SPECIAL PROVISIONS

FOR

69TH AVE NE CULVERT REPLACEMENT

FOR

CITY OF ARLINGTON

JUNE 2013

MURRAY, SMITH & ASSOCIATES, INC.
2707 Colby Avenue, Suite 1110
Everett, WA 98201
(425) 252-9003

SPECIAL PROVISIONS

The work on this project shall be accomplished in accordance with the Standard Specifications for Road, Bridge and Municipal Construction 2012 edition prepared by the Washington State Department of Transportation, hereinafter referred to as the "Standard Specifications".

The following Special Provisions are made a part of this contract and supersede any conflicting provisions of the 2012 Standard Specifications for Road, Bridge and Municipal Construction, and the foregoing Amendments to the Standard Specifications.

The Standard Specifications, as modified or supplemented by the Amendments, City of Arlington Design and Construction Standards and Specifications, and these Special Provisions, shall govern all of the Work. The deletion, amendment, alteration, or addition to any subsection or portion of the Standard Specifications is meant to pertain only to that particular portion of the section, and in no way should be interpreted that the balance of the section does not apply. In case of conflict between the various elements of the Contract Documents, refer to Section 1-04.2 of these Special Provisions for order of precedence.

Several types of Special Provisions are included in this contract; General, APWA, Local, Bridges and Structures, and Project Specific. Special Provisions types are differentiated as follows:

(date)	General Special Provision
(******)	Notes a revision to a General Special Provision and also notes a Project Specific Special Provision.
(Date APWA GSP)	APWA Special Provision
(Date COA GSP)	Local Special Provision

General Special Provisions (GSP) are similar to Standard Specifications in that they typically apply to many projects, usually in more than one Region. Usually, the only difference from one project to another is the inclusion of variable project data, inserted as a "fill-in".

APWA Special Provision are similar to General Special Provisions in that they typically apply to many projects, usually in more than one Region. However, they are modified for Local Agencies to use on smaller projects than WSDOT.

Local Special Provisions are similar to Standard Specifications in that they typically apply to many project within the City of Arlington. Usually, the only difference from one project to another is the inclusion of variable project data, inserted as a "fill-in".

Project Specific Special Provisions normally appear only in the contract for which they were developed.

DESCRIPTION OF WORK

The work to be performed under these specifications and drawings consists of the replacement of a culvert on Prairie Creek under 69th Avenue Northeast. Work shall also includes rip rap slope protection, embankment, and retaining wall construction.

The above general outline of principal features of the work does not in any way limit the responsibility of the Contractor(s) to perform all work and furnish all equipment, labor and materials required by the specifications and drawings. The drawings and specifications shall be considered and used together. Anything appearing as a requirement of either shall be accepted as applicable to both even though not so stated therein or shown.

Division 1
General Requirements

1-11 TEMPORARY WATER MANAGEMENT

Add the following new section:

1-11.1 Submittals

Prior to the Pre-Construction Meeting, the Contractor shall prepare and submit a Flow Diversion/Dewatering Plan and a Fish Exclusion Protocol Plan.

The Contractor shall include necessary fish exclusion and removal measures in the Plans. Fish exclusion activities and implementation of a Fish Exclusion Protocol Plan shall be completed under the direct, on-site supervision of a person or persons declared a certified or associate “Fisheries Professional” by the American Fisheries Society. The American Fisheries Society certification number of the Fisheries Biologist(s) shall be submitted to Owner’s Representative prior to construction.

The Flow Diversion/Dewatering Plan shall comply with all permit requirements, including seasonal restrictions.

1-11.2 Plan Contents

The Fish Exclusion Protocol Plan prepared by the Contractor shall describe how the Contractor will remove and exclude fish from the work area prior to any work within the Ordinary High Water Mark of Prairie Creek. All fish shall be removed from the project area under the direct on-site supervision of a qualified Fisheries Biologist before construction begins. Fish exclusion shall comply with all permit requirements. Unless otherwise specified in the permit requirements, capture techniques shall be used that will minimize stress to the fish. Fish shall be handled in such a manner as to avoid and minimize fish mortality. Once fish are safely removed from the site they shall be carefully relocated to a permanent open area upstream or downstream of the project limits. The Contractor shall execute the fish exclusion/removal plan methods and protocols prior to any unique flow diversion event when fish may be present in the work area.

1-11.3 Water Control and Diversion

Work in the stream will require diversions around or through the work area and may require surface and groundwater dewatering. The Contractor shall install and maintain a temporary flow diversion system that:

1. Shall not be implemented prior to implementation of the Fish Exclusion Protocol Plan for that particular reach or area.
2. Protects adjacent waters from turbidity and other water quality disturbances caused by construction.
3. Allows for removal of surface water and groundwater and other water entering the construction area.
4. Provides for Diversion/dewatering such that water flow shall be ramped such that any remaining stranded fish can be removed before the affected reach is completely devoid of surface flows.

5. Includes a schedule or process for rewatering work areas following construction.
6. Meets all permit requirements, including being sufficiently protective of fish and other aquatic wildlife.

Prior to beginning work, the Contractor shall have in-place a Flow Diversion/Dewatering Plan approved by the Owner’s Representative.

The Contractor shall provide the Owner’s Representative with a minimum of 5 days advance notice of the start of each unique dewatering/diversion activities.

Any pumps used in the diversion/dewatering system shall be placed in flat areas a sufficient distance from the channel and adequately secured (by anchoring to a tree or stake or similar). Pump intakes will be covered with 1/8-inch mesh to prevent entrainment of fish or amphibians, and shall be checked periodically for impingement of fish and amphibians. Pump discharges shall be outfitted with approved energy dissipation devices.

The Contractor will anticipate the need to provide for fish removal and flow diversions/dewatering for two surface water sources upstream of the 74th Avenue NE culvert. Water enters the work area from the southeast via Prairie Creek and from the east via an unnamed tributary.

The Contractor’s plan shall not include a provision for constant pumping. Pumping shall only be allowed during regular work hours as defined in the Contract. Pumps shall not be unattended and shall be shut down when the work site is not occupied by the Contractor.

Water shall be treated and contained as needed to adequately remove suspended sediment prior to disposal. Water shall be disposed of in an environmentally acceptable manner, in accordance with project permits, applicable laws, and such that property is not damaged.

As a contingency, sufficient backup equipment shall be maintained at the site as needed to ensure that stream flows will be diverted at all times. Contractor shall have a contingency plan for possible malfunction or failure of equipment such as pumps, plugs, piping and power source.

All activities shall be scheduled to minimize the length of time during which the dewatering and flow diversion will be necessary, and shall minimize impacts to aquatic resources.

Flows expected in Prairie Creek are as follows:

Design Storm	Flow Rate
2-year	73 cfs
10-year	151 cfs
100-year	218 cfs

These flows are based on basin characteristics and hydrologic modeling performed for the project.

The contractor will phase/meter both diversion/dewatering and rewatering activities in such a manner that downstream reaches of Prairie Creek are neither starved for flow nor subject to excessive flows that may mobilize sedimentation or cause erosion below the work areas.

The City has obtained a Hydraulic Project Approval (HPA) for the project which contains specific requirements for bypass pumping and fish capture and movement. In addition to the requirements of the HPA, any fish capture and movement must comply with the requirements in the Washington State Department of Transportation's Fish Exclusion Protocols and Standards. Specific measures to bypass the streams and move fish shall be addressed in the Flow Diversion/Dewatering Plan described in this section.

1-11.4 Measurement

No specific unit of measurement will apply to "Temporary Water Management."

1-11.5 Payment

There shall be no separate payment for "Temporary Water Management."

END OF DIVISION 1

Division 7
Drainage Structures, Storm Sewers, Sanitary Sewers, Water Mains, and Conduits

7-02 Culverts

7-02.3(6) Precast Reinforced Concrete Three-Sided Culvert

Add the following new section:

7-02.3(6)A General

This work consists of furnishing and constructing precast reinforced concrete three sided culvert with precast headwall and wing walls from Granite Precasting & Concrete, Old Castle Precast, or Hanson Pipe & Products under 69th Avenue Northeast. Provide reinforcement and portland cement concrete according to AASHTO M273M (ASTM C 850M) for precast construction. Provide watertight joint seals.

7-02.3(6)B Submittals

Submit the following items for approval within one week after the Notice to Proceed is issued.

a. Design Summary: Submit five copies of the Design Summary. The Design Summary shall be a summary sheet identifying the most important features of design including (but not limited to) the following:

   - References used.
   - Design assumptions.
   - Material specifications (i.e., f' c and fy).
   - Design loads.
   - Soil and hydraulic requirements.
   - Special staging, handling and/or installation requirements.

b. Design Calculations and Detail Plans: Submit five sets of the Design Calculations and Detail Plans. Both the Design Calculations and Detail Plans shall be signed (with the PE seal) by a Professional Engineer registered to practice in the state of Washington. Half size (11" x 17") format is required for the Detail Plans.

c. Shop Drawings: Submit five sets of the Shop Drawings. Half size (11" x 17") format is required for the Shop Drawing details.

7-02.3(6)C Installation

Care shall be taken in handling and transporting to avoid damaging pipes and their coatings. Loading and unloading shall be accomplished with the culvert sections under control at all times and under no circumstances shall the culvert sections be dropped.

All culvert sections and jointing materials shall be carefully examined for defects and no piece shall be laid that is known to be defective. Any defective piece installed shall be removed and replaced with a new pipe section in a manner satisfactory to the Owner's Representative at the Contractor's expense. Defective material shall be marked and removed from the job site before the end of the day.

The structure shall be assembled in accordance with the shop drawings and layout provided by the manufacturer.

The Contractor shall provide footings as required per the plans and specifications.

The Contractor shall provide proper bedding and backfill to avoid distortion that may create undesirable stresses in the structure or settlement of the roadway, or both. The bedding shall be free of rock formations, protrusions, frozen material, and organic material. Support base shall be inspected prior to placement of the culvert sections.

The structure shall be backfilled using gravel borrow according to Section 9-03.14(1) of the Standard Specifications.

a. Backfill materials shall be placed in symmetrical lifts on each side of the structure. The differential between the lifts on either side shall not exceed 24 inches at any time. Each layer of soil shall be placed in 6 to 8 inches loose thickness and compacted to a minimum of 95% density per ASTM D1557 (Modified Proctor).

b. Backfill soils shall be free of rocks exceeding 3 inches in diameter, frozen matter, ice, organic matter, and foreign materials.

c. If the native material has a high percentage of silt or fine sand, well graded granular material must be used in the critical backfill zone or non-woven geotextile must be used to prevent soil migration.

d. During backfilling operations, only small tracked construction equipment shall be near the structure as fill progresses above the crown and to the minimum height of cover. Cover over the structure shall be determined by measuring from the crown of the structure to the bottom of flexible pavement or to the top of rigid pavement. After adequate cover and compaction is achieved, live loads may increase at the discretion of the Engineer.

7-02.4 Measurement

Replace this section with the following:

No specific units of measurement will apply to the lump sum items for "Precast Reinforced Concrete Three-Sided Culvert."

Precast Reinforced Concrete Three-Sided Culvert

7-02.5 Payment

Replace this section with the following:

There shall be no separate payment for “Precast Reinforced Concrete Three-Sided Culvert”.

7-16 Temporary Force Main Bypass (New Section)

7-16.1 Description

The City of Arlington's Lift Station No. 2 continuously pumps wastewater in an 8-inch force main, which has generally ascending profile westward on 204th St NE, then continuing north on 69th Ave NE west of the BNSF right-of-way. The functions of the force main shall not be compromised during the course of the Work except as specified herein. Contractor shall plan and prosecute the Work such that operation of the force main operation is not interrupted for a period of time greater than two (2) consecutive hours.

The existing sewer lift station must be kept in operation through the use of existing or temporary bypass piping until the new force main piping is accepted by the Owner and capable of accepting the raw sewage.

Pumping disruptions could potentially result in the spillage or discharge of municipal wastewater. State law allows Department of Ecology (DOE) to impose civil penalties for spillage or wastewater. A person who unlawfully pollutes water as specified is subject to criminal prosecution.

Spillage or discharge of wastewater to surface waters or drainage courses is prohibited during construction. Penalties imposed on Owner as a result of any bypass caused by Contractor, his employees or subcontractors, and legal fees and other expenses to Owner resulting directly or indirectly from the bypass shall be borne in full by Contractor.

Contractor is responsible to plan, schedule, and sequence his construction activities to ensure that pumping of wastewater at all times is uninterrupted.

Contractor shall be responsible for controlling any and all leakage resulting from or integral to making all temporary and permanent piping connections, and shall provide any and all devices required to control, stop, divert, or dispose of any and all leakage.

7-16.1(1)Submittals

Submit an individual Temporary Force Main Bypass Piping Plan for approval for each Contractor planned bypass. Submit plan a minimum of two (2) weeks prior to the proposed date of installation of the temporary bypass piping. Do not construct or place temporary bypass piping until the Owner and Engineer have reviewed plans and responded in writing of their approval.

Include the following information, at a minimum, in each plan:

1. Proposed schedule for the installation of bypass piping, including dates and times for the connection to the existing force main, duration of bypass piping installation and proposed connection of the final, permanent, force main piping.
2. Drawing showing layout and routing of bypass piping, fittings, and valves with associated sizes and dimensions and any required temporary thrust restraint.

3. Material data for bypass piping, hose, valves, and fittings.
4. Contingency plan describing steps to be taken if temporary bypass piping fails or cannot be completed as planned within the allocated times.
5. Emergency contact phone numbers for the Contractor or Subcontractor responsible for the temporary bypass piping installation.
6. Assistance required of the Owner's operating personnel during shutdowns.

Results of field pressure test of temporary piping, submitted prior to temporary bypass operation.

7-16.2 Materials

Contractor shall submit proposed flexible or hard temporary piping, valves, and fittings for bypass pumping operations.

7-16.3 Construction Requirements

7-16.3(1) Temporary Piping Routing

Route temporary piping to avoid blocking construction equipment, driveway, and roadway access. Provide protection for piping and couplings where crossing access points is unavoidable.

7-16.3(2) Testing

Pressure test temporary piping to a pressure no less than 150 psi prior to placing into operation and submit test results to Engineer.

END OF DIVISION 7

Division 9
Materials

9-30 WATER DISTRIBUTION MATERIALS

9-30.1(7) Restrained Joint PVC Pipe (4 Inches and over) (New Section)

Restrained Joint PVC pipe shall be manufactured in accordance with AWWA C-900-07 (latest edition) PC 235 DR 18. The pipe shall be manufactured from PVC resin meeting cell class 12454. The pipe joint system shall meet ASTM D 3139 and the gasket be made of SBR rubber meeting ASTM F477 and be suitable for wastewater application.

The PVC pipe restraint system shall be installed in the pipe belling process and is manufactured from Ductile Iron and supplied with a corrosion resistant coating.

9-30.2 Fittings

9-30.2(6) Restrained Joints

Supplement section 9-30.2(6) with the following:

Where restrained mechanical joints are shown, such restraint shall use "Megalugs" for restraint, or approved equal. "Megalugs" shall be EBAA Iron 1000 series for ductile iron pipe and 2000 series for PVC pipe and shall be provided in quantities as may be required.

Restrained joints for pipe shall be designed for a water working pressure of 150 psi.

Restrained joint for pipe shall be capable of being deflected after assembly.

END OF DIVISION 9

APPENDIX A
Geotechnical Report

Geotechnical Engineering Services

Prairie Creek Drainage Improvement Project
Arlington, Washington

for
Murray, Smith & Associates, Inc.

December 3, 2012

GeoEngineers
Earth Science + Technology

Geotechnical Engineering Services

Prairie Creek Drainage Improvement Project
Arlington, Washington

for
Murray, Smith & Associates, Inc.

December 3, 2012

GeoEngineers

8410 154th Avenue NE
Redmond, Washington 98052
425.861.6000

Geotechnical Engineering Services

Prairie Creek Drainage Improvement Project
Arlington, Washington

File No. 5430-007-00

December 3, 2012

Prepared for:

Murray, Smith& Associates, Inc.
2707 Colby Avenue, Suite 1110
Everett, Washington 98201-3566

Attention: Nathan Hardy

Prepared by:

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NANCY L. TOCHKO
STATE OF WASHINGTON
29804
REGISTERED
PROFESSIONAL ENGINEER

12/3/2012

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Geotechnical Engineer

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Principal

NLT:SDS:nld

Four hard copies submitted

GEOENGINEERS

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APPENDICES

INTRODUCTION

This report presents the results of our geotechnical engineering services for evaluation of the soil and groundwater conditions and provides recommendations for the design and construction of the proposed five new culvert crossings for the Prairie Creek Drainage Improvement project located in Arlington, Washington. The creek crossings will be at Haskens Road, the BNSF Railroad corridor, 204th Street NE just east of BNSF, 71st Avenue NE at about the 202nd Block NE and at 74th Avenue NE near 201st Street NE. The site is shown relative to surrounding physical features on the Vicinity Map (Figure 1) the Site Plan Location Map (Figure 2) and the Site Plans (Figures 3, 4, and 5).

PROJECT DESCRIPTION

GeoEngineers understanding of the project is based on discussions with Nathan Hardy and Jenna Thelen with Murray, Smith & Associates, Inc. We understand that the City of Arlington proposes drainage improvements along Prairie Creek from north of Haskens Road to south of 74th Avenue NE. This report only focuses on the planned replacement of the existing five culverts situated at Haskens Road, under the BNSF Railroad corridor immediately east of Haskens Road, 204th Street NE about 150 feet east of BNSF, 71st Avenue NE at about the 202nd Block NE and at 74th Avenue NE near 201st Street NE.

This report was prepared in support of the preliminary design phase. We understand that agencies are requesting that the new culverts have a width of 16 feet for fish passage considerations. At this time, most of the existing culverts are planned to be replaced with box culverts with a typical width of 16 to 19 feet and lengths ranging from 68 to 130 feet. The crossing of 204th Street NE will have to be accomplished in two phases to maintain traffic on 204th Street NE. The bottom of the new culverts will be about 2 to 3 feet below the existing stream bed grade, and will require excavations on the order of about 10 to 18 feet in depth. New abutment walls may be necessary for some of the new culverts.

The crossing under the BNSF right of way is currently planned to be accomplished using trenchless technologies. However, other options for the Haskens Road and BNSF crossings are being considered including skewing the Haskens Road culvert to the north and moving the BNSF culvert north of the existing culvert, or possibly realigning the creek to the south side of 204th Street NE.

SCOPE OF SERVICES

The purpose of this study is to complete subsurface explorations at the project site and to provide geotechnical engineering conclusions and recommendations for the design and construction of the proposed improvements. GeoEngineers' geotechnical engineering services were completed in general accordance with our task order agreement executed on August 8, 2012. Our specific scope of services for this phase of the project includes the following tasks:

1. Review geologic maps and subsurface information in our files for the site vicinity.

2. Explore subsurface soil and groundwater conditions by completing five geotechnical borings. The original scope was for six borings but we were unable to complete one of the borings due to utility and right-of-way restrictions.
3. Complete laboratory testing on selected soil samples obtained from the explorations.
4. Classify the soils encountered in the explorations and evaluate pertinent engineering and physical characteristics.
5. Provide recommendations for temporary excavations, including geotechnical considerations for allowable temporary cut slopes, temporary shoring and dewatering.
6. Provide recommendations for the design of the new culverts including foundation support and lateral soil pressures. Comment on any anticipated construction difficulties identified from the results of our site studies and from our experience on projects at similar sites.
7. Assess seismic hazards at the site, including ground liquefaction and lateral spreading potential. If liquefaction is a concern, recommendations will be presented for methods to mitigate the effects of liquefaction.
8. Discuss trenchless pipeline installation issues and alternatives (pipe jacking and boring, microtunneling, HDD, etc.). Preliminary considerations for trenchless construction will be provided.
9. Provide recommendations for earthwork and site preparation including suitability of on-site soils for reuse in trench backfill, placement and compaction of trench backfill, and mitigation of unsuitable soil conditions. This will include an evaluation of the effects of weather and/or construction equipment on site soils.
10. Comment on any anticipated construction difficulties identified from the results of site studies and from experience on projects at similar sites.
11. Discuss geotechnical considerations related to groundwater conditions including anticipated seasonal fluctuations.
12. Address City of Arlington sensitive areas ordinance issues as they pertain to geotechnical and geological considerations.
13. Present our findings and recommendations in a written report with supporting site plan, boring logs, and other applicable figures.

FIELD EXPLORATIONS AND LABORATORY TESTING

Field Explorations

Subsurface soil and groundwater conditions at the site were evaluated by completing five geotechnical borings (B-1 through B-5). The borings were completed to depths ranging from 4 to 24 feet below the existing ground surface. The approximate locations of these borings are shown on the Site Plans, Figures 3, 4 and 5. Details of the field exploration program and logs of the explorations are presented in Appendix A.

Laboratory Testing

Soil samples were obtained during drilling and taken to our laboratory for further evaluation. Selected samples were tested for the determination of moisture content, fines content, and particle size distribution. A description of the laboratory testing and the test results are presented in Appendix B.

SITE DESCRIPTION

Site Geology

The site is located along the eastern side of the Stillaguamish River Valley. According to the Geologic Map of the Arlington West Quadrangle (Minard 1985), the project site is located on an outcropping of Recessional Outwash, specifically the Marysville Sand Member. The Arlington Gravel Member is present north of the project site.

The recessional outwash was deposited by meltwater flowing south from the stagnating and receding Vashon Glacier. The Marysville Sand Member is characterized by well draining stratified outwash sand, with some gravel and areas of silt and clay. This deposit is up to 65 feet thick and is generally underlain by glacial till. The Arlington gravel member consists of well drained stratified sand and gravel deposits; oxidation and iron oxide cementation are common in this unit. The deposit may be up to 85 feet thick, and is also generally underlain by glacial till.

Surface Conditions

As described previously, the focus of this report is on the five culvert crossings at Haskens Road, under the BNSF Railroad corridor immediately east of Haskens Road, 204th Street NE about 150 feet east of BNSF, 71st Avenue NE at about the 202nd Block NE and at 74th Avenue NE near 201st Street NE. The project alignment is shown on the attached Site Plans, Figures 2, 3, 4 and 5.

Haskens Road north of 204th Street NE has a commercial development north and west of the road and creek, and a ditch vegetated with blackberry bushes between the east end of the culvert and the railroad embankment. East of the railroad embankment, the creek drainage is bordered by 204th Street NE to the south and a level undeveloped parcel to the north. The undeveloped parcel has some trees and shrubs near the creek, and is vegetated with blackberry bushes and grass across the remainder of the parcel. The railroad has a spur line veering to the northeast. It does not appear that this spur line is in active use based on the vegetation present in the railroad bed. For the culvert crossing under 204th Street NE, the undeveloped parcel is present to the north, and south of 204th Street NE the creek is in a channel bordered by commercial developments to the east and a gravel access driveway to the west. The creek channel in this area is about 6 to 8 feet below the surrounding areas.

The creek channel and culvert under 71st Avenue NE cut across commercial developments with landscaping present along the road. The creek channel is vegetated with tall grass and some shrubs and trees. A low levee is present along the channel east of 71st Avenue NE with the creek channel about 3 to 5 feet below the road and levee. Where the creek crosses under 74th Avenue NE, commercial developments with landscaping are present on the west side of the street. Undeveloped property vegetated with tall grass, brush, blackberry bushes and trees is present on the east side of 74th Avenue NE in the vicinity of the creek. The creek is about 3 to 4 feet below the road and surrounding area. The road, culvert and creek elevations are as follows:

Area	Culvert Type and Size	Elevation[1] (feet)		
		Roadway	Top of Culvert	Creek Water Level[2]
Haskens Road	60-inch CMP	122 to 124	114 to 115	113 to 114
BNSF RR	60-inch Steel	124	114 to 116	114 to 115
204th Street NE	60-inch CMP	121	116 to 117	114
71st Avenue NE	60-inch CMP	119	118	117
74th Avenue NE	48-inch CMP	127	124-125	124

Notes:

1 Elevations are approximate and based off of a survey provided by Murray, Smith & Associates.

2 Creek water levels as measured by the surveyors late August 2012.

Existing utilities within or near the project areas include overhead power, business signs and communication lines and buried gas, fiber optic, storm sewer, sanitary sewer, and water.

City of Arlington Critical Areas

As part of our services for the project, we reviewed available City of Arlington information regarding critical areas for the site and surrounding area. Based on information provided by the City, we note the following:

- The liquefaction susceptibility is mapped as low to moderate by the Washington State Department of Natural Resources.
- Very small steep slope hazard areas are mapped along the south side of the creek channel south of 204th Street NE and west of Haskens Road.
- A 150-foot stream buffer is present along the Prairie Creek alignment.

SUBSURFACE EXPLORATIONS

Soil Conditions

We evaluated the subsurface conditions at the site by drilling five geotechnical borings (B-1C through B-5) to depths ranging from 4 to 24 feet below the ground surface (bgs). A detailed description of our field exploration procedures and logs of the explorations are presented in Appendix A.

With the exception of the boring B-1 completed on Haskens Road, the remaining borings were completed in unpaved areas. Borings B-1, B-1B and B-1C encountered 3 inches of asphaltic concrete underlain by 8 to 10 inches of crushed rock base course. Boring 1 was attempted at three locations as the first two attempts (borings B-1 and B-1B on the west side of the road) encountered refusal on rocks or rubble at a depth of 4 to 5.25 feet, respectively. Boring B-1C was able to penetrate this rubble/boulder zone on the east side of the road. We understand from a

City of Arlington employee that fill with large boulders was placed in this area during development of the road and surrounding area.

The subsurface conditions encountered in the remaining borings generally include 3 to 6 inches of grass/sod and root zone underlain by fill. The fill typically consists of medium dense silty sand with varying amounts of gravel. As discussed above, large boulders and possibly rubble is present in the upper portion of the fill underlying Haskens Road. The thickness of the fill layer ranges from about 6 to 10 feet and is likely associated with previous development and grading activities adjacent to the creek channel.

Recessional outwash deposits and possibly alluvial deposits from the creek or the Stillaguamish River were encountered below the fill and extended to the maximum depth explored in each boring. These deposits generally consist of medium dense to very dense sand and gravel with varying amounts of silt and cobble content. Boring B-2 encountered a soft and loose alluvial layer of silt and silty sand from a depth of about 11 to 15 feet. It should be noted that the sampler used in collecting soil samples from each boring has an internal diameter of about 1.5 inches. Therefore, the size of the sampler restricts the size of gravel we are able to sample. In most borings, the driller noted the presence of gravels by the drilling action. Notations are present on the boring logs where rough drilling occurred or where gravels were felt by the driller. Therefore, although the soil descriptions in the boring logs are mainly sands, we believe that layers of gravels are also present within these recessional outwash deposits. Large cobbles and boulders were not encountered in the borings we completed with the exception of B-1; however, large cobbles and boulders are known to occur in recessional outwash soils.

Groundwater Conditions

Groundwater was observed in some of the borings. Groundwater was observed at a depth of about 19 feet in boring B-3, at a depth of about 9 feet in boring B-4, and at a depth of about 23 feet in boring B-5. Static groundwater was not observed in borings B-1 and B-2 at the time of exploration. Groundwater observations represent conditions observed during drilling and may not represent the groundwater conditions throughout the year. Groundwater conditions are expected to fluctuate as a result of season, precipitation and other factors.

CONCLUSIONS AND RECOMMENDATIONS

General

Based on the results of our subsurface exploration program and our geotechnical evaluation, it is our opinion that the proposed drainage improvements (i.e. culvert replacements) can be successfully completed from a geotechnical perspective provided the considerations presented in this report are incorporated in the project planning and design. The key geotechnical issues for the project are summarized below:

- The culverts may be supported on shallow foundations bearing on the medium dense to dense recessional sand and gravel deposits. We anticipate that the bottom of the culvert crossing under the BNSF tracks will likely be below the level of soft alluvial deposits encountered in boring B-2. We recommend an allowable soil bearing pressure of 4,000 pounds per square foot (psf) be used for footings supported by native medium dense soil or structural fill placed over native medium dense to dense soil.

- We anticipate that pipe ramming will be the preferred trenchless technique to construct a new culvert under the BNSF tracks. However, pipe ramming would require installing two 8-footdiameter culverts as 8 feet is the maximum size for this method. Pregrouting of the embankment fill soils above the culvert would likely be required by BNSF.
- We did not complete a specific scour analyses for this project. The proposed culvert foundations should be placed deep enough to protect them from potential scour impacts.
- Shoring will likely be used to complete the excavations for the remaining culverts to minimize the impacts to the adjacent roadways.
- Difficulties in excavating and installing shoring will likely be encountered for the culvert crossing Haskens Road due to the presence of rubble and/or large boulders present in the upper 5 feet of the fill.
- Provided the creek water is diverted from the excavations, we anticipate that dewatering can typically be accomplished by open pumping using sump pumps. A higher level of pumping/dewatering may be required for the culvert excavation crossing 74th Avenue NE, where groundwater was encountered at a depth of about 9 feet during drilling and where relatively free-draining sand is present.
- We anticipate that the soils at the site can be excavated using conventional construction equipment. However, the contractor should be prepared to deal with cobbles and boulders in the outwash soils, and with rubble or large boulders in the vicinity of Haskens Road. Ideally, earthwork should be undertaken during extended periods of dry weather when the soils will be less susceptible to disturbance and provide better support for construction equipment. Dry weather construction will help reduce earthwork costs.
- Effective erosion and sedimentation control must be implemented during construction so that potential impacts to the adjacent sensitive areas are reduced. The erosion potential of the on-site soils is moderate to high. The erosion and sedimentation control measures used for this project should be in accordance with applicable regulatory standards.

The following sections of this report present our conclusions and recommendations for site development, foundation support and performance estimates for the associated site developments.

Site Preparation and Earthwork

General

We recommend that site preparation and earthwork be completed during the normally dry season of the year (generally July through September) if practical, as the workability of the soil becomes difficult and the erosion potential of the on-site soils is increased during extended periods of wet weather.

Earthwork Considerations

Asphalt, grass/sod, fill, and outwash deposits were observed in the explorations. In addition, excavations will require removal of adjacent concrete sidewalks. We anticipate that these materials can be excavated with conventional excavation equipment, such as trackhoes or dozers. Cobbles were not encountered in these explorations, with the possible exception of borings B-1 and B-1B, but were encountered in borings completed for a nearby project and boulders are known to occur in glacial outwash deposits. Therefore, the contractor should be prepared to deal with cobbles and boulders in the fill and outwash.

Clearing and Grubbing

The work area should be cleared of all surface and subsurface deleterious matter, including debris, trees, shrubs and associated stumps and root wads, and should be stripped of the sod and organic soil. The woody debris should be removed from the project site for disposal. Based on our experience, we anticipate that stripping depths will generally be less than 8 inches. The stripped vegetative material and organic soil can be stockpiled and later reused in landscaping if desirable.

Removal and demolition of existing structures should include removal of below-grade elements. Existing voids or new depressions created during site preparation should be cleaned of loose soil or debris and backfilled with structural fill.

Creek Diversion

The creek should be diverted into a tight line going around each excavation such that creek water does not enter each excavation.

Sedimentation and Erosion Control

In our opinion, the erosion potential of the undisturbed on-site soils is low to moderate as most of the adjacent areas are relatively flat and landscaped or well vegetated.

The amount and potential impacts of erosion are in part a function of the time of year construction occurs. Wet weather construction will increase the amount and extent of erosion. We expect that exposed soils will have moderate erosion potential during wet weather. It will therefore be necessary to put in place effective erosion controls during and after construction. These should include proper control of surface water runoff to prevent uncontrolled, concentrated surface water runoff over slope areas and minimizing the time of exposure in the areas stripped during construction through prompt re-vegetation.

Effective erosion and sedimentation control during construction may consist of interceptor swales and silt fences to prevent water from flowing off site. Because the runoff is likely to be silty, we recommend that the collected water be passed through a temporary desilting facility prior to discharging the water into the stormwater collection system. Completion of initial clearing and grading activities during the drier months and limiting the disturbance of the existing ground surface and vegetation where possible will also reduce the risks of erosion. Material stockpiles should be covered during wet weather to prevent erosion and soil loss. All areas disturbed during construction should be seeded and planted as soon as practical to reduce the potential for erosion.

Erosion and sedimentation control measures should be installed and maintained in accordance with applicable regulatory standards.

Subgrade Preparation

We recommend that all subgrade soils for each culvert foundation be evaluated by a representative of GeoEngineers before construction of the foundation or structural fill to identify any soft or unsuitable subgrade soils. Any soft or unsuitable subgrade soils that are observed during this evaluation should be removed and replaced with compacted structural fill. Where subgrade soils have high fines content, construction during the wet season can result in significant disturbance. In areas where high fines content are observed in the subgrade soils, we recommend 2 to 4 inches of crushed rock be placed on the prepared foundation subgrade to protect it and avoid softening the silty subgrade soils during wet weather construction. Haul roads and laydown areas should also include crushed rock surfacing to protect them during wet weather construction.

Structural Fill

All fill, whether on-site or imported soil, that will support pavement areas or foundations, or in utility trenches should meet the criteria for structural fill presented below. The suitability of soil for use as structural fill depends on its gradation and moisture content.

MATERIALS

Structural fill material quality varies depending upon its use, as described below:

1. Structural fill placed to construct embankments, to backfill utility trenches, to support culvert or wall foundations and to provide subgrade for pavement should consist of common borrow as described in Section 9-03.14(3) of the 2012 Washington State Department of Transportation (WSDOT) Standard Specifications. If structural fill is placed during wet weather, it should consist of gravel borrow as described in Section 9-03.14(1) of the 2012 WSDOT Standard Specifications.
2. Structural fill placed adjacent to below-grade and retaining walls (drainage zone) should consist of gravel backfill for walls in conformance with Section 9-03.12(2) of the 2012 WSDOT Standard Specifications.
3. Structural fill placed as crushed surfacing base course below pavements should conform to Section 9-03.9(3) of the 2012 WSDOT Standard Specifications.

USE OF ON-SITE SOILS

Most of the near-surface soils observed in the explorations generally contain a high percentage of fines (silt/clay) and are moisture-sensitive. Some of the existing fill that meet the requirements for common borrow may be suitable for use as common borrow during dry weather, provided it can be properly moisture-conditioned prior to placement. These soils will likely be limited to the sand, sand with silt, and gravel encountered in the borings.

The on-site soils that meet the requirements for common borrow are expected to be suitable for structural fill in areas requiring compaction to at least 95 percent of the maximum dry density (MDD) estimated in general accordance with American Society for Testing and Materials (ASTM) D 1557, provided the work is completed during the normally dry season (June through September) and that the soil can be properly moisture-conditioned. It may be necessary to import sand and gravel with a low fines content to achieve adequate compaction for support of pavement areas for wet weather construction. Imported structural fill consisting of sand and gravel (WSDOT gravel borrow) should be planned if construction occurs during wet weather.

The use of on-site soils that meet the requirements for common borrow as structural fill during wet weather should be planned only for areas requiring compaction to 90 percent of the MDD or less, as long as the soils are properly protected from wet weather and not placed during periods of precipitation. The contractor should plan to cover and maintain all fill stockpiles with plastic sheeting if the soil will be used as structural fill. The reuse of on-site soils is highly dependent on the skill of the contractor and the schedule, and we will work with the design team and contractor to maximize the reuse of on-site soils during the wet and dry seasons.

STRUCTURAL FILL PLACEMENT

Structural fill should generally be placed in loose lifts not exceeding about 8 to 10 inches in thickness. Each lift should be conditioned to the proper moisture content and compacted to the specified density before placing subsequent lifts. If structure fill is placed adjacent to existing slopes, the existing slope should be benched prior to the fill placement and compaction to avoid an unstable interface zone. Structural fill placed in areas used to support footings or retaining walls should be compacted to at least 95 percent of MDD as determined by the ASTM D-1557 test method. Pavement area fill, including utility trench backfill, should be compacted to at least 90 percent of MDD, except for the upper 2 feet below finished subgrade surface, which should be compacted to at least 95 percent of MDD. Structural fill to support sidewalks should be placed after the subgrade is evaluated and be compacted to at least 90 percent of MDD.

We recommend that a representative from GeoEngineers, Inc. be present during structural fill placement to observe the work and perform in-place density tests to evaluate whether or not the specified compaction is being achieved.

Culvert Excavations

General

Culvert excavation depths will range from about 10 to 20 feet, with most of the culvert replacements requiring excavations on the order of about 10 feet deep, with the exception of the two culverts under Haskens Road and the BNSF tracks. We anticipate that medium dense to dense recessional deposits will be exposed in the base of these excavations. All temporary cut slopes and shoring must comply with the provisions of Title 296 Washington Administrative Code (WAC), Part N, "Excavation, Trenching and Shoring." The contractor performing the work has the primary responsibility for the protection of workers and adjacent improvements.

Because the soils at the project consist mostly of sand and gravel with variable amounts of silt, we recommend that all excavations extending below groundwater depth be fully dewatered. Otherwise, excessive groundwater flow into excavations could cause lateral movement of the granular soils into the excavations, possibly destabilizing the excavations or causing excessive ground settlement adjacent to the excavations. Dewatering is discussed further below in the "Construction Dewatering" section of this report.

Temporary and Permanent Slopes

We anticipate that shored excavations will be required for most of the culvert replacements. However, where sloped excavations are possible, we recommend that temporary cut slopes be inclined no steeper than 1½H:1V (horizontal to vertical). Flatter slopes may be necessary if seepage is present on the face of the cut slopes or if localized sloughing occurs.

Some sloughing and raveling of the cut slopes should be expected. Temporary covering, such as heavy plastic sheeting, should be used to protect these slopes during periods of rainfall. Surface water runoff from above cut slopes should be prevented from flowing over the slope face by using curbs, berms, drainage ditches, swales or other appropriate methods.

If temporary cut slopes experience excessive sloughing or raveling during construction, it may become necessary to modify the cut slopes to maintain safe working conditions and protect adjacent facilities or structures. Slopes experiencing excessive sloughing or raveling can be flattened, they can be regraded to add intermediate slope benches, or additional dewatering can be provided if the poor slope performance is related to groundwater seepage.

Because the contractor has control of the construction operations, the contractor should be made responsible for the stability of cut slopes, as well as the safety of the excavations. Shoring and temporary slopes must conform to applicable local, state and federal safety regulations. The final configuration for temporary excavation slopes should be evaluated during construction, as it is a function of the soil and groundwater conditions encountered and the contractor's approach to excavation.

Permanent creek banks should be inclined no steeper than 3H:1V. Permanent slopes should be planted or hydroseeded as soon as practicable after grading. We recommend that all fill be placed as structural fill, as described above.

Shored Excavations

We anticipate that the excavations for the culvert replacements, plus the jacking and recovery pit for trenchless construction, will be completed using shored excavations to minimize the limits of the excavations. The jacking pit might be completed using temporary cut slopes or using shoring, depending on the existing ground surface slope in the vicinity of the receiving pit and right-of-way limitations.

Because of the diversity of available shoring systems and construction techniques, the design of temporary shoring is most appropriately left up to the contractor proposing to complete the installation. However, we recommend that the shoring be designed by an engineer licensed in Washington, and that the PE-stamped shoring plans and calculations be submitted to the City of Arlington and the Engineer for review prior to construction. The following paragraphs present general recommendations for the type of shoring system and design parameters that we conclude are appropriate for the subsurface conditions at the project.

The site soils can be retained using conventional trench shoring systems such as trench boxes, slide rail system, or sheet piles with lateral restraint. The design of temporary shoring should allow for lateral pressures exerted by the adjacent soil, and for surcharge loads resulting from structures, traffic, construction equipment, temporary stockpiles adjacent to the excavation, etc. Lateral load resistance can be mobilized through the use of braces, tiebacks, anchor blocks and passive pressures on members that extend below the bottom of the excavation. Temporary shoring used to support trench excavations typically uses internal bracing such as hydraulic shoring or trench boxes.

The lateral soil pressures acting on shoring walls will depend on the nature and density of the soil behind the wall and the inclination of the backfill surface. For walls that are free to yield at the top at least one thousandth of the height of the wall (i.e., wall height times 0.001), soil pressures will be less than if movement is restrained. We recommend that yielding walls retaining medium dense to dense fill and native soils be designed using an equivalent fluid density of 35 and 60 pounds per cubic foot (pcf), for horizontal ground surfaces and ground surfaces inclined at 1½ H:1V above the horizontal, respectively. For non-yielding (i.e., braced) systems, we recommend that the shoring be designed for a uniform lateral pressure of 26*H in psf, where H is the depth of the planned excavation in feet below a level ground surface. Similarly, for a ground surface inclined at 1½H:1V above partial shoring, we recommend that shoring be designed for a uniform lateral pressure of 46*H.

These lateral soil pressures do not include traffic, structure or construction surcharges that should be added separately, if appropriate. Shoring should be designed for a traffic influence equal to a uniform lateral pressure of 100 psf acting over the depth of the trench. If shoring is within 15 feet of active BNSF tracks, additional surcharge loading may be necessary. More conservative pressure values should be used if the designer deems them appropriate. These soil pressure recommendations are predicated upon the construction being essentially dewatered; if effective dewatering methods are used to lower the groundwater level below the bottom of the excavation, hydrostatic pressures need not be added to the soil pressures within the exposed height of shoring.

If portions of the shoring use passive elements such as anchor or reaction blocks, available soil resistance can be estimated using passive soil pressures assuming an equivalent fluid density of 350 pcf above the water table and 200 pcf below the water table.

Dewatering

The purpose of this report section is to present geotechnical and hydrogeological data that will influence temporary construction dewatering and to describe in general terms various types of dewatering techniques that may be feasible at the site. Detailed dewatering designs for construction are not within our scope of services.

As discussed above, static groundwater was observed in some of the borings at the time of exploration. Where observed, the depth to static groundwater varied from about 9 to 23 feet. We recommend the groundwater level be maintained a minimum of 2 feet below the bottom of the excavation during construction or that level necessary to stabilize the shoring. The level will depend upon the dewatering method, the size of the excavation and other factors.

Based on the soil conditions and our experience in the area, we expect that groundwater in excavations less than about 2 to 4 feet below the static groundwater level can be controlled by open pumping using sump pumps. For excavations deeper than 4 feet below the water table, or where free-draining sands and gravels are present at the base of the excavation, dewatering using well points or deep wells might be necessary. We recommend that the contractor be required to submit a proposed dewatering system design and plan layout to the City of Arlington and the Engineer for review and comment prior to beginning construction.

The level of effort required for dewatering will depend to a great extent on the time of year during which construction is accomplished and the extent to which all of the creek flow is successfully diverted around each excavation. Less seepage into the work areas should be expected if construction is accomplished in the late summer or early fall months, and correspondingly, more seepage should be expected during the wetter periods of the year. We recommend that construction be completed in the late summer or early fall months when the creek flows are typically at their lowest. In our opinion, this will result in significant cost savings for the dewatering.

A general discussion of the dewatering methods anticipated for the project is presented below.

Open Pumping

This dewatering method involves removing water that has seeped into the excavation by pumping from a sump that has been excavated at one end of the excavation or trench. Drainage ditches that are connected to the sump are typically excavated along the sidewalls at the base of the excavation or trench. The excavation for the sump and the drainage ditches should be backfilled with gravel or crushed rock to reduce the amount of erosion and associated sediment in the water pumped from the sump. In our experience, a slotted casing or perforated 55-gallon drum that is installed in the sump backfill provides a suitable housing for a submersible pump.

The amount of water removed from the excavation by open pumping should be minimized because of high turbidity levels. Temporary storage of dewatering effluent from the sumps in a settlement tank or basin may be required to meet discharge permit requirements and reduce sediment content prior to discharging the water to surface water courses.

Pumped Wells

Individually pumped wells may be considered for dewatering the construction areas. Pumped wells that have been properly installed and developed are capable of producing the high discharge rates that are necessary to dewater highly permeable sand deposits. Pumped wells are generally the most effective dewatering method in areas where dewatering to deeper than about 20 feet below the ground surface is necessary.

We recommend that all dewatering wells installed for this project be properly developed to remove fine sediment from the immediate vicinity of the well screens. Proper development is essential for producing efficient wells and greatly reduces the turbidity of the water discharged from the well. Filter packs consisting of graded sand, or sand and fine gravel should be installed around the well screens in areas where the aquifer contains a high percentage of fine sand and silt.

Well Points

Well points are effective for dewatering all types of soils, whether pumping small amounts of water from silt or large quantities of water from coarse sand and gravel. The volume of water generated by a well point system is typically less than the volume generated by a corresponding system of pumped wells because the well points are generally completed at a shallower depth. Because of the shallower completion depth, the volume of aquifer that contributes water to a well point system is less than for a comparable deep well system.

Well point systems are most suitable for dewatering shallow excavations where the water table must be lowered no more than about 20 feet bgs. Multiple well point stages are generally required beyond that depth because of the physical limitations of suction lift. Dewatering can be accomplished at depths greater than 20 feet where the excavation can be open cut to permit installation of the well point system below original grade. This technique increases the depth to which the water table can be lowered with well points.

Earthquake Engineering

Design Earthquake Parameters

The seismic design of the proposed improvements can be completed using the design criteria presented in the American Association of State Highway and Transportation Officials (AASHTO) seismic design information. The AASHTO Guide Specifications recommend a 7 percent probability of exceedance in 75 years (nominal 1,000-year earthquake) design event for development of a design spectrum. Based on these criteria, we recommend the parameters for site class, seismic zone, acceleration coefficient and spectral acceleration coefficients presented in the following table.

AASHTO SEISMIC PARAMETERS

AASHTO Seismic Parameter	Recommended Value
Site Class	D
Seismic Zone for $0.30 < S_{D1} \leq 0.50$	3
Effective Peak Ground Acceleration Coefficient $A_S = F_{pga}PGA = (1.17)(0.333)$	0.39
Design Spectral Acceleration Coefficient at 0.2 Second period $S_{DS} = F_aS_s = (1.20)(0.753)$	0.904
Design Spectral Acceleration Coefficient at 1.0 Second period $S_{D1} = F_vS_1 = (1.89)(0.255)$	0.482

Seismic Hazards

We evaluated the site conditions for seismic hazards including liquefaction, lateral spreading and seismically induced landsliding. Our evaluation indicates the site has low risk of liquefaction because of the relatively low groundwater and presence of medium dense to very dense outwash deposits below the site. Because there is a low risk of liquefaction, the site has a low risk of liquefaction-induced ground disturbance including lateral spreading. Our evaluation of seismically induced landsliding indicates that there is also a low risk for seismically induced landsliding.

Shallow Foundations

General

Based on soils observed in our explorations located near the proposed culverts, we anticipate that medium dense or denser sand and gravel soils (recessional outwash) will be present at the anticipated foundation grades, assumed to be about 12 to 17 feet below existing grades, with the exception of the 71st Street culvert which may be slightly higher. Overexcavation and replacement with structural fill may be necessary in the vicinity of the crossing under the BNSF tracks, as Boring B-2 encountered a zone of loose silty sand to soft silt from a depth of about 12 to 15 feet. We recommend that the proposed culverts be supported on conventional spread footings bearing on the native medium dense to dense sand and gravel soils observed in the borings at and below the anticipated base of the new culverts, or on properly placed and compacted structural fill that extends down to the competent native soils.

Foundation Design Parameters

Footings may be designed using an allowable soil bearing value of 4 kips per square foot (ksf) on properly compacted structural fill or native medium dense or denser sand and gravel soils. The allowable soil bearing values apply to the total of dead and long-term live loads and may be increased by up to one-third for wind or seismic loads. Footings should be at least 2-foot-wide, and should be founded a minimum of 2 feet below the level of the creek channel bottom.

Settlement

Provided all loose soil is removed and the subgrade is prepared as recommended under "Construction Considerations" below, we estimate the total settlement of shallow foundations will be on the order of ½ to 1 inch. The settlements will occur rapidly, essentially as loads are applied. Differential settlement between the bridge abutments is expected to be less than 1 inch.

Lateral Resistance

Lateral foundation loads may be resisted by passive resistance on the sides of footings and by friction on the base of the footings. For footings supported on native soils or on structural fill placed and compacted in accordance with our recommendations, the allowable frictional resistance may be computed using a coefficient of friction of 0.4 applied to vertical dead-load forces.

The allowable passive resistance of soils may be computed using an equivalent fluid density of 200 pcf (triangular distribution) if these elements are poured directly against undisturbed native soils or surrounded by structural fill. This value assumes that the hydrostatic groundwater level may at times be as high as the culvert footings. No passive resistance should be allowed for soils located on the creek-side of the culvert within 2 feet of the creek bed. The above coefficient of friction and passive equivalent fluid density values incorporate a factor of safety of about 1.5.

Construction Considerations

Subgrade disturbance may occur if footing excavations are completed during wet weather. A working mat of lean concrete or compacted crushed rock should be placed over the footing subgrade immediately following excavation to prevent softening and disturbance of the footing subgrade if construction occurs during wet weather.

If soft areas are present at the footing subgrade elevation, the soft areas should be removed and replaced with structural fill at the direction of the Geotechnical Engineer. In such instances, the zone of structural fill should extend laterally beyond the footing edges a horizontal distance at least equal to the thickness of the fill.

Given the relatively high allowable bearing pressures presented above, the condition of all footing excavations must be observed by the Geotechnical Engineer or their representative to evaluate if the work is completed in accordance with our recommendations and that the subsurface conditions are as anticipated.

Retaining/Abutment Walls

General

At this time, we do not know if the proposed culverts will require abutment walls. Abutment walls could consist of conventional concrete cantilever walls or possibly a block/keystone type of wall. bearing on shallow foundations. The following paragraphs present our recommendations for retaining walls.

For retaining walls with horizontal backslopes that are allowed to deflect about 0.002H under loading, we recommend that the walls be designed for the active earth pressure taken as an equivalent fluid density of 35 pcf for well-draining gravel backfill for walls. If the ground within 5 feet of the retaining wall rises at an inclination of 2H:1V or steeper, the retaining wall should be designed using an equivalent fluid density of 50 pcf. For adjacent slopes flatter than 2H:1V, soil pressures can be interpolated between this range of values. Other conditions should be evaluated on a case-by-case basis.

If the retaining walls are restrained against rotation, we recommend that the walls with horizontal backslopes be designed for an at-rest earth pressure taken as an equivalent fluid density of 55 pcf. If the ground within 5 feet of the retaining wall rises at an inclination of 2H:1V or steeper, the rigid retaining wall should be designed using an equivalent fluid density of 78 pcf. For adjacent slopes flatter than 2H:1V, soil pressures can be interpolated between this range of values. Rigid walls are walls that deflect less than about 0.002H under the at-rest pressure loading, where H is the height of the retaining wall. Once the wall moves approximately 0.002H, the active pressure state is achieved.

Typically, retaining walls are designed for a surcharge pressure for traffic loading. For traffic loading, we recommend that retaining walls be designed for a uniform surcharge pressure determined by increasing the height of the wall by 2 feet. Other surcharge loads should be included as appropriate.

If seismic earth pressure are considered in design we recommend that a rectangular seismic earth pressure distribution equal to 8H in psf be added to the static lateral earth pressures presented above for the rigid wall or active earth pressure condition, whichever is appropriate.

Drainage

The above lateral earth pressures assume that the backfill behind the retaining walls is drained. Drainage consisting of either a perforated drain pipe installed near the base of the retaining walls or installation of weepholes near the base of the retaining wall should be incorporated in the design. If a drain pipe is used, the drains should consist of a perforated pipe a minimum of 4 inches in diameter enveloped within a minimum thickness of 6 inches of gravel backfill for drains, WSDOT Standard Specification 9-03.12(4). Clean-outs for the collector pipe should be installed as appropriate. Alternatively, the walls can be provided with weepholes designed in accordance with WSDOT Standard Plans.

Construction Considerations

Backfill placed within 5 feet of below grade walls should be compacted to densities of at least 90 percent of the MDD obtained in accordance with the ASTM D-1557 procedure to reduce the potential for development of excess pressure on the walls. If sidewalks or pavement will be placed adjacent to the wall, we recommend that the upper 2 feet of fill be compacted to 95 percent of the MDD. Measures should be taken to prevent the buildup of excess lateral soil pressures due to over-compaction of the backfill behind the wall; for example, by using hand-operated mechanical vibrators.

Drainage Considerations

General

We recommend that all surfaces be sloped to drain away from the existing and proposed structures and improvements. Pavement surfaces and open space areas should be sloped such that the surface water is collected and routed to suitable discharge points.

We anticipate that shallow groundwater seepage may enter excavations depending on the time of year construction takes place, especially in the winter months. However, we expect that this seepage water can be handled by digging interceptor trenches in the excavations and pumping from sumps. If not intercepted and removed from the excavations, the seepage water will make it difficult to place and compact structural fill and may destabilize cut slopes.

Trenchless Crossing Considerations

General

We understand that the culvert replacement under the BNSF tracks will likely be completed using trenchless technology. Difficulties in completing trenchless in this area include the close proximity to Haskens Road which may limit the area available for the jacking or receiving pits, and the presence of rubble or large boulders in the fill underlying Haskens Road which precludes installing a longer continuous culvert using trenchless techniques under both Haskens road and the BNSF tracks.

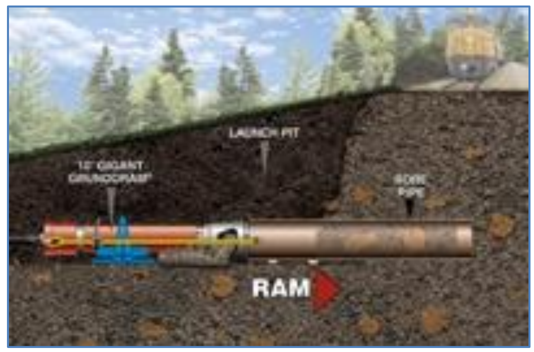
In general, we anticipate that recessional sands and gravels will be encountered along most of the alignment. Some softer silt and loose sand may also be encountered. No groundwater was observed in borings B-1C or B-2, however, based on the groundwater conditions encountered in other borings, groundwater may be present along the trenchless alignment.

Large rocks and/or rubble were encountered in the upper 5 feet of fill along the west side of Haskens Road. If the trenchless portion extends under Haskens Road, the presence of large rocks or rubble could present extreme difficulties for completing the trenchless bore. Therefore, at this time, we recommend that the trenchless section extend only under the railroad tracks and not under Haskens Road.

Due to the desired large width of the new culvert, it is our opinion that the trenchless techniques best suited for this project include pipe ramming, box culvert jacking, and tunneling using arch canopy methods. Based on our experience with completing trenchless projects under railroad embankments, we anticipate that the railroad will require the City to pregrout the embankment soils prior to initiating the trenchless project. The following sections provide a general discussion of possible trenchless techniques for this project.

Twin Pipe Ramming

Pipe ramming is a trenchless construction method that uses a large pneumatic ramming tool to drive steel pipe through a variety of soil types (see the graphic to the right). A cutting shoe can be welded to the front of the lead pipe to provide a small over cut and help reduce friction along the pipe, and to cut through the soil. Bentonite or polymer lubrication can also be used to help reduce friction during long ramming drives. The soil inside the pipe remains in place during ramming and is removed after the pipe reaches the far side of the embankment. With pipe ramming, the maximum possible size is about 8 feet in diameter.



Pipe Ramming (graphic courtesy of TT Technologies)

Therefore, if a spring line diameter of 16 feet is required, a twin barrel approach would be necessary using this method. The combined width at the spring line of the two pipes would be about 16 feet.

For railroad and embankment crossings, the steel pipe is often driven in one continuous section. Once the full length of the pipe casing has been installed, the spoils inside the casing are removed. Depending on the length of the installation, spoil from inside the casing can be removed with compressed air, jetting or augering.

One of the benefits of pipe ramming is that the spoils from inside the casing are not removed during installation (like auger boring). This results in formation of a soil plug within the casing. In soils which may have perched layers of groundwater, or in soils that are a few feet below the groundwater table, this soil plug can reduce or stop the inflow of groundwater. Additionally, the soil plug in the casing eliminates the possibility of over excavation at the face of the casing.

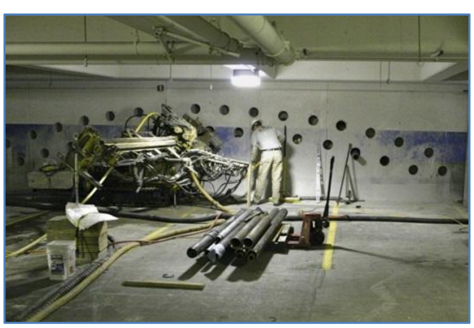
Pipe ramming is the preferred method by many railroads as no soil is removed until the casing is completely installed, lessening the risk of piping or disturbance of the railroad embankment fill soils. Pipe ramming also has the benefit of not requiring a receiving pit and the ramming pit could be placed on the east side of the railroad where there is adequate room.

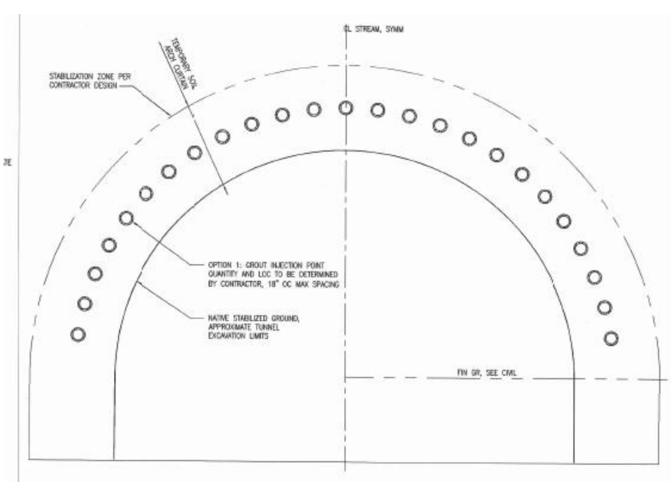
Box Culvert Jacking

Jacking a box culvert uses the same approach as jacking a pipe. A steering shield is placed in front of the first section of reinforced box culvert. Large rams are placed behind the box culvert to provide thrust. Lubrication is provided along the top and sides of the box culvert to reduce friction. As the first section of box culvert is pushed into place, the soil from the interior of the culvert is excavated. The length of box culvert that can be pushed depends on the weight of the culvert, the soils and the available sizes of rams. **This technology is not common in the PNW and to our knowledge no local contractors have rammed large box culverts.** Challenges include preventing the culvert from sinking below the desired alignment due to the weight of the box. Also, a special order box culvert would be required to have rails built in on the bottom of the culvert.

Conventional Tunneling Using Arch Canopy Methods

Conventional tunneling is typically not allowed by railroads, however, installation of a pre-grouted (or pre-frozen) arch would construct a canopy of improved ground (also known as "barrel vaulting") by drilling horizontal bores from either side of the embankment and might be allowed by the railroad. This is a method commonly used in large diameter tunneling applications. For either grout or freeze pipes, the canopy usually has a single or a double row of freeze tubes or perforated grout tubes placed in 8-inch-diameter bores, with bores spaced at 12 to 18 inches on center. For a grout canopy, the grout is pumped through the steel grout tubes and is directed to discrete areas along the tube by using packers. The result is closely spaced and possibly overlapping grouted beams with the grout tube left in place as reinforcing. For both a grout-arch and a frozen-ground arch, the beams act as support for the overlying soil mass and as soil improvement to reduce over-excavation of the tunnel. Once the arch is in place, the tunnel is excavated and a final liner is installed.





LIMITATIONS

We have prepared this report for the exclusive use by Murray, Smith & Associates, and their authorized agents for the geotechnical elements of the proposed Prairie Creek Drainage Improvements project to be located in Arlington, Washington.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering in this area at the time this report was prepared. No warranty or other conditions, expressed or implied, should be understood.

Any electronic form, facsimile or hard copy of the original document (email, text, table, and/or figure), if provided, and any attachments are only a copy of the original document. The original document is stored by GeoEngineers, Inc. and will serve as the official document of record.

Please refer to the attachment titled Report Limitations and Guidelines for Use for additional information pertaining to use of this report.

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FIGURES

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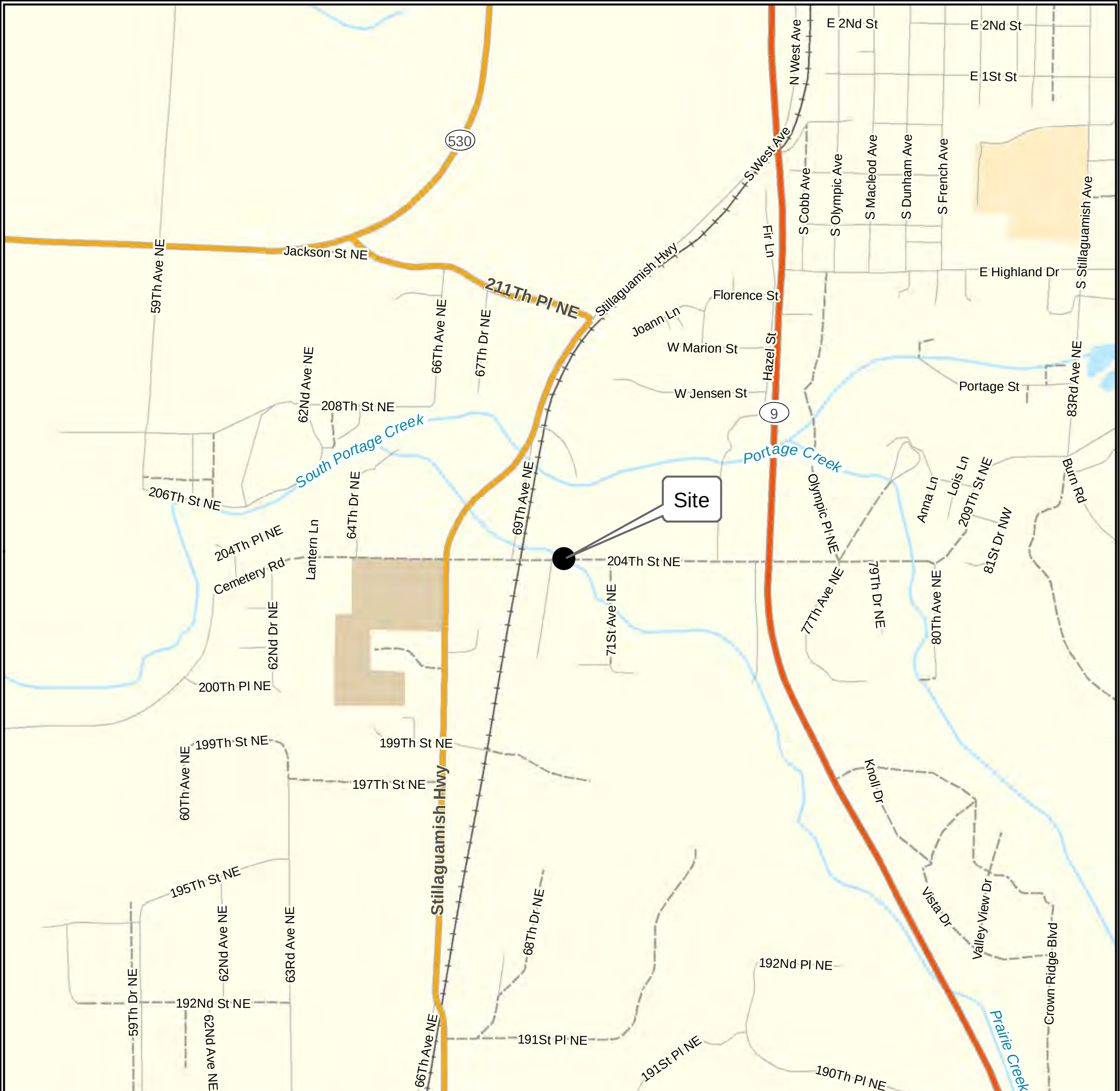

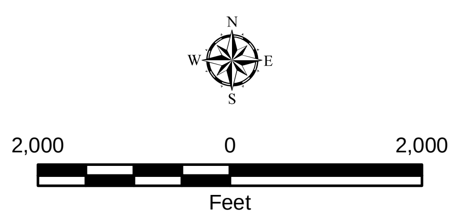

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Data Sources: ESRI Data & Maps, Street Maps 2005

Transverse Mercator, Zone 10 N North, North American Datum 1983
North arrow oriented to grid north

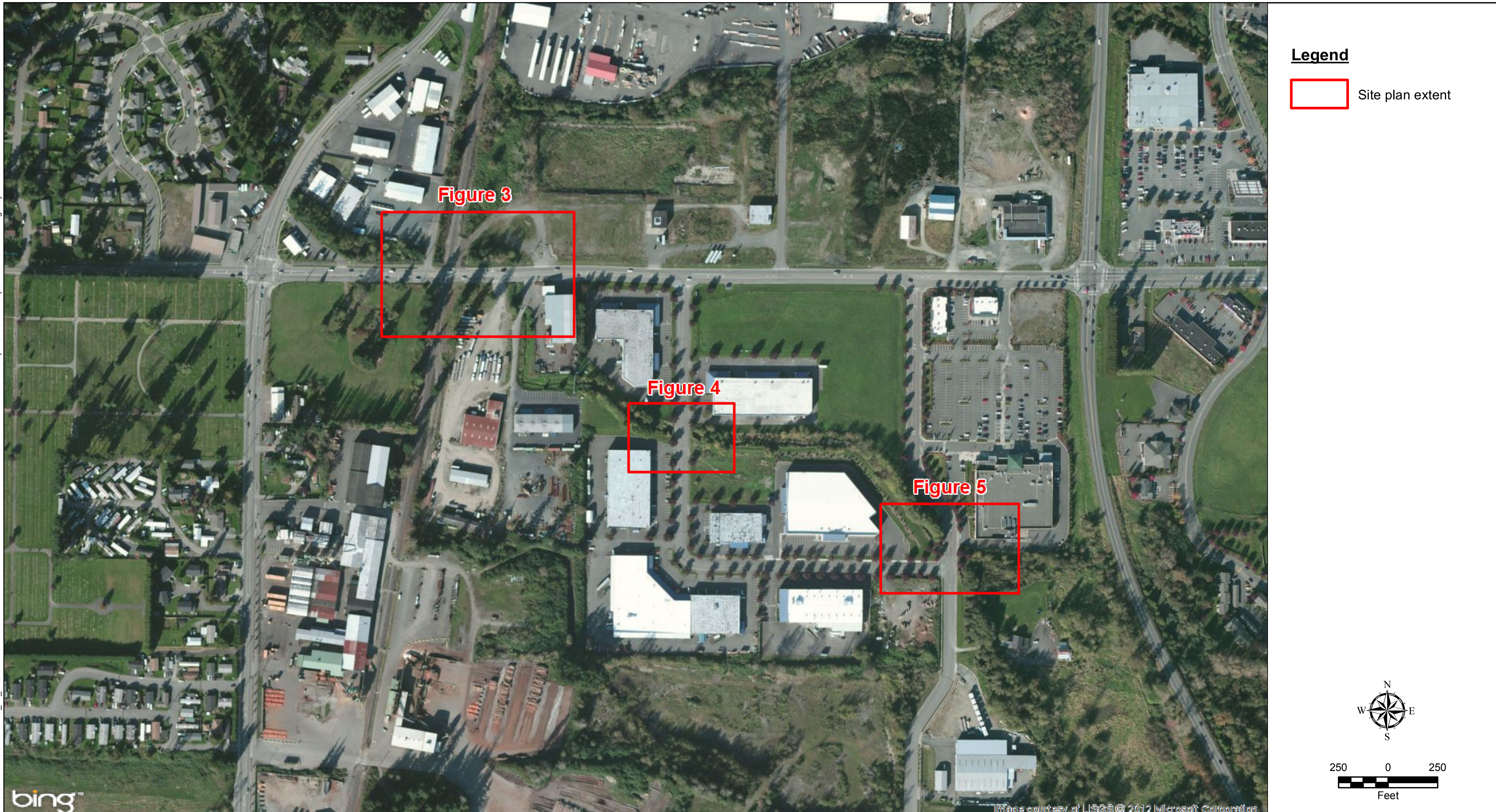
Vicinity Map

City of Arlington
Prairie Creek Drainage Improvements
Arlington, Washington

GeoEngineers

Figure 1

Office: Redmond Path: \\red\projects\5\5430007\GIS\543000700_F1_VicinityMap.mxd Map Revised: September 04, 2011 EL



Data Source:

Projection: WGS 1984 Web Mercator Auxiliary Sphere

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Site Plan Locations

Prairie Creek Drainage Improvements
Arlington, WA

GEOENGINEERS

Figure 2

Map Revised: 11 September 2012 bmagdasy

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Office: SEA

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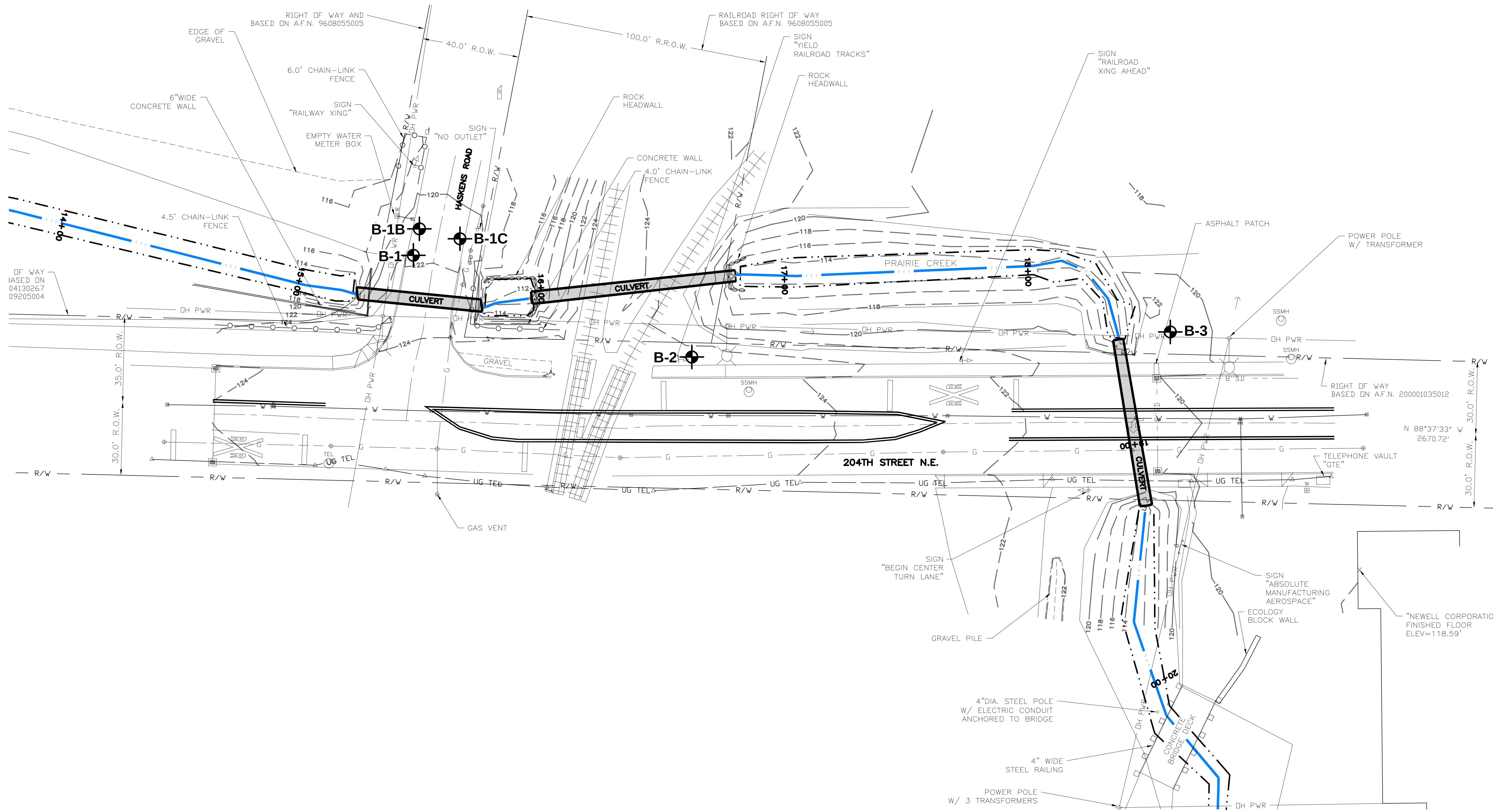

Notes

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Reference: Base survey by Metron and Associates, Inc. dated August 2012.

Legend

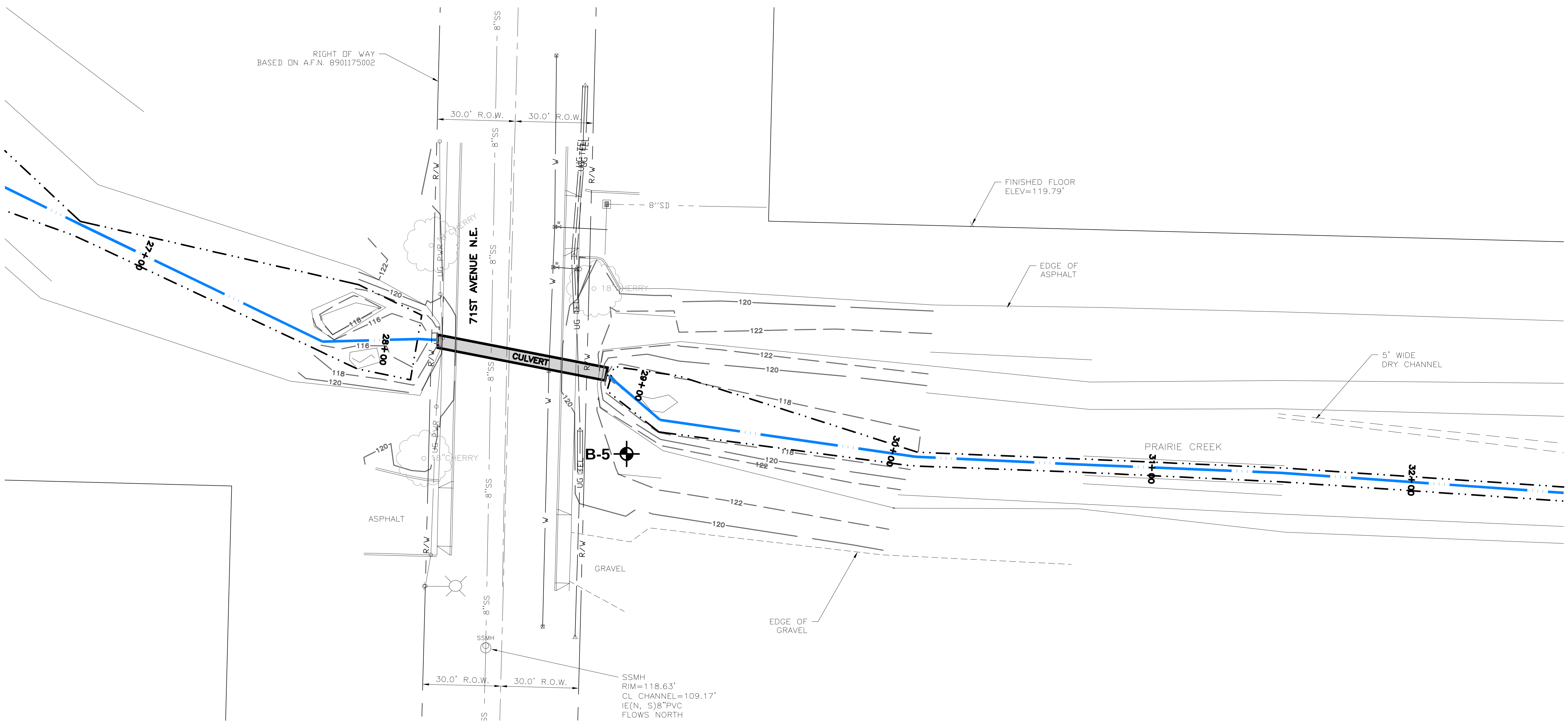
B-1 ⌖ Boring by GeoEngineers

Site Plan

Prairie Creek Drainage Improvement Project
Arlington, Washington

GeoEngineers

Figure 3



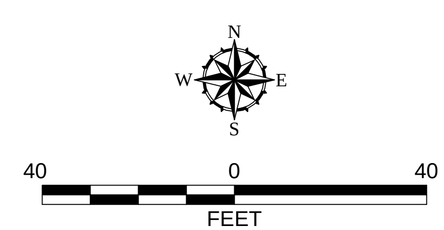
Notes

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Reference: Base survey by Metron and Associates, Inc. dated August 2012.

Legend

B-5 Boring by GeoEngineers



Site Plan

Prairie Creek Drainage Improvement Project
Arlington, Washington

GEOENGINEERS

Figure 4

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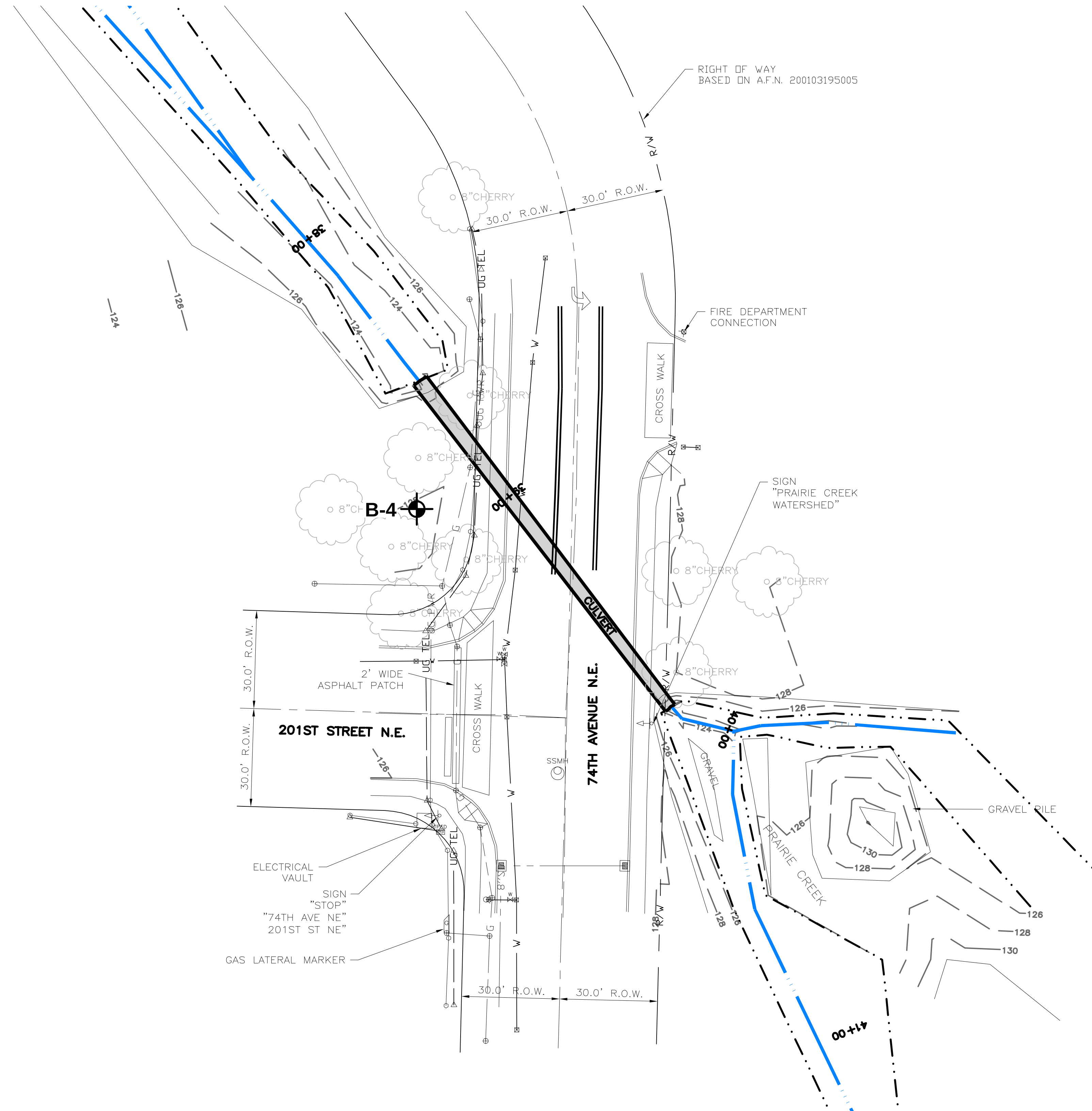

Notes

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Reference: Base survey by Metron and Associates, Inc. dated August 2012.

Legend

B-4 ⌖ Boring by GeoEngineers

N W E S

40 0 40

FEET

Site Plan

Prairie Creek Drainage Improvement Project
Arlington, Washington

GEOENGINEERS

Figure 5

APPENDICES

Appendix A
Field Explorations

APPENDIX A
FIELD EXPLORATIONS

We explored subsurface conditions at the site of the proposed culvert locations by completing five borings (B-1C through B-5). Boring 1 was attempted at three locations as the first two attempts (borings B-1 and B-1B on the west side of the road) encountered refusal on rocks or rubble at a depth of 4 to 5.25 feet. The drilling was performed by Geologic Drill on August 29, 2012.

The locations of the explorations were estimated in the field by measuring distances from site features through taping/pacing in the field. The approximate exploration locations are shown on the Site Plans, Figures 3, 4, and 5. Boring elevations were estimated based on a survey drawing provided by Murray, Smith & Associates, Inc. dated August 2012.

The borings were completed using trailer-mounted, continuous-flight, hollow-stem auger drilling equipment. The borings were continuously monitored by a representative from our firm who examined and classified the soils encountered, obtained representative soil samples, observed groundwater conditions and prepared a detailed log of each exploration.

The soils encountered in the borings were generally sampled at 2½- or 5-foot vertical intervals with a 2-inch-outside diameter split-barrel standard penetration test (SPT) sampler. The samples were obtained by driving the sampler 18 inches into the soil with an automatic 140-pound hammer falling approximately 30 inches. The number of blows required for each 6 inches of penetration was recorded. The blow count ("N-value") of the soil was calculated as the number of blows required for the final 12 inches of penetration. This resistance, or N-value, provides a measure of the relative density of granular soils and the relative consistency of cohesive soils. Where very dense soil conditions precluded driving the full 18 inches, the penetration resistance for the partial penetration was entered on the logs. The blow counts are shown on the boring logs at the respective sample depths.

Soils encountered in the borings were visually classified in general accordance with the classification system described in Figure A-1. A key to the boring log symbols is also presented in Figure A-1. The logs of the borings are presented in Figures A-2 through A-8. The boring logs are based on our interpretation of the field and laboratory data and indicate the various types of soils and groundwater conditions encountered. The logs also indicate the depths at which these soils or their characteristics change, although the change may actually be gradual. If the change occurred between samples, it was interpreted. The densities noted on the boring logs are based on the blow count data obtained in the borings and judgment based on the conditions encountered.

Observations of groundwater conditions were made during drilling. The groundwater conditions encountered during drilling are presented on the boring logs. Groundwater conditions observed during drilling represent a short-term condition and may or may not be representative of the long-term groundwater conditions at the site. Groundwater conditions observed during drilling should be considered approximate.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% RETAINED ON NO. 200 SIEVE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS
				SP	POORLY-GRADED SANDS, GRAVELLY SAND
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% PASSING NO. 200 SIEVE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS, ROCK FLOUR, CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS AND SILTS OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: Multiple symbols are used to indicate borderline or dual soil classifications

ADDITIONAL MATERIAL SYMBOLS

SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
	AC	Asphalt Concrete
	CC	Cement Concrete
	CR	Crushed Rock/ Quarry Spalls
	TS	Topsoil/ Forest Duff/Sod

Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Groundwater observed at time of exploration

Perched water observed at time of exploration

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata or geologic units

Approximate location of soil strata change within a geologic soil unit

Material Description Contact

Distinct contact between soil strata or geologic units

Approximate location of soil strata change within a geologic soil unit

Sampler Symbol Descriptions

2.4-inch I.D. split barrel

Standard Penetration Test (SPT)

Shelby tube

Piston

Direct-Push

Bulk or grab

Blowcount is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted). See exploration log for hammer weight and drop.

A "P" indicates sampler pushed using the weight of the drill rig.

Laboratory / Field Tests

%F	Percent fines
AL	Atterberg limits
CA	Chemical analysis
CP	Laboratory compaction test
CS	Consolidation test
DS	Direct shear
HA	Hydrometer analysis
MC	Moisture content
MD	Moisture content and dry density
OC	Organic content
PM	Permeability or hydraulic conductivity
PP	Pocket penetrometer
PPM	Parts per million
SA	Sieve analysis
TX	Triaxial compression
UC	Unconfined compression
VS	Vane shear

Sheen Classification

NS	No Visible Sheen
SS	Slight Sheen
MS	Moderate Sheen
HS	Heavy Sheen
NT	Not Tested

NOTE: The reader must refer to the discussion in the report text and the logs of explorations for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the specific exploration locations and at the time the explorations were made; they are not warranted to be representative of subsurface conditions at other locations or times.

KEY TO EXPLORATION LOGS

GeoEngineers

FIGURE A-1

Drilled	Start 8/29/2012	End 8/29/2012	Total Depth (ft) 4	Logged By JQS / Checked By	Driller Geologic	Drilling Method

Surface Elevation (ft) 121.0 / Vertical Datum (Approximate)	Hammer Data Auto 140 (lbs) / 30 (in) Drop	Drilling Equipment Deep Rock XL
Easting (X) / Northing (Y)	System / Datum	Groundwater: Date Measured / Depth to Water (ft) / Elevation (ft)

Notes: Auger Data: 3¼-inch I.D.

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
	0								AC	3 inches asphalt concrete			
120									CR	8 to 10 inches base course, 5/8-inch minus crushed rock			
									SM	Brown silty fine to medium sand with gravel and rubble or boulders (dense, dry to moist) (fill)			
			8	50/6"		1							Slight hydrocarbon odor

Refusal at 4 feet on rock?

Note: Please see Figure A-1 for explanation of symbols

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Log of Boring B-1

GeoEngineers

Project: City of Arlington
Project Location: Arlington, Washington
Project Number: 05430-007-00

Figure A-2
Sheet 1 of 1

Drilled	Start 8/29/2012	End 8/29/2012	Total Depth (ft)	5.25	Logged By JQS / Checked By	Driller Geologic	Drilling Method

Surface Elevation (ft) / Vertical Datum	121.0 (Approximate)	Hammer Data	Auto 140 (lbs) / 30 (in) Drop
Drilling Equipment	Deep Rock XL		
Easting (X) / Northing (Y)		System / Datum	
Groundwater: Date Measured	Depth to Water (ft)	Elevation (ft)	

Notes: Auger Data: 3¼-inch I.D.

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
	0								AC	3 inches asphalt concrete			
120									CR	8 to 10 inches base course, 5/8-inch minus crushed rock			Boring 5 feet north of B-1
									SM	Brown silty fine to medium sand with occasional gravel and rubble or boulders (medium dense, moist) (fill)			
			8	50/5"		1							Slight hydrocarbon odor
	5												

Refusal at 5.25 feet on rock?

Note: Please see Figure A-1 for explanation of symbols

Log of Boring B-1B

GeoEngineers

Project: City of Arlington

Project Location: Arlington, Washington

Project Number: 05430-007-00

Figure A-3

Sheet 1 of 1

Drilled	Start 8/29/2012	End 8/29/2012	Total Depth (ft) 24	Logged By JQS Checked By
Driller	Geologic	Drilling Method		
Surface Elevation (ft) 121.0 Vertical Datum (Approximate)	Hammer Data Auto 140 (lbs) / 30 (in) Drop	Drilling Equipment Deep Rock XL		
Easting (X) Northing (Y)	System Datum	Groundwater	Date Measured / Depth to Water (ft) / Elevation (ft)	

Notes: Auger Data: 3¼-inch I.D.

FIELD DATA

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
	0								AC	3 inches asphalt concrete			
120									CR	8 to 10 inches base course, 5/8-inch minus crushed rock			
									SM	Brown silty fine to medium sand with occasional gravel and rubble or boulders (medium dense to dense, moist) (fill)			
			12	50/6"		1							
115	5												
			12	25		2							
									SP-SM	Brown fine to medium sand with silt and gravel (dense, moist)			
110	10												
			10	61		3					4		%F = 8.6
105	15												
									SP-SM	Brown fine to medium sand with silt and occasional gravel (loose, moist) (recessional outwash)			
			16	33		4					17		SA; %F = 7
100	20												
			14	84		5							

Note: Please see Figure A-1 for explanation of symbols

Log of Boring B-1C

GeoEngineers

Project: City of Arlington
Project Location: Arlington, Washington
Project Number: 05430-007-00

Drilled	Start 8/29/2012	End 8/29/2012	Total Depth (ft) 24	Logged By JQS Checked By	Driller Geologic	Drilling Method

Surface Elevation (ft) Vertical Datum	124.0 (Approximate)	Hammer Data	Auto 140 (lbs) / 30 (in) Drop
Drilling Equipment	Deep Rock XL		
Easting (X) Northing (Y)		System Datum	
Groundwater	Date Measured	Depth to Water (ft)	Elevation (ft)

Notes: Auger Data: 3¼-inch I.D.

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
124	0								SM	Brown silty fine to medium sand with occasional gravel, organic matter and cobbles (medium dense, moist) (fill)			Drilling action indicates large gravels or cobbles
			12	27		1							
120	5												Rough drilling
			10	30		2			SW	Brown fine to coarse sand with trace silt and occasional gravel (medium dense, moist)	6		Rough drilling %F = 35
115	10												
			10	4		3			ML/SM	Dark brown silt with sand to silty fine to medium sand with occasional gravel (soft/loose, wet)	28		
110	15								SP-SM	Brown fine to coarse sand with silt and gravel (medium dense to dense, moist) (recessional outwash)			Driller notes larger gravels
			13	43		4					6		SA; %F = 10
105	20												
									SM	Brown silty fine to medium sand (medium dense, moist)			
100			5	30		5							Little recovery, with gravel?

Note: Please see Figure A-1 for explanation of symbols

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Log of Boring B-2

GeoEngineers

Project: City of Arlington
Project Location: Arlington, Washington
Project Number: 05430-007-00

Figure A-5
Sheet 1 of 1

Drilled	Start 8/29/2012	End 8/29/2012	Total Depth (ft)	24	Logged By JQS Checked By	Driller Geologic	Drilling Method

Surface Elevation (ft) Vertical Datum	121.0 (Approximate)	Hammer Data	Auto 140 (lbs) / 30 (in) Drop	Drilling Equipment	Deep Rock XL
Easting (X) Northing (Y)		System Datum			

Notes: Auger Data: 3¼-inch I.D.

Groundwater Date Measured	Depth to Water (ft)	Elevation (ft)
8/29/2012	19.0	102.0

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
121–119	0–2								SM	Brown silty fine to medium sand with occasional gravel (loose, moist) (fill?)			
	2–7		12	13		1			SP-SM	Brown silty fine to medium sand with gravel (loose to medium dense, moist) (fill?)			Driller notes large gravels at approximately 2 to 2.5 feet
	7–12		14	50/5"		2			SM	Brown silty fine to medium sand with gravel (medium dense to dense, moist) (fill?)	3.3		Driller notes gravels %F = 9
	12–17		1	50		3			SM	Dark brown silty fine to medium sand (medium dense to dense, moist)			Little recovery
	17–21		16	39		4	∇ (19.0)		SM	Brown silty fine to medium sand with trace gravel (medium dense to dense, moist) (recessional outwash)	5.4		SA; %F = 13
	21–24		18	54		5			SP	Gray-brown fine to medium sand with trace silt, occasional gravel and occasional lenses of silt (dense, moist to wet)			

Note: Please see Figure A-1 for explanation of symbols

Log of Boring B-3

GeoEngineers

Project: City of Arlington
Project Location: Arlington, Washington
Project Number: 05430-007-00

Figure A-6
Sheet 1 of 1

Drilled	Start 8/29/2012	End 8/29/2012	Total Depth (ft) 24	Logged By JQS Checked By	Driller Geologic	Drilling Method

Surface Elevation (ft) 128.0 Vertical Datum (Approximate)	Hammer Data Auto 140 (lbs) / 30 (in) Drop	Drilling Equipment	Deep Rock XL
Easting (X) Northing (Y)	System Datum	Groundwater	

Date Measured	Depth to Water (ft)	Elevation (ft)
8/29/2012	9.0	119.0

Notes: Auger Data: 3¼-inch I.D.

FIELD DATA

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
	0								SOD	3 inches sod			
									SM	Light brown silty fine to medium sand with gravel, cobbles and occasional organics (loose, moist) (fill)			
125			5	18		1				Grades to darker brown			
	5								SP-SM	Brown fine to medium sand with silt and gravel (medium dense, moist) (recessional outwash)			
120			12	20		2					7.9		Driller notes gravel at 7.5 feet %F = 8
	10						∇						
115			12	27		3			SW-SM	Gray fine to coarse sand with silt and gravel (medium dense, wet)	11		SA; %F = 8
	15												
110			14	51		4			SM	Brown silty fine to coarse sand with gravel (medium dense to dense, wet)			
	20												
105			12	34		5							

Note: Please see Figure A-1 for explanation of symbols

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Log of Boring B-4

GeoEngineers

Project: City of Arlington
Project Location: Arlington, Washington
Project Number: 05430-007-00

Drilled	Start 8/29/2012	End 8/29/2012	Total Depth (ft)	24	Logged By JQS / Checked By	Driller Geologic	Drilling Method

Surface Elevation (ft) / Vertical Datum	122.0 (Approximate)	Hammer Data	Auto 140 (lbs) / 30 (in) Drop
Drilling Equipment	Deep Rock XL		
Easting (X) / Northing (Y)		System / Datum	

Notes: Auger Data: 3¼-inch I.D.

Groundwater Date Measured	Depth to Water (ft)	Elevation (ft)
8/29/2012	22.5	99.5

FIELD DATA

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Water Level	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
	0					SOD	4 inches sod			
120		4	17	1		SM	Brown silty fine to medium sand with gravel and trace organics (medium dense, dry to moist) (fill)			
115	5	10	19	2		SP	Gray-brown fine to medium sand with trace silt and occasional gravel (medium dense, moist) (recessional outwash)	5.5		%F = 4
110	10	10	35	3		SW-SM	Grayish brown fine to coarse sand with silt and gravel to sandy gravel with silt (medium dense, moist)	6.5		SA; %F = 7
105	15	12	35	4			Grades to brown in color			
100	20	17	15	5	∇	SP-SM	Brown fine to medium sand with silt (medium dense, wet)			

Note: Please see Figure A-1 for explanation of symbols

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Log of Boring B-5

GEOENGINEERS

Project: City of Arlington
Project Location: Arlington, Washington
Project Number: 05430-007-00

APPENDIX B
Laboratory Testing

APPENDIX B
LABORATORY TESTING

General

Soil samples obtained from the explorations were transported to GeoEngineers' laboratory and examined to confirm or modify field classifications, as well as to evaluate index properties of the soil samples. Representative samples were selected for laboratory testing consisting of the determination of the moisture content, percent fines, and grain size distribution. The tests were performed in general accordance with test methods of the American Society for Testing and Materials (ASTM) or other applicable procedures.

The results of the moisture content and percent fines determinations are presented at the respective sample depths on the exploration logs in Appendix A. The sieve analyses test results are presented in Figures B-1 and B-2.

Moisture Content Testing

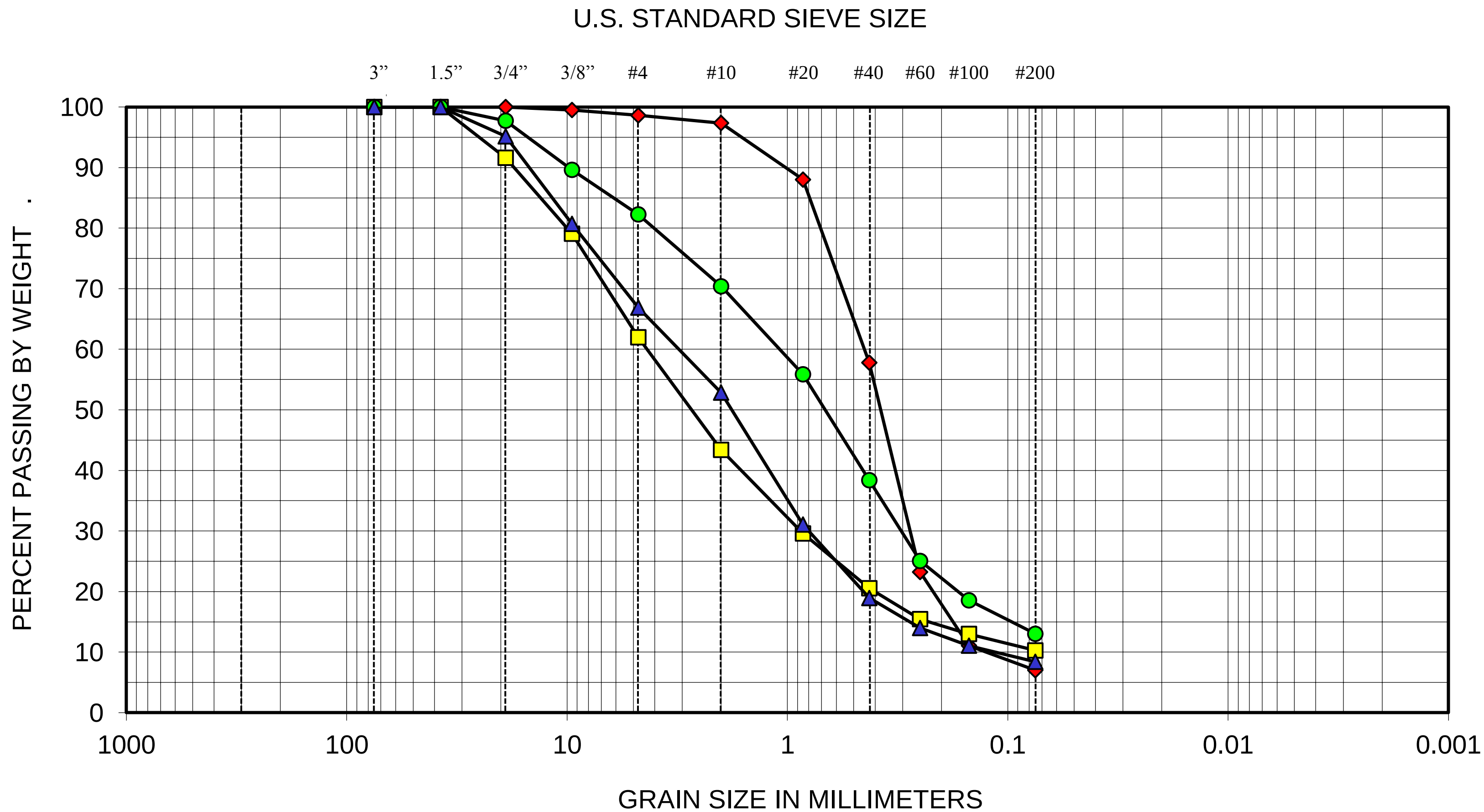
Moisture content tests were completed in general accordance with ASTM D 2216 for representative samples obtained from the explorations. The results of these tests are presented on the exploration logs in Appendix A at the depths at which the samples were obtained.

Percent Passing U.S. No. 200 Sieve

Selected samples were "washed" through the U.S. No. 200 mesh sieve to determine the relative percentages of coarse- and fine-grained particles in the soil. The percent passing value represents the percentage by weight of the sample finer than the U.S. No. 200 sieve. These tests were conducted to verify field descriptions and to determine the fines content for analysis purposes. The tests were conducted in general accordance with ASTM D 1140, and the results are shown on the exploration logs in Appendix A at the respective sample depths.

Sieve Analyses

Sieve analyses were performed on selected samples in general accordance with ASTM D 422 to determine the sample grain size distribution. The wet sieve analysis method was used to determine the percentage of soil greater than the U.S. No. 200 mesh sieve. The results of the sieve analyses were plotted and classified in general accordance with the Unified Soil Classification System (USCS).



<table>
<tr><td rowspan="2">BOULDERS</td><td rowspan="2">COBBLES</td><td colspan="2">GRAVEL</td><td colspan="3">SAND</td><td rowspan="2">SILT OR CLAY</td></tr>
<tr><td>COARSE</td><td>FINE</td><td>COARSE</td><td>MEDIUM</td><td>FINE</td></tr>
</table>

SYMBOL	EXPLORATION NUMBER	DEPTH (ft)	SOIL CLASSIFICATION
◆	B-1c	17½	Brown fine to medium sand with silt (SP-SM)
■	B-2	17½	Brown fine to course sand with silt and gravel (SP-SM)
●	B-3	17½	Brown silty fine to medium sand with gravel (SM)
▲	B-4	12½	Gray fine to course sand with silt and gravel (SW-SM)

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SIEVE ANALYSIS RESULTS

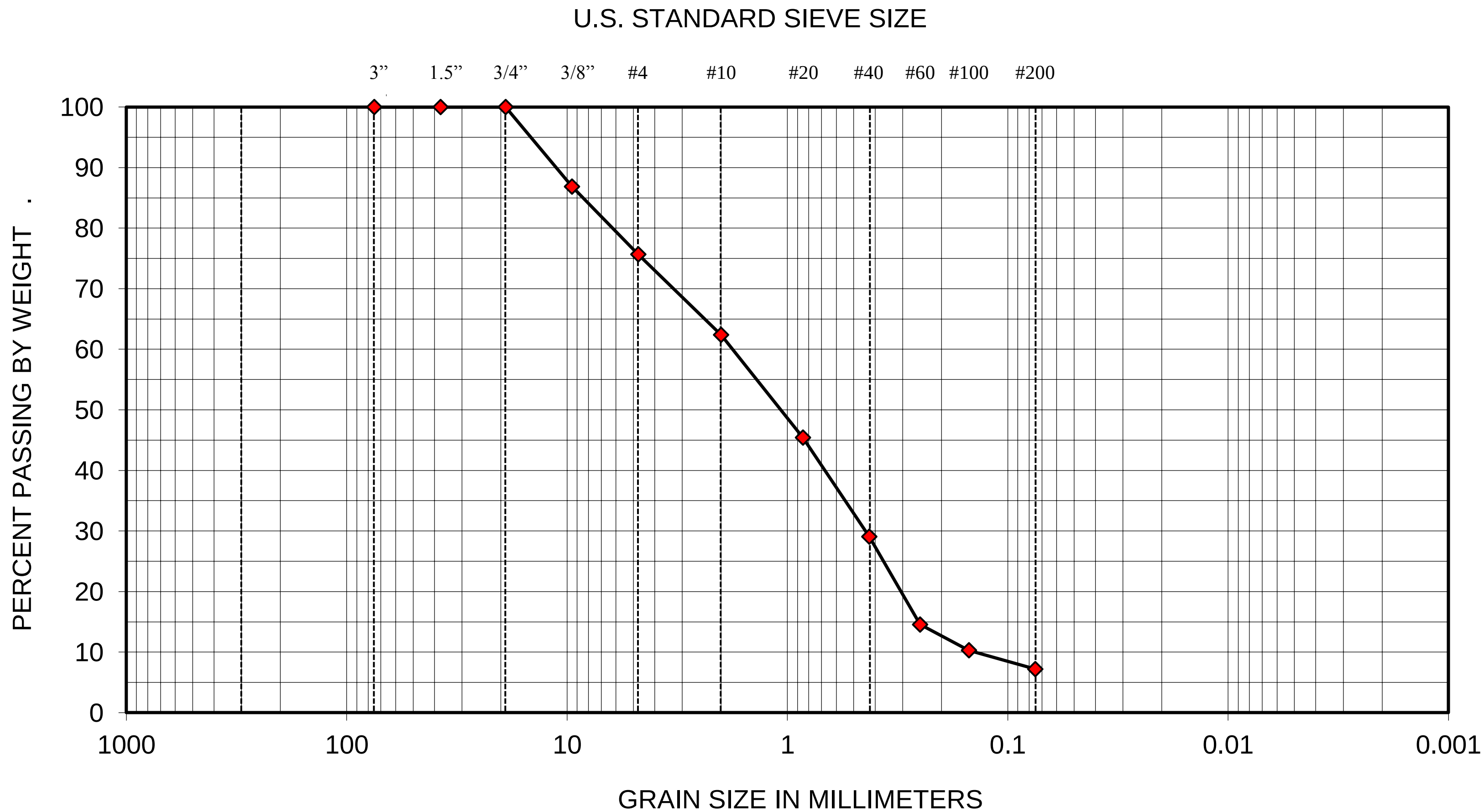
FIGURE B-1

5430-007-00 SAS: SAS 09-06-2012

GEOENGINEERS

FIGURE B-2

SIEVE ANALYSIS RESULTS



BOULDERS	COBBLES	GRAVEL		SAND			SILT OR CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE	

SYMBOL	EXPLORATION NUMBER	DEPTH (ft)	SOIL CLASSIFICATION
◆	B-5	17½	Grayish brown fine to course sand with silt and gravel (SW-SM)

Appendix C
Report Limitations and Guidelines for Use

APPENDIX C
REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

This attachment provides information to help you manage your risks with respect to the use of this report.

Geotechnical Services Are Performed For Specific Purposes, Persons and Projects

This report has been prepared for the exclusive use of The City of Arlington, Murray, Smith & Associates, Inc. and their authorized agents. This report is not intended for use by others, and the information contained herein is not applicable to other sites.

GeoEngineers structures our services to meet the specific needs of our clients. For example, a geotechnical or geologic study conducted for a civil engineer or architect may not fulfill the needs of a construction contractor or even another civil engineer or architect that are involved in the same project. Because each geotechnical or geologic study is unique, each geotechnical engineering or geologic report is unique, prepared solely for the specific client and project site. Our report is prepared for the exclusive use of our Client. No other party may rely on the product of our services unless we agree in advance to such reliance in writing. This is to provide our firm with reasonable protection against open-ended liability claims by third parties with whom there would otherwise be no contractual limits to their actions. Within the limitations of scope, schedule and budget, our services have been executed in accordance with our Agreement with the Client and generally accepted geotechnical practices in this area at the time this report was prepared. This report should not be applied for any purpose or project except the one originally contemplated.

A Geotechnical Engineering Or Geologic Report Is Based On A Unique Set Of Project-Specific Factors

This report has been prepared for the proposed Prairie Creek Drainage Improvement project in Arlington, Washington. GeoEngineers considered a number of unique, project-specific factors when establishing the scope of services for this project and report. Unless GeoEngineers specifically indicates otherwise, do not rely on this report if it was:

- not prepared for you;
- not prepared for your project;
- not prepared for the specific site explored; or
- completed before important project changes were made.

For example, changes that can affect the applicability of this report include those that affect:

- the function of the proposed structure;
- elevation, configuration, location, orientation or weight of the proposed structure;
- composition of the design team; or
- project ownership.

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[1] Developed based on material provided by ASFE, Professional Firms Practicing in the Geosciences; www.asfe.org.

If important changes are made after the date of this report, GeoEngineers should be given the opportunity to review our interpretations and recommendations and provide written modifications or confirmation, as appropriate.

Subsurface Conditions Can Change

This geotechnical or geologic report is based on conditions that existed at the time the study was performed. The findings and conclusions of this report may be affected by the passage of time, by manmade events such as construction on or adjacent to the site, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations. Always contact GeoEngineers before applying a report to determine if it remains applicable.

Most Geotechnical And Geologic Findings Are Professional Opinions

Our interpretations of subsurface conditions are based on field observations from widely spaced sampling locations at the site. Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoEngineers reviewed field and laboratory data and then applied our professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ, sometimes significantly, from those indicated in this report. Our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions.

Geotechnical Engineering Report Recommendations Are Not Final

Do not over-rely on the preliminary construction recommendations included in this report. These recommendations are not final, because they were developed principally from GeoEngineers' professional judgment and opinion. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for this report's recommendations if we do not perform construction observation.

Sufficient monitoring, testing and consultation by GeoEngineers should be provided during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether or not earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective method of managing the risks associated with unanticipated conditions.

A Geotechnical Engineering Or Geologic Report Could Be Subject To Misinterpretation

Misinterpretation of this report by other design team members can result in costly problems. You could lower that risk by having GeoEngineers confer with appropriate members of the design team after submitting the report. Also retain GeoEngineers to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering or geologic report. Reduce that risk by having GeoEngineers participate in pre-bid and preconstruction conferences, and by providing construction observation.

Do Not Redraw The Exploration Logs

Geotechnical engineers and geologists prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, but recognize that separating logs from the report can elevate risk.

Give Contractors A Complete Report And Guidance

Some owners and design professionals believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering or geologic report, but preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with GeoEngineers and/or to conduct additional study to obtain the specific types of information they need or prefer. A pre-bid conference can also be valuable. Be sure contractors have sufficient time to perform additional study. Only then might an owner be in a position to give contractors the best information available, while requiring them to at least share the financial responsibilities stemming from unanticipated conditions. Further, a contingency for unanticipated conditions should be included in your project budget and schedule.

Contractors Are Responsible For Site Safety On Their Own Construction Projects

Our geotechnical recommendations are not intended to direct the contractor’s procedures, methods, schedule or management of the work site. The contractor is solely responsible for job site safety and for managing construction operations to minimize risks to on-site personnel and to adjacent properties.

Read These Provisions Closely

Some clients, design professionals and contractors may not recognize that the geoscience practices (geotechnical engineering or geology) are far less exact than other engineering and natural science disciplines. This lack of understanding can create unrealistic expectations that could lead to disappointments, claims and disputes. GeoEngineers includes these explanatory “limitations” provisions in our reports to help reduce such risks. Please confer with GeoEngineers if you are unclear how these “Report Limitations and Guidelines for Use” apply to your project or site.

Geotechnical, Geologic And Environmental Reports Should Not Be Interchanged

The equipment, techniques and personnel used to perform an environmental study differ significantly from those used to perform a geotechnical or geologic study and vice versa. For that reason, a geotechnical engineering or geologic report does not usually relate any environmental findings, conclusions or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. Similarly, environmental reports are not used to address geotechnical or geologic concerns regarding a specific project.

Biological Pollutants

GeoEngineers' Scope of Work specifically excludes the investigation, detection, prevention or assessment of the presence of Biological Pollutants. Accordingly, this report does not include any interpretations, recommendations, findings, or conclusions regarding the detecting, assessing, preventing or abating of Biological Pollutants and no conclusions or inferences should be drawn regarding Biological Pollutants, as they may relate to this project. The term "Biological Pollutants" includes, but is not limited to, molds, fungi, spores, bacteria, and viruses, and/or any of their byproducts.

If Client desires these specialized services, they should be obtained from a consultant who offers services in this specialized field.

Have we delivered World Class Client Service?

Please let us know by visiting **www. geoengineers.com/feedback**.

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