Design-Level Geotechnical Engineering Study
Proposed West Main Street Improvement Project
Kelso, Washington

Prepared for:
Otak, Inc.

September 24, 2010
1633-00

Ash Creek Associates, Inc.
Environmental and Geotechnical Consultants

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Jeff Duquette
Senior Project Engineer, Ash Creek Associates

STUART ALBRIGHT
STATE OF WASHINGTON
27196
REGISTERED
PROFESSIONAL ENGINEER

Stuart Albright, P.E.
Principal Engineer, Ash Creek Associates

3015 SW First Avenue
Portland, Oregon 97201-4707
(503) 924-4704 Portland
(360) 567-3977 Vancouver
(503) 943-6357 Fax
www.ashcreekassociates.com

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1.0 Introduction

This report presents Ash Creek Associates, Inc.'s (Ash Creek's) geotechnical recommendations for the proposed West Main Street Improvement Project in Kelso, Washington (Figure 1).

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:

1. Office Study: We completed a review of existing geotechnical and geological information available to staff. This review included published geology maps, Natural Resources Conservation Service (NRCS) soil surveys, and past geotechnical reports for the area.
2. Field Reconnaissance: After completing our document review, we performed a surface reconnaissance of the site. This included walking available portions of the site to observe surface manifestations of geotechnically related issues associated with the proposed roadway redevelopment.
3. Subsurface Explorations: We completed a subsurface exploration program consisting of drilled borings. The borings were accomplished using a small, trailer-mounted drill rig. Logs of subsurface conditions were maintained. Samples were identified in the field and returned to Ash Creek's office for further classification and testing. Infiltration testing was completed in three borings.
4. Analysis and Report: Ash Creek prepared this geotechnical report for the project.

3.0 Limitations of Our Work

This work was performed for the exclusive use of Otak, Inc. (Otak), 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's understanding of the project is based on discussions with members of the Design Team, as well as review of the preliminary alignment plans and information provided to Ash Creek by Otak. The objectives of the West Main Street realignment project include increasing safety for all modes of transportation including bicycles and pedestrians, reducing congestion between Kelso and Longview, and stimulating community revitalization.

While several alignment alternative routes were initially considered within the Study Area, this review focuses on the Preferred Alignment (Figure 2), which consists of transitioning the existing east/west principal arterial corridor from the easterly end of West Main Street over to Catlin Street (one block south), creating a "cross-over" that transitions West Main Street traffic to SR 4 (the Ocean Beach Highway/Cowlitz Way intersection) at the westerly end of Catlin. Kelso's existing classification of Catlin is *Public Road (Local Access)*, so the proposed project would reclassify the widened portion of Catlin to a Principal Arterial.

The preferred alternative transitions between 2nd Avenue and 3rd Avenue to align West Main Street with Catlin Street so that widening occurs to the south side of Catlin Street. This alignment provides enhanced safety for the future major arterial roadway by eliminating driveways and aligning the intersection of Cowlitz Way/Ocean Beach Highway/Catlin Street, which is currently offset 27 feet. A newly configured intersection between 2nd and 3rd Avenues will be signalized and provide access from the realignment to the existing West Main Street. A new traffic signal will also be added at the 5th Avenue/ Catlin Street intersection. The new roadway will provide bike lanes and sidewalks for its length and creates an opportunity for future City projects to make improvements along existing West Main Street by making the transition from West Main to Catlin at the east end of the district.

5.0 Site Description

The project site is located in the west Kelso area of Cowlitz County, Washington. The terrain gradient encompassing the improvement project can be described as relatively flat. Road section areas were established near native grades for the majority of the project area.

The land use in the project vicinity of the east-west trending West Main Street is mixed commercial and residential. Commercial land use is primarily centered on West Main Street with higher density of residential land use along surrounding streets to the north and south. Vegetation consists primarily of grass and residential landscaping shrubs.

6.0 Site Geology

6.1 Geologic Overview

Generally, the near-surface geology within the project area consists of recent alluvium (Qal). The Qal deposits are upper-Pleistocene- to Holocene-aged and typically consist of un-dissected terrace deposits along floodplains of rivers. These deposits are primarily sand and silt in the vicinity of the Columbia River and dominantly sand and basaltic gravel along the Cowlitz River.

Based on historical well logs obtained from the Washington State Department of Ecology (DOE) EIM database, alluvial and glaciofluvial sands, silts and gravels are expected to extend below the Study Area to more than 80 feet below the ground surface (bgs). Regionally, the City of Kelso is located approximately 20 miles north of the northern extent of the Portland Basin, which extends south into Oregon. In the Kelso area, the Cowlitz River is incised into the basaltic andesite of the Columbia River Basalt Group (CRBG) which is the dominant rock type in the vicinity. Shallow sediments in the Kelso area consist primarily of Quaternary alluvial deposits.

South of the City of Kelso, the CRBG is overlain by Quaternary significant alluvial and glaciofluvial (i.e., Missoula Flood) deposits, forming the Portland Basin. The Portland Basin is a structural depression dominated by four major groups of deposits which are described by the United States Geologic Survey (USGS, 2004) as:

(1) Quaternary alluvial and glaciofluvial deposits. The rates and composition of Quaternary deposition has been significantly influenced by (USGS, 2004): (1) episodes of mountain glaciations, (2) inundation of cataclysmic flooding events (i.e., the Missoula Floods), and (3) the level of regional volcanic activity. Quaternary deposits primarily consist of silt, sand, and gravel. Significant sediments were deposited during the cataclysmic flood events of the Missoula Floods. The flood deposits consist of large cobbles and boulders near the source areas of maximum velocity, such as the mouth of the Columbia Gorge. The more slack water deposits of the same flooding events consist of coarse- to fine-grained sands and slightly silty sands. The grain size decreases with distance from the Columbia River Gorge. Pyroclastic flows or lahars did not reach the Woodland area; however, fluvial-reworked volcanic debris from Mount St. Helens Quaternary is exposed in river terraces north of Woodland.

(2) Miocene to lower Pleistocene alluvial deposits. The basin is filled with alluvial deposits of the Sandy River Mudstone and Troutdale Formation (Trimble, 1963) which are over 1,000 feet in thickness at some areas. The Troutdale Formation disconformably overlies the fine-grained Sandy River Mudstone and is composed of a variety of sedimentary materials including unconsolidated sand and gravel, cemented gravels and cobbles (conglomerate), and hard sandstones.

(3) Miocene Columbia River Basalt Group. The majority of the tholeiitic CRBG was erupted from fissures in southeastern Washington and adjacent areas of Oregon and Idaho between 16.5 and 6 million years ago (Ma; USGS, 2004). The CRBG is composed of four primary basalt formations (Imnaha Basalt, Grande Ronde Basalt, Wanapum Basalt, and Saddleback Basalt) composed of numerous flows. CRBG that is exposed in the Woodland area belongs to the Grande Ronde formation, which was the most voluminous flow (Tolan and others, 1989) of the CRBG.

(4) Paleogene bedrock. The USGS (2004) describes this unit to consist of a diverse assortment of subaerially erupted lava flows, volcaniclastic rocks, and intrusive rocks (e.g., dikes, sills, and small plugs). The unit is generally composed of a variety of basalts, andesites, and dacites.

6.2 Natural Resources Conservation Service Soil Survey of Cowlitz County

Office review of the NRCS soil survey of Cowlitz County (1971; updated 2006) indicates the presence of one probable near-surface soil unit mantling the site. In general, the NRCS only classifies soils present in the upper 4 to 6 feet bgs. NRCS identifies this soil unit as Newberg Fine Sandy Loam.

The unified soil classification of Newberg Fine Sandy Loam has been established as SM. The American Association of State Highway and Transportation Officials (AASHTO) soil classification of this material is A1, A2, and A4. Shrink-swell potential of this material is considered low. The soil profile indicates moderately rapid permeability, increasing with depth. Soil pH ranges from 5.6 to 7.2. Fines content of this material (i.e., percentage of soil particles smaller than a standard No. 200 sieve) varies from 15 to 50 percent.

7.0 Subsurface Conditions

The field exploration program consisted of five drilled borings. These explorations were conducted on July 22, 2010. The locations of these explorations are shown on Figure 2. The logs for the explorations are 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.

Borings B-1 through B-5 were advanced using a 6-inch outside-diameter (OD), hollow-stem auger. A summary of subsurface conditions encountered within our borings has been provided below.

Asphalt or Gravel Surface. The majority of the borings were advanced through the road sections of city streets (Catlin Street, 3rd Avenue SW, and 4th Avenue SW). The asphalt and base rock vary across the site, but asphalt thickness is typically up to 6 inches. The one boring (B-5) that was not advanced through road sections was advanced in the gravel-covered area adjacent to West Main Street. The gravel section in this location was underlain by approximately 12 inches each of sand and dense gravel fill.

Fill. Consistent with the urban nature of the site, the majority of the area has been regraded during prior development. The pavement and gravel surfaces are underlain by thin levels of fill which were typically not directly sampled but were evident in the drill cuttings. In general, these fills were less than 2 feet thick, but deeper variations are likely within the project corridor.

Sand. The native soils underlying the site consist of very loose to medium dense, dry to wet, brown or to gray, silty sand, or sand. This unit was encountered in our explorations at depths ranging from immediately below the surface asphalt and base rock to the maximum depth of our borings (approximately 21.5 feet bgs).

Groundwater. Groundwater was noted at the time of our explorations at depths ranging from 10 to 13 feet bgs. Excavations below the groundwater table may require dewatering. In addition, permanent structures located beneath the groundwater table should be designed to account for uplift forces resultant from buoyancy effects. We anticipate that groundwater levels could rise several feet from levels measured during the winter and early spring.

7.1 WSDOT Soil Borrow Classification

The soils present on site are only suitable for reuse under Washington State Department of Transportation (WSDOT) specifications as Common Borrow. Based on WSDOT Standard Specifications, Common Borrow may be virtually any soil or aggregate either naturally occurring or processed which is substantially free of organics or other deleterious material and is non-plastic. The specification allows for the use of more plastic Common Borrow when approved by the engineer. On WSDOT projects, this material will generally be placed at 90 percent (Method B) or 95 percent (Method C) of Standard Proctor compaction. Common Borrow will likely have a high enough fines content to be moderately to highly moisture sensitive. This moisture sensitivity may affect the design property selection if placement conditions are likely to be marginal due to the timing of construction.

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 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, Ash Creek strongly recommends 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 re-compacted. These areas should be proof-rolled and, if the areas still deflect under wheel load, should subsequently be prepared in accordance with the recommendations provided in the Wet Weather Construction Section of this report (below). 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 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), static steel drum or grid roller. We recommend that Ash Creek 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, nonwoven, 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 re-compacted 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.

Based upon results from our research and explorations, 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. There will likely be areas of the site that will require subgrade stabilization during either new embankment fill construction or road subgrade reconstruction.

Marginal Subgrade Stabilization. Due to the previously developed nature of the site, random soft fill soils are likely to be encountered during site grading. It should also be anticipated that 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 re-compaction;
- 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 re-compacted. 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. Final embankment fill slopes should not exceed finished gradients of 2H:1V.

Fills should be installed in horizontal lifts and should be compacted in accordance with WSDOT Standard Specifications 2-03.3(14B and C) Method B. The majority of the materials available for borrow within the project limits would be classified as Common Borrow.

Much of the material generated from excavations and slope cuts on the project will contain a significant quantity of cobbles and boulders. Compaction control of these materials will need to be completed using WSDOT Test Method 606.

In order to achieve acceptable levels of compaction, it is generally desirable to maintain moisture contents of Common Borrow to within the range of 3 to 4 percent of the optimum moisture content. We anticipate that the majority of the borrow soils available on this site will be at moisture contents higher than this range. During dry weather, the soils will likely need to be dried in accordance with the aeration requirements of the WSDOT Standard Specifications, Section 2-03.3(15).

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.

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 meets the requirements for Common Borrow. However, if excess moisture causes the fill to pump or weave, those areas should be aerated and re-compacted 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.

Working Pads for Marginal Subgrade Areas. The working pad for stabilizing marginal subgrade areas should consist of durable, clean, crushed rock. This material should be relatively clean, with a low percentage of fines by weight. Materials conforming to the WSDOT standards for Quarry Spalls are generally acceptable for this purpose. Typically, a separation geotextile is placed between the overexcavated subgrade and quarry spall backfill.

8.4 Erosion Control

Ash Creek 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 stormwater disposal systems. Rip-rap, gabion baskets, or similar erosion control methods may be necessary at stormwater 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 over as much of the site as possible.

8.5 Temporary Excavation Slopes

Native soils may stand in 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 Act (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. Without advance dewatering, sloped excavations below the groundwater table will not be feasible.

8.6 Trench and Excavation Dewatering

Dewatering within trenches and excavations may be required for utilities. Groundwater at the time of our subsurface explorations was observed at depths between 10 to 13 feet bgs. However, groundwater levels may rise several feet above these levels during particularly wet winter and spring months.

Soils observed within our borings consisted of sands and silty sands. These soil types will cave/ravel and flow within excavations, particularly when such excavations are close to or below the groundwater table. The challenges associated with excavation work in this soil unit will include sloughing, caving of sidewalls, and possible base heave. This will typically lead to project delays, significant increases in bedding and backfill quantities, and the possibility of heaving soil within trench base areas.

To limit the potential for sidewall sloughing, flowing soils, and base heave, we strongly recommend excavations and trench areas be dewatered via well point installations prior to commencing any excavation work. Well points should be installed and groundwater drawdown conducted such that groundwater in the excavation zone is below the design base of excavation level prior to any digging. 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.

Water generated from dewatering will require disposal in accordance with state and city regulations. Water generated from dewatering and/or well point installation can often be laden with silt. Removal of silt via temporary settling ponds, Baker Tanks, etc., will typically be required.

8.7 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. Proper dewatering and shoring of excavations and trenches must also be conducted to limit the potential for undermining roads, sidewalks, structures, etc.

8.8 Stormwater Infiltration Testing

Ash Creek understands that the project will include the construction of stormwater infiltration features. Infiltration testing was conducted in July 2010 as part of the exploration activities. Infiltration testing was conducted in borings B-3, B-4, and B-5. Field infiltration testing was undertaken to determine percolation rates for the soils underlying the project site. The tests were conducted and the infiltration rates for the various test locations were determined based on these data.

Testing procedures entailed drilling a boring immediately adjacent to the existing boring location noted above. Infiltration tests were conducted within borings drilled using a 4-inch OD, solid-stem auger to a depth of 3 feet bgs. A 5-foot (60-inch) or 7-foot (84-inch) length of 4-inch-diameter polyvinyl chloride (PVC) pipe was inserted in the base of the boring.

Groundwater was encountered at depths ranging from 10 to 13 feet bgs and could rise several feet during wet weather. As such, the use of deeper infiltration structures (trenches or sumps) is likely precluded by the Western Washington Stormwater Manual requirements for separation. For this reason, we focused our field efforts on evaluating shallow soils for infiltration.

The soils at and below test depth were allowed to saturate for at least one hour. After the saturation period, a falling head test was conducted within the hollow-stem auger or PVC pipe. The decrease in head within the casing was timed over a 6-inch interval (12- to 6-inch head).

A summary of the groundwater depth, infiltration rates, and general soil conditions at the infiltration test depth are as follows:

B-1. Groundwater was encountered at approximately 10 feet bgs. Falling Head Infiltration Tests indicated bulk rates of 0.6 inch per hour (in/hr) at a depth of 3 feet (12 inches of head). The material consisted of loose, slightly silty, fine sand.

B-4. Groundwater was encountered at approximately 11 feet bgs. Falling Head Infiltration Tests indicated bulk rates of 3.0 in/hr at a depth of 3 feet (12 inches of head) and approx 4.5 in/hr at a depth of 6 feet (12 inches of head). The material consisted of loose, silty, very fine sand at 3 feet bgs, grading to slightly silty fine to medium sand at 6 feet bgs.

B-5. Groundwater was encountered at approximately 13 feet bgs. Falling Head Infiltration Tests indicated bulk rates of 6.5 in/hr at a depth of 4 feet (12 inches of head). The material consisted of medium dense, medium to coarse sand.

Based on the results of our groundwater evaluation, it does not appear that the use of deep infiltration systems (trenches or sumps) will be feasible for this project. Although not particularly prone to infiltration, the shallow soils can support the use of raingardens or other shallow systems. In general, bulk rates used in design should not exceed 3 in/hr, and design rates should reflect reductions recommended by WSDOT and/or the Western Washington manual.

The WSDOT Highway Runoff Manual M 31-16.01 indicates several factors for site infiltration suitability. These are detailed in section 4-5.1 of M 31-16.01. A summary of these factors is indicated below:

Setback. This includes consideration of locations of existing building foundations and basements which might be adversely affected by nearby infiltration features.

Groundwater Protection. Groundwater protection Areas defined in WAC 173-200. This includes well head, natural aquifer protection, etc.

Depth to Bedrock, the Groundwater Table and Impermeable Layer. No bedrock or impermeable layers were observed in our soil explorations. However groundwater was observed in all of our borings in July 2010. The WSDOT Highway Runoff Manual indicates: "The base of all infiltration basins or trench systems must be ≥ 5 feet above the seasonal highwater mark, bedrock (or hardpan), or other low-permeability layer. A separation down to 3 feet may be considered if the design of the overflow and/or bypass structures is judged by the site professional to be adequate to prevent overtopping and meet the SSC specified in this section."

Determination of Ksat. Ksat is the saturated hydraulic conductivity in centimeters per second (cm/s). Per WSDOT procedures, it is determined by the following equation:

$$\mathrm{Log(Ksat)} = -1.57 + 1.90D10 + 0.015D60 - 0.013D90 - 2.08\,F_{fines}$$

D10, D60 and D90 are the grain sizes in millimeters for percent weights for the soil in question
F_{fines} is the percent of soil passing a standard 0.075mm sieve (No. 200 Sieve)

The following table is the average diameter and fines content values for the native silt and native sand units underlying the site.

Soil Grain Size Values for Determination of Ksat

Soil Sample Underlying the Site	D10 (mm)	D60 (mm)	D90 (mm)	F_{fines}
B-1, 3.0 feet	0.020	0.075	0.132	.6 (60%)
B-4, 3.0 feet	0.033	0.094	0.138	.45 (45%)
B-4, 6.5 feet	0.070	0.152	0.260	.12 (12%)
B-5, 5.0 feet	10.4	0.96	0.40	.013 (1.3%)

We have analyzed the gradation data in accordance with WSDOT procedures to develop design infiltration rates (including the factor of safety) for subgrade infiltration. The results are included in the table below.

Calculated Ksat Values

Soil Sample Location	log(Ksat)	Ksat cm/s	Ksat ft/day
B-1, 3.0 feet	-2.780	0.0017	4.7
B-4, 3.0 feet	-2.443	0.0036	10
B-4, 6.5 feet	-1.687	0.021	60
B-5, 5.0 feet	-0.957	0.11	300

8.9 Pavement Designs

Design thicknesses were generally based upon the WSDOT Pavement Design Guide and the 1993 edition of the AASHTO Guide for Design of Pavement Structures. The proposed materials for the project consist of Asphalt Cement Concrete (AC) over crushed rock base for the majority of the project. The pavement construction will be completed in accordance with WSDOT Standard Specifications.

We obtained design traffic from the project traffic engineers (DKS). The data includes projected vehicle levels in both 2009 and 2030 as well as current truck counts. The Average Daily Traffic (ADT) values for 1st Avenue were 13,600 vehicles in 2009 and 21,230 in 2030. For Main Street the ADT values were 14,000 vehicles in 2009 and 23,300 vehicles in 2030. The truck percentages used were 4 percent for 1st Avenue and 5 percent for Main Street. Axle load distributions were based upon truck factors contained in the WSDOT Pavement Design Manual. The total calculated design traffic level for 1st Avenue is 3.0M ESALS and for Main Street is 4.1M ESALs.

The pavement subgrade resilient modulus (MR) was developed from site CBR testing and a review of adjacent explorations in the project vicinity. The project is located in an area of urban fill and native fine-grained soils.

Based upon this information, we have estimated a conservative resilient modulus value of 4,100 pounds per square inch (psi).

The following values were used in our analyses and were developed from the WSDOT Pavement Design Guide:

Pavement Design Parameters

Design Methodology	1993 AASHTO Guide for Design of Pavement Structures, WSDOT Pavement Design Guide.
Performance Period	20 years.
Reliability	85%
Serviceability	Initial Serviceability of 4.5, Terminal Serviceability of 3.0.
Overall Standard Deviation	0.5
Subgrade Resilient Modulus	4,100 based on California Bearing Ratio Testing (ASTM D-1883).
Base Course Resilient Modulus	25,000 psi based on WSDOT.
Layer Coefficients	AC layer coefficient was 0.42 based on area experience. The crushed rock coefficient of 0.12 was determined based on the calculated modulus.
Drainage Coefficient	1.0 for surfacing, 0.9 for base.

Notes: 1. ESAL = Equivalent single axle load. 2. psi = Pounds per square inch.

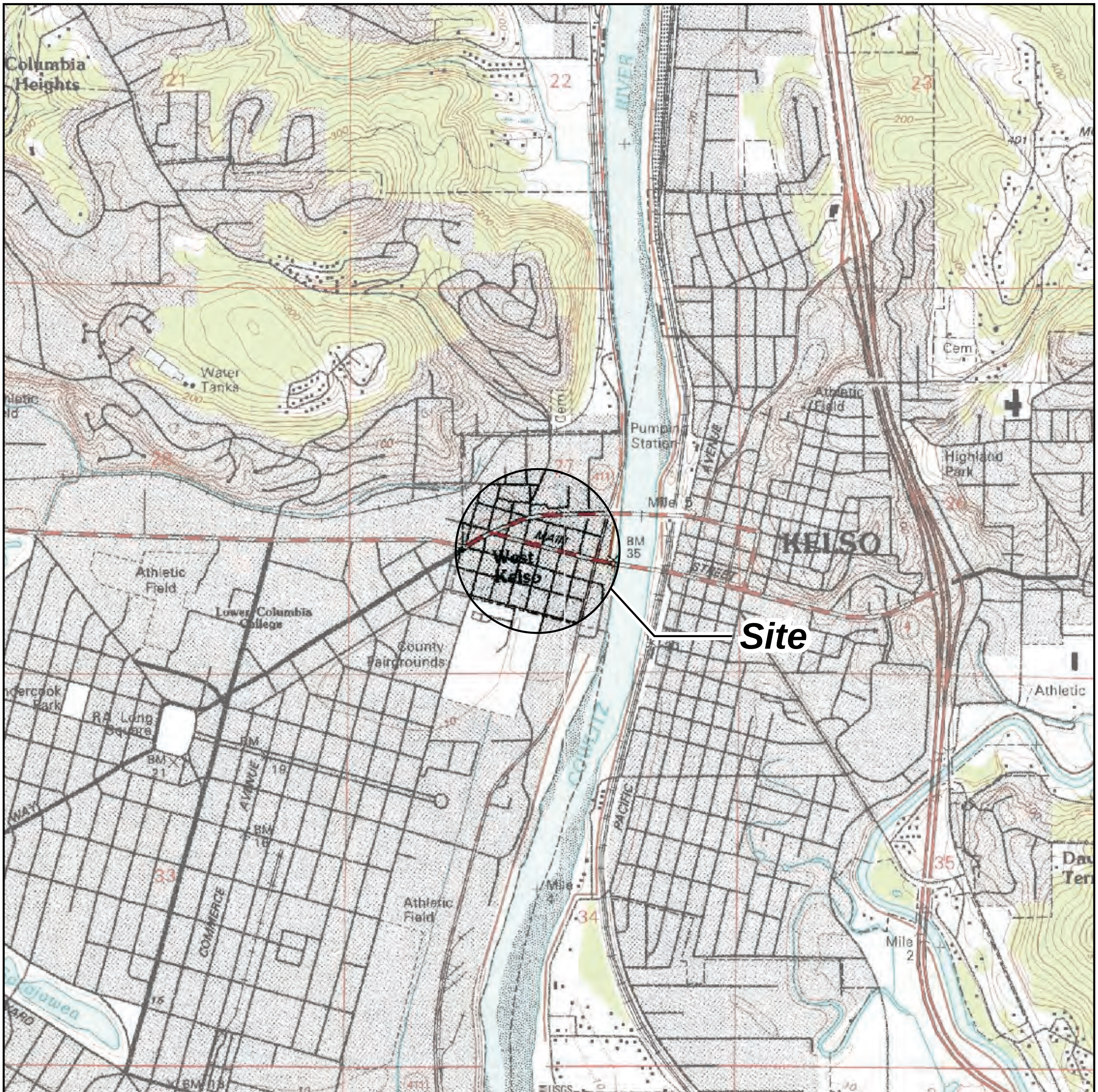
Recommended Pavement Designs. Based on the analysis documented above, we recommend the use of 8 inches of AC over 17 inches of crushed rock base (CRB) for 1st Avenue and of 9 inches of AC over 16 inches of CRB for Main Street. We anticipate that pavement construction will be completed in accordance with the WSDOT Standard Specifications.

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 be retained for construction observation of site preparation activities, including excavation and compaction.

10.0 Closing

This report presented Ash Creek's geotechnical engineering evaluation and recommendations for the proposed West Main Street Improvement Project in Kelso, Washington. 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.



Note: Base map prepared from USGS 7.5-minute quadrangle of Kelso, WA-OR, dated 1990 as provided by MSR Maps.com.

0 2,000 4,000
Approximate Scale in Feet

N

Site Location Map

West Main Street Realignment Project
Design-Level Geotechnical Evaluation
Kelso, Washington

Ash Creek Associates, Inc. Environmental and Geotechnical Consultants	Project Number	1663-00	Figure
	September 2010		1

Legend:

B-2 Boring Location

B-1 Boring/Infiltration Test Location

Preferred Alignment

NOTE: Base map prepared from 2010 - Google Imagery. Aerial dated August 14, 2005.

Site Exploration Plan

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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:
MAJOR CONSTITUENT with additional remarks; color, moisture, minor constituents, density/consistency.

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
Very soft	0 - 2
Soft	2 - 4
Medium stiff	4 - 8
Stiff	8 - 15
Very Stiff	15 - 30
Hard	>30

Moisture

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

Minor Constituents

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

Sampling Symbols

BORING AND PUSH-PROBE SYMBOLS

- ☒ Split Spoon
- ⧄ Sonic
- ⧅ Tube (Shelby, Push-Probe)
- ▥ Cuttings
- ◫ Core Run
- * No Sample Recovery
- SSA Solid Stem Auger
- HSA Hollow Stem Auger
- MR Mud Rotary

TEST PIT SOIL SAMPLES

- ☒ Grab
- ⧄ Bag
- ⧅ Shelby Tube

Key to Exploration Logs

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Project Number	1663-00	Figure
August 2010		Key

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West Main Street Realignment Project
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Boring Number:	**B-1**
Project Number:	**1663-00**
Logged By:	A. Reese
Date:	July 22, 2010
Site Conditions:	Overcast, 70° F
Drilling Contractor:	Subsurface Technolgies
Drilling Equipment:	Diedrich D-25, Truck-Mounted HSA
Sampler Type:	Split Spoon, SPT (Autohammer)
Depth to Water (ATD):	10'
Surface Elevation:	Not Measured

▲ Standard Penetration Resistance (Blows per Foot)

10 20 30 40

Depth, feet	Sample ID	Sample	Lithologic Description
		☒	Asphalt surface (~4") over base rock (~4") over silty, very fine SAND; pale brown (10YR 6/3), slightly moist, medium dense.
		☒	Becomes soft. Infiltration test conducted (Material remained dry after soak and infiltration testing).
5		☒	Becomes slightly moist and very soft.
		☒	Becomes moist and soft.
10		☒	With interbedded silt lenses, wet, and very soft.
15		☒	SIlty SAND; dark grayish brown (10YR 4/2), wet, very soft.
			Bottom of Boring at 16.5' BGS.
20			
25			
30			
35			

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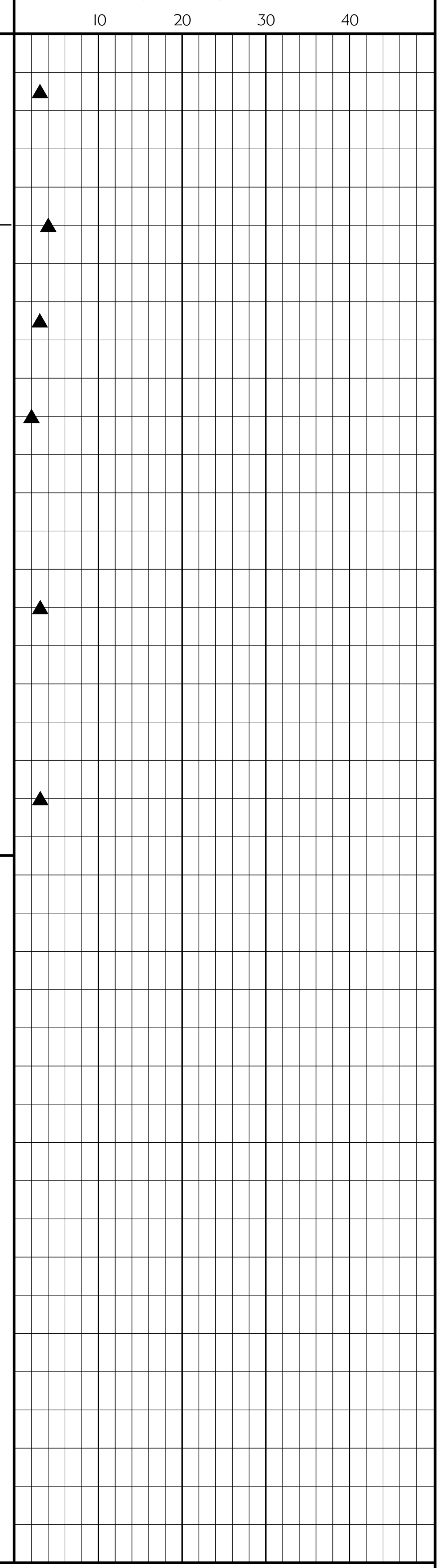
Boring Number:	**B-2**
Project Number:	**1663-00**
Logged By:	A. Reese
Date:	July 22, 2010
Site Conditions:	Overcast, 70° F
Drilling Contractor:	Subsurface Technolgies
Drilling Equipment:	Diedrich D-25, Truck-Mounted HSA
Sampler Type:	Split Spoon, SPT (Autohammer)
Depth to Water (ATD):	9'
Surface Elevation:	Not Measured

Depth, feet	Sample ID	Sample	Lithologic Description
0			Asphalt surface (~6") over base rock (~3").
		☒	Silty, fine SAND; pale brown (10YR 6/3), slightly moist, soft.
5		☒	SAND; dark yellowish brown (10YR 4/6), dry, fine to medium grained, very loose, clean.
		☒	Becomes yellowish brown (10YR 4/6), slightly silty (<5%).
10		☒	Becomes wet with a higher silt content (~20%).
15		☒	
20		☒	Becomes dark brownish gray (10YR 4/2), slightly silty (>5%).
			Bottom of Boring at 21.5' BGS.
25			
30			
35			

▲ Standard Penetration Resistance (Blows per Foot)



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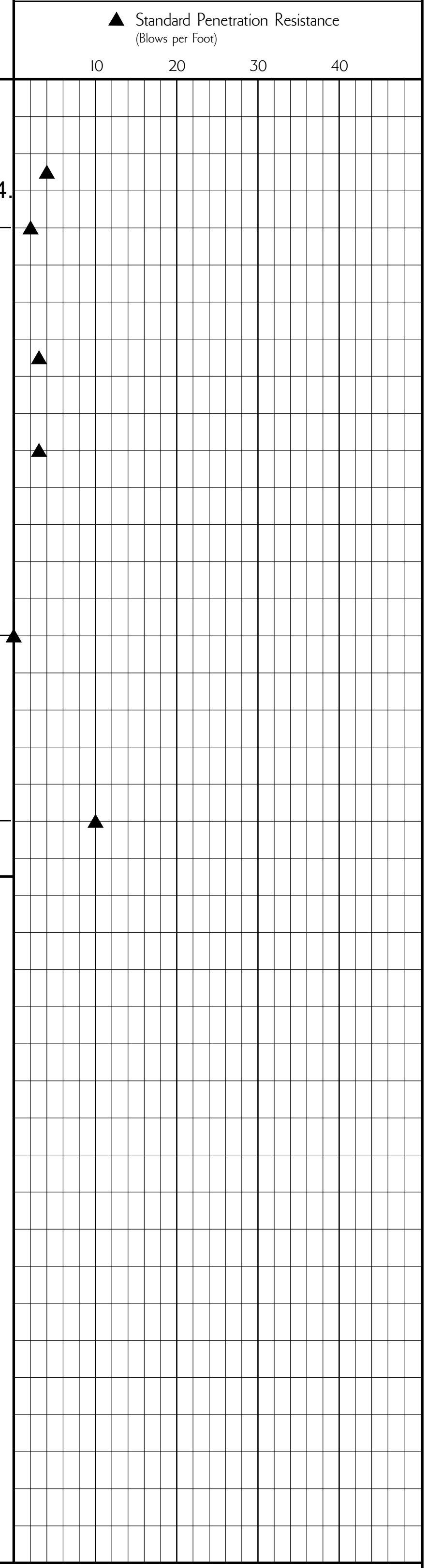
Boring Number:	**B-3**
Project Number:	**1663-00**
Logged By:	A. Reese
Date:	July 22, 2010
Site Conditions:	Overcast, 70° F
Drilling Contractor:	Subsurface Technolgies
Drilling Equipment:	Diedrich D-25, Truck-Mounted HSA
Sampler Type:	Split Spoon, SPT (Autohammer)
Depth to Water (ATD):	~10'
Surface Elevation:	Not Measured

Depth, feet	Sample ID	Sample	Lithologic Description
0			Asphalt surface (4") over base rock (4").
2.5		☒	No recovery. Very fine SAND; slightly silty (from cuttings). Resampled at 4.
4		☒	SAND; brown (10YR 4/3), slightly moist, fine to medium grained, slightly silty (~10%), very loose.
7.5		☒	Becomes more silty (~25%) Becomes (mottled (gray/brown) and moist.
10		☒	Interbedded silt and sand lenses, brown and brown/gray mottled, wet, soft to loose.
15		☒	Silty SAND; brown (10YR 3/3), wet, very soft.
20		☒	SAND; dark gray (10YR 4/1), wet, medium grained, slightly silty, loose.
21.5			Bottom of Boring at 21.5' BGS.



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West Main Street Realignment Project
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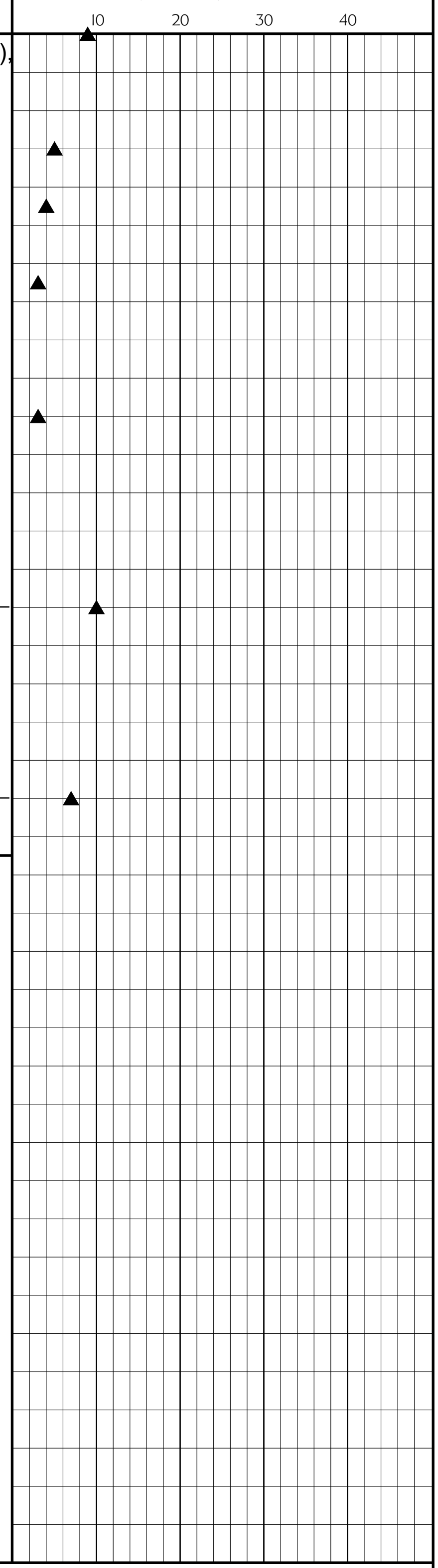
Boring Number:	**B-4**
Project Number:	**1663-00**
Logged By:	A. Reese
Date:	July 22, 2010
Site Conditions:	Overcast, 70° F
Drilling Contractor:	Subsurface Technolgies
Drilling Equipment:	Diedrich D-25, Truck-Mounted HSA
Sampler Type:	Split Spoon, SPT (Autohammer)
Depth to Water (ATD):	~11'
Surface Elevation:	Not Measured

Depth, feet	Sample ID	Sample	Lithologic Description
0		X	Asphalt surface (4") over base rock (3") over SAND; pale brown (10YR 6/3), slightly moist to dry, fine to medium grained, slightly silty (<5%), medium dense.
3		X	Becomes fine grained, slightly moist, and loose. Infiltration test conducted.
4.5		X	Becomes fine to medium grained and dry.
6			Infiltration test conducted.
6.5		X	
10		X	Becomes wet and slightly silty.
15		X	Silty SAND; dark yellowish brown (10YR 4/6), wet, medium to coarse grained, medium dense.
20		X	SAND; dark gray (10YR 4/1), wet, coarse grained, medium dense.
			Bottom of Boring at 16.5' BGS.

▲ Standard Penetration Resistance (Blows per Foot)



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West Main Street Realignment Project
Design-Level Geotechnical Evaluation
Kelso, Washington

Boring Number:	**B-5**
Project Number:	**1663-00**
Logged By:	A. Reese
Date:	July 22, 2010
Site Conditions:	Overcast, 70° F
Drilling Contractor:	Subsurface Technolgies
Drilling Equipment:	Diedrich D-25, Truck-Mounted HSA
Sampler Type:	Split Spoon, SPT (Autohammer)
Depth to Water (ATD):	~13'
Surface Elevation:	Not Measured

Depth, feet	Sample ID	Sample	Lithologic Description
0			Gravel surface (6") over sandy FILL (12") over gravel FILL (12").
2.5		X	SAND; dark yellowish brown (10YR 4/6), dry, medium grained, medium dense, clean. Infiltration test conducted.
5		X	Becomes coarse grained.
7.5		X	Becomes medium grained and loose.
10		X	Silty, very fine SAND; brown (10YR 4/3), slightly moist, loose.
15		X	Becomes wet with interbedded silt lenses.
20		X	Becomes wet and very soft.
21.5			Bottom of Boring at 21.5' BGS.

▲ Standard Penetration Resistance
(Blows per Foot)