate contains materials toxic to organisms.

Specifically, the analyses performed on the leachate samples obtained at the proposed school site compare to those reported by Shermer & Phipps in these categories.

| | |
|---|---|
| pH | All samples tested ranged from 4.6 to 6.1 probably caused by the carboxylic and phenolic compound in the leachate. |
| Specific Conductance | A semi-quantitative analysis for dissolved solids indicating a range of 190 to 260 ppm of dissolved solids in the tested samples. |

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[1] Shermer, G. D., and Phipps, James B., "A Study of Woodwaste Leachate," completed under Washington State Dept. of Ecology Contract 75-054 and 76-046 of Choker Research, Grays Harbor College, Aberdeen, Washington, 1976.

| | |
|---|---|
| Chemical Oxygen Demand (C.O.D.) | Indicator of the oxygen need in the chemical breakdown of wood waste leachate. Values from Gas Probe GP-1 through GP-4 varied from 360 to 580 mg/l. Although Shermer & Phipps have reported C.O.D. values on the order of 13,000 mg/l during an initial 10-day period of leachate production from a controlled field test site, they do indicate a C.O.D. value near 400 mg/l is typical for leachate from woodwaste fills that are one to several years old. |
| Biological Oxygen Demand (B.O.D.) | Indicator of the amount of oxygen needed in the biological breakdown components of the leachate. The two values reported for GP-1 and GP-2, 350 and 250 mg/l respectively could not be quantitatively correlated with analysis presented by Shermer & Phipps because of differing reactions with different woodwaste products. |

The analyses for total, volatile, and fixed solids give a qualitative assessment of the amount of suspended, dissolved, and organic solids in the samples tested. The values indicate the majority of solids occur as non-organic material; however, a quantitative analysis of the chemical substances and quantities of the fixed solids was not made.

Bacteriological analyses were completed on leachate samples obtained from probes installed at the northeast (NE) and Southwest (SW) corners of the site. The major importance of these tests is that the high level of total bacteria can be accounted for by bacterial multiplication in a favorable growth environment, i.e., the nutrients provided by the wood chips. Total coliform bacteria have long been used as indicators of sewage pollution, although the group includes bacteria from diverse natural sources; e.g., soil, water, and vegetation. In the samples tested, NE and SW, total coliform bacteria were greater than 240,000 and 43 per 100 ml, respectively. These values as compared to total bacteria are inconclusive in that the sample from NE could represent a raw sewage source; on the other

hand, the SW sample would not in that the high total bacteria is probably a result of bacterial multiplication as explained above and not due to coliform bacteria.

Because of the close correlation between the chemical analyses of leachate from the project site and those reported by Shermer & Phipps in a recently completed, more detailed study of woodwaste leachate, the conclusions arrived at in the more comprehensive study program apply to the proposed school site. Briefly, leachate from woodwaste can be characterized as a complex mixture of water soluble wood extractives that is often highly colored, possesses an offensive odor, and causes foaming and an oil-like sheen on associated waters. There is a high oxygen demand and often a significant reduction in the pH of associated waters.

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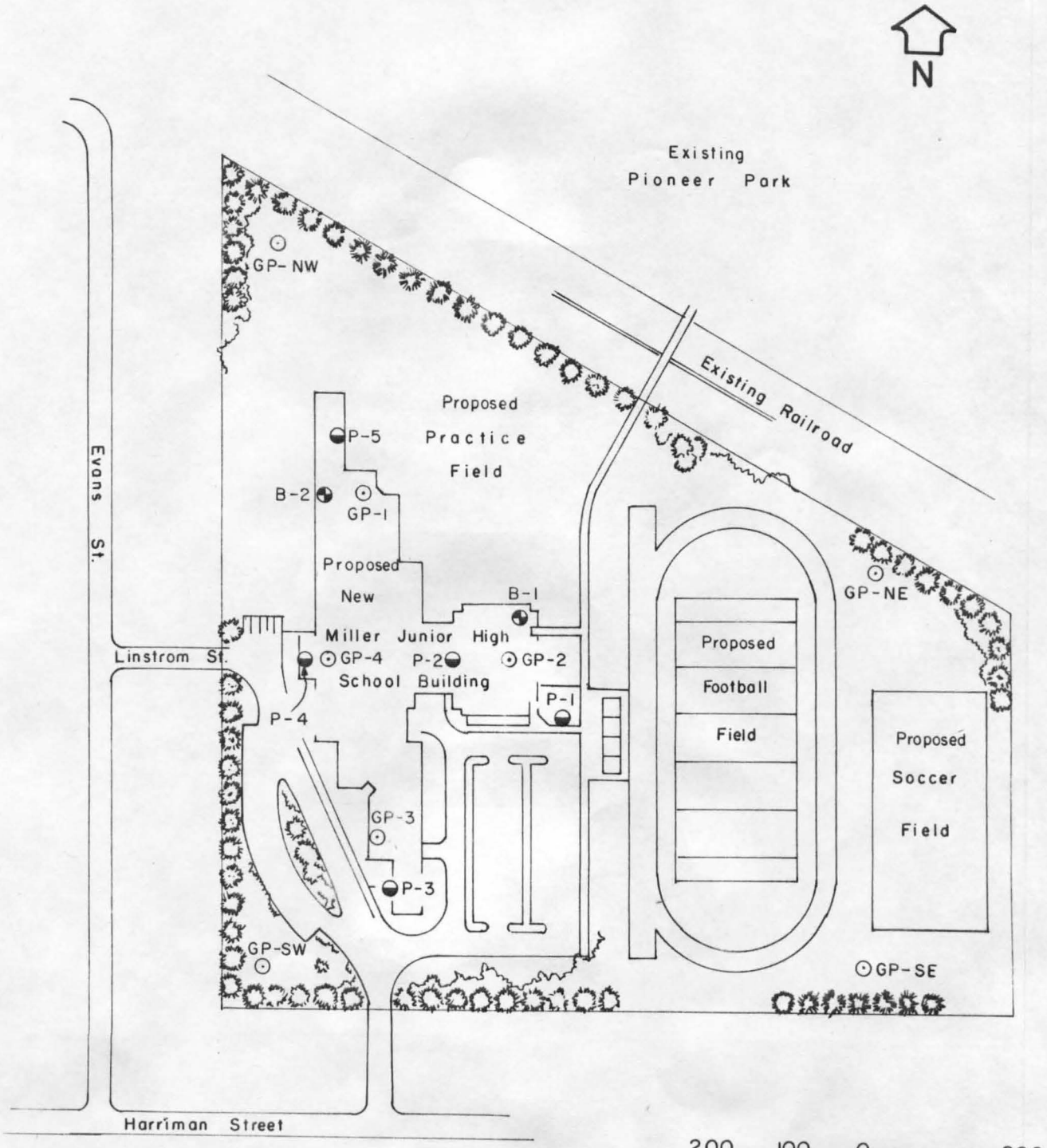
Thomas M. Gavin

THOMAS M. GAVIN
Professional Engineer

John Crowser

JOHN C. CROWSER
Professional Engineer

# SITE and EXPLORATION PLAN




LEGEND

B-1 Boring Location and Number

P-2 Dutch Cone Probe Location and Number

GP-NW Gas Probe Bore Hole Location and Letter and Number Designation

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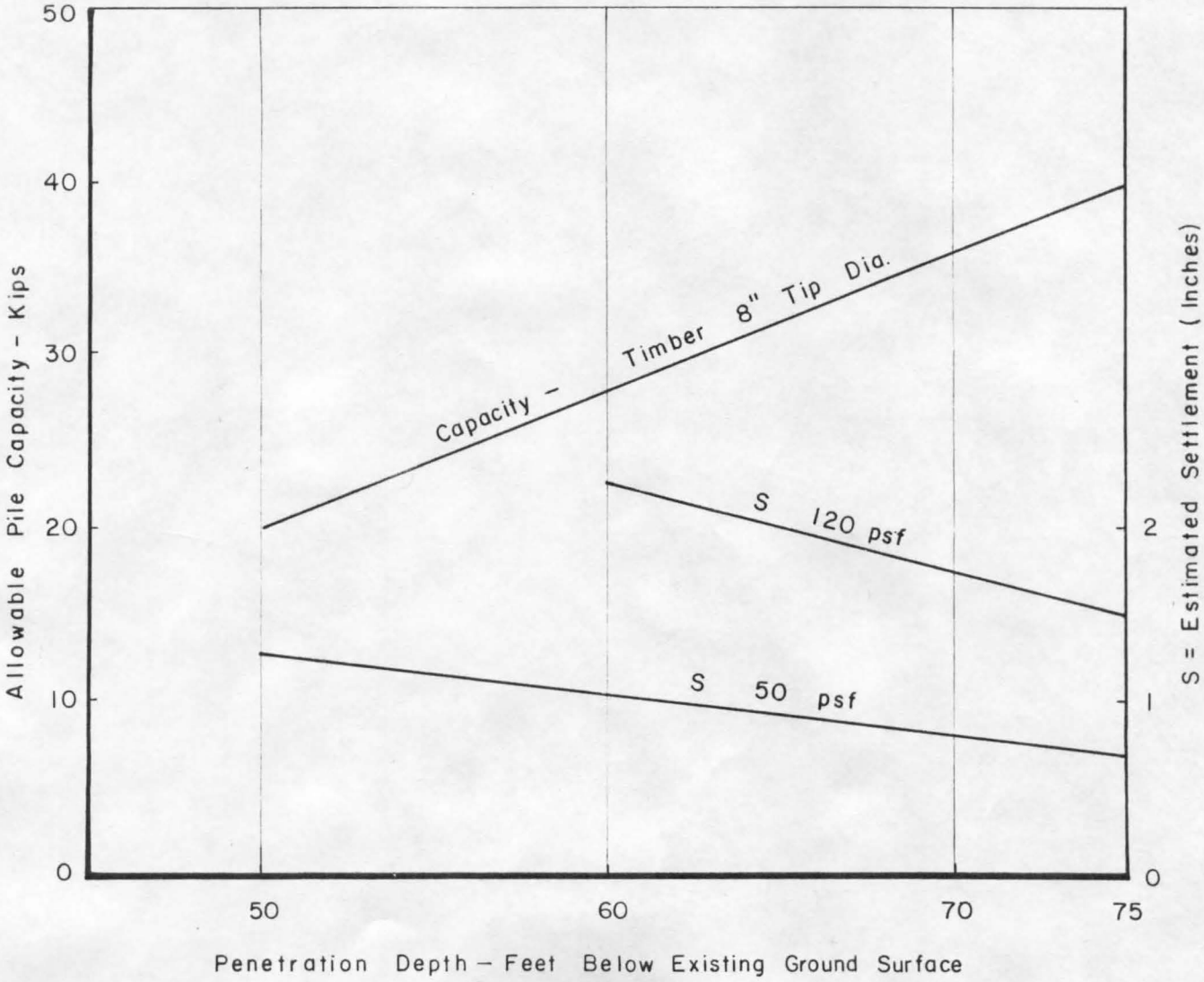
Figure 1

# VERTICAL PILE CAPACITY and ESTIMATED SETTLEMENTS




Notes: 1) 50 PSF- INDICATES TOTAL DEADLOAD PLUS LONG-TERM LIVELOADS IN POUNDS PER SQUARE FOOT OF BUILDING AREA THAT WILL BE TRANSFERRED TO PILE FOUNDATIONS.

2) ESTIMATED SETTLEMENTS ARE BASED ON A MINIMUM PILE SPACING OF 8 FEET CENTER-TO-CENTER.

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Figure 2

# APPENDIX A

## FIELD EXPLORATIONS

Field explorations at the site consisted of a general reconnaissance, two borings, five Dutch cone penetrometer probes, and the installation of eight gas and leachate sampling probes. Field techniques and results for installation of the eight sampling probes, as well as measurement and sampling of combustible gas emissions, have been described in our report dated June 11, 1976. The explorations were coordinated in the field by an engineering geologist from our firm. Mr. Roger V. LeClerc, II.

### Borings

The two borings were drilled to depths of 70½ and 104 feet on May 24 and 25, 1976, at the approximate locations shown on the Site Plan, Figure 1. The borings were made with a truck-mounted, Mobile B-61 drill rig using a 3 3/8-inch inside diameter continuous-flight, hollow-stem auger. Representative soil samples, both disturbed and relatively undisturbed, were generally obtained at 5-foot intervals. The representative but disturbed samples were obtained using the Stnadard Penetration Test procedure. This test is a means of determining the relative density of granular soil and the consistency of cohesive soil, and consists of driving a standard 2-inch O.D., split-barrel sampler a distance of 18 inches using a 140-pound hammer, free falling 30 inches. The number of blows of the hammer required to drive the sampler the last 12 inches is the Standard Penetration Resistance, shown graphically on the boring logs. All samples were classified in the field, placed in air-tight jars and transported to our laboratory for detailed examination and classification. In compressible SILT materials, several relatively undisturbed thin-walled steel tube samples were also obtained. The

ends of the tubes were sealed with caps and returned to our laboratory where they were extracted and tested. The drilling and sampling of borings were continuously observed by our engineering geologist who also prepared field boring logs on the basis of an examination of the samples secured and the excavated material, as well as the action of the drilling equipment. The Boring Logs, presented on Figures A-1 through A-3, are based on inspection of the samples secured, laboratory test results, and field boring logs.

Dutch Cone Penetrometer Probes

The five Dutch cone probes, designated P-1 through P-5 were advanced to depths ranging from 95 to 124 feet on May 7 through 13 at the approximate locations shown on the Site Plan, Figure 1. The Dutch cone used was the Begemann, type, the principles of which are shown in Figure A-4. The system is mounted on a truck which provides the necessary reaction for the applied loads.

From the results of the penetrometer probes, a direct correlation is obtained between the point resistance of the cone and bearing capacity of the soil, and between the sleeve friction and frictional characteristics of the soil. The relative density or consistency of the soil being probed is empirically related to the cone penetration resistance, $q_c$. Further, comparing the values of $q_c$, sleeve friction ($f_s$) and the friction ratio ($FR = f_s/q_c$ in percent), leads to an interpretive soil classification. The soil classification chart developed by Schmertmann for use with the Dutch cone penetrometer results is presented on Figure A-5. Generally, a friction ratio value of less than 2 indicates sand; a value between 2 and 4 indicates a silt-sand mixture, clayey sand, or silt; and values greater than 4 are indicative of a clayey silt or clay. The descriptive soil interpretations presented on the probe logs have been developed using Figure A-5 as a guideline with modifications according to correlations of soil types disclosed in other projects within Western Washington and careful evaluation of the probe results. The results of the penetrometer probes accomplished for this study and interpretive soil logs are presented in Figures A-6 through A-10.

Leachate Sampling

A sample of ground water and leachate was obtained from each of the eight shallow gas and leachate sampling probes on June 3, 1976. Samples were obtained by inserting the end of a short length of plastic tubing into the probe and lowering it to a few inches below the water level. With the other end of the tubing attached to a small hand-operated bilge pump, the fluid was drawn through the tubing by the pump and discharged into a jar for transportation to the laboratory. Each jar and the entire sampling apparatus were rinsed with distilled water prior to sampling of each probe.

# LEGEND

| SYMBOL | DESCRIPTION |
|---|---|
| ⊠ | 2.0 inch O.D. Split Spoon Sample |
| ⧅ | 3.0 inch O.D. Thin-wall Sample |
| ● | Moisture Content, Percent by Weight |
| ▲ | Standard Penetration Resistance N-value (140 pound weight, 30" drop, blows per foot) |
| ⊢—⊣ | Indicates Water Content At Plastic Limit (lower value) and Liquid Limit (higher value) |
| UCS / WW | Unconfined Compressive Strength - tons per sq. ft / Wet Unit Weight - Pounds per Cubic Foot |
| | WOOD WASTE FILL |
| | ORGANIC SILT - Clayey SILT (OH - MH) with Organic Material and Occasional Fine Sandy Zones |
| | VERY DENSE SAND |

1. Soil descriptions are interpretive and actual changes may be gradual
2. Water level, if indicated, is for date indicated and will probably vary with time of year

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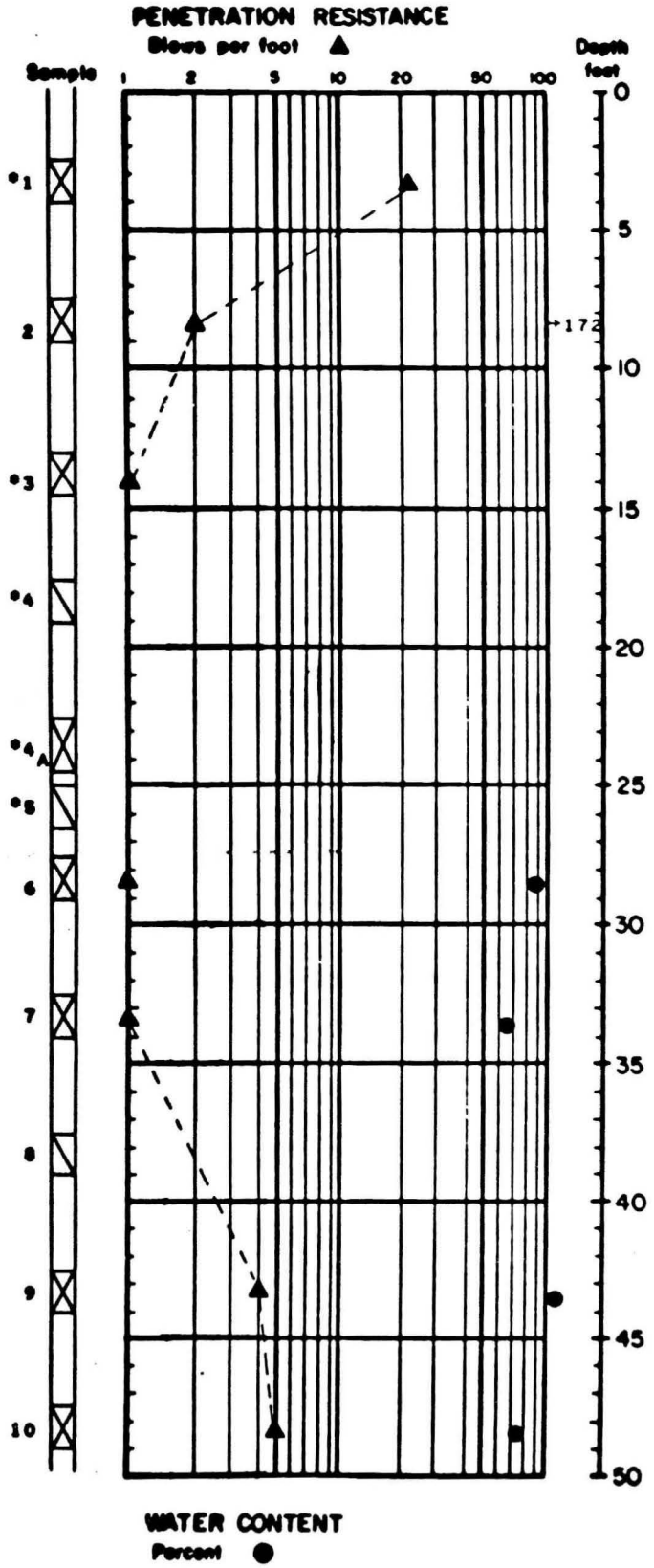
Figure A-1

# BORING LOG B-1




## SOIL INTERPRETATION

FILL — SILTY SAND

WOOD WASTE FILL

LOOSE, WET TO SATURATED, WOOD CHIPS, LARGER WOOD FRAGMENTS, ROCK FRAGMENTS, AND OTHER DEBRIS.

ORGANIC SILT

VERY SOFT, SATURATED, GRAY, CLAYEY SILT (OH-MH) WITH ORGANIC MATERIAL AND OCCASIONAL FINE SANDY ZONES.

MEDIUM STIFF

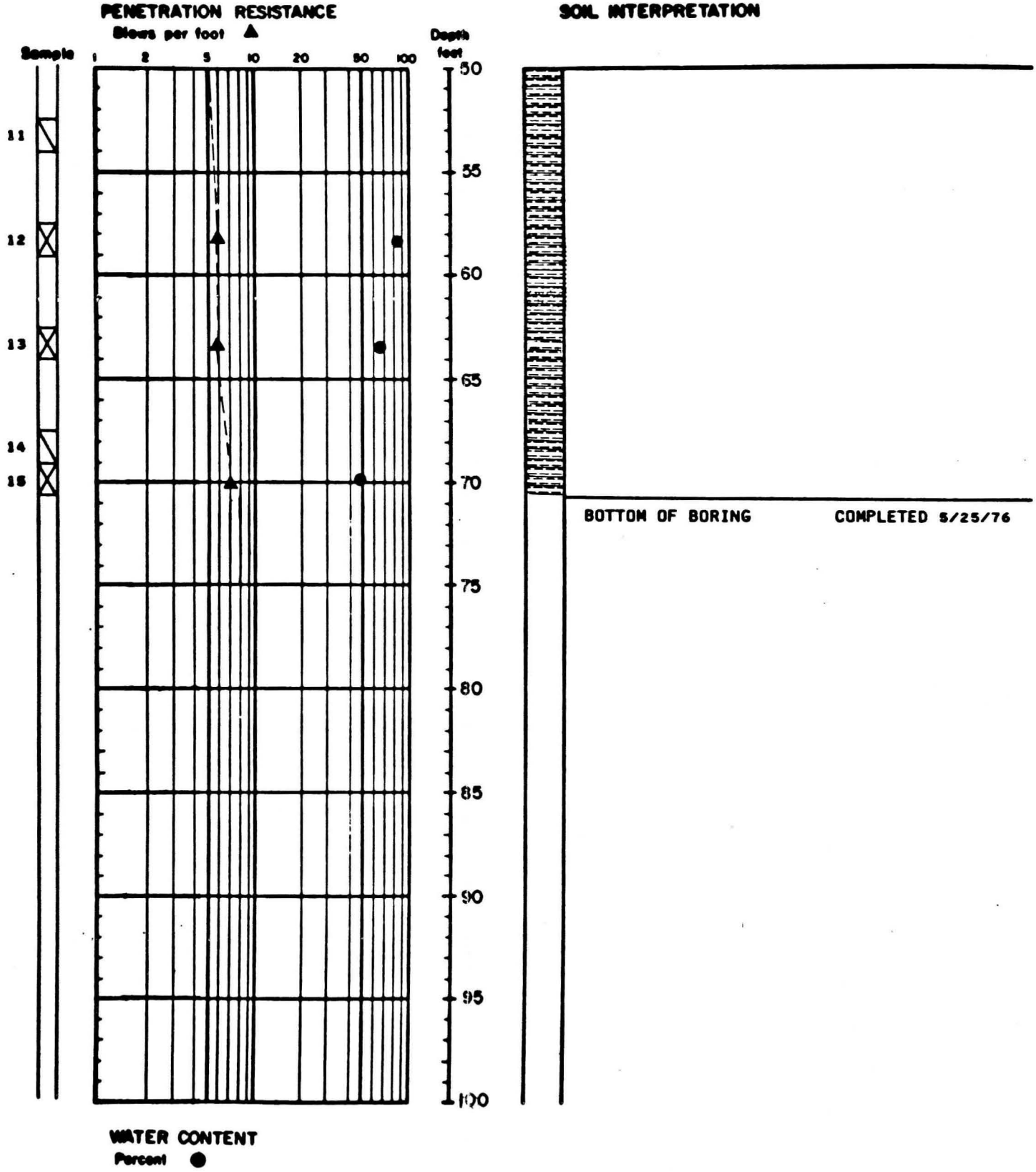



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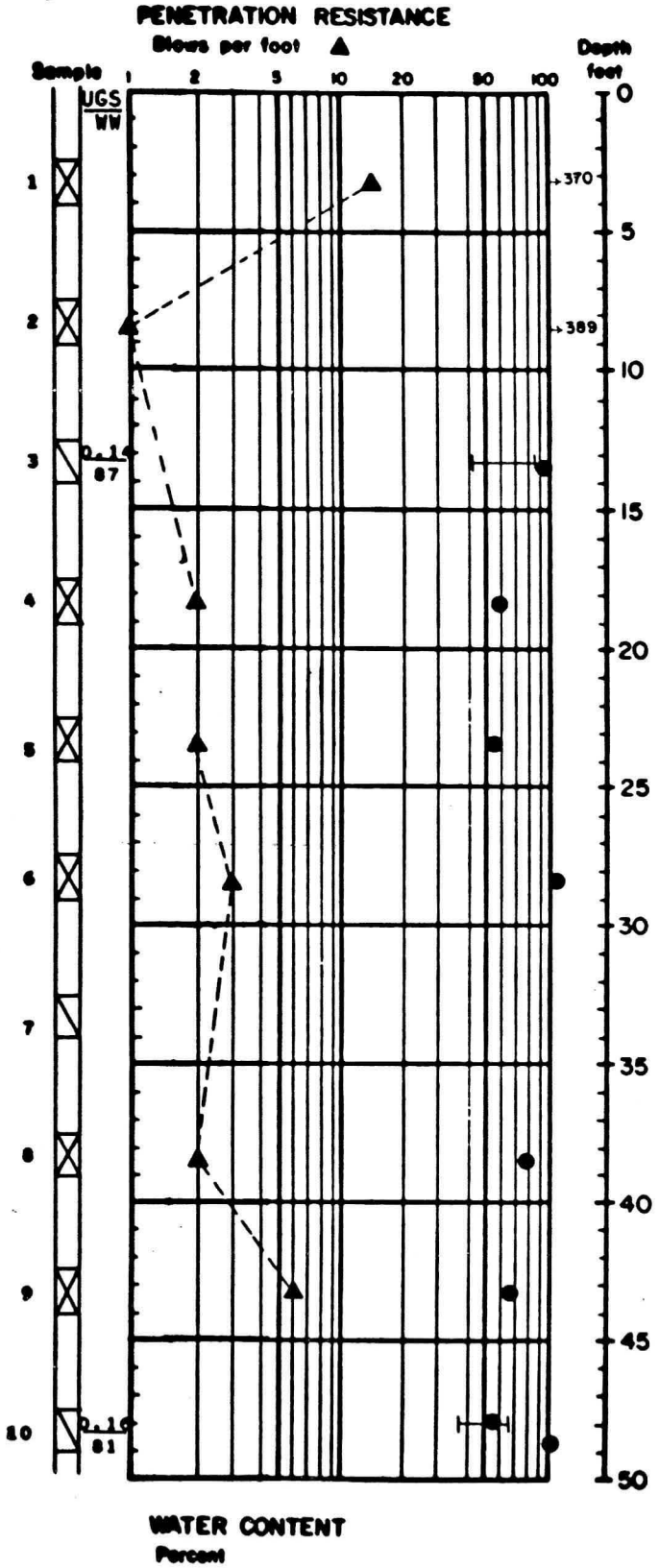
Figure A-2

# BORING LOG B-2




## SOIL INTERPRETATION

WOOD WASTE FILL

LOOSE, WET TO SATURATED, WOOD CHIPS, LARGER WOOD FRAGMENTS, ROCK FRAGMENTS, AND OTHER DEBRIS.

ORGANIC SILT

VERY SOFT, GRAY, CLAYEY SILT (OH-MH) WITH ORGANIC MATERIAL AND OCCASIONAL FINE SANDY ZONES.

MEDIUM STIFF

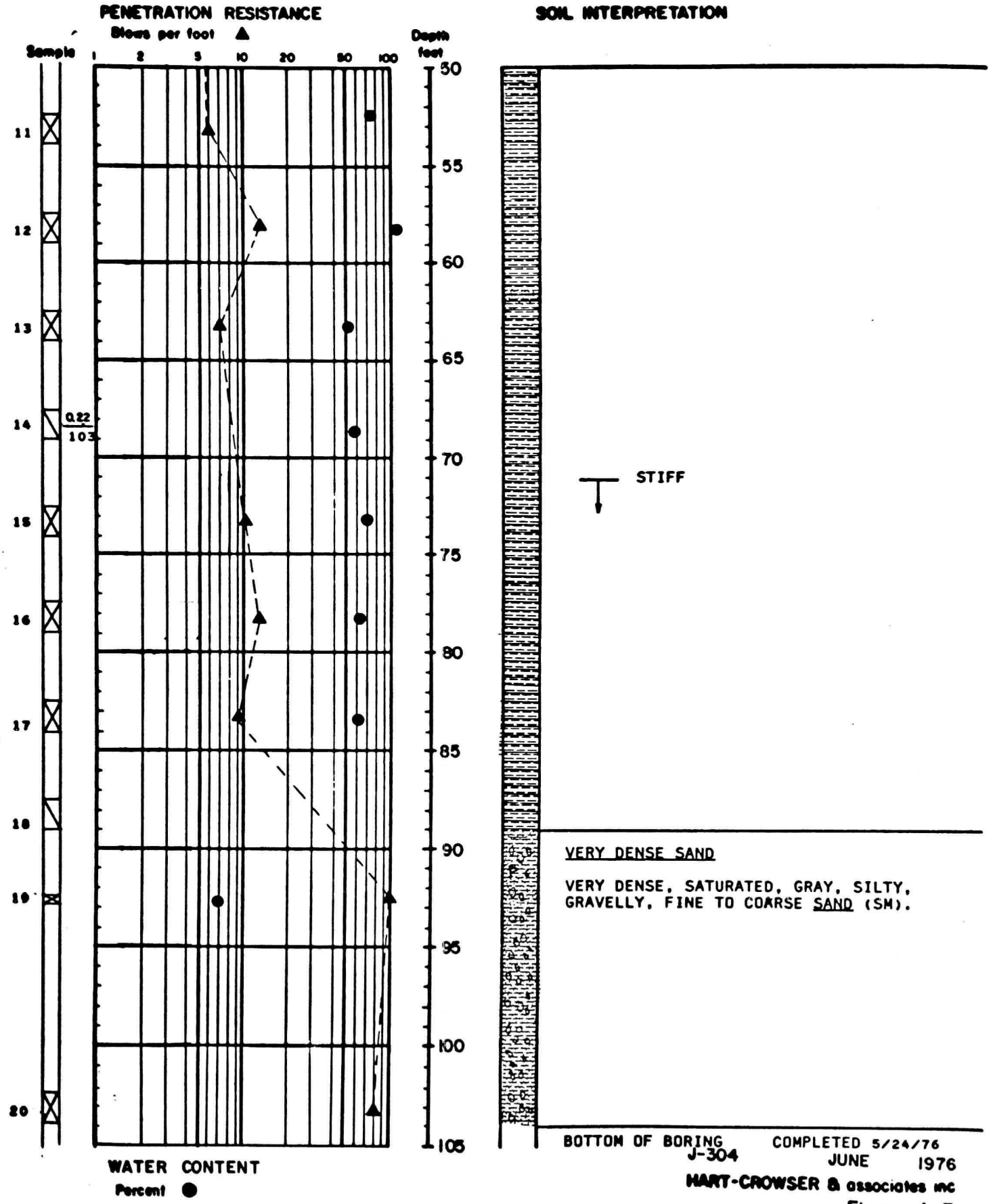

PENETRATION RESISTANCE
Blows per foot
Sample
1
2
5
10
20
50
100
Depth feet
50
55
60
65
70
75
80
85
90
95
100
105
11
12
13
14
15
16
17
18
19
20
0.22
103
WATER CONTENT
Percent
SOIL INTERPRETATION
STIFF
VERY DENSE SAND
VERY DENSE, SATURATED, GRAY, SILTY, GRAVELLY, FINE TO COARSE SAND (SM).
BOTTOM OF BORING
COMPLETED 5/24/76
J-304
JUNE 1976
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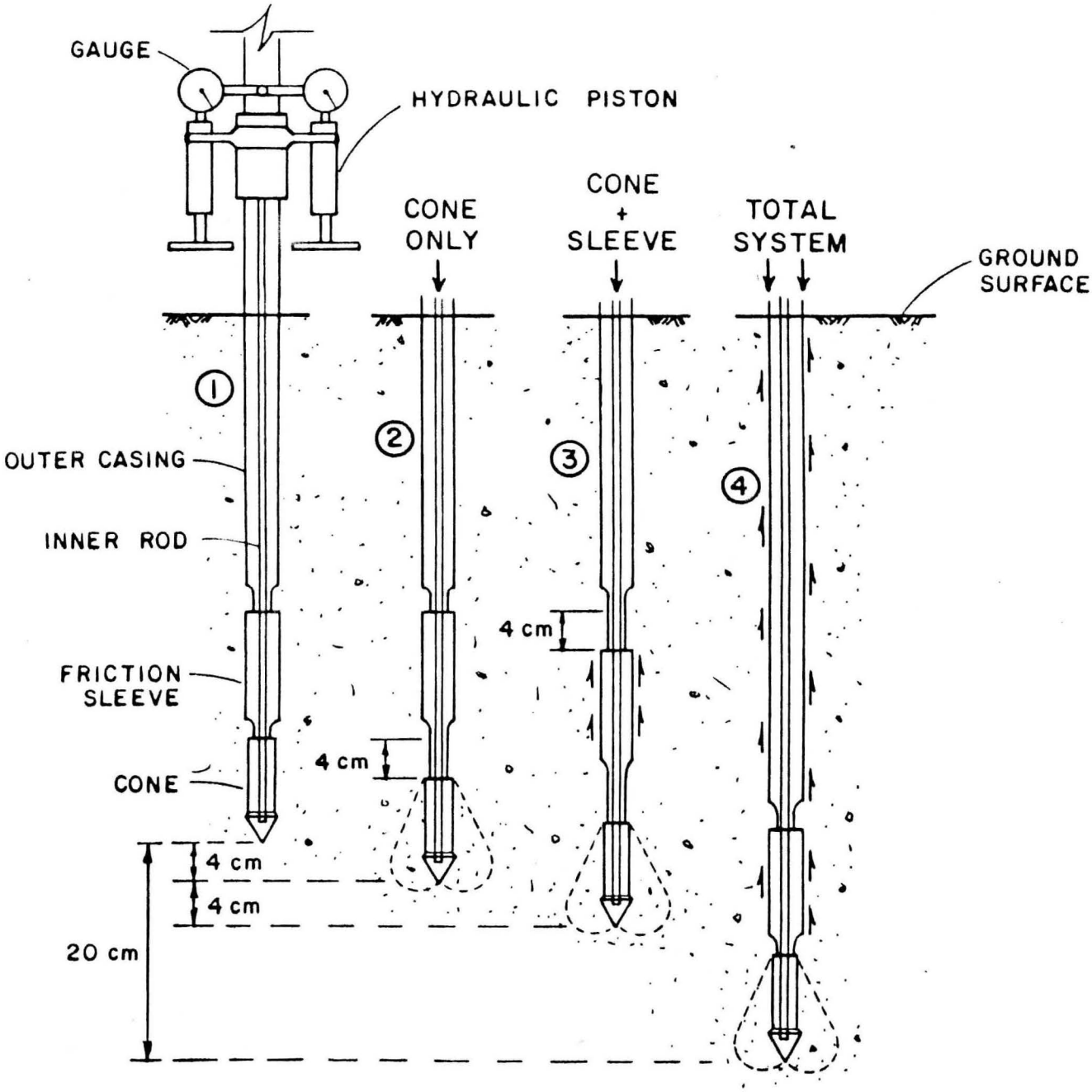
Figure A-3

# PRINCIPLE OF DUTCH CONE PENETROMETER




## PROCEDURE

1. The cone as shown in position 1 is pushed down by the inner rod for a depth of 4 cm, and the point resistance measured.
2. From position 2 the point is again pushed 4 cm and the combined point resistance and side friction is measured by causing the sleeve to be lowered as shown in position 3.
3. The total system is then advanced 20 cm by pushing the outer casing to position 4 from which the next test is accomplished.

## NOTES:

1. The cone apex angle = 60°
   section area = 10 cm
2. Length of friction sleeve = 13 cm
   diameter = 36 mm

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Figure A-4

# SOIL CLASSIFICATION CHART
## (Schmertmann 1969)

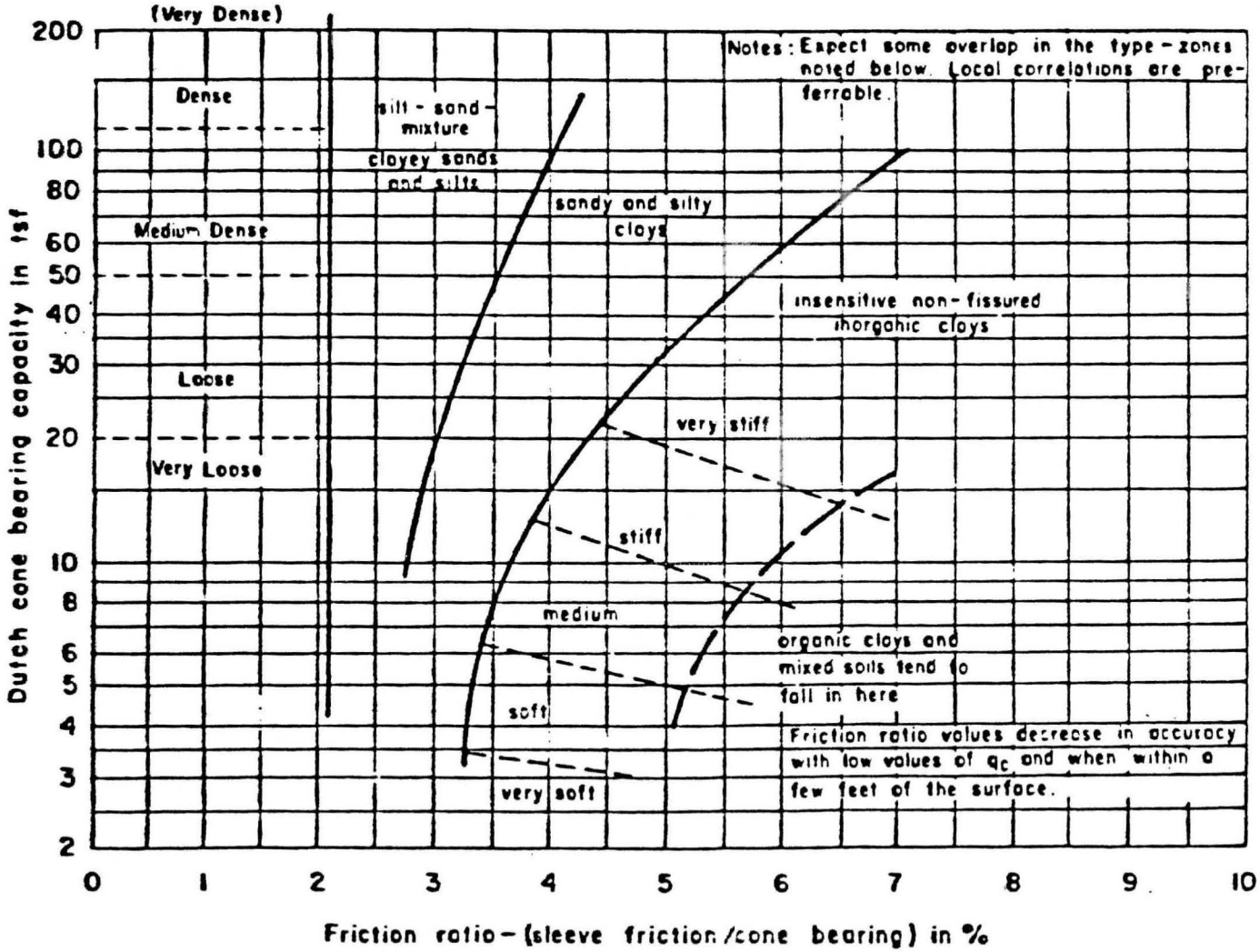


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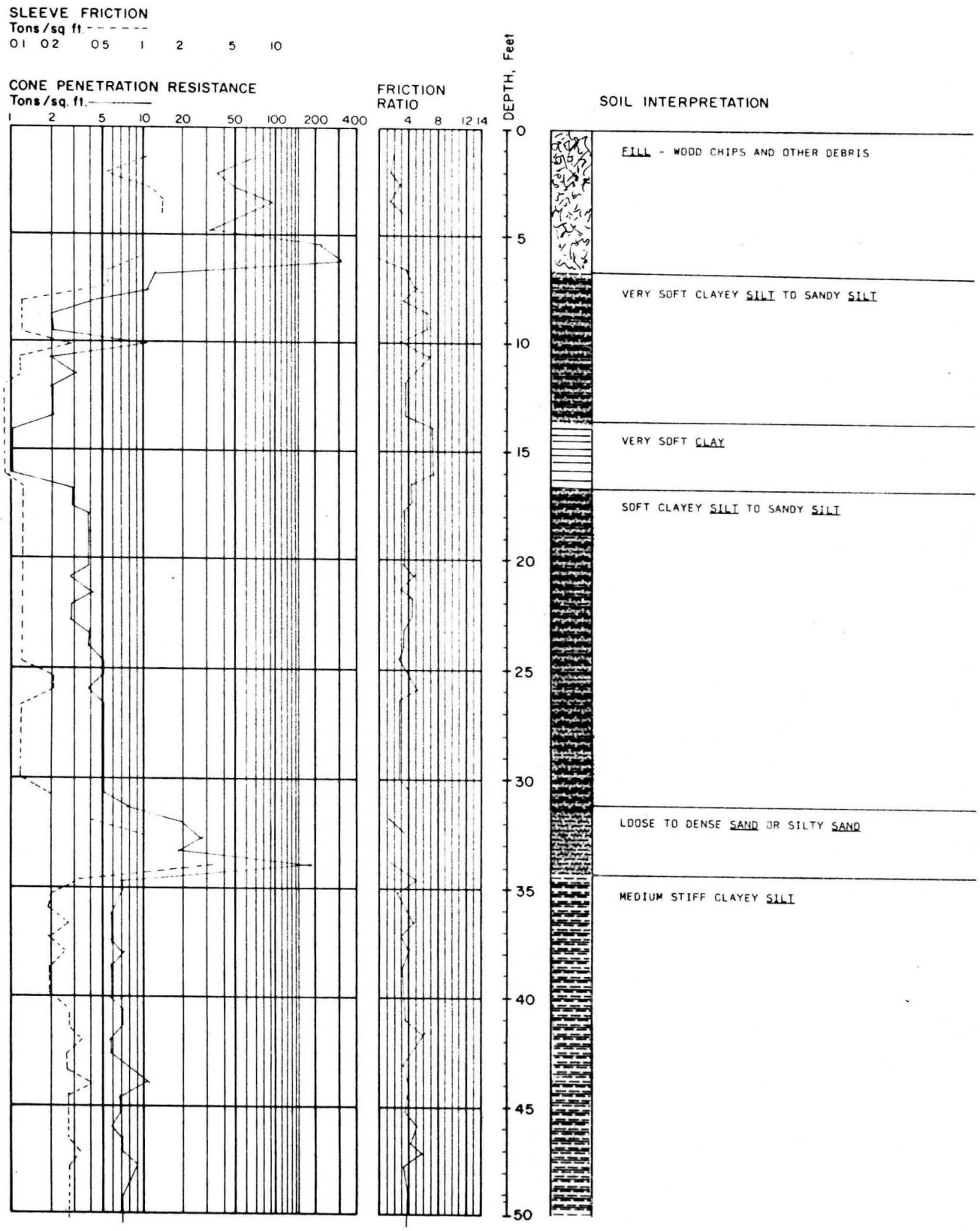
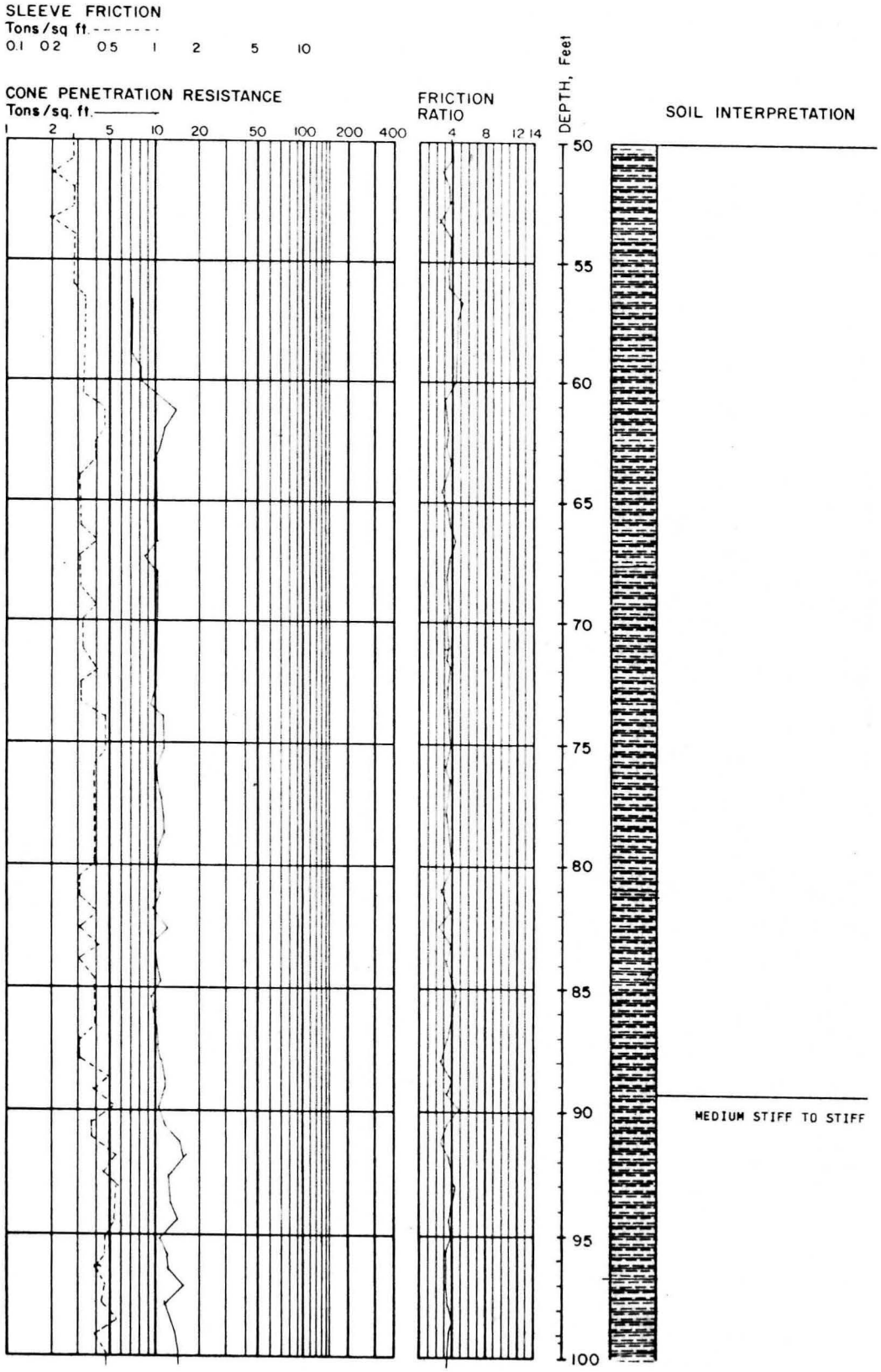
Figure A-5

# LOG OF PROBE P-1




SLEEVE FRICTION
Tons/sq ft.
0.1 0.2 0.5 1 2 5 10
CONE PENETRATION RESISTANCE
Tons/sq. ft.
1 2 5 10 20 50 100 200 400
FRICTION RATIO
4 8 12 14
DEPTH, Feet
50
55
60
65
70
75
80
85
90
95
100
SOIL INTERPRETATION
MEDIUM STIFF TO STIFF

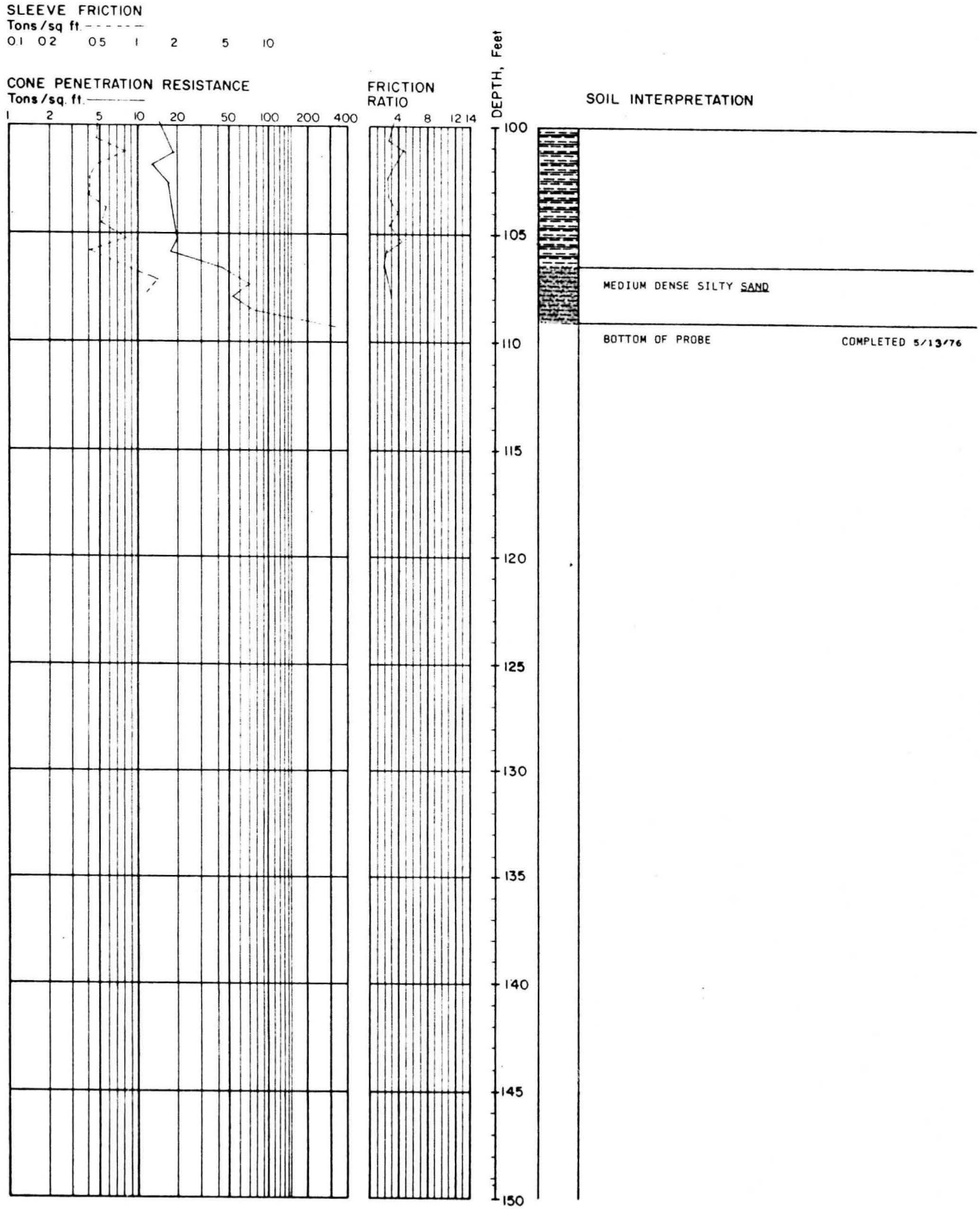



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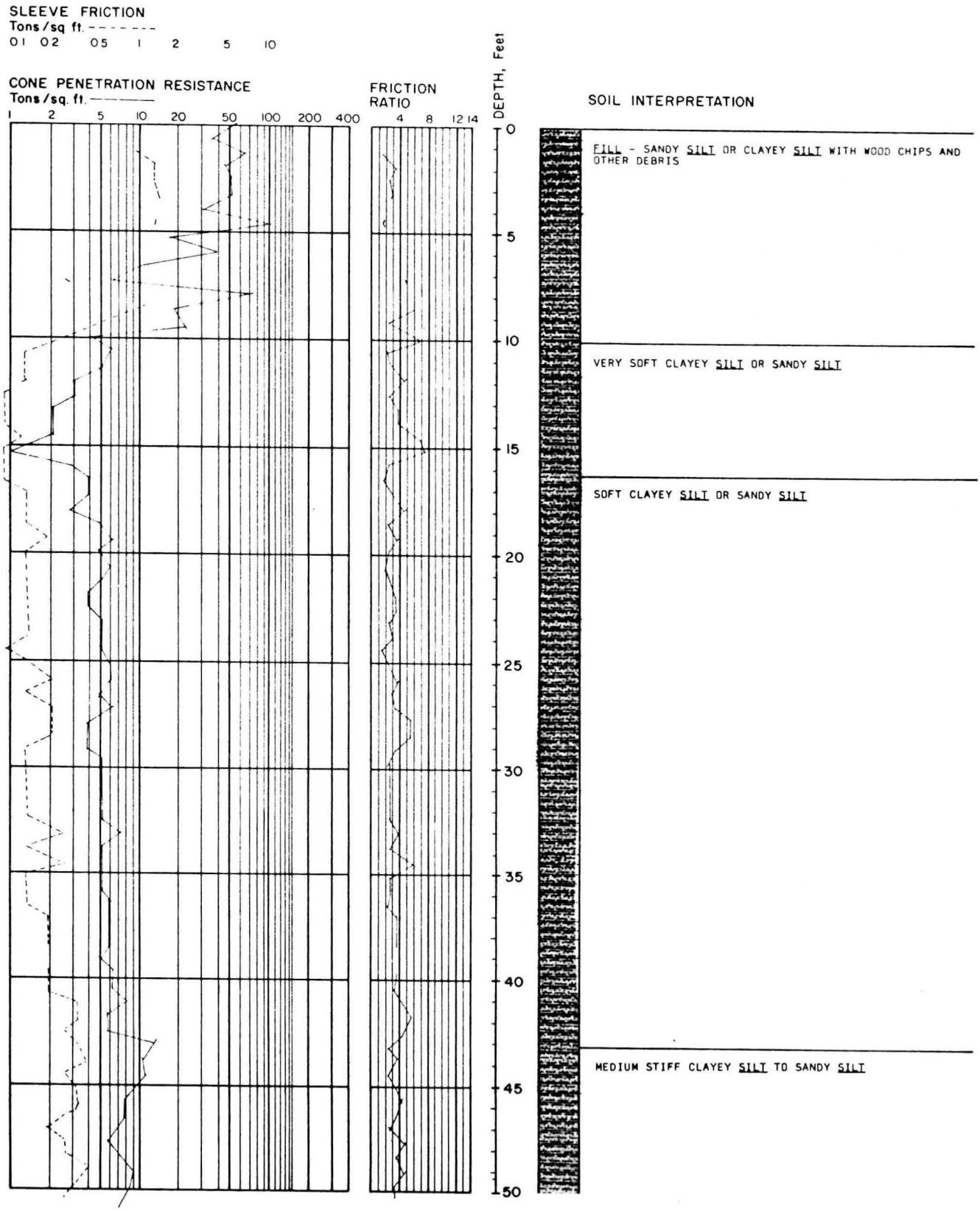
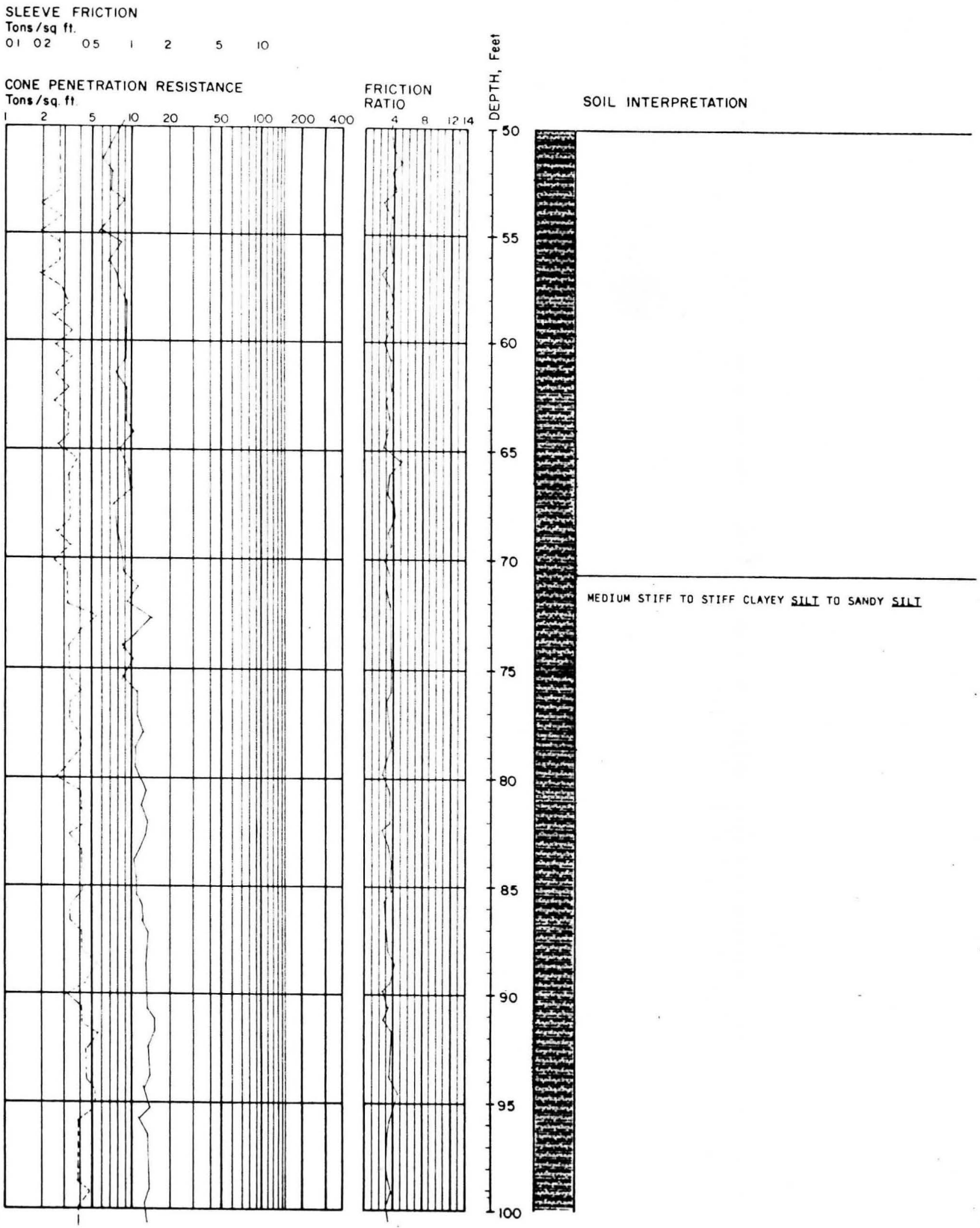
Figure A-6

# LOG OF PROBE P-2




SLEEVE FRICTION
Tons/sq ft.
01 02 05 1 2 5 10
CONE PENETRATION RESISTANCE
Tons/sq. ft.
1 2 5 10 20 50 100 200 400
FRICTION RATIO
4 8 12 14
DEPTH, Feet
50
55
60
65
70
75
80
85
90
95
100
SOIL INTERPRETATION
MEDIUM STIFF TO STIFF CLAYEY SILT TO SANDY SILT

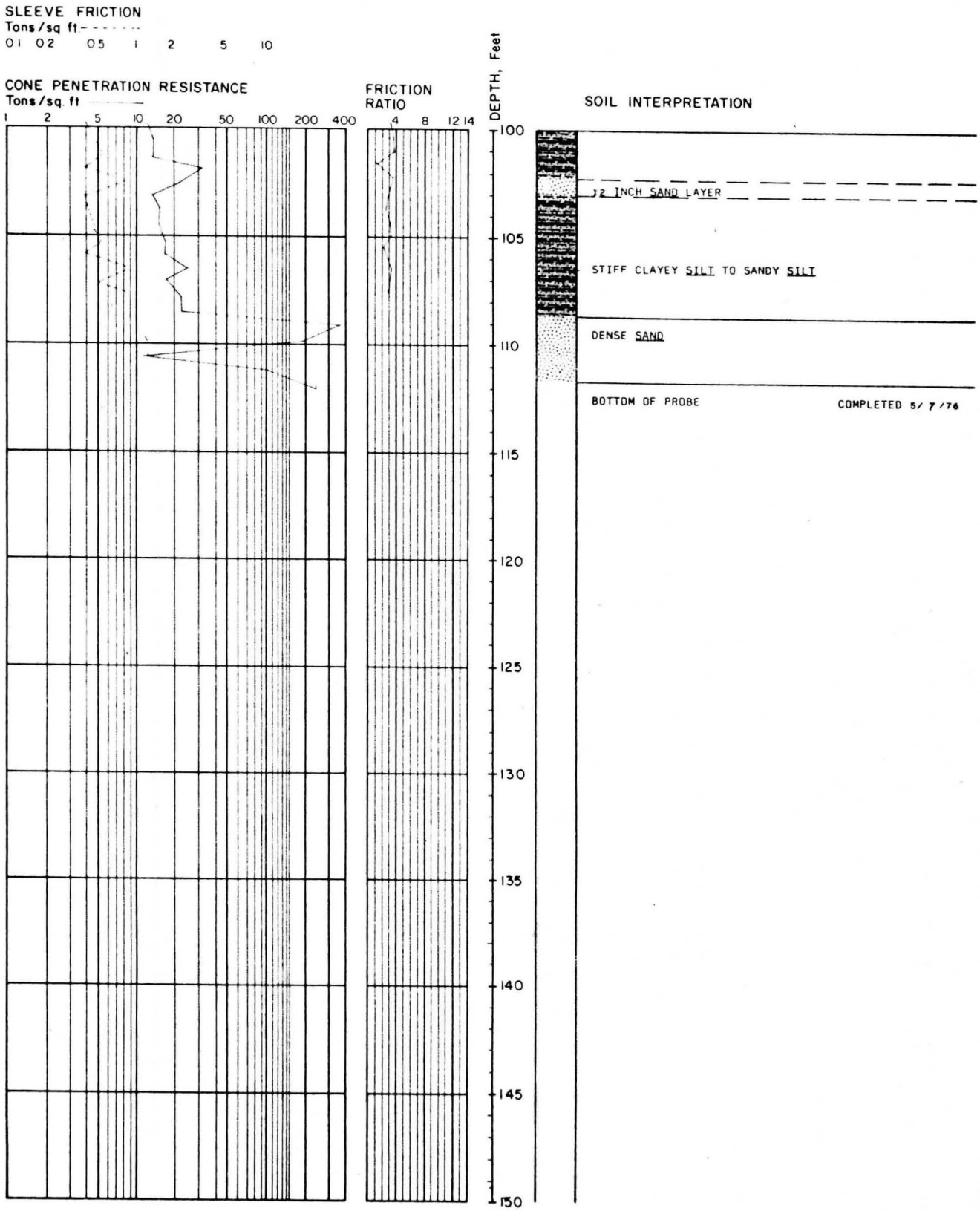



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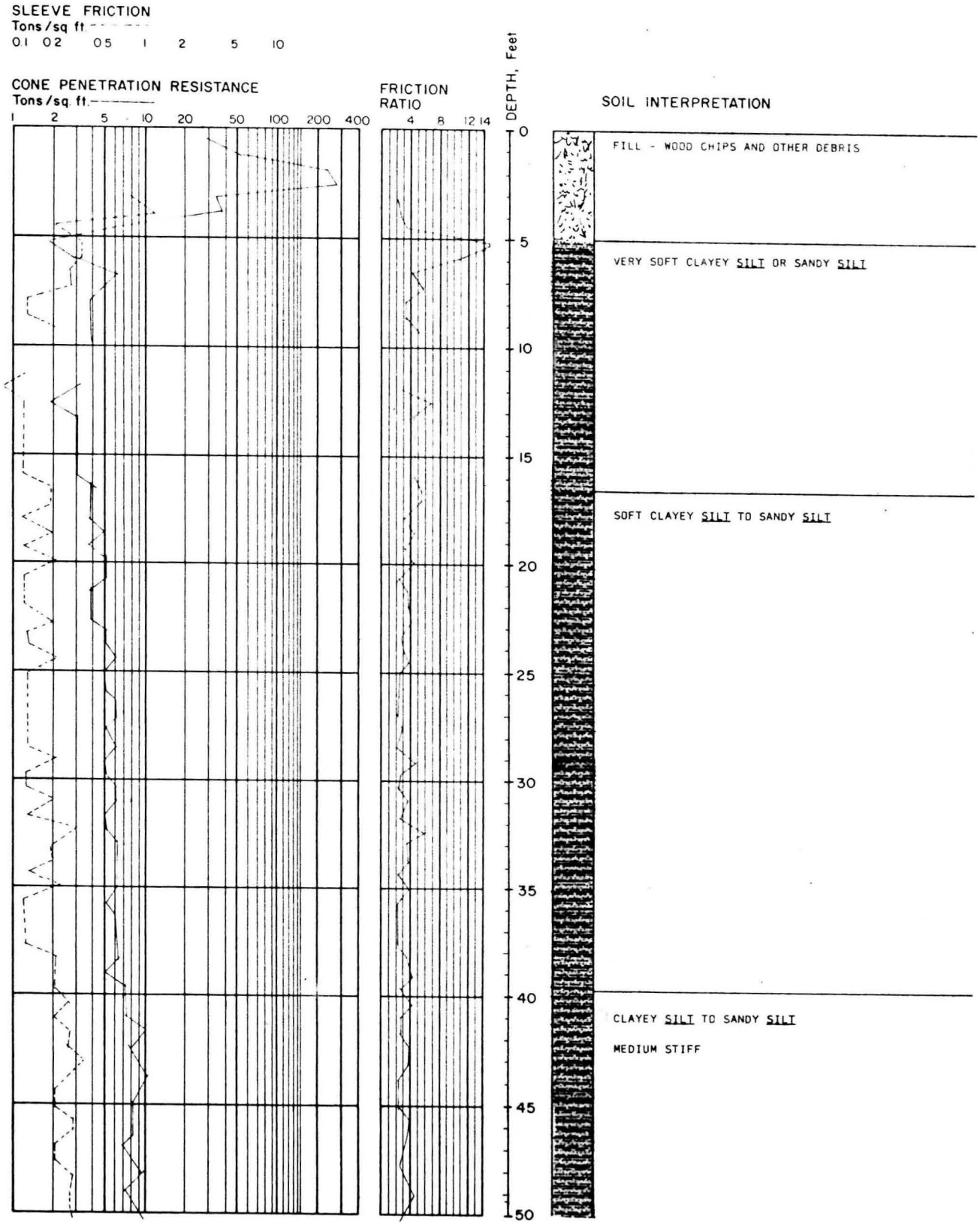
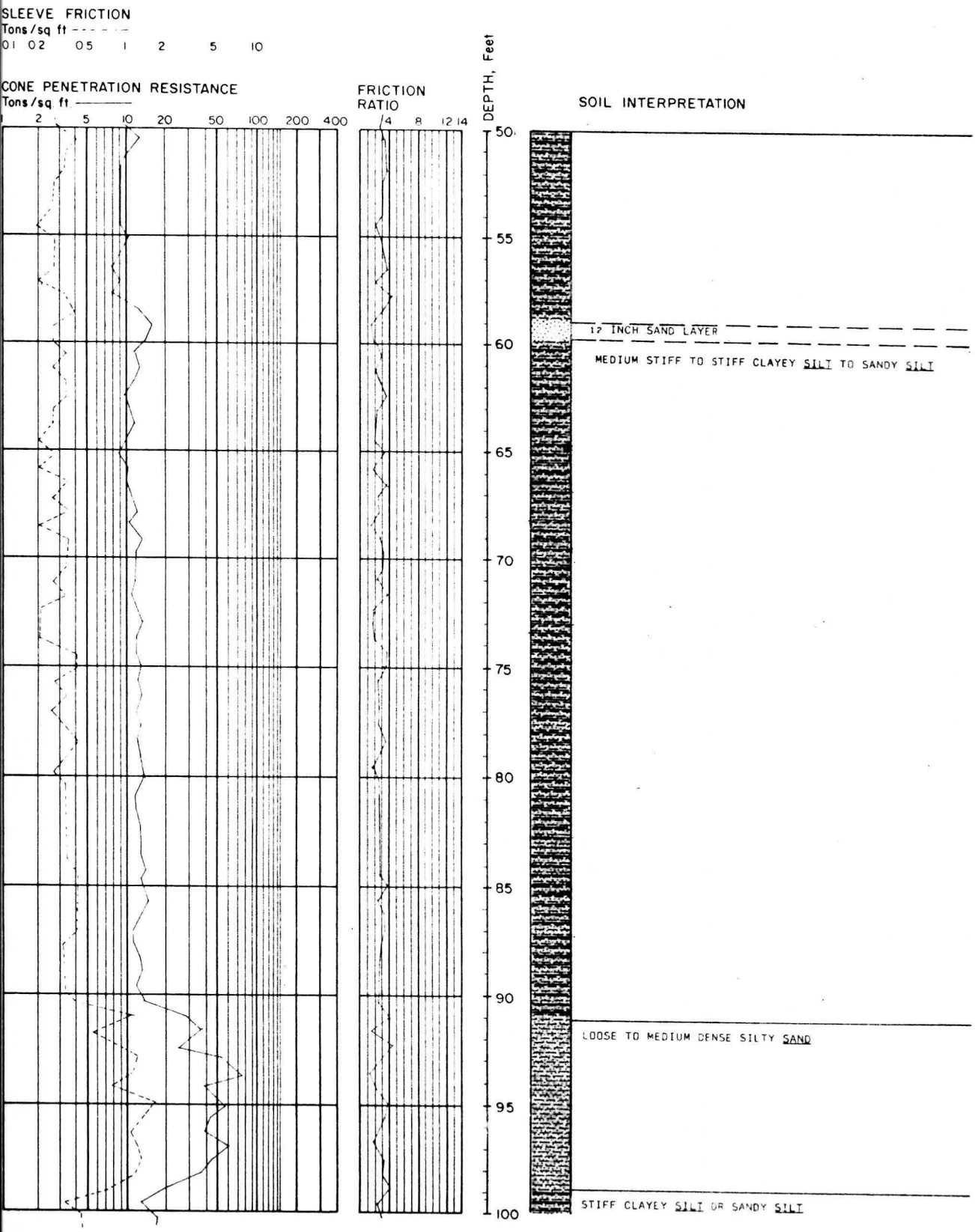
Figure A-7

# LOG OF PROBE P-3




SLEEVE FRICTION
Tons/sq ft
0.1 0.2 0.5 1 2 5 10
CONE PENETRATION RESISTANCE
Tons/sq ft
1 2 5 10 20 50 100 200 400
FRICTION RATIO
2 4 8 12 14
DEPTH, Feet
50
55
60
65
70
75
80
85
90
95
100
SOIL INTERPRETATION
12 INCH SAND LAYER
MEDIUM STIFF TO STIFF CLAYEY SILT TO SANDY SILT
LOOSE TO MEDIUM DENSE SILTY SAND
STIFF CLAYEY SILT OR SANDY SILT

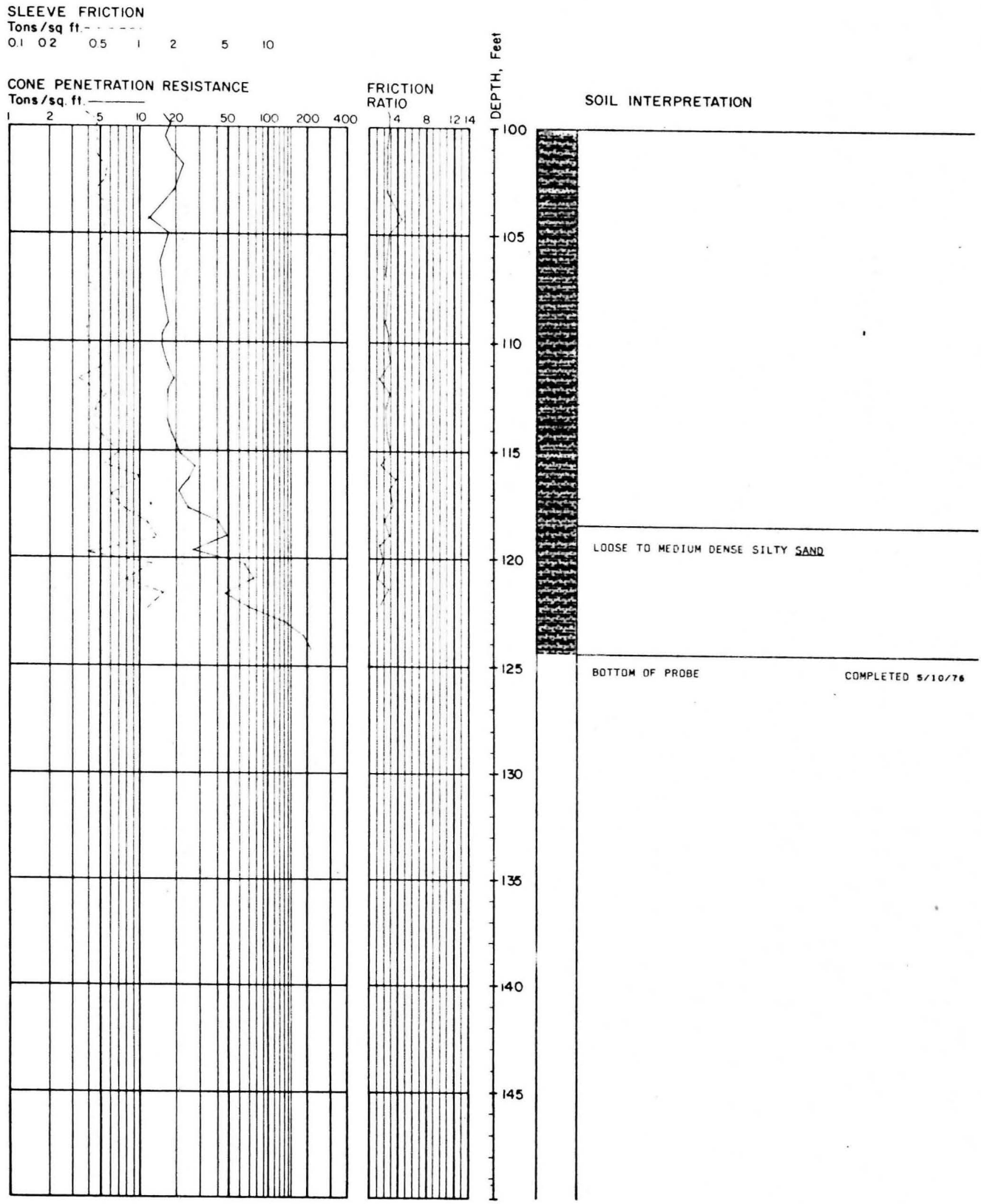



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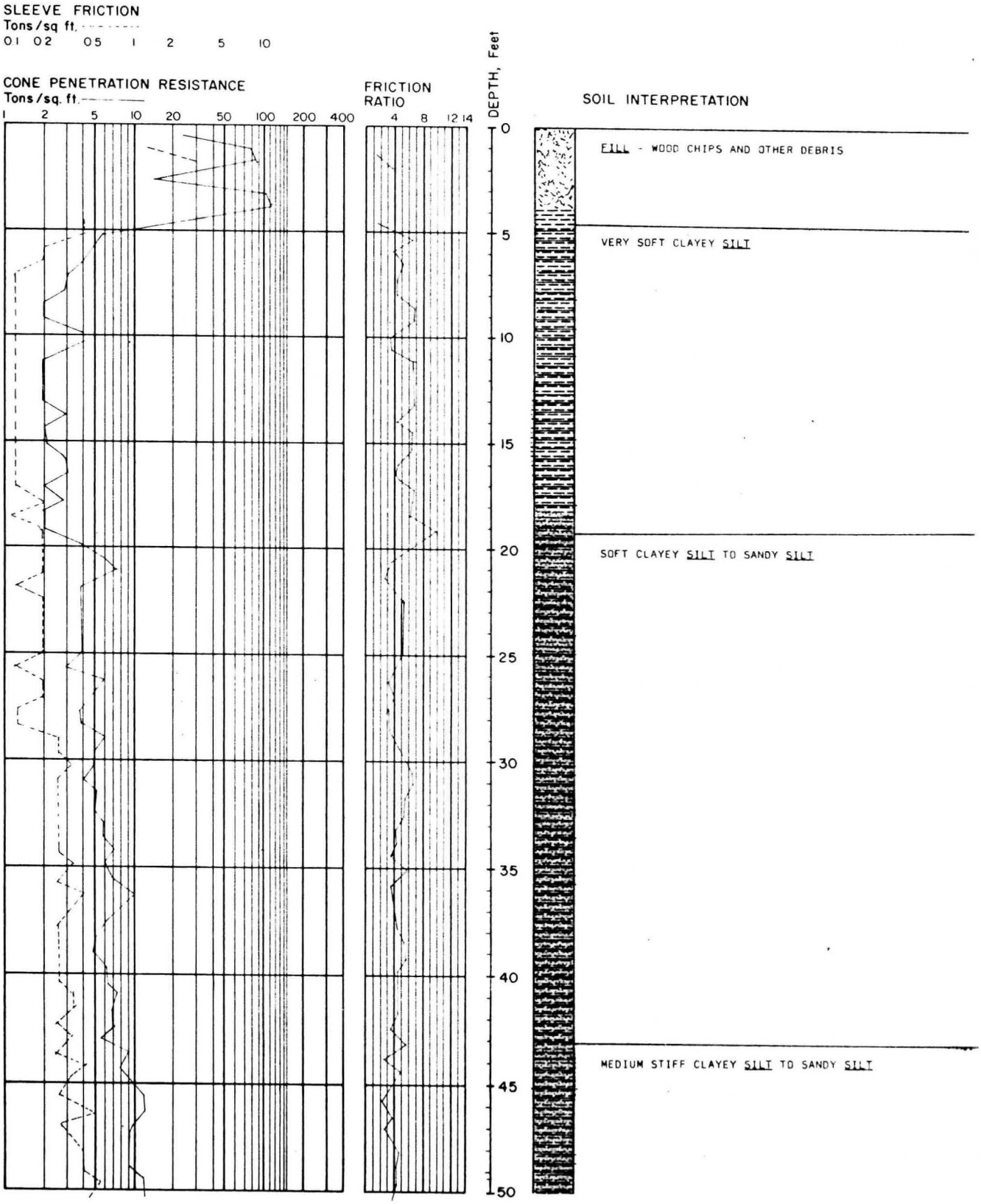
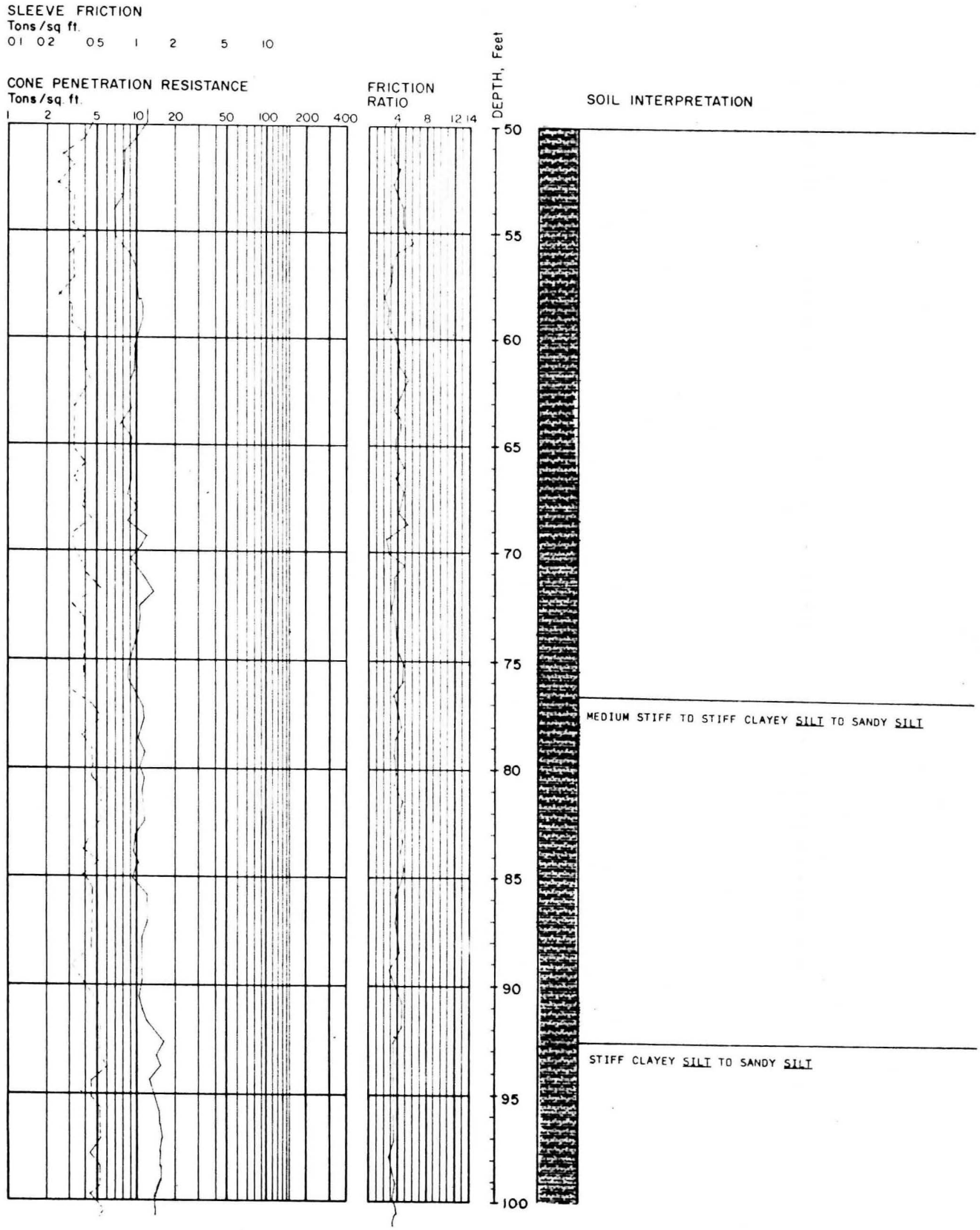
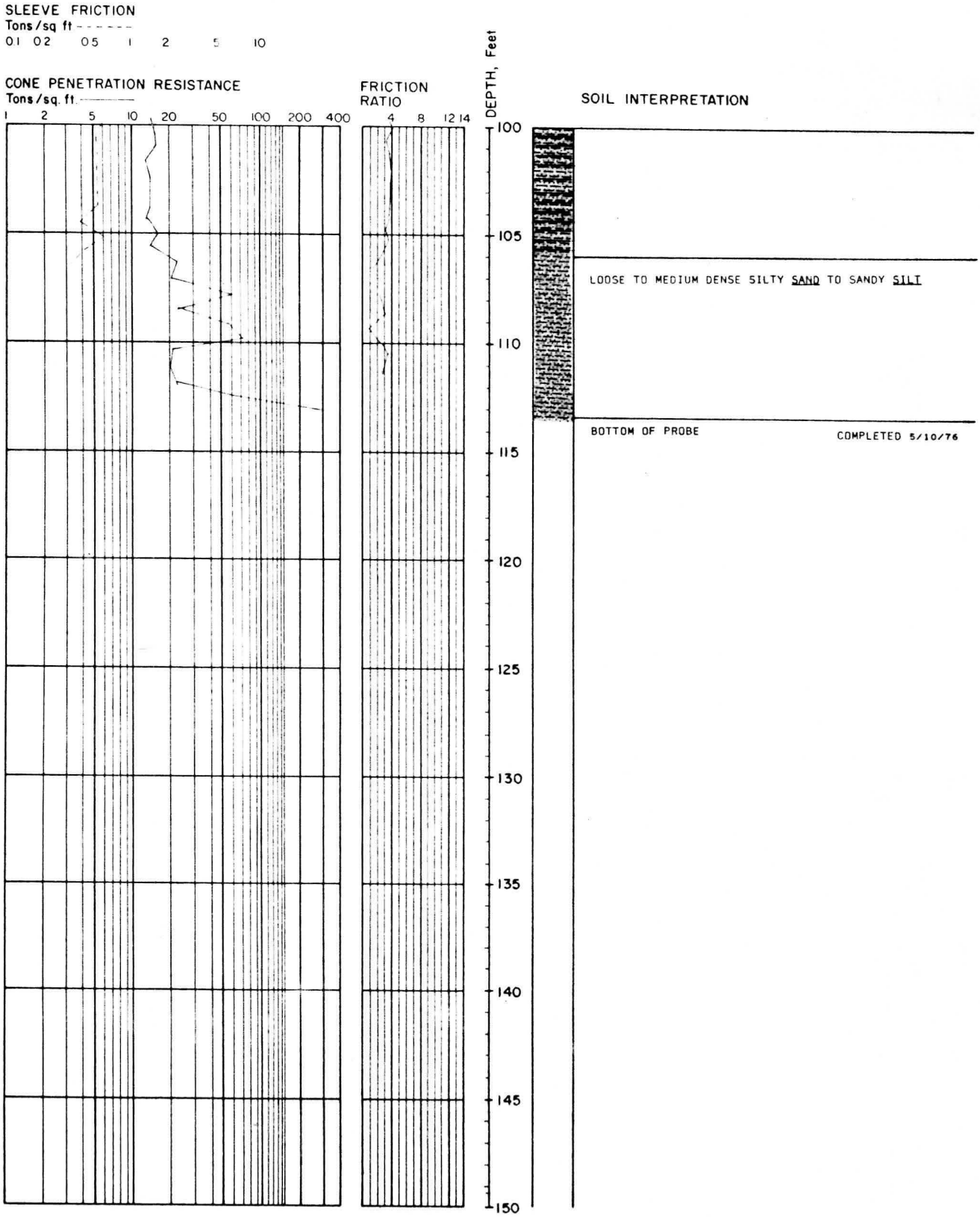
Figure A-8

# LOG OF PROBE P-4




SLEEVE FRICTION
Tons/sq ft.
0.1 0.2 0.5 1 2 5 10
CONE PENETRATION RESISTANCE
Tons/sq. ft.
1 2 5 10 20 50 100 200 400
FRICTION RATIO
4 8 12 14
DEPTH, Feet
50
55
60
65
70
75
80
85
90
95
100
SOIL INTERPRETATION
MEDIUM STIFF TO STIFF CLAYEY SILT TO SANDY SILT
STIFF CLAYEY SILT TO SANDY SILT




'RETATION

) CHIPS AND OTHER DEBRIS

LAYEY SILT

SILT TO SANDY SILT

CLAYEY SILT TO SANDY S

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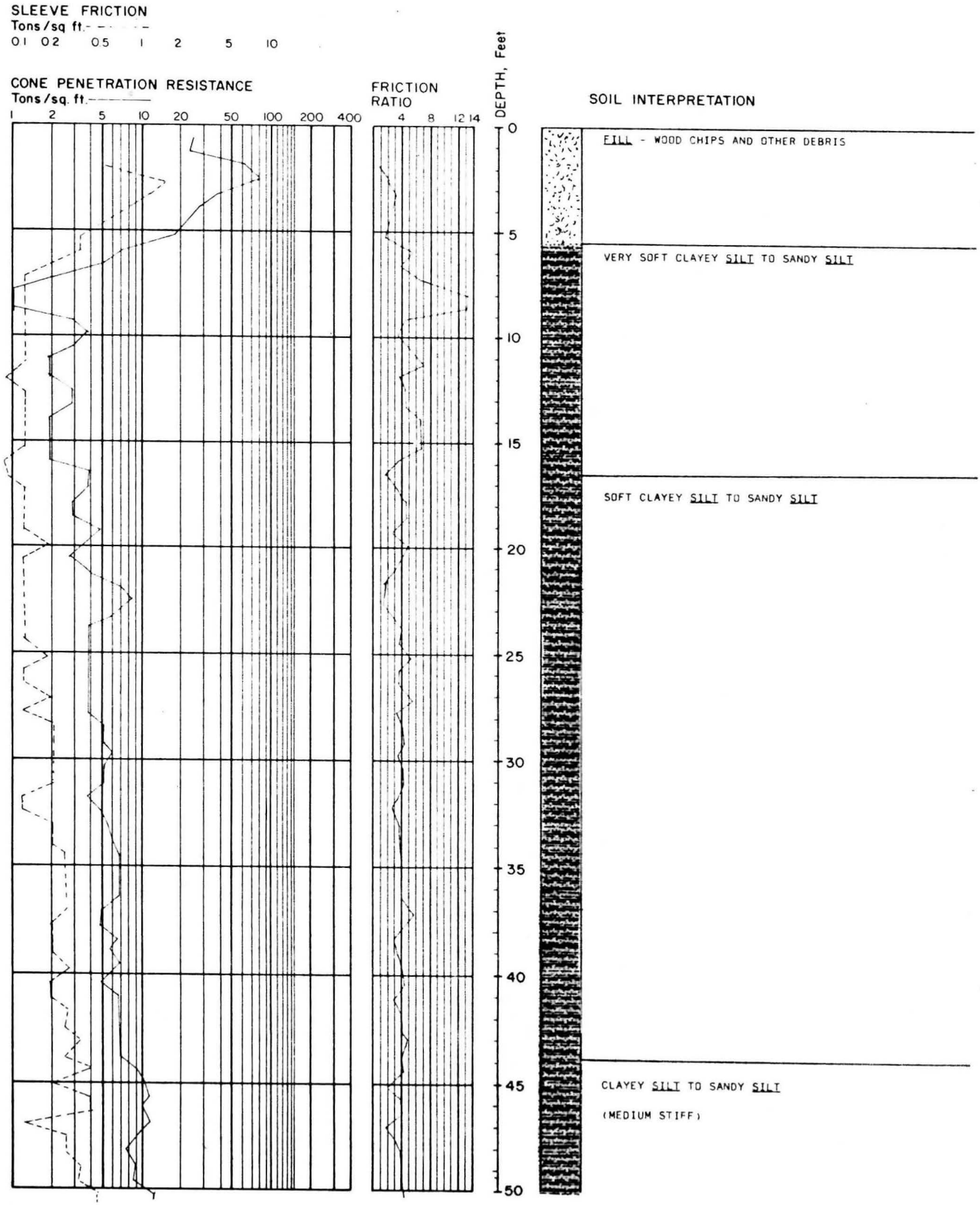
Figure A-9

# LOG OF PROBE P-5



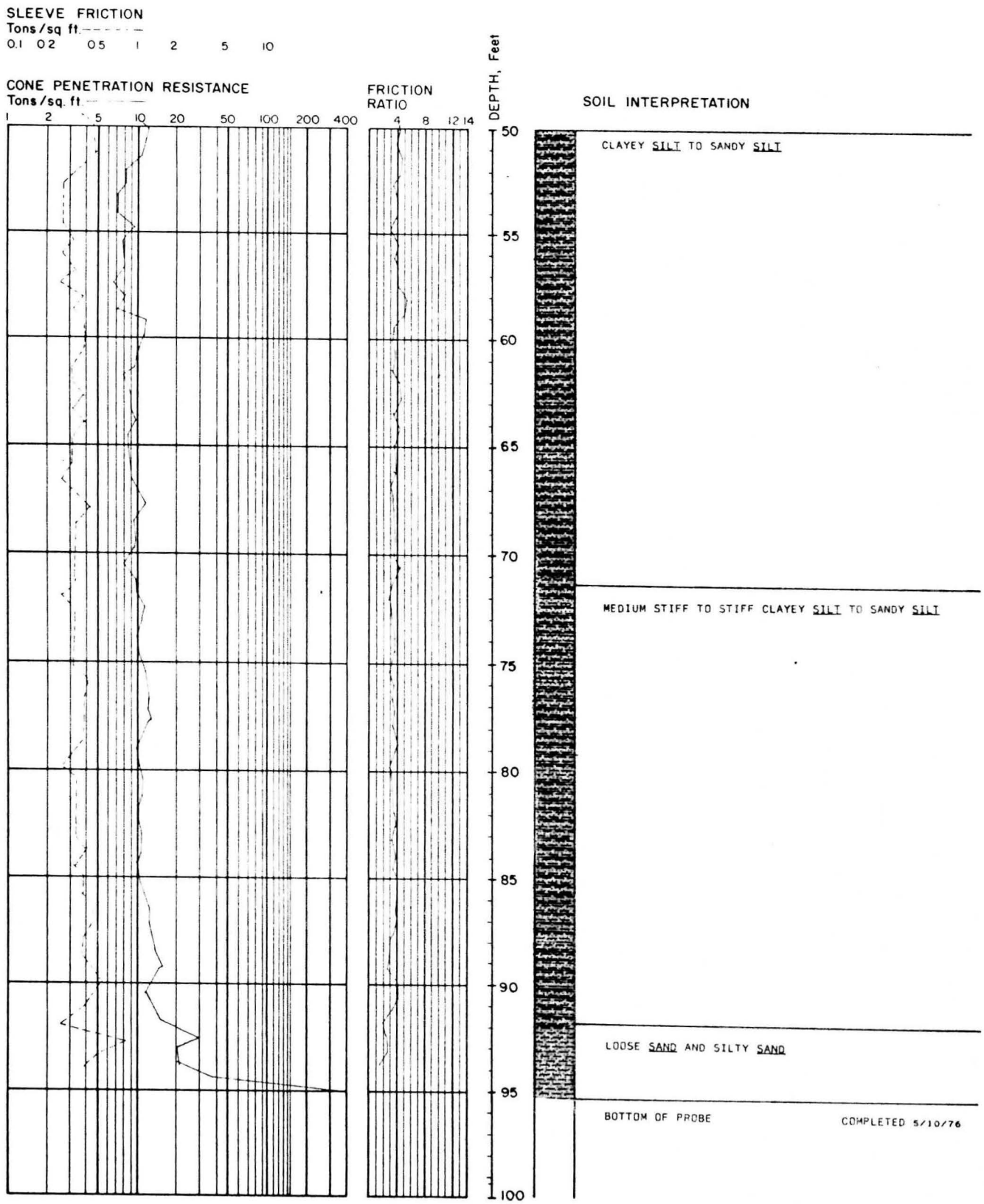



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Figure A-10

# APPENDIX B

## LABORATORY TESTING

A laboratory testing program was developed to establish the basic index and engineering properties of site soils. Particular attention was directed to determining the consolidation and shear strength characteristics of the compressible SILT soils.

### Classification Tests

All samples were visually reclassified in our laboratory and the field logs verified and modified as required. In addition, the natural water content was determined for most samples and Atterberg limits performed on selected samples for the purposes of (1) identification and correlation of the soils, and (2) providing basic engineering properties. Natural water contents are presented on the logs of borings, Appendix A. Atterberg limits are presented on the plasticity chart, Figure B-1.

### Unconfined Compression Tests

Unconfined compression tests were performed on selected, relatively undisturbed samples by subjecting a cylindrical specimen to axial compression until failure occurs at peak compressive stress. The undrained shear strength of a cohesive soil tested in this manner is generally taken as one-half its unconfined compressive strength. Wet unit weight and natural moisture content were also determined for all specimens. Unconfined compression test results and wet unit weights are presented on the logs of borings, Appendix A.

## Direct Shear Tests

Direct shear tests were performed on selected samples to determine the shear strength characteristics of the SILT soils The test series was performed on relatively undisturbed soils placed into a 2.5-inch diameter shear box. A normal load was applied and the sample allowed to consolidate with access to water prior to testing. Shearing was conducted at a constant strain rate of 0.001 inches per minute with shear stress measured using a calibrated proving ring. Direct shear test results, plotted in terms of normal and shear stress, are presented on Fig. B-2.

## Consolidation Tests

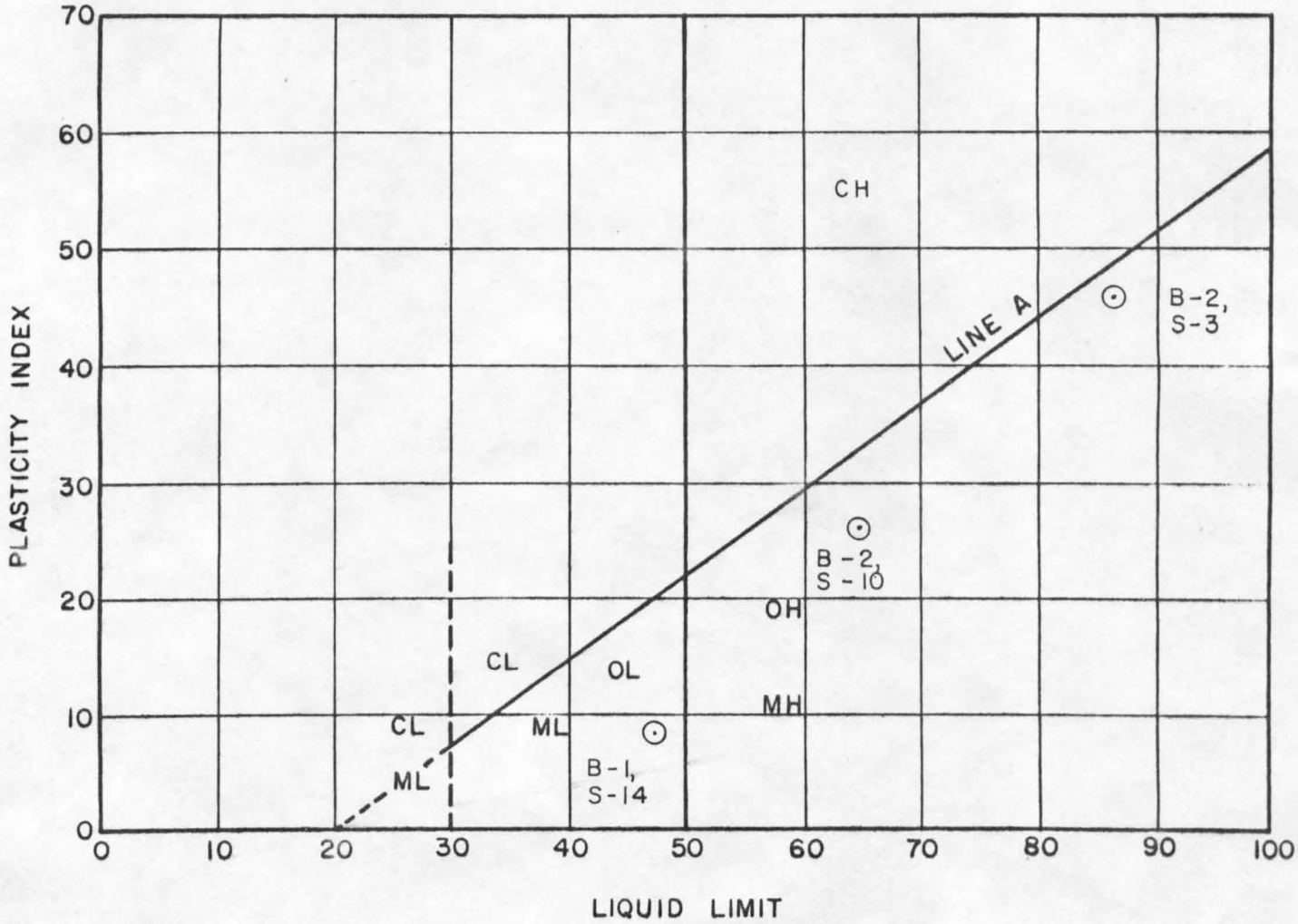
Three consolidation tests were performed on selected samples of compressible SILT soils to provide data for making settlement estimates. Relatively undisturbed samples were carefully trimmed and fitted into a rigid ring with porous stones placed on the top and bottom of the sample to allow drainage or absorption of water to approximate possible saturation under field conditions.

Vertical loads were then applied in increments with each load being allowed to consolidate prior to adding the following increment. Measurements of the time compression were obtained during each load increment and rebound was measured during the unloading portion. Consolidation test results, plotted in terms of percent consolidation (strain) versus applied load (stress), are presented on Figs. B-3 through B-5.

## Groundwater and Leachate Analysis

Chemical and bacterial analyses were performed by Laucks Testing Laboratories Inc. of Seattle, Washington. The laboratory test results are presented in a certificate from Laucks Testing Laboratory on Figure B-6.

# PLASTICITY CHART




| | | LL | PL | PI | |
|---|---|---|---|---|---|
| B-2, S-3 | ⊙ | 87 | 41 | 46 | Organic Clayey SILT (OH-MH) |
| B-2, S-10 | ⊙ | 65 | 38 | 27 | Organic Clayey SILT (OH-MH) |
| B-1, S-14 | | 47 | 38 | 9 | Organic SILT (OL-ML) |

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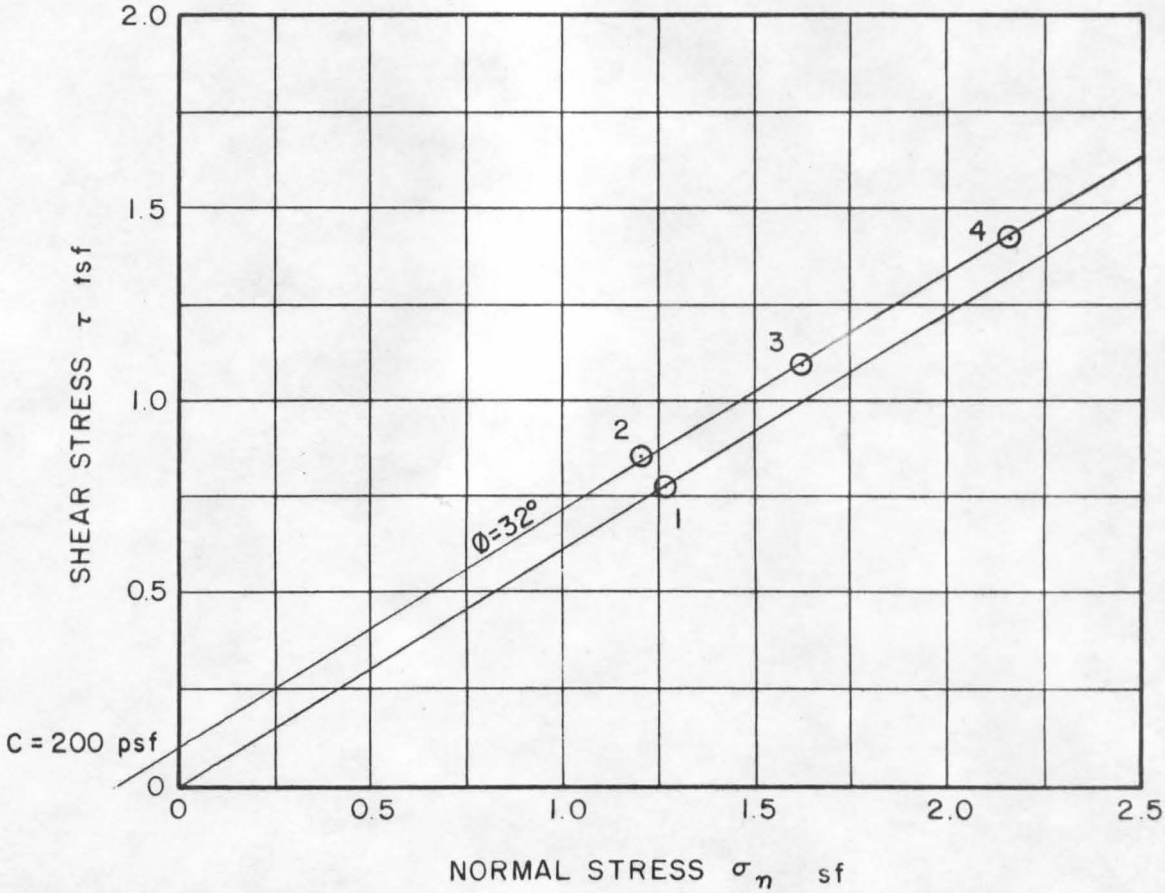
Figure B-1

# DIRECT SHEAR TEST RESULTS




| TEST NO. | SAMPLE | DEPTH FEET | UNDISTURBED SAMPLE WET UNIT WEIGHT (PCF) | NATURAL % W.C. | ATTERBERG LIMITS LL | PL | PI | CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|
| 1 | B-1, S-8 | 37.5-39 | 90.7 | 55 | – | – | – | |
| 2 | B-2 S-10 | 47.5 -49 | | | 65 | 38 | 27 | Organic Clayey SILT (OH-MH |
| 3 | " | " | 100.1 | 57 | " | " | " | " " " |
| 4 | " | " | 99.9 | 51 | " | " | " | " " " |

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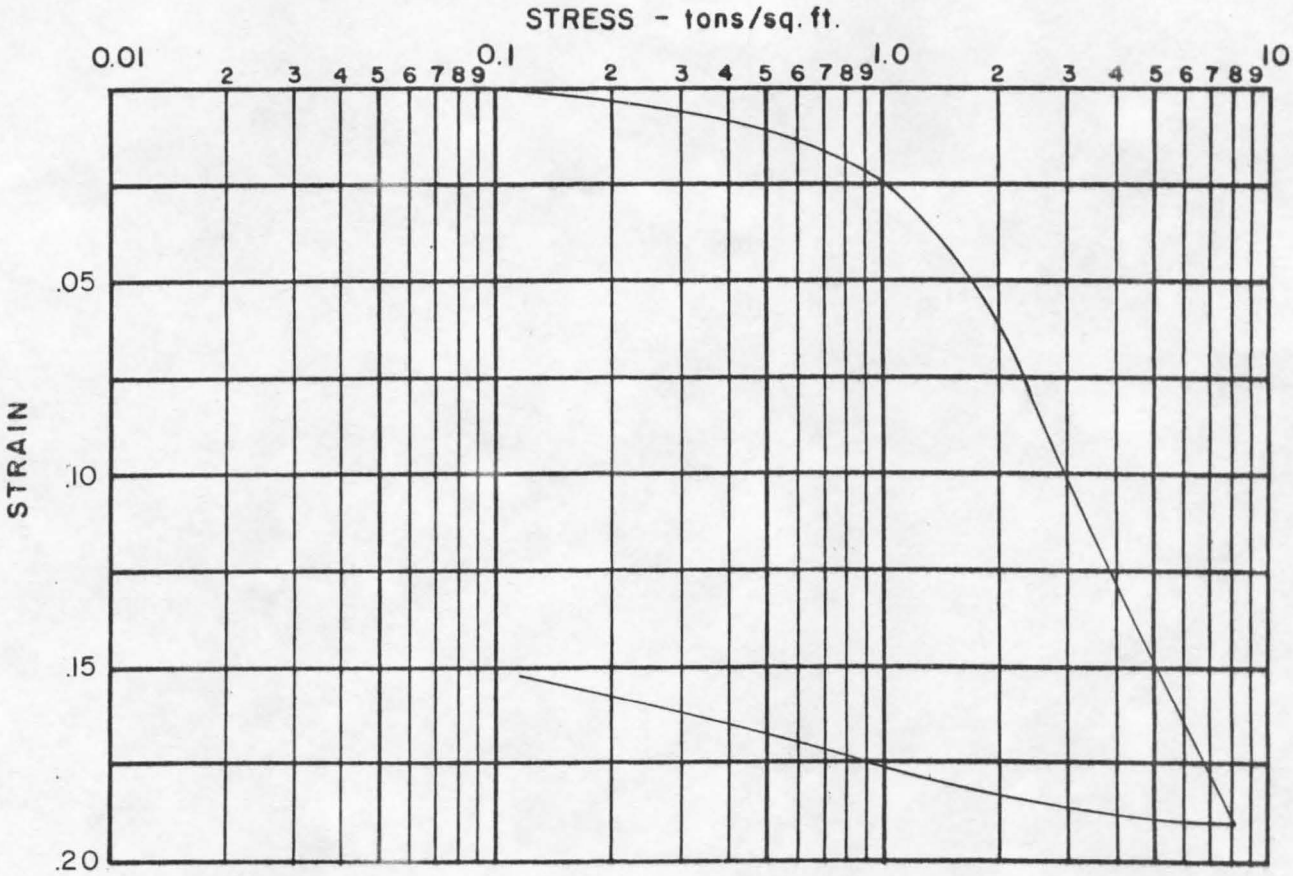
Figure B-2

# CONSOLIDATION TEST RESULTS




| SAMPLE | DEPTH, feet | % WATER CONTENT Before | After | ATTERBERG LIMITS LL | PL | PI | CLASSIFICATION |
|---|---|---|---|---|---|---|---|
| B-1, S-14 | 67.5-69.0 | 62 | 51 | 47 | 38 | 9 | Organic SILT (OL-ML) |

WET UNIT WEIGHT = 100.1 pcf

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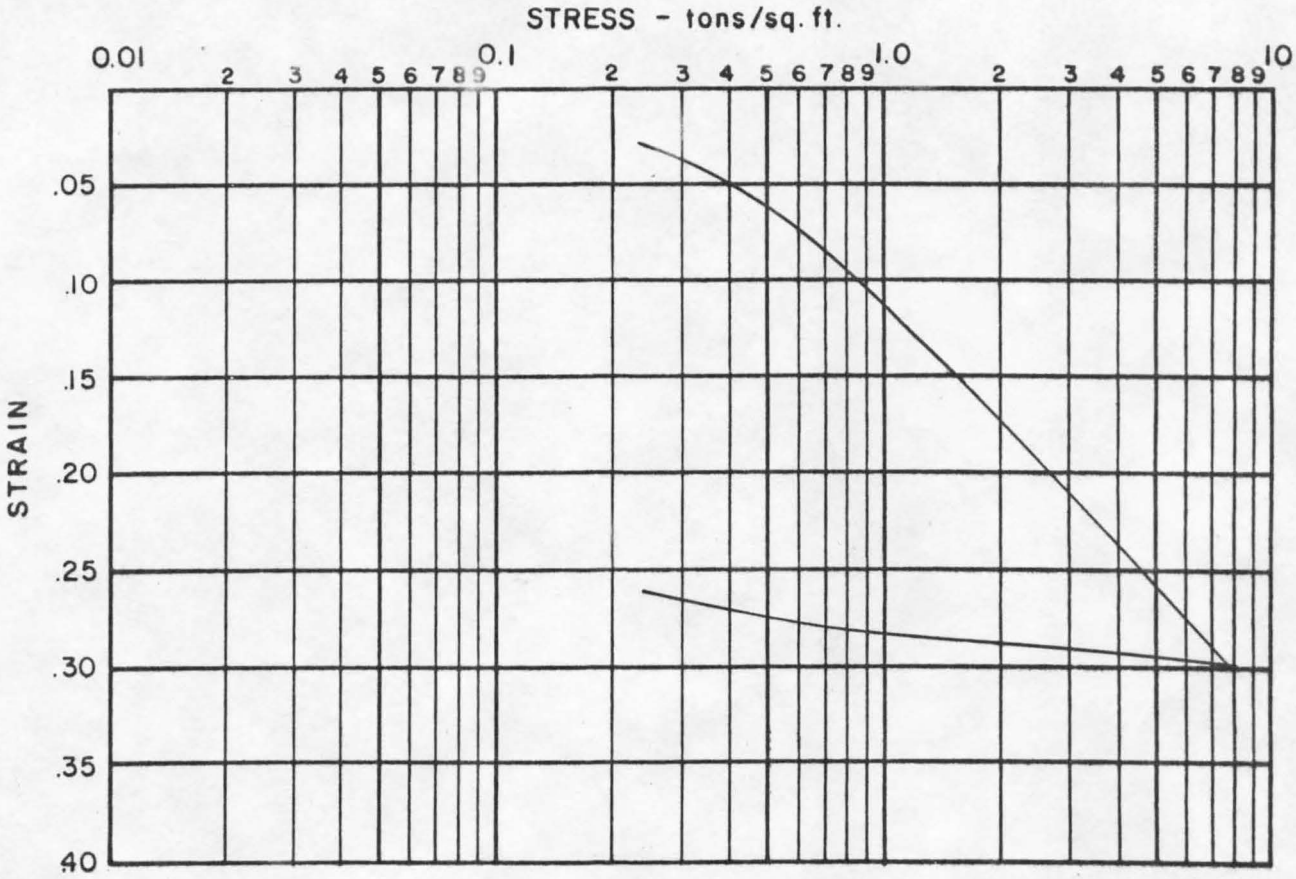
Figure B-3

# CONSOLIDATION TEST RESULTS




| SAMPLE | DEPTH, feet | % WATER CONTENT Before | After | ATTERBERG LIMITS LL | PL | PI | CLASSIFICATION |
|---|---|---|---|---|---|---|---|
| B-2, S-3 | 12.5-14.0 | 90 | 51 | 87 | 41 | 46 | Organic Clayey SILT (OH-MH |

WET UNIT WEIGHT = 98.5 pcf

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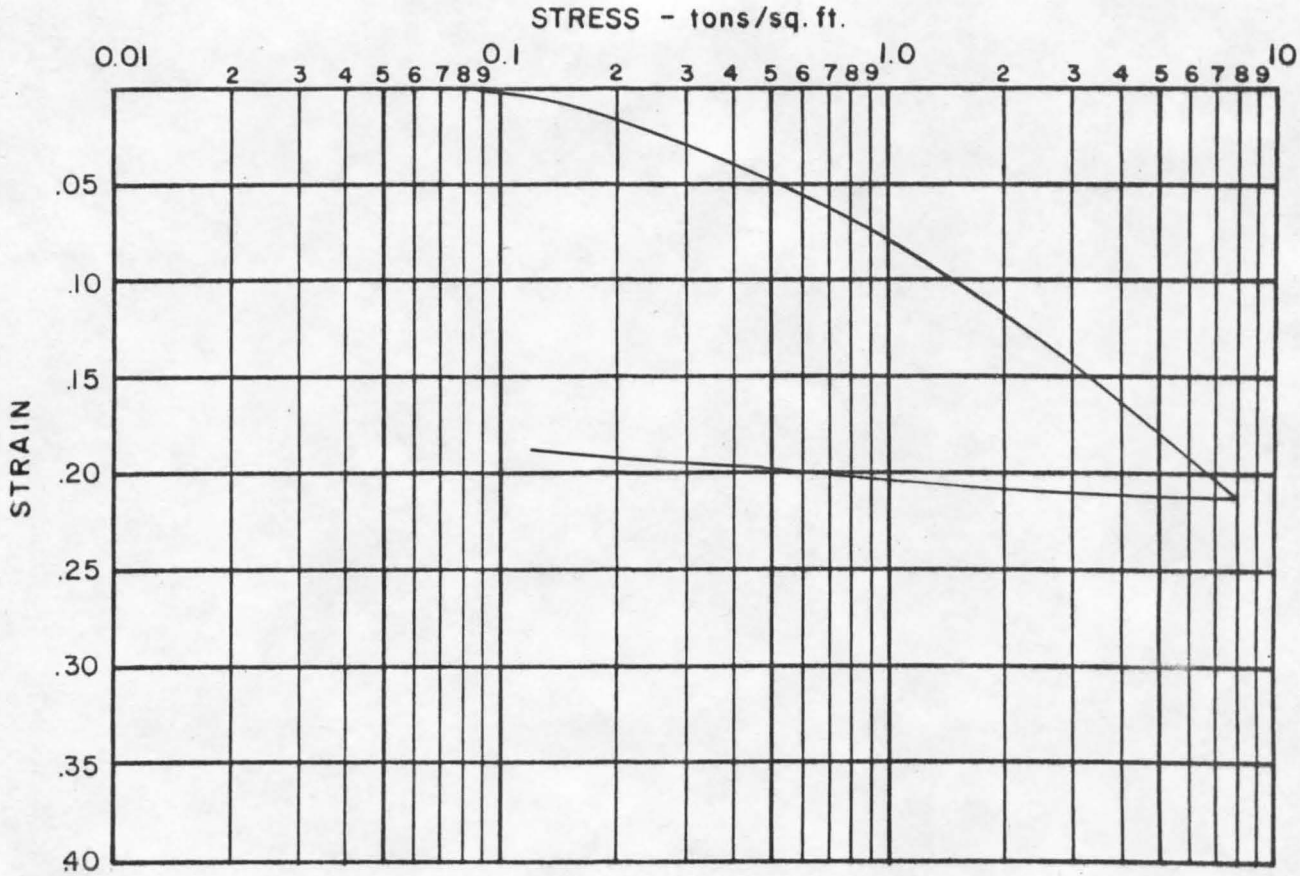
Figure B-4

# CONSOLIDATION TEST RESULTS




| SAMPLE | DEPTH, feet | % WATER CONTENT Before | After | ATTERBERG LIMITS LL | PL | PI | CLASSIFICATION |
|---|---|---|---|---|---|---|---|
| B-2, S-10 | 47.5-49.0 | 57 | 28 | 65 | 38 | 27 | Organic Clayey SILT (OH-MH) |

WET UNIT WEIGHT = 105.1 pcf

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HART-CROWSER & associates inc.

Figure B-5

# CERTIFICATE

# LAUCKS TESTING LABORATORIES

INCORPORATED

(206) 622-0727
1008 WESTERN AVENUE
SEATTLE, WASHINGTON 98104

CHEMISTS
SAMPLERS • INSPECTORS
ASSAYERS • SPECTROGRAPHERS
BIO-CLINICAL CHEMISTRIES

LABORATORY NO. 56490

DATE June 21, 1976

CLIENT Hart Crowser & Associates
Washington Plaza
Suite 1414
Tacoma, Washington 98402

RECEIVED
JUN 22 1976
HART - CROWSER

REPORT ON GROUND WATER

SAMPLE IDENTIFICATION Submitted: 6/11/76

Marked: Job #J-304

TESTS PERFORMED AND RESULTS:

| Identification | pH, glass electrode | Specific Conductance micromhos/cm | Chemical Oxygen Demand, milligrams/L | 5 Day B.O.D. Milligram per Liter |
|---|---|---|---|---|
| GP-1 | 5.7 | 370 | 520 | 350 |
| GP-2 | 5.5 | 225 | 460 | 250 |
| GP-3 | 4.6 | 330 | 580 | -- |
| GP-4 | 5.3 | 380 | 360 | -- |
| NE | 6.0 | 225 | -- | -- |
| NW | 6.2 | 505 | -- | -- |
| SE | 6.0 | 420 | -- | -- |
| SW | 6.1 | 555 | -- | -- |

| | Milligrams/Liter | | |
|---|---|---|---|
| Identification | Total Solids | Volatile Solids | Fixed Solids |
| GP-1 | 800 | 195 | 605 |
| SE | 5950 | 1740 | 4210 |

Fixed solids from above samples were analyzed qualitatively spectrographically, with results as follows:

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Figure B-6

CERTIFICATE

# LAUCKS TESTING LABORATORIES

INCORPORATED

MAin 2-0727
1008 WESTERN AVENUE
SEATTLE, WASHINGTON 98104

LABORATORY NO. 56490



Hart Crowser & Associates

| | GP-1 | SE |
|---|---|---|
| Major constituents | Magnesium & Calcium | Magnesium & Calcium |
| Minor constituents | Silicon & Sodium | Silicon |
| Trace constituents | Lead, Aluminum, Molybdenum, Nickel, Titanium, Manganese, Potassium, Chromium, and Vanadium | Lead, Aluminum, Sodium, Titanium, Manganese, Potassium Chromium, Vanadium |

Bacteriological Analyses

| | NE | SW |
|---|---|---|
| Total Bacteria Count per ml | 300,000 | 1,900,000 |
| Total Coliform Count, M.P.N. per 100 ml of sample | greater/240,000 | 43 |
| Fecal Coliform Count M.P.N. per 100 ml of sample | less/3 | less/3 |
| Fecal Strep. M.P.N. per 100 ml of sample | 4,600 | less/3 |

Respectfully submitted,

LAUCKS TESTING LABORATORIES, INC.

J. M. Owens

J. M. Owens

JMO:po

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