SUBSURFACE EXPLORATION AND GEOTECHNICAL ENGINEERING

PROPOSED RETAIL STORE

Aberdeen, Washington

Prepared for

KKPFF Consulting Engineers

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CITY OF ABERDEEN

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August, 1992

RZA AGRA, Inc.
Engineering & Environmental Services

AGRA
Earth & Environmental Group

Subsurface Exploration and Geotechnical Engineering Report

Proposed Retail Store

Aberdeen, Washington

Prepared for

KPFF Consulting Engineers

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August 1992

W-8401

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SUBSURFACE EXPLORATION AND GEOTECHNICAL ENGINEERING REPORT
PROPOSED RETAIL STORE
ABERDEEN, WASHINGTON

1.0 SUMMARY

The proposed project construction is considered feasible with respect to subsurface conditions encountered at the project site. A brief summary of the project geotechnical considerations is presented below:

- Our subsurface explorations consisted of drilling ten hollow-stem auger borings, pushing five Dutch cone penetrometer probes, and excavating 8 test pits. At the time of our site evaluation, several parcels which comprise portions of the project site were not explored due to access restrictions.

- Based on the explorations accomplished for this study, the site is covered with various types of fill soils and organic debris which varies in thickness from approximately 3 to 12 feet. Generally consistent subsurface conditions were encountered beneath the fill and debris. The explorations encountered native, interbedded alluvial loose to medium dense sands and soft to medium stiff silt, clayey silts and organic silts overlying deeper deposits of medium dense to dense sands and medium stiff to stiff silts. Very soft to medium stiff organic silt were encountered directly beneath the fill and varied in thickness. These interbedded soils typically extended about 48 to 52 feet below the existing ground surface. These silt beds are considered relatively compressible. Between depths of approximately 49 to 57 feet, a medium dense sand layer was encountered in all of our deep explorations. Underlying the medium dense sand, we encountered interbedded medium dense sands and medium stiff to stiff silts which extended to the full depths explored.

- Preliminary design grades call for the placement of approximately 2 feet of fill over the building pad area in order to achieve the proposed floor subgrade elevation of 14.5 feet. This subgrade elevation would be coincident with the top of dike elevation along the south side of the site.

- In our opinion, the subsurface soils and organic debris encountered in our explorations are not suitable to support the proposed structure on shallow foundations without experiencing severe total and adverse differential settlement. For this reason, we recommend that the proposed building be pile-supported to improve long-term foundation performance. A

structurally supported floor slab is also recommended, in order to minimize differential settlement between the floor slab and the pile supported building.

- The native site soils are highly silty and therefore, moisture sensitive. The use of imported granular fill is recommended for structural fill placement below the proposed structure as well as beneath parking areas. The use of on-site materials for fill would be difficult or impossible due to their plasticity, organic content, and moisture content.

- Over the life of the pavement, we expect the site to settle several inches causing pavement surface degradation and increased maintenance.

- Groundwater was encountered at all of our boring locations at the time of our subsurface exploration program. Groundwater levels varied from 7 to 15 feet beneath the existing ground surface. We anticipate that most water levels would have risen to within 7 or 8 feet of the ground surface if the borings were left open long enough to allow water levels to come to equilibrium. Groundwater conditions should be expected to vary throughout the year due to seasonal variations, fluctuations of river level, and on-and off-site uses.

This summary is presented for introductory purposes only and should be used in conjunction with the full text of this report. The project description, site conditions and our detailed geotechnical design recommendations are presented in the text of this report. The field exploration procedures and detailed logs of the explorations are presented in this report as Appendix A. Laboratory procedures and test results are presented in Appendix B while Appendix C presents the calculations used for AASHTO pavement design and the Geotechnical Investigation Fact Sheet and Design Criteria.

2.0 PROJECT DESCRIPTION

The proposed project site is located in Aberdeen, Washington. It is bounded to the north by an active railroad line, to the south by the Chehalis River, to the west by seaport facilities, and to the east by the Wishkah Mall. The site can be accessed from the north by Newell and Chehalis Streets, from the west by Heron Street or from the east via the Wishkah Mall. The location of the site is presented on Figure 1, Vicinity Map.

Based on the information provided to us at this point, we understand that the proposed building would be a single story structure with concrete slab-on-grade floors, utilizing a combination of load bearing concrete

block walls and steel columns, as described in the owner's "Geotechnical Investigation Report Requirements" for type A store. Column loads are anticipated to range up to 100 kips, and wall loads would range between 4 and 6 kips per lineal foot. Floor slab loads may range from 100 to 250 psf. The approximate area of the proposed building would be on the order of 116,000 square feet. Parking areas, truck loading/unloading areas, access drives and landscaped areas would also be constructed in conjunction with this project.

The purpose of this evaluation was to establish general subsurface conditions at this site from which conclusions and recommendations for foundation design, pavement design, and general earthwork construction recommendations for the project could be formulated. The scope of work consisted of a field exploration program, geotechnical engineering analyses, laboratory testing, and preparation of this report. In the event that there are any changes in the nature, design, elevation, or location of the proposed structure, the conclusions and recommendations contained in this report should be reviewed by RZA-AGRA and modified, as necessary, to reflect those changes. This report has been prepared in accordance with generally accepted geotechnical engineering practice for the exclusive use of KPFF Consulting Engineers and their agents for specific application to this project.

3.1 Surface Conditions

The proposed project site presently has several buildings on the northern half, all of various size and differing uses. The southern half is currently vacant. At least 14 structures occupied the site at the time of our exploration. In general, the site was fairly flat with a maximum topographic relief on the order of 2 feet. Ground surface elevations range from approximately 11.3 to 14 feet. Previously placed fills were observed across the entire site. The exposed, shallow surficial fills typically consisted of pit-run sand and gravel or crushed gravel. The majority of the site appears to have been developed in the past.

3.2 Subsurface Conditions

Based on materials encountered during our field exploration program it appears that the near-surface conditions are variable across the site particularly in the upper 3 to 12 feet where man-placed fill soils and wood debris was encountered. At our exploration locations, 3 to 12 feet of man-placed fill soils and organic debris were observed overlying the native soils. Typically, fill material varied from sawdust, lumber, logs, concrete, ash and other debris to sandy silt, silt, gravelly sand, sandy gravel, or silty sand. In most test pits, logs, lumber, and debris such as brick, metal, and concrete were observed. In some cases, test pits were terminated due to impenetrable thicknesses of buried logs. Such conditions were encountered in test pits TP-1 and TP-7. Underlying the fill soils and debris, interbedded alluvial loose to dense sands and soft to medium stiff silts, clayey silts and organic silts were encountered as shallow as 3½ feet and extended to the

bottom of our explorations 80 feet below the existing ground surface. Man-placed fills and organic waste should also be expected in other locations on this site, not covered by our exploration program.

The native materials encountered on the subject site are interpreted to be of alluvial origin. Alluvial materials are characteristically variable in thickness and areal extent, and also vary in consistency and density as a result of the dynamic depositional environment in which they accumulated. Our subsurface exploration encountered variable thickness interbeds ranging from loose to dense sands and silty sands to soft to medium stiff silts, clayey silts and organic silts. Borings and probes which were advanced throughout the upper compressible deposits encountered very soft to medium stiff organic silts to about 48 to 52 feet deep and medium dense to dense sands from about 49 to 57 feet beneath the ground surface. Interbedded medium stiff to stiff silts and medium dense sands were encountered below 57 feet to the full depths explored. Some of the silty layers encountered at depth contained organic matter and are probably of organic origin. These materials are considered highly compressible and exhibit long-term secondary compression characteristics. The boring, test pit, and probe logs, the Site and Exploration Plan, Figure 1, are included with this report for referencing depths and thicknesses of soft compressible soils at the exploration locations.

Man-placed fills were encountered in all of our explorations. These fill materials contained various types of soil, lumber, logs, saw dust, ash, concrete, and other debris. Our borings encountered up to 12 feet of man-made fill containing sawdust, logs, lumber, ash, concrete and other debris. Man-placed fills should also be expected in other locations on this site, not covered by our exploration program.

According to the U.S. Department of Agriculture, Soil Conservation Service, soil survey map of Grays Harbor County Areas, the native site soils are described as Udorthents. These soils occur in diked tidelands according the SCS. The soil is classified within soil hydrologic group C which indicates the soil has a slow infiltration rate when wet and a slow rate of water transmission.

3.3 Groundwater

Groundwater was encountered in borings B-1 through B-8 at approximate elevations ranging from 0 to 7 feet or about 7 to 15 feet beneath the ground surface. We anticipate that groundwater elevations would have risen to about elevation 7 feet if borings had been left open long enough for water levels to come to equilibrium. Moderate groundwater seepage was also observed in test pit TP-3 about 4 feet beneath the ground surface. Groundwater levels should be expected to fluctuate to higher elevations on an intermittent

basis, and higher during high precipitation periods or years. Fluctuations of groundwater are likely to occur as a result of seasonal variations in precipitation, river elevations, changes in site utilization and other factors.

4.0 CONCLUSIONS AND RECOMMENDATIONS

In our opinion, development as proposed is feasible from a geotechnical standpoint, utilizing pile foundations and structural floor support. In their present condition, the site soils are unsuitable for support of conventional footings and slab-on-grade floor system without excessive post-construction settlement. Slab-on-grade floor support would only be possible if the underlying wood debris is removed and replaced with structural fill and then preloaded with an additional surcharge. Due to the amount of material which would have to be removed and replaced to construct a slab-on-grade floor, as well as constructing a preload surcharge, we recommend consideration of pile foundations with a structural floor slab. Even if the slab on grade alternative is considered, differential settlement between the building foundations and floor slab would be in excess of ½ inch which exceeds the specified maximum in the Geotechnical Investigation Report Requirements.

4.1 Site Preparation

We understand that about 2 feet of fill will be placed above the existing fill soils to elevate the floor subgrade to elevation 14.5 feet. The existing weeds growing in the building area should be bladed flat with a dozer prior to placing the fill. In pavement areas, all surficial vegetation should be completely removed.

At least 14 buildings will be demolished to accomplish construction of the proposed retail facility. We anticipate that all or some of these structures are founded on timber piles. Once the buildings have been demolished, we recommend that all of the timber piles be pulled and the resulting hole backfilled with pea gravel. If pulling the piles is not considered cost-effective, the pile tops should be cut off at least 3 feet below finished grade. It is customary to cut the pile butts at an angle rather than flat across the top to reduce the effects of settlement on pavement structures. There is a risk of pavement cracking over cutoff piles due to long-term settlement.

We also anticipate that several utilities will require abandonment or relocation. In either case, the utilities should be removed, relocated, or abandoned in-place in accordance with applicable State and local laws. All trench backfilling should be performed in accordance with the recommendations presented in the Structural Fill section of this report.

Site preparation and initial construction activities should be planned to minimize disturbance to the existing ground surface. We recommend heavy construction equipment be kept off existing soft soils where they are exposed or close to the surface. Heavy construction traffic should be limited to areas where the structural fill is in place.

4.2 Structural Fill

All structural fill placed on the project site as well as that used for trench backfill should be placed in accordance with the recommendations herein for structural fill. In our opinion, structural fill placed to raise site grades to finish-floor subgrade elevations should consist of imported, granular sand and gravel. Structural fill should be placed in loose lifts not exceeding 8 inches in thickness. Individual lifts should be compacted such that a density of at least 90 percent of the modified Proctor maximum density (ASTM:D 1557) is achieved. We recommend that a representative from RZA AGRA, Inc. be present during the placement of structural fill to observe the work and perform a representative number of in-place density tests. In this way, the adequacy of earthwork may be evaluated as grading and utility construction progresses.

The suitability of soils used for structural fill depends primarily on the gradation and moisture content of the soils when it is placed. As the fines content (that portion passing the U.S. No. 200 sieve) increases, soil becomes increasingly sensitive to small changes in moisture content and adequate compaction becomes more difficult or impossible to achieve. Silty soils containing more than about 5 percent fines by weight cannot be consistently compacted to a firm, non-yielding condition when the moisture content is more than about 2 percent above optimum. Import granular fill is recommended below all foundations, floor slabs and for use in raising site grades and preload fill. The use of clean, granular soils would be necessary for fill use to complete wet weather site work in either foundation or pavement areas. We also recommend that pavements be underlain by a minimum 18 inch thickness of imported, clean granular fill. Clean granular fill soils should consist of well-graded sand with gravel (as defined by ASTM: D 2487) containing less than 5 percent fines by weight based on the fraction passing the U.S. No. 4 sieve.

We recommend that clean, granular fill soils as described above be specified for all fill likely to be exposed to wet conditions. The specifications should also state that clean, granular fill soils must be capable of being compacted to a firm, non-yielding condition at the recommended compaction level during wet weather or wet site conditions.

4.3 Pile Foundations

In our opinion, it is possible to support the proposed structure on driven-timber or augercast piles. The selection of a pile foundation type will depend on time constraints and economics. For lightly loaded structures, low capacity driven timber piles are most often cost-effective. We have provided driven timber and augercast pile recommendations in the following paragraphs for the owners use in evaluating the pile foundation options.

Piles would gain their vertical compressive capacity mainly from side friction between the pile and the native soils with some minor contribution from tip end-bearing. All piles should be installed to a tip elevation of -36 feet to -40 feet, in order to gain end-bearing support in medium dense sands. We recommend a maximum allowable vertical compressive capacity for an 8-inch tip diameter, timber pile of 20 tons. For a 10-inch tip diameter timber pile, we recommend 25 tons allowable capacity. Alternatively, 16-inch diameter augercast (AC) concrete piles would provide a maximum allowable compressive capacity of 40 tons. These pile capacities include a factor of safety of at least 2.5. If adequately installed, the pile foundations are anticipated to settle less than 1 inch, with differential settlements on the order of ½ inch between pile-supported foundation elements. The above allowable compressive capacities are based on minimum pile spacings of 3D, where D is the average diameter of the piles. If pile groups of more than 9 piles per cap are planned, we should be consulted for reductions of group capacity.

For fixed-head timber piles, we recommend designing for an ultimate lateral capacity of 5 tons per pile for a limiting deflection of approximately 1 inch. The depth to zero deflection has been calculated to be approximately 14 feet beneath the pile cap. This lateral capacity assumes that structural stiffness of the pile cap/floor network prevents rotation of the pile top. We recommend the design criteria be utilized with the actual anticipated loads to determine the deflections and maximum moments. These calculated values should be analyzed with respect to the tolerable deflections of the structure and the strength of the pile material (with a suitable safety factor) to resist the moments and shear.

For fixed augercast piles, we recommend designing for an ultimate lateral capacity of 10 tons per pile for a limiting deflection of approximately 1 inch. The depth to zero deflection has been calculated to be approximately 20 feet beneath the pile cap.

It should be noted that the above formulations do not contain a factor of safety. Further, a number of circumstances can alter the actual deflection values obtained in the field as compared to the calculated values as previously mentioned. Pile group action can have an influence. At the minimum pile spacing,

actual pile deflections in the group may be almost twice that calculated for an individual pile. The effects of group action tend to disappear when piles are spaced 8 or more diameters, center to center, in the direction of the load and at three or more diameters normal to the direction of the load. Piles subjected to repetitive, cyclic loading may result in deflection magnitudes up to twice that of a pile subjected to a static load. For these piles, we would recommend values of lateral pile capacity equal to half of those values presented be used to establish lateral capacity design.

Regardless of the pile type selected, based on our explorations, significant obstructions will likely be encountered during pile installation. Nested buried logs were encountered in some of the test pits, which in two instances led to abandonment of deeper test pit excavation. Additionally, our explorations encountered debris in the existing fill. Where obstructions to pile driving or drilling are encountered close to the surface, typically, a backhoe or trackhoe can excavate and remove the obstruction. Alternatively, spudding or predrilling may be required to advance through debris or buried logs.

We recommend that fills placed to raise building pad grades be completed at least 90 days prior to pile installation. Assuming that approximately 2 feet of fill is placed across the entire site, we estimate that settlement on the order of 2 inches will occur during this 90-day period. As the upper compressible soils and fill materials consolidate and move downward with respect to the relatively immobile pile, the soils surrounding the pile impart shear stresses acting downward on the surface of the pile. The result is a real additional load, in addition to structural load, known as "negative skin friction", or "downdrag". Downdrag loads are anticipated to be minor on the site if the 90-day waiting period is followed. The allowable capacities of the piles presented previously in this report have been reduced to account for the effects of "downdrag".

Uplift pile capacity develops as a result of side friction between the pile and adjacent soils. Uplift load resistance can be provided by side friction from that portion of the pile embedded in the a medium dense to dense native sands and soft to stiff cohesive site soils. We recommend utilizing ultimate uplift capacities of 8 and 10 tons for the recommended 8 and 10 inch diameter timber piles, respectively and 20 tons for 16-inch diameter augercast piles installed to design depth. Uplift capacity for timber piles will likely be limited by structural capacity of the piles and pile cap connection. We recommend using a safety factor of 1.5 or more depending on loading conditions. These values neglect the dead weight of the pile. The structural adequacy of the pile section should be verified to resist the uplift loads. AC piles, in particular, must be reinforced to fully mobilize their uplift capacity.

All timber piles should be pressure-treated conforming to standard specifications outlined in ASTM:D 25. In addition, we recommend a minimum tip diameter of 8 inches for all timber piles. We recommend that timber piles be driven with a diesel, air or steam hammer, having a rated energy of not less than 15,000 foot pounds and not more than 26,000 foot pounds. Due to the low density and consistency of the native soils immediately below existing ground surface, use of a diesel hammer may not be appropriate as continued effective operation of the hammer may be difficult. The actual pile capacity should be verified in the field based on an appropriate dynamic pile-driving formula. Pile "set-up" should be checked during initial driving after a minimum 24-hour waiting period.

Augercast piles (AC piles) may be utilized to support the proposed structure. AC piles would minimize the potential for high vibration levels in the soils and the resultant risk of damage to adjacent utilities and buildings. Due to the potential for increased downdrag loads on AC piles, we recommend that their installation be continuously monitored and that capacities be confirmed by load testing.

AC piles are formed by drilling to a predetermined depth with a continuous flight, hollow-stem auger. Cement grout is then pumped down the auger under pressure as the auger is withdrawn. Reinforcing steel can then be installed in the unset concrete column to provide lateral and tension capabilities. Reinforcing steel should be structurally designed for lateral and uplift requirements. To provide pile continuity, at least one reinforcing bar should be installed the full length of the pile. The contractor should be required to stagger the pile drilling and grouting operations, such that all piles within 10 feet of a pile being drilled have set for at least 24 hours. We recommend a minimum AC pile spacing of 4 feet on center.

Since AC pile continuity and capacity are highly dependent upon the specific installation procedures, we recommend that installation of all AC piles be observed by a representative of our firm. Our representative would observe the contractor's operations, collect and interpret the installation data, and determine possible modifications to pile penetration depths or installation, if necessary. Also, a qualified representative understands the implications of varying from the established design criteria and would respond accordingly to minimize delays. In order to provide an evaluation of augercast pile installations, we recommend that the contractor be required to provide a pressure gauge in the grout line between the pump and the auger, and a means of determining the quantity of grout used per pile, such as a calibrated stroke counter on the grout pump. It may be necessary to periodically confirm the nature of the bearing materials during pile installation by withdrawing the auger at selected pile locations to examine the material retained on the auger cutting teeth.

Augercast pile grouting is especially critical on this site due to the soft soils present at depth. We recommend that the piles be grouted uniformly to a volume of at least 130 percent of the nominal 16-inch diameter volume, and a head of at least 20 feet of grout be maintained at all times. With the soft soils on the site, grout quantities of 150 to 200 percent of the nominal drilled diameter may be encountered. Any interruptions in the grout pressure would require redrilling at least 1-foot below the auger tip and regrouting the affected interval. Excessive grout take at any one zone, indicative of formation of a grout bulb in soft soils or voids in the fill, should be minimized. In extreme cases, this can require the installation of a casing or other modifications to the typical augercast procedures. This can be assessed further during a test pile program.

We recommend an augercast pile/driven pile load test program prior to production pile installation. Test pile installation data as well as load test data would be reviewed to develop the most cost-effective combination of pile length and capacity. We recommend consideration of a combination of compressive, lateral, and tensile (uplift) load tests at a minimum of two locations on the site. Test piles should be installed for the pile type selected for the project. Pile testing should be completed prior to ordering production piles, so that adjustments to pile length can be made, if necessary.

Compressive, lateral and tensile load testing should be completed in accordance with procedures outlined in ASTM specifications D-1143, D-3966, and D-3689, respectively. We recommend that RZA AGRA be retained to observe and monitor test pile installation, supervise and perform the load tests, and analyze the data collected. A report would be issued following completion of the testing describing and documenting test pile installation methods and results, load test procedures and results, and other information as appropriate.

Table 1 presents our recommended test pile program. Test and reaction piles should be installed in the same manner as the production piles. Provided the test and reaction piles are installed to the recommended depths, uplift capacity tests could be performed on the reaction piles.

Table 1

Recommend Test Pile Program

Compressive:	2 to evaluate bearing capacity at 40 and 53 feet depths (possible reductions in design depths)
Lateral:	1 at any test location if required by structural engineer
Uplift:	1 at 40 feet depth if required by structural engineer

4.4 Structural Floors

We estimate the long-term consolidation of the soft organic silt and clayey silt layers beneath the fill, long-term settlement could be on the order of 3 to 4 inches or more. A slab-on-grade floor could experience significant cracking and warping due to both total and differential settlement resulting from this anticipated consolidation. We therefore recommend utilizing a structural floor slab with the building proposed at the subject site. The floor subgrade area should be prepared as described in the Site Preparation section of this report, and structural fill should be placed and compacted as described in the Structural Fill section.

Disturbance of soils beneath the floor area and grade beams should be minimized, to the extent possible. Typical pile installation procedures and construction methods often create widespread disturbance of soft subgrades, and some remedial earthwork should be anticipated prior to casting the floors and grade beams. This could include removal and replacement of soft or disturbed, wet soils with structural fill, placement of a working surface of gravel, quarry spalls, or possibly placement of a lean concrete mud slab. During dry site conditions, we anticipate that slabs can be cast atop a gravel layer placed after removal and replacement of excessively soft soils.

Provided the floor subgrade soils are not disturbed after construction of the grade beams and structural floor, some passive pressure resistance will develop against the grade beams. We recommend using a maximum allowable passive pressure of 150 pcf against the grade beams neglecting the upper 1 foot of embedment.

4.5 Capillary Break

We recommend that floor slabs be founded on a minimum of 6 inches of clean granular structural fill to serve as a capillary break. Capillary break material should have less than 3 percent fines based on that fraction passing the U.S. No. 4 Sieve with at least 30 percent retained on the U.S. No. 4 Sieve. An impervious moisture barrier should also be placed between the floor slab and the capillary break.

Capillary break material should not be placed on disturbed soils which may allow the material to settle and mix with the disturbed soil. This would reduce the effectiveness of the capillary break and possibly create moisture problems in the floor slabs.

4.6 Pavement Design

It must be recognized that pavement design is a compromise between high initial cost and little maintenance on side and low initial cost coupled with a need for periodic repairs. As a result, the owner will need to take

part in the development of an appropriate pavement section. Critical features which govern the durability of the surface include the stability of the subgrade, the presence or absence of moisture, free water and organics, the fines content of the subgrade soils, the traffic volume, the frequency of use by heavy vehicles and in this case, the possibility that piling from existing or previous structures may be beneath the pavement section.

The on-site native soils are highly silty and exhibit low subgrade support characteristics when saturated even if initially compacted to a high degree. Based on in-situ cone penetration and SPT tests of the near-surface soils, and on laboratory tests performed on a representative soil sample, we have assigned a design CBR value of 1 percent for the near-surface native soils in an undisturbed state. Imported granular fill is recommended for pavement subgrade support. We do not recommend utilizing native soils as pavement subgrade.

Based on the information provided us, standard duty pavement design should be based on a 20-year life of approximately 11,279 equivalent 18 kip single-axle loads. For heavy-duty pavement sections, a 20-year traffic volume of 30,697 equivalent 18 kip single-axle loads has been specified. Using the design criteria contained in The Guide for Design of Pavement Structures by AASHTO (1986), the structural number (SN) used for design of heavy duty asphalt pavement (20-year design life) would be 3.2 and 2.7 for light duty pavement design.

Although a variety of pavement sections could be utilized, we recommend a minimum asphalt concrete thicknesses of 3 inches underlain by a minimum of 6 inches of crushed aggregate base over 13½ inches of properly compacted clean, granular subbase fill for heavy duty pavement sections. For light duty pavement sections, we recommend 3 inches of AC pavement over 6 inches of compacted crushed base course over 9 inches of compacted sand and gravel subbase. The 3 inches of asphalt in the light-duty section will provide additional durability even after the pavement has experienced some settlement compared with a more typical 2-inch thick section. We understand that imported subbase soils will actually be thicker than we recommended. A 2-inch thick asphalt section would probably be structurally adequate until settlement occurs. The thinner 2-inch asphalt section will more likely require additional maintenance than that of a 3-inch thick asphalt section. The actual pavement design selected should consider the subgrade material provided for support and may best be made after design site grades are finalized.

The pavement recommendations presented above are based on the characteristics of the subgrade soils and should perform as expected with respect to pavement support. However, over the life of the pavement

we expect the site to settle several inches. Settlement of this magnitude typically causes "birdbath" type depressions in the surface, altering surface runoff patterns and ultimately causing cracking. As the site settles, abandoned piling will begin to cause distress in the pavement which will manifest itself by cracking and warping. This will increase the long-term maintenance costs for the pavement.

4.7 Seismic Criteria

Seismic design of the structure requires the selection of a numerical coefficient of soils structure iteration, designated "S" in the 1991 edition of the Uniform Building Code, Section 2333 (b,c,d). Based on the soil conditions encountered at the borings at the site and public published geologic mapping, we recommend the use of an S factor equalling 2.0, as specified for Soil Profile Type S_4 in the 1991 Uniform Building Code. Soil Profile Type S_4 applies where 40 feet or more of soft clay is present. Based on Figure 23-2 of the 1991, UBC, it appears the site is located within Seismic Zone 2B.

Detailed liquefaction analyses were not included in the scope of work for this project, and would not typically be undertaken for a project of this magnitude in this region. Our subsurface investigation, however, indicated the presence of loose to medium dense, saturated, silty fine sands and silts below the water table, which are commonly encountered in alluvial valleys in the Puget Sound region. There is a potential for liquefaction of loose sand deposited below the water table during a seismic event of sufficient magnitude and duration. Due to the predominately silty nature of loose deposits on this site (with relatively high fines contents), the liquefaction potential appears to be limited, and is considered lower than other alluvial sites across the state, including the Ports of Seattle and Tacoma, and the Auburn-Kent area. Liquefaction risk decreases with increased density, and the capacities of the pile foundations recommended for support of the structure are not expected to be adversely affected by liquefaction.

4.8 Drainage and Utility Considerations

Prior to construction, any site surface water should be routed away from the construction areas as much as possible. We recommend that buildings be provided with a perimeter footing drain system to collect available water. Footing drains should consist of at least a 4-inch diameter perforated pipe with an envelop of pea gravel or washed rock. The drainpipe should lead away from the buildings via gravity to a storm sewer or other suitable discharge. Site grades should be planned to slope away from the buildings. Roof and surface runoff should not discharge into the footing drain system. Instead, a separate tight-line drain network should be installed or splash blocks may be used. It should be noted that perched water may develop on portions of the site that could adversely effect pavement life. Once grades are set, we should review the plans to see if subsurface drains would be appropriate in these areas.

Settlement-sensitive utilities beneath the building could be suspended from the structures. Soil loads on the pipes could be estimated by RZA AGRA when depths and diameters are available. Soil loads can be significant, especially with deep pipe burial. If it is necessary to reduce soil loads, lightweight backfill or possibly no backfill could be considered as alternatives to conventional burial. In our opinion, conventional burial would consist of overexcavating a minimum of 1 foot below bottom of trench elevations and constructing a pipe foundation with imported granular soils. The actual amounts of overexcavation will be a function of the pipe burial depth, the type of exposed subgrade soils, and the type of pipe to be installed. Based on the type of soils encountered in our explorations, we estimate that pipe foundation fill soils could be as thick as 2 feet.

Based on the groundwater conditions at the time of our explorations, utility trench excavations which extend below about 4 feet from existing grade may encounter seepage. Deep excavations may require dewatering. Dewatering should be the responsibility of the contractor on this project.

Even with the trench bedding recommendations presented above, some settlement of utilities should be expected to occur. Therefore, we recommend that all utilities which drain by gravity be constructed at a steeper slope to minimize the potential for loss of flow gradients within the pipes.

5.0 CLOSURE

The conclusions and recommendations presented in this report are based on the explorations accomplished for this study. The number, locations and the depths of the explorations were completed within the site constraints so as to yield the information to formulate our recommendations. The integrity of the proposed project depends on proper site preparation, foundation construction, and earthwork. We would be available to provide geotechnical engineering services during earthwork and foundation phases of the project. We would also be available to provide inspection services in accordance with the owner's standard specification requirements. If variations are observed at that time, we would be available to provide additional geotechnical recommendations so as to minimize delays as the project develops.

It has been a pleasure to provide you with this information. If you have any questions or require further information, please do not hesitate to call at your convenience.

Respectfully submitted,

RZA AGRA, Inc.

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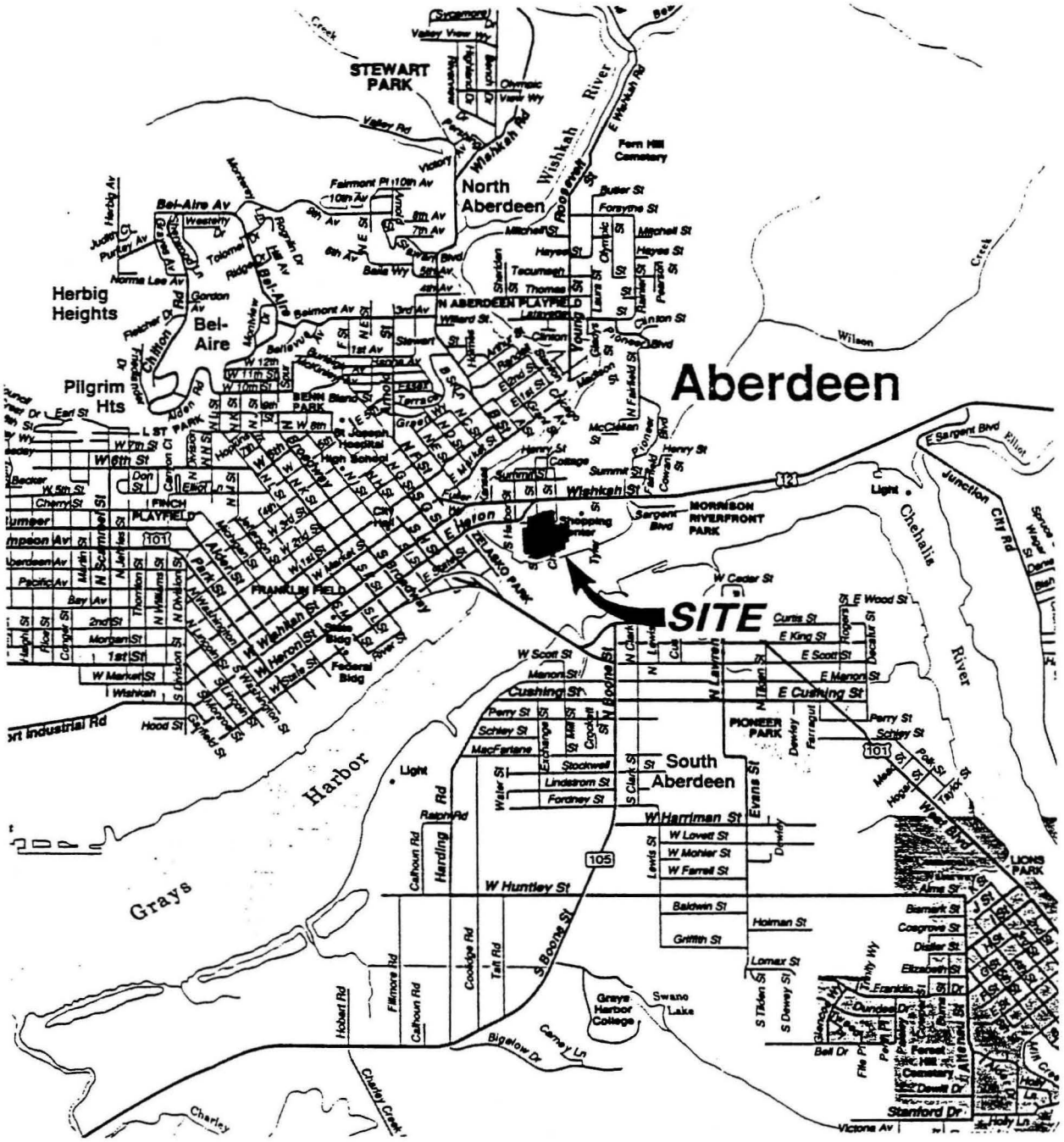
Enclosures: Figure 1 - Location Map
Figure 2 - Site and Exploration Plan
Appendix A - Subsurface Exploration Procedures and Logs
Appendix B - Laboratory Test Procedures and Results
Appendix C - AASHTO Pavement Design
Appendix D - Geotechnical Investigation Fact Sheet and Foundation Design Criteria

TAJ/JEZ/LAD



RZA-AGRA
ENGINEERING & ENVIRONMENTAL SERVICES

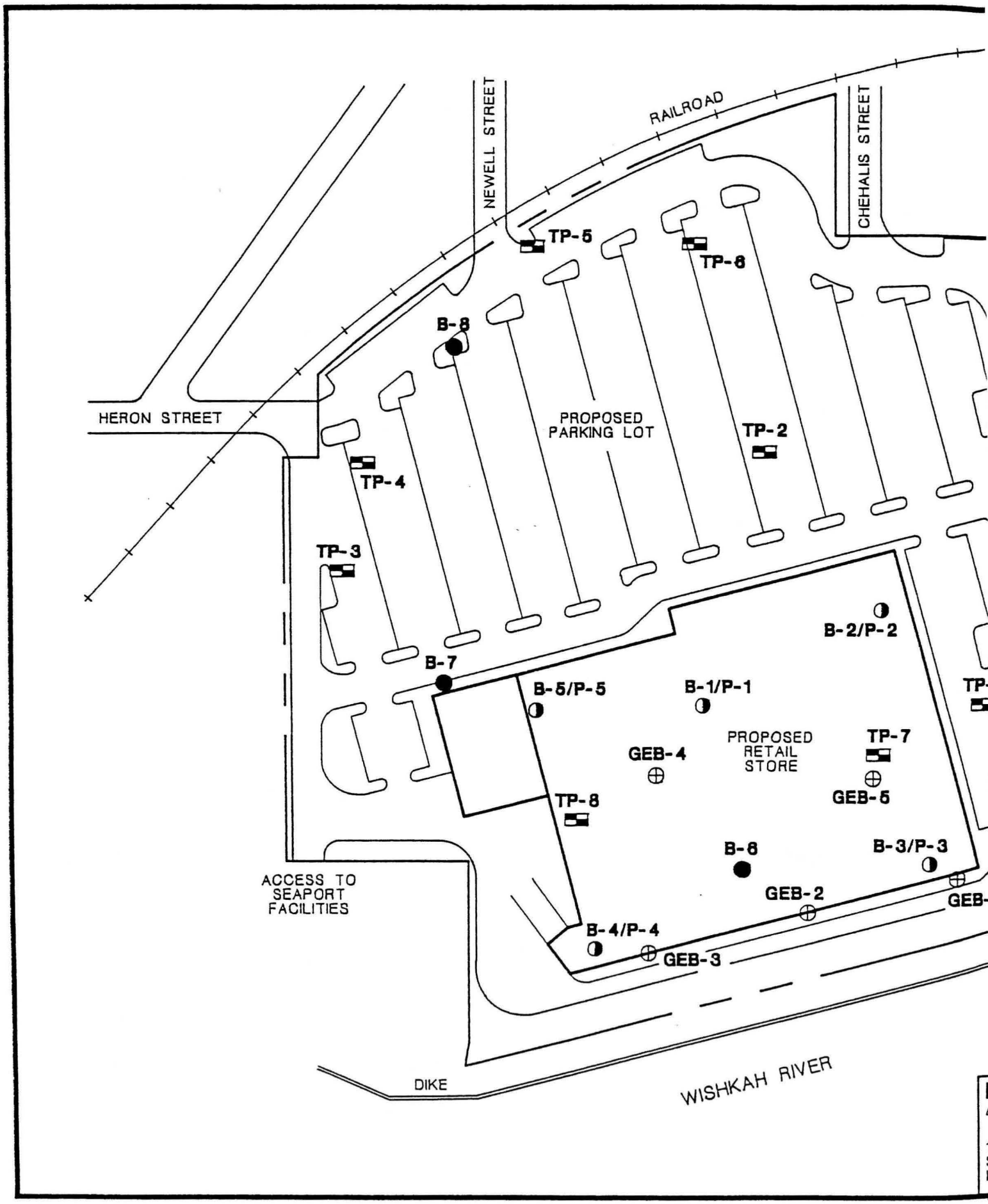
11335 N.E. 122nd Way
Suite 100
Kirkland, Washington
98034-6918

W.O.	W-8401
DESIGN	TDW
DRAWN	MJF
DATE	AUG 1992
SCALE	N.T.S.

PROPOSED RETAIL STORE
ABERDEEN, WASHINGTON

LOCATION MAP

FIGURE 1


RAILROAD
NEWELL STREET
CHEHALIS STREET
TP-5
TP-6
B-8
HERON STREET
PROPOSED
PARKING LOT
TP-2
TP-4
TP-3
B-2/P-2
B-7
B-5/P-5
B-1/P-1
PROPOSED
RETAIL
STORE
TP-7
GEB-4
GEB-5
TP-8
B-6
B-3/P-3
ACCESS TO
SEAPORT
FACILITIES
GEB-2
B-4/P-4
GEB-3
DIKE
WISHKAH RIVER

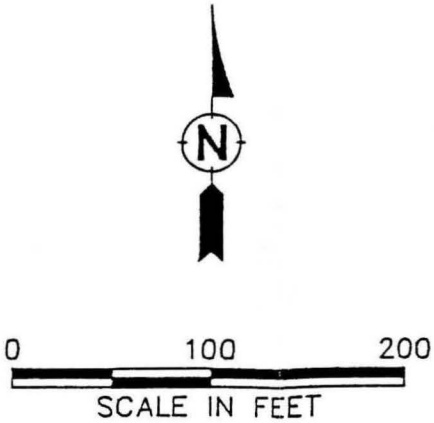
LEGEND

B-8 — BORING NUMBER AND LOCATION

B-5/P-5 — BORING AND DUTCH CONE PENETROMETER NUMBER AND LOCATION

TP-8 — TEST PIT NUMBER AND LOCATION

GEB-5 — BORING NUMBER AND APPROXIMATE LOCATION (GEOENGINEERS)



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Kirkland, Washington 98034-6918

W.O.	W-8401
DESIGN	TDW
DRAWN	MJF
DATE	AUG 1992
SCALE	1"=100'

PROPOSED RETAIL STORE
ABERDEEN, WASHINGTON

SITE AND EXPLORATION PLAN

FIGURE 2

DRAWING NO. \8A\W-8401\SITE.DWG

APPENDIX A

W-8401

SUBSURFACE EXPLORATION

The field exploration program conducted for this study consisted of advancing a series of hollow-stem auger borings, Dutch Cone penetrometer probes, and backhoe-excavated test pits. The approximate exploration locations are illustrated on the Site and Exploration Plan, Figure 2. The locations were obtained in the field by taping from existing site features. The locations should be considered accurate to the degree implied by the method used.

Hollow Stem Auger Borings

The borings were drilled between 29 and 30 July 1992 by a local exploration drilling company under subcontract to our firm. The borings consisted of advancing a 4-inch inside diameter, hollow-stem auger with a truck-mounted drill rig. During the drilling process, samples were obtained at generally 2 ½ or 5 foot depth intervals. The borings were continuously observed and logged by an engineering geologist from our firm.

Undisturbed samples were obtained by pushing a 3-inch outside diameter, seamless steel Shelby tube into the soil by the hydraulic system on the drill rig in accordance with ASTM:D 1587. Since the thin wall tube is pushed rather than driven, the sample obtained is considered relatively undisturbed. The samples were classified in the field by examining each end prior to sealing with plastic caps. The samples were then transported to our laboratory where they were extruded for further classification and laboratory testing.

Disturbed samples were obtained by using the Standard Test Procedure as described in ASTM:D 1586. This test and sampling method consists of driving a standard 2-inch outside diameter split barrel sampler a distance of 18 inches into the soil with a 140 pound hammer free falling a distance of 30 inches. The number of blows for each 6 inch interval is recorded. The number of blows required to drive the sampler the final 12 inches is considered the Standard Penetration Resistance ("N") or blow count. The blow count is presented graphically on the boring logs in this appendix. If a total of 50 blows is recorded within one 6 inch interval, the blow count is recorded as 50 blows for the number of inches of penetration. The resistance, or "N" value, provides a measure of the relative density of granular soils or the relative consistency of cohesive soils.

Static Cone Penetrometer Probes

Seven static cone penetrometer probes or Dutch cone tests were performed for this project between 29 and 31 July 1992 by a local exploration company under subcontract to our firm. The equipment used for this

test consists of a cone and friction sleeve which are advanced hydraulically by rods reacting against a drill truck. The static cone penetration test is performed as follows: 1) the cone is pushed down by an inner rod and the point resistance is recorded; 2) the cone and the sleeve and are then pushed and their combined resistance is measured; 3) the cone resistance is subtracted from the total resistance to provide the frictional resistance. A direct correlation between point resistance and the bearing capacity of the soils is obtained. The relative density or consistency of the soil probed is empirically related to the cone resistance. Comparing the cone bearing capacity and the friction ratio (sleeve friction/cone bearing) provides and interpretive soil classification based on the Dutch Cone soil classification chart prepared by J.H. Schmertman, 1969. The descriptive soil interpretations presented on the static cone penetrometer probe logs have been developed by using this classification chart as a guideline. Modifications to the classifications were developed according to correlations of soil types disclosed in the adjacent borings performed on the site and careful interpretation of the probe results. The detailed interpretive logs of the static cone penetrometer probes accomplished for this study are presented subsequently.

TEST PITS

The test pit exploration phase of the program consisted of eight test pit explorations (TP-1 through TP-8), excavated by a rubber tired backhoe provided by Bill Sell Trucking and Excavating of Aberdeen, Washington under contract to RZA-AGRA on 31 July 1992. These test pit excavations permitted a detailed evaluation of the character of the shallow subsurface soils underlying the area where the proposed building and parking lot are to be located. Also, they are more representative indicators of true near-surface character than soil exploration drilling since they allow a continuous visual observation of all soil layers encountered.

Each test pit was continuously logged and observed by an experienced engineering geologist from our firm. In-situ strength and quality attributes of materials encountered were estimated by our field observer based on experience with similar soils and the difficulty incurred during excavation. Relative density estimates are presented parenthetically on the test pit logs since the densities are not based on the actual SPT blowcounts. Representative samples of the soils in the test pits were retrieved, classified in the field and transported to our laboratory for a detailed evaluation and classification. The test pit logs are presented in this appendix and the soil descriptions are based on the inspection of the samples secured, field logs and laboratory test.

PROJECT: *Proposed Retail Store Aberdeen, WA* W.O. *W-8401* BORING NO. *B-1*

DEPTH (feet)	SOIL DESCRIPTION Approximate ground surface elevation: *13.6 feet*	SAMPLE TYPE	SAMPLE NUMBER	GROUND WATER	STANDARD PENETRATION RESISTANCE (Blows per foot)	TESTING
0	*Dense, dry, brown, sandy GRAVEL (Fill)*					
	Medium stiff, moist, dark gray, SILT (Fill)		S-1			
5	*difficult drilling / gravel or wood*					
	Becomes sandy SILT			ATD		
	Becomes, saturated, dark gray to black, clayey SILT with some woodwaste and organic matter		S-2			(107)
10	*Boring terminated at approximately 9 feet*					
15						
20						
25						
30						

LEGEND

2-inch OD split-spoon sample

ATD Groundwater level at time of drilling

MOISTURE CONTENT

Plastic limit — Natural — Liquid limit

RZA AGRA, Inc
Engineering & Environmental Services
11335 NE 122nd Way, Suite 100
Kirkland, Washington 98034-6918

Drilling started: *29 July 1992* Drilling completed: *29 July 1992* Logged by: *TDW*

PROJECT: *Proposed Retail Store*
Aberdeen, WA

W.O. *W-8401* BORING NO. *B-2*

DEPTH (feet)	SOIL DESCRIPTION Approximate ground surface elevation: *13.3 feet*	SAMPLE TYPE	SAMPLE NUMBER	GROUND WATER	STANDARD PENETRATION RESISTANCE (Blows per foot)	TESTING
0	*Medium dense, damp, brown, GRAVEL with some sand (Fill)*					
	Medium stiff, wet, brown, SAWDUST and WOOD CHIPS (Fill)		*S-1*		(293)	
5						
	Soft, wet, dark gray, SILT with interbeds of sawdust		*S-2*		(189)	*PL=53* *LL=104* *PI=51*
10	*Boring terminated at approximately 9 feet*					
15						
20						
25						
30						

LEGEND

2-inch OD split-spoon sample

MOISTURE CONTENT

Plastic limit — Natural — Liquid limit

RZA AGRA, Inc
Engineering & Environmental Services

11335 NE 122nd Way, Suite 100
Kirkland, Washington 98034-6918

Drilling started: *29 July 1992* Drilling completed: *29 July 1992* Logged by: *TDW*

PROJECT: Proposed Retail Store Aberdeen, WA

W.O. W-8401 BORING NO. B-3

Page 1 of 1

DEPTH (feet)	SOIL DESCRIPTION Approximate ground surface elevation: 11.6 feet	SAMPLE TYPE	SAMPLE NUMBER	GROUND WATER	STANDARD PENETRATION RESISTANCE (Blows per foot)	TESTING
0	Dense, damp, brown, GRAVEL with some sand (Fill)					
			S-1		66	
5				ATD		
	Becomes saturated		S-2			
10						
	Medium stiff, wet, dark gray, SILT, with some wood chips		S-3		(64)	
15	Boring terminated at approximately 14 feet					
20						
25						
30						

LEGEND

2-inch OD split-spoon sample

ATD Groundwater level at time of drilling

MOISTURE CONTENT

Plastic limit — Natural — Liquid limit

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Engineering & Environmental Services

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Kirkland, Washington 98034-6918

Drilling started: 29 July 1992 Drilling completed: 29 July 1992 Logged by: TDW

PROJECT: Proposed Retail Store Aberdeen, WA

W.O. W-8401 BORING NO. B-4

Page 1 of 1

DEPTH (feet)	SOIL DESCRIPTION (Approximate ground surface elevation: 12.4 feet)	SAMPLE TYPE	SAMPLE NUMBER	GROUND WATER	STANDARD PENETRATION RESISTANCE (Blows per foot)	TESTING
0	Dense, dry to damp, brown, sandy GRAVEL (Fill)					
			S-1		Blowcount Overstated 50/6"	
5						
	Very loose, saturated, black SAND with some silt (Fill)		S-2	ATD		
10						
	Medium stiff, wet, dark gray, SILT with trace wood chips		S-3		(62)	PL=59 LL=89 PI=30
15	Boring terminated at approximately 14 feet					
20						
25						
30						

LEGEND

2-inch OD split-spoon sample

ATD Groundwater level at time of drilling

MOISTURE CONTENT

Plastic limit — Natural — Liquid limit

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Engineering & Environmental Services
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Kirkland, Washington 98034-6918

Drilling started: 29 July 1992 Drilling completed: 29 July 1992 Logged by: TDW

PROJECT: *Proposed Retail Store Aberdeen, WA* W.O. *W-8401* BORING NO. *B-5*

DEPTH (feet)	SOIL DESCRIPTION Approximate ground surface elevation: *13.5 feet*	SAMPLE TYPE	SAMPLE NUMBER	GROUND WATER	STANDARD PENETRATION RESISTANCE (Blows per foot)	TESTING
0	*Medium dense, dry, dark gray, crushed gravel (Fill)*					
	Loose, wet, green-gray, SAND (Fill)		S-1			
5	*Cuttings are wet silt and sawdust (Fill)*			ATD		
	Soft, saturated, gray, clayey SILT with some organics (Native)		S-2			
10	*Boring terminated at approximately 9 feet*					
15						
20						
25						
30						

MOISTURE CONTENT: Plastic limit — Natural — Liquid limit

LEGEND

- 2-inch OD split-spoon sample
- 3-inch OD Dames & Moore sample using 300 lbs. Hammer
- ATD Groundwater level at time of drilling
- Consolidation test

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Engineering & Environmental Services
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Kirkland, Washington 98034-6918

Drilling started: *29 July 1992* Drilling completed: *29 July 1992* Logged by: *TDW*

PROJECT: Proposed Retail Store Aberdeen, WA

W.O. W-8401 BORING NO. B-6

DEPTH (feet)	SOIL DESCRIPTION Approximate ground surface elevation: 12.7 feet	SAMPLE TYPE	SAMPLE NUMBER	GROUND WATER	STANDARD PENETRATION RESISTANCE (Blows per foot)	TESTING
0	Dense, damp, brown, sandy GRAVEL (Fill)		S-1		Blowcount Overstated 50/5"	
5	Logs, wood debris, and gravel (Fill)		S-2		Blowcount Overstated 50/3"	
10	Soft, wet grading to saturated, dark gray SILT with trace organic material (Native)		S-3		(67)	
15			P-1	ATD		
			S-4		(74)	
20	Loose, saturated, dark gray, silty SAND with trace woodchips and sawdust		S-5			
25	Becomes medium dense		P-2			
30	(continued)		S-6			

MOISTURE CONTENT

Plastic limit — Natural — Liquid limit

LEGEND

- 2-inch OD split-spoon sample
- 3-inch OD Shelby sampler
- ATD Groundwater level at time of drilling
- Hydrometer analysis

RZA AGRA, Inc
Engineering & Environmental Services
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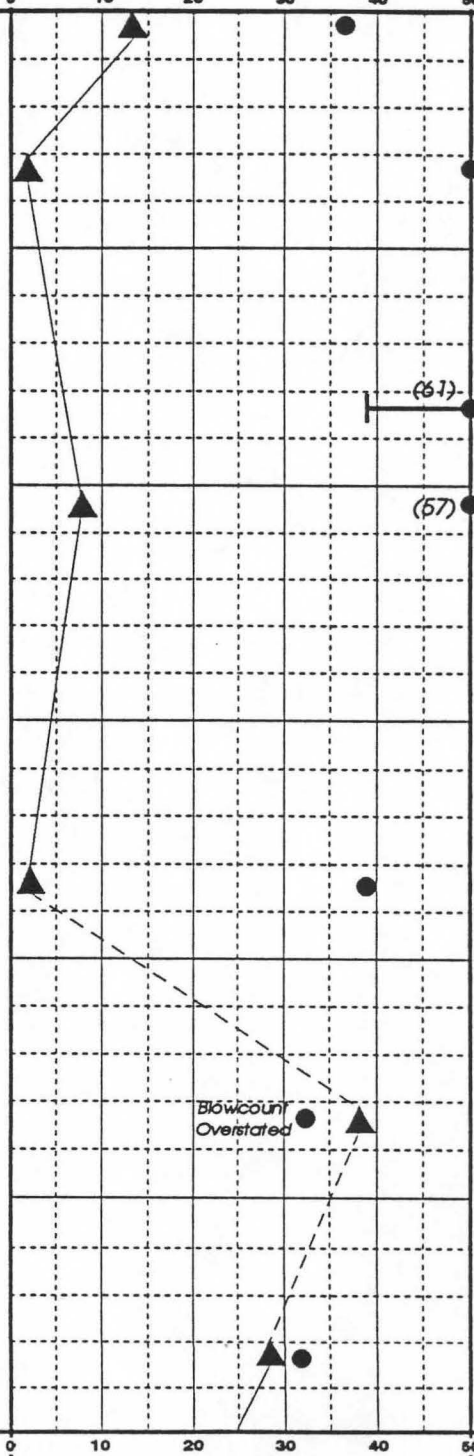
Kirkland, Washington 98034-6918

Drilling started: 30July 1992 Drilling completed: 30 July 1992 Logged by: TDW

PROJECT: *Proposed Retail Store Aberdeen, WA* W.O. *W-8401* BORING NO. *B-6*

DEPTH (feet)	SOIL DESCRIPTION (Approximate ground surface elevation: 12.7 feet)	SAMPLE TYPE	SAMPLE NUMBER	GROUND WATER	STANDARD PENETRATION RESISTANCE (Blows per foot)	TESTING
30	*Silty SAND as above*		S-6			
	Soft, saturated, dark gray, SILT with some sand and trace organic material					
			S-7			
35						
			P-3		(61)	LL=68 PI=29
40	*Become medium stiff*		S-8		(67)	
	Very soft, saturated, dark gray, sandy SILT					
45						
			S-9			
50						
	Medium dense, saturated, black SAND with some silt		S-10		Blowcount Overstated	
55						
			S-11			
60	*(continued)*					



LEGEND

- 2-inch OD split-spoon sample
- Groundwater level at time of drilling (ATD)
- Consolidation test
- 3-inch OD Shelby sampler
- Hydrometer analysis
- Grain size analysis

RZA AGRA, Inc
Engineering & Environmental Services
11335 NE 122nd Way, Suite 100
Kirkland, Washington 98034-6918

Drilling started: *30July 1992* Drilling completed: *30 July 1992* Logged by: *TDW*

PROJECT: Proposed Retail Store Aberdeen, WA

W.O. W-8401 BORING NO. B-6

DEPTH (feet)	SOIL DESCRIPTION Approximate ground surface elevation: 12.7 feet	SAMPLE TYPE	SAMPLE NUMBER	GROUND WATER	STANDARD PENETRATION RESISTANCE (Blows per foot)	TESTING
60	Medium dense, saturated, black, silty SAND		S-12			
65	Boring terminated at approximately 64 feet					
70						
75						
80						
85						
90						

LEGEND

- 2-inch OD split-spoon sample
- Groundwater level at time of drilling (AID)
- 3-inch OD Shelby sampler
- Grain size analysis
- Hydrometer analysis

MOISTURE CONTENT: Plastic limit — Natural — Liquid limit

RZA AGRA, Inc
Engineering & Environmental Services
11335 NE 122nd Way, Suite 100
Kirkland, Washington 98034-6918

Drilling started: 30 July 1992 Drilling completed: 30 July 1992 Logged by: TDW

PROJECT: Proposed Retail Store Aberdeen, WA W.O. W-8401 BORING NO. B-7

DEPTH (feet)	SOIL DESCRIPTION Approximate ground surface elevation: 14.0 feet	SAMPLE TYPE	SAMPLE NUMBER	GROUND WATER	STANDARD PENETRATION RESISTANCE Blows per foot 0 10 20 30 40 50	Page 1 of 1 TESTING
0	Medium dense, damp, gray, GRAVEL (Fill)					
	Loose, wet, glossy black, wood or coal cinders and ash (Fill)		S-1			
5				ATD		
	Becomes very loose, saturated		S-2			
10	Boring terminated at approximately 9 feet					
15						
20						
25						
30						

LEGEND

2-inch OD split-spoon sample

ATD Groundwater level at time of drilling

MOISTURE CONTENT

0 10 20 30 40 50

Plastic limit Natural Liquid limit

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Engineering & Environmental Services
11335 NE 122nd Way, Suite 100
Kirkland, Washington 98034-6918

Drilling started: 30 July 1992 Drilling completed: 30 July 1992 Logged by: TDW

PROJECT: *Proposed Retail Store Aberdeen, WA* W.O. *W-8401* BORING NO. *B-8*

DEPTH (feet)	SOIL DESCRIPTION Approximate ground surface elevation: *12.0 feet*	SAMPLE TYPE	SAMPLE NUMBER	GROUND WATER	STANDARD PENETRATION RESISTANCE Blows per foot (0 10 20 30 40 50)	Page 1 of 1 TESTING
0	*Medium dense, moist, dark gray, sandy GRAVEL (Fill)*					
	Soft, wet, brown, organic SILT (Native)	X				
5						
	Very soft, saturated, gray SILT with some clay and organic material (Native)		S-1	ATD		
10	*Boring terminated at approximately 9 feet*					
15						
20						
25						
30						

LEGEND

I 2-inch OD split-spoon sample

X Sample not recovered

ATD Groundwater level at time of drilling

MOISTURE CONTENT (0 10 20 30 40 50)

Plastic limit — Natural — Liquid limit

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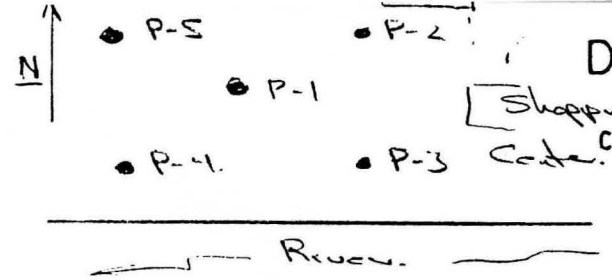
Engineering & Environmental Services
11335 NE 122nd Way, Suite 100
Kirkland, Washington 98034-6918

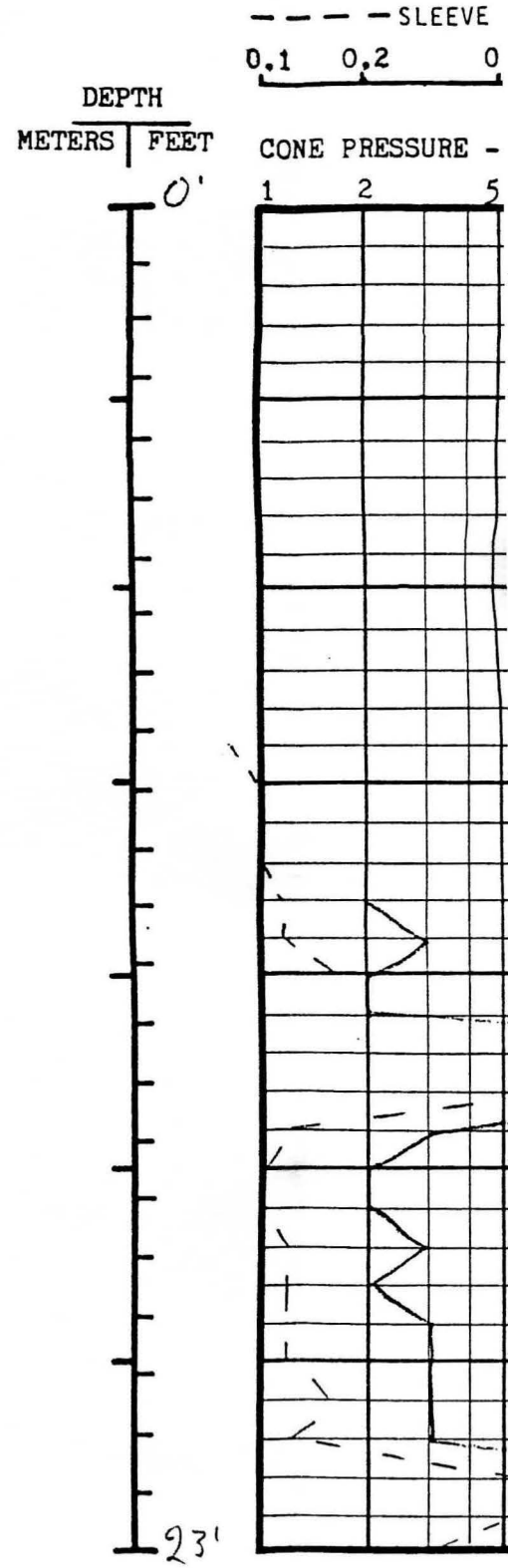
Drilling started: *30 July 1992* Drilling completed: *30 July 1992* Logged by: *TDW*



DUTCH CONE LOG & CALCULA

CLIENT Rittenhouse-Zeman LOCATION Aberdeen, WA

NO. RODS	DEPTH IN METERS	DIAL READINGS TOTAL SYSTEM	CONE ONLY	CONE + SLEEVE	SLEEVE READING —	CONE PRESS. KG/CM²	SLEEVE FRICT. KG/CM²	FRICT. RATIO %
	0.2							
	1.0							
	2.0							
		60	40	NR		5		
		20	0	0				
	3.0	40	20	30	10	3	.07	3.3
		40	10	25	15	2	.10	5.0
		20	0	15	15		.10	
		30	10	25	15	2	.10	6.5
		70	20	40	20	3	.13	4.3
	4.0	40	10	30	20	2	.13	8.5
		50	10	35	25	2	.17	
		840	800	NR		81		1.8
		520	180	400	220	19	1.47	4.2
		480	20	140	120	3	.80	4.3
	5.0	620	10	30	20	2	.13	5.0
		180	10	25	15	2	.10	5.0
		140	20	35	15	3	.10	4.3
		140	10	30	20	2	.13	6.5
		160	20	40	20	3	.13	4.3
	6.0	160	20	40	20	3	.13	4.3
		160	20	40	20	3	.13	5.7
		140	20	45	25	3	.17	4.3
		280	260	280	20	27	.13	2.0
		400	170	250	80	18	.53	2.9
	7.0	480	160	240	80	17	.53	1.9



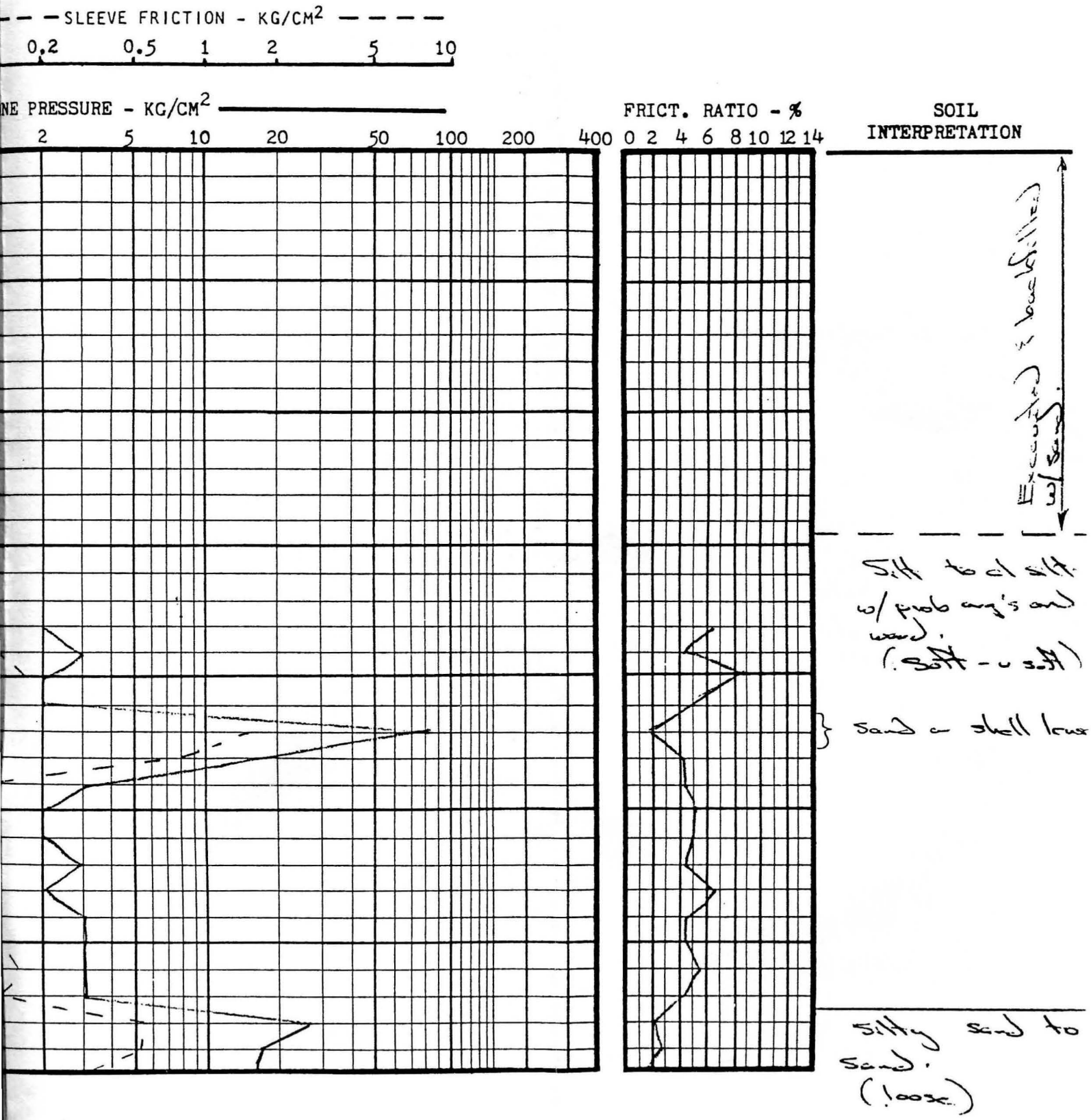
NOTE: 1 KG/CM² = 1.024 TONS/FT.²

1 KG/CM² = .01 Bar

Vandehey Soil Explorations is in the business of soil exploration and test drilling, and is not a soils consulting firm. The services performed by Vandehey Soil Explorations consist of soil exploration and obtaining soil test data by use of the "Dutch Cone Penetrometer" and conventional test drilling and Vandehey Soil Explorations warrants only that its soil exploration and testing equipment is in proper and good operating order, that proper operating procedures for use of the "Dutch Cone Penetrometer" and test drilling equipment are implemented in the use of said machines, and that data readings are accurately reported. No reliance should be placed on soil interpretive conclusions arrived at by Spencer Vandehey and Vandehey Soil Explorations but rather, all data reported should be reviewed and interpreted by qualified soil engineers.

ALCULATION SHEET

deen, WA JOB NO _____ PROBE NO P-1 DATE 7-29-92 SHEET 1 of 4



VANDEHEY SOIL EXPLORATIONS
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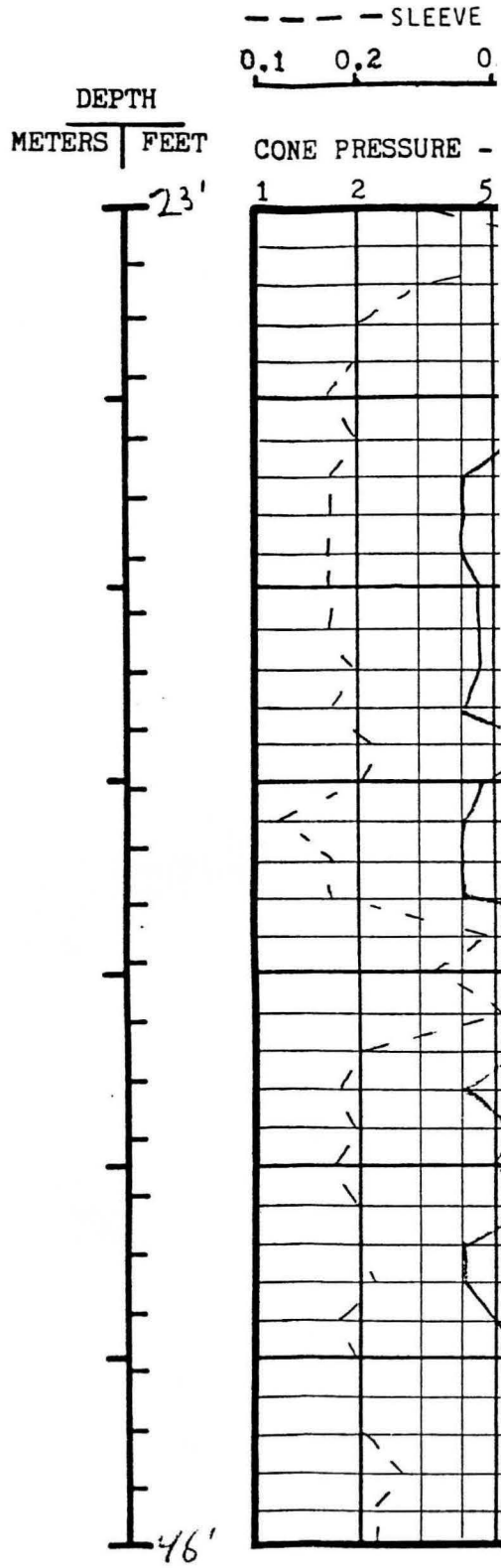
DUTCH CONE LOG & CALCULAT

CLIENT Rittenhouse-Zeman LOCATION Aberdeen WA

NO. RODS	DEPTH IN METERS	DIAL READINGS TOTAL SYSTEM	CONE ONLY	CONE + SLEEVE	SLEEVE READING —	CONE PRESS. KG/CM²	SLEEVE FRICT. KG/CM²	FRICT. RATIO %
	7.2	970	620	670	50	6½	.33	1.5
		520	100	240	140	12	.93	2.8
		460	50	100	50	7	.33	2.9
		560	50	80	30	7	.20	2.9
	8.0	350	45	75	30	6½	.20	3.6
		320	40	65	25	6	.17	3.3
		310	20	50	30	4	.20	4.3
		300	20	45	25	4	.17	4.3
		350	20	45	25	4	.17	4.3
	9.0	360	25	50	25	4½	.17	3.8
		370	25	50	25	4½	.17	3.8
		370	25	50	25	4½	.17	4.4
		360	20	50	30	4	.20	4.3
		400	40	75	25	4	.17	3.3
	10.0	400	25	60	35	4½	.23	4.4
		400	20	50	30	4	.20	3.3
		400	20	40	20	4	.13	4.3
		400	20	45	25	4	.17	4.3
		540	140	165	25	16	.17	2.9
	11.0	540	110	180	70	13	.47	2.5
		540	60	110	50	8	.33	6.6
		580	40	120	80	6	.53	3.3
		580	20	50	30	4	.20	4.3
		500	35	60	25	5½	.17	3.6
	12.0	500	30	60	30	5	.20	4.4
		530	50	75	25	7	.17	2.9
		510	20	50	30	4	.20	5.0
		500	20	50	30	4	.20	5.8
		550	20	65	35	5	.23	3.4
	13.0	560	40	65	25	6	.17	3.2
		570	35	65	30	5½	.20	3.6
		580	40	70	30	6	.20	3.3
		580	35	65	30	5½	.20	4.9
		620	40	80	40	6	.27	3.8
	14.0	620	45	80	35	6½	.23	3.5

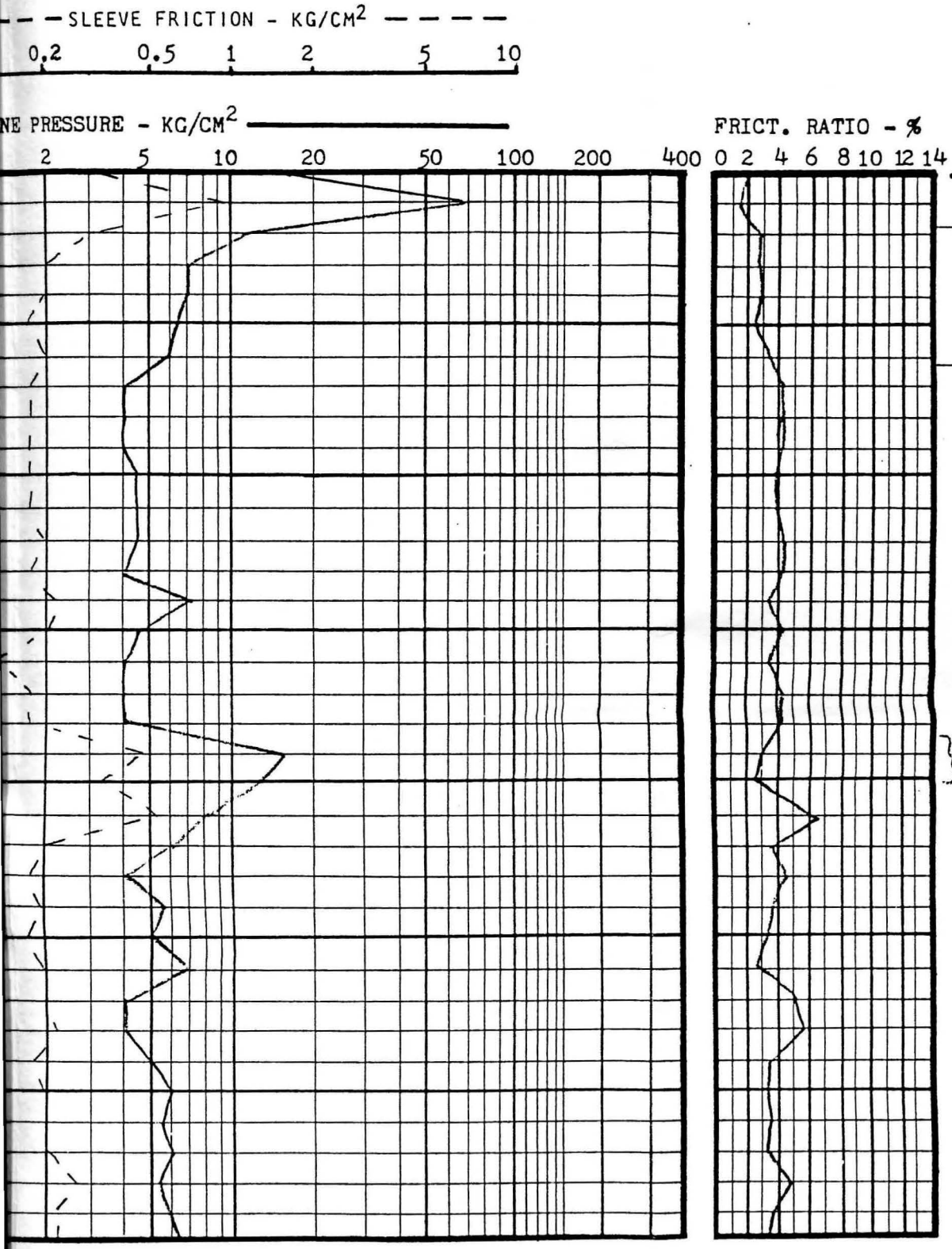


NOTE: 1 KG/CM² = 1.024 TONS/FT.²

Vandehey Soil Explorations is in the business of soil exploration and test drilling, and is not a soils consulting firm. The services performed by Vandehey Soil Explorations consist of soil exploration and obtaining soil test data by use of the "Dutch Cone Penetrometer" and conventional test drilling and Vandehey Soil Explorations warrants only that its soil exploration and testing equipment is in proper and good operating order, that proper operating procedures for use of the "Dutch Cone Penetrometer" and test drilling equipment are implemented in the use of said machines, and that data readings are accurately reported. No reliance should be placed on soil interpretive conclusions arrived at by Spencer Vandehey and Vandehey Soil Explorations but rather, all data reported should be reviewed and interpreted by qualified soil engineers.

LCULATION SHEET

en UA JOB NO _____ PROBE NO P-1 DATE 7-29-92 SHEET 2 of 4



SOIL INTERPRETATION

See prev. page

Silt to sa. silt. (soft)

Silt to cl silt w/ poss s cgs (soft)

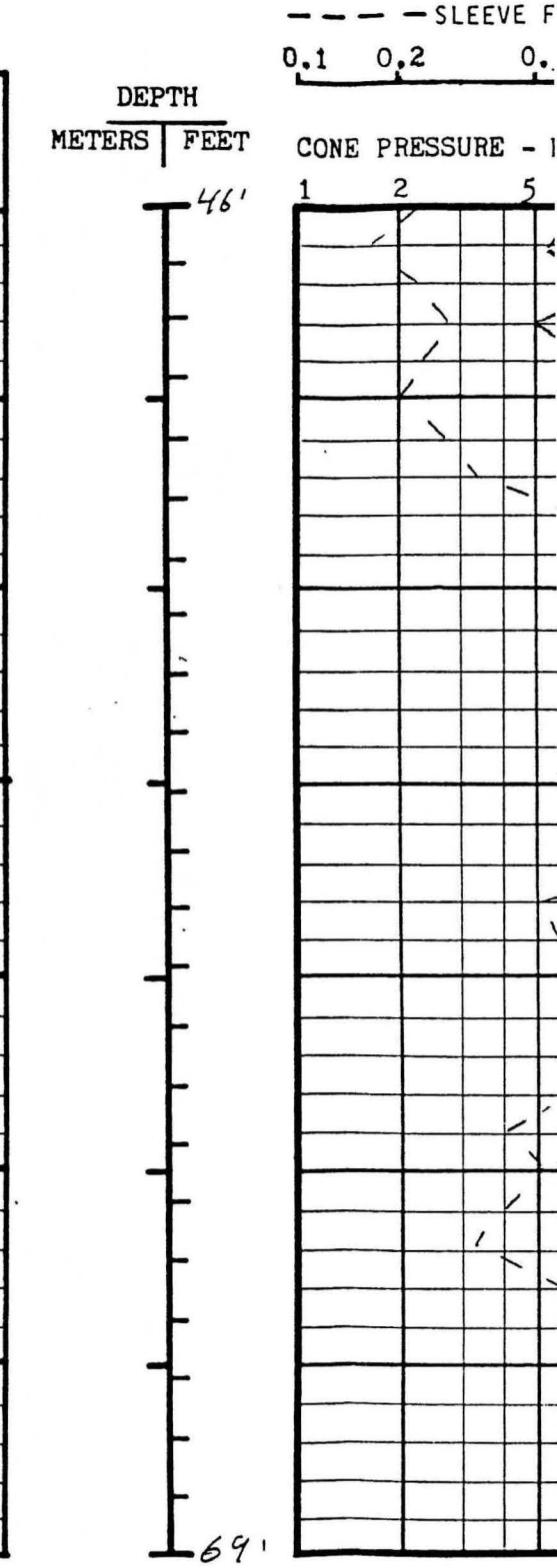
SM-ML, soft - m stiff

DUTCH CONE LOG & CALCULAT

CLIENT Rittenhouse-Zeman LOCATION Aberdeen, MA J

	NO. RODS	DEPTH IN METERS	DIAL READINGS TOTAL SYSTEM	DIAL READINGS CONE ONLY	DIAL READINGS CONE + SLEEVE	SLEEVE READING —	CONE PRESS. KG/CM²	SLEEVE FRICT. KG/CM²	FRICT. RATIO %
		14.2	630	25	60	35	5½	.23	3.1
			620	60	85	25	9	.17	2.6
			640	20	55	35	5	.23	5.4
			700	40	80	40	7	.27	3.3
		15.0	680	35	70	35	6½	.23	3.1
			700	60	90	30	9	.20	3.0
			720	70	110	40	10	.27	2.3
			880	600	650	50	63	.33	1.1
			1700	950	1050	100	98	.67	0.7
		16.0	2300	1300	1400	100	133	.67	0.8
			2600	1000	1150	150	103	1.00	0.7
			3400	1550	1650	100	158	.67	0.8
			2800	600	780	180	63	1.20	1.4
			2500	550	680	130	58	.87	1.5
		17.0	2700	440	570	130	47	.87	1.6
			1900	820	960	140	85	.93	1.1
			1500	320	460	140	35	.93	3.8
			1000	210	410	200	24	1.33	2.2
			1600	400	480	80	43	.53	1.4
		18.0	1300	350	440	90	38	.60	1.9
			800	210	330	110	30	.73	2.9
			1100	730	360	130	26	.87	3.6
R			1200	520	660	140	55	.93	1.2
			1200	130	230	100	16	.67	2.5
		19.0	1000	160	220	60	19	.40	2.8
			1050	120	200	80	15	.53	2.7
			820	70	130	60	10	.40	3.3
			950	140	240	50	22	.33	2.1
R			1200	450	540	90	48	.60	
RS		20.0	1100	450	NR		48		1.3
			1500	360	450	90	39	.60	2.6
			1400	160	310	150	19	1.00	3.5
			1200	280	380	100	31	.67	1.9
			1300	320	410	90	35	.60	1.9
		21.0	1200	190	280	100	21	.67	2.9

NOTE: 1 KG/CM² = 1.024 TONS/FT.²



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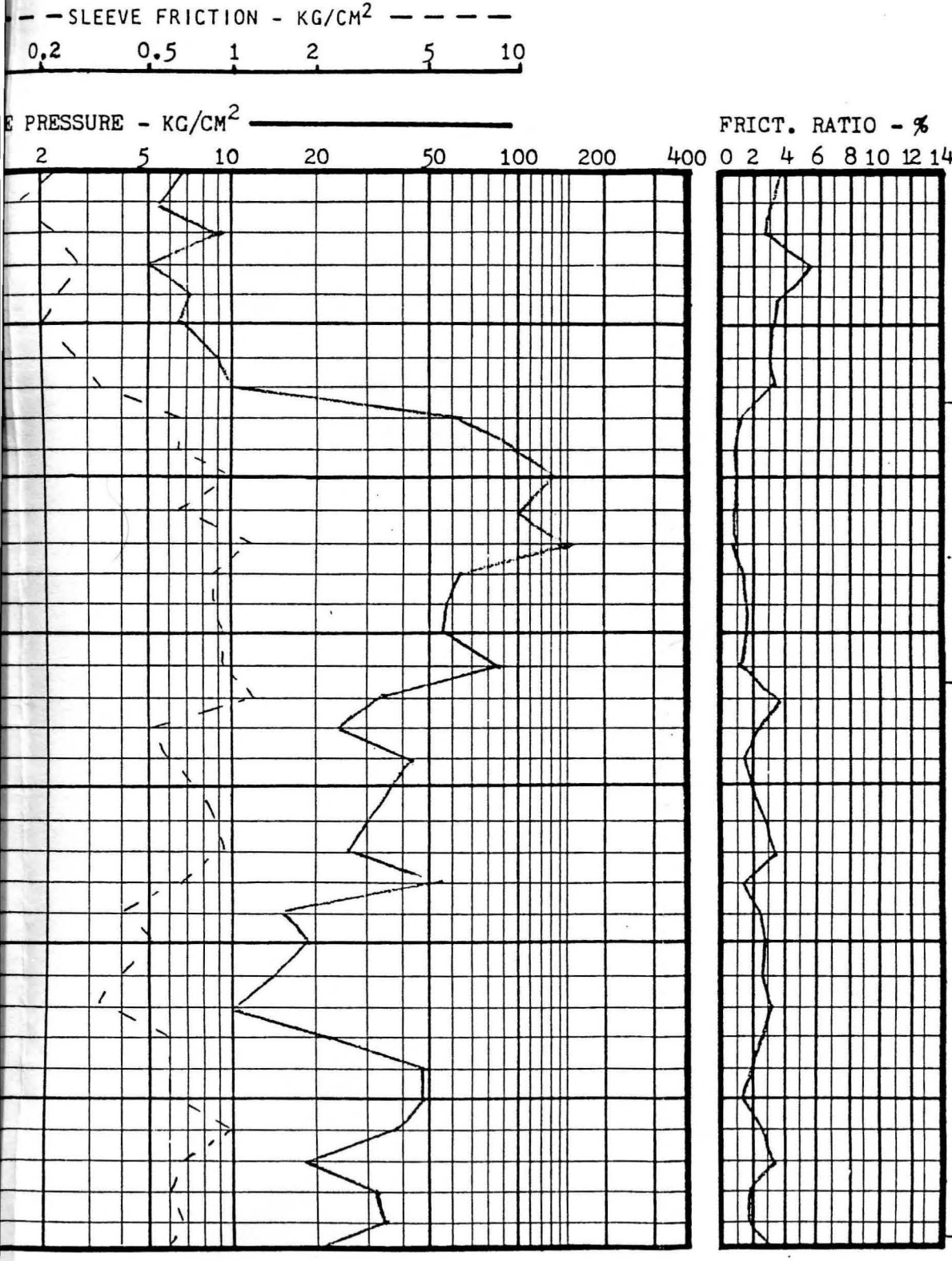
LCULATION SHEET

...ern, WA JOB NO _____ PROBE NO P-1 DATE 7-29-92 SHEET 3 of 4



SOIL INTERPRETATION

cl silt to silt (soft - m stiff)

Sand. m dense - dense

m dense.

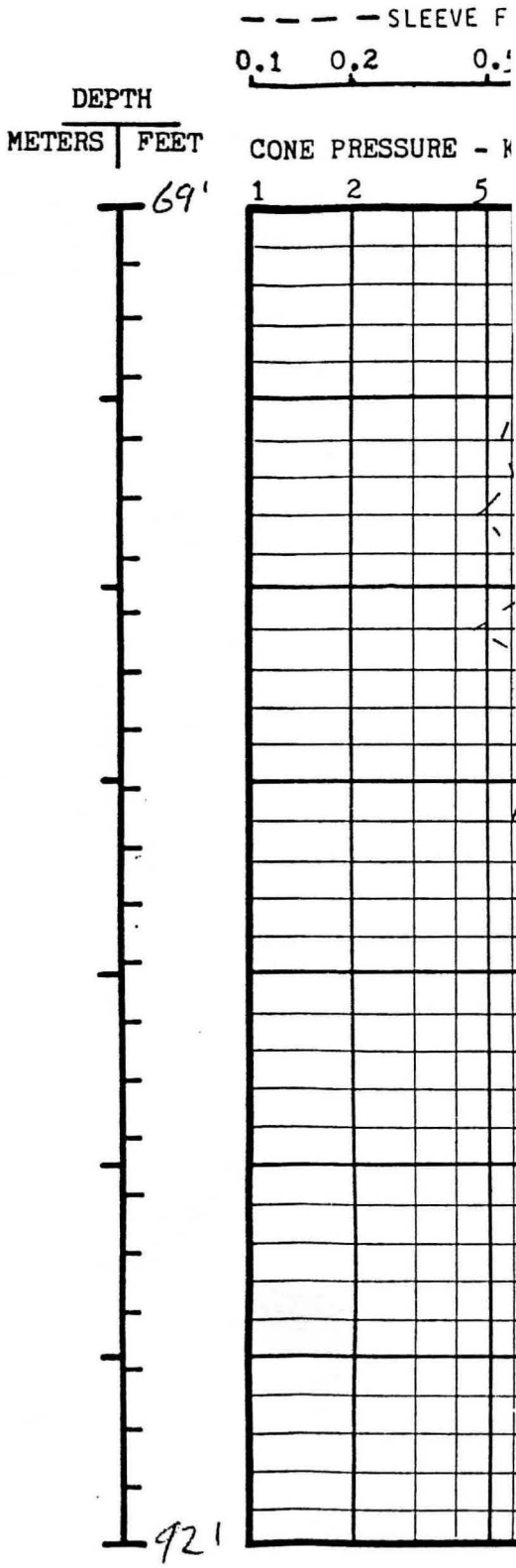
Alternating layers of silt, sand & SM-ML. (m dense to m stiff - stiff)

See next page.

DUTCH CONE LOG & CALCULAT

CLIENT Rittenhouse-Zeman LOCATION Aberdeen, WA JI

NO. RODS	DEPTH IN METERS	DIAL READINGS TOTAL SYSTEM	DIAL READINGS CONE ONLY	DIAL READINGS CONE + SLEEVE	SLEEVE READING —	CONE PRESS. KG/CM²	SLEEVE FRICT. KG/CM²	FRICT. RATIO %
	21.2	1150	180	370	90	32	.60	2.5
		1200	180	300	120	22	.80	3.3
R		1050	200	310	110	24	.73	3.3
		1400	350	470	120	39	.80	2.4
	22.0	1500	560	700	140	60	.93	1.2
		1400	260	370	110	30	.73	2.2
		1400	220	320	100	26	.67	2.8
		1200	100	210	110	14	.73	3.4
		1200	130	200	70	17	.47	3.9
	23.0	1000	250	350	100	29	.67	2.5
		1400	540	650	110	58	.73	0.8
		1200	420	490	70	46	.47	1.6
		1200	370	480	140	41	.73	2.1
		1700	900	1030	130	94	.87	1.5
	24.0	1600	200	410	210	24	1.40	2.8
		1600	220	320	100	26	.67	2.3
		1800	260	350	90	30	.60	
	25.0							
	26.0							
	27.0							
	28.0							



NOTE: 1 KG/CM² = 1.024 TONS/FT.²

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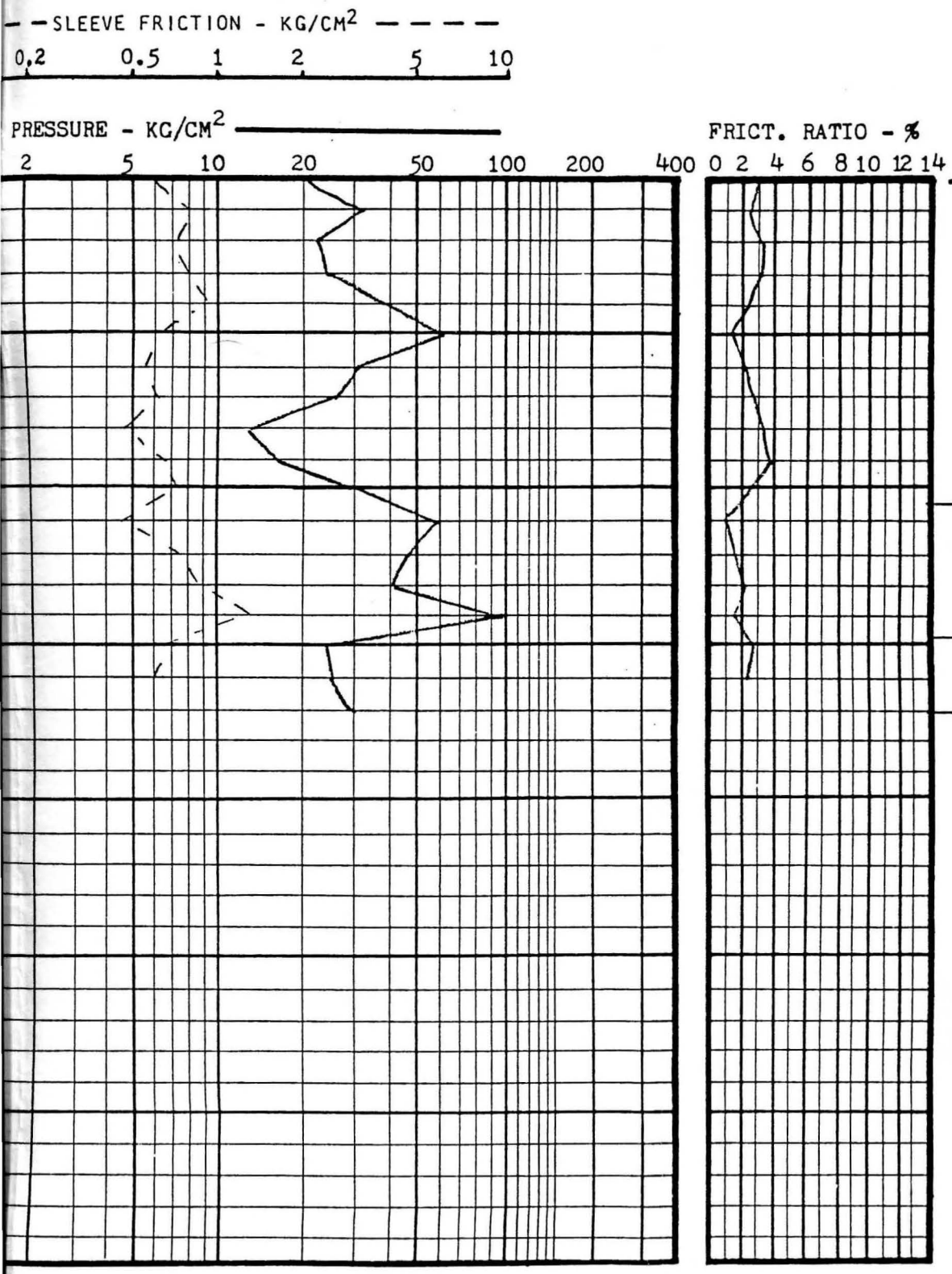
LCULATION SHEET

en, WA JOB NO _____ PROBE NO P-1 DATE 7-29-92 SHEET 4 of 4



SOIL INTERPRETATION

Silt w/ layers of SM-ML & sand. (stiff & medium

Sand to sa w/s silt. (medium dense)

Sandy silt to silty sand.

Terminated @ 80 ft.

VANDEHEY SOIL EXPLORATIONS

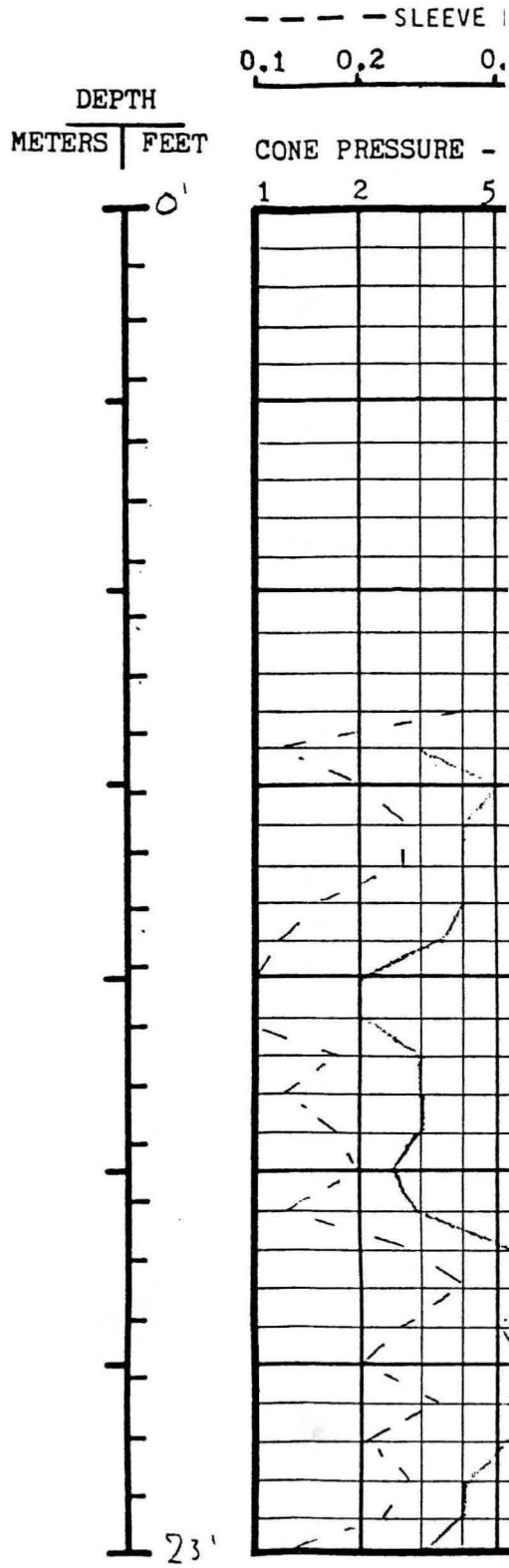
RT 2, BOX 25 • BANKS, OREGON • PHONE AREA CODE (503) 324-3261

DUTCH CONE LOG & CALCULAT

CLIENT Rittenhouse-Zeman LOCATION Aberdeen, WA

NO. RODS	DEPTH IN METERS	DIAL READINGS TOTAL SYSTEM	CONE ONLY	CONE + SLEEVE	SLEEVE READING —	CONE PRESS. KG/CM²	SLEEVE FRICT. KG/CM²	FRICT. RATIO %
	0.2							
	1.0							
	2.0	180	40	NR		5		
		150						
		160	20	80	60	3	.40	4.3
	3.0	160	40	60	20	5	.12	4.0
		100	30	60	30	4	.20	6.7
		110	30	70	40	4	.27	6.8
		130	30	70	40	4	.27	4.3
		160	25	50	25	3½	.17	3.7
	4.0	120	10	30	20	2	.13	5.0
		100	10	25	15	2	.10	3.5
		120	20	30	10	3	.07	5.7
		130	20	45	25	3	.17	4.3
		180	40	40	20	3	.13	5.7
	5.0	170	15	40	25	2½	.17	7.0
		170	20	50	30	3	.20	4.3
		200	40	70	20	6	.13	4.5
		220	60	100	40	4	.27	6.4
		240	40	100	60	5	.40	5.4
	6.0	260	40	70	40	6	.27	5.3
		400	40	140	30	12	.20	2.8
		300	40	90	50	4	.33	4.0
		310	30	60	30	4	.20	6.8
		400	30	70	40	4	.27	5.8
	7.0	360	20	55	35	3	.23	4.8

NOTE: 1 KG/CM² = 1.024 TONS/FT.²



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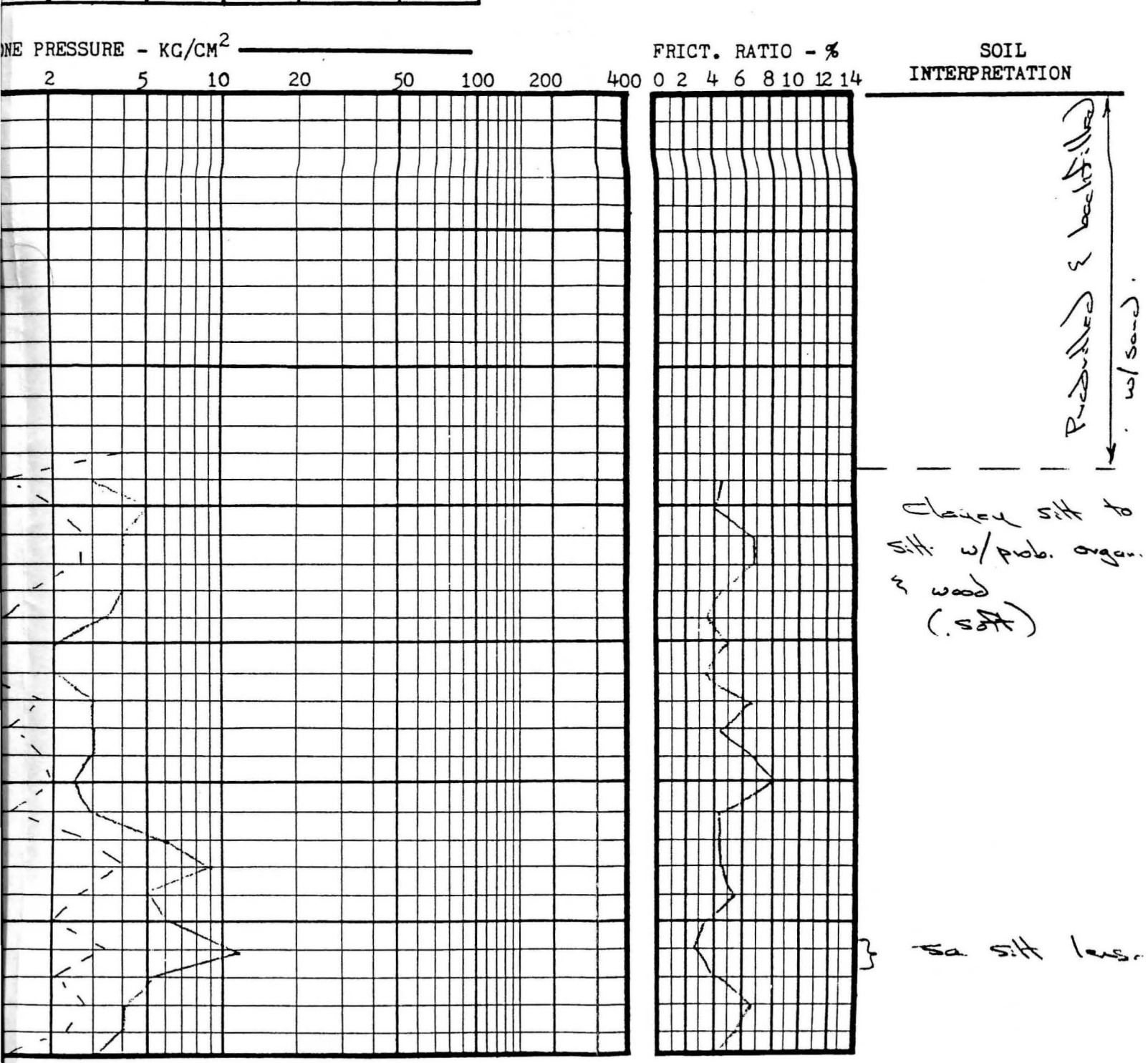
ALCULATION SHEET

deen, WA JOB NO _____ PROBE NO P-2 DATE 7-31-92 SHEET 1 of 3

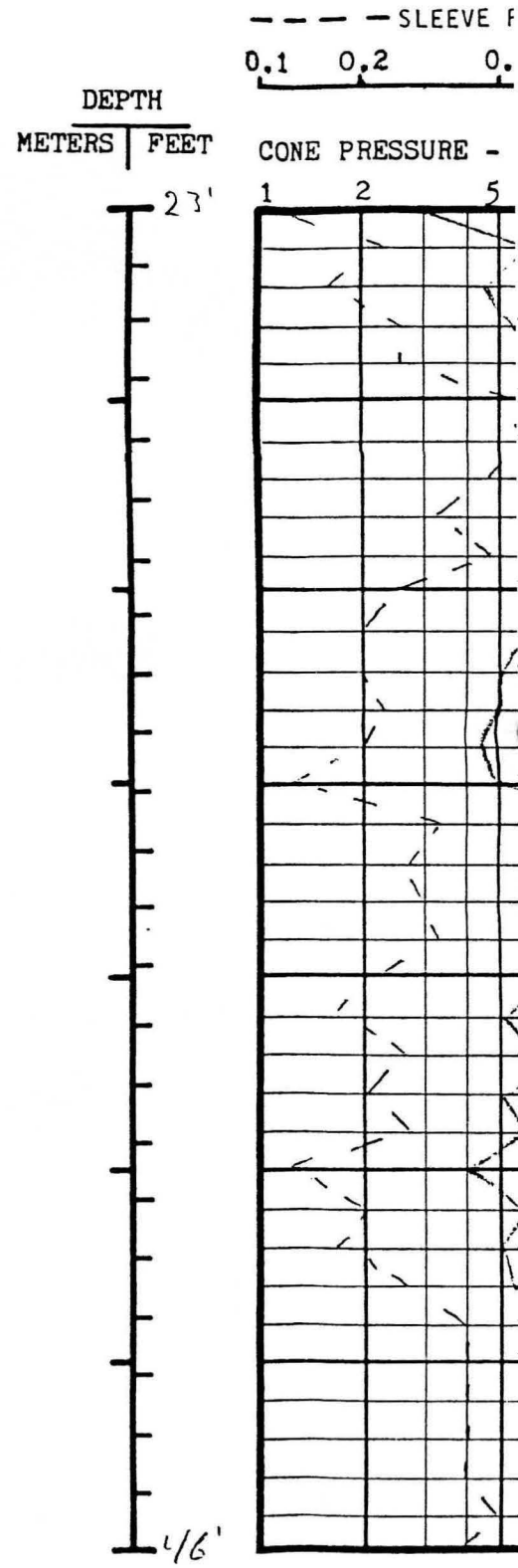


VANDEHEY SOIL EXPLORATIONS
RT. 2, BOX 25 • BANKS, OREGON • PHONE AREA CODE (503) 324-3261

DUTCH CONE LOG & CALCULAT

CLIENT R. Hemmer Tavern LOCATION Aberdeen, WA J

NO. RODS	DEPTH IN METERS	DIAL READINGS TOTAL SYSTEM	CONE ONLY	CONE + SLEEVE	SLEEVE READING —	CONE PRESS. KG/CM²	SLEEVE FRICT. KG/CM²	FRICT. RATIO %
	7.2	340	40	60	20	6	.13	3.5
		340	25	60	35	4½	.23	3.8
		340	25	60	25	5½	.17	4.9
R		420	100	140	40	12	.27	2.3
	8.0	580	220	260	40	21	.27	2.2
		600	240	320	80	26	.53	2.3
		580	100	190	90	12	.60	3.9
		640	70	140	70	9	.47	3.7
		640	110	160	50	13	.33	3.6
	9.0	570	50	120	70	7	.41	3.9
		600	40	80	40	6	.27	3.3
		630	30	60	30	5	.20	4.0
		620	30	60	30	5	.20	4.6
		620	25	60	35	4½	.23	4.4
	10.0	640	30	60	30	5	.20	2.6
		740	100	120	20	12	.13	2.8
		780	140	190	50	16	.33	1.7
		720	40	80	40	6	.27	5.0
		1000	120	165	45	14	.30	2.4
	11.0	900	50	100	50	7	.33	3.3
		600	30	65	35	5	.23	3.4
		780	50	75	25	7	.17	3.9
		720	30	70	40	5	.27	4.0
		720	40	70	30	6	.20	4.5
	12.0	720	20	60	40	4	.27	3.3
		720	60	80	20	6	.13	3.3
		720	30	60	30	5	.20	3.4
		720	25	60	35	4½	.17	4.9
		760	200	240	40	22	.27	1.8
	13.0	570	120	180	60	14	.40	2.9
		760	110	170	60	13	.40	3.1
		1000	170	210	60	17	.40	2.4
		800	140	200	60	16	.40	2.5
		1200	180	240	60	20	.40	2.4
	14.0	1200	140	210	70	15	.47	3.7



NOTE: 1 KG/CM² = 1.024 TONS/FT.²

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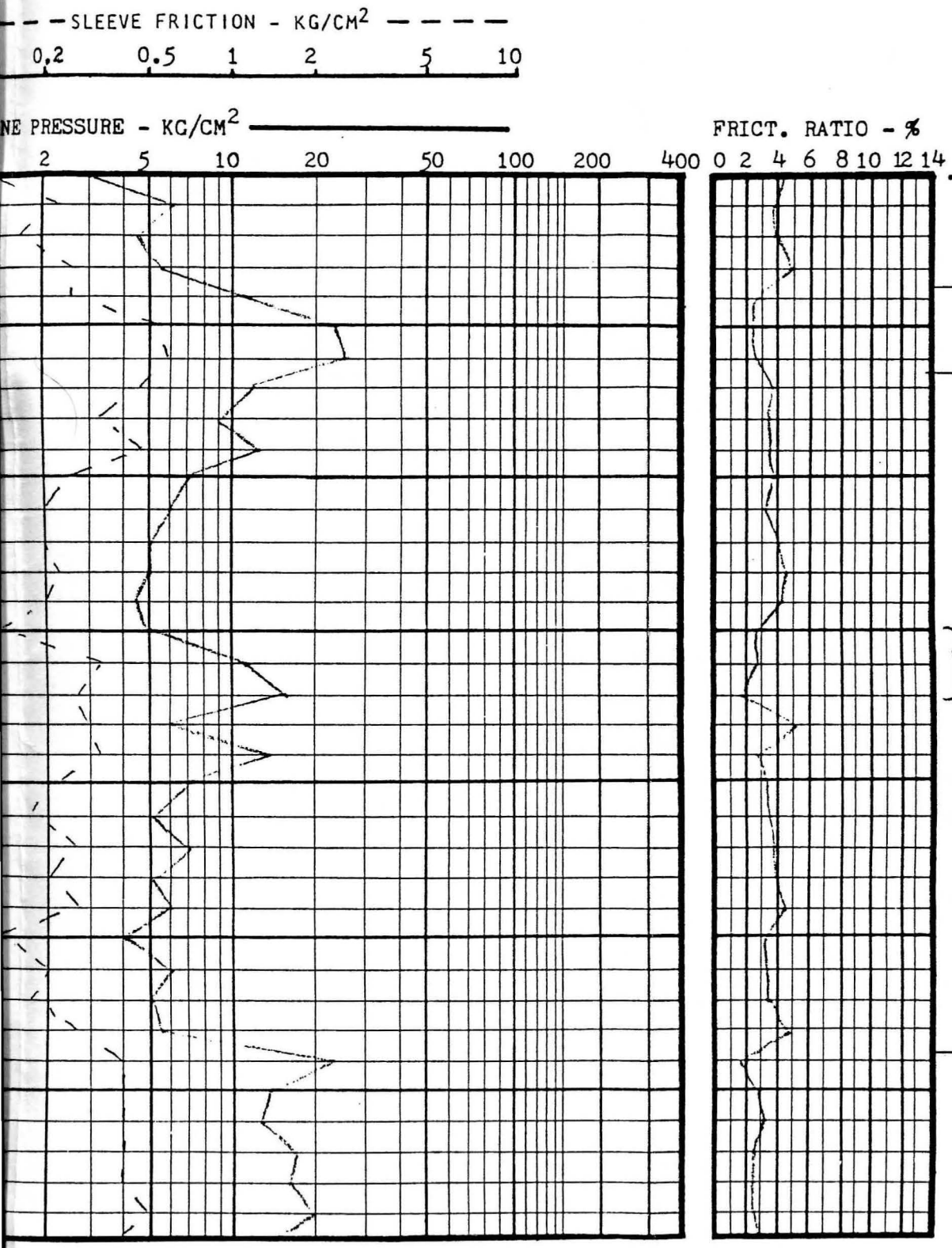
ALCULATION SHEET

dson, WA JOB NO _____ PROBE NO P-2 DATE 7-31-92 SHEET 2 of 3



SOIL INTERPRETATION

See prev page

Silty sand (loose)

silt to cl silt (m stiff)

soft - m stiff.

SM-ML layer

Silty sand to sand silt. (loose - m dense)

VANDEHEY SOIL EXPLORATIONS
RT 2, BOX 25 • BANKS, OREGON • PHONE AREA CODE (503) 324-3261

DUTCH CONE LOG & CALCULAT

CLIENT R. Hentlouer-Zeman LOCATION Aberdeen, WA JO

NO. RODS	DEPTH IN METERS	DIAL READINGS TOTAL SYSTEM	CONE ONLY	CONE + SLEEVE	SLEEVE READING —	CONE PRESS. KG/CM²	SLEEVE FRICT. KG/CM²	FRICT. RATIO %
	14.2	1150	130	190	60	16	.40	2.9
		1200	200	270	70	23	.47	2.9
		1200	240	340	100	27	.67	2.5
		1500	400	500	100	43	.67	1.9
	15.0	1600	420	540	120	45	.80	1.6
		1800	640	750	110	67	.73	1.0
		2000	950	1050	100	98	.67	1.0
		2200	950	1000	150	98	1.00	0.8
		2700	950	1050	100	98	.67	1.1
	16.0	2800	680	940	160	71	1.07	1.1
		2600	720	840	120	75	.80	1.8
		2400	620	820	200	65	1.33	0.9
		2000	610	900	90	64	.60	1.3
		2400	580	750	170	61	1.13	1.5
	17.0	2400	620	960	140	85	.93	1.3
		2400	400	560	160	43	1.07	2.6
		2400	320	490	170	35	1.13	2.5
		2000	450	580	130	48	.87	1.7
		1700	240	410	120	32	.80	3.1
	18.0	1500	150	300	150	18	1.00	3.7
		1300	110	210	100	14	.67	3.8
		1200	390	460	80	41	.53	2.0
→		1200	300	420	120	33	.80	
	19.0							
	20.0							
	21.0							

DEPTH

METERS	FEET
	46'
	69'

— — — — SLEEVE F

0.1 0.2 0.5

CONE PRESSURE - K

1 2 5

NOTE: 1 KG/CM² = 1.024 TONS/FT.²

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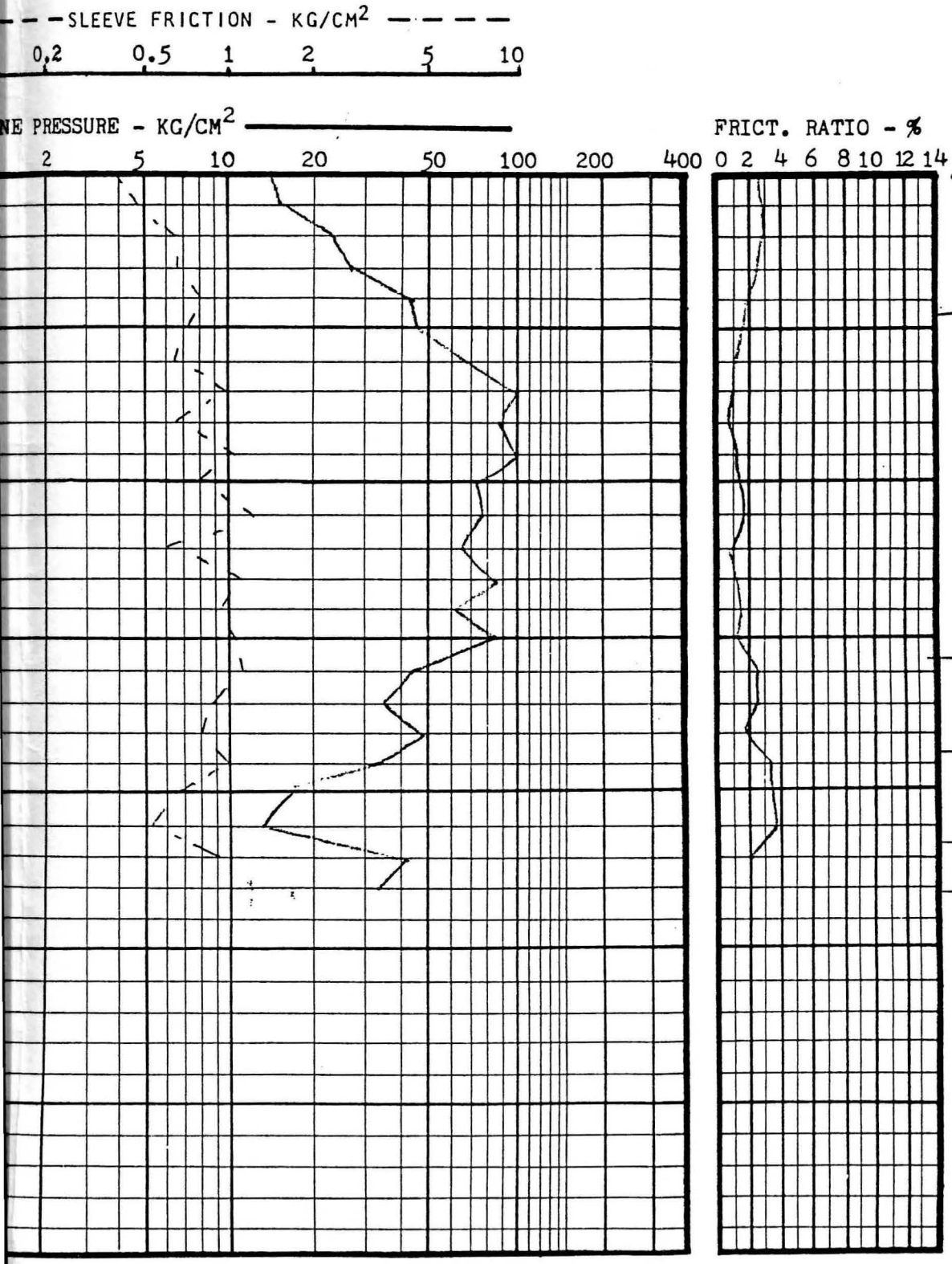
ALCULATION SHEET

dorin, WA JOB NO _____ PROBE NO P-2 DATE 7-3-92 SHEET 3 of 3



SOIL INTERPRETATION

See prev page.

Sand (m dense)

Sa silt to si sa (m dense)

Silt to cl silt. (stiff)

Sand to si sand.

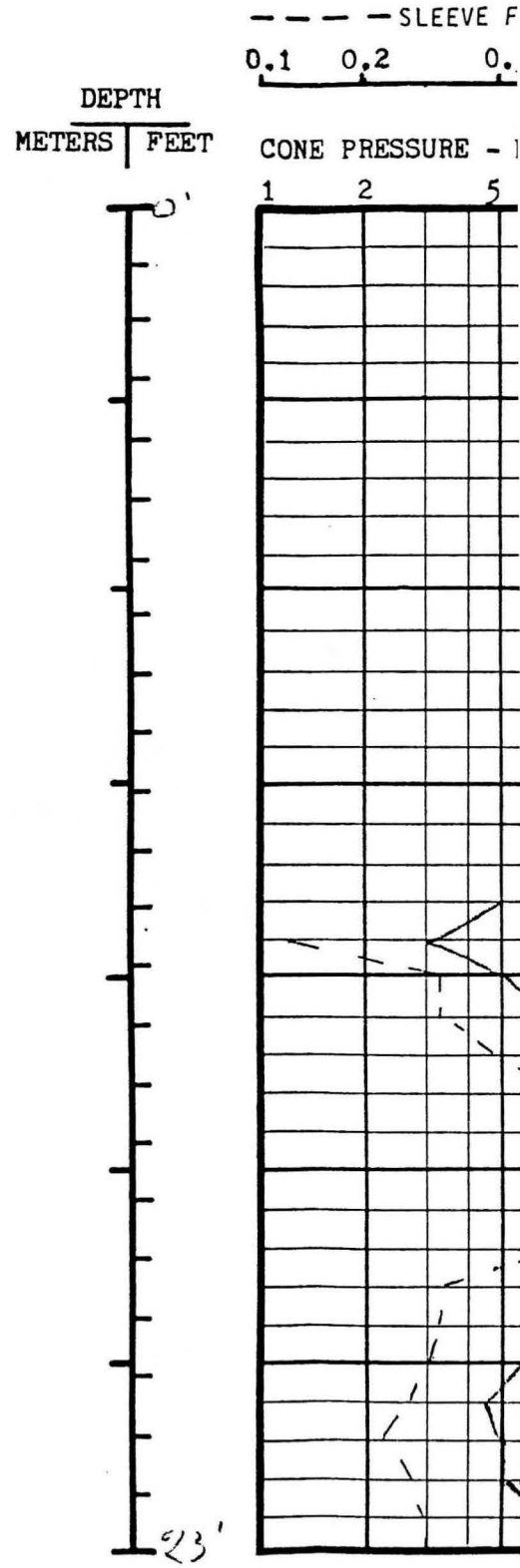
Terminated @ 61'

VANDEHEY SOIL EXPLORATIONS
RT. 2, BOX 25 • BANKS, OREGON • PHONE AREA CODE (503) 324-3261

DUTCH CONE LOG & CALCULAT

CLIENT Rittenhouse-Zeman LOCATION Aberdeen, WA J

NO. RODS	DEPTH IN METERS	DIAL READINGS TOTAL SYSTEM	CONE ONLY	CONE + SLEEVE	SLEEVE READING —	CONE PRESS. KG/CM²	SLEEVE FRICT. KG/CM²	FRICT. RATIO %
	0.2							
		180	100	NR				
	1.0	300	100	110	10	11	.07	
		120	40	NR		5		
	2.0	70	0	0				
		60						
		60						
	3.0							
		140	50	NR				
		140	40	50	10	5	.07	
		160	20	NR		3		4.3
	4.0	130	40	60	20	5	.13	6.6
		150	60	110	50	7	.33	4.7
		180	70	120	50	8	.33	5.9
		180	70	140	70	8	.47	9.1
		380	150	260	110	16	.73	4.6
	5.0	280	50	160	110	6	.73	
S		260	300	NR		31		2.6
		350	70	140	120	8	.80	6.1
		330	50	160	110	6	.73	5.5
		290	50	100	50	6	.33	5.5
	6.0	270	50	100	50	6	.33	5.0
		280	35	80	45	4½	.30	6.0
		310	40	80	40	5	.27	4.6
		330	40	75	35	5	.23	5.4
		360	60	100	40	7	.27	4.3
	7.0	350	50	95	45	6	.30	5.0



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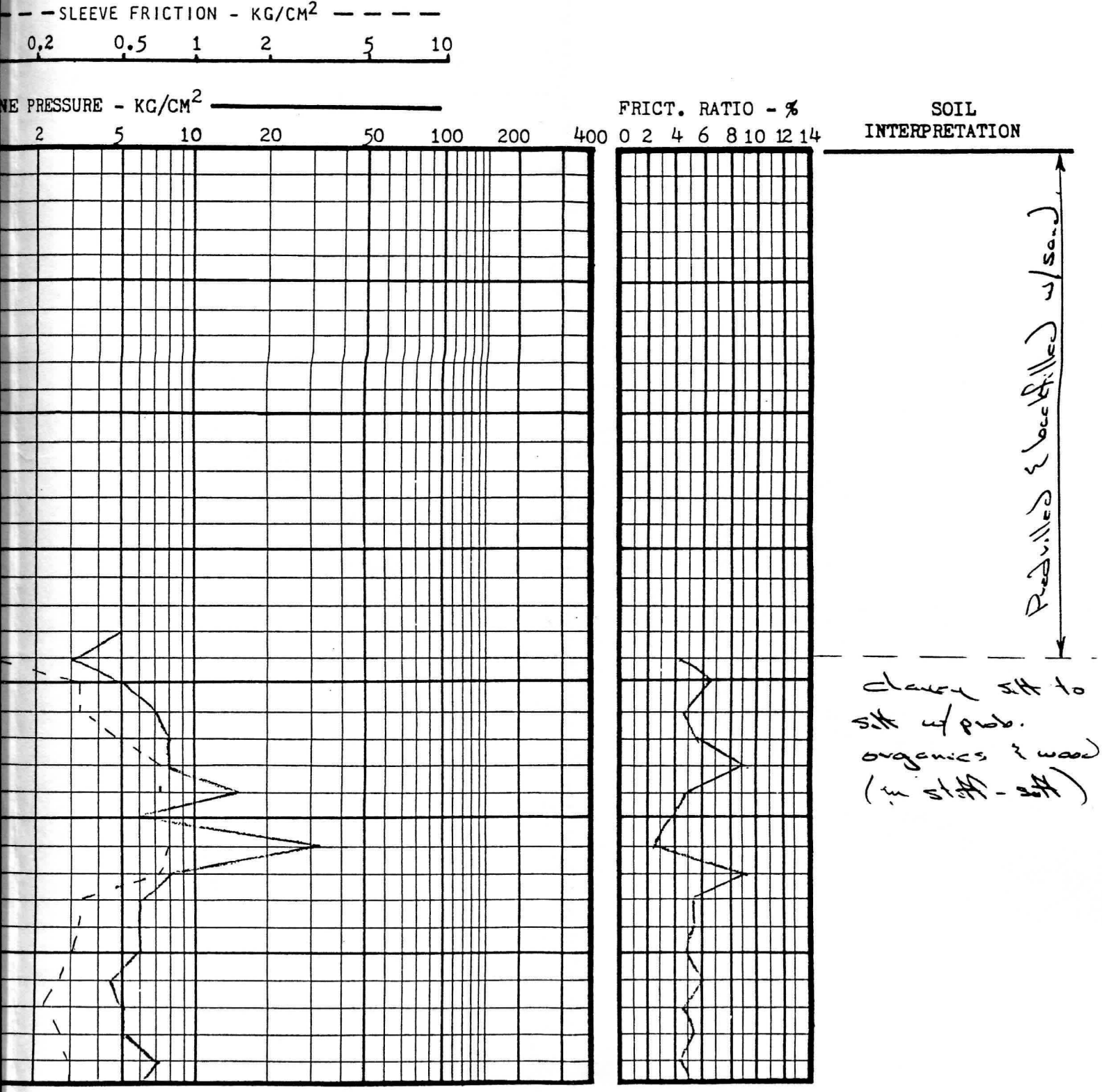
LCULATION SHEET

deen, WA JOB NO _____ PROBE NO P-3 DATE 7-30-92 SHEET 1 of 3

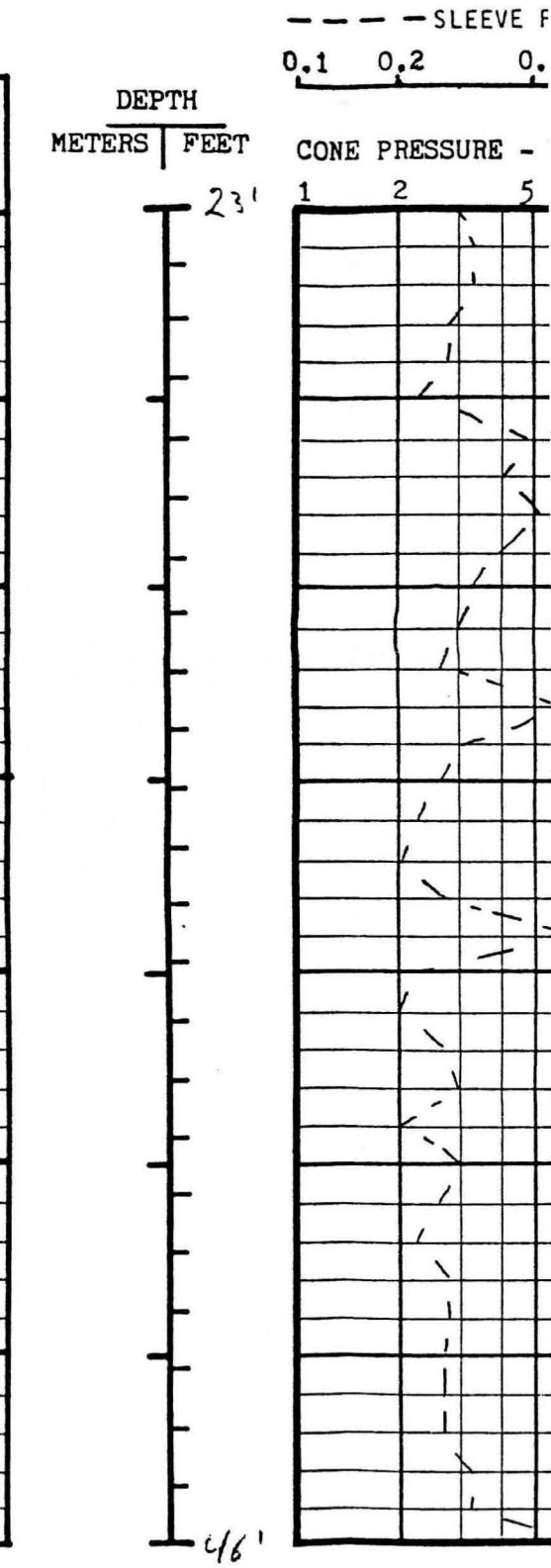


VANDEHEY SOIL EXPLORATIONS
RT 2, BOX 25 • BANKS, OREGON • PHONE AREA CODE (503) 324-3261

DUTCH CONE LOG & CALCULAT

CLIENT Rittenhouse-Zeman LOCATION Aberdeen, WA J

NO. RODS	DEPTH IN METERS	DIAL READINGS TOTAL SYSTEM	DIAL READINGS CONE ONLY	DIAL READINGS CONE + SLEEVE	SLEEVE READING —	CONE PRESS. KG/CM²	SLEEVE FRICT. KG/CM²	FRICT. RATIO %
	7.2	410	115	90	45	6½	.30	5.1
		440	70	120	50	9	.33	3.7
		580	50	100	50	7	.33	3.9
		540	60	100	40	8	.27	3.4
	8.0	570	50	90	40	7	.27	3.3
		600	80	115	35	10	.23	4.7
		590	60	130	70	8	.47	5.0
		870	320	380	60	34	.40	1.6
		820	160	240	80	18	.53	2.2
	9.0	850	80	140	60	10	.40	3.3
		950	70	120	50	9	.33	3.3
		790	60	105	45	8	.30	3.4
		750	110	150	40	13	.27	4.6
		820	70	160	90	9	.60	3.3
	10.0	770	40	85	45	6	.30	4.5
		850	40	80	40	6	.27	3.8
		760	45	80	35	6½	.23	3.1
		740	40	70	30	6	.20	4.5
		1000	340	380	40	36	.27	1.9
	11.0	800	50	150	100	7	.67	3.3
		810	40	75	35	6	.23	3.3
		1000	70	100	30	9	.20	3.0
		820	65	105	40	8½	.27	3.5
		820	55	100	45	7½	.30	2.7
	12.0	920	50	110	30	10	.20	3.0
		880	55	100	45	7½	.30	3.6
		900	45	85	40	6½	.27	3.5
		920	45	80	35	6½	.23	4.2
		960	55	95	40	7½	.27	3.6
	13.0	1000	40	80	40	6	.27	4.5
		1000	40	80	40	6	.27	4.5
		1000	40	80	40	6	.27	4.5
		1050	40	80	40	6	.27	5.5
		1050	100	150	50	12	.33	2.8
	14.0	1400	150	200	50	17	.33	2.9



NOTE: 1 KG/CM² = 1.024 TONS/FT.²

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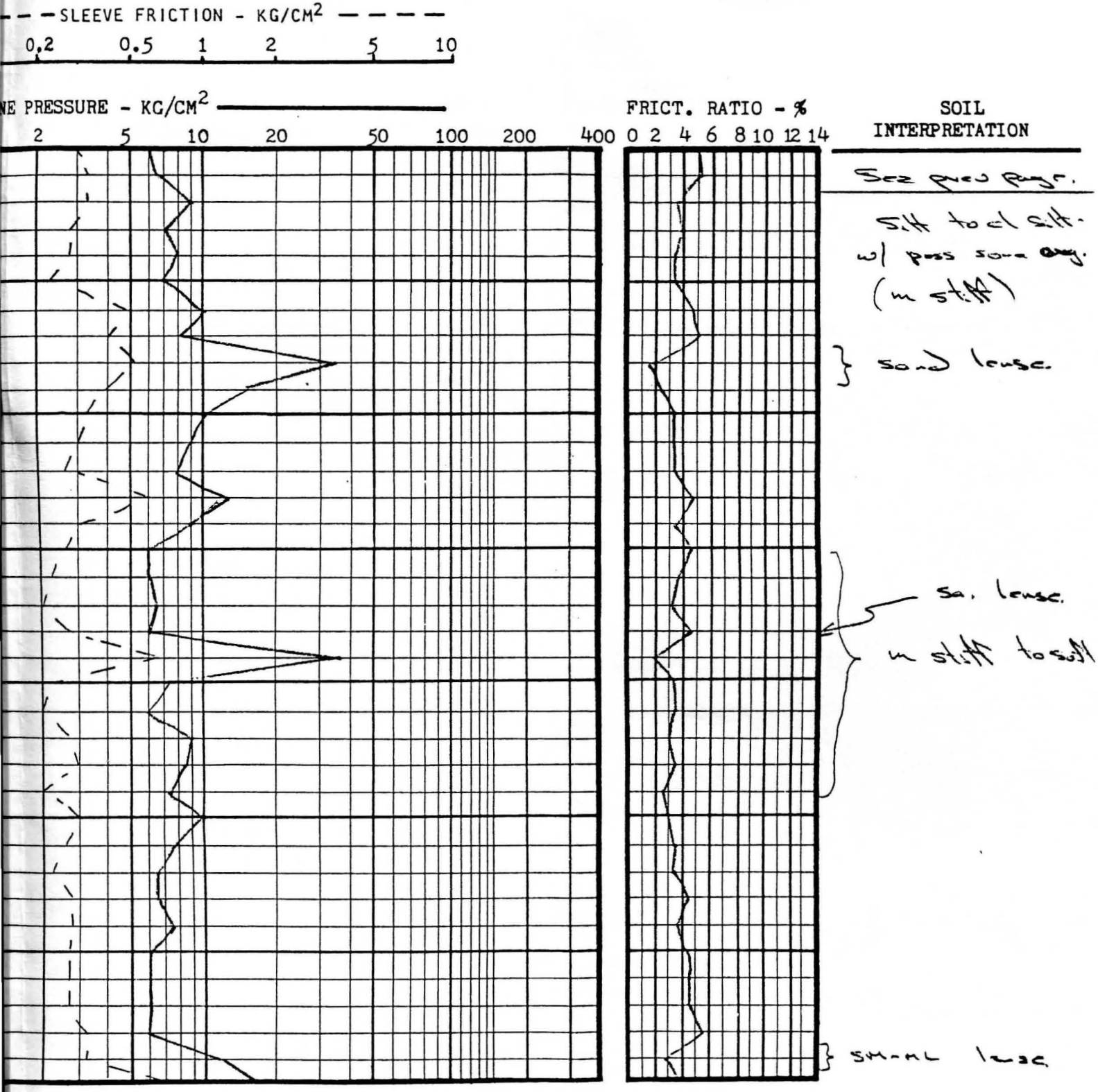
ALCULATION SHEET

rdeen, WA JOB NO _____ PROBE NO P-3 DATE 7-30-92 SHEET 2 of 3



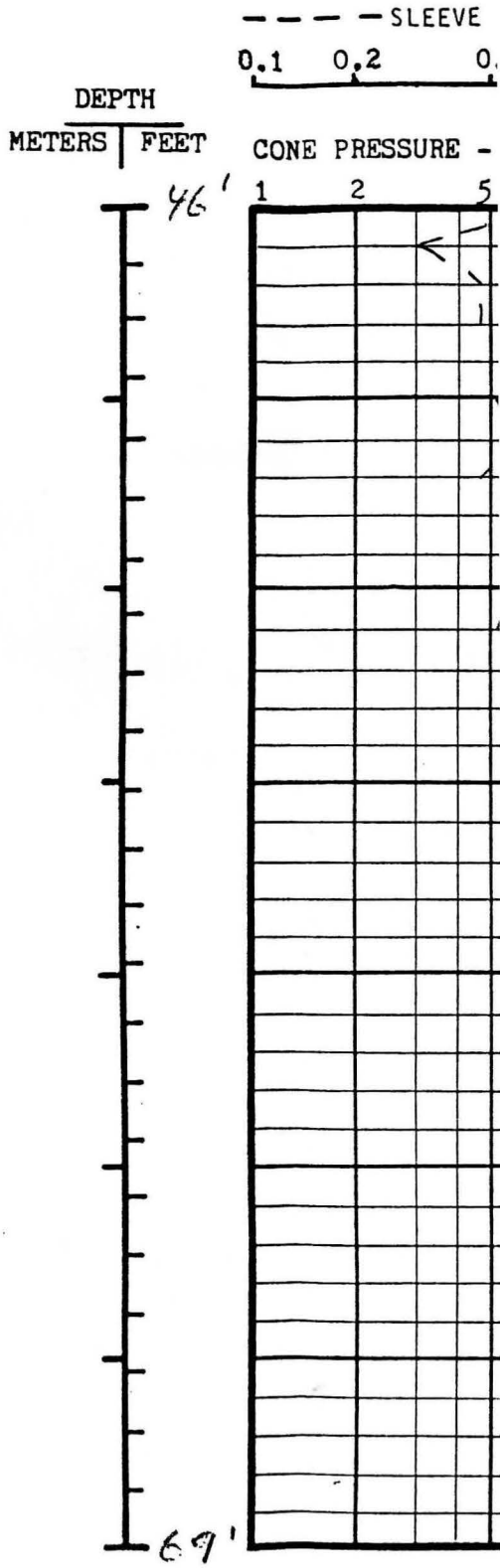
VANDEHEY SOIL EXPLORATIONS

RT. 2, BOX 25 • BANKS, OREGON • PHONE AREA CODE (503) 324-3261

DUTCH CONE LOG & CALCULAT

CLIENT Kettenhofer-Zeman LOCATION Aberdeen, WA

NO. RODS	DEPTH IN METERS	DIAL READINGS TOTAL SYSTEM	DIAL READINGS CONE ONLY	DIAL READINGS CONE + SLEEVE	SLEEVE READING —	CONE PRESS. KG/CM²	SLEEVE FRICT. KG/CM²	FRICT. RATIO %
	14.2	1200	60	160	100	9	.67	3.3
		1200	155	200	45	18½	.30	2.5
		1500	750	820	70	78	.47	0.6
		1800	570	640	70	60	.47	2.2
	15.0	1700	280	480	200	31	1.33	1.7
		1800	160	240	80	19	.53	3.2
		1600	220	310	90	25	.60	1.9
		1600	360	430	70	39	.47	1.5
		2000	760	850	90	79	.60	1.5
	16.0	1800	320	500	180	35	1.20	1.9
		1700	240	340	100	27	.67	2.0
		1900	270	300	80	25	.53	3.5
		1500	220	350	130	25	.87	2.4
		1700	980	370	90	31	.60	2.8
	17.0	2400	1200	1330	130	123	.87	0.8
		3200	1700	1850	150	173	1.00	1.2
		3400	1800	2100	300	187	2.00	0.8
		3500	1000	1200	200	103	1.33	1.4
		3200	480	680	220	49	1.47	1.1
	18.0	3200	350	430	80	38	.53	1.9
		2200	480	590	110	51	.73	2.0
		1600	270	380	150	25	1.00	2.4
		1600	410	500	90	44	.60	
	19.0							
	20.0							
	21.0							



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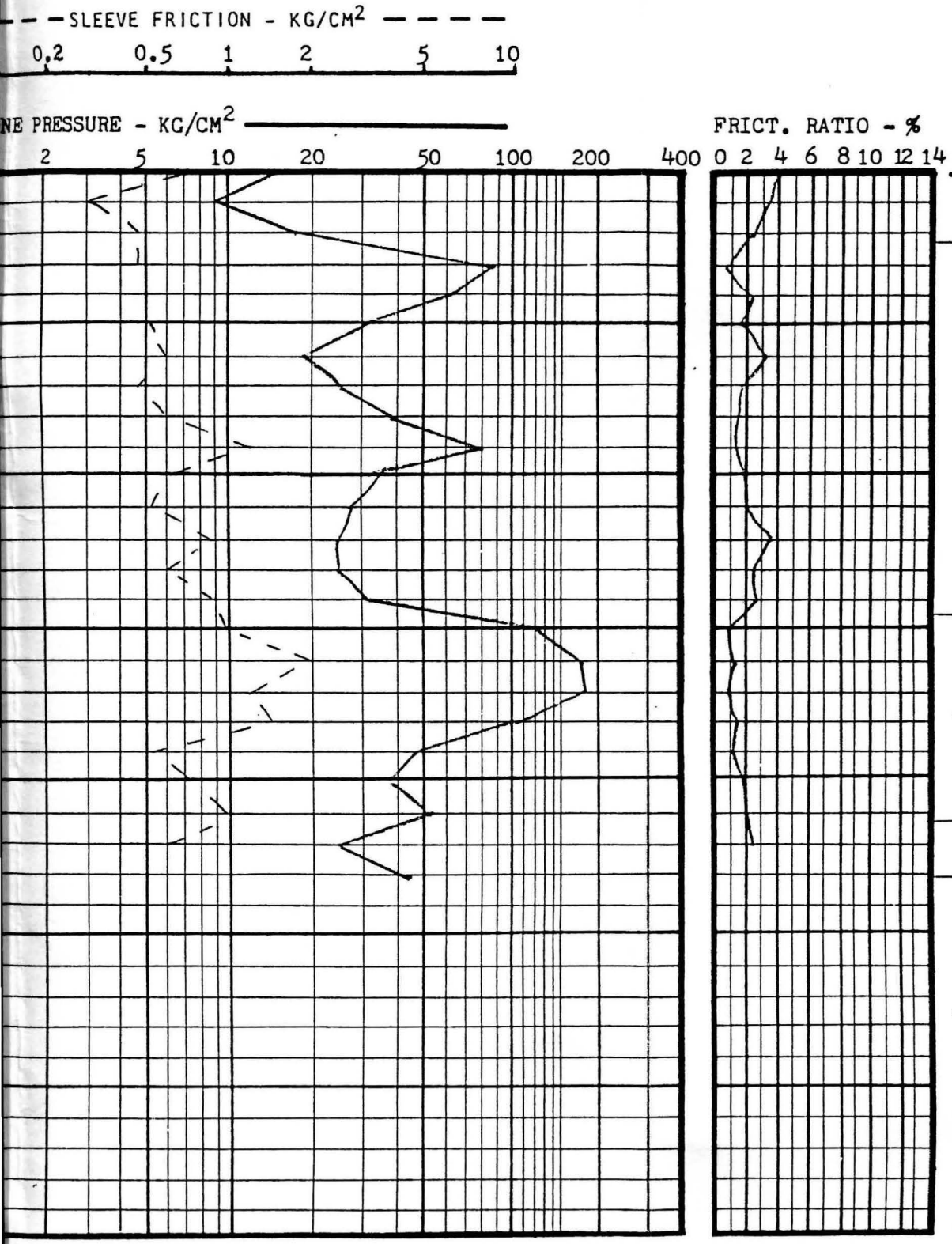
LCULATION SHEET

Keen, WA JOB NO _____ PROBE NO P-3 DATE 7-30-92 SHEET 3 of 3



SOIL INTERPRETATION

See prev page

Alternating layers of sand, silt & SM-ML. m dense & m stiff-stiff.

Sand. (dense.) grading to m dense.

Silty sand to sandy silt.

Terminated @ 61 ft.

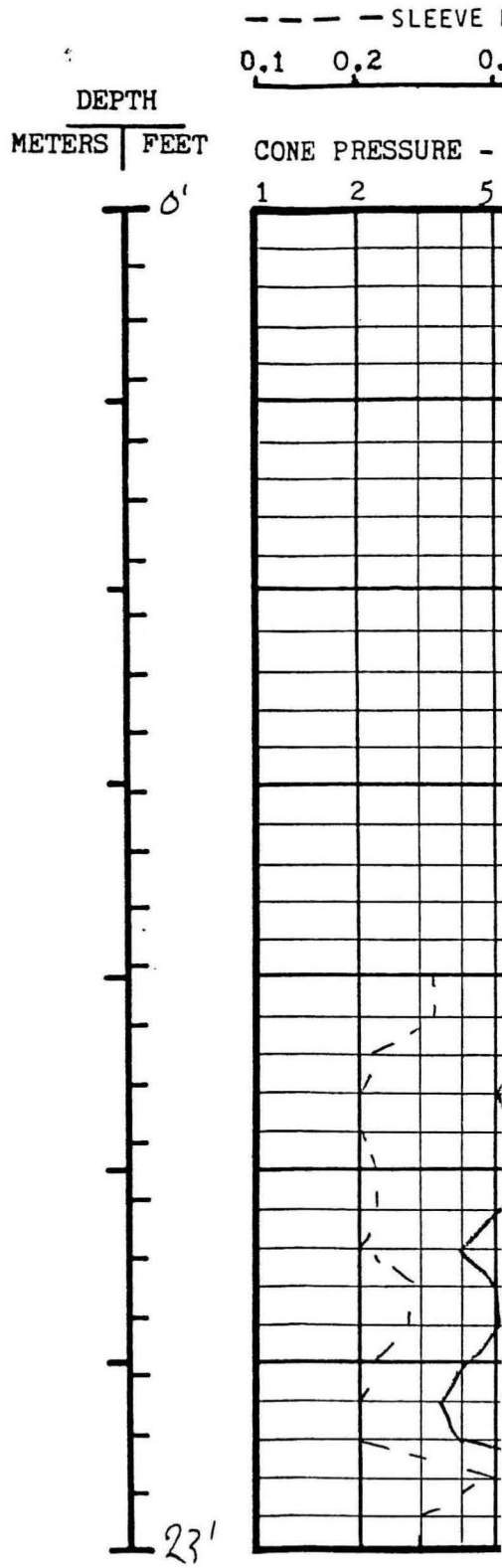
VANDEHEY SOIL EXPLORATIONS
RT. 2, BOX 25 • BANKS, OREGON • PHONE AREA CODE (503) 324-3261

DUTCH CONE LOG & CALCULAT

CLIENT Rittenhaso-Zeman LOCATION Aberdeen, WA

NO. RODS	DEPTH IN METERS	DIAL READINGS TOTAL SYSTEM	CONE ONLY	CONE + SLEEVE	SLEEVE READING —	CONE PRESS. KG/CM²	SLEEVE FRICT. KG/CM²	FRICT. RATIO %
	0.2							
		150						
		200	170	NR		18		0.7
	1.0	600	130	150	20	14	.13	0.9
		400	50	70	20	6	.13	
		600						
		650						
	2.0	340	70	NR		8		
		180						
		100						
		70	0					
		50						
	3.0	40						
		30						
		40						
		50						
		0						
	4.0	80	70	—				
		120	70	120	50	8	.33	4.1
		130	50	100	50	6	.33	3.8
		120	40	75	35	5	.23	4.0
		140	50	80	30	6	.20	3.3
	5.0	140	50	80	30	6	.20	3.8
		120	40	75	35	5	.23	4.6
		130	30	65	35	4	.23	5.0
		130	40	70	30	5	.20	5.4
		160	40	80	40	5	.27	5.4
	6.0	170	30	70	40	4	.27	5.8
		170	25	60	35	3½	.23	5.7
		160	30	60	30	4	.20	5.0
		220	110	140	30	12	.20	4.2
		240	65	140	75	7½	.50	4.0
	7.0	240	40	85	45	5	.30	6.0

NOTE: 1 KG/CM² = 1.024 TONS/FT.²



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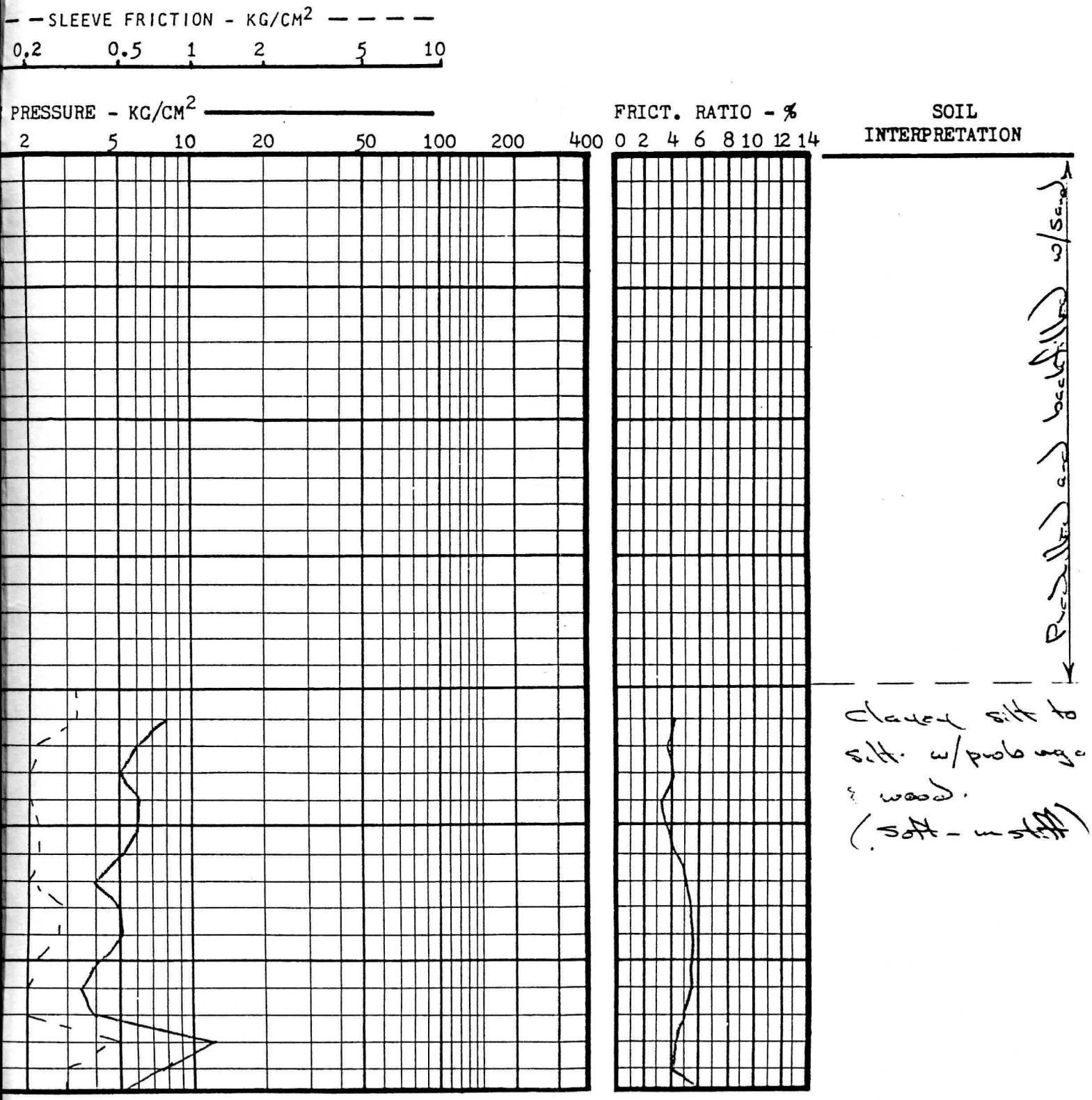
LCULATION SHEET

een, WA JOB NO _____ PROBE NO P-4 DATE 7-30-92 SHEET 1 of 3



VANDEHEY SOIL EXPLORATIONS

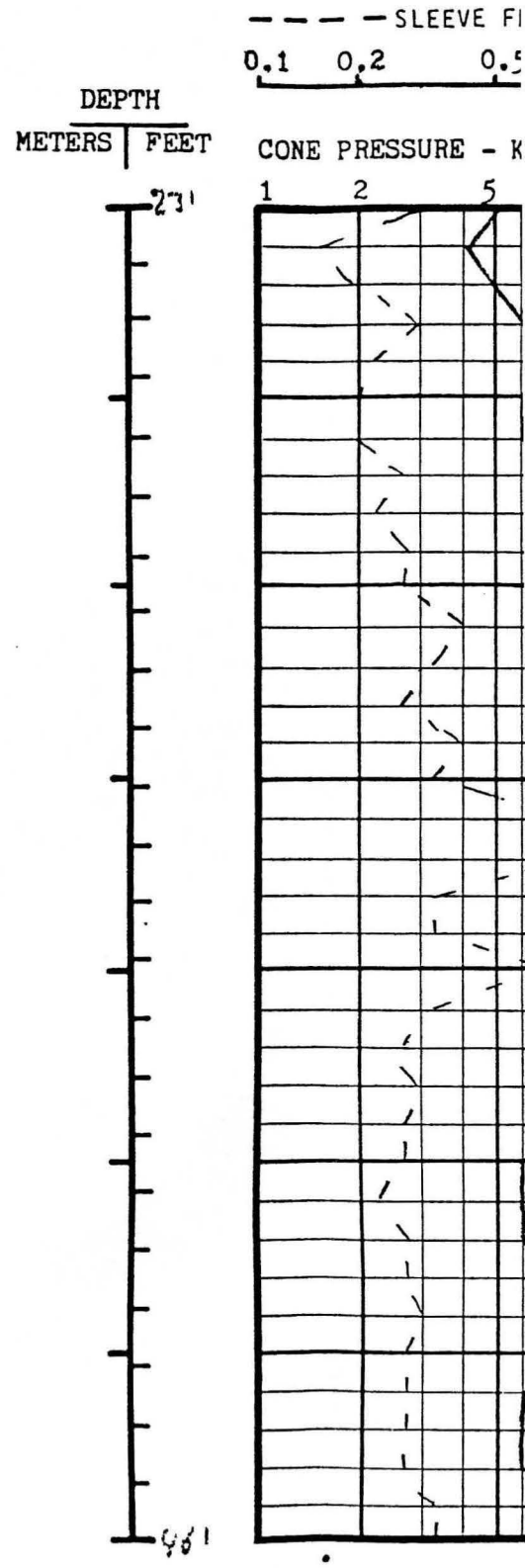
RT. 2, BOX 25 • BANKS, OREGON • PHONE AREA CODE (503) 324-3261

DUTCH CONE LOG & CALCULAT

CLIENT R. Hirshorn-Zeman LOCATION Aberdeen, WA JO

NO. RODS	DEPTH IN METERS	DIAL READINGS TOTAL SYSTEM	CONE ONLY	CONE + SLEEVE	SLEEVE READING —	CONE PRESS. KG/CM²	SLEEVE FRICT. KG/CM²	FRICT. RATIO %
	7.2	240	20	65	45	4	.30	4.3
		250	30	55	25	5	.17	4.0
		260	40	70	30	6	.20	5.0
		350	55	100	45	7½	.30	3.1
	8.0	350	40	75	35	6	.23	3.3
		360	40	70	30	6	.20	3.3
		380	50	80	30	7	.20	3.9
		410	50	90	40	7	.27	3.3
		460	50	85	35	7	.23	3.9
	9.0	480	50	90	40	7	.27	3.9
		520	60	100	40	8	.27	5.0
		530	70	130	60	9	.40	3.7
		560	70	120	50	9	.33	3.0
		660	80	120	40	10	.27	4.0
	10.0	700	80	140	60	10	.40	3.3
		800	390	440	50	41	.33	2.0
		1050	480	600	120	50	.80	1.6
		850	100	220	120	12	.80	2.5
		1000	130	180	50	15	.33	2.2
	11.0	850	300	350	50	32	.33	2.1
		640	60	160	100	8	.67	4.1
		670	50	100	50	7	.33	3.9
		760	50	90	40	7	.27	4.3
		640	55	100	45	7½	.30	3.6
	12.0	630	40	80	40	6	.27	4.5
		610	40	80	40	6	.27	3.8
		600	40	75	35	6	.23	4.5
		620	50	90	40	7	.27	3.9
		660	55	95	40	7½	.27	4.0
	13.0	670	55	100	45	7½	.30	3.6
		700	40	80	40	6	.27	4.5
		700	40	80	40	6	.27	4.5
		740	40	80	40	6	.27	4.5
		800	100	140	40	12	.27	2.8
	14.0	820	100	150	50	12	.33	2.8

NOTE: 1 KG/CM² = 1.024 TONS/FT.²



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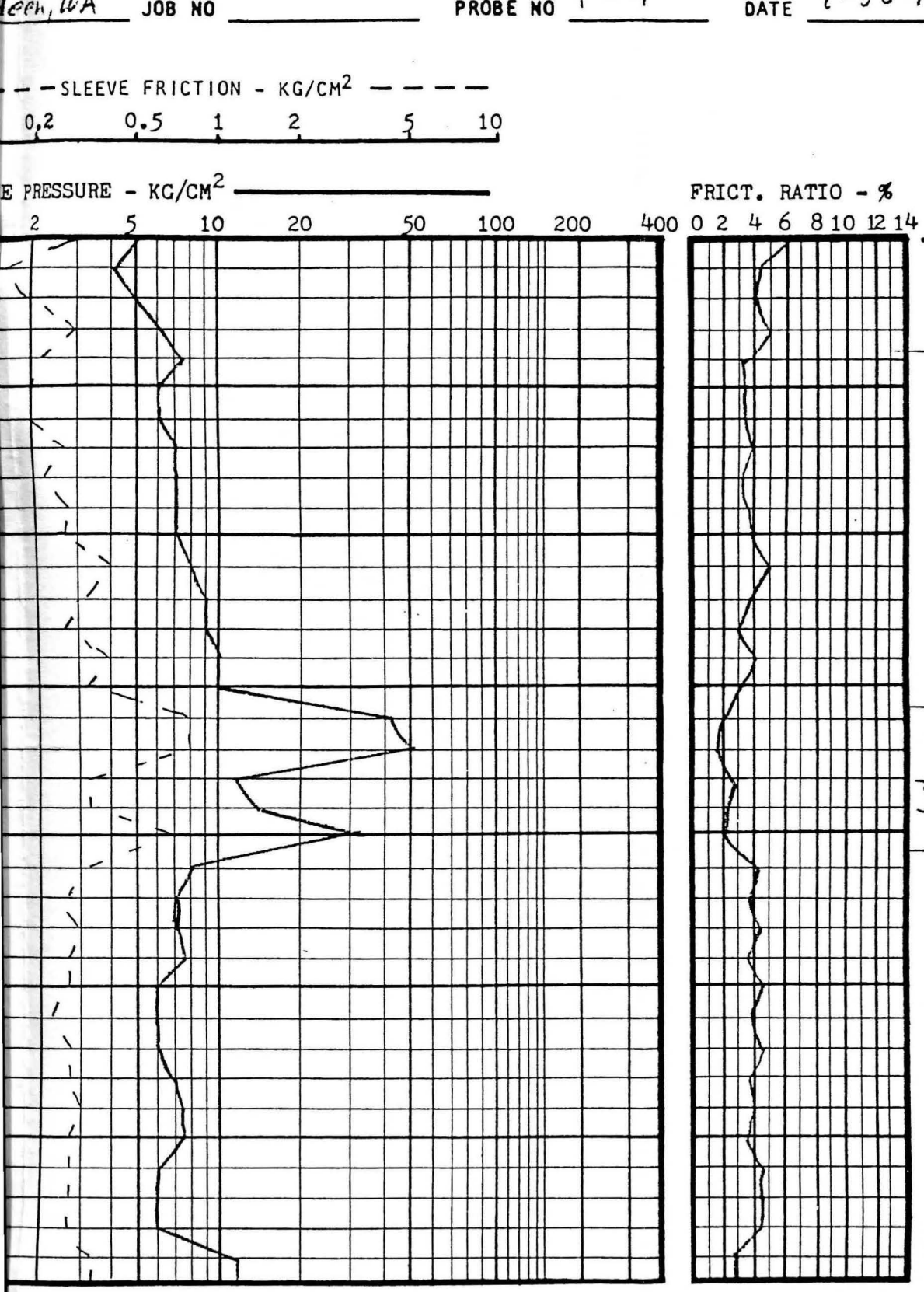
LCULATION SHEET

leen, WA JOB NO _____ PROBE NO P-4 DATE 7-30-92 SHEET 2 of 3



SOIL INTERPRETATION

See prev. page.

Silt to cl silt w/ poss some org's (soft - m stiff)

m stiff.

Sand to sand w/s silt (loose - m dense) } sm - ml.

Silt to cl silt. (m stiff - soft.)

m stiff.

VANDEHEY SOIL EXPLORATIONS
RT. 2, BOX 25 • BANKS, OREGON • PHONE AREA CODE (503) 324-3261

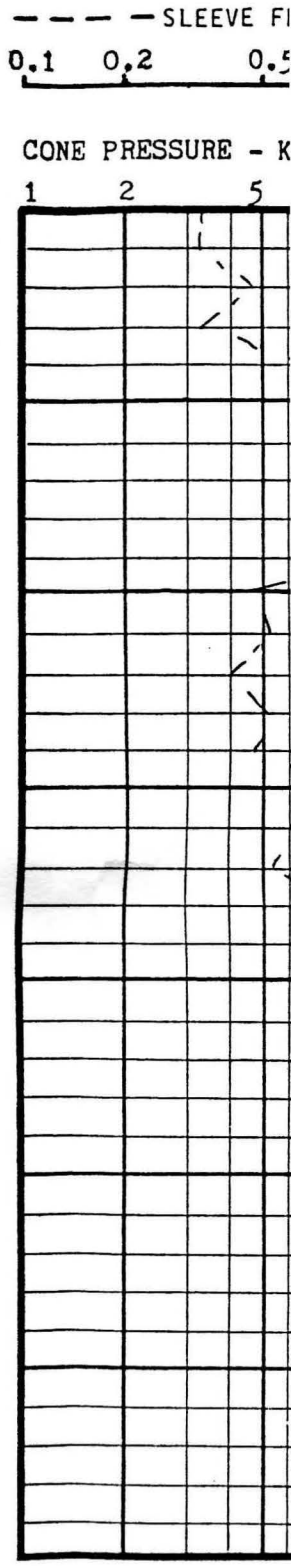
DUTCH CONE LOG & CALCULAT

CLIENT Rittenhouse Zeman LOCATION Aberdeen, WA JO

	NO. RODS	DEPTH IN METERS	DIAL READINGS TOTAL SYSTEM	CONE ONLY	CONE + SLEEVE	SLEEVE READING —	CONE PRESS. KG/CM²	SLEEVE FRICT. KG/CM²	FRICT. RATIO %
		14.2	820	45	95	50	7.2	.33	4.4
			860	50	100	50	8	.33	5.9
			880	50	120	70	8	.47	4.1
			1300	260	310	50	29	.33	2.3
		15.0	1400	420	520	100	45	.67	1.9
			2100	700	830	130	73	.87	1.8
			1700	800	1000	200	83	1.33	0.8
			2000	350	450	100	38	.67	2.1
			2000	620	740	120	65	.80	2.0
		16.0	1500	110	310	200	14	1.33	3.4
			1200	100	170	70	13	.47	4.1
			1200	80	160	80	11	.53	3.6
			820	100	160	60	13	.40	4.1
			1200	140	220	80	17	.53	2.8
		17.0	1200	360	430	70	39	.47	2.2
R			1200	350	480	130	38	.87	1.6
			1400	590	680	90	62	.60	0.9
			1600	520	600	80	55	.53	1.5
R			1600	520	640	120	55	.80	1.2
		18.0	2900	1800	1900	100	183	.67	0.5
			3300	1800	1950	150	183	1.00	0.7
→			2900	1000	1200	200	103	1.33	1.0
			3000	850	1000	150	88	1.00	
		19.0							
		20.0							
		21.0							

DEPTH METERS | FEET

46' … 69'



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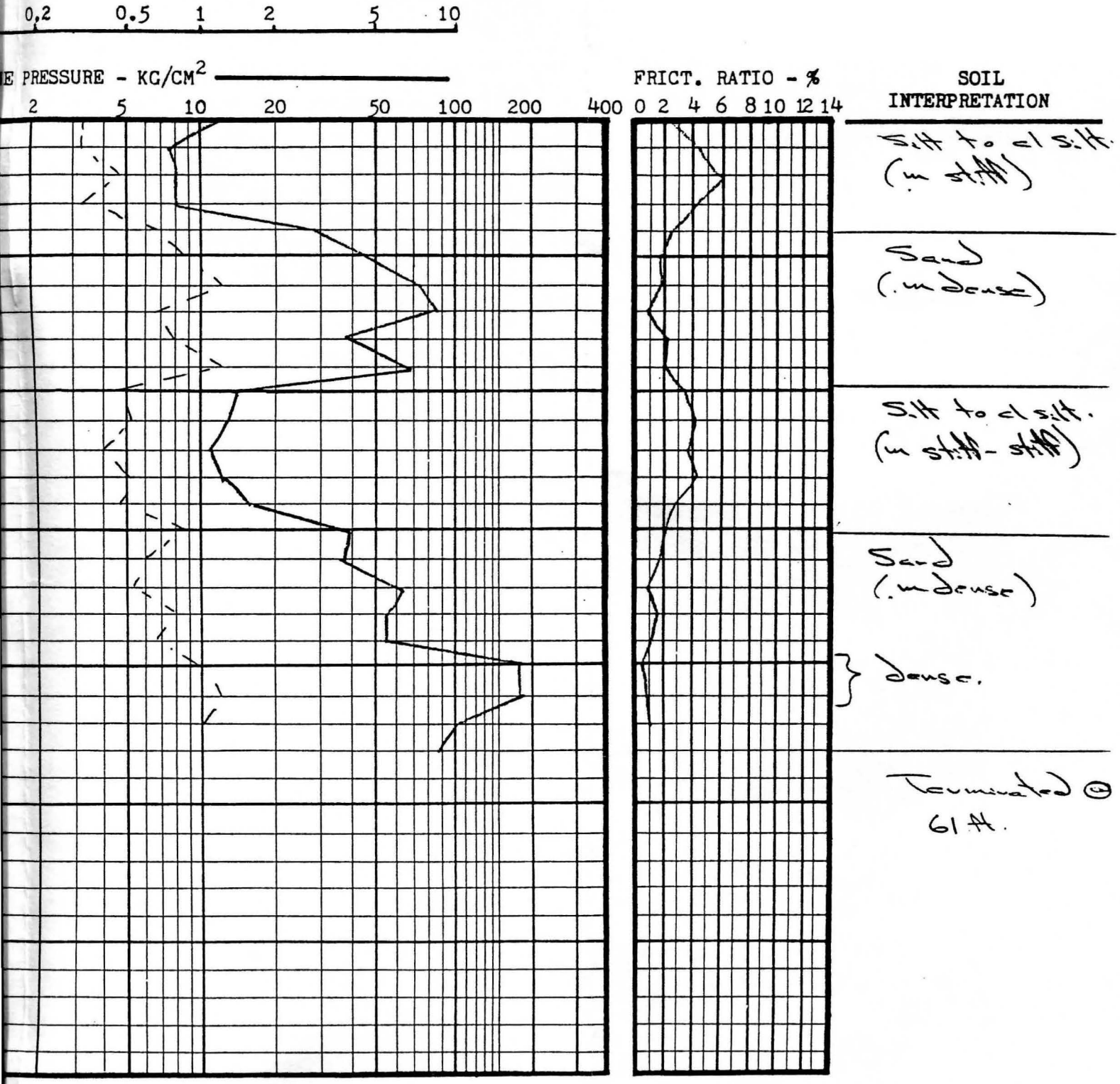
LCULATION SHEET

Aberdeen, WA JOB NO _____ PROBE NO P-4 DATE 10-4 SHEET 3 of 3

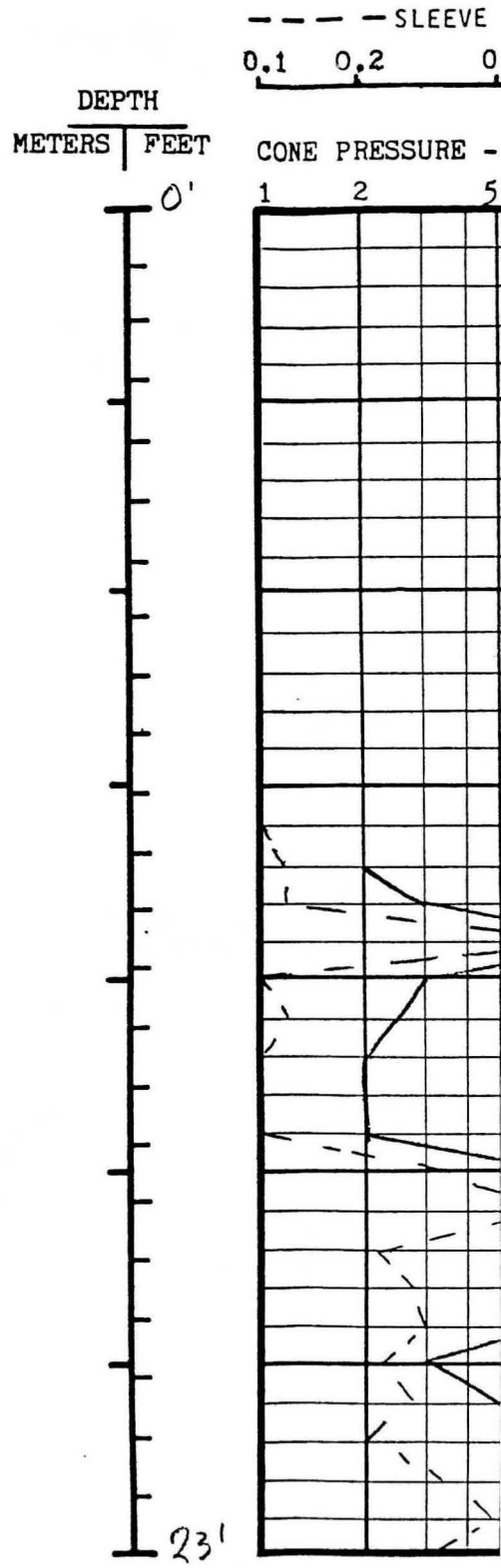


VANDEHEY SOIL EXPLORATIONS
RT. 2, BOX 25 • BANKS, OREGON • PHONE AREA CODE (503) 324-3261

DUTCH CONE LOG & CALCULAT

CLIENT Rittenhouse-Zeman LOCATION Aberdeen, WA

NO. RODS	DEPTH IN METERS	DIAL READINGS TOTAL SYSTEM	CONE ONLY	CONE + SLEEVE	SLEEVE READING —	CONE PRESS. KG/CM²	SLEEVE FRICT. KG/CM²	FRICT. RATIO %
	0.2							
	1.0							
	2.0							
		140	40	NR		5		
		100	0	0				
		140	0	0				
	3.0	40	20	NR		3		
		20	0	15				
		20	10	25	15	2	.10	6.5
		50	20	40	20	3	.13	4.3
R		120	100	120	20	11	.13	7.9
	4.0	80	20	150	130	3	.87	3.3
		90	15	30	15	2½	.10	5.2
		220	10	30	20	2	.13	5.0
		50	10	25	15	2	.10	3.5
		80	10	20	10	2	.07	5.0
	5.0	100	65	80	15	7½	.10	4.4
		340	180	230	50	14	.23	3.7
		220	55	160	105	6½	.70	3.5
		260	45	80	35	5½	.23	4.9
		300	60	100	40	7	.27	4.3
	6.0	200	20	65	45	3	.30	7.7
		180	40	75	35	5	.23	5.4
		200	40	80	40	5	.27	4.0
		200	50	80	30	6	.20	5.5
		280	130	180	50	14	.33	3.4
	7.0	280	50	120	70	6	.47	5.5



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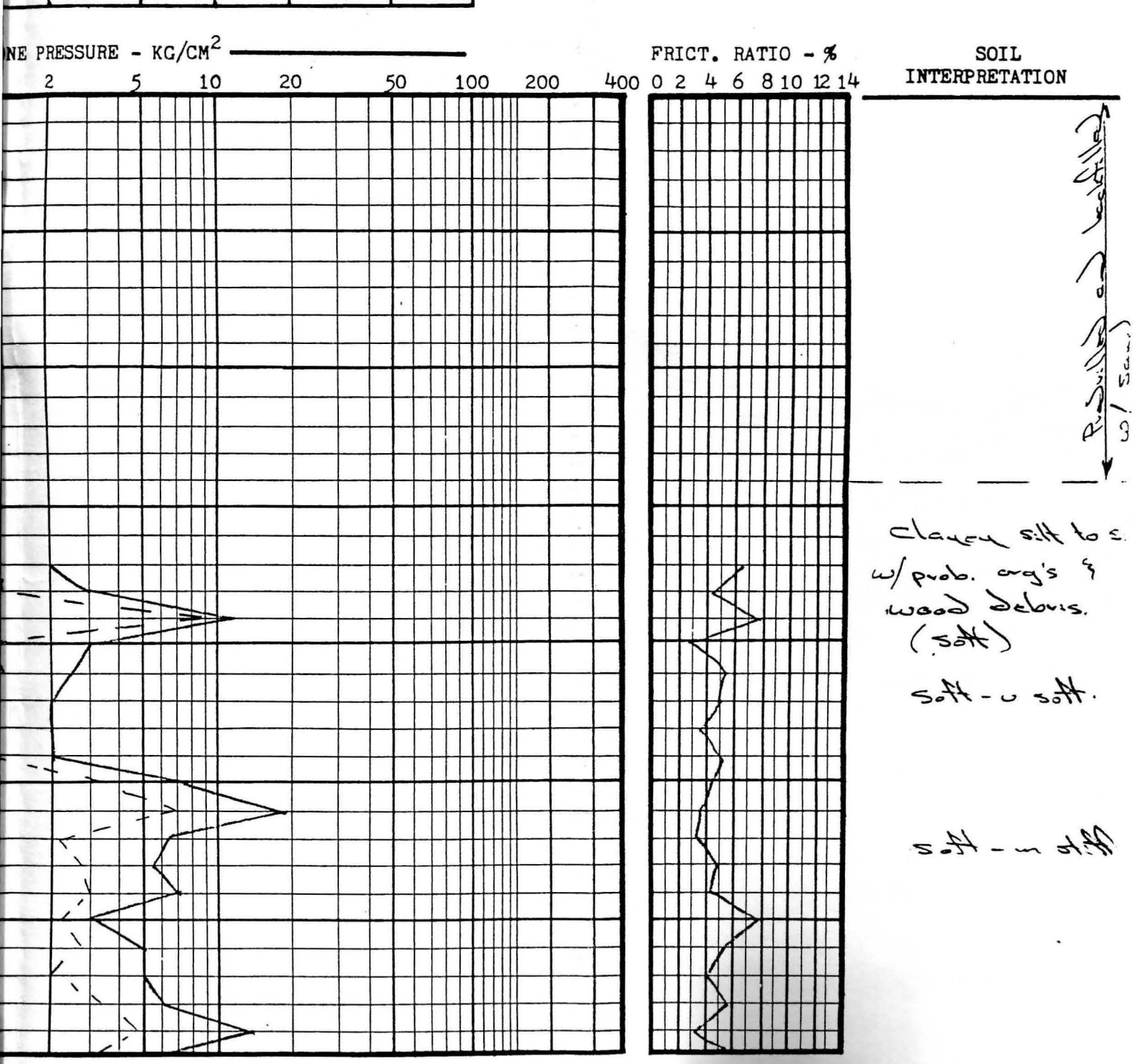
ALCULATION SHEET

deen, WA JOB NO _____ PROBE NO P-5 DATE 7-30-92 SHEET 1 of 3



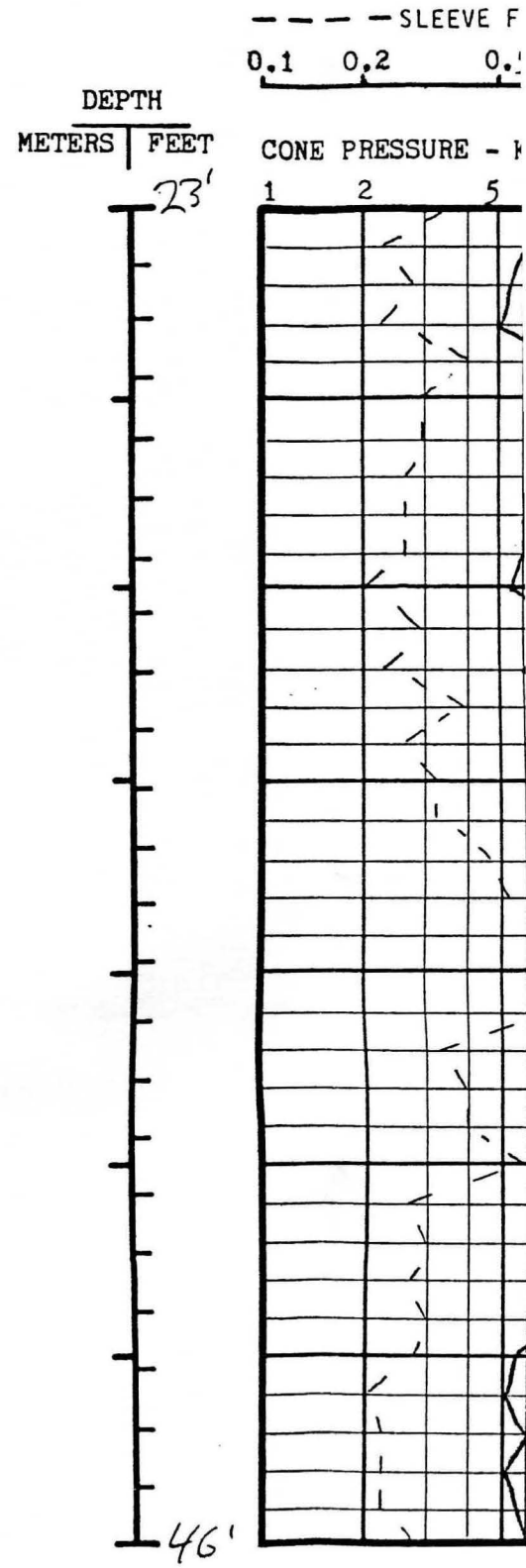
VANDEHEY SOIL EXPLORATIONS
RT. 2, BOX 25 • BANKS, OREGON • PHONE AREA CODE (503) 324-3261

DUTCH CONE LOG & CALCULAT

CLIENT Rittenhouse-Zeman LOCATION Aberdeen, WA J

NO. RODS	DEPTH IN METERS	DIAL READINGS TOTAL SYSTEM	CONE ONLY	CONE + SLEEVE	SLEEVE READING —	CONE PRESS. KG/CM²	SLEEVE FRICT. KG/CM²	FRICT. RATIO %
	7.2	270	40	90	50	6	.33	3.8
		270	35	70	35	5½	.23	4.9
		270	30	70	40	5	.27	4.6
		340	65	100	35	8½	.23	4.7
	8.0	360	60	120	60	8	.40	3.8
		400	50	95	45	7	.30	4.3
		400	40	85	45	6	.30	4.5
		420	40	80	40	6	.27	4.5
		460	40	80	40	6	.27	4.5
	9.0	460	35	75	40	5½	.27	3.6
		520	60	90	30	8	.20	3.8
		520	40	85	45	6	.30	3.8
		550	60	95	35	8	.23	5.0
		900	180	240	60	20	.40	1.4
	10.0	900	240	280	40	26	.27	1.3
		950	370	420	50	34	.33	0.8
		1050	300	350	50	32	.33	1.5
		1050	320	390	70	34	.47	1.6
		1050	180	260	80	20	.53	4.0
	11.0	1050	160	280	120	18	.80	3.3
		1200	170	260	90	19	.60	3.8
		900	60	170	110	8	.73	4.1
		900	100	150	50	12	.33	3.3
		820	220	980	60	24	.40	6.7
	12.0	1900	80	140	60	10	.40	6.0
		800	50	140	90	7	.60	3.9
		800	50	90	40	7	.27	4.3
		840	50	95	45	7	.30	3.9
		800	60	100	40	8	.27	3.8
	13.0	840	35	50	45	5½	.30	4.9
		840	30	70	40	5	.27	4.0
		750	40	70	30	6	.20	3.8
		840	30	65	35	5	.23	4.6
		860	35	70	35	5½	.23	4.2
	14.0	860	40	75	35	6	.23	4.5

NOTE: 1 KG/CM² = 1.024 TONS/FT.²



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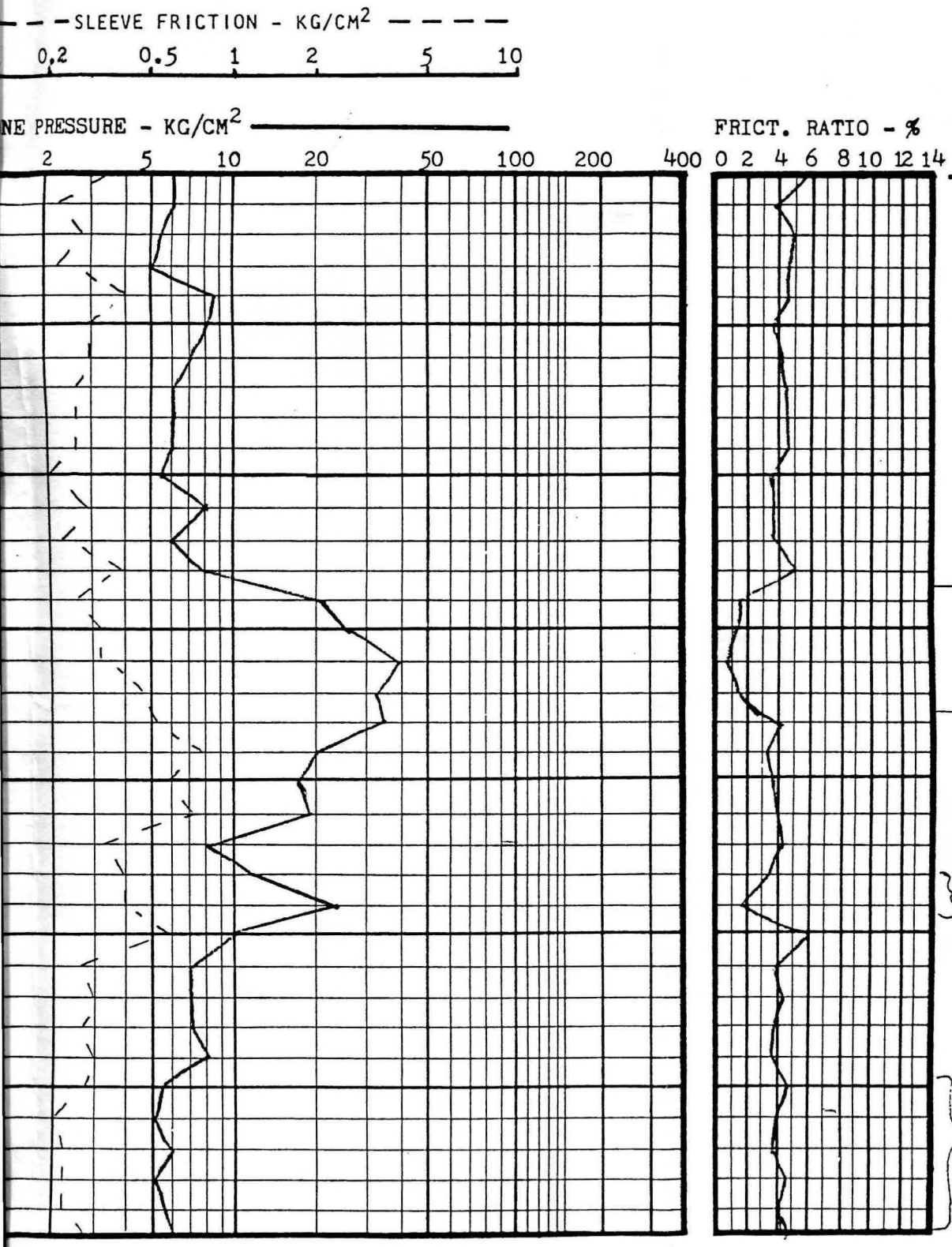
ALCULATION SHEET

dam, WA JOB NO _____ PROBE NO P-5 DATE 7-30-92 SHEET 2 of 3



SOIL INTERPRETATION

Clayey silt to silt w/ prob organics. (m stiff - soft)

Sand (loose - m dense

Silt to cl silt. (m stiff)

} Sand layer.

} soft - m stiff

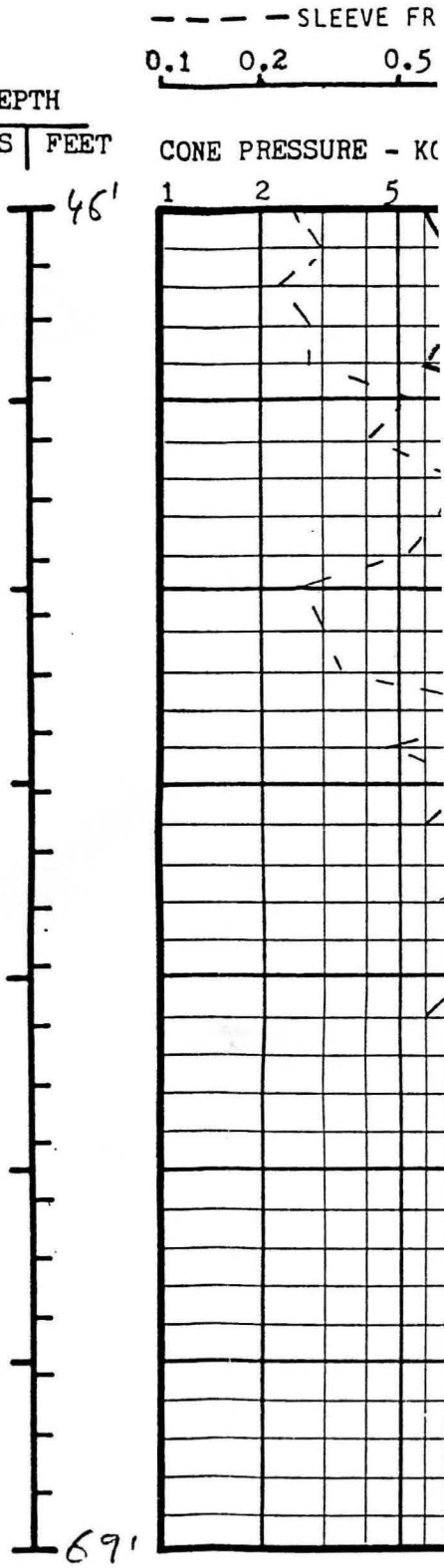
VANDEHEY SOIL EXPLORATIONS

RT 2, BOX 25 • BANKS, OREGON • PHONE AREA CODE (503) 324-3261

DUTCH CONE LOG & CALCULATI

CLIENT Rettenhouse-Lewis LOCATION Aberdeen, WA JO

NO. RODS	DEPTH IN METERS	DIAL READINGS TOTAL SYSTEM	CONE ONLY	CONE + SLEEVE	SLEEVE READING —	CONE PRESS. KG/CM²	SLEEVE FRICT. KG/CM²	FRICT. RATIO %
	14.2	860	40	80	40	7	.27	4.3
		900	40	85	45	7	.30	3.3
		950	40	75	35	7	.23	3.9
		900	30	70	40	6	.27	4.5
	15.0	900	100	140	40	13	.27	4.1
		900	90	160	80	11	.53	3.6
R		1000	250	310	60	28	.40	2.6
		1100	120	230	110	15	.73	4.5
		1200	160	260	100	19	.67	2.8
	16.0	1250	40	120	80	7	.53	3.9
		1050	40	80	40	7	.27	
S		1050	50	XR		53		0.6
		2200	1100	1150	50	113	.33	1.1
		1900	750	940	190	78	1.27	0.6
	17.0	2300	650	720	70	68	.47	1.2
		2000	380	500	120	41	.80	1.5
		2000	400	490	90	43	.60	2.6
		1600	200	370	170	23	1.13	2.9
		1500	220	320	100	25	.67	2.9
	18.0	1400	300	410	110	33	.73	2.4
		1200	170	290	120	26	.80	3.0
		1200	210	300	90	24	.60	
	19.0							
	20.0							
	21.0							

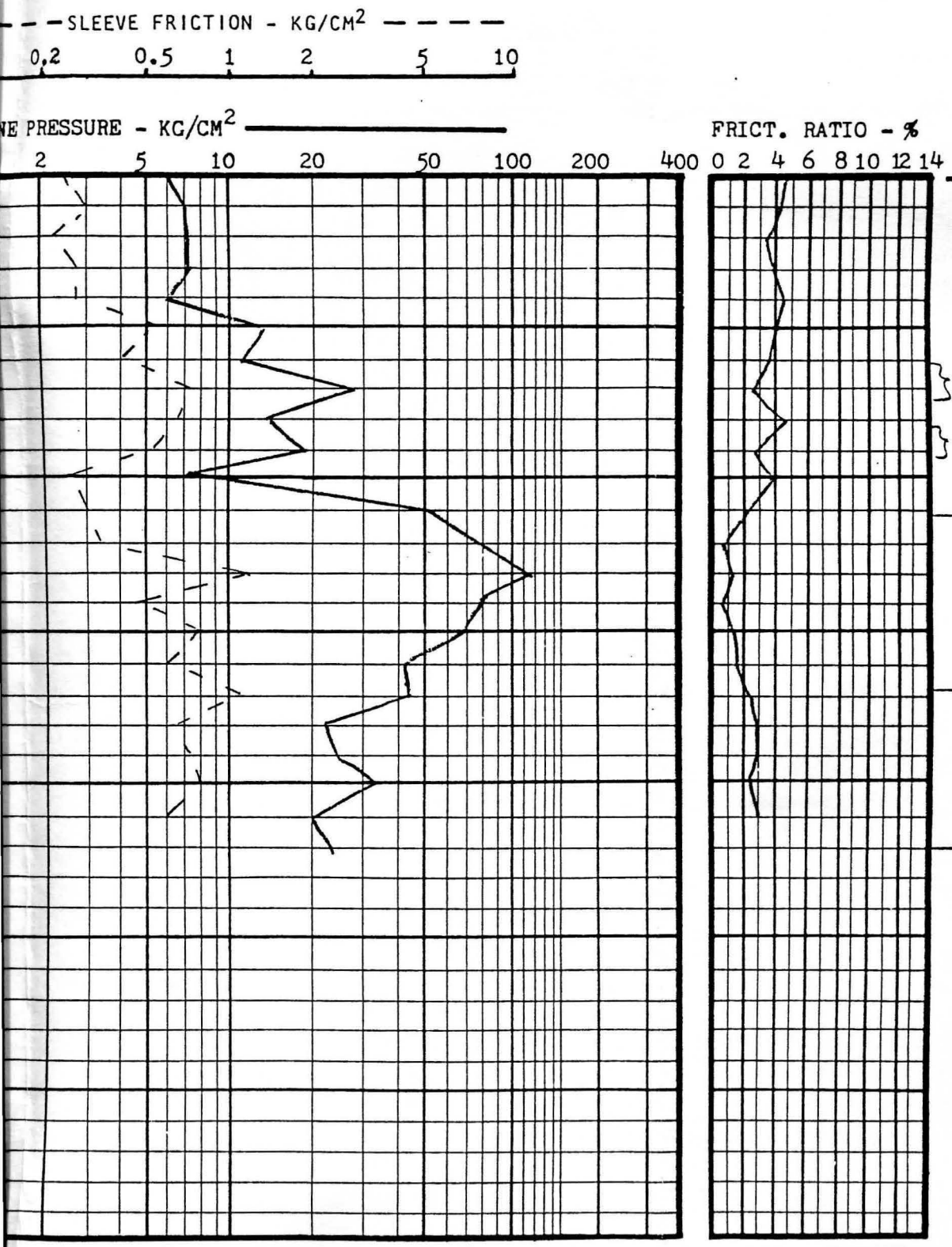


NOTE: 1 KG/CM² = 1.024 TONS/FT.²

Vandehey Soil Explorations is in the business of soil exploration and test drilling, and is not a soils consulting firm. The services performed by Vandehey Soil Explorations consist of soil exploration and obtaining soil test data by use of the "Dutch Cone Penetrometer" and conventional test drilling and Vandehey Soil Explorations warrants only that its soil exploration and testing equipment is in proper and good operating order, that proper operating procedures for use of the "Dutch Cone Penetrometer" and test drilling equipment are implemented in the use of said machines, and that data readings are accurately reported. No reliance should be placed on soil interpretive conclusions arrived at by Spencer Vandehey and Vandehey Soil Explorations but rather, all data reported should be reviewed and interpreted by qualified soil engineers.

ALCULATION SHEET

ern. WA JOB NO _____ PROBE NO P-5 DATE 7-30-92 SHEET 3 of 3



SOIL INTERPRETATION

- silt to cl silt. (m stiff - soft)
- m stiff
- SM-ML.
- ML-SM.
- Sand (m dense)
- Silt to sa. silt. (stiff)
- Terminated @ 60½'

VANDEHEY SOIL EXPLORATIONS

RT 2, BOX 25 • BANKS, OREGON • PHONE AREA CODE (503) 324-3261

TEST PIT LOGS

Depth (feet)	Soil Classification	W-8401
	Test Pit TP-1	
	Approximate ground surface elevation: 29 feet	
0.0 - 2.0	Medium dense to dense, damp, brown, sandy GRAVEL (Fill)	
2.0 - 6.0	Wood debris, bark, wood scraps, 1 to 2-foot diameter logs and other wood debris (Fill)	
	Refusal due to logs at 6.0 feet.	
	No caving observed	
	No seepage observed	
	Test Pit TP-2	
	Approximate ground surface elevation: 11.9 feet	
0.0 - 1.5	Medium dense, damp, light-brown, sandy SILT (Fill).	
1.5 - 2.5	Soft, wet, black organic SILT (Fill).	
2.5 - 3.5	Sawdust fill.	
3.5 - 10.0	Soft to very soft, wet to saturated, gray SILT with some organics and sand.	
	Small logs and branches from 5 to 7 feet.	
	Test pit terminated at 10.0 feet	
	Minor caving at 5 to 7 feet.	
	Slight seepage at 7 feet.	
	Test Pit TP-3	
	Approximate ground surface elevation: 13.5 feet	
0.0 - 1.5	Medium dense, moist, sandy GRAVEL (crushed rock fill).	
1.5 - 2.5	Soft, wet, black, organic SILT (Fill)	
2.5 - 4.5	Loose, saturated, blue-gray, fine SAND (Fill)	
	Piece of a brick found at 3 feet.	
4.5 - 7.0	Sawdust fill.	
7.0 - 10.0	Soft, saturated, gray, SILT with some organic material.	
	Test pit terminated at 10.0 feet	
	Slight caving at 4 feet.	
	Moderate seepage at 4 feet.	

Depth (feet)	Soil Classification
	Test Pit TP-4
	Approximate ground surface elevation: 12.5 feet
0.0 - 4.0	Medium dense, wet, brown and black, silty SAND with some gravel (fill).
4.0 - 5.0	Soft, wet to saturated, gray SILT with some organics (Fill)
5.0 - 6.0	Sawdust and some boards (Fill)
6.0 - 8.0	Soft, saturated, gray, SILT with some organic material.
	Test pit terminated at 8.0 feet
	No caving observed
	No seepage observed
	Test Pit TP-5
	Approximate ground surface elevation: 13.0 feet
0.0 - 0.5	Crushed rock (Fill)
0.5 - 2.0	Medium dense, moist to wet, black shiny, gravelly SAND with some silt (fill).
2.0 - 4.0	Wet sawdust fill.
4.0 - 7.0	Very soft, saturated, gray, SILT with some organic material.
	Test pit terminated at 7.0 feet
	No caving observed
	No seepage observed
	Test Pit TP-6
	Approximate ground surface elevation: 12.5 feet
0.0 - 0.5	Medium dense, dry, light-brown, silty SAND (Fill).
0.5 - 2.0	Medium dense, moist, gray, sandy GRAVEL (Fill)
	Railroad tie at 1.5 feet.
2.0 - 3.0	Soft, wet, black organic SILT (Fill)
3.0 - 4.0	Sawdust and wood debris (Fill)
4.0 - 8.0	Very soft, wet to saturated, gray, SILT with some organic material.
	Test pit terminated at 8.0 feet
	No caving observed
	No seepage observed

Depth (feet)	Soil Classification
	Test Pit TP-7
	Approximate ground surface elevation: 13.0 feet
0.0 - 2.0	Dense, damp, brown, sandy GRAVEL (fill).
2.0 - 4.3	Logs and wood debris (Fill)
	Steel cable at 3 feet.
	Refusal at 4.5 feet due to logs and cable.
	No caving observed
	No seepage observed
	Test Pit TP-8
	Approximate ground surface elevation: 13.0 feet
0.0 - 2.0	Dense, damp, brown, sandy GRAVEL (Fill)
2.0 - 6.0	Medium dense, moist to wet, dark gray, debris (bricks, metal, concrete) with some gravelly SAND (Fill)
	Wood debris observed from 4 to 6 feet.
6.0 - 8.0	Soft, wet, gray, SILT with some organics.
	Test pit terminated at 8.0 feet
	No caving observed
	No seepage observed

Date excavated: 31 July 1992 Logged by: Todd D. Wentworth

APPENDIX B

LABORATORY TEST PROCEDURES AND RESULTS

APPENDIX B

W-8401

LABORATORY TESTING PROCEDURES

A series of laboratory tests were performed during the course of this study to evaluate the index and geotechnical engineering properties of the subsurface soils. Descriptions of the types of test performed are given below.

Visual Classification

Samples recovered from the exploration locations were visually classified in the field during the exploration program. Representative portions of the samples were carefully packaged in watertight containers and transported to our laboratory where the field classifications were verified or modified as required. Visual classification was done in general accordance with the Unified Soil Classification system. Visual soil classification includes color, relative moisture content, soil type based on grain size, and accessory soil types included in the sample. Soil classifications are presented on the exploration logs in Appendix A.

Moisture Content Determinations

Moisture content determinations were performed on representative samples obtained from the explorations in order to aid in identification and correlation of soil types. The determinations were made in general accordance with the test procedures described in ASTM:D 2216. The results of the tests are shown on the exploration logs in Appendix A.

Atterberg Limits

Atterberg limits are used primarily for classification and indexing of cohesive soils. The liquid and plastic limits are tow of five Atterberg limits and are defined as the moisture content of a cohesive soil at arbitrarily established limits for liquid and plastic behavior, respectively. Liquid and plastic limits were established for selected samples in a general accordance with ASTM:D 423 and ASTM:D 424, respectively. The results of the Atterberg Limits are presented on a plasticity chart in this appendix where the plastic index (liquid limit minus plastic limit) is related to the liquid limit. The plastic limits and liquid limits are also presented adjacent to the appropriate samples on the exploration logs in Appendix A.

Grain Size Analysis

A grain size analysis indicates the range in diameter of soil particles included in a particular sample. Grain size analyses were performed on representative samples in general accordance with ASTM:D 422. The results of the grain size determinations for the samples were used in classification of the soils, are presented in this appendix.

Consolidation Test

A one-dimensional consolidation test was performed in general accordance with ASTM:D 2435 on a selected sample of the site soils to provide data for developing settlement estimates. The undisturbed soil sample was carefully trimmed and fit into a rigid ring. Porous stones were placed on both the top and bottom of the sample to allow drainage. Vertical loads were then applied to the sample incrementally in such a way that the sample was allowed to consolidate under each load increment. The rebound of the sample during unloading was also measured. The results of the consolidation test are presented in this appendix as a plot of percent consolidation (strain) versus the log of applied load (stress).

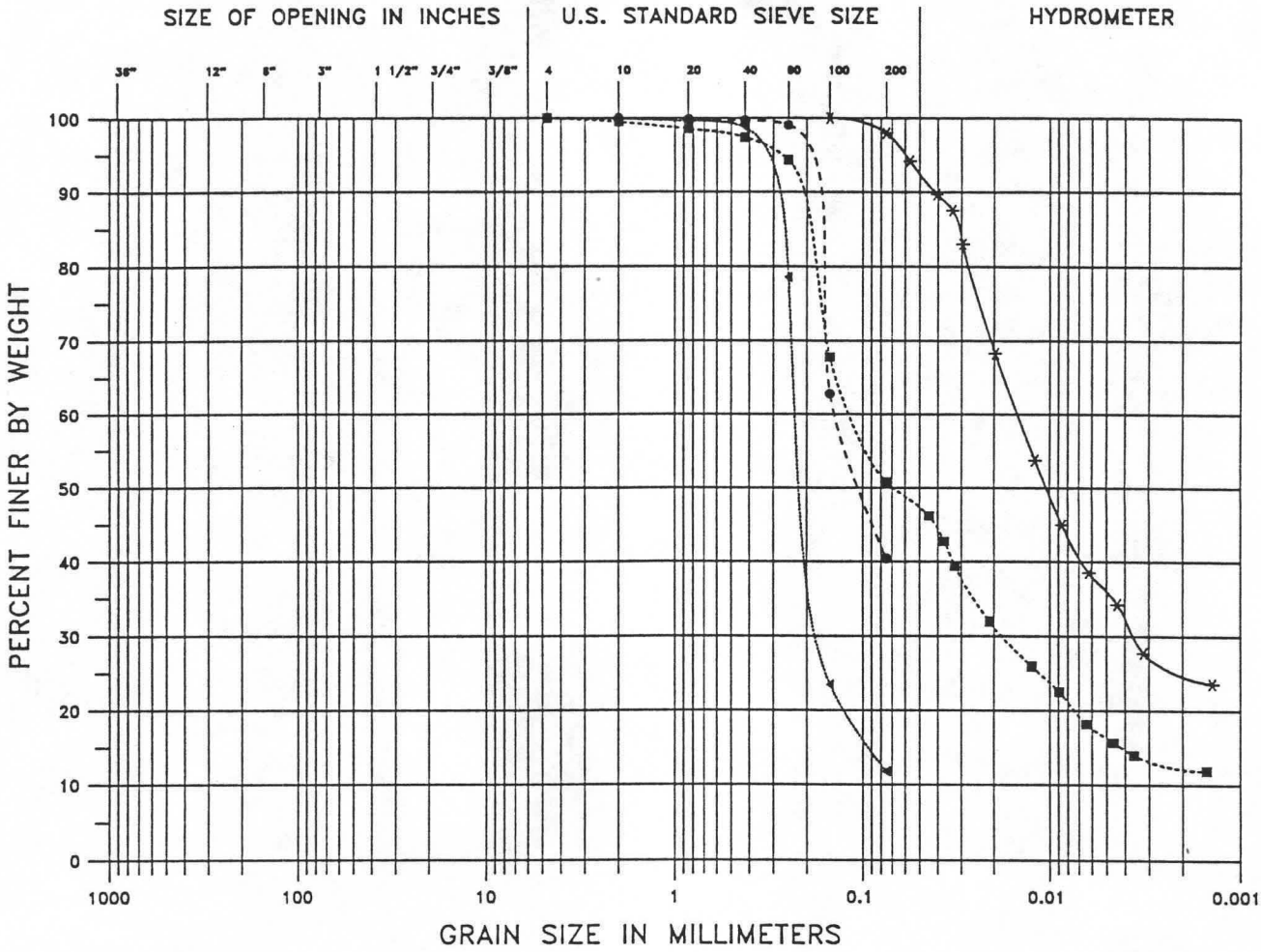
California Bearing Ratio Tests

A California Bearing Ratio test was performed on a composite sample of the site soils in general accordance with ASTM:D 1883-73, to provided an evaluation of the relative quality and support characteristics of subgrade soils. Representative portions from the sample were compacted in a mold, in general accordance with ASTM:D 1557-78 to provided a moisture-density relationship curve. Following compaction, a 15-pound surcharge was applied to each sample which was then totally immersed in water and allowed to soak for a period of 72 to 96 hours, during which time it was monitored for swell. At the end of this period the sample was removed, drained and a vertical load applied to the surcharged soil with a penetration piston at a constant rate of strain. Measurements of the applied vertical load were obtained at selected penetration depths. CBR test results and moisture-density relationships plotted in terms of percent water content versus percent corrected CBR and dry density are presented in this appendix.

Soil Chemical Analytical Tests

For use in determining corrosion potential of the site soils, we submitted soil samples for pH and resistivity tests. Samples were submitted to Northwest Agricultural Consultants in Kennewick, Washington. Samples from borings were submitted for pH testing while samples from borings were submitted for resistivity testing. The results are presented at the end of Appendix B.

GRAIN SIZE DISTRIBUTION



		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
BOULDERS	COBBLES	GRAVEL		SAND			FINE GRAINED	

	Exploration	Sample Number	Depth	Moisture	Fines	Soil Description
✕—✕—✕—✕—✕	B-6	S-4	18'	74%	98%	
●--●--●--●--●	B-6	S-6	30'	36%	40%	
■---■---■---■---■	B-6	S-9	45'	39%	51%	
◄—◄—◄—◄—◄	B-6	S-10	50'	32%	12%	

Project: Proposed Retail Store
Work Order: W-8401
Date: 8/14/92

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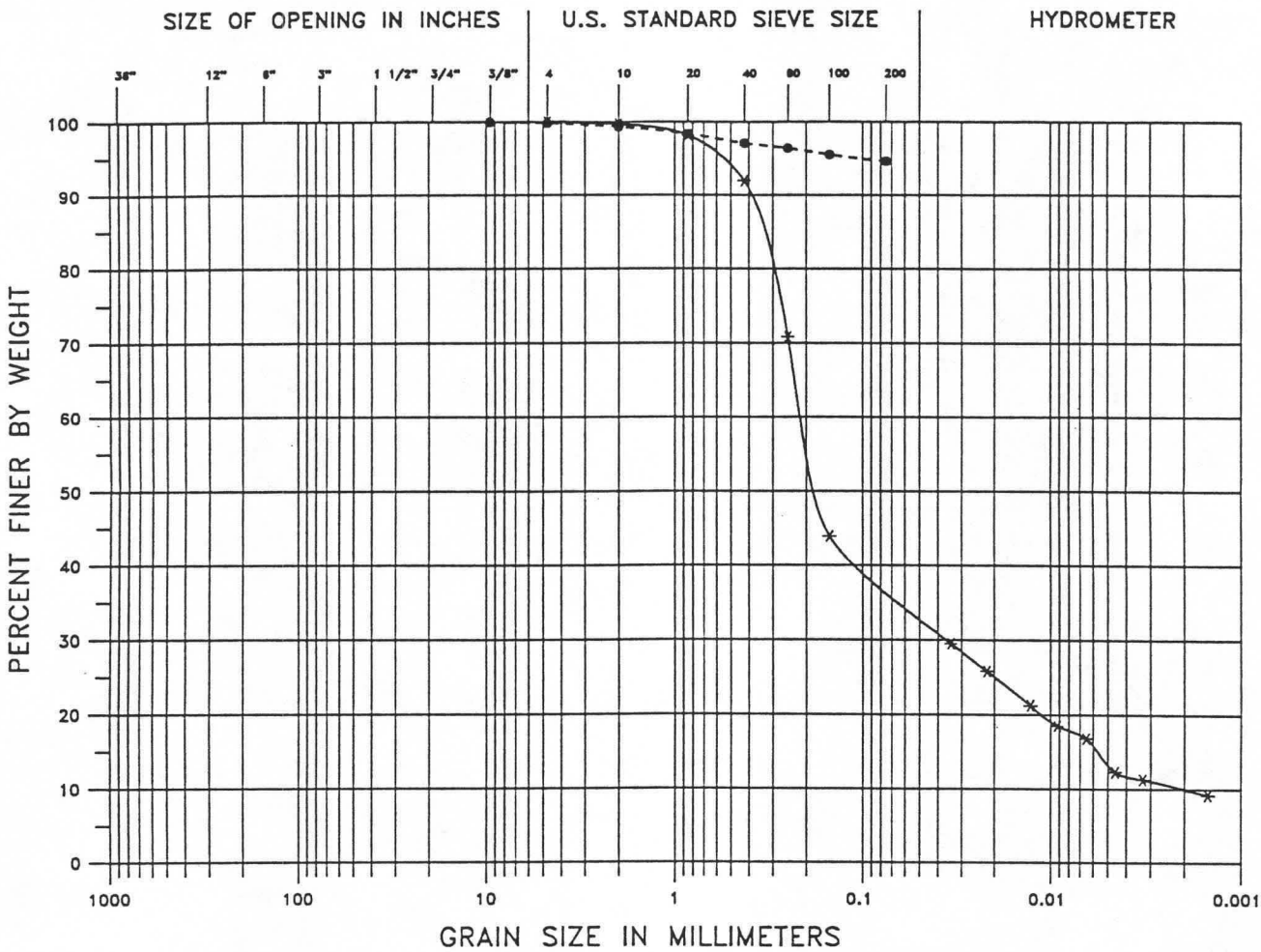
ENGINEERING & ENVIRONMENTAL SERVICES
11335 N.E. 122nd Way
Suite 100
Kirkland, Washington 98034-6918

GRAIN SIZE DISTRIBUTION



		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
BOULDERS	COBBLES	GRAVEL		SAND			FINE GRAINED	

	Exploration	Sample Number	Depth	Moisture	Fines	Soil Description
×—×—×—×—×	B-6	S-12	63'	38%	41%	
●- - ●- - ●- - ●	TP-2	S-2	5'	126%	95%	
■---■---■---■						
◄—◄—◄—◄						

Project: Proposed Retail Store
Work Order: W-8401
Date: 8/14/92

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11335 N.E. 122nd Way
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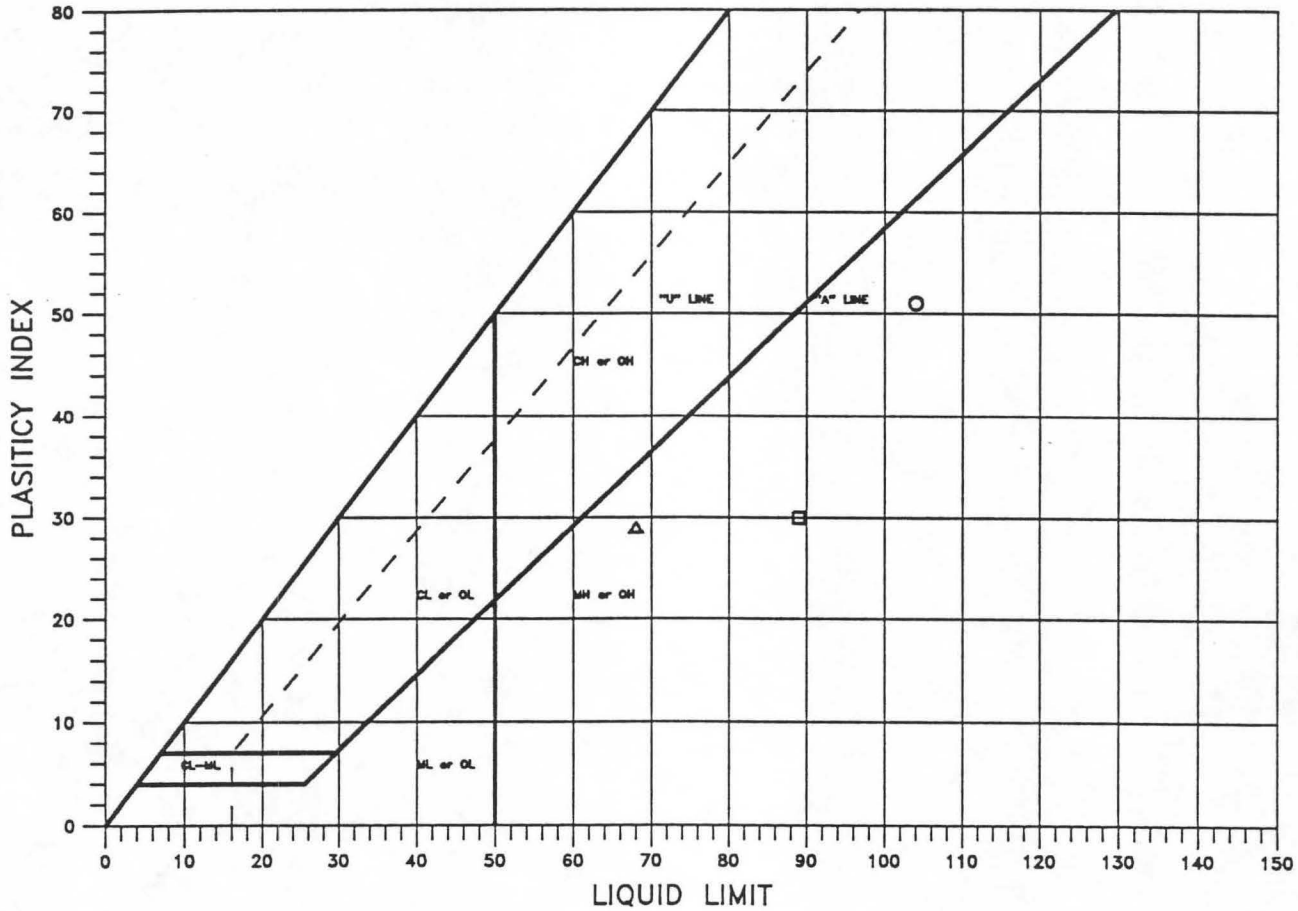
Kirkland, Washington 98034-6918

PLASTICITY CHART



Symbol	Sample Type	Sample Number	Depth (Ft.)	Moisture	Liquid Limit	Plastic Limit	Plastic Index	Passing 200
○	B-2	S-2	8'	189%	104	53	51	
□	B-4	S-3	13'	62%	89	59	30	
△	B-6	P-3	38'	61%	68	39	29	
◇								
✥								
☆								
⊕								

B - Boring Tp - Testpit Ha - Hand Auger Gb - Grab Bag X - Other

Project: Proposed Retail Store
Work Order: W-8401
Date: 8/14/92

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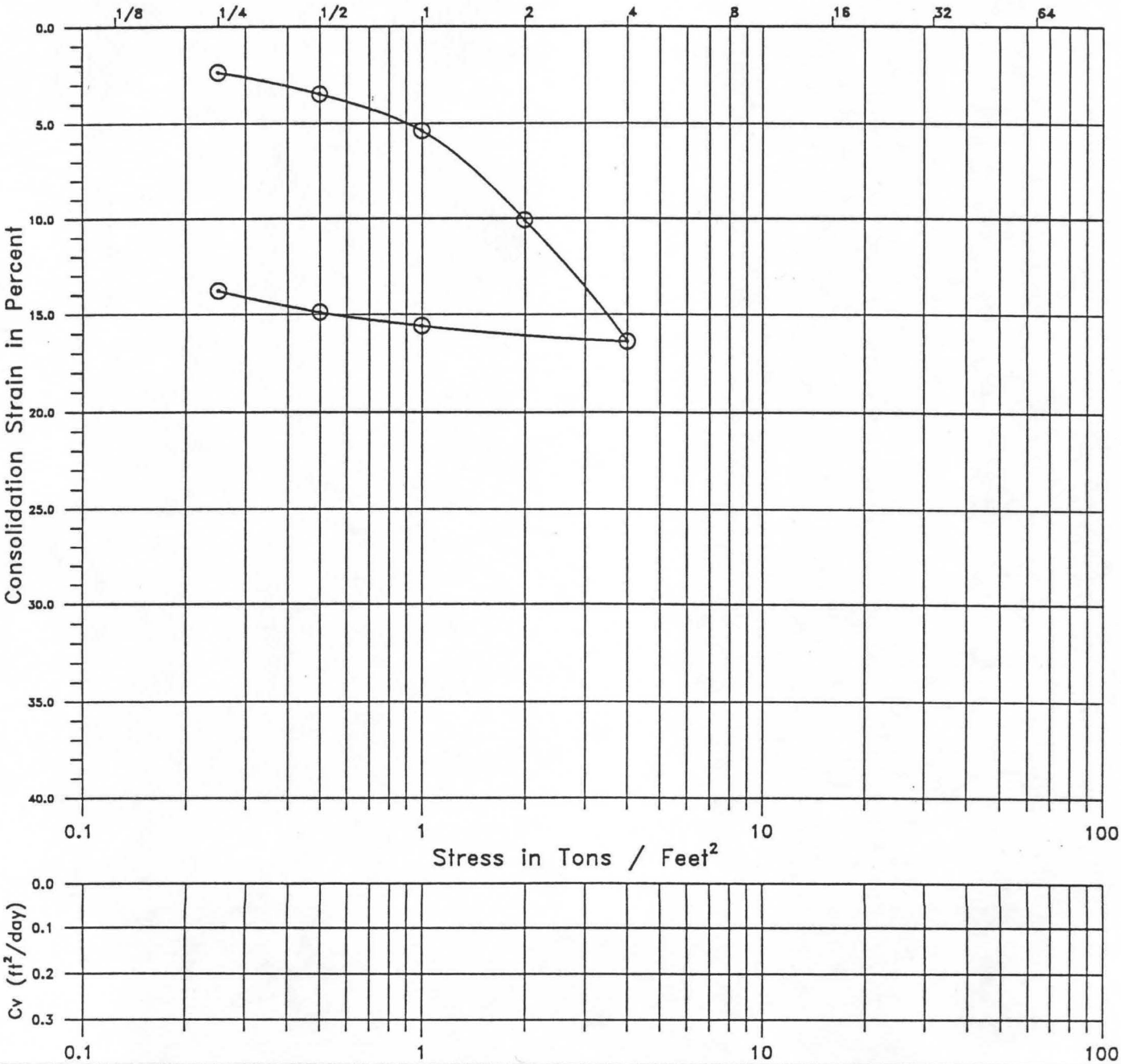
Consolidation Test



Project:	Grimes Project Aberdeen		
Work Order:	W-8401		
Date:	7/17/92		
B-6 S-2		Depth:	37.5'
Moisture:		Density:	
LL:		PL:	

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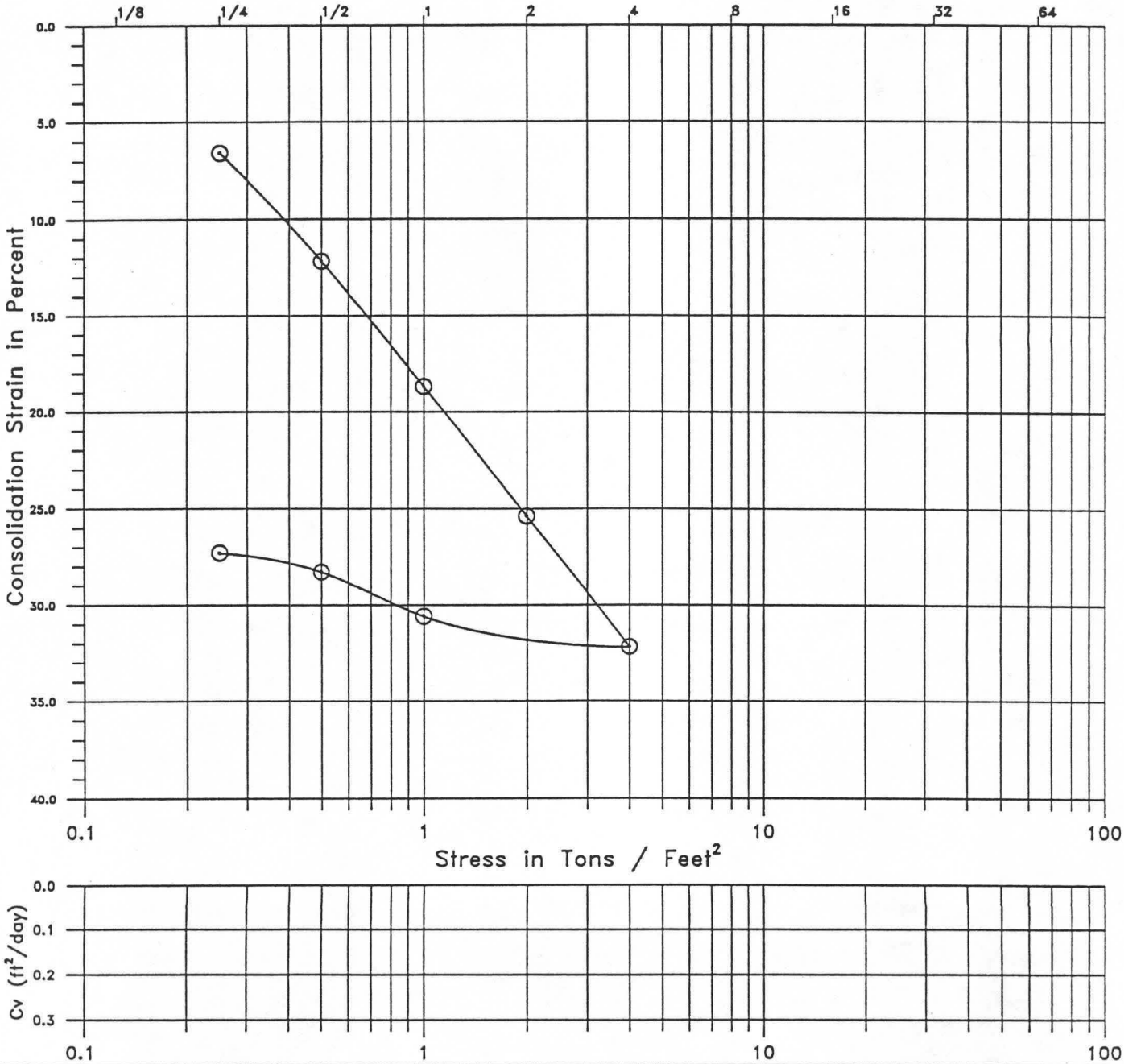
Consolidation Test



Project:	Grimes Project Aberdeen		
Work Order:	W-8401		
Date:	7/17/92		
B-5 S-1		Depth:	8'
Moisture:		Density:	
LL:		PL:	

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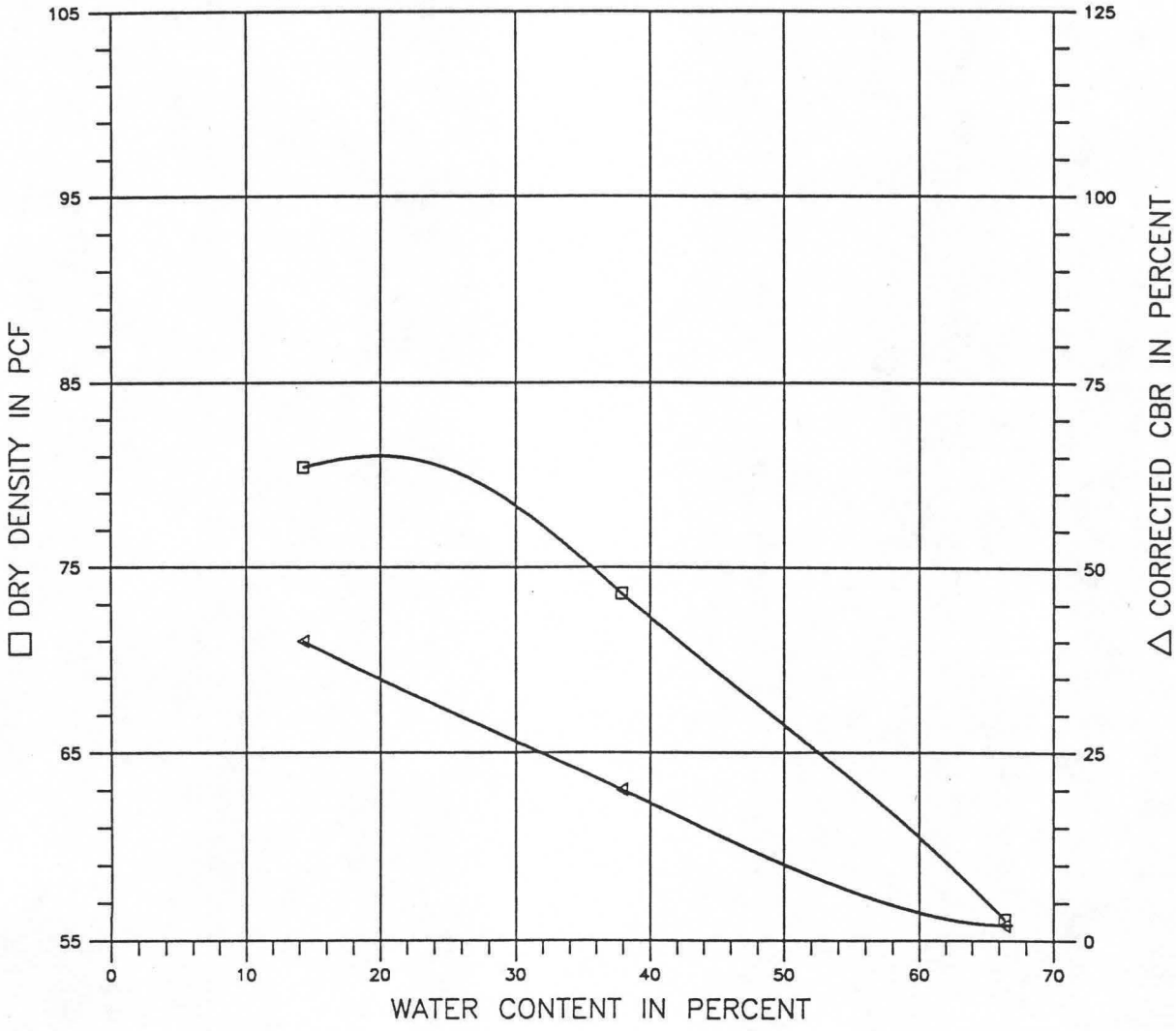
CALIFORNIA BEARING RATIO (CBR) TEST
ASTM: D 1883



Sample Type	Sample Number	Depth (Ft.)	Natural Moisture	Optimum Moisture	Soil Description
TP-2	S-2	5'	--	21%	

B - Boring Tp - Testpit Ha - Hand Auger Gb - Grab Bag X - Other

Project: Proposed Grimes Project Aberdeen
Work Order: W-8401
Date: 8/17/92

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11335 N.E. 122nd Way
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Kirkland, Washington 98034-6918

W-8401 MOISTURE CONTENT DETERMINATIONS			
Boring No.	Sample No.	Depth (feet)	Moisture Content (%)
B-1	S-1	3	30.6
B-1	S-2	8	32.0
B-2	S-1	3	293.0
B-2	S-2	8	189.3
B-3	S-2	8	9.3
B-5	S-2	8	123.0
B-6	S-1	3	3.8
B-6	S-3	13	66.8
B-6	S-4	18	73.7
B-6	S-7	33	50.2
B-6	S-3	37.5	61.0

SOIL CHEMICAL ANALYSIS RESULTS			
Sample Location	Depth (feet)	pH	Resistivity (ohm-cm)
TP-2, S-1	2	5.6	5,200
B-3, S-1	3		15,000
B-5, S-1	3	8.0	3,800

APPENDIX C

AASHTO PAVEMENT DESIGN

AASHTO 1986 METHOD FOR DESIGN OF PAVEMENT STRUCTURES

FOR: Grimes Project - Aberdeen, WA

DESIGN LIFE: 20 years

DESIGN CALIFORNIA BEARING RATIO: 1%

INPUT VALUES FOR STRUCTURAL NUMBER (SN)	REFERENCE
Estimated ESAL (20 yrs) = 30,697	Specified
Reliability (R) = 85%	Specified
Standard Normal Deviate (Z_r) - = 1.037	I-62
Overall Standard Deviation (S_o) = 0.50	I-62, III-51
Reselient Modulus (M_R) = 1,500 psi	I-14
Effective Resilient Modulus (M_R, seasonally adjusted) = 1,500 psi	II-14
Initial Serviceability (P_o)= 4.2	II-12
Terminal Serviceability (P_t) = 2.0	Specified
Design Serviceability Loss (PSI) = 2.2	II-12
Structural Number (SN) = 3.2 and 2.7	II-35
Input values for thickness calculations	
Asphalt layer coefficient (a_1) = 0.33	II-19
Base course layer coefficient (a_2) = 0.12	II-20
Base course drainage coefficient (m_2) = 1.20	II-26
Compacted Roadbed layer coefficient (a_3) = 0.09	II-23
Compacted Roadbed drainage coefficient (m_3) = 1.10	II-26

Recommended Pavement Section Thicknesses (inches)

	Asphalt Concrete	Crushed Base Course	Compacted Subbase
Standard	3	6	9
Heavy	3	6	13½

APPENDIX D

GEOTECHNICAL DESIGN FACT SHEET AND FOUNDATION DESIGN CRITERIA

GEOTECHNICAL INVESTIGATION FACT SHEET

This form shall be included in the Geotechnical Investigation Report as an Appendix.

PROJECT: Grimes LOCATION: Aberdeen, WA
(City) (State)

Geotechnical Investigation Report submitted By: RZA-AGRA, INC.

SCS Soil Hydrologic Group C Soil Name Udorthents

Infiltration (Circle One): Poor Fair Good Very Good

Ground Water Level: Below 2' Wet Season Below 4' Dry Season

Topsoil/Stripping Depth 6 inches or less Undercut Required (Circle One): Yes No

Compaction Method (Circle one): Standard Proctor / Modified Proctor

Minimum Compaction Required for Upper 1 Feet of Stripped Subgrade, if necessary:

Building Area 92 % Outlot Area 92 %

Parking Area 92 % Waste Area 92 %

Compaction Equipment Type and Weight: Vibratory Roller

Compaction Tests: 1 Test for Each 5000 Sq. Ft. each Lift

Structural Fill Maximum Lift Thickness 10 inches (Measured loose)

Subgrade design CBR (or LBR) value = 1%.

COMPONENT	ASPHALT		CONCRETE	
	STANDARD	HEAVY	STANDARD	HEAVY
Stabilized Subgrade (If Applicable) (HRT)	NA	NA	NA	NA
Subbase Material	9"	13½"	9"	13½"
Base Material (Stone, Sand/shell, etc.)	6"	6"	6"	6"
Asphaltic Base Course (If Applicable)	NA	NA	NA	NA
Leveling/Binder Course (If Applicable)	3"	3"	6"	6"
Surface Course	2"	3"	6"	6"

Note: This information should not be used separately from other portions of this Soil Investigation Report.

FOUNDATION DESIGN CRITERIA

This form shall be included in the Geotechnical Investigation Report as an Appendix

PROJECT: GRIMES

LOCATION: Aberdeen, WA
(City) (State)

GEOTECHNICAL ENGINEER: RZA-AGRA, INC.

SOILS REPORT DATED: 31 August 1992

Recommended
Option

FOUNDATION OPTIONS: 1 Driven timber piles

2 Augercast concrete piles

BEARING PRESSURE: See test for piling capacities

MINIMUM FOOTING SIZES: See text for piling diameters and lengths

MIN. FOOTING EMBEDMENT: See text for piling embedment

FROST DEPTH: 18 inches

TOTAL SETTLEMENT (1" MAX): 1 inch or less

DIFFERENTIAL SETTLEMENT (1/2" MAX): 1/2 inch

SLAB: POTENTIAL VERTICAL RISE (1/2" MAX): less than 1 inch settlement

VAPOR BARRIER: yes

CAPILLARY BARRIER: 6 inches sand/gravel WSDOT Specs 9-03.12(4), 13, 15, 16

SUBGRADE REACTION MODULUS: 3 pci (native soil)

PERIMETER DRAINS REQ'D: Yes see report text

CONCRETE: Type I X Type II ___ Type III ___ Other

SPECIAL COMMENTS:

NOTE: This information should note be used separately from other portions of this soil investigation report.