# FIELD EXPLORATION AND GEOTECHNICAL ENGINEERING STUDY
# PROPOSED J.A. JONES GRAVING DOCK
# ALEXANDER AVENUE AND SOUTH LINCOLN AVENUE ON BLAIR WATERWAY
# PORT OF TACOMA, WASHINGTON

J-988

HART CROWSER & associates inc.
GEOTECHNICAL ENGINEERING

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March 26, 1981

Storch Corporation/Engineers
17930 Pacific Highway South, Suite 516
Seattle, Washington 98188

Attn: Mr. Erik Prestegaard, P.E.

Re: Field Exploration and Geotechnical Engineering Study
Proposed J.A. Jones Graving Dock
Alexander Avenue and South Lincoln Avenue on Blair Waterway
Port of Tacoma, Washington

Gentlemen:

We are pleased to submit herein eight copies of the above referenced report. This report contains the results of our field exploration and laboratory testing programs together with geotechnical engineering conclusions and recommendations.

The field exploration program disclosed relatively uniform conditions at the site consisting of dredged sand fill soils overlying natural deltaic deposits. The natural soil sequence disclosed in the explorations consisted of alternating strata of soft to medium stiff clayey silts and medium dense sands. Dense to very dense, fine to medium sands were disclosed at depth. Engineering studies were directed toward evaluating excavation slope stability for the proposed graving dock as well as evaluating foundation support conditions for pontoon construction and for the appurtenant staging areas.

General recommendations are also included for dewatering the proposed graving dock excavation, lateral pressures against the sheet pile closure structure and required embedment depth, and recommendations for construction monitoring and observation.

We appreciate this opportunity to be of service to you in this regard and look forward to continuing involvement in this exciting project. Should you have any questions regarding this report or other matters, plase feel free to call at your convenience.

Sincerely,

HART-CROWSER & ASSOCIATES, INC.

Kurt W. Groesch

KURT W. GROESCH, P.E.
Project Engineer

KWG:gpa

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LAUCKS TESTING LABORATORIES, INC.

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# FIELD EXPLORATION AND GEOTECHNICAL ENGINEERING STUDY
# PROPOSED J.A. JONES GRAVING DOCK
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# PORT OF TACOMA, WASHINGTON

## INTRODUCTION

This report presents the results of field explorations and field laboratory testing programs in conjunction with a geotechnical engineering study for construction of a graving dock in the Port of Tacoma, Washington. The general location of the project is shown on Figure 1, Vicinity Map. This study was carried out to evaluate the feasibility of a conceptual design from a geotechnical standpoint and to outline potential areas of design or construction problems which warrant further study.

The purpose of the study was to evaluate the subsurface conditions relative to the proposed graving dock construction. This report provides geotechnical engineering recommendations for slope stability of the proposed excavation slopes, field permeability testing and preliminary dewatering design, lateral pressures and required embedment depth for the sheet pile closure structure and analysis of alternate foundation types, allowable bearing pressures and anticipated settlement for the graving dock, shipway and casting yard areas. In addition to the above, recommendations are included for additional services for construction monitoring and observation.

This study was authorized by Mr. Erik Prestegaard, P.E., of Storch Corporation/Engineers. Based on the conditions encountered in the field explorations, additional laboratory and geotechnical engineering studies were subsequently verbally authorized by Mr. Prestegaard for a more detailed slope stability and consolidation analysis.

This report has been prepared for exclusive use of Storch Corporation/Engineers for specific application to the proposed graving dock. The report has been prepared in accordance with generally accepted geotechnical engineering practice. No other warranty, expressed or implied, is made.

The field exploration program consisted of site reconnaissance visits, six Dutch cone penetrometer probes, three hollow-stem auger borings, three piezometer installations, groundwater level monitoring for in place permeability testing and groundwater sampling. The Dutch cone penetrometer probes, the hollow-stem auger borings, and piezometer installations were accomplished by Subterranean, Inc. under subcontract to Hart-Crowser & Associates. The field exploration procedures are detailed in Appendix A. Detailed interpretive logs of the explorations are presented on Figure 2, Site and Exploration Plan.

The initial laboratory testing program consisted of visual classification and moisture content determinations on all samples recovered. Grain size analyses were performed on selected samples within the granular soils encountered below the proposed excavation level for correlation with field permeability testing.

Based on the results of preliminary geotechnical analyses and in light of the extensive near surface deposits of soft cohesive soils encountered at the site, detailed index, shear strength and consolidation tests were performed on selected samples. Laboratory test procedures are presented in Appendix B with detailed test results shown on Figures B-2 through B-11. In addition, samples of the site groundwater were tested to determine potential incrustation or corrosive problems associated with the long term operation of a dewatering system. The results of these tests are tabulated in Appendix B.

The subsurface conditions inferred from the explorations and the soil properties determined by field and laboratory testing provide the basis for the geotechnical analyses and recommendations accomplished for this study.

## SITE AND PROJECT DESCRIPTION

The proposed site for the graving dock lies on the north side of the Blair Waterway in the Port of Tacoma, Washington. The project site is bounded on the north by Alexander Avenue, and on the east and west by other commercial and industrial properties. The location of the project site is shown on the Vicinity Map, Figure 1. The project area and proposed layout is shown on Figure 2, Site and Exploration Plan. The existing ground surface elevation across the site is somewhat variable. Topographic information obtained from the Port of Tacoma show ground surface elevations vary from elevation 15 to 20 feet (MLLW=0). A typical ground surface elevation on the order of 18 feet

has been used in the engineering analyses. The project site was sparsely wooded at the time of the explorations and has been subsequently cleared. Some ponded surficial water was encountered at the site. Due to the saturated soft surficial soils, a bulldozer was required to assist in moving the exploration equipment.

Plans call for excavating below existing site grade to about elevation (-)4 feet within the graving dock area. The plan dimensions of the excavation at elevation (-)4 feet would be about 650 feet by 480 feet. A 2-foot thickness of slag would be established within the excavation as a working surface. Segmental pontoon units for the Hood Canal Bridge would then be constructed within the graving dock utilizing mainly precast concrete elements. Typical pontoons would be 360 feet by 60 feet in plan, and each would weight about 13,000 kips. It is understood that up to five pontoons would be constructed in one cycle, and that possibly three construction cycles would be necessary to complete the project. When one construction cycle is complete, the excavation would be temporarily flooded, the pontoons floated out and towed for assembly at the bridge site. The preliminary plans call for a driven steel sheet pile closure structure. After the excavation is flooded, the sheet piles will be extracted to allow access to the Blair Waterway. The steel sheet piles will be redriven to effect closure and the excavation pumped out to initiate the next construction cycle.

Adjacent to the graving dock to the west would be a casting yard for the precast concrete elements which make up the pontoons. Discussions with Storch Corporation/Engineers indicate the largest elements would be 18 feet by 27 feet in plan and each

would weigh about 135 kips. East of the graving dock, it is planned to construct a shipway to allow construction and launching of concrete anchors for the floating bridge. Typical anchors would weigh on the order of 2,400 kips and would be 45 feet in diameter. It is understood the anchors would be cast on skids. The anchors would be launched by sliding down the sloped shipway. It is understood that completed anchors would draw about 20 feet, and the shipway would extend to elevation (-)15 feet.

GEOLOGIC CONDITIONS

The project area lies within the Puyallup River Delta which was originally a tidal marsh prior to development of the Port of Tacoma. The major growth of the delta is presumed to have occurred during the very large meltwater discharges and rising sea level elevations accompanying the retreat of the Vashon Glaciation. Changes in sea level, discharge rate and sediment load have resulted in irregular rate of delta front advancement. A rapidly advancing delta would consist primarily of granular soils while periods of slow or no growth would result in deposition of siltier soils. The distribution of soils indicated by borings advanced at this project and other areas within the Puyallup delta indicate alternating rates of growth resulting in stratified deposition of silts and sands. The natural soil sequence is locally overlain by varying amounts of man-placed fill soils.

## SUBSURFACE CONDITIONS

Six Dutch cone probes and three hollow-stem auger borings were advanced in the project area. Based on the conditions disclosed in the explorations, a Generalized Subsurface Profile, Figure 3, has been developed. The location of the profile is shown on Figure 2. Generalized soil descriptions are presented next to graphical soil logs corresponding to their appropriate elevations.

The explorations advanced in the area of the graving dock indicated relatively uniform conditions as illustrated on the profile. In general, the soil stratigraphy encountered consisted of varying amounts of dredged sand and other fill soils overlying natural soils. The natural soil sequence consisted of very soft to soft clayey silts with organics extending to about elevation (-)3± feet underlain by medium dense to dense sands. These sands extend to about elevation (-)16± feet and are underlain by soft to medium stiff cohesive soils. Beneath cohesive soils at an elevation of about (-)26 ± feet, dense to very dense granular soils with occasional silt layers or silty zones were encountered which extended to the full depths explored.

The groundwater conditions encountered at the site were monitored at the time of installation of the piezometers when the field permeability tests were performed, and by the use of a continuous water level recording mechanism. The groundwater levels which were measured are indicated on the boring logs with the date measured at the appropriate elevation. Generally, groundwater

levels were encountered between 8 and 12 feet below the existing ground surface. The continuous recording device indicated 0.6 feet of variation at the location of Boring B-1 over a period of three days. It must be noted that groundwater levels may fluctuate due to variations in rainfall, tides, temperature and other factors at the time measurements are made.

Local variations from boring to boring from the above outlined conditions do exist. These generalized conditions summarized above were based on the subsurface conditions encountered at nine widely spaced exploration locations. The transition between differing soil types may be gradual. The nature and extent of variations from these conditions may not become apparent until construction begins or further explorations are undertaken. Should significant variations from the generalized conditions become evident, it will be necessary to reevaluate the following design considerations and recommendations of this report.

## GEOTECHNICAL ENGINEERING STUDY: CONCLUSIONS AND RECOMMENDATIONS

In the event that any changes in the nature, design or location of the elements or components of the graving dock or appurtenant facilities are planned, the conclusions and recommendations contained in this report should be reviewed and the conclusions modified or verified to reflect the changes. This report is written to determine the feasibility of a conceptual design from a geotechnical standpoint. This report will outline potential design and construction problems and outline areas for further study.

Excavated Slope Stability

The graving dock will be constructed by excavating about 20 feet below existing site grade. Preliminary site layout and the conceptual design were based on side slopes for the excavation of 2H:1V (Horizontal to Vertical). A preliminary evaluation of slope stability based on field shear strength test values indicated the proposed side slopes were potentially unstable due to the presence of very soft to soft clayey silts which were found to extend to about the planned depth of excavation. Based on the results of the preliminary analysis, a further sophisticated laboratory shear strength test program and a computer slope stability analysis were deemed warranted.

Properties of the in situ soils were evaluated based on the laboratory test procedures presented in Appendix B. The soil strength parameters and the planned slope configuration were input as data for the computer analysis. The program used was a commercial derivative of ICES-SLOPE. The program allows estimation of factor of safety against sliding for circular failure arcs.

In general terms, the calculated factor of safety is the ratio of the available shear resistance to the forces which act to produce landsliding. When the soil strength is equal to the slide producing forces, a factor of safety of 1.0 would exist and the slope would be in a condition of incipient failure.

Although the computer analysis is mathematically accurate, many uncertainties are associated with the assumed subsurface conditions, soil shear strength, and the location of the most critical failure surface. To provide a margin of safety against these uncertainties, a calculated factor of safety on the order of 1.5 for nonseismic conditions is considered

desirable in generally accepted geotechnical engineering practice. The slope stability analyses were predicated on the installation and successful operation of the system.

The first case studied by computer methods was the stability of a 2H:1V (Horizontal to Vertical) cut slope. The minimum calculated factor of safety against a rotational failure assuming quick (undrained) failure mechanism was about 1.1. Based on the above discussion, this was deemed not acceptable for a long term condition, but indicated such a slope may stand on a temporary basis.

To improve the stability of the excavated slope, the effects of using a stabilizing buttress constructed by a locally available slag material were studied. This material is already planned to be used to provide a working surface in the excavation bottom. Discussions with Mr. John Hodges of J. A. Jones indicated a maximum slope angle of 2½H:1V could be tolerated from space constraints. To reduce the quantities of material used in the buttress, a dual slope angle was analyzed. The slope configuration studied consisted of a 2 foot blanket of slag extending down from the ground surface on a 2H:1V slope to a point 8 feet above the planned final depth of excavation. This material would serve to protect the cut slope from erosion. From this point, the slag buttress is extended to the point where a 2½H:1V slope from the existing ground surface would intercept the proposed final excavated grade. This slope configuration was chosen for the purpose of allowing a uniform 2H:1V base slope for ease of excavation using dragline methods, and at the same time achieve the stabilizing effects of the slag material at the toe of the slope (where the failure arc exits in a cohesive soil). The slope configuration and location of

failure surfaces for the minimum calculated factors of safety are shown on Figure 4, Excavation Slope Configuration. The minimum factor of safety calculated for the slag buttress slope assuming a quick (undrained) failure mechanism is 1.3. This factor of safety against sliding may be improved by moving the point where the slope angle for the slag buttress changes upslope, by flattening overall the slope angle, or by reducing the forces which tend to produce sliding by removing load by excavating at the top of the slope. The calculated minimum factor of safety for a quick (undrained) failure mechanism for the composite slope with the ground surface elevation at (+)15 feet was about 1.4.

Because of the relatively long term use (1½ years ±) of the graving dock, the stability of the proposed excavated slope was also studied using the long term (drained or effective) strength parameters for the various soil units. The minimum calculated factor of safety for a major deep seated failure was 1.7. The slope stability analysis also indicated that shallow sloughing failures at the interface of the slag and underlying cohesive soils are somewhat more likely to occur. To improve the calculated factor of safety, the slag blanket thickness would have to be increased above the 2-foot thickness planned. This would significantly increase the overall slag quantities, and these costs should be weighed against the risk of isolated surficial failures which may be remedied by a regular maintenance program.

The factors of safety indicated for the quick (undrained) failure mechanism with the slag blanket is somewhat lower than what is desirable from a generally accepted geotechnical engineering

practice. The same discussion applies to the surficial sloughing failures. Consequently, it is our opinion that some risk exists relative to either the quick (undrained) failure mechanism for the buttressed slope or the surficial sloughing failure. The minimum calculated factor of safety against a slow (drained) failure mechanism is considered acceptable.

Preliminary Dewatering Considerations

A dewatering system will be necessary to excavate the graving dock area and to construct the bridge pontoons in the "dry". Based upon conversations with Erik Prestegaard, the facility will be in operation for up to 18 months and the excavation will be flooded at least once and dewatered a second time.

The excavation is located within 80 feet of the Blair Waterway and is anticipated to be approximately 650 by 480 feet in aerial extent with 555 feet fronting on the Blair Waterway (Figure 3). Existing grade is at approximately 18 feet MLLW with the anticipated excavation bottom to be approximately elevation (-)4 feet.

To evaluate the general subsurface conditions related to site dewatering, borings and Dutch cone probes, in situ hydraulic conductivity (permeability) tests, grain size analyses, water quality sampling with chemical analyses and water level measurements were accomplished. The field procedures are contained in Appendix A, while laboratory procedures and test results are shown in Appendix B.

The boring and Dutch cone probes indicate that the geologic conditions are relatively consistent beneath the site. A representative geologic profile is listed below while geologic cross-sections are shown on Figure 3.

Table I: Representative Geologic Profile

| Elevation (Feet msl) | Depth (Feet) | Thickness (Feet) | Soil Type |
|---|---|---|---|
| +18 to +9 | 0 - 9 | 9 | Sand |
| + 9 to -3 | 9 -21 | 12 | Clay/Silt |
| - 3 to -16 | 21-34 | 13 | Sand |
| -16 to -26 | 34-44 | 10 | Clay/Silt |
| -26 to ?* | 44-?* | ?+ | Sand |

*Extended to full depths explored.

Based on the anticipated excavation depth and subsurface conditions, the bottom of the excavation will penetrate through the upper silt layer and into the top several feet of the middle sand layer. Therefore, it is our opinion that the major goal of the dewatering system on this site will be to reduce the heads (artesian pressure) within the middle sand layer.

Groundwater levels within the middle sand layer were measured in three observation wells installed to an approximate elevation of (-)10 feet MLLW during our field explorations. The geologic data indicates that the middle sand layer is hydraulically coupled with the Blair Waterway and variation in heads within this layer will be largely dependent upon tidal fluctuations.

A conversation with personnel of the National Oceanographic and Atmospheric Administration suggests that normal high high tides reach a level of elevation +10 to +13 feet, but that tidal levels of +15 to +16 feet could occur during intense storms.

Observations made by Hart-Crowser in early March, 1981 indicate a high groundwater elevation of approximately +12 feet. However, it is recommended that a groundwater elevation of up to +15 feet be anticipated. Using a +15 foot elevation and an excavation bottom of (-)4 feet and assuming that the water level would be lowered to (-)6 (2 feet below the excavation bottom) a head reduction of approximately 21 feet should be achieved by a dewatering system.

Hydraulic conductivity (permeability) tests (k) were conducted in Borings B-1, B-2 and B-3. The falling head, basic time lag method (Hvorslev, 1951) was used to determine k values. These values are listed below:

Table 2: Hydraulic Conductivity Values

| | |
|---|---|
| B-1 | $2.5 \times 10^{-4}$ cm/sec. |
| B-2 | $6.6 \times 10^{-4}$ cm/sec. |
| B-3 | $2.5 \times 10^{-4}$ cm/sec. |

Grain size analyses were conducted on selected samples. The results of these analyses are shown on Figures B-2 through B-4.

An approximate plan flow net (Figure 5) was constructed to determine the general flow paths of water into the dewatered excavation. The flow net indicates that the major source of water to the excavation will be from the Blair Waterway and that lesser volumes will be derived from areas away from the waterway.

Water samples were obtained from each of the three borings and submitted to Laucks Testing Laboratories of Seattle for analysis. The results of the analyses are contained in Appendix B.

- General Recommendations

It is our opinion that wells or well points used in combination with an interior French drain system could be used to reduce heads if adequate vacuum lifts can be maintained. Based upon the hydrogeologic data, a dewatering system capable of lowering the head within the middle sand layer approximately 21 feet should be installed. This 21 foot head reduction translates into a minimum lowered groundwater level of approximately 24 feet below the existing grade. Heads within wells or well points will probably be at a lower level to achieve the necessary head reduction.

The well or well point system should be fully penetrating through the middle sand layer. It is not recommended that the lower sand layer be penetrated in that pumpage from this layer will not significantly aid in head reduction of the middle sand layer. Penetration of the lower layer will result in increased pumpage volumes and may cause problems be improving the hydraulic coupling of the middle and lower sand layers.

Well or well point spacing can be aided by using the flow net shown in Figure 5. It is anticipated that the major flow will be derived from the Blair Waterway, therefore, the closest well spacing may be necessary along this side of the excavation.

It is recommended that the heads within the middle sand strata be monitored through the use of observation wells while the dewatering system is in operation, and that these wells indicate reduction in the heads at or below the planned final depth of excavation, prior to mass excavation. After the well or well point system has reduced the head within the middle sand layer below the bottom of excavation and excavation has been completed, French drains could be installed. These French drains could improve drainage within the excavation bottom and provide an increased safety factor to the dewatering operation. The quality of the pumped water needs to be considered for the successful long term operation of the system. The results and water quality considerations are discussed in a following section.

It should be noted that head reduction within the middle sand should be completed prior to excavation near the upper silt/middle sand contact. If the heads within the middle sand are not reduced sufficiently before excavation, "sand boils" may occur which may preclude the use of spread footings.

It is recommended that Hart-Crowser & Associates be afforded the opportunity to review the design of the planned dewatering system. It is also recommended that Hart-Crowser & Associates observe the installation of the dewatering system and observe and monitor the groundwater levels in the various observation wells to evaluate the performance of the planned dewatering system.

## Water Quality

Groundwater samples were obtained from Borings B-1, B-2 and B-3 for chemical analysis. The purpose of the analyses was to provide data to assess possible operational difficulties of the dewatering system related to incrustation and/or corrosion. In addition, analyses for organic chemical and phenols were accomplished to assess possible difficulties with pumped water disposal.

The field sampling procedures are described in Appendix A. The Laucks Testing Laboratories analytical results are contained in Appendix C.

## Foundation Support

- Graving Dock

In the graving dock excavation, it appears feasible to support the pontoons by spread footings founded in the medium dense to dense sand which was encountered in various explorations at or about the planned depth of excavation. In the graving dock area, current excavation plans call for removal of about 18 to 20 feet of soil down to an elevation of (-)4 feet. A slag blanket of about 2 feet in thickness is planned to facilitate traffic across the bottom of the excavation.

The settlement response for the underlying foundation soils was modeled by two methods. The settlement response of the granular soils was modeled using elastic methods developed by Schmertmann for long footings founded over sand deposits using criteria from the Dutch cone penetrometer probes. The settlement behavior of the cohesive soils was modeled using the results

of one-dimensional consolidation tests performed in our laboratory on undisturbed samples recovered from the borings. Because of the reduction in vertical stresses achieved by the excavation, the consolidation response was modeled using the slope of the recompression curve for the cohesive soils. The predicted elastic settlements in the granular soils were also smaller due to the effects of preloading.

2¼ ksf

The settlement response was evaluated for two cases. The first case studied was a uniform distributed load equal to the dead weight of the completed pontoon (13,000 kips) applied over the plan area of a pontoon. Predicted settlements for this case ranged from about 1 inch to about 1½ inches based on a probe by probe basis.

Additionally, the settlement response for a long strip footing three feet in width loaded to 4,000 psf (pounds per square foot) was evaluated. Estimated settlement magnitudes on the order of ½-inch to 1¼-inches were indicated. Differential settlement of the pontoon supports on the order of 3/4-inch may be anticipated. Discussions with Mr. Erik Prestegaard and Mr. Joel Dean of Storch Corporation/Engineers indicate this range of settlement would be considered tolerable. It should be noted that some variation in the contact between the upper cohesive unit and the dense sand strata may be anticipated. Local overexcavation and replacement of unsuitable foundation soils with compacted structural fill may be required.

The above settlement analysis and recommended bearing pressures are predicated on footings founded in dense undisturbed natural sands as disclosed in the explorations. Of particular concern is the potential detrimental effects of loosening the dense sand strata planned for footing support by indiscriminate sump pumping

or by particle migration due to seepage forces imposed by unrelieved artesian pressures in the dense sand strata. Uncontrolled excavation prior to relieving these artesian pressures could result in "blowing" the bottom of the excavation and could force pile support for the pontoons within the affected areas. Areas which are excavated prior to installation and operation of the dewatering system may be considered suspect. It is recommended that the integrity of the granular soils be evaluated after completion of the mass excavation for spread footing support. A quick and reliable method would be a number of additional Dutch cone penetrometer probes within the excavation limits. These additional explorations could be advanced relatively economically and provide immediate data for analysis.

- Casting Yard

Pursuant to conversations with Mr. Joel Dean and Mr. Erik Prestegaard, we have also studied the settlement response of spread footings or slabs for support of forms for precast concrete elements for the pontoons. Based on the above mentioned conversations, it appears that the heaviest element would impose loads of 5 kips per lineal foot distributed over an area 18 feet wide and 27 feet long. This results in a uniformly distributed load on the order of 0.3 ~~ksf~~ which (kips per square foot).

The settlement response of footings in the casting yard were modeled using elastic parameters as derived from the Dutch cone penetrometer probes. The settlement response due to this uniformly distributed load is estimated to be on the order of 1½ inches within the upper cohesive soils.

The soils in this area will not have been relieved by excavation such as those beneath the graving dock. These soils must be considered normally consolidated, and as such will experience significantly greater deformation for a given load than those beneath the excavation.

?
uniform

These settlement values are predicated on assuming the existing subsurface conditions in the casting yard are uniform and similar to those disclosed in the borings and probes. No explorations were accomplished in the casting yard and variations from the assumed conditions may exist. It is recommended that the subsurface conditions be explored to verify these assumptions.

Significantly larger settlement may occur and the potential for bearing capacity failures exists if footings are founded directly in soft cohesive soils or if the dredged sand fill is absent or thins. In addition, settlement magnitudes of the soils in the casting yard may be significantly greater if the imposed loads on organic soils are sufficient to initiate secondary consolidation within the organic soils. The amount of secondary consolidation which may occur would depend on the nature and distribution of the organic soils, the configuration and duration of the loads, and other factors. Secondary consolidation is a long term phenomena. It is recommended that provisions for leveling the forms on a periodic basis be included in the design.

- Shipway

It is planned to construct and launch concrete anchors for the planned floating bridge on a shipway to be located immediately to the east of the proposed graving dock. Preliminary plans for support of the shipway call for the use of prefabricated steel

pipe piling (16 inches in diameter with 0.375-inch wall) driven with a closed end on a 10-foot grid. The piles would be connected by prefabricated panels. The panels and piles for the inclined portion of the shipway would support the anchors only briefly as the anchor is moved down the shipway on skids.

Preliminary plans for the shipway call for a 10 percent grade. The inclined section of the shipway will begin at a point 200 feet from the Blair Waterway, and will extend to elevation (-)15 feet. Assuming a uniform 2H:1V slope for the natural soil slope into the Blair Waterway, the piles for the final portion of the shipway (below about elevation (-)7.5) will extend above the natural soil slope. A maximum unsupported length of about 30 feet for piles may be anticipated.

Typical concrete anchors for the bridge would each weigh about 2,400 kips. Anchors are planned to be up to 45 feet in diameter. The average contact pressure over the plan anchor area would be on the order of 1.5 ksf. An areal load of this extent and magnitude would cause significant settlement in the upper clay silt soils, would most likely trigger secondary consolidation in organic soils found within the upper clay silts and approaches the theoretical bearing capacity for the upper soft cohesive soils. Due to the above mentioned criteria, it was deemed necessary to support the planned anchor loads on deep foundations.

The pile parameters are derived from Dutch cone penetrometer results and empirical correlation with Standard Penetration test results from the nearest explorations advanced in the graving dock area. No explorations were advanced in the area of the shipway and variations from the conditions assumed in the

analyses may exist. It is recommended that the subsurface conditions be further explored in the shipway area prior to the final design of shipway support elements.

It appears that about 16 piles would be utilized to support the anchor loads at a given time, and a vertical load on the order of about 75 tons per pile may be imposed. If the piles are founded in the middle sand strata between about elevation 0 to (-)15 feet, the vertical loads transferred by shear and end bearing from the piles will be imposed on the underlying cohesive soils. Utilizing the consolidation and the Dutch cone penetrometer parameters for the lower cohesive soils, pile settlements on the order of 7 to 8 inches may occur due to consolidation settlement of the underlying cohesive soils.

Additional settlement from secondary consolidation within any organic soils within the cohesive deposit may be anticipated. Both consolidation and secondary settlement are long term and time dependent, and would occur if the anchor loads would be imposed and maintained for extended periods of time.

Preliminary discussions with Mr. Ted Bohlander of Storch Corporation/Engineers indicates three or more anchors may be cast simultaneously, and that casting times of three to four months may be required per anchor. A significant portion of the consolidation settlement could occur over the time period needed for casting and fabricating the anchors, and secondary consolidation may be triggered within the cohesive soils during this time period, as well. For areas which will support the anchor loads over extended periods of time, it may be advisable to support the imposed loads on piles which penetrate the lower cohesive soils with pile tips bearing in the dense to very dense sands disclosed at about elevation (-)35 feet.

With the planned shipway configuration and the assumed subsurface conditions, pile support of the temporary anchor loads may be achieved in the medium dense to dense sand strata disclosed between about elevation 0 and (-)15 for the northernmost inclined portion of the shipway. It is recommended that a minimum pile embedment on the order of 10 feet into the medium dense to dense sands be achieved, and that a minimum overall pile embedment on the order of 20 feet be maintained. Proceeding down the shipway, at a point roughly 100 feet from the Blair Waterway, it may be necessary to extend the piles to bear in the lower dense to very dense sands disclosed at about elevation (-)35 feet, as the planned shipway grade begins to encoach on the dense sand bearing strata. Individual vertical pile capacity should be verified in the field by the method outlined in the following paragraph. Piles which do not reach the required vertical capacity in the middle sand strata should be driven to the required vertical capacity with tips in the lower sand deposit.

Because of possible variations in soil conditions, we recommend the vertical pile capacity be verified in the field based on a dynamic pile driving formula. Such a formula should allow for variations in various physical factors such as the hammer energy, size, type and length of piles, and modulus of elasticity of the pile materials. An appropriate formula would be the Danish ($S_o$) pile driving formula. The $S_o$ pile driving formula is presented below:

$$Q = \left[ \frac{a\ E_r}{S + \frac{1}{2} S_o} \right]$$

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 = $\frac{\text{delivered energy}}{\text{rate energy}}$ (about 0.8)

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

$L$ = length of pile, feet

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

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

It is recommended that a factor of safety of 2.0 be applied to the ultimate capacity as determined by the pile driving formula to achieve the design value based on initial driving. Engineering experience indicates that the granular soils may experience partial liquefaction during pile driving operations. This may result in a partial temporary loss of strength. As the pore pressures generated during pile driving dissipate, the soil shearing resistance will tend to increase to original values. Because of this effect, pile resistance derived from dynamic formula based on initial driving may not accurately indicate the long term capacity of the driven piles. If a factor of safety of 2.0 is not achieved at the anticipated bearing depth during initial driving, a number of piles should be redriven for a short distance following a waiting period. Dynamic pile capacity of redriven piles should be evaluated using a factor of safety of 2.5.

Because of the uncertainties in the subsurface conditions at the shipway location, a program of test driving of piling at different locations along the shipway may be warranted to evaluate potential different driving and bearing conditions along the shipway alignment.

The installation of all piles should be observed by a competent soils engineer or engineering geologist. Observation of pile handling, pile hammer operating characteristics, and pile penetration resistance would make it possible to confirm the penetration depth, monitor variations in subsurface conditions, and evaluate the pile capacity using dynamic pile driving relations. Because of Hart-Crowser & Associates familiarity with the subsurface conditions and design assumptions, it is recommended that Hart-Crowser & Associates monitor the installation of the piles and evaluate the pile driving results.

Sheet Piles for Closure

The preliminary plans call for a driven steel sheet pile closure structure. The approximate location and dimension of the structure is shown on Figure 2. The pontoons will be completed in the "dry" within the dewatered graving dock excavation, and the graving dock will then be subsequently flooded. At that time, sheet piles which effect closure will be extracted and the pontoon elements floated out to the Blair Waterway.

Sheet piles will resist loads imposed both by lateral earth pressures and those due to unbalanced hydrostatic pressures. The lateral soil pressure which act against the sheet pile walls depend on the nature of the natural soils as well as drainage conditions behind the wall. The sheet pile walls have been analyzed for two cases; those piles driven along the sides of the closure structure, and those piles in the closure section.

The forces which act on the sheet pile walls which would be driven along the sides of the closure structure are shown on Figure 6. These forces are based on the average conditions disclosed in the borings and Dutch cone penetrometer probes. It is recommended that the actual subsurface conditions at the proposed closure section be verified prior to final design. Discussions in the USS Steel Sheet Piling Design Manual indicate cantilever sheet pile walls are generally limited by economics to a maximum height of about 15 feet. For the proposed closure structure, the proposed sheet pile wall will extend from about (+)18 to (-)2, somewhat beyond the limits of a cantilever wall. Therefore, the wall was analyzed by methods outlined for anchored bulkheads utilizing the free earth support method.

The results of the analyses indicated the sheet piles should extend beyond the dense sand strata by about 5 feet into the lower cohesive soils. It is common practice to increase sheet pile penetration by 20 to 40 percent to provide a factor of safety against "kick-out" at the toe. For the wall configuration shown on Figure 6, an anchor pull on the order of 8 kips per lineal foot would be required for stability of the wall for an anchor located 4 feet below the top of the wall. The required section modulus can be determined by drawing the shear diagram and finding the moment at the point of zero shear. The required section may be modified to reflect the relative wall stiffness by methods outlined by Rowe.

The lateral loading imposed against the sheet piles which effect the closure is a function of the hydrostatic pressures imposed by tidal fluctuations. These pressures and the resisting forces on the sheet pile wall are shown on Figure 6. Because of the wall height, the wall was initially analyzed for cantilever support. Embedment depths indicated from the analyses were quite large, with stability for the sheet pile walls requiring embedment depths well in excess of 20

feet. The wall was subsequently analyzed as if it were an anchored bulkhead. Sheet pile embedment depths on the order of about 10 feet were indicated. An anchor pull or other lateral resisting force from braces or others means on the order of 4½ kips per lineal foot of wall was required for the wall configuration shown on Figure 6. From a seepage standpoint, it is recommended that the sheet piles for the closure section be driven to a minimum embedment of 5 feet into the lower cohesive deposit. This would coincide closely to the 40 percent increase for a factor of safety for resistance to "kick-out". Sizing of the sheet piles should be done according to the provisions previously outlined. A similar analysis should be completed for the maximum storm tide level. An increase in sheet pile penetration of 20 percent for the more extreme loading condition is considered warranted.

## RECOMMENDATIONS FOR ADDITIONAL SERVICES

The recommendations included in this report are based on the results of nine widely spaced explorations. Site specific information regarding subsurface conditions in the casting yard, along the proposed shipway and for the closure structure were not disclosed in the preliminary study. In addition, the potential for disturbance of the medium dense to dense sand strata planned for spread footing support of the pontoons due to uncontrolled excavation and sump pumping prior to installation and successful operation of the dewatering system, is apparent.

It is, therefore, recommended that the subsurface conditions which were assumed for the geotechnical analyses for the various project components be verified by further field explorations. Additional geotechnical studies utilizing the actual subsurface conditions disclosed for the various components of the project would be warranted if significant changes are noted. The relative effects of

disturbance within the sand strata planned for spread footing support within the graving dock should be evaluated after the mass excavation is complete. Areas which are excavated prior to installation and operation of the dewatering system should be evaluated further at that time.

It is recommended that Hart-Crowser & Associates provide periodic construction and geotechnical engineering services during the earthwork, dewatering and pile driving operations at the site. The purpose of observing the mass excavation would be to evaluate stable side slopes for final grading and placement of slag blanket and buttress material. It is recommended that Hart-Crowser & Associates observe and monitor the groundwater levels in various observation wells to evaluate the performance of the planned dewatering system. It is also recommended that Hart-Crowser & Associates be provided an opportunity to review the design of the planned dewatering system.

It is recommended that Hart-Crowser & Associates observe the pile installation for support of the shipway and anchors and evaluate vertical pile capacity using dynamic pile driving relations. Because of the uncertain subsurface conditions in the vicinity of the shipway, a program of test driving piles is recommended at different locations to evaluate potential different driving and bearing conditions along the shipway alignment.

It is recommended that Hart-Crowser & Associates be afforded the opportunity for a general review of final plans and specifications in order that the geotechncial recommendations contained in this report be properly interpreted and implemented in the design and specifications.

HART-CROWSER & ASSOCIATES, INC.

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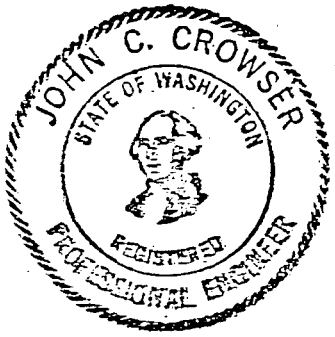
Project Engineer

MATTHEW G. DALTON
Sr. Project Hydrogeologist

JOHN C. CROWSER, P.E.
Sr. Vice President

KWG:MGD:JCC:gpa



# VICINITY MAP PORT OF TACOMA

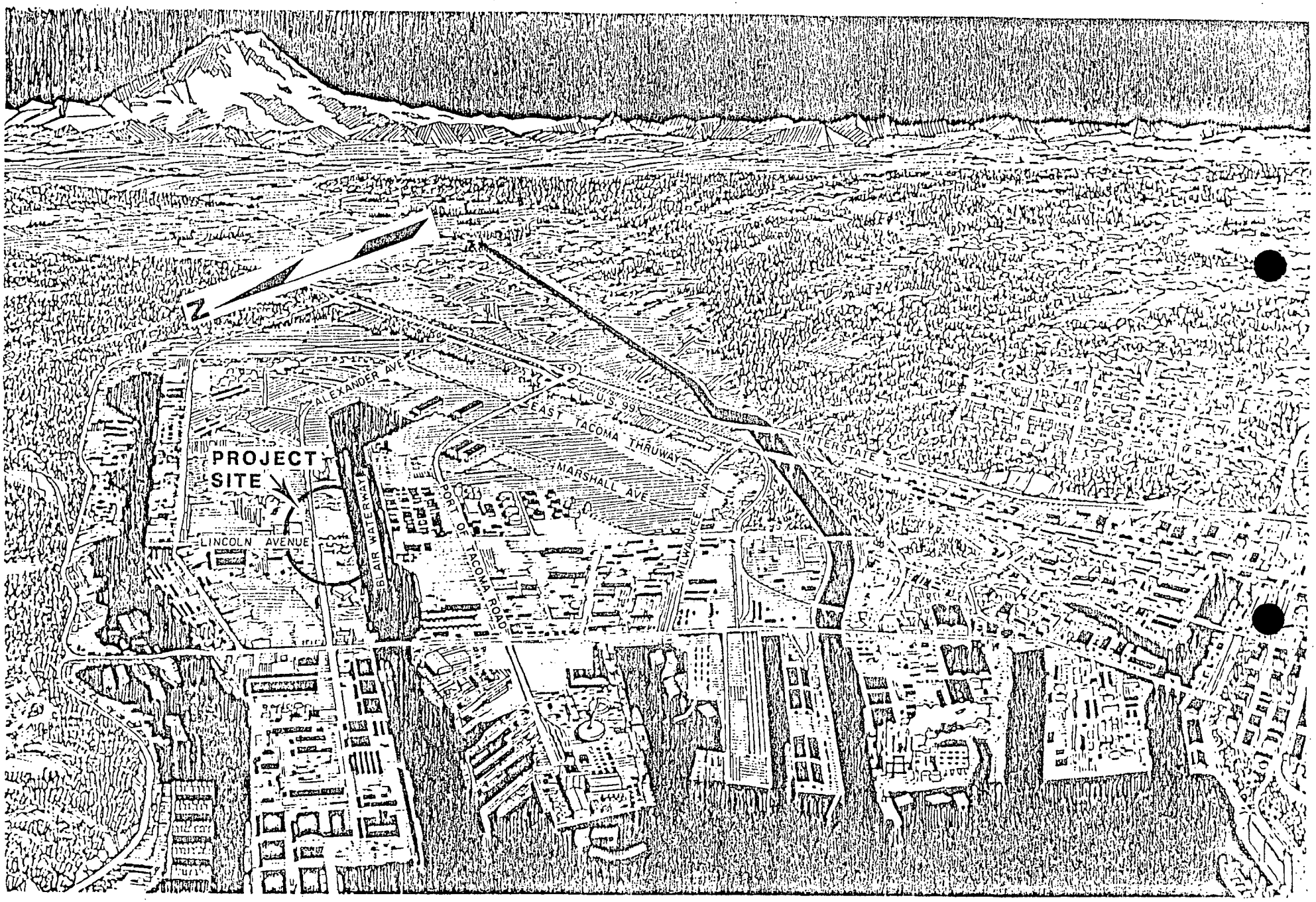



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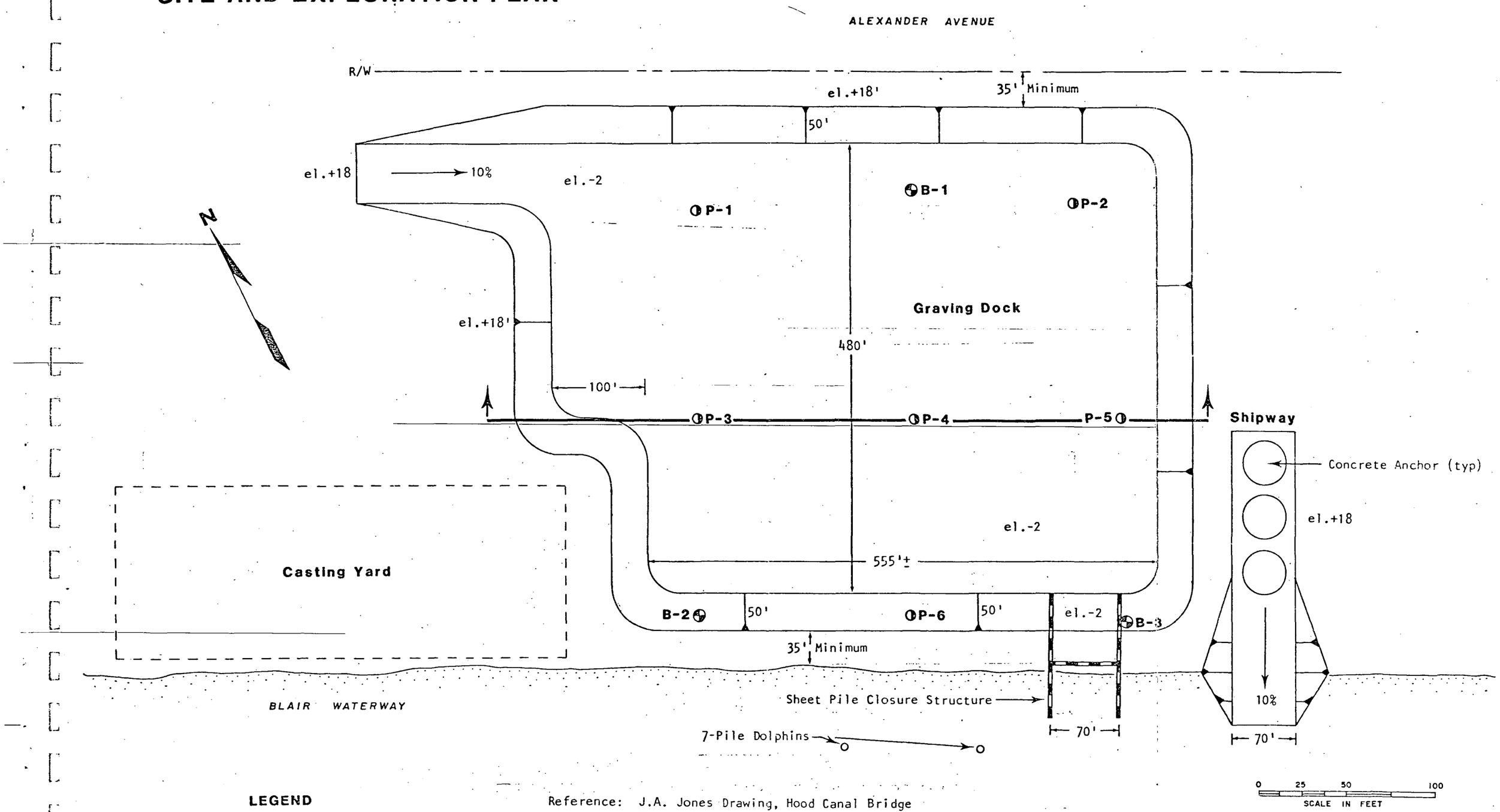
Figure 1

# SITE AND EXPLORATION PLAN




**LEGEND**

**P-1** Probe Number and Approximate Location

**B-1** Boring Number and Approximate Location

Reference: J.A. Jones Drawing, Hood Canal Bridge Tacoma Fabrication Facility, REP 81021

Note: Drawing modified to reflect 2½:1 side slopes and 35' setback from Blair Waterway.

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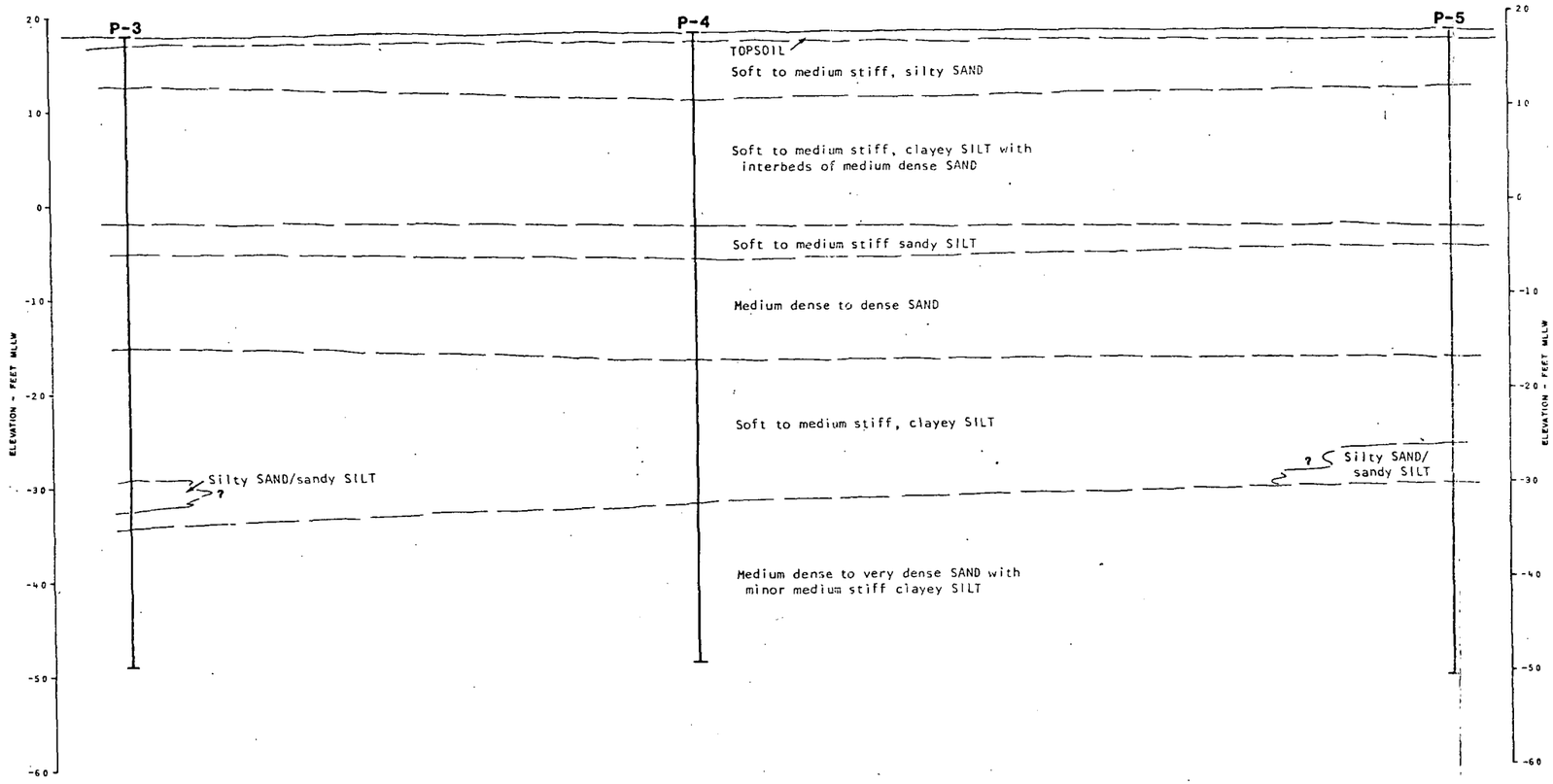
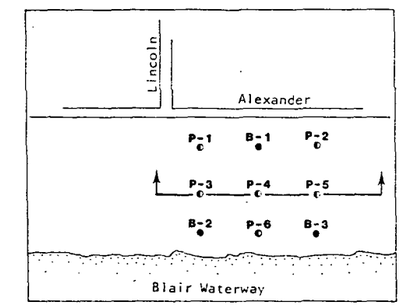
**Figure 2**

## GENERALIZED SUBSURFACE PROFILE




**PLAN VIEW**
NOT TO SCALE

Note:
1. Assumed Ground Surface Elevation +18' MLLW
2. The stratum lines are based upon interpolation between borings and may not represent actual subsurface conditions.

0 20 40 80
SCALE IN FEET
VERTICAL EXAGGERATION = 4X

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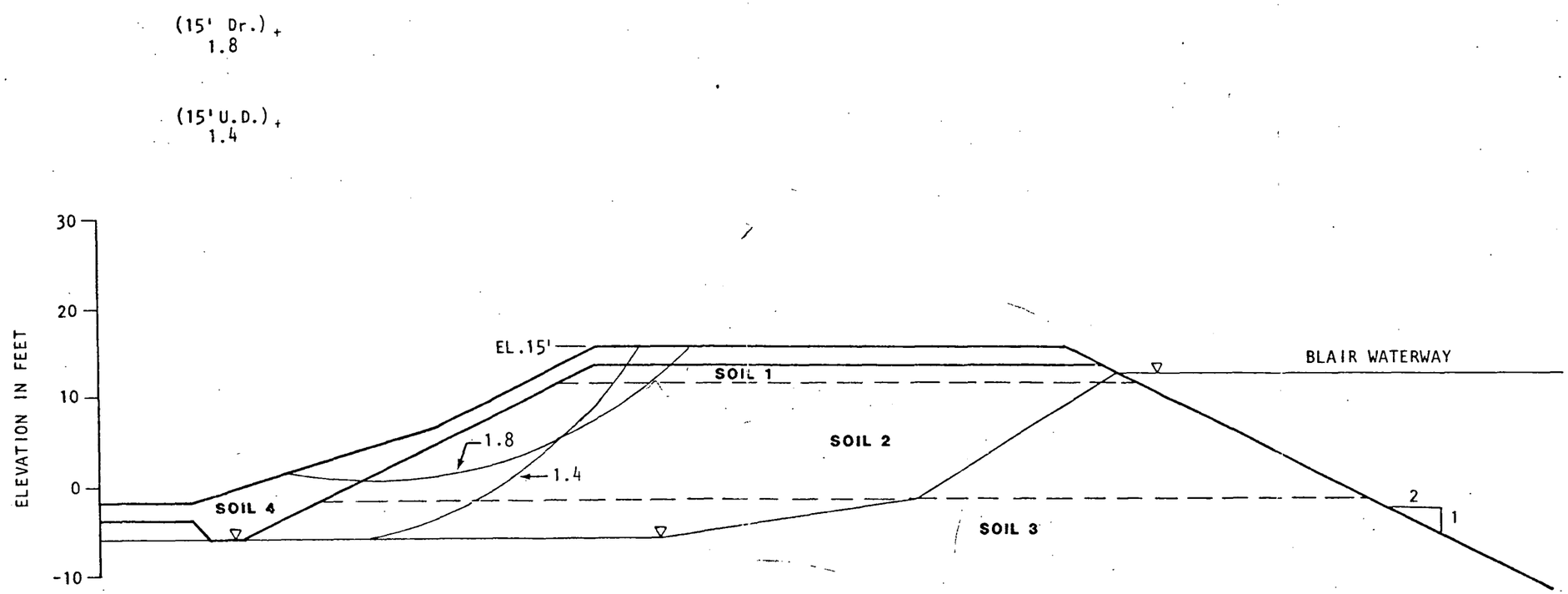
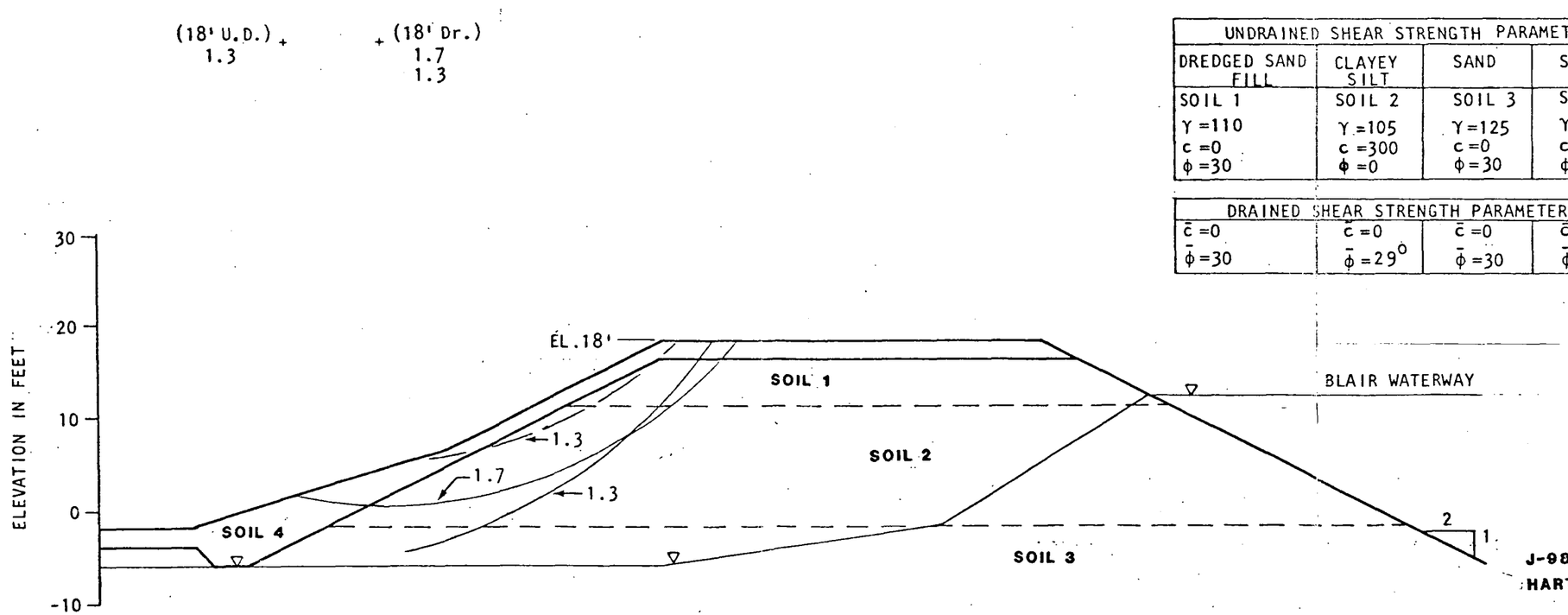
**Figure 3**

# EXCAVATION SLOPE CONFIGURATION




| UNDRAINED SHEAR STRENGTH PARAMETERS | | | |
|---|---|---|---|
| DREDGED SAND FILL | CLAYEY SILT | SAND | SLAG FILL |
| SOIL 1 | SOIL 2 | SOIL 3 | SOIL 4 |
| $\gamma = 110$ | $\gamma = 105$ | $\gamma = 125$ | $\gamma = 135$ |
| $c = 0$ | $c = 300$ | $c = 0$ | $c = 0$ |
| $\phi = 30$ | $\phi = 0$ | $\phi = 30$ | $\phi = 35$ |

| DRAINED SHEAR STRENGTH PARAMETERS | | | |
|---|---|---|---|
| $\bar{c} = 0$ | $\bar{c} = 0$ | $\bar{c} = 0$ | $\bar{c} = 0$ |
| $\bar{\phi} = 30$ | $\bar{\phi} = 29^{\circ}$ | $\bar{\phi} = 30$ | $\bar{\phi} = 35$ |

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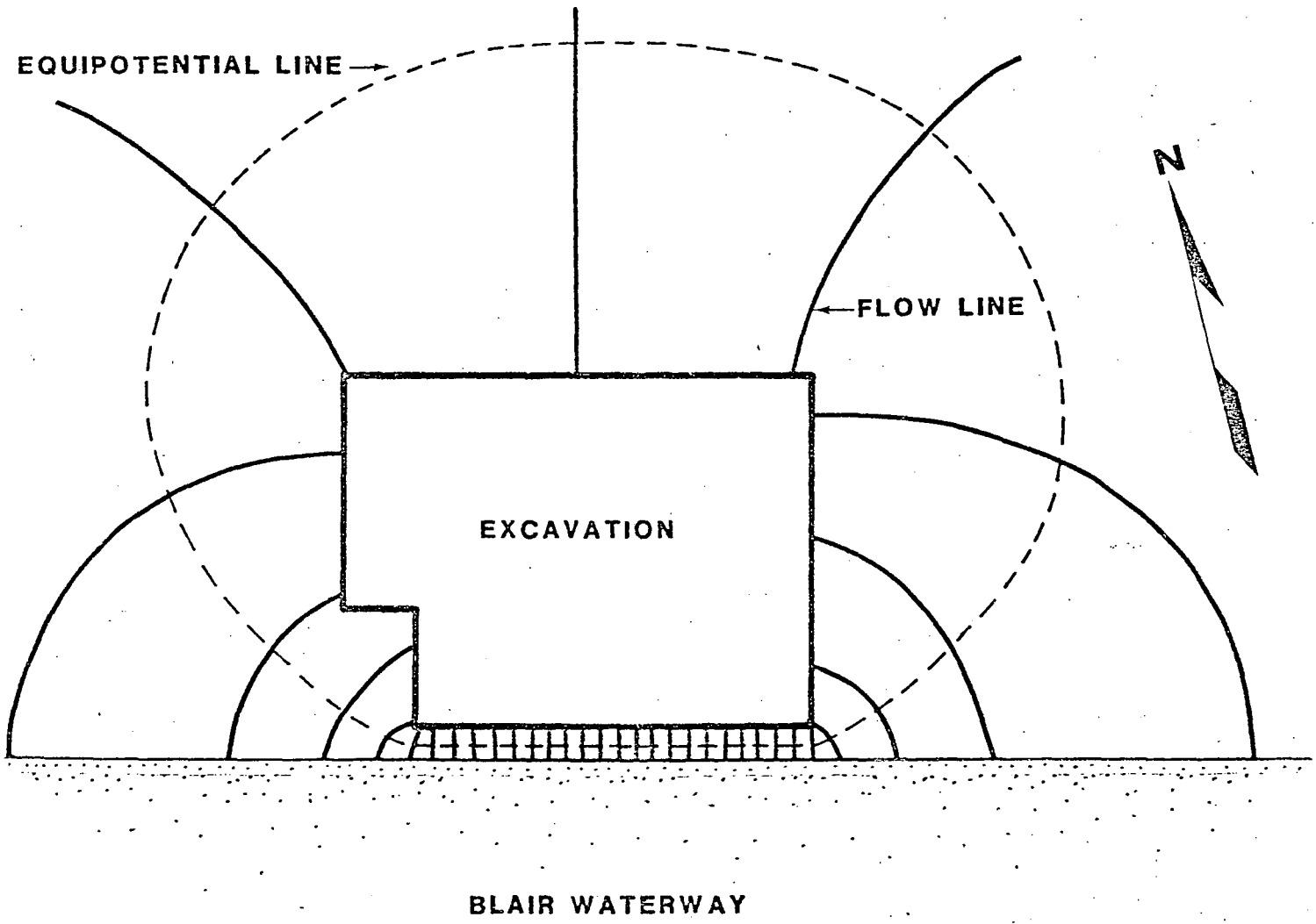
Figure 4

# APPROXIMATE FLOW NET




(Not to Scale)

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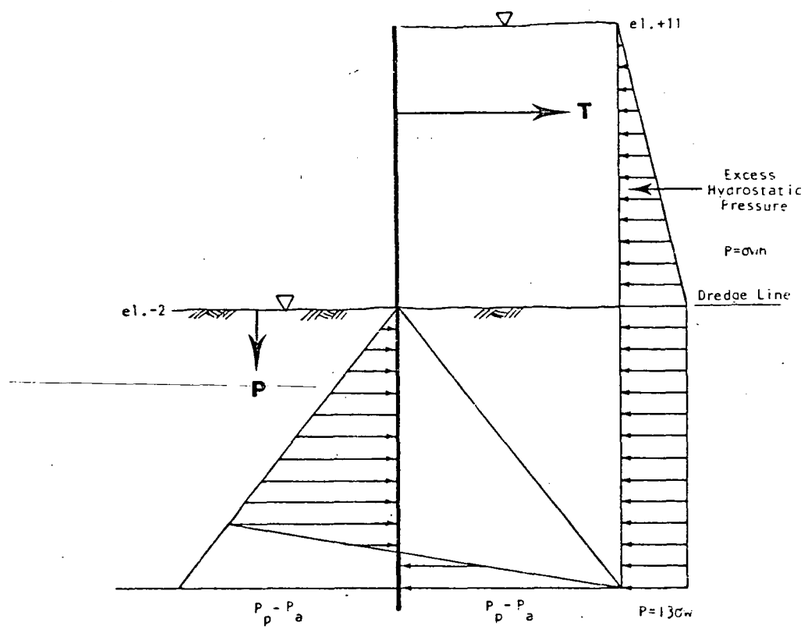
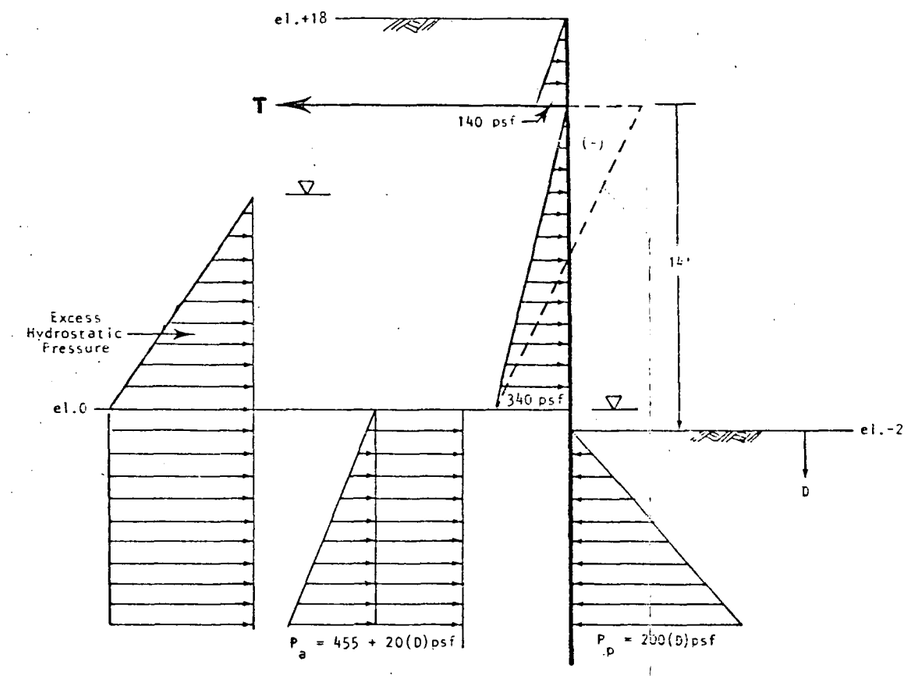
Figure 5

# LATERAL PRESSURE DIAGRAMS

**A. Sheet Piles for Closure Structure**




**B. Sheet Piles along Side of Closure Structure**




Reference: United States Steel, Steel Sheet Piling Design Manual

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Figure 6

# APPENDIX A
# FIELD EXPLORATIONS

The field exploration program accomplished for this study consisted of six Dutch cone penetrometer probes, three hollow-stem auger borings, three piezometer installations, field permeability testing and groundwater sampling. The locations of the explorations are shown on Figure 2, Site and Exploration Plan. Detailed interpretive logs of the explorations are shown on Figures A-3 through A-11. Soil stratification was generalized from drilling observations and samples recovered from the exploration program. Material changes shown on the exploration logs are often gradational in nature and conditions may vary at locations distant from where the explorations were undertaken.

The location of the Dutch cone probes and the auger borings were staked in the field by representatives of Hart-Crowser & Associates. The locations of the explorations were found by taping from physical features in the field. The elevations for the exploration locations were determined by interpolation from topographic information furnished by the Port of Tacoma. The locations and elevations of the explorations should be considered accurate to the degree implied by the method used.

The Dutch cone probes were completed between February 9 and 16, 1981. The auger borings were drilled on February 11, 12 and 16, 1981. The field permeability testing was completed on February 19 and 20, 1981, while the water quality samples were collected on March 5, 1981.

## Dutch Cone Probes

The Dutch cone probes, designated P-1 through P-6, were advanced to depths ranging from 52 to 68.5 feet below the existing ground surface.

The Dutch cone probes were accomplished by Subterranean, Inc. of Gig Harbor, Washington under subcontract to Hart-Crowser & Associates. The principles of the Dutch cone system is shown on Figure A-1. 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 in Figure A-2.

Generally, a friction ratio value 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 indicate a clayey silt or clay. The descriptive soil interpretations presented on the probe logs have been developed using Figure A-2 as a guideline with modifications according to correlations of soil types disclosed in borings performed at the site and other locations in the vicinity and careful interpretation of the probe results. The results of the penetrometer probes accomplished for this study and interpretive soil logs are presented in Figures A-3 through A-8.

## Soil Test Borings

The soil test borings, designated B-1 through B-3, were advanced to depths ranging from 79 to 83½ feet below the existing site grade. The borings were also advanced using hollow-stem auger methods by Subterranean, Inc. of Gig Harbor, Washington under subcontract to Hart-Crowser & Associates, Inc.

The borings were continuously observed and the soil conditions were logged by Mr. Russ Prior, an engineering geologist from Hart-Crowser & Associates. Representative split spoon and Shelby tube samples were obtained in the field. These samples were transported to the Seattle laboratory of Hart-Crowser & Associates for further classification and testing. A detailed log noting the stratification encountered in each boring has been prepared, and is shown on Figures A-9 through A-11.

Representative soil samples were generally obtained continuously in zones of particular interest and at 5 foot intervals elsewhere. The representative, disturbed samples were obtained using the Standard Penetration test procedure in Borings B-1 through B-3. This test is a means of determining the relative density of granular soil and the consistency of a 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 each 6-inch increment was recorded by our geologist. The number of blows required to drive the sampler the last 12 inches is the Standard Penetration Resistance shown on the Boring Logs, Figures A-9 through A-11.

To obtain high quality samples of the cohesive and compressible organic soils for strength and consolidation testing 3 inch Ø thin wall steel Shelby tubes were hydraulically pushed into the cohesive subsoils at the desired depths. This sampling method reduces the sample disturbance which occurs when a sample is driven, as in the Standard Penetration Test procedure.

## Field Permeability Testing

Three hydraulic conductivity tests (field permeability tests) were conducted at the Borings B-1, B-2, B-3 locations shown on Figure 2. Each test hole was drilled adjacent to a sampled boring using a hollow-stem auger. A 5 foot long 10-slot PVC screen with associated flush coupled riser pipe was telescoped down the center of the auger

so that the mid-point of the screen was approximately in the vertical middle of the middle sand layer. A sand filter was placed adjacent to the screened section and the auger was withdrawn. Development was accomplished using compressed air which lifted silt particles to the surface.

After each test well was installed and developed a falling head test was conducted (February 19 and 20, 1981). The test consisted of rapidly raising the static level within the well by adding water and noting the falling levels with time. At each hole three trials were conducted. The data was analyzed using the Basic Time Lag method developed by Hvorslev (1951).

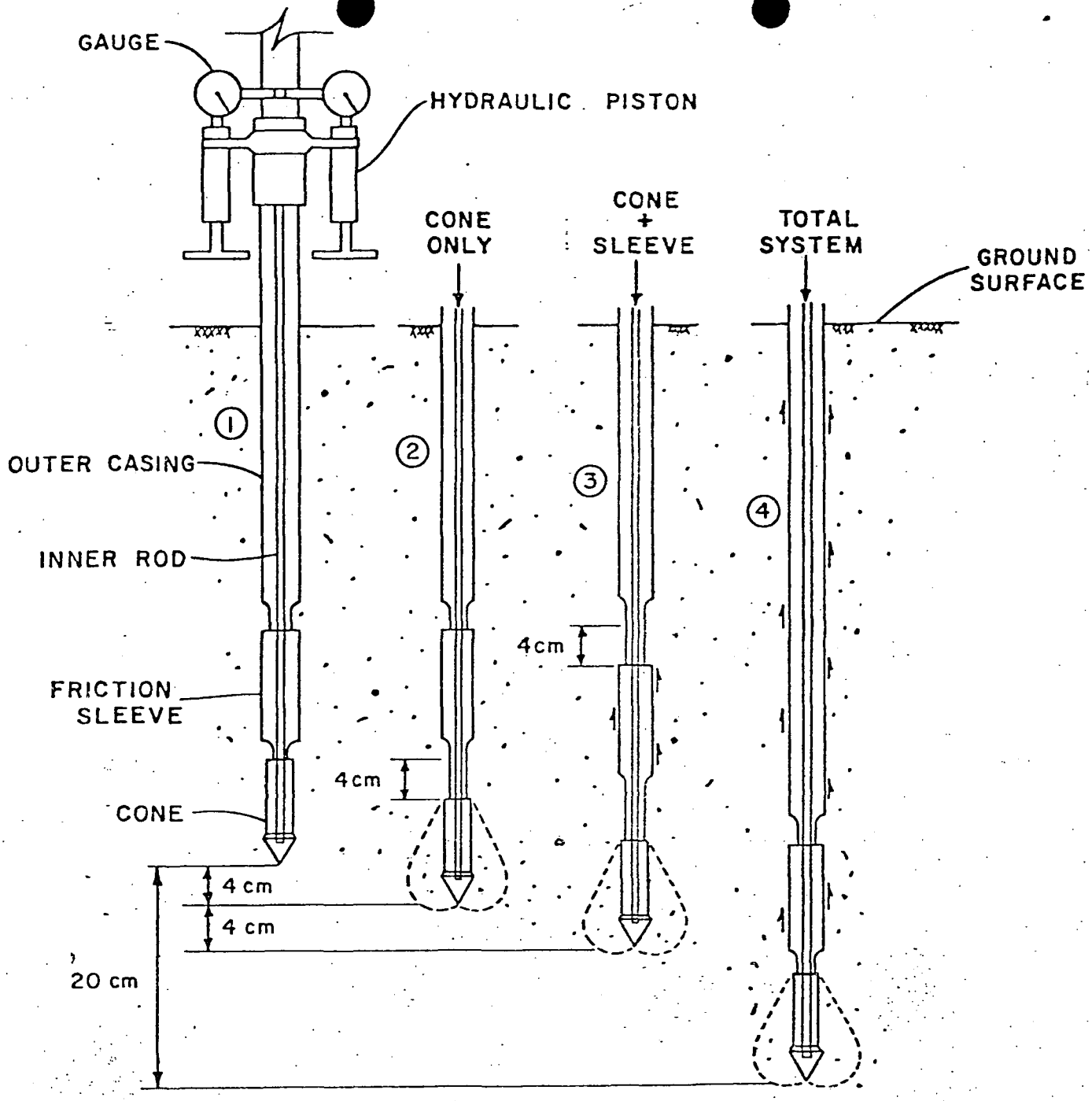
Water Quality Sampling

Water quality sampling was conducted on March 5, 1981. Each boring was sampled using a nitrogen-gas displacement pump. Prior to sampling three casing volumes were removed. The samples were placed in sample containers with appropriate preservatives, and transported to the testing laboratory the same day. The Laucks Testing Laboratories of Seattle provided the sample containers and preservatives and conducted the analyses.

## REFERENCES CITED

Hvorslev, M.J., 1951, Time Lag and Soil Permeability in Ground-Water Observations; Waterways Experiment Station Bull No. 36; Vicksburg, Mississippi.

# 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 4cm, and the point resistance measured.
2. From position 2 the point is again pushed 4cm 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 20cm by pushing the outer casing to positon 4 from which the next test is accomplished.

NOTES:

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

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Figure A-1

# DUTCH CONE SOIL CLASSIFICATION CHART

after Schmertmann, 1969

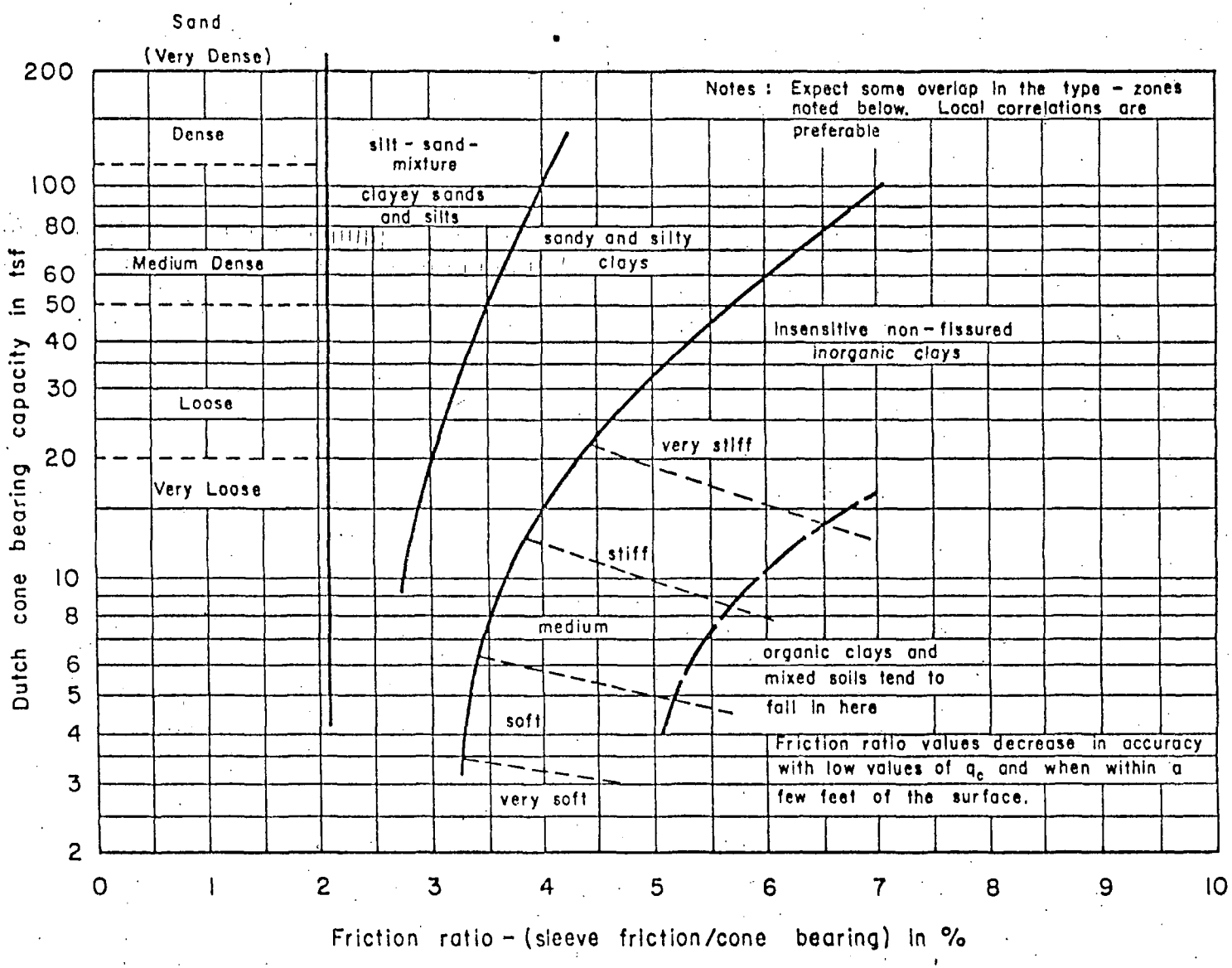


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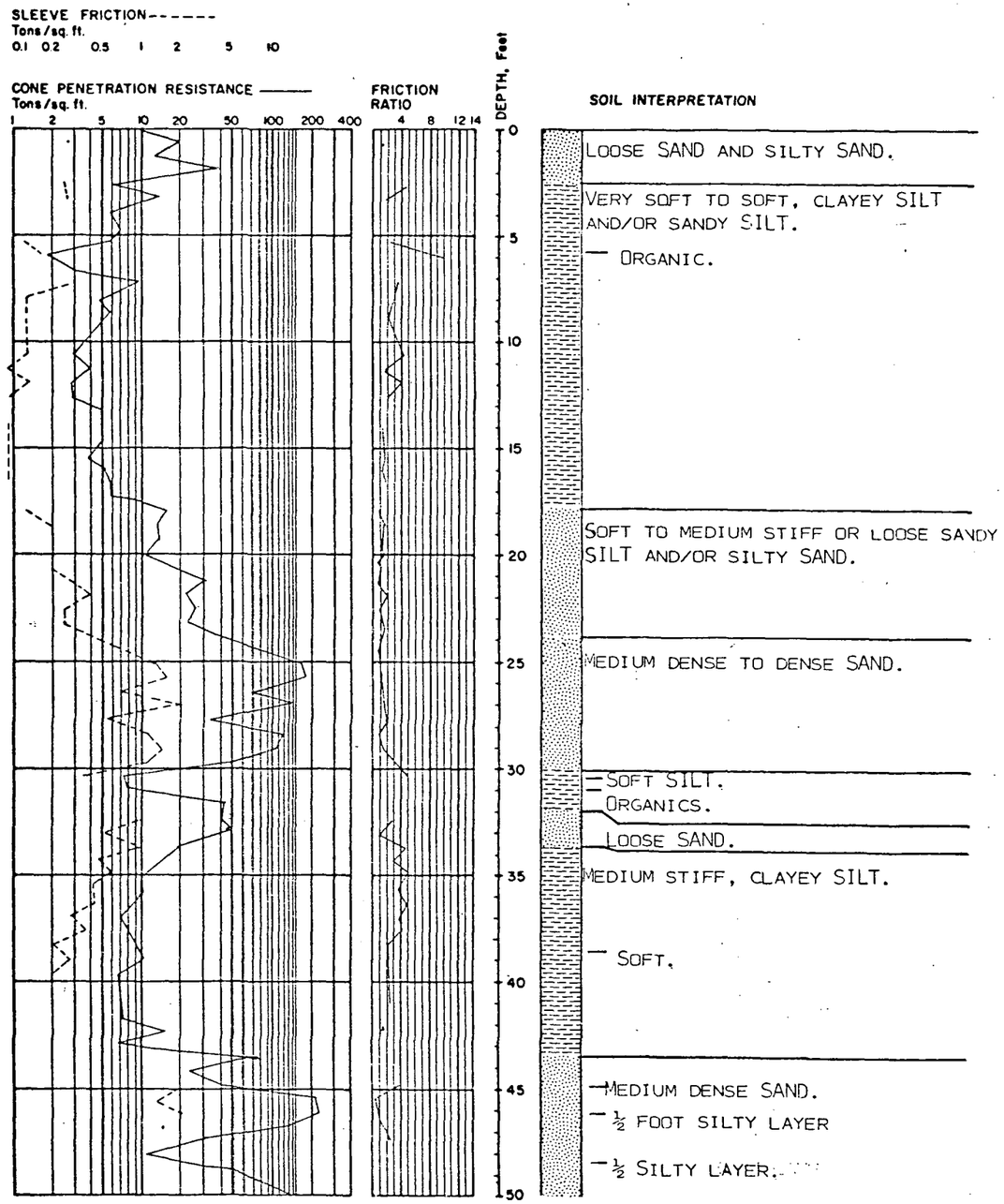
Figure A-2

# LOG OF PROBE P-1



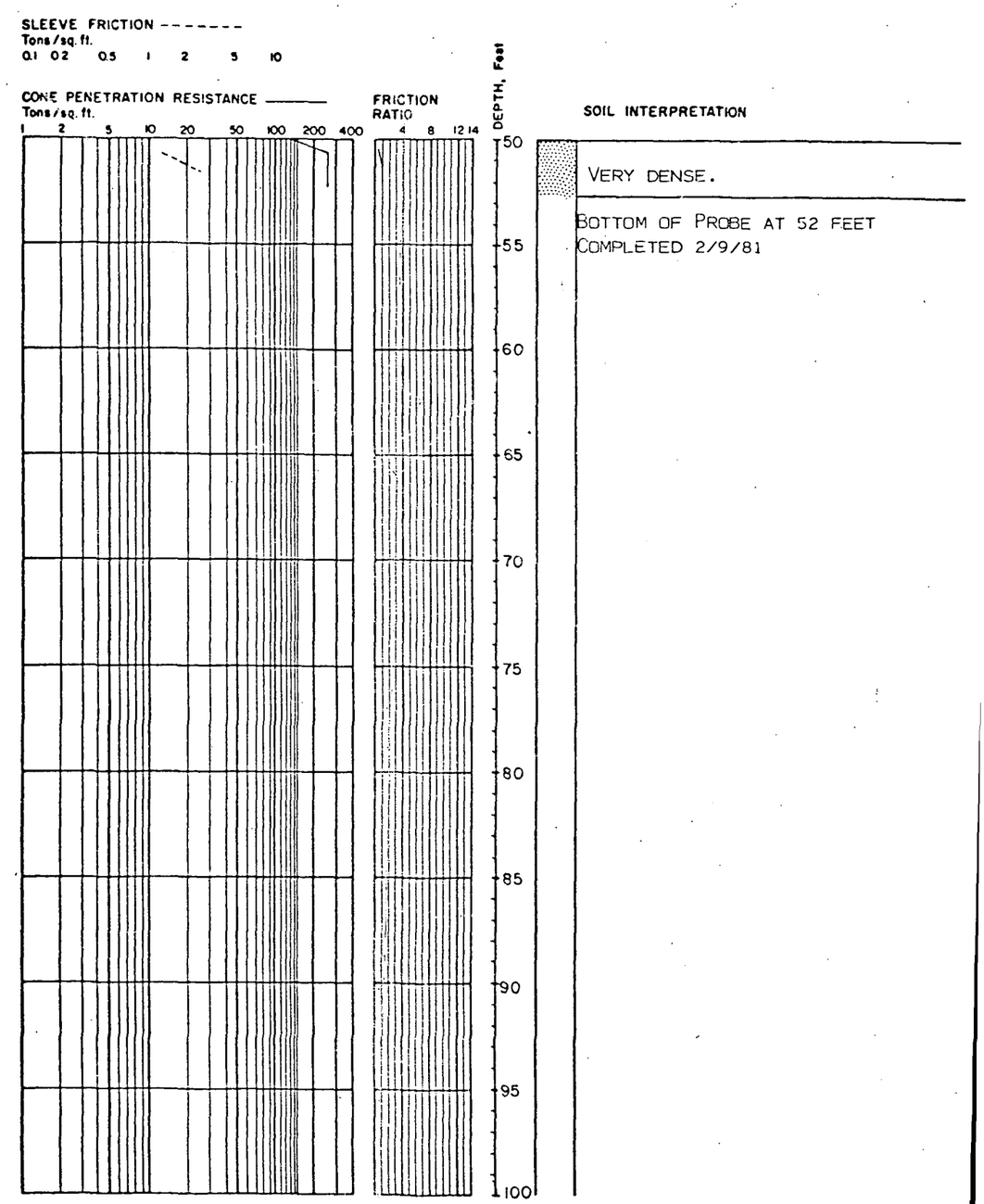




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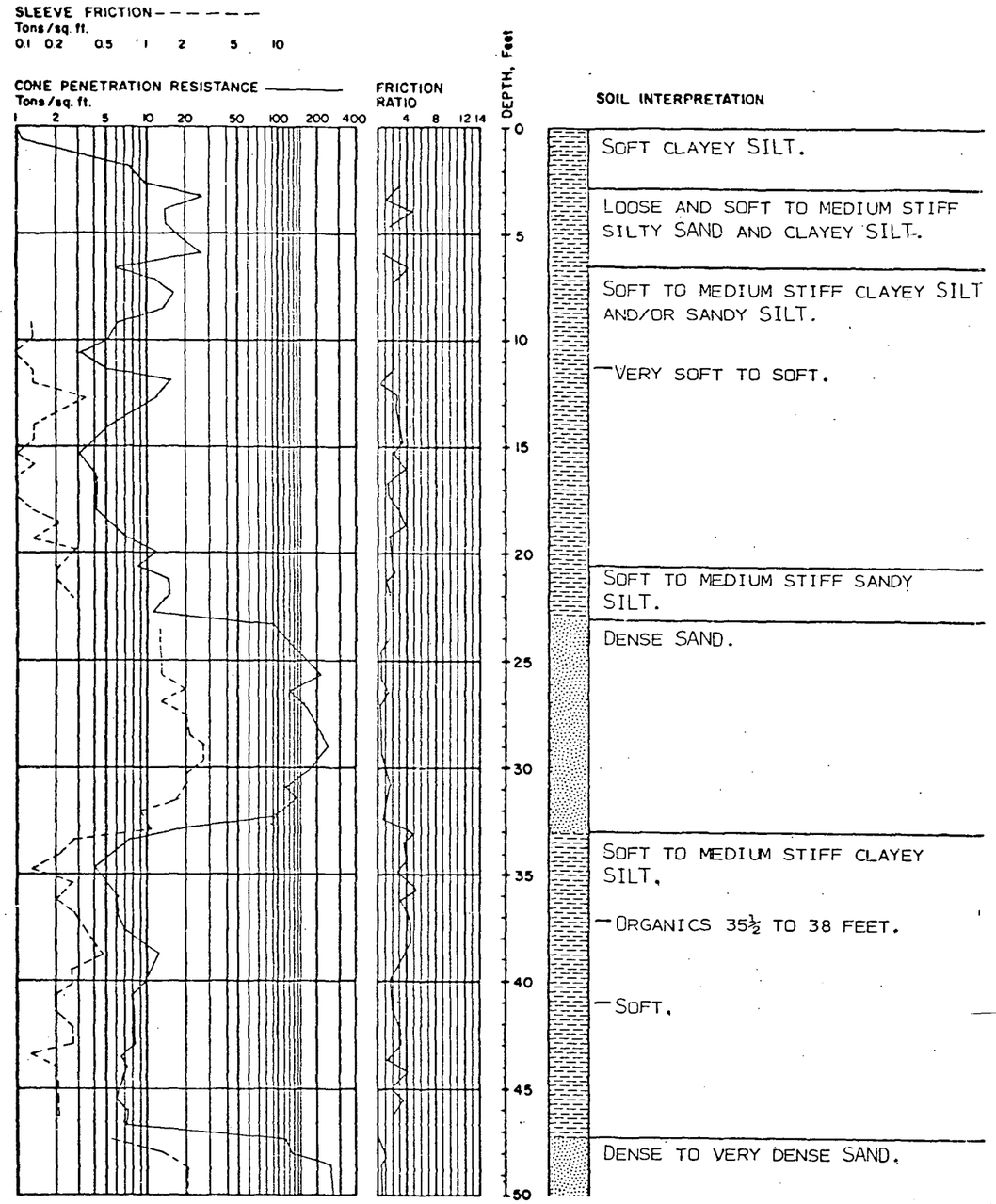
Figure A-3

# LOG OF PROBE P-3



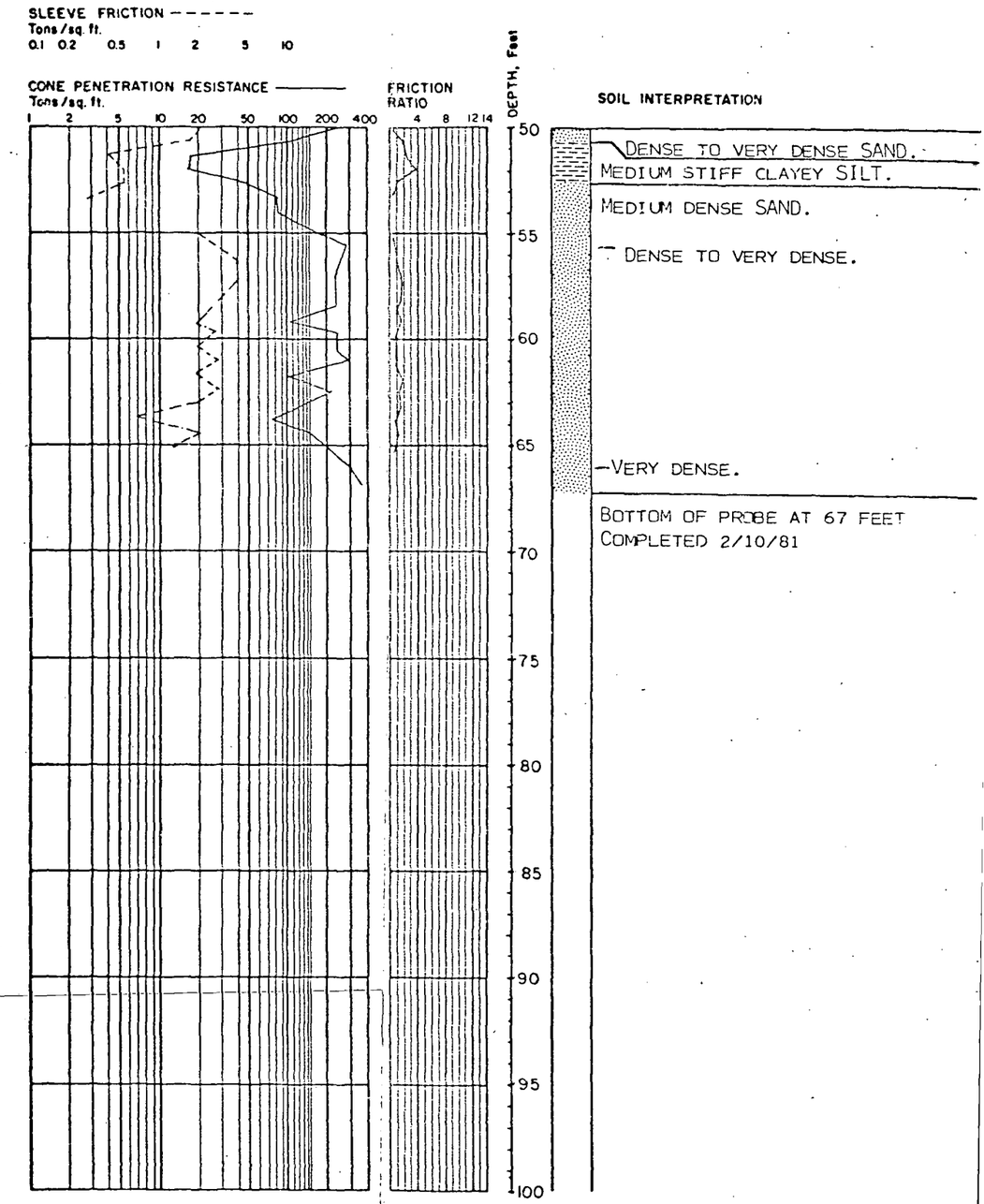




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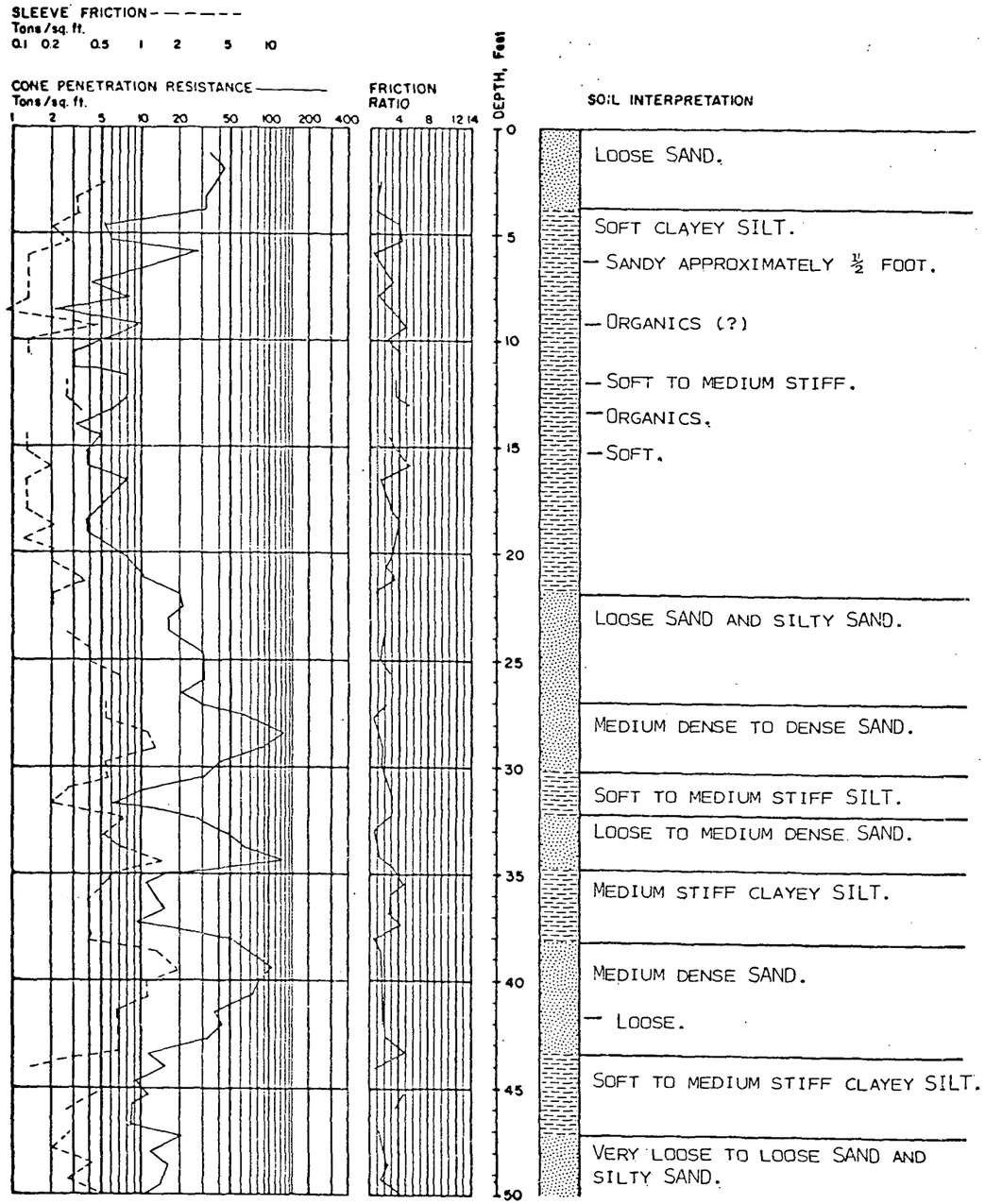
Figure A-5

# LOG OF PROBE P-2




SOIL INTERPRETATION

LOOSE SAND.

SOFT CLAYEY SILT.
- SANDY APPROXIMATELY ½ FOOT.
- ORGANICS (?)
- SOFT TO MEDIUM STIFF.
- ORGANICS.
- SOFT.

LOOSE SAND AND SILTY SAND.

MEDIUM DENSE TO DENSE SAND.

SOFT TO MEDIUM STIFF SILT.

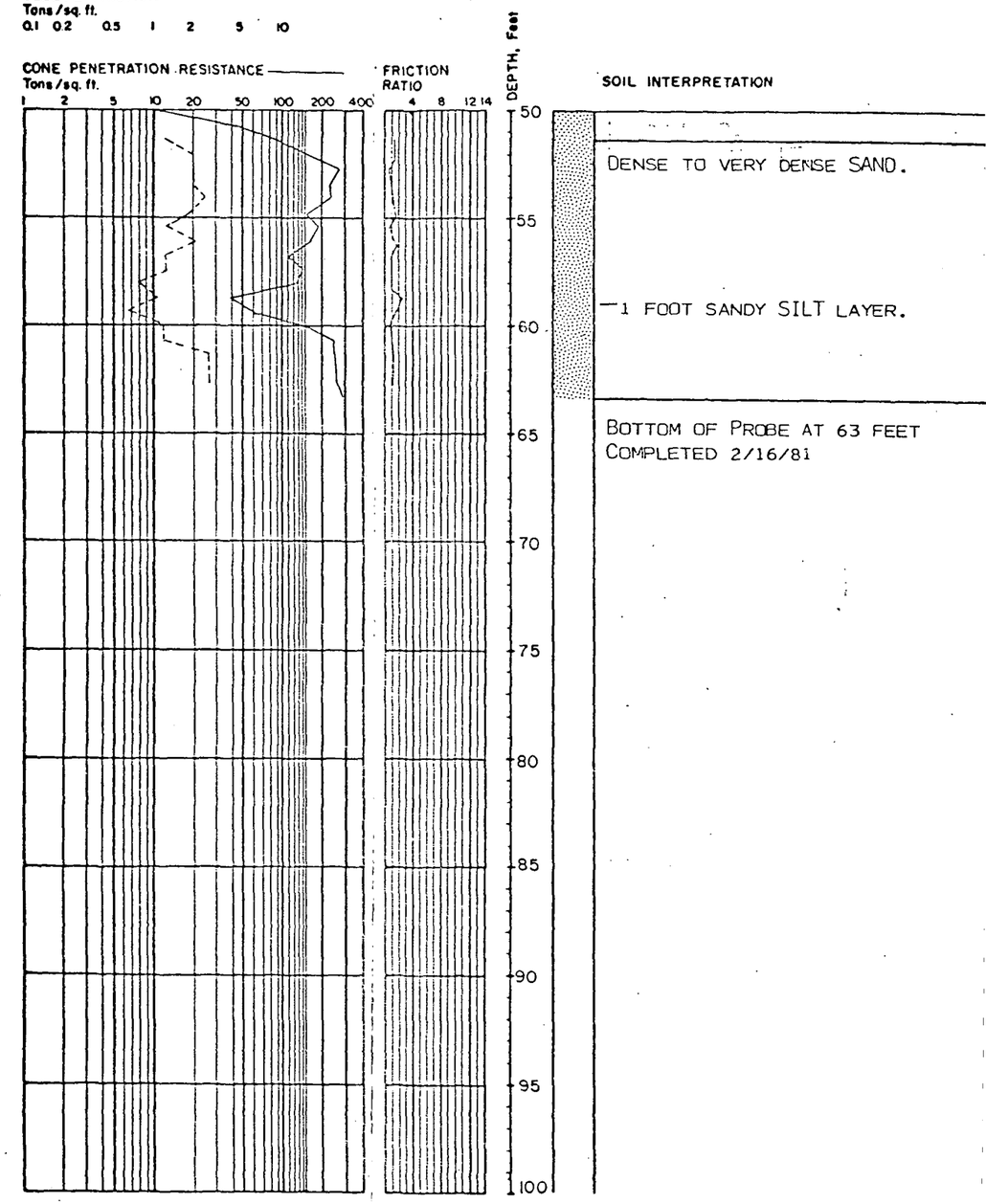
LOOSE TO MEDIUM DENSE SAND.

MEDIUM STIFF CLAYEY SILT.

MEDIUM DENSE SAND.
- LOOSE.

SOFT TO MEDIUM STIFF CLAYEY SILT.

VERY LOOSE TO LOOSE SAND AND SILTY SAND.




SOIL INTERPRETATION

DENSE TO VERY DENSE SAND.

- 1 FOOT SANDY SILT LAYER.

BOTTOM OF PROBE AT 63 FEET
COMPLETED 2/16/81

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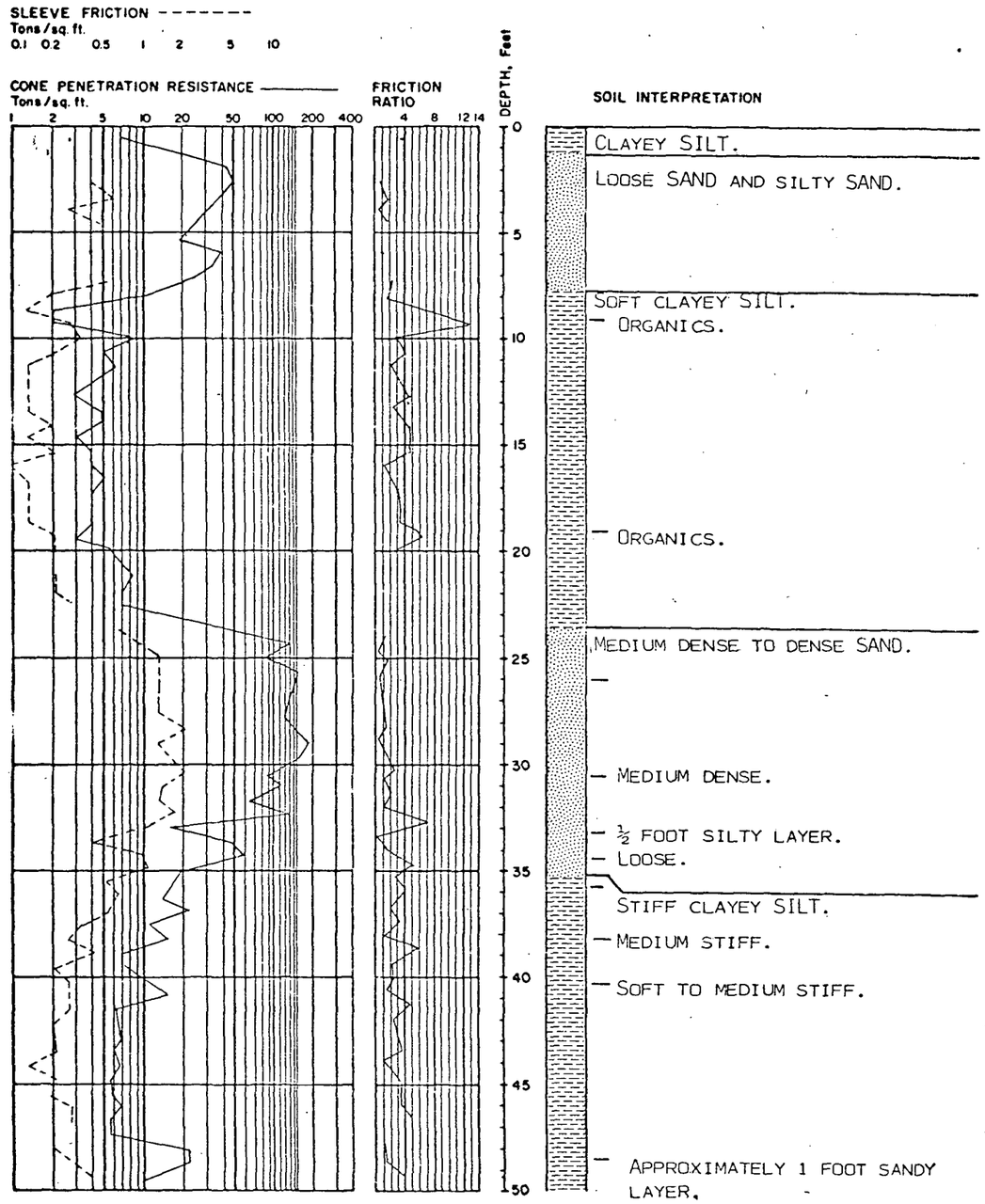
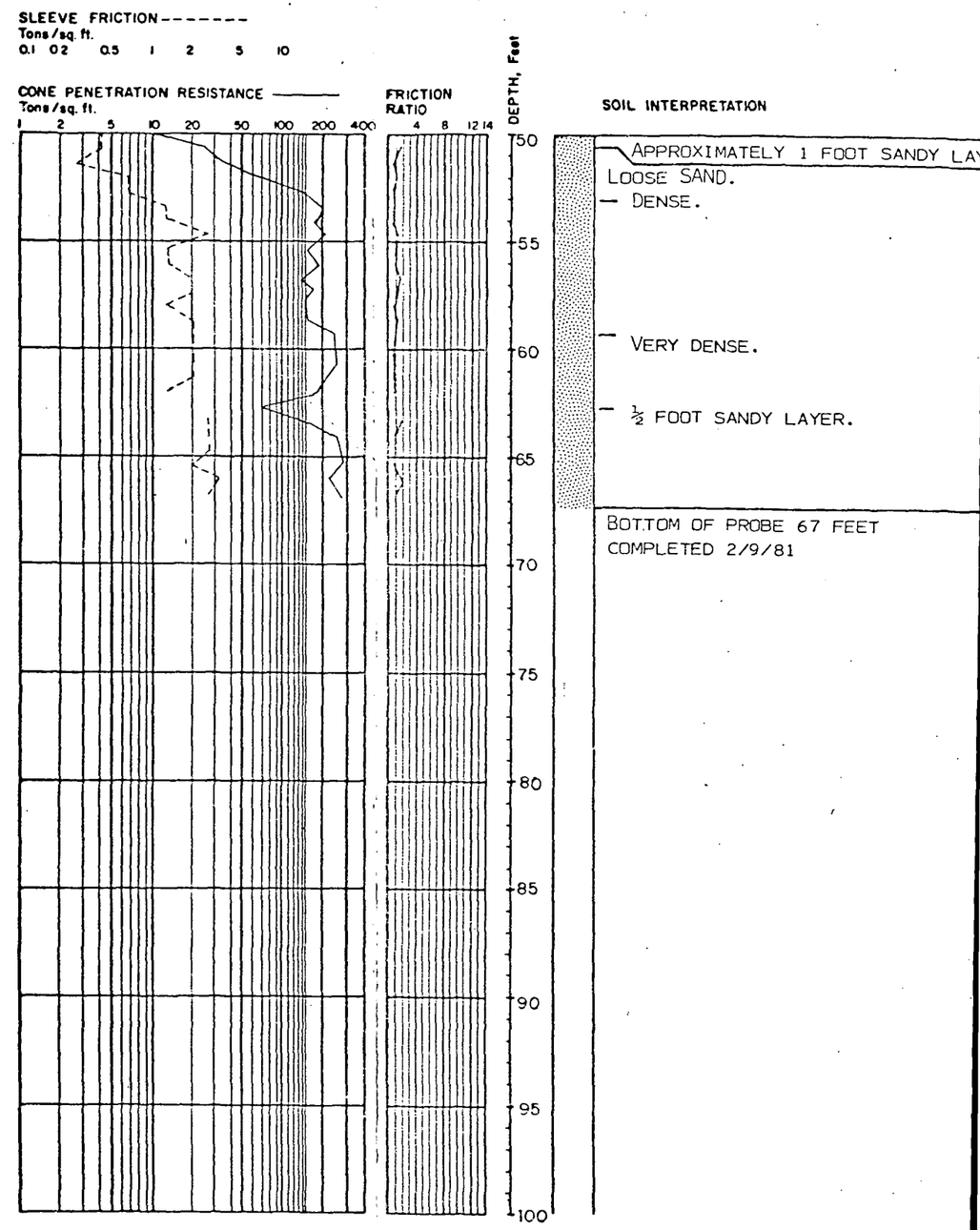
Figure A-4

# LOG OF PROBE P-4



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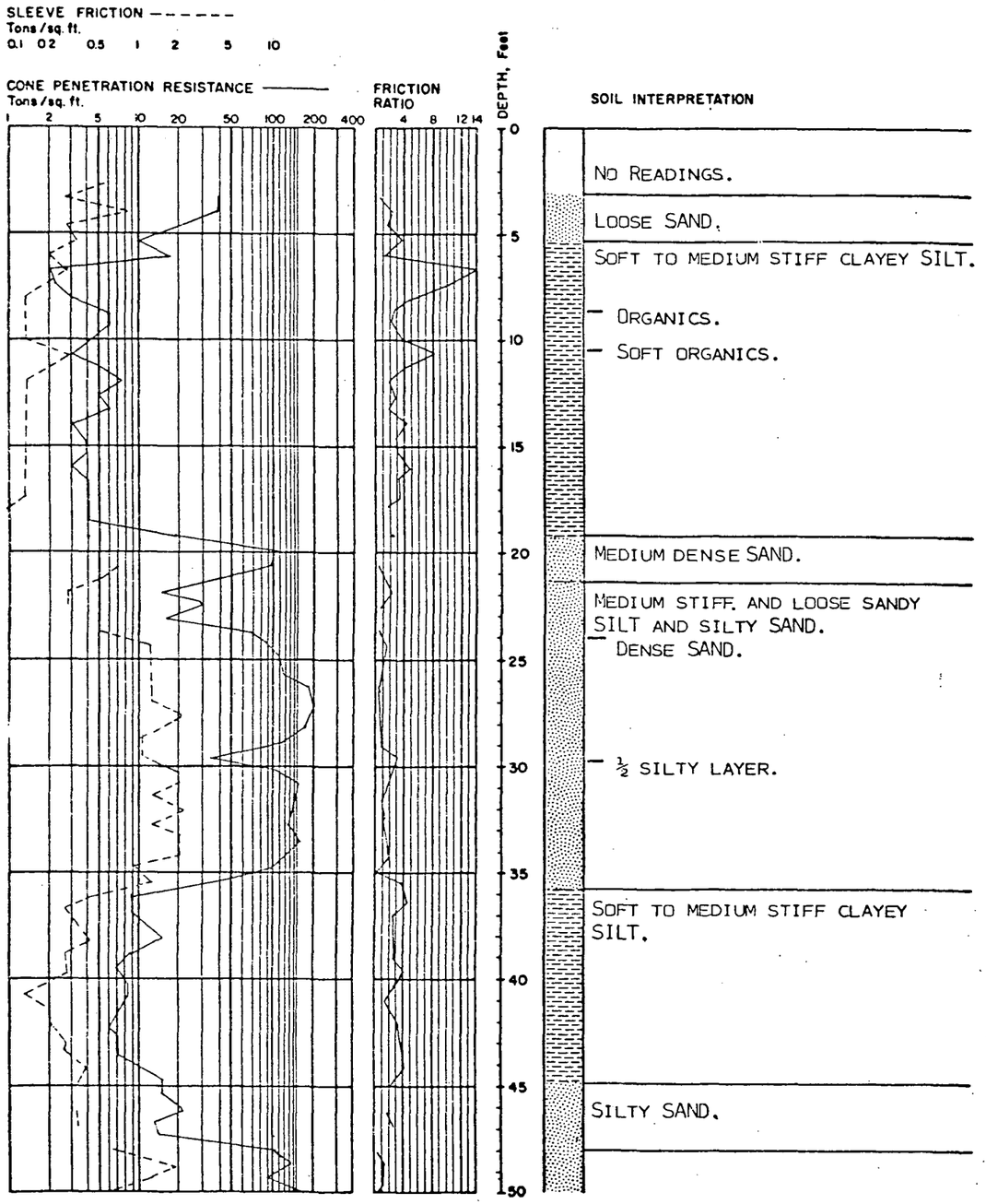


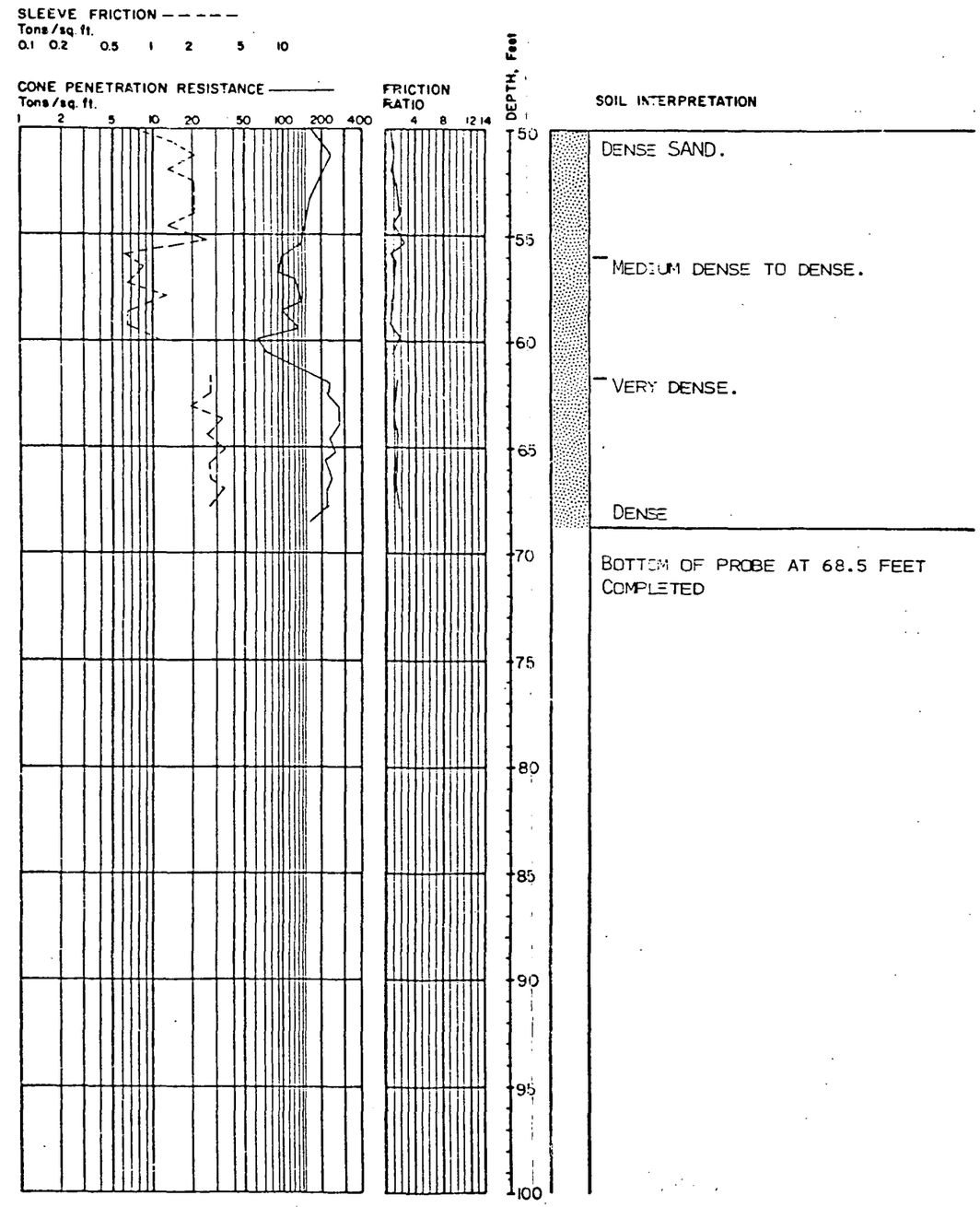


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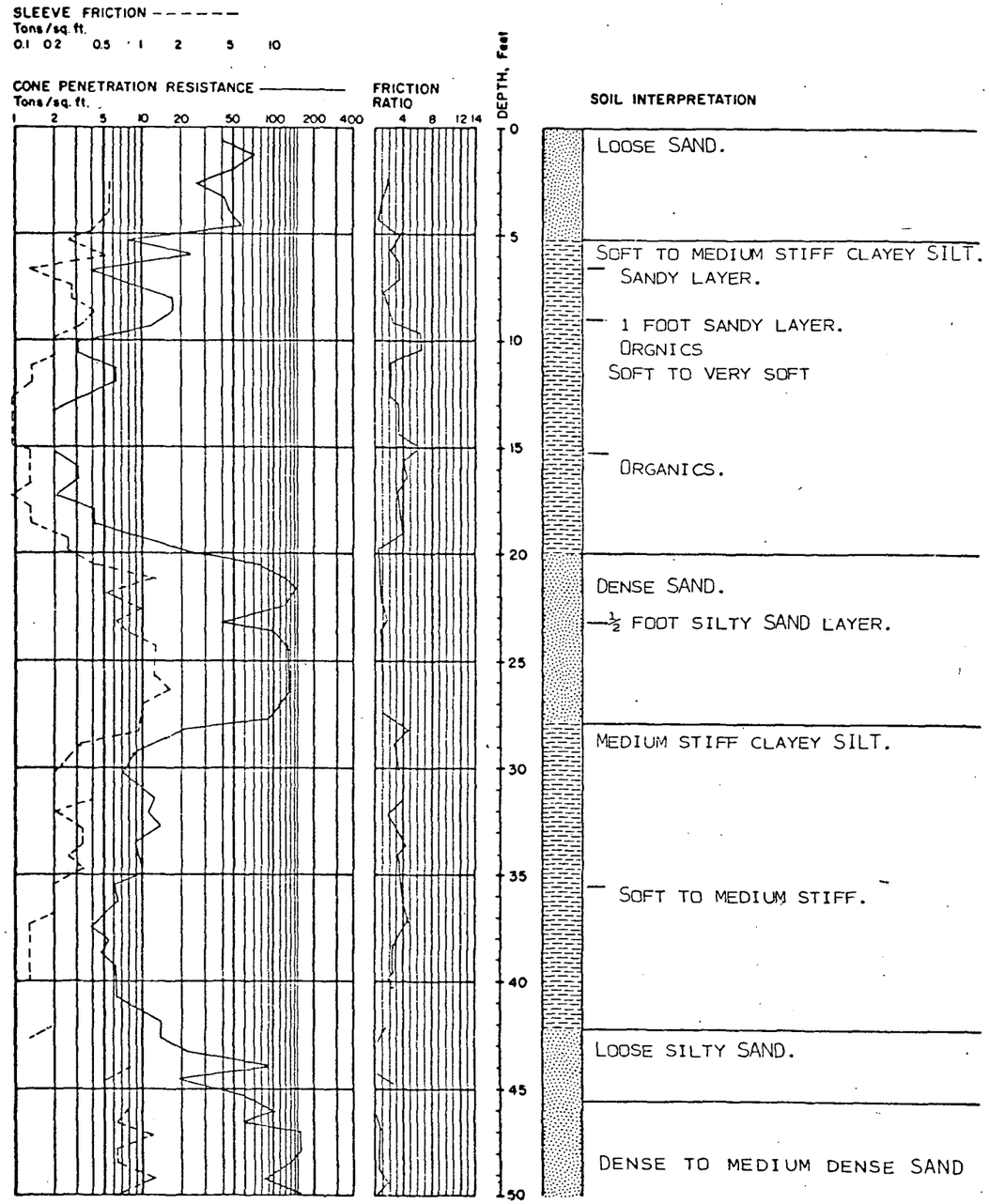
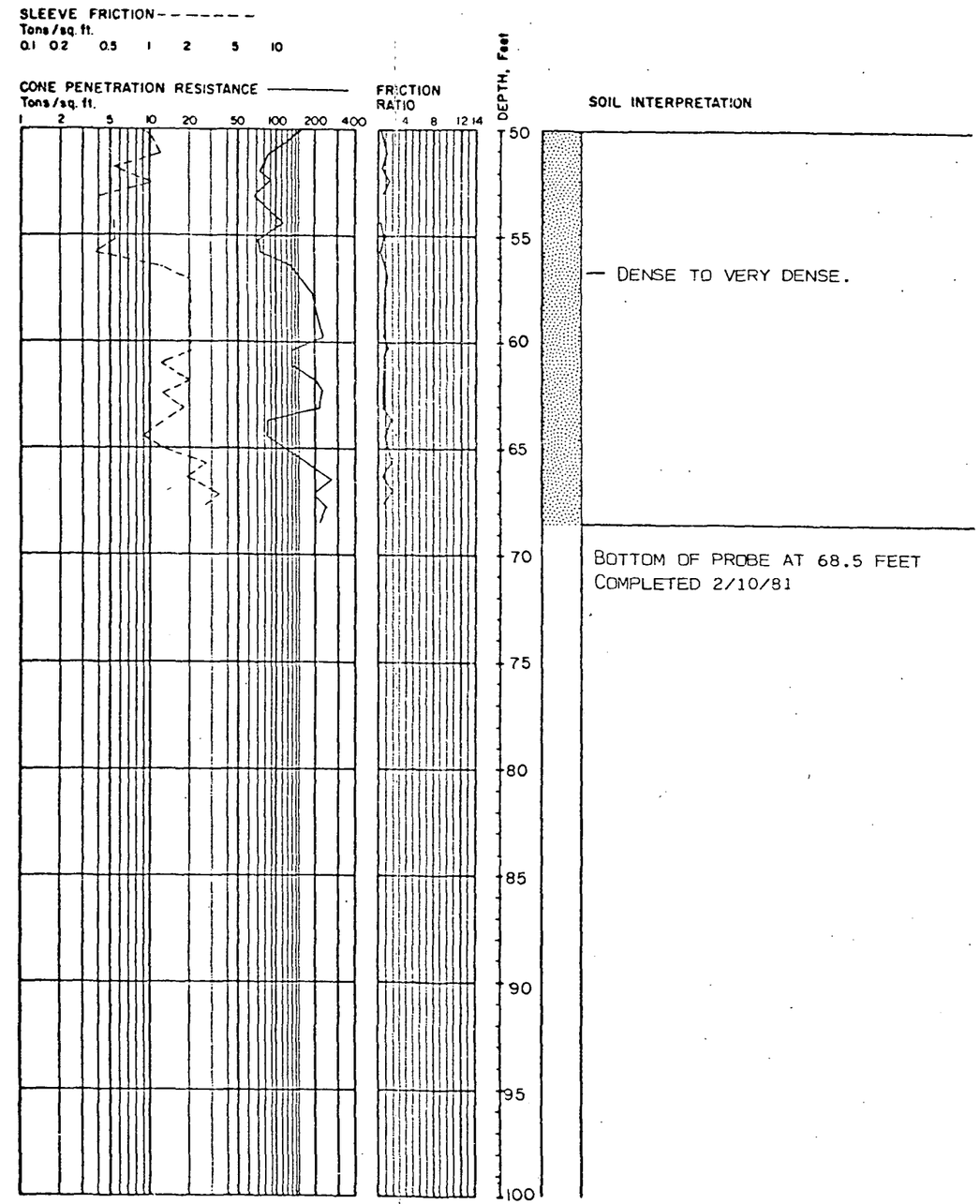
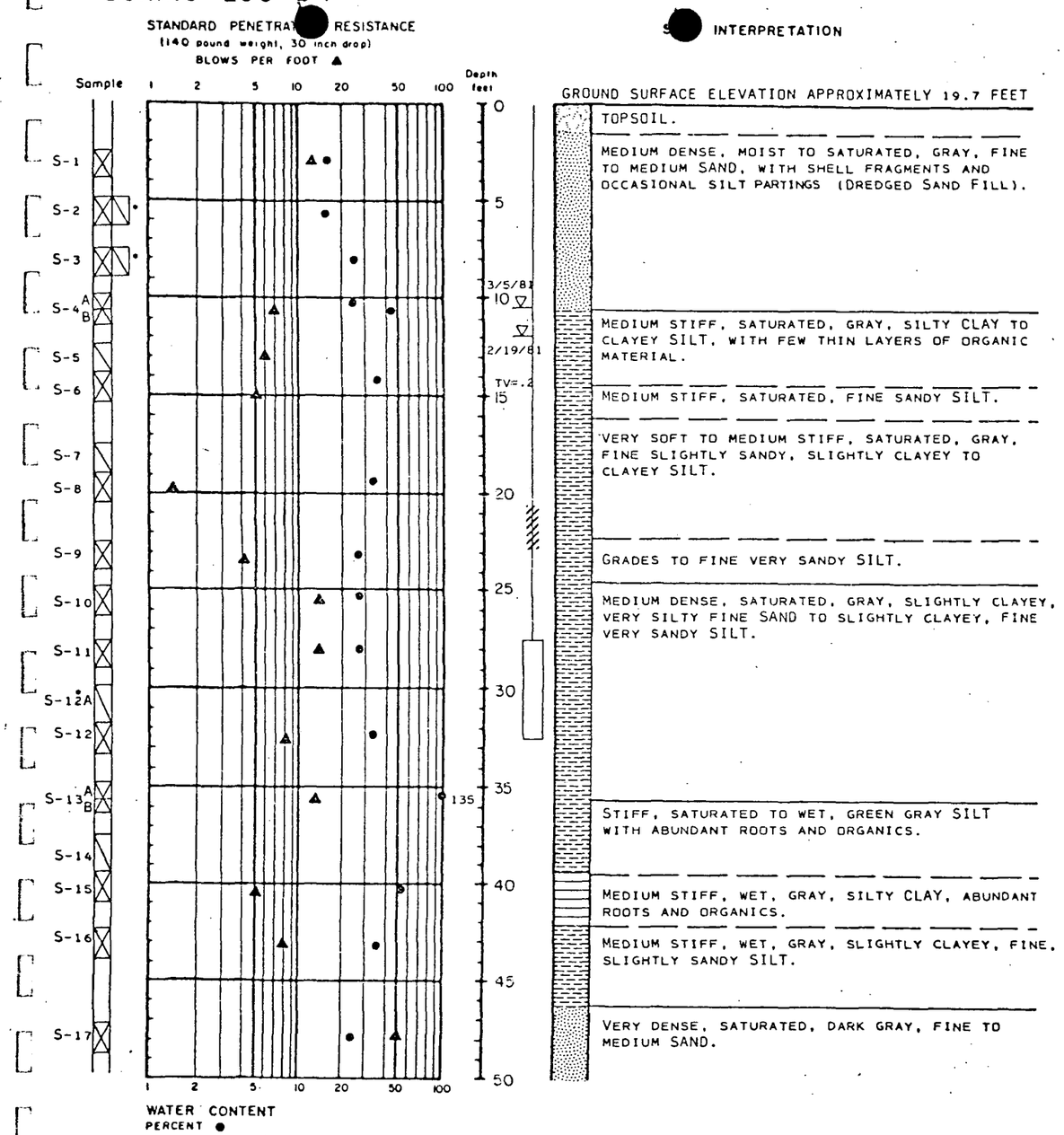
Figure A-7

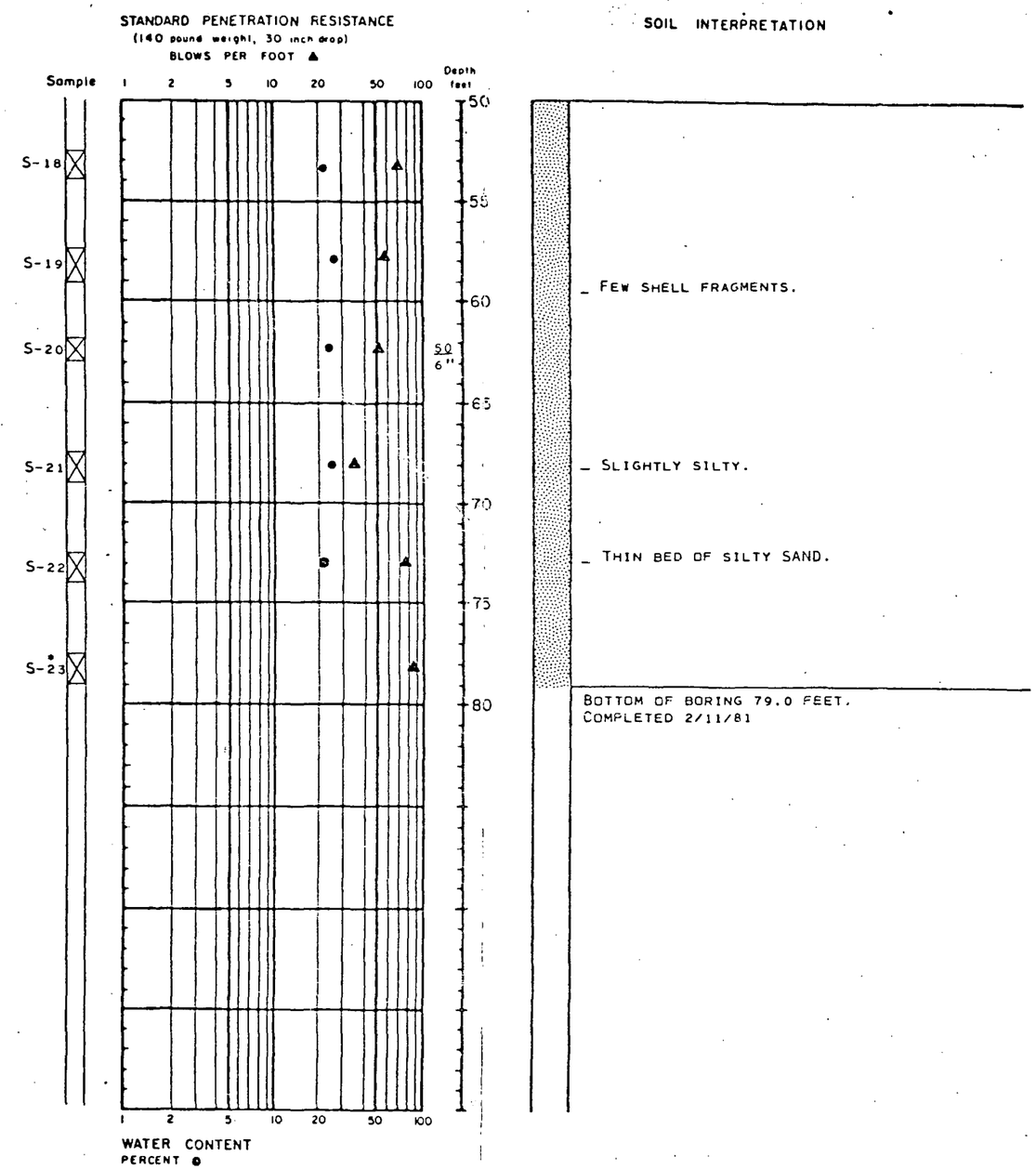
# LOG OF PROBE P-6



# BORING LOG B-1




**LEGEND**

- ⊠ 2" O.D. Split Spoon Sample
- ⊠ 3" O.D. Shelby Sample
- • No Sample Recovery
- Bentonite Seal
- ATD At Time of Drilling
- Water Level
- Observation Well
- Liquid Limit
- Plastic Limit
- PP Pocket Penetrometer (tsf)
- tv Torvane (tsf)

NOTE: Soil descriptions are interpretive and actual changes may be gradual.

**LEGEND**

- ⊠ 2" O.D. Split Spoon Sample
- ⊠ 3" O.D. Shelby Sample
- • No Sample Recovery
- ATD At Time of Drilling
- Water Level
- Observation Well
- Liquid Limit
- Plastic Limit
- PP Pocket Penetrometer (tsf)
- tv Torvane (tsf)

NOTE: Soil descriptions are interpretive and actual changes may be gradual.

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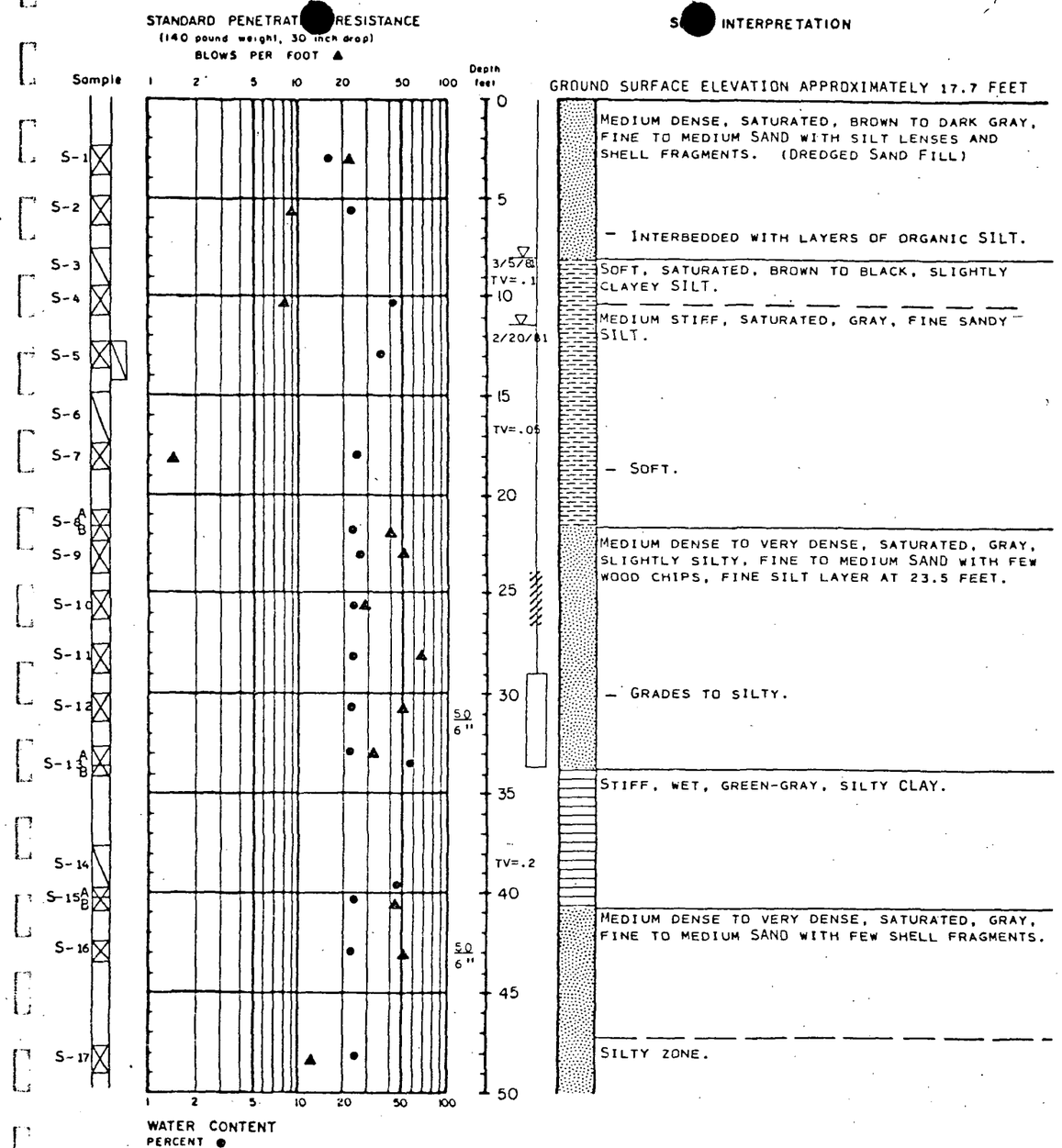
Figure A-9

# BORING LOG B-2




## SOIL INTERPRETATION

GROUND SURFACE ELEVATION APPROXIMATELY 17.7 FEET

MEDIUM DENSE, SATURATED, BROWN TO DARK GRAY, FINE TO MEDIUM SAND WITH SILT LENSES AND SHELL FRAGMENTS. (DREDGED SAND FILL)

– INTERBEDDED WITH LAYERS OF ORGANIC SILT.

SOFT, SATURATED, BROWN TO BLACK, SLIGHTLY CLAYEY SILT.

MEDIUM STIFF, SATURATED, GRAY, FINE SANDY SILT.

– SOFT.

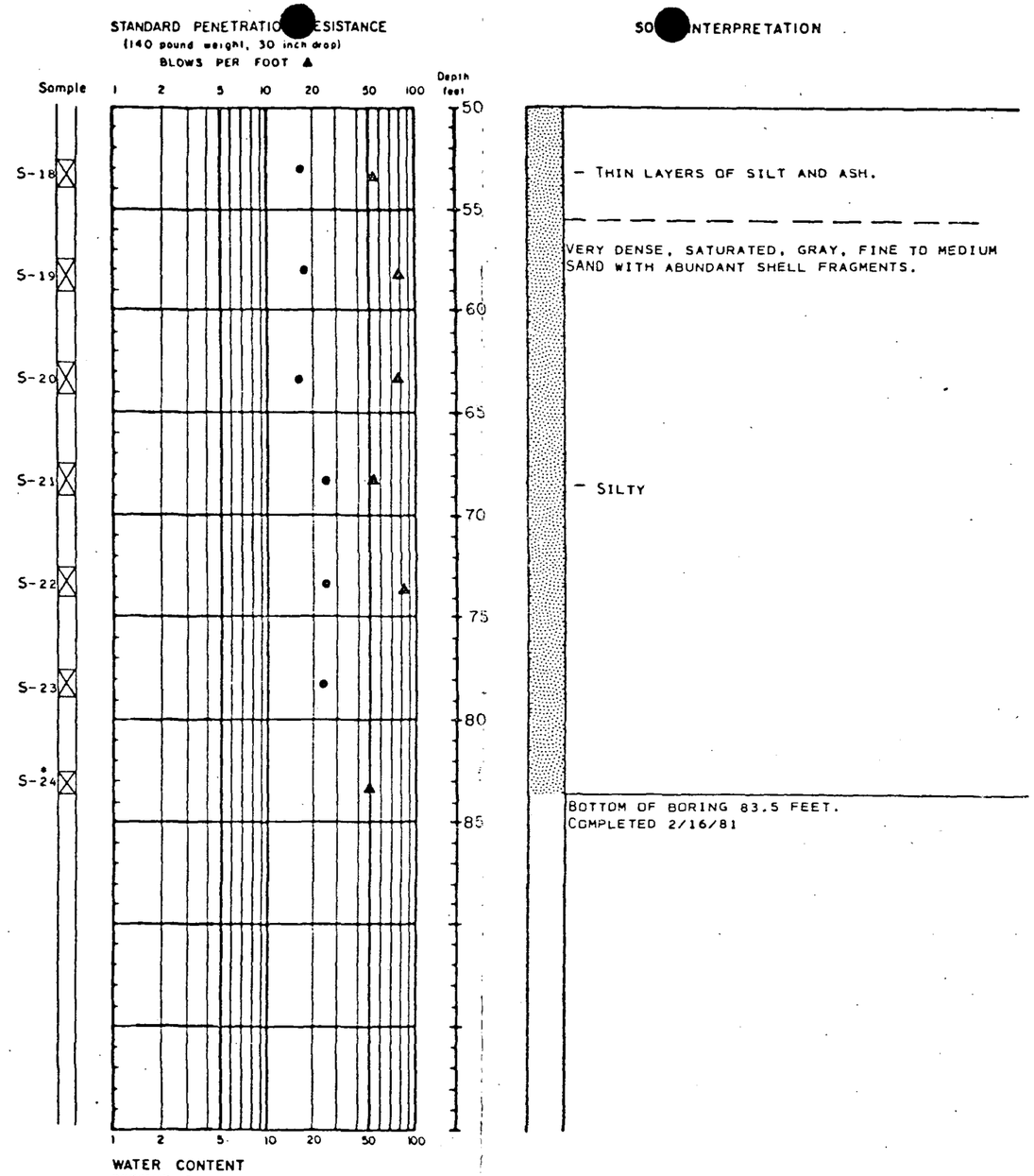
MEDIUM DENSE TO VERY DENSE, SATURATED, GRAY, SLIGHTLY SILTY, FINE TO MEDIUM SAND WITH FEW WOOD CHIPS, FINE SILT LAYER AT 23.5 FEET.

– GRADES TO SILTY.

STIFF, WET, GREEN-GRAY, SILTY CLAY.

MEDIUM DENSE TO VERY DENSE, SATURATED, GRAY, FINE TO MEDIUM SAND WITH FEW SHELL FRAGMENTS.

SILTY ZONE.




## SOIL INTERPRETATION

– THIN LAYERS OF SILT AND ASH.

VERY DENSE, SATURATED, GRAY, FINE TO MEDIUM SAND WITH ABUNDANT SHELL FRAGMENTS.

– SILTY

BOTTOM OF BORING 83.5 FEET.
COMPLETED 2/16/81

LEGEND

- ⊠ 2" O.D. Split Spoon Sample
- ⊠ 3" O.D. Shelby Sample
- • No Sample Recovery
- Bentonite Seal
- ATD At Time of Drilling
- Water Level
- Observation Well
- Liquid Limit
- Plastic Limit
- PP Pocket Penetrometer (tsf)
- tv Torvane (tsf)

NOTE: Soil descriptions are interpretive and actual changes may be gradual.

LEGEND

- ⊠ 2" O.D. Split Spoon Sample
- ⊠ 3" O.D. Shelby Sample
- • No Sample Recovery
- ATD At Time of Drilling
- Water Level
- Observation Well
- Liquid Limit
- Plastic Limit
- PP Pocket Penetrometer (tsf)
- tv Torvane (tsf)

NOTE: Soil descriptions are interpretive and actual changes may be gradual.

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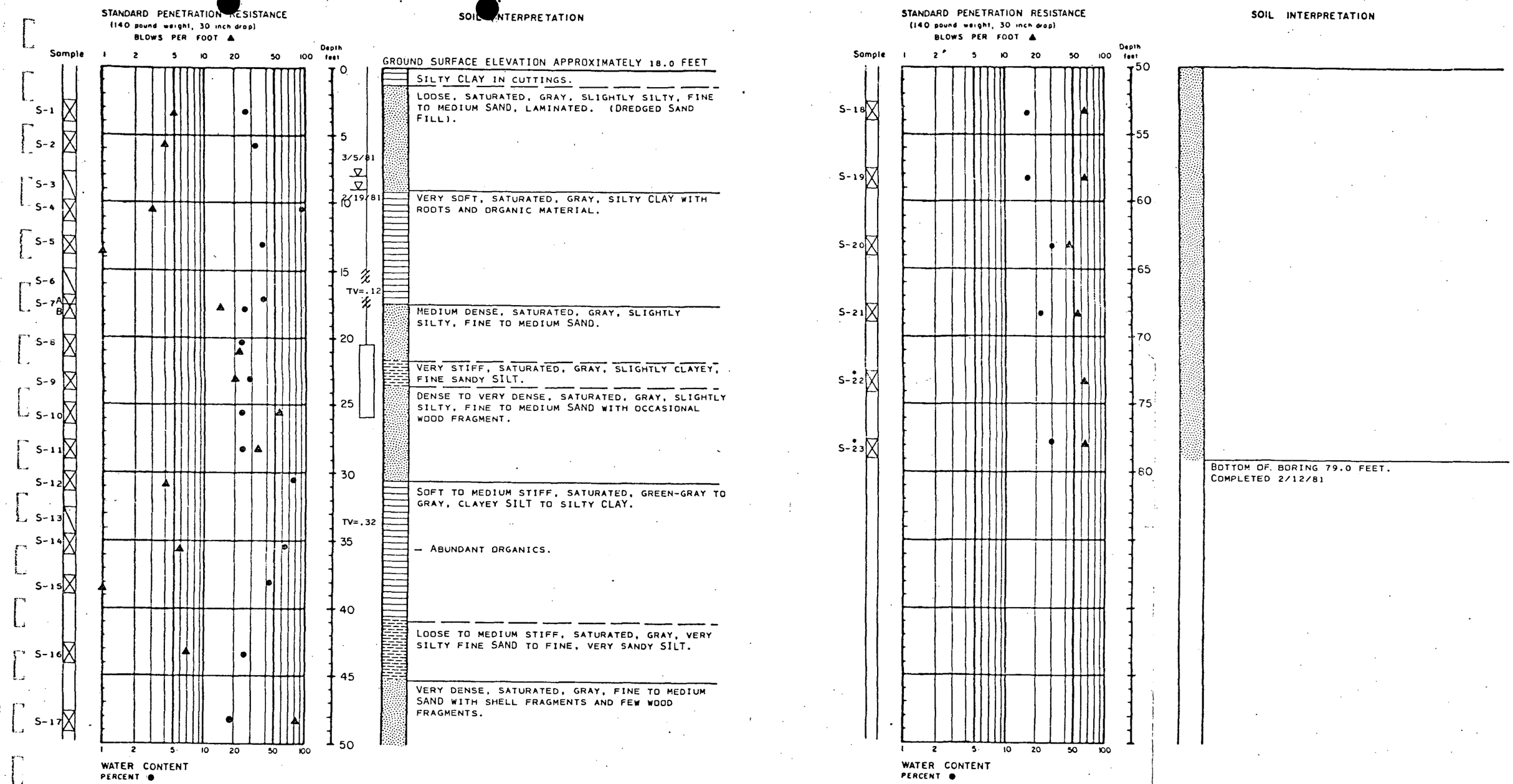
Figure A-10

# BORING LOG B-3




LEGEND

- ⊠ 2" O.D. Split Spoon Sample
- ⊠ 3" O.D. Shelby Sample
- • No Sample Recovery
- Bentonite Seal
- ATD At Time of Drilling
- Water Level
- Observation Well
- Liquid Limit
- Plastic Limit
- pp Pocket Penetrometer (tsf)
- tv Torvane (tsf)

NOTE: Soil descriptions are interpretive and actual changes may be gradual.

LEGEND

- ⊠ 2" O.D. Split Spoon Sample
- ⊠ 3" O.D. Shelby Sample
- • No Sample Recovery
- ATD At Time of Drilling
- Water Level
- Observation Well
- Liquid Limit
- Plastic Limit
- pp Pocket Penetrometer (tsf)
- tv Torvane (tsf)

NOTE: Soil descriptions are interpretive and actual changes may be gradual.

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Figure A-11

# APPENDIX B
# LABORATORY TESTS

Laboratory tests were performed on representative soil samples recovered from the borings to establish soil moisture conditions and geotechnical engineering properties. The Unified Soil Classification system, presented in Figure B-1, was used as a basis to describe the soil samples. Laboratory tests, consisting of further visual classification, moisture content determinations, grain size analyses, Atterberg limit tests, consolidation tests and shear strength tests were performed as described below.

## Visual Classification

Soil samples recovered in the explorations were transported to the laboratory where detailed classification was accomplished. The visual portion of the laboratory examination of the samples was similar to the field examination, except additional time was utilized and the samples were viewed in a relatively controlled environment. Visual classification included consistency or density, color, moisture content, major soil type and the modifying fractions in the sample. Classification was then made subject to modification after additional testing such as moisture content and grain size analyses had been accomplished as deemed appropriate.

## Moisture Content Determinations

Moisture contents were determined for each sample recovered in the borings in general accordance with ASTM D 2216-71 as soon as possible following their arrival in our Seattle laboratory. The results of these tests are shown adjacent to each soil sample on the Boring Logs, Figures A-9 through A-11. In addition, the

moisture contents of samples subjected to other testing have been determined and are presented along with the various test results which follow in this appendix.

## Grain Size Analyses

Grain size analyses were performed on representative samples generally in accordance with test procedures described in ASTM D 422-63. The wet sieve analysis method was used for most samples and determines the size distribution greater than the No. 200 mesh sieve. The size distribution for particles less than the No. 200 mesh sieve was determined by the Hydrometer method for a selected number of samples. The results of the tests are presented as curves in Figures B-2 through B-4 plotting percent finer by weight versus grain size. Each curve is identified as to boring number and sample number with a complete written description presented in tabulated form at the bottom of the figure.

## Atterberg Limits

Atterberg limit determinations were accomplished for selected samples of the silt, clayey silt and silty clay. The liquid limit was determined in general accordance with ASTM D 423-66 and the plastic limit was determined in general accordance with ASTM D 424-59. The results of the Atterberg limit analyses and the plasticity characteristics are summarized on the Plasticity Chart, Figure B-5, which relates the plasticity index (liquid limit minus the plastic limit) to the liquid limit. The results of the Atterberg limit tests are also shown where applicable on figures presenting various other test results.

## Consolidation Tests

Consolidation tests were performed on selected samples of the clayey silt, silty clay and the organic or peaty soils to provide data for making settlement calculations. Undisturbed samples were carefully trimmed and fit into a rigid ring with porous stones placed on the top and bottom of the sample to allow drainage. 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 in Figure B-6.

## Pocket Penetrometer and Torvane

The Pocket Penetrometer and Torvane procedure provide a quick approximate test of the consistency (undrained shear strength) of a cohesive soil sample. The Pocket Penetrometer device consists of a calibrated spring mechanism which measures penetration resistance of a small diameter (¼ inch) steel tip over a given distance. The tip is pressed against the soil and the resistance measured at the required penetration (¼ inch) is correlative with the unconfined compressive strength of the soil.

The Torvane device consists of a small diameter plate with eight equally spaced and radially arranged vanes. The vanes are pressed into the soil and the device is rotated. The vanes force a shear failure to take place over the area of the face of the plate, and the resistance at failure as measured by a calibrated spring is correlative to the undrained shear strength of the sample tested.

The results of the Pocket Penetrometer and Torvane tests are shown on the logs of borings corresponding to the depth of the sample.

Unconsolidated-Undrained Triaxial Test

The unconsolidated undrained triaxial test (UU) is a method used to measure the undrained strength of the soil at approximately the in situ pressure and moisture conditions. The tests were performed by trimming a specimen from Shelby tube samples to a length of about 5½ to 6 inches. The sample is encased in a rubber membrane and then placed in the triaxial cell. An all-around pressure is applied hydraulically and is about equal in magnitude to the overburden pressure. An axial vertical load is then applied at a constant strain rate to the sample. The stress-strain behavior is recorded until failure takes place without allowing drainage from the specimen. The failure stress is taken as the maximum load on the samples. The specific test results are shown on Figures B-7 and B-8. The shear strength is considered to be one-half the maximum deviator stress based on a $\phi = 0^{o}$ concept utilizing a total stress analysis.

Direct Shear Tests

Direct shear tests were performed on selected samples in general accordance with ASTM D 3080. Relatively undisturbed samples were trimmed and fit into a direct shear ring having an inside diameter of 2.5 inches. The top and bottom surfaces of the samples were fit with perforated metal plates. After applying the desired normal force, the specimen were submerged. The specimen were then failed using a loading device at a controlled rate of strain of

0.0025 inches per minute. In each test sufficient sample strain was achieved to evaluate the peak strength of the sample. The results of the direct shear tests are presented in Figures B-9 and B-10.

Consolidated Undrained Triaxial Tests

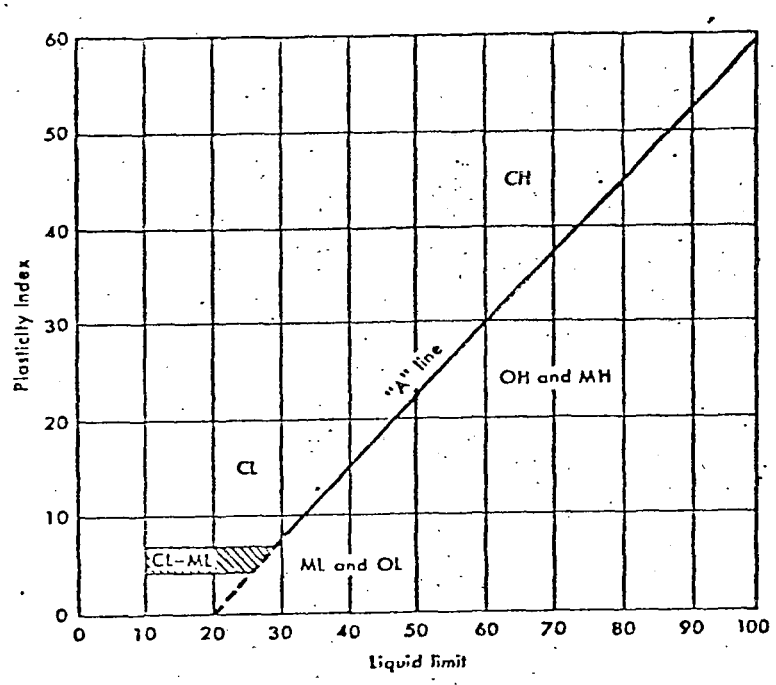
The consolidated undrained triaxial (CU) test with pore pressure measurements is a method of measuring both the undrained and the drained strength behavior of the soils at stress levels greater than the existing in situ stress conditions. The test specimen preparation for the CU test is the same as for the UU test. When the sample is placed in the triaxial test cell, the all around pressure is applied hydraulically and drainage from the sample is allowed through porous stones located at the top and bottom of the sample. The soil structure adjusts to the increased pressure by the mechanism of consoldation and movement of pore water from the sample takes place. When consolidation is completed, drainage lines from the sample are closed and the sample is sheared to failure by application of increasing vertical axial load at a constant strain rate as in the UU test.

During the shearing process, the magnitude of pore water pressure developed is recorded. From the data, two separate plots are developed. The total stress plot measures the undrained (total) shear strength for the soil for varying consolidation pressures. The effective stress plot measures the drained shear stength (total stress minus pore water pressure) with varying consolidation pressures. The data is plotted using principal stress versus shear stress as Mohr's circles and the tangent to the Mohr's circles for a test series represents the angle of internal friction (Ø). The intercept along the vertical axis is the apparent cohesion (c).

The results of the undrained test data are shown on the lower plot of the Triaxial Compression Test form, using solid lines, and the drained behavior is indicated using dashed lines. Specific test results are shown in Figure B-11, Triaxial Compression Tests.

# UNIFIED SOIL CLASSIFICATION SYSTEM

| Major divisions | | | Group symbols | Typical names | Laboratory classification criteria | |
|---|---|---|---|---|---|---|
| Coarse-grained soils (More than half of material is larger than No. 200 sieve size) 0.08 mm | Gravels (More than half of coarse fraction is larger than No. 4 sieve size) | Clean gravels (Little or no fines) | GW | Well-graded gravels, gravel-sand mixtures, little or no fines | Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows: Less than 5 per cent ..... GW, GP, SW, SP; More than 12 per cent ..... GM, GC, SM, SC; 5 to 12 per cent ..... Borderline cases requiring dual symbols** | $C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3 |
| | | | GP | Poorly graded gravels, gravel-sand mixtures, little or no fines | | Not meeting all gradation requirements for GW |
| | | Gravels with fines (Appreciable amount of fines) | GM* d | Silty gravels, gravel-sand-silt mixtures | Atterburg limits below "A" line or P.I. less than 4 | Above "A" line with P.I. between 4 and 7 are borderline cases requiring use of dual symbols |
| | | | GM* u | | | |
| | | | GC | Clayey gravels, gravel-sand-clay mixtures | Atterburg limits above "A" line with P.I. greater than 7 | |
| | Sands (More than half of coarse fraction is smaller than No. 4 sieve size) | Clean sands (Little or no fines) | SW | Well-graded sands, gravelly sands, little or no fines | | $C_u = \frac{D_{60}}{D_{10}}$ greater than 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3 |
| | | | SP | Poorly graded sands, gravelly sands, little or no fines | | Not meeting all gradation requirements for SW |
| | | Sands with fines (Appreciable amount of fines) | SM* d | Silty sands, sand-silt mixtures | Atterburg limits below "A" line or P.I. less than 4 | Limits plotting in hatched zone with P.I. between 4 and 7 are borderline cases requiring use of dual symbols. |
| | | | SM* u | | | |
| | | | SC | Clayey sands, sand-clay mixtures | Atterburg limits above "A" line with P.I. greater than 7 | |
| Fine-grained soils (More than half of material is smaller than No. 200 sieve) 0.08 mm | Silts and clays (Liquid limit less than 50) | | ML | Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity | | |
| | | | CL | Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays | | |
| | | | OL | Organic silts and organic silty clays of low plasticity | | |
| | Silts and clays (Liquid limit greater than 50) | | MH | Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts | | |
| | | | CH | Inorganic clays of high plasticity, fat clays | | |
| | | | OH | Organic clays of medium to high plasticity, organic silts | | |
| | Highly organic soils | | Pt | Peat and other highly organic soils | | |




Plasticity Chart

*Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is based on Atterburg limits; suffix d used when LL is 28 or less and the P.I. is 6 or less; the suffix u used when LL is greater than 28.

**Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group symbols. For example: GW-GC, well-graded gravel-sand mixture with clay binder.

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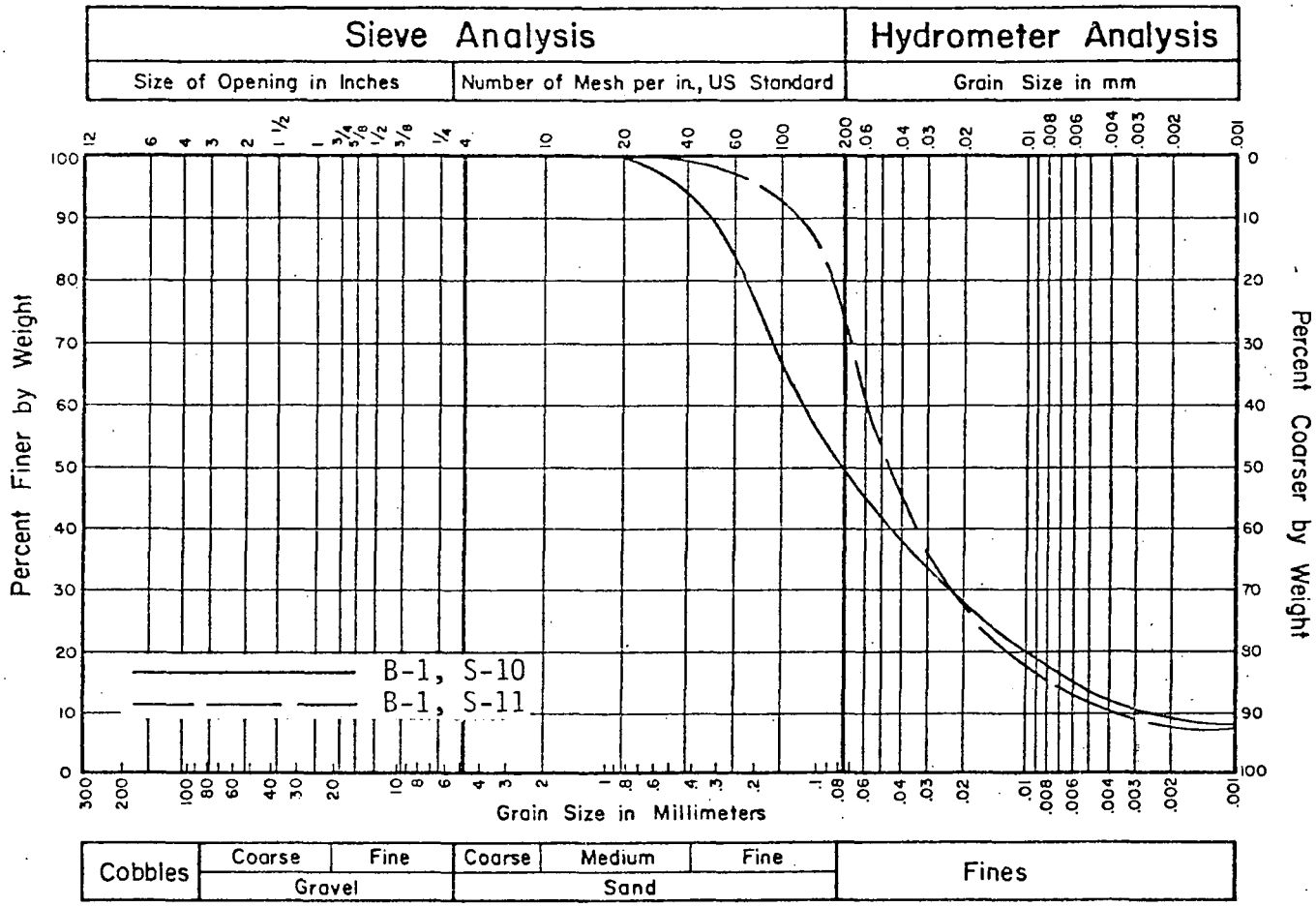
FIGURE B-1

# GRAIN SIZE CLASSIFICATION




| Sample | Depth-ft | U.S.C. | Classification | Nat. W.C. % | Atterberg Limits LL | PL | PI |
|---|---|---|---|---|---|---|---|
| B-1 S-20 | 25.0 - 26.5 | SM | SLIGHTLY CLAYEY, VERY SILTY FINE SAND. | 29 | | | |
| B-1 S-11 | 27.5 - 29.0 | ML | SLIGHTLY CLAYEY, VERY SANDY SILT. | 31 | | | |

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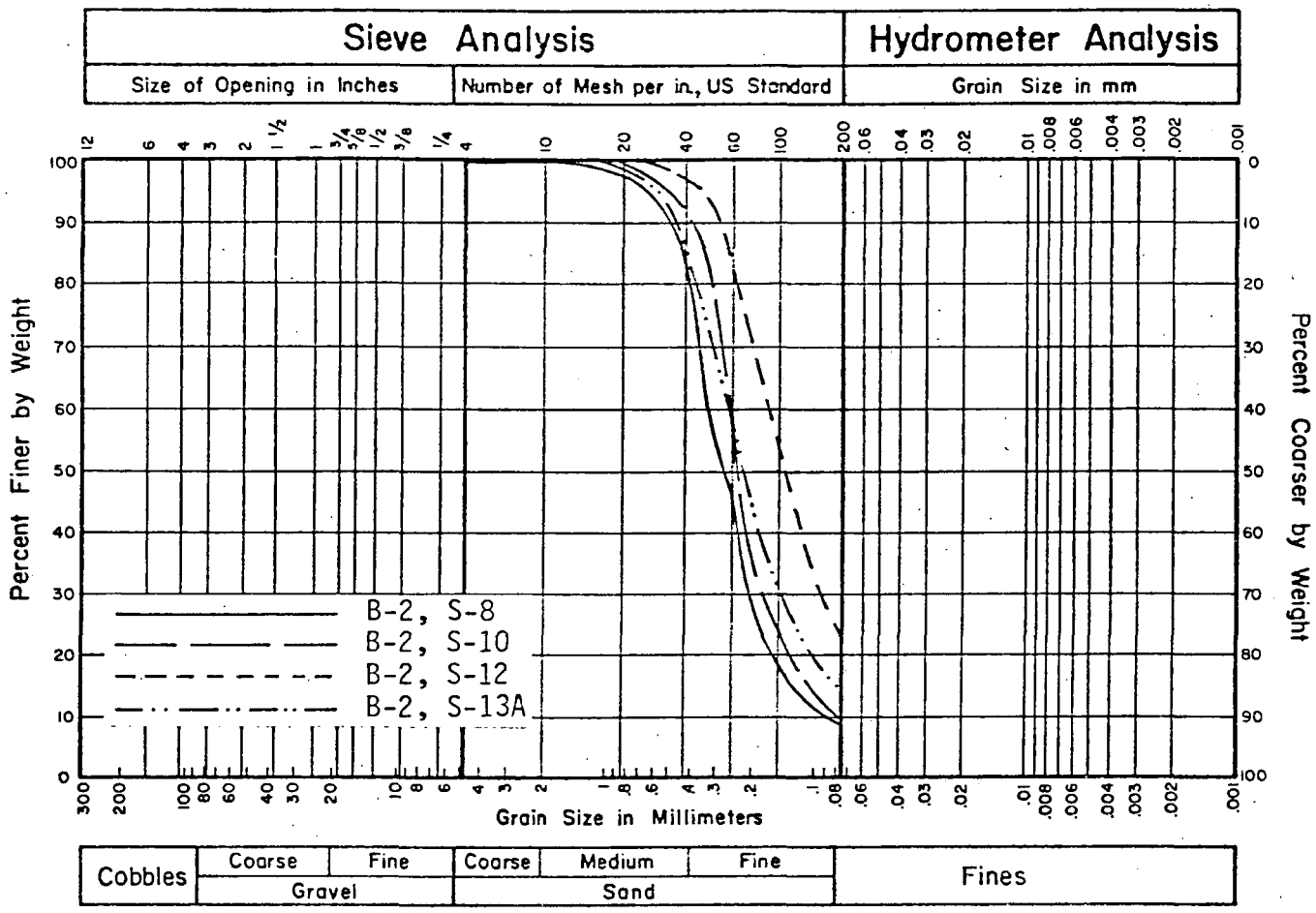
Figure B-2

# GRAIN SIZE CLASSIFICATION




| Sample | Depth-ft | U.S.C. | Classification | Nat. W.C. % | Atterberg Limits LL | PL | PI |
|---|---|---|---|---|---|---|---|
| B-2 S-8 | 21.0 - 22.5 | SM | SLIGHTLY SILTY, FINE TO MEDIUM SAND. | 25 | | | |
| B-2 S-10 | 25.0- 26.5 | SP-SM | SLIGHTLY SILTY, FINE SAND. | 29 | | | |
| B-2 S-12 | 30.0 - 31.0 | SM | SILTY FINE SAND | 24 | | | |
| B-2 S-13A | 32.5 - 33.0 | SM | SILTY FINE TO MEDIUM SAND. | 23 | | | |

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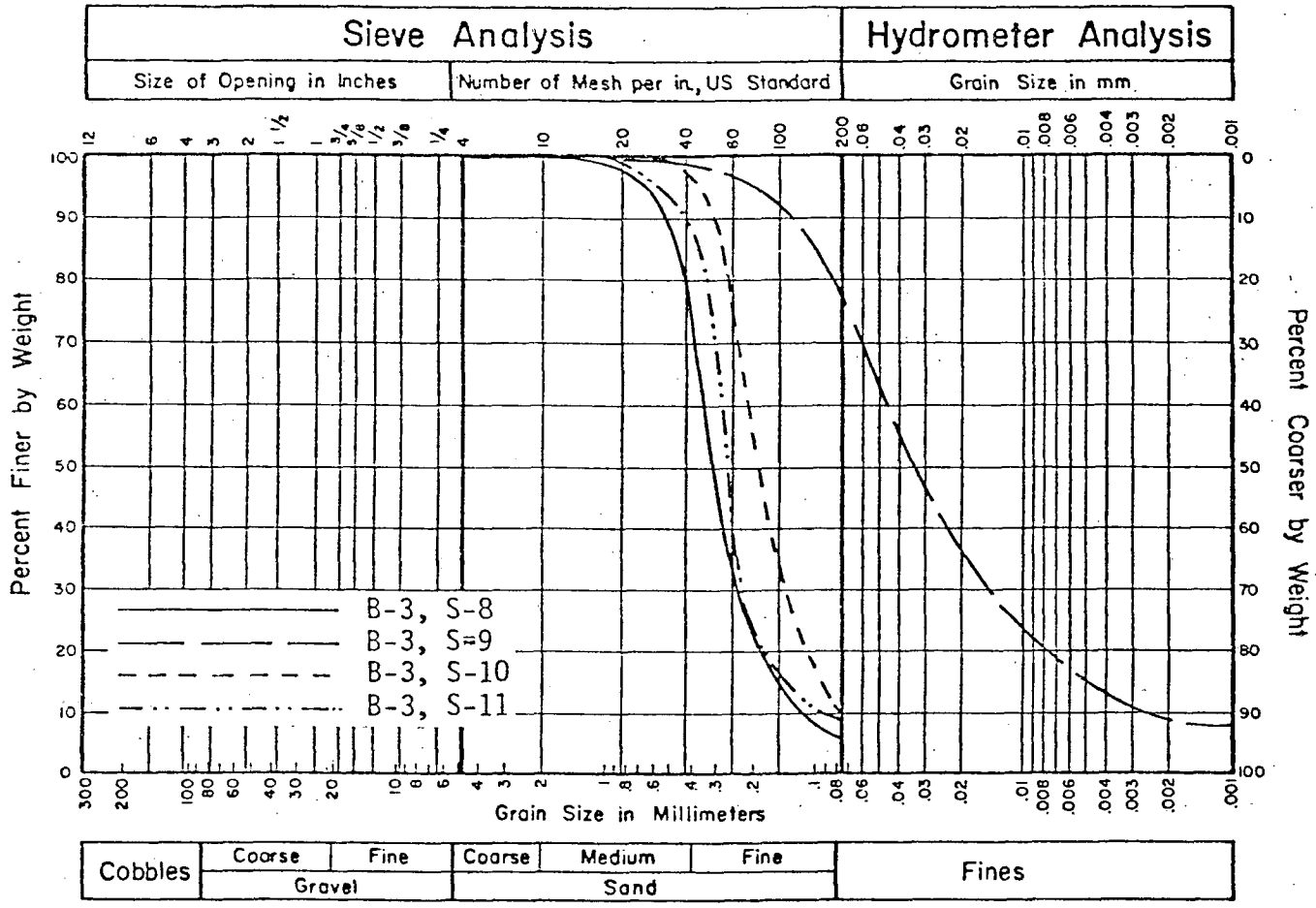
Figure B-3

# GRAIN SIZE CLASSIFICATION




| Sample | Depth-ft | U.S.C. | Classification | Nat. W.C. % | Atterberg Limits LL | PL | PI |
|---|---|---|---|---|---|---|---|
| B-3 S-8 | 20.0 - 21.5 | SP-SM | SLIGHTLY SILTY, FINE TO MEDIUM SAND. | 27 | | | |
| B-3 S-9 | 22.5 - 24.0 | ML | SLIGHTLY CLAYEY, SANDY SILT. | 31 | | | |
| B-3 S-10 | 25.0 - 26.5 | SP-SM | SLIGHTLY SILTY FINE SAND. | 28 | | | |
| B-3 S-11 | 27.5 - 29.0 | SP-SM | SLIGHTLY SILTY, FINE TO MEDIUM SAND. | 29 | | | |

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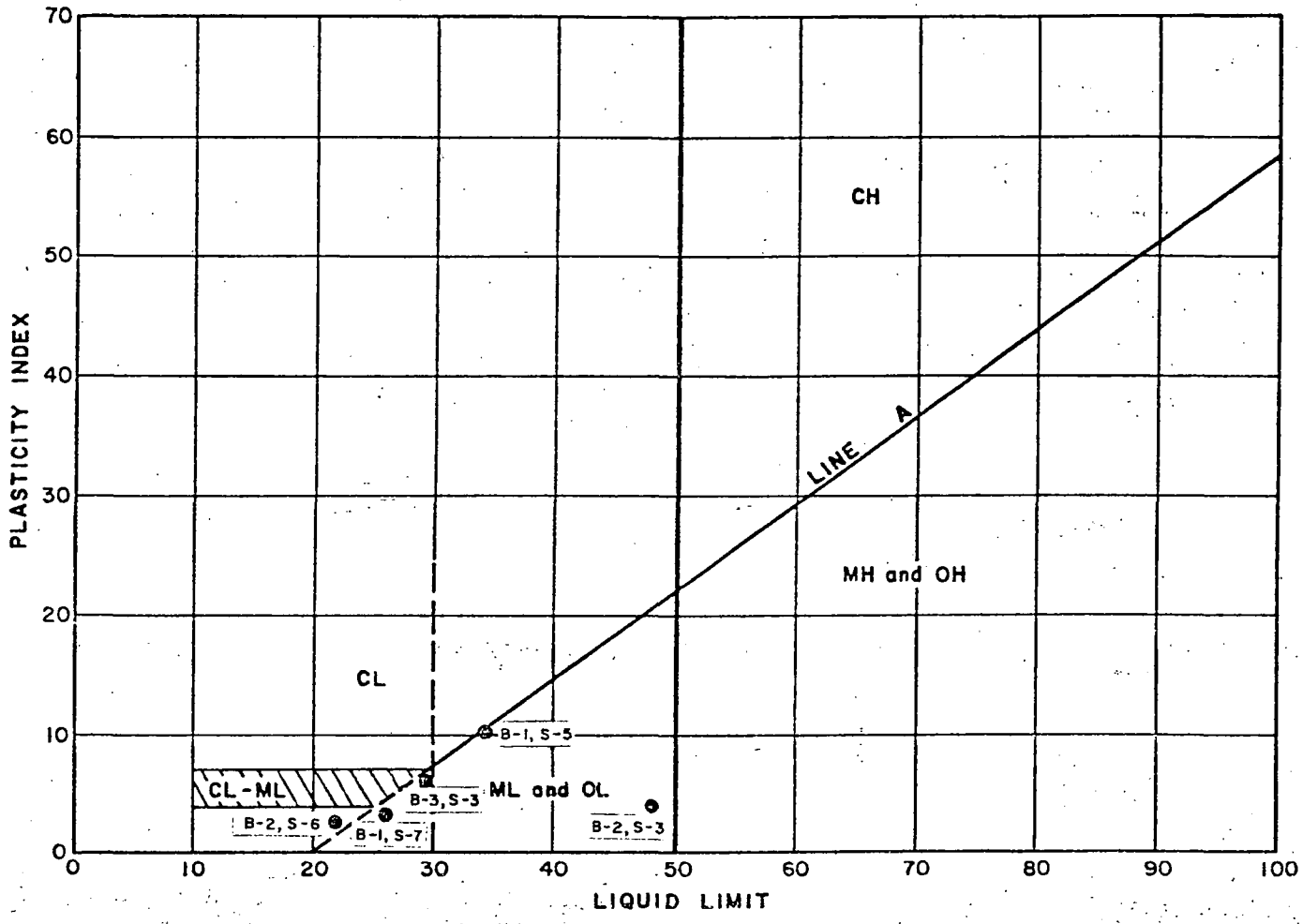
Figure B-4

# PLASTICITY CHART




| Boring | Sample | Depth | W.C. | L.L. | P.L. | P.I. | Classification | U.S.C. |
|---|---|---|---|---|---|---|---|---|
| B-1 | S-5 | 14.0-14.5 | | 34 | 24 | 10 | SANDY SILT. | ML |
| B-2 | S-3 | 8.6 - | | 48.3 | 44.1 | 4.2 | SANDY SILT | ML - OL |
| B-1 | S-7 | 19.0-19.5 | | 26 | 23 | 3 | SANDY SILT. | ML |
| B-2 | S-3 | 8.6-9.5 | | 48.3 | 44.6 | 4.7 | SANDY SILT. | ML-OL |
| | | 16.5 | | | | | | |
| B-2 | S-6 | 16.0-16.5 | | 22 | 20 | 2 | SANDY SILT | ML |
| B-3 | S-3 | 9.4-10.0 | | 30 | 23 | 7 | SANDY SILT. | ML |

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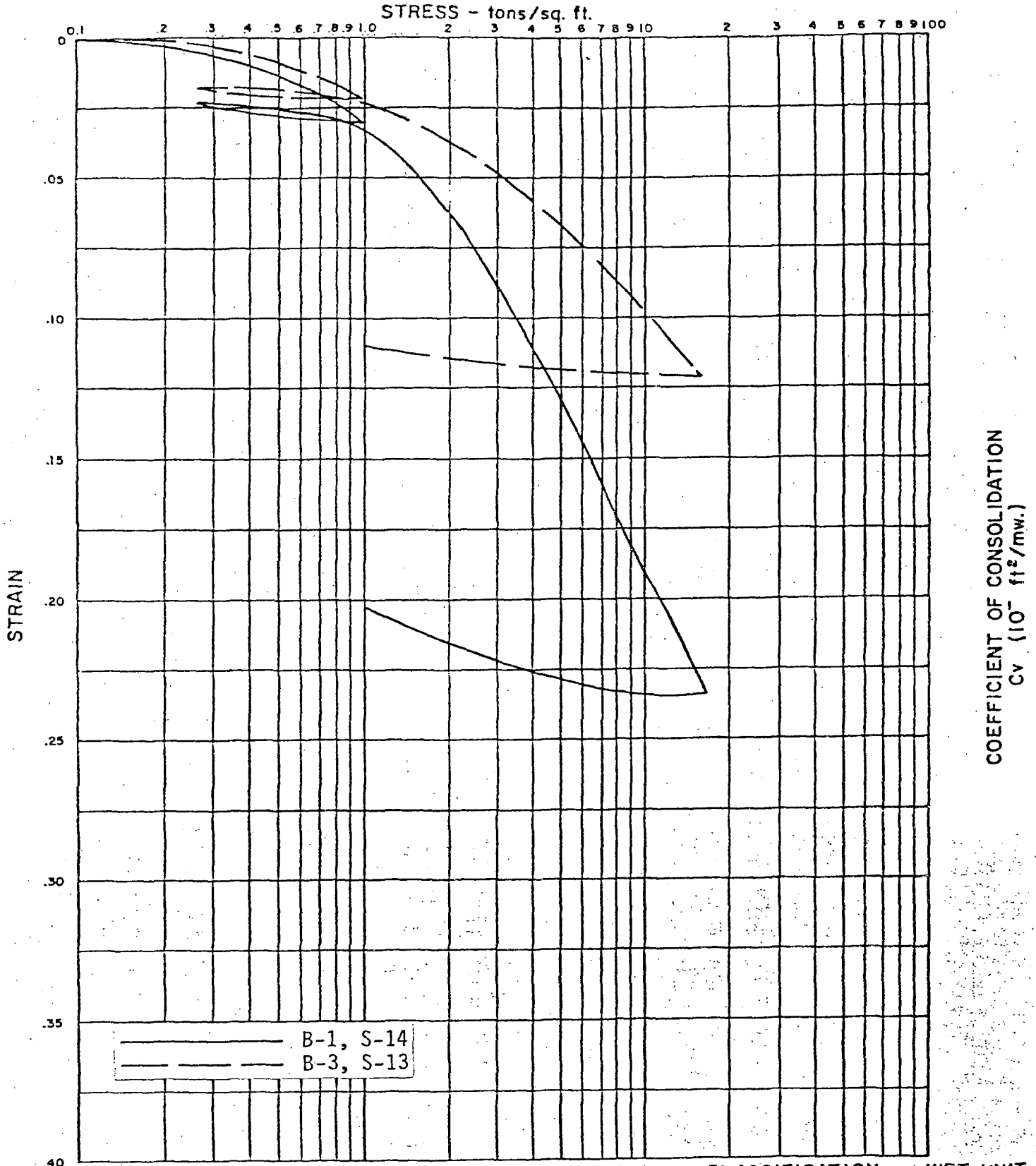
Figure B-5

# CONSOLIDATION TEST RESULTS




| SAMPLE | DEPTH feet | % WATER CONTENT Before | After | ATTERBERG LIMITS LL | PL | PI | CLASSIFICATION | WET UNIT WEIGHT |
|---|---|---|---|---|---|---|---|---|
| B-1, S-14 | 39.4-39.8 | 65 | 47 | | | | | 98.9 |
| B-3, S-13 | 33.6-33.7 | 45 | 36 | | | | | 108.4 |

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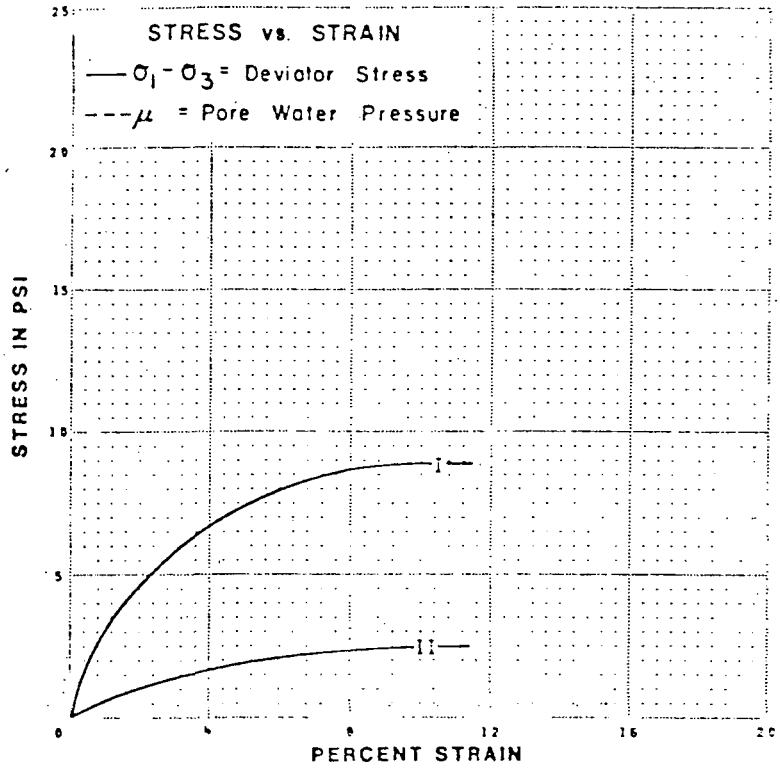
Figure B-6

# TRIAXIAL COMPRESSION TESTS




**Sample Classification and Remarks:**

UNCONSOLIDATED-UNDRAINED (UU) CHAMBER PRESSURE APPLIED WITHOUT CONSOLIDATION. SAMPLE SHEARED WITHOUT DRAINAGE.

| TEST NO. | | I | II | III |
|---|---|---|---|---|
| BORING NO. | | B-2 | B-3 | |
| SAMPLE NO. | | S-3 | S-3 | |
| DEPTH | | 8.6-9.5 | 9.4-10.0 | |
| CELL PRESSURE, PSI | | 7.5 | 7.5 | |
| CONSOLIDATION PRESSURE, PSI | | 7.5 | 7.5 | |
| HEIGHT, INCHES | | 6.24 | 6.01 | |
| DIAMETER, INCHES | | 2.85 | 2.78 | |
| WET UNIT WT., PCF | | 94.7 | 114.4 | |
| W.C. %, BEFORE TEST | | 60.2 | 42.8 | |
| W.C. %, AFTER TEST | | 59.7 | 41.3 | |
| ATTERBERG LIMITS % | LL | 48 | 30 | |
| | PL | 44 | 23 | |
| | PI | 4 | 7 | |

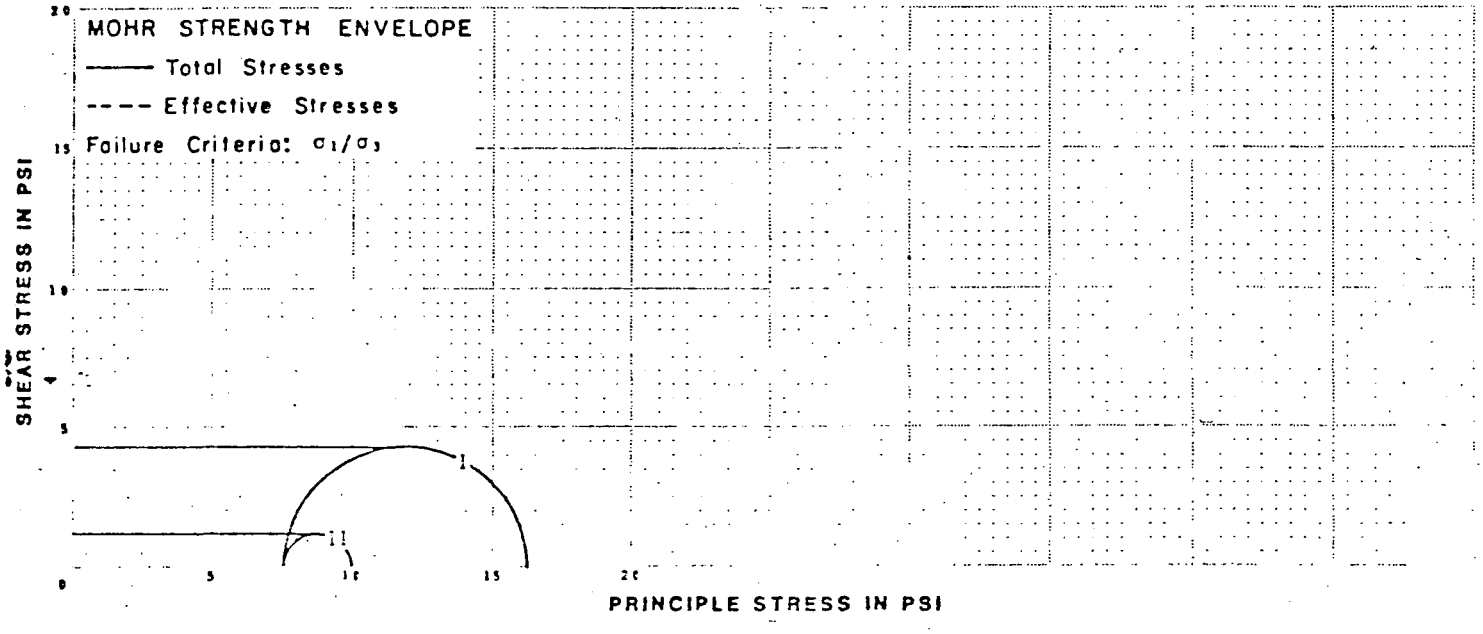



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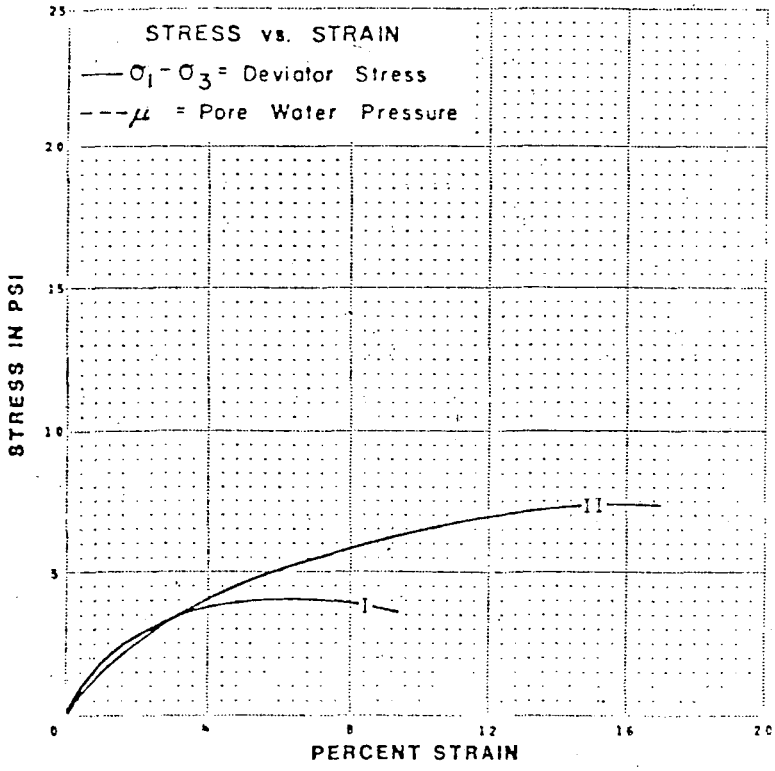
Figure B-7

# TRIAXIAL COMPRESSION TESTS




**Sample Classification and Remarks:**

UNCONSOLIDATED-UNDRAINED (UU) CHAMBER PRESSURE APPLIED WITHOUT CONSOLIDATION SAMPLE SHEARED WITHOUT DRAINAGE.

| TEST NO. | | I | II | III |
|---|---|---|---|---|
| BORING NO. | | B-1 | B-1 | |
| SAMPLE NO. | | S-5 | S-7 | |
| DEPTH | | 14.0-14.5 | 19.0-19.5 | |
| CELL PRESSURE, PSI | | 10.0 | 15.0 | |
| CONSOLIDATION PRESSURE, PSI | | 10.0 | 15.0 | |
| HEIGHT, INCHES | | 6.43 | 6.43 | |
| DIAMETER, INCHES | | 2.80 | 2.77 | |
| WET UNIT WT., PCF | | 103.6 | 118.9 | |
| W.C. %, BEFORE TEST | | 57.5 | 35.9 | |
| W.C. %, AFTER TEST | | 56.9 | 35.5 | |
| ATTERBERG LIMITS % | LL | 39 | 26 | |
| | PL | 24 | 23 | |
| | PI | 10 | 3 | |

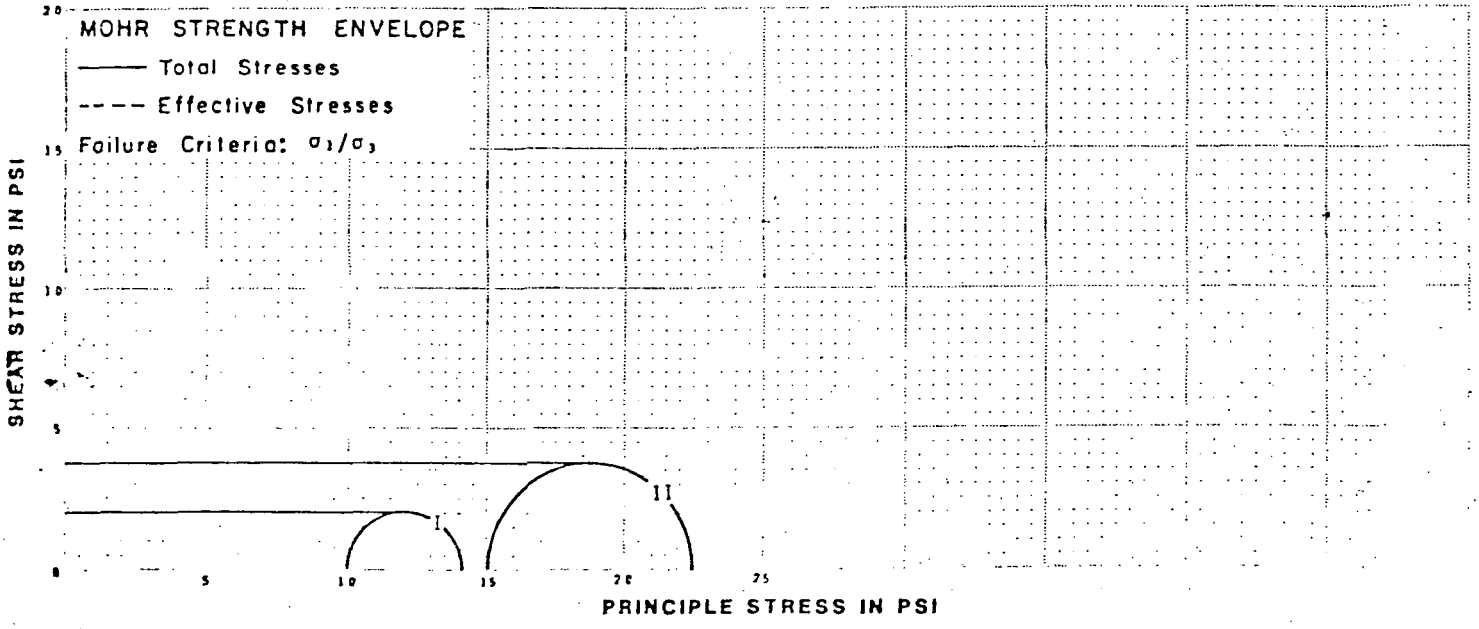



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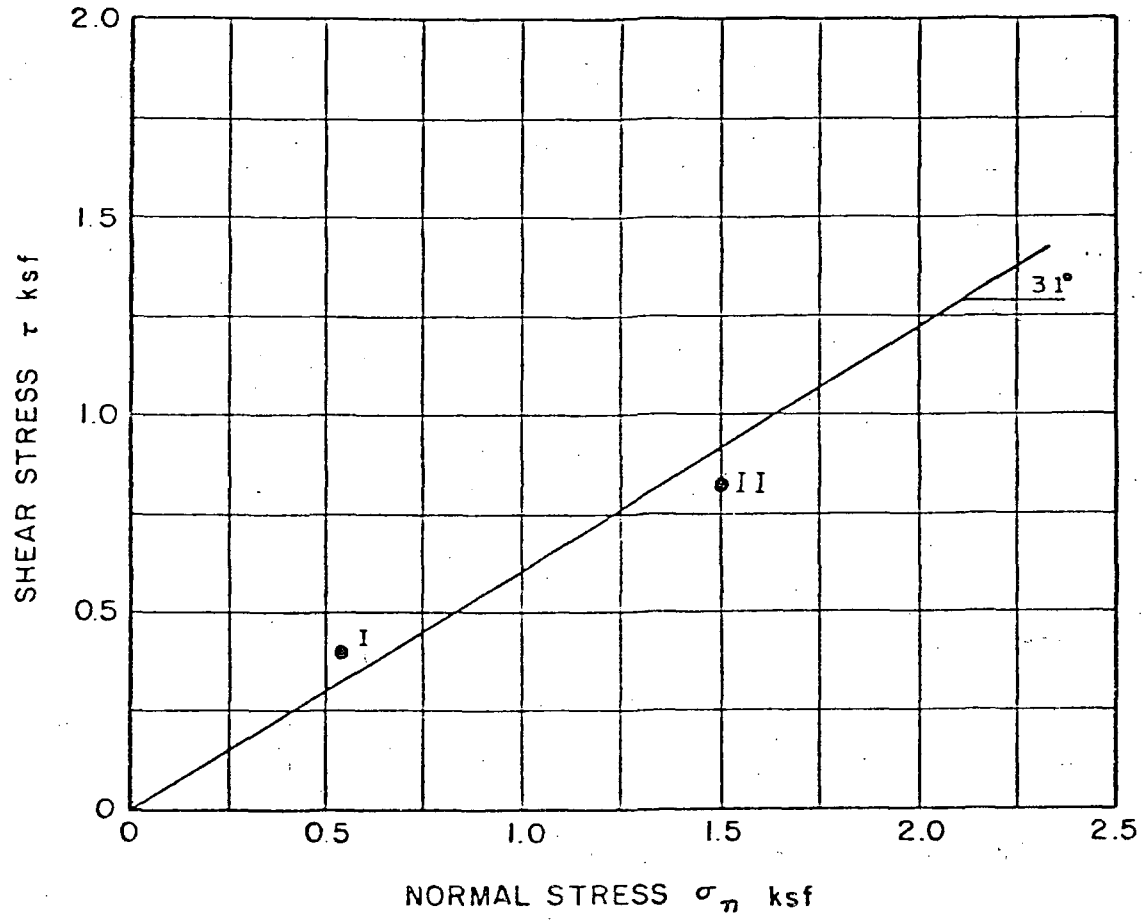
Figure B-8

# DIRECT SHEAR TEST RESULTS



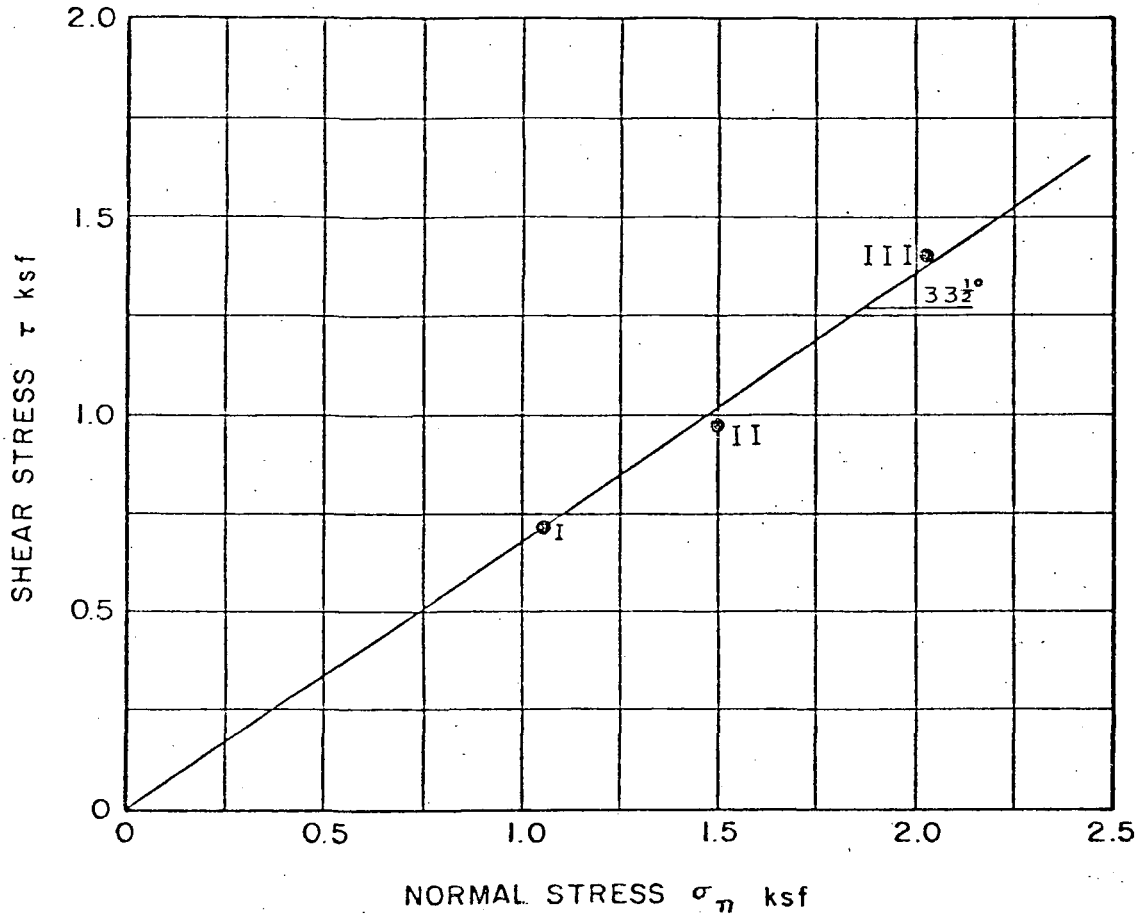

| TEST NO. | SAMPLE | DEPTH FEET | COMPACTION | %W.C. | ATTERBERG LIMITS LL | PL | PI | CLASSIFICATION | WET UNIT WEIGHT (pcf) |
|---|---|---|---|---|---|---|---|---|---|
| | B-1, S-5 | 14.0-14.2 | | | 34 | 24 | 10 | SANDY SILT. | |
| | I | 14.2 | | 49 | | | | | 103 |
| | II | 14.0 | | 72 | | | | | 92 |

J-988 March 1981

HART-CROWSER & associates inc.

Figure 8-9

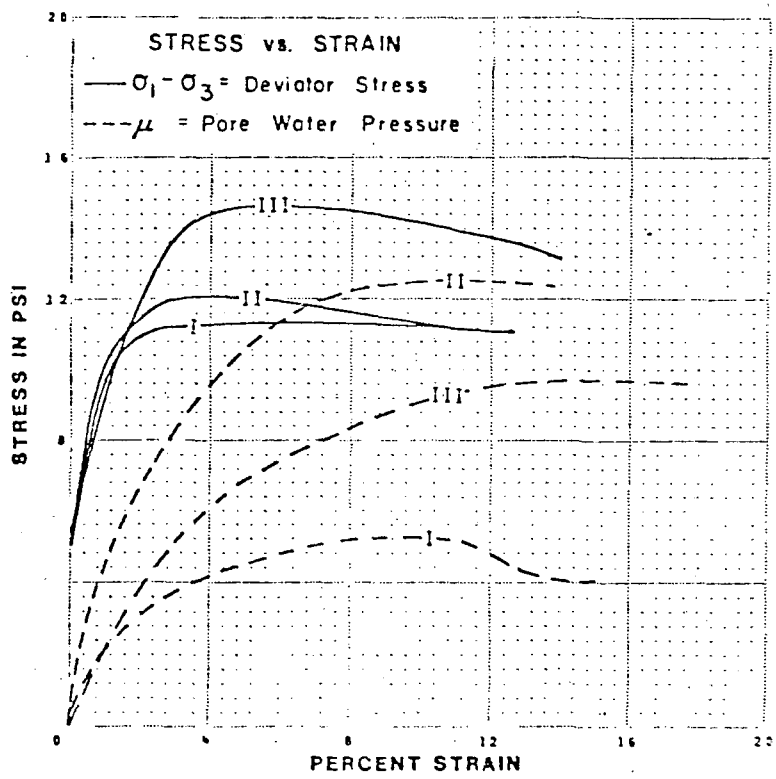
# DIRECT SHEAR TEST RESULTS




| TEST NO. | SAMPLE | DEPTH FEET | COMPACTION | %W.C. | ATTERBERG LIMITS LL PL PI | CLASSIFICATION | WET UNIT WEIGHT (pcf) |
|---|---|---|---|---|---|---|---|
| | B-1, S-7 | | | | | CLAYEY SILT | |
| | I | 18.9-19.0 | | 40 | | | 109 |
| | II | 18.8-18.9 | | 38 | | | 118 |
| | III | 18.7-18.8 | | 38 | | | 111 |

J-988 March 1981
HART-CROWSER & associates inc.
Figure B-10

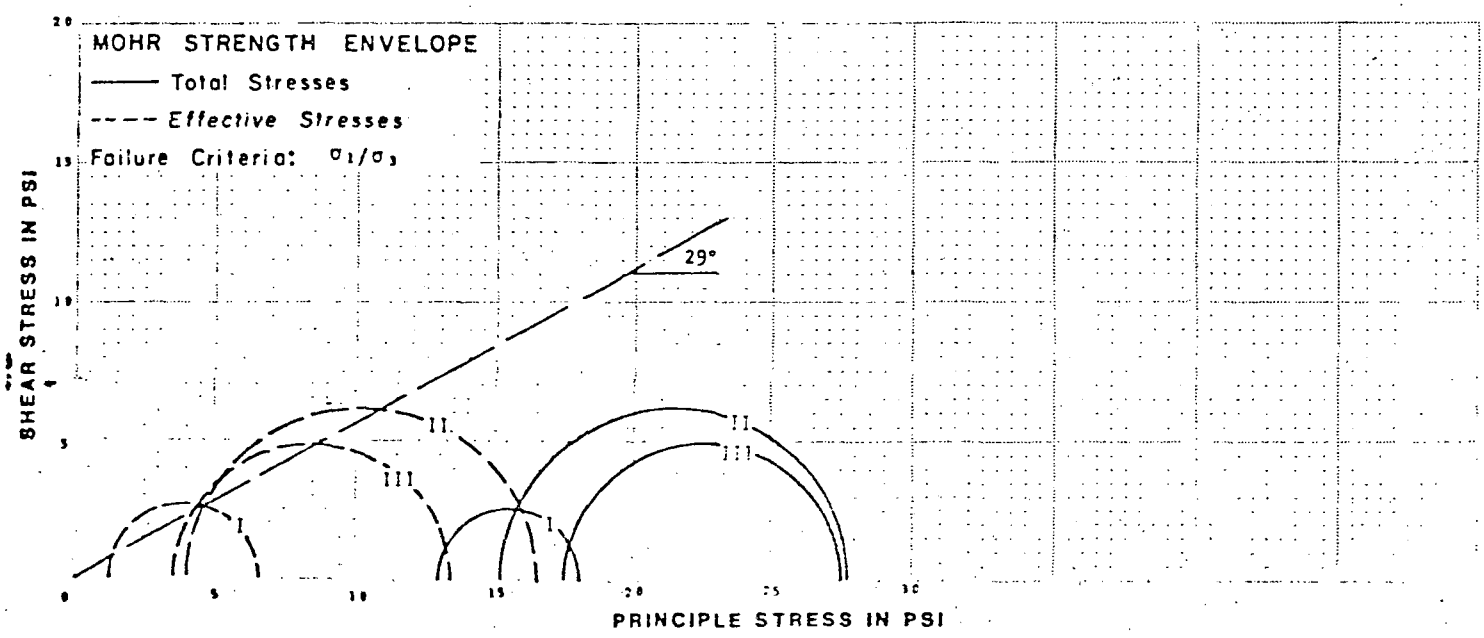
# TRIAXIAL COMPRESSION TESTS




**Sample Classification and Remarks:**

CONSOLIDATED-UNDRAINED (CU)
ALL SAMPLES CONSOLIDATED TO $D_{50}$
AND SHEARED UNDER CONSOLIDATION PRESSURE.
STRAIN RATE .010 IN/MIN. 5 PSI BACK
PRESSURE. ALL SAMPLES WERE VERY SOFT.

| TEST NO. | | I | II | III |
|---|---|---|---|---|
| BORING NO. | | B-2 | B-2 | B-2 |
| SAMPLE NO. | | S-6 | S-6 | S-6 |
| DEPTH | | 15.3-15.8 | 16.0-16.5 | 16.5-17.0 |
| CELL PRESSURE, PSI | | 12.5 | 15 | 17.5 |
| CONSOLIDATION PRESSURE, PSI | | 5.0 | 5.0 | 5.0 |
| HEIGHT, INCHES | | 6.39 | 6.16 | 6.59 |
| DIAMETER, INCHES | | 2.82 | 2.80 | 2.77 |
| WET UNIT WT., PCF | | 118.9 | 122.1 | 122.7 |
| W.C. %, BEFORE TEST | | 31.2 | 33.8 | 31.6 |
| W.C. %, AFTER TEST | | 29.5 | 29.8 | 28.3 |
| ATTERBERG LIMITS % | LL | 22 | 22 | 22 |
| | PL | 20 | 20 | 20 |
| | PI | 2 | 2 | 2 |




J-988 March 1981

HART-CROWSER & associates Inc.

Figure B-11

J-988

# APPENDIX C

## LAUCKS TESTING LABORATORIES, INC.

Laucks
Testing Laboratories, Inc.
1008 Western Avenue, Seattle, Washington 98104 (206) 622-0727

# Certificate

Chemistry, Microbiology, and Technical Services

CLIENT Hart-Crowser & Associates, Inc.
1910 Fairview Avenue East
Seattle, WA 98102

LABORATORY NO. 72951

DATE March 20, 1981

REPORT ON WATER

SAMPLE IDENTIFICATION Marked: 1) B-1 RCP 3/5 3:12
2) B-2 RCP 3/5 8:00 AM
3) B-3 RCP 3/5 11:10

TESTS PERFORMED AND RESULTS:

| | 1 | 2 | Maximum Contamination Levels* |
|---|---|---|---|
| Nephelometer Turbidity, units | 40 | 100 | 1.0 |
| Color, units | 500 | 100 | 15 |
| Specific Conductance, micromhos/cm at 25°C | 17,000 | 16,000 | 700 |
| | parts per million | | |
| Arsenic | Less/.01 | Less/.01 | .05 |
| Barium | Less/.25 | Less/.25 | 1.0 |
| Cadmium | .024 | .019 | .01 |
| Chromium | .020 | .014 | .05 |
| Iron | 4.9 | 4.1 | .3 |
| Lead | .18 | .16 | .05 |
| Manganese | 2.2 | 1.4 | .05 |
| Mercury | Less/.001 | Less/.001 | ..002 |
| Selenium | Less/.005 | Less/.005 | .01 |
| Silver | .024 | .020 | .05 |
| Sodium | 2,700 | 2,300 | 20.0 |
| Fluoride | .8 | 1.0 | 2.0 |
| Nitrate as N | .2 | Less/.2 | 10.0 |
| Chloride | 7,400 | 6,600 | --- |
| Sulfate as $SO_4$ | 40 | 25 | --- |

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Laucks Testing Laboratories, Inc.
1008 Western Avenue Seattle Washington 98104 (206) 622-0727
Chemistry, Microbiology and Technical Services

# Certificate

Hart-Crowser & Associates, Inc.

PAGE NO. 2

LABORATORY NO. 72951

| | 1 | 2 | 2b** | 2c** | 3 | 3b*** |
|---|---|---|---|---|---|---|
| pH, glass electrode at 25°C | 6.0 | 7.3 | --- | --- | 6.9 | --- |
| | | | parts per million | | | |
| Phenols | ND | ND | --- | --- | ND | --- |
| Calcium | 370 | 260 | --- | --- | 80 | --- |
| Total Dissolved Solids | 15,000 | 14,000 | --- | --- | 8,000 | --- |
| Alkalinity as $CaCO_3$ | 620 | 1,400 | --- | --- | 1,100 | --- |
| Hardness as $CaCO_3$ | 4,000 | 3,100 | --- | --- | 1,000 | --- |
| Dissolved Oxygen | ND | 0.1 | ND | 0.1 | 0.1 | 0.1 |

VOLATILES

| | concentration, ug/L | | |
|---|---|---|---|
| | 1 | 2 | 3 |
| acrolein | ND | ND | ND |
| acrylonitrile | ND | ND | ND |
| benzene | ND | ND | ND |
| carbon tetrachloride | ND | ND | ND |
| chlorobenzene | ND | ND | ND |
| 1,2-dichloroethane | ND | ND | ND |
| 1,1,1-trichloroethane | ND | ND | ND |
| 1,1-dichloroethane | ND | ND | ND |
| 1,1,2-trichloroethane | ND | ND | ND |
| 1,1,2,2-tetrachloroethane | ND | ND | ND |
| chloroethane | ND | ND | ND |
| bis(chloromethyl)ether | ND | ND | ND |
| 2-chloroethylvinyl ether | ND | ND | ND |
| chloroform | ND | ND | ND |
| 1,1-dichloroethylene | ND | ND | ND |
| 1,2-trans-dichloroethylene | ND | ND | ND |
| 1,2-dichloropropane | ND | ND | ND |
| 1,3-dichloropropylene | ND | ND | ND |
| ethylbenzene | ND | ND | ND |

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Chemistry Microbiology and Technical Services

Certificate

Hart-Crowser & Associates, Inc.

PAGE NO. 3

LABORATORY NO. 72951

| | concentration, ug/L | | |
|---|---|---|---|
| | 1 | 2 | 3 |
| methylene chloride | tr | tr | tr |
| methyl chloride | ND | ND | ND |
| methyl bromide | ND | ND | ND |
| bromoform | ND | ND | ND |
| dichlorobromomethane | ND | ND | ND |
| trichlorofluoromethane | ND | ND | ND |
| dichlorodifluoromethane | ND | ND | ND |
| chlorodibromomethane | ND | ND | ND |
| tetrachloroethylene | ND | ND | ND |
| toluene | tr | ND | ND |
| trichloroethylene | tr | tr | tr |
| vinyl chloride | ND | ND | ND |

*Per current E.P.A. and State of Washington regulations.
**Duplicate of sample #2
***Duplicate of sample #3

Explanation of Symbols

ND = None Detected
tr = Trace (Less than 10 ug/L)

| Lower Limits of Detection (ND) | mg/L |
|---|---|
| Phenols | .005 |
| Dissolved Oxygen | 0.1 |

The lower limit of detection for all of the volatiles, with the exception of acrolein and acrylonitrile is 1 ug/L. Acrolein and acrylonitrile have a limit of detection of 100 ug/L.

Respectfully submitted,

J. M. Owens

JMO:ks

J. M. Owens

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Laucks
Testing Laboratories, Inc.
1008 Western Avenue, Seattle, Washington 98104 (206) 622-0727

# Certificate

Chemistry, Microbiology, and Technical Services

CLIENT Hart Crowser & Associates, Inc.
1910 Fairview Avenue East
Seattle, WA 98102

LABORATORY NO. 72951-b

DATE March 20, 1981

REPORT ON WATER

SAMPLE IDENTIFICATION 3) B-3 RCP 3/5 11:10

| TESTS PERFORMED AND RESULTS: | Your Sample | Maximum Contamination Levels* |
|---|---|---|
| Nephelometer Turbidity, Units | 25 | 1 |
| Color, Units | 150 | 15 |
| Specific Conductance, micromhos/cm at 25°C | 9,000 | 700 |
| | parts per million | |
| Arsenic | <.01 | .05 |
| Barium | <.25 | 1.0 |
| Cadmium | .015 | .01 |
| Chromium | .022 | .05 |
| Iron | 4.7 | .3 |
| Lead | .08 | .05 |
| Manganese | 1.6 | .05 |
| Mercury | <.001 | .002 |
| Selenium | <.005 | .01 |
| Silver | <.01 | .05 |
| Fluoride | .6 | 2.0 |
| Nitrate, as N | .4 | 10.0 |
| Sodium | 1,400. | 20.0 |
| Chloride | 3,200. | --- |
| Sulfate as $SO_4$ | 12. | --- |

*per current E.P.A. and State of Washington regulations.

Respectfully submitted,

Laucks Testing Laboratories, Inc.

J. M. Owens

JMO:bg

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# REPORT

# OF

# PRELIMINARY
# SUB-SURFACE SOILS INVESTIGATION

CAP SANTE WATERWAY IMPROVEMENTS

ANACORTES, WASHINGTON

FOR

SNELSON-ANVIL, INC.
P. O. BOX 70
ANACORTES, WASHINGTON

ANVIL CORPORATION

JOB NO. S-79-837
SEPTEMBER, 1979

ENGINEERING DESIGN
PROJECT MANAGEMENT
TESTING LABORATORIES
INSPECTION

ANVIL CORPORATION

1675 W. BAKERVIEW ROAD
BELLINGHAM, WA 98225
(206) 671-1450

September 24, 1979
Job No. 79-837

Snelson-Anvil, Inc.
P. O. Box 70
Anacortes, Washington

Attention: Mr. L. K. Levorsen

Project: Cap Sante Waterway Improvements, Anacortes, Washington

Report of: Preliminary Sub-Surface Soils Investigation

## INTRODUCTION

This report presents the results of our Preliminary Sub-surface Soils Investigation at the above site. The investigation was conducted both on-shore and off-shore in order to determine the physical characteristics of the soils, especially pertaining to dredging and disposal operations. Studies were also made regarding the engineering properties of material removed by dredging and deposited on the land.

This study provides the following:

1. A general evaluation of sub-surface conditions.
2. Relative ease or difficulty of dredging the bottom sediment lying near shore of Fidalgo Bay.
3. Existing sub-surface soil conditions surrounding the abandoned Scott Paper Mill.
4. Engineering characteristics of soil deposited on the paper mill property on and off the shore line.
5. Recommendations for construction procedures to be followed in order to provide working areas for construction and loading of modules.

## BACKGROUND INFORMATION

This preliminary investigation is aimed at providing general information as to the feasibility of the development of the area. More detailed soil studies will be needed later when specific design studies of facilities are made.

The site lies on the relatively protected northwest shore of Fidalgo Bay, on the east side of Anacortes, Washington. A small boat harbor (Cap Sante Marina) has been developed just to the north of the subject area. The boat basin has been protected with two wooden pile breakwaters and a rock jetty. The Cap Sante Waterway, providing a passageway from the boat basin to deep water east of Cap Sante, is maintained with a width of 150' and a depth of 15'. (All elevations in this report are in reference to Elevation 0.0, which is the mean lower low water, and is equivalent to Elevation 90.36 of the City of Anacortes datum.)

Snelson-Anvil has developed a modular assembly and barge loading facility just to the south of the area.

The Corps of Engineers dredged a barge channel to depth - 18 in 1975-6 east of the outer harbor line.

The property is the site of the former Scott Paper Company mill. Numerous structures are present in various states of disrepair. One large pile of wood processing waste is present on the north side of the property. This material is currently being slowly hauled away.

Possible site developments include the following:

1. Construction of a side-loading wharf along the north shore of the paper mill property.
2. Construction of pier facilities along the east shore of the paper mill property just south of the existing breakwaters.
3. Construction of barge-loading facilities similar to Snelson-Anvil's present north loading area.
4. Development of present paper mill property into some light industrial usage, especially types needing adjacent pier facilities.

A study of the Dames & Moore Report, No. 7923-001-05, of January 17, 1975, and the Roger Lowe Associates Report of January 30, 1976, was made in order to correlate the information contained therein. Those reports covered test borings made on Snelson-Anvil's property to the south of the area covered by the present investigation.

TEST METHODS

Eight (8) test pits were dug with a rubber-tired backhoe at the locations shown in the attached Test Hole Location Plan. Test pits were dug to depths of up to 12' below existing grade.

Twelve (12) test holes were drilled off-shore with a truck-mounted rotary

hydraulic drilling rig. The rig was supported on a 30' x 70' barge especially equipped for drilling holes through a hatch in the center of the barge. The barge was positioned with anchors and/or mooring cables. The test holes were advanced either to a dense sand or to elevation -20. Hole 18 was drilled to --37.6.

Representative samples were obtained in order to classify the soils. Samples were tested for moisture content by ASTM Method D-2216 and for Atterberg Limits by ASTM Methods D-423 and D-424. The % passing the #200 sieve was determined by ASTM D-1140. These tests were used to classify the soils by ASTM D-2487 and D-2488.

Standard penetration tests were made in accordance with ASTM Method D-1586, at about 5' intervals in all off-shore test holes.

All sampling and testing was performed under the supervision of the undersigned, and our representative was present during all operations.

TEST RESULTS

The detailed test results are presented in the accompanying Soils Logs. The on-shore test pits showed the material to consist primarily of wood mill waste, logs, organic matter, etc. Test Pit 2 did encounter a deposit of dense sandy silt at an elevation of +5.5, however, Test Pit 3 showed considerable gravelly sand below 2', which was probably an old granular fill.

The off-shore borings showed the primary sediment to consist of an inorganic silt, Classification ML, that is in a soft condition. The "0" (zero) blow count shown in the Logs for this material means that the weight of the drilling rods and sampler was sufficient to drive the sampler at least 1' without any blows of the drop hammer. This deposit is deepest in the area furthest from shore (TH 19), and becomes shallower as the shoreline is approached.

Below the deposit of soft inorganic silt, the soil consists of a light grey sandy silt that becomes denser and coarser with depth. The top of this stratum is located at Elevation -20.4 in TH 15, -16.3 in TH 14, -20.9 in TH 9, -12.7 in TH 8, and -11.4 in TH 9. This deposit classifies as an SM-ML. It has probably been consolidated by the tremendous pressure of the ice formed during the Fraser glaciation some 11,000 years ago. This glacier at times was up to 5000' thick, and the ground surface was depressed some 300 to 400' below sea level.

Test Holes 18 and 19 also showed the sediment contains no shell fragments, but the relative quantity increases the closer one comes to shore.

Test Hole 11 showed a pocket of coarser sand with gravel at an elevation of -15.5, although the blow count was 0.

The Dames and Moore and Roger Lowe reports indicated that along the shoreline, the dense sandy silt extends to an elevation of about -13. Below -13 the soil consists of a very hard clayey silt with blow counts well above 100/ft.

## DISCUSSION AND RECOMMENDATIONS

### DREDGING

The area more than 1000' east of the shoreline should be easy to dredge by clamshell or hopper to an elevation of -20. This includes the area beyond the outer harbor line north of the Snelson-Anvil/Port of Anacortes property line, and more than 200' beyond the inner harbor line south of that property line. The area south of the property line contains soft sediment to at least -15 everywhere east of the inner harbor line.

Dredging would be more difficult in the dense sandy silt below -11 in the vicinity of Test Holes 8 and 9.

Because the silt deposit contains little clay, dredging will not produce an excessive amount of turbidity.

If sufficient area for fill can be established along the shoreline south of the Port's property line, the dredged material could be deposited by using a bulkhead for containment. Exposure to air and drainage would result in rapid firming of the silty soil. It would probably be desirable to place the coarser sandy silt at elevations below 0.

The process of drying and firming could be accelerated by crust manipulation such as described by Haliburton of the Corps of Engineers in Technical Report DS-78-11 of the Dredged Material Research Program of the Waterways Experiment Station.

Any excess of dredged material over that placed on the shoreline should be disposed of in designated areas of open water by bottom dumping.

### ENGINEERING PROPERTIES OF DREDGED FILL

The fine sandy silt nature of the sediment is not good foundation material. However, after dewatering and densifying, it could be used as a foundation for an industrial work area, provided sufficient stabilization procedures were followed.

Assuming the silty soil could be brought to a CBR value of at least 3, an adequate work area for heavy industrial traffic could be achieved by the placement of at least 30" of pit run gravel, 10" of crushed base course, and a wearing surface either of crushed rock top course or asphaltic concrete. Some settlement of the underlying fill could be expected, but would occur in a very short time.

CONCLUSIONS

Additional test borings may be needed when specific structures are planned. If there are any questions regarding this preliminary investigation, please contact the undersigned.

Respectfully submitted,

ANVIL CORPORATION

Wallace L. Rankin, P.E.
Soils Engineer

WLR/mp

# ANVIL CORPORATION

PROJECT Cap Sante Waterway CLIENT Snelson-Anvil

LOCATION Anacortes, WA

JOB NO. 79-837

## SOILS LOG

| HOLE NO. | LOCATION | ENGINEER |
|---|---|---|
| 1 | N554,550 E1,570,260 | G. Richardson |
| **DRILL TYPE** | **BIT TYPE** | **DRILLER** |
| Backhoe | | – |
| **SURFACE ELEV.** | **TOTAL DEPTH** | **WATER LEVEL** |
| 14.6 | 9.0' | 8.0' |
| **DATE STARTED** | **DATE COMPLETED** | **WEATHER** |
| August 23, 1979 | August 23, 1979 | Cloudy |

| ELEV. 14.6 | DEPTH FT. | SAMPLE NO. | N B/FT. | MOIST % | DESCRIPTION | SYMB |
|---|---|---|---|---|---|---|
| | | | | | Dark brown sandy silt | |
| | | 1 | | | highly organic - large logs | |
| | | | | | wood chips + misc. debris | |
| | | | | | moist - loose | |
| 6.6 | 8.0 | | | | | |
| 5.6 | 9.0 | | | | | |
| | | | | | Bottom of Hole | |

# ANVIL CORPORATION

PROJECT Cap Sante Waterway CLIENT Snelson-Anvil

LOCATION Anacortes, WA

JOB NO. 79-837

## SOILS LOG

| HOLE NO. | LOCATION | ENGINEER |
|---|---|---|
| 2 | N554,650 E1,570,140 | G. Richardson |
| **DRILL TYPE** | **BIT TYPE** | **DRILLER** |
| Backhoe | – | – |
| **SURFACE ELEV.** | **TOTAL DEPTH** | **WATER LEVEL** |
| 14.0 | 10.0' | 8.5' |
| **DATE STARTED** | **DATE COMPLETED** | **WEATHER** |
| August 23, 1979 | August 23, 1979 | Cloudy |

| ELEV. | DEPTH FT. | SAMPLE NO. | N B/FT. | MOIST % | DESCRIPTION | SYMB |
|---|---|---|---|---|---|---|
| 14.0 | | | | | | |
| | 1.5 | 1 | | | Light grey gravelly silty sand some wood fiber-moist-med.dense | |
| | | 2 | | | Dark brown wood fiber, saw dust and wood chips<br>Moist - loose | |
| 5.5 | 8.5 | | | | | |
| 4.0 | 10.0 | 3 | | | Light grey brown gravelly sandy silt saturated-dense | |
| | | | | | Bottom of Hole | |

ANVIL 78

# ANVIL CORPORATION

PROJECT Cap Sante Waterway CLIENT Snelson-Anvil

LOCATION Anacortes, WA

JOB NO. 79-837

## SOILS LOG

| HOLE NO. | LOCATION | ENGINEER |
|---|---|---|
| 3 | N554,800 E1,570,190 | G. Richardson |
| **DRILL TYPE** | **BIT TYPE** | **DRILLER** |
| Backhoe | – | – |
| **SURFACE ELEV.** | **TOTAL DEPTH** | **WATER LEVEL** |
| 13.0 | 10.0' | 8.0' |
| **DATE STARTED** | **DATE COMPLETED** | **WEATHER** |
| August 23, 1979 | August 23, 1979 | Cloudy |

| ELEV. | DEPTH FT. | SAMPLE NO. | N B/FT. | MOIST % | DESCRIPTION | SYMB |
|---|---|---|---|---|---|---|
| 13.0 | | | | | | |
| | 2.0 | 1 | | | Dark brown gravelly sandy silt highly organic – wood chips moist – loose | |
| | | 2 | | | Boulders – shale<br>Light grey gravelly sand moist – dense<br>Fill | |
| 5.0 | 8.0 | | | | | |
| 3.0 | 10.0 | | | | | |
| | | | | | Bottom of Hole<br>Buried wood retaining wall exposed W. of hole | |

ANVIL 78

# ANVIL CORPORATION

PROJECT Cap Sante Waterway CLIENT Snelson-Anvil

LOCATION Anacortes, WA

JOB NO. 79-837

## SOILS LOG

| HOLE NO. | LOCATION | ENGINEER |
|---|---|---|
| 4 | N555,120 E1,570,210 | G. Richardson |
| **DRILL TYPE** | **BIT TYPE** | **DRILLER** |
| Backhoe | – | – |
| **SURFACE ELEV.** | **TOTAL DEPTH** | **WATER LEVEL** |
| 13.0 | 11.5' | 8.0 |
| **DATE STARTED** | **DATE COMPLETED** | **WEATHER** |
| August 23, 1979 | August 23, 1979 | Cloudy |

| ELEV. | DEPTH FT. | SAMPLE NO. | N B/FT. | MOIST % | DESCRIPTION | SYMB |
|---|---|---|---|---|---|---|
| 13.0 | | | | | Dark brown gravelly silty sand highly organic-logs, bricks | |
| | | | | | some chemicals-sulfur & gypsum | |
| | | 1 | | | moist - loose | |
| | | | | | Concentrated wood scrap | |
| 5.0 | 8.0 | | | | | |
| 1.5 | 11.5 | | | | | |
| | | | | | Bottom of Hole | |

# ANVIL CORPORATION

PROJECT Cap Sante Waterway CLIENT Snelson-Anvil

LOCATION Anacortes, WA

JOB NO. 79-837

## SOILS LOG

| HOLE NO. | LOCATION | ENGINEER |
|---|---|---|
| 5 | N555,340 E1,570,350 | G. Richardson |
| **DRILL TYPE** | **BIT TYPE** | **DRILLER** |
| Backhoe | – | – |
| **SURFACE ELEV.** | **TOTAL DEPTH** | **WATER LEVEL** |
| 15.0 | 10.5' | 9.0' |
| **DATE STARTED** | **DATE COMPLETED** | **WEATHER** |
| August 23, 1979 | August 23, 1979 | Cloudy |

| ELEV. | DEPTH FT. | SAMPLE NO. | N B/FT. | MOIST % | DESCRIPTION | SYMB |
|---|---|---|---|---|---|---|
| 15.0 | | | | | Dark brown gravelly silty sand highly organic-logs & wood chips. Some chemicals-sulfur-bricks | 2 wood piling |
| | | 1 | | | moist - loose | |
| | | | | | concentrated wood scrap | |
| 6.0 | 9.0 | | | | | |
| 4.5 | 10.5 | | | | | |
| | | | | | Bottom of Hole | |

# ANVIL CORPORATION

PROJECT Cap Sante Waterway CLIENT Snelson-Anvil

LOCATION Anacortes, WA

JOB NO. 79-837

## SOILS LOG

| HOLE NO. | LOCATION | ENGINEER |
|---|---|---|
| 6 | N555,740 E1,570,720 | G. Richardson |
| **DRILL TYPE** | **BIT TYPE** | **DRILLER** |
| Backhoe | – | – |
| **SURFACE ELEV.** | **TOTAL DEPTH** | **WATER LEVEL** |
| 17.0 | 12.0' | 10.0' |
| **DATE STARTED** | **DATE COMPLETED** | **WEATHER** |
| August 23, 1979 | August 23, 1979 | Cloudy |

| ELEV. 17.0 | DEPTH FT. | SAMPLE NO. | N B/FT. | MOIST % | DESCRIPTION | SYMB |
|---|---|---|---|---|---|---|
| | | | | | Dark grey brown gravelly silty sand-high ash or carbon organics logs and misc. wood | |
| | | 1 | | | moist - loose | |
| | | | | | less ash - more wood | |
| 7.0 | 10.0 | | | | | |
| 5.0 | 12.0 | | | | high wood fiber | |
| | | | | | Bottom of Hole | |

ANVIL 79

# ANVIL CORPORATION

PROJECT Cap Sante Waterway
LOCATION Anacortes, WA
CLIENT Snelson-Anvil
JOB NO. 79-837

## SOILS LOG

| HOLE NO. | LOCATION | ENGINEER |
|---|---|---|
| 7 | N556,050 E1,570,400 | G. Richardson |
| **DRILL TYPE** | **BIT TYPE** | **DRILLER** |
| Backhoe | - | - |
| **SURFACE ELEV.** | **TOTAL DEPTH** | **WATER LEVEL** |
| 17.2 | 12.0' | 10.5' |
| **DATE STARTED** | **DATE COMPLETED** | **WEATHER** |
| August 23, 1979 | August 23, 1979 | Cloudy |

| ELEV. | DEPTH FT. | SAMPLE NO. | N B/FT. | MOIST % | DESCRIPTION | SYMB |
|---|---|---|---|---|---|---|
| 17.2 | | | | | | |
| | 0.5 | | | | Dark brown silty sand-topsoil | |
| | | 1 | | | Light brown gravelly silty sand moist - dense little organics | |
| | 5.0 | | | | | |
| | | 2 | | | Dark brown gravelly sand highly organic - wood fiber and misc. wood moist - loose | |
| 6.7 | 10.5 | | | | high concentrations of wood fibers | |
| 5.2 | 12.0 | | | | Bottom of Hole | |

Strong sulfurous odor @ water table where organics were present

ANVIL 78

# ANVIL CORPORATION

PROJECT Cap Sante Waterway CLIENT Snelson-Anvil

LOCATION Anacortes, WA

JOB NO. 79-837

## SOILS LOG

| HOLE NO. | LOCATION | ENGINEER |
|---|---|---|
| 7A | N556,040 E1,569,850 | G. Richardson |
| **DRILL TYPE** | **BIT TYPE** | **DRILLER** |
| Backhoe | – | – |
| **SURFACE ELEV.** | **TOTAL DEPTH** | **WATER LEVEL** |
| 17.5 | 12.0' | 11.0' |
| **DATE STARTED** | **DATE COMPLETED** | **WEATHER** |
| August 23, 1979 | August 23, 1979 | Cloudy |

| ELEV. | DEPTH FT. | SAMPLE NO. | N B/FT. | MOIST % | DESCRIPTION | SYMB |
|---|---|---|---|---|---|---|
| 17.5 | | | | | Heavy overburden | |
| | | | | | Dark grey brown gravelly silty sand | |
| | | 1 | | | High organic - wood fiber and misc. wood | |
| | | | | | pockets of wood fiber | |
| 6.5 | 11.0 | | | | | |
| 5.5 | 12.0 | | | | Bottom of Hole | |

# ANVIL CORPORATION

PROJECT Cap Sante Waterway
CLIENT Snelson-Anvil
LOCATION Anacortes, WA
JOB NO. 79-837

## SOILS LOG

| | | |
|---|---|---|
| HOLE NO. 8 | LOCATION N556,100 E1,571,400 | ENGINEER G. Richardson |
| DRILL TYPE Mobile B-75 | BIT TYPE 2" split tube Hollow stem auger | DRILLER Powell |
| SURFACE ELEV. -5.7 | TOTAL DEPTH 8.5' | WATER LEVEL Tide + 6.8 |
| DATE STARTED 9/4/79 - 1530 | DATE COMPLETED 9/4/79 | WEATHER Overcast-windy |

| ELEV. 0.0 | DEPTH FT. | SAMPLE NO. | N B/FT. | MOIST % | DESCRIPTION | SYMB | |
|---|---|---|---|---|---|---|---|
| | | | | | Water | | |
| -5.7 | 0.0 | | | | Light grey sandy silt | | LL = 29, NP<br>-#200 = 71%<br>Class = ML<br>Ds = 74 pcf |
| | 3.0 | 1 | 0 | 47.1 | saturated<br>soft | | |
| -12.7 | 7.0 | 2 | 26 | 16.2 | Light grey silty sand<br>saturated - very dense | | LL = 14 NP<br>-#200 = 47%<br>Class = SM<br>Ds = 117 pcf |
| -14.2 | 8.5 | | | | Bottom of Hole | | |

ANVIL 72

# ANVIL CORPORATION

PROJECT Cap Sante Waterway

CLIENT Snelson-Anvil

LOCATION Anacortes, Washington

JOB NO. 79-837

## SOILS LOG

| | | |
|---|---|---|
| HOLE NO. 9 | LOCATION N 555,600 E 1,571,200 | ENGINEER Richardson |
| DRILL TYPE Mobile B-75 | BIT TYPE 2" split tube Hollow Stem Auger | DRILLER Powell |
| SURFACE ELEV. -6.9 | TOTAL DEPTH 14.5' | WATER LEVEL Tide + 4.1 |
| DATE STARTED 9/6/79 0750 | DATE COMPLETED 9/6/79 | WEATHER Mild-partly sunny |

| ELEV. 0.0 | DEPTH FT. | SAMPLE NO. | N B/FT. | MOIST % | DESCRIPTION | SYMB | |
|---|---|---|---|---|---|---|---|
| | | | | | Water | | |
| -6.9 | 0.0 | | | | Light grey inorganic silt saturated soft | | Class = ML Ds = 63 pcf |
| | | 1 | 0 | 62.5 | | | |
| -11.4 | 4.5 | | | | Light grey fine sandy silt saturated very dense | | |
| | | 2 | 26 | 27.2 | | | Class = ML Ds = 97 pcf |
| | 9.5 | | | | | | |
| | | 3 | 29 | 27.5 | | | |
| -21.4 | 14.5 | | | | Bottom of Hole | | |

# ANVIL CORPORATION

PROJECT Cap Sante Waterway
LOCATION Anacortes, Washington
CLIENT Snelson-Anvil
JOB NO. 79-837

## SOILS LOG

| HOLE NO. | LOCATION | ENGINEER |
|---|---|---|
| 10 | N 555,100 E 1,571,200 | Richardson |
| **DRILL TYPE** Mobile B-75 | **BIT TYPE** 2" split tube Hollow Stem Auger | **DRILLER** Powell |
| **SURFACE ELEV.** -6.6 | **TOTAL DEPTH** 12.0' | **WATER LEVEL** Tide + 1.9 |
| **DATE STARTED** 9/6/79 0850 | **DATE COMPLETED** 9/6/79 | **WEATHER** Mild-partly sunny |

| ELEV. 0.0 | DEPTH FT. | SAMPLE NO. | N B/FT. | MOIST % | DESCRIPTION | SYMB | |
|---|---|---|---|---|---|---|---|
| | | | | | Water | | |
| -6.6 | 0.0 | | | | Light grey fine sandy silt<br>Saturated<br>Soft | | Class = ML |
| | | 1 | 2 | 52.7 | | | Ds = 70 pcf |
| | 6.0 | | | | | | |
| | | | | | Some shells | | |
| -17.6 | | 2 | 12 | 41.8 | Medium density | | Ds = 79 pcf |
| -18.6 | 12.0 | | | | Bottom of Hole | | |

# ANVIL CORPORATION

PROJECT Cap Sante Waterway CLIENT Snelson-Anvil

LOCATION Anacortes, Washington

JOB NO. 79-837

## SOILS LOG

| HOLE NO. | LOCATION | ENGINEER |
|---|---|---|
| 11 | N 554,600 E 1,571,200 | Richardson |
| **DRILL TYPE** Mobile B-75 | **BIT TYPE** 2" split tube Hollow stem auger | **DRILLER** Powell |
| **SURFACE ELEV.** -7.0 | **TOTAL DEPTH** 13.5' | **WATER LEVEL** Tide + 8.0 |
| **DATE STARTED** 9/4/79 1715 | **DATE COMPLETED** 9/4/79 | **WEATHER** Overcast - Windy |

| ELEV. 0.0 | DEPTH FT. | SAMPLE NO. | N B/FT. | MOIST % | DESCRIPTION | SYMB | |
|---|---|---|---|---|---|---|---|
| | | | | | Water | | |
| -7.0 | 0.0 | | | | Light grey fine sandy silt<br>Saturated<br>Some Shells | | Class = ML<br>Ds = 78 pcf |
| | 4.0 | 1 | 0 | 42.6 | | | |
| | 8.0 | 2 | 0 | 12.8 | Light grey gravelly sand pocket<br>Loose | | Class = SW |
| -19.5 | | | | | | | |
| -20.5 | 13.5 | 3 | 25 | 24.3 | Light grey sandy silt-very dense<br>Bottom of Hole | | Class = ML<br>Ds = 102 pcf |

ANVIL 73

# ANVIL CORPORATION

PROJECT Cap Sante Waterway
LOCATION Anacortes, Washington
CLIENT Snelson-Anvil
JOB NO. 79-837

## SOILS LOG

| | | | |
|---|---|---|---|
| HOLE NO. 12 | LOCATION N 554,600 E 1,571,400 | ENGINEER Richardson | |
| DRILL TYPE Mobile B-75 | BIT TYPE 2" split tube Hollow Stem Auger | DRILLER Powell | |
| SURFACE ELEV. -7.3 | TOTAL DEPTH 13.5' | WATER LEVEL Tide + 7.7 | |
| DATE STARTED 9/4/79 1623 | DATE COMPLETED 9/4/79 | WEATHER Overcast-windy | |

| ELEV. 0.0 | DEPTH FT. | SAMPLE NO. | N B/FT. | MOIST % | DESCRIPTION | SYMB | |
|---|---|---|---|---|---|---|---|
| | | | | | Water | | |
| -7.3 | 0.0 | | | | Light grey inorganic silt saturated soft some shells | | LL = 31, PI = 1 |
| | | 1 | 0 | 57.2 | | | -#200 = 96% |
| | 3.0 | | | | | | Class = ML |
| | | | | | | | Ds = 66 pcf |
| | | 2 | 0 | 47.5 | | | Ds = 74 pcf |
| | 8.0 | | | | Becoming coarser | | LL = 14, NP |
| | | | | | | | -#200 = 67% |
| | | 3 | 0 | 23.2 | | | Class = ML |
| | | | | | | | Ds = 103 pcf |
| -20.8 | 13.5 | | | | Bottom of Hole | | |

ANVIL 78

# ANVIL CORPORATION

PROJECT Cap Sante Waterway

CLIENT Snelson-Anvil

LOCATION Anacortes, Washington

JOB NO. 79-837

## SOILS LOG

| | | |
|---|---|---|
| HOLE NO. 13 | LOCATION N 555,100 E 1,571,400 | ENGINEER Richardson |
| DRILL TYPE Mobile B-75 | BIT TYPE 2" split tube Hollow stem auger | DRILLER Powell |
| SURFACE ELEV. -7.4 | TOTAL DEPTH 16.5' | WATER LEVEL Tide = +0.6 |
| DATE STARTED 9/6/79 0930 | DATE COMPLETED 9/6/79 | WEATHER Mild-partly sunny |

| ELEV. 0.0 | DEPTH FT. | SAMPLE NO. | N B/FT. | MOIST % | DESCRIPTION | SYMB | |
|---|---|---|---|---|---|---|---|
| | | | | | Water | | |
| -7.4 | 0.0 | | | | Light grey fine sandy silt saturated soft | | Class = ML |
| | | 1 | 0 | 39.7 | some shells | | Ds = 81 pcf |
| | 6.5 | | | | | | |
| | 11.5 | 2 | 2 | 49.7 | | | Ds = 72 pcf |
| -20.9 | 13.5 | | | | Light grey sandy silt Moist - Dense | | Class = ML-SM Ds = 112 pcf |
| | | 3 | 33 | 18.4 | | | |
| -23.9 | 16.5 | | | | Bottom of Hole | | |

ANVIL 76

# ANVIL CORPORATION

PROJECT Cap Sante Waterway CLIENT Snelson-Anvil

LOCATION Anacortes, Washington

JOB NO. 79-837

## SOILS LOG

| HOLE NO. | LOCATION | ENGINEER |
|---|---|---|
| 14 | N 555,600 E 1,571,400 | Richardson |
| **DRILL TYPE** | **BIT TYPE** | **DRILLER** |
| Mobile B-75 | 2" split tube Hollow stem auger | Powell |
| **SURFACE ELEV.** | **TOTAL DEPTH** | **WATER LEVEL** |
| -7.3 | 14.5' | Tide + 8.2 |
| **DATE STARTED** | **DATE COMPLETED** | **WEATHER** |
| 9/5/79 1750 | 9/5/79 | Misty-overcast |

| ELEV. 0.0 | DEPTH FT. | SAMPLE NO. | N B/FT. | MOIST % | DESCRIPTION | SYMB | |
|---|---|---|---|---|---|---|---|
| | | | | | Water | | |
| -7.3 | 0.0 | | | | Light grey fine sandy silt | | Class = ML |
| | | 1 | 0 | 53.6 | saturated soft | | Ds = 69 pcf |
| | 4.5 | | | | some shells | | |
| -16.3 | 9.0 | 2 | 17 | 14.6 | Light grey silty sand | | Ds = 100 to 120 pcf |
| | | | | | Medium to dense | | Class = SM |
| | | | | | Some shells | | |
| | | 3 | 13 | 26.8 | | | |
| -21.8 | 14.5 | | | | Bottom of Hole | | |

ANVIL 78

# ANVIL CORPORATION

PROJECT Cap Sante Waterway CLIENT Snelson-Anvil

LOCATION Anacortes, Washington

JOB NO. 79-837

## SOILS LOG

| HOLE NO. | LOCATION | ENGINEER |
|---|---|---|
| 15 | N 556,200 E 1,571,700 | Rricharson |
| **DRILL TYPE** | **BIT TYPE** | **DRILLER** |
| Mobile B-75 | Hollow Stem Auger | Powell |
| **SURFACE ELEV.** | **TOTAL DEPTH** | **WATER LEVEL** |
| -6.9 | 14.0' | Tide + 4.1 |
| **DATE STARTED** | **DATE COMPLETED** | **WEATHER** |
| 9/5/79 1430 | 9/5/79 | Misty-overcast |

| ELEV. 0.0 | DEPTH FT. | SAMPLE NO. | N B/FT. | MOIST % | DESCRIPTION | SYMB | |
|---|---|---|---|---|---|---|---|
| | | | | | Water | | |
| -6.9 | 0.0 | | | | Light grey fine sandy silt | | Class = ML |
| | | 1 | 0 | 47.6 | Saturated | | Ds = 74 pcf |
| | 4.0 | | | | Soft | | |
| | | | | | Some shells | | |
| | 9.0 | 2 | 0 | 49.0 | | | Ds = 73 pcf |
| -20.4 | 13.5 | 3 | 11 | 42.0 | Light grey sandy silt-medium | | Ds = 79 pcf |
| -20.9 | 14.0 | | | | Bottom of Hole | | |

ANVIL 76

# ANVIL CORPORATION

PROJECT Cap Sante Waterway

CLIENT Snelson-Anvil

LOCATION Anacortes, Washington

JOB NO. 79-837

## SOILS LOG

| | | |
|---|---|---|
| HOLE NO. 16 | LOCATION N 556,100 E 1,572,000 | ENGINEER Richardson |
| DRILL TYPE Mobile B-75 | BIT TYPE 2" Split Tube Hollow Stem Auger | DRILLER Powell |
| SURFACE ELEV. -7.3 | TOTAL DEPTH 14.0' | WATER LEVEL Tide + 3.2 |
| DATE STARTED 9/5/79 1400 | DATE COMPLETED 9/5/79 | WEATHER Misty-overcast |

| ELEV. 0.0 | DEPTH FT. | SAMPLE NO. | N B/FT. | MOIST % | DESCRIPTION | SYMB | |
|---|---|---|---|---|---|---|---|
| | | | | | Water | | |
| -7.3 | 0.0 | | | | Light grey fine sandy silt<br>Saturated | | Class = ML<br>Ds = 78 pcf |
| | | 1 | 0 | 43.3 | | | |
| | 4.0 | | | | Soft<br>Some Shells | | |
| | | 2 | 0 | 52.8 | Some sand lenses | | Ds = 69 pcf |
| | 9.0 | | | | | | |
| | | 3 | 0 | 63.8 | | | Ds = 62 pcf |
| -21.3 | 14.0 | | | | Bottom of Hole | | |

ANVIL 76

# ANVIL CORPORATION

PROJECT Cap Sante Waterway CLIENT Snelson-Anvil

LOCATION Anacortes, Washington

JOB NO. 79-837

## SOILS LOG

| | | |
|---|---|---|
| HOLE NO. 18 (sheet 1 of 2) | LOCATION N 556,090 E 1,572,620 | ENGINEER Richardson |
| DRILL TYPE Mobile B-75 | BIT TYPE 2" Split Tube Hollow stem auger | DRILLER Powell |
| SURFACE ELEV. -11.1 | TOTAL DEPTH 26.5' | WATER LEVEL Tide -1.1 |
| DATE STARTED 9/5/79 1050 | DATE COMPLETED 9/5/79 | WEATHER Rainy-overcast |

| ELEV. 0.0 | DEPTH FT. | SAMPLE NO. | N B/FT. | MOIST % | DESCRIPTION | SYMB | |
|---|---|---|---|---|---|---|---|
| | | | | | Water | | |
| -11.1 | 0.0 | | | | Light grey inorganic silt<br>Saturated<br>Soft | | LL = 42 PI = 7<br>#200 = 93%<br>Class = ML |
| | | 1 | 0 | 84.2 | | | Ds = 51 pcf |
| | 6.5 | | | | No shells | | |
| | | 2 | 0 | 77.2 | | | |
| | 11.5 | | | | | | Ds = 55 pcf |
| -25 | 13.9 | | | | Continued on Sheet 2 | | |

ANVIL 78

# ANVIL CORPORATION

PROJECT Cap Sante Waterway CLIENT Snelson-Anvil

LOCATION Anacortes, Washington

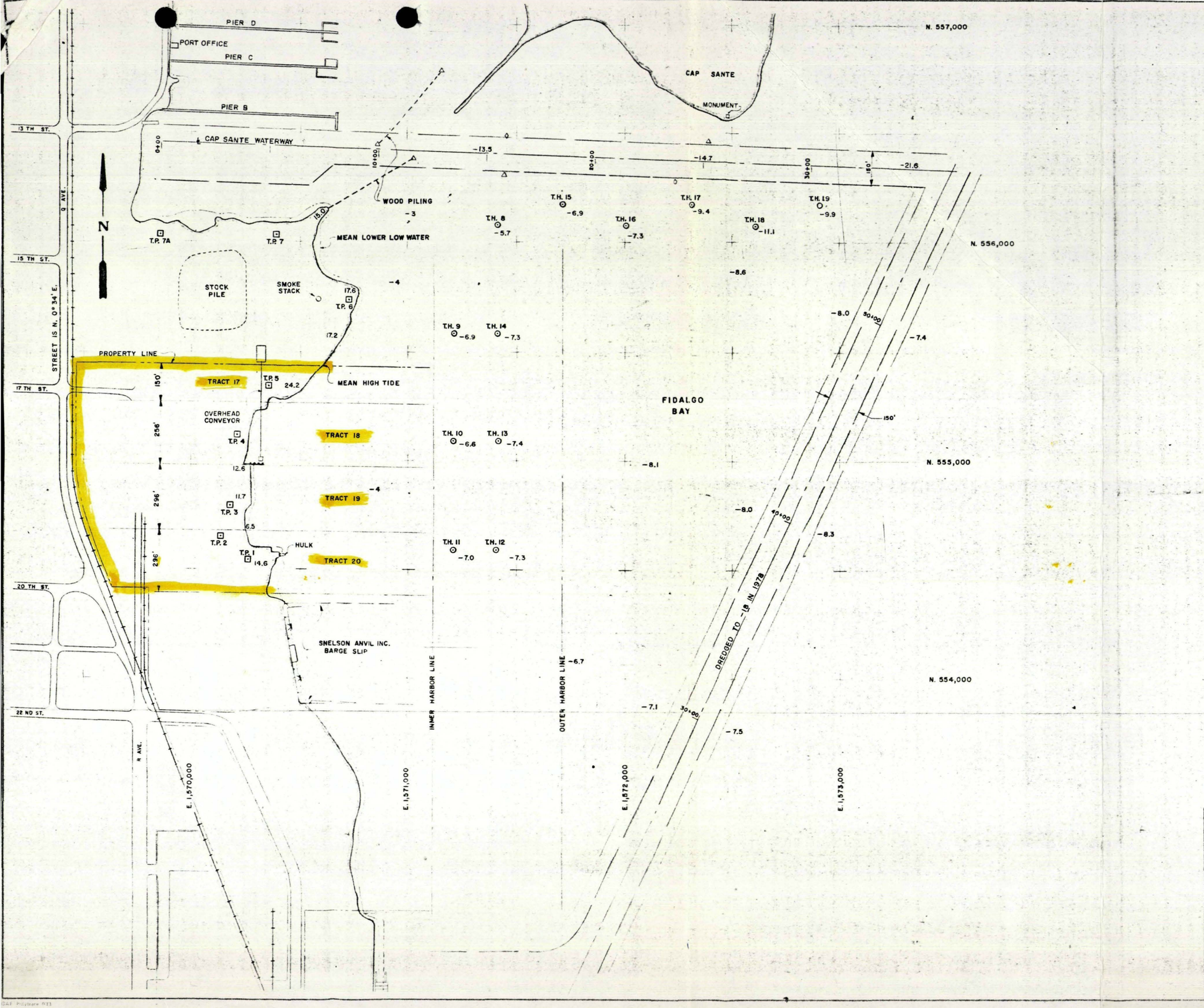
JOB NO. 79-837

## SOILS LOG

| HOLE NO. | LOCATION | ENGINEER |
|---|---|---|
| 19 | N 556,170 E 1,572,920 | Richardson |
| **DRILL TYPE** | **BIT TYPE** | **DRILLER** |
| Mobile B-75 | 2" Split Tube Hollow Stem Auger | Powell |
| **SURFACE ELEV.** | **TOTAL DEPTH** | **WATER LEVEL** |
| -9.9 | 10.5' | Tide -0.4 |
| **DATE STARTED** | **DATE COMPLETED** | **WEATHER** |
| 9/5/79 0910 | 9/5/79 | Rainy-overcast |

| ELEV. | DEPTH FT. | SAMPLE NO. | N B/FT. | MOIST % | DESCRIPTION | SYMB | |
|---|---|---|---|---|---|---|---|
| 0.0 | | | | | Water | | |
| -9.9 | 0.0 | | | | Light grey inorganic silt<br>Saturated<br>Soft | | Class = ML |
| | 5.5 | 1 | 0 | 63.5 | | | Ds = 62 pcf |
| | | | | | No Shells | | |
| -20.4 | 10.5 | 2 | 0 | 68.0 | | | Ds = 59 pcf |
| | | | | | Bottom of Hole | | |

ANVIL 76




## NOTES

**HORIZONTAL REFERENCE**
LAMBERT GRID PROJECTION – WASHINGTON NORTH ZONE

**VERTICAL REFERENCE**
M.L.L.W. = 0. ELEVATIONS IN FEET
CITY OF ANACORTES DATUM EL. 90.36 = 0.00

## LEGEND

⊙ TEST HOLE
⊡ TEST PIT

| REV NO. | DATE | DESCRIPTION | DR. BY | CHECKED BY | PROJ. ENGR. APPR. | CLIENT APPR. |
|---|---|---|---|---|---|---|

**ANVIL** CORPORATION
1815 W. BAYSHORE RD.
BELLINGHAM WA 98225
(206) 671-1459

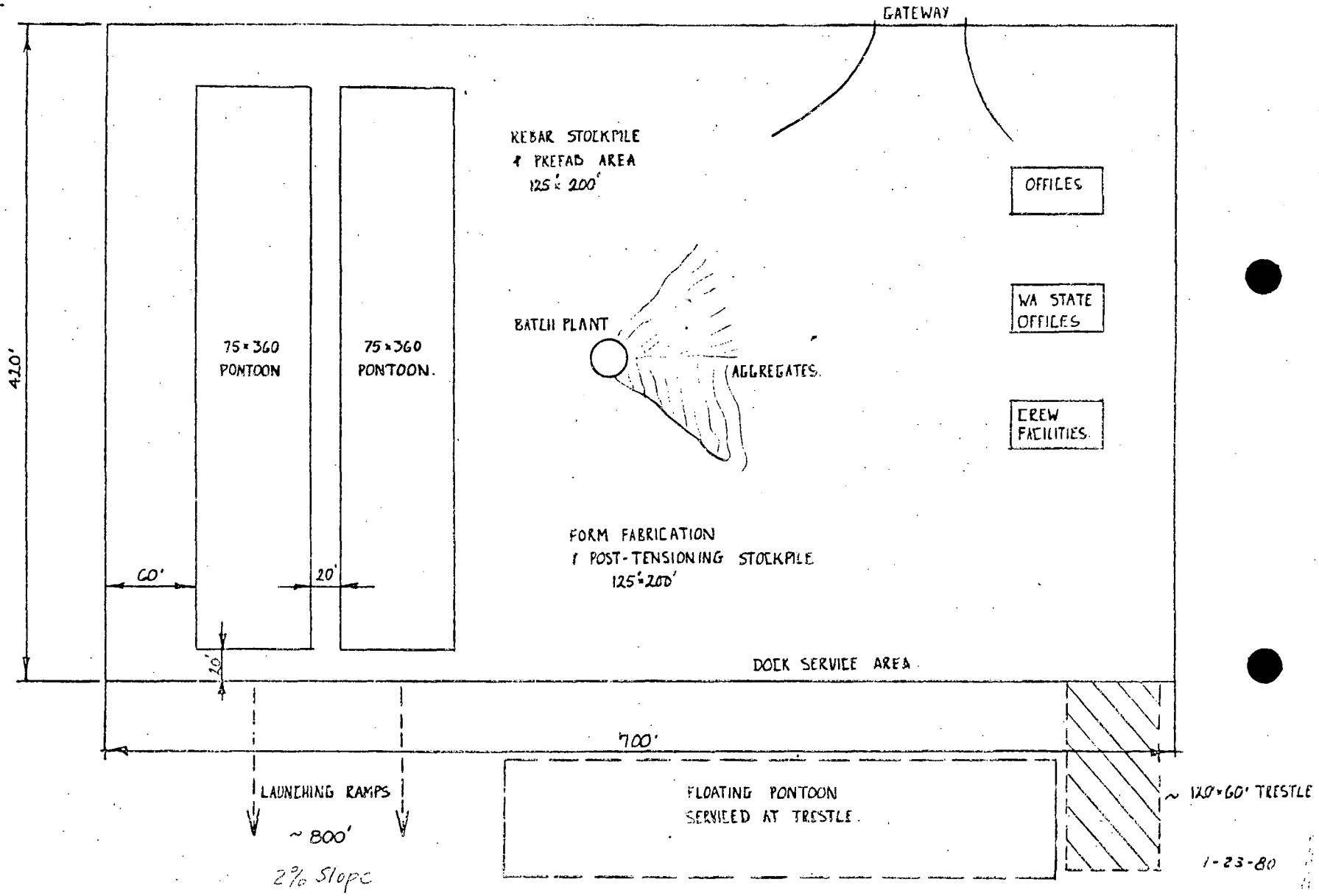
# TEST HOLE LOCATION PLAN

CAP SANTE WATERWAY IMPROVEMENTS
ANACORTES, WASHINGTON

| SCALE | DATE | DRAWING NO. |
|---|---|---|
| 1" = 200' | 10-8-87 | CD - 847 - 2 |

GENERAL LAYOUT - PONTOON CASTING YARD ~ 6¾ ACRES

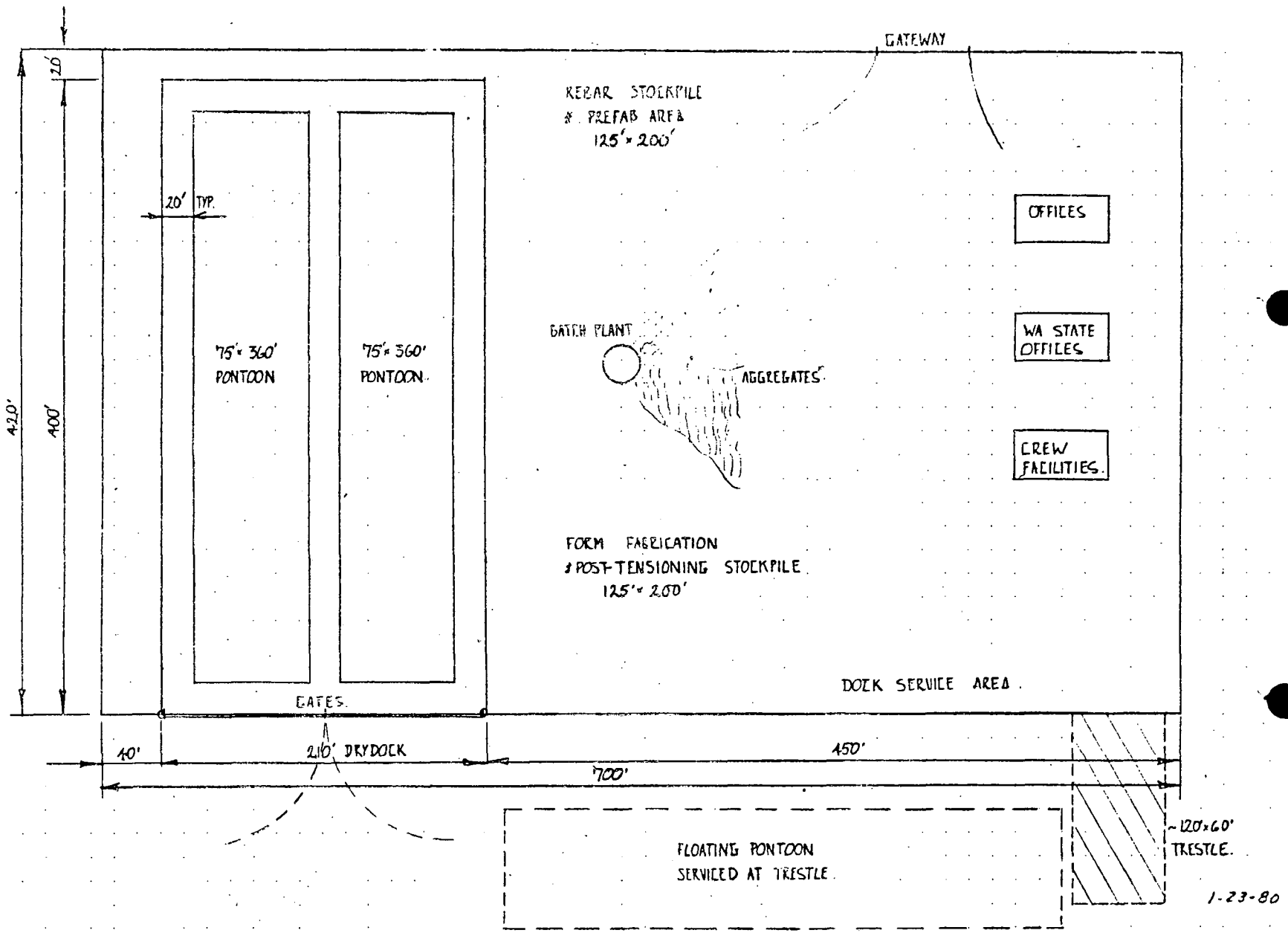
END LAUNCH




1-23-80

GENERAL LAYOUT - PONTOON CASTING YARD ~ 3-3/4 ACRES.

DRYDOCK LAUNCH.




1-23-80