

Interim
Soil SR 4/17/91
an
RECEIVED

APR 18 1991

DEPT. OF ECOLOGY

**Geotechnical Report and
Environmental Summary
Proposed TRAMCO Complex
Expansion - Paine Field
Everett, Washington**

March 1991

**Sierra Construction Company, Inc./
TRAMCO
Paine Field
3226 111th Street S.W.
Everett, WA 98204**



SHANNON & WILSON, INC.

400 N. 34th St., Suite 100
P.O. Box C-30313
Seattle, WA 98103
(206) 632-8020

TABLE OF CONTENTS

	<u>Page</u>
1.0 INTRODUCTION AND SCOPE OF WORK	1
1.1 Geotechnical Services	1
1.2 Environmental Services	2
2.0 SITE DESCRIPTION	2
3.0 PROJECT DESCRIPTION	3
3.1 Geotechnical Aspects	3
3.2 Environmental Aspects	4
3.3 General	4
4.0 FIELD PROGRAM	4
5.0 GEOTECHNICAL LABORATORY PROGRAM	4
6.0 PREVIOUS EXPLORATIONS AND INFORMATION	5
7.0 CHEMICAL LABORATORY ANALYSIS	5
8.0 SUBSURFACE CONDITIONS	5
9.0 DESIGN RECOMMENDATIONS	6
9.1 General	6
9.2 Foundation and Slab Support Alternatives	7
9.3 Shallow Foundations	8
9.4 Drilled Piers	10
9.4.1 Design Parameters	10
9.4.2 Installation of Drilled Piers	11
9.4.3 Augercast Pile Design Parameters	12
9.4.4 Installation of Augercast Piles	14
9.5 Concrete Hangar and Approach Slab Support	14
9.6 Concrete Floor Slabs	16
9.7 Site Grading	16
9.7.1 General	16
9.7.2 Site Preparation	16
9.7.3 Excavations and Fills	17
9.7.4 Temporary Excavation Slopes	19

TABLE OF CONTENTS (continued)

	<u>Page</u>
9.8 Retaining Walls and Foundation Walls	20
9.9 Drainage	21
9.10 Wet Weather Considerations	22
9.11 Utilities	23
10.0 ADDITIONAL STUDIES	23
11.0 DOCUMENT REVIEW AND CONSTRUCTION OBSERVATIONS	24
12.0 LIMITATIONS	24

**GEOTECHNICAL REPORT AND ENVIRONMENTAL SUMMARY
PROPOSED TRAMCO EXPANSION
PAINE FIELD, EVERETT, WASHINGTON**

1.0 INTRODUCTION AND SCOPE OF WORK

This report summarizes the results of geotechnical explorations and recommendations, and environmental explorations and chemical laboratory testing for a proposed hangar, attached shop/office building, parking garage, and warehouse, to be located at the Snohomish County Airport (Paine Field) in Everett, Washington. The purpose of our work was to explore and identify the general subsurface soil and groundwater conditions at the site, provide geotechnical engineering recommendations related to structure and pavement foundations, discuss construction considerations as related to the geotechnical aspects of the project, and evaluate the potential for petroleum hydrocarbon contamination beneath the area where the new structures are to be located.

Shannon & Wilson, Inc. was retained by Sierra Construction Company, Inc. to perform the following scope of work:

1.1 Geotechnical Services

- Drill 8 borings and excavate 4 test pits at the site. Perform field California Bearing Ratio (CBR) tests at the test pit locations. During the field program, an additional boring was added in the planned parking structure area to better define subsurface conditions.
- Perform geotechnical laboratory tests on select soil samples for purposes of classification and to determine pertinent engineering properties, including moisture content determinations, grain size analyses, and laboratory CBR tests.
- Provide a description of the field explorations and geotechnical laboratory tests performed.
- Provide a summary of subsurface conditions, a site plan showing the exploration locations, and a log of each exploration.
- Provide recommendations for foundation support, allowable bearing pressures, resistance to lateral loads, and anticipated total and differential settlements.
- Provide recommendations for subgrade preparation for slabs on grade, including modulus of subgrade reaction values for use in designing new concrete slabs.

- Discuss earthwork, fill, and excavation considerations, including suitability of reusing on-site soils as fill, compaction recommendations, and wet-weather earthwork considerations.
- Provide earth pressures on below grade structure and retaining walls.
- Discuss subsurface drainage considerations, as appropriate.

1.2 Environmental Services

- Drill 3 borings for the collection of soil samples; install permanent monitoring wells in the boreholes for the collection of representative groundwater samples for laboratory analysis.
- Perform soil and water chemical laboratory testing for (1) petroleum hydrocarbons (gasoline and diesel) by EPA Modified Method 8015; (2) aromatic compounds benzene, toluene, ethylbenzene, and xylenes (BTE and X) by EPA Methods 8020 for soil and 602 for groundwater; and (3) total lead (for soil samples only) by Inductively Coupled Plasma Spectroscopy (ICP). Upon completion of field activities, testing for aromatic compounds in groundwater was repeated in an effort to duplicate initial results.
- Present the results of the field program and chemical tests and provide conclusions and recommendations regarding the data collected.

The above services were conducted in general accordance with our proposal dated January 14, 1991 and verbally authorized by Sierra Construction Company, Inc. (Sierra) on January 14, 1991. Throughout the course of this study, additional work was necessitated by the results obtained in the field. This additional work was authorized by Sierra, prior to completing such work.

2.0 SITE DESCRIPTION

The proposed site property consists of an approximately 30-acre parcel located at the Snohomish County Airport (Paine Field) in Everett, Washington. In particular, the site is approximately bounded at the southern end by Navy lease area (located to the south of 112th Street S.W.), by 109th Street S.W. to the north, by 31st Avenue W. to the east, and by an undeveloped area that extends to runway 34L to the west. The site is generally gently sloping and developed with numerous buildings and roadways. The existing topography is such that the ground surface generally slopes downward to the south. Maximum relief over the site is estimated to be 30 feet.

According to available utility plans and from markings made by utility companies in conjunction with our field work, numerous underground utilities traverse throughout the site. In addition, existing underground storage tanks are reportedly present at the site. We understand that these tanks will be removed before construction and appropriate studies will be conducted prior to or during their removal.

3.0 PROJECT DESCRIPTION

3.1 Geotechnical Aspects

We understand that the proposed project involves the construction of a new hangar, an attached two-story shop/office building, a three-story above grade parking garage, a warehouse, new concrete ramps and approach slabs, and a retaining wall along the north side of the site. The proposed hangar structure will be approximately 1,000 feet by 250 feet in plan dimension with a finished floor elevation of 582 feet (local datum). Maximum height of the hangar will be about 90 feet. The attached two-story shop/office building will be an L-shaped structure located along the south and east sides of the hangar. This structure will have a width of approximately 120 feet and a total length of approximately 1,250 feet. The three-story parking garage will be located just south of 112th Street S.W. and west of 31st Avenue W. The parking garage will have plan dimensions of approximately 300 feet by 500 feet. The finished floor elevation of this structure has not been determined, but is expected to be between elevation 572 and 582 feet. The warehouse is tentatively planned to be located to the west of the parking garage. The warehouse will have plan dimensions of approximately 360 feet by 300 feet and a finished floor elevation near 582 feet.

A new retaining wall will be constructed along the south side of 109th Street S.W. in order to make the required excavation to the hangar finished floor elevation. Maximum wall height and excavation depth will be about 15 feet.

The majority of the exterior walls of the proposed hangar and shop/office structures are to consist of tilt-up concrete construction to a height of 30 feet and metal siding above this height. The north side of the hangar, however, will consist of a series of sliding steel doors about 50 feet in height that will ride on tracks at floor level. These retractable doors will serve as an entrance way for aircraft. The parking garage will consist of post-tensioned concrete construction.

The floor of the structures and the approach ramps and staging areas will consist of slab on grade concrete.

Loads for the proposed structures are not known at this time. However, hangar and shop/office building loads are anticipated to be similar to the existing hangar and attached shop/office building located to the northeast of the subject site. We understand that these existing structures have approximately the following loads: main truss column loads in the hangar of approximately 1,400 kips each, exterior column loads of up to 150 kips, exterior wall loads from 1 to 5 kips per lineal foot (klf), and loads of about 5 klf for the sliding steel doors.

3.2 Environmental Aspects

Previous explorations by Converse Consultants NW indicated the presence of residual petroleum hydrocarbon compounds in the site soils and groundwater at three sites located to the north and east of the proposed structures. We understand that the Corps of Engineers will direct the cleanup of these sites. The purpose of the current study was to evaluate the potential for residual hydrocarbons to the west of 31st Avenue W., where the new structures are to be located.

3.3 Additional Considerations

If there are changes in project design or location, the conclusions and recommendations presented may not be applicable. If changes are made, we should be retained to review our conclusions and recommendations and to provide a written modification or confirmation.

4.0 FIELD PROGRAM

Subsurface conditions were explored by drilling 12 exploratory borings and excavating 4 test pits. The borings were drilled to depths ranging from 30.5 to 45.5 feet below the existing ground surface between January 24 and 30, 1991. The test pits were excavated to depths ranging from 6 to 9 feet on February 6, 1991. Approximate locations of borings and test pits are shown on the Site and Exploration Plan, Figure 1. Logs of the explorations are presented in Appendix A, along with details of the exploration program.

5.0 GEOTECHNICAL LABORATORY PROGRAM

Laboratory tests were performed on soil samples obtained in the field to aid in classifying the soils and to determine pertinent engineering properties. The laboratory testing program included

visual classification, moisture content determinations, grain-size analyses, and laboratory California Bearing Ratio (CBR) tests. Details and results of the laboratory testing program are included as Appendix B to this report.

6.0 PREVIOUS EXPLORATIONS AND INFORMATION

To aid in identifying subsurface conditions beneath the proposed site, existing information and previous explorations were compiled and evaluated to develop and/or confirm geotechnical conditions at the site and environmental laboratory test results. The primary sources utilized were previous geotechnical engineering reports from Converse Consultants NW and Earth Consultants, Inc. and an environmental report from Converse Consultants NW. Logs of existing explorations from these reports are included in Appendix C. Approximate locations of these explorations are shown on Figure 1.

7.0 CHEMICAL LABORATORY ANALYSIS

Soil and groundwater samples from borings BH-103, BH-107, and BH-111 were analyzed by Freidman & Bruya, Inc., Seattle, Washington. Results of the chemical laboratory analysis are included as Appendix D to this report.

8.0 SUBSURFACE CONDITIONS

In general, subsurface conditions consisted of sod and topsoil or asphalt pavement overlying fill, all of which are underlain by native, dense to very dense, glacial soil deposits. The exploration logs should be consulted for detailed descriptions of the conditions encountered in the explorations.

Specifically, up to about 3 inches of asphalt pavement and up to about 6 inches of sod and topsoil were observed in the explorations. Below these materials, fill soils were observed to depths of up to 15 feet. Fill soils varied in type and density with little continuity between material types over short distances. Fill soils were typically classified as slightly gravelly to gravelly, silty sand and for the most part appeared to be reworked native soils. In a few of the explorations, a highly organic layer was encountered below the fill soils. This layer was interpreted as being the original ground surface (topsoil horizon).

In borings BH-101, BH-108 and BH-110, up to 7 feet of medium dense native soils were observed below fill before encountering dense to very dense glacial soils. However, generally these glacial soils were encountered immediately below the fill and any old topsoil horizon. Contours of the estimated elevation of the top of the glacial soils are presented on Figure 1. The glacial soils were classified as slightly gravelly to gravelly, silty sand. As indicated in Table B-1 in Appendix B, the percentage of fines (soil smaller than the No. 200 U.S. Standard Sieve size) varied from 38 to 65 percent on five different tested samples. However, based on our experience, the actual percentage may be as low as 15 to 20 percent and higher than 65 percent.

Fill soils were observed to be moist to wet. Native, glacial soils were generally classified as moist, although occasionally wet.

The lines designating the interface between soil materials on the exploration logs should be considered as reasonably accurate only at the exploration locations. Soil conditions at other locations may differ from conditions occurring at these areas. In addition, the transition between the material types may be abrupt or gradual at other locations.

Groundwater levels encountered during drilling and excavation and measured thereafter in observation wells installed in 5 of the boreholes are shown on the logs of the explorations. These levels should only be considered reasonably accurate at the locations measured and at the time of measurement. Seasonal groundwater fluctuations are anticipated to occur depending on the amount of rainfall and infiltration and percolation of surface water. It should be noted that heavy groundwater seepage was observed in test pit, TP-1. The seepage was so heavy that the excavation could not be bailed with the bucket of the backhoe. In addition, the sides of the excavation caved because of this heavy groundwater inflow.

9.0 DESIGN RECOMMENDATIONS

9.1 General

The results of environmental explorations and chemical laboratory analysis suggest that elevated concentrations of benzene and total xylenes in groundwater samples obtained from monitoring wells BH-107 and BH-111 do exist. However, the results could not be duplicated in subsequent sampling and testing, which may indicate that the wells are on the fringe of a plume of dissolved hydrocarbons. The extent of this plume could not be established based on the limited exploration program conducted over the site. Based on the results of the environmental sampling and chemical laboratory analysis (included in Appendix D), it is our opinion that construction is still

possible at the site. However, it is likely that collection and treatment of groundwater during excavation and from perimeter drains for footings and retaining walls following construction will be required. It is anticipated that the economic feasibility of construction will be dependent on the extent and degree of residual hydrocarbons in groundwater and the volume of groundwater that could be generated during construction. At this time, these parameters are not known and additional studies are recommended, as described in Section 10.0 of this report. For the purpose of this report, it is assumed that treating groundwater is an acceptable economical alternative and construction of the proposed structures will proceed. If this is not the case, alternative approaches will be necessary, such as raising structure grades and accomplishing construction in a manner to avoid encountering groundwater.

On the basis of the information obtained during this exploration program, it is our opinion that the site is suitable for construction of the proposed facilities. Portions of the structures can be supported on shallow foundations with concrete slabs on grade. However, in areas, deep fill soils or other unsuitable materials were encountered that are not suitable for foundation support. In these areas, foundations should be extended down to the glacial soils by a deep foundation system. Appropriate foundation types include augercast piles and drilled piers. Groundwater seepage could be significant both during and after construction. Therefore, both temporary and permanent drainage provisions are recommended. Specific design parameters and recommendations for each of the above items are presented in the subsequent sections of this report.

9.2 Foundation and Slab Support Alternatives

We understand that the hangar, attached two-story shop/office building, and parking garage will have relatively high compressive loads and are sensitive to settlement. The warehouse is a relatively light, flexible structure and can tolerate a greater amount of total and some differential movement than the other structures. Based on the estimated elevations to the dense to very dense glacial soil deposits shown on Figure 1, all four of the structures are located such that the depth to these soils ranges from 0 to up to 15 feet below the proposed final floor elevations.

Figure 1 presents contours of estimated elevations of the bearing soils based on available explorations completed in the project area. These depths are approximate as boring locations and elevations were compiled from a variety of sources, including those listed in Appendix C, and variations in depth to the surface of the bearing as shown by the contours should be anticipated. The contours shown on Figure 1 should only be used for preliminary planning and cost estimation purposes. Actual depths to bearing soils beneath the structures should be determined in the field by a qualified geotechnical engineer during construction.

In order to reduce the potential for post-construction settlement, we recommend that foundations for the proposed hangar, shop/office building, and parking garage, extend down and bear in the dense to very dense glacial soils. In areas, this can be accomplished by conventional, shallow spread and strip footings. However, in other areas either overexcavation and replacement with a lean mix concrete monolith or the use of deep foundations will be required. The actual method used will depend on the depth to the dense glacial soils, the ability to control subsurface groundwater, and the economics of overexcavation and replacement versus deep foundation support. At locations where there is a transition of foundation support from shallow to deep or vice versa, the structural engineer should anticipate some differential settlement and consider this potential movement in the design of the structure(s).

In our opinion, the warehouse can be supported on shallow foundations bearing in either the medium dense native soils or dense to very dense glacial soils or in properly placed and compacted structural fill placed on top of these soils. We understand that this structure is relatively flexible and thus there is no reason to deepen the footings through any new fill placed for floor slab construction. Because of the differences in soil strength characteristics, there will be a potential for post-construction differential settlement. This should be taken into account by the structural engineer. In addition, in order to avoid confusion during construction (by encountering differing or unanticipated subsurface conditions necessitating the need for foundation redesign), all footings should be designed for the lower allowable soil bearing pressure (i.e., structural fill).

Concrete slabs on grade for the new ramps and approach slabs may be placed on existing fill soils with some subgrade preparation. In our opinion, floor slabs for structures should be supported directly on medium dense native soils or dense to very dense glacial soils (existing fill and old topsoil horizons removed first), on properly placed and compacted structural fill placed on top of these soils (i.e., overexcavate and replace with structural fill), or be designed as a structural slab supported by a deep foundation system. Again, the actual method used will depend on the depth to suitable soils, the ability to control subsurface groundwater, and the economics of overexcavation and replacement versus deep foundation support. If the owner is willing to accept the risk of future settlements, the hangar floor slab may be placed on existing fill soils.

Specific details and design parameters are presented in the following sections.

9.3 Shallow Foundations

Conventional footings should be founded in bearing soils, defined herein as the dense to very dense, glacial soil deposits, or on a lean mix concrete monolith placed on the bearing soils. The exception to the above is for footings for the warehouse, which may be founded in structural fill that is placed immediately on top of the medium dense native soils or bearing soils (i.e., fill and non-suitable native soils are removed and structural fill compacted on top of this excavated surface). New fill should not be placed over the surface of the existing soils for warehouse footings.

If loose, wet, or disturbed soils are encountered at foundation subgrades, these soils should be removed to expose the undisturbed foundation soils, and the resulting overexcavation backfilled with lean concrete (or compacted structural fill as an option for warehouse footings only). The base of all excavations should be dry and free of loose soil at the time of concrete placement.

Individual column footings and continuous wall footings bearing in the undisturbed bearing soils may be designed using a maximum allowable soil bearing pressure of 8,000 pounds per square foot (psf). Warehouse footings may be designed using an allowable soil bearing pressure of 4,000 psf. This lower value reflects the founding of footings in structural fill or medium dense native soils as encountered in boring BH-101. Recommended bearing pressures may be increased by one-third to include short-term loads such as those from wind or earthquakes. Column footings and continuous wall footings should have minimum widths of 24 inches. All footings should be embedded at least 24 inches below the lowest adjacent exterior grade or the interior floor slab grade.

Individual footings designed for the bearing pressures noted above are anticipated to settle a maximum of about 1/2- to 3/4-inch for the former case and 1-inch for the latter case. Differential settlement between adjacent footings could approach the total settlement. The majority of this settlement is expected to take place during the construction period, as the load is applied.

Lateral loads may be resisted by friction along the base of foundations and by passive soil resistance against buried foundations and walls. Footings cast directly in the undisturbed bearing soils or on properly compacted structural fill placed on top of these soils may be designed using a coefficient of base friction of 0.4. This value includes a factor of safety of 1.5. Passive soil resistance may be calculated based on an equivalent fluid density of 300 pounds per cubic foot (pcf). This value includes a factor of safety of 1.5 in order to reflect limited lateral deformations of less than one percent of the embedded depth. This value is based on the assumption of a

horizontal surface beyond the footing or wall of at least two times the depth of embedment in the direction of wall movement. Passive resistance should be ignored in the upper 12 inches if not covered by floor slabs or pavements or ignored entirely if future development will result in removal of the soils providing resistance.

9.4 Drilled Piers

9.4.1 Design Parameters

Foundation support at heavy column locations may be provided by drilled piers that penetrate into the dense to very dense glacial soils. The piers should penetrate a minimum distance of 5 feet into the dense to very dense glacial soils (bearing soils) or to a minimum depth of 10 feet below the final slab-on-grade elevation, whichever is greater.

The piers will derive support from end bearing and frictional resistance in the bearing soils. Straight-shafted drilled piers of 4-foot- and 5-foot-diameter may be designed for an allowable load of 390 and 490 kips, respectively. These values include a factor of safety of 2. The allowable loads are for frictional resistance over a 5-foot length of the pile. The allowable downward capacity of the piers are for total dead plus live loads and may be increased by one-third to accommodate the transient portion only of loads that include wind and/or seismic forces.

The vertical downward capacity recommended above is based on soil strength characteristics only. Pier capacities based on the strength of pier materials should be determined by the structural engineer.

Piers should be spaced a minimum center-to-center distance of three pier diameters to reduce group effects. For the anticipated loads stated earlier, we estimate that settlement of properly installed piers, as described above, will be on the order of 1/2-inch.

Lateral resistance and deflection of pier foundations are governed primarily by the soil materials along the upper portion of the pier. At the present time, lateral loads are not known. Therefore, we have provided lateral capacity based on assumed 1/4-inch of lateral deflection. Recommended allowable pier capacity and related structural moment information are provided in Table 1. This table is based on the assumption of a 20-foot-long pier embedded 5 feet into the glacial soils. If design conditions are significantly different or if the assumed deflection is inappropriate for structural design, we should be retained to provide additional recommendations.

TABLE 1
LATERALLY LOADED DRILLED PIER DESIGN DATA

	<u>4-Foot- Diameter</u>	<u>5-Foot- Diameter</u>
<u>Axial Load (kips)</u>	390	490
FREE HEAD		
Lateral Capacity, kips ^(a)	22	24
Maximum Moment, foot-kips	128	142
Depth to Negative Deflection, feet	17	17
Depth to Maximum Moment, feet	9	9
FIXED HEAD		
Lateral Capacity, kips ^(a)	100	160
Maximum Positive Moment, foot-kips	4	0
Maximum Negative Moment, foot-kips	1,175	2,183
Depth to Max Pos Moment, feet	18	>20
Depth to Max Neg Moment, feet	0	0
Depth to Negative Deflection, feet	20	>20

(a) Lateral capacity is based on 1/4-inch deflection at the ground surface for the assumed axial load.

For strain levels compatible with the described lateral deflections, we recommend a lateral passive pressure equivalent to that of a fluid with a density of 300 pounds per cubic foot (pcf) and a coefficient of base friction of 0.4 be used for calculating lateral soil resistance against buried grade beams and pier caps. The passive lateral soil resistance and coefficient of friction include a factor of safety of 1.5.

9.4.2 Installation of Drilled Piers

Based on our subsurface explorations, it is our opinion that the drilled piers may be excavated with conventional equipment, such as an auger on a hydraulic kelly bar, for the depths required. However, limited test borings cannot accurately define soil conditions at all locations of a site. Although no large boulders or other obstructions were encountered in the explorations, it is our experience that fill and glacial soils can contain obstructions that cannot be removed with conventional soil augering equipment.

The soils encountered in the borings are wet and in areas contain few fines. During our field explorations, soils would often slough into those boreholes that encountered water once the augers were removed. Thus, sloughing should be expected if water is encountered and casing of the hole will likely be required. Casing should be installed such that it penetrates into the

glacial soils, to seal off water for the short term case. Casing will increase the time and cost required for installation and should be included in construction estimates.

No more than 2 inches of standing water should be allowed at the bottom of a pier shaft at the time concrete is placed. Casing removal during concrete placement should be performed by an experienced contractor so that a void (due to arching) is not created by having too much concrete in the casing. Drilling and concrete placement adjacent to piers that have not fully cured may cause a disturbance. We recommend that no truck, heavy equipment, or new pier construction be allowed within 8 feet of piers less than 12 hours old.

9.4.3 Augercast Pile Design Parameters

Foundation support at lighter column locations and wall locations may be provided by augercast piles that penetrate into the dense to very dense glacial soils. The augercast piles should penetrate a minimum distance of 5 feet into the dense to very dense glacial soils (bearing soils) or to a minimum depth of 10 feet below the final slab-on-grade elevation, whichever is greater.

The piles will derive support from end bearing and frictional resistance in the bearing soils. Augercast piles of 12-inch- and 16-inch-diameter may be designed for an allowable load of 40 and 60 kips, respectively. These values include a factor of safety of 2. The allowable loads are for frictional resistance over a 5-foot length of the pile. The allowable downward capacity of the piers are for total dead plus live loads and may be increased by one-third to accommodate the transient portion only of loads that include wind and/or seismic forces.

The vertical downward capacity recommended above is based on soil strength characteristics only. Pile capacities based on the strength of pile materials should be determined by the structural engineer.

Piles should be spaced a minimum center-to-center distance of three pile diameters to reduce group effects. For the anticipated loads stated earlier, we estimate that settlement of properly installed piles, as described above, will be less than 1/2-inch.

Lateral resistance and deflection of pile foundations are governed primarily by the soil materials along the upper portion of the pile. Recommended allowable pile capacity and related structural moment information are provided in Table 2. This table is based on the assumption of a 20-foot-long pile embedded 5 feet into the glacial soils. This table is based on the assumption of a 20-foot-long pile embedded 5 feet into the glacial soils. If design conditions are significantly different or if the assumed deflection is inappropriate for structural design, we should be retained to provide additional recommendations.

TABLE 2
LATERALLY LOADED AUGERCAST PILE DESIGN DATA

	<u>4-Foot-Diameter</u>	<u>5-Foot-Diameter</u>
<u>Axial Load (kips)</u>	40	60
FREE HEAD		
Lateral Capacity, kips ^(a)	4	6
Maximum Moment, foot-kips	12	22
Depth to Negative Deflection, feet	9	11
Depth to Maximum Moment, feet	5	6
FIXED HEAD		
Lateral Capacity, kips ^(a)	10	18
Maximum Positive Moment, foot-kips	10	25
Maximum Negative Moment, foot-kips	34	88
Depth to Max Pos Moment, feet	7	9
Depth to Max Neg Moment, feet	0	0
Depth to Negative Deflection, feet	11	15

(a) Lateral capacity is based on 1/4-inch deflection at the ground surface for the assumed axial load.

For strain levels compatible with the described lateral deflections, we recommend a lateral passive pressure equivalent to that of a fluid with a density of 300 pounds per cubic foot (pcf) and a coefficient of base friction of 0.4 be used for calculating lateral soil resistance against buried grade beams and pile caps. The passive lateral soil resistance and coefficient of friction include a factor of safety of 1.5.

9.4.4 Installation of Augercast Piles

Augercast piles are installed by rotating a hollow-stem auger to the depth required. Concrete grout is then pumped under pressure through the hollow stem as the auger is withdrawn from the hole. After placement of the grout, we recommend that a continuous reinforcing bar (or reinforced cage, as necessary) be installed along the pile for the full length as an indicator that each pile consists of a continuous column of grout. The quality and strength of augercast piles are highly dependent on the installation procedure and the experience of the contractor. We recommend that pile installation be performed only by a contractor specializing in augercast piles and who has experience in penetrating into the very dense glacial soils as encountered in the Seattle area.

Although no large boulders or other obstructions were encountered in the explorations, it is our experience that fill and glacial soils can contain obstructions that cannot be removed with conventional soil augering equipment. If obstructions are encountered during pile installation, it may be necessary to move the pile locations. Therefore, the structural engineer should develop contingency plans that can quickly be implemented during construction.

The grout pressures and equipment loads during pile installations can disturb piles that have not fully cured. We recommend that no heavy equipment or new pile construction be allowed within 8 feet of piles less than 12 hours old.

Because the pile is cast below the ground, judgment and experience are used as a basis for determining the acceptability of a particular pile. Drilling characteristics, auger withdrawal rate, grout pressures, and the quantity of grout pumped per pile are all used in addressing the suitability of individual piles. To aid in these evaluations, the contractor should provide a calibrated grout pump and pressure gauge.

9.5 Concrete Hangar and Approach Slab Support

Concrete ramp and approach slabs may be designed as a slab on grade on top of the existing fill soils. In addition, because the concrete slab for the hangar will likely be on the order of 12 inches thick (comparable to the existing hangar slab thickness), it may be designed as a slab-on-grade on top of existing fill soils if the owner is willing to assume the risk of future settlements. As noted in the logs of our explorations the fill soils are variable and cannot be completely documented without performing a comprehensive and costly exploration program. It is our experience that undocumented fills commonly contain deleterious materials and are placed with little or no control on material type and compaction effort. Therefore, there is a potential for post-construction settlement under the slab. Because the fill has been in place for many years, this settlement would likely result from deterioration of pockets of organics rather than mass settlement.

Certain measures can be taken to reduce the risks associated with placing a slab-on-grade on these soils. Such measures include isolating the floor slab from columns and foundation walls and providing for a thicker slab and additional reinforcement to reduce cracking that may result from differential settlement of the fill soils under the slab. In addition, proof-rolling and removal and replacement of soft and loose soils, as described previously, may locate areas where poor performance could occur in the future.

Field CBR tests were performed in three of the test pit excavations, for a better evaluation of subgrade strengths. In test pit TP-2, CBR's of 19 and 23 were obtained in well compacted fill. In test pit, TP-3, a CBR of 100+ was obtained in native, very dense glacial soils. In test pit, TP-4, a CBR of 32 was measured in well compacted fill. In addition, one laboratory CBR test was performed on fill soils recompacted at the natural moisture content, which was approximately 5 percent above the natural moisture content. The results indicated a CBR of 5. This value is an indication of the low subgrade strength that can exist if fill is not properly placed and compacted, and is the reason that fill moisture contents are recommended to be no greater than 2-1/2 percent above the optimum moisture content during fill placement (see the Excavations and Fills Section of this report, Section 9.7.3). Therefore, it will be necessary to perform continuous and thorough compaction testing during fill placement.

Based on the CBR test results, we recommend a modulus of subgrade reaction, k , of 220 pounds per cubic inch (pci) for existing fill soils that are determined to be dense and unyielding from proof-rolling and for new fill soils that will be placed and compacted according to the recommendations in this report. For subgrades consisting of undisturbed, dense to very dense glacial soils, directly below the planned concrete section, a value of 500 pci may be used.

We recommend that at least 6 inches of select granular fill (Table 3, Section 9.7.3) be placed beneath all slabs on grade to act as a capillary break for moisture and to provide uniform subgrade support. This material should meet the gradational requirements specified in Table 3 and be compacted to at least 95 percent of the ASTM D-1557 maximum dry density. The actual thickness should be based on the design required for support, especially in areas where high loads are anticipated, and that required for frost protection in accordance with Snohomish County airport design recommendations, as the native soils are frost susceptible. In areas where moisture is undesirable, we also recommend that a moisture vapor barrier, overlain by 2 inches of clean sand fill, be placed over this select granular fill base.

9.6 Concrete Floor Slabs

Concrete floor slabs for the shop/office building, parking garage, and warehouse should either be designed as a structural slab or as a slab on grade bearing on either the medium dense native soils, the dense to very dense glacial soils or properly compacted structural fill placed on these soils. Because these slabs will be thinner and less reinforced than the slabs supporting aircraft, it is not recommended to support such slabs on the existing fill or loose native soils.

We recommend that at least 6 inches of select granular fill be placed beneath all slabs on grade to act as a capillary break for moisture and to provide uniform subgrade support. This material should meet the gradational requirements specified in Table 3 in Section 9.7.3 and be compacted to at least 95 percent of the ASTM D-1557 maximum dry density. In areas where moisture is undesirable, we also recommend that a moisture vapor barrier, overlain by 2 inches of clean sand fill, be placed over this select granular fill base.

9.7 Site Grading

9.7.1 General

Site grading, as described in this section, includes all excavations and fills necessary to bring the site to the proposed elevations, including fill to support building foundations and slabs, and backfill of foundation elements and retaining walls.

It is recommended that site grading and earthwork be performed during the drier, warmer months of the year. Based on our experience on similar sites, significant overages in cost and delays to the project can occur if site grading is attempted during wetter times of the year.

9.7.2 Site Preparation

Prior to any grading, the existing structures on the site should be demolished and removed from the site. Excavations that are incidental to demolition should be backfilled with compacted materials as specified in the "Excavations and Fills" section of this report (Section 9.7.3). All utility lines that traverse under proposed slabs on grade in the hangar and approach/ramp slab areas that are larger than 6 inches in diameter and within a depth of 10 feet of the proposed slab should be removed and backfilled with properly compacted structural fill or abandoned in place by filling the pipe with a slurry concrete mix.

We recommend that trees and brush be cleared and roots and stumps be removed from building areas, parking areas, and all areas to be graded. The topsoil that mantles the site is loose and organic, and should be removed from the site except in landscape areas. The depth of this removal is variable over the site. For quantity estimating purposes only, we suggest that an average stripping depth of 6 inches be used. Roots and stumps will likely extend deeper than the depth estimated above and should be completely removed from the site in the above specified areas. Topsoil is not considered suitable for reuse as fill other than landscape fill and should be removed from the site or stockpiled for reuse in landscaping areas. We also recommend that existing asphalt and concrete pavements be removed from building and pavement areas. In

addition, in the shop/office building, parking garage, and warehouse, all fill, old topsoil material, and loose native soils should be removed if a slab-on-grade floor will be used.

In fill or at-grade areas, the exposed soil surface after stripping and prior to fill placement should be compacted in place to at least 95 percent of the maximum dry density as determined by ASTM D-1557 and to a dense and unyielding condition. The compacted surface should then be proof-rolled with a fully loaded, tandem-axle, 10-yard dump truck. Areas that are soft, loose, or yielding should be further compacted or removed and reconditioned or replaced with compacted structural fill. Typically, a depth of 2 to 3 feet is adequate for overexcavation of soft and loose soils detected during the proof-rolling operation to provide a suitable subgrade for placing a pavement.

Areas that will require excavation to achieve subgrade elevations should be proof-rolled, as described above, after the subgrade elevation is reached. Care should be taken to avoid disturbing subgrade soils and supporting soils that will remain in place. Any areas that are disturbed should be reconditioned or replaced and recompact to a dense and unyielding condition.

9.7.3 Excavations and Fills

Excavation may be difficult in the unweathered glacial soil deposits and may require the use of a tractor-mounted ripper on areal cuts and a hoe-ram or hand-operated pavement breaker in small excavations such as those for utilities. In addition, although cobbles and boulders were not encountered in our explorations, historically excavations in glacial soil deposits have encountered these materials.

Structural fill is defined herein as all fill that will be placed beneath foundations, slabs, and pavements. The on-site glacial soils are suitable for use as structural fill provided they are free of organics or other deleterious materials and are placed at a moisture content near optimum to permit proper compaction. In order to provide suitable subgrade support, this moisture content should be no more than 2-1/2 percent above the optimum moisture content as determined from ASTM D-1557. If there is not a sufficient quantity of on-site material available for the fill quantities required or if weather conditions are such that the soils cannot be placed and compacted to the fill compaction standards recommended in this report, imported structural fill material will be required.

Imported structural fill should be a well graded sand or sand and gravel with less than 5 percent of the material that passes the 3/4-inch sieve passing the number 200 sieve. In addition the fill material should have a maximum particle size of 3 inches. An example of an imported structural fill material that has been used successfully under wet weather conditions is summarized in Table 3 below. The gradation of this material will be defined herein as select granular fill.

TABLE 3
SELECT GRANULAR FILL

<u>U.S. Standard Sieve Size</u>	<u>Percent Passing by Dry Weight</u>
3-inch	100
3/4-inch	50 - 100
No. 4	25 - 65
No. 10	10 - 50
No. 40	0 - 20
No. 200	0 - 5*

* Less than five percent passing based on the minus 3/4-inch fraction

All fill should be placed in uniform, horizontal layers not exceeding 8 inches in loose thickness for heavy equipment compactors and 4 inches for hand-operated mechanical compactors and compacted to meet the criteria in Table 4.

TABLE 4

RECOMMENDED FILL COMPACTION STANDARDS

<u>Fill Location</u>	<u>Percent Compaction</u>
Beneath foundations, floor slabs, and pavements	95
Exterior wall backfill	90
Subgrade prior to filling	95
Utility trenches	same as adjacent fill

* Expressed as a percentage of the maximum dry density determined by ASTM D-1557

If subgrade or fill soils become loosened or disturbed, additional excavation to expose dense, undisturbed soils and replacement with properly compacted structural fill will be required. The contractor may reduce disturbance by the following:

- Limiting construction traffic over supporting soils
- Providing gravel "working mats"
- Sloping excavated surfaces to promote runoff
- Trenching and providing brow ditches above cut slopes
- Sealing the exposed surface by rolling with a smooth drum compactor or rubber- tire roller at the end of each working day and removing wet surface soils prior to commencing filling the following day

Structural fill should extend laterally a minimum distance of 15 feet beyond building limits, roadways, and other improvements.

9.7.4 Temporary Excavation Slopes

Sloped temporary construction excavations may be used where planned excavation limits will not undermine existing structures, interfere with other construction, or extend beyond construction limits. Where there is not enough area for sloped excavations, temporary shoring should be provided.

The "safe" slope for the excavation of all soil will depend on the following factors: 1) the presence and abundance of groundwater; 2) the type and density of the soils; 3) the depth of excavation; 4) surcharge loading adjacent to the excavation such as that from excavated material, existing structures, or construction equipment; and 5) the time of construction. Construction slope values required for stability and safety depend on a careful evaluation of all of the above factors. Because of the many variables involved, the actual slope values required for stability in open excavations can only be estimated prior to construction.

Based on the subsurface conditions encountered in the explorations and the anticipated 15-foot maximum excavation depths, it is our opinion that sloped temporary excavations, in the absence of water, may be made at 0.5H:1V (horizontal to vertical) to 1H:1V in the glacial soils and at 1H:1V to 1.5H:1V in fill soils. If groundwater is flowing or seeping into the excavation, it should be expected to cause an unstable condition in the side slopes. If wetted by surface water, the slopes may be subject to erosion.

Excavation slopes should not be shown on the plans but instead made the responsibility of the contractor since the contractor is continuously at the site and is able to observe the nature and conditions of the subsurface materials encountered, including groundwater, and has responsibility for methods, sequence, and schedule of construction. If instability is detected, slopes should be flattened or shored. Regardless of the construction method used, all excavation work should be accomplished in compliance with applicable local, state, and federal safety codes.

9.8 Retaining Walls and Foundation Walls

We recommend that foundation walls and retaining walls be backfilled with select granular fill as specified under the "Site Grading" section of this report (Section 9.7). A perforated, rigid collection pipe (such as PVC or an approved equivalent) should be placed at the base of walls in a blanket of drain gravel meeting the gradational requirements specified in Table 5 below, and wrapped in a drainage geotextile such as Mirafi 140N or equivalent. Brittle, corrugated pipes that can crush under backfill weight or under the weight of construction equipment are not recommended. Collected water from the drains should be tightlined to the storm drain system. Figure 2 presents a detail of construction recommendations for walls.

TABLE 5

DRAIN GRAVEL

<u>U.S. Standard Sieve Size</u>	<u>Percent Passing by Dry Weight</u>
3/8-inch	100
1/4-inch	30 - 50
No. 8	0 - 5

We recommend that unrestrained walls that are free to yield at least 0.1 percent of the wall height be designed for an active earth pressure distribution equivalent to a fluid having a density of 35 pcf. Restrained walls should be designed to resist an at-rest equivalent fluid density of 55 pcf. These pressures are based on a horizontal backfill surface adjacent to the wall and do not include hydrostatic pressures. Surcharge loads, including construction and traffic loads and loads from stockpiled material, should be added to these values. Horizontal earth pressure values may be computed by multiplying the vertical surcharge load at a particular depth by 0.3 for yielding walls and by 0.5 for restrained walls. These values also assume a horizontal backfill surface.

The allowable bearing pressure for retaining wall footings may be taken as the value given under the "Shallow Foundations" section of this report (Section 9.3) provided the same foundation preparation is performed. In addition, an equivalent fluid density for passive resistance and an allowable coefficient of base friction may be used as specified in this section. It may be necessary to extend the retaining wall base below typical embedment depths at the southeastern corner of the proposed wall location because of the presence of deep fill soils in this area.

Care should be exercised when compacting backfill against retaining and foundation walls. To reduce temporary construction loads on the walls, heavy equipment should not be used for placing and compacting fill within a region as determined by a 0.5H:1V line drawn upward from the bottom of the wall, or within 3 feet of the wall, whichever is greater. We recommend that hand-operated compaction equipment be used in these areas.

9.9 Drainage

We recommend that perimeter foundation drains be installed around the structures to collect shallow perched groundwater and surface water infiltration and to reduce the potential for water to enter beneath the floor slabs. Drains should be constructed as recommended in Section 9.8,

"Retaining Walls and Foundation Walls", and as shown on Figure 2. Convenient cleanouts should be provided to increase the useful life of the drains.

Roof downspouts should not be permitted to discharge into foundation drains or into foundation bearing soil. Collected water should be directed away and downslope of the building, or into the storm drain system. In addition, the exterior ground surface adjacent to the proposed building should be sloped to promote proper drainage and to direct surface runoff away from the building.

During construction it will be necessary to control seepage, rainwater and runoff with a system of drainage ditches and swales. The site should be graded to promote drainage at all times. This will especially be necessary if deep overexcavation and replacement will be attempted in the southeastern corner of the shop/office building, in the northeastern corner of the warehouse, and in the parking garage area. Deep cut-off trenches, that penetrate at least 2 feet into the glacial soils or two feet below planned excavation limits (whichever is greater) may be required in these areas in order to allow excavation to proceed in the dry. The trenches should be lined with a drainage geotextile, such as Mirafi 140N or equivalent, and be filled with 1- to 2-inch drainage gravel. As noted previously, test pit TP-1 encountered large quantities of groundwater inflow. Therefore, any cut-off trenches that are installed should be done well in advance of construction and in the drier months of the year. Figure 3 presents a detail of a cut-off subdrain system. Such trenches may make the cost of overexcavation uneconomical, especially for the parking structure, where large quantities of groundwater are anticipated and deep overexcavation required.

Bales of straw and/or geotextile silt fences should be used to control soil movement and erosion, as appropriate.

9.10 Wet Weather Considerations

The on-site soils contain a significant percentage of silt and fine sand that make them particularly sensitive to moisture with regard to fill placement and are easily disturbed by construction equipment. These soils may degrade to a slurry-like consistency when subjected to construction traffic or otherwise disturbed in wet conditions. As indicated on the logs of the explorations, soils were classified as moist to wet above the proposed excavation elevations. Therefore any grading that utilizes the on-site soils should be accomplished in dry weather and may require spreading and drying to lower the soil moisture content into a range suitable for compaction.

Construction traffic should be restricted to frequently used routes. The on-site soils will likely provide a suitable working surface in dry weather conditions, however, after continual repetitions by wheel loads the material can degrade rapidly, especially in the presence of water.

Gravel working surfaces are recommended once the subgrade area for floor slabs and pavements are exposed. In cut areas, it may be desirable to leave the subgrade one to two feet above the planned grade so that disturbed soils resulting from construction operations can be removed prior to paving without requiring the need for overexcavation and additional fill. If foundations are constructed during wetter times of the year, or if seepage is significant, a thin layer of lean-mix concrete should be placed at the base of the footings to reduce the possibility of disturbing the subgrade soils.

9.11 Utilities

Utility trenching should be in accordance with the Washington State Department of Transportation and American Public Works Association (WSDOT/APWA) Standard Specifications. Achieving the required compaction during backfilling operations may not be possible during wet weather conditions, and even during dry weather conditions, because of the moisture sensitivity of the on-site soils, the small confines of the trench, and the limited time frame for aerating soils. Therefore, the use of imported structural fill should be anticipated.

Any new utilities that are placed below the concrete slabs on grade should be designed to withstand vertical loads from backfill soils and from airplane wheel loads.

10.0 ADDITIONAL STUDIES

Prior to construction, additional environmental explorations and chemical laboratory testing are recommended to better characterize site conditions. Additional environmental services should include:

- Determination of the groundwater flow gradient at the site. This would be accomplished by surveying the vertical and horizontal location of each well installed at the site and measuring the static groundwater elevation at each well. A computer contoured gradient map of the upper surface of the local groundwater table would then be utilized for further field investigative work.
- Determination of past locations of petroleum hydrocarbon facilities at the site by conducting a historical research of Airport Facility records.

- Installation of additional groundwater monitoring wells installed at strategic locations as determined from information obtained from the above exercises. This would include additional laboratory analyses of soil and groundwater.
- Determination of aquifer flow characteristics, including transmissivity and/or permeability, by completing field "slug" tests.

In addition, it may be desirable to complete a geotechnical boring in the northeastern corner of the proposed warehouse. As shown on Figure 1, the contours of estimated elevation of bearing soils changes rapidly in this area. An additional exploration may help better determine the type of foundation support selected for this structure and therefore the cost of construction.

11.0 DOCUMENT REVIEW AND CONSTRUCTION OBSERVATIONS

We recommend that we be retained to review those portions of the plans and specifications that pertain to foundations and earthwork to determine whether they are consistent with the recommendations in this report. Although this phase of project design is often neglected, substantial savings in cost or future cost overruns during construction can be corrected during this phase, prior to construction bidding. In addition, because of the size of this project and the number of structures involved, more than one foundation alternative has been given for each structure in order to allow TRAMCO to construct the most economical facilities. These alternatives have conditions and thus warrant review before construction. Typically, such a review can be performed for about \$1,000.

We recommend that monitoring, testing, and consultation be provided by our firm during construction to confirm that the conditions encountered are consistent with those indicated by our explorations, to provide expedient recommendations should conditions be revealed during construction that differ from those anticipated, and to evaluate whether earthwork activities comply with contract plans and specifications. Such activities would include observation of subgrade preparation for foundations, floor slabs, and pavements; observation of deep foundation installation; observation of fill placement and compaction testing; and other geotechnical related earthwork activities.

12.0 LIMITATIONS

The analyses, conclusions, and recommendations contained in this report are based on site conditions as they presently exist and further assume that the explorations (both previous and

current) are representative of the subsurface conditions throughout the site, i.e., the subsurface conditions everywhere are not significantly different from those disclosed by the explorations. If, during construction, subsurface conditions different from those encountered in these explorations are observed or appear to be present, we should be advised at once so that we can review these conditions and reconsider our recommendations, where necessary. If there is a substantial lapse of time between the submission of this report and the start of construction at the site, or if conditions have changed due to natural causes or construction operations at or adjacent to the site, or appear to be different from those described in our report, we recommend that we review our report to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse.

Within the limitations of scope, schedule and budget, the analyses, conclusions and recommendations presented in this report were prepared in accordance with generally accepted professional geotechnical engineering principles and practice in this area at the time this report was prepared. We make no other warranty, either express or implied. These conclusions and recommendations were based on our understanding of the project as described in this report and the site conditions as observed at the time of our explorations.

Unanticipated soil conditions are commonly encountered and cannot be fully determined by merely taking soil samples or making test borings and pits. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. Therefore, some contingency fund is recommended to accommodate such potential extra costs.

This report was prepared for the exclusive use of the Owner, Architect, and Engineer for specific application to the design of the project at this site as it relates to the geotechnical aspects discussed herein. The data and report should be provided to prospective contractors for their information, but our report, conclusions and interpretations should not be construed as a warranty of subsurface conditions included in this report.

The scope of our services did not include any environmental assessment or evaluation regarding the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater or air, on or below the site, other than that described within the scope of services in this report.

We appreciate the opportunity to be of service on this project, and look forward to continued involvement. If you have any questions, or require additional services, please do not hesitate to contact us.

SHANNON & WILSON, INC.

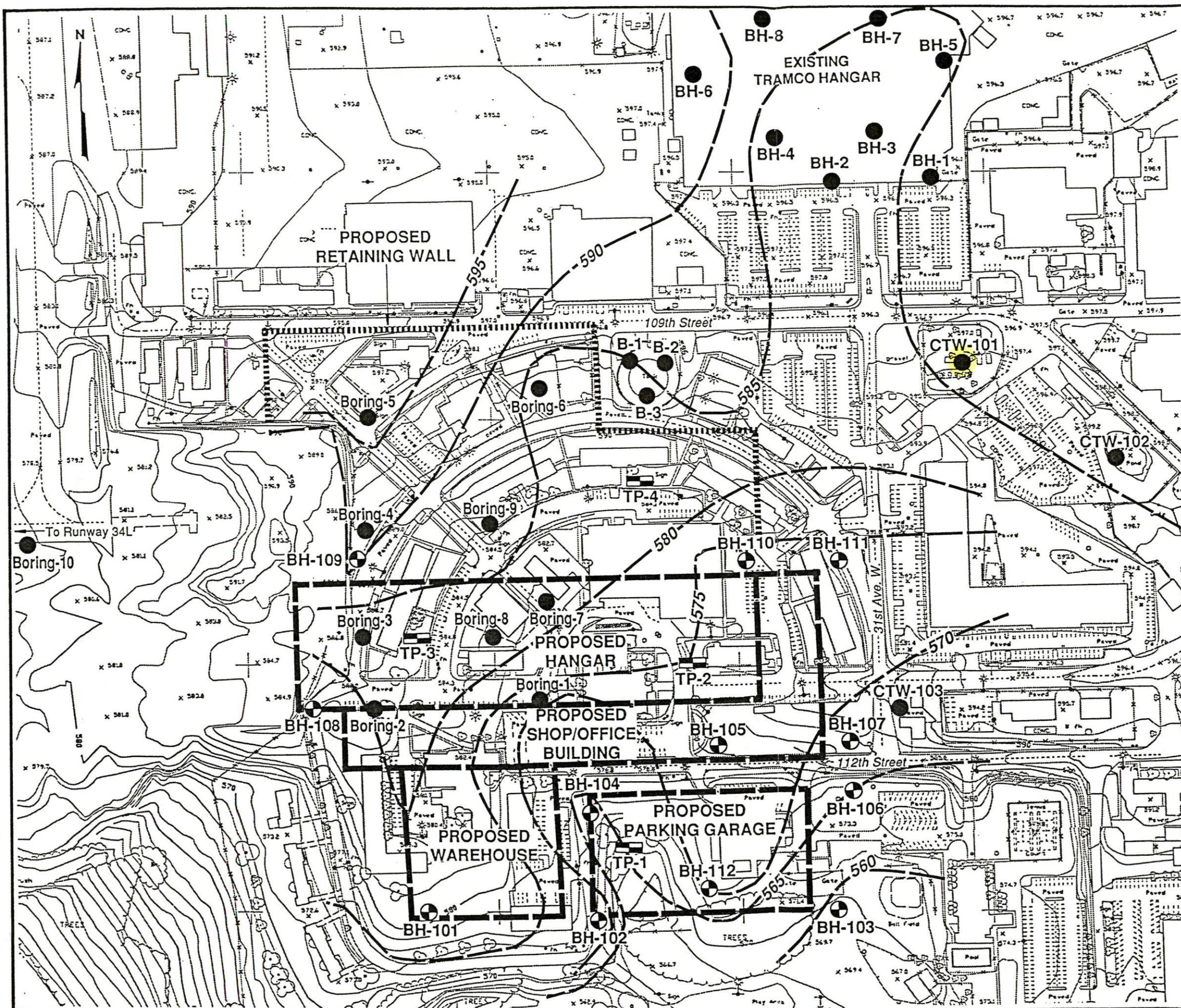


Gregory R. Fischer, P.E.
Senior Engineer



Herman H. (Tex) Druebert, P.E.
Vice President

GRF:HHD/grf

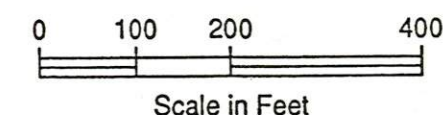


LEGEND

- Previous Boring Designation and Approximate Location
- BH-1 - By Converse Consultants NW, 2-88
- B-1 - By Converse Consultants NW, 11-88
- CTW-101 - By Converse Consultants NW, 3-90
- Boring-1 - By Earth Consultants, Inc., 4-90
- BH-101 ● Boring Designation and Approximate Location
- TP-1 ■ Test Pit Designation and Approximate Location
- Estimated Elevation of Dense to Very Dense, Native Glacial Soil Deposits

NOTES

1. Plan taken from undated topographic plan provided by Sierra Construction Company.
2. Elevations of dense to very dense native glacial soil deposits are approximate and should be confirmed during construction.
3. Contours shown are generalized from materials encountered in the borings. Variations between the contours shown and actual conditions may exist.



TRAMCO Complex Expansion
Paine Field - Everett, Washington

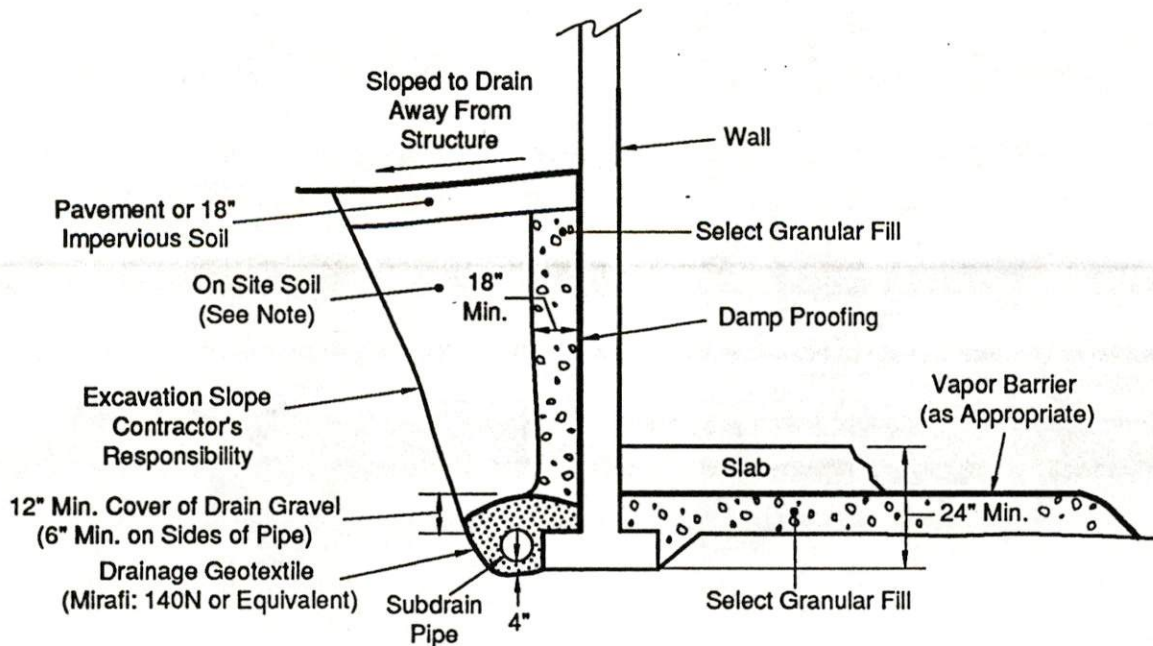
SITE AND EXPLORATION PLAN

February 1991

W-5751-01

SHANNON & WILSON, INC.
Geotechnical Consultants

FIG. 1



Not to Scale

DRAIN GRAVEL

<u>Sieve Size</u>	<u>% Passing by Weight</u>
3/8"	100
1/4"	30 to 50
No. 8	0 to 5

SELECT GRANULAR FILL

<u>Sieve Size</u>	<u>% Passing by Weight</u>
3"	100
3/4"	50 to 100
No. 4	25 to 65
No. 40	0 to 20
No. 200	0 to 5*

*Based on Minus
3/4" Fraction

SUBDRAIN PIPE

4" minimum diameter perforated or slotted pipe;
tight joints; sloped to drain (6"/100' min. slope);
provide clean-outs.

Perforated pipe holes (3/16" to 1/4" dia.) to be
in lower half of the pipe with lower quarter
segment unperforated for water flow.

Slotted pipe to have 1/8" maximum width slots.

NOTE

If wet conditions render on-site soil unsuitable for
compaction, backfill the zone shown above with
free-draining granular soil with not more than 5%
(by weight based on minus 3/4" portion) passing
No. 200 sieve with no plastic fines.

TRAMCO Complex Expansion
Paine Field - Everett, Washington

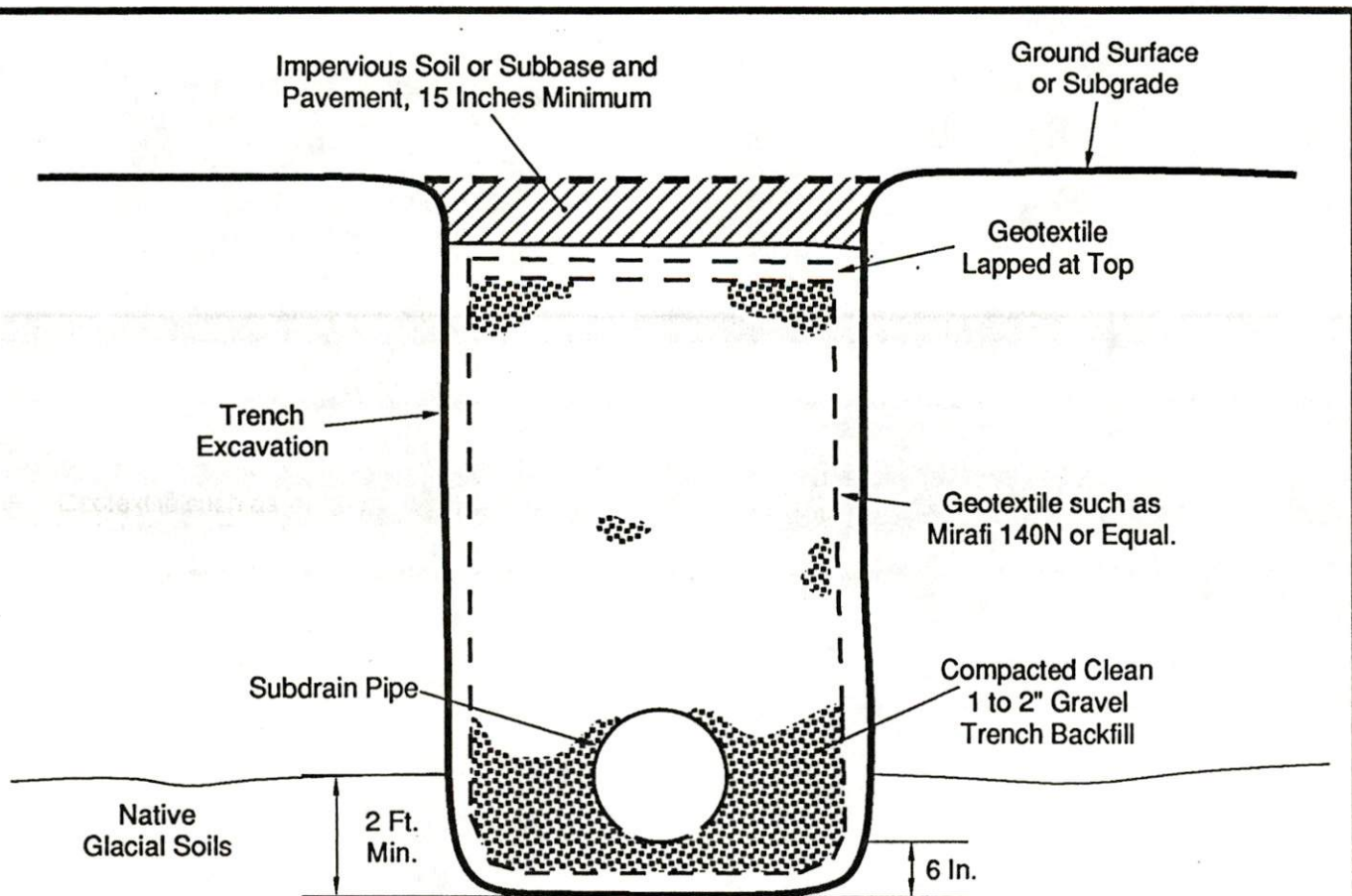
WALL DRAINAGE AND BACKFILLING

February 1991

W-5751-01

SHANNON & WILSON, INC.
Geotechnical Consultants

FIG. 2



Not to Scale

SUBDRAIN PIPE

Perforated or slotted pipe; tight joints; sloped to drain; provide clean-outs; min. dia.: 4 inches.

Perforated pipe holes (1/8-in. to 1/4-in. dia.) to be in lower half of pipe with lower quarter segment unperforated for water flow.

Slotted pipe to have 1/8-in. max. width slots.

Cut-off subdrains should be placed outside of structure locations.

TRAMCO Complex Expansion
Paine Field - Everett, Washington

TYPICAL SUBDRAIN INSTALLATION

February 1991

W-5751-01

SHANNON & WILSON, INC.
Geotechnical Consultants

FIG. 3


SHANNON & WILSON, INC.

 Geotechnical Consultants
 Engineering and Applied Geosciences

*Over
 35 Years of
 Excellence*

400 North 34th Street, Suite 100 • P.O. Box C-30313 • Seattle, WA 98103 • (206) 632-8020 • Fax: (206) 547-0386

March 4, 1991

 Sierra Construction Company, Inc./TRAMCO
 Paine Field
 3226 11th Street S.W.
 Everett, Washington 98204

Attn: Mr. Chris Fusetti

**RE: GEOTECHNICAL REPORT AND ENVIRONMENTAL SUMMARY, PROPOSED
 TRAMCO COMPLEX EXPANSION, PAINE FIELD, EVERETT, WASHINGTON**

Enclosed is our geotechnical report and environmental summary for the referenced project. This report presents the results of our field explorations and geotechnical and chemical laboratory testing; provides geotechnical recommendations for design; and discusses construction considerations as related to the geotechnical aspects of the project. Prior to construction, additional explorations and chemical laboratory testing are recommended to better characterize site conditions. The recommendations in this report, which were developed from the field and chemical laboratory results, are based on very limited environmental data and are intended for preliminary planning and cost estimation purposes only. These recommendations should be confirmed or modified based on the results of additional studies.

We appreciate the opportunity to be of service to you on this project and look forward to our continued involvement through final design and construction.

Sincerely,

SHANNON & WILSON, INC.

 Herman H. (Tex) Druebert, P.E.
 Vice President

GRF:HHD/grf

W5751-01.LTR/GRF-lkd/cbt

Seattle • Everett • Fairbanks • Anchorage • St. Louis

PRESIDENT: Earl A. Sibley, P.E.

SR. VICE PRESIDENT: Atef A. Azzam, P.E.; Raymond P. Miller, P.E.; Harvey W. Parker, P.E.; George Yamane, P.E.

VICE PRESIDENT: Herman H. (Tex) Druebert, P.E.; Richard H. Gates, P.E.; W. Paul Grant, P.E.; Leland B. Jones, P.E.; Thomas E. Kirkland, P.E.; Gerry Millar, R.G.; Frank W. Pita, P.E., P.G.; Ming-Jiun (Jim) Wu, P.E.

CONSULTANT: William L. Shannon, P.E.

APPENDIX A
FIELD EXPLORATION PROGRAM

APPENDIX A

FIELD EXPLORATION PROGRAM

Subsurface conditions for this project were explored by drilling 12 borings and excavating 4 test pits at the approximate locations shown on the Site and Exploration Plan, Figure 1. The borings were performed from January 24, to 30, 1991 and advanced to depths ranging from 30.5 to 45.5 feet below the existing ground surface. The test pits were performed on February 6, 1991 and advanced to depths ranging from 6 to 9 feet. The results of our exploration program are presented on the boring logs (Figures A-1 to A-12) and test pit logs (Figures A-13 to A-16) at the end of this appendix.

A representative from our firm was present throughout the field work program to observe the explorations, assist in sampling, and to prepare descriptive logs of the explorations. Soils were classified in general accordance with ASTM D-2488, "Standard Recommended Practice for Description of Soils (Visual-Manual Procedure)". The final exploration logs represent our interpretation of the contents of the field logs and the results of laboratory testing.

The explorations were located in the field by hand taping and pacing relative to existing physical features. The approximate ground surface elevation at the exploration locations, as presented on the exploration logs, were interpolated from an existing untitled, undated topographic plan provided by Sierra. The location and elevation of the explorations should be considered accurate to the degree implied by the method used.

The borings were drilled with a truck-mounted Mobile B-61 drill rig. Hollow stem augers, with a 3-3/8-inch inside diameter, were used to advance each of the borings. Sampling was performed through the hollow stem of the augers.

Standard Penetration Tests (SPT) were taken at 2-1/2- to 5-foot depth intervals with a 2-inch outside diameter split-spoon sampler in general accordance with ASTM D-1586. The split-spoon sampler was driven into the soil a distance of 18 inches with a 140-pound hammer freely falling from a height of 30 inches. Recorded blows for each 6 inches of penetration are shown on the boring logs. Where greater than 50 blows were required to drive the sampler in a 6-inch increment, the test was stopped and the penetration recorded for the 50 blow interval. The number of blows required to drive the sampler the last 12 inches is the Standard Penetration Resistance. This resistance, or blow count, provides a qualitative measure of the relative density of cohesionless soils and consistency of cohesive soils. Representative portions of the split-spoon

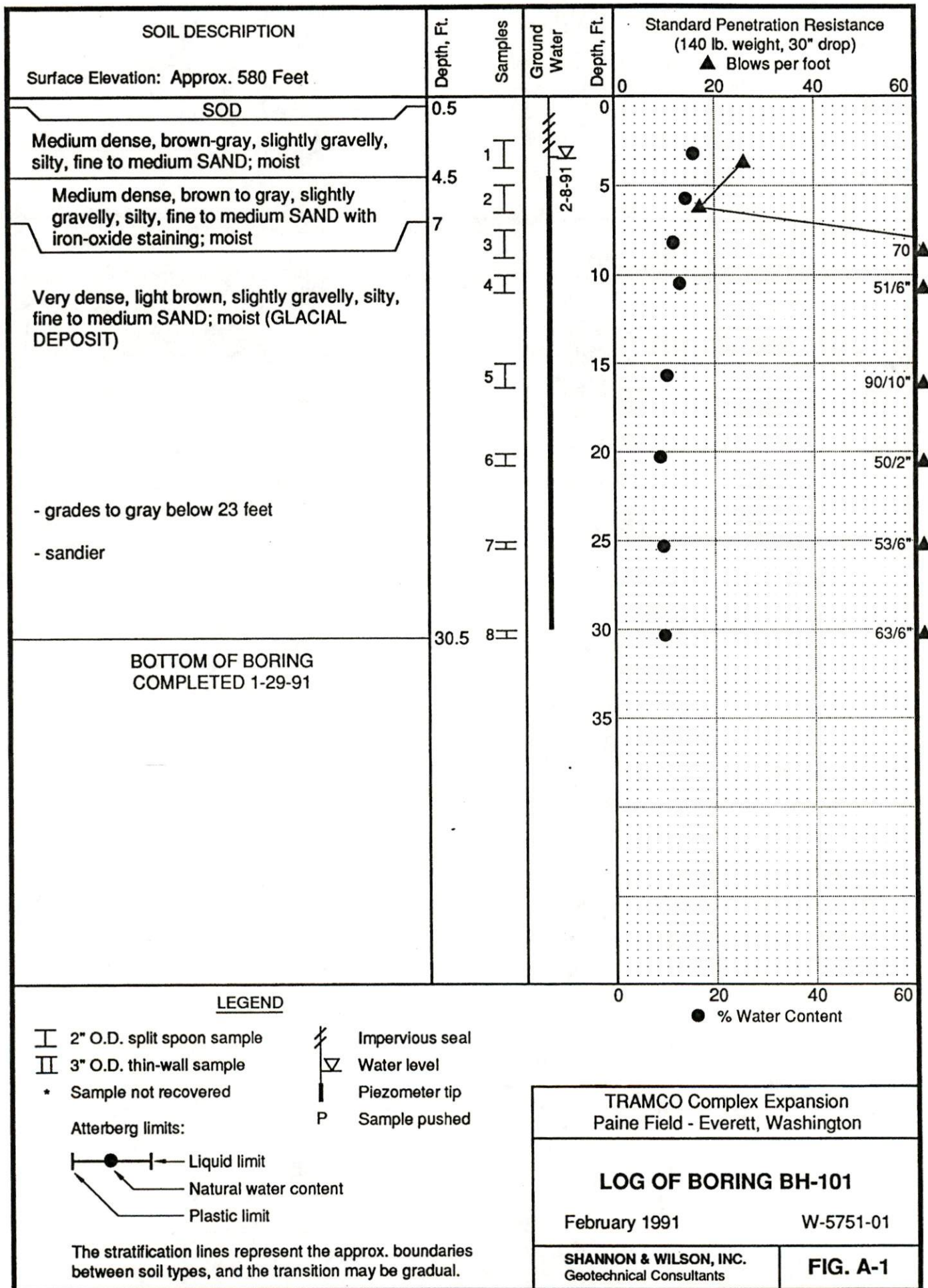
samples were placed in jars, sealed, and transported to our laboratory for further observation and testing.

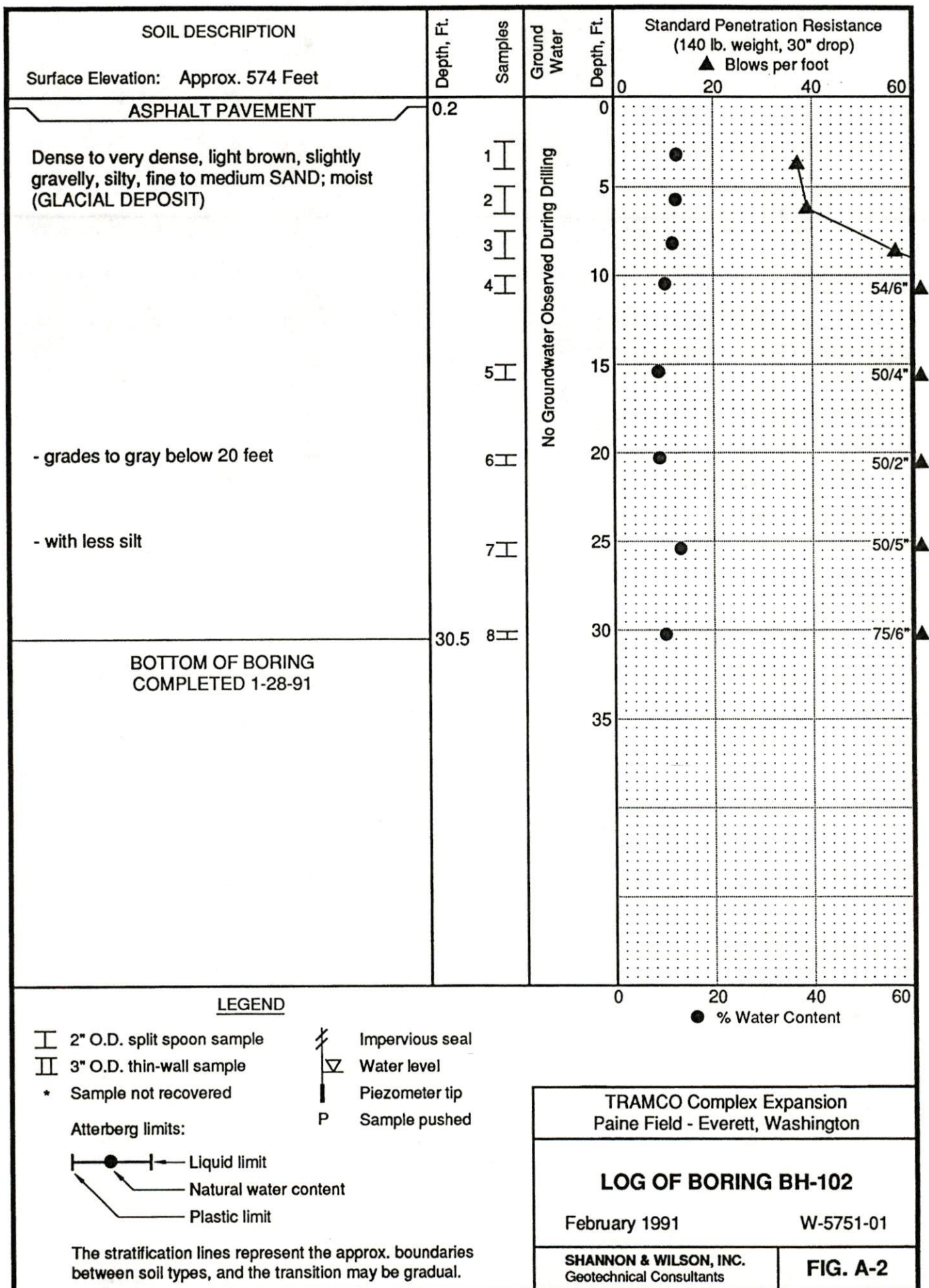
Upon completion of drilling the borings, observation and monitoring wells were installed in 5 of the boreholes to determine groundwater levels at the boring locations and for use in obtaining groundwater samples. The wells consisted of 3/4-inch and 2-inch inside diameter PVC pipe with 10- to 35-foot long screen intervals. After placing the pipe, a filter pack material consisting of sand was placed to above the screen. A bentonite seal was placed above the sand (and below the screen, as appropriate). A cast iron valve box was placed at the surface and grouted into the ground. Water levels along with the date of measurement and construction configurations are shown on the individual exploration logs.

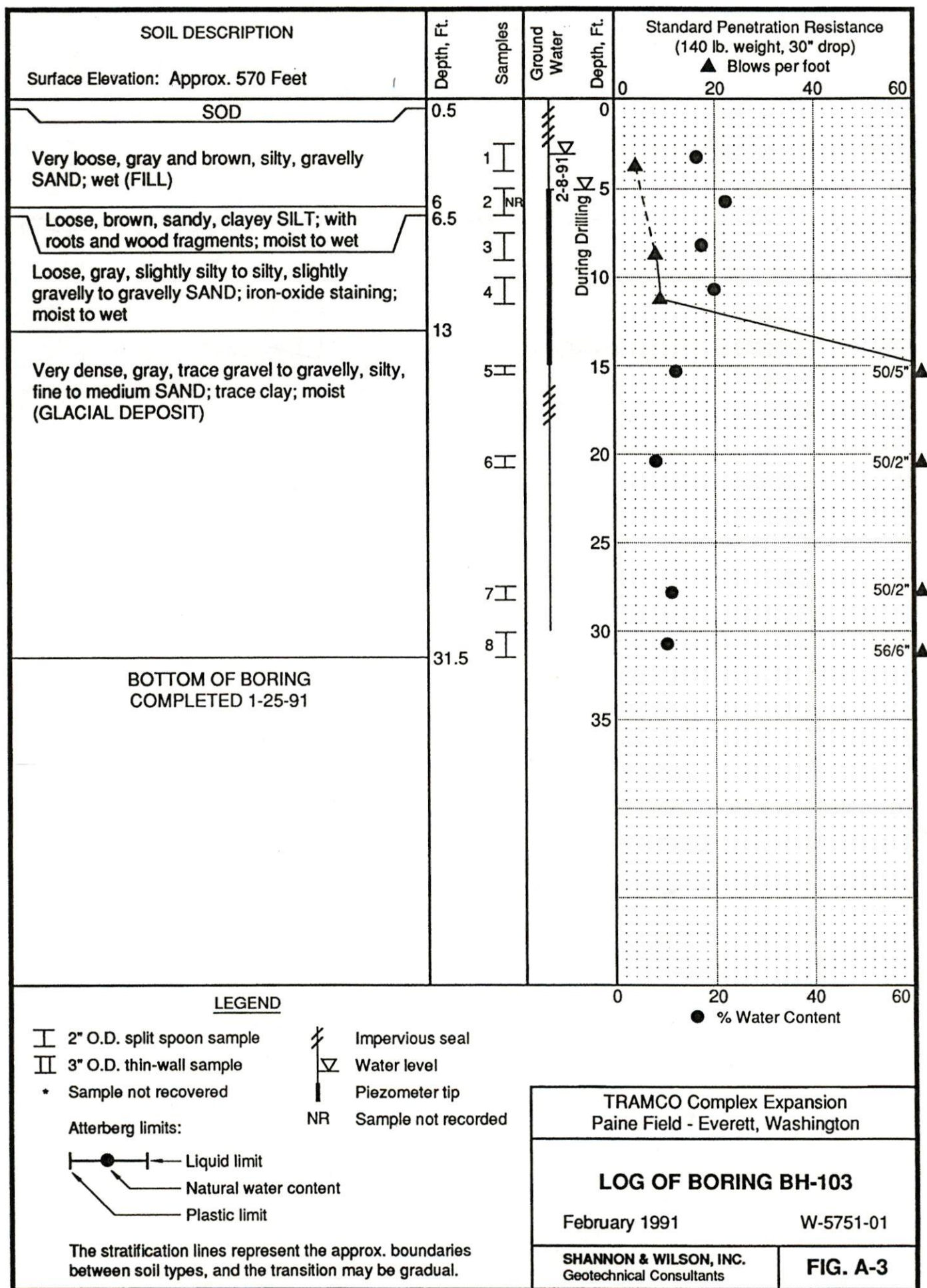
The test pits were excavated with a Case 680L rubber-tire backhoe provided by the Snohomish County airport. Test pits allow direct visual observation of the subsurface soils on the sides of an excavated trench. Representative samples of soil types encountered were placed in plastic bags, sealed, and transported to our laboratory for further classification and testing. The relative density and consistency of the soils were estimated by our field representative. Four field California Bearing Ratio (CBR) tests were conducted in the test pits for evaluation of pavement subgrade strengths. The tests were performed in general accordance with ASTM D-4429. The results of the field CBR tests are presented in Table A-1 below.

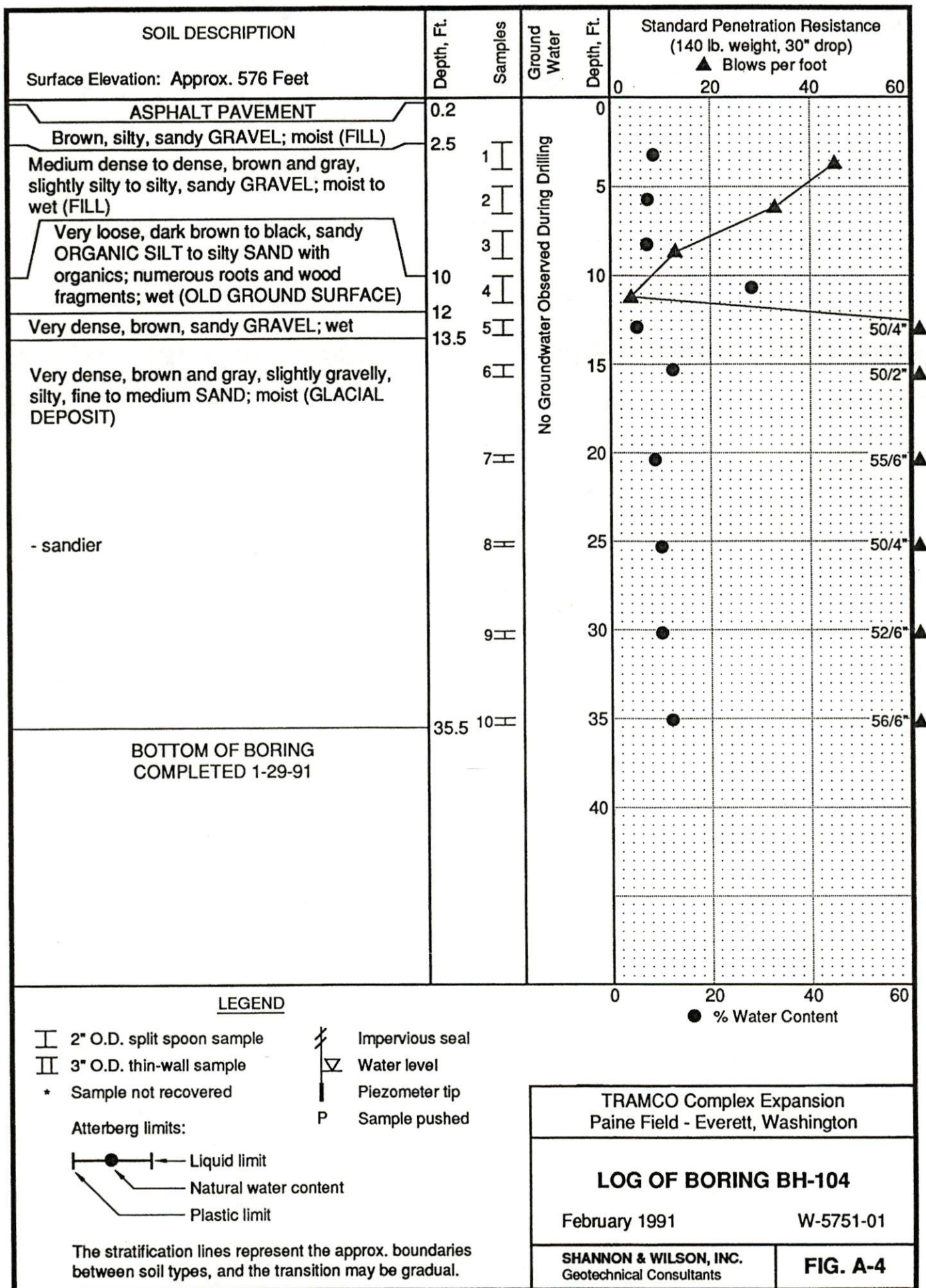
TABLE A-1
RESULTS OF FIELD CBR TESTS

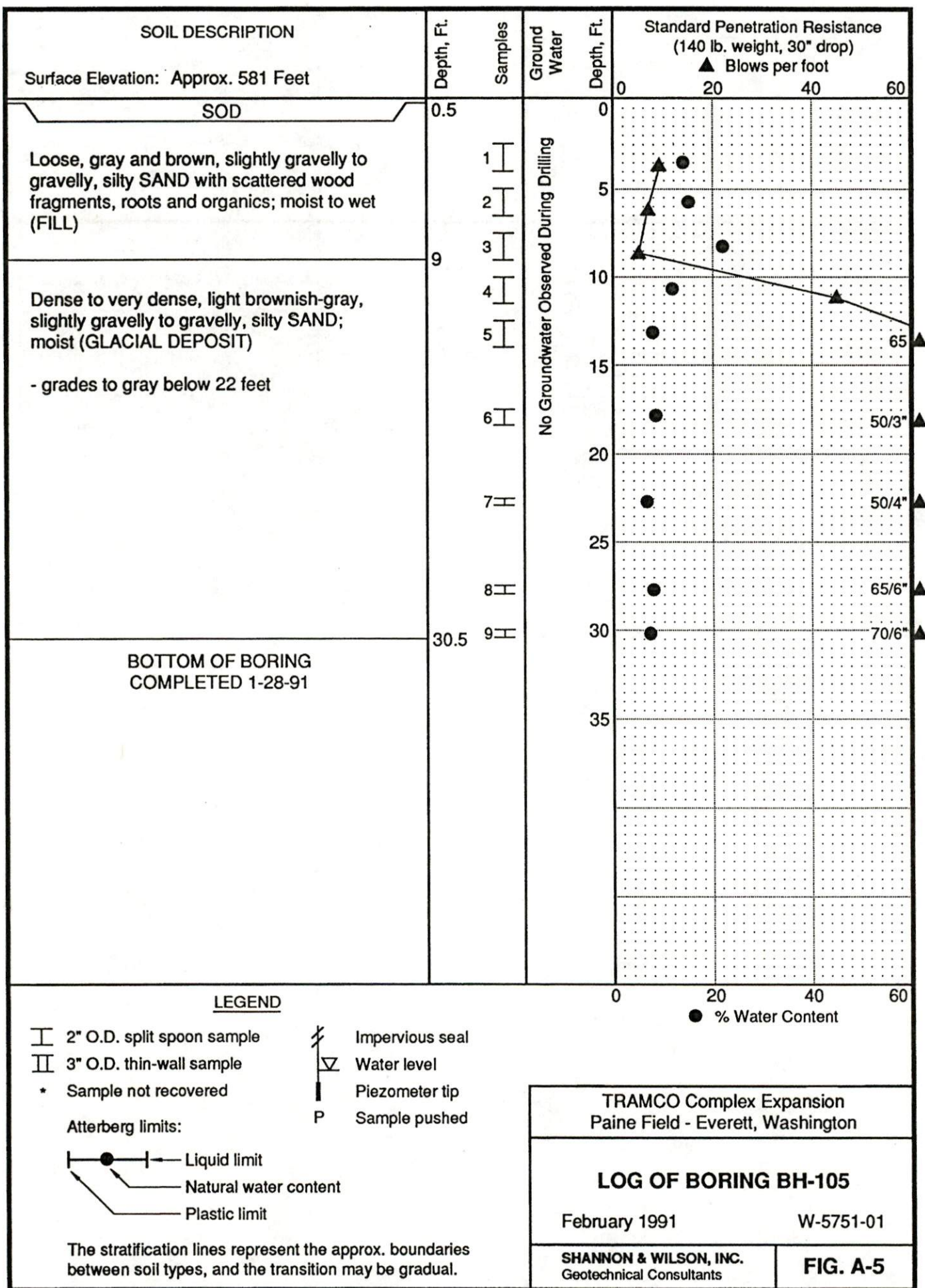
<u>CBR Test</u> #	<u>Test Pit</u>	<u>Depth (ft)</u>	<u>CBR</u>
1	TP-2	1.8	23
2	TP-2	1.8	19
3	TP-3	2.6	100+
4	TP-4	1.4	32

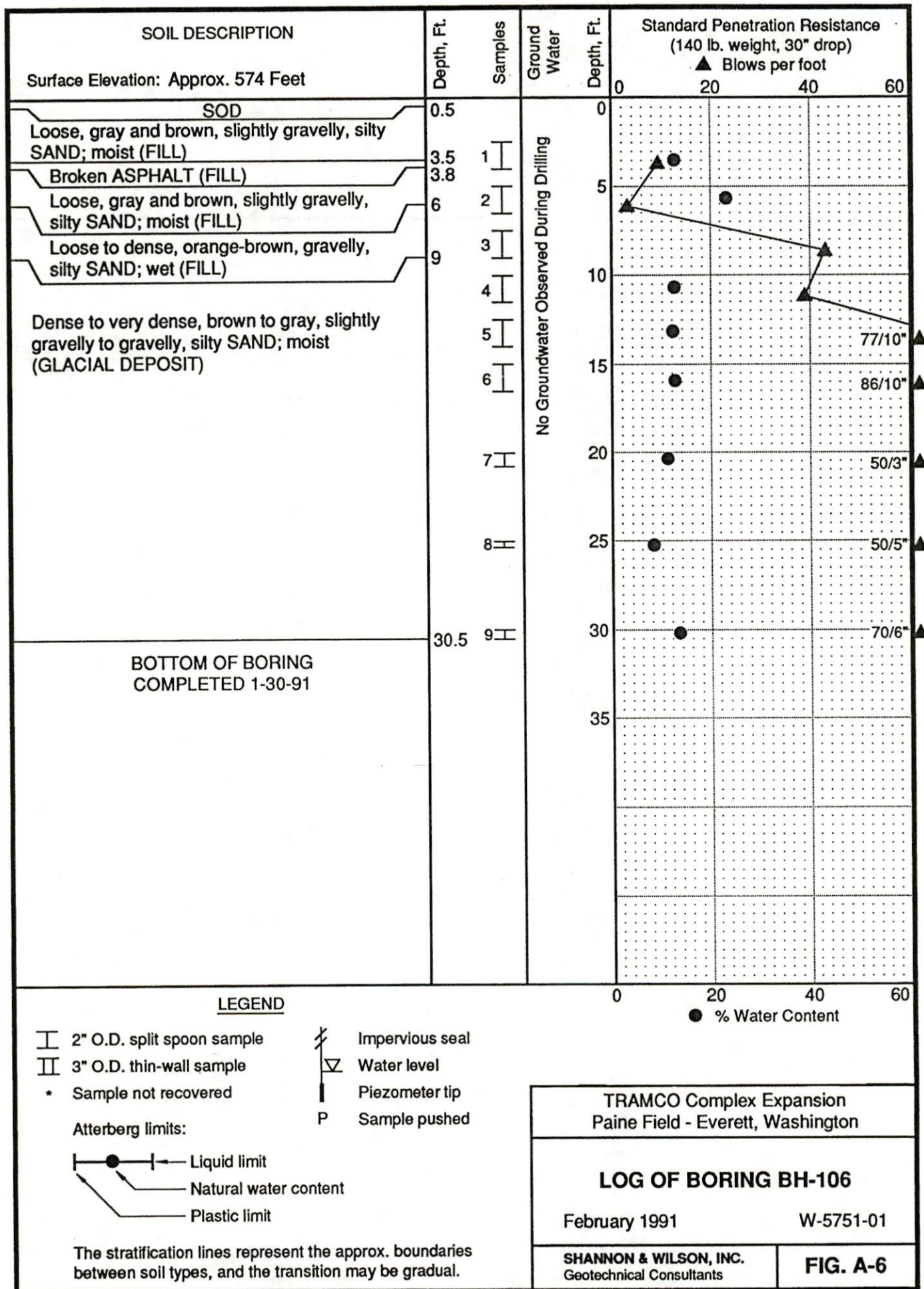


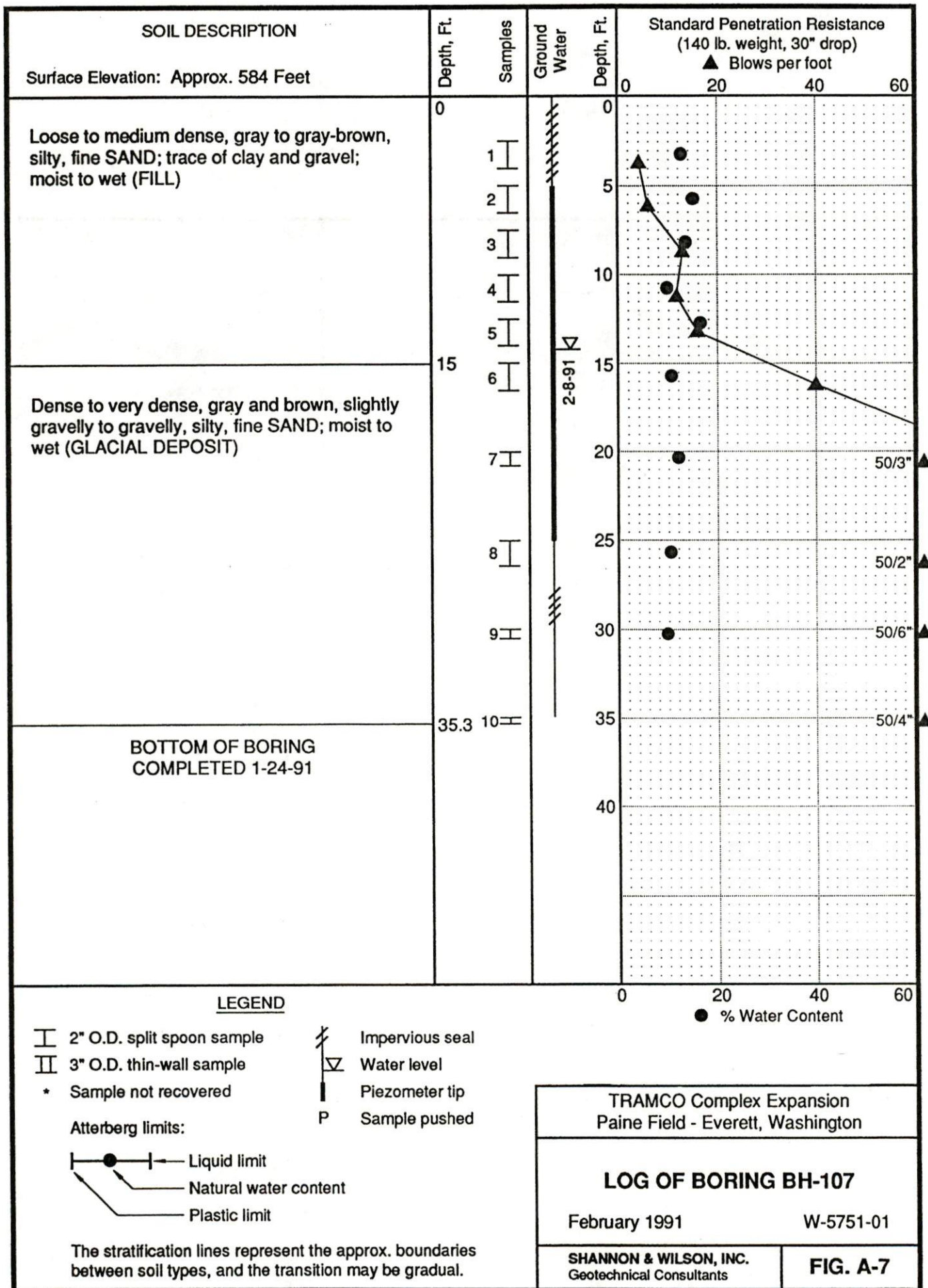


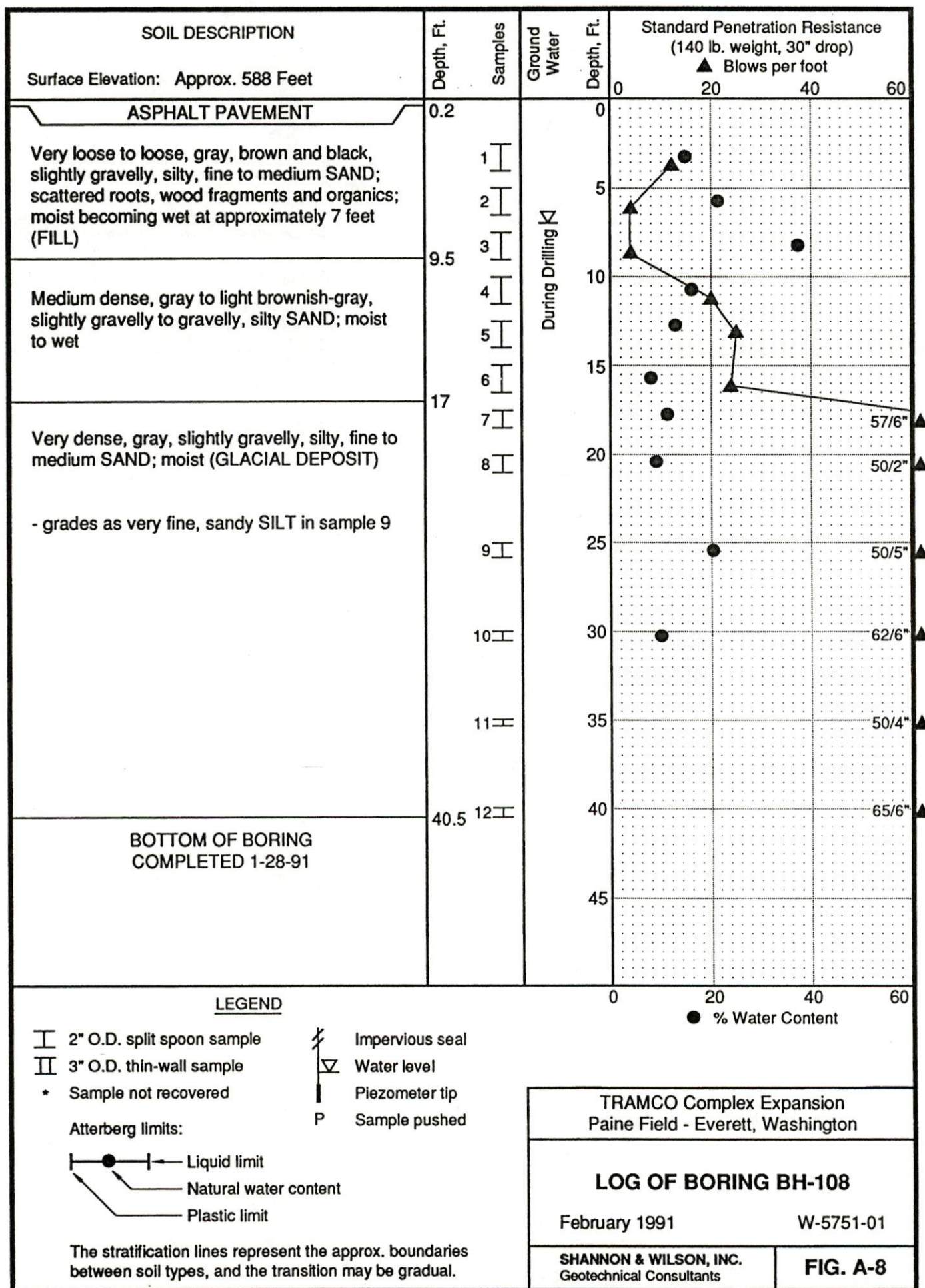


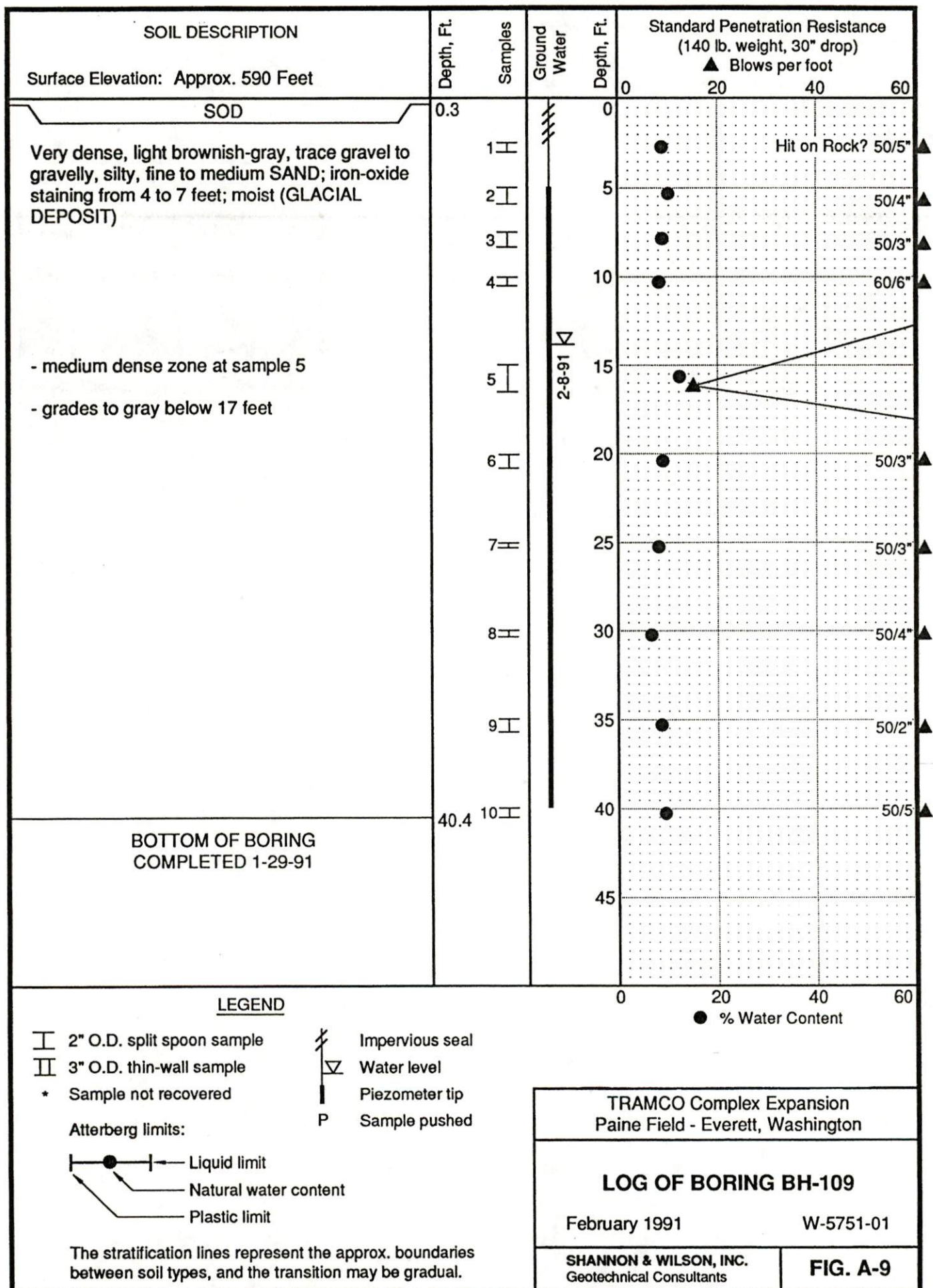


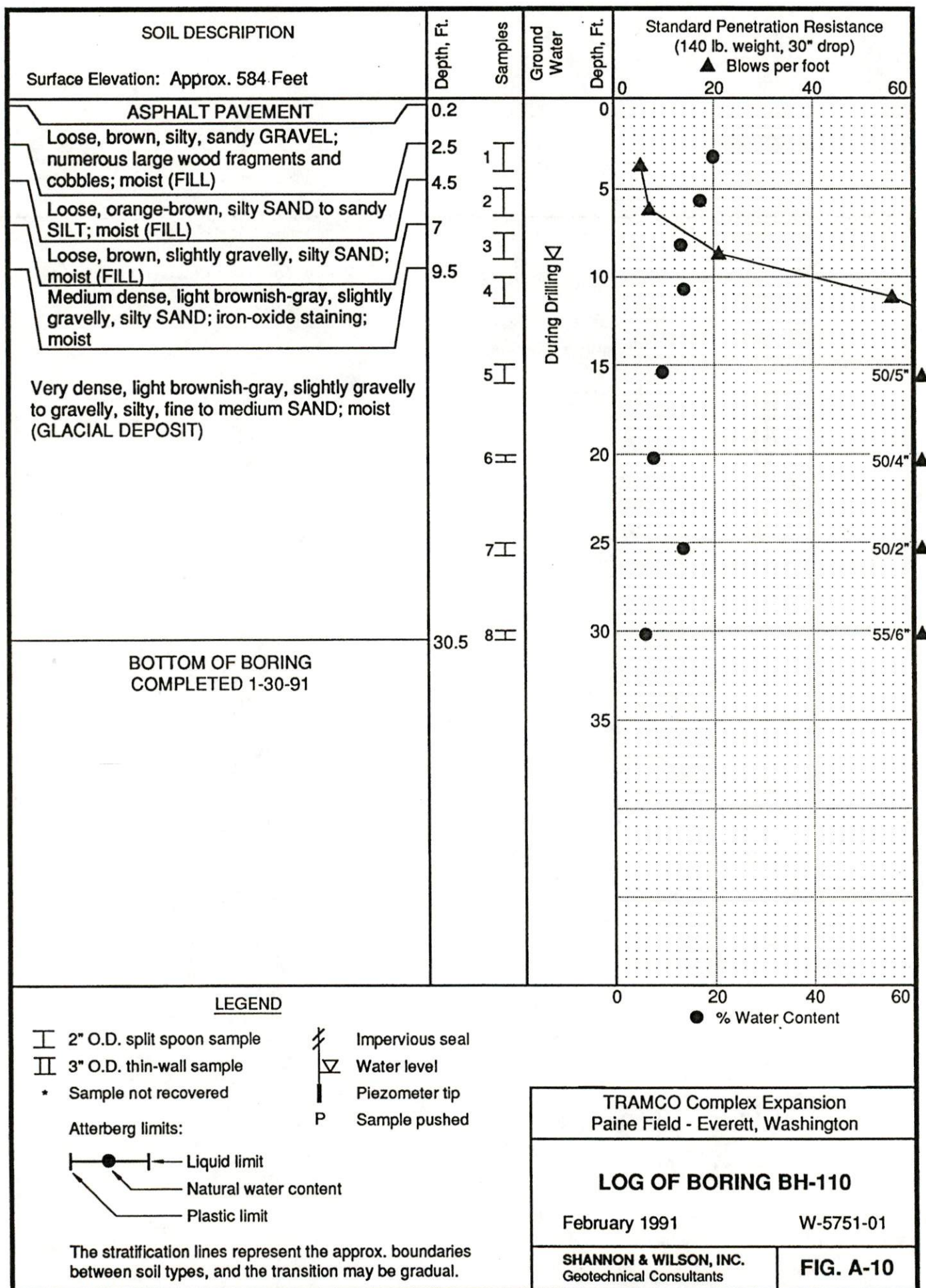


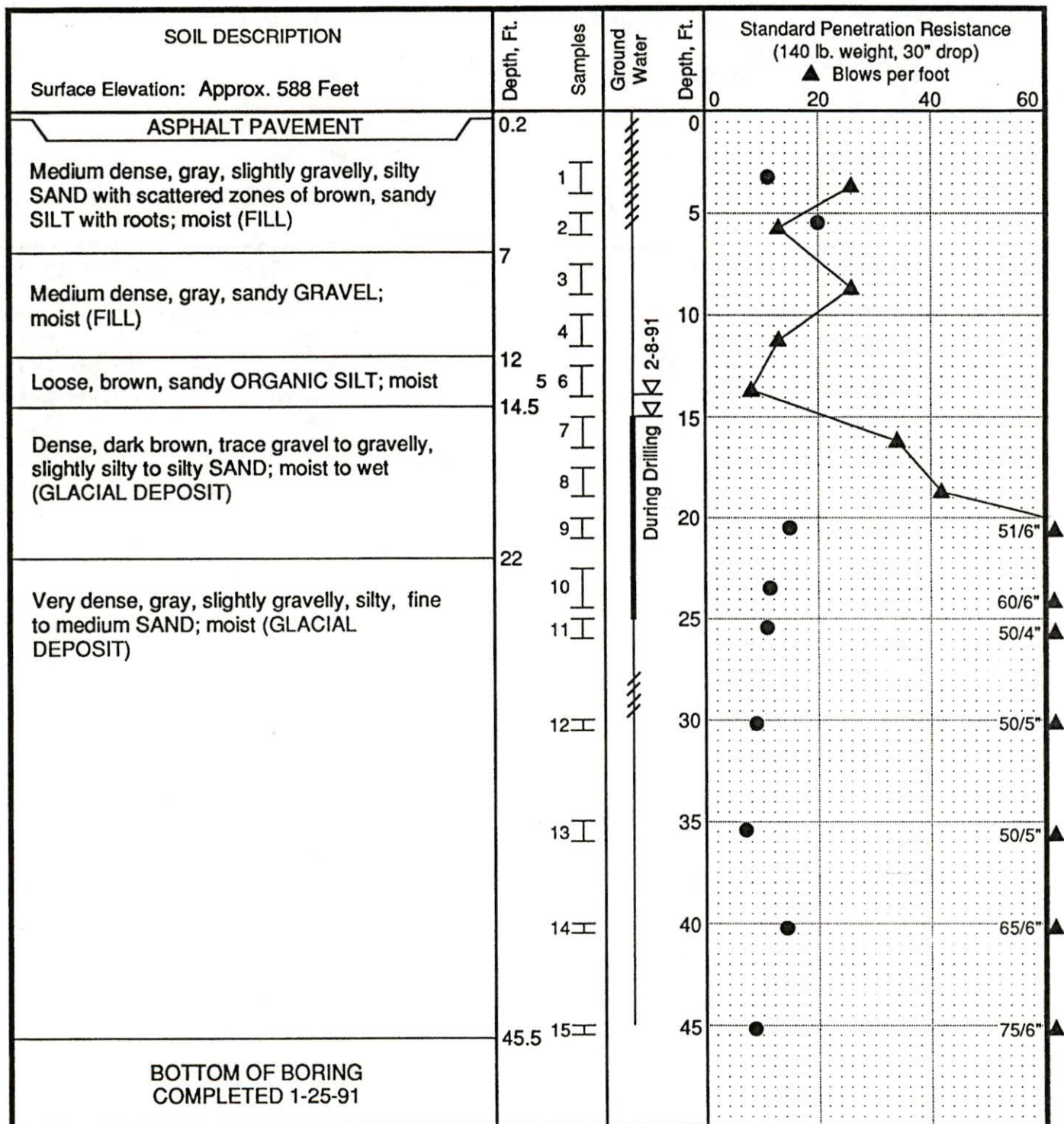












LEGEND

- | | | | |
|---|----------------------------|---|-----------------|
| | 2" O.D. split spoon sample | | Impervious seal |
| | 3" O.D. thin-wall sample | | Water level |
| * | Sample not recovered | | Piezometer tip |
| | | P | Sample pushed |
- Atterberg limits:
-
- Liquid limit
 — Natural water content
 — Plastic limit

The stratification lines represent the approx. boundaries between soil types, and the transition may be gradual.

TRAMCO Complex Expansion
Paine Field - Everett, Washington

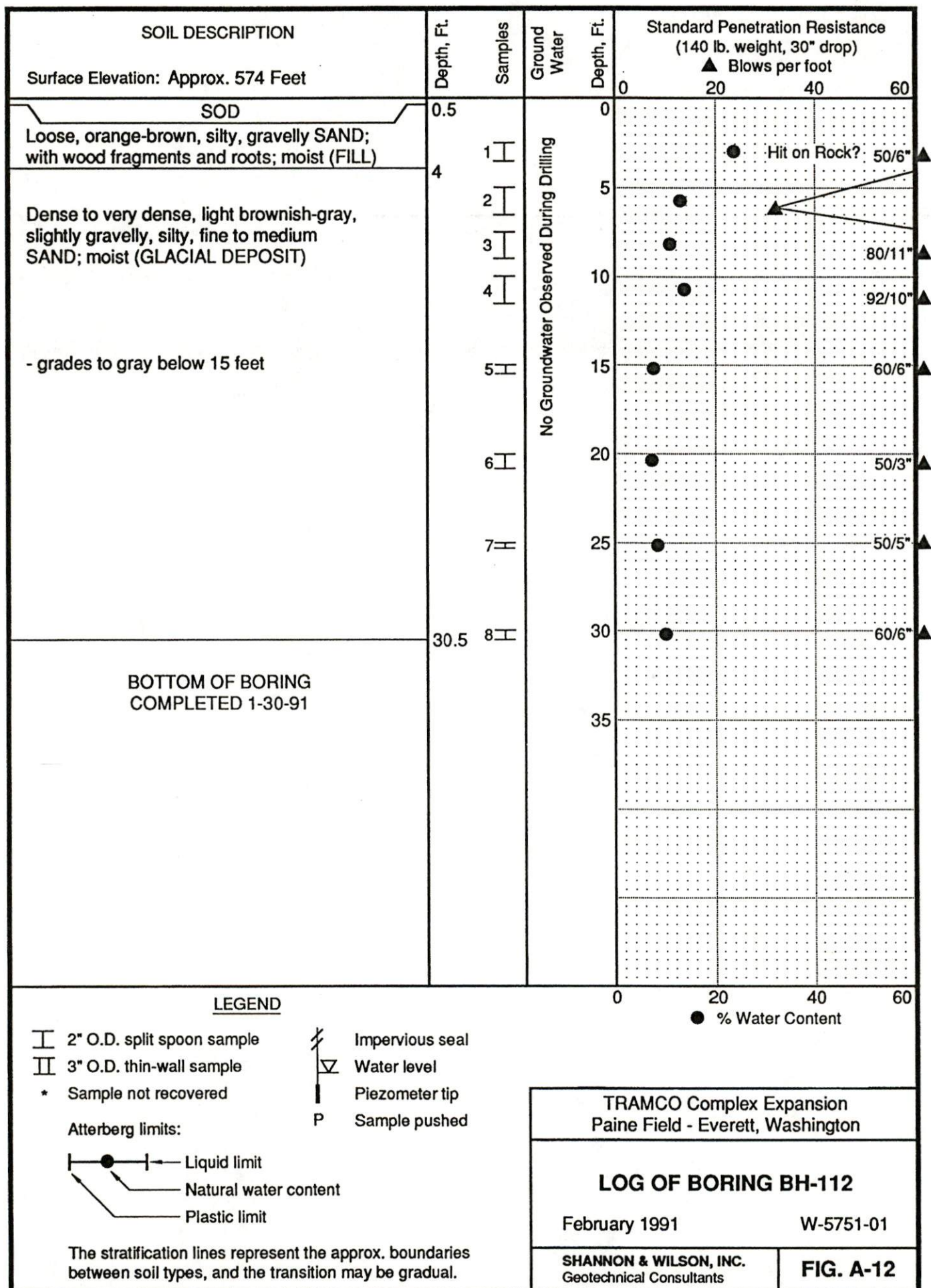
LOG OF BORING BH-111

February 1991

W-5751-01

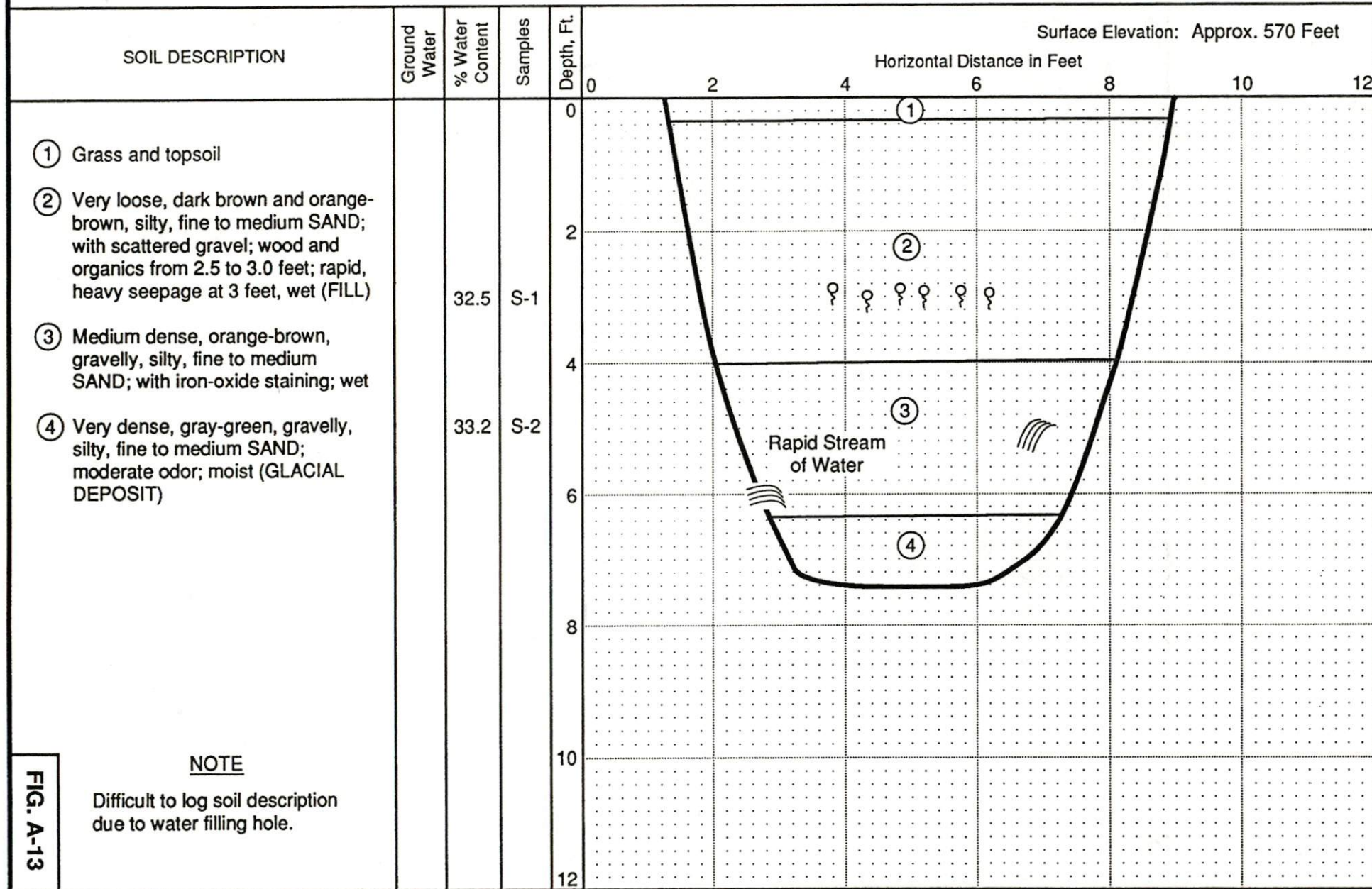
SHANNON & WILSON, INC.
Geotechnical Consultants

FIG. A-11



LOG OF TEST PIT TP-1

PROJECT: TRAMCO Complex Expansion



LOG OF TEST PIT TP-2

PROJECT: TRAMCO Complex Expansion

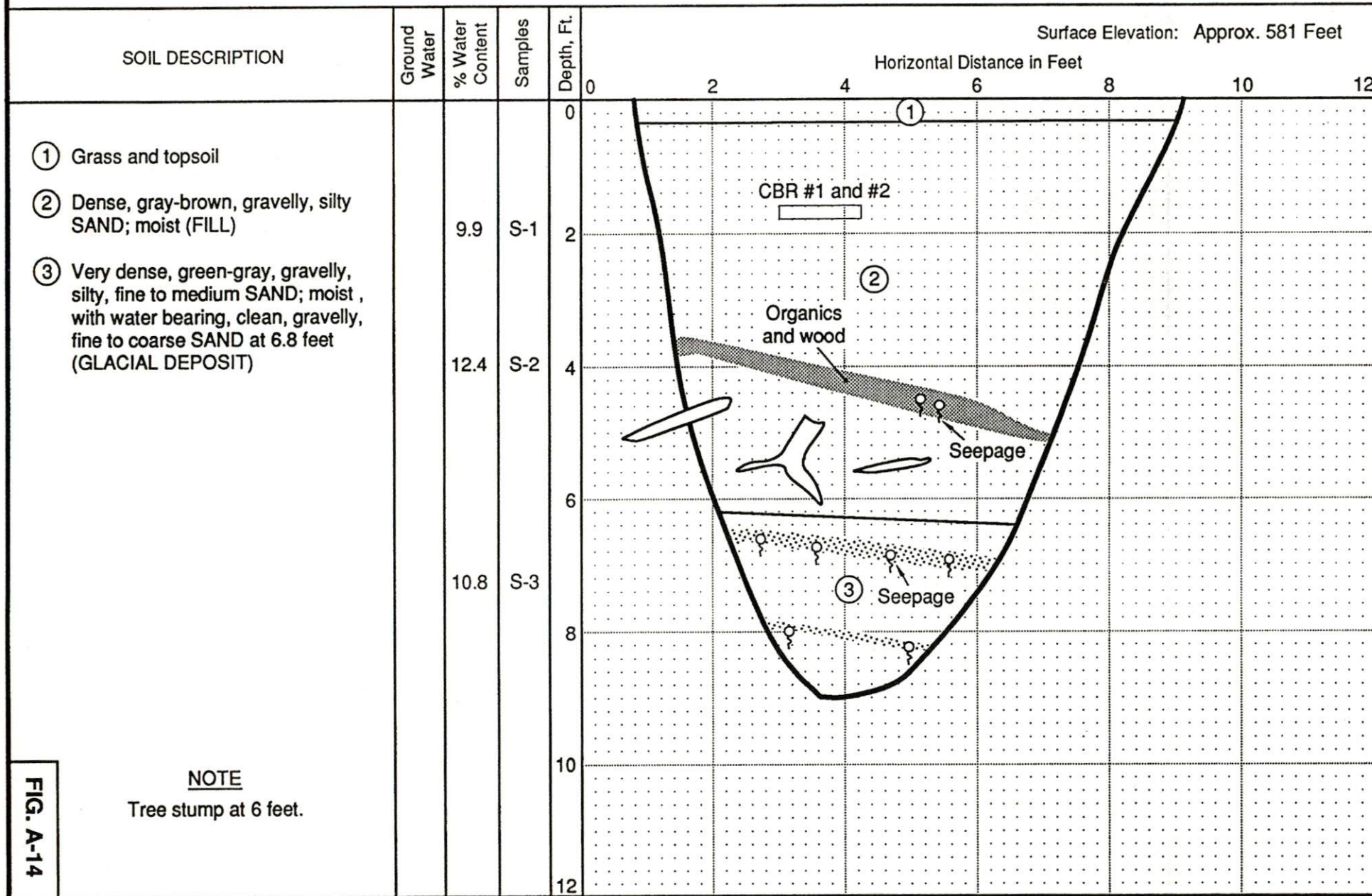


FIG. A-14

SHANNON & WILSON, INC.
Geotechnical Consultants

JOB NO: W-5751-01

DATE: 2-6-91

LOCATION: See Plan

LOG OF TEST PIT TP-3

PROJECT: TRAMCO Complex Expansion

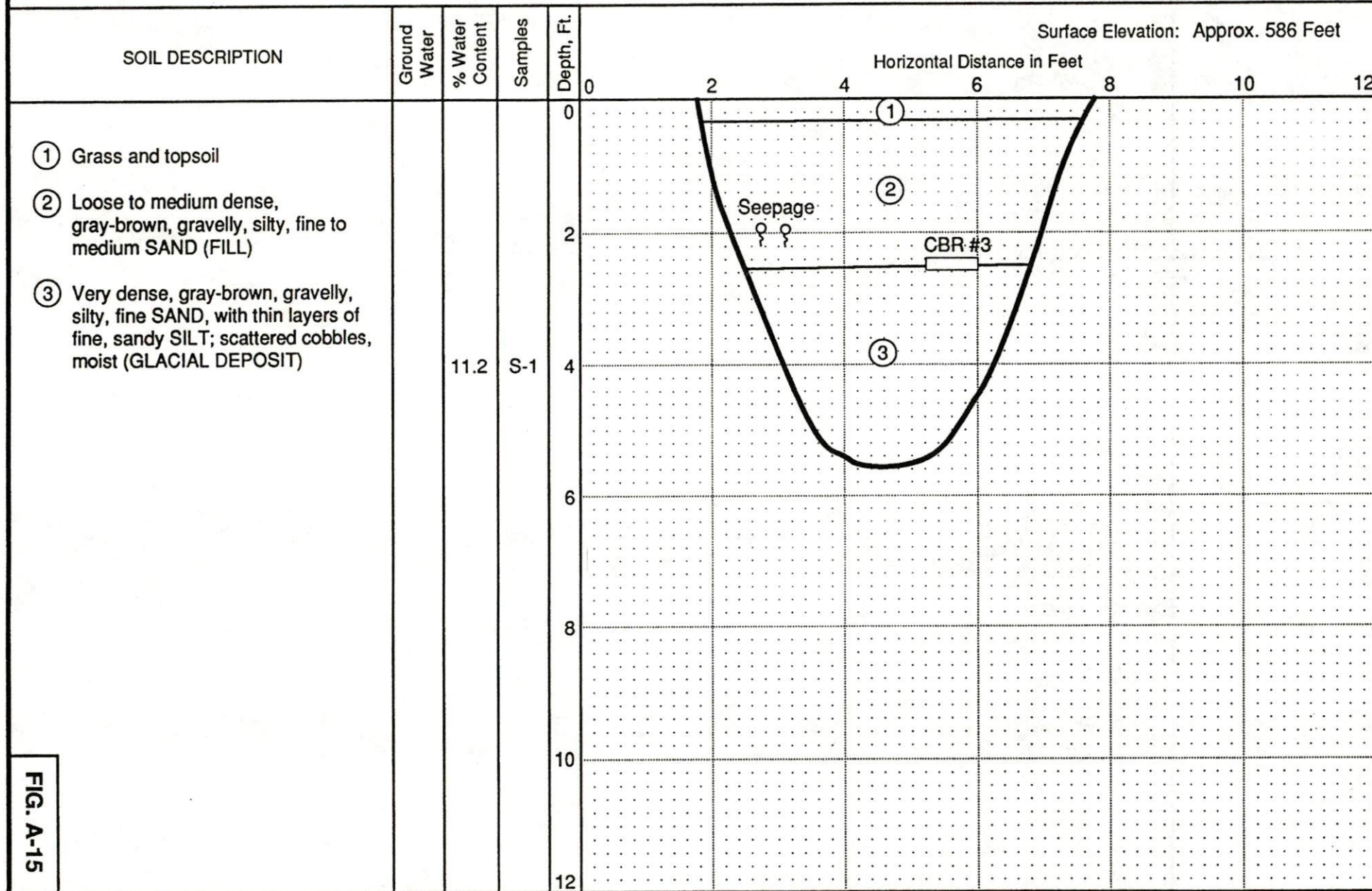


FIG. A-15

SHANNON & WILSON, INC.
Geotechnical Consultants

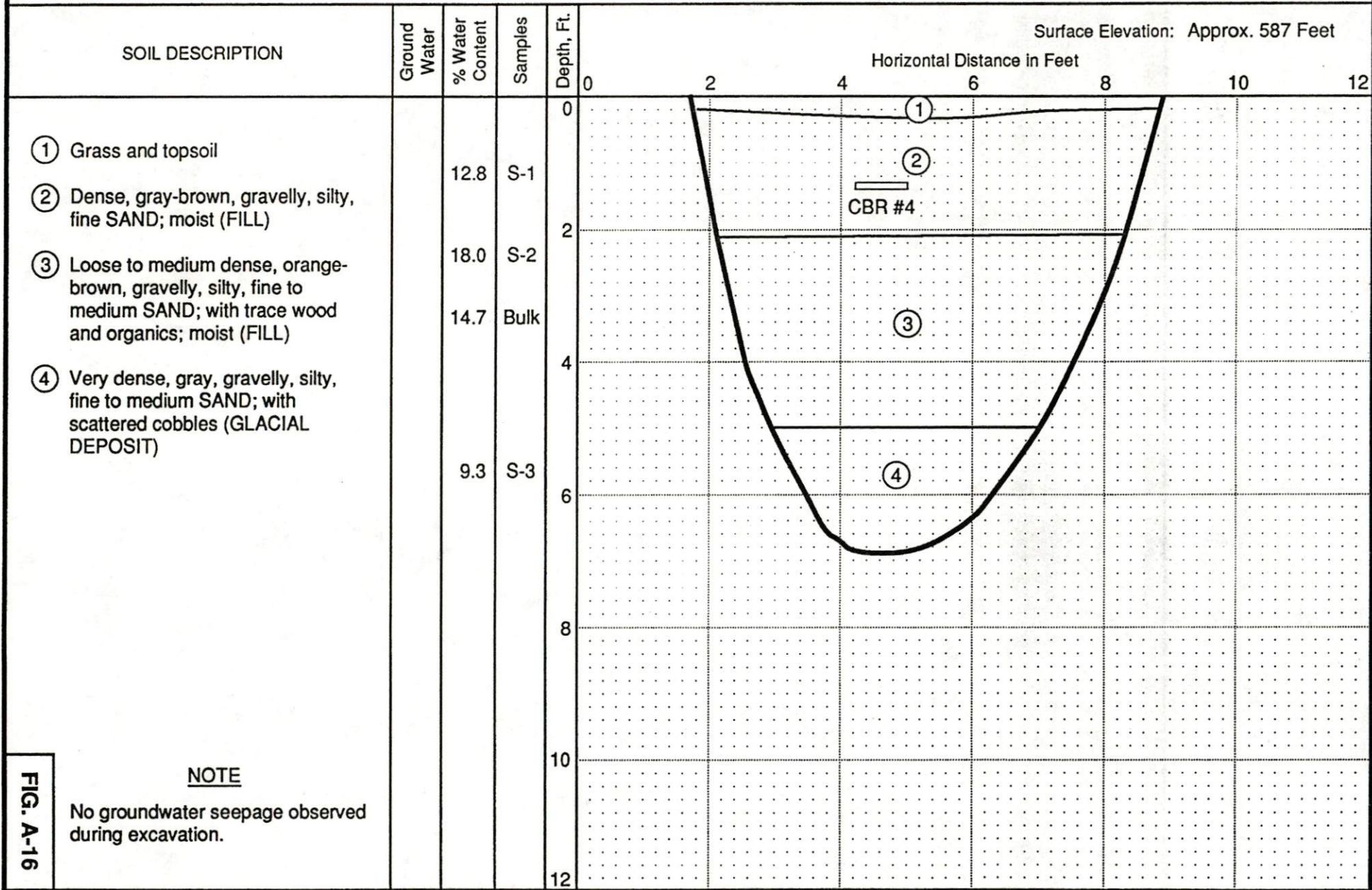
JOB NO: W-5751-01

DATE: 2-6-91

LOCATION: See Plan

LOG OF TEST PIT TP-4

PROJECT: TRAMCO Complex Expansion



APPENDIX B
GEOTECHNICAL LABORATORY TESTING

APPENDIX B

GEOTECHNICAL LABORATORY TESTING

Soil samples were tested to develop parameters for use in evaluating subsurface conditions and preparing geotechnical engineering recommendations for the proposed project. The laboratory testing program included visual classification, moisture content determinations, grain size analyses, and laboratory CBR tests. All tests, excluding the CBR tests, were performed in our laboratory. Testing was performed in general accordance with the American Society of Testing and Materials (ASTM) standard test procedures.

Classification tests were performed on selected samples to aid in determining index properties of tested samples and to permit correlation of engineering properties of tested samples with similar soil types. The soils that were tested were checked against the field log classifications and updated where appropriate in general accordance with ASTM D-2487, "Standard Test Method for Classification of Soils for Engineering Purposes".

Moisture content determinations were performed on selected soil samples in general accordance with ASTM D-2216. The test results were used in classification and correlation of the various soils encountered at the site. The results of these tests are shown on the individual logs of the explorations.

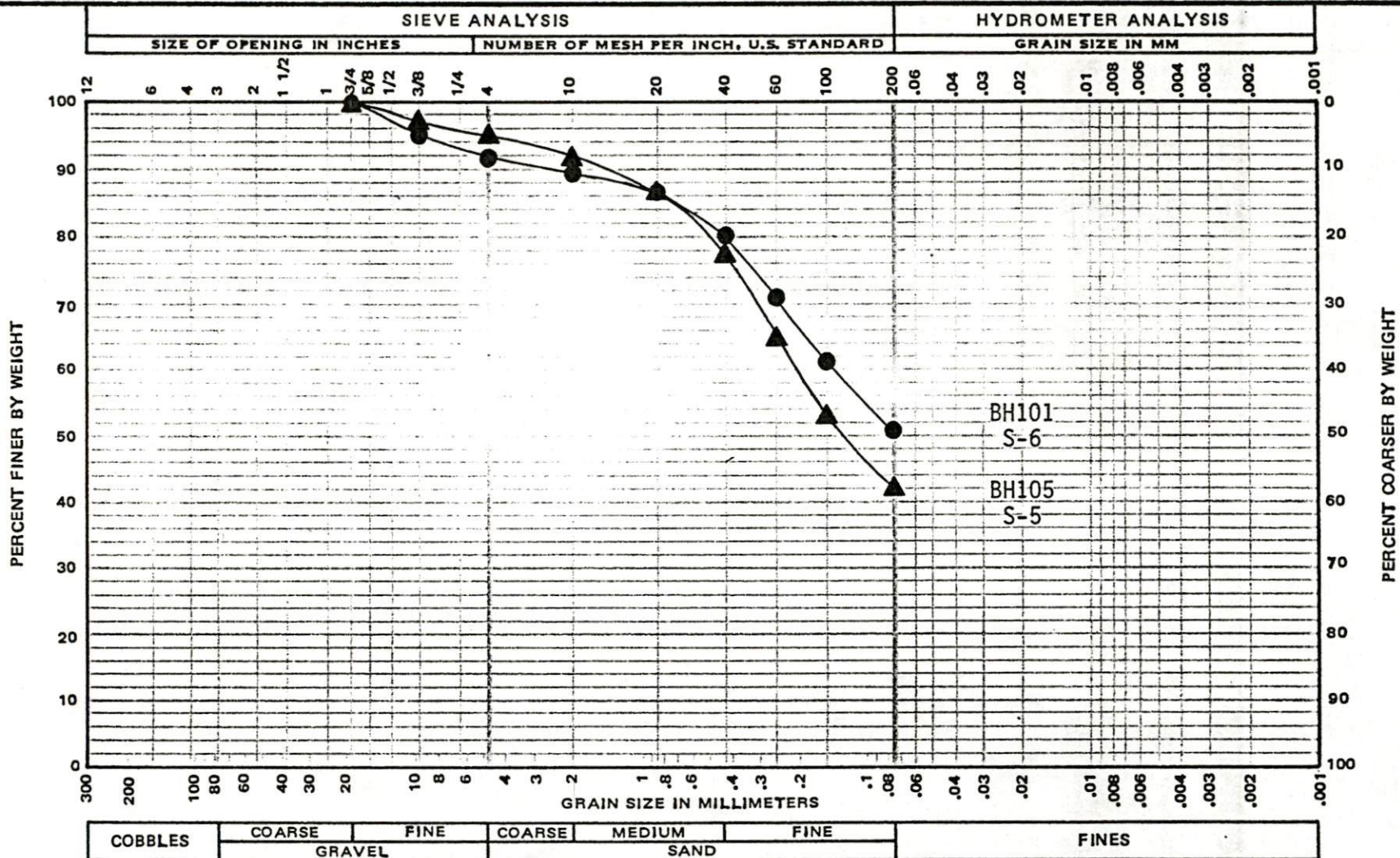
Sieve analyses were performed on selected samples to determine grain size distributions. The tests were performed in general accordance with ASTM D-422. The grain size curves are shown at the end of this appendix on Figure B-1. In addition, the percentage of fines of select soil samples was determined by measuring the percentage of soil passing the number 200 sieve. The results of these measurements are tabulated in Table B-1.

A laboratory California Bearing Ratio (CBR) test was performed on one sample obtained from the test pits for use in evaluating subgrade strengths for pavement design. The test was performed on recompacted silty sand fill from test pit, TP-4, at a depth of 3 feet. The test was performed at the natural moisture content in general accordance with ASTM D-1883. The results of this test indicated a CBR value of 5.

TABLE B-1

PERCENTAGE OF FINES

<u>Boring</u>	<u>Sample</u>	<u>Percent Passing No. 200 Sieve</u>
BH-101	6	50
BH-101	7	42
BH-105	5	42
BH-106	7	38
BH-108	9	65



SAMPLE NO.	DEPTH-FT.	U.S.C.	CLASSIFICATION	NAT. W.C. %	LL	PL	PI	Proposed Tramco Complex Expansion Paine Field, Everett, Washington
BH-101 S-6	20.0 - 20.8	SM	Gray, silty SAND	8.8				
BH-105 S-5	12.5 - 14.0	SM	Gray/Brown, silty SAND	12.1				

GRAIN SIZE DISTRIBUTION

APPENDIX C

PREVIOUS EXPLORATIONS AND INFORMATION

APPENDIX C

PREVIOUS EXPLORATIONS AND INFORMATION

To aid in identifying subsurface conditions beneath the proposed project site, existing information and previous explorations were compiled and evaluated to develop and/or confirm geotechnical conditions at the site and environmental laboratory test results. The primary sources utilized were boring logs from previous geotechnical engineering reports from Converse Consultants NW titled, "Geotechnical Design Report, Proposed Airplane Maintenance Hangar" and dated February 29, 1988 and "Geotechnical Design Report, Proposed Standpipe" and dated November 14, 1988 (Figures C-1 through C-11) and Earth Consultants, Inc. titled, "Geotechnical Engineering Study, TRAMCO Airplane Maintenance Hangar" and dated May 10, 1990 (Figures C-12 through C-21) and an environmental report from Converse Consultants NW titled, "Preliminary Soil and Groundwater Assessment, Three Sites at Paine Field" and dated April 25, 1990 (Figures C-22 through C-24). Approximate locations of these explorations are shown on Figure 1. Because these explorations were performed by others, and because their locations were scaled from other site plans, the accuracy of the soil types and exploration locations cannot be confirmed.

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF ACTUAL CONDITIONS ENCOUNTERED.

DEPTH IN FEET	SAMPLE NO. SAMPLE	BLOWS/6"	OTHER TESTS**	FIELD MOISTURE % OF DRY WEIGHT	DRY DENSITY PCF	DESCRIPTION	SYMBOL	MOISTURE	CONSISTENCY
0						SAND (Fill); gray, fine to coarse, trace silt	SP	wet	loose
1A		3		27					
5		3							
		5				SANDY SILT (Fill); brown to dark brown, fine sand, scattered wood fragments, trace fine gravel	ML	moist to wet	medium stiff
2A		1		36					
		1							
3A		3		15					
		4				SILTY SAND; gray, fine sand, trace medium to coarse sand and fine gravel	SM	2/23/66 moist to wet	very soft medium dense
10		4							
		12				SILTY SAND (Glacial Till); grayish-brown, predominantly fine sand, trace to few fine gravel	SM	moist	very dense
		50/6"				- gray below 13 feet		ATD ∇	
15		22							
		35							
		29							
6A		50/5"							
20									
7A		50/3"							
25						Bottom of boring at depth 23.0' Observation well installed at bottom of boring			
30						Pavement Section: 5-1/2" Portland Cement concrete slab			
35									
40									

* A. 2" split-spoon sampler

B. 3" O.D. thin-wall sampler

C. 3-1/4" O.D. x 2-1/2" liner

** A - Atterberg, C - consolidation, DS - direct shear,

D. 3-1/2" O.D. split barrel sampler X. sample not recovered

G - grain size, T - triaxial, P - permeability



water level
impervious seal
piezometer tip

PROPOSED AIRPLANE MAINTENANCE HANGAR
Everett, Washington
for TRAMCO

Project No.

88-35112

Drawing No.

A-1



Converse Consultants

Geotechnical Engineering
and Applied Sciences

FIG. C-1

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF ACTUAL CONDITIONS ENCOUNTERED.

DEPTH IN FEET	SAMPLE NO. SAMPLE	BLOWS/6"	OTHER TESTS**	FIELD MOISTURE % OF DRY WEIGHT	DRY DENSITY PCF	DESCRIPTION	SYMBOL	MOISTURE	CONSISTENCY
0						SILTY SAND (Fill); gray, predominantly fine sand, trace to few fine gravel	SM	moist to wet	loose
1A		8				- brown below 5 feet			
5		5							
2A		3							
3A		3				SANDY SILT/SILTY SAND (Fill); gray, fine sand, trace fine gravel, scattered wood frag., slight organic odor	ML/ SM	moist	medium dense
10		5			17	- dk. brown to brown w/organic odor & scattered wood frag. below 10.5'			loose
4A		12			30			after drilling moist	medium dense
5A		4			15	SILTY SAND; brown, predominantly fine sand, trace to few fine gravel, with iron stains		moist ATD	medium dense
15		3							
6A		9							
7A		12				SILTY SAND (Glacial Till); grayish-brown, predominantly fine sand, few fine to coarse gravel		moist	very dense
20		12							
8A		12				- with iron stains			
25		35				Bottom of boring at depth 23.0'			
		50/6"				Backfilled with cuttings			
30						Pavement Section: 8" Portland Cement concrete slab			
35									
40									

* A. 2" split-spoon sampler

B. 3" O.D. thin-wall sampler

C. 3-1/4" O.D. x 2-1/2" liner

D. 3-1/2" O.D. split barrel sampler X. sample not recovered

** A - Atterberg, C - consolidation, DS - direct shear,

G - grain size, T - triaxial, P - permeability



water level

impervious seal

piezometer tip

PROPOSED AIRPLANE MAINTENANCE HANGAR
Everett, Washington
for TRAMCO

Project No.

88-35112

Drawing No.

A-2



Converse Consultants

Geotechnical Engineering
and Applied Sciences

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF ACTUAL CONDITIONS ENCOUNTERED.

DEPTH IN FEET	SAMPLE NO. SAMPLE	BLOWS/6"	OTHER TESTS**	FIELD MOISTURE % OF DRY WEIGHT	DRY DENSITY PCF	DESCRIPTION	SYMBOL	MOISTURE	CONSISTENCY
0						SILTY SAND (Fill); gray, predominantly fine sand, few fine gravel	SM	moist	medium dense
1A	9	11							
5		13							
2A	14	15							dense
3A	21	14				- with scattered wood fragments			
10		18							
4A	4	9	16			SANDY SILT/SILTY SAND; gray & brown mottled, streaked with iron stains	ML/ SM	moist to wet	medium dense
5A	15	15	11			SILTY SAND (Glacial Till); grayish-brown, predominantly fine sand, trace to few fine gravel	SM	2/23/00 moist	very dense
15		40							
6A	47	50/5'				- gray below 18 feet			
20								ATD	
7A	31	50/5'							
25									
8A	50/5'								
30									
9A	50/4'								
35						Bottom of boring at depth 33.0' Observation well installed at bottom of boring			
40									

Pavement Section: 13" Portland Cement concrete slab
6" 1-1/2 inch gravel

* A. 2" split-spoon sampler

B. 3" O.D. thin-wall sampler

C. 3-1/4" O.D. x 2-1/2" liner

D. 3-1/2" O.D. split barrel sampler X. sample not recovered

** A - Atterberg, C - consolidation, DS - direct shear,

G - grain size, T - triaxial, P - permeability



water level
impervious seal
piezometer tip

PROPOSED AIRPLANE MAINTENANCE HANGAR
Everett, Washington
for TRAMCO

Project No.

88-35112

Drawing No.

A-3



Converse Consultants

Geotechnical Engineering
and Applied Sciences

FIG. C-3

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF ACTUAL CONDITIONS ENCOUNTERED.

DEPTH IN FEET	SAMPLE NO. SAMPLE	BLOWS/ft	OTHER TESTS**	FIELD MOISTURE % OF DRY WEIGHT	DRY DENSITY PCF	DESCRIPTION	SYMBOL	MOISTURE	CONSISTENCY
0									
1A	21					SILTY SAND (Fill); brown & gray, predominantly fine sand, trace to few fine gravel - with fragments of wood & charcoal & slight organic odor from 5-8'	SM	moist	dense
2A	22								
3A	25								
5	11								medium dense
6A	9	19							
7A	9	11							
8A	11					SILTY SAND (Fill); gray, fine, layered w/iron stained sand lenses	SM	moist	medium dense
9A	8					SILTY SAND; gray, predominantly fine, trace to few fine gravel, with iron stains	SM	moist	medium dense
10	4								
11	7								
12	10	11							
13	32	G	13			SILTY SAND (Glacial Till); grayish-brown, predominantly fine sand, trace to few fine gravel	SM	moist	very dense
15	50/6"								
16A	50/3'								
17						- gray below 20 feet		2/23/00	▽
18									
19									
20									
21									
22									
23									
24									
25									
26									
27									
28									
29									
30									
31									
32									
33									
34									
35									
36									
37									
38									
39									
40									

Pavement Section: 13" Portland Cement concrete slab

* A. 2" split-spoon sampler

B. 3" O.D. thin-wall sampler

C. 3-1/4" O.D. x 2-1/2" liner

D. 3-1/2" O.D. split barrel sampler X. sample not recovered

** A - Atterberg, C - consolidation, DS - direct shear,

G - grain size, T - triaxial, P - permeability



water level
impervious seal
piezometer tip

PROPOSED AIRPLANE MAINTENANCE HANGAR
Everett, Washington
for TRAMCO

Project No.

88-35112

Drawing No.

A-4



Converse Consultants

Geotechnical Engineering
and Applied Sciences

FIG. C-4

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF ACTUAL CONDITIONS ENCOUNTERED.

DEPTH IN FEET	SAMPLE NO. SAMPLE	BLOWS/6"	OTHER TESTS** FIELD MOISTURE % OF DRY WEIGHT DRY DENSITY PCF	DESCRIPTION	SYMBOL	MOISTURE	CONSISTENCY
0							
1A	14 18 18	17		SILTY SAND (Fill); gray, predomi- nantly fine sand, trace to few fine gravel - with organic odor & charcoal & wood fragments from depth 3'-4'	SM	moist	dense
5							
2A	17 12 10			SILTY SAND (Fill); green-gray, predominantly fine sand, trace to few fine gravel - brown below depth 7 feet	SM	moist ATD Δ	medium dense
10						after drilling Δ	
3A	50/0"			SILTY SAND (Glacial Till); gray, predominantly fine sand, trace to few fine gravel	SM	moist	very dense
15							
4A	47 27 50/6"	13					
20							
5A	50/1"						
25				Bottom of boring at depth 22.5' Backfilled with cuttings			
30				Pavement Section: 15" Portland Cement concrete slab			
35							
40							

* A. 2" split-spoon sampler

B. 3" O.D. thin-wall sampler

C. 3-1/4" O.D. x 2-1/2" liner

D. 3-1/2" O.D. split barrel sampler X. sample not recovered

** A - Atterberg, C - consolidation, DS - direct shear,

G - grain size, T - triaxial, P - permeability



water level

impervious seal

piezometer tip

PROPOSED AIRPLANE MAINTENANCE HANGAR
Everett, Washington
for TRAMCO

Project No.

88-35112

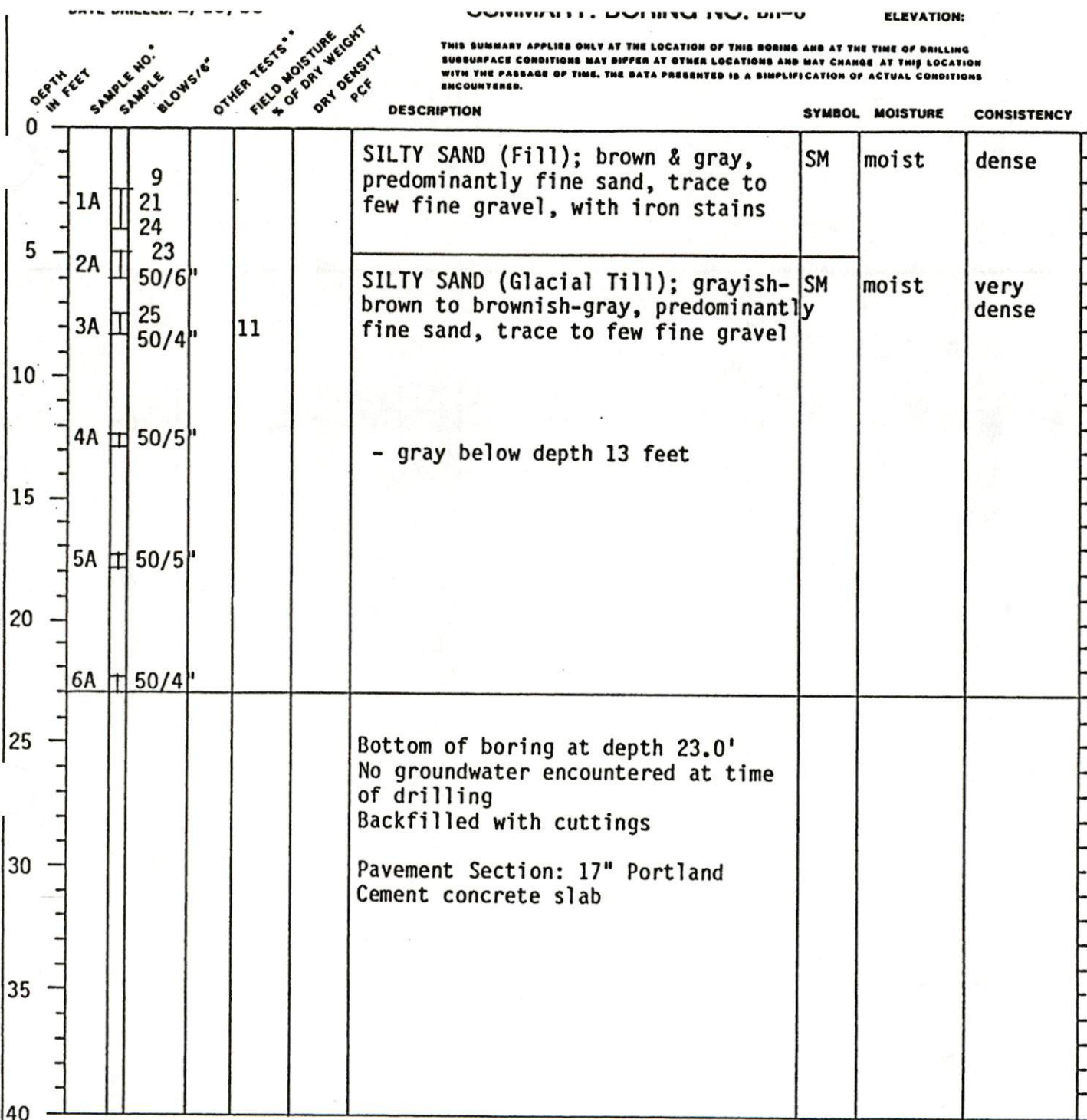
Drawing No.

A-5



Converse Consultants

Geotechnical Engineering
and Applied Sciences



* A. 2" split-spoon sampler
 B. 3" O.D. thin-wall sampler C. 3-1/4" O.D. x 2-1/2" liner ** A - Atterberg, C - consolidation, DS - direct shear,
 D. 3-1/2" O.D. split barrel sampler X. sample not recovered G - grain size, T - triaxial, P - permeability

water level
 impervious seal
 piezometer tip

PROPOSED AIRPLANE MAINTENANCE HANGAR
 Everett, Washington
 for TRAMCO

Project No.
 88-35112

FIG. C-6

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF ACTUAL CONDITIONS ENCOUNTERED.

DEPTH IN FEET	SAMPLE NO. SAMPLE	BLOWS/ft	OTHER TESTS**	FIELD MOISTURE % OF DRY WEIGHT	DRY DENSITY PCF	DESCRIPTION	SYMBOL	MOISTURE	CONSISTENCY
0						SILTY SAND (Fill); gray, predomi- nantly fine sand, trace to few fine gravel	SM	moist	medium dense
1A	6	10							
5		11				ORGANIC SILT; dark brown to black, with scattered wood fragments	OH	moist	stiff
2A	9	14	A	82					
	7		A	30					
10						SILTY CLAY; mottled gray & brown - PP=1.0	ML	after drilling moist	stiff
3A	7	15		17		SILTY SAND (Glacial Till); mottled gray & brown, predominantly fine sand, trace to few fine gravel, with iron stains	SM	moist to wet	dense
15		23						ATD	
4A	7	35	G	15		SILTY SAND; brown & gray, fine to medium, little silt	SM	moist to wet	very dense
20		50/5"							
5A		50/4"				SILTY SAND (Glacial Till); gray, predominantly fine sand, trace to few fine gravel	SM	moist to wet	very dense
25									
6A		50/6"							
30						- with clayey fines	SC		
7A		50/3"							
35						Bottom of boring at depth 33.0' Backfilled with cuttings			
40						Pavement Section: 9-1/2" Portland Cement concrete slab			

PP=Pocket penetrometer in tsf

* A. 2" split-spoon sampler

B. 3" O.D. thin-wall sampler

C. 3-1/4" O.D. x 2-1/2" liner

** A - Atterberg, C - consolidation, DS - direct shear,

D. 3-1/2" O.D. split barrel sampler X. sample not recovered G - grain size, T - triaxial, P - permeability



water level

impervious seal

piezometer tip

PROPOSED AIRPLANE MAINTENANCE HANGAR
Everett, Washington
for TRAMCO

Project No.

88-35112

Drawing No.

A-7



Converse Consultants

Geotechnical Engineering
and Applied Sciences

FIG. C-7

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF ACTUAL CONDITIONS ENCOUNTERED.

DEPTH IN FEET	SAMPLE NO. SAMPLE	BLOWS/6"	OTHER TESTS**	FIELD MOISTURE % OF DRY WEIGHT	DRY DENSITY PCF	DESCRIPTION	SYMBOL	MOISTURE	CONSISTENCY
0									
1A	12 7 4					SILTY SAND (Fill); brown & gray, predominantly fine sand, trace to few fine gravel, with scattered wood fragments	SM	moist	medium dense
5									
2A	2 3 6					POORLY GRADED SAND (Fill); brown & gray, fine to medium, trace fine gravel	SP	moist	loose
10									
3A	23 50/6'					SILTY SAND/SANDY SILT; brown, fine, with iron stains	SM/ ML	moist	medium dense
15									
4A	23 26 50/5'				13	SILTY SAND (Glacial Till); grayish-brown, predominantly fine sand, trace to few fine gravel	SM	moist to wet	very dense
20									
5A	50/4'					- gray below depth 22 feet		after drilling ATD	
25									
6A	50/6'				11	- with clayey fines below depth 25 feet	SM/ SC		
30									
7A	50/5'								
35						Bottom of boring at depth 33.0' Backfilled with cuttings Pavement Section: 5" Asphaltic Concrete; 6" 1-1/2 inch gravel; 12" Silty gravel with sand			
40									

* A. 2" split-spoon sampler

B. 3" O.D. thin-wall sampler

C. 3-1/4" O.D. x 2-1/2" liner

** A - Atterberg, C - consolidation, DS - direct shear,

D. 3-1/2" O.D. split barrel sampler X. sample not recovered

G - grain size, T - triaxial, P - permeability



water level

impervious seal

piezometer tip

PROPOSED AIRPLANE MAINTENANCE HANGAR
Everett, Washington
for TRAMCO

Project No.

88-35112

Drawing No.



Converse Consultants

Geotechnical Engineering
and Applied Sciences

A-8

DATE DRILLED: 11/2/88

SUMMARY: BORING NO. B-1

ELEVATION: 596

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING.
SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION
WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF ACTUAL CONDITIONS
ENCOUNTERED.

DEPTH IN FEET	SAMPLE NO. SAMPLE	BLOWS/6"	OTHER TESTS** FIELD MOISTURE % OF DRY WEIGHT DRY DENSITY PCF	DESCRIPTION	SYMBOL	MOISTURE	CONSISTENCY
0				2" asphaltic concrete over 6" silty sand & gravel base course			
1A	4		16	FILL SILTY SAND; brown and gray, fine to coarse, predominantly fine sand, some silt, trace to few gravel	SM	moist	loose to medium dense
2A	5 11 10		14				
3A	10 19 30	G	12	GLACIAL DEPOSITS SILTY SAND; brownish-gray, fine to coarse, predominantly fine sand, some silt, trace to few gravel	SM	moist	very dense
4A	50/4"		9				
5A	50/6"		9	- grades to gray -			
25				Bottom of boring at depth 23 feet. Backfilled with grout to ground surface No groundwater encountered during drilling.			
30							

* A. 2" split-spoon sampler

B. 3" O.D. thin-wall sampler

C. 3-1/4" O.D. x 2-1/2" liner

**A - Atterberg, C - consolidation, DS - direct shear,

D. 3-1/2" O.D. split barrel sampler

X. sample not recovered

G - grain size, T - triaxial, P - permeability

water level
impervious seal
piezometer tip

PROPOSED STANDPIPE

Paine Field - Everett, Washington
for TRAMCO

Project No.

88-35214

Figure No.

A-1



Converse Consultants NW

Geotechnical Engineering
and Applied Sciences

FIG. C-9

DATE DRILLED: 11/2/88

SUMMARY: BORING NO. B-2

ELEVATION: 596

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING.
SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION
WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF ACTUAL CONDITIONS
ENCOUNTERED.

DEPTH IN FEET	SAMPLE NO. SAMPLE	BLOWS/6"	OTHER TESTS** FIELD MOISTURE % OF DRY WEIGHT DRY DENSITY PCF	DESCRIPTION	SYMBOL	MOISTURE	CONSISTENCY
0				2" asphaltic concrete over 6" silty sand & gravel base course			
1A	9 9 7		14	FILL - SILTY SAND; brown, dark brown and gray, fine to coarse, predominantly fine sand, some silt, few gravel, with wood chips, layers of sandy silt with organics	SM	moist	loose to medium dense
2A	1 3 7		18				
3A	16 34 50/2"		12	GLACIAL DEPOSITS SILTY SAND; gray, fine to coarse, predominantly fine sand, some silt, trace to few gravel	SM	moist	very dense
4A	19 50/5"		11	- grades brownish-gray			
5A	37 50/5"		11				
6A	50/4"		11				
25				Bottom of boring at depth 23 feet. Backfilled with grout from bottom to 7' depth. $\frac{3}{4}$ " dia. PVC piezometer installed to 7' with slotted section from 5' to 7', sand filter pack from 3' to 7'. Grouted from 3' to ground surface, surface monument installed. No groundwater encountered during drilling or measurement on 11/4/88			
30							
35							

* A. 2" split-spoon sampler

B. 3" O.D. thin-wall sampler

C. 3-1/4" O.D. x 2-1/2" liner

D. 3-1/2" O.D. split barrel sampler

X. sample not recovered

**A - Atterberg, C - consolidation, DS - direct shear,

G - grain size, T - triaxial, P - permeability

water level
impervious seal
piezometer tip

PROPOSED STANDPIPE
Paine Field - Everett, Washington
for TRAMCO

Project No.

88-35214

Figure No.

A-2



Converse Consultants NW

Geotechnical Engineering
and Applied Sciences

FIG. C-10

DATE DRILLED: 11/2/88

SUMMARY: BORING NO. B-3

ELEVATION: 594

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING.
SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION
WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF ACTUAL CONDITIONS
ENCOUNTERED.

DEPTH IN FEET	SAMPLE NO. SAMPLE	BLOWS/ft	OTHER TESTS**	FIELD MOISTURE % OF DRY WEIGHT	DRY DENSITY pcf	DESCRIPTION	SYMBOL	MOISTURE	CONSISTENCY
0						2" asphaltic concrete over 6" silty sand & gravel base course			
1A	9			11		FILL			
5	2A	11 6		14		SILTY SAND; brown and gray, fine to coarse, predominantly fine sand, some silt, trace gravel - with wood chips below 5' - grades gray below 7'	SM	moist	medium dense
	3A	4 9		13					
10	4A	7 14	G	13		GLACIAL DEPOSITS			
	5A	12 19		14		SILTY SAND; gray, fine to coarse, predominantly fine sand, some to mostly silt, trace gravel	SM	moist	dense
15	6A	12 17							
		39		10		SILTY SAND; brownish-gray, fine to coarse, predominantly fine sand, some silt, few fine gravel	SM	moist	very dense
20		50/2"							
	7A	50/5"		14					
25						Bottom of boring at depth 23 feet. Backfilled with grout to ground surface. No groundwater encountered during drilling.			
30									
35									

* A. 2" split-spoon sampler

B. 3" O.D. thin-wall sampler

C. 3-1/4" O.D. x 2-1/2" liner

**A - Atterberg, C - consolidation, DS - direct shear,

D. 3-1/2" O.D. split barrel sampler

X. sample not recovered

G - grain size, T - triaxial, P - permeability



water level
impervious seal
piezometer tip

PROPOSED STANDPIPE
Paine Field - Everett, Washington
for TRAMCO

Project No.

88-35214

Figure No.

A-3



Converse Consultants NW

Geotechnical Engineering
and Applied Sciences

FIG. C-11

BORING NO. 1

Logged By SD

Date 4-11-90

Elev. 100' ±*

Graph	US CS	Soil Description	Depth (ft.)	Sample	(N) Blows Ft.	W (%)	
	sm	(6" topsoil and sod)					
		Gray silty SAND with some gravel, moist, medium dense	5	I	15	10	
		-brown, wet		I	14	13	
		-very dense	10				
		-hard drilling		H	50/4"	8	
		-very dense	15				
		-very dense	20	H	50/5"	9	
		-very dense	25	H	50/4"	8	
		-very dense	30	H	50/3"	11	
		-sand lense		H	50/3"	9	
		-very dense	35				
		-very dense		T	50/2"	4	

Boring terminated at 38 feet below existing grade.
No groundwater encountered during drilling.
Boring backfilled with cuttings and bentonite.
*Elevation of SE building corner assumed.
Other elevations relative to this.

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis, and judgement. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.



Earth Consultants Inc.

Consultants in Engineering, Geology & Environmental Sciences

BORING LOG

TRAMCO WAREHOUSE
SNOHOMISH COUNTY, WASHINGTON

Proj. No. 4858

Drwn. GLS

Apr '90

Checked SD

Date 4-25-90

Plate A2

BORING NO. 2

Logged By SD

Date 4-11-90

Elev. 103'±

Graph	US CS	Soil Description	Depth (ft.)	Sample	(N) Blows Ft.	W (%)
	SC	(3" asphalt concrete) Dark brown clayey SAND, moist, loose (Fill) -charred wood	5	I	9	22
	sm	Brown silty SAND, trace organics, wet, loose to medium dense		I	80	13
	sm	Gray silty SAND, wet, very dense	10	I		
	sp	Tan SAND, some silt, wet, very dense		I	87/11"	10
	sm	Tan silty SAND, little gravel, moist, very dense -very dense -gray -very dense -gravel -partially cemented -large boulder -very dense -very dense -partially cemented	15 20 25	I II II T	50/4" 50/1/2" 50/4"	10 9 7

Boring terminated at 28 feet below existing grade.
No groundwater encountered during drilling.
Boring backfilled with cuttings, bentonite and
concrete.

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis, and judgement. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.



Earth Consultants Inc.
CONSULTING ENGINEERS, GEOLOGISTS & ENVIRONMENTAL SCIENTISTS

BORING LOG

TRAMCO WAREHOUSE
SNOHOMISH COUNTY, WASHINGTON

Proj. No. 4858

Drwn. GLS

Apr '90

Checked SD

Date 4-25-90

Plate A3

FIG. C-13

BORING NO. 3

Logged By SD

Date 4-11-90

Elev. 105'±

Graph	US CS	Soil Description	Depth (ft.)	Sample	(N) Blows Ft.	W (%)	
	sm	(6" topsoil and sod) Brown silty SAND, little gravel, wet, loose		I	PUSH	14	
	sm	Gray silty SAND with some gravel, moist, very dense	5	II	50/4"	10	
		-brown -very dense	10	II	50/5"	9	
		-gray -very dense	15	I	50/3"	9	
		-no gravel	20	II	62/6"	9	
		-no gravel -very dense -rock	25	II	50/4"	8	
		-sand becoming finer grained -very dense	30	II	50/4"	8	
		-little gravel -very denes	35	I	50/4"	8	
		-very dense					

Boring terminated at 38 feet below existing grade.
No groundwater encountered during drilling.
Boring backfilled with cuttings and bentonite.

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis, and judgement. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.



Earth Consultants Inc.
Consultants in Engineering, Geology & Environmental Sciences

BORING LOG

TRAMCO WAREHOUSE
SNOHOMISH COUNTY, WASHINGTON

Proj. No. 4858

Drwn. GLS

Apr '90

Checked SD

Date 4-26-90

Plate A4

BORING NO. 4

Logged By SD

Date 4-10-90

Elev. 108'±

Graph	US CS	Soil Description	Depth (ft.)	Sample	(N) Blows Ft.	W (%)	
	sm	(3" asphalt concrete) -cobbles					
		Gray silty SAND with some gravel, moist, very dense	5	I	50/3"	7	
		-very dense					
		-very dense		I	60/6"	8	
			10				
		-more sand					
		-very dense		I	52/6"	8	
		-gravel	15				
		-very dense		I	55/6"	7	
			20				
		-very dense		I	50/2"	8	
			25				
		-very dense		I	50/4"	7	
			30				
		-very dense		I	53/6"	8	
			35				
	sp-sm	Gray gravelly SAND, moist, very dense					
		-very dense		I	80/6"	5	

Boring terminated at 38 feet below existing grade.
No groundwater encountered during drilling.
3/4" PVC standpipe installed to bottom of boring.
Lower 2 feet slotted. Boring backfilled with
sand, bentonite and cuttings.

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis, and judgement. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.



Earth Consultants Inc.
Geotechnical, Laboratory, Geologics & Environmental Sciences

BORING LOG

TRAYCO WAREHOUSE
SNOHOMISH COUNTY, WASHINGTON

Proj. No. 485E

Drwn. GLS

Apr '90

Checked SD

Date 4-26-90

Plate A5

FIG. C-15

BORING NO. 5

Logged By SD

Date 4-13-90

Elev. 110'±

Graph	US CS	Soil Description	Depth (ft.)	Sample	(N) Blows Ft.	W (%)	
	sm	(6" topsoil and sod) Brown silty SAND with some gravel, moist, very dense					
		-very dense	5	I	75	9	
		-gray					
		-very denes		I	50/6"	7	
			10				
		-very dense		I	50/6"	9	
			15				
		-very dense		I	50/4"	9	
			20				
		-very dense		I	50/5"	8	
			25				
				I	50/5"	8	
			30				
				I	50/4"	8	
			35				
				I	50/5"	8	

Boring terminated at 38 feet below existing grade.
No groundwater encountered during drilling.
Boring backfilled with cuttings, concrete and bentonite.

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis, and judgement. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.



Earth Consultants Inc.
Consultants of Engineers, Geologists & Environmental Scientists

BORING LOG

TRAMCO WAREHOUSE
SNOHOMISH COUNTY, WASHINGTON

Proj. No. 4858

Drwn. GLS

Apr '90

Checked SD

Date 4-27-90

Plate A6

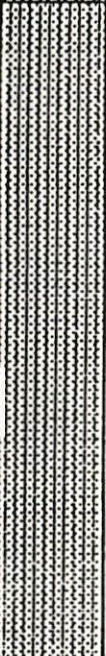
FIG. C-16

BORING NO. 6

Logged By SD

Date 4-13-90

Elev. 110'±

Graph	US CS	Soil Description	Depth (ft.)	Sample	(N) Blows Ft.	W (%)
	SC	(6" topsoil and sod) Brown clayey SAND with some gravel, moist, medium dense (Fill) -medium dense	5	I	13	14
		-medium dense -wood fragments	10	I	26	13
	SM	-wood debris in cuttings				
		Tan, silty SAND with some gravel, moist, very dense -ochre staining	15	I	65	14
		-very dense	20	I	50/5"	9
		-very dense	25	I	50/5"	10
		-very dense	30	I	50/5"	12
		-very dense	35	I	50/4"	12
		-very dense		T	90/9"	13

Boring terminated at 39 feet below existing grade.
No groundwater encountered during drilling.
Boring backfilled with cuttings and bentonite.

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis, and judgement. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.



Earth Consultants Inc.
Geotechnical Engineers, Geologists & Environmental Scientists

BORING LOG

TRAMCO WAREHOUSE
SNOHOMISH COUNTY, WASHINGTON

Proj. No. 4858

Drwn. GLS

Apr '90

Checked SD

Date 4-27-90

Plate A7

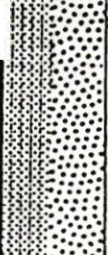
FIG. C-17

BORING NO. 7

Logged By SD

Date 4-10-90

Elev. 104' ±

Graph	US CS	Soil Description	Depth (ft.)	Sample	(N) Blows Ft.	W (%)	
	sm	(3" of asphalt) Light brown silty SAND with little gravel, moist, dense		I	75/8"	15	
	SM	-rock in sample tube	5				
		-mild hydrocarbon odor in cuttings		I	75/6"	8	
		-grades to gray:	10				
		-hard drilling		I	50/2"	9	
		-very dense	15				
		-very dense	20	I	65/6"	7	
		-very dense	25	I	85/5"	9	
		-very dense	30	I	50/5"	7	
		-very dense	35	I	65/6"	9	
	sm-sp	Gray SAND, some silt, little gravel, moist, very denes		I	62/6"	6	

Boring terminated at 38 feet below existing grade.
No groundwater encountered during drilling.
Boring backfilled with cuttings and concrete.

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis, and judgement. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.



Earth Consultants Inc.
Geotechnical Engineers, Geologists, & Environmental Scientists

BORING LOG

TRAMCC WAREHOUSE
SNOHOMISH COUNTY, WASHINGTON

Proj. No. 4858

Drwn. GLS

Apr '90

Checked SD

Date 4-27-90

Plate A8

FIG. C-18

BORING NO. 8

Logged By SD

Date 4-10-90

Elev. 103'±

Graph	US CS	Soil Description	Depth (ft.)	Sample	(N) Blows Ft.	W (%)	
	sm	(3" asphalt concrete) Light brown silty SAND with little gravel, moist, very dense -grades to gray -very dense -very dense	5		50/3"	8	
			10		52/6"	8	
		-3" thick sand lense -very dense	15		55/6"	7	
		-very dense	20		65/6"	10	
		-very dense	25		60/6"	8	
		-drilling becoming very difficult					
					50/5"	6	

Boring terminated at 28 feet below existing grade.
No groundwater encountered during drilling.
3/4" PVC standpipe installed to bottom of boring.
Lower 2 feet slotted. Boring backfilled with sand,
bentonite and cuttings.

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis, and judgement. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.



Earth Consultants Inc.
Geotechnical Engineering, Geology & Environmental Sciences

BORING LOG

TRAYCO WAREHOUSE
SNOHOMISH COUNTY, WASHINGTON

Proj. No. 4858

Drwn. GLS

Apr '90

Checked SD

Date 4-27-90

Plate A9

BORING NO. 9

Logged By SD

Date 4-13-90

Elev. 105'±

Graph	US CS	Soil Description	Depth (ft.)	Sample	(N) Blows Ft.	W (%)	
	sm	(6" topsoil and sod)					
		Gray silty SAND, some gravel, moist, very dense		II	57/6"	7	
			5				
		-3" thick sand lense -very dense		II	50/3"	10	
			10				
		-very dense		II	50/6"	8	
			15				
		-very dense		II	50/3"	8	
			20				
		-1" thick sand lense -very dense		II	50/3"	10	
			25				
		-very dense		II	50/2"	10	
		-drilling becoming easier -no gravel -very dense		II	50/2"	9	
			30				
		-very dense		II	50/4"	8	
			35				

Boring terminated at 38 feet below existing grade.
No groundwater encountered during drilling.
Boring backfilled with cuttings and bentonite.

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis, and judgement. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.



Earth Consultants Inc.
Geotechnical Engineers, Geologists & Environmental Scientists

BORING LOG

TRAMCO WAREHOUSE
SNOHOMISH COUNTY, WASHINGTON

Proj. No. 4858

Drwn. GLS

Apr '90

Checked SD

Date 4-27-90

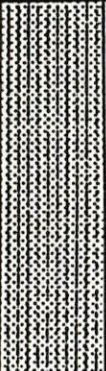
Plate A10

BORING NO. 10

Logged By SD

Date 4-16-90

Elev. 95'±

Graph	US CS	Soil Description	Depth (ft.)	Sample	(N) Blows Ft.	W (%)	
	sm	(3" sod) Tan silty SAND, some gravel, moist, very dense -very dense	5	I	83/11"	11	
		-very dense -grades to gray	10	II	50/5"	8	
		-very dense	15	III	50/6"	8	

Boring terminated at 15.5 feet below existing grade.
Groundwater encountered at 15 feet during drilling.
3/4" PVC standpipe installed to bottom of boring.
Lower 2 feet slotted. Boring backfilled with sand,
bentonite, cuttings and concrete.

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis, and judgement. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.



Earth Consultants Inc.
Geotechnical Engineers, Geologists & Environmental Scientists

BORING LOG

TRAMCO WAREHOUSE
SNOHOMISH COUNTY, WASHINGTON

Proj. No. 4858

Drwn. GLS

Apr '90

Checked SD

Date 4-27-90

Plate All

FIG. C-21



Converse NW

Monitoring Well Geologic & Construction Log

Project Number
90-35124

Well Number

CTW-101

Sheet 1 of 1

Project Preliminary Soil & Groundwater Assessment

Elevation (Top of Well Casing)

Water Level Elev. NGVD

585.13

Drilling Contractor Geoboring

Drilling Method HSA

Location Everett, Washington

Surface Elevation NGVD 595.00

Start Date March 28, 1990

Finish Date March 28, 1990

Depth feet	Well Construction	Lab Tests	Blows/ 6"	OV Reading	Description
	locking, water tight, flush metal monument concrete grout annular seal				SILTY SAND WITH GRAVEL (Fill); gray-brown, fine to medium; medium dense, moist
2	bentonite seal				
	blank well casing 4" ID PVC schedule 40	C	1 2 3	347 ppm	SAND (Fill); gray, fine to coarse, trace silt; medium dense, moist; strong petroleum odor slight visible sheen
4					
	well screen 4" ID PVC schedule 40; .010 slot width		1 1 1	47 ppm	SILTY SAND WITH GRAVEL (Fill); gray, fine to medium; very loose, wet; strong petroleum odor and visible sheen
6					
			1 0 1	40 ppm	same; strong petroleum odor and visible sheen
8					
	4/3/90	C	3 9 23	257 ppm	SILTY SAND WITH GRAVEL (Glacial Till); gray-brown, fine to medium, trace cobbles; dense, moist; strong petroleum odor with discolored soil layers
10	filter pack 10/20 silica sand				
			16 50/ 6"	25 ppm	
12	8" thread end plug				
			9 20 30	12 ppm	SAND WITH GRAVEL (Glacial Till); gray, fine to medium, some silt, with irregular laminations of fine sand; dense, moist; slight petroleum odor no visible sheen
14					
	bottom of boring was backfilled with granular bentonite	C	18 31 35	10 ppm	
16					
18					
					Bottom of boring at depth 19 feet

ST - Sampler Type:

4" ID Split Spoon

Bulk Grab Sample

Drive Barrel

Lab Tests:

S - Soil Properties

C - Chemical Properties

Water Level

Logged by: DAY

Approved by: EWM

Figure No. B-1

FIG. C-22



Converse NW

Monitoring Well Geologic & Construction Log

Project Number
90-35124

Well Number

CTW-102

Sheet 1 of 1

Project Preliminary Soil & Groundwater Assessment

Location Everett, Washington

Elevation (Top of Well Casing)

Surface Elevation NGVD 598.00

Water Level Elev. NGVD

596.25

Start Date March 28, 1990

Drilling Contractor Geoboring

Finish Date March 28, 1990

Drilling Method HSA

Depth feet	Well Construction	Lab Tests	Blows/ 6"	OM Reading	Description
	locking, water tight, flush metal monument concrete grout annular seal				GRAVEL 3/8-inch crushed
2	bentonite seal 4/3/90				SILTY SAND WITH GRAVEL (Fill); gray-brown, fine to medium, trace cobbles; loose, wet; no petroleum odor or visible sheen
	blank well casing 4" ID PVC schedule 40		2 0 4	0 ppm	
4					SAND (Fill); gray, fine to coarse, trace silt; loose, wet; no petroleum odor or visible sheen
	well screen 4" ID PVC schedule 40; .010 slot width	C	1 1 2	0 ppm	
6					
8		C	2 3 6	42 ppm	-grades with some silt; slight to moderate odor with moderate visible sheen
	filter pack 10/20 silica sand				
10		C	3 3 4	9 ppm	SAND (Fill); gray, fine to coarse, trace silt; loose, wet; slight petroleum odor and visible sheen
12	6" threaded end plug				SILTY SAND WITH GRAVEL (Glacial Till); gray, fine to medium, trace cobbles; very dense, moist; no petroleum odor or visible sheen
			50 50/ 2"	0 ppm	
14	due to installation problems and heaving sands monitoring well was relocated approximately 5 feet west of original location and drilled to a depth of 12 feet		8 50/ 5"	0 ppm	
16					Bottom of boring at depth 15.8 feet
18					

ST - Sampler Type:

4" ID Split Spoon

Bulk Grab Sample

Drive Barrel

Lab Tests:

S - Soil Properties

C - Chemical Properties

Water Level

Logged by: DAY

Approved by: EWM

Figure No. B-2



Converse NW

Monitoring Well Geologic & Construction Log

Project Number

90-35124

Well Number

CTW-103

Sheet 1 of 2

Project Preliminary Soil & Groundwater Assessment

Location Everett, Washington

Elevation (Top of Well Casing)

Surface Elevation NGVD 590.00

Water Level Elev. NGVD

582.10

Start Date April 1, 1990

Drilling Contractor Geoboring

Finish Date April 1, 1990

Drilling Method HSA

Depth feet	Well Construction	Lab Tests	Blows/6"	OVM Reading	Description
2	locking, water tight, flush metal monument concrete grout annular seal	C	5 9 8	5 ppm	ASPHALT 3-inches SILTY SAND WITH GRAVEL (Fill); gray, fine to medium; very loose, very moist; strong petroleum odor and slight visible sheen (boring restarted to the west from original location, encountered 6-inch diameter steel fuel line at 2.5 feet below ground surface)
4	bentonite seal		7 6 6	0 ppm	-grades to gray mottled brown, trace organic debris; no petroleum odor or visible sheen
6					-grades with zones or layers of organic debris; no petroleum odor or visible sheen
8	4/3/90 blank well casing 4" ID PVC schedule 40	C	3 2 3	15 ppm	SAND WITH SILT (Fill); gray-brown, fine to medium; loose, very moist
10		C	1 1 2	63 ppm	SILTY SAND WITH GRAVEL (Fill); brown, fine to medium, trace roots; loose moist; slight petroleum odor no visible sheen
12	well screen 4" ID PVC schedule 40; .010 slot width	C	2 2 4	10 ppm	SILTY SAND (Fill); gray, fine to coarse, few gravel, trace organic debris; very loose, wet; slight petroleum odor, no visible sheen
14			3 3 3	0 ppm	
16			3 6 9	0 ppm	ORGANIC SILT (Old Ground Surface); dark-brown, trace organics, roots, and gravel; stiff, wet; no petroleum odor or visible sheen
18	filter pack 10/20 silica sand				

ST - Sampler Type:

4" ID Split Spoon

Bulk Grab Sample

Drive Barrel

Lab Tests:

S - Soil Properties

C - Chemical Properties

Water Level

Logged by: DAY

Approved by: EWM

Figure No. B-3

FIG. C-24

Converse NW		Monitoring Well Geologic & Construction Log			
		Project Number 90-35124	Well Number CTW-103	Sheet 2 of 2	
Project Preliminary Soil & Groundwater Assessment			Location Everett, Washington		
Elevation (Top of Well Casing) _____			Surface Elevation NGVD 596.00		
Water Level Elev. NGVD 582.10			Start Date April 1, 1990		
Drilling Contractor Geoboring			Finish Date April 1, 1990		
Drilling Method HSA					

Depth feet	Well Construction	Lab Tests	Blows/ 6"	OVM Reading	Description
			6	0 ppm	SILTY SAND WITH GRAVEL (Glacial Sediments); gray, fine to coarse, trace roots; dense, wet; no petroleum odor or visible sheen
			12		
			14		
-22	 8" thread end plug				SILTY SAND (Glacial Sediments); gray, fine to medium, trace to little gravel, thinly bedded with silty sand with gravel; very dense, very moist; no petroleum odor or visible sheen
-24					
-26			18	0 ppm	Bottom of boring at depth 26 feet
			50/ 6"		
-28					
-30					
-32					
-34					
-36					
-38					

ST - Sampler Type: 4" ID Split Spoon Bulk Grab Sample Drive Barrel	Lab Tests: S - Soil Properties C - Chemical Properties Water Level	Logged by: DAY Approved by: EWM Figure No. B-3
--	--	---

APPENDIX D
CHEMICAL LABORATORY ANALYSIS

APPENDIX D

CHEMICAL LABORATORY ANALYSIS

GENERAL

Select soil and groundwater samples in borings BH-103, BH-107, and BH-111 were tested by an analytical laboratory, Freidman & Bruya, Inc. Details and results of the chemical laboratory tests are presented in the following sections.

RESULTS OF GROUNDWATER ANALYSIS

Three permanent groundwater monitoring wells were constructed at the locations noted on Figure 1 and constructed as shown on the boring logs. Monitoring wells were installed and developed (i.e., evacuation of 35 gallons of groundwater from each well) at locations identified as BH-103, BH-107, and BH-111 and sampled on February 1 and 8, 1991. Upon completion of development processes at each well, representative samples were obtained and submitted to the project laboratory for analysis by EPA Method 602 (benzene, toluene, ethylbenzene, and xylenes) and by EPA Method 8015 for total petroleum hydrocarbons. Table D-1 lists the results of testing performed on February 1 and 8, 1991.

Elevated concentrations of benzene and total xylenes were reported by the project laboratory in groundwater samples obtained from BH-111 on February 1, 1991. The levels of benzene and total xylenes are above the Washington State Department of Ecology (WDOE) Model Toxic Control Act (MTCA) Method A Cleanup Levels (for groundwater), effective February 28, 1991. Groundwater samples obtained from monitoring wells BH-103 and BH-107 each contained concentrations of benzene, toluene, ethylbenzene and total xylenes (BTE&X) below current regulatory guidelines. Each of the wells sampled contained concentrations of total petroleum hydrocarbons, as determined by EPA Method 8015, below 0.8 parts per million (ppm), which is below the 1 ppm WDOE MTCA guideline (refer to Table D-1 for additional information).

Groundwater samples were again obtained from monitoring wells BH-103, BH-107, and BH-111 on February 8, 1991 in an effort to reconfirm the findings of February 1, 1991. The results of testing conducted on February 1, 1991 in wells identified as BH-107 and BH-111 could not be duplicated. This may be attributed to the fact that these wells may be on the outer fringe of a plume of dissolved hydrocarbons at the site and that aggressive purging (such as, volumes in excess of 35 gallons) as performed for the first sampling period would be required to duplicate conditions sampled on February 1, 1991 at these locations.

RESULTS OF SOIL ANALYSIS

Twenty-four individual soil samples were collected from borings BH-103, BH-107, and BH-111 during soil drilling activities of January 24 and 25, 1991. Soil samples were obtained in general accordance with our proposal dated January 14, 1991.

Table D-2 lists the results of analyses performed on the collected soil samples. The analytical data suggests that the sampled soils contained concentrations of petroleum hydrocarbons and lead below current regulatory cleanup levels (MTCA, Chapter 173-340 WAC, Method A Cleanup Levels - Industrial Soil, effective February 28, 1991).

D-1
GROUNDWATER RESULTS FROM TRAMCO PAINE FIELD PROJECT (1)
PAGE 1 OF 1

SAMPLE LOCATION & DATE	BENZENE (ppb) (2)	TOLUENE (ppb)	ETHYL BENZENE (ppb)	TOTAL XYLENE (3) (ppb)	TOTAL PETROLEUM HYDROCARBON EPA METHOD 8015 (ppm) (4)	QA RELATIONSHIP
02/01/91						
PF-BH103-42-W-0	<1	<1	<1	<1		Duplicate Sample
PF-BH103-43-W-1	<1	<1	<1	<1		
PF-BH107-46-W-0	<1	<1	<1	<1		Duplicate Sample
PF-BH107-47-W-1	<1	<1	<1	<1		
PF-BH111-50-W-0	22	<1	<1	517		Duplicate Sample
PF-BH111-51-W-1	19	<1	<1	479		
PF-BH103-44-W-0					<0.8	
PF-BH103-45-W-1					<0.8	Duplicate Sample
PF-BH107-48-W-0					<0.8	
PF-BH107-49-W-1					<0.8	Duplicate Sample
PF-BH111-52-W-0					<0.8	
PF-BH111-53-W-1					<0.8	Duplicate Sample
02/08/91						
PF-BH103-54-W-0	<1	<1	<1	<1	NT (5)	
PF-BH107-55-W-0	2	<1	<1	371	NT	
PF-BH111-57-W-1	<2	<2	<2	<2	NT	

< Depicts below detection limit, detection limit reported.

Notes:

- 1) As reported by Friedman and Bruya, Inc.; 02-7-91 & 02-12-91.
- 2) Parts Per Billion (ppb).
- 3) Xylene reported as m,p & o isomers; reported here as total xylenes.
- 4) Parts Per Million (ppm).
- 5) Not Tested (NT).
- 6) Original laboratory report and correspondence available at the S&W Seattle office.
- 7) Benzene maximum concentrations in water is 5 ppb; (WDOE, Model Toxics Control Act (MTCA), Chapter 173-340 WAC, 02-28-91, Method A Cleanup Levels - Groundwater).
- 8) Toluene maximum concentrations in water is 40 ppb; (WDOE, MTCA Method A Cleanup Levels - Groundwater).
- 9) Ethylbenzene maximum concentrations in water is 30 ppb; (WDOE, MTCA Method A Cleanup Level - Groundwater).
- 10) Xylenes maximum concentrations in water is 20 ppb; (WDOE, MTCA Method A Cleanup Level - Groundwater).
- 11) Benzene, toluene, ethylbenzene and xylene concentrations determined by EPA Method 602.
- 12) TPH is the abbreviation for total petroleum hydrocarbons as diesel as determined by EPA Modified Method 8015.
- 13) TPH maximum concentrations in water is 1 ppm; (WDOE, MTCA Method A Cleanup Level - Groundwater).

SAMPLE LOCATION & DATE	BENZENE (ppb) (2)	TOLUENE (ppb)	ETHYL BENZENE (ppb)	TOTAL XYLENE (3) (ppb)	EPA METHODS 5030 & 8015 (4) (As Gasoline) (ppm)	EPA METHOD 8015 (GC/FID) (5) (As Diesel) (ppm)	LEAD By ICP (6) (ppm)	QA RELATIONSHIP
01-24-91								
PF-BH107-007-S-0						<10	16	
PF-BH107-009-S-0						<10	14	
PF-BH107-013-S-0						<10	10	
PF-BH107-014-S-1						<10	12	Duplicate
PF-BH107-008-S-0	<1	<1	<1	<1				
PF-BH107-010-S-0	<1	<1	<1	<1				
PF-BH107-011-S-0	<1	<1	<1	<1				
PF-BH107-012-S-1	<1	<1	<1	<1				Duplicate
01-25-91								
PF-BH111-034-S-0					<1	<10	9.7	
PF-BH111-036-S-0					<1	<10	7.5	
PF-BH111-039-S-1					<1	<10	7.5	Duplicate
PF-BH111-041-S-0					<1	<10	6.8	
PF-BH103-022-S-0					<1	<10	12	
PF-BH103-025-S-0					<1	18 (7)	21	
PF-BH103-026-S-1					<1	<10	12	Duplicate
PF-BH103-027-S-0					<1	<10	7.8	
PF-BH111-033-S-0	<1	<1	<1	<1				
PF-BH111-035-S-0	<1	<1	<1	<1				
PF-BH111-037-S-1	<2	<2	<2	<2				Duplicate
PF-BH111-038-S-0	<1	<1	<1	<1				
PF-BH103-021-S-0	<1	<1	<1	<1				
PF-BH103-023-S-0	<1	<1	<1	<1				
PF-BH103-024-S-1	<1	<1	<1	<1				Duplicate
PF-BH103-028-S-0	<1	<1	<1	<1				

SEE NOTES ON PAGE 2 OF 2.

Notes for Table D-2; Page 2 of 2.

< Depicts below detection limit, detection limit reported.

- 1) As reported by Friedman and Bruya, Inc.; 01-30-91.
- 2) Parts Per Billion (ppb).
- 3) Xylene reported as m,p & o isomers; reported here as total xylenes.
- 4) Parts Per Million (ppm) as Gasoline.
- 5) Parts Per Million (ppm) as Diesel.
- 6) Lead in ppm as reported by Inductively Coupled Plasma Spectroscopy (ICP); maximum concentration in soil is 1000 ppm, (WDOE, Model Toxics Control Act (MTCA) Chapter 173-340 WAC, 02-28-91, Method A Cleanup Levels - Industrial Soil).
- 7) A small amount of a material heavier than the analyte was present.
- 8) Original laboratory report and correspondence available at the S&W Seattle office.
- 9) Benzene maximum concentrations in soil is 500 ppb; (WDOE, MTCA Method A Cleanup Levels - Industrial Soil).
- 10) Toluene maximum concentrations in soil is 40,000 ppb; (WDOE, MTCA Method A Cleanup Levels - Industrial Soil).
- 11) Ethylbenzene maximum concentrations in soil is 20,000 ppb; (WDOE, MTCA Method A Soil Cleanup Level - Industrial Soil).
- 12) Xylenes maximum concentration in soil is 20,000 ppb; (WDOE MTCA, Method A Soil Cleanup Level - Industrial Soil).
- 13) Benzene, toluene, ethylbenzene and xylene concentrations determined by EPA Method 8020.
- 14) TPH (Diesel) maximum concentration in soil is 200 ppm; (WDOE, MTCA Method A Cleanup Level - Industrial Soil).
- 15) TPH (Gasoline) maximum concentration in soil is 100 ppm; (WDOE, MTCA Method A Cleanup Level - Industrial Soil).

Important Information About Your Geotechnical Engineering Report

A GEOTECHNICAL ENGINEERING REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical engineering report is based on a subsurface exploration plan designed to incorporate a unique set of project-specific factors. These typically include: the general nature of the structure involved, its size and configuration; the location of the structure on the site and its orientation; physical concomitants such as access roads, parking lots, and underground utilities, and the level of additional risk which the client assumed by virtue of limitations imposed upon the exploratory program. To help avoid costly problems, consult the geotechnical engineer to determine how any factors which change subsequent to the date of the report may affect the recommendations.

Unless your consulting geotechnical engineer indicates otherwise, your geotechnical engineering report should not be used:

- ☐ when the nature of the proposed structure is changed; for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one;
- ☐ when the size or configuration of the proposed structure is altered;
- ☐ when the location or orientation of the proposed structure is modified;
- ☐ when there is a change of ownership; or
- ☐ for application to an adjacent site.

Geotechnical engineers cannot accept responsibility for problems which may develop if they are not consulted after factors considered in their reports have changed.

MOST GEOTECHNICAL "FINDINGS" ARE PROFESSIONAL ESTIMATES.

Site exploration identifies subsurface conditions only at those points where samples are taken and when they are taken, but the physical means of obtaining subsurface data precludes the determination of precise conditions. Consequently, the information obtained is intended to be sufficiently accurate for design, but is subject to interpretation. Additionally, data derived through sampling and subsequent laboratory testing are extrapolated by the geotechnical engineer who then renders an opinion about overall subsurface conditions, their likely reaction to proposed construction activity, and appropriate foundation design. Even under optimal circumstances actual conditions may differ from those opined to exist, because no geotechnical engineer, no matter how qualified, and no subsurface exploration program, no matter how comprehensive, can reveal what is hidden by earth, rock and time. For example, the actual interface between materials may be far more gradual or abrupt than the report indicates, and actual conditions in areas not sampled may differ from predictions. Nothing can be done to prevent the unanticipated, but steps can be taken to help minimize their impact. For this reason, most experienced owners retain their geotechnical consultant through the construction stage, to identify variances, conduct additional tests which may be needed, and to recommend solutions to problems encountered on site. Prudent owners establish contingencies to account for such variations in subsurface conditions as exposed during construction.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be modified by constantly changing natural forces. Because a geotechnical engineering report is based on conditions which existed at the time of subsurface exploration, construction decisions should not be based on a geotechnical engineering report whose adequacy may have been affected by time. Speak with the geotechnical consultant to learn if additional tests are advisable before construction starts. For example, groundwater conditions commonly vary seasonally.