A report prepared for

Consoer Townsend and Associates
733 Market Street, Suite 500
Tacoma, Washington 98402

GEOTECHNICAL ENGINEERING STUDY
PROPOSED STORAGE FACILITY
TACOMA, WASHINGTON

AGI Project No. 15,523.001.02

by

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I. INTRODUCTION

A. General

This report presents the results of our geotechnical engineering study in support of your preparation of a RCRA Part B application for a proposed storage tank and barrel storage facility at Northwest Processing. The plant site is located at 1707 Alexander Way in Tacoma, Washington, approximately as shown on the Vicinity Map, Figure 1.

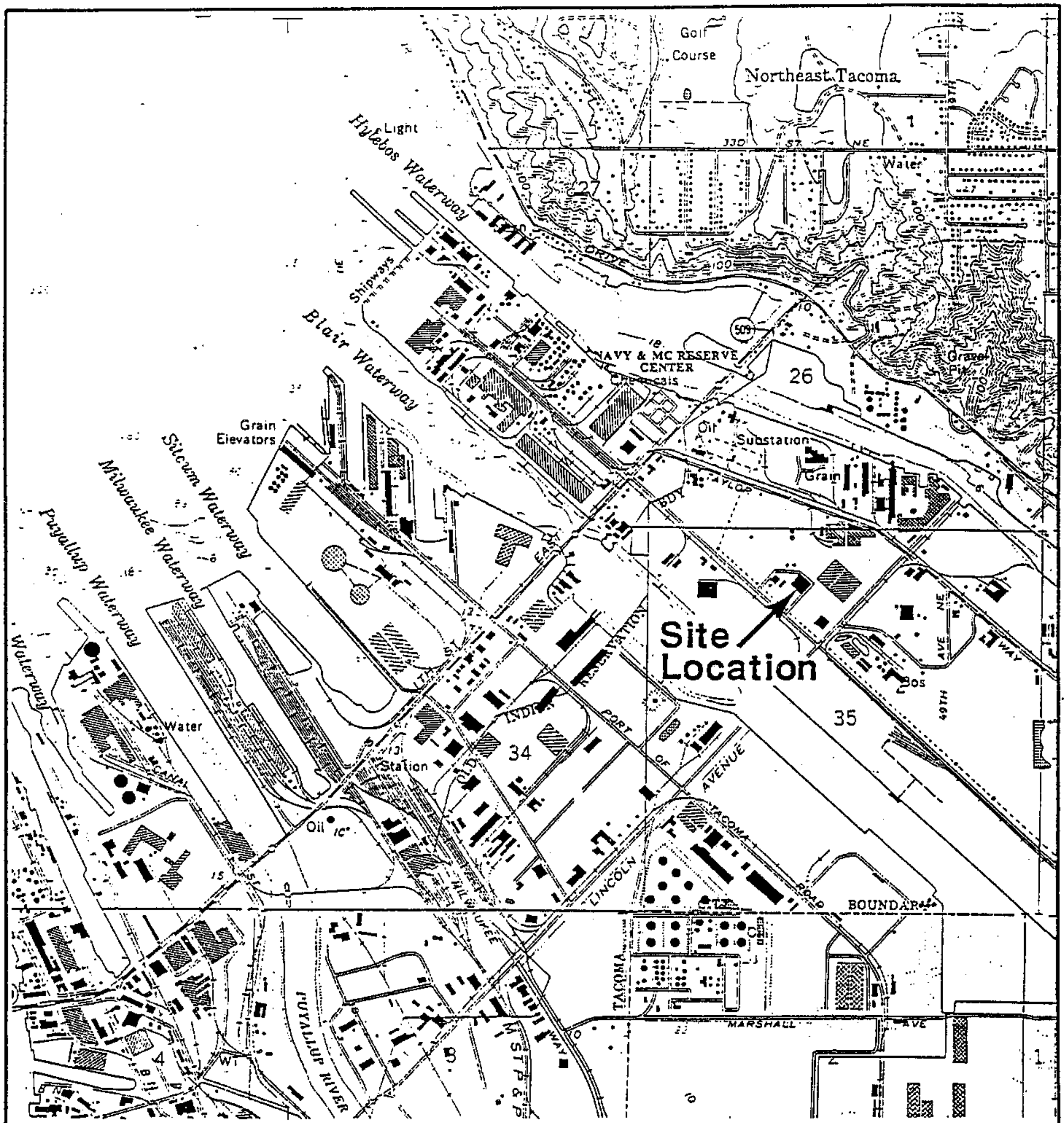
B. Project Description

We understand the proposed storage facilities will include six above-ground 10,000-gallon horizontal storage tanks, each measuring about 10 feet in diameter by 27 feet in length, and a barrel storage area as shown on the Site Plan, Figure 2. The tanks will be supported on a concrete mat measuring roughly 48 feet by 50 feet in plan. The drum storage area will be supported on an adjacent concrete mat measuring approximately 48 feet by 27 feet. Preliminarily, contact pressures beneath the tank storage area will be on the order of 400 pounds per square foot (psf). Although contact pressures for the drum storage area were not provided at the time of our study, we understand they will be less than 400 psf.

C. Scope of Services

The purpose of our study is to provide recommendations for site preparation and foundation support for the proposed storage facility based on subsurface exploration, laboratory testing, and engineering analyses. Specifically, our scope of services includes the following:

- Exploration of subsurface conditions at the project area by two borings.
- Laboratory testing to assess pertinent physical characteristics of the soils encountered. The testing program included direct shear, consolidation, Atterberg Limits, and moisture-density tests.
- Recommendations for site preparation and fill placement, including evaluation of suitability of on-site soils for use as fill and estimates of the need for, and depth of, overexcavation required to remove and replace unsuitable soils, if present.
- Evaluation of liquefaction potential of site soils.



Reference: USGS 7.5' Quadrangle titled "Tacoma North, Wash." dated 1981.

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Vicinity Map

Consoer Townsend & Assoc./Northwest Processing
Tacoma, Washington

FIGURE **1**

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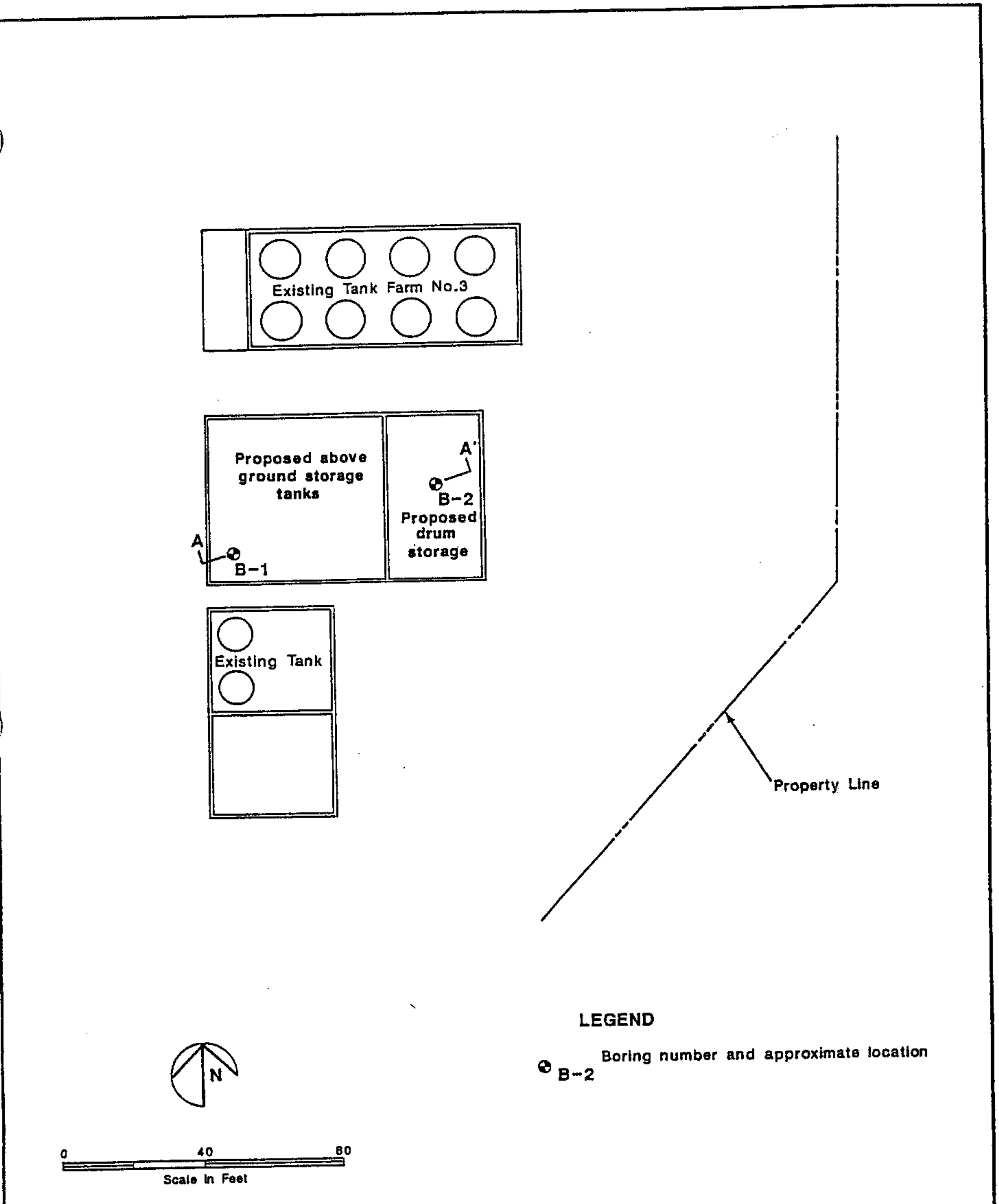

Reference: Site Plan prepared by Consoer Townsend & Associates, undated.

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Site Plan

Consoer Townsend & Assoc./Northwest Processing
Tacoma, Washington

FIGURE **2**

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- Recommendations for foundation design for the proposed storage areas. We provide recommendations for design and installation of timber pile foundations to mitigate liquefaction potential and consolidation settlement potential. Recommendations include allowable capacity versus penetration criteria and estimates of pile foundation settlement.

- Preparation of this report containing our findings, conclusions, and recommendations.

II. SITE CONDITIONS

A. Surface

Current plans indicate the proposed storage facility will be located approximately 5 feet north of Tank Farm No. 4 and approximately 18 feet south of Tank Farm No. 3, as shown on Figure 2. The relatively level site is currently covered with gravel. Relief is on the order of 1 foot.

B. Subsurface

Subsurface conditions were explored by 2 borings drilled to depths of 44 and 59 feet below current site grade. Approximate locations of the explorations are shown on Figure 2. A generalized subsurface profile depicting our interpretation of site stratigraphy is presented on the Cross Section, Figure 3. A description of the field equipment and procedures in addition to logs of the explorations are contained in Appendix A. Descriptions of laboratory tests and results are contained in Appendix B.

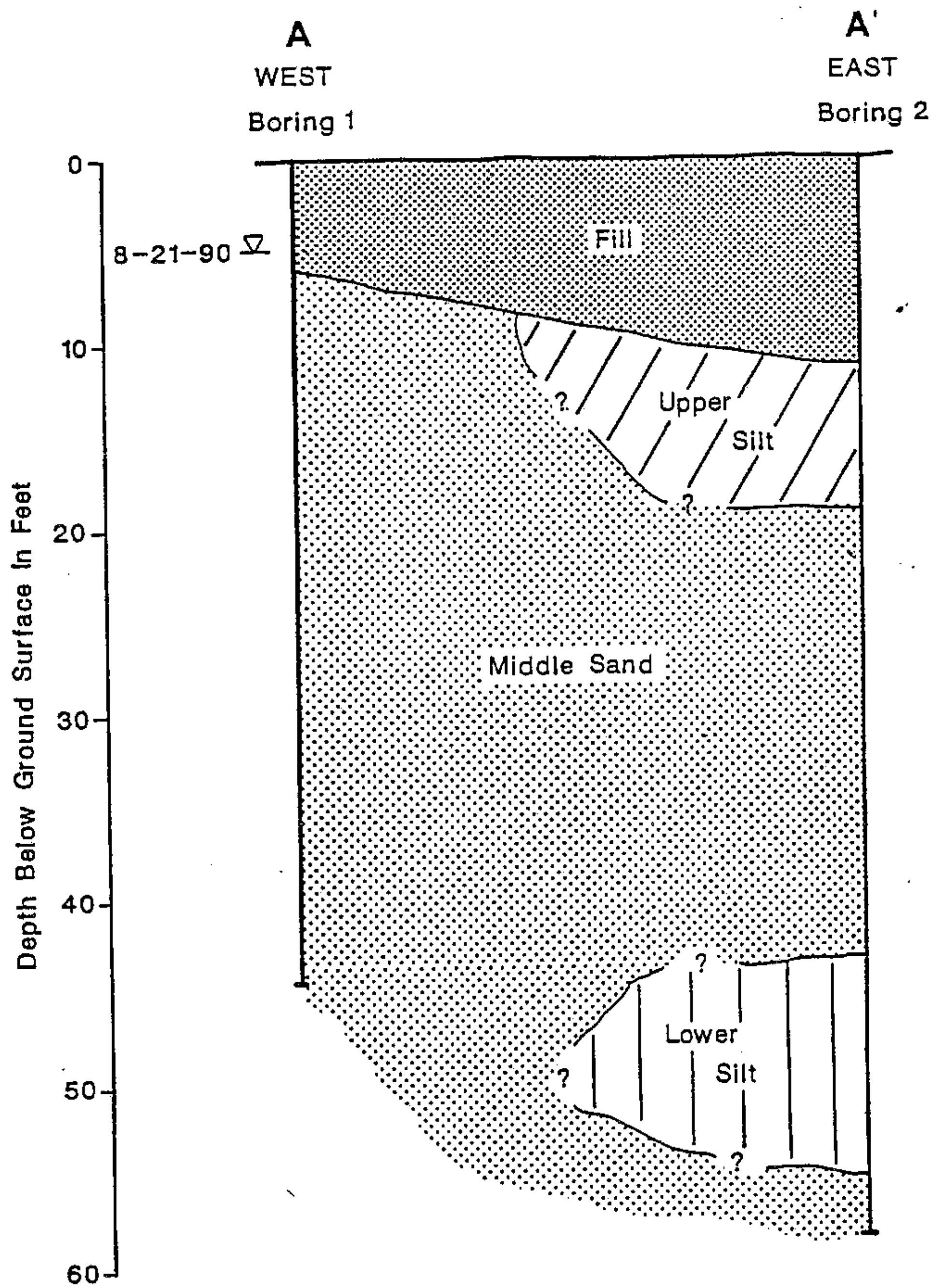
Subsurface deposits comprise four distinct units described below and summarized in Table 1. Unit designations are based on commonly used geologic terminology.

FILL: Fill was encountered in both borings extending from the ground surface to depths ranging from 6 to 11 feet. Fill includes a surface layer of gravel underlain by sandy silt at the location of Boring 1 and silty sand and sandy silt at Boring 2. Miscellaneous debris was noted in the Fill including a slight petroleum odor at Boring 2. Fill is inherently variable, but typically has moderate strength and compressibility.

UPPER SILT: Upper Silt was encountered in Boring 2 between depths of 11 and 19 feet and consists of soft silt and clay. The Upper Silt is characterized by low strength and moderate to high compressibility.

SAND: Loose to dense sand was encountered beneath Fill in Boring 1, and Upper Silt in Boring 2. The depth to the Sand varied from 6 to 19 feet in Borings 1 and 2, respectively. The Sand is characterized by moderate strength and moderate to low compressibility.

LOWER SILT: Lower Silt was encountered in Boring 2 between depths of approximately 42 and 54 feet below site grade. This unit is medium stiff to stiff, with moderate strength and low to moderate compressibility. Layers of Sand were encountered beneath the Lower Silt to the maximum depth explored at the location of Boring 2, suggesting the Lower Silt is not continuous beneath the project area.



Scale: 1"=10' Vertical
1"=20' Horizontal

Explanation:
This cross section is a diagrammatic interpretation of subsurface conditions based on interpolation and extrapolation of data from borings.
Actual conditions are substantially more complex than depicted and will vary between borings.

Applied Geotechnology Inc. Geotechnical Engineering Geology & Hydrogeology	**Cross Section A-A'** Consoer Townsend & Assoc./Northwest Processing Tacoma, Washington				FIGURE **3**
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C. Groundwater

Groundwater was encountered in Borings 1 and 2 at a depth of approximately 5 feet below ground surface. We expect site groundwater levels fluctuate in response to seasonal rainfall variations.

III. DISCUSSION AND CONCLUSIONS

A. General

Based on the results of our site exploration, laboratory testing, and engineering analyses, we conclude there is a moderate to high potential for seismically-induced liquefaction of certain zones of sand deposits underlying the project area. We further conclude that contact pressure from a soil-supported mat foundation could cause consolidation of compressible fine-grained deposits, resulting in some differential settlement of the tank and barrel storage areas, in addition to the adjacent Tank Farm No. 4 concrete mat. On this basis, we recommend the tank and barrel storage facility be supported by driven timber piles that penetrate into the Sand deposit encountered during our subsurface exploration. The use of driven pile foundations will eliminate the need to overexcavate and replace unsuitable near-surface compressible soils and should reduce liquefaction potential of certain soil zones through which the piles penetrate. However, if piping extends from the new facility to existing facilities supported by shallow foundations, the mechanical or piping design should provide for potential differential settlement of such pipe systems.

Geotechnical considerations and design recommendations for site preparation and foundation design are discussed in further detail in the following sections. Specific design and construction recommendations are provided in Sections IV and V, respectively.

B. Earthwork

Specific plans were not available at the time we prepared this report. However, we anticipate that relatively minor cutting and filling will be required in conjunction with construction of the proposed storage facility. If utility installation is required, backfill should consist either of on-site soils or imported Select Fill. Results of our study indicate on-site soils are generally unsuitable for reuse as structural fill unless they are conditioned to at or near optimum moisture content before placement and compaction, and then only during dry weather conditions. We recommend select sand and gravel, as specified in Section V, be imported for use as backfill if construction is undertaken during wet weather.

Excavations extending more than about 3 to 4 feet below current site grade could encounter substantial groundwater. We therefore recommend the contractor be apprised of the possible need to dewater.

C. Seismic Risk and Liquefaction

A thorough seismic risk analysis for the project is beyond our scope of services. However, we believe that earthquake response is an important factor for this site and should be carefully considered in design.

Historically, the Puget Sound region has been subjected to frequent earthquakes of moderate intensity and is indicated in the Uniform Building Code as Seismic Zone 3. Two earthquakes that resulted in significant damage occurred in 1949 near Olympia and in 1965 near Seattle. The April 13, 1949 earthquake is the largest recorded earthquake in the region, reaching a magnitude of 7.1 on the Richter scale. It was felt over 150,000 square miles (390,000 square kilometers), and resulted in eight deaths and many injuries. The April 29, 1965 earthquake of magnitude 6.5 was felt over 130,000 square miles (340,000 square kilometers) and resulted in widespread damage in the Seattle area. Ground failures attributed to liquefaction occurred in many waterfront areas.

Based on seismic risk analyses we performed for similar sites in the Puget Sound region, we believe peak ground accelerations at the Northwest Processing site would be on the order of 0.15g to 0.25g for a lower and an upper level event, respectively. These earthquakes would have respective magnitudes of approximately 6.25 and 7.5. The lower level event is one with a 50 percent probability of exceedance during a 50-year design life. This is an event similar to the 1965 Seattle earthquake, and one that most structures should be designed to survive with little damage. The upper level earthquake represents a less likely (10 percent probability of exceedance) event for which major structures, such as nuclear power plants, should be designed.

The predominantly fine to medium-grained gradation and loose consistency of zones within the Sand, together with the relatively shallow groundwater table, makes portions of this unit potentially susceptible to liquefaction during a seismic event. A shallow spread foundation system deriving its support from soil above this unit could experience significant post construction settlement as a result of ground failures due to liquefaction.

The liquefaction potential of the soils underlying the Northwest Processing site was evaluated by an empirical method using a direct correlation between observed ground motion and SPT data. We considered a typical soil profile with the groundwater table 5 feet below ground surface. For the lower level earthquake, our results are as follows:

Lower Level Earthquake: A comparison of the induced cyclic shear stresses with the available cyclic shear strength indicates liquefaction during a lower level earthquake would likely not occur within the Sand, except possibly for localized areas of particularly loose soil. Results of the liquefaction analysis are shown on Liquefaction Potential, Figures 4 and 5. The impact of the proposed structure would be related to ground settlement, reflected by settlement of the storage facility if a shallow mat foundation is utilized. Although the magnitude of such settlement is difficult, if not impossible, to reasonably estimate, we believe there is a low to moderate risk that for a mat foundation, differential settlement-related damage could result from localized ground settlement induced by liquefaction.

Installation of timber piles will transfer foundation load beneath the upper liquefiable soils. Furthermore, pile installation should densify site soils, reducing liquefaction potential.

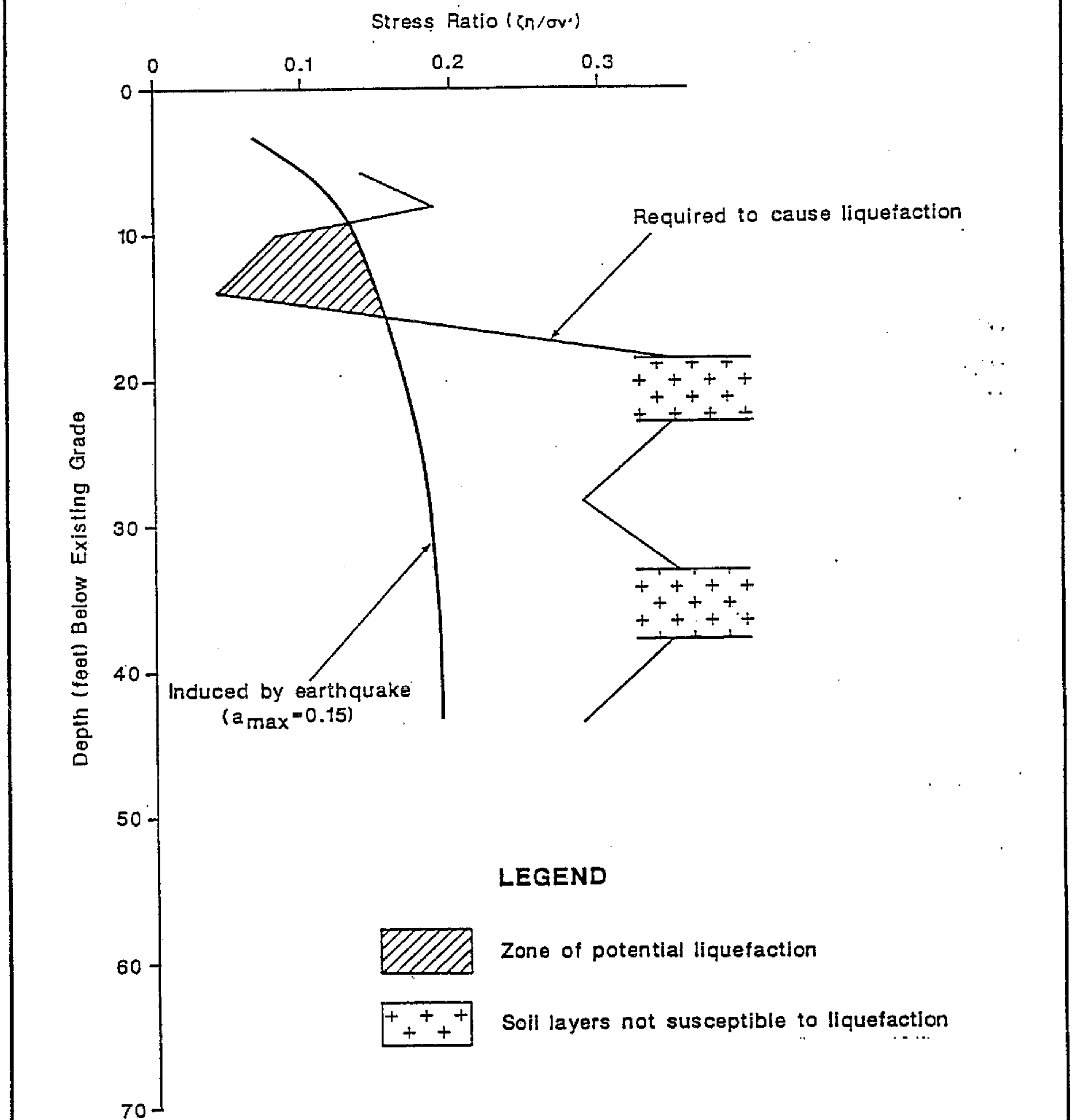
It is our opinion the consequences of liquefaction in lower portions of the Sand unit are minor. If liquefaction does occur at depths greater than about 40 feet, it might be reflected by small, but relatively uniform areal settlements at the ground surface.

D. Timber Pile Foundations

We evaluated several pile types including auger-cast (cast-in-place) piles, concrete piles, closed-end steel pipe piles, and timber piles for support of the proposed tank and barrel storage facility. We eliminated auger-cast piles from consideration due to soil liquefaction potential and the need to reduce that potential by use of driven displacement piles. In view of the relatively low load per pile that will be imposed by the proposed facility, it is our opinion that timber piles are the most economical deep foundation. It is our further opinion that timber piles driven to depths on the order of 25 to 30 feet below current site grade will provide adequate downward and lateral supporting capacity for the proposed storage facility. Specific design and installation recommendations are contained in following sections.

E. Frost Penetration

Typical frost penetration depths in the Pacific Northwest are in the range of 6 to 12 inches below ground surface. Accordingly, shallow spread and mat foundations, and utilities susceptible to frost, should be embedded at least 18 inches below lowest adjacent finished grade to reduce potential disturbance and adverse effects related to frost penetration.



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Liquefaction Potential @ Boring 1
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FIGURE **4**

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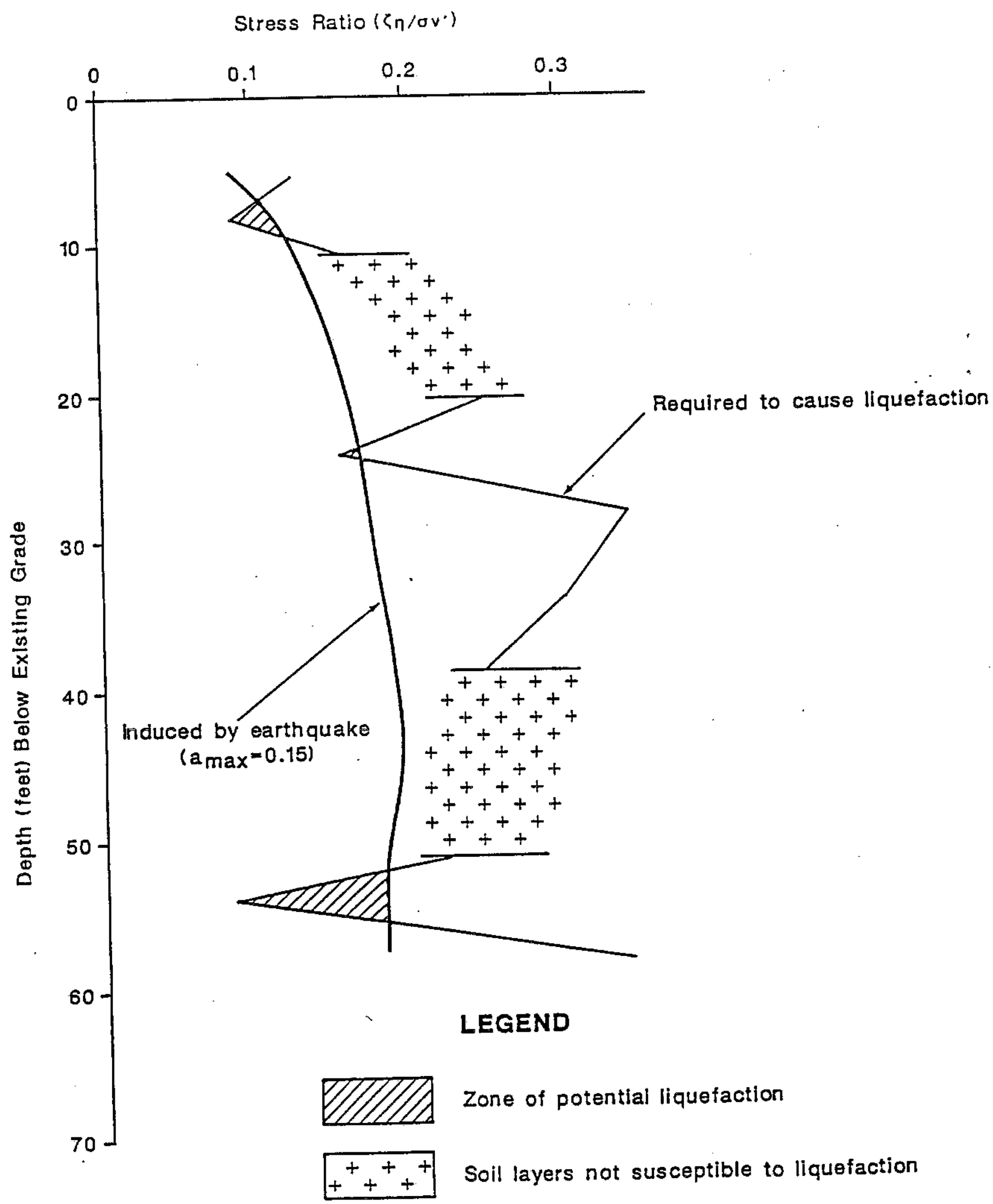

Applied Geotechnology Inc. Geotechnical Engineering Geology & Hydrogeology	**Liquefaction Potential @ Boring 2** Consoer Townsend & Assoc./Northwest Processing Tacoma, Washington			FIGURE **5**
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F. Site Drainage

Surface water runoff can be controlled during construction by careful excavation procedures. Typically, such procedures include construction of upgrade perimeter drainage ditches or low earth berms and use of temporary sumps to collect seepage and prevent water from damaging exposed subgrades. All collected water must be directed, under control, to a permanent positive discharge system.

We emphasize that you inform your contractor of the possibility of encountering groundwater in utility and other excavations. The contractor should be prepared to dewater excavations sufficiently to permit completion of below grade installations and construction.

G. Limitations and Additional Services

This report has been prepared for the exclusive use of Consoer Townsend and Associates, Northwest Processing, and their consultants, for this project only. The analyses, conclusions, and recommendations in this report are based on conditions encountered at the time of this study and our experience and engineering judgement. AGI cannot be responsible for the interpretation of the data contained herein by others.

Our services were performed in a manner consistent with that level of care and skill ordinarily exercised by members of the profession currently practicing under similar conditions in the area. No other warranty, expressed or implied, is made.

We must presume the conditions encountered are representative of the entire property. However, you should be aware that subsurface conditions may vary between exploration locations, and with time, unanticipated conditions can and often do occur. If differing conditions are exposed during construction, or the design is modified, we should be requested to reevaluate our recommendations and to provide a written confirmation or modification, as necessary. We cannot be responsible for the applicability of our recommendations if not afforded this opportunity. To allow for these eventualities, we recommend a contingency be provided in both your construction budget and schedule.

We recommend we be retained to review final project plans and specifications to verify the intent of our recommendations is properly interpreted and included. We also recommend we be retained to provide construction monitoring services during the geotechnical phases of project construction. This will allow us to verify subsurface conditions are as anticipated, and to observe and test the Contractor's work as your representative.

IV. DESIGN RECOMMENDATIONS

A. General

The following paragraphs present our design recommendations for use by your consultants on this project. It is important to remember, however, that for satisfactory and successful construction of this project, these recommendations must be applied in their entirety and in conjunction with the construction recommendations provided in Section V of this report.

B. Timber Pile Foundation

We recommend timber piles with a minimum tip diameter of 8 inches be installed for support of the proposed tank and barrel storage facility. We further recommend the piles penetrate into the Sand encountered by our borings to a depth of not less than 25 feet and not more than 35 feet below current site grade. Specific design recommendations are presented below:

- Pile Types
  - Minimum 8-inch tip diameter, pressure-treated timber piles
- Bearing Layer
  - Dense Sand
- Safety Factors

  The allowable downward pile capacities include a safety factor of about 3 for end bearing and 1.5 for side friction with an overall safety factor of at least 2.0. The allowable uplift capacities include a safety factor of about 2.0.

- Timber Pile Design Criteria

Diameter (Inches)	Allowable Downward Capacity (Kips)	Allowable Uplift Capacity (Kips)	Recommended Minimum Penetration Below Existing Site Grade (Feet)
Minimum 8-inch tip	40	20	30

Pile lengths will vary due to the variable depth to and density of supporting Sand and presence of dense zones may inhibit or prevent pile installation to the recommended minimum penetration. We anticipate pile penetrations below grade will range from 25 to 35 feet. However, the contractor should be prepared to install somewhat longer piles in the event of local nonconformities in the stratigraphy as revealed during this investigation. Piles that do not meet the recommended penetrations must be evaluated by AGI so that we can advise the structural engineer of reduced allowable capacities and need for installation of additional piles.

Allowable pile capacities are for the total of all dead and live loads and may be increased by one-third for temporary short term wind but not seismic loads.

Axial pile capacities are based on soil strength and approximate structural capacity. The structural engineer should verify allowable loads based on actual loading conditions.

- Pile Groups
  - Pile Group Efficiency: 1.00
  - Minimum Center to Center Pile Spacing: 3 pile diameters
- Lateral Loads

  The following lateral load characteristics were evaluated assuming a fixed slope (no rotation) condition at the top of a single timber pile. Allowable lateral loads include a safety factor of at least 1.5.

Allowable Lateral Load for 1/4-Inch Deflection (Kips)	Approximate Point of Fixity Below Mat (Feet)
4	6

Allowable Lateral Load for 1/2-Inch Deflection (Kips)	Approximate Point of Fixity Below Mat (Feet)
8	8

If additional lateral load resistance is needed, a proportion of available passive resistance between the mat and adjacent soil can be utilized. We recommend an equivalent fluid density of 180 and 240 pcf be used to estimate allowable passive pressure resistance in conjunction with lateral pile deflections of 1/4 inch and 1/2 inch, respectively. Lateral pile capacity and passive soil resistance at other lateral deflections less than 1/2 inch may be estimated by linear interpolation.

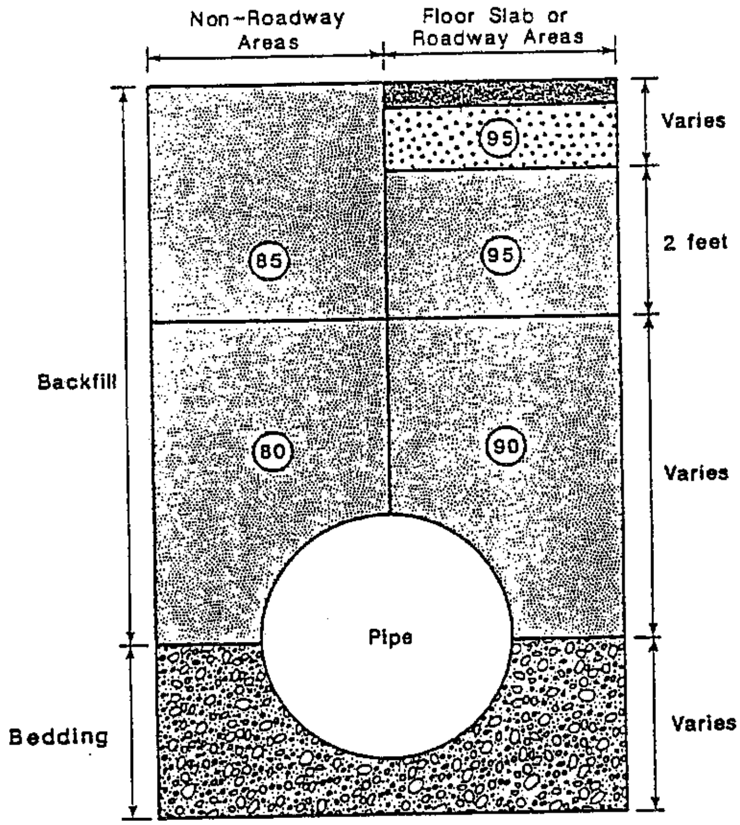
- Pile Settlement
  - Estimated total settlement of single pile = 1/2 inch or less
  - Estimated total settlement of pile group = 3/4 inch or less

C. Slopes

- Temporary Slopes
  - Cut: Maximum 1-1/2:1 (horizontal:vertical)
  - Fill: Maximum 1-1/2:1
- Permanent Slopes
  - Cut: Maximum 2:1 (horizontal:vertical)
  - Fill: Maximum 2:1

D. Utility Lines

All utility line excavations, particularly those beneath pavements should be properly backfilled with compacted Select Fill to the specified degree of compaction. Figure 6 provides a pictorial representation of our recommended design.



LEGEND


Asphalt/Concrete Pavement/ Concrete Floor Slab


Base Material/Slab Base Rock


Backfill; compacted on-site soil or imported select fill material


Bedding material; material type depends on type of pipe and laying conditions. Bedding should conform to the manufacturers recommendations for the type of pipe selected.

95 Minimum percentage of maximum laboratory dry density as determined by ASTM Test Method D 1557-78 (Modified Proctor).

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Typical Utility Trench Fill

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FIGURE **6**

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V. CONSTRUCTION RECOMMENDATIONS

A. General

1. Description

This section presents our detailed recommendations for geotechnical aspects of construction. Specifically, we cover earthwork and drainage. Our design criteria are based on these construction recommendations; therefore, these recommendations should be included in the project specifications in their entirety.

2. Standard Specifications

Where possible, we refer to the 1988 Edition of the State of Washington Standard Specifications for Road, Bridge, and Municipal Construction.

3. Geotechnical Engineer

We recommend that you retain us as the Geotechnical Engineer during construction to observe and test the geotechnical aspects of the contractor's work. This will allow us to compare the actual conditions encountered with those anticipated by this investigation and to modify our recommendations, if necessary. We will be present at the site intermittently to check that the Contractor's work conforms with the geotechnical aspects of the plans and specifications. Our daily field reports and final report from an important record of construction. However, observation and testing by the Geotechnical Engineer should not in any way release the Contractor from the responsibility of performing his work in such a manner as to provide a satisfactory job that meets the requirements of the Project Plans and Specifications or from meeting his contractual obligations to the owner.

4. Geotechnical Report

This report has been prepared for design purposes for Consoer Townsend and Associates and their design consultants for this project only. The data and report should be provided to the Contractors for their bidding or estimating but not as a warranty of the subsurface conditions. We cannot be responsible for the interpretation by others of the information contained in this report.

5. Construction Site Safety

Our scope of services did not include construction safety practices and this report is not intended to direct construction means, methods, techniques, sequences or procedures, except as specifically described, and then only for consideration in design, not for construction guidance. The Contractor should be made responsible for construction site safety and compliance with local, state, and federal requirements.

B. Earthwork

1. General

Earthwork consists of clearing and stripping, excavation, fill placement and compaction, utility trench backfilling, structural excavating and backfilling, and all subsidiary work necessary to complete the grading of the developed areas to conform with the lines, grades, and slopes shown on the plans.

Terms used in this section are defined as follows:

- Percent Compaction is the required in-place dry density of the material, expressed as a percentage of the maximum dry density of the same material as determined by ASTM Test Method D1557 (Modified Proctor).
- Optimum Moisture Content is the moisture content (percent by dry weight) corresponding to the maximum dry density of the same material as determined by ASTM Test Method D1557.
- On-site Fill is the soil obtained at the project site during excavation (following clearing and stripping) free of organic contaminants, perishable material, and rocks or lumps greater than 6 inches in maximum dimension.
- Select Fill is imported soil consisting of clean, free-draining sand and gravel containing less than 3 percent fine material (passing the No. 200 sieve).
- Moisture Sensitive Soil is on-site soil containing more than 3 percent silt or clay (passing the No. 200 sieve).

2. Quality Control

The Contractor should be made responsible for quality control of the earthwork. As the Geotechnical Engineer provided by the owner, we will observe and test the Contractor's work for conformance with the geotechnical aspects of the plans and specifications. The Contractor should be required to provide us with every reasonable facility for checking the workmanship for conformance. We will prepare a daily record of our observations and tests which will be made available to the Contractor and to the owner. You may wish to consider making the Contractor responsible for retesting of his work when it fails to meet the requirements of the plans and specifications and for the associated costs of such retesting.

The Contractor should submit samples of each of the required earthwork materials to the Geotechnical Engineer for evaluation and approval prior to use. The samples should be submitted at least four days prior to using the materials and sufficiently in advance of the work to allow the Contractor to identify alternate sources if the material proves to be unsatisfactory.

3. <u>Seepage Control</u>

Runoff or groundwater seepage that would interfere with the Contractor's work should be controlled during construction. Control may consist of temporary drainage ditches or subsurface drains. Installation of such measures should be the Contractor's responsibility.

4. <u>Clearing and Stripping</u>

The building and paved areas of the site should initially be cleared and stripped of all existing trees, vegetation, and debris. These materials should not be reused as Select or On-site Fill. They should be removed from the site and disposed.

5. <u>General Excavation</u>

General excavation consists of removing unsuitable on-site soils to sufficient depth to achieve design grades. In general, although most of the on-site soils are Moisture Sensitive Soils, they can be excavated using standard earthmoving equipment. If earthwork operations are performed during periods of wet weather, these soils will deteriorate rapidly and will create difficulties in the excavation process.

After the design grade is reached, the exposed soils should be scarified to a depth of at least 6 inches, moisture conditioned if necessary, and then compacted to at least 95 Percent Compaction. Additional excavation may be required to remove soft or extremely weak materials that are exposed during the recompaction operation. Any soft soils encountered during the compaction operation should, if possible, be recompacted or excavated and replaced with Select Fill compacted to 95 Percent Compaction.

All final surfaces exposed by the complete excavation shall be finished true to line and grade and present a smooth, firm surface.

After rainfall, construction equipment should be prevented from traveling on the exposed site subgrade until the soils have been allowed to dry sufficiently. Otherwise, traffic activity on the wetted subgrade will degrade the exposed materials and result in additional excavation of the disturbed materials. Such additional overexcavation should be performed at the Contractor's cost.

6. <u>Fill and Backfill Placement Compaction</u>

This section covers the placement and compaction of earth materials to achieve the planned grades. Fill or backfill material should consist of Select Fill. We do not recommend using the native materials for On-site Fill.

Fill material should be placed in layers 8 inches or less in loose thickness, moisture conditioned if necessary, and compacted to 95 Percent Compaction. If field density tests indicate the required Percent Compaction has not been obtained, the fill material should be reconditioned as necessary and recompacted to the required Percent Compaction before placing any additional material.

7. Utility Trench Backfilling and Compaction

This section covers backfilling of all utility trenches. The Contractor should be responsible for the safety of his men working in utility trenches.

We recommend that all utility trenches, but particularly those greater than 4 feet in depth, be supported in accordance with state and federal safety regulations.

All utility trench backfill material should consist of Select fill. Utility trench backfill should be placed and compacted as described in the preceding section of this report. Backfill should be compacted to at least 95 Percent Compaction. Particular care should be taken to make sure bedding or fill material is properly compacted in place to provide adequate support to the pipe. Jetting or flooding is not a substitute for mechanical compaction and should not be allowed.

C. Pile Foundations

1. General

This section covers the furnishing and installation of a driven pile foundation system consisting of pressure treated timber piles at the locations and to the elevations shown on the project plans or described in the specifications.

Terms used in this section are defined as follows:

- Collars and Plates: Material for bearing collars and splices shall meet the requirements of ASTM A36. Work shall conform to the details indicated on the drawings or equivalent details approved by the Engineer.
- Timber Pile: A pressure treated wooden pile having a minimum tip diameter of 8 inches and a taper of not less than 1 inch in 10 feet and meeting the requirements of ASTM D25 for grade, quality, size, straightness, manufacture, seasoning, and tolerances.
- Strapping: High tensile steel banding wrapped at least twice around the pile butt, about 12 inches below the top of pile, and firmly connected and fixed in place. Strapping helps to prevent the pile butt from splitting and brooming during driving.

- <u>Pile Redrive</u>: Occurs when a pile which has been driven to the approximate design elevation, and has been allowed to sit in-place undisturbed for a set period of time while pore water pressures are allowed to dissipate, is redriven with the same driving equipment for several feet. This allows the engineer to better determine the actual pile capacity.

2. <u>Contractor's Responsibility</u>

All piling should be furnished and installed by a Contractor experienced in similar work and qualified to install piles in accordance with the plans and specifications. The Contractor should be responsible for providing all necessary labor, supervision, materials, tools and equipment to locate, install and cut off or build up piles for the foundations in accordance with the plans and specifications. He should also be responsible for using all available data, including but not limited to, the project geotechnical report, the plans and specifications, indicator pile driving records, other pile driving records or summaries of piles driven on nearby projects, pile driving behavior, or pile heave, and should consider all of the information together.

The Contractor should also be responsible for driving the piles to obtain the specified driving criteria, and for providing equipment and piling lengths that anticipate variations in the strength of the subsurface soils that could reasonably be expected based on all the information available.

3. <u>Quality Control and Monitoring of Installation</u>

The Contractor's work should be performed under the full time observation of the Geotechnical Engineer who will be present during the driving operations to observe the work and correlate driving behavior with subsurface data. The Geotechnical Engineer shall at all times have access to the work, and the Contractor should furnish the Geotechnical Engineer with every reasonable facility for checking the workmanship for conformance with the geotechnical aspects of the plans and specifications. The Contractor should be responsible for marking each pile in 1-foot increments, starting at the tip, and should use enlarged numerals to indicate the length of the pile at 5-foot intervals. The markings should be such that they are not obliterated or made unreadable during slinging and handling. The piles should be oriented in the leads so that the marking are visible for monitoring purposes.

The Geotechnical Engineer will keep a complete written record of each pile driven, noting the date, time, type, location, diameter, wall thickness, tip and butt size, verticality, and driving record with blow counts. The record should also reflect any work stoppages during driving and details of the hammer and ancillary equipment being used. Copies of these records should be provided to the Contractor, the City's representative, and the Architect for review.

The Contractor should be responsible for notifying the Geotechnical Engineer at least 48 hours in advance of starting or restarting any pile driving work; he should not proceed with the work unless the Geotechnical Engineer is present on the site.

4. Submittals

The Contractor shall submit the following information before beginning work on the site:

- Shop drawings for concrete piles and pile caps showing placement of steel and any special embedded or attached lifting devices. (Any other pick-up points, or other pick-up method shall also be shown on drawings.) Drawings should conform to ACI 351.
- Before any concrete is placed in either piles or pile caps, Contractor should furnish for approval a concrete mix design for each type of concrete to be used.
- Manufacturer of fabricated piling.
- Mill certificates at the time the pile is delivered to the site.
- List of equipment intended to be utilized in the driving, noting hammer type, size, cushion types, lead lengths, and crane capacities.

5. Driving Equipment

- Hammer: Timber piles may be driven with either a steam or air operated pile hammer. Drop hammers should not be used. The hammer furnished should have a capacity at least equal to the hammer manufacturer's recommendation for the total weight of pile and character of subsurface conditions to be encountered. The driving energy of the hammer should not be less than 15,000 foot pounds per blow. All pile driving hammers should be maintained by the Contractor in good operating condition and operated according to the manufacturer's recommendations. Sufficient pressure shall be maintained at the steam hammer so that (1) for double-acting hammer, the number of blows per minute during and at the completion of driving of a pile is equal approximately to that at which the hammer is rated; (2) for single-acting hammer, there is a full upward stroke of the ram; and (3) for differential type hammer, there is a slight rise of the hammer base during each upward stroke.

- <u>Obstructions</u>: Piles that encounter refusal before reaching design tip elevation should be abandoned or pulled and a new pile driven.

- <u>Tolerances</u>: Piles should be driven by the Contractor as accurately as possible in the correct location. The final position of the pile head should not deviate more than 6 inches from the location indicated on the plans and should not deviate from the vertical by more than 1/4 inch per five feet of length. The Contractor should not pull piles into place after driving.

- <u>Heave Monitoring</u>: The Contractor should survey the piles before any other pile is driven within a 25-foot radius of that pile. The piles should be marked, or the elevation of the butt after cut off determined, so that the pile's elevation may be monitored for heave. The elevations should be determined within 0.01 foot and referenced to a permanent bench mark located on a fixed structure that will be unaffected by construction or pile driving activity on this site. Pile elevations should be resurveyed before timber decking or floating breakwater sections are installed and after driving of any piles within a 25-foot radius is complete. Piles that heave more than 1/2 inch should be redriven.

  Immediately after driving is completed any remaining steel strapping should be removed from the butts of timber piling and disposed. All timber pile heads (butts) should then be "fresh headed" at the elevations indicated on the plans, and the exposed timber coated with at least two coats of hot creosote.

7. <u>Pile Redriving</u>

Before beginning any redriving, the Contractor should be responsible for marking the 3-foot length of piles nearest the ground or water surface in 1-inch intervals. The depth should be clearly marked with enlarged numerals at each 6-inch increment.

When pile redriving has been satisfactorily completed the Contractor should, where necessary, fresh head and treat the top of timber or concrete piles as described in the preceding paragraph.

8. <u>Correction of Defective Work</u>

During driving the Geotechnical Engineer will examine the piles for alignment, crimping, buckling, splitting, visible breakage, or other irregularities. Piles that fail to meet the requirements of the plans and specifications, or for any other justifiable reason are unacceptable, shall be considered defective and will be rejected.

The Contractor should submit his plans for correcting any defective work to the Engineer for approval before correcting any of the work.

Additional piles should be driven by the Contractor to replace defective piles. All design, construction and inspection costs associated with pile replacement or changes in the foundation elements by reason of improper or defective pile installation, should be at the Contractor's expense.

Rejected piles should remain in the ground and should be cut or broken off at mudline or at least 2 feet below the final cutoff elevation. The Contractor should backfill any resulting hole with Select Fill or crushed rock before proceeding with any other work.

APPENDICES

APPENDIX A

Field Exploration

Subsurface conditions were explored at the site on August 21, 1990 using a truck-mounted, Mobile B-61 hollow-stem auger drill rig. Approximate boring locations are shown on the Site Plan, Figure 2. Borings were located in the field by measuring from existing structures. Boring elevations were estimated based on an assumed datum. Our field engineer monitored the drilling operations, logged the borings, and obtained relatively undisturbed soil samples for visual examination and subsequent laboratory testing. Soil samples were classified in accordance with the Unified Soil Classification System, as shown on the Soil Classification/Legend, Plate A1.

Soil samples were collected using either a split barrel sampler (2.4-inch inside diameter and 3.0-inch outside diameter), equipped with brass liner rings, or with a Standard Penetration Test (SPT) split-spoon sampler with dimensions in accordance with ASTM D1586. The split barrel sampler was driven with a 300-pound hammer falling 30 inches and the SPT sampler was driven with a 140-pound hammer falling 30 inches. Blow counts were recorded for the lower 12 inches of each sample.

Samples were sealed to limit moisture loss, labeled, and returned to our laboratory for further examination and testing. Summary logs of the borings, modified to reflect the results of laboratory examination and testing, are presented on Plates A2 through A5. The stratification lines, shown on the individual logs, represent the approximate boundaries between soil types; actual transitions may be either more gradual or more severe. The conditions depicted are for the date and location indicated only, and it should not necessarily be expected they are representative of conditions at other locations and times.

APPENDIX B

Laboratory Testing

A. General

We conducted laboratory tests on several representative soil samples to better assess the soil classification of the units encountered and to evaluate general physical properties and engineering characteristics. A brief description of the tests performed for this study is provided below. It is important to note that these test results may not accurately represent in-situ soil conditions. Our recommendations are based on interpretation of these test results and their use in guiding our engineering judgement. AGI cannot be responsible for the interpretation of these data by others.

In general accordance with our General Conditions, the soil samples for this project will be discarded after a period of 30 days following completion of this report unless we are otherwise directed in writing.

B. Soil Classification

Soil samples are visually examined in the field by our representative at the time they are obtained. They are subsequently packaged and returned to our laboratory where they are reexamined and the original description is checked and verified or modified. The samples are described in general accordance with the Unified Classification System, ASTM Test Method D-2487-83 and test data. The resulting descriptions are provided at the appropriate sample location on the individual boring log and are qualitative only. The attached Soil Classification/Legend, Appendix A, Plate A1, provides pictorial symbols that match the written descriptions.

C. Direct Shear Test

We performed direct shear tests on two of the relatively undisturbed samples to estimate the shear strength of the Sand soils. The tests were performed in general accordance with ASTM Test Method D-3080-72(79) on samples at the field moisture conditions and/or under increased moisture conditions. A normal load, appropriate to the anticipated foundation conditions, was applied to the test sample and the sample was then sheared under a constant strain control. The results of these tests are presented on Plates B-1 and B-2, Direct Shear Test.

D. One Dimensional Consolidation Test

To estimate compressibility of the soft soils underlying the site, we performed a one dimensional consolidation tests on a relatively undisturbed sample. These test, which was performed in general accordance with ASTM Test Method D-2435-80, was conducted on fully saturated samples. The results obtained provide an indication of the degree of plastic deformation of the soil with time and aid in making an approximation of the magnitude and rate of settlement of the compressible soils under the design loads with time. The results are presented on Plate B3, Consolidation Test.

E. Atterberg Limits

Because of the large amounts of fines in the sampled soils from the field, we deemed it necessary to perform several Atterberg Limit tests on the finer materials to evaluate plasticity characteristics and as an aid in accurate classification of the soils. These tests include the liquid and plastic limits which were performed in accordance with ASTM Test Methods D-423-66(72) and D-424-59(71), respectively. The Plastic Index, the difference between the liquid and plastic limits, is then estimated. The results of the liquid limit provide a measure of the tested soils shear strength and is analogous to the direct shear test. When coupled with the plastic index, the results help us to classify the in-place soils on the basis of these soil characteristics. The results of these tests are presented on Plate B4, Atterberg Limits.

F. Moisture Density

Moisture content and dry density tests were performed on several samples obtained from the boring. The purpose of these tests is to approximately ascertain the in-place moisture content and the associated dry unit weight (dry density) of the soil sample tested. The moisture content is determined in general accordance with the ASTM Test Method D-2216-80 and the dry unit weight is computed on the basis of this result and the volume of the sample container. The information obtained assists us by providing qualitative information regarding soil strength and compressibility. The results of these tests are presented at the appropriate sample depth on the boring log.

DISTRIBUTION

2 Copies

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733 Market Street, Suite 500
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Attention: Mr. Richard Foss

Quality Control/Technical Review by:

Vincent Lascko, P.E.
Associate Engineer

JMS/JBH/tag

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS					TYPICAL NAMES
COARSE GRAINED SOILS (MORE THAN HALF IS LARGER THAN NO. 200 SIEVE)	GRAVELS (MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE)	CLEAN GRAVELS WITH FINES LESS THAN 5%	GW		WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES
			GP		POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES
		GRAVELS WITH OVER 12% FINES	GM		SILTY GRAVELS, POORLY GRADED GRAVEL-SAND-SILT MIXTURES
			GC		CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES
	SANDS (MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE)	CLEAN SANDS WITH LESS THAN 5% FINES	SW		WELL GRADED SANDS, GRAVELLY SANDS
			SP		POORLY GRADED SANDS, GRAVELLY SANDS
		SANDS WITH OVER 12% FINES	SM		SILTY SANDS, POORLY GRADED SAND-SILT MIXTURES
			SC		CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES
FINE GRAINED SOILS (MORE THAN HALF IS SMALLER THAN NO. 200 SIEVE)	SILTS AND CLAYS (LIQUID LIMIT LESS THAN 50)		ML		INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL		INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL		ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS (LIQUID LIMIT GREATER THAN 50)		MH		INORGANIC SILTS, MICACEOUS OR DIATOMACIOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH		INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH		ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			Pt		PEAT AND OTHER HIGHLY ORGANIC SOILS

LEGEND

SAMPLE

- ■ "Undisturbed"
- ▨ Bulk/Grab
- ◫ Not Recovered
- ◪ SPT

CONTACT BETWEEN UNITS

- Well Defined Change
- Gradational Change
- Obscure Change
- End of Exploration

BLOWS/FOOT

Hammer is 300 pounds with 30-inch drop, unless otherwise noted

S - SPT Sampler (2.0 Inch O.D.) with 140 pound hammer, 30-inch drop

T - Thin Wall Sampler (2.8 Inch Sample)

H - Split Barrel Sampler (2.4 Inch Sample)

MOISTURE DESCRIPTION

- Dry - Considerably less than optimum for compaction
- Moist - Near optimum moisture content
- Wet - Over optimum moisture content
- Saturated - Below water table, in capillary zone, or in perched groundwater

LABORATORY TESTS

- Consol - Consolidation
- LL - Liquid Limit
- PL - Plastic Limit
- Gs - Specific Gravity
- SA - Size Analysis
- TxS - Triaxial Shear
- TxP - Triaxial Permeability
- Perm - Permeability
- Po - Porosity
- MD - Moisture/Density
- DS - Direct Shear
- VS - Vane Shear
- Comp - Compaction
- UU - Unconsolidated • Undrained
- CU - Consolidated • Undrained
- CD - Consolidated • Drained

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Soil Classification/Legend

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Tacoma, Washington

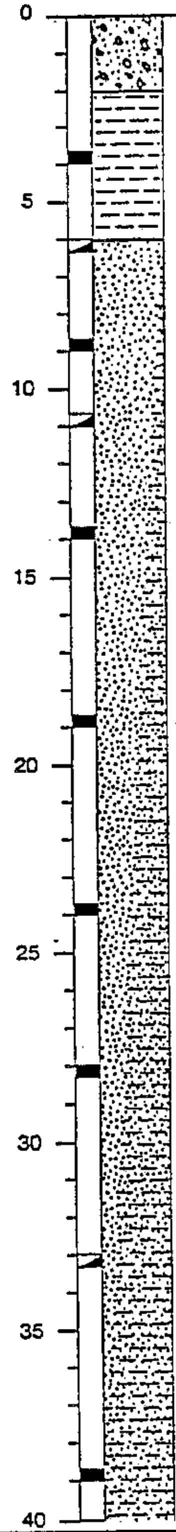
PLATE **A1**

JOB NUMBER	DRAWN	APPROVED	DATE	REVISED	DATE
15,523.001	SES	JMA	12 Sep. 90		

Equipment 8-inch Hollow-stem Auger (HSA)

Land Surface Elevation 100 feet (assumed) Date 8/21/90

Laboratory Tests	Moisture Content (%)	Dry Density (pcf)	Blows per Foot
MD	31.3	80	9
			9
MD	23.8	98	12
			6
			2
MD	21.6	104	29
DS,MD	18.7	104	44
MD	20.3	99	23
			44
MD	29.8	88	31



BROWN GRAVEL (GP) dense, moist; some sand (Fill).

DARK BROWN SANDY SILT (ML) stiff, moist; trace wood and gravel (Fill).

BLACK SAND (SP-SM) loose to medium dense, saturated; fine to medium grained, trace silt.

Grades with increasing silt.

Grades with occasional dense zones.

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Log of Boring 1 (0'-40')

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Tacoma, Washington

PLATE **A2**

JOB NUMBER	DRAWN	APPROVED	DATE	REVISED	DATE
15.523.001	SES	JMA	27 August 90		

Equipment: 8-inch HSA

Land Surface Elevation: 100 feet (assumed) Date: 8/22/90

Laboratory Tests	Moisture Content (%)	Dry Density (pcf)	Blows per Foot	Depth (feet)	Sample	Description
				40		
MD	24.4	102	28			Grades with clayey sand lenses.
				45		Groundwater encountered at approximately 5-foot depth during drilling.
						Boring backfilled with bentonite on 8/21/90.
				50		
				55		
				60		
				65		
				70		
				75		
				80		

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Log of Boring 1 (40'-44')

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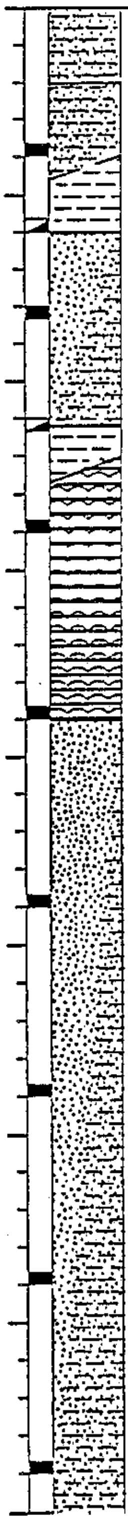
PLATE **A3**

JOB NUMBER	DRAWN	APPROVED	DATE	REVISED	DATE
15.523.001	SES	JMA	27 August 90		

Equipment 8-inch HSA

Land Surface Elevation 100 feet (assumed) Date 8/21/90

Laboratory Tests	Moisture Content (%)	Dry Density (pcf)	Blows per Foot	Depth (feet)	Description
				0	BROWN SILTY SAND (SM) medium dense, moist; some gravel (Fill).
					DARK BROWN SILTY SAND (SM) dense, moist; some gravel, debris, slight petroleum odor (Fill).
MD	13.1	109	59		
				5	BROWN SANDY SILT (ML) stiff, saturated; some gravel (Fill).
			10		
					BLACK SAND (SP-SM) loose, saturated; fine to medium grained, trace silt (Fill).
MD	20.8		7		
				10	
			3		Interlayered with Brown Peat at 11 feet.
					GRAY SILT (ML) soft, saturated.
					GRAY ORGANIC SILT (OH) soft, saturated.
CONSOL LL/PL, MD	55.7	67	2		
				15	
					Grades with some wood.
MD	60.4	65	21		
				20	GRAY SAND (SP-SM) medium dense, saturated; fine to medium grained, trace silt.
MD	23.6	102	13		
				25	
DS, MD	22.2	106	83		Grades with occasional dense zones.
				30	
MD	25.9	98	28		
				35	
			25		
				40	



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Log of Boring 2 (0'-40')

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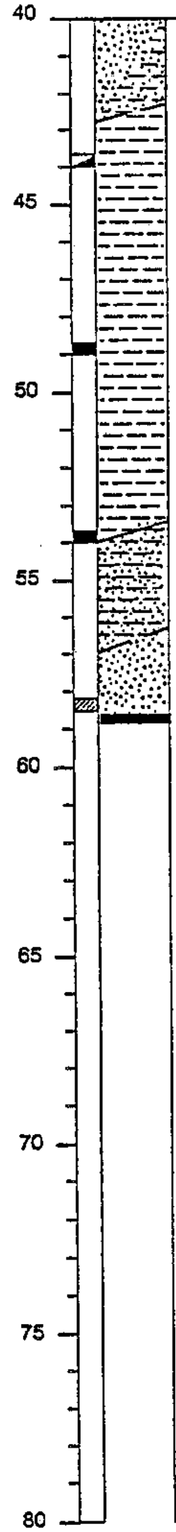
PLATE **A4**

JOB NUMBER	DRAWN	APPROVED	DATE	REVISED	DATE
15.523.001	SES	JMA	27 August 90		

Equipment 8-inch HSA

Land Surface Elevation 100 feet (assumed) Date 8/21/90

Laboratory Tests	Moisture Content (%)	Dry Density (pcf)	Blows per Foot	Depth (feet)
			9	
MD	29.8	95	35	
MD	23.6	104	11	
			96	



GRAY SILT (ML) stiff, saturated; some sand.

Grades with increasing sand.

GRAY SILTY SAND (SM) medium dense, saturated; fine to medium grained.

BLACK SAND (SP) dense, saturated; fine to coarse grained.

Groundwater encountered at approximately 5-foot depth during drilling.

Boring backfilled with bentonite on 8/21/90.

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Log of Boring 2 (40'-59')

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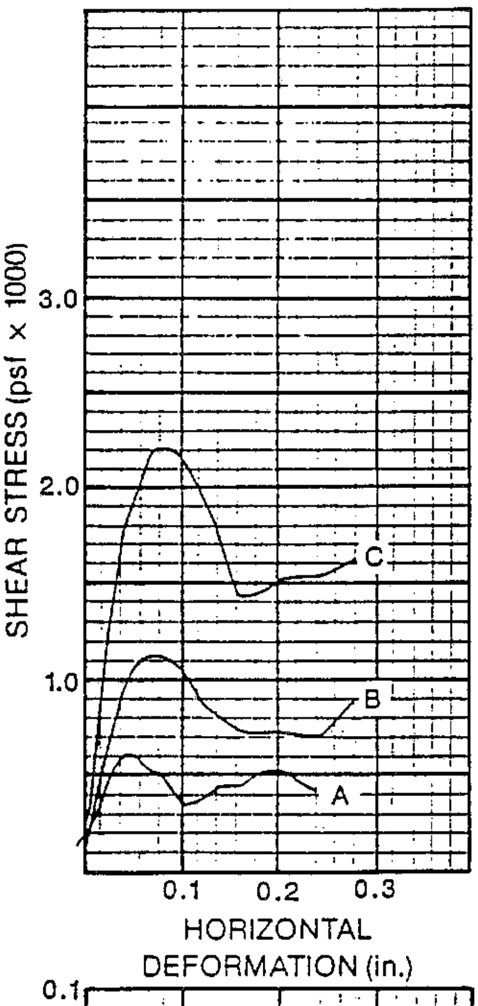
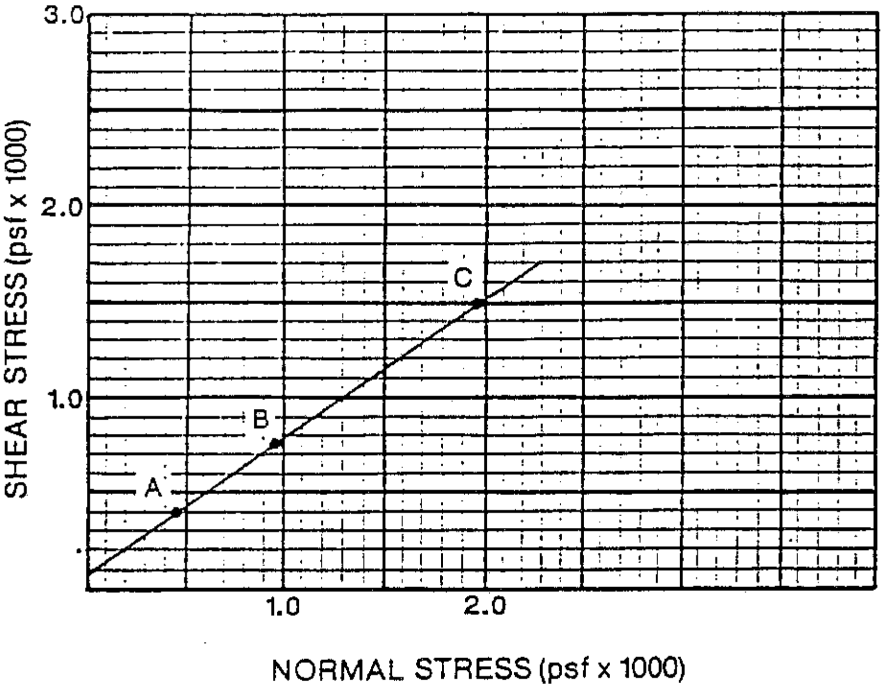
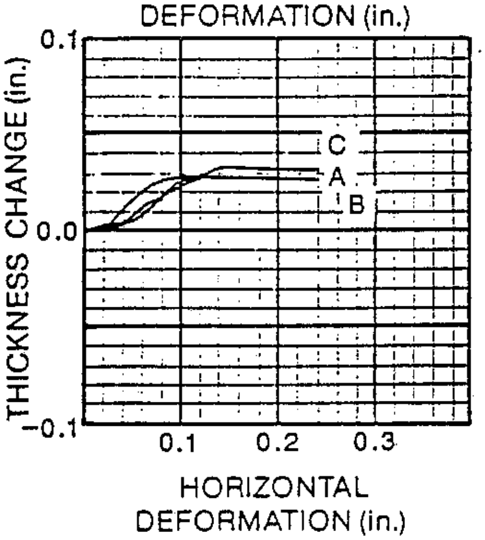
PLATE **A5**

JOB NUMBER	DRAWN	APPROVED	DATE	REVISED	DATE
15,523.001	SES	JMA	27 August 90		

Reference ASTM D 3080







$\phi' =$ 35° (ultimate)
$C' =$ 100 psf (apparent)

TEST TYPE: Consolidated/Drained   Controlled Strain

PHYSICAL CONDITIONS		TEST NO. A	TEST NO. B	TEST NO. C
INITIAL	Height (in)	1.000	1.000	1.000
	Water Content (%)	18.7	18.7	18.7
	Void Ratio	0.617	0.672	0.646
	Saturation (%)	99.2	91.5	97.4
	Dry Density (pcf)	103.5	100.1	101.7
BEFORE TEST	Time for 50% Consolidation (min)			
	Time for 95% Consolidation (min.)			
	Void Ratio after Dry Density (pcf)			
FINAL	Water Content (%)	27.7	27.9	26.4
	Void Ratio	0.631	0.668	0.651
	Saturation (%)	100.0	100.0	100.0
Normal Stress (psf)		484.8	969.5	1973.7
Maximum Shear (psf)		620.0	1142.0	2218.0
Time to Failure (min)				
Sample Source		B1 @ 24 feet (depth)		
Classification		SP-SM		G_s 2.68

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Direct Shear Test

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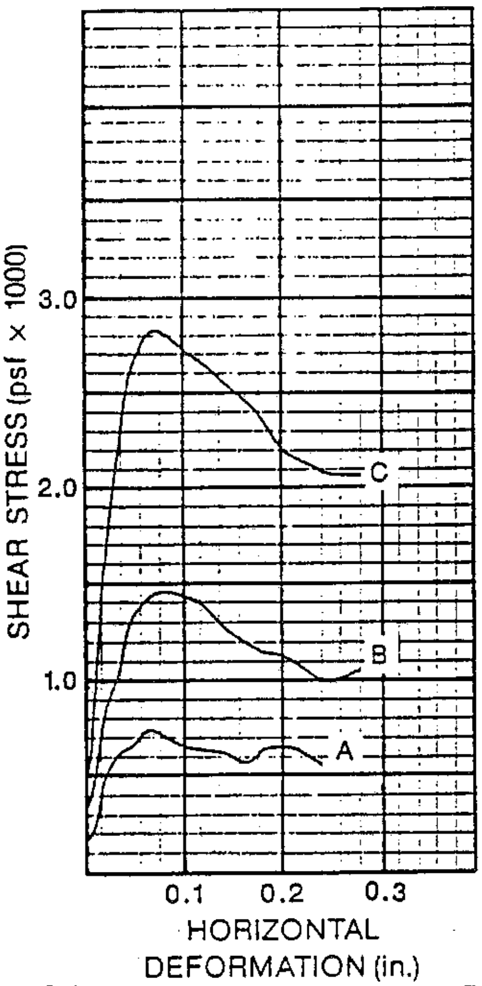
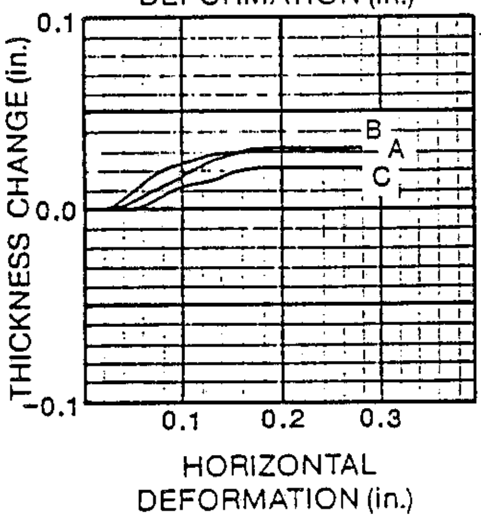
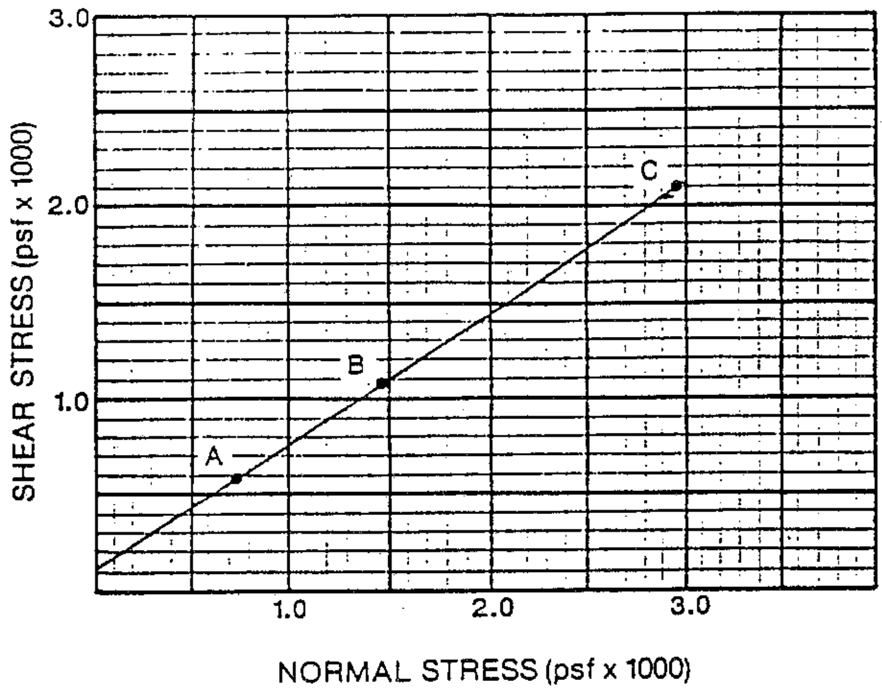
PLATE **B-1**

JOB NUMBER	DRAWN	APPROVED	DATE	REVISED	DATE
15,523.001	MCT	Alut	10 Sep. 90		

Reference ASTM D 3080







$\phi' = 38°$ (ultimate)
$C' =$ 100psf (apparent)

TEST TYPE: Consolidated/Drained Controlled Strain

PHYSICAL CONDITIONS		TEST NO. A	B	C
INITIAL	Height (in)	0.990	1.000	1.00
	Water Content (%)	22.2	22.2	22.2
	Void Ratio	0.580	0.603	0.60
	Saturation (%)	99.9	99.5	34.51
	Dry Density (pcf)	105.9	104.4	104.4
BEFORE TEST	Time for 50% Consolidation (min)			
	Time for 95% Consolidation (min.)			
	Void Ratio after Dry Density (pcf)			
FINAL	Water Content (%)	23.8	24.7	23.5
	Void Ratio	0.594	0.612	0.363
	Saturation (%)	100.0	100.0	100.0
Normal Stress (psf)		727.1	1488.9	2977.8
Maximum Shear (psf)		717.0	1454	2810.0
Time to Failure (min)				
Sample Source		B2 @ 29 feet (depth)		
Classification		SP-SM		G_s2.68

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Direct Shear Test

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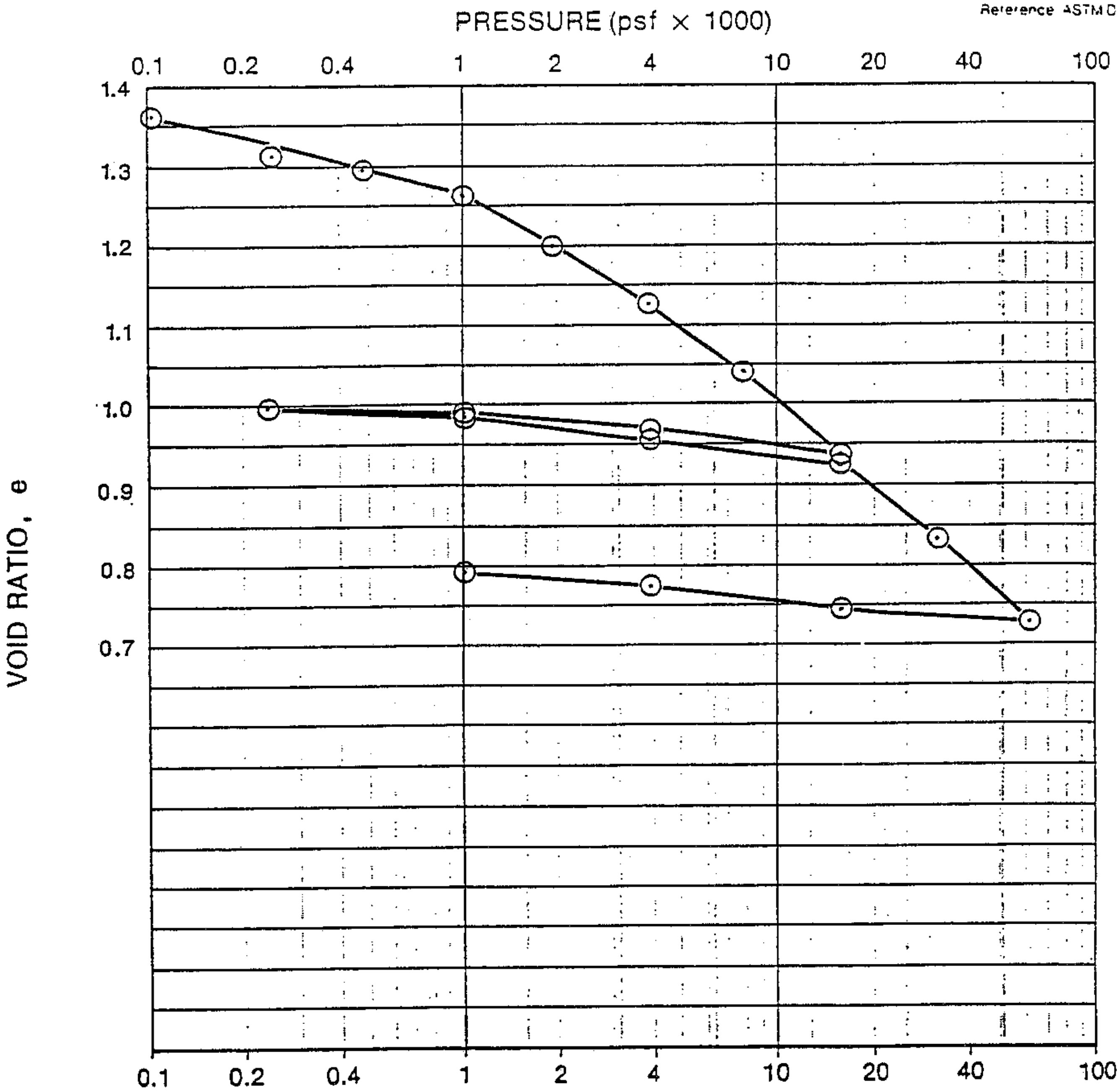
PLATE
B-2

JOB NUMBER	DRAWN	APPROVED	DATE	REVISED	DATE
15,523.001	MCT	Jms	10 Sep. 90		

Reference ASTM D 2435



VOID RATIO — PRESSURE CURVE

Type of Specimen D+M		Condition		Before Test		After Test	
Diameter (in.) 2.41	Height (in.) 0.743	Water Content	w_o	55.7 %	w_f	29.6 %	
Overburden Press., P_o psf		Void Ratio	e_o	1.481	e_f	0.789	
Preconsol. Press., P_c psf		Saturation	S_o	100.0 %	S_f	100.0 %	
Compression Index, C_c		Dry Density	γ_d	67.2 pcf	γ_d	93.2 pcf	
LL 51	PL 31	PI 20		G_s 2.67 (measured)			
Classification Organic Silt (OH)				Source B2 @ 13.5 feet (depth)			

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Consolidation Test

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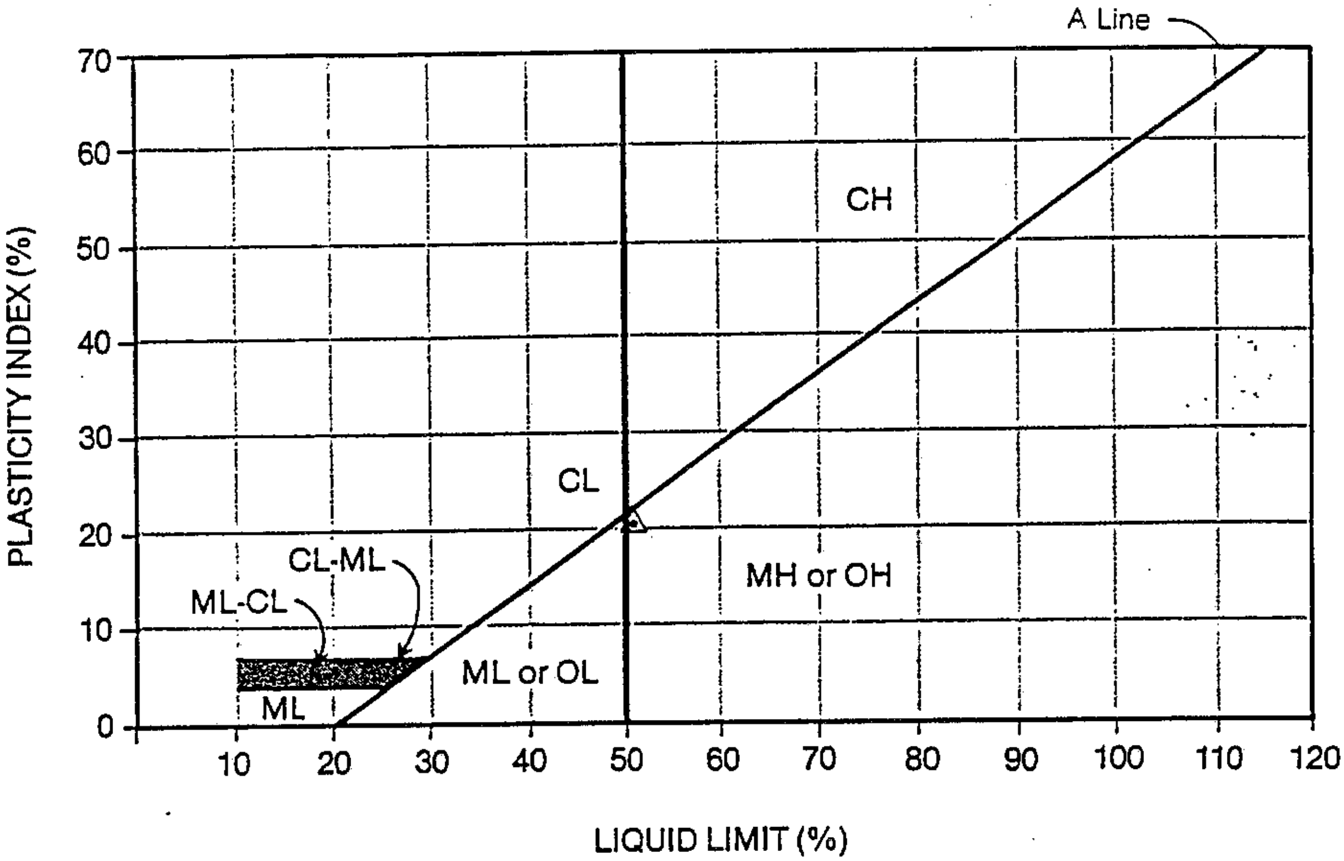
Tacoma, Washington

PLATE **B-3**

JOB NUMBER	DRAWN	APPROVED	DATE	REVISED	DATE
15,523.001	MCT	[signature]	10 Sep. 90		

Reference: ASTM D 423, 424



Symbol	Source	Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
▲	B2 @ 13.5 feet (depth)	Organic Silt (OH)	53.9	51	20	

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Atterberg Limits
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PLATE
B-4

JOB NUMBER	DRAWN	APPROVED	DATE	REVISED	DATE
15,523.001	MCT	[signature]	10 Sep. 90		