GEOTECHNICAL REPORT
TAYLOR CREEK CULVERTS PHASE 2
SEATTLE, WASHINGTON

Work Authorization No.: C399315
November 2004

Seattle
Public Utilities
Materials Laboratory

707 South Plummer Street
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GEOTECHNICAL REPORT
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1.0 INRODUCTION

1.1 GENERAL

This report presents the results of Seattle Public Utilities (SPU) Materials Laboratory's geotechnical investigation for proposed Taylor Creek Culverts Phase 2 Project. The project location, existing site layout, and locations of borings completed for this project are shown on Figure 1. A profile showing our interpretation of subsurface conditions along the project alignment is presented as Figure 2. Our understanding of the planned project is based on conversations with the design team, background materials provided by the project team, and review of site conditions. The following sections describe the results of our investigation and the key geotechnical issues related to the project.

1.2 SCOPE OF WORK

Our scope of work included background review of existing data surrounding the site, performing a utility locate and subsurface investigation with four borings, laboratory testing, and geotechnical analyses to provide design and construction recommendations for proposed modifications at Taylor Creek. Authorization for this work was provided by Fitsum Aberra on May 6, 2004.

1.3 PROJECT DESCRIPTION

The focus of the Taylor Creek Phase 2 project is to provide fish passage past the barriers near Rainier Ave S so that salmon can reach high quality habitat in Lakeridge Park. SPU evaluated several alternatives for achieving the project goals. The elements of the preferred alternative are illustrated on Figure 3 and include:

- Construction of a fish ladder downstream of the driveway dam between 10020 and 10028 68th Ave S;
- Installation of a new 8-foot diameter by about 14-foot deep maintenance hole (MH) at the intersection of a 3 by 6-foot box culvert that crosses beneath Rainier Avenue South and a 36-inch diameter culvert that passes beneath the 10005 Rainier Ave S property. The MH will be a cast-in-place base "saddle" MH to allow MH construction without bypassing flows in the culvert; and,

- Replacement of an existing 42-inch diameter culvert with a new 4 by 6-foot corrugated steel culvert. The new culvert will extend from the north end of the existing 3 by 6-foot box culvert (north of the Rainier Avenue South right-of-way) and continue beneath a private driveway.

Early (30 percent) preliminary drawings showed the fish ladder consisting of a series of concrete weir walls. The design has evolved to include sheet piling weir walls for the upstream (southern) portion of the fish ladder and rock weirs for the downstream (northern) portion. Preliminary plan and profile drawings of the planned improvements are attached as Figures 4 and 5.

2.0 FIELD AND LABORATORY INVESTIGATIONS

SPU Materials Laboratory personnel conducted subsurface explorations at the site by drilling four soil borings (B-1 through B-4) on June 9, 2004. The borings were completed to a maximum depth of 25 feet below the existing ground surface. Approximate locations of the explorations are shown on Figure 1. The borings were sited in an attempt to provide a representative subsurface profile along the alignment of the site. Two borings (B-1 and B-2) were located within the area of the planned fish ladder. One boring (B-3) was located at the proposed MH on the south side of Rainier Avenue S, and the final boring (B-4) was located in the vicinity of the planned culvert replacement beneath the driveway just north of Rainier Avenue S. Boring B-4 was located as close to the culvert as possible, given the presence of underground and overhead utilities, and traffic considerations.

The borings were drilled by Geologic Drill using a trailer-mounted and portable drill rig with hollow stem auger drill tooling. A standpipe piezometer[1] was installed in B-2 and B-3 to facilitate measurement of groundwater level fluctuations. Appendix A describes the field exploration methodology in greater detail, and includes logs of the explorations completed for this study (Figures A-2 through A-5). A key to the terms and symbols used on the logs is presented as Figure A-1.

Geotechnical laboratory tests were conducted on selected soil samples to characterize certain physical properties of the on-site soils. Laboratory testing included determination of natural moisture content and grain size distribution. The moisture content test results are displayed on the summary boring logs in Appendix A. The grain size distributions test results are shown in Appendix B.

[1] The piezometers should be decommissioned in accordance with WAC 173-160-460(2) at the time of construction. The SPU Materials Laboratory should be contacted 10 working days in advance to provide the services of the Drilling Contractor.

3.0 SITE CONDITIONS

3.1 SURFACE CONDITIONS

The site spans across Rainier Avenue South in a residential neighborhood in South Seattle. It is located near the southwest shoreline of Lake Washington. The prevailing surface topography in the site vicinity slopes downward toward Lake Washington. Most of the site has been filled to create the current grades of Rainier Avenue S and the private property south of the roadway. As a result of the filling, the site is relatively flat except for the localized topographic relief described below.

The existing surface conditions are illustrated on Figures 1 and 3. The south portion of the site is a heavily vegetated, open drainage course of Taylor Creek where the fish ladder is proposed. The creek channel is bordered by a former residence (10020 68th Avenue S) to the east and a road (68th Avenue S) to the west. The creek channel is approximately 8 feet below the road grade. The creek enters the site from the south via a drop beneath a driveway and exits the open drainage course via a 36-inch culvert. The creek banks are mildly sloped except for an approximately 4-foot high cut bank just downstream of the drop.

Downstream (north) of the open drainage course, the 36-inch culvert continues beneath an existing apartment building (10005 Rainier Ave S) and transitions to a 3-foot tall by 6-foot wide box culvert at the south edge of the Rainier Avenue S right-of-way. Rainier Avenue S is a multi-lane roadway with asphalt overlay, concrete curbs and sidewalks. The northern portion of Rainier Avenue S is built on an approximately 9-foot-thick embankment. North of Rainier Avenue S, the embankment slopes steeply down for a vertical distance of about 9 feet to a private, asphalt-paved access road. The 3- by 6-foot culvert ends at the north right-of-way line and transitions to a 42-inch culvert, which passes beneath the private access road. The creek re-emerges from the end of the 42-inch culvert and into another open drainage course north of the access road. The open drainage course carries Taylor Creek flows through private property and into Lake Washington.

3.2 EARTHQUAKES AND SEISMICITY

The Puget Sound region is known to be seismically active. Large earthquakes have occurred several times in recent history, such as the 1949 Olympia (magnitude 7.2), 1965 Seattle (magnitude 6.5), and 2001 Nisqually (magnitude 6.8) Earthquakes. Furthermore, geologists are learning more about potential seismic sources and their corresponding hazards in the area, such as the Cascadia Subduction Zone and the Seattle Fault. Consequently, moderate to high levels of earthquake shaking should be anticipated during the design life of the project.

The United States Geological Survey has calculated and mapped ground shaking levels for the United States for events with various probabilities. For this site, the peak ground acceleration with a ten percent probability of exceedance in fifty years (a common design-level event for major public works structures) is 0.32 g (USGS, 2002). Although this seismic hazard is influenced by several earthquake sources and magnitudes, it is most strongly influenced by a magnitude 6.75 event occurring on the Seattle Fault. The Seattle Fault is not defined by a discreet fault, but rather by a zone of fault splays. The site is considered to be adjacent to the Seattle Fault zone.

For developments on private property, the Seattle Department of Planning and Development (DPD) requires that the design criteria include a 100-year return period seismic event (DPD Director's Rule 3-93, 1993). To satisfy the 100-year design criteria, the Director's Rule allows the use of magnitude 6.5 design level earthquake with a peak horizontal ground acceleration equal to 0.20g for alluvial soils, such as those found at this site. Therefore, our recommendations relating to seismic design are based on this criterion.

3.3 Subsurface Conditions

3.3.1 General Geology

To gain an understanding of subsurface conditions, we reviewed background information from previous studies completed in the area to complement information gained from our subsurface investigation. Refer to the Reference list at the end of this report. Review of a published geologic map (Waldron et al, 1962) indicates that the surficial deposits of the surrounding area are mainly recent alluvium. Beyond the immediate area of Taylor Creek, surface soils are mapped as Glacial Till. Our explorations are in general agreement with the mapped conditions. The recent alluvium was deposited by Taylor Creek under moderate to low energy conditions and can be expected to be an interbedded unit of sand, gravel and silt with organics. Because it was not overridden by glacial ice during the last glaciation, it tends to be relatively loose. Glacial Till, which is likely present beneath the recent alluvium, is very dense and competent, having been overridden by up to about 3,000 thousand feet of glacial ice.

Portions of the site are mapped as a Potential Liquefaction Area or Landslide Prone Area in the City of Seattle Environmentally Critical Areas Map Folios. Given the topography at the site, it is our opinion that there is a very low likelihood of the project being impacted by a landslide. Potential liquefaction hazards are discussed below in Section 4.3.

3.3.2 Site Specific Subsurface Conditions

In general, we encountered Fill and Alluvium during our explorations. These units are described below. Our interpretation of subsurface conditions along the project alignment is shown graphically on Figure 2.

Fill: Loose to medium dense, Fill was encountered to a depth of 13 feet in borings B-3 and B-4 beneath Rainier Avenue South. The Fill classifies as Gravel with Sand (GW) [2], Silty Sand with Gravel (SM) and Sandy Silt (ML) with varying percentages of organic debris. The strength and compressibility of the Fill soils is variable. They are generally moisture sensitive, meaning that they will degrade and become difficult to work when wet. Though not encountered in our explorations, older fill has been known to contain large debris such as timbers, logs, concrete slabs, and other unexpected materials.

Alluvium: Alluvium was encountered throughout the total depth of B-1 and B-2, and beneath the Fill soils in B-3 and B-4. The Alluvium encountered ranged in density from very loose to dense, and is interbedded with layers that classify as Peat (Pt), Sandy Silt (ML), Silty Sand (SM), Sand with Silt (SP-SM), Sand (SP) and Gravel with Sand (GW). The strength and compressibility of the Alluvium soils is variable. In general, the density of the alluvium increases with depth.

Groundwater: Groundwater was encountered at stream level in borings B-1 and B-2 and approximately 9 to 13 feet below the surface grade of Rainier Avenue South in borings B-3 and B-4, respectively. We anticipate that groundwater levels will fluctuate seasonally in response to rainfall, stream flows, and other factors.

4.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our investigations and analyses, it is our opinion that the proposed project is feasible from a geotechnical perspective, provided the recommendations of this report are properly incorporated in design and construction. Key geotechnical considerations for the project include:

- Excavation, shoring and foundations for the proposed new MH on the north side of Rainier Avenue S;
- Excavation, shoring and foundations for the proposed culvert extension on the south side of Rainier Avenue S; and,
- Sheet piling proposed for the south portion of the fish ladder.

These and other issues are discussed in the following sections.

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[2] Unified Soil Classification System (USCS), ASTM D2487 and D2488

4.1 Culvert and MH Structure

4.1.1 Excavations and Shoring

Excavation and shoring will be required for the proposed MH and culvert on the south and north side of Rainier Avenue S, respectively. Based on our characterization of subsurface conditions and our experience in similar geologic environments, we anticipate that the on-site soils can be excavated with conventional excavating equipment. Though not observed in our explorations, the Contractor should be prepared to encounter rubble, wood, and other debris, as these items are often found in older fill areas. In addition, the Contractor should implement a construction-dewatering plan due to the presence of groundwater at levels exceeding 3 feet above the final excavation elevations.

For planning purposes, we recommend assuming a groundwater table at approximately Elevation 26 (9 feet below surface) at the location of the proposed MH and Elevation 20 (6 feet below surface) at the proposed new culvert location. We understand the current plan calls for the MH being approximately 14 feet deep, which is about 5 feet below the groundwater table at that location. Consequently, the Contractor's construction-dewatering plan should enable MH installation without causing unnecessary disturbance to the foundation subgrade and maintain excavation stability. This may require an external dewatering system, such as a pumping well or well point outside of the excavation footprint. The Contractor is responsible for designing the dewatering system in accordance with Section 7-17.3(1)A3 of the City of Seattle *Standard Specifications* (City of Seattle, 2003). The design should be completed by a geologist or geotechnical engineer who is experienced in the area of construction dewatering.

Groundwater levels at the proposed new culvert are within a foot of the invert elevation, so dewatering for the culvert is expected to be straightforward. We expect that dewatering for the culvert will be handled adequately with the use of sumps and pumps within the excavation.

Protective systems will be required for the MH and culvert excavations for worker safety and to support adjacent utilities and pavements. They should be designed and implemented in accordance with Section 7-17.3(1)A7 of the *Standard Specifications*. Provided the MH excavation is properly dewatered, Trench Safety Systems (7-17.3(1)A7a) the should provide appropriate protection of nearby facilities during construction. However, if particularly sensitive facilities are nearby that are prone to even minor disturbance, Support Systems (7-17.3(1)A7b) should be considered.

4.1.2 Structure Foundations, Backfill and Compaction

The exposed subgrade soils at the locations of the MH and culvert are expected to be <u>unsuitable</u> for support of the planned structures, and will require improvement. The subgrade improvement measures described below should be completed during construction in the presence of a Materials Laboratory geotechnical engineer.

The MH will be installed without interruption of flows in the culvert. This will be accomplished by casting the MH base to the spring line of the culvert, setting the MH, then removing the top portion of the culvert.

The MH is not particularly sensitive to settlement. The designer has stated that up to 1-inch of total settlement is tolerable. Therefore, the subgrade for the base should be improved by overexcavating 24 beneath the planned MH base level and replacing the excavated soul with Pipe Bedding CDF in accordance with Section 9-01.5 of the *Standard Specifications*. The overexcavation and CDF backfill should extend at least 18 inches beyond the outside diameter of the MH.

The culvert foundation subgrade should be improved by overexcavating 6 inches below the planned bottom of the culvert, compacting the subgrade in-place, and placing a woven stabilization geotextile[3] on the exposed subgrade. The overexcavated area should then be backfilled with a compact layer of Type 2 Mineral aggregate. Type 2 consisting of recycled materials should <u>not</u> be allowed. The standard culvert bedding in accordance with Section 7-03 of the *Standard Specifications* should be placed above the Type 2 layer.

The onsite soils to be excavated contain a relatively high percentage of fines (material passing the U.S. Standard No. 200 sieve) making them sensitive to moisture and difficult to compact when wet. Therefore, depending on weather conditions at the time of construction, imported backfill material may be required. Imported backfill material should consist of material containing less than 5 percent fines (passing the No. 200 sieve), such as Type17 Mineral Aggregate as described in the Section 9-03.14 of the *Standard Specifications*. Controlled Density Fill (CDF) could also be used (Section 9-01.5).

Backfill should be placed and compacted as described in Section 7-17.3(3) of the *Standard Specifications*. During placement of the initial lifts, the backfill should not be dropped directly on pipes and should be placed uniformly around structures to avoid unbalanced lateral loads. Furthermore, heavy vibratory equipment should be used with care to avoid damage to structures, pipes, and adjacent utilities. Backfill should be placed in a dry excavation. Placement of fill into standing water should not be allowed.

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[3] The geotextile should meet the requirements of Standard Specification 9-05.22.

4.1.3 MH and Culvert Design Considerations

Assuming that the subgrade improvement recommendations in the previous section are followed, the MH can be designed assuming an allowable bearing pressure of 2,500 pounds per square foot (psf), and the culvert can be designed assuming an allowable bearing pressure of 4,000 psf. The recommended maximum allowable bearing pressure may be increased by 1/3 for short term transient conditions such as seismic loading. Assuming construction is accomplished as recommended herein, and for the loads anticipated, we estimate total and differential settlement of the MH and culvert will be less than ¼-inch and ¾-inch, respectively. It is anticipated that the majority of the estimated settlement will occur during construction, as loads are applied.

The MH structure should be designed for horizontal pressures from an at-rest equivalent fluid weight of 60 pounds per cubic foot (pcf). Other loads acting on the walls or on the retained soil near the walls should be properly incorporated in the design calculations. Traffic loads can be accounted for using an additional uniform horizontal surcharge of 110 psf. This assumes a uniform vertical traffic load of 250 psf on the surface. The structure should be designed to resist hydrostatic pressures for the portions that will be below the water table.

The culvert will have minimal cover and should be designed to support appropriate traffic and overburden loads.

4.1.4 Pavement Restoration

We assume that pavements will be restored to match the existing pavement section. Backfill materials and compaction of the subgrade below the pavements should be in accordance with the recommendations described in the Section 4.1.2. Removed pavement areas should be restored in accordance with the appropriate City of Seattle *Standard Plans for Municipal Construction* (2000) and Section 9 of SDOT's *Street and Sidewalk Pavement Opening and Restoration Rules* (1997), assuming a competent subgrade. Pavement restoration should be planned for the area of the excavation plus a distance of D/2 from the perimeter of the excavation. Exact limits of pavement restoration should be determined after backfilling of the excavation.

4.2 Fish Ladder

4.2.1 Sheet Piles

Sheet piling cells are planned for construction of the upstream (southern) three pools in the fish ladder. Sheet piling was selected to contain the upper pools, maintain their structural stability and to provide relative water-tightness in the portion of the ladder where more soil retention is required. Also, it is anticipated that dredging, if necessary, will occur from the southernmost pools, so the cells will need to retain adjacent soil under a variety of pool depths while supporting loads of maintenance vehicles needed to perform the dredging.

Based on our interpretation of the subsurface conditions, the installation of sheet piles as planned is feasible from a constructability perspective. It should be anticipated that sheet pile installation will result in noticeable disturbance to the site, since relatively large equipment is required. The Contractor should be required to submit a sheet pile-driving plan, detailing the equipment to be used, and access requirements. Any restrictions in terms of site disturbance should be communicated in the specification relating to the pile-driving plan. We recommend that a Materials Lab geotechnical engineer review the pile-driving plan and provide at least part-time monitoring of the pile installation.

Under the current plan as shown on Figure 4, the maximum exposed wall height to be retained by the sheet piling is 5 feet. Based on our interpretation of subsurface conditions in the area of the proposed sheet piling, we recommend utilizing the soil properties in Table 1 for design.

Table 1 – Soil Parameters for Sheet Pile Design

Elevation	**Total Unit Weight (pcf)**	**Buoyant Unit Weight (pcf)**	**Soil Angle of Internal Friction, ϕ (deg)**	**Soil/Sheet Pile Interface Friction Angle, δ (deg)**
Above El. 20	105	43	28	11
Below El. 20	125	63	32	15

Notes:

1)pcf = pounds per cubic foot; deg = degrees

2)Cohesion, c = 0 for all soils

3)Assume active pressures on the wall, based on the fact that the wall will yield slightly when loaded.

Additional sheet pile design considerations include:

- Assume the groundwater level is at ground surface behind the wall (retained side) and at the dredge line in front of the wall (pond side) for design;
- Apply a factor of safety of 1.5 to the passive pressure calculated from the soil parameters in Table 1;
- Neglect the passive resistance of the "wedge" of soil shown in front of the walls on Figures 4 and 5;

- Include relevant external loads in the design. Uniform loads on the ground surface should be multiplied by 0.35 and applied horizontally to the exposed portion of the wall. For example, a 250 psf traffic surcharge would be applied horizontally as a uniform 88 psf pressure acting over the 5-foot exposed portion of the wall;

- If seismic inertial forces are to be considered in the design, they should be applied as an additional uniform horizontal load of 6.5H psf, where H equals the exposed wall height. The seismic load applies only to the exposed portion of the wall. The recommended inertial force is based on a 100-year design level seismic event in accordance with DPD Director's Rule 3-93.

- External loads such as traffic loads do not need to be evaluated in conjunction with seismic loads since the likelihood of them occurring simultaneously is very low.

4.2.2 Rock Weirs

Rocks used in the weirs should be of sufficient size to resist anticipated hydraulic forces from the creek without excessive movement, and be of sufficient quality such that they do not degrade excessively over time. We recommend that rocks used in the weirs meet the requirements of *Standard Specification* 9-03.17.

4.2.3 Seepage

Past experience has shown that seepage around impermeable flow retention features can result in concentrated internal erosion of creek banks from piping. There is a potential for this to occur around the ends of the sheet piling on the open sides of the cells (west side). This potential can be reduced by enclosing the cells on the west side or possibly extending the piling further into the creek bank.

We understand that it is SPU's experience that seepage through the rock weirs does not present a problem when the successive pool elevation differences are minimal. In earlier SPU projects, we understand that seepage between rocks in the weir reduces with time as fine creek-bed material fills the spaces between the rocks. However, we anticipate that sedimentation may be reduced considerably with the installation of the sheet piling, reducing the potential for the rock weirs to become self-sealed. If this is the case, it may become necessary to import material to use in sealing the weirs.

4.3 Seismic Considerations

For projects of this type, SPU has rarely considered potential seismic impacts in past designs. However, permit requirements may dictate consideration of seismic impacts on the private property portions of the project. This section presents seismic considerations so the project team is informed of potential seismic impacts, and can consider them in the design if they choose to do so. In our opinion, potential seismic impacts are not critical to the success of the project.

For this project, the principle seismic hazards are liquefaction and seismically induced settlement. In our opinion, ground rupture and seismically induced landslides are not likely to impose significant impacts to the project. Ground motion response applies to the sheet pile walls and is addressed in Section 4.2.1.

Liquefaction occurs when a loose, saturated granular deposit is shaken and the pore water pressure within the soil increases. This increase in pore pressure causes a reduction in strength. Sand boils, flotation of underground structures, and settlement can also result from this phenomenon.

Based on the subsurface data obtained for the project, we performed liquefaction analyses for the upper 25 feet of soil. The analyses were based on a peak ground acceleration (PGA) of 0.20g and a magnitude 6.5 event and indicated a very low likelihood that liquefaction was possible in the looser zones that contained fewer fines. Based on our understanding of the geology in the site vicinity, it is unlikely that significant liquefaction will occur beneath a depth of 25 feet.

Based on our analyses, it is our opinion that sand boils or flotation of underground structures will not occur due to the level of seismic shaking considered. Seismic settlement can be expected to be on the order of less than ½-inch at the ground surface due to a design level earthquake (PGA = 0.2g, M = 6.5). This settlement is expected to be inconsequential to the performance of the project.

5.0 LIMITATIONS AND ADDITIONAL SERVICES

This report was prepared in accordance with generally accepted professional principles and practices in the field of geotechnical engineering at the time the report was prepared. The scope of our work did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous or toxic substances in the soil, surface water, or groundwater at this site.

This report should be provided in its entirety as a reference to prospective Contractors for bidding and estimating purposes; however, the conclusions and interpretations presented in this report should not be construed as a warranty of the subsurface conditions. Factual information, such as the that presented in the appendices, should be included in the Project Manual for the Contractor's information and their own interpretation.

This report is issued with the understanding that the information and recommendations contained herein are brought to the attention of the appropriate design team personnel and incorporated into the project plans and specifications, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

We recommend that the SPU Materials Laboratory be retained to review the plans and specifications and verify that our recommendations have been interpreted and implemented as intended. Sufficient geotechnical monitoring, testing, and consultation should be provided during construction to confirm that the conditions encountered are consistent with those indicated by explorations and to verify that the geotechnical aspects of construction comply with the contract plans and specifications. Recommendations for design changes will be provided should conditions revealed during construction differ from those anticipated.

We appreciate the opportunity to be of service.

Sincerely,
SPU MATERIALS LABORATORY

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EXPIRES 11/13/06

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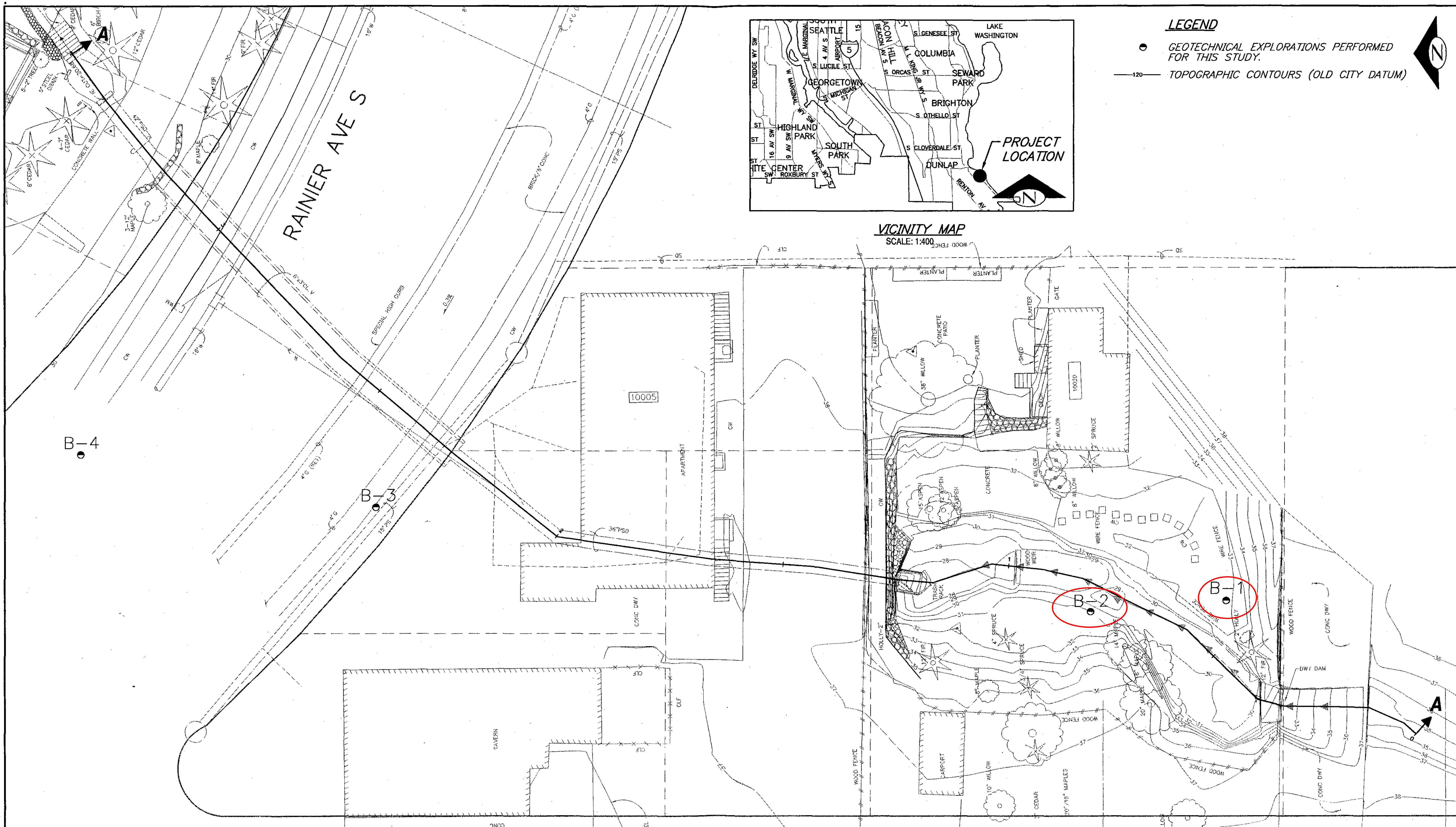

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TAYLOR CREEK CULVERTS PHASE II

SITE AND EXPLORATION MAP

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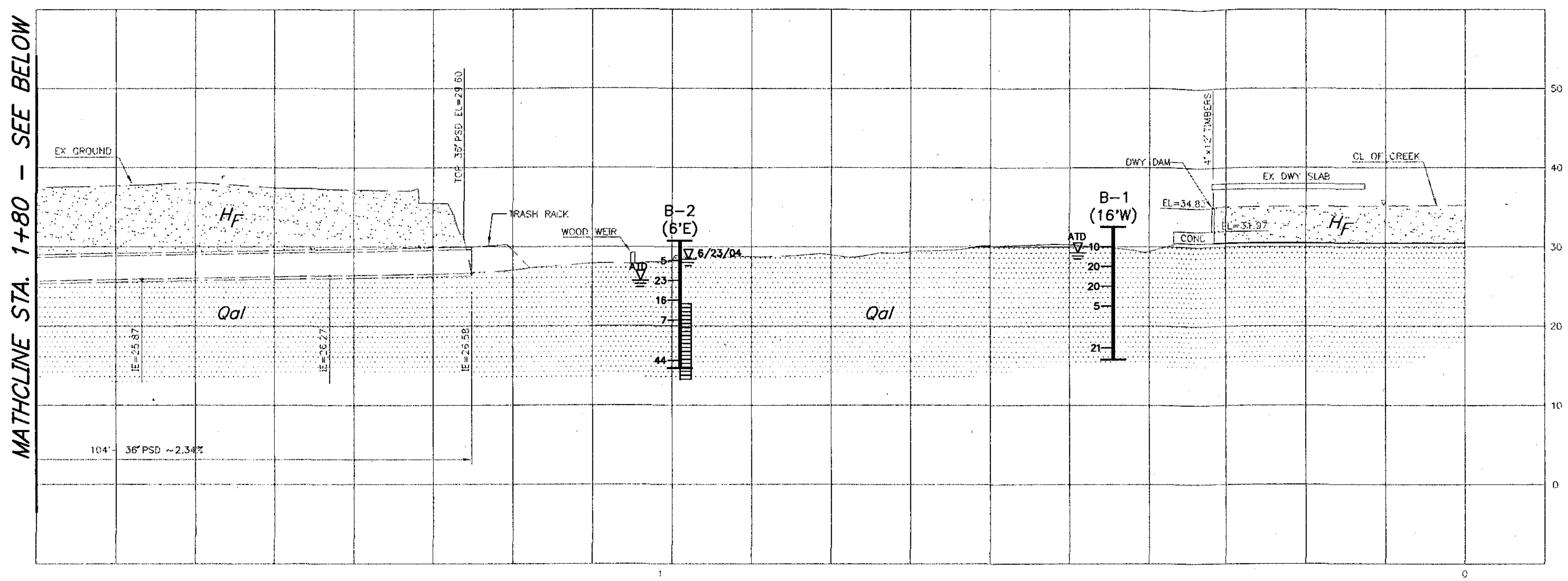
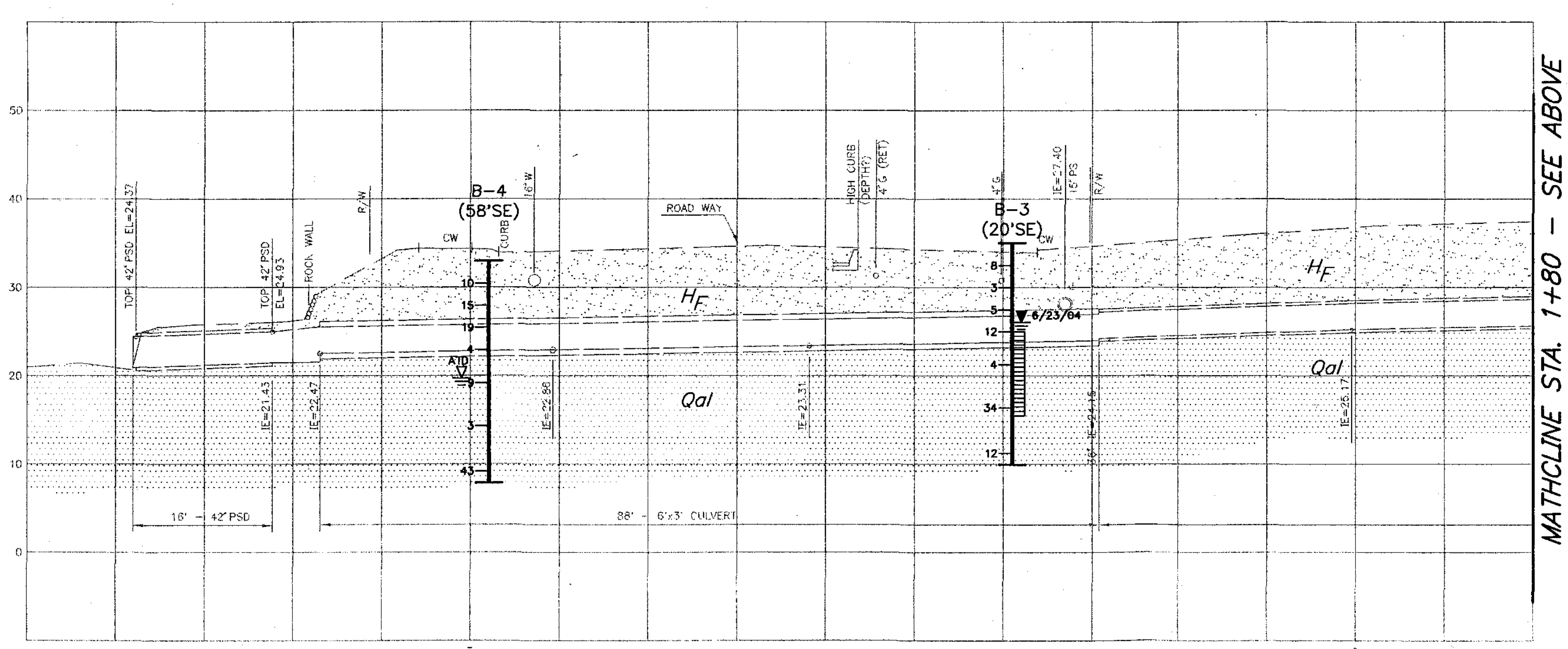
FIGURE 1



SUBSURFACE PROFILE
A–A'



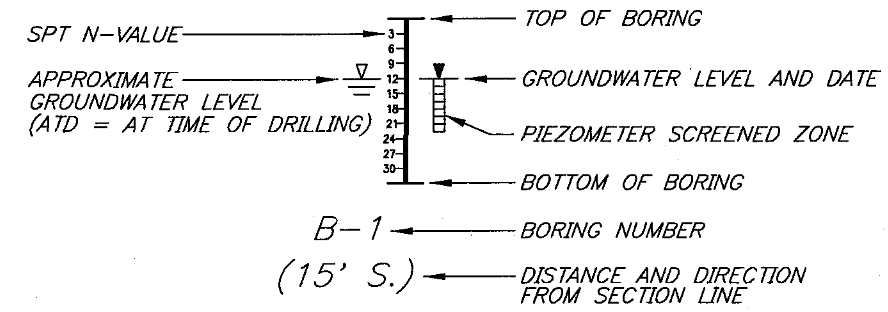
SUBSURFACE PROFILE
A–A' (CONT.)

GEOLOGIC UNIT DESCRIPTIONS

H_F = FILL

Qal = ALLUVIUM

LEGEND



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0 10 20
HORIZONTAL SCALE IN FEET

TAYLOR CREEK CULVERTS PHASE II

SUBSURFACE PROFILE A-A

C399315

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FIGURE
2

APPENDIX A

FIELD EXPLORATION PROGRAM

APPENDIX A

FIELD EXPLORATION PROGRAM

APPENDIX A

FIELD AND LABORATORY INVESTIGATIONS

Subsurface conditions were explored by advancing four soil borings (B-1 through B-4) on June 9, 2004. The borings were drilled to a maximum depth of 25 feet below the ground surface. The approximate locations of the explorations are illustrated on Figure 1 in the main body of the text. The explorations were located in the field by measuring relative to prominent existing features near the site. The approximate ground surface elevations at the explorations are based on the topography data shown on the preliminary plans. The locations and elevations of the explorations should be considered accurate only to the degree implied by the methods used.

The borings were advanced by Geologic Drill of Nine Mile Falls, Washington using a portable Acker Soil Mechanic drill rig for borings B-1 and B-2, and a trailer-mounted Deep Rock XL on borings B-3 and B-4. Hollow-stem auger drilling techniques were employed. The results of the explorations are summarized on the individual summary boring logs, which are included in this Appendix as Figures A-2 through A-5. A key to the symbols and terms used on the summary logs is presented as Figure A-1.

Soil samples were obtained from all borings at 2½ to 5-foot depth intervals using the Standard Penetration Test (SPT, ASTM D-1586). The 2.0-inch outside diameter (OD) SPT sampler was driven into the soil a distance of 18 inches using a 140-pound drive hammer falling a distance of 30 inches. The hammer was operated using a rope-and-cathead system. Recorded blows for each 6 inches of sampler penetration (blow counts) are shown on the summary logs in this appendix. The standard penetration resistance, SPT N-value, is the sum of the blow counts for the second and third 6-inch interval. The N-value provides a qualitative measure of the relative density of cohesionless soil, or the relative consistency of fine-grained soils. Representative portions of all recovered samples were placed in sealed containers and transported to our laboratory for further observation and testing.

A 2-inch diameter Schedule 40 PVC standpipe piezometer was installed in B-2 and B-3 to facilitate measurements of groundwater levels and their fluctuations. The piezometer also provides the capability to perform slug testing to aid in dewatering design, if needed. The piezometers were screened as shown graphically on the summary logs. A traffic-rated, flush-mount surface casting was installed at the surface. The piezometer should be properly decommissioned in accordance with WAC 296-155 before the conclusion of the project.

An SPU Materials Laboratory representative was present throughout the field exploration 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 Practice for Description and identification of Soils (Visual-Manual Procedure)*. The summary exploration logs represent our interpretation of the contents of the field logs and the results of laboratory testing. The stratigraphic contacts shown on the individual summary logs represent the approximate boundaries between soil types; actual transitions may be more gradual. The subsurface conditions depicted are only for the specific dates and locations reported, and therefore, are not necessarily representative of other locations and times.

UNIFIED SOIL CLASSIFICATION SYSTEM - ASTM D2488

MAJOR DIVISION			GROUP SYMBOL	LETTER SYMBOL	GROUP NAME
COARSE GRAINED SOILS CONTAINS MORE THAN 50% FINES	**GRAVEL AND GRAVELLY SOILS** MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVEL WITH ≤ 5% FINES		GW	Well-graded GRAVEL
				GP	Poorly graded GRAVEL
		GRAVEL WITH BETWEEN 5% AND 15% FINES		GW-GM	Well-graded GRAVEL with silt
				GW-GC	Well-graded GRAVEL with clay
				GP-GM	Poorly graded GRAVEL with silt
				GP-GC	Poorly graded GRAVEL with clay
		GRAVEL WITH ≥ 15% FINES		GM	Silty GRAVEL
				GC	Clayey GRAVEL
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	SAND WITH ≤ 5% FINES		SW	Well-graded SAND
				SP	Poorly graded SAND
		SAND WITH BETWEEN 5% AND 15% FINES		SW-SM	Well-graded SAND with silt
				SW-SC	Well-graded SAND with clay
				SP-SM	Poorly graded SAND with silt
				SP-SC	Poorly graded SAND with clay
		SAND WITH ≥ 15% FINES		SM	Silty SAND
				SC	Clayey SAND
FINE GRAINED SOILS CONTAINS MORE THAN 50% FINES	SILT AND CLAY	LIQUID LIMIT LESS THAN 50		ML	Inorganic SILT with low plasticity
				CL	Lean inorganic CLAY with low plasticity
				OL	Organic SILT with low plasticity
		LIQUID LIMIT GREATER THAN 50		MH	Elastic inorganic SILT with moderate to high plasticity
				CH	Fat inorganic CLAY with moderate to high plasticity
				OH	Organic SILT or CLAY with moderate to high plasticity
HIGHLY ORGANIC SOILS				PT	PEAT soils with high organic contents
TOPSOIL				TP	TOPSOIL

NOTES:

1) Sample descriptions are based on visual field and laboratory observations using classification methods of ASTM D2488. Where laboratory data are available, classifications are in accordance with ASTM D2487.

2) Solid lines between soil descriptions indicate change in interpreted geologic unit. Dashed lines indicate stratigraphic change within the unit.

3) Fines are material passing the U.S. Std. #200 Sieve.

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FIGURE 3
(Sheet 1 of 2)

KEY :

SAMPLING METHOD

	2-inch OD SPT Split Spoon Sample with 140-lb hammer falling 30 inches (ASTM D1586).
	No Recovery.
	Shelby Tube Sample (ASTM D1587).
	3-inch OD Split Spoon Sample (California Sampler) with 300-lb hammer falling 30-inches.
	Grab Sample.
	Non Standard (As noted on log).
	Core Run.

Note: Symbol Length Represents Sample Recovery

LABORATORY TEST

AL	Atterberg Limits
FC	Fines Content
GSD	Grain Size Distribution (Sieve and/or Hydrometer)
ENV	Environmental Testing
SG	Specific Gravity
MD	Moisture Density Relationship (Proctor Test)
C	Consolidation
UCS	Unconfined Compression Strength
Perm	Hydraulic Conductivity Test (As noted on Log)

COMPONENT DEFINITIONS

COMPONENT	SIZE RANGE
Boulders	Larger than 12 in
Cobbles	3 in to 12 in
Gravel	3 in to No. 4 (4.75 mm)
Coarse gravel	3 in to 3/4 in
Fine gravel	3/4 in to No. 4 (4.75 mm)
Sand	No. 4 (4.75 mm) to No. 200 (0.075 mm)
Coarse Sand	No. 4 (4.75 mm) to No. 10 (2.00 mm)
Medium Sand	No. 10 (2.00 mm) to No. 40 (0.425 mm)
Fine Sand	No. 40 (0.425 mm) to No. 200 (0.075 mm)
Silt and Clay	Smaller than No. 200 (0.075 mm)

COMPONENT PROPORTIONS

DESCRIPTIVE TERMS	RANGE OF PROPORTION
Trace	Less than 5%
Few	5 - 15%
Little	15 - 30%
Some	30 - 50%
Mostly	50 - 100%

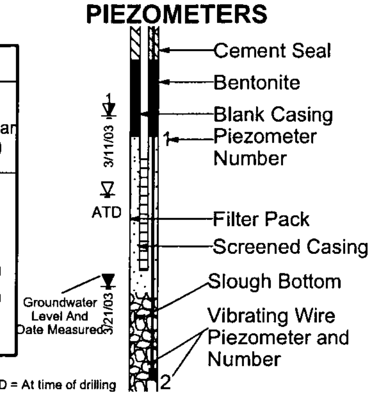
MOISTURE CONTENT

DRY	Absence of moisture, dusty, dry to the touch
MOIST	No visible water, near optimum moisture content.
WET	Visible free water,usually soil is below water table.

RELATIVE DENSITY OR CONSISTENCY VERSUS SPT N - VALUE

COHESIONLESS SOILS			COHESIVE SOILS		
Density	N (blows/ft)	Approximate Relative Density	Consistency	N (blows/ft)	Approximate Undrained Shear Strength (psf)
Very Loose	0 to 4	0 - 15	Very Soft	0 to 2	< 250
Loose	4 to 10	15 - 35	Soft	2 to 4	250 - 500
Medium Dense	10 to 30	35 - 65	Medium Stiff	4 to 8	500 - 1000
Dense	30 to 50	65 - 85	Stiff	8 to 15	1000 - 2000
Very Dense	over 50	85 - 100	Very Stiff	15 to 30	2000 - 4000
			Hard	over 30	> 4000

PIEZOMETERS



ATD = At time of drilling

SOIL STRATIFICATION AND STRUCTURE

STRATA	DESCRIPTION
Parting	Less than 1/16 inch thick
Seam	1/16 to 1/2 inch thick
Layer	1/2 to 12 inch thick
Scattered	Less than 1 occurrence per foot
Numerous	More than 1 occurrence per foot

STRUCTURE	DESCRIPTION
Stratified	Alternating layers of varying material or color with layers at least 1/4 inch thick;note thickness
Laminated	Alternating layers of varying material or color with layers less than 1/4 inch thick;note thickness
Fissured	Breaks along definite planes of fracture with little resistance to fracturing
Slickensided	Fracture planes appear polished or glossy, sometimes striated
Blocky	Cohesive soil that can be broken down into small angular lumps which resist further breakdown
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay; note thickness
Homogenous	Same color throughout

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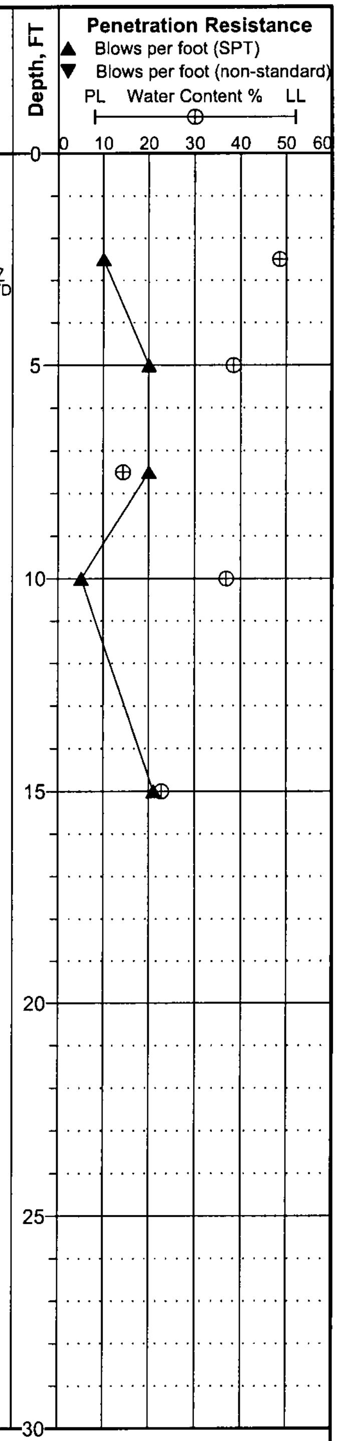
FIGURE 3
(Sheet 2 of 2)

SOIL DESCRIPTION

Surface Elevation: 33 Site Datum

Depth, FT	Soil Description	USCS	Samples	Blows/6"	Lab tests	Water level
0	Surface is topsoil, blackberries.					
	ALLUVIUM Stiff, brown, SANDY PEAT, little silt; moist to wet; tip is wet, numerous wood fragments.	PT	1	1,4,6		ATD
5	Medium dense, gray, SAND WITH SILT AND GRAVEL; wet; numerous organics, pockets of sand, wood fragments.	SP-SM	2	5,8,12		
	Becomes scattered pockets of silt, scattered organics.		3	8,7,13	GSD	
10	Medium stiff, gray, SANDY SILT, trace gravel; wet; numerous organics, pockets of sand, wood fragments.	ML	4	2,2,3		
15	Medium dense, gray, SAND WITH GRAVEL, trace silt; wet; fractured gravel in sampler.	SP	5	12,11,10		
	Bottom of boring at 16.5 feet. Hole filled with bentonite chips.					



Date Completed: June 9, 2004
Driller: Geologic Drill
Equipment: Acker Soil Mechanic
Drilling Method: 2.25-in. ID HSA
Hammer System: Rope & Cathead

Approximate Location: 78 feet east of centerline of 68th Avenue South & 260 feet south of centerline of Rainer Avenue South. (N: 1290795, E: 189498)

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LOG OF BORING B-1

C399315 **FIGURE A-2**

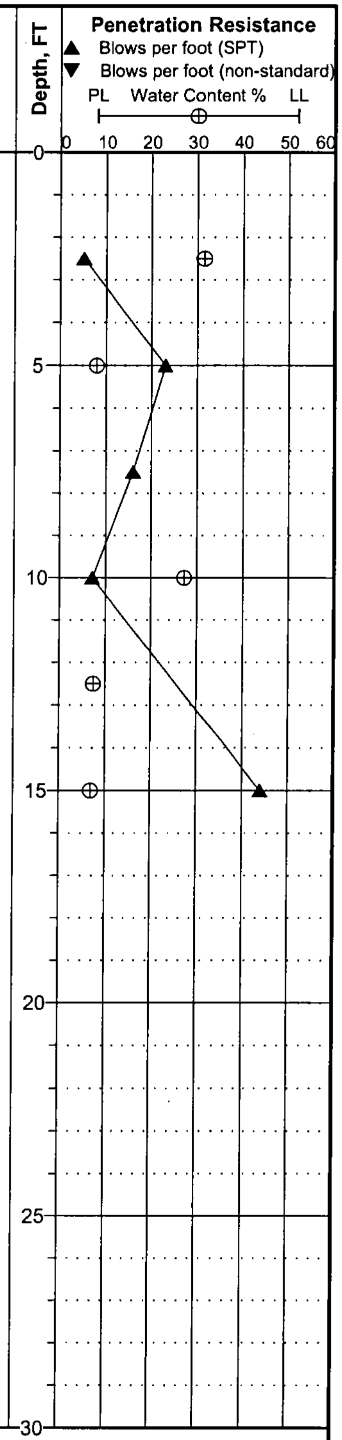
Logged by: TMS Reviewed by: HHH Sheet 1 of 1

SOIL DESCRIPTION

Surface Elevation: 31 Site Datum

Depth, FT	Soil Description	USCS	Samples	Blows/6"	Lab tests
0–4	**ALLUVIUM** Medium stiff, dark brown, SANDY PEAT, few silt, trace gravel; moist to wet; numerous roots, scattered pockets of black organics.	PT	1	1,2,3	
4–9.5	Medium dense, gray, GRAVEL WITH SAND, trace silt; wet; gravel is rounded.	GW	2	10,11,12	GSD
	Bottom 6 inches is gray silt, middle 4 inches is wood fragments.		3	6,9,7	
9.5–16	Loose, gray, SAND WITH GRAVEL, trace silt; wet; scattered organics, scattered pockets of gray silt.	SP	4	8,5,2	
	Becomes dense, gravel is rounded.		5	26,24,20	GSD
16	Bottom of boring at 16 feet. Driller overdrilled to 17 feet for piezometer install.				

Ground Water: 6/23/04; ATD



Date Completed: June 9, 2004
Driller: Geologic Drill
Equipment: Acker Soil Mechanic
Drilling Method: 2.25-in. ID HSA
Hammer System: Rope & Cathead

Approximate Location: 75 feet east of centerline of 68th Avenue South & 220 feet south of centerline of Rainer Avenue South. (N: 1290808, E: 189533)

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LOG OF BORING B-2

C399315 FIGURE A-3

Logged by: TMS Reviewed by: HHH Sheet 1 of 1

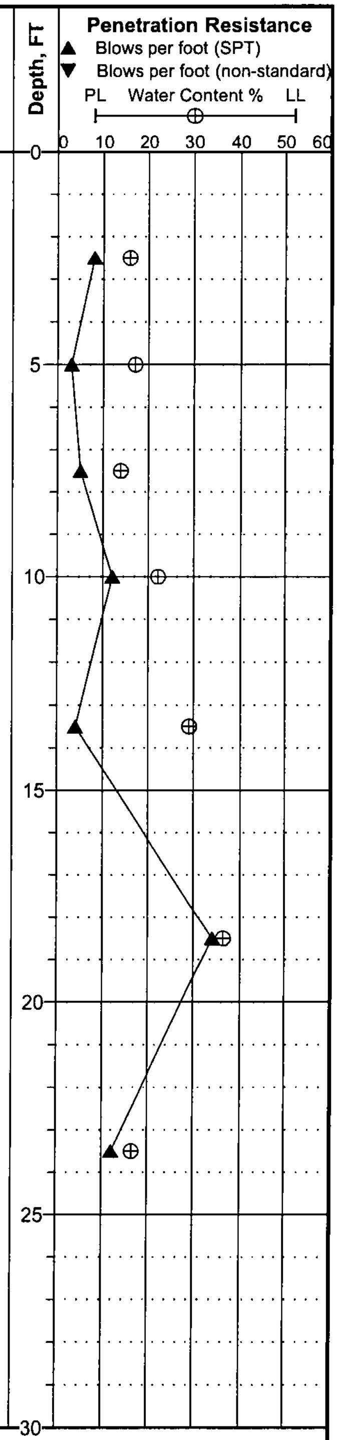
LOG OF BORING TAYLOR CREEK.GPJ SEATTLE PUML.GDT 11/5/04

SOIL DESCRIPTION

Surface Elevation: 35 Site Datum

Depth, FT	Soil Description	USCS	Sample	Blows/6"	Lab tests
0	2-inch asphalt overlay, 3-inch brick over 8-inch concrete.				
	FILL Medium stiff, brown, SANDY SILT WITH GRAVEL; moist; scattered rust staining and pockets of silt.	ML	1	3,4,4	
5	Very loose, gray-brown, SILTY SAND WITH GRAVEL; moist; scattered rust staining and pockets of silt.	SM	2	2,1,2	
	Becomes loose, moist to wet, numerous rust staining, scattered pockets of gray sandy silt and sand.		3	2,2,3	
10	Becomes medium dense, brown, wet, numerous wood fragments, scattered fibers and pockets of gray silt.		4	3,5,7	FC
	ALLUVIUM Very loose to loose, brown, SILTY SAND WITH GRAVEL; wet; numerous wood fragments up to 2 inches, numerous pockets of sand with gravel.	SM	5	8,2,2	GSD
15	Becomes medium dense, large wood fragment, fractured gravel in sampler, scattered pockets of organics.		6	13,26,8	GSD
20			7	10,6,6	
25	Bottom of boring at 24 feet. Piezometer installed.				
30					

Ground Water: ATD; 6/23/04



Date Completed: June 9, 2004
Driller: Geologic Drill
Equipment: Trailer-Mounted Deep Rock XL
Drilling Method: 3.25-in. ID HSA
Hammer System: Rope & Cathead

Approximate Location: 96 feet east of centerline of 68th Avenue South & 50 feet south of centerline of Rainer Avenue South. (N: 1290855, E: 189666)

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LOG OF BORING B-3

C399315

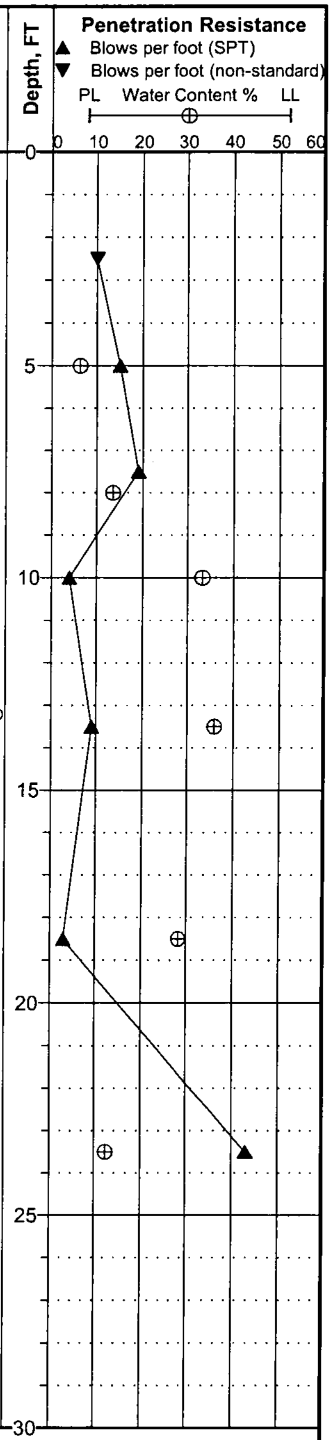
FIGURE A-4

Logged by: PS/TMS   Reviewed by: HH   Sheet 1 of 1

LOG OF BORING TAYLOR CREEK.GPJ SEATTLE PUML.GDT 11/5/04

SOIL DESCRIPTION	Depth, FT	USCS	Samples	Blows/6"	Lab tests
Surface Elevation: 33 Site Datum	0				
FILL Dense, light brown, GRAVEL WITH SAND, trace silt; dry.		GW			
Medium dense, light brown, SILTY SAND WITH GRAVEL; dry to moist; scattered organics.		SM		10,6,4	
	5		1	4,8,7	
Numerous organics, scattered rust staining and pockets of gray sandy silt.			2	8,10,9	
Soft, dark brown, SANDY SILT, trace fine gravel; moist; burned debris and organics.	10	ML	3	2,2,2	
ALLUVIUM Loose, dark gray, SILTY SAND WITH GRAVEL; wet.		SM	4	6,5,4	GSD
	15				
Becomes very loose, gray-brown SILTY SAND, few fine gravel, scattered organics/organic seams.			5	1,1,2	GSD
	20				
Dense, brown, GRAVEL WITH SAND, trace silt; wet.		GP	6	16,20,23	
Bottom of boring at 25 feet. Boring backfilled with mixture of cuttings and bentonite.	25				
	30				

ATD



Date Completed: June 9, 2004
Driller: Geologic Drill
Equipment: Trailer-Mounted Deep Rock XL
Drilling Method: 3.25-in. ID HSA
Hammer System: Rope & Cathead

Approximate Location: 112 feet east of centerline of 68th Avenue South & 40 feet north of centerline of Rainer Avenue South. (N: 1290887, E: 189725)

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LOG OF BORING B-4

C399315 **FIGURE A-5**

Logged by: PS Reviewed by: HHH Sheet 1 of 1

LOG OF BORING TAYLOR CREEK.GPJ SEATTLE PUML.GDT 11/5/04

APPENDIX B

LABORATORY TESTING PROGRAM

APPENDIX B

LABORATORY TESTING PROGRAM

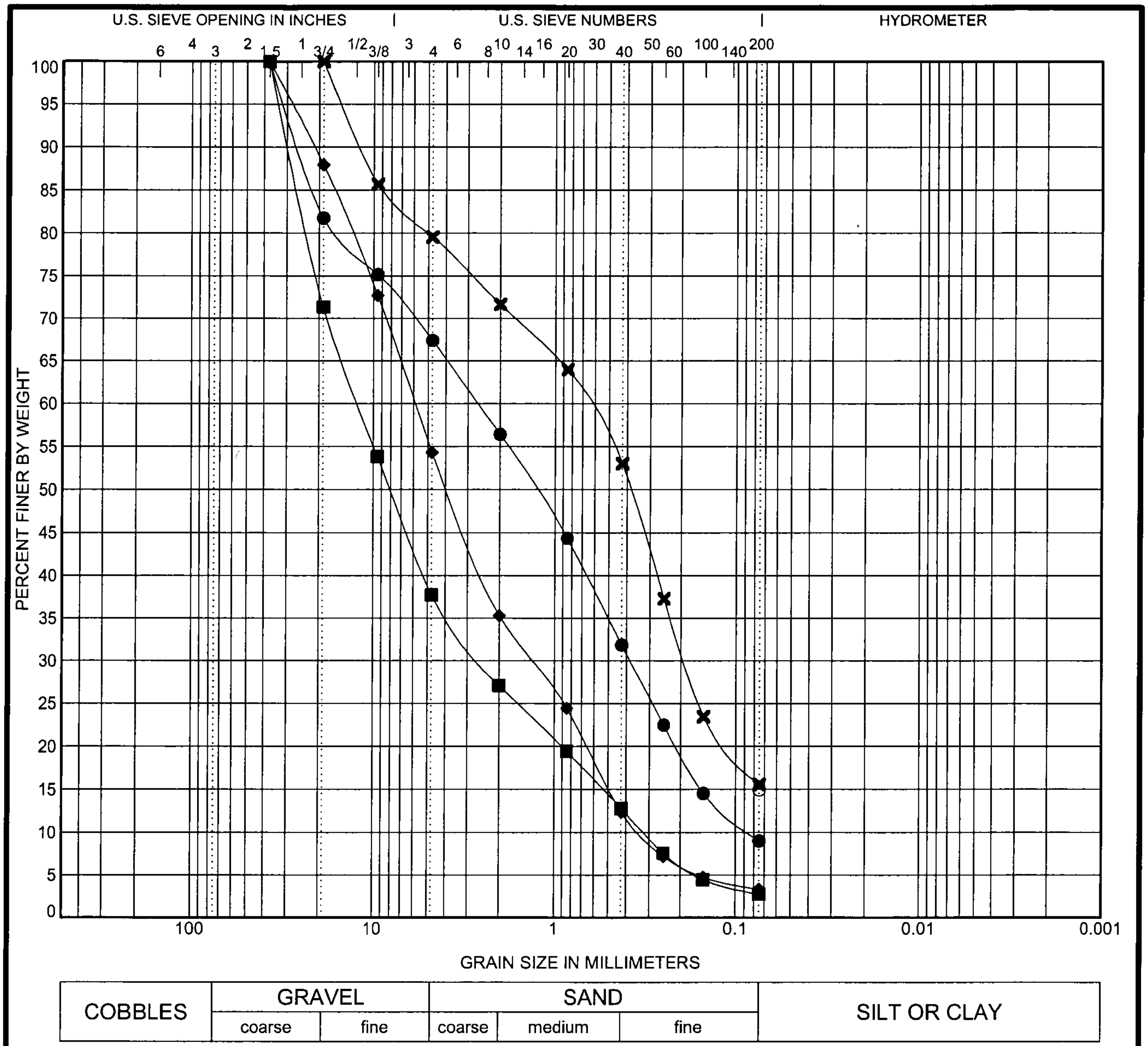
SPU Materials Laboratory representatives performed laboratory tests on selected soil samples collected during our field investigation. The laboratory tests were conducted in general accordance with appropriate ASTM test methods. The test procedures and test results are discussed below.

Natural Water Content

Natural water content determinations were made on selected soil samples in general accordance with ASTM D2216, *Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass*. Test results are graphically indicated at the appropriate sample depth on the summary logs in Appendix A.

Grain Size Distribution

The grain size distribution of selected samples was analyzed in general accordance with ASTM D422, *Standard Test Method for Particle-Size Analysis of Soils*. Results of grain size analyses are plotted on Figure B-1 of this Appendix. The soil samples tested for grain size distribution are indicated on the summary logs.



	Sample ID	Depth	Classification	LL	PL	PI	Cc	Cu
●	B-1; 3	7.5	Gray, SAND WITH SILT AND GRAVEL (SP-SM)				0.7	31.2
■	B-2; 2	5.0	Gray, GRAVEL WITH SAND, trace silt (GW)				1.7	37.9
◆	B-2; 5	15.0	Gray, SAND WITH GRAVEL, trace silt (SP)				0.9	17.7
○	B-3; 4	10.0	Brown, SLTY SAND WITH GRAVEL (SM)					
✖	B-3; 5	13.5	Brown, SILTY SAND WITH GRAVEL (SM)					

	Sample ID	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-1; 3	7.5	37.5	2.65	0.38	0.085	32.6	58.4	9.0	
■	B-2; 2	5.0	37.5	12.14	2.53	0.32	62.3	35.0	2.8	
◆	B-2; 5	15.0	37.5	5.88	1.32	0.333	45.7	51.0	3.3	
○	B-3; 4	10.0	0.075						15.0	
✖	B-3; 5	13.5	19	0.66	0.19		20.4	64.0	15.6	

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GRAIN SIZE DISTRIBUTION (ASTM D422)

Taylor Creek Phase 2
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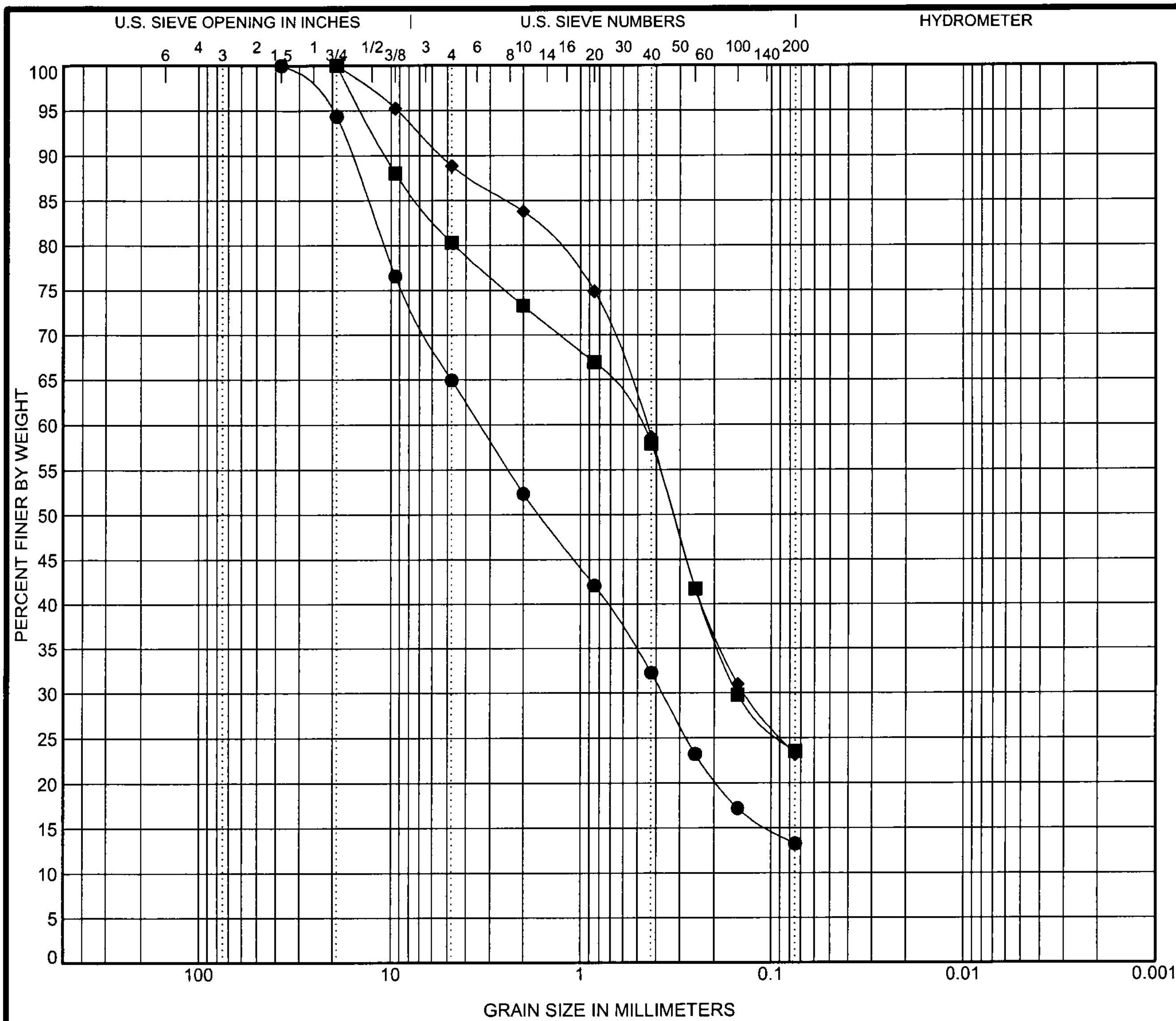
WA No.: C399315 November 2004 **FIGURE: B-1**

US GRAIN SIZE TAYLOR CREEK.GPJ US EVAL.GDT 11/5/04



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample ID	Depth	Classification	LL	PL	PI	Cc	Cu
●	B-3; 6	18.5	Brown, SILTY SAND WITH GRAVEL (SM)					
■	B-4; 4	13.5	Dark gray, SILTY SAND WITH GRAVEL (SM)					
◆	B-4; 5	18.5	Dark gray-brown, SILTY SAND, few fine gravel (SM)					

	Sample ID	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-3; 6	18.5	37.5	3.38	0.37		35.0	51.7	13.3	
■	B-4; 4	13.5	19	0.5	0.15		19.7	56.8	23.5	
◆	B-4; 5	18.5	19	0.45	0.14		11.1	65.7	23.2	

US GRAIN SIZE TAYLOR CREEK.GPJ US EVAL.GDT 11/5/04

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GRAIN SIZE DISTRIBUTION (ASTM D422)

Taylor Creek Phase 2
Seattle, Washington

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November 2004

FIGURE: B-2