Design Level Geotechnical Engineering Study Ridgefield Rail Overpass Project Ridgefield, Washington

Prepared for:
Jacobs Engineering Group, Inc.

September 13, 2007
1161-00

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Environmental and Geotechnical Consultants

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1.0 Introduction

This report presents Ash Creek Associates' geotechnical recommendations for the proposed Ridgefield Rail Overpass Project.

The significant aspects of this report have been arranged in the following manner:

- Project understanding;
- Subsurface conditions; and
- Engineering conclusions.

2.0 Scope of Services

Our scope of services for this project included the following:

- Surficial reconnaissance;
- Review of existing geotechnical data in the area;
- Subsurface Explorations;
- Geotechnical engineering analyses; and
- Preparation of this report.

3.0 Limitations of Our Work

This work was performed for the exclusive use of Jacobs Engineering, their clients, and their consultants for specific application to this project and site. We performed this work in accordance with generally accepted professional practices in the same or similar localities, related to the nature of the work accomplished, at the time the services were performed. No other warranty, express or implied, is made.

4.0 Project Understanding

Ash Creek Associates' understanding of the project is based upon discussions with members of the Design Team as well as review of the proposed project alignment and information provided to Ash Creek Associates by Jacobs Engineering. The improvement project will consist of the construction of a vehicle bridge over the railroad tracks to eliminate two at-grade crossings and to provide access to the Ridgefield waterfront and the Port of Ridgefield's proposed mixed use development. The site location has been indicated on Figure 1. The extent of the project has been indicated on Figure 2.

5.0 Site Description

On the east, the site consists of a small existing section of Pioneer Street that slopes moderately down to the west. The existing roadway terminates some distance west of Main Avenue. The central portion of the site consists of a moderately steep, west-facing bluff that is approximately 30 feet high. The railroad right-of-way is located near the base of the bluff. West of the railroad, the remainder of the site consists of a relatively flat area adjacent to Lake River. The majority of this portion of the site is occupied by a marina and former log treating facility.

The selected alignment for the proposed bridge consists of an extension of Pioneer Street. We understand the current selected alignment is as follows:

- Approach via Pioneer Street;
- Bridge over the railroad, turning northward to parallel the railroad, and reaching current grade on the north portion of the current marina property; and
- At-grade roadway the remainder of the distance to the Port of Ridgefield.

6.0 Site Geology

6.1 Geologic Overview

The soil and rock formations underlying the bluffs are composed of the Tertiary-age (Lower Pliocene) Troutdale Formation. During the Tertiary Period, cataclysmic uplifting of the Columbia River Basalt and high velocity floods and debris flows washed granular materials into the Portland Basin. The Troutdale Formation is on the order of 5 to 7 million years old and is composed of a variety of sedimentary materials including unconsolidated sand and gravel, cemented gravels and cobbles (conglomerate), and hard sandstones. The formation is in excess of 1,000 feet thick in places and the surface is often deeply weathered to fine-grained sandy silts and clayey silts. On steep slopes, the fine fractions of the Troutdale Formation are prone to massive landsliding.

Because of the nature of the Troutdale Formation's variable consistency and partial cementation, springs can seep from the bluff face during the wet season. Often the water flows beneath the vegetation mat and only manifests itself in cut slopes and ditches.

The materials at the base of the bluffs, beneath and west of the railroad embankment, are composed of gravels and sands most likely derived from past erosion of the bluffs by the ancient Columbia River. Further from the bluffs, near Lake River, we anticipate that the near-surface soils grade into more sandy and silty materials. Groundwater is expected to closely reflect the Lake River elevation during the summer months and rise several feet during the winter months.

The materials observed along the bluffs and in the ditches along the railroad right-of-way confirm the presence of the gravelly phase of the Troutdale Formation. We did not observe any gross stability problems associated with undisturbed native soils, but relatively severe past erosion has occurred from storm water runoff at the terminus of Pioneer Street. The drainage has been rip-rapped and has energy dissipation boulders at the outfall

6.2 United States Soil Conservation Service Soil Survey of Clark County

Office review of the United States Soil Conservation Service (USCS) soil survey of Clark County (1972) indicates the presence of two major near-surface native soil units mantling the site. In general, the USCS only classifies soils present in the upper 4 to 6 feet of material mantling a site. USCS identifies these soil units respectively as Hillsboro Silt Loam and Sauvie Silt Loam. A summary of the soil properties of these units, as well as the approximate extents over the project area, is described below.

Hillsboro Silt Loam. This soil unit is present at the top of the bluff over the eastern end of the project. Unified soil classification of this unit is established as ML; the equivalent AASHTO classification is A4. Shrink-swell potential is considered moderate. Soil fines contents (i.e., percentage of soil particles smaller than a standard No. 200 sieve) vary from 65 to 70 percent, and pH levels fall into the range of 5 to 6. Plasticity Index (PI) typically ranges between 6 and 10.

Sauvie Silt Loam, 3 to 8 Percent Slopes. This soil unit is mapped over the flatter, western portion of the site. Unified soil classification of this unit is established as ML to SM, and the AASHTO classification is A4 to A6. Shrink-swell potential is considered low; fines contents (i.e., percentage of soil particles smaller than a standard No. 200 sieve) vary from 45 to 90 percent, and pH levels are classified as falling in the range of 6.1 to 7.3. A high groundwater table typically characterizes this soil unit.

6.3 Seismicity and Earthquake Sources

The seismicity of the Clark County area, and hence the potential for the project site ground shaking, is controlled by three separate fault mechanisms. These include the Cascadia Subduction Zone (CSZ), the mid-depth intraplate zone, and the relatively shallow crustal zone. Descriptions of these potential earthquake sources are presented below.

The Cascadia Subduction Zone. The CSZ is located offshore and extends from northern California to British Columbia. Within this zone, the oceanic Juan De Fuca Plate is being subducted beneath the continental North American Plate to the east. The interface between these two plates is located at a depth of approximately 15 to 20 kilometers (km). The return interval for large subduction zone earthquakes is believed to be 300 to 500 years. Evidence suggests that the most recent subduction zone event took place approximately 300 years ago. Geomatrix's study (1995) suggests the maximum earthquake associated with the CSZ is moment magnitude (M_w) 8 to 9. A subduction zone earthquake of M_w 8.5 was assumed for the purposes of this report.

The Intraplate Zone. The intraplate zone encompasses the portion of the subducting Juan De Fuca Plate located at a depth of approximately 30 to 50 km below western Washington and Oregon. Very low levels of seismicity have been observed within the intraplate zone in Oregon; however, much higher levels of seismicity within this zone have been recorded in Washington and California. Historical activity associated with the intraplate zone includes the 1949 Olympia (M_w 7.1), the 1965 Puget Sound (M_w 6.5), and the 2001 Nisqually (M_w 6.8) earthquakes. Based on the data presented within the Geomatrix (1995) report, an earthquake of M_w 7.25 has been chosen to represent the seismic potential of the intraplate zone.

The recent (February 28, 2001) seismic event near the town of Nisqually, Washington (the epicenter of which was between Tacoma and Olympia, approximately 10 miles northeast of Olympia) has been classified as an intraplate-type seismic event. The Nisqually quake resulted in 320 reported injuries and over $2 billion in property damages. The magnitude of the Nisqually Quake was 6.8. The focus of this quake was approximately 30 miles deep. It was felt strongly in Portland and Vancouver, as well as in British Columbia.

Near-Surface Crustal Sources. The third source of seismicity that can result in ground shaking within the greater Portland area is near-surface crustal earthquakes occurring within the North American Plate. The historical seismicity of crustal earthquakes in southwest Washington is higher than the seismicity associated with the CSZ and the intraplate zone. The 1993 Scotts Mills (M_w 5.6) and Klamath Falls (M_w 6.0) earthquakes were crustal earthquakes.

7.0 Subsurface Conditions

A field exploration program consisting of four drilled borings was conducted on June 12 and June 24, 2007. Further, the owner of the property at 1310 Main Street provided us with a number of drilling logs completed by PBS Engineering and Environmental (PBS) for the redevelopment of that site. These included the log of a monitoring well installed adjacent to the proposed bridge abutment. The locations of the subsurface explorations are shown on Figure 2. Logs of the subsurface explorations have been included in Appendix A of this report. The attached logs describe soils and various engineering properties of soils encountered during exploration. Descriptions are based upon *in situ* testing, laboratory testing, and field classification of soil samples.

A summary of subsurface conditions encountered within our subsurface explorations has been provided below. The subsurface conditions within the project area have been divided into two distinct units. The typical subsurface profile observed in each of these four zones has been summarized below.

7.1 Zone 1: Bluff Top and Face

The subsurface conditions observed adjacent to the terminus of the existing Pioneer Street and within the bluff face consisted of the following:

Fill. The area adjacent to Pioneer Street has been filled to construct the roadway extension. The fills encountered consisted of soft to medium-stiff, brown silty clay. The fills appear to contain some organics and may not have been placed as structural fill. Nearly 10 feet of fill was encountered in boring B-4. Fill soils were not encountered down the bluff face in the PBS B-9 boring.

Native Silts, Clays, and Sands. The fills are directly underlain by stiff, damp to moist, alluvial soils. Near Pioneer Street, the soils generally consisted of stiff silts and clays. Mid-slope in the PBS B-9 boring, the near-surface soils were logged as silty fine sand overlying clays. The clay soils are underlain by a layer of medium-dense, fine to medium sand.

Troutdale Formation Gravels. The clays, silts, and sands are underlain at depth by dense to very dense, moist to wet, sandy gravels. This formation conforms to the Troutdale Formation in the area and may be partially or fully cemented. Cobbles and boulders are common in the Troutdale Formation and may be encountered in excavations. The surface of the Troutdale Formation gravels was encountered at a depth of 43 feet in Ash Creek Associates' B-4 boring and at 20 feet in the PBS B-9 boring.

Groundwater Conditions. The mud-rotary drilling techniques employed for boring B-4 do not allow for an accurate determination of groundwater table elevations during drilling. The monitoring well installed by PBS (their boring B-9) recorded a groundwater reading at a depth of 15.5 feet below the ground surface (bgs) on March 27, 2007.

7.2 Zone 2: Lowland

The subsurface conditions observed on the relatively flat, lower portion of the site consisted of the following:

Fills. The surface of the majority of this portion of the site has been regraded for the construction of roadways, parking areas, and other facilities. As such, the upper 2 to 3 feet have been disturbed or consist of imported fills. The fills appear to consist of locally derived silts and sands. The majority of fills encountered do not appear to have been placed as structural fills.

Native Silts and Sands. The fills are underlain by a mixture of silts and sands with some gravels and organics. This soil appears to be derived from a combination of mass wasting (erosion and landsliding) of the adjacent bluff and flood plain deposition. The soils are generally very soft to medium-stiff in consistency.

Troutdale Formation Gravels. The near-surface soils are underlain at depth by dense to very dense, moist to wet, sandy gravels. This formation conforms to the Troutdale Formation in the area and may be partially or fully cemented. Cobbles and boulders are common in the Troutdale Formation and may be encountered in excavations. The surface of the Troutdale Formation gravels was encountered at depths ranging from 25 to 39 feet bgs.

Groundwater Conditions. The mud-rotary drilling techniques employed do not allow for an accurate determination of groundwater table elevations during drilling. Groundwater is expected to closely reflect the Lake River elevation during the summer months and rise several feet during the winter months.

8.0 Conclusions and Recommendations

Our recommendations are based on our current understanding of the project. If the nature or location of the planned construction changes, Ash Creek Associates should be contacted so that we may confirm or revise our recommendations.

8.1 Site Preparation

We have provided recommendations for wet weather and dry weather construction, as well as other geotechnical concerns and issues relative to the project site. Because of the moisture-sensitive near-surface soils and the presence of shallow groundwater over much of the site, Ash Creek Associates strongly recommends that site grading and utility trenching be conducted during dry weather conditions. The optimum time for site grading and trench work generally falls between late June and late September.

Dry Weather Construction. We recommend that compaction criteria for structural fills, embankment fills, and trench backfills be based upon ASTM D-1557 (Modified Proctor) testing. Embankment fills, structural fills, and backfills should be compacted to 92 percent of the material's maximum dry density. Landscape fills and nonstructural berms should be compacted to approximately 85 percent of the material's maximum dry density. Compaction of grid-reinforced fills should adhere to proprietary specifications. This often entails slightly reduced compaction requirements adjacent to the backside of block walls.

Even during dry weather, some areas of the road subgrade may become soft or may "pump" (deflect under wheel load), particularly in cuts, poorly drained areas, abandoned drainage ditches, swales, old fills, and areas subjected to frequent heavy construction traffic loads. Soft or wet areas present at finished road subgrade elevations should first be scarified, tilled, dried, and recompacted. These areas should be proof-rolled and if the area still deflects under wheel load, it should subsequently be prepared in accordance with the recommendations provided in the Wet Weather Construction Section of this report. Overexcavation of soft road subgrade areas can generally be limited to 1 to 2 feet. A non-woven geotextile may also be applied to road subgrade areas where overexcavation of soft soils is required.

Overexcavated soft areas should be backfilled with clean granular stabilization rock. Stabilization rock should consist of clean bank-run gravel, diced rock, or pit-run quarry rock. Nominal material size should be 2 to 4 inches (minus).

Wet Weather and Wet or Soft Subgrade Construction Methods. During wet weather, or when adequate moisture control is not possible, it will be necessary to install a granular working blanket to support construction equipment and provide a firm base on which to place subsequent fill and pavement. Commonly, the working blanket consists of a bank-run gravel or pit-run quarry rock. Nominal material size should not exceed 4 inches (minus). Materials conforming to the Washington State Department of Transportation (WSDOT) standards for Gravel Borrow are generally acceptable for this purpose.

As an alternative to a granular working blanket, it may be possible to substitute a certain percentage of the overall working blanket thickness with a cement-treated soil base. Based upon our past experience with cement-treated working blankets, it is likely that cement content will be in the range of 6 to 8 percent by weight.

After installation, the working blanket should be compacted by a minimum of four complete passes with a moderately heavy (15,000 pounds [lbs.]) static steel drum or grid roller. We recommend that we be retained to observe granular working blanket installation and compaction.

The working blanket must provide a firm base for subsequent fill installation and compaction. It has been our experience that a minimum of 1 to 2 feet of working blanket is normally required, depending on the gradation and angularity of the working blanket material. This assumes the material is placed on a relatively undisturbed subgrade in accordance with the preceding recommendations and that it is not subjected to frequent heavy construction traffic.

Portions of the site used as haul routes for heavy construction equipment will require a thicker working blanket in order to protect the fine-grained subgrade.

A heavy-grade, non-woven, soil stabilization geotextile that conforms to WSDOT specifications should be installed on fine-grained subgrade to prevent silt and clay from contaminating and pumping the granular working blanket.

Construction practices can greatly affect the amount of working blanket necessary. In addition, the use of a cement-treated soil subgrade can significantly reduce the amount of granular working blanket required. By using tracked equipment and granular haul roads, the working blanket area can be minimized. If dump trucks and rubber-tired equipment are allowed random access across the site, a thicker working blanket may be required. Normally the design, installation, and maintenance of a granular working blanket are the responsibilities of the Earthwork Contractor.

Proof-Rolling of Road Subgrades. Regardless of which method of subgrade preparation is used (i.e., wet weather or dry weather), we recommend the prepared subgrade be proof-rolled with a fully loaded dump truck or other suitable equipment prior to fill placement or base course installation. Any area that pumps, weaves, or appears soft and muddy should be scarified, dried, and recompacted or overexcavated, and backfilled with compacted granular fill. If a significant length of time passes between fill placement and commencement of construction operations, or if significant traffic has been routed over these areas, we recommend the subgrade be similarly proof-rolled again before any foundation or pavement installation is allowed.

Subgrade conditions over the majority of the road alignment appear to consist of moderately-stiff native mineral soils or soft to medium-stiff embankment fill soils. Based upon results from our research and reconnaissance, the native mineral soils and existing embankment fills will function adequately as road subgrade if prepared in accordance with the recommendations outlined in this report. Some sections of the project will encounter special construction challenges associated with marginal subgrade conditions. There will likely be other limited areas of the site that will require subgrade stabilization during either new embankment fill construction or road subgrade reconstruction.

Marginal Subgrade Stabilization. It should also be anticipated that other limited areas of marginal subgrade will manifest themselves during construction. When encountered, these areas should be assessed on a case-by-case basis to determine the best approach for stabilization. There are several common approaches to stabilization of road subgrade which typically can include:

- Subgrade scarification, aeration/drying followed by recompaction;
- Stabilization via overexcavation and replacement of soft areas with clean crushed rock or pit-run material. This option is sometimes employed in combination with placement of a geo-grid or geo-fabric over marginal subgrade areas prior to placement of stabilization rock; and

- Stabilization via in-place cement treatment. Typically, subgrade stabilization via cement treatment entails the use of a 5 to 7 percent cement content addition by dry unit weight. The cement additive is mixed into subgrade soils in-place with rippers, tillers, and scarifiers. Following mixing, the treated soils are subsequently recompacted. Practical depth of in-place treatment is usually 12 to 18 inches below surface grade. Cement treatment is not appropriate for organic soils.

In any of the above-described approaches, subgrade stabilization can typically be limited to depths of approximately 1 to 2 feet below design subgrade elevations.

8.2 Embankment Fills and Structural Fills

Embankment and structural fills should be installed on a subgrade that has been prepared in accordance with the above recommendations. Fills should be installed in horizontal lifts not exceeding 8 inches in thickness (loose - prior to compaction), and should be compacted to at least 92 percent of the material's maximum dry density as determined by ASTM D-1557 (Modified Proctor) testing. The compaction criteria may be reduced to 85 percent in nonstructural landscape or nonstructural berms. The road base below the asphalt section and the upper 12 inches of road subgrade should be compacted to 95 percent as determined by ASTM D-1557.

Materials that cannot be moisture-density tested due to oversized rock fragments should be compacted by a minimum of four passes with a moderately heavy (15,000 Ibs.) drum roller. This material should subsequently be observed for its performance under heavy wheel loads. Any area that pumps or deflects excessively should be prepared in accordance with our previous recommendations.

In order to achieve acceptable levels of compaction, it is generally desirable to maintain moisture contents of fine-grained fill soils within the range of 3 to 4 percent of the optimum moisture content.

Each compacted layer of structural fill or road embankment fill should be observed for excessive deflection or reaction under moving loaded equipment to verify no soft or pumping areas remain in any layer. Areas that are noted to deflect excessively should be prepared in accordance with the dry and wet weather grading recommendations provided above.

Structural fills or embankment fills placed over ground with slopes in excess of 5H:1V should be keyed and benched into existing slopes. Seeps encountered during grading on sloping ground should be intercepted via area drains. Outfalls for such drains should be routed to the toe of such slopes and should not be allowed to drain freely over slopes. Area drains are typically field-designed on a case-by-case basis. Usually seeps will be intercepted via 6-inch perforated drain pipes surrounded by clean crushed rock or drain rock fill.

A summary of recommended compaction specifications is provided in the table below.

Fill Compaction Specifications

Material	Percent of Maximum Dry Density (ASTM D-1557)
Fine-Grained Fill	92
Landscaping Fill	85
Clean Granular Fill	95
Pavement Subgrade	95

8.3 Fill Material Recommendations

Structural Fills During Summer Grading. During dry weather, road embankment fills and other structural fills may consist of virtually any relatively well-graded soil that is free of debris, organic matter, and high percentages of clay or clay lumps, and which can be compacted to the preceding specifications. However, if excess moisture causes the fill to pump or weave, those areas should be dried and recompacted or removed and backfilled with compacted granular fill. To achieve adequate compaction during wet weather, or if proper moisture content cannot be achieved by drying, we recommend fills consisting of well-graded, clean granular soils (sand or sand and gravel). Fill materials corresponding to WSDOT specifications for Select Borrow or Gravel Borrow will generally be appropriate for wet weather grading.

Wet Weather Grading and Subgrade Stabilization Fills. Because moisture levels are difficult to control in fine-grained soils and soil drying via aeration is not realistically an option, structural fill constructed during the wet season should consist of clean, durable crushed rock, or clean granular fill. Typically, wet weather grading conditions should be assumed to exist between the months of mid-October through early to late June.

Wet Weather Grading with Cement-Treated Soils. An alternative to the use of granular fill is cement treatment of native soils to be employed in structural fill. This is accomplished using specialized spreaders and mixers and is often more cost effective than imported granular fill. Soil cement treatment is typically a Contractor-related means-and-methods item. This type of soil treatment is typically conducted by spreading Portland cement over the surface of the soils to be treated. The Portland cement is subsequently tilled or disced into soils via specialized mixing equipment. Ideal mixing depths are typically between 12 and 18 inches below finished subgrade elevations, dependent upon the Contractor's equipment and construction approach. Percentage of cement additive to soils being treated in this manner often varies depending upon soil moisture content and soil clay content. It has been Ash Creek Associates' past experience with the native soils in the project vicinity that 5 to 7 percent cement additive by total weight will be required to achieve acceptable compaction levels and soil stiffness within fills, subgrades, or haul routes. Employing local earthwork contractors with experience in soil cement treatment will typically minimize construction delays and budget overages associated with wet weather grading.

Pavement Base Rock. Crushed rock utilized in these areas should consist of clean, 5/8- to 1-1/2-inch (minus) durable crushed rock. Fines content should not exceed the maximum allowable by WSDOT (i.e., maximum allowable by weight material passing a standard No. 200 sieve).

Trench Backfill. Utility conduits should be bedded in material meeting the WSDOT specification for Gravel Backfill for Pipe Zone Bedding. Trench backfill should be lightly compacted above breakable conduits within two pipe diameters or 18 inches, whichever is greater. Trench backfill underlying pavements or other settlement-sensitive structures or features should consist of durable, clean crushed rock with nominal size between 5/8 inch (minus) and 1-1/2 inches (minus). This material should be relatively clean, with a low percentage of fines by weight. Fines content should not exceed the maximum allowable by WSDOT (i.e., maximum allowable by weight material passing a standard No. 200 sieve).

Working Pads for Marginal Subgrade Areas and Wet Weather Grading. The working pad for wet weather construction should consist of durable, clean crushed rock, bank-run, or pit-run material. Nominal size should be between 1-1/2 and 4 inches (minus). This material should be relatively clean, with a low percentage of fines by weight. Materials conforming to the WSDOT standards for Gravel Borrow are generally acceptable for this purpose.

8.4 Areal Settlements

Areal Settlements. Areal settlements for fills constructed to maximum heights of 6 to 8 feet or less are estimated to be less than approximately 1 inch. This assumes fill construction over fill native mineral soil subgrades. If fills are constructed in accordance with Ash Creek Associates' recommendations regarding fill compaction, subgrade stabilization, and optimal moisture levels for fill placement, the majority of areal fill settlement is expected to occur during fill construction.

8.5 Foundation Support

In order to support the relatively large foundation loads associated with the bridge abutments and bents, we recommend that the bridge be placed on drilled shafts. Deep foundations constructed for the project should derive their support from the Troutdale Formation gravels. Due to the presence of shallow water and granular soils, the shallow soils would be subject to caving and heaving during the construction of drilled shafts. Drilled shaft construction would likely require the use of casing or bentonite slurry to maintain the bearing surfaces.

For the purposes of the following section, we have assumed that structural foundation support will rely upon foundation elements bearing within the underlying Troutdale gravels. Based on discussions with the design team, we understand that the drilled shafts will support a service level load of 3,000 kips with an ultimate capacity of 7,500 kips needed.

Vertical Drilled Shaft Capacities

Drilled Shaft Diameter (feet)	Embedment Depth in Troutdale (feet)	Uplift Capacity (kips)
5	50	600
6	20	400
8	10	500

The allowable vertical capacities presented for piers may be increased by one-third for short-term transient loads (wind and seismic loading conditions).

Lateral Load Capacity of Piers. Lateral loading on pier caps can be resisted by the lateral load carrying capacity of foundation piers. We have computed allowable lateral pile capacities based on a maximum deflection of 1/2 inch. For the 6-foot-diameter drilled shaft, embedded a minimum of 20 feet into the Troutdale Formation gravels, the maximum lateral load at a deflection of 1/2 inch was 550 kips.

8.6 Erosion Control

Ash Creek Associates recommends that finished cut and fill slopes be protected immediately following grading with vegetation, gravel, or other approved erosion control methods. Water should not be allowed to flow over slope faces or drop from outfalls, but should be collected and routed to storm water disposal systems. Rip-rap, gabion baskets, or similar erosion control methods may be necessary at storm water outfalls or to reduce water velocity in ditches. Silt fences should be established and maintained throughout the construction period. Silt fence barriers should be established downslope from all construction areas to protect natural drainage channels from erosion and/or siltation. To decrease erosion potential, care should be taken to maintain native vegetation and organic soil cover in as much of the site as possible.

8.7 Temporary Excavation Slopes

Native soils may stand near vertical slopes for short periods of time; however, they may collapse suddenly and without warning. Precautions in utility trench and other excavations will be required due to the potential for caving/sloughing within native soils underlying the site. Any excavations deeper than 4 feet should be sloped or shored in accordance with Occupational Health and Safety Administration (OSHA) regulations. Normally, shoring systems are Contractor-designed and -installed items.

In general, temporary excavation slopes may be suitable in areas where adjacent improvements are not located within a horizontal distance equal to the depth of the excavation (measured from the top of the excavation). Unsupported temporary excavation slopes within native fine-grained soils or fill soils should not exceed slopes of 1H:1V. Actual slopes used during construction should be determined by the Contractor on a case-by-case basis.

For the majority of the alignment, the use of coffin-box-type shoring may lead to significant sloughing and caving. For dewatered excavations and excavations above the seasonal groundwater table, it may be feasible to use such an approach, provided that the Contractor has made provisions to address sidewall sloughing and caving. Without advance dewatering, sloped excavations below the groundwater table will not be feasible.

8.8 Trench and Excavation Dewatering

Dewatering within trenches and excavations will likely be required for deeper utilities. Our experience in the area indicates that attempting to excavate below the groundwater table without dewatering typically leads to large amounts of sidewall caving, project delays, significant increases in bedding and backfill quantities, and the possibility of heaving soil within trench base areas. Based on our experience in the area, we do not recommend dewatering via sumps and/or small pumps for this project. Groundwater depths and the permeability of native soils below the groundwater table are expected to preclude a typical "low-tech" approach to trench dewatering.

8.9 Excavations Next to Railroad Alignments

It may prove necessary to install utilities adjacent to existing railroad alignments. In general, ground surface settlements as a function of short-term construction excavation work are relatively small beyond an oblique projection starting from the bottom of the trench and projecting toward the ground surface at an angle of 45 degrees. Excavations that are located closer to the railroad tracks than the limits discussed should be structurally shored.

8.10 Trenching In the Vicinity of Existing Structures

Surface settlements as a function of short-term construction excavation work are typically small beyond an oblique projection starting from the bottom of the trench and projecting toward the ground surface at an angle of 45 degrees (1H:1V projection from the base of the trench to the ground surface). We recommend that a setback, as defined by the above-described 45-degree projection line, be established between existing houses and commercial buildings and the bottoms of any proposed utility trenches.

8.11 Pavement Design Recommendations

The following preliminary pavement design recommendations are based upon Ash Creek Associates' experience with similar soil types in the vicinity of the project. It is Ash Creek Associates' recommendation that pavement design for this site be based upon relevant soil properties and constraints outlined in the following tables.

Pavement Design Parameters

Design Methodology	1993 AASHTO Guide for Design of Pavement Structures
Performance Period	20 years
Traffic Loading	Total traffic 16,000 ADT both ways. From WSDOT: 4 percent trucks, 1 ESAL per truck; total of 2.4M ESALs.
Reliability	90 percent
Serviceability	Initial Serviceability of 4.2, Terminal Serviceability of 2.5.
Overall Standard Deviation	0.44
Subgrade Resilient Modulus	4,500 based on local experience.
Base Course Resilient Modulus	25,000 pounds per square inch (psi) based on WSDOT.
Layer Coefficients	AC layer coefficient was 0.44 based on area experience. The crushed rock coefficient of 0.14 was determined based on the calculated modulus. For CTS, a value of 0.35 was calculated.
Drainage Coefficient	A value of 1 was used for the conventional and CTS pavements.

Based on discussions with the design team, Ash Creek Associates has prepared pavement designs for asphalt concrete (AC) over crushed rock pavements, Portland cement concrete (PCC) pavements, or full-depth asphalt concrete placed directly on a Portland cement amended subgrade (CTS).

Pavement Designs

Pavement Type	Surfacing Thickness (feet)	Base Thickness (feet)
AC/Crushed Rock	0.5	1.25
PCC/Crushed Rock	0.6	0.5
AC/Cement Modified Subgrade	0.4	1.0

The CTS design is based on the use of cement treatment of native soils to provide pavement support. This is accomplished using specialized spreaders and mixers and is often more cost effective than imported granular fill. Soil cement treatment is typically a Contractor-related means-and-methods item. This type of soil treatment is typically conducted by spreading Portland cement over the surface of the soils to be treated. The Portland cement is subsequently tilled into soils via specialized mixing equipment. Ideal mixing depths are typically between 12 to 18 inches bgs, dependent upon the Contractor's equipment and construction approach. A 12-inch thickness is the most common for projects of this nature. Percentage of cement additive to soils being treated in this manner often varies dependent upon soil moisture content and soil clay content. It has been Ash Creek Associates' past experience with the native soils in the project vicinity that 5 to 7 percent cement additive by total weight will be required to achieve acceptable compaction levels and soil stiffness within fills, subgrades, or haul routes. Employing local Earthwork Contractors with experience in soil cement treatment will typically minimize construction delays associated with wet weather grading. Cement treatment should be specified to be in accordance with "Suggested Specifications for Soil Cement Base Course", available from the Portland Cement Association. Compaction of pavement subgrades should be based on the maximum dry density as determined by ASTM D-1557 (modified proctor) testing. The cement modified subgrade soil should be compacted to 95 percent. Asphalt concrete should be compacted in excess of minimum WSDOT standard as determined by ASTM D-2041. This design is intended for use on public streets. If possible, construction traffic should be limited to unpaved and untreated roadways, or specially constructed haul roads. If this is not possible, the pavement design selected from the pavement design table should include an allowance for construction traffic.

8.12 Retaining Wall Design

The following guidelines for restrained and non-restrained walls assume that the associated recommendations regarding drainage, compaction, and other issues will be implemented. The design parameters in this section are for conventional retaining walls. If alternative retaining wall systems are proposed, Ash Creek Associates should be contacted for additional soil parameters.

Restrained Walls. Restrained walls are any walls that are prevented from rotation during backfilling. Walls with corners and those that are restrained by a floor slab or roof fall into the category of restrained walls. We recommend that restrained walls be designed for pressures developed from the equivalent fluid weights shown in the following table.

Restrained Wall Pressure Design Recommendations

Backfill Slope Horizontal:Vertical	Equivalent Fluid Weight (pounds per cubic foot [pcf])
Level	45
3H:1V	55

These pressures represent our best estimates of actual pressures that may develop and do not contain a factor of safety. These pressures are assumed to act horizontally (normal to the wall). This is based on the assumption that drainage membranes or impervious wall coatings will prevent friction between the wall and backfill. These pressures assume retaining wall backfill material is well drained. If traffic loads are expected within a horizontal distance from the top of the wall equal to the wall height, uniform lateral earth pressure acting horizontally on restrained walls equal to 90 pounds per square foot (psf) should be added to earth loads acting on the wall.

Non-Restrained Walls. Non-restrained walls have no restraint at the top and are free to rotate about their base. Most cantilever retaining walls fall into this category. We recommend that non-restrained walls be designed for pressures developed from the equivalent fluid weights shown in the following table.

Non-Restrained Wall Pressure Design Recommendations

Backfill Slope Horizontal:Vertical	Equivalent Fluid Weight (pcf)
Level	35
3H:1V	40

These pressures represent our best estimate of actual pressures that may develop and do not contain a factor of safety. These pressures assume retaining wall backfill material is well drained. If traffic loads are expected within a horizontal distance from the top of the wall equal to the wall height, uniform lateral earth pressure acting horizontally on unrestrained walls equal to 70 psf should be added to earth loads acting on the wall.

Seismic Lateral Earth Pressure. Lateral earth pressure acting on a retaining wall should be increased to account for seismic loadings. These pressures may be approximated by an evenly distributed pressure which is applied over the entire back of the wall. Using a design acceleration coefficient of 0.17 (this is equal to 1/2 of the peak horizontal ground acceleration) and a wall height "H" of up to 25 feet, we recommend that the seismic loadings be based on the surcharge pressures given in the following table.

Seismic Surcharge Design Pressure Recommendations

Design Condition	Seismic Pressure Surcharge (psf)
Active Earth Pressure	9H
At-Rest Earth Pressure	20H

These pressures represent our best estimate of actual pressures that may develop and do not contain a factor of safety. These pressures assume retaining wall backfill material is well drained.

Retaining Wall Backfill. Backfill behind retaining walls should consist of free-draining granular material. To minimize pressures on retaining walls, we recommend the use of well-graded crushed rock backfill with less than 5 percent by weight passing the No. 200 sieve. Use of other material could increase wall pressures. Overcompaction of this fill can greatly increase lateral soil pressures. We therefore recommend that this fill be compacted to approximately 90 percent of the material's maximum density as determined by ASTM D-1557 (Modified Proctor) testing.

We recommend that foundations or major loads not be placed within the zone that extends back from the base of retaining walls at a 1H:1V slope. Foundation loads located within this zone will significantly increase lateral pressures acting on retaining walls. In addition, backfill behind retaining walls is typically compacted to lower levels than normal structural fill. Some settlement is typical of retaining wall backfill. Foundations within a wall backfill zone will also be subjected to settlement.

Retaining Wall Drainage. Retaining walls will require drainage in order to alleviate lateral fluid forces on the walls. The drains should be protected by a filter fabric to prevent internal soil erosion and potential clogging.

Retaining Wall Bearing and Sliding Resistance. The majority of the project's retaining structures are anticipated to be found over a medium-stiff to stiff silt subgrade that transitions into a medium-dense silty sand. Retaining wall footings can be sized based upon an allowable bearing capacity of 2.5 kips per square foot (ksf). This allowable bearing capacity assumes the foundation is established on firm native subgrade soils, below all topsoil, and that frost heave depths are established in accordance with jurisdictional codes and procedures.

For sliding resistance the soils underlying spread footing can be assumed to have an ultimate coefficient of friction of 0.4.

Passive soil pressure can be developed along the sides of footings if the granular backfill is used around footings and the backfill is compacted to at least 95 percent of the material's maximum dry density as determined by ASTM D-1557 (Modified Proctor) testing. An equivalent passive fluid weight of 300 pcf can be used for resistance against sliding.

8.13 Mechanically Stabilized Embankment Retaining Wall Design

Mechanically Stabilized Embankment (MSE) retaining wall foundations should be designed for an allowable bearing capacity of 2,500 psf. This allowable bearing capacity assumes wall foundation pads or crushed rock leveling pads are established over firm native subgrade or structural fill. A passive pressure in resistance to lateral loads of 300 pcf equivalent fluid weight may be employed for MSE retaining walls embedded below finished surface grades. For sliding resistance, the soils underlying spread footing can be assumed to have an ultimate coefficient of friction of 0.4.

MSE retaining wall backfill can consist of any material that conforms to grid manufacturing specifications. The best long-term performance for MSE walls will result from use of a clean crushed rock backfill within the reinforced zone. We therefore recommend that MSE retaining wall backfill consist of clean, durable, free-draining crushed rock material. Use of clean crushed rock backfill within the geo-grid reinforced zone provides two advantages:

1) High shear strength, and thus increased long-term internal and global wall stability; and
2) Good drainage characteristics and thus reduced potential for developing hydraulic pressure over time along the backside of the wall.

Other backfill types will function within geo-grid reinforced walls subject to soil strength parameters outlined below. Whatever the final choice for the geo-grid reinforced backfill zone, we recommend that a drainage layer consisting of clean crushed rock be employed in the backfill zone immediately behind the back of the retaining wall. The drainage layer should extend a minimum of 12 to 18 inches laterally into the wall backfill zone. This drainage layer should consist of clean, well-graded crushed rock or drain rock material with less than 5 to 6 percent material by weight passing the No. 200 sieve. Use of other material could increase lateral pressures acting on the grid-reinforced wall. Overcompaction of the reinforced backfill adjacent to the wall can also greatly increase lateral soil pressures acting on the wall.

Typically the grid and wall block manufacturer will specify a recommended setback zone behind the wall in which a low compaction level should be adhered to. In many cases, the level of compaction in this zone will be between 90 and 92 percent of the maximum density determined in accordance with ASTM D-1557. Proprietary specifications will often call out that smaller compaction equipment such as light-weight self-propelled compactors, hand-operated vibratory skidders, or jumping jacks be employed in the zone immediately adjacent to the back of the retaining wall blocks.

Wall Drains. We recommend that a 6-inch-diameter perforated drain pipe be established at the heel of the geo-grid retaining wall foundations or along the top of the leveling pad or footing. The perforated pipe should be encapsulated in clean, free-draining crushed rock. A filter fabric or silt sock should be used to prevent internal soil erosion and potential clogging of the drains.

Backfill Soil Strength Design Recommendations. Recommended soil strength parameters for use in geo-grid reinforced retaining wall design are summarized in the following table. Soil cohesion should be assumed as zero.

MSE Wall Soil Strength Design Recommendations

Backfill Type	Design Friction Angle (phi)	Moist Soil Unit Weight (gamma)
Crushed Rock	40 degrees	135 pcf
Clean Sand	30 degrees	115 pcf
Native Silt or Sandy Silt	28 degrees	125 pcf

Traffic Surcharging Loads. If traffic loads are expected within a horizontal distance from the top of the geo-grid wall equal to the wall height, a uniform lateral earth pressure acting horizontally on geo-grid reinforced walls equal to 70 psf should be added to earth loads acting on the wall. If backslopes behind geo-grid reinforced walls are not horizontal, additional soil surcharge acting on the geo-grid retaining wall should be incorporated into global wall stability assessments.

Grid Reinforcement and Future Utilities. Grid reinforcement can be relatively fragile with respect to future excavation work into the reinforced zone. For this reason, any wall design involving grid-reinforced retaining features needs to take alignment of future underground utilities into account. Trenching through in-place grid reinforcement will destroy the integrity of a retaining system and destabilize the wall system. It is therefore critical to consider the impact of employing grid-reinforced retaining features with respect to future utility alignments.

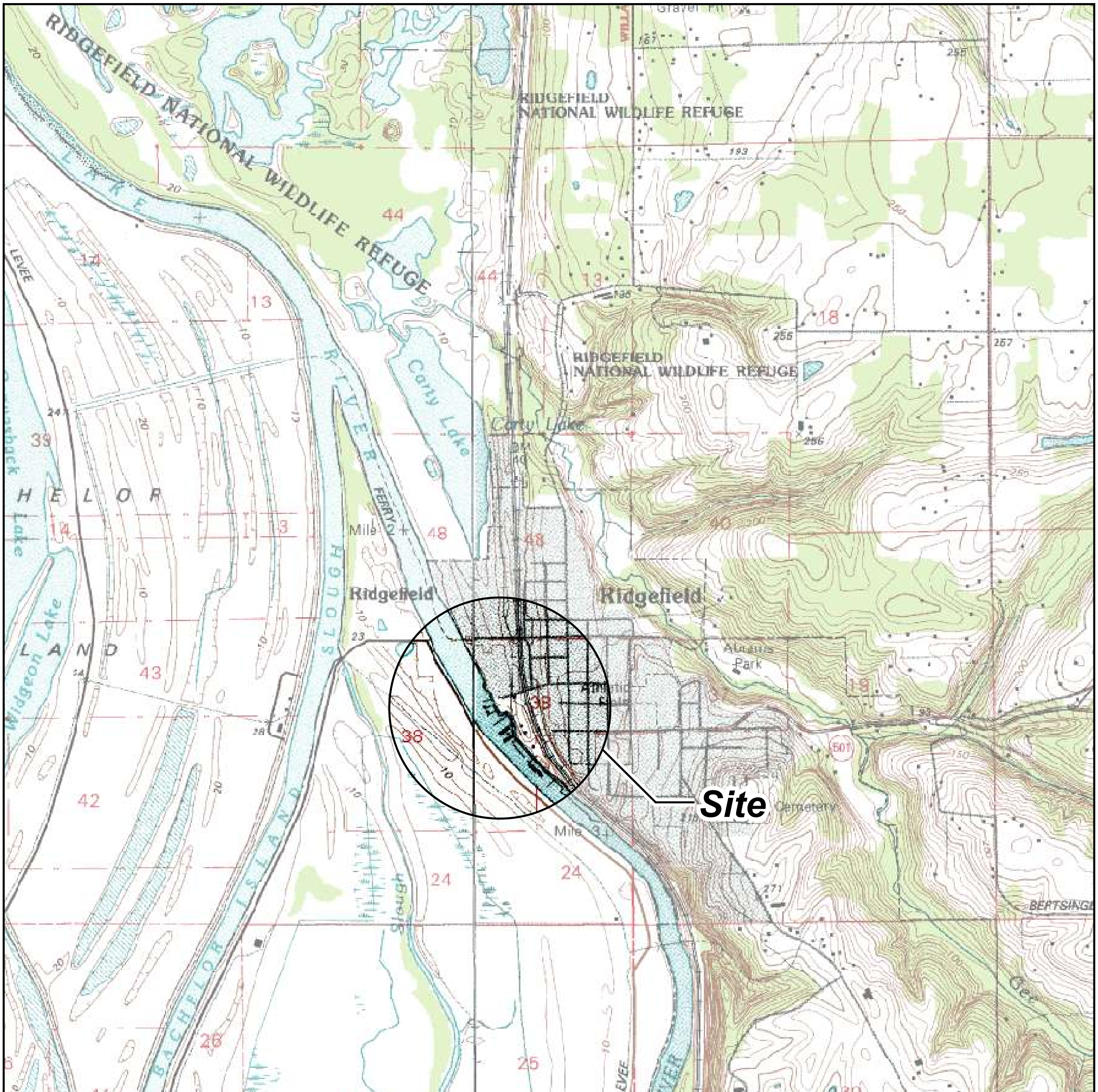
Seismic Loading. Lateral earth pressure acting on a retaining wall should be increased to account for seismic loadings. We recommend using a design horizontal acceleration coefficient of Kh = 0.17 (this is equal to 1/2 of the peak horizontal ground acceleration). The vertical acceleration coefficient (Kv) can be assumed as 1/2 of the horizontal acceleration component (although Kv, when taken as 1/2 Kh, affects dynamic active earth force [PAE] by less than 10 percent. Seed and Whitmann concluded that vertical accelerations can be ignored when the Mononobe-Okabe Earth Pressure Method is used to estimate PAE).

9.0 Additional Geotechnical Services

In order to correlate preliminary soil data with the actual soil conditions encountered during construction, and to assess construction conformance to our report, we recommend Ash Creek Associates be retained for construction observation of site preparation activities including excavation and compaction.

10.0 Closing

This report presented Ash Creek Associates' geotechnical engineering evaluation and recommendations for the proposed Ridgefield Rail Overpass Project. We trust this report meets your needs. If you have any questions, or if we can be of further assistance, please call. We look forward to working with you in the future.



Base map prepared from USGS 7.5-minute quadrangle of Ridgefield, WA, dated 1990 as provided by Topozone.

0 2,000 4,000

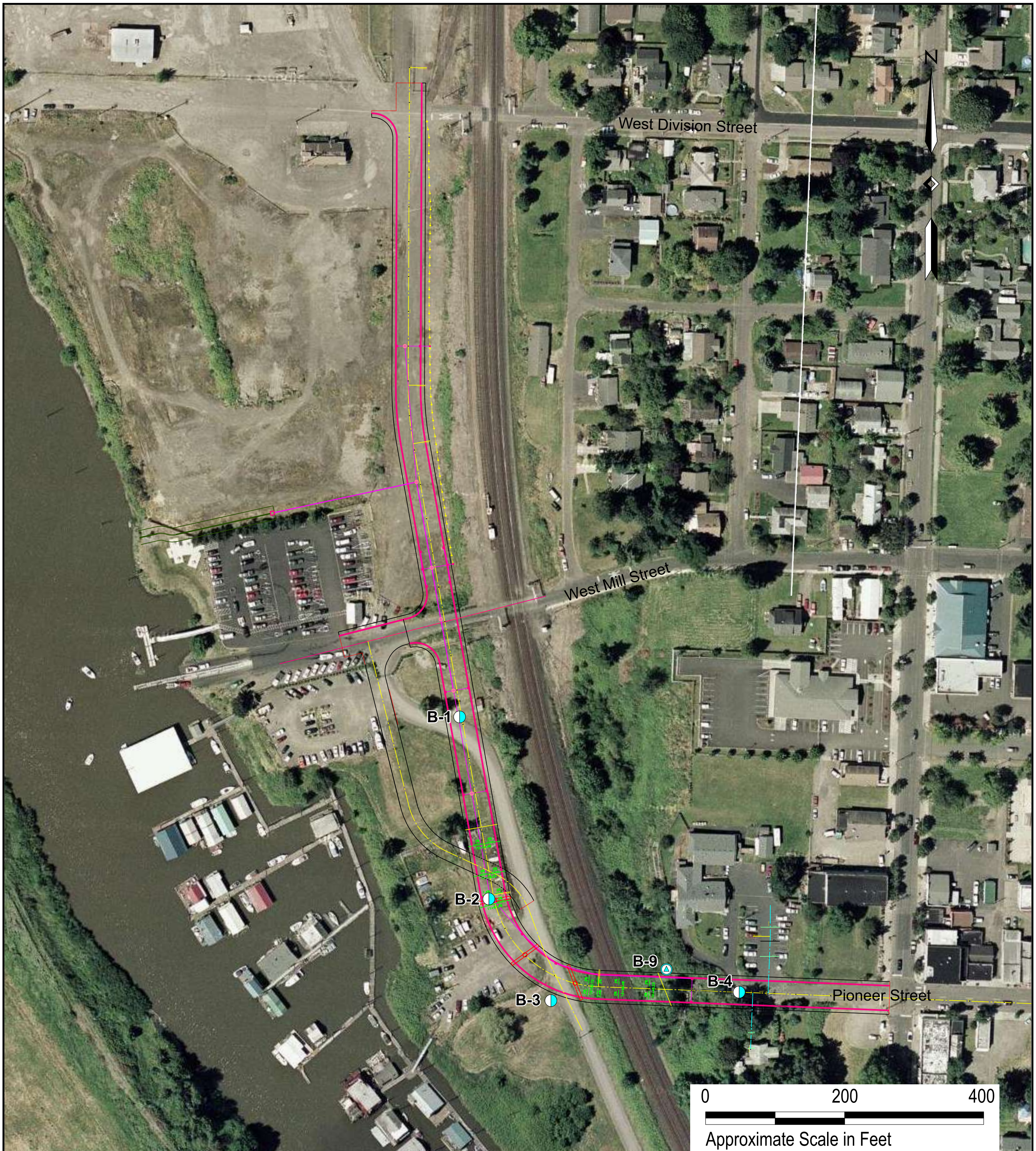
Scale in Feet

N

Site Location Map

Geotechnical Engineering Study
Ridgefield Rail Overpass Project
Ridgefield, Washington

Ash Creek Associates, Inc. Environmental and Geotechnical Consultants	Project Number	1161-00	Figure
	August 2007		1



Legend:

B-1 Boring Location

B-9 Boring Location (PBS)

Note: Aerial photo provided by Google Maps.

Site Exploration Plan

Geotechnical Engineering Study
Ridgefield Rail Overpass Project
Ridgefield, Washington

Ash Creek Associates, Inc.
Environmental and Geotechnical Consultants

Project Number	1161-00	Figure
August 2007		2

Appendix A

Subsurface Exploration Logs

Sample Descriptions

Classification of soils in this report is based on visual field and laboratory observations which include density/consistency, moisture condition, and grain size, 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 with additional remarks.

Density/Consistency

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

SAND and GRAVEL Density	Standard Penetration Resistance in Blows/Foot
Very loose	0 - 4
Loose	4 - 10
Medium dense	10 - 30
Dense	30 - 50
Very dense	>50

SILT or CLAY Density	Standard Penetration Resistance in Blows/Foot	Approximate Shear Strength in TSF
Very soft	0 - 2	<0.125
Soft	2 - 4	0.125 - 0.25
Medium stiff	4 - 8	0.25 - 0.5
Stiff	8 - 15	0.5 - 1.0
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 Symbols

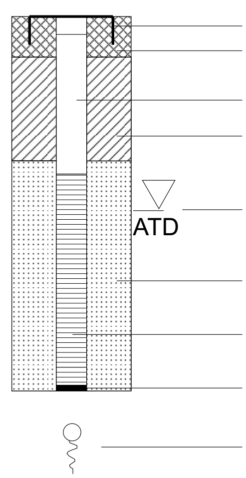
BORING AND GEOPROBE® SYMBOLS

⊠	Split Spoon
⧅	Tube (Shelby, Geoprobe®)
▥	Cuttings
◫	Core Run
▤	Temporarily Screened Interval
N	Standard Penetration Resistance
*	No Sample Recovery
P	Tube Pushed, Not Driven
PID	Photoionization Detector Reading
W	Water Sample
⬭	Sample Submitted for Chemical Analysis

TEST PIT SOIL SAMPLES

⊠	Grab (Jar)
⧄	Bag
⧅	Shelby Tube

Groundwater Observations and Monitoring Well Construction



Key to Exploration Logs

Geotechnical Engineering Study
Ridgefield Rail Overpass Project
Ridgefield, Washington

Ash Creek Associates, Inc.
Environmental and Geotechnical Consultants

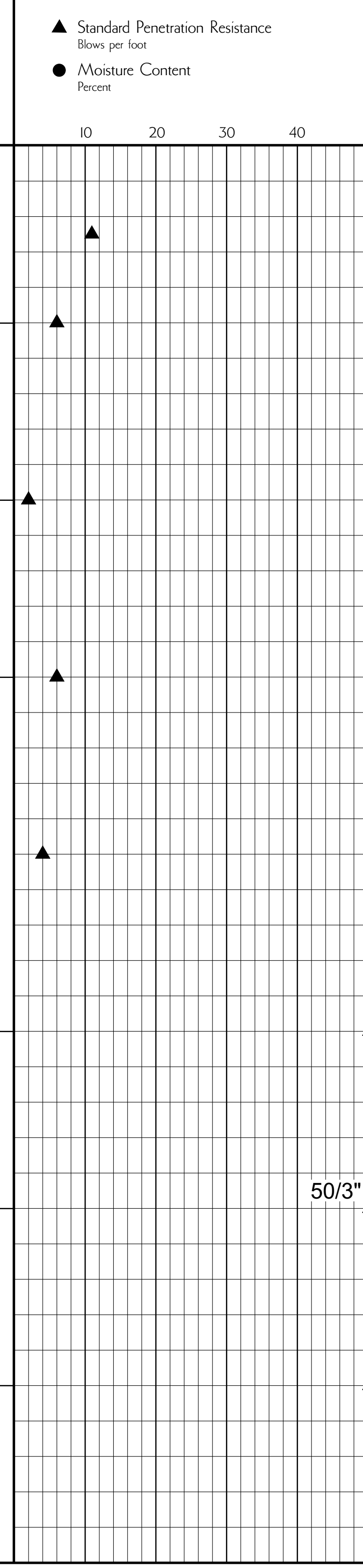
Project Number	1161-00	Figure
August 2007		Key

Ash Creek Associates, Inc.
Environmental and Geotechnical Consultants

Ridgefield Rail Overpass Project
Ridgefield, Washington

Log Of Exploration Number:	**B-1**
Project Number:	**1161-00**
Boring Location: See Figure 2	Surface Elevation: Not Measured
Drilling Contractor: Boart Longyear	Date Started: June 12, 2007
Drilling Method: 4.5" OD Tricone Mud Rotary	Date Finished: June 12, 2007
Drilling Equipment: Mobile B-59	Logged By: J. Duquette

Depth, feet	Sample ID	Sample	Material Description
	S-1	X	Stiff, wet, gray SILT.
5	S-2	X	Medium stiff, moist to wet, gray, clayey SILT.
10	S-3	X	No Recovery. Logged from cuttings. (Soft), wet, gray, clayey SILT.
15	S-4	X	Medium stiff, wet, gray, clayey SILT.
20	S-5	X	Coarse sand on tip of shoe.
25	S-6	X	Very dense, wet, tan-brown or gray, sandy GRAVEL with a little silt. Slow drilling below ~25.0'.
30	S-7	X	Very dense, wet, gray SAND and GRAVEL. Very little recovery.
			Layers of soft silt, dense gravel, and loose sand from ~32.0' to 34.0'.
			Very dense below 34.0'.
35	S-8	X	Very dense, wet, brown SAND.
			Gravel at 37.0'.



Ash Creek Associates, Inc. Environmental and Geotechnical Consultants	Ridgefield Rail Overpass Project Ridgefield, Washington	Log Of Exploration Number: **B-1** Project Number: **1161-00**
Boring Location: See Figure 2		Surface Elevation: Not Measured
Drilling Contractor: Boart Longyear		Date Started: June 12, 2007
Drilling Method: 4.5" OD Tricone Mud Rotary		Date Finished: June 12, 2007
Drilling Equipment: Mobile B-59		Logged By: J. Duquette

Depth, feet	Sample ID	Sample	Material Description	Standard Penetration Resistance (Blows per foot) / Moisture Content (Percent)
	S-9		Very dense, wet, gray, sandy GRAVEL. Very little recovery.	50/2"
45	S-10		Very dense, wet, gray GRAVEL. Very little recovery. Gravel fragments lodged in shoe.	50/2"
50	S-11		Very dense, wet, gray, sandy GRAVEL. Very little recovery.	50/1"
			Boring Terminated at ~50.0' BGS. No Groundwater or Seepage Encountered.	
55				
60				
65				
70				
75				

Ash Creek Associates, Inc.
Environmental and Geotechnical Consultants

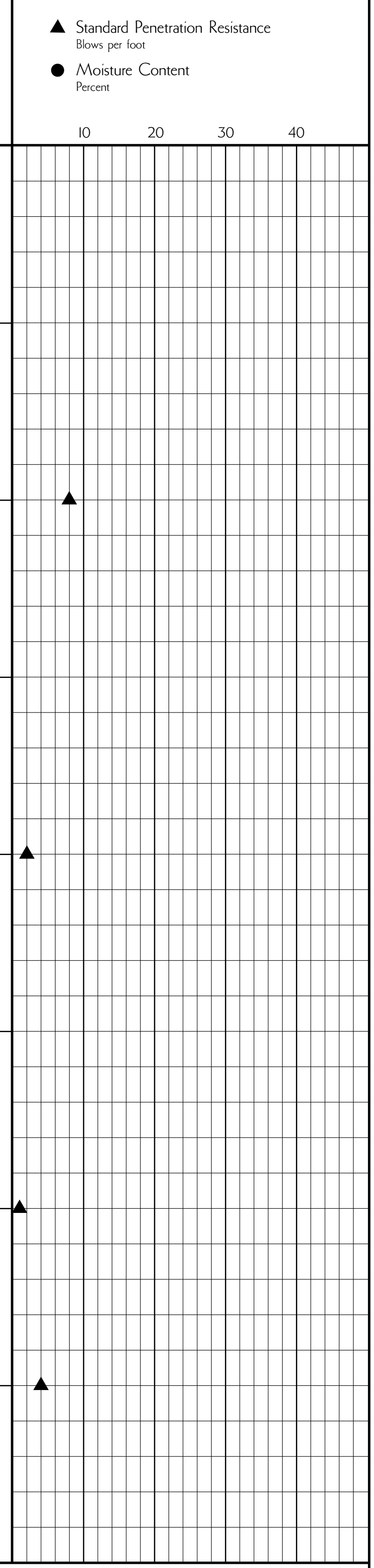
Ridgefield Rail Overpass Project
Ridgefield, Washington

Log Of Exploration Number: **B-2**

Project Number: **1161-00**

Boring Location: See Figure 2	Surface Elevation: Not Measured
Drilling Contractor: Boart Longyear	Date Started: June 12, 2007
Drilling Method: 4.5" OD Tricone Mud Rotary	Date Finished: June 12, 2007
Drilling Equipment: Mobile B-59	Logged By: J. Duquette

Depth, feet	Sample ID	Material Description
5	S-1	(Medium stiff), moist, gray SILT.
10	S-2	Medium stiff, moist, gray SILT with trace organics.
15	S-3	(Medium stiff), moist, gray, clayey SILT.
20	S-4	(Very soft), wet, gray SILT.
25	S-5	(Medium stiff), wet, gray SILT.
30	S-6	Very loose, wet, gray SILT with fine sand and trace wood fragments.
35	S-7	Loose, wet, gray, silty, fine SAND with trace wood fragments.
		Gravel at 39.0'. Slow drilling below ~40.0'.



Ash Creek Associates, Inc.
Environmental and Geotechnical Consultants

Ridgefield Rail Overpass Project
Ridgefield, Washington

Log Of Exploration Number: **B-2**

Project Number: **1161-00**

Boring Location: See Figure 2	Surface Elevation: Not Measured
Drilling Contractor: Boart Longyear	Date Started: June 12, 2007
Drilling Method: 4.5" OD Tricone Mud Rotary	Date Finished: June 12, 2007
Drilling Equipment: Mobile B-59	Logged By: J. Duquette

▲ Standard Penetration Resistance Blows per foot

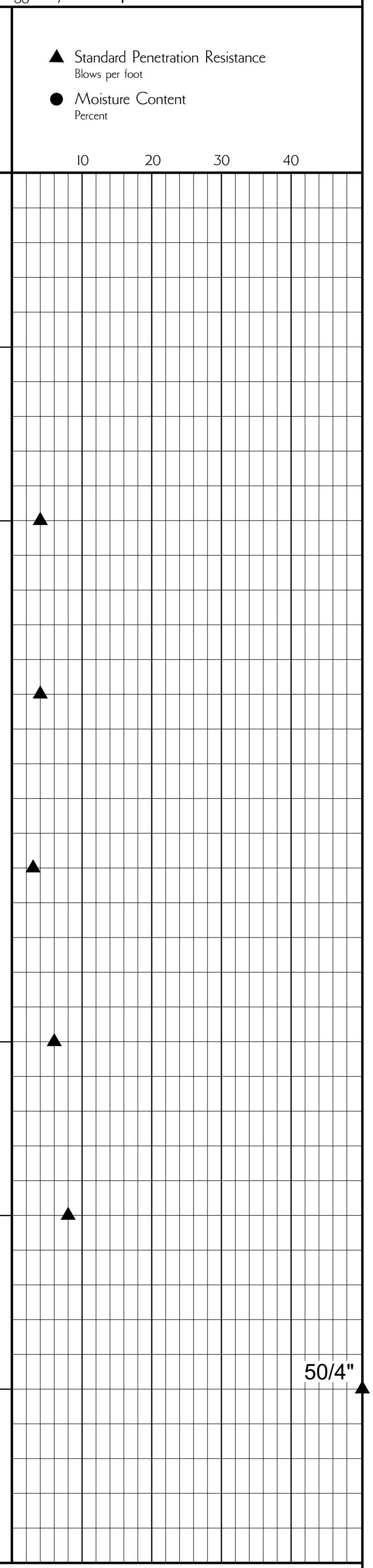
● Moisture Content Percent

Depth, feet	Sample ID	Material Description	Standard Penetration Resistance (10, 20, 30, 40)
	S-9	Very dense, wet, gray GRAVEL. Very little recovery.	50/3"
45	S-10	Very dense, wet, gray GRAVEL. Very little recovery.	50/3"
50	S-11	Very dense, wet, gray GRAVEL. Very little recovery.	50/3"
		Boring Terminated at ~50.0' BGS. No Groundwater or Seepage Encountered.	
55			
60			
65			
70			
75			

Ash Creek Associates, Inc. Environmental and Geotechnical Consultants	Ridgefield Rail Overpass Project Ridgefield, Washington	Log Of Exploration Number: B-3
		Project Number: 1161-00

Boring Location: See Figure 2	Surface Elevation: Not Measured
Drilling Contractor: Boart Longyear	Date Started: June 12, 2007
Drilling Method: 4.5" OD Tricone Mud Rotary	Date Finished: June 12, 2007
Drilling Equipment: Mobile B-59	Logged By: J. Duquette

Depth, feet	Sample ID	Material Description
5	S-1	(Soft to medium stiff), moist to wet, gray SILT.
10	S-2	Soft to medium stiff, wet, gray SILT with trace organics.
15	S-3	
20	S-4	A little very fine sand.
25	S-5	Medium stiff, wet, gray SILT and a little fine sand.
		Increasing fine sand below 28.0'.
30	S-6	Medium dense, wet, gray, silty fine SAND.
35	S-7	Very dense, wet, gray GRAVEL. Very little recovery.



Ash Creek Associates, Inc. Environmental and Geotechnical Consultants	Ridgefield Rail Overpass Project Ridgefield, Washington	Log Of Exploration Number: **B-3** Project Number: **1161-00**

Boring Location: See Figure 2	Surface Elevation: Not Measured
Drilling Contractor: Boart Longyear	Date Started: June 12, 2007
Drilling Method: 4.5" OD Tricone Mud Rotary	Date Finished: June 12, 2007
Drilling Equipment: Mobile B-59	Logged By: J. Duquette

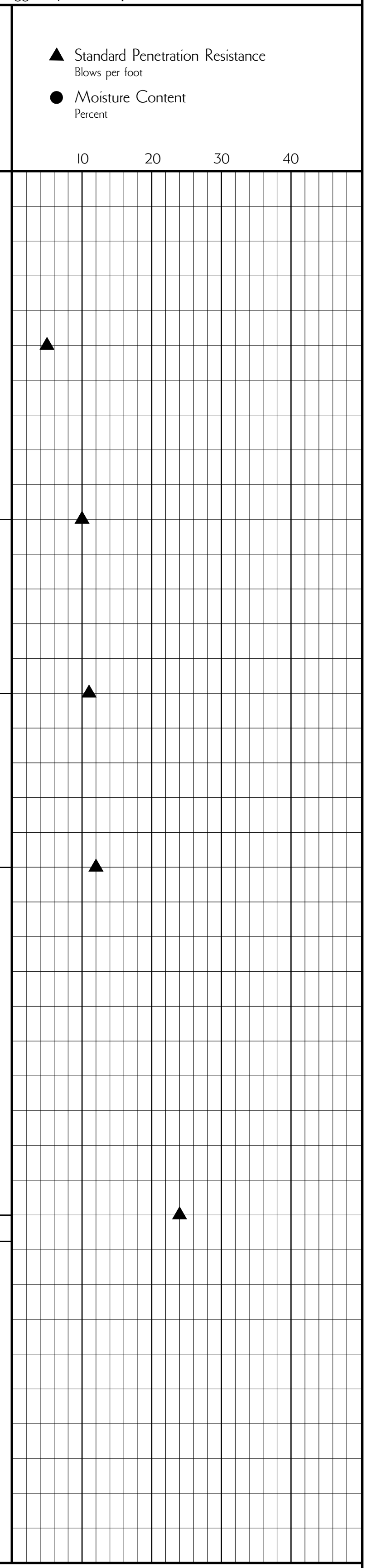
Depth, feet	Sample ID	Sample	Material Description	▲ Standard Penetration Resistance (Blows per foot) ● Moisture Content (Percent) 10 20 30 40
	S-8		Very dense, wet, gray GRAVEL. Very little recovery.	50/2"
45	S-9			50/1"
			Very dense, wet, gray GRAVEL. Very little recovery.	
			Boring Terminated at ~45.0' BGS. No Groundwater or Seepage Encountered.	
50				
55				
60				
65				
70				
75				

Ash Creek Associates, Inc.
Environmental and Geotechnical Consultants

Ridgefield Rail Overpass Project
Ridgefield, Washington

Log Of Exploration Number: **B-4**	Project Number: **1161-00**
Boring Location: See Figure 2	Surface Elevation: Not Measured
Drilling Contractor: Boart Longyear	Date Started: June 24, 2007
Drilling Method: 4.5" OD Tricone Mud Rotary	Date Finished: June 24, 2007
Drilling Equipment: Mobile B-59	Logged By: J. Duquette

Depth, feet	Sample ID	Sample	Material Description
5	S-1	X	Soft to medium stiff, damp, brown, silty CLAY with trace organics. Fill.
10	S-2	X	Stiff, damp, blue-gray, rust mottled CLAY to silty CLAY.
15	S-3	X	Stiff, damp to moist, blue-gray CLAY to silty CLAY.
20	S-4	X	Stiff, damp, blue-gray CLAY.
25			
30	S-5	X	Stiff, damp to moist, yellow-brown, silty CLAY.
			Stiff, damp, yellow-brown SILT.
35			



Ash Creek Associates, Inc.
Environmental and Geotechnical Consultants

Ridgefield Rail Overpass Project
Ridgefield, Washington

Log Of Exploration Number:	**B-4**
Project Number:	**1161-00**

Boring Location: See Figure 2	Surface Elevation: Not Measured
Drilling Contractor: Boart Longyear	Date Started: June 24, 2007
Drilling Method: 4.5" OD Tricone Mud Rotary	Date Finished: June 24, 2007
Drilling Equipment: Mobile B-59	Logged By: J. Duquette

▲ Standard Penetration Resistance
Blows per foot
● Moisture Content
Percent

Depth, feet	Sample ID	Sample	Material Description	Standard Penetration Resistance (10 / 20 / 30 / 40)
	S-6		Very dense, damp to moist, tan-brown SAND with a trace of fine silt.	▲ ~25
			Encountered dense gravel.	
	S-7		Very dense, wet, gray, coarse, sandy GRAVEL.	▲ >50
45				
	S-8			50/3" ▲
			Boring Terminated at ~48.0' BGS. No Groundwater or Seepage Encountered.	
50				
55				
60				
65				
70				
75				

PBS
1310 Main St.
Vancouver, WA 98660
Phone: (360) 690-4331
Fax: (360) 696-9064

RIDGEFIELD VILLAGE
RIDGEFIELD, WASHINGTON

PBS PROJECT NUMBER: 72477.000

APRIL 2007

DEPTH FEET	MATERIAL DESCRIPTION	ELEV DEPTH	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	Loose, dark brown, silty fine to medium SAND, moist, subangular, roots to 0.5' bgs (Flood Deposits)			Flush mount monument with one foot of concrete backfill
2.5	Becomes brown, has moderate orange mottles		4	1-inch standpipe piezometer
5.0			7	Bentonite chips
7.5	Medium stiff, brown, fine sandy silty CLAY with trace angular medium sand; moist, medium plasticity, heavy orange mottles	7.5	5	
10.0	Becomes stiff		12	
11.2	Medium dense, brown, fine to medium SAND, moist, subangular	11.2		
15.0			31	▽ Groundwater measured at 15.5 feet bgs on 03/27/07
16.0	Dense, dark greenish gray, medium SAND with some gravel, subangular, wet	16.0		
16.5	Very dense, gray, sandy GRAVEL, moist, subangular, fresh gravels (Alluvium)	16.5		
20.0			26-37-50/5"	
22.5				10/20 Sand
25.0	Becomes dense		48	1-inch slotted standpipe
30.0	Becomes very dense		50/6"	
35.0				

BORING METHOD: Mud Rotary
DRILLED BY: Subsurface Technologies, LLC
BORING BIT DIAMETER: 5 7/8"
LOGGED BY: P. Hughes
COMPLETED: 3/27/07

BORING B-9

PBS
1310 Main St.
Vancouver, WA 98660
Phone: (360) 690-4331
Fax: (360) 696-9064

RIDGEFIELD VILLAGE
RIDGEFIELD, WASHINGTON

PBS PROJECT NUMBER: 72477.000

APRIL 2007

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEV DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 50 100)	INSTALLATION AND COMMENTS
35.0						50/3"	
37.5							Bentonite chips
40.0						40-50/50.5"	
		40.9 feet, bottom of boring	40.9				
42.5							
45.0							
47.5							
50.0							
52.5							
55.0							
57.5							
60.0							
62.5							
65.0							
67.5							
70.0							

BORING METHOD: Mud Rotary
DRILLED BY: Subsurface Technologies, LLC
BORING BIT DIAMETER: 5 7/8"
LOGGED BY: P. Hughes
COMPLETED: 3/27/07

BORING B-9
(continued)