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Geotechnical and Hydrogeologic Study
Port Angeles Graving Dock
Port Angeles, Washington

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GEOTECHNICAL AND HYDROGEOLOGIC STUDY
PORT ANGELES GRAVING DOCK
PORT ANGELES, WASHINGTON

INTRODUCTION

This report presents the results of our geotechnical and hydrogeologic study to provide design, construction, and dewatering recommendations for the proposed graving dock project located in Port Angeles, Washington. This report supersedes our November 13, 2002, memorandum "Summary of Field Pumping Test and Dewatering Design" and the preliminary geotechnical engineering recommendations provided in a draft report dated November 18, 2002.

The first several pages of our report summarize the purpose and scope our work. The main body of the report presents our geotechnical design and construction dewatering recommendations. This is followed by appendices, which present current and previous explorations, laboratory test results, and the groundwater pumping tests data.

Our report includes the following:

- Project Understanding;
- Executive Summary;
- Purpose and Scope of Work;
- Site Geology;
- Subsurface Conditions;
- Seismic Conditions;
- Engineering Analysis and Recommendations; and
- Appendices that present:
  - Current geotechnical field exploration logs (Appendix A);
  - Laboratory test procedures and results (Appendix B);
  - Current and previous environmental borings (Appendix C); and
  - Data and analysis for the groundwater pumping test (Appendix D).

PROJECT UNDERSTANDING

The proposed Port Angeles Graving Dock site is located along the Port Angeles waterfront, just south of Ediz Spit. A Vicinity Map is provided on Figure 1. It is designed to serve as a graving dock for construction of the eastern half of the SR 104 Hood Canal Floating Bridge concrete pontoons. The graving dock will also

be used in the future for construction of SR 520 floating bridge replacement pontoons.

We understand that the current preferred alternative of the proposed graving dock has a plan dimension of 460 by 905 feet. A 170-foot-wide deep channel will be constructed along the eastern part of the graving dock with a top of slab at elevation –15 feet. An upper slab at elevation 10 feet, where most of the pontoons will be constructed, will form the remainder of the graving dock. A wall will be constructed along the entire perimeter of the dock with the top of wall at elevation 28 feet. After constructing the pontoons, they will be floated out of the graving dock into the Port Angeles Harbor by flooding the graving dock with water up to elevation 27 feet. It is our understanding that the construction of the pontoons will last about 5 to 6 months, and the flooding operation to float the pontoons will only last a few days.

EXECUTIVE SUMMARY

The following summarizes the principal conclusions and recommendations contained within this report. Refer to subsequent sections of the report for further discussion of each point, as well as for other recommendations.

Explorations and Testing

Our field program included eight geotechnical borings, designated H-1-02 through H-8-02. Monitoring wells were installed in three of the eight geotechnical borings. A pumping test was performed in a large-diameter dewatering well (PW-02). An array of observation wells (OW-1 through OW-3) and monitoring wells were used to provide site-specific data for the drawdown response to pumping of groundwater. Exploration locations are shown on Figure 2.

Subsurface Conditions

There are five major soil units underlying the project site. From the ground surface downward, these units are:

- Soil Unit 1 - SAND/GRAVEL (upper aquifer);
- Soil Unit 2 - Silty SAND (upper aquitard);
- Soil Unit 3 - Sandy SILT (upper aquitard);
- Soil Unit 4 - Sand to Gravelly SAND (lower aquifer); and
- Soil Unit 5 - Glacially Overridden Soils (lower aquitard).

The depth to groundwater varies from 6.3 to 8.9 feet, which corresponds to elevation 4.8 to 5.7 feet. Figure 3 shows an elevation contour map of top of bearing layer (Soil Unit 5). Figures 4 through 8 illustrate a series of generalized subsurface cross sections.

Seismic Conditions

Two levels of earthquake have been considered in this study, including:

- An earthquake with a 10 percent probability of exceedance in a 50-year period (475-year return period) and a corresponding Peak Ground Acceleration (PGA) of 0.28g.

- An earthquake with a 10 percent probability of exceedance in a 20-year period (190-year return period), with a corresponding PGA of 0.14g.

Widespread liquefaction may occur in the silty Sandy and the sandy Silt soils (Soil Units 2 and 3) under the 475-year earthquake. Under the 190-year earthquake, only local and relatively isolated pockets of soils may liquefy.

Dewatering

Dewatering of the proposed layout and construction scenario for the graving dock can be accomplished to achieve the main objectives for safe construction in dry conditions. However, some of the on-site soils will not yield much water to gravity drainage and may benefit from more proactive dewatering systems (such as vacuum wellpoints or eductor systems) that are more appropriate than dewatering wells for reducing pore pressures in lower permeability soils. A conceptual dewatering schematic is illustrated on Figure 9.

The pumping test demonstrated the presence of a hydrologically significant lower sand horizon (Soil Unit 4) at depth, which, though relatively thin, appeared to exist across most of the site. The presence of this relatively permeable layer constitutes a potential source of uplift pressures beneath the base of the deep excavation. Depressurization of this layer as part of the dewatering approach may provide improved drainage of the poorly draining soils that exist above and below final grade in the deep channel section of the proposed excavation.

Retaining Structures

Feasible retaining wall alternatives include sheet piles, secant soldier pile walls, or a hybrid sheet pile/solider pile wall system. Ground freezing is likely not a long-term solution based on cost comparisons.

Deep Channel Wall

The walls surrounding the deep channel will be subjected to five different lateral loading cases.

(1) Construction Phase;
(2) Operational Phase;
(3) Super Flood Phase;
(4) Earthquake; and
(5) Post-Earthquake (Liquefaction).

Given the variation in soil conditions and the size of the deep channel, we have identified three segments designated as C-1 through C-3 (refer to Figure 10) along the perimeter of the channel, with each segment representing different soil conditions. Earth pressure parameters for wall segments C-1 through C-3 are presented in Tables 2 through 4, respectively. For illustrative purpose, the subsurface profile along segment C-1 and the corresponding earth pressure diagrams are presented on Figures 11 and 12. An important assumption associated with these earth pressure diagrams is that the lower aquifer will be permanently depressurized inside and beneath the deep channel, as discussed subsequently.

General assumptions and seismic-related loadings for each of the five loading cases should be used in conjunction with the soils parameters presented in Tables 2 through 4.

Upper Slab Wall

The super flood phase will likely govern the design of the upper slab wall. Two segments (U-1 and U-2) were identified as shown on Figure 10, each corresponding to different subsurface conditions. Table 5 and 6 presents earth pressure parameters for Segments U-1 and U-2, respectively. During the super flood case, the water pressure is assumed to start at elevation 27 feet and extends to the bottom of the sheet pile. This assumption implies that the slab will have joints or cracks such that the underlying soils are hydraulically connected to the water in the graving dock.

Seepage Cutoff

The deep channel walls should extend into the glacially overridden soils, which will provide adequate seepage cut off. For the upper slab walls, we recommend that the wall be embedded in Soil Unit 2 with the tip of the wall elevation at –17 feet.

Deadman and Tieback Recommendations

The sheet piles surrounding the deep channel will be supported with either deadman (concrete or sheet pile) or tieback anchors. The recommended static and seismic earth pressures for concreter deadman are shown on Figure 14. For the sheet pile deadman, the earth pressures parameters presented in Tables 2 through 4 should be used in the design.

Two criteria govern the deadman location. The first criterion is that the deadman should be located behind a line that extends from the point of zero moment on the sheet pile wall to the ground surface with an inclination that is equal to the soil friction angle measured from the horizontal. The second criterion is that the passive wedge acting on the deadman does not overlap with the active wedge acting on the sheet pile wall of the deep channel.

The main disadvantage of a tieback system is soil liquefaction. It is expected that tieback bond length would be located within the silty Sands (Soil Unit 2) and the sandy Silts (Soil Unit 3) where wide-spread liquefaction may occur under the 475-year earthquake, leading to a significant loss in tieback capacities. In this case, the tieback alternative may not be feasible, unless the bond length can be extended into the deeper no-liquefiable soils (Soil Unit 5).

Excavation Stability of Walls

Both the deep channel and the upper slab walls will have adequate factors of safety against global instability provided embedment recommendations presented above are implemented in the design.

Graving Dock Slabs

Deep Channel Slab

The super flood phase would likely be the critical case for slab settlements. Our analyses suggest the settlements would vary from about 1 inch toward the southern edge of the channel to 2 to 4 inches toward the northern edge of the channel. Settlements in the vicinity of the northern edge of the channel may cause downdrag on some piles supporting the gate.

Assuming the water pressure in Soil Unit 4 may equal the hydrostatic pressure outside the dock during the operational phase, our analyses suggest factors of safety of 0.55 and 0.9 against uplift along the southern and the northern edges of the channel, respectively. The vertical stress required to achieve a factor of safety of 1.25 against uplift varies linearly from 1,000 psf along the southern

edge to 500 psf along the northern edge of the deep channel. A pressure relief system alone or combined with a structural system can be used to reduce uplift impacts. The structural system may consist of piles, micropiles, or tiedowns embedded into the glacially overridden soils to resist uplift forces. Regardless of the use a structural system, a pressure relief system will be required to avoid reduction of passive earth pressures acting on the sheet pile wall.

Upper Slab

Settlement would vary across the upper slab from about 1 inch toward the southeast corner to 6 inches toward the northwest corner. We understand that these settlements are tolerable, and therefore, preloading is not required. These settlements would occur due to the compression of the soils located mainly below the tip (elevation –17 feet) of the sheet pile for the upper slab. Therefore, downdrag forces are not anticipated on the sheet piles for the upper slab. For the deep channel sheet piles, settlement-induced downdrag would be insignificant given the much higher axial capacity of the sheet piles.

The estimated settlement due to liquefaction would range from 6 to 10 inches under the 475-year earthquake. Under the 190-year earthquake, local zones of liquefiable zones may induce 2 or 3 inches of settlement.

Pile Foundation for Gate Structure

The preliminary design calls for 24-inch-diameter open-end structural steel pipe piles with a 3/4-inch pile wall thickness. We understand that vertical piles will be used to resist both the axial and lateral loads.

Axial Load Capacity

In addition to the uplift forces from the superstructure, the piles should be designed to resist uplift forces due to the water pressure within Soil Unit 4. The additional uplift stress will be about 1,000 and 1,300 psf on the base of the pile cap at elevation –22 and –25 feet, respectively.

The recommended <u>ultimate skin friction</u> values (in compression and tension) are as follows:

- Between elevation –25 and –43 feet, use 200 psf for tension, ignore resistance for compression to account for settlement-induced downdrag on piles due to static and/or liquefaction;

- Between elevation -43 and -53 feet, use 2,000 psf; and

- Below elevation -53 feet, use 3,000 psf.

Use ultimate end bearing (in compression) of 200 kips (assuming inside-fit cutting shoe and no plug)

Use FS = 2.0 for both tension and compression with pile capacity verified by pile driving analyzer (PDA) and CAPWAP analysis on at least 2 to 5 percent of the production piles. Group effects for axial loads will not be significant provided that piles are spaced at least 2½D.

Laterally Load Capacity

The p-y curve soil parameters for single pile under static and post-earthquake cases are provided in Table 7. Group reduction factors for lateral analysis should be incorporated into the analysis, in accordance to the procedure outlined in the *WSDOT Bridge Design Manual*, as revised July 2000.

Walls Attached to the Gate Foundation

Two concrete walls along the east and west sides of the gate will be attached to its foundation. The top of the wall long the east will be at elevations 14 feet with a total height of about 30 feet. Along the west side, the top of the wall will at elevation 21 feet with a total height of about 37 feet. The recommended earth pressures are presented in Table 9.

PURPOSE AND SCOPE OF WORK

The purpose of our work is to provide the Washington State Department of Transportation (WSDOT) and their design consultants with geotechnical engineering and hydrogeologic recommendations related to the design and construction of the proposed graving dock.

Scope of Work

Our scope of work for this project included:

- Evaluate the groundwater regime and estimate the amount of water that can be expected in the graving dock excavation;

- Install a test well and observation wells and perform a groundwater pumping test;
- Develop conceptual construction methods that would allow for bottom slab construction in the dry or wet construction with seals;
- Present recommendations for a dewatering system for construction and operation of the graving dock;
- Assess seepage rates including water quantity and quality that can be expected in excavations if a dewatering system is used and not used, during both construction and operation;
- Assess treatment that may be required for discharge from excavations, well points, graving dock valves, and pumping systems, during both construction and operation;
- Drill seven geotechnical borings and collect representative soil sample for laboratory index testing;
- Present recommendations for feasible wall types for the graving dock walls;
- Develop earth pressures and water pressures for wall design;
- Present tieback or deadmen recommendations, if needed;
- Develop recommendations for wall stability and constructability;
- Define seismic design parameters and seismic design recommendations;
- Present foundation recommendations for the graving dock floor;
- Develop recommendations for seepage forces and uplift forces acting on the walls and floor;
- Evaluate excavation stability based on wet construction and dry construction, bottom heave, and side wall stability;
- Evaluate deep foundation alternatives, such as piles, shafts, and micro piles to carry uplift loads and compressive loads applied to the graving dock;
- Develop special provisions and plan sheets to support geotechnical recommendations;

- Review the contract documents and seal all geotechnical plans; and
- Prepare interim memoranda and final geotechnical/hydrogeologic report.

The scope of work also includes environmental drilling, sampling, and testing. Results of environmental investigation are presented in a report under separate cover dated October 25, 2002. Additional test results on soil samples obtained from the latest round of geotechnical explorations are presented in the environmental report addendum dated December 6, 2002.

SUBSURFACE EXPLORATIONS AND TESTING

Our field program consisted of eight geotechnical borings (designated H-1-02 through H-8-02), three observation wells (designated OW-1 through OW-3), and a large diameter test well (designated PW-02). Boring H-8-02 was drilled as a result of a survey error and was terminated at a relatively shallow depth. The locations of the explorations are shown on Figure 2. The geotechnical borings were drilled to depths ranging from 44 to 90 feet using hollow-stem augers. Monitoring wells were installed in three (H-5-02 through H7-02) of the geotechnical borings. Observation wells OW-1 and OW-3 were drilled to a depth of 20 feet. Observation well OW-2 was drilled to a depth of 35 feet. The logs of the geotechnical borings, the test well, and the observation wells are presented in Appendix A.

The laboratory testing consisted of water content determination, grain size analyses, hydrometer tests, and Atterberg limits. The results of the water content determinations and the Atterberg limits are presented on the boring logs. The test procedures and results of the Atterberg limits and the grain size analyses are presented in Appendix B.

As part of our scope of work, Hart Crowser previously performed an environmental investigation for the project site. Five environmental borings were drilled for the environmental investigation (refer to Figure 2). Groundwater monitoring wells were also installed in these borings. Physical laboratory testing was performed on samples obtained in one of the current environmental boring (HC-NW-02), and consisted of water content determination and grain size analyses. Additionally, Hart Crowser drilled 10 environmental borings for a previous study in 1988. Locations of the 1988 borings, which are located in the vicinity of the subject dock, are shown on Figure 2. The current and previous environmental borings are presented in Appendix C.

Given the depth of proposed construction below the water table, the program also included an investigation into the feasibility of dewatering the site, and determination of the recommended methods for achieving dewatering objectives. A pumping test was performed in a large-diameter dewatering well installed at the site, and an array of observation wells was used to provide site-specific data for the drawdown response to pumping of groundwater. Results and analysis of the pumping test data and observed drawdown responses are presented in Appendix D. This information is used to develop a proposed dewatering concept for the project and to evaluate the performance of potential alternative methods. The current graving dock layout and construction concept have been taken into account in developing the current dewatering system design.

SITE GEOLOGY

Based on our understanding of the site from current and previous explorations, the site is situated on filled ground that forms a relatively flat lowland area adjacent to Port Angeles Harbor. The property was used for most of the 20th century as a sawmill and lumber processing yard. A 150- to 200-foot-high bluff, located just south of the property, forms the boundary of the uplands to the south. The site area was part of the intertidal zone prior to being filled.

In general, the site stratigraphy from the current ground surface downward includes recent fills consisting of dredged silts, sands, and gravel as well as a localized area of wood chips. The dredged fills overlie natural beach deposits consisting of interbedded silts, sands, and gravel, which in turn overlie glacially overridden and consolidated sediments at depth. Materials encountered in our explorations were near-surface fill material including wood debris over intertidal and beach deposits of sand and gravel with interbeds of cleaner sand as well as silt layers. These intertidal and beach deposits generally contain shell fragments but are difficult to distinguish from the overlying dredged fill, which also contains shell fragments in places.

The bluff and the soils underlying the original beach deposits consist of an interlayered and very dense sequence of glacially derived sediments ranging from relatively permeable sands and gravels to hard silts of very low permeability.

Much of the fill was placed over the original beach deposits prior to the 1920s. The fill is reported to be dredge material, and at the site, consists of sandy gravel and gravelly sand. Based upon the explorations at the site, the upper portions of the fill (generally above 5 feet in depth) consist of a loose to medium dense

mixture of sand with silt and gravel, and containing varying amounts of bark and wood debris, coarse gravel, and angular riprap used as ballast on the dirt log haul roads.

SUBSURFACE CONDITIONS

Soil Conditions

Based on current and previous explorations, there are five major subsurface units underlying the subject site. A contour map of top of bearing elevations and a series of generalized subsurface profiles are illustrated on Figures 3 though 8. The following sections describe each of the major five soil units as encountered in current and previous borings.

Soil Unit 1 - SAND/GRAVEL (Upper Aquifer)

This unit underlies a thin surficial fill, and extends to depths varying from 6 to 25 feet, with an average depth of about 18 feet (elevation –5 feet). For simplicity, the overlying fill layer and this unit are presented as one soil unit on the generalized cross sections. The relative density of most of the soils in this unit varies from medium dense to dense, occasionally very dense. The inferred high relative density, based on blow counts in the borings, may be attributed to the presence of gravel and may not be representative of actual soil density.

Soil Unit 2 - Silty SAND (Upper Aquitard)

This unit was encountered underlying Soil Unit 1 and consists of loose to medium dense, occasionally dense, silty Sand. Within the footprint of the graving dock, the thickness of this unit ranges from 10 to 35 feet. Based on results of laboratory grain size analyses, the fines content in this unit ranges from 24 to 40 percent. The unit is entirely submerged under the groundwater table.

Soil Unit 3 - Sandy SILT (Upper Aquitard)

This unit was only encountered in borings drilled along the northern side of the graving dock (refer to Cross Section A-A'). In current and previous environmental borings, this unit was not differentiated from the overlying silty Sand. The boundaries between Soil Units 2 and 3 at the locations of the environmental borings, however, were interpreted based on the distinct difference in SPT blow counts between the two soil units. The unit consists of very soft to soft, sandy Silt, with thickness varying from 20 to 30 feet.

Based on two combined grain size (grain size and hydrometer) tests, the sand content is 25 to 28 percent; the silt content is 60 to 65 percent; and the clay content is about 10 to 12 percent. However, this soil unit appears to be of very low plasticity or non-plastic.

In boring H-1-02, the blow counts of this layer were mostly zero (refer to Cross Section A-A'). Although we believe these low blow counts were not caused by heave or negative water head in the bore hole, it may warrant additional cone penetration testing for verification of this anomalous ground condition.

Soil Unit 4 - SAND to Gravelly SAND (Lower Aquifer)

This unit appears to be thin, generally about 2 to 5 feet in thickness. This soil unit was NOT noted in many of the earlier borings, particularly in the environmental borings. This can be attributed in part to its limited thickness. The results of the groundwater pumping test indicate that this unit is relatively permeable, suggesting that it may be relatively clean of fines, in spite of the "silty" description in some of the boring logs.

Soil Unit 5 - Glacial Overridden Soils (Lower Aquitard)

This unit mainly consists of hard, sandy Silt with interbedded layers of very dense, silty Sand. Occasionally, this unit contains layers of very dense, gravelly Sand. This unit is a suitable bearing layer for the sheet pile around the deep channel and for the deep foundations supporting the graving dock gate. Figure 3 presents a contour map showing the elevation of the top of the bearing layer. As shown on the map, the elevation of top of this unit dips toward the north, with about 25 feet difference in elevation across the site. Throughout this report this unit is referred to as the "bearing layer " or "lower aquifer," depending on the context of our recommendations.

Groundwater Conditions and Tidal Fluctuations

Soils below the water table are saturated and under hydrostatic conditions controlled primarily by the average tide level and supplemented by surface recharge due to the infiltration of rainfall on land. These conditions lead to groundwater levels that are slightly higher than average water levels in Port Angeles Harbor.

Upper Aquifer

Soil Unit 1 forms an upper aquifer above an average depth of 18 feet (elevation -5 feet) consisting of moderately permeable silty sand and gravel. This water-

bearing zone contains the water table that is slightly above mean sea level in the adjacent Port Angeles Harbor. The predominant groundwater flows are expected to occur in this upper aquifer, which may have a direct source of recharge at or near the shoreline.

Prior to the start of the pumping test (see Appendix D) on October 30, 2002, the water table in the upper aquifer was measured in monitoring wells at elevation 5.65 ±0.2 feet. The water table at the test location appears to be unaffected by tidal fluctuations in Port Angeles Harbor. Based on "static" water level data observed in the various wells, the water table in the upper aquifer is essentially flat, with a very slightly sloping surface toward the harbor giving an estimated hydraulic gradient of 1 in 250 (0.004 ft/ft; H:V).

Measurements of tidal fluctuation in the upper aquifer (unaffected by pumping) were made in monitoring well HC-NW-02 during the pumping test (October 29-30, 2002). Although this well is the closest to the sea wall, it showed little if any discernible tidal influence. It should be noted that the effect of tidal fluctuations dissipates rapidly with increasing distance inland from the shoreline, especially in unconfined aquifers, where the larger storativity (drainable porosity) provides a greater damping effect than in a confined aquifer (see below). Also, construction of the adjacent seawall may limit the hydraulic connection between the shallow groundwater in the upper aquifer and the water in Port Angeles Harbor.

In either event, the dewatering design includes the assumption that the hydraulic connection is fully effective at the shoreline. This constitutes a factor of safety for the recharge effect provided by the presence of water in Port Angeles Harbor, proximally adjacent to the proposed excavation. The assumed presence of this recharge source does not produce large amounts of recharge flow that would preclude dewatering efforts, but it ensures that conservative conditions are considered in the design of the dewatering system.

Upper Sand/Silt Aquitard

The fine sand and silt horizons of Soil Units 2 and 3, below an average depth of 18 feet (elevation -5 feet), forms a lower permeability aquitard in which groundwater flow is relatively limited. Laboratory grain size and hydrometer tests (see Appendix B) show this aquitard to contain a substantial proportion of silt, such that its hydraulic conductivity is likely lower than was indicated from earlier soil descriptions. The hydraulic conductivity of fine sand and silt layer estimated on the basis of grain size analysis (Appendix C) is 1×10^{-5} cm/sec, but depositional layering could introduce anisotropy in the vertical plane, such that the vertical permeability may be somewhat lower.

Despite the relatively low hydraulic conductivity, saturated conditions and hydrostatic pore pressures within these loose/soft silty sandy soils represent a potential issue for the stability and workability of the deep channel excavation subgrade at elevation –20 feet, and will need to be adequately dewatered to provide stable working conditions in the excavations of the deep channel and gate structure. Gravity drainage of the silt/sand aquitard will be slow; it is likely to take between 2 to 4 weeks to obtain sufficient drainage of these soils that will increase their strength to permit excavation in the dry. Reduction of pore pressures within these soil units is also required outside of the deep channel excavation to reduce lateral loads on the sheet pile wall proposed for ground support and final construction of the channel.

The installation of dewatering systems in advance of excavation work, coupled with the time constraints that limit the speed of excavation, usually provide sufficient time for drainage to occur before potentially saturated soils are encountered at depth.

Lower Sand Aquifer

The water level data and pumping test response from monitoring well H-5-02 revealed the presence of a thin deep aquifer layer beneath the silt/sand aquitard. The data also demonstrate the lateral continuity between this aquifer and the body of water in Port Angeles Harbor, although its presence was not recognized in some of the previous site explorations.

The proposed sheet pile wall forming lateral support for the deep channel section for the project is expected to fully penetrate this aquifer and provide a cutoff from lateral sources of recharge. However, if the lateral connection persists through construction, excess water pressure (i.e., a pressure head of +6 feet) in this layer at depth below the base of the channel excavation (subgrade elevation -20 feet) may cause potential problems of uplift and heave. For prudent design analysis, it must be assumed that the pressure head at depth is not dissipated and that it acts on the base of a relatively impermeable plug of soil between the subgrade and the top of the aquifer. If the hydrostatic pressure on the base of the soil plug exceeds the total weight of the soil plug, an uplift condition may exist. Therefore, it will be necessary to depressurize the lower aquifer to reduce excess water pressures and increase the factor of safety against uplift and heave.

Lower Aquitard

No water level data were collected from Soil Unit 5, which forms a deep aquitard composed of hard silt that appears to be glacially compacted. From a

hydrogeologic perspective, this layer is considered to form a low-permeability boundary beneath the entire site. Sheet piles driven into this layer should form an effective hydraulic cutoff for lateral flow in all overlying soil units. However, some allowance should be made for possible pile separation or deviation that could create "windows" for groundwater leakage through the sheet piles.

SEISMIC CONDITIONS

Site Seismicity

The graving dock site is located along the northern coastline (along Strait of Juan de Fuca) of the Olympic Peninsula of Washington State. Major earthquakes in the region are generally associated with shallow-subduction plate tectonic activity off the west coast. Research within the last few decades indicates that earthquakes with magnitudes of up to 9.0 are capable of occurring in the coastal areas. No shallow subduction earthquakes have been recorded within the region in the recent historical period. The most recent large (at least magnitude 8 or more) event appears to have occurred in 1700, based on ethnographic data.

In addition to the large subduction earthquakes, several active and potentially active faults are present within the Strait of Juan de Fuca region (Mosher and Johnson 2000). Seismic events on these faults are generally attributed to the intraplate seismicity within the Juan de Fuca plate. They are similar in nature to the historical Puget Lowland earthquakes, including the April 13, 1949, Olympia earthquake (Richter magnitude 7.0); the April 29, 1965, Seattle earthquake (Richter magnitude 6.5); and the February 28, 2001, Nisqually earthquake (Richter magnitude 6.8).

The seismic hazards from both of these sources are included in the probabilistic USGS seismic hazard mapping project that forms the basis of the seismic acceleration map in the WSDOT Bridge Design Manual.

Seismic Design Considerations

For seismic design, an acceleration coefficient of 0.28 g is interpolated from the current acceleration map in the WSDOT Bridge Design Manual. The recommended acceleration coefficient is based on expected ground motion at the project site that has a 10 percent probability of exceedance in a 50-year period (475-year return period).

Design response spectra presented in AASHTO (1996 and 1998) are considered appropriate for seismic design of the structures on this project. A Type III soil profile response spectrum is recommended for seismic design. An associated Site Coefficient of 1.5 is recommended.

We understand there may be interest in designing the graving dock for a life expectancy shorter than that used for highway bridges. For comparative purposes, we developed the seismic acceleration at the site that would be consistent with a 20-year life span for the graving dock. Applying a 10 percent probability of exceedance in 20-year period (190-year return period), the acceleration coefficient was determined to be 0.14 g, or about one half that of the 10 percent probability of exceedance in a 50-year period value that is used for bridges with life expectancies on the order of 75 to 100 years. The lower acceleration value would result in lower Monobe-Okobe seismic earth pressures and less potential for liquefaction, which may govern the design of the graving dock walls.

Liquefaction Potential

Liquefaction of saturated sands occurs when the sands are subject to cyclic loading. The cyclic loading causes the water pressure to increase in the sand, reducing the intergranular stresses. As the intergranular stresses are reduced, the shearing resistance of the sand decreases. If pore pressures develop to the point where the effective stresses acting between the grains become zero, the soil will behave like a viscous fluid. Under this condition soil flow is possible. The effect of liquefaction can range from reduced shear strength to viscous fluid behavior.

The liquefaction potential of saturated sands is evaluated mainly on soil gradation, relative density, and the depth of the deposit. The potential for liquefaction is highest for saturated loose, fine- to medium-grained sands and silty sands above a depth of about 40 feet. The potential for liquefaction at the Port Angeles Graving Dock site was assessed using the procedure originally developed by Seed and Idriss (1982) and later updated by Youd and Idriss (2001).

Based on the available subsurface information, we conclude that liquefaction could develop in the upper two soil units under an earthquake with a 475-year return period. In Soil Unit 1, liquefaction would likely be local and relatively isolated. For the silty Sand in Soil Unit 2, a more wide-spread liquefaction may occur, particularly in the vicinity of borings H-5-02 and H-6-02, leading to significant loss of soil strength and ground settlement.

For the sandy Silt (Soil Unit 3), assessing the potential for liquefaction is quite complex. Based on our visual and laboratory classification, this soil has very low plasticity or is non-plastic. Assuming this would behave like sands, the Seed and Idriss procedure indicates that wide-spread liquefaction is likely to occur under the 475-year earthquake but unlikely to occur under the 190-year earthquake.

Lateral Spreading

In the case of liquefaction, lateral spreading may occur on sites adjacent to bodies of water (similar to the subject site) or on gently sloping sites. Using the equation presented in Youd et al. (2002), our engineering analyses indicate that negligible (less than 0.2 inch) lateral deformations may occur under the 475-year earthquake due to the vicinity of the site to the water. This magnitude of deformation should not significantly impact the proposed graving dock. The subject site is relatively flat and, therefore, lateral spreading is not anticipated due to the effect of sloping.

ENGINEERING ANALYSES AND RECOMMENDATIONS

This section describes the engineering analyses conducted to support the design recommendations for construction dewatering and for the geotechnical aspects of retaining structures, graving dock slabs, and pile foundations.

Dewatering Considerations

Construction dewatering and the control of groundwater levels or pore water pressures to ensure safe construction and operation of the proposed graving dock facility present a number of significant challenges due to the ground conditions and high water table described above. A number of dewatering methods have been considered in the context of the project to achieve the desired degree of dewatering. The relative merits and drawbacks of each method are described below. A recommended approach to dewatering for each stage of excavation, construction, and operation is then described.

Recommendations are based mainly on the results of a single pumping test, which helped to reveal the complexity of the hydrogeologic conditions of the site, and take into consideration drilling conditions in the loose fine-grained soils that could limit the efficiency of installed dewatering systems. Given the potential difficulties that could be encountered, these recommendations should be considered as a starting point rather than a definitive design, with steps taken to measure the effectiveness of each part of the recommended system as it is installed and operated. Modifications can then be implemented as necessary

based on drawdown responses observed in piezometers that measure water pressures at key locations and depths both inside and outside the deep channel section of the graving dock.

Dewatering Methods

Since groundwater exclusion will be achieved at the graving dock mainly by using sheet pile walls, this description concentrates on suitable methods for consideration at the Port Angeles site that will work together with the sheet pile walls.

Dewatering Wells. The first choice for dewatering this type of facility is typically the installation of dewatering wells drilled to between 1½ and 2 times the depth of the proposed excavation (i.e., at least 60 feet deep). Dewatering wells with individual submersible pumps are best suited to ground conditions exhibiting moderate to high permeability, such as Soil Unit 1. Provided that wells are designed to be efficient and are adequately screened and developed, multiple wells typically spaced 50 to 150 feet apart can effectively pump groundwater from permeable soils to create reasonably large cones of depression that coalesce together, lowering the water level over relatively broad areas. However, to be effective in this application, relatively permeable deposits need to be present above and below foundation grade.

The limited depth of Soil Unit 1 and the presence of less permeable materials (Soil Units 2 and 3) above the deeper foundation grade of elevation -20 feet at the Port Angeles site limits the use of this approach. Dewatering wells would be effective in lowering water levels within the upper aquifer to a depth of no more than 15 to 18 feet (i.e., to lower the water table by approximately 10 feet). The effectiveness of the wells will be adversely impacted by the interface between Soil Units 1 and 2 that have lower permeability. The contrast in soil permeability prevents the drawdown cone from dropping below this interface, even if the wells are significantly deeper. Greater lowering of the groundwater levels within the silt/sand aquitard by using dewatering wells will thus be prevented, and a dewatering system consisting only of conventional dewatering wells will not be effective.

Vacuum Wellpoints. Wellpoints are small-diameter wells that are drilled or jetted to depths of between 25 and 30 feet but on a much closer spacing than dewatering wells (typically between 5 and 10 feet). Wellpoints are connected to a common suction manifold to which a vacuum is applied using a vacuum-assisted centrifugal wellpoint pumpset. Performance of the system is limited by the constraint of maximum suction lift in the manifold, which is around 25 feet. Water in the wellpoints cannot be lowered beyond this depth. However,

additional stages of wellpoint installations with headers and wellpoints installed at lower elevations within the excavation can achieve greater depths of dewatering.

Wellpoints are suited to draining fine sands and silts such as Soil Units 2 and 3, where the application of a vacuum to the soil mass can assist the dewatering action, improving the strength of loose or soft materials by lowering pore water pressures and increasing effective stresses. This approach is expected to be workable inside the deep channel excavation; however, the depth limitation imposed by limited suction lift would restrict the performance of wellpoints installed outside the sheet piles. Wellpoints installed with headers at the ground surface (elevation 13 feet) could dewater soils down to an elevation no lower than -12 feet.

Eductor Systems. Eductor systems overcome the depth constraint that limits the application of vacuum wellpoint systems, and allow the dewatering of lower-permeability soils that do not yield water readily via gravity drainage to wells or wellpoints. Eductor systems feature modified wellpoints that include a jet-pump orifice at the tip of each wellpoint, and require a second manifold system that provides pressurize water to each wellpoint. The venturi action of the jet-pump nozzle provides significant suction and pumping action at the tip, which draws water into the eductor wellpoint.

At the Port Angeles site, eductor systems offer the most effective solution for dewatering the fine sand and silt of Soil Units 2 and 3. To achieve dewatering of these units to the depths required, eductors will need to be installed at least 50 to 60 feet deep outside the sheet pile walls of the deep channel section.

Sump Pumping. Sumps are essentially shallow wells that are excavated typically a few feet below the water table to extract excess water locally to aid excavation work. They usually perform best for shallow excavations below the water table in moderate to high permeability soils, but can induce significant loss of fines if used with coarse filter materials in fine-grained or silty sediments.

The use of sumps or shallow wells in parts of the Port Angeles site may be appropriate for lowering groundwater levels in the shallow upper gravelly sand aquifer due to the limited depth of this formation. Sumps may be needed to remove groundwater that is "perched" at the interface between Soil Units 1 and 2. Sumps may also be effective in controlling locally wet areas, such as sand or gravel pockets; however, they will not be effective for dewatering the fine sand and silt of Soil Units 2 and 3.

Dewatering System Design Recommendations

This section provides a description of the dewatering issues and recommended approaches considered to facilitate excavation and construction of the graving dock, or to improve safety conditions for specific stages of the work. As part of the dewatering approach, piezometers should be installed at key locations to provide direct indication of the drawdowns being achieved, and to allow the effectiveness of system components to be assessed as dewatering progresses.

The primary dewatering objective for the revised layout is to lower the water table beneath the deep channel section of the future graving dock to permit excavation and base-slab construction in dry conditions. Under the revised structural design concepts prepared by KPFF, this objective will be achieved in concert with the installation of sheet pile walls around the perimeter of the deeper excavation. External dewatering of the soils outside these sheet piles is required to reduce hydrostatic pressure on the sheet pile wall.

Dewatering Outside of the Deep Sheet Pile Walls

Design of the sheet pile wall requires a reduction in groundwater levels outside the excavation to reduce lateral hydrostatic loads, until sufficient horizontal reaction can be obtained from the casting of the lower graving dock floor slab and transfer struts. This dewatering objective requires lowering of groundwater levels in all soils around the outside of the sheet pile structure that will form the deeper walls of the channel section in the eastern portion of the graving dock.

The ground conditions at the Port Angeles site present a significant challenge to successful dewatering because the relatively permeable Soil Unit 1 overlies the saturated but much less permeable soils of Soil Units 2 and 3, both of which must be dewatered. Dewatering outside of the deep sheet pile walls cannot be achieved to a sufficient depth using vacuum wellpoints. Wellpoints will be limited to around 10 feet of drawdown (below the static water table) due to the change in soil conditions that occurs at a depth of around 18 feet. The resulting partial reduction of hydrostatic loads on the outside of the sheet pile walls would not be sufficient in that additional bracing for the sheet pile walls would be required during the excavation phase.

The recommended approach for lowering external water levels within the finer soils of Soil Units 2 and 3 is a system of eductor wells. Eductor wells should initially be installed on 10-foot centers, set between the dead-man tiebacks and installed from the ground surface after the tieback trench has been backfilled. Eductor wells should be drilled to depths of between 50 and 60 feet (i.e., to

elevations between -37 and -47 feet), screened from a depth of 50 to 20 feet, and connected to injection and vacuum headers laid on the ground surface.

Eductors may need to be installed within pre-drilled holes (at least 8-inch-diameter) and completed with 4-inch-diameter PVC casings and screens so that sufficient filter material can be placed around the casing to provide a vertical conduit for water from the upper aquifer to percolate down to the wellscreen. Drillholes may need to have temporary casing installed during construction to keep the borehole annulus free from fines and prevent clogging of the filter material by the loose fine sands and slits. Water inflows from the aquitard are likely to be insufficient to ensure effective development of the filter material.

The eductor system is anticipated to extract a total flow of up to 20 gpm from the silt/sand aquitard and should be effective in lowering external water levels below the required elevation of -20 feet, provided that the system is not "swamped" by lateral recharge and inflow to the area of dewatering via the more permeable materials of Soil Unit 1. Lateral flow in the upper aquifer is expected to be around 100 gpm for full drawdown and this should be assumed as the design flow for the eductor system, although it will be reduced by the presence of the deadman sheet pile wall. However, it may not be possible to capture all shallow groundwater flow in the upper aquifer using the deeper eductor wells alone. A few feet of saturation may potentially remain above the silty fine sand layer due to the remnant groundwater mounds that will exist between each eductor well.

Depending on the amount of remnant groundwater remaining in the upper aquifer, additional measures in the form of shallow (25-foot-deep) dewatering wells or vacuum wellpoints may also be required in the assumed worst case to adequately dewater Soil Unit 1. The need for additional wells or wellpoints should be determined by measuring water levels in piezometers installed to the base of Soil Unit 1, and spaced at approximately 100-foot centers along the outside of the deep sheet pile walls. Additional dewatering efforts, if required, could be contracted on a force-account basis.

Dewatering of the Deep Channel Excavation

To facilitate construction of the floor slab and transfer struts forming the lower section of the graving dock, dewatering is required to produce dry working conditions at a base grade elevation of -20 feet. Transfer struts will be cast at this elevation, with the top of the slab at elevation -15.5 feet. Groundwater levels need to be lowered ideally 2 feet below the deepest part of the excavation, i.e., to elevation -22 feet, to ensure dry conditions and avoid surface softening.

Penetration of the sheet pile wall through the sand/silt aquitard and the underlying thin aquifer (Soil Unit 4) will provide a hydraulic cutoff to potential groundwater inflow via both the upper and lower aquifers. Groundwater in the soil mass contained by the sheet piles should be effectively isolated in this manner. Given the potential for achieving a full hydraulic cut-off with the deep sheet pile walls, the need for dewatering of the soils contained between the sheet piles must be considered. Although the soils will be isolated from lateral sources of recharge, they will still be fully saturated with entrained pore water.

Excavation contractors may be tempted to rely on sump pumping to deal with the remnant pore water that is contained within the soils inside the sheet pile walls, once the cut-off has become effective. While this approach may realize savings on the substantial cost of internal dewatering, the potential disadvantages should not be overlooked. Loose, silty, fine sands and soft to very soft silts of Soils Units 2 and 3 that are fully saturated will degrade rapidly when disturbed. Excavation without the benefit of proactive dewatering could result in the base of the excavation becoming potentially unworkable, and having insufficient integrity to support the construction of transfer struts or the placement of structural fill/draining layer material. However, sumps may be effective for removing entrained water from the more permeable upper aquifer in Soil Unit 1.

The potential construction risks can be eliminated with the inclusion of a properly designed proactive dewatering system. Without such a system, the added construction costs in terms of delays, difficulty in materials handling, and constraints on the mobility of equipment in the excavation can rapidly add up and substantially exceed the investment in up-front dewatering costs.

Given the target elevation of -22 feet for dewatering in the deeper section of the graving dock, our recommendation is to install a system of eductor wells inside the sheet pile wall to remove entrained water and firm up the soft silts and loose sands of Soil Unit 3. Eductor wells have the advantage over other alternatives in that one system should be capable of producing the required dewatering.

To minimize interference with excavation and construction activities, the system on the inside the western wall could be installed first, immediately following the installation of the sheet pile wall separating the upper and lower sections of the graving dock. This could allow eductor headers to be laid on the excavated ground surface (or upper slab) at around elevation 10 feet, thus avoiding the need to suspend the system from the sheet piles.

Since the sheet piles form a cutoff to lateral flow, eductor wells located inside the deeper section of sheet pile wall should operate more effectively than outside the excavation, especially once the water contained within the sheet piles in the overlying aquifer of Soil Unit 1 has been completely removed. If excavation and construction schedules allow, it may be more cost-effective to operate this single system along the western side of the deep channel for 2 to 3 weeks and monitor its effectiveness in a series of piezometers installed in the central and eastern parts of the deep channel area.

An extended period of operation (at least 2 to 4 weeks) may be required before the internal eductor systems are able to fully lower water levels across the full area of the deep channel section. The system will need to remove at least 2.5 million gallons of entrained water, plus any leakage through the sheet piles and upward flow from below. To achieve full drawdown within a month, the system will need to operate continuously at a rate of at least 60 gpm. Piezometers should be installed to allow system performance and effectiveness to be monitored, and to permit system modifications in a timely manner, if required.

Other Alternative Dewatering Methods. As an alternative to the eductor systems, dewatering inside sheet pile walls forming the deep channel section of the graving dock excavations could be accomplished using other methods in a series of stages or options:

- With shallow wells or sumps to drain water by gravity from the upper more permeable aquifer soils, followed by wellpoints installed from above the lowered water table:

  - Shallow wells or sumps could be drilled or excavated from the water table (elevation 7 feet) to the base of the upper aquifer (elevation -5 feet). The effectiveness of this system would be limited by the interface with loose, silty, fine sand/sandy silt below approximate elevation -5 feet that is not expected to drain effectively.

  - Wellpoints could be jetted from within the partial excavation to a depth of 25 feet (i.e., to elevation –18 to –27 feet), set back inside the sheet piles by approximately 2 feet, and connected to vacuum headers hung from brackets welded to the sheet piles (see Figure 9).

  This is expected to be the least costly method for dewatering the excavation but it carries the risk that the wellpoint headers may not be installed low enough for dewatering of the subgrade soils to be fully effective down to the desired depth.

- Alternatively, with two stages of wellpoints:

  - The first stage of wellpoints installed to maximum depth (30 feet) once the excavation reaches the water table at a depth of around 6 feet below ground level (elevation 7 feet).

  - As the excavation proceeds to the top of the loose, silty, fine sand/sandy silt aquitard, a second tier of wellpoints will be required to be installed between the first tier, and jetted to a depth of 25 feet (i.e., to elevation -30 feet).

  This option has a high likelihood for success, but is likely to be significantly more expensive than a single ring of conventional wellpoints installed from the deepest practical elevation, and will likely exceed the cost of a single eductor system. Two wellpoint rings will also impose the greatest interference to excavation and construction work in the deep channel section and are, therefore, not recommended.

To be fully effective, internal wellpoints need to be installed as low as possible inside the sheet pile walls; however, this work will be constrained by saturated conditions below the original water table (elevation 7 feet). Sump pumping is likely to help in removing water only from the upper aquifer materials within the excavation, as the lateral source of recharge will have been cut off by the sheet pile walls. The internal wellpoints will need to be installed as low as possible if it is to effectively reduce pore pressures in the subgrade soils beneath the transfer struts, which will also benefit from the cut-off effect of the sheet pile walls.

Deep Aquifer Depressurization

The deep sheet piles are expected to extend down to and through the medium dense to dense sand layer of Soil Unit 4 that forms a thin aquifer beneath the fine sand/silt layer, and penetrate into the hard underlying silt of Soil Unit 5 by 5 to 10 feet However, the complete effectiveness of a hydraulic cut-off at this depth cannot be guaranteed, since pile separation or the presence of a few windows at depth could provide the small amount of lateral inflow needed to maintain high potentiometric pressure levels within the lower aquifer (Soil Unit 4). The dewatering design therefore must consider the need for depressurization of the groundwater in this deep layer to maintain the stability of the overlying soils in the critical excavated condition.

Pressure relief of groundwater in the lower aquifer layer will be required to ensure that the plug of soils contained beneath the base of the final excavation within the sheet piles is not subject to heave due to high groundwater head at

depth. This could be achieved with a group of deep wells installed inside the excavation to penetrate at least 3 feet into the underlying deep silt aquitard (Soil Unit 5), generally below elevation –47 feet. Wells could be installed:

1. Either from the ground surface (60+ feet deep) prior to the start of excavation. However, the wells would be exposed and vulnerable to damage throughout the period of excavation; or

2. From a lower elevation inside the partially completed excavation, depending on the performance and effectiveness of the deep cut-off and the other methods of internal dewatering, as determined by ongoing piezometer measurements.

Wells would be relatively widely spaced, between 100 and 200 feet apart. A layout of eight deep wells strategically located within the excavation is proposed. Based on the results of the pumping test, we expect that well yields could vary between 5 and 50 gpm, with the effectiveness of the system dependent on the individual well yields that are obtained. Actual well yields will depend on the thickness and permeability of the lower aquifer at each well location, and the effectiveness of the hydraulic cut-off achieved by the sheet piles. Well construction and development will also be a key factor given the difficult drilling conditions at the site and the risk for gravel pack materials to become clogged with fine sand and silt from the aquitard layer. Temporary drillhole casing to the full depth may be required to ensure that the filter material does not become clogged.

To ascertain the performance of deep well depressurization as work proceeds, water pressures within the lower aquifer layer should be monitored using piezometers located centrally within the excavation. If the initial system of eight wells does not appear to be lowering water levels as expected (taking into consideration actual well yields achieved), then additional wells may be required.

Although the deep depressurization system will be drawing water from an aquifer that is directly connected to Port Angeles Harbor, flowrates will be limited by the permeability of the soil, such that sufficient loss in head (hydraulic gradient within the soil mass) can be sustained between Port Angeles Harbor and the depressurization system due to the hydraulic resistance of the aquifer. Flow rates should be further limited by the cut-off provided by the sheet piles, subject to their degree of effectiveness. Fluctuations due to tide levels will be small (less than 1 foot) and will be translated into corresponding periodic minor changes in flow rate from the deep wells.

Dewatering for the Gate and Pump/Fish Exit Structure

Construction of the gate and pump/fish exit structure are expected to take place within a temporary cofferdam constructed as a northward extension of the deeper channel section of the graving dock. We understand this portion of the dewatering work may be contractor-designed. Depending on the timing of this work and the related channel dredging to provide access to the gate area, the general recommendations on dewatering of the deeper channel section likely apply to this area as well.

The cofferdam for this area is expected to provide a hydraulic cut-off at depth, but the subgrade soils contained within the cofferdam will still require dewatering. Seepage flow paths will be correspondingly shorter, which will give rise to higher flowrates and greater pumping needed to achieve the same drawdowns as in the deep channel section.

Long-Term Control of Groundwater Pressures

Post-construction and long-term performance of the graving dock has also been considered in terms of the need for control of pore pressures in the foundation soils beneath both the upper and lower sections of the floor slab during conditions that include super flood, dry, and transitional phases of graving dock operation. The potential effects of these conditions on wall and slab integrity are considered, and alternative recommendations are provided relating to structures that are resistant to uplift forces and for the relief of excess hydrostatic pressures.

To prevent heave and uplift of the lower floor slab in the graving dock, we recommend the installation of below-slab drainage layer with a network of perforated underslab drainage pipes that will allow subgrade pore pressures to dissipate. The drainage layer should comprise at least 12 inches of clean coarse sand and gravel (free of fines), laid on the subgrade below the floor slab, and spanning between or above the transfer struts. Perforated drainage pipes (4-inch-diameter) should be laid within the drainage layer between each transfer strut. The current design calls for a peripheral drainage trench cast into the lower slab, where the underslab drainage pipes would discharge.

The volume of pore water inflow to the underslab drainage system during dry dock conditions is expected to be relatively low, assuming a reasonably effective sheet pile cut-off, and should be manageable as a small but continuous baseflow of no more than 5 to 10 gpm. This is likely to be a lot less than the flow rates for stormwater generated from rainfall on the lower slab, and should not be an issue in sizing the pumps. The current design calls for four sumps equipped with submersible pumps.

Eductor wells installed for the internal dewatering of the deep channel section could be incorporated into the permanent pressure-relief scheme, rather than being decommissioned or removed. The current floor slab design includes a perimeter drainage channel cast into the floor of the graving dock (see Figure 9). The trench section of the slab could be cast around the installed wellpoints while leaving them in place. Wellpoint riser pipes could then be cut off at (or slightly above) the drain invert level, allowing them to "bleed" water into the drainage channel, thus helping to dissipate excess pore pressures in the subsoils directly below the drain.

Passively flowing wellpoints would not be as effective in lowering underslab pore pressures as ones operated with active vacuum pumping, hence the need for underslab drainage. When the graving dock is flooded, the relief holes and cut-down wellpoints could allow water pressures to equalize on either side of the floor slab, thus reducing potential heave and uplift or settlement problems. Alternatively, the drainage pipes and wells/wellpoints could be sealed with one-way or flap valves to prevent sediment from entering the subsurface drainage facilities.

Similar drainage facilities are not required beneath the upper slab for dry operations but the super flood condition will potentially recharge the shallow soils above the water table, and potentially create excess pressures beneath the upper slab during pump-down of the dock. This assumes that the upper slab will not be waterproof, with cracks and construction joints allowing flood water to seep through and enter underlying (unsaturated) soils.

The most significant outflow component during super flooding is likely to be the water volume required to saturate the soils beneath the upper slab that are above the water table. Assuming a vadose zone thickness of 4 feet and a fillable porosity of 15 percent, this represents a pore volume of over 200,000 cubic feet. If the filling process above the upper slab takes 56 hours, the average seepage flow required to saturate the soil in the vadose zone below the slab is over 450 gpm. Leakage through the upper slab at this rate would depend on the existence of sufficient pathways through cracks and construction joints.

Transient effects of pore filling and drainage related to the cycle of dock flooding and emptying are likely to exceed the overall rates of seepage through the subsoils and beneath the upper sheet pile wall. Steady-state values estimated for this rate of seepage are between 50 and 100 gpm, using both a simple flownet analysis and assuming large-scale radial flow. However, this simplified analysis ignores any leakage through the sheet piles.

Dewatering System Monitoring

Construction monitoring of dewatering system performance and effectiveness is a key component to assuring a successful project. Monitoring should include:

- Documentation of the installation: well depths, screen locations, pump settings;
- Operating hours and flowrates for pumps;
- Water levels in piezometers and monitoring wells; and
- Periodic sampling of discharge flows for water quality parameters.

Ideally, system monitoring should be incorporated into the design and implementation approach, given the difficult ground conditions. Potential uncertainties in soil properties and system performance may require more or less dewatering effort than is envisioned in this design recommendation.

Water Treatment and Disposal

Water flows from the dewatering system are expected to be relatively clean despite the presence of localized contamination hot spots in the surface soils. Flows from the system may be initially turbid, especially during the development of individual wells and during system restarts, but should otherwise produce clear, sediment-free water. Salinity levels do not appear to be elevated, based on water quality monitoring during the pumping test (Table 1), but wells closer to the shoreline are more likely to produce brackish or saline water.

Discharges from the dewatering system should be initially passed through one or more Baker tanks that will allow suspended sediment to settle out, and enable the detection of surface films, which could indicate petroleum contamination. Baker tanks can be fitted with cartridge filters on their discharge lines to deal with specific contaminants. Alternatively, dewatering discharges could be routed to the stormwater system, and subjected to treatment train that includes oil separation, retention ponds, and an infiltration bioswale.

Disposal may be permissible to Port Angeles Harbor but this is likely to require approval from the Washington State Department of Ecology and/or Department of Fish and Wildlife. It would be preferable to discharge to the Port's stormwater system, in which case the quality of the water should meet any applicable discharge requirements.

Alternatively, disposal to any local sewer system may be considered but this option may incur discharge fees based an assumed waste treatment parameters, even if the water is clean. Sewer districts also commonly impose discharge volume limitations, especially during the winter months when the sewer system capacity is reserved to handle peak flows.

Water should be sampled for potential contaminants, per the requirements of the agency overseeing the disposal of water. Sampling should occur soon after system start-up, and should be conducted with expedited lab turnaround to ensure that pumping contaminated waters is kept to a minimum.

Side Effects of Dewatering

This section considers potential side effects of dewatering, including induced settlements, pumping of contaminated groundwater, and the potential drawdown effect on existing water supply wells in the vicinity of the project site.

Structural Settlements

Reduction of water levels associated with construction dewatering increases the effective stress in soils below the water table. Generally these stress levels occur within the range of stress history already experienced by the soil. However, if the new levels of effective stress achieved through dewatering exceed the stress history for the soil at a given depth, additional settlements could occur. Settlement occurring beneath the foundations of sensitive buildings or other structures could cause structural damage. We understand that there are no major structures within the immediate vicinity of the site.

For the graving dock, depending on construction schedule, external dewatering to relieve hydrostatic loads on the sheet pile wall for the deep channel could occur after all or some of the upper slab is cast. This could potentially subject the upper slab to some differential settlements (2 to 3 inches), decreasing in magnitude away from the lower dock. We understand that these settlements are tolerable.

Contaminated Soils and Groundwater

We received preliminary environmental test results for soil samples from the recent geotechnical borings. Based on our discussions with WSDOT and our review of the boring logs and field notes, we selected soil samples for analyses primarily in the upper 5 to 10 feet and one per boring. However, two of the borings exhibited potential environmental impacts (odor and staining) so we analyzed two samples in those borings (H-3-02 and H-4-02). In addition, five

environmental borings were previously advanced for WSDOT, and soil samples were collected and chemically analyzed. Groundwater monitoring wells were also installed in these five borings and groundwater samples were collected and analyzed as part of the environmental investigation.

The chemical results indicated low concentrations (73 and 54 mg/kg) of TPH in the diesel-range in H-3-02 between 3 and 9 feet below grade. Only one of the soil samples in H-4-02 had detectable concentrations of TPH. This was the shallow soil sample at 2.5 to 4.0 feet and the concentration was 640 mg/kg TPH in the heavy oil-range. The other H-4-02 sample at 12.5 to 14.0 feet had no detectable concentrations of TPH. We did not have a soil sample between these two samples because there was no soil recovery due to silt and wood jammed into the sampling shoe.

Only one soil sample from the five borings advanced as part of the environmental investigation, was found to have a detectable concentration of 30 mg/kg TPH in the diesel range, between 5.0 and 6.5 feet (HC-SE-02). The soil samples above and below this depth contained non-detectable concentrations of TPH. None of the groundwater samples analyzed contained any detectable concentrations at TPH, semivolatiles, or PAHs.

There were no detectable concentrations of semivolatiles, including PAHs, in any of the soil samples analyzed. Based on this information and our field observations, there does not appear to be any wide-spread contamination in the soils in the areas sampled by the geotechnical borings or the previous five environmental borings.

Given that contaminated soils are relatively rare at the site, and are generally present only at or around the water table, we do not expect that any of the dewatering activities described above would result in significant mobilization of subsurface contaminants. It is considered highly unlikely that such contaminants, if present, would be detectable in the water being pumped from the proposed dewatering system.

Impacts to Adjacent Wells

A record review was completed to address the potential for drawdown in the upper aquifer caused by external dewatering of the sheet pile wall to impact water levels or production rates in any existing water supply wells in the vicinity. The review included a query of the Washington State Department of Health and Department of Ecology public records. Documents indicate the closest water supply or irrigation well to be greater than one mile away from the site. Moreover, the closest well is located on the bluff to the south, significantly

above the elevation of the site. Dewatering of the graving dock excavation will not affect any of the water supply or irrigation wells located by this record review.

Retaining Structures

The construction of the graving dock will require a relatively watertight permanent wall along the entire perimeter of the graving dock, and the western side of the deep channel. The depth of excavation in the deep channel area is expected to be about 28 to 30 feet. Along the outside perimeter of the upper slab, the perimeter wall will mainly support water pressure during the super flood period. The following sections present discussions about wall type alternatives and design recommendations on the lateral earth pressures.

Types of Permanent Retaining Walls

The purpose of the retaining walls is to provide hydraulic cut off in addition to support lateral loads. Sheet piles and continuous secant soldier piles represent two feasible alternatives to support the deep channel excavation and the upper slab walls. A hybrid wall consisting of soldier piles to resist lateral forces and sheet piles to cut off seepage is technically feasible but may not be economical. Additionally, given the permanent use of these walls, ground freezing is likely not feasible in our opinion.

The sheet pile wall alternative is likely more favorable because it provides a continuous section for the above and below grade walls. However, it may have some limitations regarding its flexural capacity and the ability to penetrate dense to very dense gravelly soils. Hard driving through dense granular soils could potentially be a source of leakage due to sheeting ripping out of the interlocks.

A secant soldier pile wall could be constructed below grade to provide hydraulic cut off and lateral support for the deep channel. A separate retaining structure would be required for the above-grade wall. The above-grade wall may include a cast-in-place concrete wall or a Mechanically Stabilized Earth (MSE) wall. Some special detailing will likely be required to provide a watertight connection between the above- and below-grade walls.

The preliminary design by KPFF, the structural engineer of the project, calls for sheet pile walls. The recommended earth pressures presented below are applicable to both types of walls provided they are supported by deadman or with a single level of tiebacks.

Deep Channel Walls

The walls surrounding the deep channel will be subjected to five different lateral loading cases.

- **Construction Phase (No Slab and Transfer Beam Support).** Groundwater outside the deep channel will be lowered to the bottom of excavation elevation (elevation –22 feet).

- **Operational Phase.** Groundwater outside the dock will be at about elevation 7 feet. The floor slab and transfer struts will provide lateral support.

- **Super Flood Phase (Water at Elevation 27 Feet).** During this phase, the direction of loading will be reversed. In other words, the soils outside the dock will support the wall by mobilizing passive pressures rather than active pressures. For this case, the wall will behave as a cantilevered wall. The deadman/tieback, the floor slab and transfer struts will not provide support to the wall.

- **Earthquake.** Two levels of earthquakes, 475-year and 190-year events, have been considered in our engineering analyses as described previously. Because of the relatively short period of the super flood phase (a few days every 5 to 6 months), we have assumed that earthquake loading is only relevant to the operational phase.

- **Post-Earthquake (Liquefaction).** Based on our analyses, wide-spread liquefaction may only occur under the 475-year earthquake. Again this case is only relevant to the operational phase. It is important to note that we have assumed that liquefaction would occur after peak shaking (i.e., earthquake loading and liquefaction would not be combined).

Design Wall Sections and Corresponding Earth Pressures

Given the variations in soil conditions and the size of the deep channel, we have provided multiple design wall sections along the perimeter to achieve a cost-effective design. Along the perimeter of the channel, we have identified three segments designated as C-1 through C-3 (refer to Figure 10). Earth pressure parameters for each segment are presented in Tables 2 through 4.

- **Segment C-1.** Widespread liquefaction may occur within Soil Unit 2 along this segment under the 475-year earthquake. Therefore, the post-earthquake scenario should be considered in the design. The sandy Silt (Soil Unit 3) was

not encountered in borings H-6-02 drilled in the vicinity of this segment. Soil parameters for this segment are presented in Table 2. For illustrative purpose, the subsurface profile along this segment and the corresponding earth pressure diagrams are presented on Figures 11 and 12.

An important assumption associated with these earth pressure diagrams is that the lower aquifer will be permanently depressurized as discusses subsequently. As illustrated on these figures, the earth pressures directions near the wall toe are applicable for the **free earth support** method, which will be discussed subsequently. For the **fixed earth support** method, assuming adequate embedment, the active and passive pressures near the tip of the pile would be on the opposite sides of what were shown on these figures.

- **Segment C-2.** This segment may represent the assumed worst-case scenario for all the loading cases because of the presence of the medium stiff sandy silt (Soil Unit 3) and the depth to the top of the bearing layer. Soil parameters for this segment are presented in Table 3.

- **Segment C-3.** This segment is characterized by the relatively dense Soil Unit 2 (i.e., no global liquefaction) and by the relatively shallow load bearing layer (refer to Figure 3). Additionally, the sandy Silts (Soil Unit 3) were not encountered in borings drilled in the vicinity of this segment. Soils parameters for this segment are presented in Table 4.

Earth Pressures Assumptions

The following general assumptions and seismic-related loadings should be used in conjunction with the soils parameters presented in Tables 2 through 4.

1—Construction Phase

- Groundwater table inside the excavation is at least 2 feet below the bottom of excavation;
- Neglect top 2 feet of passive pressure; and
- Effect of seepage pressure beneath the wall can be ignored assuming embedment in hard SILT (Lower Aquitard).

2—Operational Phase

- Groundwater table outside the dock is at elevation 7 feet;
- Groundwater table inside the dock is at the bottom of underslab drain; and

- Passive pressure mobilized in the construction phase will remain effective on the wall. In other words, placing the concrete floor slab and the transfer struts would not reduce the passive pressures mobilized during construction.

3—Super Flood Phase

- Assume the water pressure starts at elevation 27 feet and extends to the bottom of the sheet pile. This assumption implies that the slab will have joints or cracks such that the underlying soils are hydraulically connected to the water in the channel.

4—Earthquake Loading

- 475-year Earthquake
  - **Active Pressures.** Add distributed pressure approximately 5.5Z psf, where Z is the depth to the bottom of the excavation.
  - Ignore hydrodynamic effect within the Sand and Gravel unit (minor effect).
  - **Passive Pressures.** Use K_P (ultimate) to estimate passive resistance. K_P (ultimate)= K_P (design) x 1.5 (FS) x 0.9 (reduction due to earthquake based on M-O method).

- 190-year Earthquake
  - **Active Pressures.** Add distributed pressure =2.5Z psf, where Z is the depth to the bottom of the excavation.
  - **Passive Pressures.** Use K_P (design). The reason for that is that the earthquake loading would reduce K_P by about 5 percent based on M-O method. On the other hand, K_P will increase as the wall moves outward. So, earthquake loading would have opposite effects on K_P, which can be practically equal. For simplicity use K_P (design).

5—Post-Earthquake (Liquefaction) - Segments C-1 and C-2 Only

- Liquefaction due to 475-year Earthquake

  No change in water and active earth pressures except for the silty Sand and the sandy Silt (Soil Units 2 and 3).

  - **Active Pressures.** Active earth pressure at the top and bottom of the liquefiable layer should be calculated using the following equation:

$$p_a = \sigma'_v - 2 * S_r$$

Where:
p_a = effective active earth pressure;
σ'_v = effective vertical stress; and
S_r = residual shear strength (refer to Tables 2 and 3). Note that the effective active earth pressure should not be less than the static active earth pressure.

- **Passive Pressures.** Use K_P (ultimate) rather than K_P (design) for soils except for liquefiable soils (Soil Units 2 and 3).

$$K_p(\text{ultimate}) = 1.5 * K_p(\text{design})$$

For liquefiable soils , use the following equation to calculate passive pressure at the top and bottom of the liquefiable layer:

$$p_p = \sigma'_v + 2 * S_r$$

Where:
p_p = effective passive earth pressure;
σ'_v = effective vertical stress; and
S_r = residual shear strength (refer to Tables 2 and 3). Note that the effective passive earth pressure should not be higher than the static passive earth pressure.

- 190-year Earthquake. No global liquefaction.

6—Surcharge Loading

Refer to Figure 11 for distributed surcharge loading. For other cases, refer to Figure 13.

7—Fill Behind the Wall

The upper 6 to 7 feet below grade will be excavated to install the deadman anchors spaced 10 feet on center. On-site, granular soils can be reused for backfill. Feasible compaction effort depends on the moisture conditions of these soils, weather conditions, and the project schedule. In Tables 2 and 4, we presented parameters of on-site granular soils compacted to 90 or 95 percent of the maximum dry density as determined by the modified Proctor maximum ASTM D 1557 test procedure. The project specifications should be consistent with the soil parameters used in the design. Note that soils within the passive zone of the deadman should compacted to 95 percent of the maximum dry density as discussed subsequently.

Methods of Sheet Pile Wall Design

There are two conventional methods for designing sheet pile walls: (1) free earth support and (2) fixed earth support. Geotechnical considerations for each method are discussed below.

Free Earth Support. This design method assumes that the pile tip is free to rotate (i.e., point of zero moment is at the bottom of the wall). The point of zero moment is critical in determining the location of the deadman as discussed subsequently. The assumption of free rotation implies that that passive pressures will only be mobilized on one side of the wall as shown on the Figures 11 and 12. This assumption can be realistic for a certain wall penetration, below which wall fixity starts to develop gradually till it reaches full fixity at a certain penetration. A Rowe's moment reduction can be applied for sheet pile walls designed using the free earth support provided the walls are NOT embedded in very loose to loose granular soils. We understand that this method will require less penetration into the load bearing layer than the fixed earth support for this project.

Fixed Earth Support. This design method assumes the pile tip is not free to rotate (i.e., fixed). In this case, the wall is deformed in such a way that the point of zero moment is somewhere, depending on the soil stiffness, between the bottom of excavation and the tip of the sheet pile. In this case, the passive pressures will be mobilized on both sides of the wall. The Rowe's moment reduction is NOT applicable for this design method. Two approaches are normally used to analyze sheet pile using this method: (1) equivalent beam method and (2) deflection method. The deflection approach is typically performed using computer software.

Appropriate Design Method. From a geotechnical perspective, both methods are applicable to the site, each with its advantages and disadvantages. The main advantage of the free earth method is the shallower penetration, which will reduce the sheet pile driving through the glacial soils (Soil Unit 5). The main advantage of the fixed earth support method is that the glacial soils (Soil Unit 5) will provide adequate fixity for the sheet piles walls without relatively deep penetration.

However, it is our understanding that the governing factors are the location of the anchor (dependent on the point of zero moment), the maximum bending moment, and the anchor force. According to KPFF, the use of the fixed earth support, as compared to the free earth support method, results in: (1) the point of zero moment is higher, (2) the maximum bending moment is lower, and (3) the anchor force is lower. Therefore, it is our understanding that the fixed

earth support method has been used in the design of the sheet piles for the deep channel.

Minimum Embedment into Glacial Soils. The passive coefficients presented in Tables 2 through 4 include a factor of safety of 1.5. According to Teng (1962), which is the reference used by AASHTO (1996) on this subject, an additional factor of safety is NOT required for the wall embedment. However, for wall segments C-1 and C-2 where liquefiable soils overlie the glacial soils, we recommend that the sheet piles be embedded at least 5 feet into the glacial soils. For wall segment C-3, we recommend that the sheet pile be embedded at least 3 feet into glacial soils to provide adequate seepage cutoff.

Sheet Pile Drivability

We performed a wave equation analysis to assess the drivability of the proposed sheet pile section based on KPFF's preliminary sheet pile design. We assumed a Delmag D-46 impact hammer and a 80-foot-long AZ48 section (50 ksi) with about 15 feet of embedment (friction distribution in the lower 20 percent of shaft) into the glacial soils.

For a 15-foot embedment, the ultimate axial capacity of the AZ48-section would be about 300 kips. The analyses suggests a range of blow counts between 20 to 50 blows per foot (bpf) and maximum compressive stresses of 33 to 41 ksi, for a predicted axial capacity of 300 to 600 kips. The higher load was examined to account for uncertainties in estimating the load capacity. Based on our discussion with local pile contractors, we feel that a reasonable maximum sheet pile penetration depth into the glacial soils would be about 10 feet, but probably no greater than 15 feet.

Note that using a box section will require more energy because of the increase of the surface area and the potential developing of a plug. Therefore, a 10-foot penetration of a box section into the glacial soils may not be feasible.

Effect of Super Flood on Pore Water Pressure

This issue is only relevant to the western wall of the deep channel (top of the wall at elevation 10 feet). During the super flood phase, the pore water pressure within the soil mass behind the wall may increase to correspond to the water level at elevation 27 feet. Following the super flood phase, the soil mass may retain excess pore water pressures for some time, which would increase the lateral pressures on the wall.

This may occur in Soil Unit 2 and/or Soil Unit 3, which have relatively low permeability. However, the build-up of pore water pressure would be mainly due to the consolidation process of these layers rather than due to hydraulic connection between the water above and below the slab. Because the loading period is short (a few days), the excess pore water pressure would dissipate relatively fast (within a few days) following the end of the super flood phase.. Therefore, we do not recommend any special measures to reduce this excess pore water pressures.

Upper Slab Walls

The upper slab walls will extend up to elevation 28 feet. The water inside the graving dock will be at elevation 27 feet. It is our understanding that the current design calls for a 30-foot-wide berm behind the upper slab walls to laterally support the perimeter wall during the super flood phase. The top of the berm as shown on the preliminary drawings provided by KPFF will be at elevation 21 feet (i.e., about 8 feet above the existing grade).

For the upper slab walls, we have not considered earthquake and post-earthquake loadings because of the low probability that the design earthquake would occur during the super flood level. During the operational phase, the wall will only support the proposed berm. However, the super flood phase will likely govern the design of the sheet pile wall.

The perimeter wall surrounding the upper slab can be divided into two wall segments (U-1 and U-2) to reflect varying soil conditions. The limits of the two segments are shown on Figure 10. Earth pressure parameters for segments U-1 and U-2 are presented in Tables 5 and 6, respectively. Note that for wall segments supported by the proposed earth berm, the passive earth pressure coefficients for relatively deep layers were reduced to account for the limited width of the earth berm (about 30 feet).

On-site granular soils can be used to construct the berm. Feasible compaction effort depends on the moistures conditions of these soils, weather conditions, and the project schedule. In Tables 5 and 6, we presented parameters for on-site granular soils compacted to 90 or 95 percent of the maximum dry density as determined by the WSDOT Test Method No. 606 (for soils with at least 30 percent gravel) or by AASHTO T99 (for soils with less than 30 percent gravel). The project specifications should be consistent with the soil parameters used in the design.

Embedment Factor of Safety. The passive coefficients presented in Tables 5 and 6 include a factor of safety of at least 1.5. According to Teng (1962), which is

the reference used by AASHTO (1996) on this subject, an additional factor of safety is not required for the wall embedment.

Water Pressure during the Super Flood Case

Active Side (Inside the Dock). Assume the water pressure starts at elevation 27 feet and extends to the bottom of the sheet pile. We assume that the slab will have joints or cracks that allow hydraulic connection between the top and bottom of the slab.

Passive Side. Assume groundwater table is at elevation 7 feet.

Seepage Cutoff

Deep Channel Walls. It is anticipated that the walls would extend into the glacially overridden soils, which are predominantly silts. These soils are relatively impervious and would provide adequate seepage cut off.

Upper Slab Walls. We recommend that the wall be embedded in Soil Unit 2 (Sand/Silt aquitard) with the tip of the wall being at elevation –17 feet (i.e., the depth of penetration is about 27 feet below top of slab). Based on results of the pumping test, this permeability of this layer is estimated to be about 1×10^{-5} cm/sec, which is adequate to cut off seepage during the super flood phase.

Additionally, the 27-foot-deep penetration (from elevation 10 to –17 feet) will provide an adequate factor of safety (greater than 2.0) against heave in soils located outside the dock based on NAVFAC DM-7.01.

Deadman and Tieback Design Recommendations

The sheet piles surrounding the deep channel will be supported with either deadman (concrete or sheet pile) or tiebacks. The current design calls for a continuous sheet pile deadman with anchors spaced at 10-foot on center. Along the western wall, the deadman will be entirely below the groundwater table. Elsewhere, the deadman will be partially below the groundwater table. As an alternative to deadman, tiebacks might be used.

Deadman - Earth Pressures

The recommended static and seismic earth pressures for concrete deadman are shown on Figure 14. For the sheet pile deadman, the earth pressures parameters presented in Tables 2 through 4 should be used in the design.

In our earth pressure recommendations, we assume that the excavation for the deadman anchors within the passive wedge will be backfilled with granular soils compacted to 95 percent of the maximum dry density as determined by the WSDOT Test Method No. 606 (for soils with at least 30 percent gravel) or by AASHTO T99 (for soils with less than 30 percent gravel). The passive wedge is determined by a line inclined at (45° $-\phi'/2$) from the horizontal and extending from the bottom of concrete deadman or from the point where passive pressure reverses its direction (sheet pile deadman) to the ground surface.

Deadman - Location

Two criteria govern the deadman location. The first criterion is that the deadman should be located behind a line that extends from the point of zero moment on the sheet pile wall to the ground surface with an inclination that varies from one soil unit to another. Through each soil unit, the line inclination is equal to the soil friction angle of this layer measured from the horizontal. By placing the deadman behind this line, no additional horizontal loads will be acting on the sheet pile perimeter wall under static conditions.

The second criterion is that the passive wedge acting on the deadman does not overlap with the active wedge acting on the sheet pile wall of the deep channel. The passive wedge is estimated by a line inclined at (45° $-\phi'/2$) from the horizontal, and the active wedge is estimated by a line inclined at (45° + $\phi'/2$) from the horizontal. The top of deadman should be at least 2 feet below grade.

Tiebacks

Tiebacks would be required in the case that the deadman embedded to a practical depth does not provide adequate lateral support. The main disadvantage of a tieback system is soil liquefaction. Given the practical tieback inclination and the anticipated no load zone limits, it is expected that tieback bond length would be located within the silty Sands (Soil Unit 2) and the sandy Silts (Soil Unit 3). As mentioned previously, wide-spread liquefaction may occur within Soil Units 2 and 3 under the 475-year earthquake, leading to a significant loss in tieback capacities. In this case, the tieback alternative may not be feasible, unless the bond length can be extended into the deeper non-liquefiable soils (Soil Unit 5).

In the case of the designing the graving dock based on the 190-year earthquake, the tieback alternative would be feasible because no wide-spread liquefaction is anticipated. Within Soil Units 2 and/or 3, the bond length of the anchor should be determined using an allowable unit friction of 0.5 ksf and a maximum pullout capacity of 2.0 kips/ft, assuming gravity-grouted tiebacks. Secondary or pressure

grouting can be used to increase the allowable unit friction. Along the southern edge of the channel, higher bond capacity can be achieved if anchors are installed in the glacial soils.

Excavation Stability

Deep Channel Walls. For the deep channel excavation, it is expected that the sheet piles would be embedded several feet into the glacial overridden soils. Therefore, global instability is not anticipated.

Upper Slab Walls. We performed global stability analyses for segment U-1 (refer to Figure 10), which represents the assumed worst-case scenario. As discussed above, the super flood phase is the only case considered in the analyses. Soils parameters presented in Table 7 were used in the analyses except for the berm, which was not considered in the analysis (conservative assumption). The stability analysis results suggest that the wall would have an adequate factor of safety (greater than 2.0).

Graving Dock Slabs

Deep Channel Slab

Settlement and Bearing Pressure

Total and differential settlements would likely govern the slab design. Due to the excavation for the deep channel, the reduction in effective stress reduction on the soils underlying the slab is about 2,000 psf. During the operational phase, the weight of the heaviest pontoon will cause about 600 psf of vertical stress. During the super flood phase the weight of the water (42 feet of water head) would increase the effective stresses on the soils underlying the slab by about 2,680 psf. In this case, we have assumed that Soil Units 2 and 3 are NOT hydraulically connected to water above the slab. Therefore, the super flood will be the critical case. For the 2,680 psf increase in vertical stress, about 2,000 psf will be re-compression (i.e., stress within the range of stress history already experienced by the soil) and the reminder 700 psf will exceed the preconsolidation stress.

Because of the variation of the depth to the top of bearing and the presence of the sandy Silt soils, settlement would vary across the site. Our engineering analyses suggest the settlement would vary from about 1 inch toward the southern edge of the channel to 2 to 4 inches toward the northern edge of the channel. Settlement in the vicinity of the northern edge of the channel may cause downdrag on some piles supporting the gate.

Modulus of Subgrade. The slab will be mostly supported on saturated, loose to medium dense, silty Sand (Soil Unit 2). Therefore, we recommend a subgrade modulus of 50 pci.

Slab Uplift

As described in the dewatering section, a complete hydraulic cut off cannot be guaranteed. Therefore, the water pressure in Soil Unit 4 may reflect the water pressure outside the dock during the operational phase. Our analyses suggest that the factor of safety against uplift can be as low as 0.5 along the southern edge of the channel. The factor of safety increases to about 0.9 near the northern edge of the deep channel where the deep aquifer is relatively deep. The vertical stress required to achieve a factor of safety of about 1.25 against uplift varies linearly from 1,000 psf along the southern edge to about 500 psf along the northern edge of the deep channel.

A pressure relief system as described in the dewatering section is required to reduce the risk of slab uplift. Combined with a relief system, the slab can be supported against uplift using piles, micropiles, or tiedowns, embedded into the glacially overridden soils. Because of their stiffness, piles or micropiles will also carry the compression loads (about 2,680 psf during the super flood phase) acting on the slab. Tiedowns should be designed for only uplift forces.

It should be noted that a pressure relief system is required regardless of the use of piles, micropiles, or tiedowns, for the purpose of not reducing the passive pressures acting on the sheet piles.

Pile Recommendations

Design recommendations for driven pipe piles, are provided as follows:

- **Ultimate Skin Friction in Compression and Tension.** 2,000 psf below top of bearing unit shown on Figure 3.

- **Ultimate End Bearing in Compression.** 400 ksf (assumes inside-fit cutting shoe and no plug).

- **ASD Minimum Factor Safety.** Use FS = 2.0 for both tension and compression provided that pile capacity should be verified by pile driving analyzer (PDA) and CAPWAP analysis.

Micropiles and Tiedowns

Our preliminary recommendations for gravity-grouted micropiles and tiedowns are as follows:

- **Ultimate Skin Friction in Compression (only micropiles) and Tension.** 2,000 psf below top of bearing unit shown on Figure 3.

- **Ultimate End Bearing in Compression.** End bearing is typically ignored in micropiles.

Secondary or pressure grouting can be used to increase the allowable unit friction. Further discussions regarding tiedowns and micropiles installation, factor of safety, testing, and location could be provided to the design team, if required.

Upper Slab Walls

The top of the slab will be at elevation 10 feet, which is about 3 feet below existing grade. During the super flood phase, the slab will support 17 feet of water, about 1,100 psf, with a net (water stress minus the removed 3 feet of soils) increase of effective pressure of about 725 psf. Because the pontoons will float during the super flood phase, their weights will not be acting on the slab. Based on our engineering analyses, settlement would vary across the upper slab from about 1 inch toward the southeast corner to 6 inches toward the northwest corner.

The major source of settlement in the vicinity of the northwest corner is the compression of the silty soils encountered in boring H-1-02. As discussed previously in the ***Subsurface Conditions*** section, the subsurface conditions encountered in boring H-1-02 could be further verified by cone penetration testing. It should be noted that dewatering for the deep channel would not significantly impact the groundwater level in the vicinity of the northwest corner.

Settlement in the vicinity of the deep channel would result in down drag forces on the sheet pile wall. These forces, however, would be insignificant given the much higher axial capacity of the sheet piles.

Modulus of Subgrade. The slab will be mostly supported on saturated, medium dense to dense Sand/Gravel (Soil Unit 1). Therefore, we recommend a subgrade modulus of 125 pci.

Preloading

Preloading would be an option for reducing settlement. However, we understand that the estimated settlement presented above is considered tolerable. Therefore, preloading is not required.

Slab Settlements Due to Liquefaction

Typically, liquefaction-induced settlement varies from 2 to 5 percent of the thickness of the liquefiable layer (Ishihara and Yoshimine 1992). Based on soil conditions in borings H-5-02 and H-6-02, the estimated settlement due to liquefaction would range from 6 to 10 inches under the 475-year earthquake. Under the 190-year earthquake, local liquefiable zones may induce 2 or 3 inches of settlement.

Pile Foundation for Gate Structure

Foundation Type Alternatives

Open-end, structural steel pipe piles represent the most feasible alternative for supporting the gate structure of the new graving dock when considering the axial and lateral load demands on the piles, and the need to achieve adequate penetration below very dense soils to attain the required pile capacity. Other pile types such as precast concrete or closed end steel pipes were considered but were ruled out because these piles will likely be difficult to be installed to a sufficient depth in the glacial deposits. Shafts are also considered feasible, but are likely more expensive than the driven pile. Shallow foundations are not feasible due to both settlement and bearing capacity concerns.

Discussions with the structural engineer, KPFF, and the results of our preliminary analysis suggest that 24-inch-diameter open-end structural steel pipe piles likely will be used for the project. We understand that vertical piles will be used to resist both the axial and lateral loads.

To achieve the required compressive, uplift, and lateral capacities, the piles should be driven open-ended with and inside-fit cutting shoe. We understand the piles will be structural steel and not filled with reinforced concrete, similar to ferry dock wingwall piles. For drivability, we recommend using a pile section no thinner than ¾-inch unless Hart Crowser performs a wave equation drivability analysis.

Axial Capacity

In general, we understand that the piles will be spaced at least 3-pile diameters on center. The top of the piles is anticipated to be at about elevation -25 feet, or about 40 feet below the existing ground surface. Each pile may be required to resist up to 100 tons in axial tension, and 150 to 200 tons in axial compression.

We recommend that the ultimate compression and tension capacities of 24-inch-diameter pipe piles be computed using the following parameters:

- Ultimate skin friction (in compression and tension):
  - Between elevation –25 and –43 feet, use 200 psf for tension only, ignore resistance for compression to account for settlement-induced downdrag on piles due to static and/or liquefaction
  - Between elevation -43 and -53 feet, use 2,000 psf
  - Below elevation -53 feet, use 3,000 psf
- Ultimate end bearing (in compression):
  - 200 kips (assumes inside-fit cutting shoe and no plug)
- **ASD Minimum Factor Safety.** Use FS = 2.0 for both tension and compression with pile capacity verified by pile driving analyzer (PDA) and CAPWAP analysis on at least 2 to 5 percent of the production piles.
- **LRFD Strength Limit State Resistance Factor.** Use 0.75 for both tension and compression with pile capacity verified by pile driving analyzer (PDA) and CAPWAP analysis on at least 2 to 5 percent of the production piles. Resistance factors for service and extreme limit states, a resistance factor of 1.0 should be used.
- Group effects for axial loads will not be significant providing that piles are spaced at least 2½D.

Laterally Loaded Pile Analyses

The behavior of structural steel pipe piles subjected to lateral loads was evaluated using the computer software LPILE. As noted previously, the sandy Silt (Soil Unit 3) may experience global liquefaction under the 475-earthquake. The p-y curve parameters for single pile under static and post-earthquake cases

are shown in Table 7. For the earthquake case, the soil parameters for the static case can be used with corresponding additional seismic loads.

Group reduction factors for lateral analysis were also incorporated into our analysis, in accordance to the procedure outlined in the *WSDOT Bridge Design Manual*, as revised July 2000. The factors account for pile interaction effects due to proximity and are a function of pile spacing based on pile diameter, D. Assuming a pile spacing of 6-foot on center, or 3 times the pile diameter, a reduction factor of 0.5 for the subgrade reaction modulus was selected (see Table 8). Also, the soil friction angle for each unit was reduced according to the procedure outlined in the bridge manual.

The results of our analysis for the static condition indicate approximately 0.5 inch of lateral pile top deflection when a 50 kip lateral load is applied to the pile top. For the post-earthquake (liquefaction) case, the analyses indicate approximately 0.8 inch of deflection. It should be noted that piles longer than 40 feet likely would be required to meet the axial load demand.

Minimum Tip Elevations

Because the piles must resist axial compression, uplift, and lateral loads, a minimum tip elevation should be specified in the contract as the lowest elevation needed to limit lateral deflection under lateral loading. A minimum tip elevation for uplift requirements based on the ultimate capacities presented above may be deeper than that needed to limit lateral deflection. If so, the deeper minimum tip elevation should be specified, with the provision that the elevation could be modified on the basis of PDA and CAPWAP data during construction. Hart Crowser should be contacted after pile design is finalized to assess the potential for overdriving to reach the minimum tip elevation.

Walls Attached to the Gate Foundation

Based on the design information provided by KPFF, two concrete walls along the east and west sides of the gate will be attached to its foundation. The top of the wall along the east will be at elevations 14 feet with a total height of about 30 feet. Along the west side, the top of the wall will at elevation 21 feet with a total height of about 37 feet. Temporary cofferdams will be used to construct these walls.

These walls will be subjected to lateral hydrostatic and soils pressures. On-site granular soils can be used to backfill behind the walls. Static and seismic earth pressures will depends on the level of compaction of these soils, and on whether

the top of the wall is free to rotate (yielding wall) or not (non-yielding). Table 9 presents our recommended earth pressures assuming:

- Two levels of compaction, 90 and 95 percent of the maximum dry density as determined by WSDOT Test Method No. 606 (for soils with at least 30 percent gravel) or by AASHTO T99 (for soils with less than 30 percent gravel);
- Two levels of earthquakes as described previously; and
- Yielding and non-yielding wall.

It should be noted that the seismic pressures for non-yielding walls presented in Table 9 are less than what AASHTO recommends as a *first approximation*. It has been observed after major earthquakes that seismic pressures are typically overestimated, particularly for rigid walls. Accordingly, in our opinion, the seismic earth pressure values presented in Table 9 are quite reasonable given the significance of the graving dock as compared to other structures such as bridges.

For surcharge loads, cases illustrated on Figure 11 and 13 are only applicable for yielding walls. For non-yielding walls, the surcharge lateral pressures are greater by a factor of **1.4**.

RECOMMENDED ADDITIONAL GEOTECHNICAL EXPLORATIONS

The variable subsurface conditions and revised layout of the graving dock may warrant additional explorations in the following areas to further characterize the subsurface conditions at specific locations within the site.

- **Graving Dock Gate.** Boring H-4-02 in the vicinity of the gate was terminated at a depth of 78 feet, which corresponds to elevation –63 feet. Based on our recommended ultimate pile skin friction, the piles would have to be driven below the bottom of boring H-4-02, to achieve the required tension capacity. Although we do not anticipate ground conditions to change beyond the current boring depth, a deeper boring may be necessary to avoid potential contractor claims.

- **Northwest Corner of Upper Slab.** As mentioned previously, the zero blow counts of the sandy Silt encountered in boring H-1-02 appear to be anomalous compared to other borings at the site. The presence of such low blow count would affect the depth of the sheet pile and settlement.

Depending on the impact of subsurface uncertainties on the design of the graving dock, additional borings may or may not be warranted. For example, if the upper slab can sustain significant settlement as much as 6 inches throughout the super flood periods and the depth of sheet pile is governed by seepage, then no additional boring would be necessary in the vicinity of boring H-01-02.

CONSTRUCTION CONSIDERATIONS

- Temporary shoring and/or slopes will be required during construction of the pile caps. The design and construction of temporary shoring/slopes should be the responsibility of the contractor, but the need for the shoring should be indicated in the plans. Groundwater will be encountered in the excavations and seals will be required. Sumps and dewatering should be designed in consideration of the dewatering section of this report.

- Test piles should be driven in accordance with Section 6-05.3(10) of the Standard Specs. A minimum of one test pile should be driven at each end of the gate pile cap. Test piles should be monitored using PDA and analyzed using CAPWAP prior to commencing production pile driving.

Other construction considerations will discussed in the final report following advances in current the feasibility studies.

LIMITATIONS OF OUR WORK

We completed this work in general accordance with our Task Assignment Y-7672, No. 4 issued by WSDOT. Our draft report is for the exclusive use of WSDOT and their design consultants for specific application to the subject site. We completed this study in accordance with generally accepted geotechnical practices for the nature and conditions of the work completed in the same similar localities at the time the work was performed. We make no other warranty, express or implied.

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Tables

Table 1
Discharge Water Quality from PW-02 during Pumping Test

Date	Time	Temperature in degrees C	EC[a] in mS/cm[b]	% Salinity	pH	Turbidity in NTU[c]	Dissolved Oxygen in mg/L
10/31/2002	10:00	11.80	3.68	0.18	8.49	20	5.69
	10:43	13.00	2.58	0.13	8.16	138	4.34
	11:43	12.90	2.69	0.14	7.85	14	4.46
	13:20	13.00	2.82	0.13	8.19	<10	3.50
	15:30	13.00	2.92	0.15	8.03	<10	3.21
	17:00	12.60	2.95	0.14	8.02	<10	3.85
	18:00	12.40	2.95	0.14	7.98	<10	4.27
11/1/2002	1:00	11.40	2.99	0.14	7.76	<10	3.40
	5:00	12.10	3.02	0.14	7.68	<10	2.33
	7:30	12.50	3.00	0.15	7.75	<10	2.85
	9:45	12.80	3.00	0.16	7.66	<10	2.75

Notes:
a- EC = Electrical Conductivity
b- mS/cm = milliSiemens per Centimeter
c- NTU = Nephelometric Turbidity Unit

Table 2 - Soil Parameters - Deep Channel
Wall Segment C-1

Soil Unit	Layer	Depth to Top of Layer[a] in Feet	Total Unit Weight in pcf	Friction Angle in Degrees	K_a	K_p (design)	Residual Shear Strength in psf (liquefiable soils- 475-year event)[b]
-	Compacted Granular Fill	0 (at ground surface)	130 (123)[c]	37 (34)[c]	0.23 (0.26)[c]	4.5 (3.9)[c]	No Liquefaction
1	Existing Sand/Gravel	6	125	35	0.25	4.0	No Liquefaction
2	Loose to Medium Dense Silty Sand	22	120	32	0.28	3.5	***570***
5	Dense Sand and Hard Sandy Silt	50	135	38	0.22	5.0	No Liquefaction

Notes:
Refer to the text for earthquake loading and assumptions.
For earth pressure diagrams (operational and earthquake cases and post-earthquake), refer to Figures 10 and 11
a- Assuming top of slab is at elevation 10.0 feet
b- Soils at this location would not experience global liquefaction under an earthquake with 10% in 20 years (190-year event)
c- The first value corresponds to 95 percent compaction and the second value (in parenthesis) corresponds to 90 percent compaction.

Table 3 - Soil Parameters - Deep Channel
Wall Segment C-2

Soil Unit	Layer	Depth to Top of Layer in Feet	Total Unit Weight in pcf	Friction Angle in Degrees	Ka	Kp (design)	Residual Shear Strength in psf (liquefiable soils- 475-year event)[a]
-	Compacted Granular Fill	0 (at ground surface)	130 (123)[b]	37 (34)[b]	0.23 (0.26)[b]	4.5 (3.9)[b]	No Liquefaction
1	Existing Sand/Gravel	6	125	35	0.25	4.0	No Liquefaction
2	Loose to Dense Silty Sand	22	125	35	0.25	4.0	No Liquefaction
3	Very Soft to Medium Stiff Sandy Silt	35	115	28	0.33	2.6	**500**
5	Hard Sandy Silt	55	135	38	0.22	5.0	No Liquefaction

Note:
Refer to the text for earthquake loading and assumptions.
a- Soils at this location would not experience global liquefaction under an earthquake with 10% in 20 years (190-year event)
b- The first value corresponds to 95 percent compaction and the second value (in parenthesis) corresponds to 90 percent compaction

Table 4 - Soil Parameters - Deep Channel
Wall Segment C-3

Soil Unit	Layer	Depth to Top of Layer in Feet	Total Unit Weight in pcf	Friction Angle in Degrees	K_a	K_p (design)
-	Compacted Granular Fill	0 (at ground surface)	130 (123)[a]	37 (34)[a]	0.23 (0.26)[a]	4.5 (3.9)[a]
1	Existing Sand/Gravel	6	130	37	0.23	4.5
2	Loose to Medium Dense Silty Sand	20	125	35	0.25	4.0
5	Dense Sand and Hard Sandy Silt	50	135	38	0.22	5.0

Note:
Refer to the text for earthquake loading and assumptions
a- The first value corresponds to 95 percent compaction and the second value (in parenthesis) corresponds to 90 percent compaction

Table 5 - Soil Parameters - Upper Slab
Wall Segment U-1

Soil Unit	Layer	Depth to Top of Layer in Feet	Total Unit Weight in pcf	Friction Angle in Degrees	K_a	K_p (design)
-	Berm (outside the dock)	8 feet high above ground surface	130 (123)[a]	37 (34)[a]	0.23 (0.26)[a]	4.5 (3.9)[a]
1	Existing Sand/Gravel	0.0 (ground surface)	120	32	0.28	3.5
2	Loose to Medium Dense Silty Sand	20	120	32	0.28	3.5 (2.6)[b]
3	Very Soft Sandy Silt	32	110	26	0.36	2.2 (1.6)[b]
4	Medium Dense Sand	60	120	32	0.28	3.5 (2.6)[b]
5	Dense Sand and Hard Sandy Silt	68	135	38	0.22	5.0 (3.8)[b]

Notes:
Refer to the text for earthquake loading and assumptions.
a- The first value corresponds to 95 percent compaction and the second value (in parenthesis) corresponds to 90 percent compaction
b- The K_p in parenthesis is reduced to account for the limited width of the earth berm and should be used where the earth berm is accounted for

Table 6 - Soil Parameters - Upper Slab
Wall Segment U-2

Soil Unit	Layer	Depth to Top of Layer in Feet	Total Unit Weight in pcf	Friction Angle in Degrees	K_a	K_p (design)
-	Berm (outside the dock)	8 feet high above ground surface	130 (123)[a]	37 (34)[a]	0.23 (0.26)[a]	4.5 (3.9)[a]
1	Existing Sand/Gravel	0.0 (ground surface)	125	35	0.25	4.0
2	Medium Dense Sand	20	120	32	0.28	3.5 (2.6)[b]
5	Dense Sand and Hard Sandy Silt	52	135	38	0.22	5.0 (3.8)[b]

Notes:
Refer to the text for earthquake loading and assumptions
a- The first value corresponds to 95 percent compaction and the second value (in parenthesis) corresponds to 90 percent compaction
b- The K_p in parenthesis is reduced to account for the limited width of the earth berm and should be used where the earth berm is accounted for

Table 7 - P-y Curve Parameters Used in LPILE Analysis

Top of pile: -25 feet

				STATIC ANALYSIS				
Soil Layer	Bottom of Layer Elevation in Feet	Soil Type	Soil Type (KSOIL)	Effective Unit Weight of Soil in pci	Cohesion in psi	Axial Strain ε_{50}	Friction Angle in Degrees[a]	Modulus of Subgrade Reaction in pci[b]
1	-43	SAND	4	0.03	0 (500)[d]	N/A	28	25 (5)[c]
2	-65	SAND	4	0.04	0	N/A	38	125

Notes

a- The friction angle presented in this table should be reduced to account for group effect as outlined in the 2000 WSDOT Bridge Design Manual

b- The modulus of subgrade reaction presented in this table should be reduced to account for group effect. The group effect reduction factors are presented in Table 8

c- The value in parenthesis (5) should be used for post-earthquake case

d- The post-liquefaction residual strength of soil layer 2 is 500 psf. Due to the low overburden stress, shear strength calculated using friction angle is less than the residual strength. Therefore, the friction angle of 28 degrees is recommended for the LPILE analysis of the liquefaction case.

Table 8 - Group Reduction Factors for Lateral Analysis

Pile Spacing[a]	Group Reduction Factor
6D	0.9
5D	0.8
4D	0.65
3D	**0.5**
2D	0.4

Note:
a- As a function of pile diameter, D.

Table 9 - Earth Pressures for Walls Attached to the Gate Foundation

Compaction Level (Modified Proctor)	Wall Case	Static in pcf[a]	Seismic Pressure in psf[b] (in terms of H[c])	
			190-Year earthquake	475-Year earthquake
90 Percent	Yielding	32 (16)[d]	2.5	5.4
	Non-Yielding	54 (27)[d]	5.4	13
95 Percent	Yielding	30 (15)[d]	2.4	5.2
	Non-Yielding	52 (26)[d]	5.2	12

Notes:

a- Static Equivalent Density

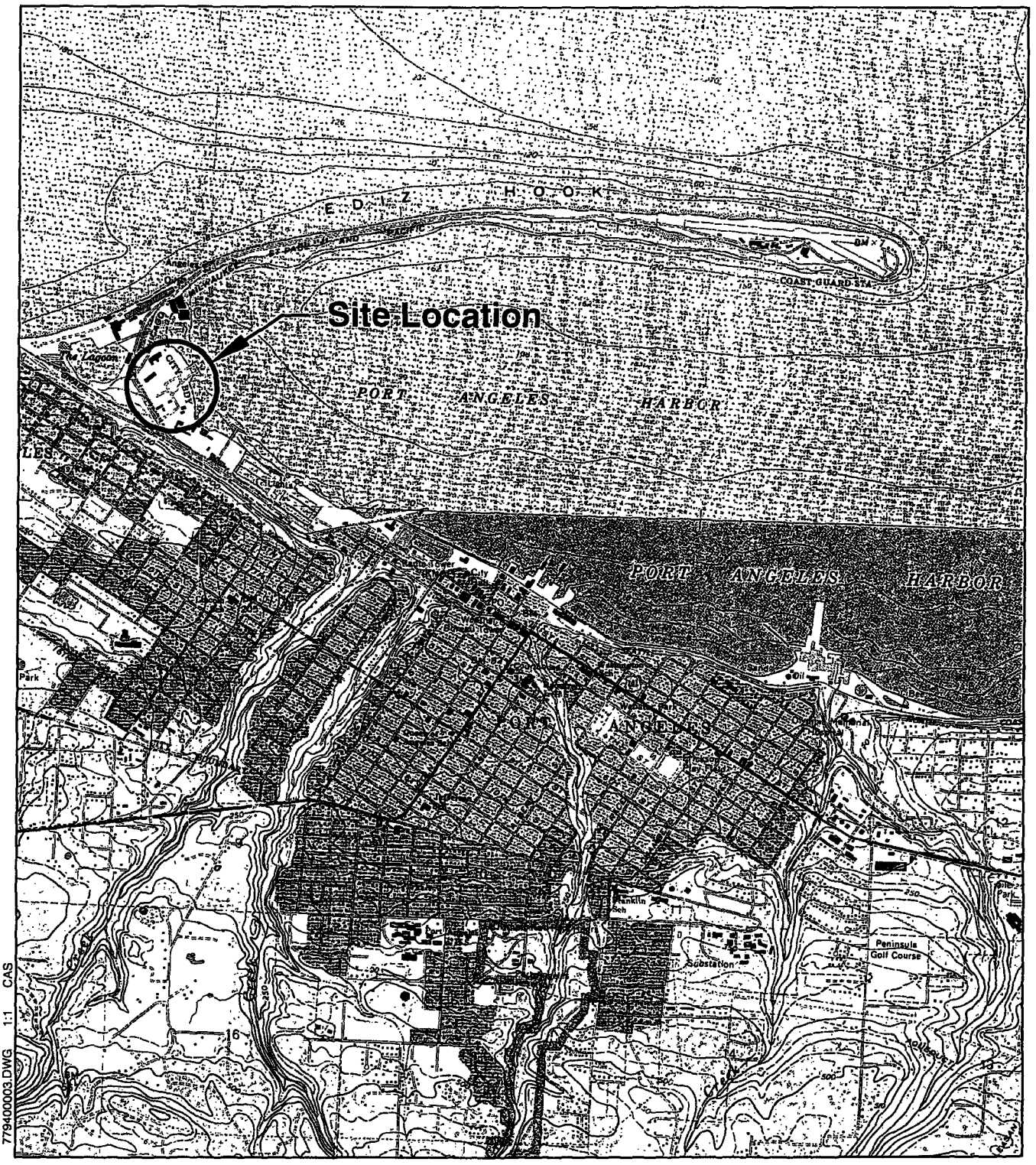
b- Seismic Uniform Earth Pressure

c- H = Wall Height in Feet

d- The first value corresponds to soils above groundwater and the second value (in parenthesis) corresponds to soils below groundwater

Figures

Vicinity Map



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Note: Base map prepared from USGS 7.5-minute quadrangle maps Port Angeles, and Ediz Hook, Washington.

0 3,000 4,000

Scale in Feet

N

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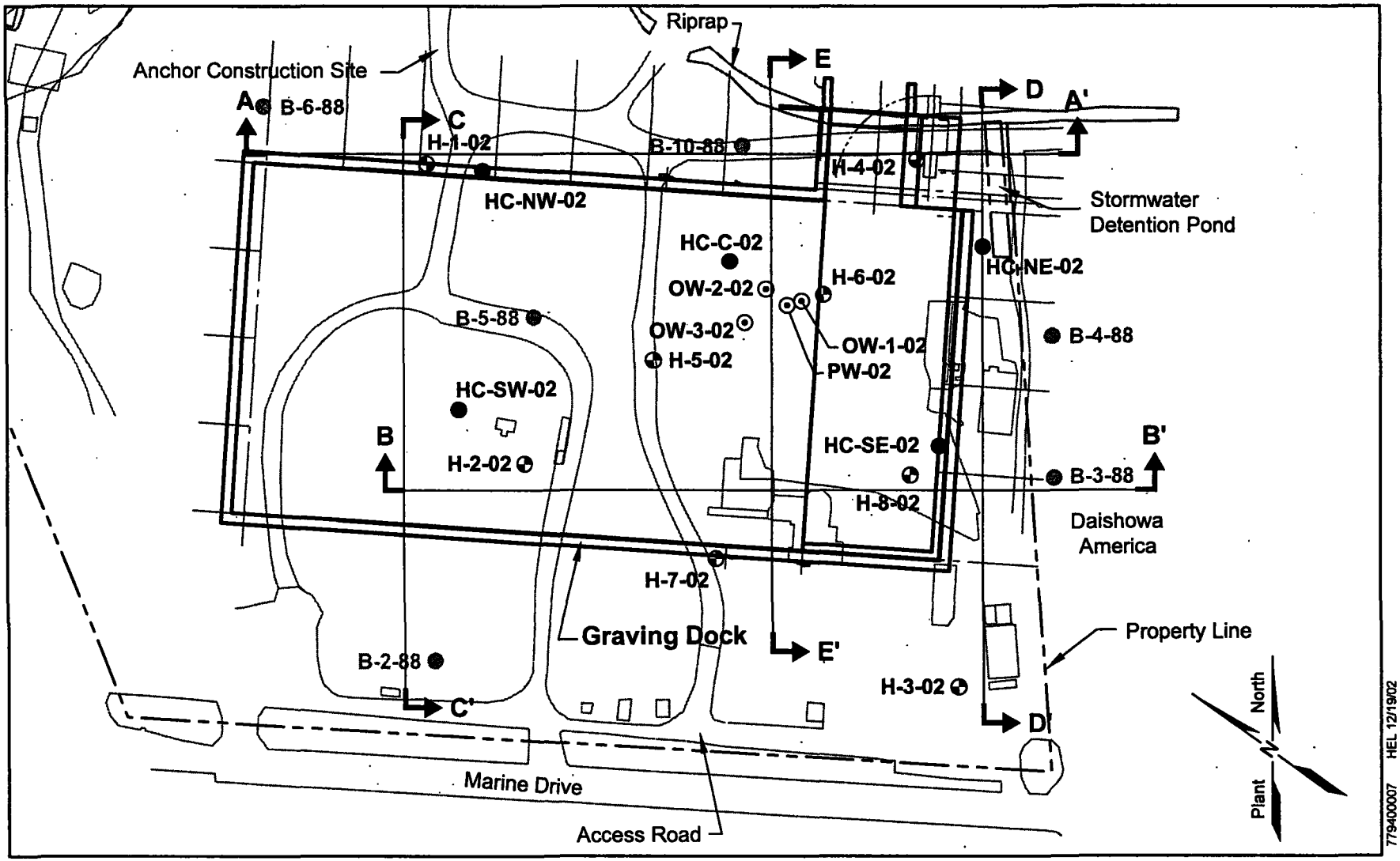
Figure 1

Site and Exploration Plan



Notes:
1) Current exploration locations were surveyed by WSDOT except geotechnical boring H-8-02 (refer to Appendix A for explanation).
2) Base map including the location of the graving dock was provided by KPFF dated 12-10-02.

Exploration Location and Number

H-1-02	Geotechnical Boring (Current Study)
OW-2-02	Wells (Current Study)
HC-SW-02	Environmental Boring (Current Study)
B-2-88	Environmental Boring (1988 Study)

B B' Cross Section Location and Designation

0 200 400
Scale in Feet

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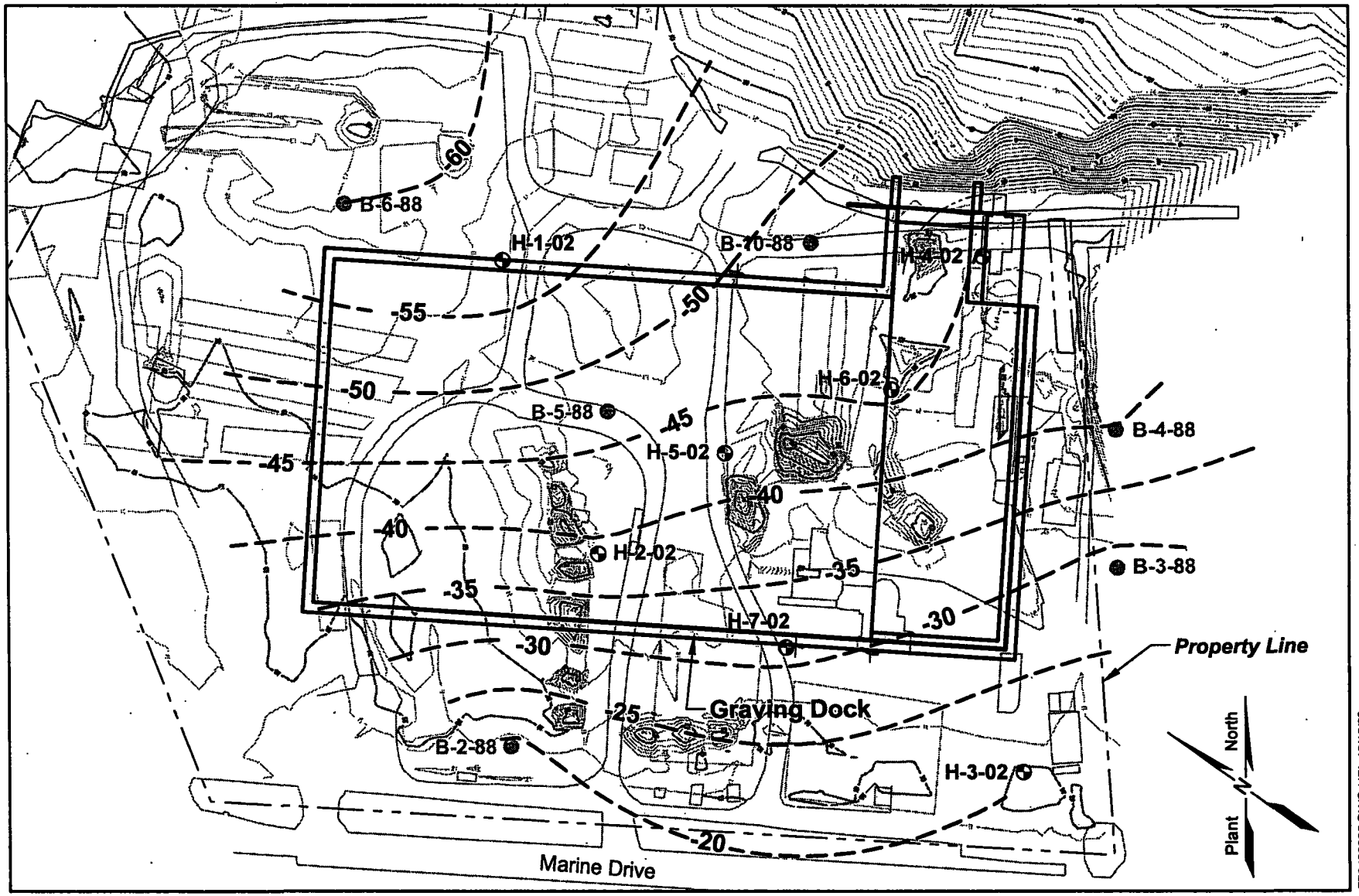
Figure 2

Top of Bearing Layer Elevation Contour Map



Note: Elevation contours are based on borings shown on this figure.

H-3-02 ⊕ Exploration Location and Number (Current Study)

B-2-88 ● Exploration Location and Number (1988 Study)

--40-- Elevation Contour in Feet of Top of Bearing Layer

0 200 400
Scale in Feet

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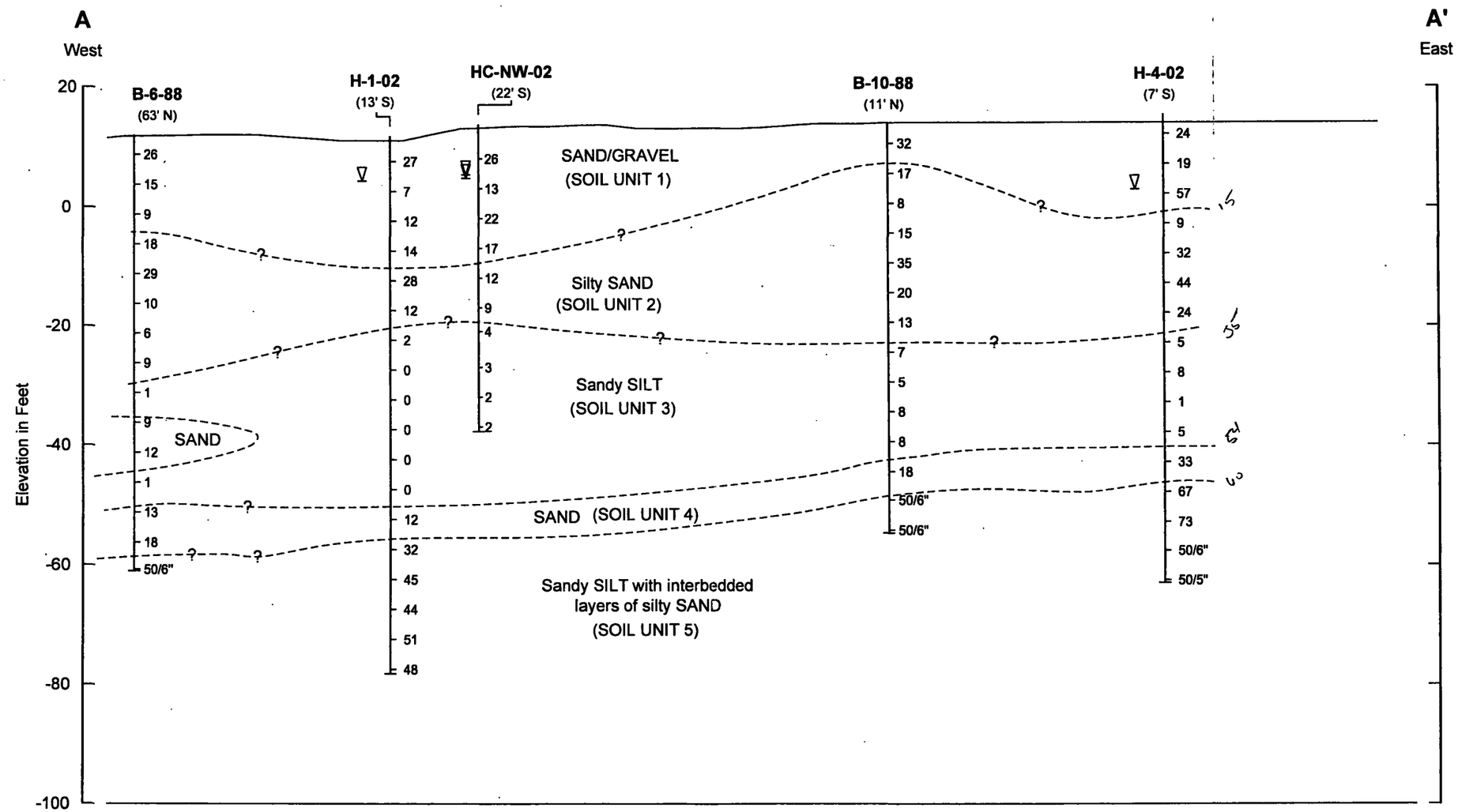
Figure 3

Cross Section A-A'



Legend:

HC-1-02 (13' S) — Exploration Number (Offset Distance and Direction)

Exploration Location

Water Level

9 — Standard Penetration Resistance in Blows per Foot

Horizontal Scale in Feet: 0, 100, 200

Vertical Scale in Feet: 0, 20, 40

Vertical Exaggeration x 5

Note:
Contacts between soil units are based upon interpolation between borings and represent our interpretation of subsurface conditions based on currently available data.

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Figure 4

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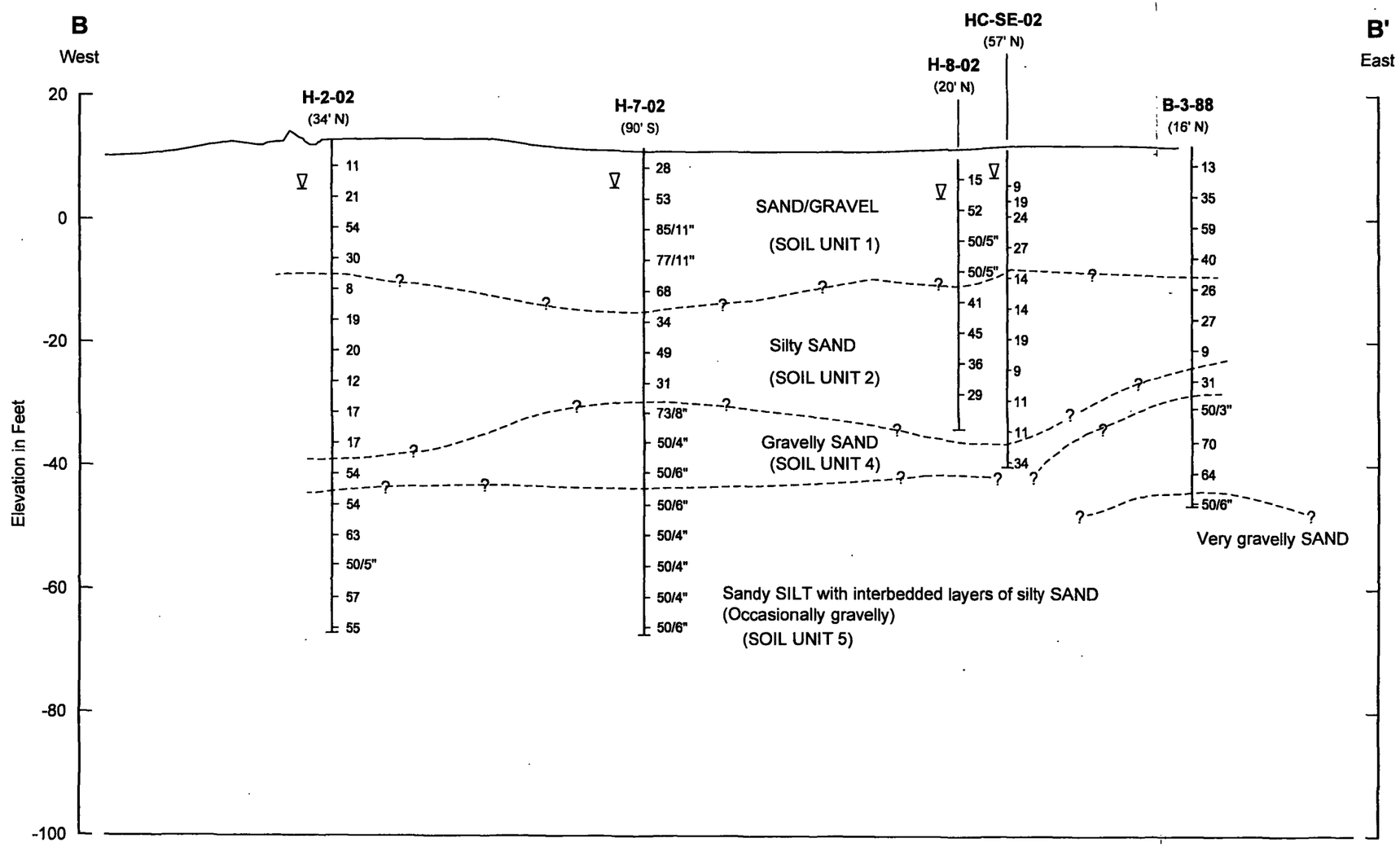
Cross Section B-B'



Legend:

HC-2-02 (34' N) — Exploration Number (Offset Distance and Direction)

Exploration Location

Water Level

9 — Standard Penetration Resistance in Blows per Foot

Horizontal Scale in Feet: 0 100 200

Vertical Scale in Feet: 0 20 40

Vertical Exaggeration x 5

Note:
Contacts between soil units are based upon interpolation between borings and represent our interpretation of subsurface conditions based on currently available data.

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Figure 5

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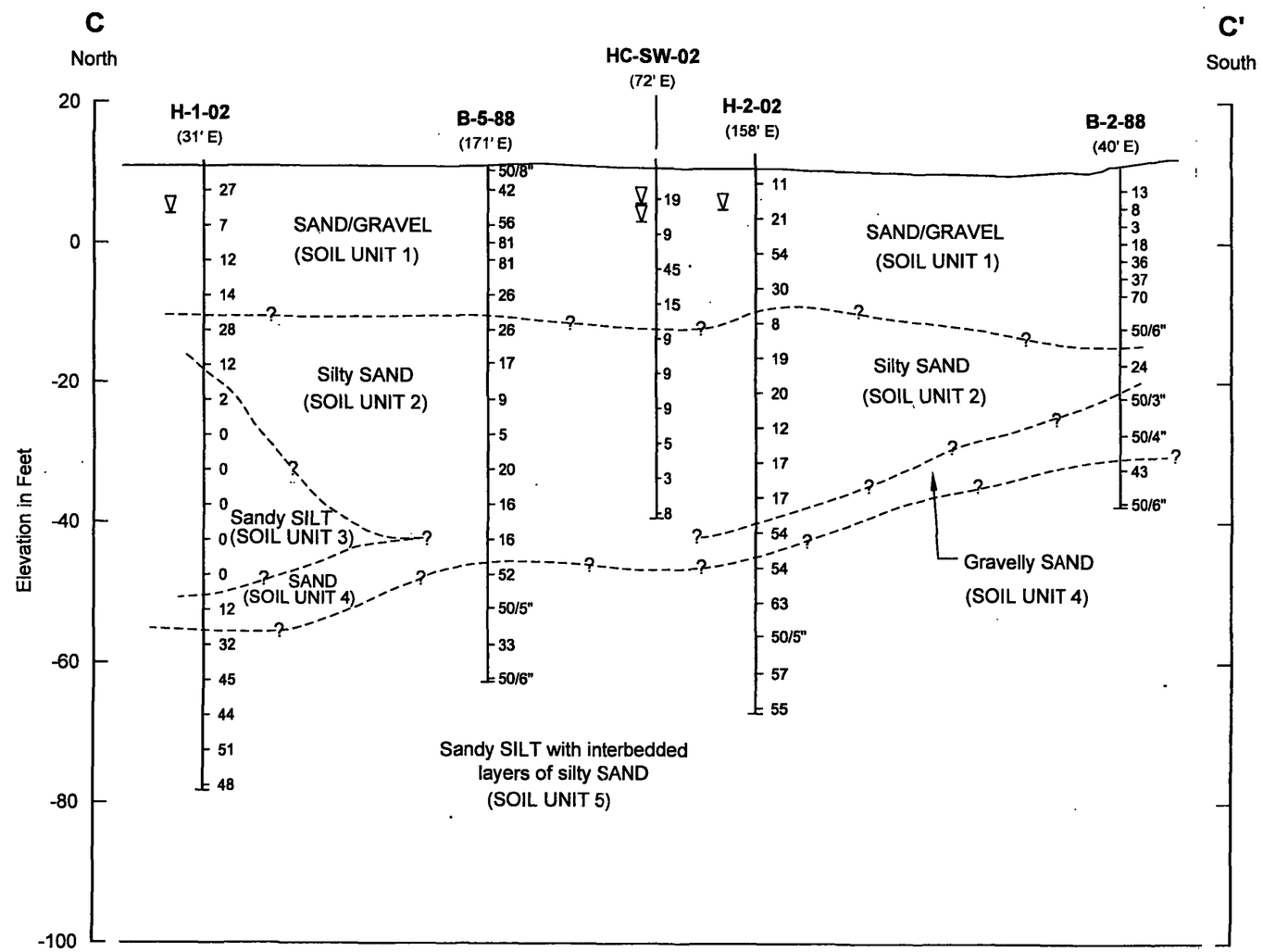
Cross Section C-C'



Legend:

HC-1-02 (31' E) — Exploration Number (Offset Distance and Direction)

Exploration Location

Water Level

9 — Standard Penetration Resistance in Blows per Foot

Horizontal Scale in Feet: 0 100 200

Vertical Scale in Feet: 0 20 40

Vertical Exaggeration x 5

Note:
Contacts between soil units are based upon interpolation between borings and represent our interpretation of subsurface conditions based on currently available data.

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Figure 6

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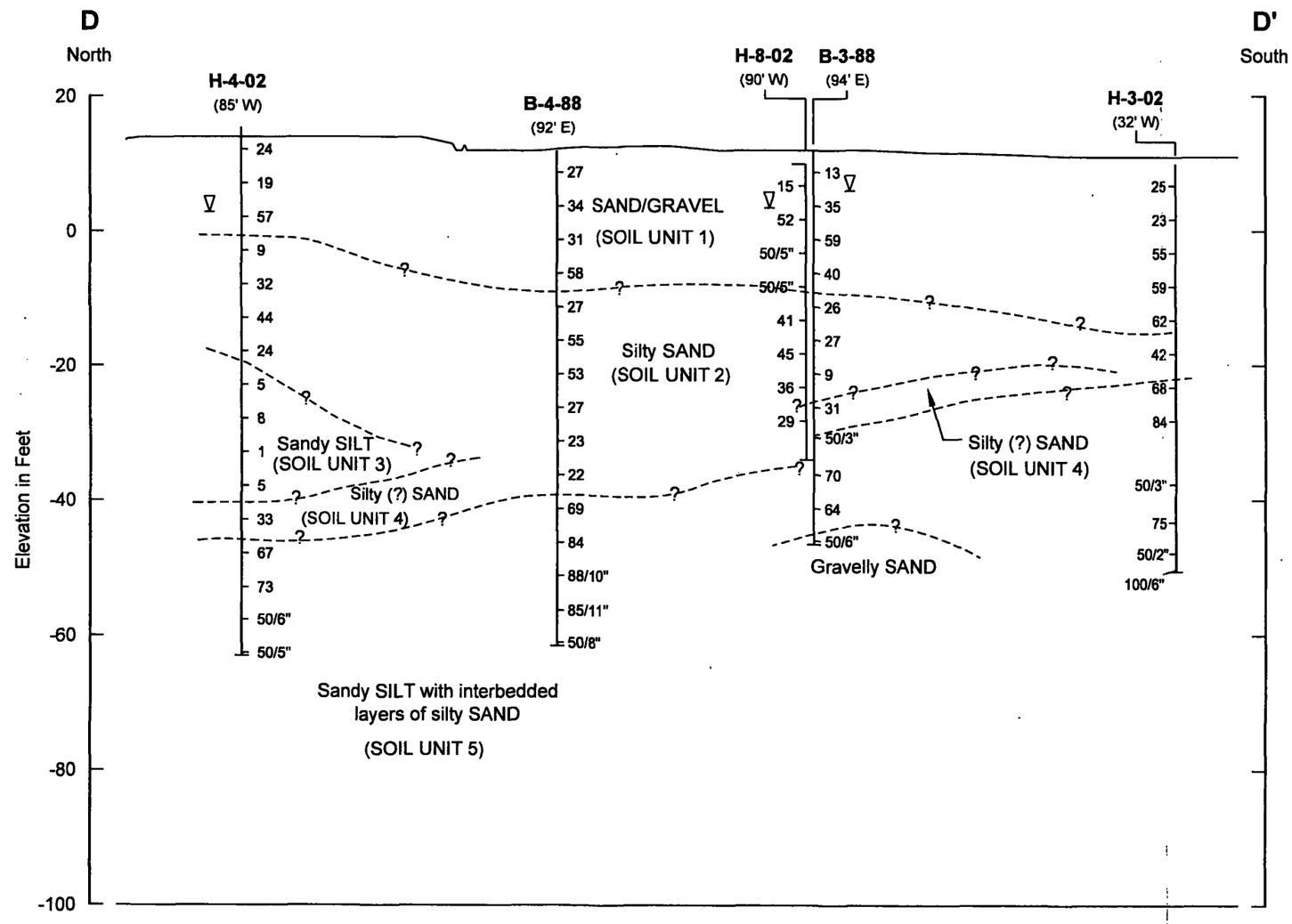
Cross Section D-D'



Legend:

HC-4-02 (85' W) — Exploration Number (Offset Distance and Direction)

Exploration Location

Water Level

9 — Standard Penetration Resistance in Blows per Foot

Horizontal Scale in Feet: 0 — 100 — 200

Vertical Scale in Feet: 0 — 20 — 40

Vertical Exaggeration x 5

Note:
Contacts between soil units are based upon interpolation between borings and represent our interpretation of subsurface conditions based on currently available data.

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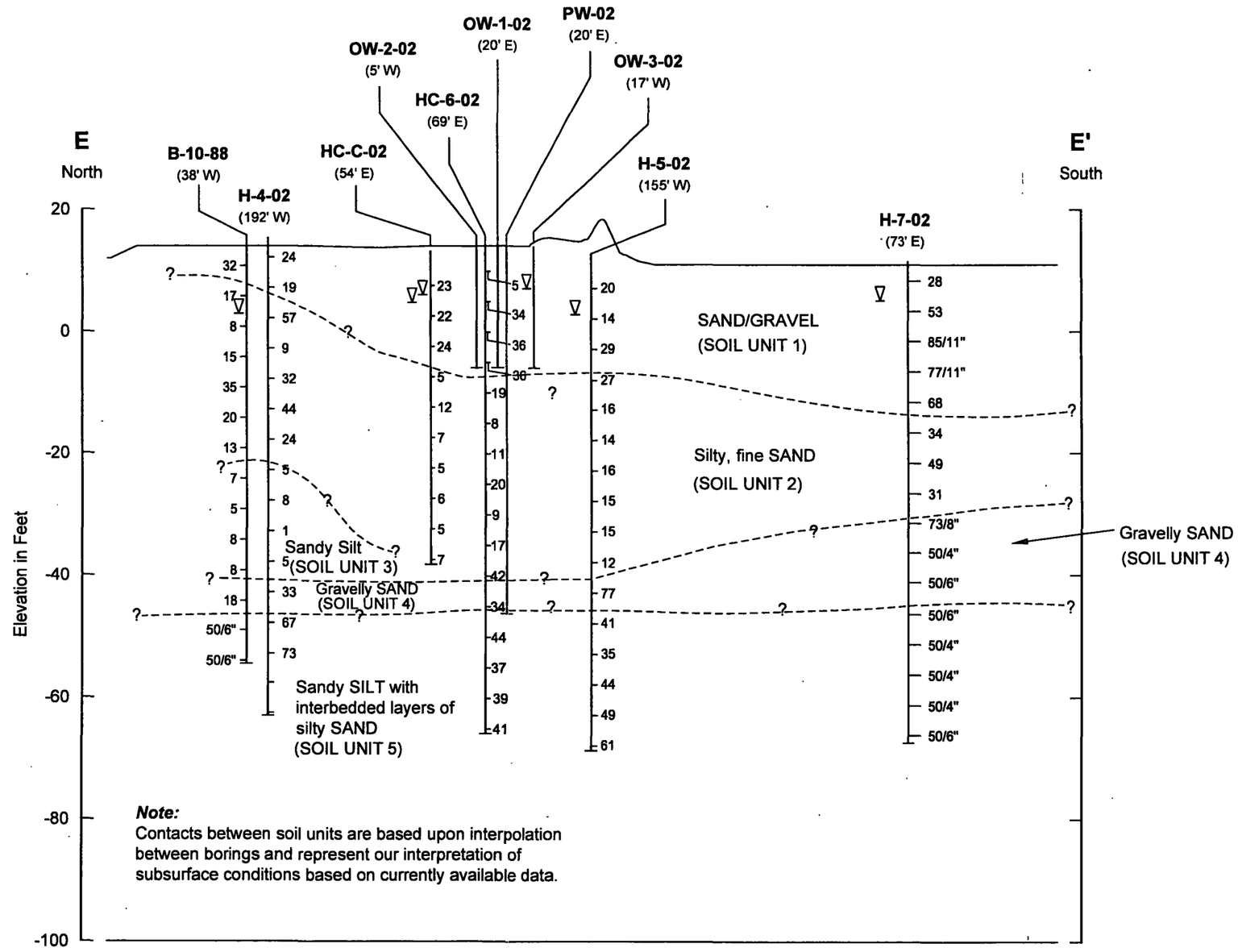
Figure 7

Cross Section E-E'



Note:
Contacts between soil units are based upon interpolation between borings and represent our interpretation of subsurface conditions based on currently available data.

Legend:

HC-7-02 Exploration Number
(73' E) (Offset Distance and Direction)

Exploration Location

Water Level

9 Standard Penetration Resistance in Blows per Foot

Horizontal Scale in Feet
0 100 200
0 20 40
Vertical Scale in Feet
Vertical Exaggeration x 5

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Figure 8

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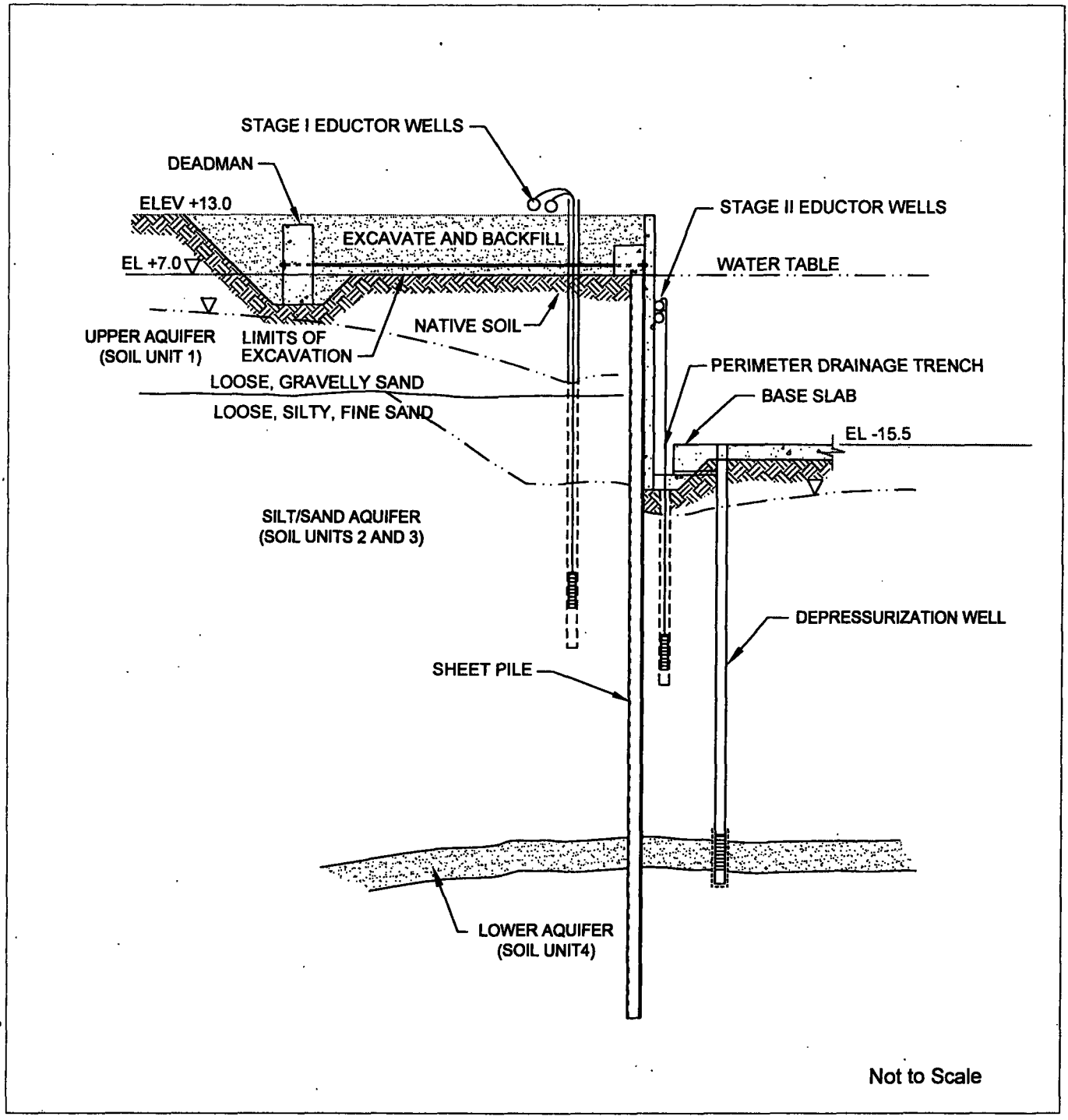
Conceptual Dewatering Schematic



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Note: Base map prepared from drawing provided by KPFF Consulting Engineers, named "Pagd-exihibit-3.dwg", dated October 22, 2002.

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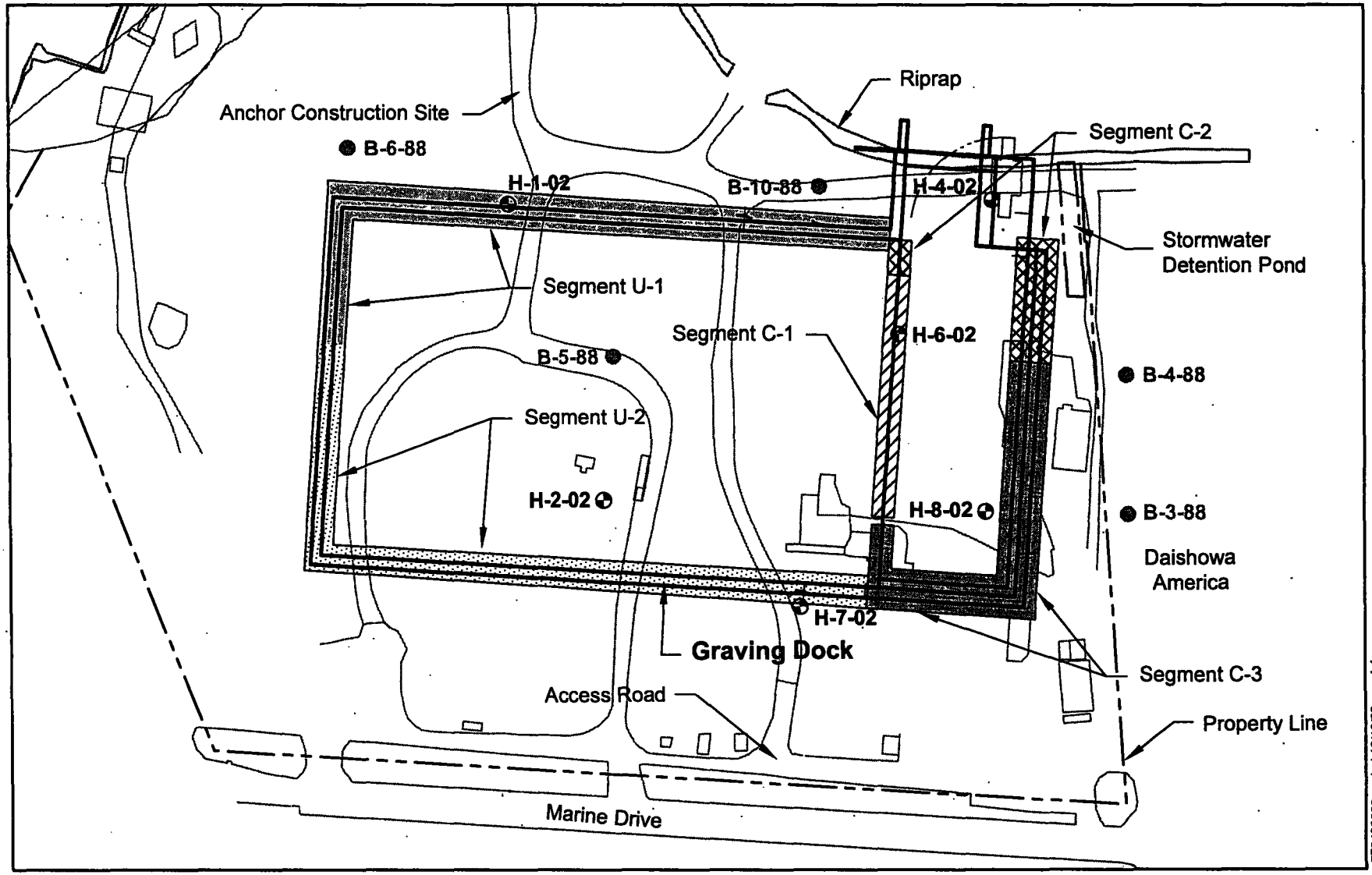
Figure 9

Wall Segments Corresponding to Different Earth Pressures



Note: Subsurface profiles for the different wall segments are mainly based on borings shown on this figure.

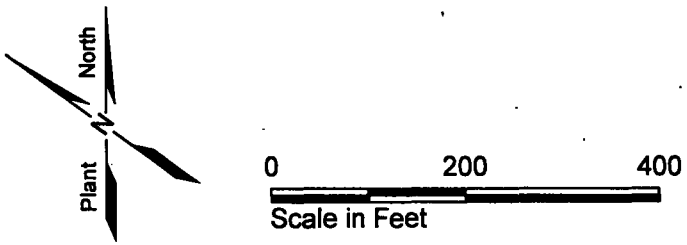


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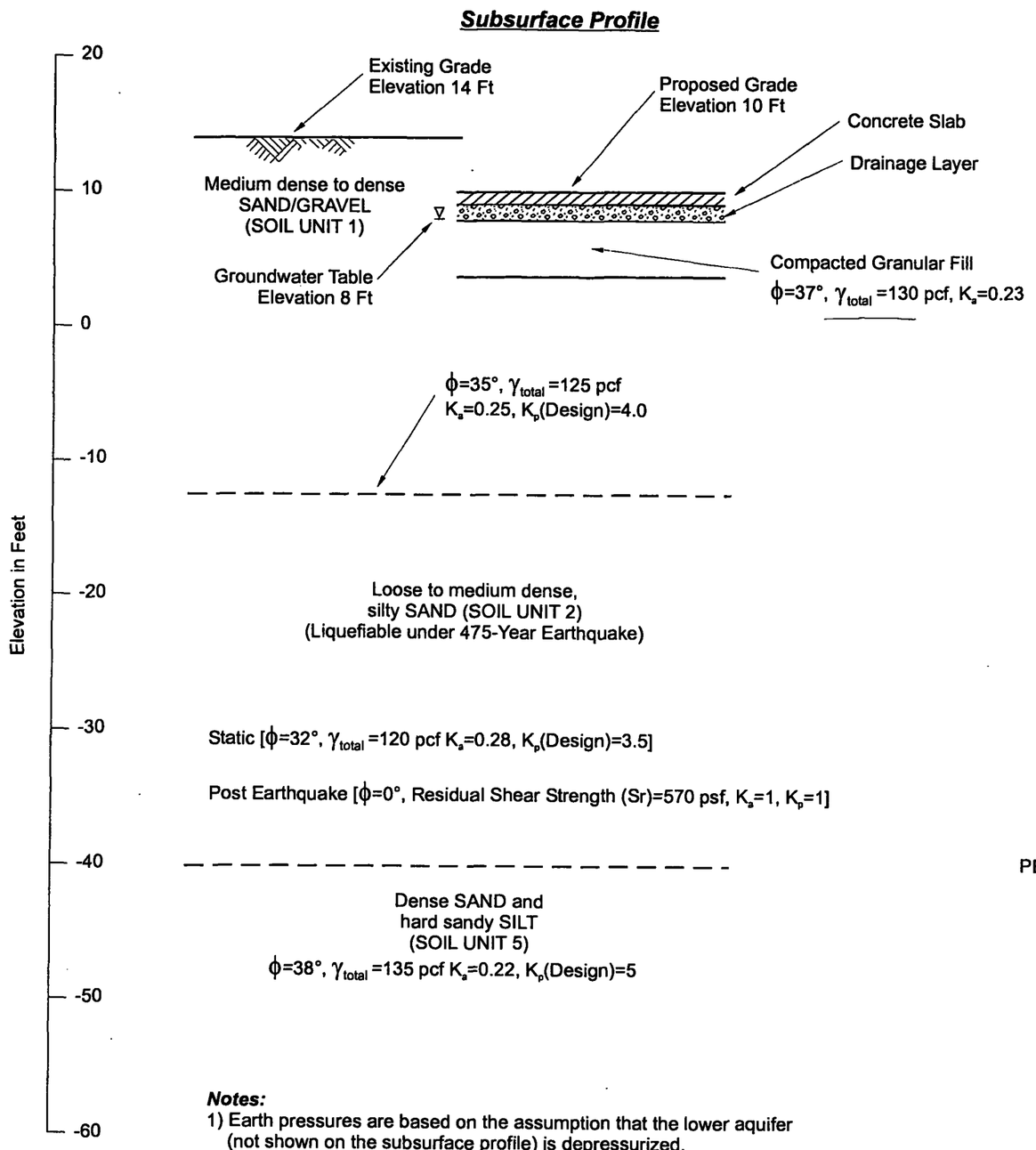
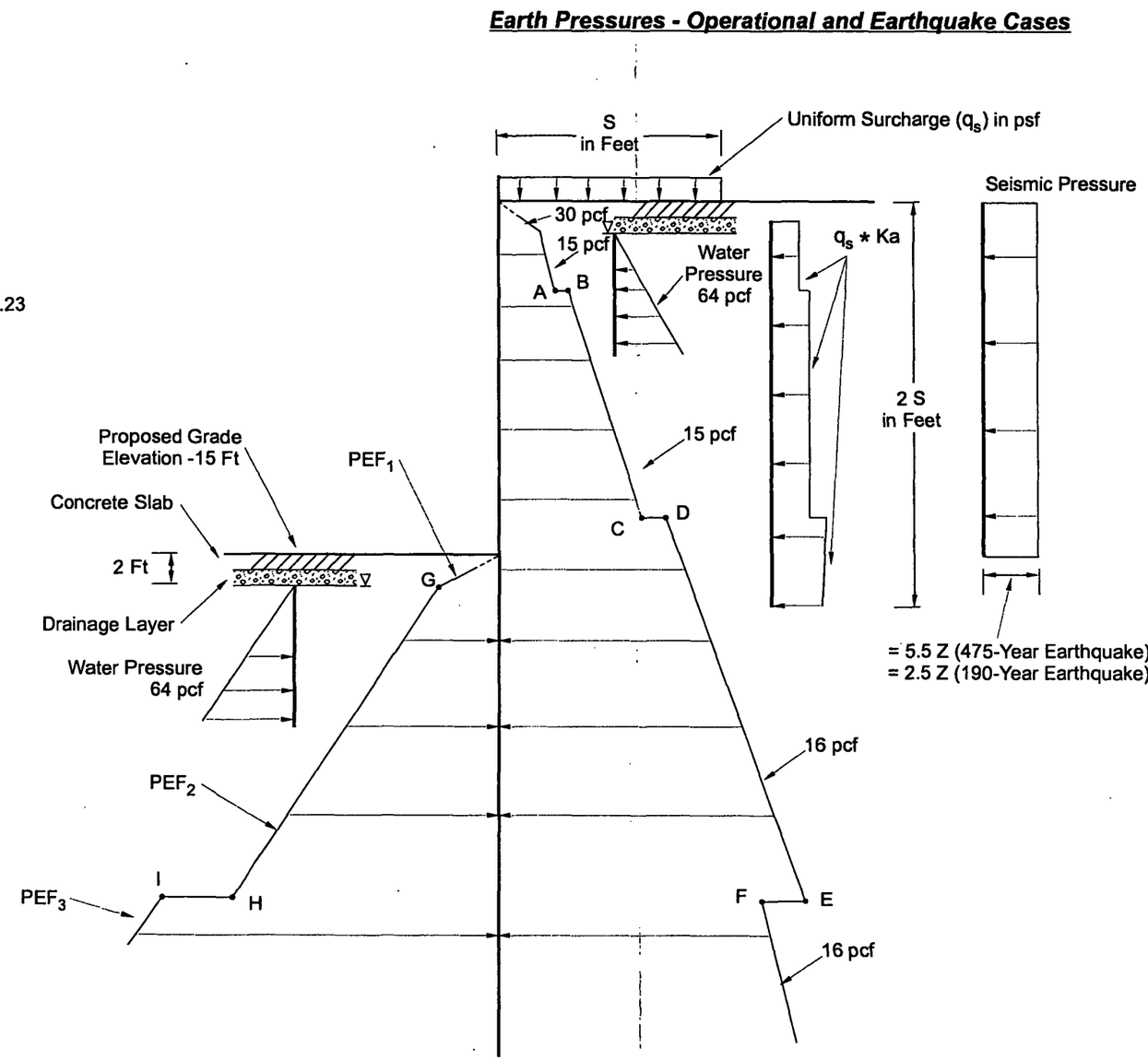
Figure 10

Subsurface Profile and Earth Pressure (Operational and Earthquake)
Segment C-1





Passive Equivalent Fluid Density
(PEF) in pcf

	Static	190-Year Earthquake	475-Year Earthquake
PEF_1	580	580	790
PEF_2	196	196	265
PEF_3	355	355	530

Earth Pressure (EP):

at "B"=EP at "A" * 0.25/0.23

at "D"=EP at "C" * 0.28/0.25

at "F"=EP at "E" * 0.22/0.28

at "I"=EP at "H" * 5/3.5

Notes:
1) Earth pressures are based on the assumption that the lower aquifer (not shown on the subsurface profile) is depressurized.
2) Location of segment C-1 is shown on Figure 10.
3) Subsurface profile is based on boring H-6-02.
4) For post-earthquake (liquefaction) case, refer to Figure 12.
5) For other surcharge loading cases, refer to Figure 13.
6) Earth pressures shown on this figure are only applicable to cantilever walls and walls supported by single level of tiebacks or deadman.

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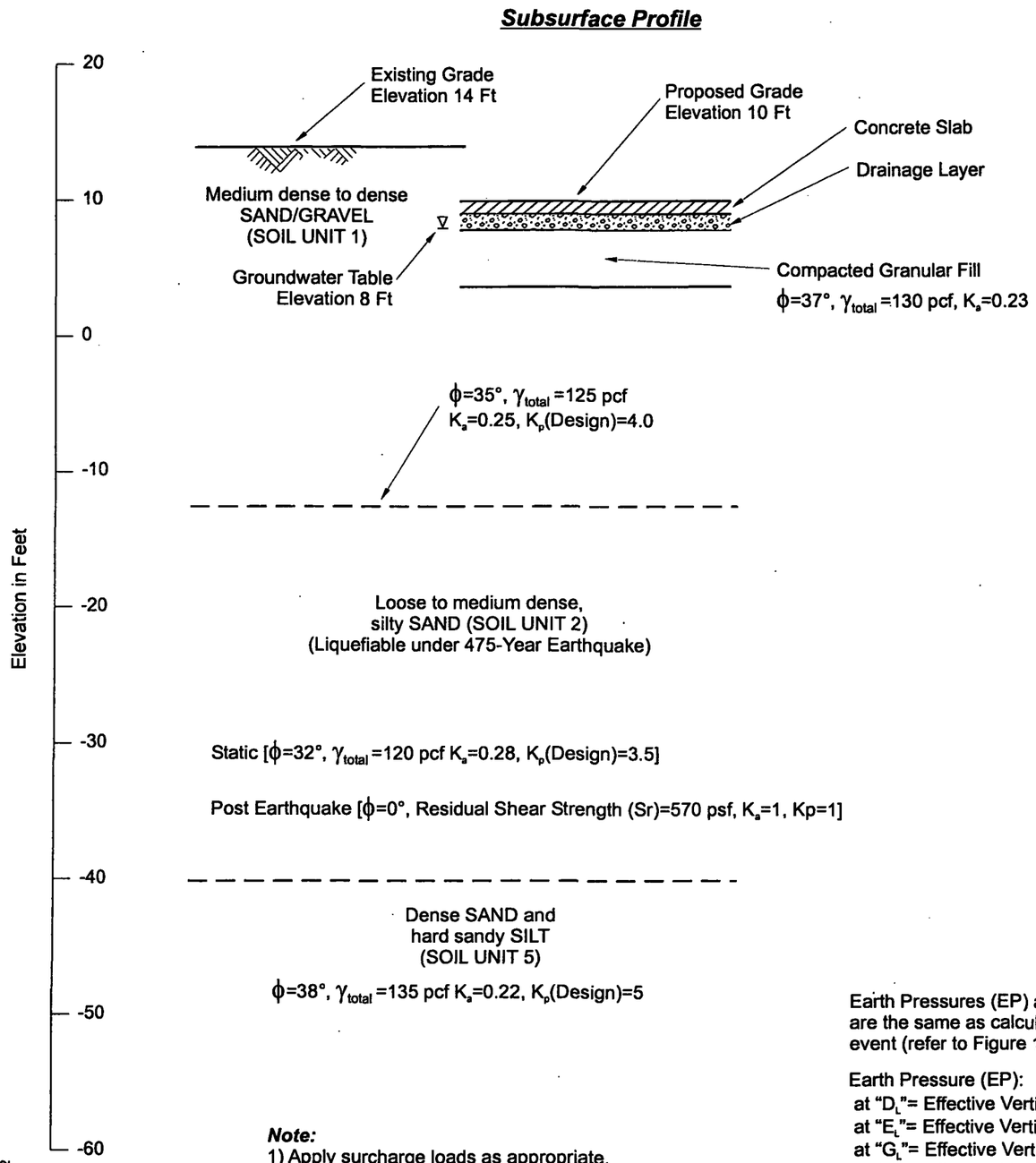
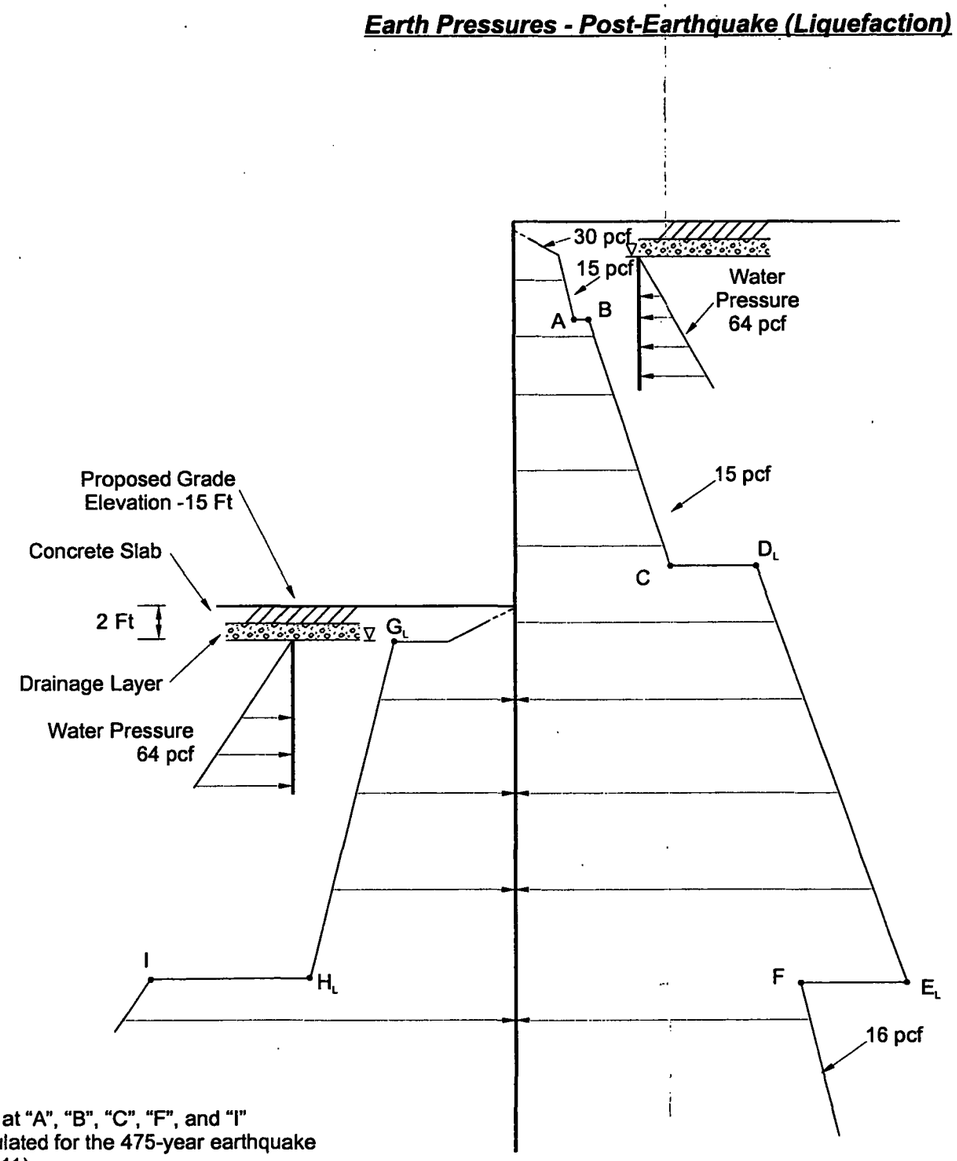
Figure 11

Subsurface Profile and Earth Pressure (Post-Earthquake)
Segment C-1





Earth Pressures (EP) at "A", "B", "C", "F", and "I" are the same as calculated for the 475-year earthquake event (refer to Figure 11).

Earth Pressure (EP):
at "D_L"= Effective Vertical Stress at "D_L"-2 * Sr (where Sr=Residual Shear Strength = 570 psf) ≥ EP at "D" on Figure 9
at "E_L"= Effective Vertical Stress at "E_L"-2 * Sr ≥ EP at "E" on Figure 9
at "G_L"= Effective Vertical Stress at "G_L"+2 * Sr ≤ EP at "G" on Figure 9
at "H_L"= Effective Vertical Stress at "H_L"+2 * Sr ≤ EP at "H_L" on Figure 9

Note:
1) Apply surcharge loads as appropriate.
2) Earth pressures are based on the assumption that the lower aquifer (not shown on the subsurface profile) is depressurized.

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Figure 12

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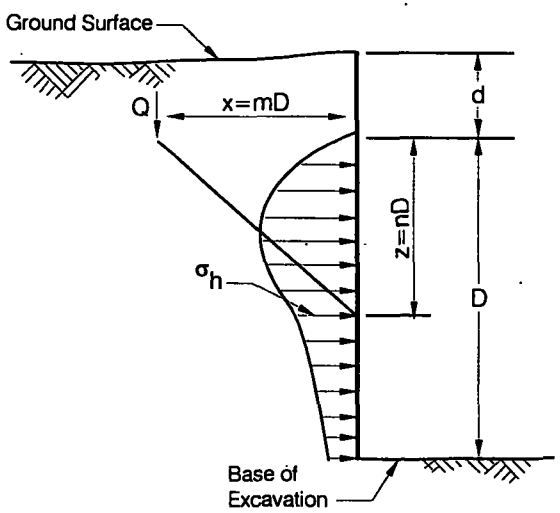
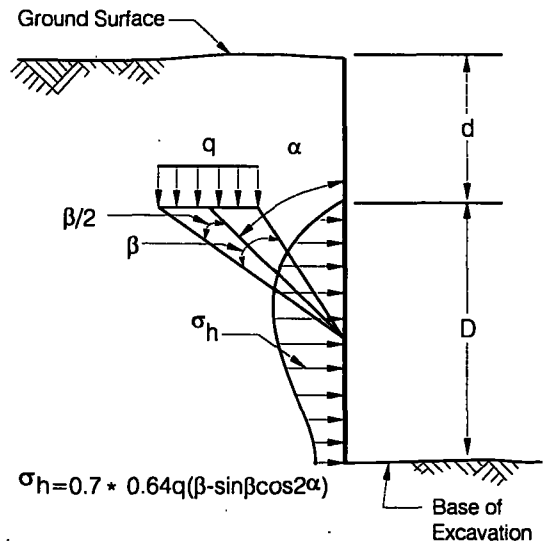
Surcharge Pressures

Determination of Lateral Pressure Acting on Permanent Walls Due to:

A. Small Isolated Footing Cross Section View

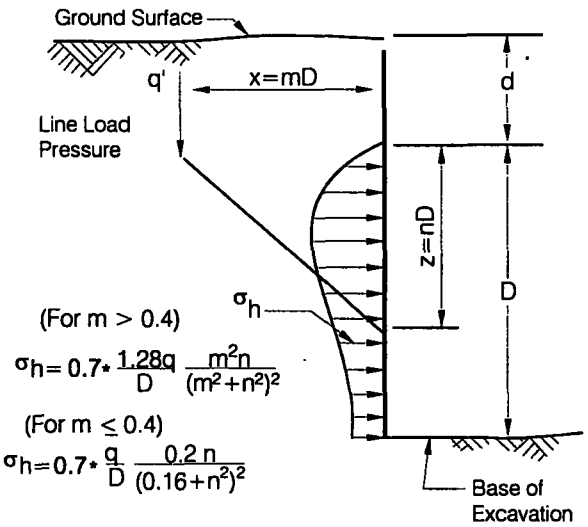


B. Large Isolated Footing Cross Section View



$\sigma_h = 0.7 * 0.64q(\beta - \sin\beta\cos 2\alpha)$

C. Continuous Wall Footing Parallel to Excavation Cross Section View



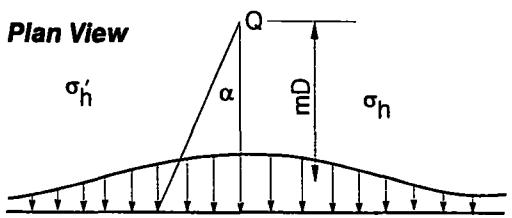
(For m > 0.4)

$$\sigma_h = 0.7 * \frac{1.28q}{D} \frac{m^2 n}{(m^2+n^2)^2}$$

(For m ≤ 0.4)

$$\sigma_h = 0.7 * \frac{q}{D} \frac{0.2\,n}{(0.16+n^2)^2}$$

Plan View



$$\sigma_h' = \sigma_h \cos^2(1.1\alpha)$$

(For m > 0.4)

$$\sigma_h = 0.7 * \frac{1.77Q}{D^2} \frac{m^2 n^2}{(m^2+n^2)^3}$$

(For m ≤ 0.4)

$$\sigma_h = 0.7 * \frac{0.28Q}{D^2} \frac{n^2}{(0.16+n^2)}$$

Definition and Units

Q	Footing Load in Pounds
D	Excavation Depth below Footing in Feet
d	Depth to Base of Footing in Feet
σ_h	Lateral Soil Pressure in PSF
q	Unit Loading Pressure in PSF
α, β	Radians

Notes:
1. Lateral pressures due to adjacent structures should be added to lateral pressures on Figures 11 and 12.
2. Wall footings acting other than parallel to the excavation can be treated as series of discrete point loads, using approach A.

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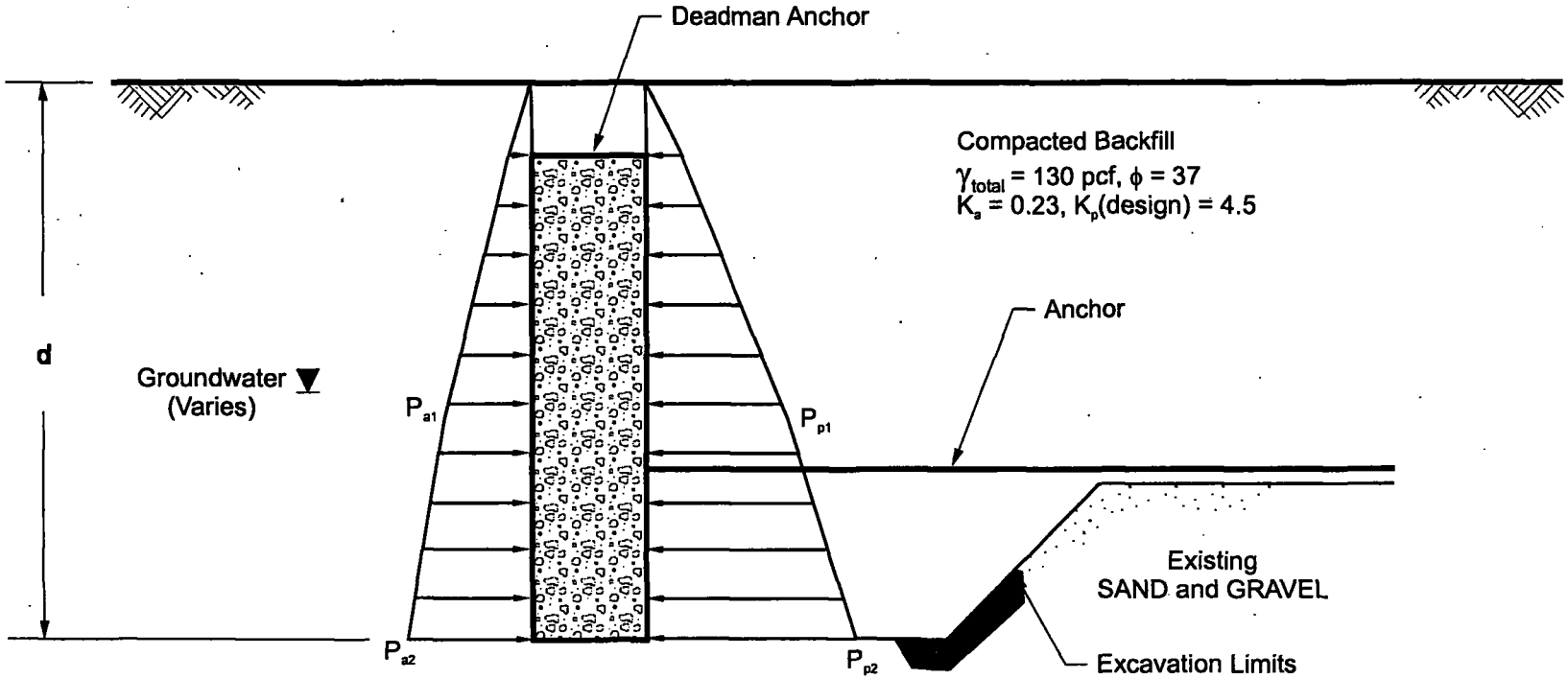
12/02
Figure 13

Deadman Earth Pressures



Note:
Anchor location should be at the location of the resultant earth pressures acting on the vertical face of the deadman.

Seismic Pressures:

475-Year Earthquake: Add active distributed pressure = 5.3d (psf)
where d is the depth to the bottom of the deadman
for passive resistance, use K_p(ultimate)

190-Year Earthquake: Add active distributed pressure = 2.5d (psf)
for passive resistance, use K_p(design)

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Figure 14
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APPENDIX A
FIELD EXPLORATION METHODS AND ANALYSIS

APPENDIX A
FIELD EXPLORATIONS METHODS AND ANALYSIS

This appendix documents the processes Hart Crowser uses in determining the nature of the soils underlying the project site addressed by this report. The discussion includes information on the following subjects:

- Explorations and Their Location;
- The Use of Auger Borings; and
- Standard Penetration Test (SPT) Procedures.

Explorations and Their Location

Our field program consisted of eight geotechnical borings (designated H-1-02 through H-8-02), three observation wells (designated OW-1 through OW-3); and a large diameter test well (designated PW-02). Appendix D includes details regarding the test well. Boring H-8-02 was drilled as a result of a survey error and was terminated at a relatively shallow depth (44 feet). The location and the elevation of boring H-8-02 were not surveyed by WSDOT.

The exploration logs within this appendix show our interpretation of the drilling, sampling, and testing data. They indicate the depth where the soils change. Note that the change may be gradual. In the field, we classified the samples taken from the explorations according to the methods presented on Figure A-1 - Key to Exploration Logs. This figure also provides a legend explaining the symbols and abbreviations used in the logs.

Locations of Explorations. Figure 2 shows the location of the explorations. This report shows the actual locations and ground surface elevations, presented on the exploration logs, as they were established during a site survey by WSDOT.

The Use of Auger Borings

With depths ranging from 44 to 90 feet below the ground surface, the eight hollow-stem auger borings were drilled from October 28 to 31, 2002. The three observation wells and the test well were drilled on October 29, 2002.

The borings and the observation wells used a 4-inch inside diameter hollow-stem auger and were advanced with a truck-mounted drill rig subcontracted by Hart Crowser. Engineering geologists from Hart Crowser continuously observed the drilling. Detailed field logs were prepared of each boring. Using the Standard Penetration Test (SPT), we obtained samples at 2-1/2- to 5-foot-depth intervals.

The logs of the borings, the test well, and the observation wells are presented on Figures A-2 through A-13 at the end of this appendix.

Standard Penetration Test (SPT) Procedures

This test is an approximate measure of soil density and consistency. To be useful, the results must be used with engineering judgment in conjunction with other tests. The SPT (as described in ASTM D 1586) was used to obtain disturbed samples. This test employs a standard 2-inch outside diameter split-spoon sampler. Using a 140-pound hammer, free-falling 30 inches, the sampler is driven into the soil for 18 inches. The number of blows required to drive the sampler the last 12 inches only is the Standard Penetration Resistance. This resistance, or blow count, measures the relative density of granular soils and the consistency of cohesive soils. The blow counts are plotted on the boring logs at their respective sample depths.

Soil samples are recovered from the split-barrel sampler, field classified, and placed into watertight jars. They are then taken to Hart Crowser's laboratory for further testing.

In the Event of Hard Driving

Occasionally very dense materials preclude driving the total 18-inch sample. When this happens, the penetration resistance is entered on logs as follows:

Penetration less than six inches. The log indicates the total number of blows over the number of inches of penetration.

Penetration greater than six inches. The blow count noted on the log is the sum of the total number of blows completed after the first six inches of penetration. This sum is expressed over the number of inches driven that exceed the first 6 inches. The number of blows needed to drive the first six inches are not reported. For example, a blow count series of 12 blows for 6 inches, 30 blows for 6 inches, and 50 (the maximum number of blows counted within a 6-inch increment for SPT) for 3 inches would be recorded as 80/9.

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Key to Exploration Logs

Sample Description

Classification of soils in this report is based on visual field and laboratory observations which include density/consistency, moisture condition, grain size, and plasticity estimates and should not be construed to imply field nor laboratory testing unless presented herein. Visual-manual classification methods of ASTM D 2488 were used as an identification guide.

Soil descriptions consist of the following:
Density/consistency, moisture, color, minor constituents, MAJOR CONSTITUENT, additional remarks.

Density/Consistency

Soil density/consistency in borings is related primarily to the Standard Penetration Resistance. Soil density/consistency in test pits is estimated based on visual observation and is presented parenthetically on the test pit logs.

SAND or GRAVEL Density	Standard Penetration Resistance (N) in Blows/Foot	SILT or CLAY Consistency	Standard Penetration Resistance(N) in Blows/Foot	Approximate Shear Strength in TSF
Very loose	0 - 4	Very soft	0 - 2	<0.125
Loose	4 - 10	Soft	2 - 4	0.125 - 0.25
Medium dense	10 - 30	Medium stiff	4 - 8	0.25 - 0.5
Dense	30 - 50	Stiff	8 - 15	0.5 - 1.0
Very dense	>50	Very stiff	15 - 30	1.0 - 2.0
		Hard	>30	>2.0

Moisture

Dry	Little perceptible moisture
Damp	Some perceptible moisture, probably below optimum
Moist	Probably near optimum moisture content
Wet	Much perceptible moisture, probably above optimum

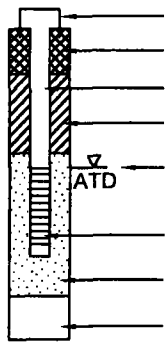
Minor Constituents	Estimated Percentage
Not identified in description	0 - 5
Slightly (clayey, silty, etc.)	5 - 12
Clayey, silty, sandy, gravelly	12 - 30
Very (clayey, silty, etc.)	30 - 50

Legends

Sampling Test Symbols

Boring Samples		Test Pit Samples	
☒	Split Spoon	☒	Grab (Jar)
⧅	Shelby Tube	⧄	Bag
▥	Cuttings	⧅	Shelby Tube
◫	Core Run		
*	No Sample Recovery		
P	Tube Pushed, Not Driven		

Groundwater Observation Wells



- Monument
- Surface Seal
- Riser Pipe
- Bentonite
- Groundwater Level on Date or at Time of Drilling (ATD)
- Well Screen
- Sand Pack
- Native Material
- Groundwater Seepage (Test Pits)

Test Symbols

GS	Grain Size Classification
CN	Consolidation
UU	Unconsolidated Undrained Triaxial
CU	Consolidated Undrained Triaxial
CD	Consolidated Drained Triaxial
QU	Unconfined Compression
DS	Direct Shear
K	Permeability
PP	Pocket Penetrometer Approximate Compressive Strength in TSF
TV	Torvane Approximate Shear Strength in TSF
CBR	California Bearing Ratio
MD	Moisture Density Relationship
AL	Atterberg Limits
	Water Content in Percent Liquid Limit Natural Plastic Limit
PID	Photoionization Detector Reading
CA	Chemical Analysis
DT	*In Situ* Density Test

HC Standards\Standard Report Figures\A-1's\A-1 Standard

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Figure A-1

Boring Log H-1-02

Northing (ft): 423863.51
Easting (ft): 998553.41

Soil Descriptions
Approximate Ground Surface Elevation in Feet: 11.8

Depth in Feet

Medium dense, moist, gray, slightly silty, slightly gravelly, fine to medium SAND.

Loose to medium dense, wet, gray, very gravelly SAND.

Becomes slightly gravelly.

Becomes silty and gravelly.

Medium dense, wet, gray, silty, fine SAND with shell fragments.

Very soft, wet, gray, slightly clayey, sandy SILT with shell fragments.

Wood debris in sample.

Medium dense, wet, gray, silty, fine SAND with shell fragments and wood.

Rough drilling from a depth of 66.5 to 68 feet.

Hard, wet, gray SILT with scattered organic material.

Dense, wet, gray, very silty, fine SAND.

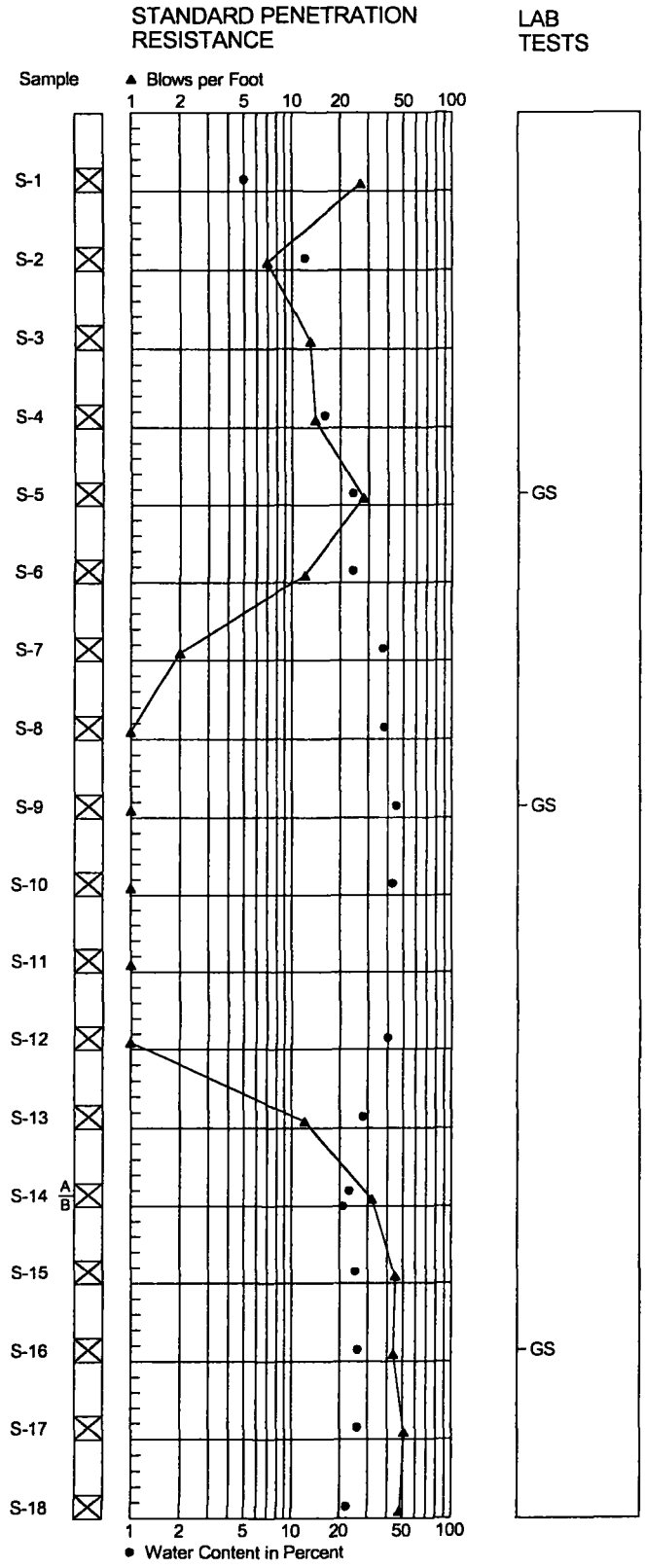
Hard, wet, gray, slightly sandy, very clayey SILT with scattered organic material.

Becomes non-sandy.

Silty, fine SAND at tip of sampler.

Becomes sandy.

Bottom of Boring at 90.0 Feet.
Completed 10/30/02.



BORING LOG 7794BL2.GPJ HC_CORP.GDT 12/17/02

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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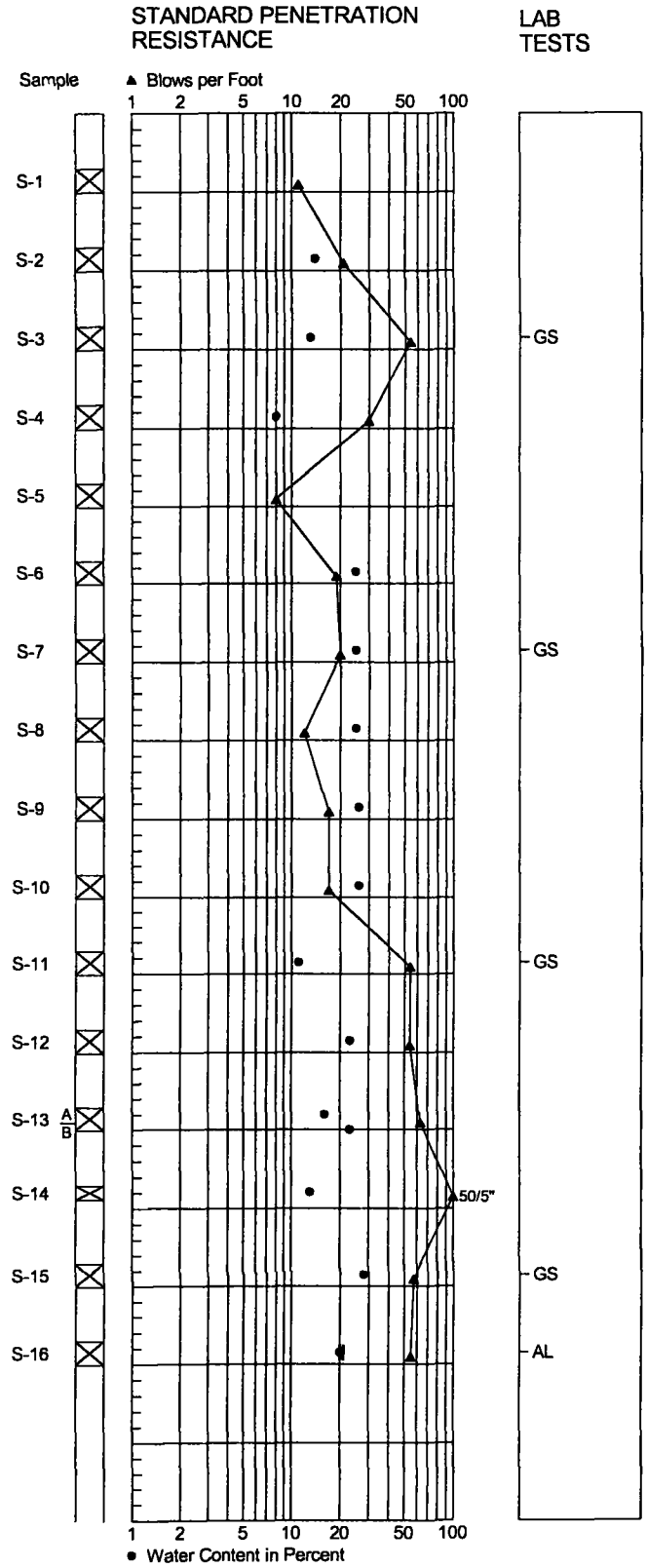
Figure A-2

Boring Log H-2-02

Northing (ft): 423531.11
Easting (ft): 998308.07

Soil Descriptions
Approximate Ground Surface Elevation in Feet: 12.9

Soil Description	Depth in Feet
Medium dense, moist, gray, slightly silty, slightly gravelly, fine to medium SAND.	0
Medium dense, wet, gray, slightly silty, very gravelly SAND.	5 ▽ ATD
Very dense, wet, gray, gravelly SAND.	10
Dense, wet, gray, slightly silty, very sandy GRAVEL.	15
Loose to medium dense, wet, gray, silty, fine SAND, with shell fragments.	20
Very dense, wet, gray, gravelly, silty, fine SAND.	25
Hard, moist, gray, slightly sandy, clayey SILT.	30
Interbedded layer of very dense, wet, dark gray, silty, fine to medium SAND.	35
Interbedded layer of very dense, wet, dark gray, slightly silty, gravelly SAND.	40
Interbedded layers of organic material.	45
Interbedded layers of silty, fine SAND.	50
Bottom of Boring at 80.0 Feet.	55
Completed 10/30/02.	60
	65
	70
	75
	80
	85
	90



BORING LOG 7794BL2.GPJ HC_CORP.GDT 12/19/02

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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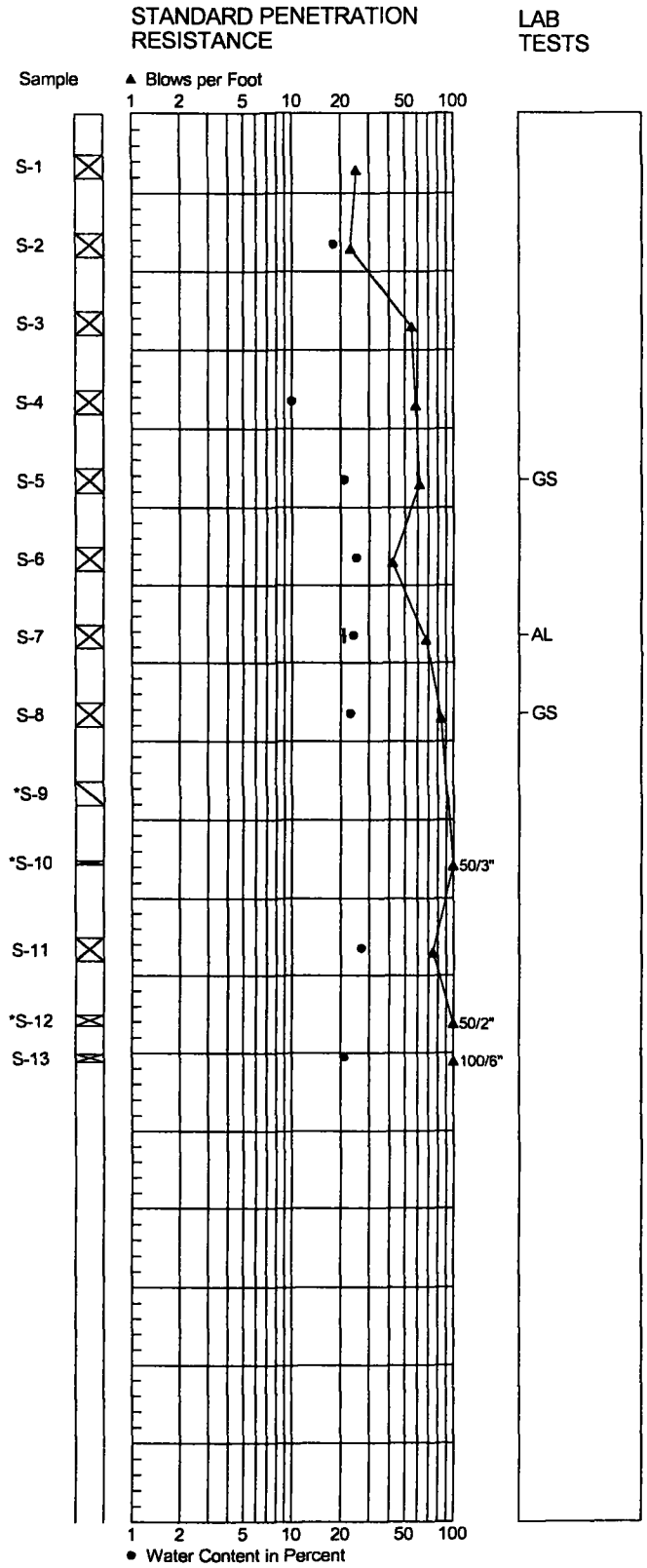
Figure A-3

Boring Log H-3-02

Northing (ft): 422897.63
Easting (ft): 998401.54

Soil Descriptions
Approximate Ground Surface Elevation in Feet: 9.7

Depth in Feet	Soil Description
0	4 inches of Asphalt over damp, gray, silty, fine to medium SAND.
5 (ATD)	Very stiff, moist to wet, gray, very silty, fine SAND with trace gravels and wood debris.
	Water added at a depth of 5 feet.
10	Medium dense, wet, gray, silty, very sandy GRAVEL with scattered root fragments.
15	Very dense, wet, gray, slightly silty, very sandy GRAVEL.
	12 inches of heave at S-3.
20	6 inches of heave at S-4. Mud added to hole.
25	Dense, wet, gray, very silty, fine SAND.
30	Scattered gravels at a depth of 31 feet.
	Hard, moist, gray, sandy SILT with scattered shell fragments.
35	Scattered gravels at a depth of 34 feet.
40	Becomes slightly sandy.
60	Becomes sandy.
	Bottom of Boring at 60.5 Feet.
	Completed 10/28/02.



BORING LOG 7794BL2.GPJ HC_CORP.GDT 11/12/02

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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Figure A-4

Boring Log H-4-02

Northing (ft): 423342.8
Easting (ft): 998932.95

Soil Descriptions
Approximate Ground Surface Elevation in Feet: 15.2

Depth in Feet

Damp, brown, slightly gravelly, sandy SILT over medium dense, damp, dark gray, gravelly, silty SAND with shell fragments.

SILT with organic material.

Very dense, wet, dark gray, silty, very sandy GRAVEL with scattered wood fragments.

Loose, wet, dark gray, very silty, fine SAND.

Brief gravelly drill action at 18 feet.

Dense to medium dense, wet, gray, very silty, fine SAND with shell fragments.

Medium stiff, wet, gray, slightly clayey, very sandy SILT with shell fragments and trace organic material.

Becomes very soft and sandy.

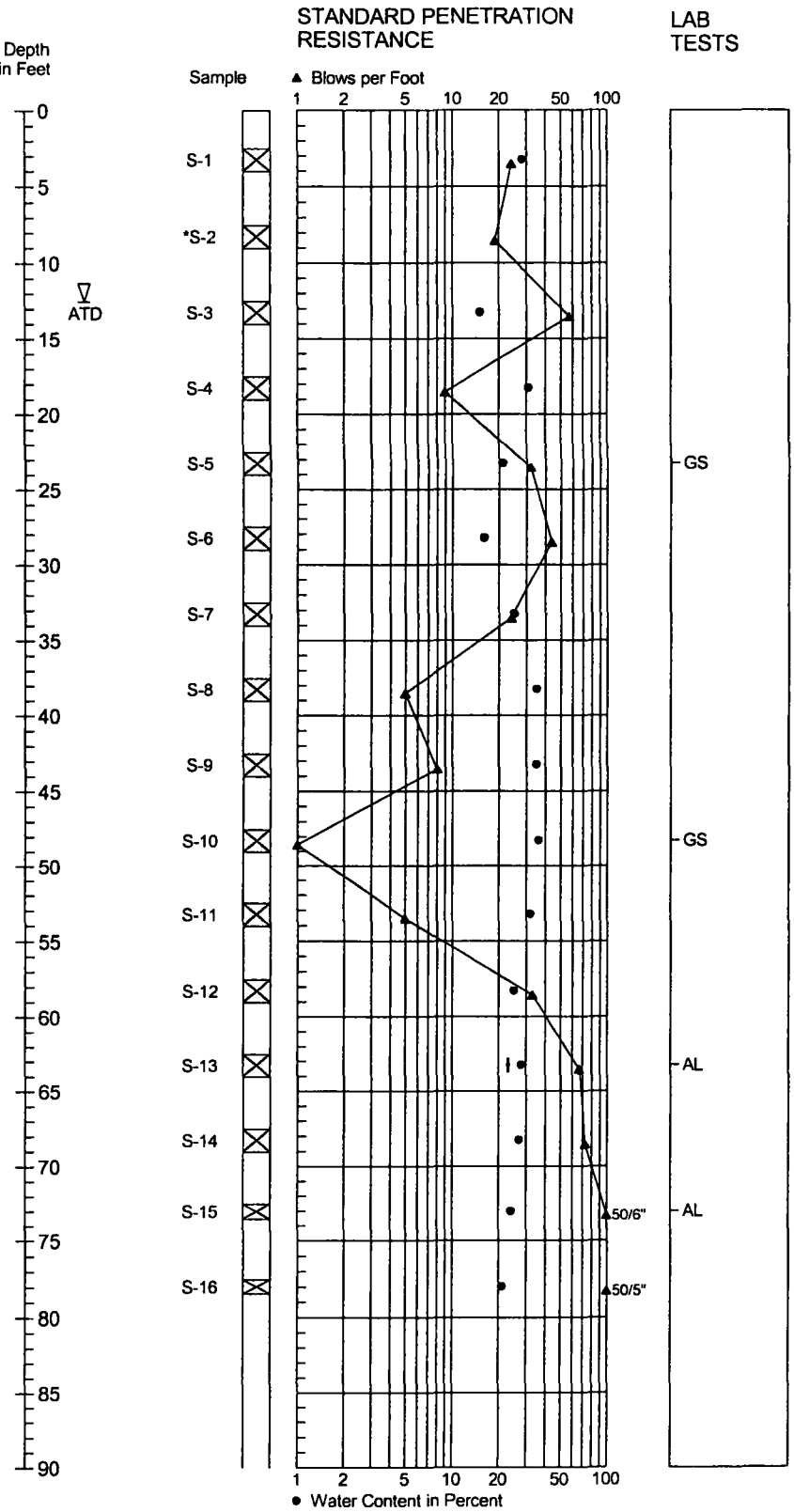
Becomes medium stiff.

Dense, wet, gray, very silty, fine SAND with shell fragments and trace gravels.

Hard, moist, gray, sandy SILT with shell fragments.

Very dense, moist, gray, very silty, fine SAND.

Bottom of Boring at 78.4 Feet.
Completed 10/31/02.



BORING LOG 7794BL2.GPJ HC_CORP.GDT 11/12/02

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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Figure A-5

Monitoring Well Log H-5-02

Northing (ft): 423471.57
Easting (ft): 998516.39

Soil Descriptions
Approximate Ground Surface Elevation in Feet: 12.8

Moist, gray, slightly silty, very sandy GRAVEL with organic material (wood). (FILL)

Medium dense, moist to wet, gray, slightly silty, very gravelly, fine to medium SAND.

Medium dense, wet, gray, very gravelly SAND.

Medium dense, wet, gray, silty, fine SAND with shell fragments.

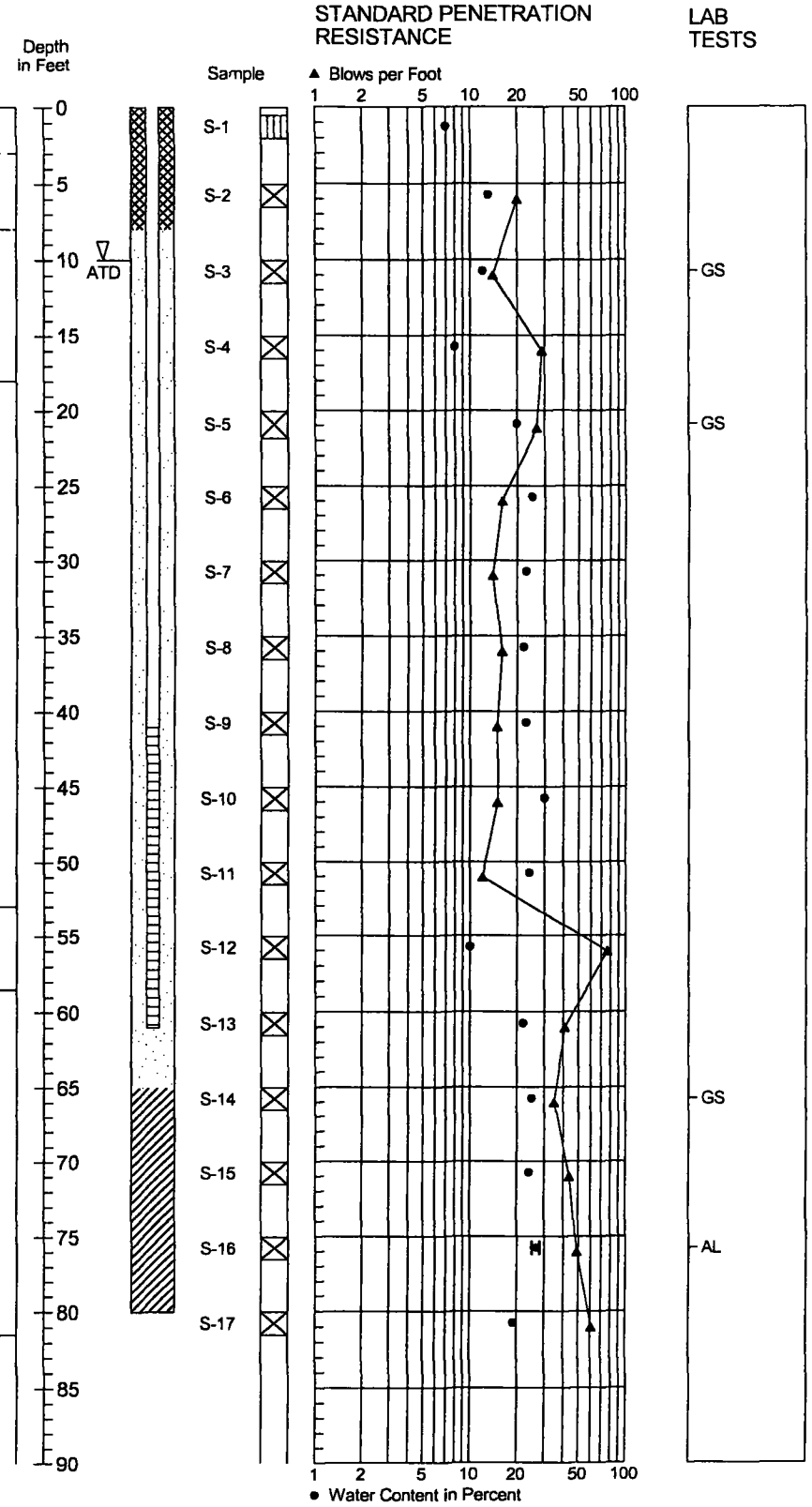
Very dense, wet, dark gray, silty, very sandy, fine GRAVEL with shell fragments.

Hard, wet, gray, slightly clayey, sandy to very sandy SILT.

Becomes moist and non-sandy.

Becomes sandy with scattered organic material.

Bottom of Boring at 81.5 Feet.
Completed 10/28/02.



BORING LOG 7794BL2.GPJ HC_CORP.GDT 11/13/02

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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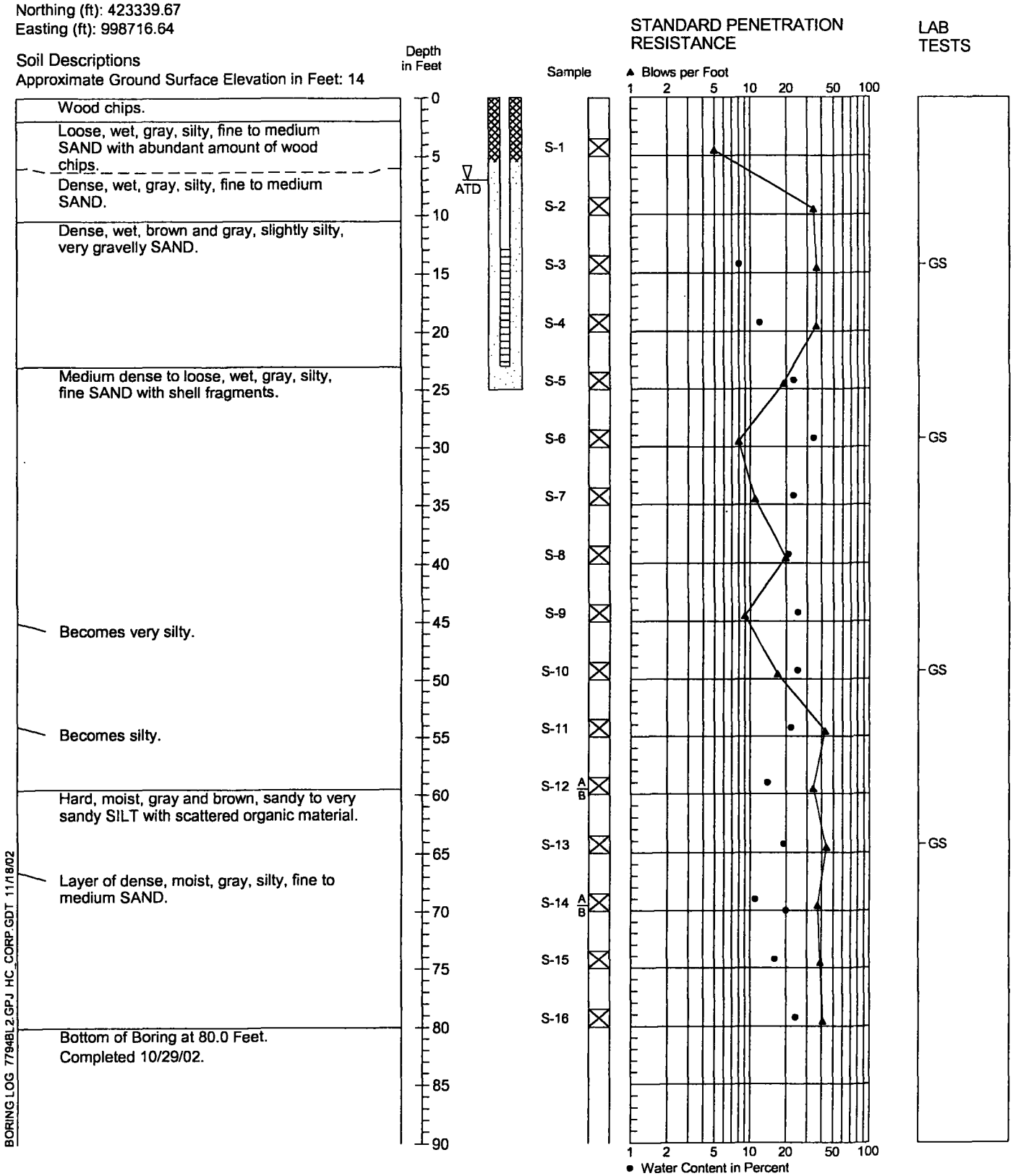
Figure A-6

Monitoring Well Log H-6-02

Northing (ft): 423339.67
Easting (ft): 998716.64

Soil Descriptions
Approximate Ground Surface Elevation in Feet: 14

Soil Descriptions
Wood chips.
Loose, wet, gray, silty, fine to medium SAND with abundant amount of wood chips.
Dense, wet, gray, silty, fine to medium SAND.
Dense, wet, brown and gray, slightly silty, very gravelly SAND.
Medium dense to loose, wet, gray, silty, fine SAND with shell fragments.
Becomes very silty.
Becomes silty.
Hard, moist, gray and brown, sandy to very sandy SILT with scattered organic material.
Layer of dense, moist, gray, silty, fine to medium SAND.
Bottom of Boring at 80.0 Feet. Completed 10/29/02.



BORING LOG 7794BL2.GPJ HC_CORP.GDT 11/18/02

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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Figure A-7

Monitoring Well Log H-7-02

Northing (ft): 423253.94
Easting (ft): 998353.13

Soil Descriptions
Approximate Ground Surface Elevation in Feet: 11.7

Damp, gray, gravelly, very silty, fine to medium SAND over medium dense, wet, brown and gray, very sandy GRAVEL.

Medium dense, wet, gray, very silty, fine SAND.

Very dense, wet, gray, slightly silty, very gravelly SAND.

Becomes gravelly.

Very dense, wet, gray, very gravelly SAND.

Becomes slightly silty.

Dense, wet, gray, silty, fine SAND.

Very dense, wet, gray, slightly silty, gravelly SAND.

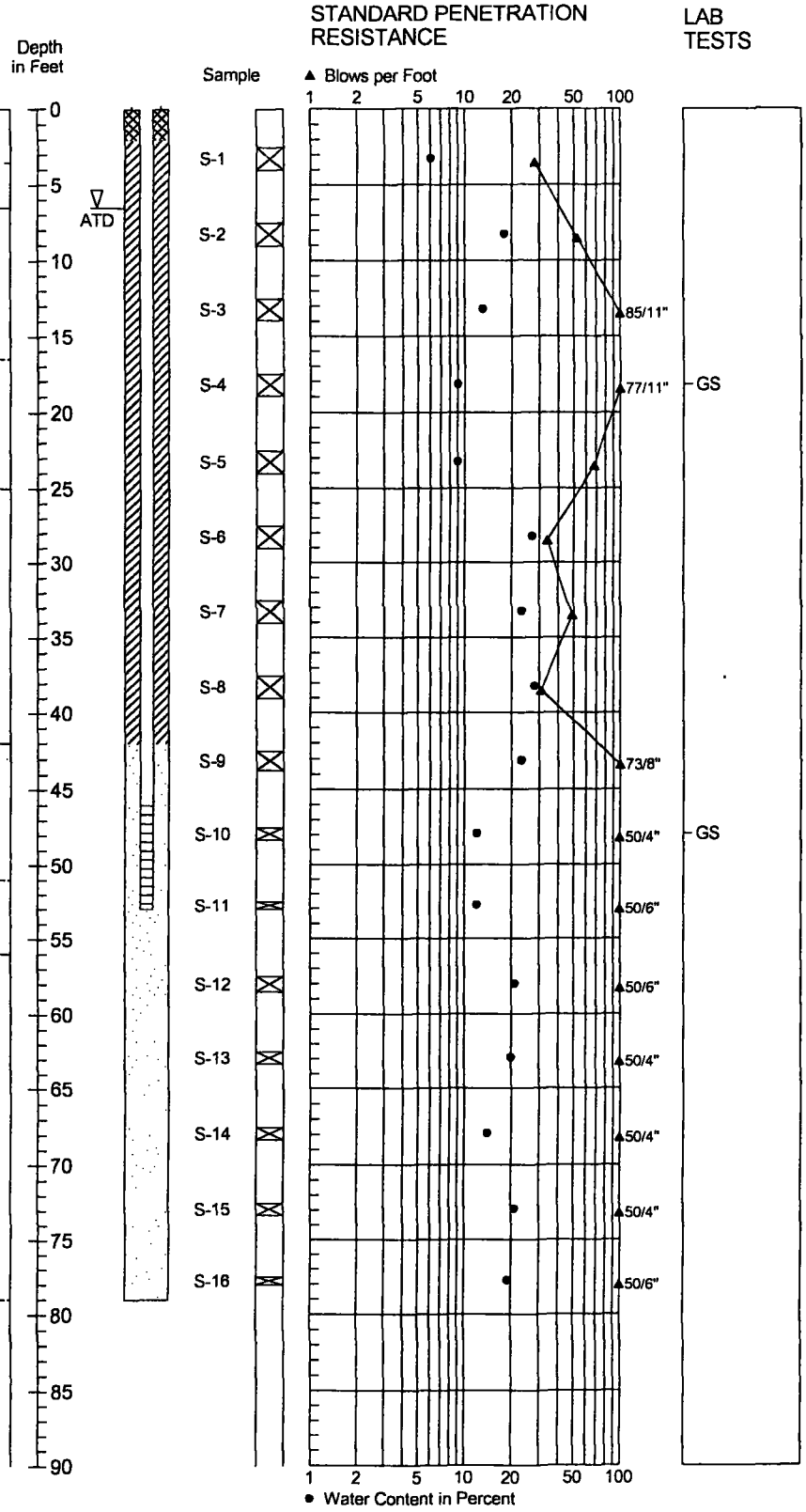
Very dense, wet, gray, silty, very sandy GRAVEL.

6 inches of heave, water added.

Hard, moist to wet, gray, very sandy SILT.

Interbedded layer of very dense, wet, gray, gravelly, very silty, fine to medium SAND.

Bottom of Boring at 79.0 Feet.
Completed 10/30/02.



BORING LOG 7794BL2.GPJ HC_CORP.GDT 11/18/02

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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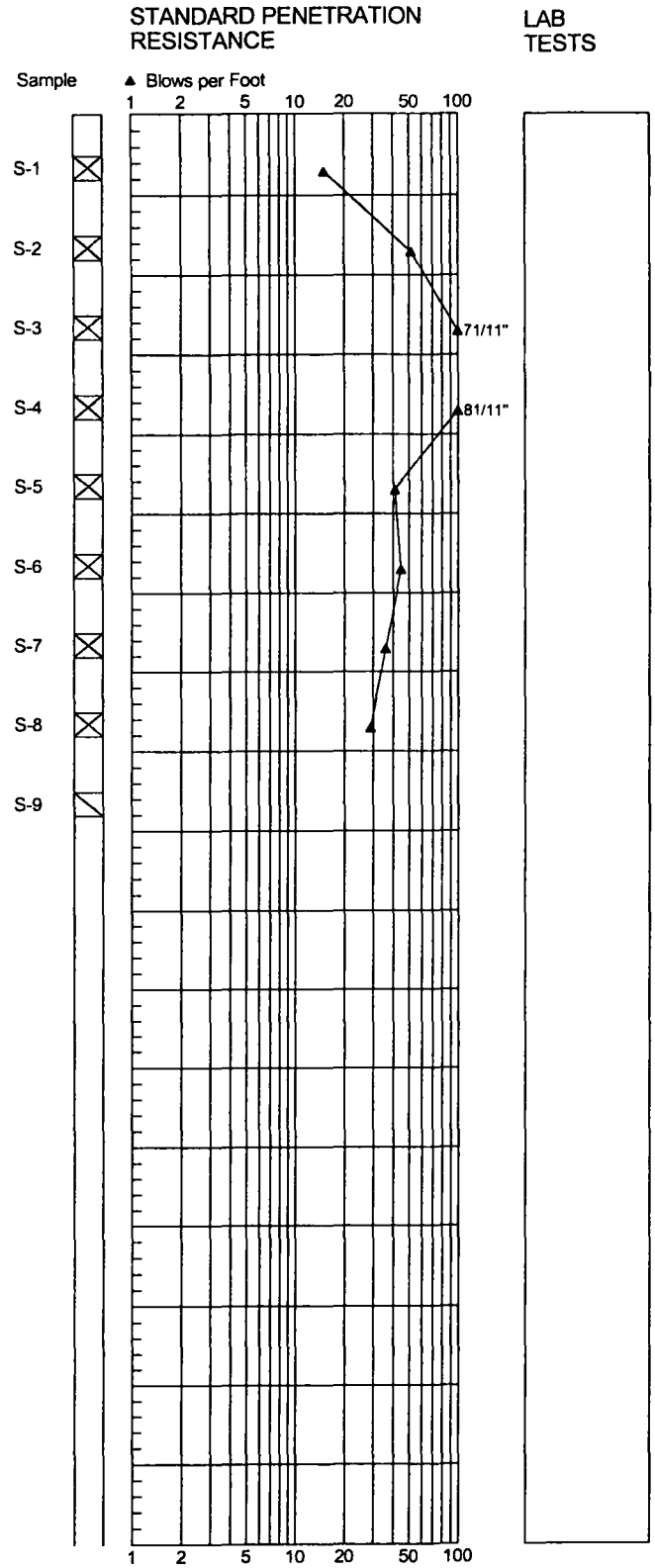
Figure A-8

Boring Log H-8-02

Soil Descriptions
Approximate Ground Surface Elevation in Feet: ~12.13

Soil Descriptions	Depth in Feet
Medium dense, moist, gray SAND and GRAVEL.	0
Very dense, wet, gray, slightly silty, fine to medium, sandy GRAVEL to gravelly SAND.	5 (ATD)
Dense to medium dense, wet, gray, very silty SAND with scattered wood and shells.	25
Bottom of Boring at 44.0 Feet. Completed 10/28/02.	45



BORING LOG 7794BL2.GPJ HC_CORP.GDT 12/19/02

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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Figure A-9

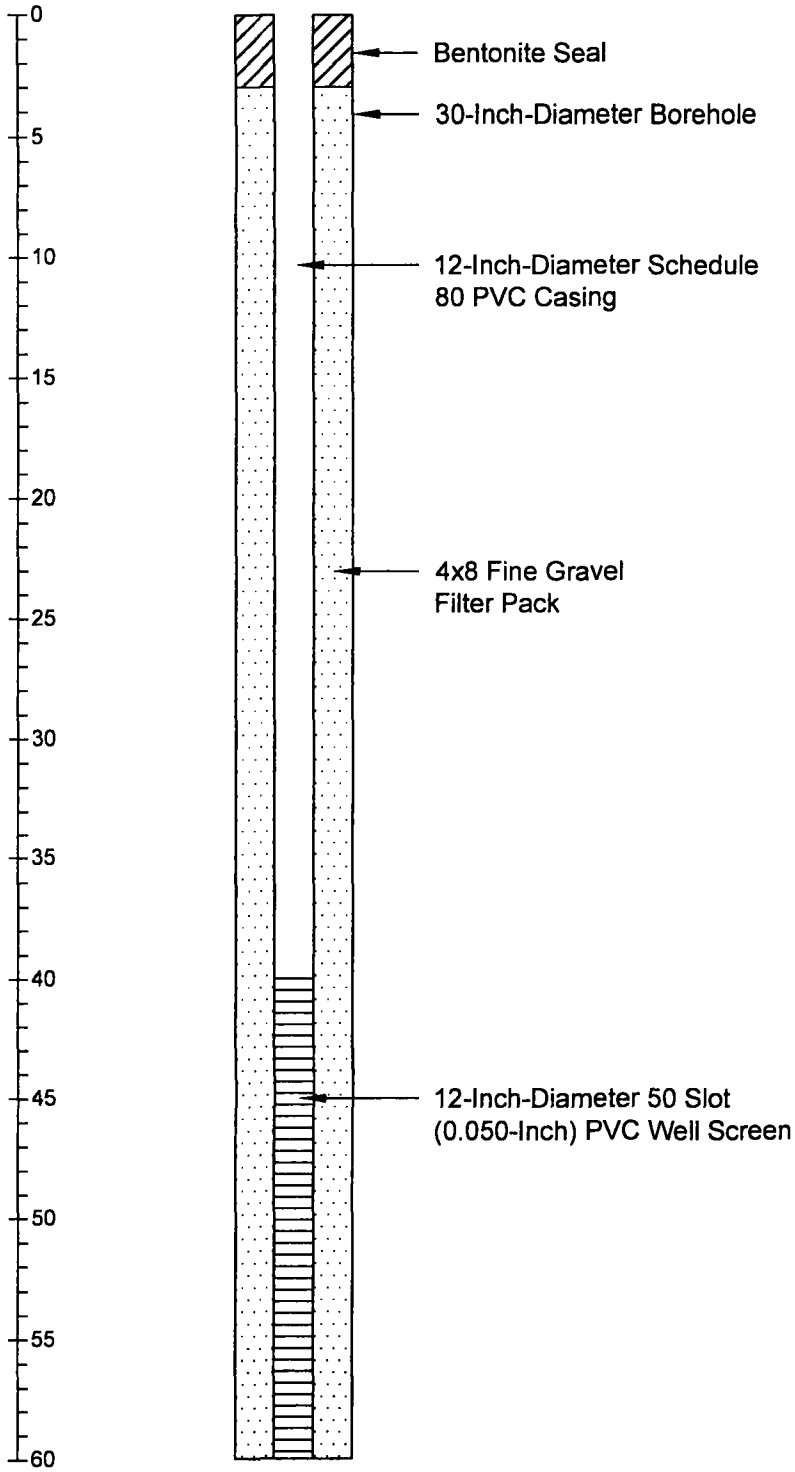
Well Completion Diagram PW-02

Northing (ft): 423370.49
Easting (ft): 998677.27

Soil Descriptions

Ground Surface Elevation in Feet: 13.27

Soil Descriptions	Depth Sample in Feet
(Dense), wet, brown and gray, slightly silty, very gravelly SAND.	0 – 23
(Medium dense to loose), wet, gray, silty, fine SAND.	23 – 55
(Dense), wet, gray, slightly silty, gravelly SAND.	55 – 60



Bottom of Boring at 60.0 Feet.
Completed 10/29/02.

Note: Borehole was not logged. Soil descriptions interpreted from nearby logged borehole and Cross Section E-E'.

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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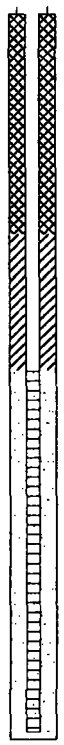
Figure A-10

Monitoring Well Log OW-1-02

Northing (ft): 423358.16
Easting (ft): 998693.02

Soil Descriptions
Approximate Ground Surface Elevation in Feet: 12.8

Soil Descriptions	Depth in Feet	Sample	STANDARD PENETRATION RESISTANCE ▲ Blows per Foot (1, 2, 5, 10, 20, 50, 100)	LAB TESTS
Drilled unsampled to install well.	0			
	5			
	10			
	15			
Bottom of Boring at 20.0 Feet. Completed 10/29/02.	20			
	25			
	30			
	35			
	40			



BORING LOG 7794MW.GPJ HC_CORP.GDT 12/9/02

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Figure A-11

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

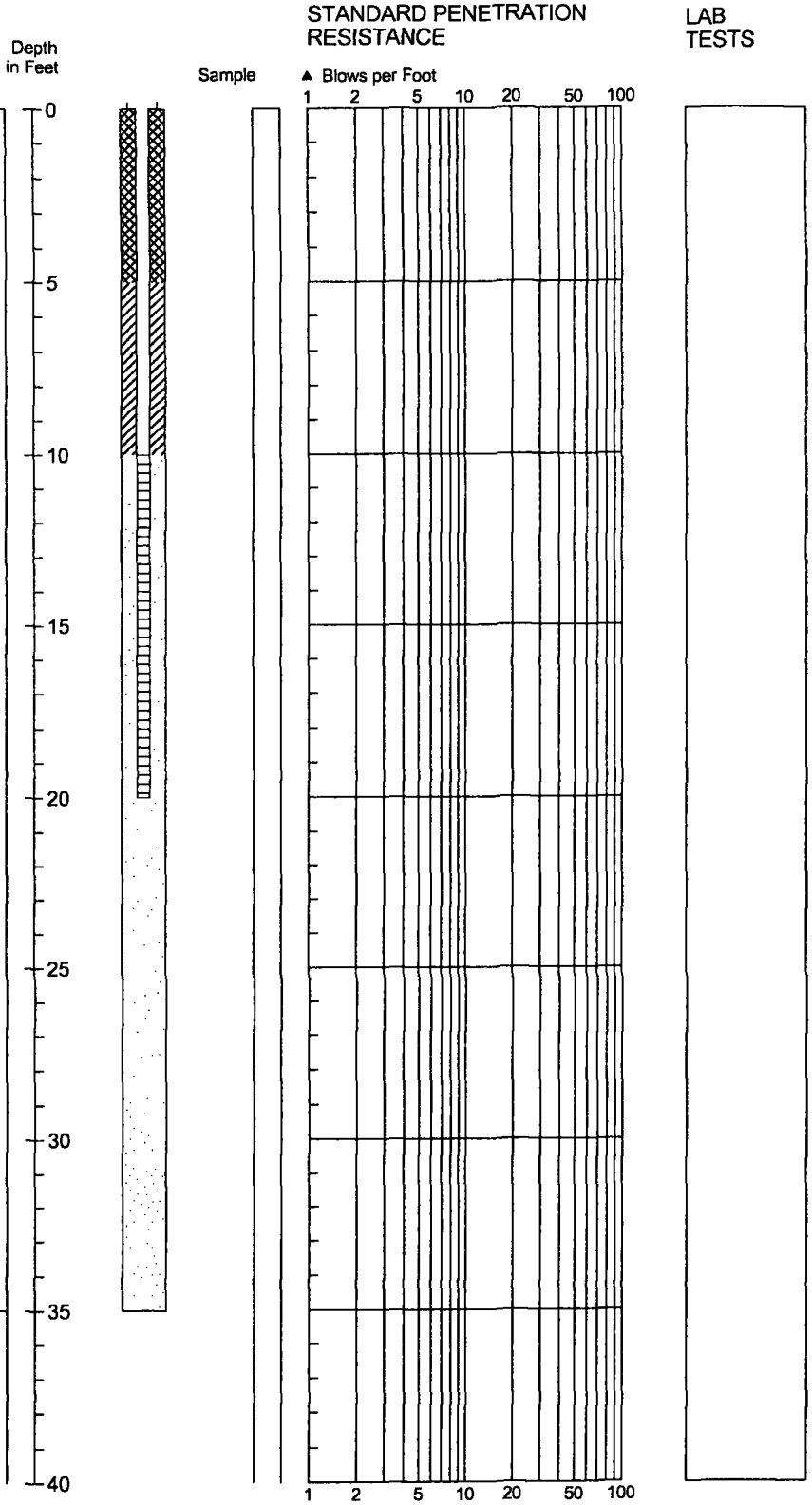
Monitoring Well Log OW-2-02

Northing (ft): 423405.46
Easting (ft): 998678.63

Soil Descriptions
Approximate Ground Surface Elevation in Feet: 15.6

Drilled unsampled to install well.

Bottom of Boring at 35.0 Feet.
Completed 10/29/02.



BORING LOG 7794MW.GPJ HC_CORP.GDT 12/9/02

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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7794 10/02

Figure A-12

Monitoring Well Log OW-3-02

Northing (ft): 423402.41
Easting (ft): 998626.46

Soil Descriptions
Approximate Ground Surface Elevation in Feet: 14.6

Drilled unsampled to install well.

Bottom of Boring at 20.0 Feet.
Completed 10/29/02.

Depth in Feet: 0, 5, 10, 15, 20, 25, 30, 35, 40

Sample

STANDARD PENETRATION RESISTANCE
▲ Blows per Foot
1 2 5 10 20 50 100

LAB TESTS

BORING LOG 7794MW.GPJ HC_CORP.GDT 12/9/02

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Figure A-13

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

APPENDIX B
LABORATORY TESTING PROGRAM

APPENDIX B
LABORATORY TESTING PROGRAM

A laboratory testing program was performed for this study to evaluate the basic index and geotechnical engineering properties of the site soils. The tests performed and the procedures followed are outlined below.

Soil Classification

Field Observation and Laboratory Analysis. Soil samples from the explorations were visually classified in the field and then taken to our laboratory where the classifications were verified in a relatively controlled laboratory environment. Field and laboratory observations include density/consistency, moisture condition, and grain size and plasticity estimates.

The classifications of selected samples were checked by laboratory tests such as Atterberg limits determinations and grain size analyses. Classifications were made in general accordance with the Unified Soil Classification (USC) System, ASTM D 2487, as presented on Figure B-1.

Water Content Determinations

Water contents were determined for most samples recovered in the explorations in general accordance with ASTM D 2216, as soon as possible following their arrival in our laboratory. Water contents were not determined for very small samples nor samples where large gravel contents would result in values considered unrepresentative. The results of these tests are plotted at the respective sample depth on the exploration logs. In addition, water contents are routinely determined for samples subjected to other testing. These are also presented on the exploration logs.

Atterberg Limits (AL)

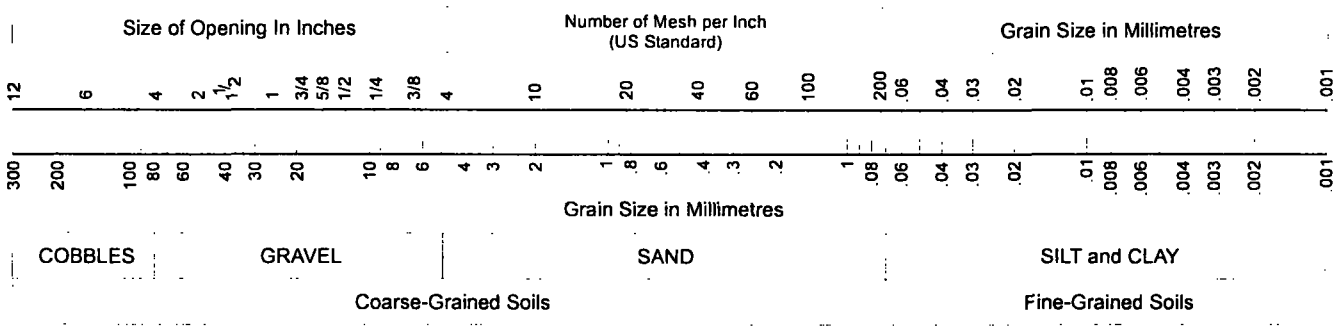
We determined Atterberg limits for selected fine-grained soil samples. The liquid limit and plastic limit were determined in general accordance with ASTM D 4318-84. The results of the Atterberg limits analyses and the plasticity characteristics are summarized in the Liquid and Plastic Limits Test Report, Figures B-2. This relates the plasticity index (liquid limit minus the plastic limit) to the liquid limit. The results of the Atterberg limits tests are shown graphically on the boring logs as well as where applicable on figures presenting various other test results.

Grain Size Analysis (GS)

Grain size distribution was analyzed on representative samples in general accordance with ASTM D 422. Wet sieve analysis was used to determine the size distribution greater than the U.S. No. 200 mesh sieve. The size distribution for particles smaller than the No. 200 mesh sieve was determined by the hydrometer method for a selected number of samples. The results of the tests are presented as curves on Figures B-3 through B-10 plotting percent finer by weight versus grain size.

F:\docs\jobs\7794\WSDOT PAGD Geo Report(Final).doc

Unified Soil Classification (USC) System
Soil Grain Size



Coarse-Grained Soils

G W	G P	G M	G C	S W	S P	S M	S C
Clean GRAVEL <5% fines	*	GRAVEL with >12% fines		Clean SAND <5% fines	*	SAND with >12% fines	
GRAVEL >50% coarse fraction larger than No. 4				SAND >50% coarse fraction smaller than No. 4			
Coarse-Grained Soils >50% larger than No. 200 sieve							

G W and S W $\left(\frac{D_{60}}{D_{10}}\right.$ >4 for G W, >6 for S W$\left.\right)$ & $1 \leq \frac{(D_{30})^2}{D_{10} \times D_{60}} \leq 3$

G P and S P Clean GRAVEL or SAND not meeting requirements for G W and S W

G M and S M Atterberg limits below A line with PI <4

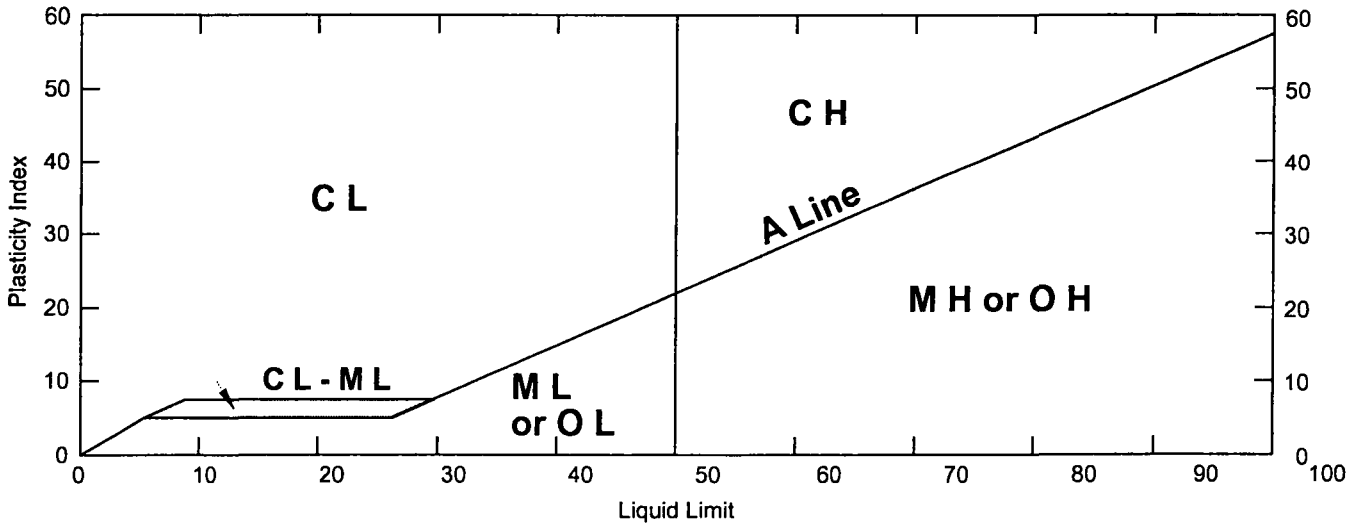
G C and S C Atterberg limits above A Line with PI >7

* Coarse-grained soils with percentage of fines between 5 and 12 are considered borderline cases requiring use of dual symbols.

D_{10}, D_{30}, and D_{60} are the particles diameter of which 10, 30, and 60 percent, respectively, of the soil weight are finer.

Fine-Grained Soils

M L	C L	O L	M H	C H	O H	Pt
SILT	CLAY	Organic	SILT	CLAY	Organic	Highly Organic Soils
Soils with Liquid Limit <50%			Soils with Liquid Limit >50%			
Fine-Grained Soils >50% smaller than No. 200 sieve						



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HC Standards\Standard Report Figures/Grain Size (B-1).CDR

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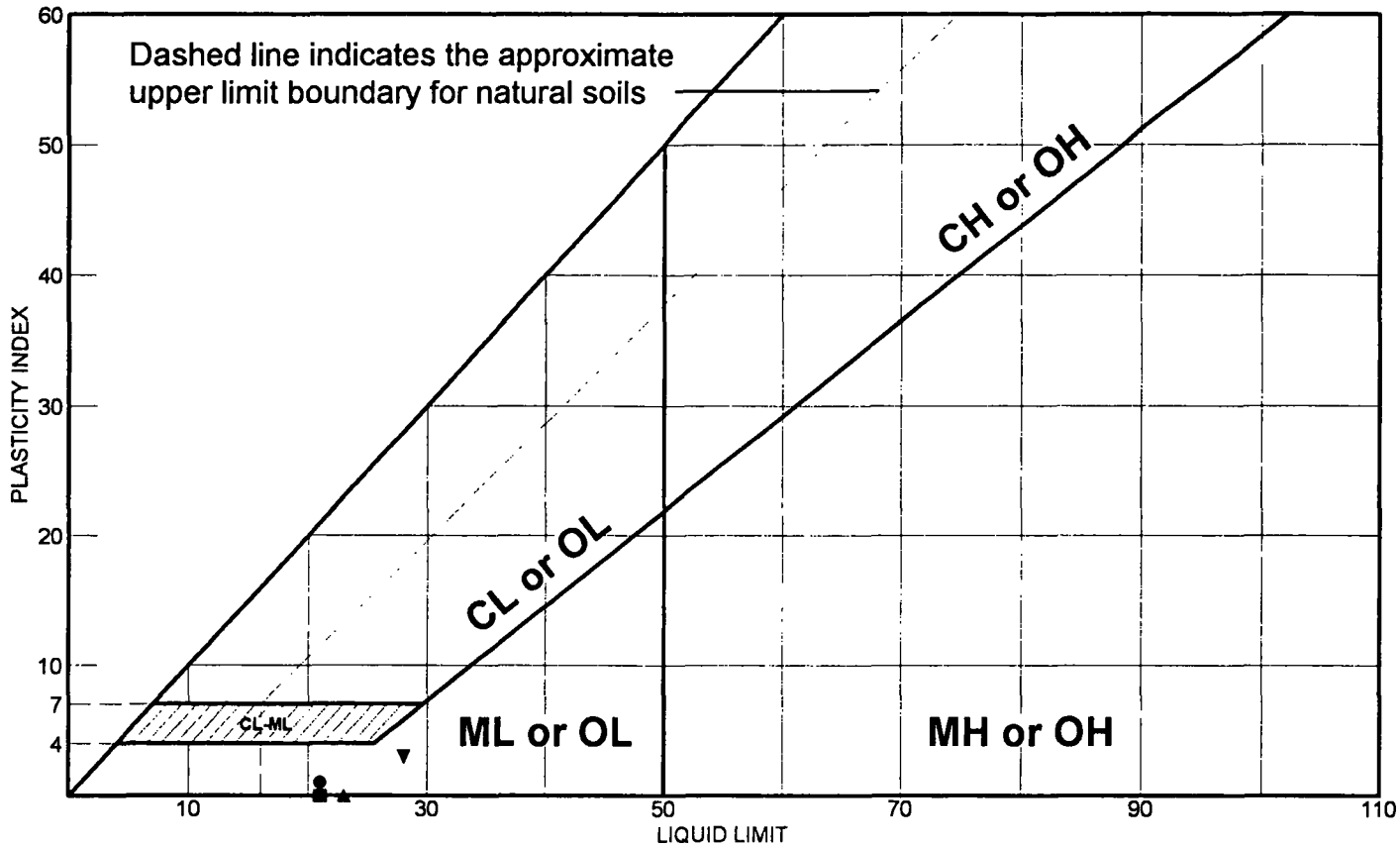
Figure B-1

LIQUID AND PLASTIC LIMITS TEST REPORT



Location + Description	LL	PL	PI	-200	USCS
● Source: H-2-02 Sample No.: S-16 SILT	21	20	1		ML
■ Source: H-3-02 Sample No.: S-7 SILT	21	21	0		ML
▲ Source: H-4-02 Sample No.: S-13 SILT	23	23	0		ML
◆ Source: H-4-02 Sample No.: S-15 SILT	21	22	NP		ML
▼ Source: H-5-02 Sample No.: S-16 SILT	28	25	3		ML

Remarks:

●

■

▲

◆

▼

Project: Port Angeles Graving Dock

Client: Washington State Department of Transportation

Location: Port Angeles, WA

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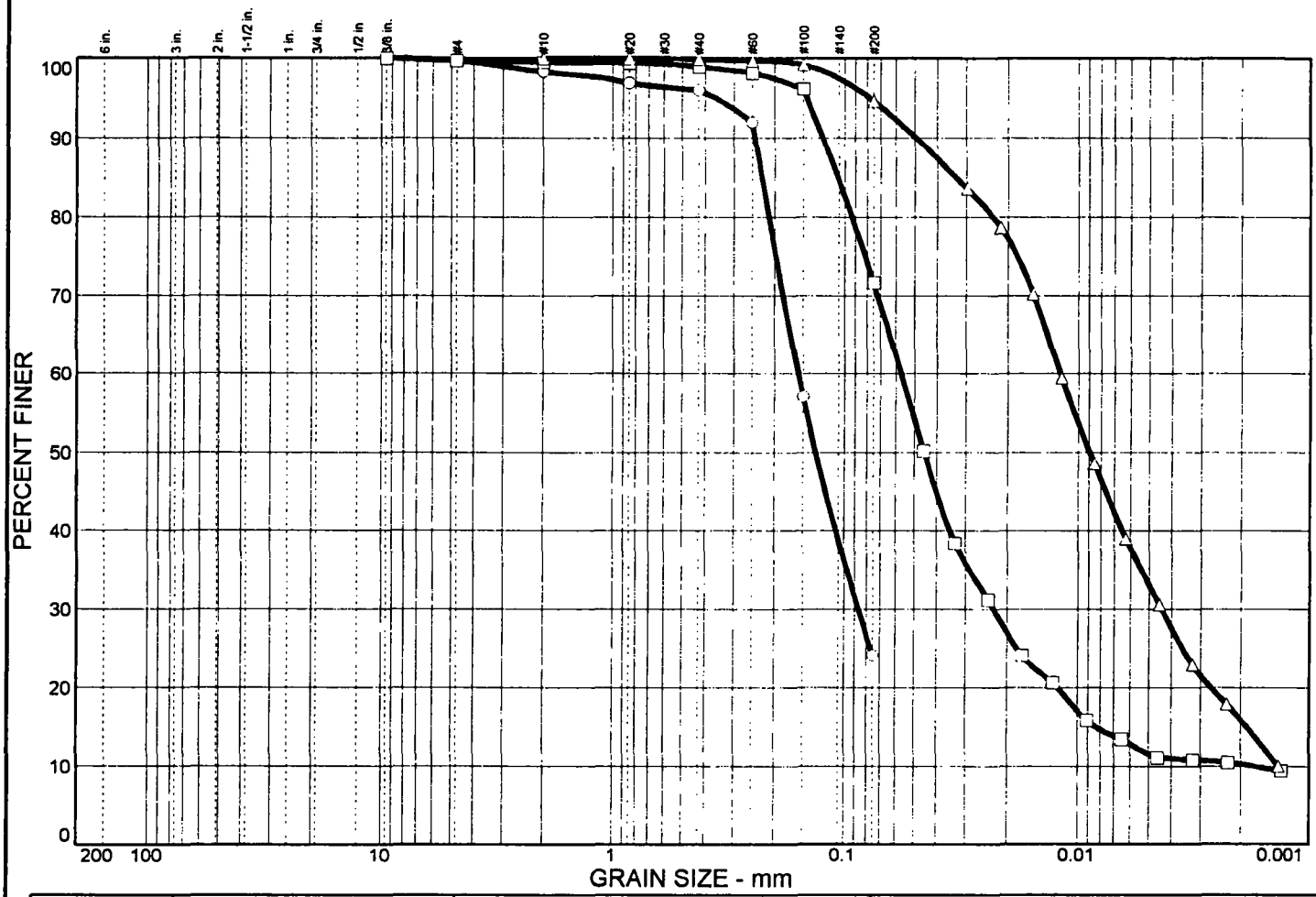
Figure No. B-2

Particle Size Distribution Report



	% COBBLES	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	0.0	0.3	1.4	2.3	71.9	24.1	
□	0.0	0.0	0.3	0.2	0.6	27.2	60.2	11.5
△	0.0	0.0	0.0	0.0	0.1	5.2	61.6	33.1

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			0.227	0.157	0.132	0.0864				
□			0.107	0.0569	0.0453	0.0227	0.0084	0.0017	5.25	32.99
△			0.0333	0.0120	0.0089	0.0044	0.0019	0.0014	1.19	8.70

MATERIAL DESCRIPTION	USCS	NAT. MOIST.
○ Silty, fine SAND	SM	24%
□ Slightly clayey, sandy SILT	ML	45%
△ Slightly sandy, very clayey SILT	ML	26%

Remarks:

○

□

△

Project: Port Angeles Graving Dock

Client: Washington State Department of Transportation

○ **Source:** H-1-02 **Sample No.:** S-5

□ **Source:** H-1-02 **Sample No.:** S-9

△ **Source:** H-1-02 **Sample No.:** S-16

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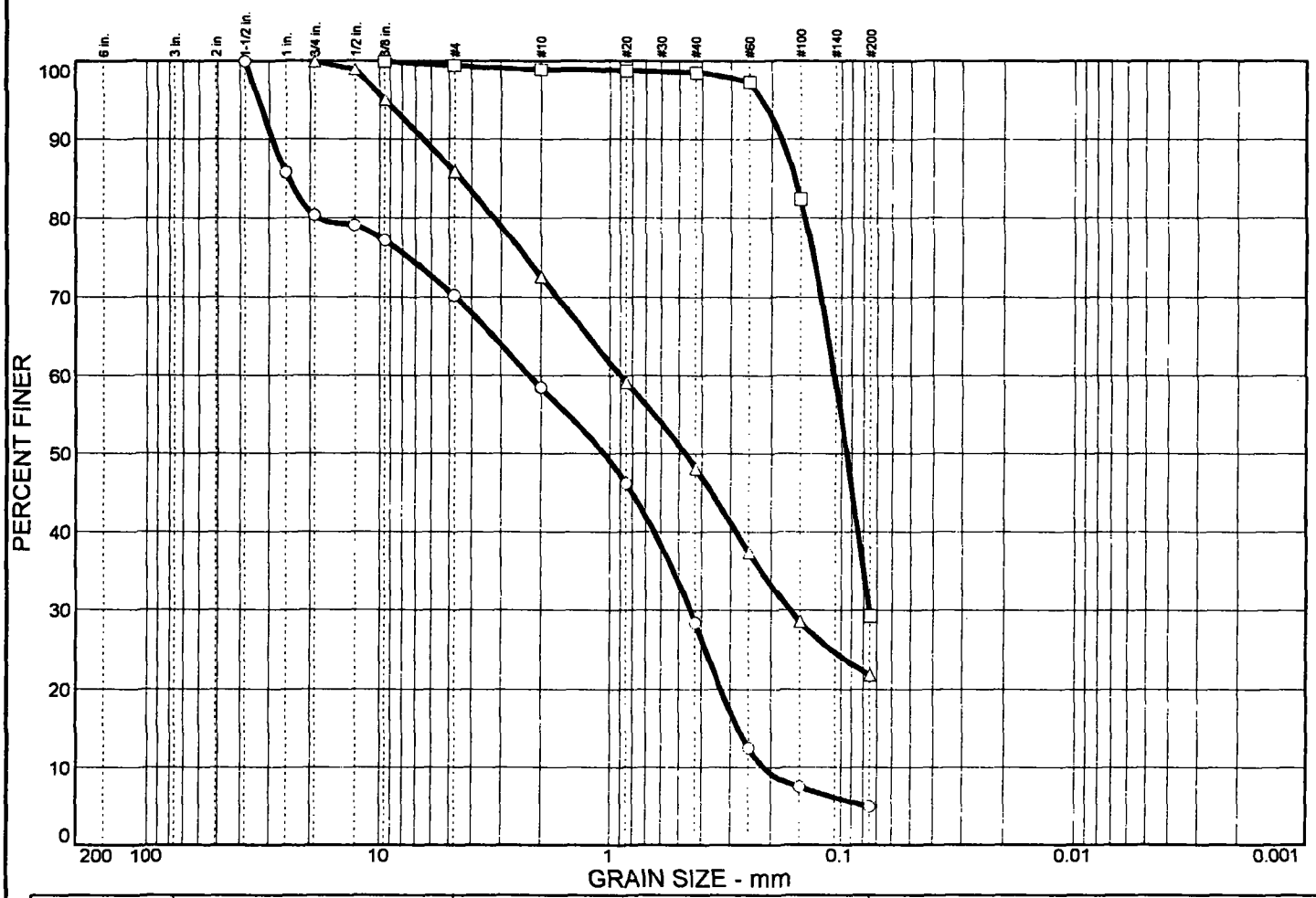
Figure No. B-3

Particle Size Distribution Report



	% COBBLES	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	19.7	10.2	11.6	30.1	23.5	4.9	
□	0.0	0.0	0.6	0.5	0.4	69.2	29.3	
△	0.0	0.0	14.2	13.2	24.5	26.3	21.8	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			24.7	2.24	1.07	0.447	0.278	0.214	0.42	10.44
□			0.159	0.107	0.0947	0.0756				
△			4.49	0.896	0.472	0.165				

MATERIAL DESCRIPTION	USCS	NAT. MOIST.
○ Gravelly SAND	SP	13%
□ Silty, fine SAND	SM	25%
△ Gravelly, silty, fine SAND	SM	11%

Remarks:

○ Small sample size

□

△ Small sample size

Project: Port Angeles Graving Dock

Client: Washington State Department of Transportation

○ **Source:** H-2-02 **Sample No.:** S-3

□ **Source:** H-2-02 **Sample No.:** S-7

△ **Source:** H-2-02 **Sample No.:** S-11

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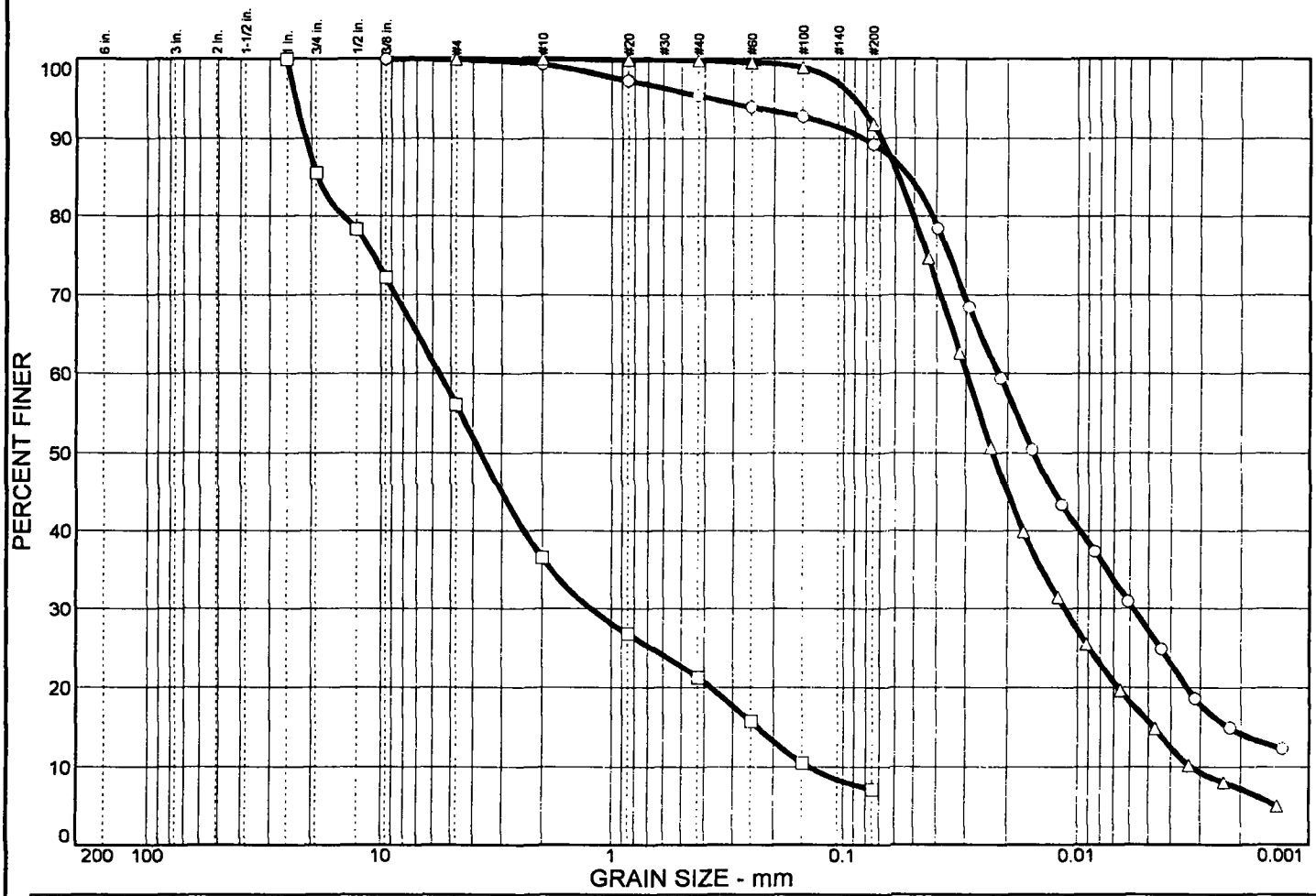
Figure No. B-4

Particle Size Distribution Report



	% COBBLES	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	0.0	0.1	0.6	3.9	6.2	62.0	27.2
□	0.0	14.5	29.4	19.6	15.3	14.0	7.2	
△	0.0	0.0	0.0	0.0	0.2	8.0	76.1	15.7

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			0.0526	0.0218	0.0154	0.0058	0.0023			
□			18.8	5.61	3.70	1.23	0.235	0.140	1.92	39.96
△			0.0583	0.0298	0.0230	0.0114	0.0048	0.0033	1.32	9.11

MATERIAL DESCRIPTION	USCS	NAT. MOIST.
○ Slightly sandy, clayey SILT	ML	28%
□ Slightly silty, very gravelly SAND	SW-SM	11%
△ Slightly sandy, clayey SILT	ML	23%

Remarks:

○

□ Small sample size

△

Project: Port Angeles Graving Dock

Client: Washington State Department of Transportation

○ **Source:** H-2-02 **Sample No.:** S-15

□ **Source:** H-3-02 **Sample No.:** S-5

△ **Source:** H-3-02 **Sample No.:** S-8

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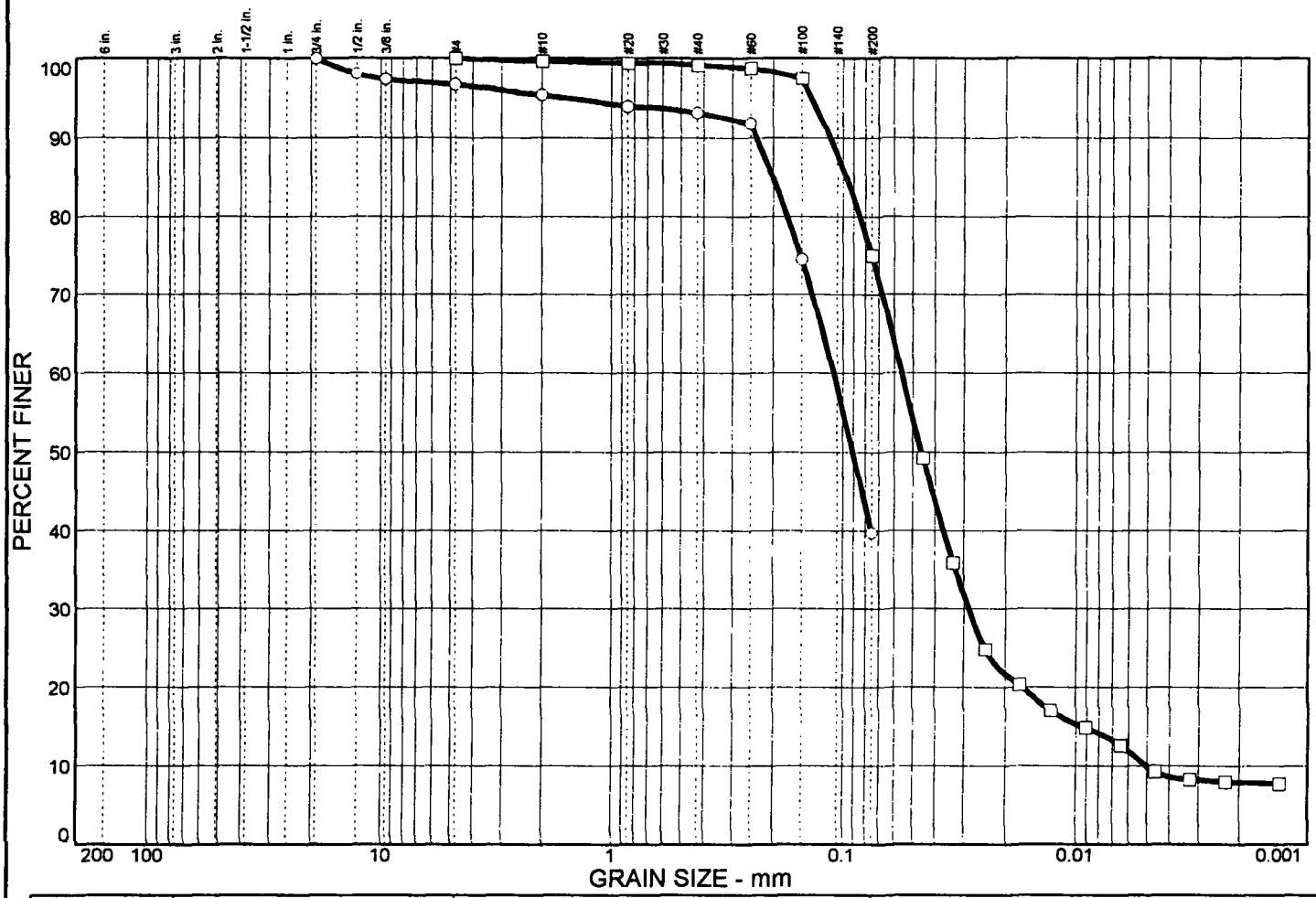
Figure No. B-5

Particle Size Distribution Report



	% COBBLES	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	0.0	3.4	1.3	2.2	53.4	39.7	
□	0.0	0.0	0.0	0.4	0.5	24.1	65.0	10.0

X	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			0.201	0.110	0.0906					
□			0.0968	0.0558	0.0459	0.0287	0.0095	0.0050	2.94	11.11

MATERIAL DESCRIPTION	USCS	NAT. MOIST.
○ Very silty, fine SAND	SM	21%
□ Slightly clayey, sandy SILT	ML	36%

Remarks:

○

□

Project: Port Angeles Graving Dock

Client: Washington State Department of Transportation

○ **Source:** H-4-02 **Sample No.:** S-5

□ **Source:** H-4-02 **Sample No.:** S-10

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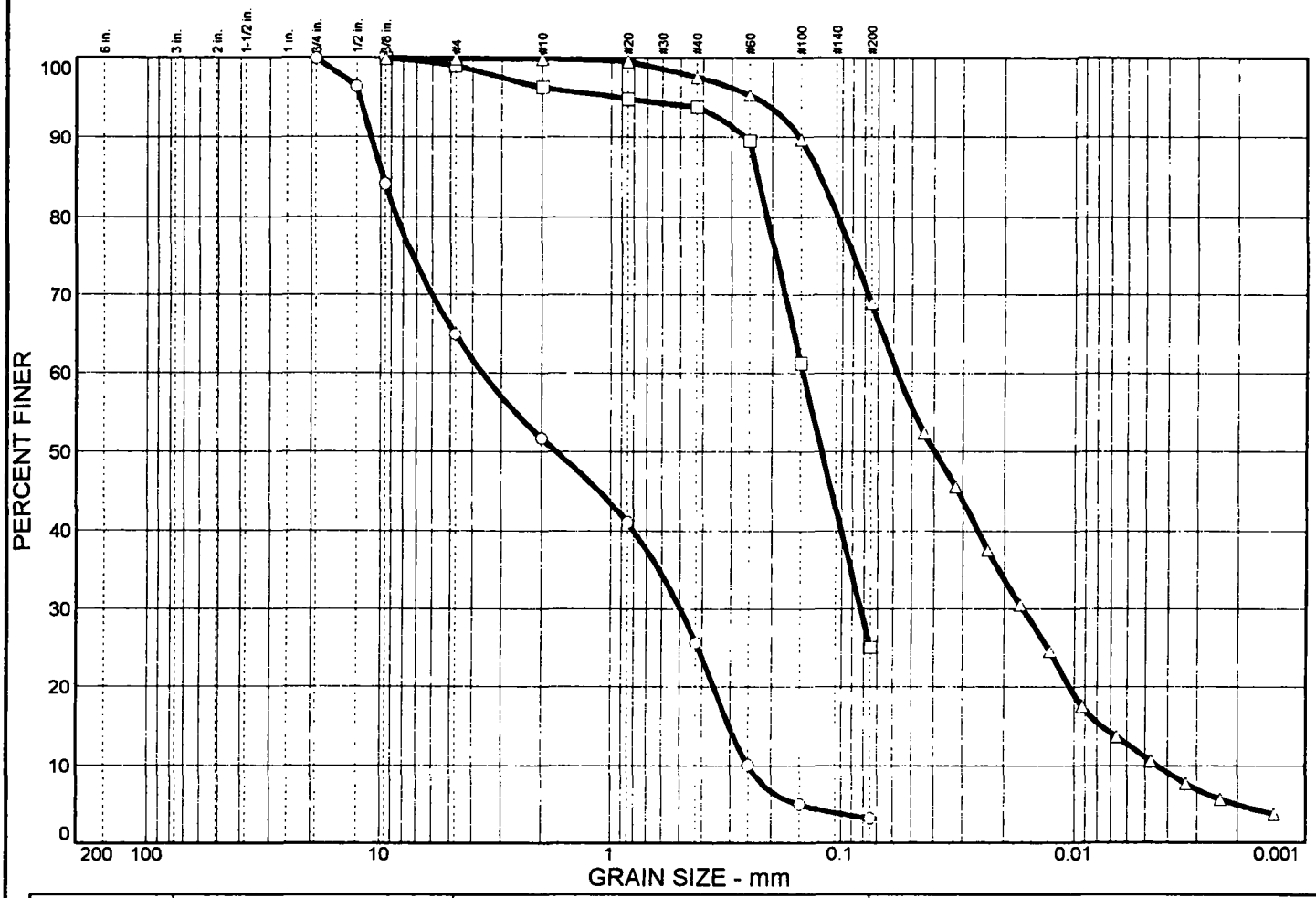
Figure No. B-6

Particle Size Distribution Report



	% COBBLES	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	0.0	35.1	13.2	26.1	22.4	3.2	
□	0.0	0.0	1.0	2.7	2.5	68.8	25.0	
△	0.0	0.0	0.1	0.0	2.3	28.7	57.8	11.1

X	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			9.75	3.62	1.73	0.497	0.304	0.250	0.27	14.49
□			0.230	0.146	0.122	0.0827				
△			0.123	0.0577	0.0398	0.0166	0.0076	0.0044	1.08	13.01

MATERIAL DESCRIPTION	USCS	NAT. MOIST.
○ Very gravelly SAND	SP	12%
□ Silty, fine SAND	SM	20%
△ Slightly clayey, very sandy SILT	ML	25%

Remarks:

○ Small sample size

□

△

Project: Port Angeles Graving Dock

Client: Washington State Department of Transportation

○ **Source:** H-5-02 **Sample No.:** S-3

□ **Source:** H-5-02 **Sample No.:** S-5

△ **Source:** H-5-02 **Sample No.:** S-14

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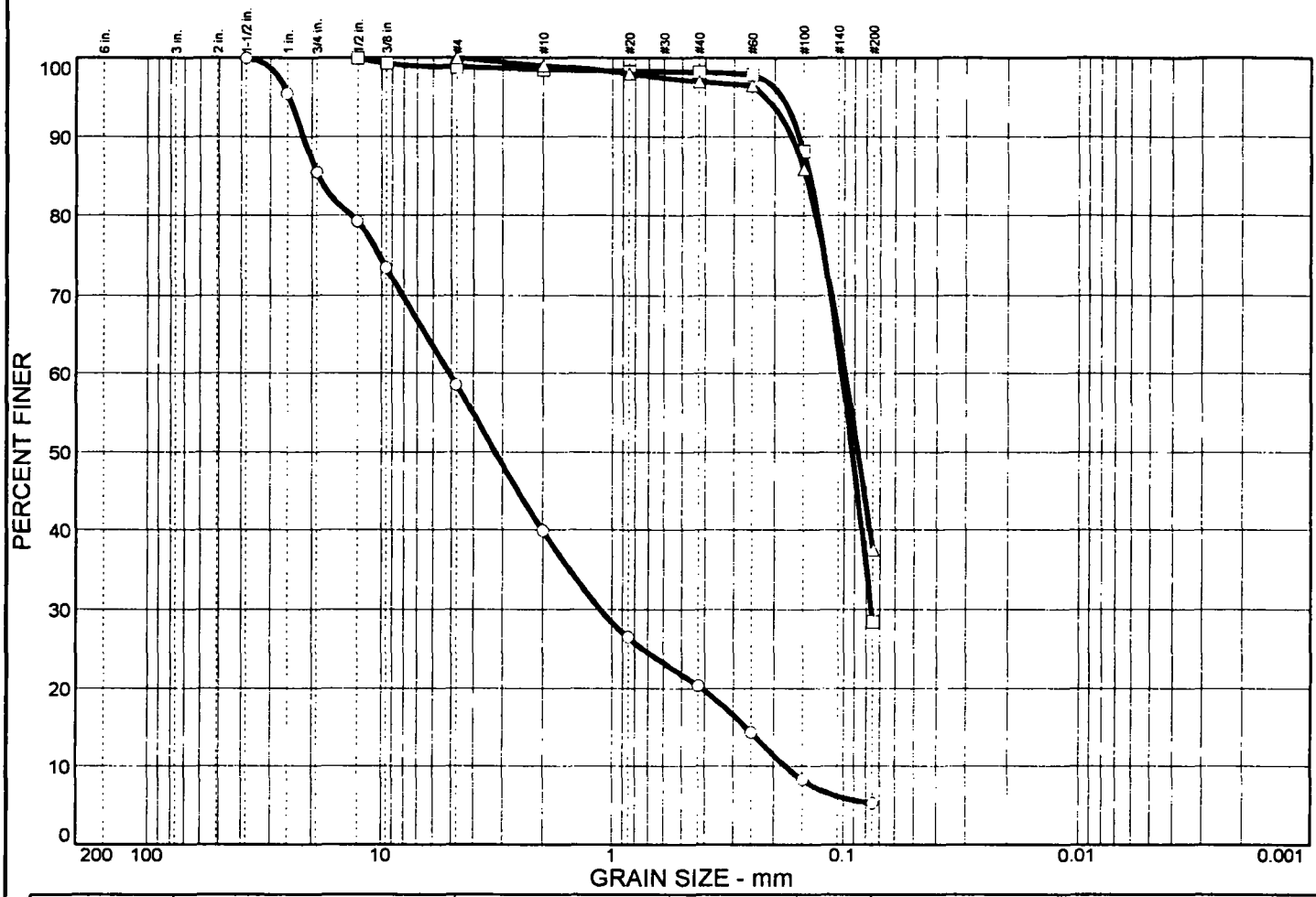
Figure No. B-7

Particle Size Distribution Report



	% COBBLES	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	14.6	26.9	18.6	19.6	15.0	5.3	
□	0.0	0.0	1.2	0.3	0.3	69.7	28.5	
△	0.0	0.0	0.0	1.0	2.0	59.5	37.5	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			18.8	5.09	3.23	1.12	0.264	0.177	1.40	28.72
□			0.141	0.102	0.0923	0.0761				
△			0.147	0.0988	0.0871					

MATERIAL DESCRIPTION	USCS	NAT. MOIST.
○ Slightly silty, very gravelly SAND	SW-SM	8%
□ Silty, fine SAND	SM	34%
△ Very silty, fine SAND	SM	25%

Remarks:

○ Small sample size

□

△

Project: Port Angeles Graving Dock

Client: Washington State Department of Transportation

○ **Source:** H-6-02 **Sample No.:** S-3

□ **Source:** H-6-02 **Sample No.:** S-6

△ **Source:** H-6-02 **Sample No.:** S-10

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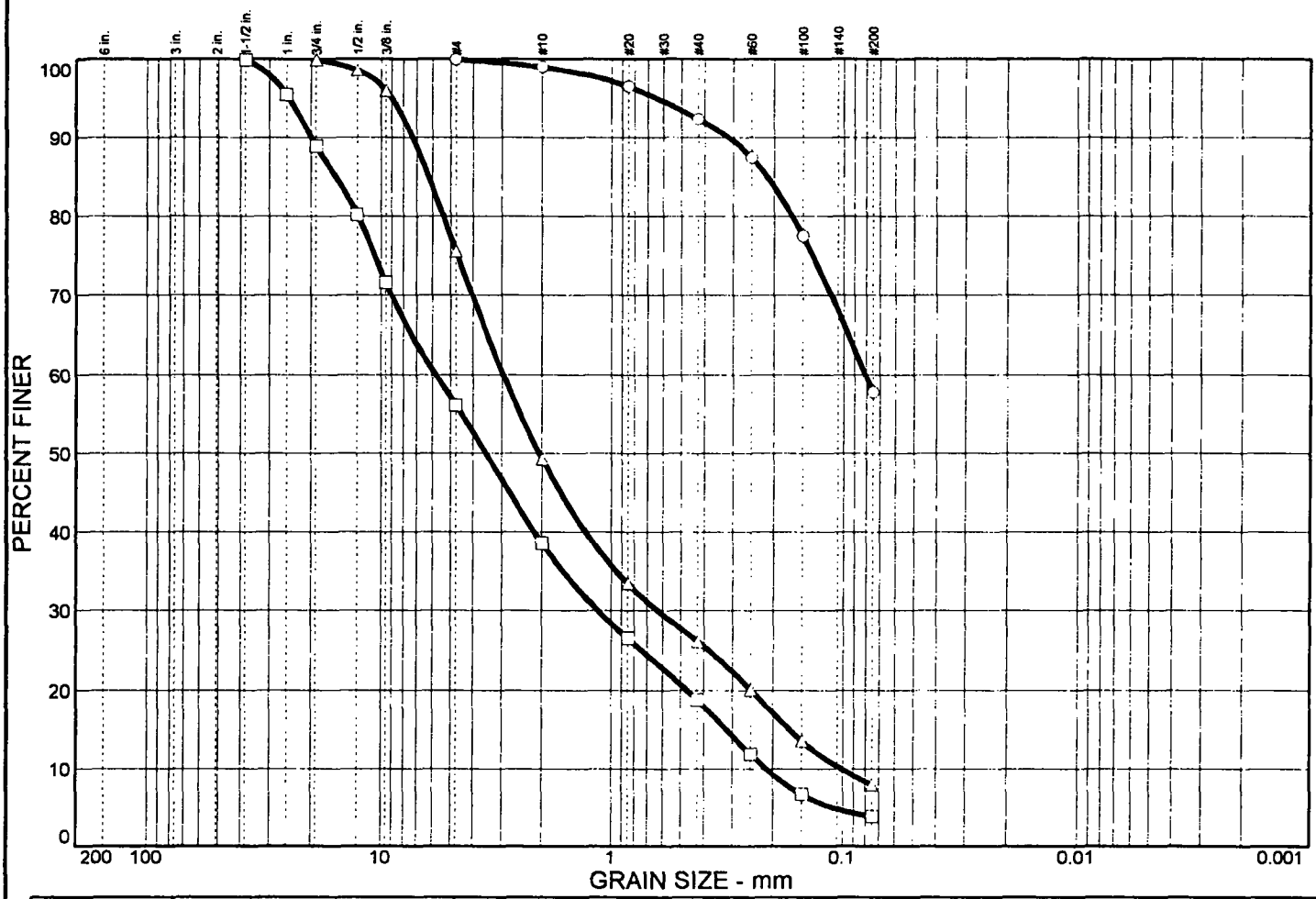
Figure No. B-8

Particle Size Distribution Report



	% COBBLES	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	0.0	0.0	1.0	6.7	34.5	57.8	
□	0.0	11.1	32.7	17.6	19.8	14.9	3.9	
△	0.0	0.0	24.4	26.3	23.2	18.2	7.9	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			0.214	0.0806						
□			15.8	5.77	3.52	1.14	0.317	0.213	1.05	27.07
△			6.23	2.95	2.06	0.627	0.169	0.101	1.32	29.18

MATERIAL DESCRIPTION	USCS	NAT. MOIST.
○ Very sandy SILT	ML	19%
□ Very gravelly SAND	SW	9%
△ Slightly silty, gravelly SAND	SW-SM	12%

Remarks:

○

□ Small sample size

△ Small sample size

Project: Port Angeles Graving Dock

Client: Washington State Department of Transportation

○ **Source:** H-6-02 **Sample No.:** S-13

□ **Source:** H-7-02 **Sample No.:** S-4

△ **Source:** H-7-02 **Sample No.:** S-10

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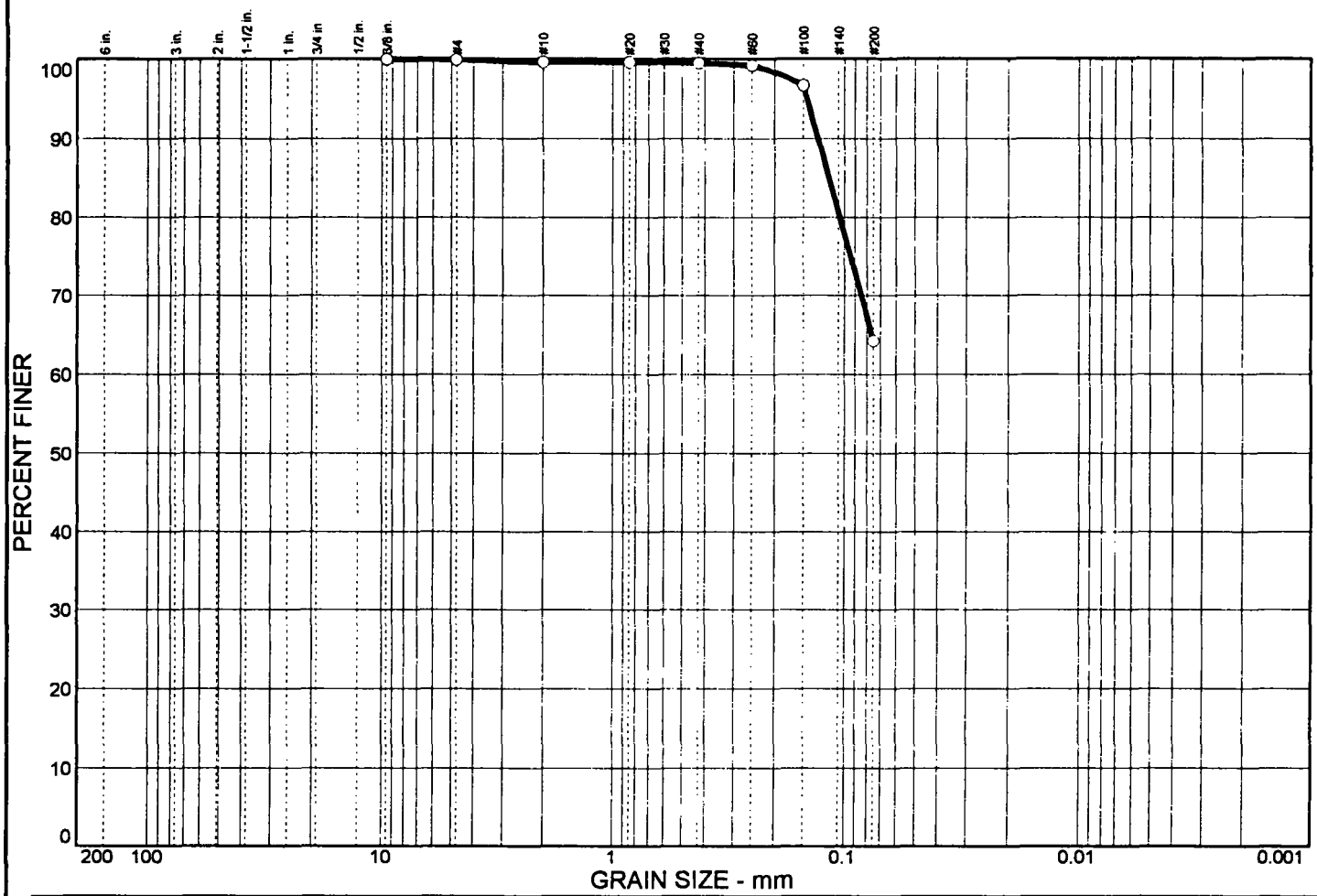
Figure No. B-9

Particle Size Distribution Report



	% COBBLES	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	0.0	0.1	0.3	0.1	35.2	64.3	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			0.116							

MATERIAL DESCRIPTION	USCS	NAT. MOIST.
○ Very sandy SILT	ML	36%

Remarks:

○

Project: Port Angeles Graving Dock

Client: Washington State Department of Transportation

○ **Source:** HC-NW-02 **Sample No.:** S-9

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Figure No. ***B-10***

APPENDIX C
PREVIOUS EXPLORATION LOGS

Boring Log & Monitoring Well HC-NE-02 (HC-NE-PA)

Northing (ft): 4232060.65
Easting (ft): 998890.13

Soil Descriptions
Approximate Ground Surface Elevation in Feet: 14.5

(Loose), damp, brown, slightly silty SAND with abundant wood, trace gravel and strong creosote-like odor.

Medium dense, moist, brown WOOD CHIPS with scattered Sand and Gravel. Strong creosote-like odor.

Very loose, wet, gray, slightly gravelly, fine SAND with trace wood and burnt odor.

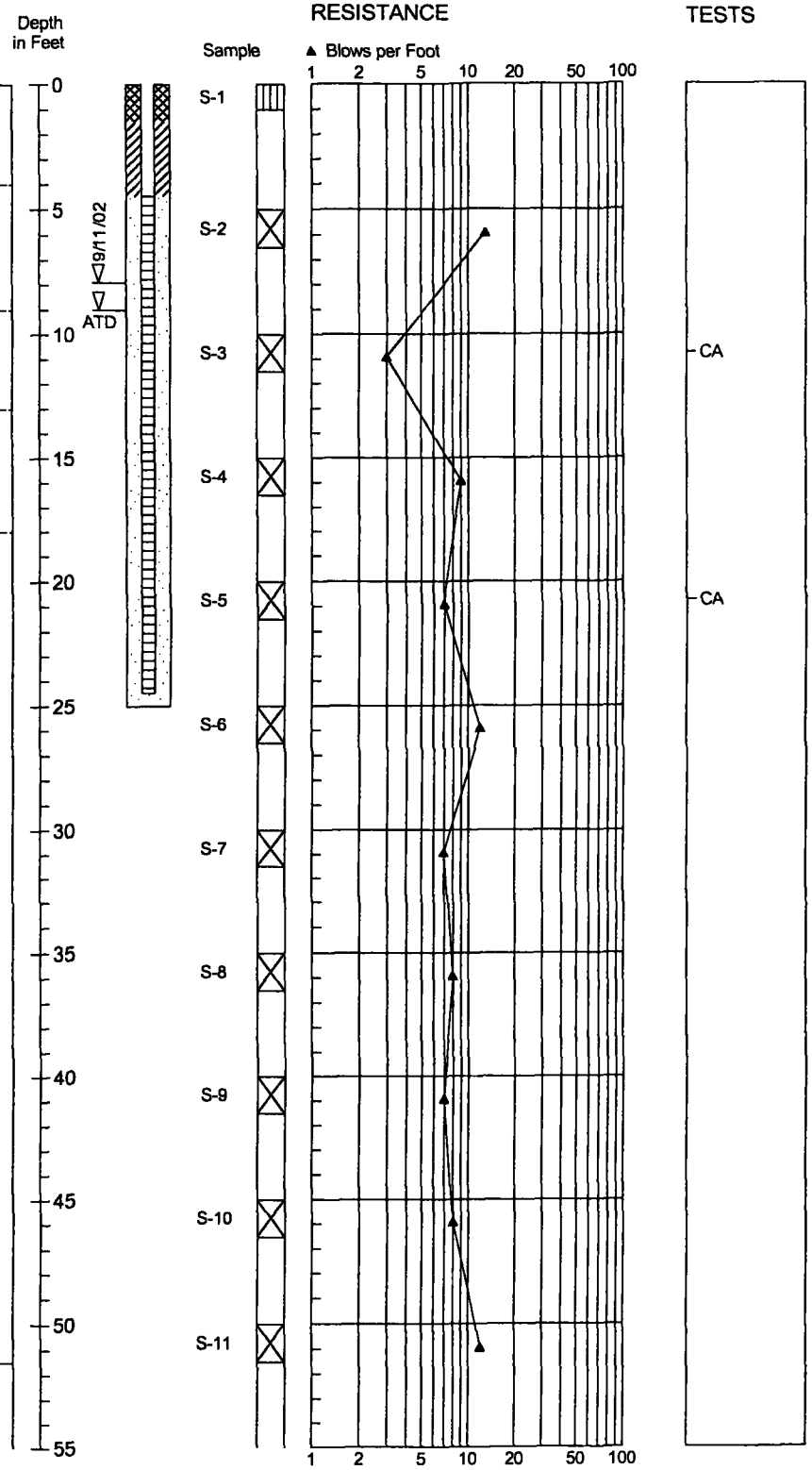
Loose, wet, gray-brown WOOD CHIPS with trace Sand and Gravel. Minor sheen and burnt odor.

Loose to medium dense, wet, gray, gravelly, fine to medium SAND with scattered wood and shell fragments.

Abundant shell fragments.

Bottom of Boring at 51.5 Feet.
Completed 09/10/02.

Groundwater sample collected for chemical analysis.



BORING LOG 7794BL.GPJ HC_CORP.GDT 1/28/03

HARTCROWSER

7794 09/02

Figure C-1

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

Boring Log & Monitoring Well HC-SE-02 (HC-SE-PA)

Northing (ft): 423100.99
Easting (ft): 998644.41

Soil Descriptions
Approximate Ground Surface Elevation in Feet: 11.3

Grass over (medium dense), dry, light brown, silty, very gravelly SAND.

Black, oily SAND in cuttings with strong hydrocarbon odor.

Loose, wet, dark brown, very gravelly SAND with staining and hydrocarbon odor.

Medium dense, wet, gray, gravelly to very gravelly, medium to coarse SAND.

No odor below this depth.

Abundant shell fragments.

Stiff, moist, gray, slightly clayey SILT.

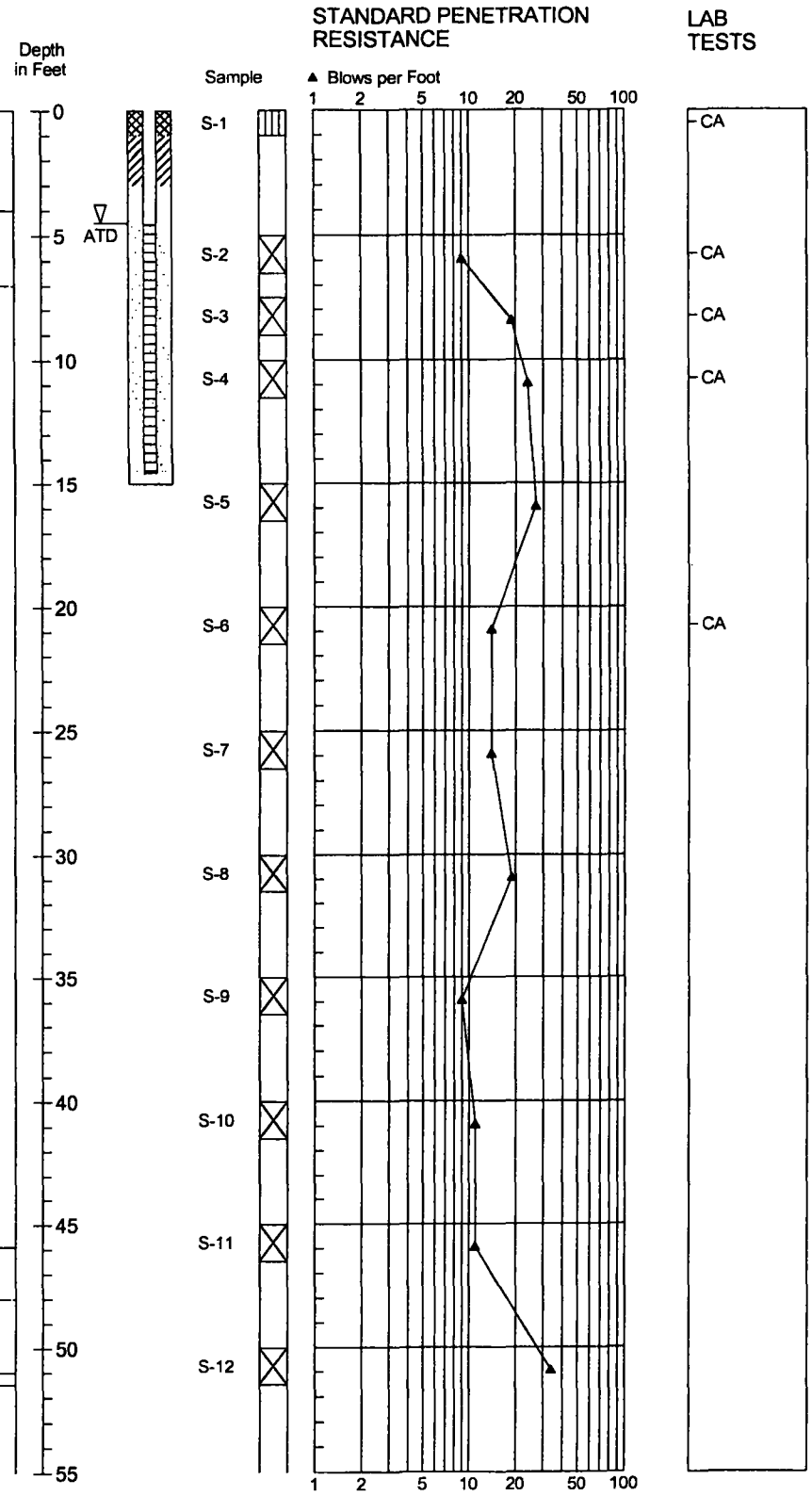
Medium dense, moist, gray, fine SAND.

Very stiff, damp, gray, clayey SILT.

Bottom of Boring at 51.5 Feet.

Completed 09/10/02.

Groundwater sample collected for chemical analysis.



BORING LOG 7794BL.GPJ HC_CORP.GDT 1/28/03

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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7794 09/02

Figure C-2

Boring Log & Monitoring Well HC-C-02 (HC-C-PA)

Northing (ft): 423465.41
Easting (ft): 998680.97

Soil Descriptions
Approximate Ground Surface Elevation in Feet: 13.2

(Loose), damp, brown, slightly silty SAND with abundant wood chips.

Medium dense, damp to wet, gray-green grading brown, gravelly SAND to sandy GRAVEL.

Medium dense, wet, red-brown, very gravelly, coarse SAND with scattered wood fibers.

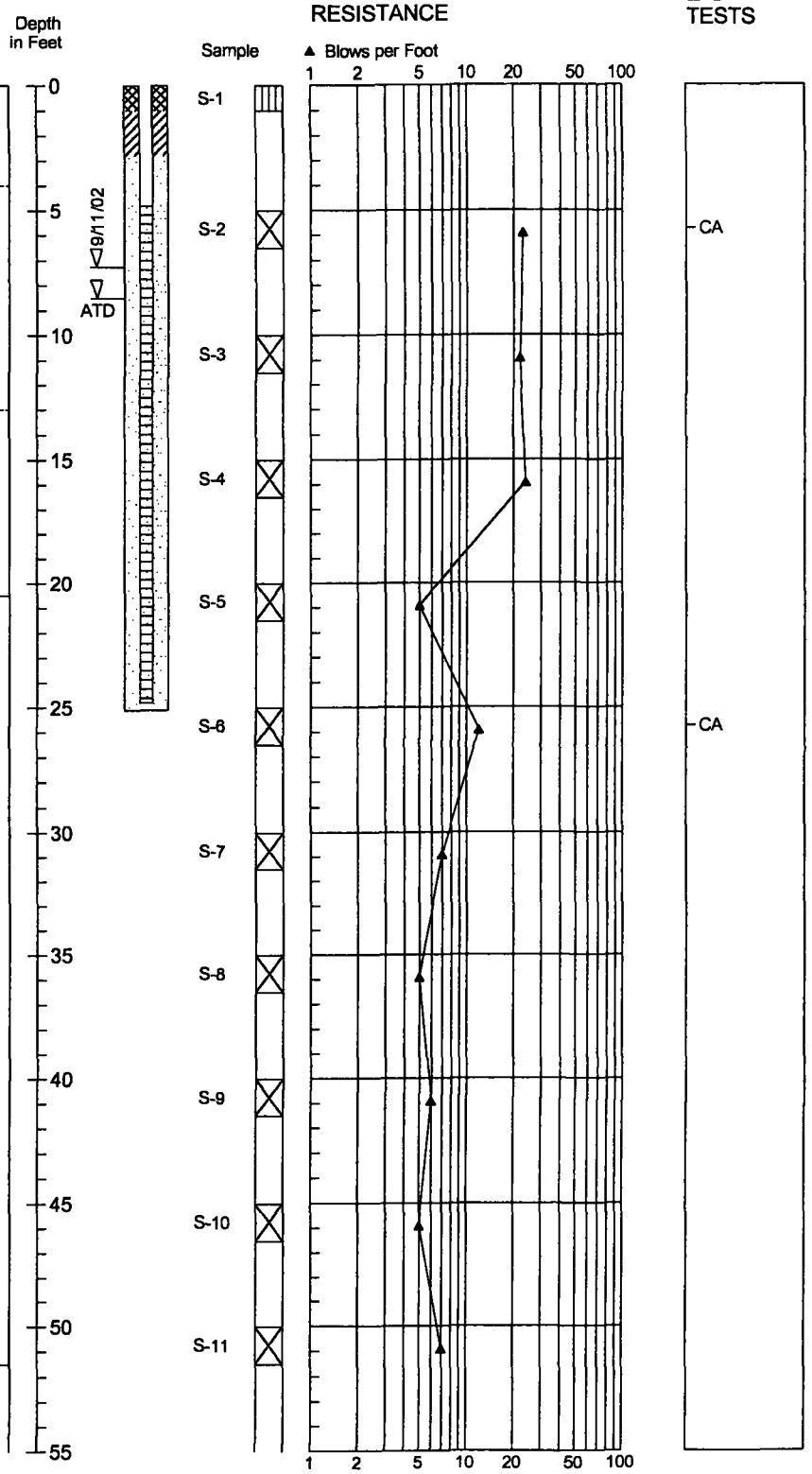
Loose, brown, sandy GRAVEL.

Loose to medium dense, wet, gray, slightly gravelly to non-gravelly SAND.

Grading to loose, medium SAND with scattered shell fragments.

Bottom of Boring at 51.5 Feet.
Completed 09/10/02.

Groundwater sample collected for chemical analysis.



BORING LOG 7794BL.GPJ HC_CORP.GDT 1/28/03

HARTCROWSER

7794 09/02

Figure C-3

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

Boring Log & Monitoring Well HC-NW-02 (HC-NW-PA)

Northing (ft): 423798.58
Easting (ft): 998588.4

Soil Descriptions
Approximate Ground Surface Elevation in Feet: 13.8

(Very loose), damp, brown WOOD CHIPS with trace of Sand.

Medium dense, damp to wet, red-brown grading gray SAND with abundant Wood Chips.

Medium dense, wet, gray, gravelly SAND with layers of fine Sand.

Medium dense, wet, gray, coarse sandy GRAVEL with scattered shell fragments.

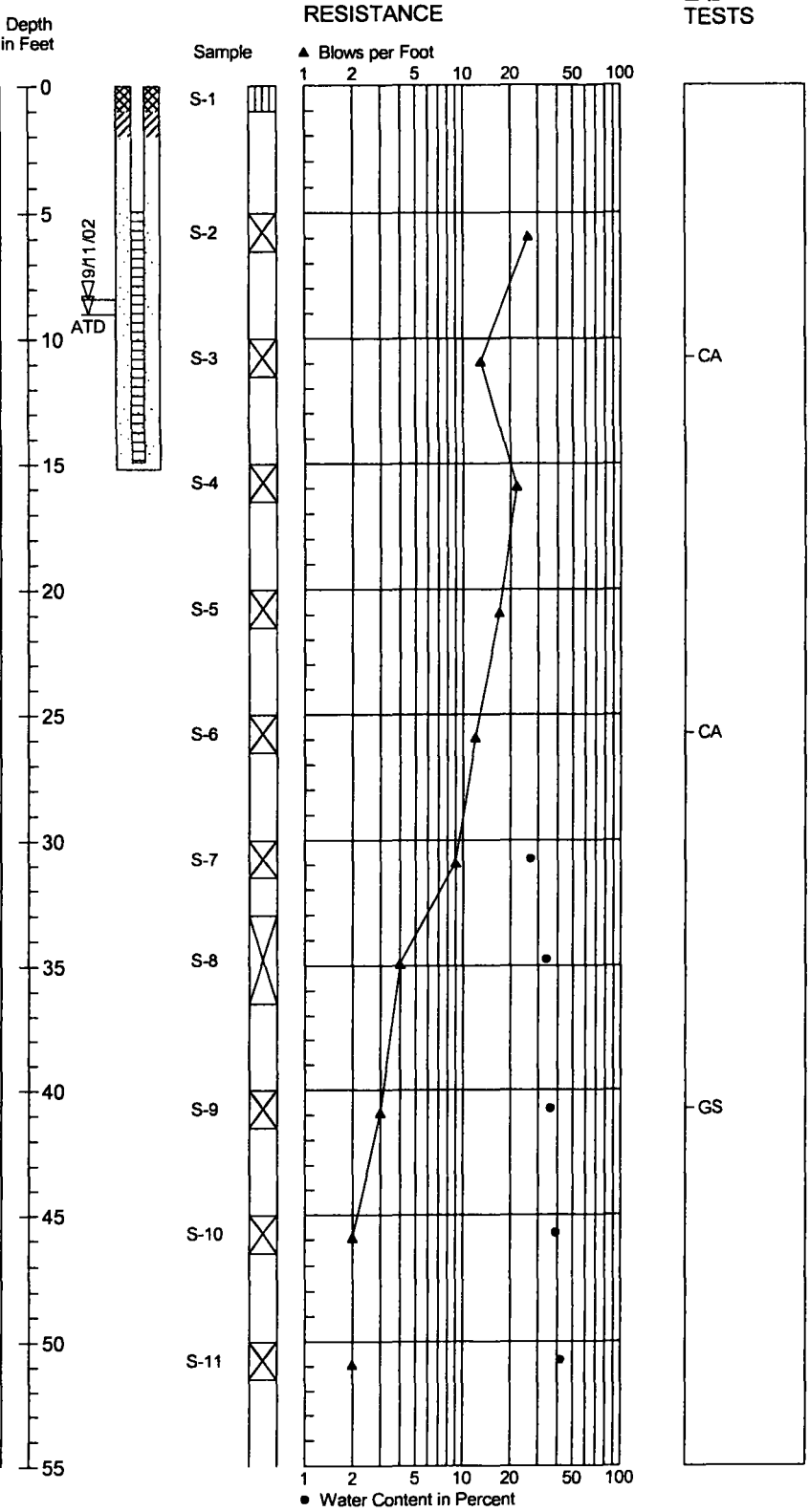
Medium dense, wet, gray, fine to medium SAND with scattered shell fragments.

Grading to loose with slight hydrogen sulfide odor.

Soft, wet, gray, very sandy SILT.

Pieces of fibrous wood.

Bottom of Boring at 51.5 Feet.
Completed 09/10/02.



Groundwater sample collected for chemical analysis.

BORING LOG 7794BL.GPJ HC_CORP.GDT 1/28/03

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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Figure C-4

Boring Log & Monitoring Well HC-SW-02 (HC-SW-PA)

Northing (ft): 423642.39
Easting (ft): 998315.1

Soil Descriptions
Approximate Ground Surface Elevation in Feet: 12.1

(Loose), damp, brown, slightly silty, slightly gravelly SAND with abundant wood chips.

Medium dense to dense, damp to wet, dark brown to gray, gravelly to very gravelly SAND.

Loose, wet, gray, fine SAND with medium SAND layers.

Scattered shell fragments.

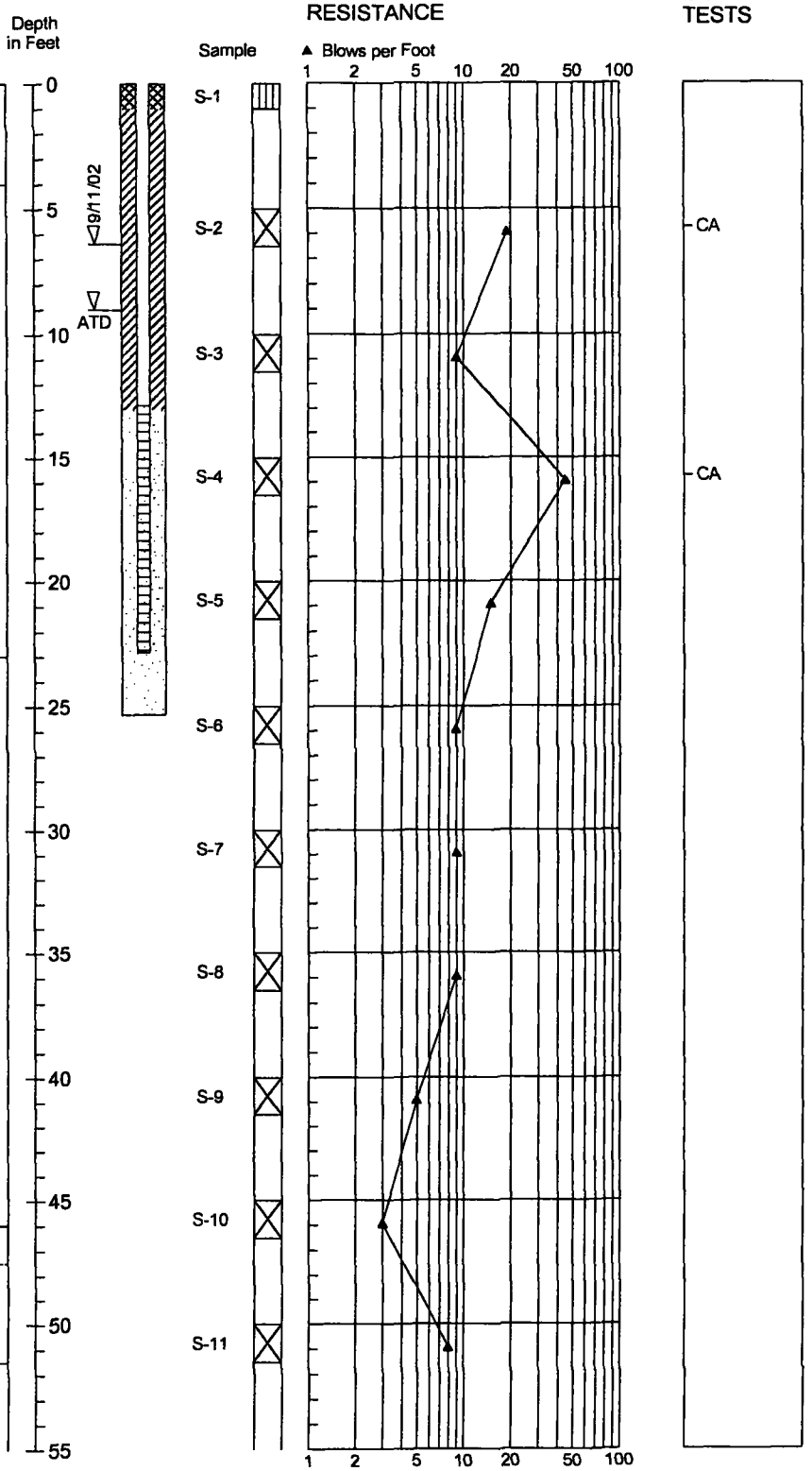
Slight hydrogen sulfide odor.

WOOD CHIPS layer.

Loose, wet, gray, fine to medium SAND with scattered shells and slight hydrogen sulfide odor.

Bottom of Boring at 51.5 Feet.
Completed 09/10/02.

Groundwater sample collected for chemical analysis.



BORING LOG 7794BL.GPJ HC_CORP.GDT 1/28/03

HARTCROWSER

7794 09/02

Figure C-5

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

Boring Log B-1

SOIL DESCRIPTIONS

Ground Surface Elevation in Feet

Medium dense, moist, gray to black, medium to fine SAND with trace gravel and silt. (FILL)

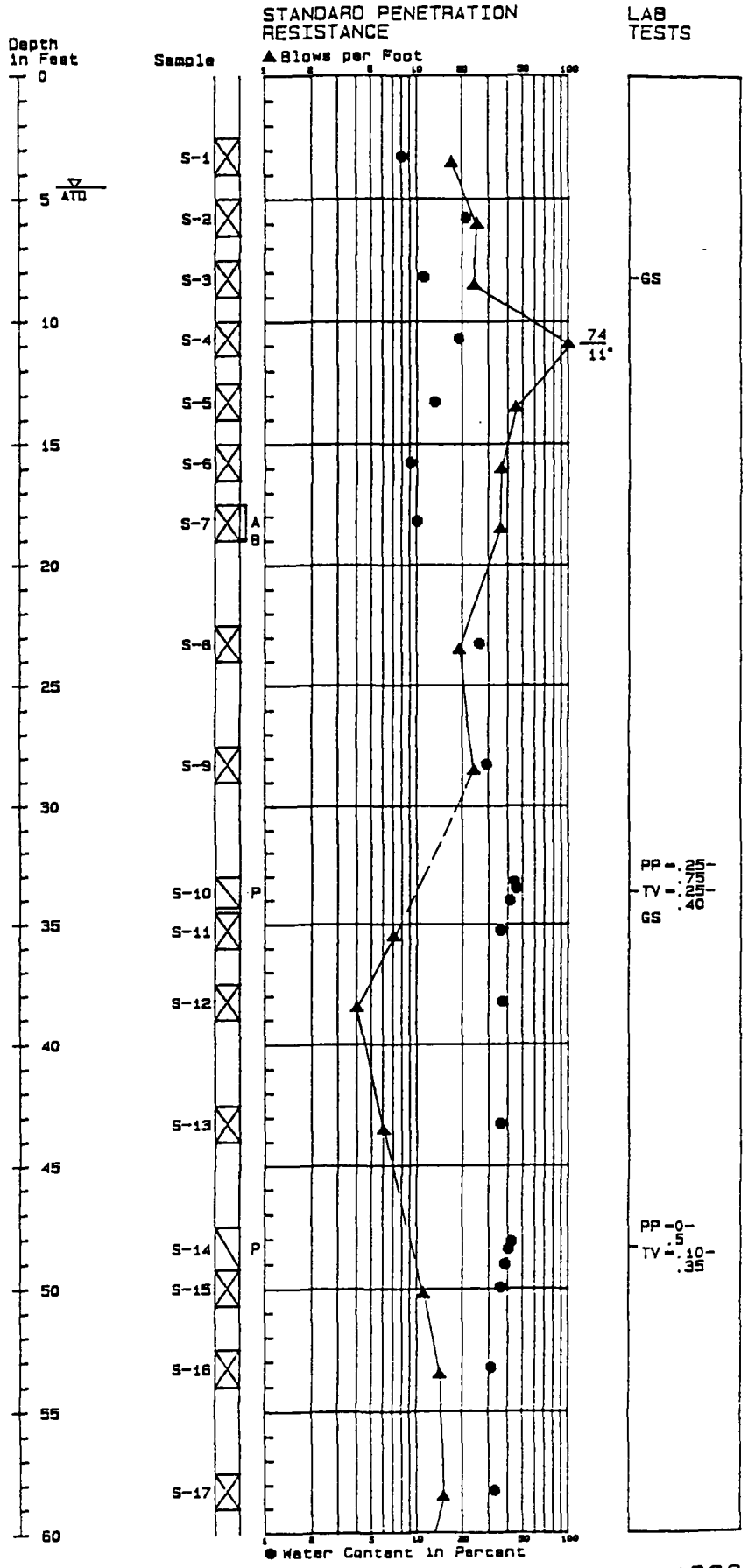
Medium dense, wet, gray, gravelly SAND and sandy GRAVEL.

- Grades dense.

Medium dense, wet, gray, silty, fine SAND with organics, wood, and shell fragments.

- Becomes loose to medium stiff, very silty SAND to fine sandy SILT.

- Becomes medium dense.



J-2160 May 1988

HART-CROWSER & associates, inc.

Sheet 1 of 2 Figure C-6

Boring Log B-1

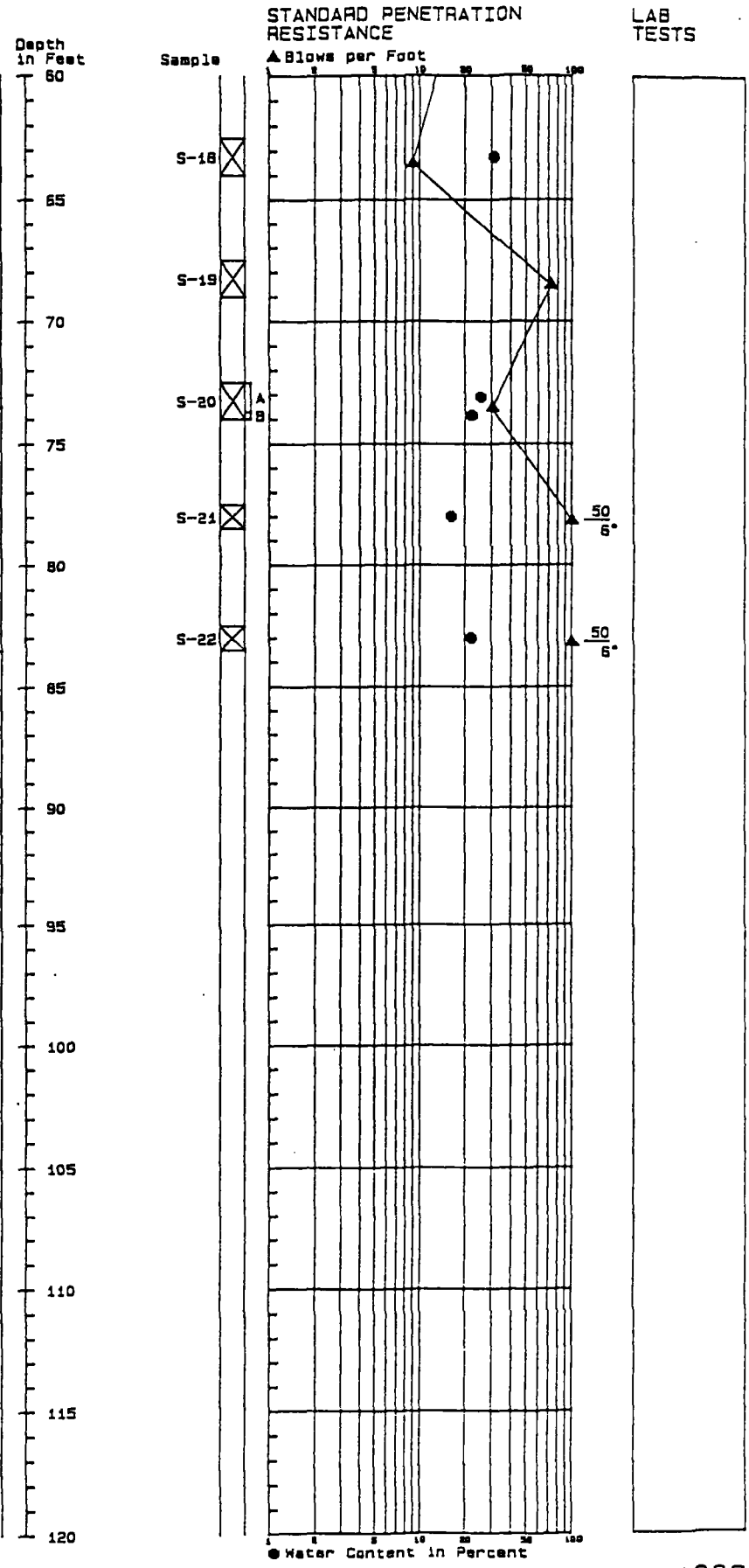
SOIL DESCRIPTIONS

Ground Surface Elevation in Feet

Loose to medium dense, wet, gray, silty, fine SAND with wood and shell fragments.

Very dense to hard, wet, gray, slightly gravelly, very silty SAND to fine sandy SILT.

Bottom of Boring at 83.5 Feet.
Completed 5/9/88.



1. Refer to Figure C-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Ground water level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

J-2160 May 1988

HART-CROWSER & associates, inc.

Sheet 2 of 2 Figure C-6

Boring Log B-2

SOIL DESCRIPTIONS

Ground Surface Elevation in Feet

Medium dense, moist to wet, oxidized brown to brown, gravelly, silty to very silty, fine SAND. (FILL)

Loose, wet, gray, gravelly SAND and black, silty, fine SAND interlayered. (FILL)

Loose, wet, brown and black, gravelly, silty SAND with substantial wood chips varying from <5% to nearly 100%. (FILL)

Medium dense to dense, wet, gray, gravelly SAND to very sandy GRAVEL.

- Grades very dense.

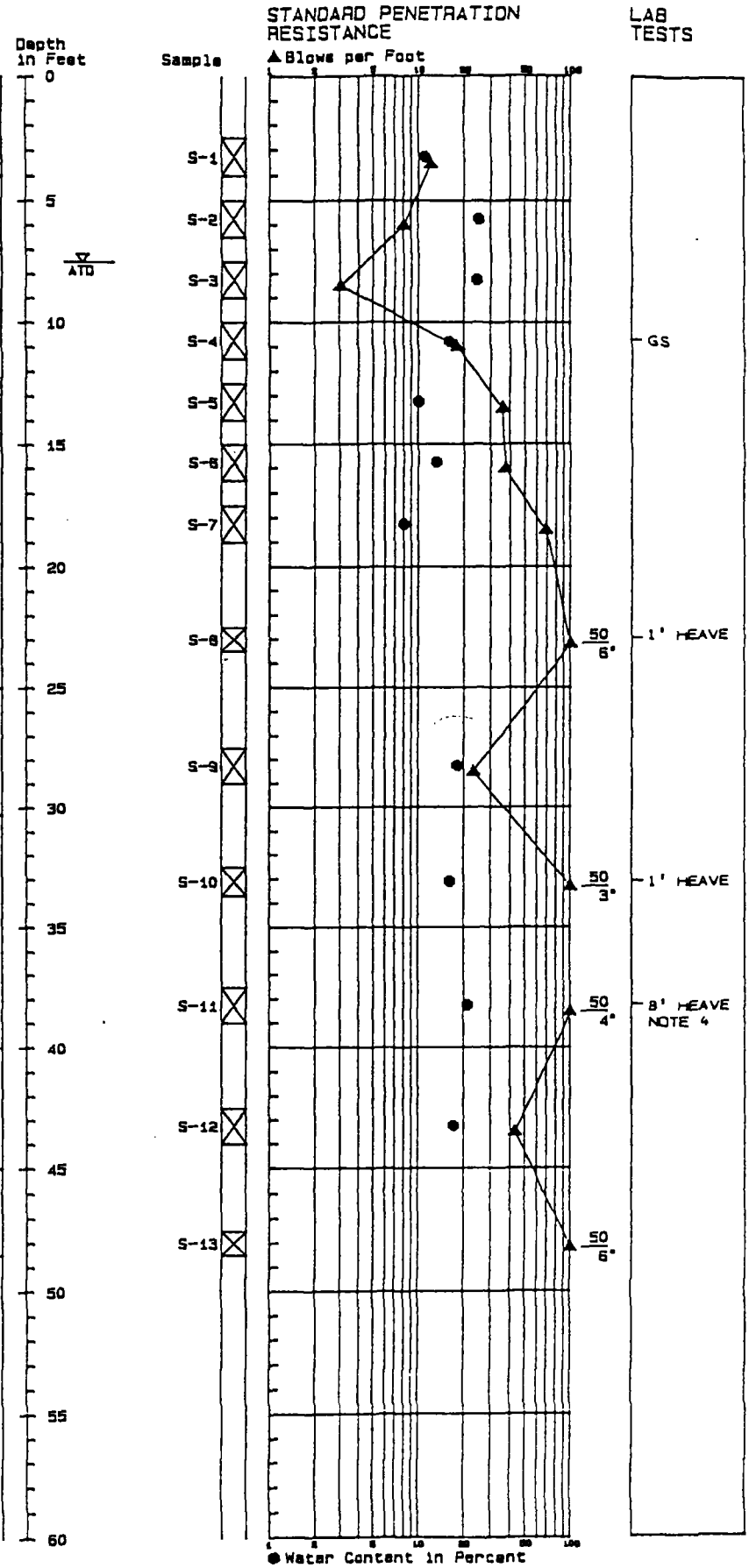
Medium dense to very dense, wet, gray, trace to slightly silty, fine SAND.

Very dense, wet, gray, trace to slightly gravelly, fine to medium SAND to SAND with trace wood.

Dense, wet, gray, silty, fine SAND with wood and shell fragments.

Very dense, wet, dark gray, interbedded, very silty, fine SAND and fine sandy SILT with trace gravel.

Bottom of Boring at 48.5 Feet. Completed 5/9/88.



1. Refer to Figure C-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Ground water level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.
4. BLOW COUNT MAY NOT BE REPRESENTATIVE OF ACTUAL MATERIAL DENSITY.

J-2160 May 1988

HART-CROWSER & associates, inc.

Figure C-7

Boring Log B-3

SOIL DESCRIPTIONS

Ground Surface Elevation in Feet

ASPHALT.

Loose, moist, dark gray to black, silty, gravelly SAND. (FILL)

(Medium dense to very dense), wet, gray, gravelly SAND to very sandy GRAVEL.

Medium dense, wet, gray, fine SAND with shell fragments, scattered fine gravel and wood.

- Grades to loose.

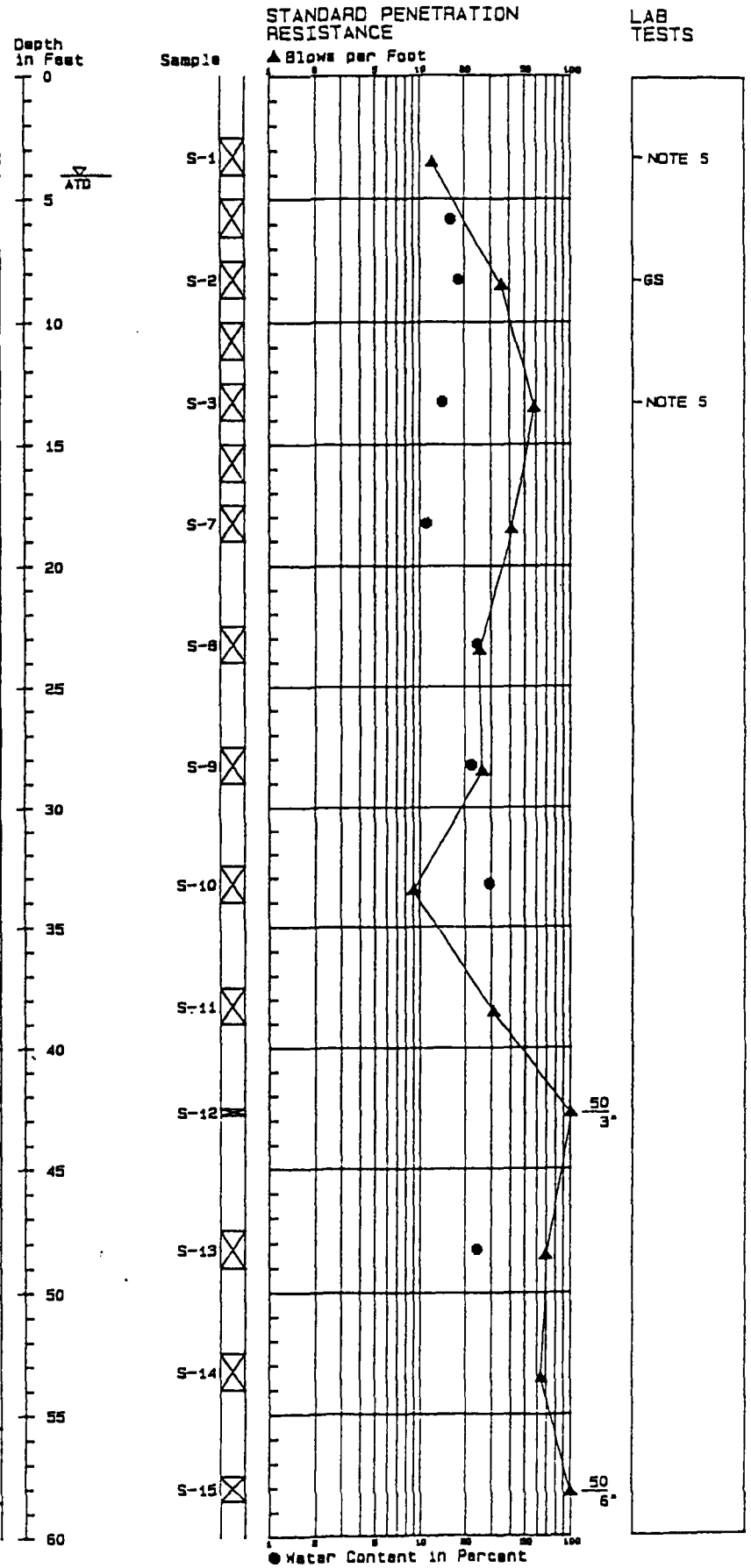
- Grades to dense.

Very dense, wet, gray, interbedded fine SAND and SILT with scattered gravels and shells.

Very dense, wet, gray, interbedded silty, fine SAND, gravelly, silty, fine SAND and hard, gray, moist to wet SILT.

Very dense, wet, gray, very gravelly SAND.

Bottom of Boring at 58.5 Feet.
Completed 5/10/88.



1. Refer to Figure C-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Ground water level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.
4. BLOW COUNT MAY NOT BE REPRESENTATIVE OF ACTUAL MATERIAL DENSITY.

J-2160 May 1988

HART-CROWSER & associates, inc.

Figure C-8

Boring Log B-4

SOIL DESCRIPTIONS

Ground Surface Elevation in Feet

Medium dense, moist to wet, black to gray, slightly silty, gravelly, medium to fine SAND with scattered rip rap, cobbles, and wood. (FILL)

Dense to very dense, wet, gray-brown, very gravelly SAND to very sandy GRAVEL.

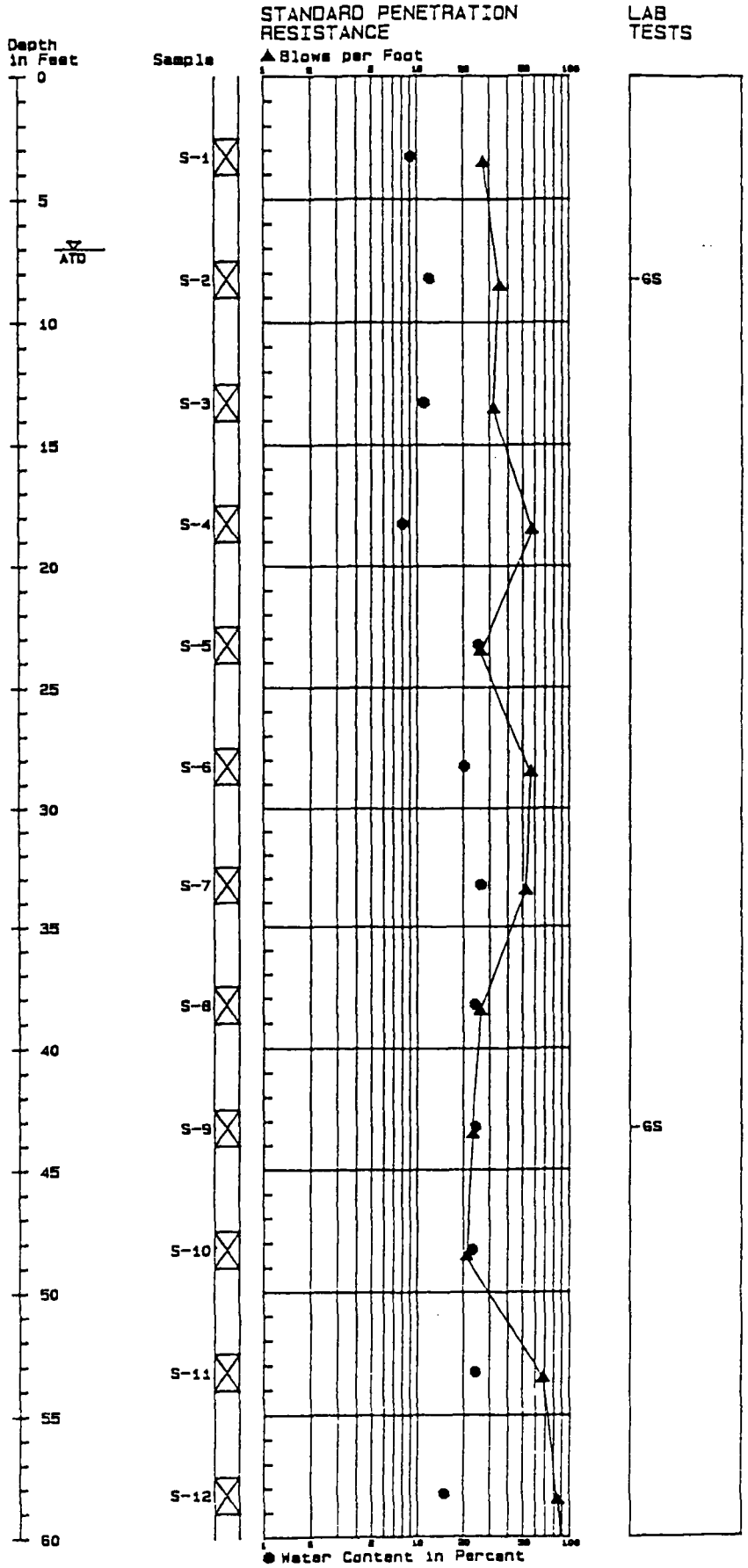
Medium dense, wet, gray, silty, very fine SAND with scattered gravel, wood, and shell fragments.

- Grades to very dense.

- Grades to medium dense.

Hard, wet, gray SILT to slightly fine sandy SILT with trace gravels.

Very dense, wet, gray, trace to slightly silty, slightly gravelly SAND to fine to medium SAND.



J-2160 May 1988

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Sheet 1 of 2 Figure C-9

Boring Log B-4

SOIL DESCRIPTIONS

Ground Surface Elevation in Feet

Very dense (hard), moist to wet, gray, interbedded, finely laminated SILT, fine sandy SILT with trace gravels, and slightly gravelly SAND.

Bottom of Boring at 73.5 Feet.
Completed 5/11/88.

Depth in Feet: 60, 65, 70, 75, 80, 85, 90, 95, 100, 105, 110, 115, 120

Sample: S-13, S-14, S-15

STANDARD PENETRATION RESISTANCE

▲ Blows per Foot

S-13: 88 / 10"
S-14: 85 / 11"
S-15: 50 / 6"

● Water Content in Percent

LAB TESTS

1. Refer to Figure C-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Ground water level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

J-2160 May 1988

HART-CROWSER & associates, inc.

Sheet 2 of 2 Figure C-9

Boring Log B-5

SOIL DESCRIPTIONS

Ground Surface Elevation in Feet

(Very dense), damp to moist, brown, silty, gravelly, medium to fine SAND with gravel to 6 inches and angular rip rap to 12 inches. (FILL)

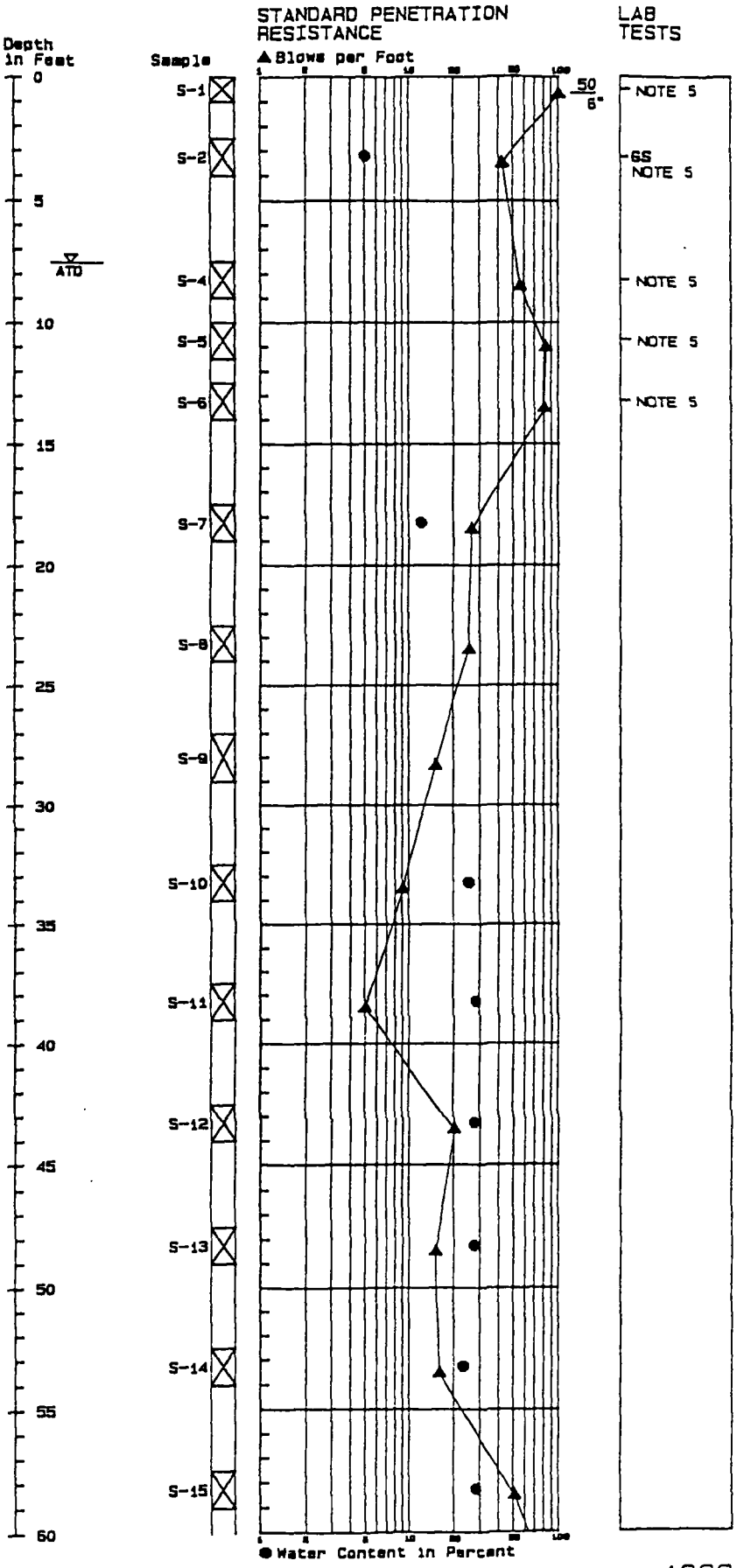
(Medium dense to dense), wet, gray, gravelly SAND becoming sandy GRAVEL.

Medium dense, wet, gray, trace to slightly silty, fine SAND with shell fragments.

- Becomes loose.

- Occasional wood.

Hard, moist, gray, fine sandy SILT.



J-2160 May 1988

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Sheet 1 of 2 Figure C-10

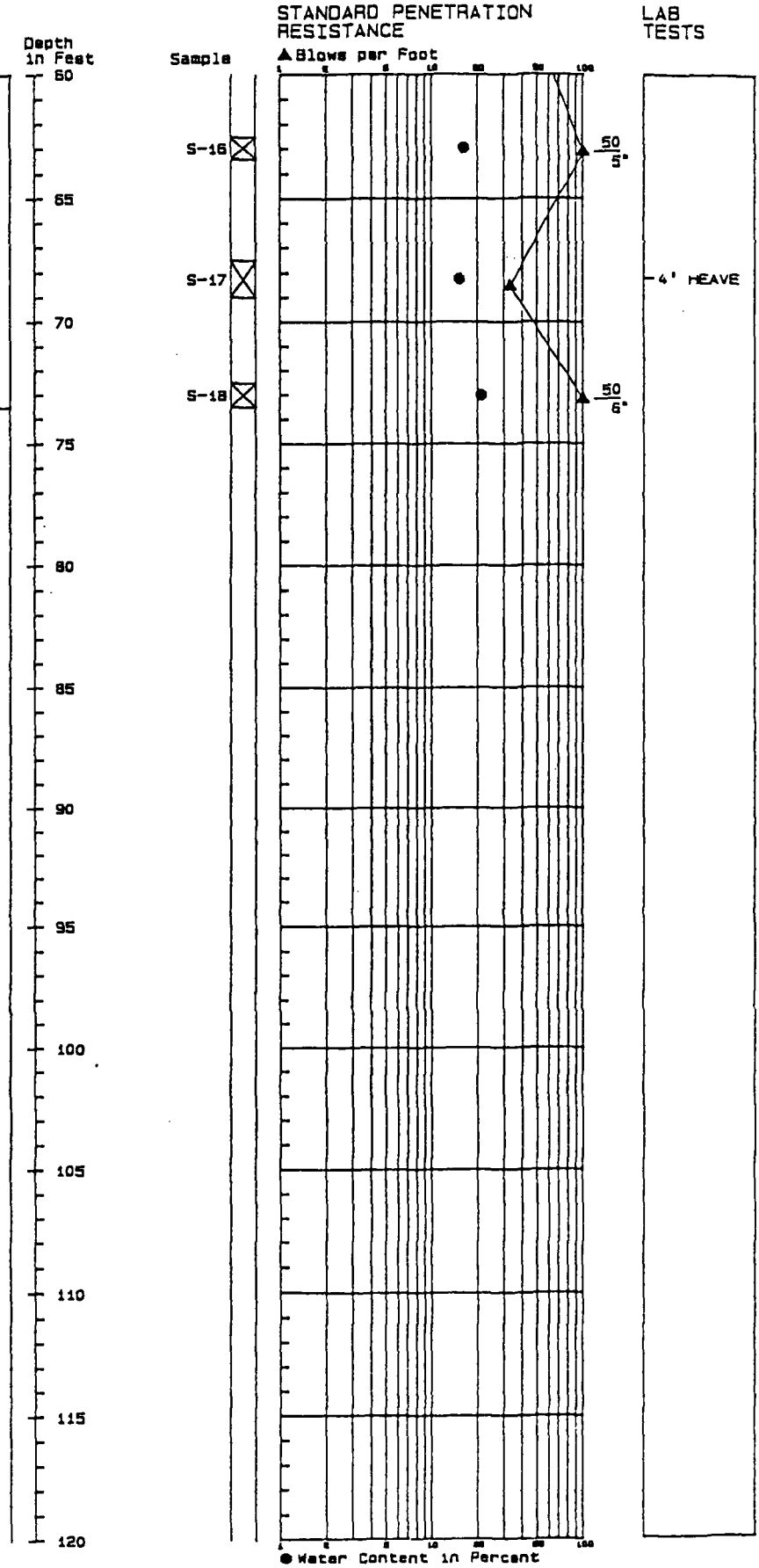
Boring Log B-5

SOIL DESCRIPTIONS

Ground Surface Elevation in Feet

Very dense (hard), wet, gray, interbedded, silty, fine SAND and fine sandy SILT with scattered shell fragments and wood.

Bottom of Boring at 73.5 Feet.
Completed 5/12/88.



1. Refer to Figure C-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Ground water level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.
4. BLOW COUNT MAY NOT BE REPRESENTATIVE OF ACTUAL MATERIAL DENSITY.

J-2160 May 1988

HART-CROWSER & associates, inc.

Sheet 2 of 2 Figure C-10

Boring Log B-6

SOIL DESCRIPTIONS

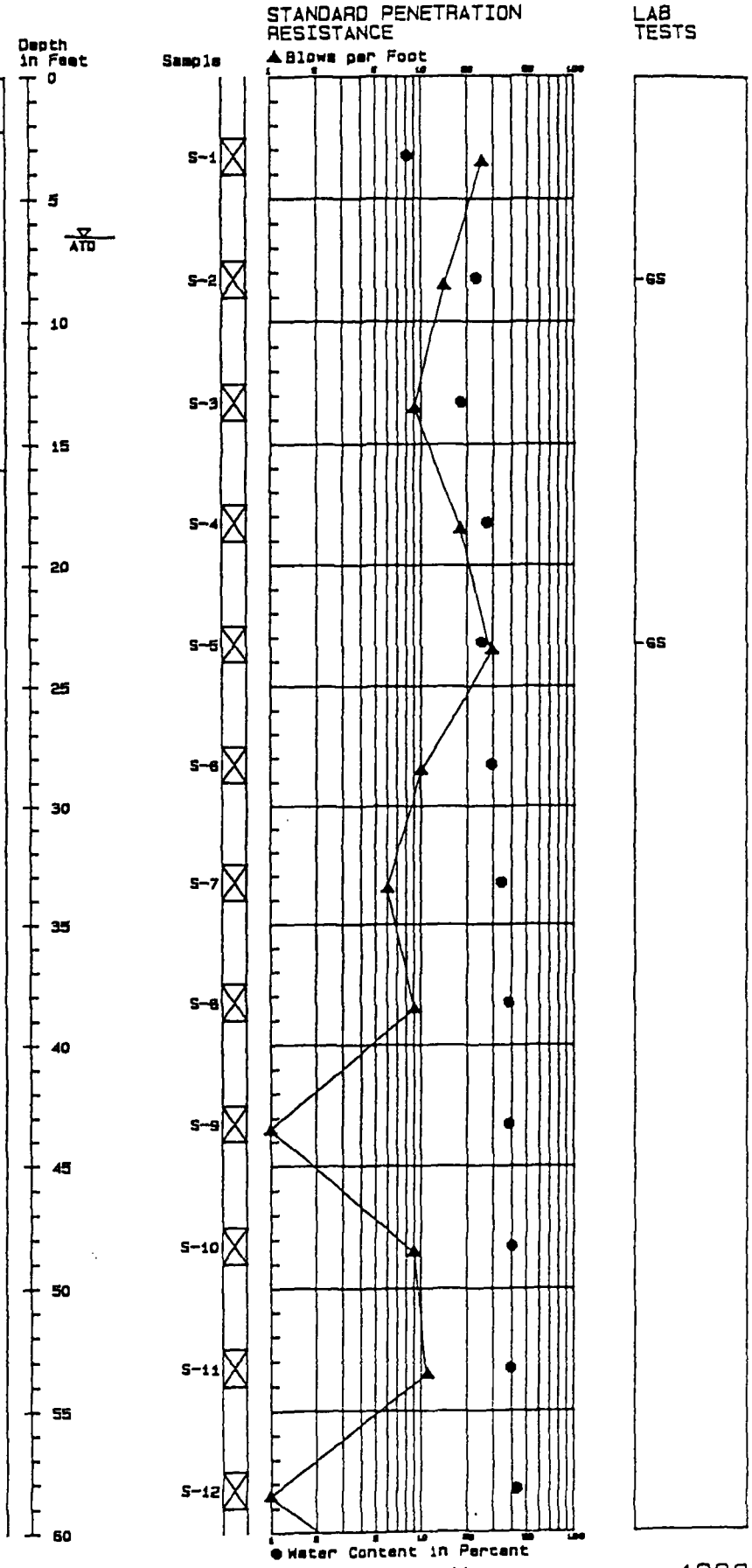
Ground Surface Elevation in Feet

Wet, black, silty, gravelly, sandy FILL with substantial wood.

Medium dense, moist to wet, gray, gravelly SAND interlayered with silty, fine SAND with shells and wood. (FILL)

Medium dense, wet, gray, trace to slightly silty, fine SAND with wood and shells.

- Grades more silty.
- Becomes loose.
- Grades to very silty.
- Very loose.
- Very loose.



J-2160 May 1988

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Sheet 1 of 2 Figure C-11

Boring Log B-6

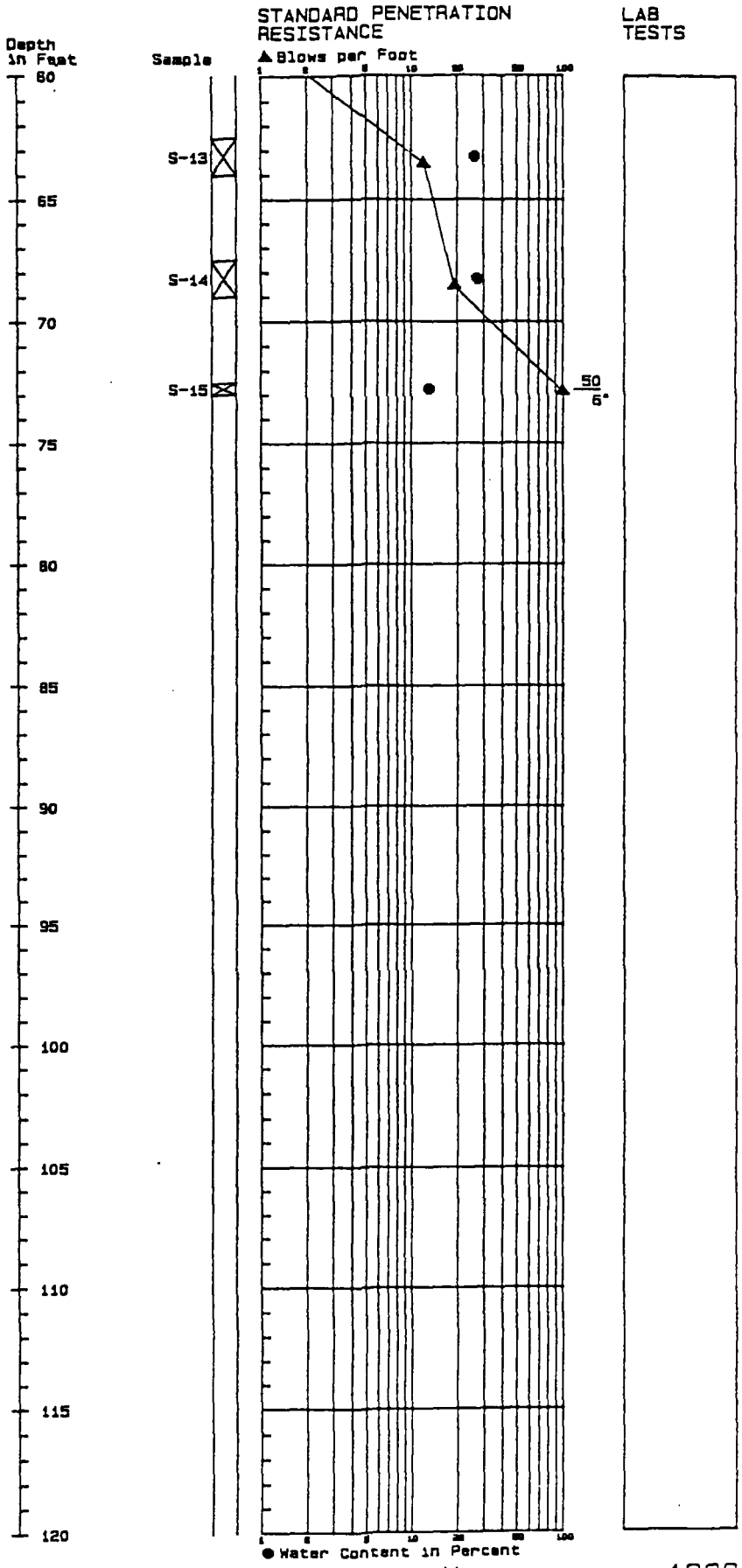
SOIL DESCRIPTIONS

Ground Surface Elevation in Feet

Medium dense, wet, gray, slightly silty to silty, fine SAND with wood and shell fragments.

Very dense, wet, gray, trace to slightly gravelly SAND with occasional shells.

Bottom of Boring at 73.0 Feet.
Completed 5/13/88.



1. Refer to Figure C-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Ground water level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

J-2160 May 1988

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Sheet 2 of 2 Figure C-11

Boring Log B-10

SOIL DESCRIPTIONS

Ground Surface Elevation in Feet

Dense, wet, gray, black and brown, silty, gravelly, medium to fine SAND with wood. (FILL)

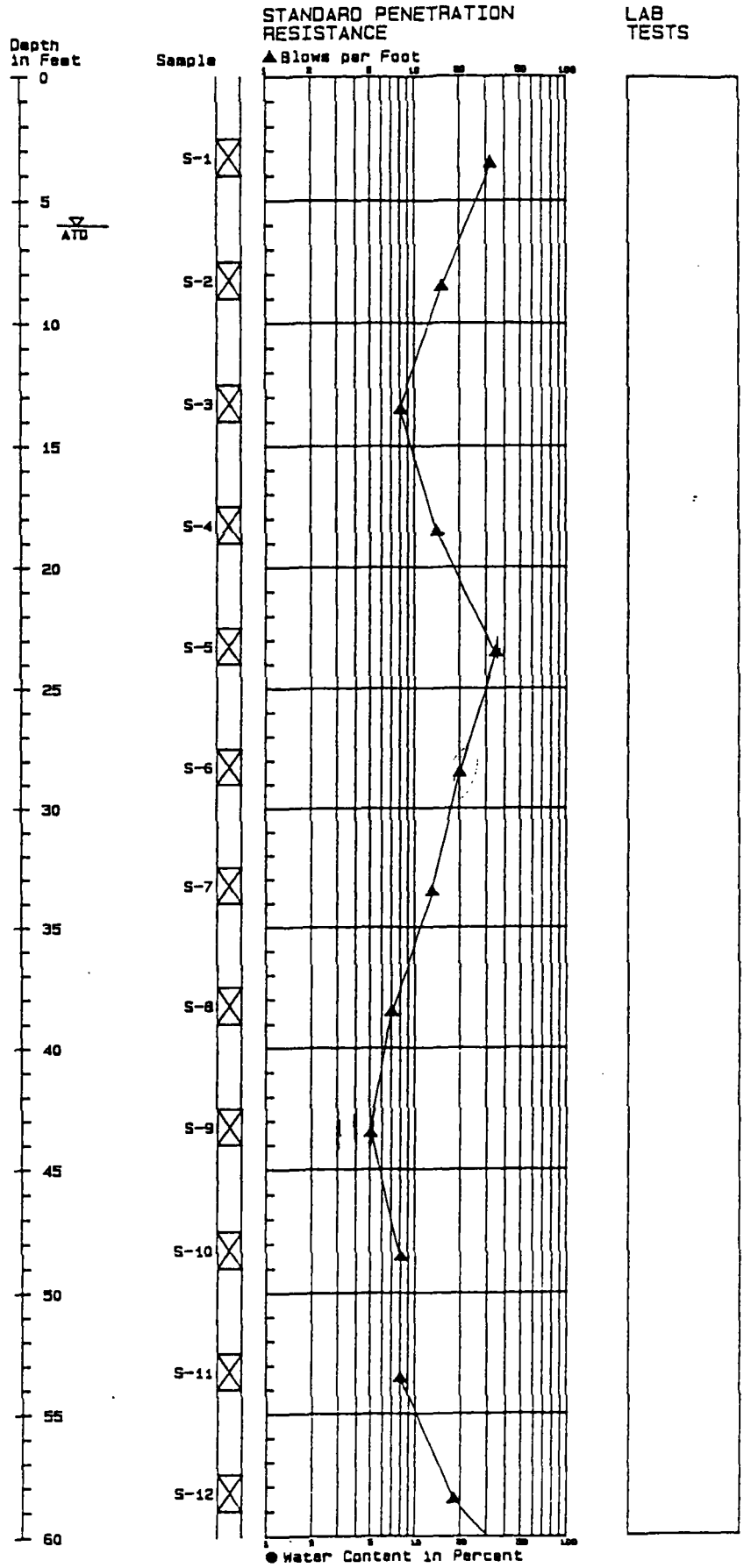
Medium dense, wet, gray, trace to slightly silty, slightly gravelly, fine SAND grading to fine SAND with wood and shell fragments.

- Loose.

- Dense.

Loose, wet, gray, slightly silty to silty, fine SAND with trace wood and shell fragments.

- Becomes medium dense.



J-2160 May 1988

HART-CROWSER & associates, inc.

Sheet 1 of 2 Figure C-12

Boring Log B-10

SOIL DESCRIPTIONS

Ground Surface Elevation in Feet

Very dense, wet, gray, interbedded, slightly silty to silty, fine SAND, clean fine SAND, and SILT with trace wood and shell fragments.

Bottom of Boring at 68.5 Feet.
Completed 5/17/88.

Depth in Feet: 60, 65, 70, 75, 80, 85, 90, 95, 100, 105, 110, 115, 120

Sample: S-13, S-14

STANDARD PENETRATION RESISTANCE

▲ Blows per Foot

S-13: 50/6"

S-14: 50/6"

● Water Content in Percent

LAB TESTS

1. Refer to Figure C-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Ground water level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

J-2160 May 1988

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Sheet 2 of 2 Figure C-12

APPENDIX D
GROUNDWATER PUMPING TEST

APPENDIX D
GROUNDWATER PUMPING TEST

Introduction

Hart Crowser performed a groundwater pumping test as part of the investigations to gain information on hydraulic characteristics of the subsurface soils that will control the selection and performance of appropriate dewatering systems for the proposed Graving Dock. Given the depth of the proposed dock, and the presence of fine sandy soils that were expected to have permeability toward the low end of the range that is amenable to gravity drainage, the test well was designed to draw water from the full sequence of soils present to a depth of approximately 60 feet. This is consistent with the original intention to dewater the site to a depth of around 30 feet below ground surface to permit slab construction on final grade at around elevation -12 feet.

Test Layout and Well Installation

The test set-up included the drilling and installation of a test well (PW-02) at a central location (see Figure 2) near one of the existing monitoring wells (H-C-02) drilled for the environmental investigation (Hart Crowser 2002a). The test well was constructed by Malcolm Drilling of Kent, WA on October 28 and 29, 2002, using a tracked Watson piling/dewatering crane/rig complete with 30-inch-diameter bucket auger and 80-foot telescopic kelly. This approach allowed the well to be drilled and completed in less than 12 hours, to meet tight schedule requirements for the implementation of fieldwork.

The well drilling method featured placement of a 20-foot-long starter casing to prevent collapse of the upper borehole, with water added throughout the drilling operation to maintain a positive head in the borehole relative to groundwater level in the formation. The well was completed by placing a 12-inch-diameter PVC well-casing and screen assembly to the full depth of 60 feet, the lower 20 feet comprising machine-slotted wellscreen with a standard slot width of 0.050 inch. The borehole annulus around the wellscreen and casing assembly up to the surface was backfilled with 4 x 8 fine gravel (from Glacier NW) to provide a filter pack. The well was then developed by over-pumping to remove fine sediment and drilling debris. Refer to Figure A-10 for a well schematic.

The capacity of a well constructed in this manner is in excess of 400 gpm, with minimal head losses through the well screen. The screen slot size is well matched to the 4 x 8 fine gravel filter pack, which in turn meets the standard filter criteria (e.g., Cedegren 1989) for permeability and resistance to piping. The conductivity of the gravel pack is on the order of 0.2 cm/sec. However, the

actual discharge obtained from the well depends mainly on the permeability of the screened formation(s), and the amount of drilling debris entrained within the gravel pack that is not removed during well development.

The standard drilling method used for the test well appears to have created a liquefied slurry within the borehole as drilling was advanced through the loose fine sand and slot silts of Soil Units 2 and 3. The implications of this were not fully realized when the well was constructed by placing materials into this slurry, with the result that a large quantity of fines appear to have been retained within the gravel pack. Dewatering contractors rely on groundwater from the formation to flow through the filter pack and dislodge drilling debris during well development. However, in this case it appears that there was insufficient flow from the formation to effect full cleaning of the screen, except perhaps over a short length of the hole opposite the deep aquifer.

To avoid such situations with dewatering wells on the project, it may be necessary to drill with casing and then flush out all drilling debris with clean water before well construction, or to drill with a synthetic polymer that may perform better than water as a drilling fluid that will maintain the integrity of the borehole wall and allow effective removal of fines.

Observation and Monitoring Wells

Three observation wells (OW-1, OW-2, and OW-3) were installed at distances of 20, 35, and 60 feet, respectively, from the test well to monitor the drawdown response to pumping and thus define the cone of depression produced. These observation wells were originally scheduled for placement below final grade to a depth of around 50 feet within the silty fine sand stratum, but the high silt content of this soil layer necessitated a field adjustment of well completions to monitor water levels in shallower, more permeable soils. The observation wells were installed in unlogged holes drilled to a depth of 20 feet by Holt Drilling of Puyallup, Washington, using the hollow-stem auger method. A 2-inch-diameter well casing and screen assembly was then placed through the auger, and the annular space backfilled with 10/20 Colorado Silica Sand. After completion, the monitoring wells were developed using a "whale" submersible pump to remove fine sediment and drilling debris. Refer to Figure A-11 through A-14 for well schematics.

Three additional monitoring wells completed the pumping test layout. The existing monitoring well H-C-02 was included in the test monitoring program, at a distance of 95 feet from the test well. Additional monitoring wells were installed in two new geotechnical borings, H-5-02 and H-6-02, located at 50 and 190 feet from the test well, respectively. A deep monitoring well completion

was made in H-5-02, with the 20-foot-long well screen located at a depth of 41 to 61 feet. A shallow monitoring well completion was made in H-6-02, with the 10-foot-long wellscreen located at a depth of 13 to 23 feet. Logs for the new geotechnical borings, including well completions are shown on Figures A-6 and A-7. The configuration of the pumping test monitoring wells is listed in Table D-1.

Water Level Monitoring

The pumped well (PW-02) and observation and monitoring wells within the pumping test layout were equipped with Mini-Troll© combined pressure-transducers and data loggers. These were set to record water levels throughout the testing period. In addition, a remote monitoring well (HC-NW-02) was selected for background monitoring and to observe potential tidal fluctuations that may affect the pumping test. To obtain accurate tidal data, a Mini-Troll logger also was placed off one of the docks in Port Angeles Harbor, protected within a PVC stilling well.

Graphical summaries of all water level records obtained are included in this appendix. Water level monitoring data, reduced in the form of hydrographs for the pumping well and the observation and monitoring wells, are depicted on Figures D-1a through D-8a. Figure D-9a depicts the tide level recorded during the test period.

Pumping Test Implementation

Following the completion of well development and placement of the monitoring instruments, a step drawdown test was performed in well PW-02 on October 30, 2002. The purpose of the step drawdown test is to ascertain the performance characteristics of the pumping well, and allow the selection of an appropriate pumping rate for the 24-hour constant rate pumping test.

Observations during well installation and development suggested that sustainable flow rates from the test well would be relatively low — on the order of 5 to 10 gpm. The step drawdown test therefore consisted of four hour-long pumping steps, starting at 3 gpm, and ramping up by approximately 1 gpm to a maximum rate of 6 gpm. The drawdown response in PW-02 to these pumping rates is shown on Figure D-1a (left portion of plot), and depicts water levels dropping progressively with each step.

The step drawdown test has not been fully analyzed at this time due to the low yields of water from the deeper sediments, which are thought to have created less than optimal conditions for pumping from the upper aquifer. The loose/soft

very silty nature of the sand/silt aquifer caused high sediment loads to remain in suspension within the wellbore during well construction and these materials appears to have been entrained within the fine gravel pack, significantly reducing its permeability. As a result, large drawdowns within the pumping well casing do not appear to have been fully communicated to the formation at the outside of the wellbore, resulting in lower drawdowns and lower flowrates than would be optimal for dewatering operations. (This suspicion was later confirmed by extrapolating distance-data from the constant-rate test back to the pumping well radius; see Figure D-9b). However, small drawdowns were observed in the six observation and monitoring wells of the pumping test layout during the step drawdown test, indicating that usable pumping test data could be obtained by proceeding with the constant rate test at a low but steady flowrate.

The constant rate test was initiated at 10 a.m. on October 31, 2002, and continued uninterrupted at a pumping rate of 5 gpm through the rest of the day and night for a period of 24 hours. High quality drawdown data were obtained from test well and the observation and monitoring wells in the vicinity of PW-02, as indicated by the drawdown plots shown in the lower sections of Figures D-1 through D-8. Tidal fluctuations in Port Angeles Harbor were monitored throughout the test period, as shown on Figure D-9a.

Pumping Test Analysis

The drawdown data obtained from the observation and monitoring wells during the constant rate pumping test have been analyzed to determine aquifer conditions at the test well location and to estimate site-specific values for key aquifer characteristics that will control the performance of dewatering systems at the site. The analysis is primarily graphical, with drawdown plotted on semi-logarithmic graphs against the log of the elapsed time (in minutes) since the start of the pumping test (see Figures D-1b through D-8b).

Wellbore Storage

Drawdowns during the early stages of the test, up to about 100 minutes, are dominated by the depletion of wellbore storage within the pumping well (see Figure D-1b). This constitutes a transitional pumping period from the start of the test when the flow from the pump is almost entirely derived from the declining water level within the well casing.

Shallow Aquifer Responses

As the test progresses and an increasing head difference is established between the water level in the well and the groundwater level in the surrounding

formation, the rate of groundwater inflow steadily increases until it approaches 100 percent of the flow from the pump. At this time, wellbore storage is exhausted and the water level in the pumped well reaches a pseudo-steady level with a drawdown of about 22 feet below the static level at the start of the test. From this point onward, the flowrate drawn into the well from the surrounding formation is effectively constant.

The effect of wellbore storage is manifest on most of the semi-logarithmic observation well drawdown records as a progressively increasing drawdown trend. An inflection occurs when the effect of wellbore storage is overcome, typically after 100 minutes, but later for the more distant wells in the shallow aquifer. Drawdowns at this time are considered to represent the aquifer response to constant pumping, and have been analyzed using the distance-drawdown method devised by Cooper and Jacob (1946), as shown on Figure D-9b.

For a uniform homogeneous aquifer pumped at a constant rate, the drawdown measured at a given time in a series of observation wells at different radial distances should plot on semi-logarithmic scales as a straight line. The slope of this line is inversely proportional to the transmissivity of the aquifer. The projected intercept of the line at zero drawdown is proportional to the ratio of aquifer storativity and transmissivity. These values are calculated as shown in Table D-2.

As the constant rate test continued beyond 100 minutes, the aquifer response should normally appear in the observation well data as a straight line of constant slope on the semi-logarithmic drawdown plots (Figures D-2b through D-8b). However, a number of these plots exhibit trends that are curved upward, leading to higher-than expected drawdowns by the end of the test. There are a number of potential causes for this effect, the most likely being limited lateral extent of the aquifer that likely pinches out to the south, and may be constrained by the sea wall structure to the north.

The effect of the increased drawdowns that are interpreted to result from one or more lateral boundaries is depicted on Figure D-9b with a second distance-drawdown trend to represent the cone of depression after 1,000 minutes. For a uniform aquifer, this second line should be parallel to the first drawn at 100 minutes. The increased steepness of this second line is consistent with the presence of lateral boundaries that limit the extent of the aquifer, increasing the observed drawdowns.

The inferred aquifer condition with one or more lateral boundaries is likely beneficial to the proposed dewatering operations in that it increases the

performance efficiency of any installed dewatering system, with greater drawdown achieved for a given installation capacity.

Deep Aquifer Responses

The drawdown response observed in the deep monitoring well installed in H-5-02 is substantially different from those observed in the shallow monitoring wells. Firstly, the drawdown response is greater in magnitude, given the distance from the pumped well (190 feet). Secondly, H-5-02 shows tidal fluctuations that affect the drawdown response. These conditions are not uncommon for a confined aquifer that is buried beneath a confining layer, such as the thick silt/sand aquitard present at the Port Angeles site.

The semi-logarithmic drawdown plot shows a muted wellbore storage effect (wellbore storage reduces within increasing distance from the well), followed by a relatively straight-line portion that is considered to be the aquifer response to pumping. Applying the time-drawdown analysis method of Cooper & Jacob (1946) to this section of the drawdown curves gives values for the aquifer characteristics listed in Table D-2.

After approximately 180 minutes of pumping, the drawdown trend in H-5-02 flattens off and appears to stabilize, then follows a muted tidal response, in contrast to the drawdown trend observed in the upper aquifer. The stabilization of drawdown, combined with the tidal response, strongly indicates that the cone of depression in the potentiometric surface for the lower aquifer has likely reached the body of surface water in Port Angeles Harbor, and that this provides a lateral recharge source to the lower aquifer.

Further calculations were performed as shown in Table D-2 to corroborate this interpretation. The image-well theory of Stallman (Ferris et al. 1962) was used to synthesize a matching drawdown trend that would be observed at H-5-02 with a recharge boundary, and to infer the apparent distance of the recharge boundary from PW-02, assuming that it lies in the direction of Port Angels Harbor (normally drawdown data from at least two monitoring wells are needed to resolve the location and direction of a recharge boundary).

The distance to the recharge boundary used in the drawdown simulation that best matches the observed drawdown trend and inflection point in the H-5-02 data (using the values of T and S from the time-drawdown analysis) was around 250 feet, which is somewhat less than the actual distance from PW-02 to the sea wall. This calculation provides a partial support for the above interpretation of conditions in the deep aquifer layer, but requires further analysis. The assumptions behind this calculation include a fully confined aquifer; however,

the sand/silt aquitard that overlies the thin lower sand aquifer layer could produce what are known as "leaky" confined aquifer conditions, whereby the small amount of vertical leakage from the aquitard becomes significant over the large area of the cone of depression. The effects of a recharge boundary and a confined leaky aquifer condition create very similar drawdown trends, and can be difficult to resolve. However, they create similar conditions for dewatering design, so full resolution of this issue is not necessary.

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Cedegren, H.R. 1989. "Seepage Drainage and Flow Nets, " Wiley, New York, 465 pp.

Cooper, H.H., and C.E. Jacob 1946. "A Generalized Graphical Method for Evaluating Formation Constants and Summarizing Well Field History," American Geophysical Union Transactions, vol. 27, pp. 526-534.

Ferris, J.G., D.B. Knowles, R.H. Brown, and R.W. Stallman 1962. "Theory of Aquifer Tests: U.S. Geological Survey Water-Supply Paper," 153&E, p. E69-E174.

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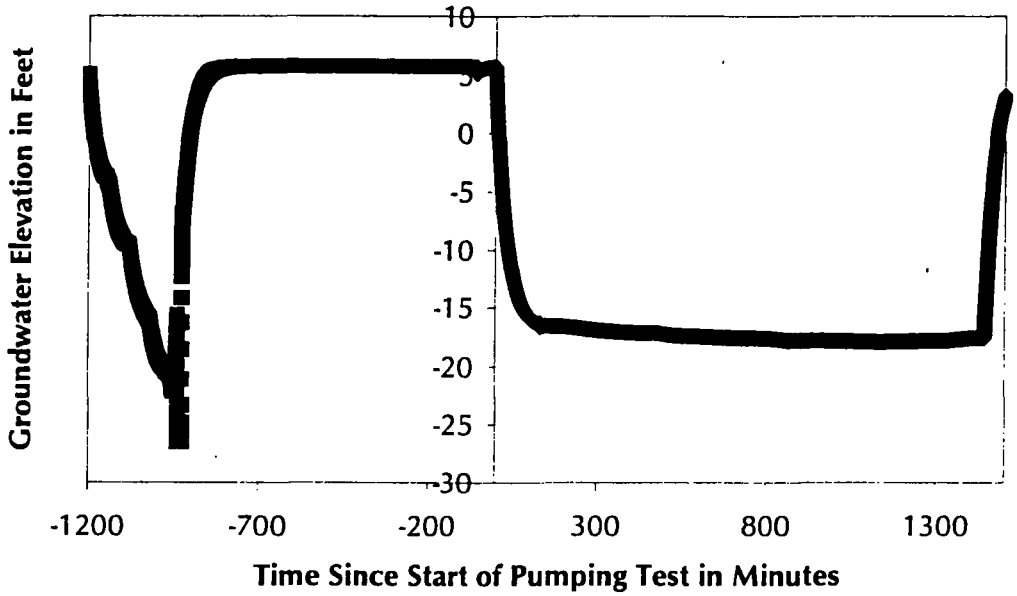
Pumping Test Data — Pumping Well PW-02

a) Hydrograph



b) Semi-Log Time Drawdown Plot

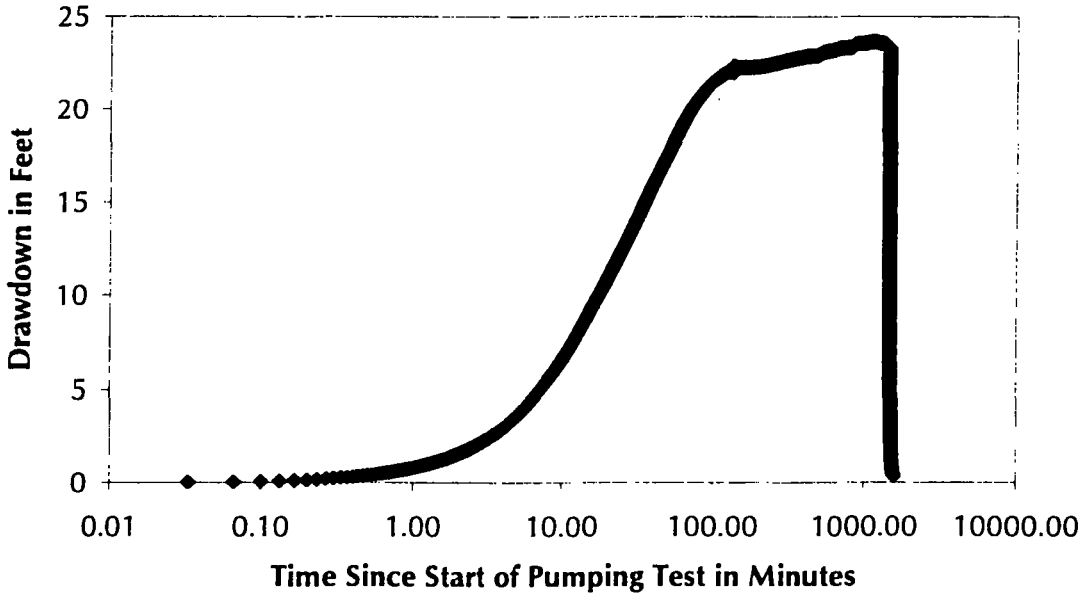


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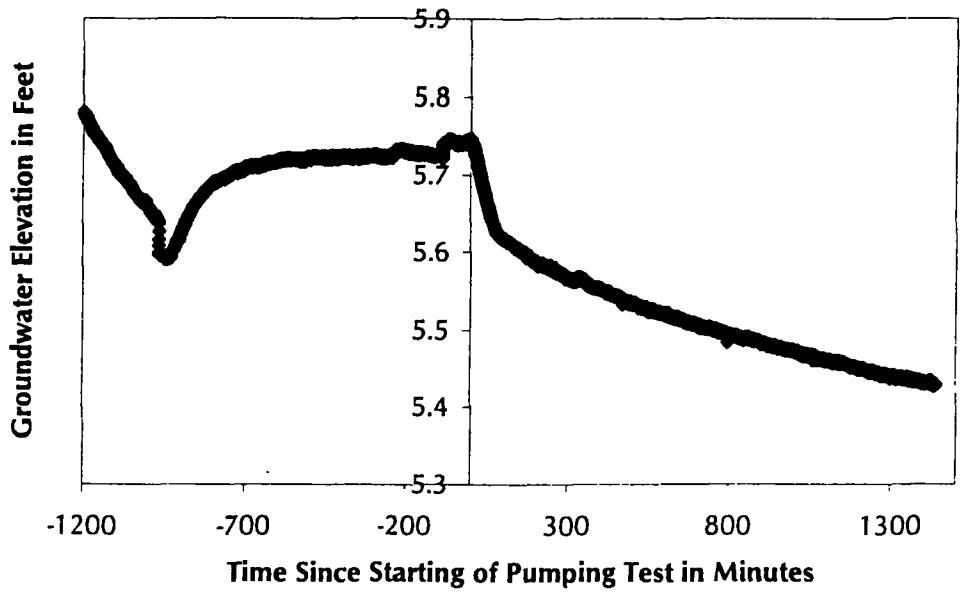
Figure D-1

Pumping Test Data — Observation Well OW-1-02

a) Hydrograph



b) Semi-Log Time Drawdown Plot

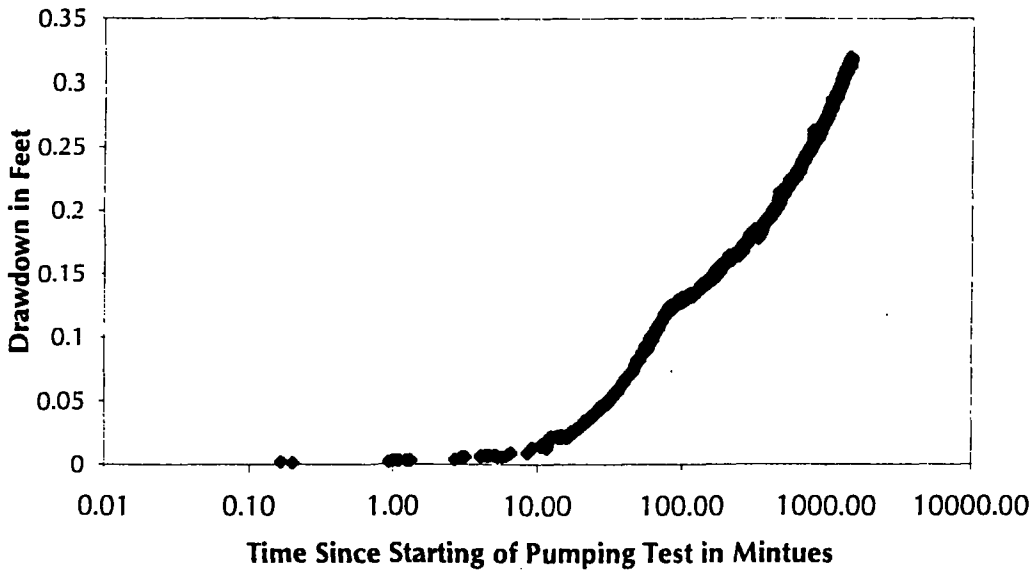


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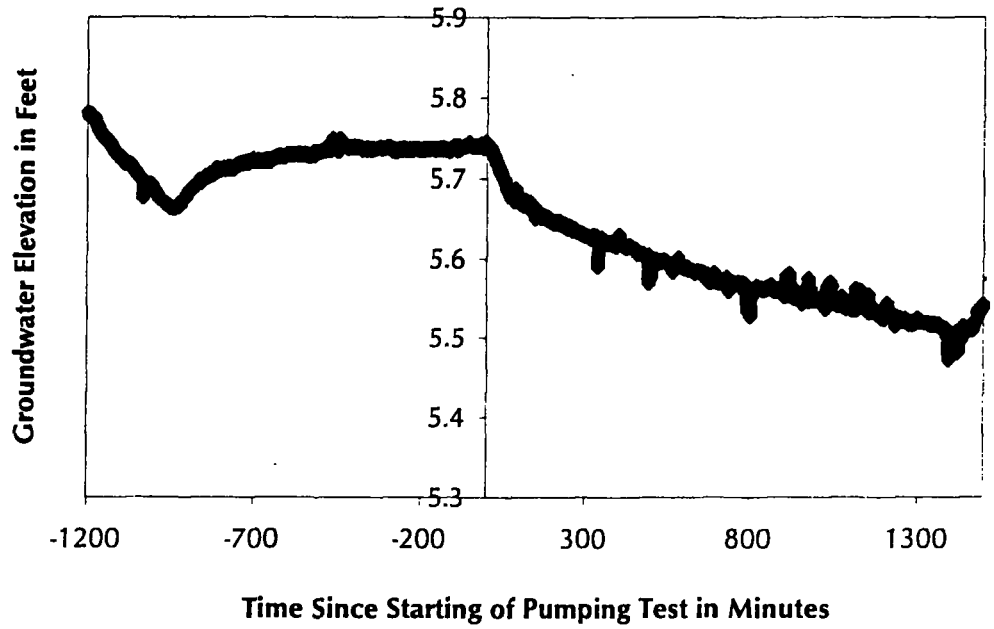
Figure D-2

Pumping Test Data — Observation Well OW-2-02

a) Hydrograph



b) Semi-Log Time Drawdown Plot

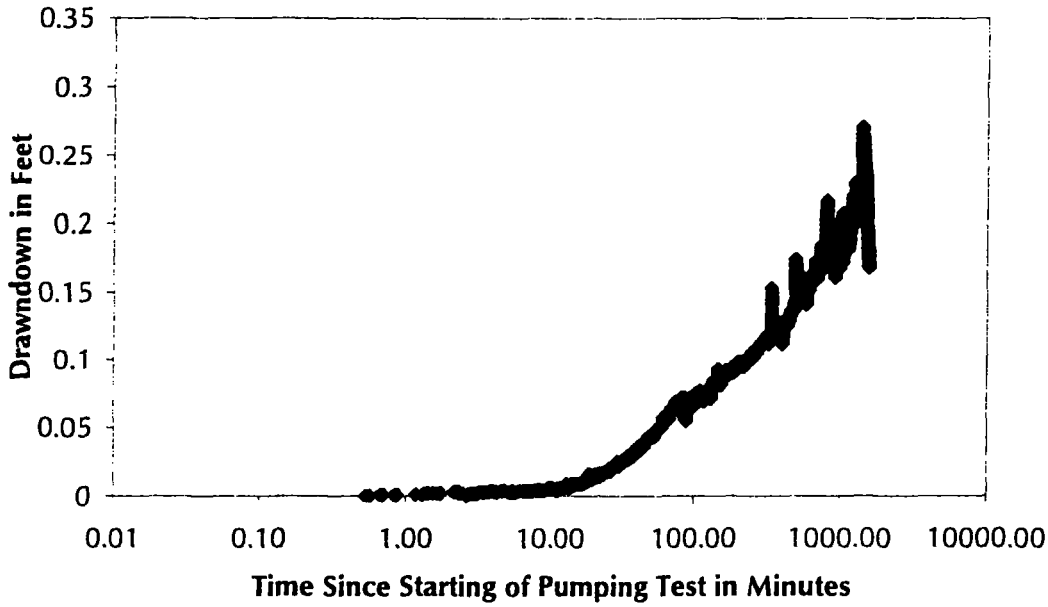


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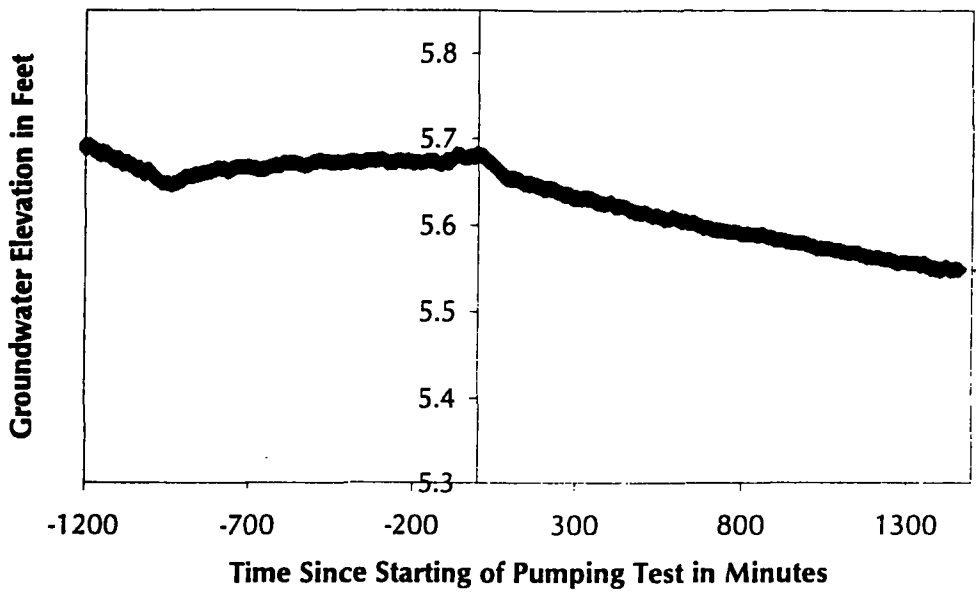
Figure D-3

Pumping Test Data — Obervation Well OW-3-02

a) Hydrograph



b) Semi-Log Time Drawdown Plot

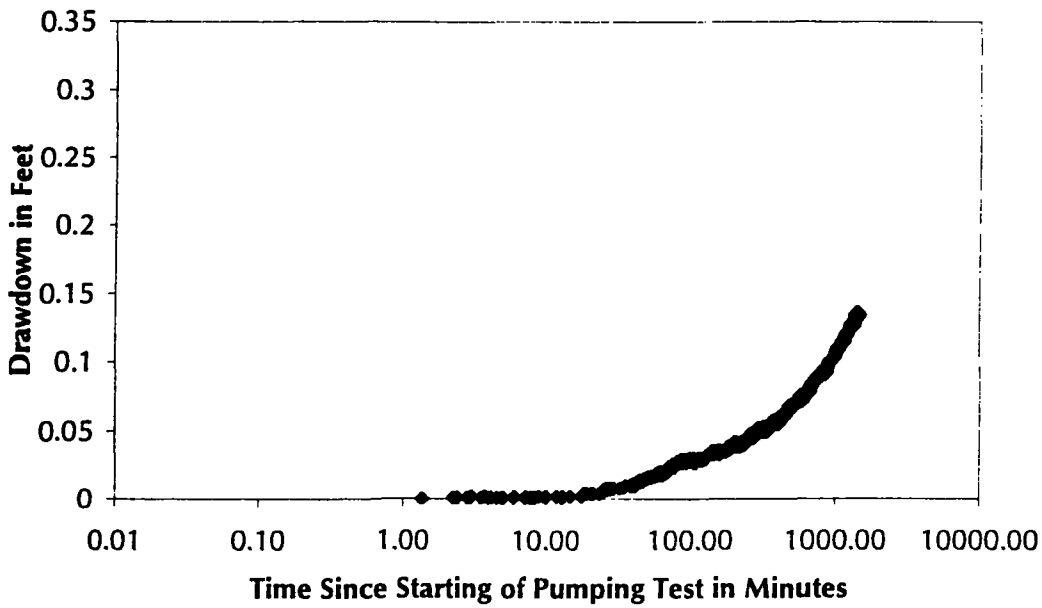


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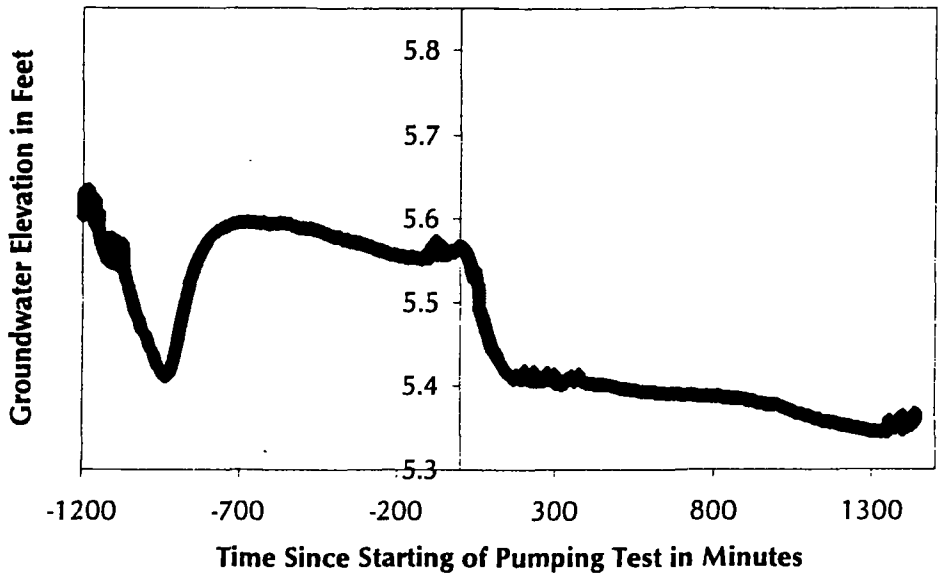
Figure D-4

Pumping Test Data — Monitoring Well H-5-02

a) Hydrograph



b) Semi-Log Time Drawdown Plot

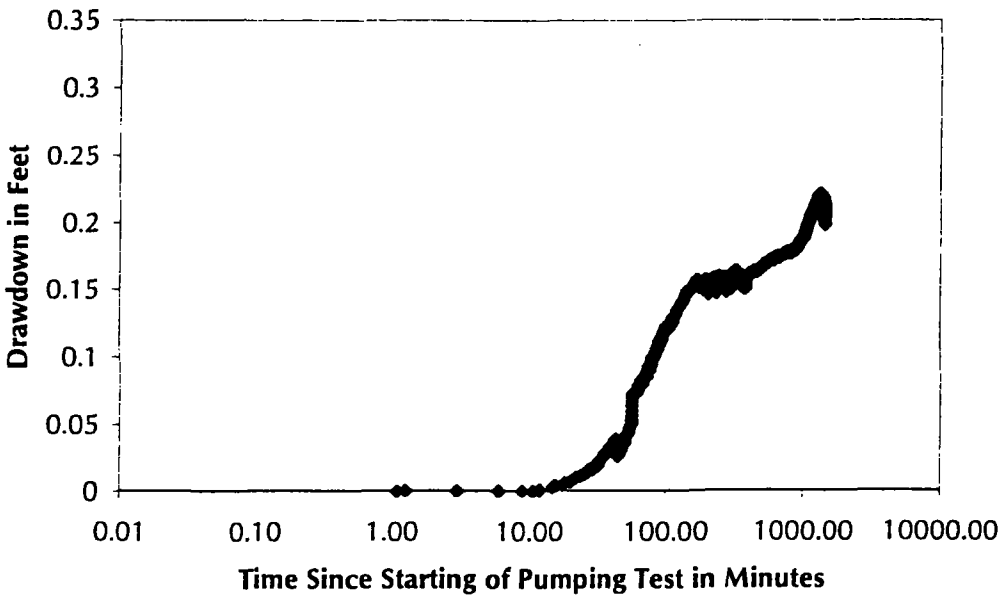


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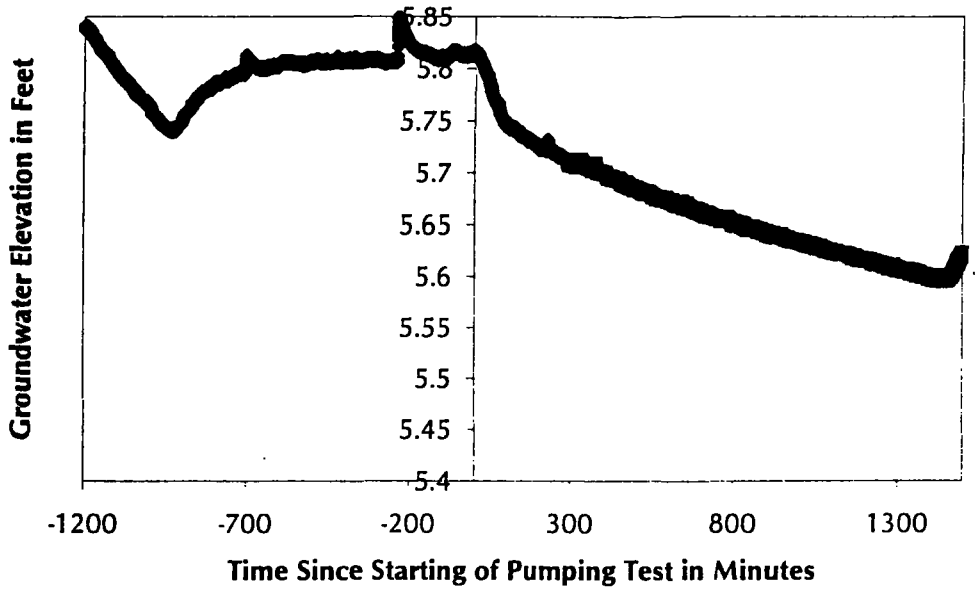
Figure D-5

Pumping Test Data — Monitoring Well H-6-02

a) Hydrograph



b) Semi-Log Time Drawdown Plot

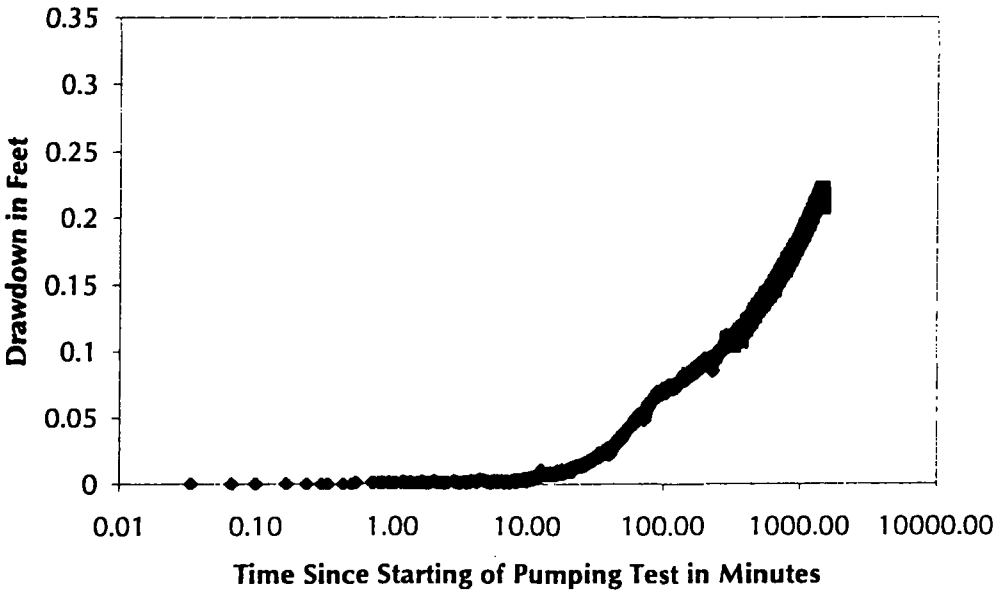


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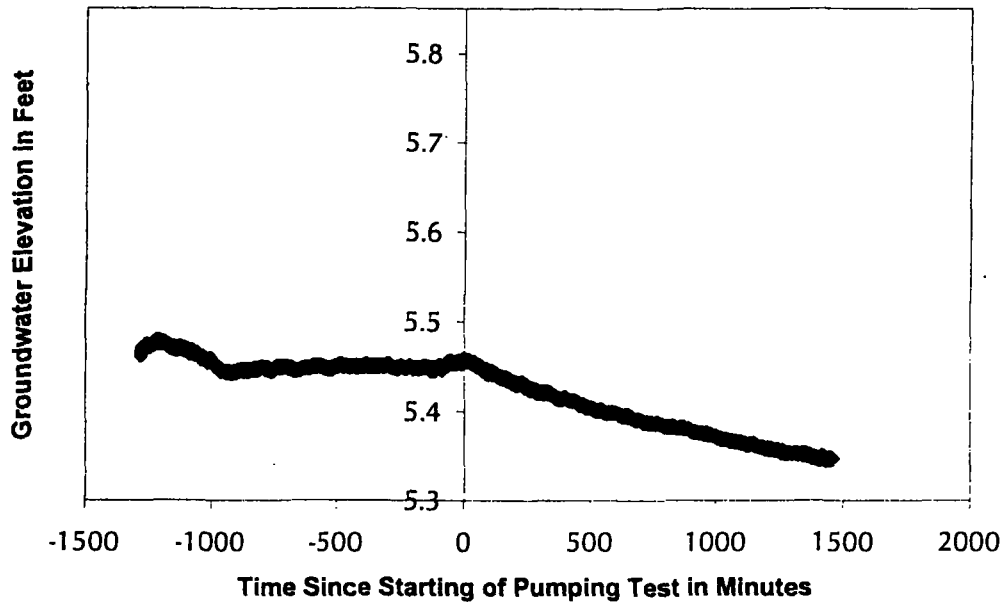
Figure D-6

Pumping Test Data — Monitoring Well HC-C-02

a) Hydrograph



x

b) Semi-Log Time Drawdown Plot

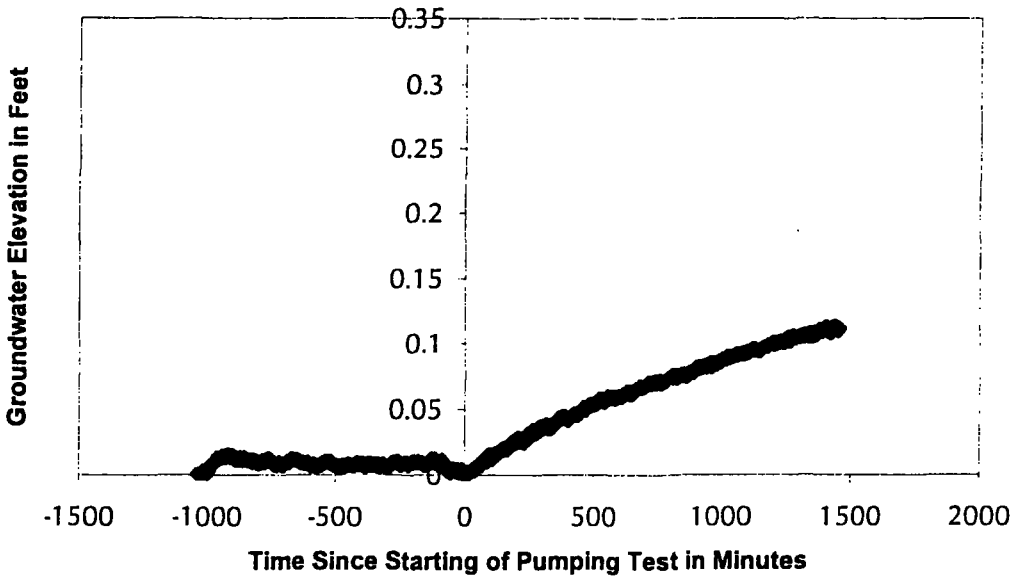


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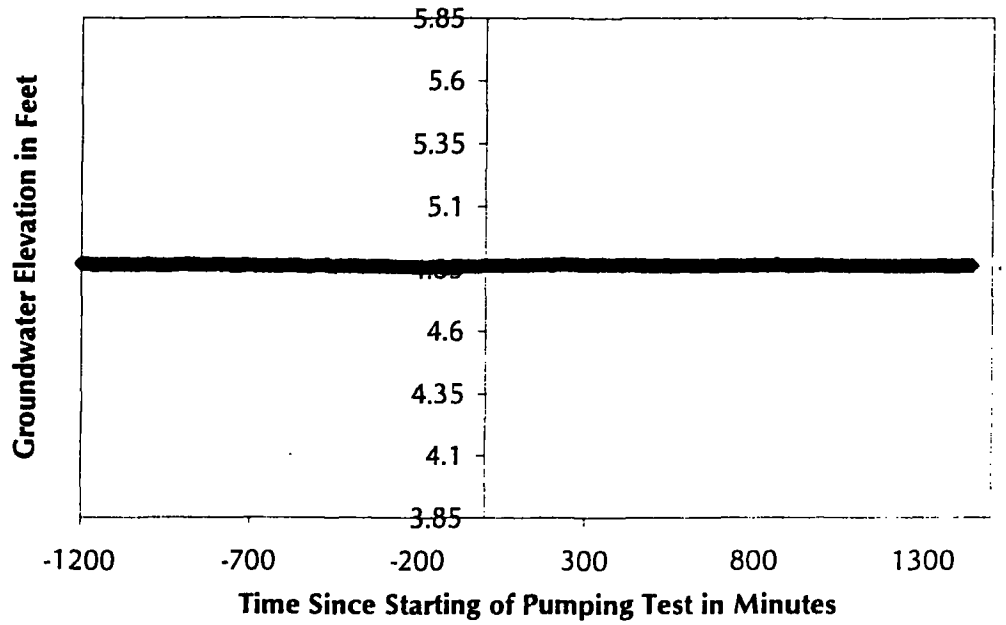
Figure D-7

Pumping Test Data — Monitoring Well HC-NW-02

a) Hydrograph



b) Semi-Log Time Drawdown Plot

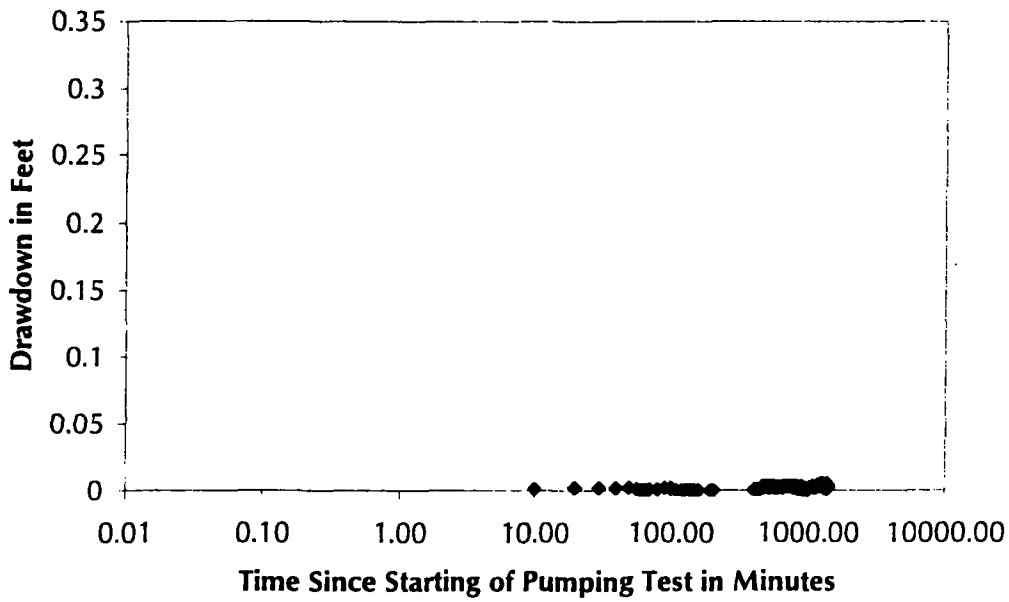


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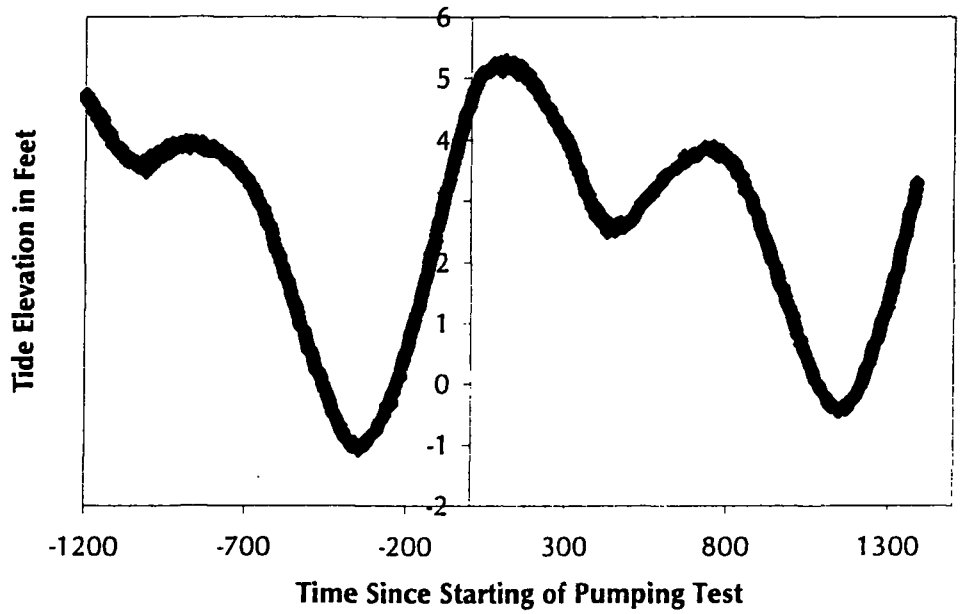
Figure D-8

Pumping Test Data

a) Tide Level



b) Distance Drawdown Analysis

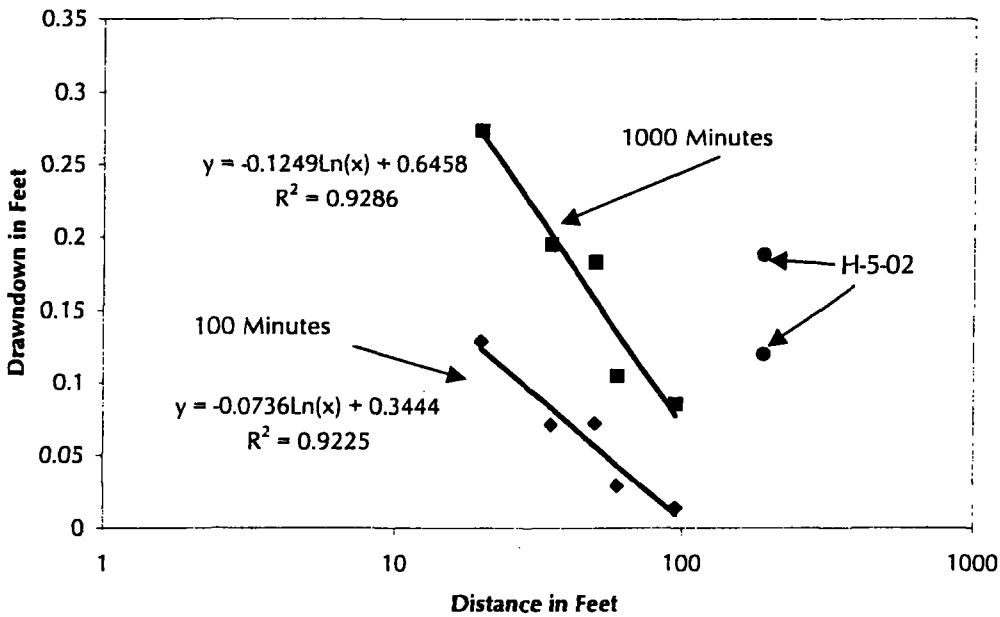


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Figure D-9