Geotechnical Engineering Services

Illabot Creek Bridge Relocation
Rockport Cascade Road
Rockport, Washington

for

Skagit River Systems Cooperative

June 30, 2011

GeoEngineers
Earth Science + Technology

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June 30, 2011

Skagit River Systems Cooperative
c/o TranTech Engineering, LLC
626 120th Avenue NE, Suite B 100
Bellevue, Washington 98005

Attention: Khashayar Nikzad, Ph.D., PE

Subject: Geotechnical Engineering Services
Illabot Creek Bridge Relocation
Rockport Cascade Road
Rockport, Washington
File No. 11129-004-00

We are pleased to submit two copies of our report, "Geotechnical Engineering Services, Illabot Creek Bridge Relocation, Rockport Cascade Road, Rockport, Washington." Our geotechnical services were completed in general accordance with our scope of services which was included in the signed agreement for the project. Our services were authorized by Khashayar Nikzad of TranTech Engineering, LLC on May 2, 2011. Preliminary results of our study were discussed with the design team as information became available.

We appreciate the opportunity to work with you on this project. Please call if you have any questions regarding this report.

Sincerely,
GeoEngineers, Inc.

J. Robert Gordon, PE
Principal

AJH:JRG:nlu

Geotechnical Engineering Services

Illabot Creek Bridge Relocation
Rockport Cascade Road
Rockport, Washington

File No. 11129-004-00

June 30, 2011

Prepared for:

Skagit River Systems Cooperative
c/o TranTech Engineering, LLC
626 120th Avenue NE, Suite B 100
Bellevue, Washington 98005

Attention: Khashayar Nikzad, Ph.D., PE

Prepared by:

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GEOENGINEERS

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INTRODUCTION AND SCOPE

This report presents the results of our geotechnical services for the proposed construction of new bridge crossing(s) for Illabot Creek along Rockport Cascade Road east of Rockport, Washington. A vicinity map showing the approximate location is provided in Figure 1.

Bridge scour and hydraulic engineering analyses were completed concurrently by GeoEngineers. The results are presented in a separate report with the results incorporated into the geotechnical considerations of this report.

The existing bridge is constructed as part of a manmade diversion of the original Illabot Creek channel. This project will return the creek to its natural channel approximately 350 to 500 feet west of the existing bridge and channel. An alternative analysis was completed that included one long single span bridge, two and three single span shorter bridges. It is our understanding that the preferred alternatively is two bridges. The bridges will be approximately 30 feet wide. The abutments will likely be protected against scour with a mat of rip-rap armor. Project design was completed at a preliminary level in order to evaluate the alternatives and relative costs; however, additional analyses, design and recommendations will be appropriate during final design for the project.

The purpose of our geotechnical engineering services was to explore the surface and subsurface soil and groundwater conditions as a basis to develop preliminary geotechnical design recommendations for the bridge construction. Our scope of geotechnical engineering services included drilling three borings, completing laboratory testing on the samples obtained from the explorations, and providing geotechnical conclusions and recommendations for preliminary design and construction of the proposed bridge(s) to determine costs for grant application.

SITE CONDITIONS

Geology

We reviewed the “Geologic Map of the Sauk River 30- by 60-Minute Quadrangle, Washington” by R.W. Tabor, dated 2002. According to the map, the site is mapped as Quaternary alluvial fan deposits.

We observed very limited alluvial fan deposits at the site during our reconnaissance. We encountered glacial till in our explorations. Glacial till typically consists of a dense to very dense, nonsorted mixture of clay, silt, sand, gravel, cobbles and boulders. The distribution and quantity of cobbles and boulders is unpredictable in these glacial soils. Boulders ranging up to 10 to 20 feet in diameter have been observed in glacial soils within the Puget Sound region. Gravel, cobbles and boulders (up to several feet in diameter) were observed randomly within the existing channel and throughout the area during our reconnaissance; however, we did not observe significant thickness of surficial alluvial fan deposits.

Surface Conditions

The site is located on Rockport Cascade Road approximately 4.2 miles east of the intersection with SR 530. Rockport Cascade Road is a low-volume asphalt paved road with little to no shoulder. The road is approximately 18 feet wide and used for access to local single family residences. Near the creek crossing, the road is an embankment leading up to the bridge. However, in general, the site terrain is relatively level with a slight slope downward to the north. An existing 24-inch corrugated plastic pipe (CPP) is located within the footprint of the proposed new bridge. At the time of our visit in late April 2011 we did not observe any water in the historic channel.

The surrounding property is undeveloped with no adjacent residences. Vegetation along the sides of the road consists of small to large deciduous and evergreen trees with shrubs, ferns, and grasses. We observed several cobbles and boulders associated with the historic channel on either side of the roadway. Significant large blowdown has occurred; the tree roots are very flat suggesting the tree roots were able to achieve very little penetration and glacial till was exposed at the base of the up-ended trees.

Subsurface Explorations

Subsurface soil and groundwater conditions were evaluated by drilling three borings at the locations shown in the Site and Exploration Plan, Figure 2. The borings were completed on April 26, 2011 to depths ranging from 9.5 to 30 feet below the existing ground surface (bgs). The borings were completed using a track-mounted drill rig subcontracted to GeoEngineers, Inc. The approximate locations of the explorations are shown in Figure 2. Details of the field exploration program, laboratory testing, and the boring logs are presented in Appendix A.

The borings were planned to be terminated at approximately 30 feet bgs. Boring B-2 encountered refusal at 10 feet bgs and B-3 encountered refusal at 8 feet bgs during original drilling. We moved B-2 6 feet east and encountered refusal at approximately 20 feet bgs; we moved B-3 15 feet east and encountered refusal at approximately 9.5 feet bgs. It is likely that the borings were terminated on cobbles/boulders in the till matrix.

Subsurface Conditions

Soil Conditions

The borings were completed from the roadway in the center of the westbound lane. Boring B-1 was completed at a potential east bridge abutment. Approximately 6 inches of asphalt concrete was encountered at the ground surface in Boring B-1. Beneath the asphalt we encountered dense sand with silt and gravel fill soil to approximately 5 feet bgs. Medium dense silty sand with gravel was encountered from 5 feet bgs to approximately 12 feet bgs. A log was encountered in the sample at 10 feet bgs. We conclude that this material is either fill or reworked native soils; based on the surrounding topography and conditions encountered in adjacent borings there is no clear topographical feature or other evidence as to why the fill would extend so deep. This material could be stream deposits; however, the presence of a similar silt content to the native soils suggest that this is also unlikely. Beneath the fill we encountered dense to very dense silty sand with gravel and occasional cobbles which we interpret to be glacial till.

B-2 encountered 4 inches of asphalt concrete overlying dense sand and gravel fill to a depth of 3 feet bgs. Beneath the fill we encountered dense to very dense silty sand with gravel and occasional cobbles which we interpret to be glacial till. At a depth of 10 feet we encountered a cobble or boulder. We moved the boring 6 feet to the east and were able to advance the boring to 20 feet prior to encountering refusal.

B-3 encountered 6.5 inches of asphalt concrete overlying dense sand and gravel fill to a depth of 1 foot bgs. Beneath the fill we encountered dense to very dense silty sand with gravel and occasional cobbles which we interpret to be glacial till. At a depth of 8 feet we encountered a cobble or boulder. We moved the boring 15 feet to the east and were able to advance the boring to 9.5 feet prior to encountering refusal.

Groundwater Conditions

Groundwater was not encountered at any of the boring locations. Our explorations were not left open long enough to allow groundwater to stabilize. The groundwater conditions should be expected to vary as a function of season, the rise and fall of the creek, precipitation, and other factors. The glacial till is considered to be essentially impermeable because of the fines content and glacial consolidation (high density). A perched groundwater condition will occur within the weathered zone during the winter, or surface water will occur where no weathered zone exists.

CONCLUSIONS AND RECOMMENDATIONS

General

We conclude that the new bridge may be supported by conventional shallow footings bearing on the dense to very dense glacial till. We reviewed the available plans to the existing bridge, which are dated 1970. The existing bridge is supported on shallow spread footings. The plans indicate that a footing subgrade elevation of 303 feet was used for design, which was to be a minimum 3 feet below the planned channel elevation of 306 feet. The present bridge and abutments have performed satisfactorily with no evidence of scour.

A summary of the site preparation, design, and construction considerations for the proposed project is provided below. This summary is presented for introductory purposes only and should be used in conjunction with the complete recommendations presented in this report.

- We recommend construction occur during summer/early fall months to minimize construction costs.
- Shallow spread footings can be used for foundations bearing on dense glacial till soil. Foundations may be designed with maximum allowable bearing capacity of 6,000 pounds per square foot (psf).
- On-site fill soil may be considered for reuse provided it meets specifications set forth for suitable structural fill material. Use of these soils will likely require segregation of the oversized material prior to placement.

Seismic Considerations

Seismicity

The site is located within the Puget Sound region, which is seismically active. Seismicity in this region is attributed primarily to the interaction between the Pacific, Juan de Fuca and North American plates. The Juan de Fuca plate is subducting beneath the North American plate. It is thought that the resulting deformation and breakup of the Juan de Fuca plate might account for the deep focus earthquakes in the region. Hundreds of earthquakes have been recorded in the Puget Sound area. In recent history, four of these earthquakes were large events: (1) in 1946, a Richter magnitude 7.2 earthquake occurred in the Vancouver Island, British Columbia area; (2) in 1949, a Richter magnitude 7.1 earthquake occurred in the Olympia area; (3) in 1965, a Richter magnitude 6.5 earthquake occurred between Seattle and Tacoma; and (4) in 2001, a Richter magnitude 6.8 earthquake occurred near Olympia.

Research has concluded that historical large magnitude subduction-related earthquake activity has occurred along the Washington and Oregon coasts. Evidence suggests several large magnitude earthquakes (Richter magnitude 8 to 9) have occurred in the last 1,500 years, the most recent of which occurred about 300 years ago. No earthquakes of this magnitude have been documented during the recorded history of the Pacific Northwest. Current codes account for these large earthquakes in the design.

Fault Hazards

Local design practice in Puget Sound and local building codes include the possible effect of local known faults in the design of structures. The site is located approximately 2 miles from a concealed high angle fault which is unnamed. The Straight Creek Fault Zone is located approximately 3 miles east of the site. It is our opinion that the faults likely represent low risk of ground fault rupture at the project site.

Seismic Zone and LRFD Parameters

We understand that the 2008 version of the AASHTO LRFD design manual will be used to design the replacement bridge. The design earthquake has a 7 percent probability exceedance in 75 years (i.e. a 1000-year recurrence interval). We recommend the project site be classified as Site Class C and that the following seismic parameters be used based on the seismic data provided in the LRFD manual:

TABLE 1. SPECTRAL RESPONSE ACCELERATIONS (SRAS)

(SRA) and Site Coefficients	PGA	Short Period	1-Second Period
Mapped SRA	PGA = 0.25	S_S = 0.57	S_1 = 0.19
Site Coefficients	F_{pga} = 1.16	F_a = 1.17	F_v = 1.62
Design SRA	A_s = 0.28	S_{DS} = 0.66	S_{D1} = 0.30

Note:

1. Site Class C Description: Very dense soil and soft rock (N > 50).

Liquefaction Potential

Liquefaction refers to a condition where vibration or shaking of the ground, usually from earthquake forces, results in the development of excess pore pressures in saturated soils and subsequent loss of strength. This can result in vertical oscillations and/or lateral spreading of the affected soils with accompanying surface subsidence and/or heaving. In general, soils which are susceptible to liquefaction include loose to medium dense clean to silty sands, which are saturated (i.e., below the water table).

The foundation soils encountered within our explorations include dense to very dense silty sand and sand with silt, gravels, cobbles, and boulders. Any underling soils over the local bedrock will have also been glacially consolidated. It is our opinion that the foundation soils have a low susceptibility to liquefaction.

Shallow Foundation Recommendations

General

Based on the conditions encountered in the explorations, it is our opinion that the proposed bridge can be supported on a shallow foundation supported on the native soils. GeoEngineers performed hydraulics and scour analyses based on the preferred alternative (Alternative 2 with two bridges), Scour components considered include long-term degradation, contraction scour, and abutment scour. Scour related to long term degradation and contraction scour was determined to be negligible for this project and is consistent with field observations. Abutment scour was determined to be between 6 to 8 feet; the equations for estimating abutment scour are widely considered over-predictive for practical application. Therefore, we concluded in our scour report that setting the bridge foundations a nominal 2 feet below the design channel elevation in combination with abutment protection should provide for adequate protection against scour while minimizing the amount of excavation needed to construct the bridge foundations to withstand the ultimate scour elevations. The design channel elevations for the two channels are 306.1 feet for the west channel and 307.1 feet for the east channel; therefore, we recommend a minimum footing subgrade elevation of 304 and 305 feet for the west and east channels respectively.

Foundation Subgrade Preparation

We recommend the foundation be constructed on the undisturbed glacial till: very dense silty sand with gravel and occasional cobbles based on the results of the explorations. Boring B-1 encountered a log at 10 feet bgs (approximately Elevation 304.5 feet) indicating that a fill/disturbed zone extends to approximately 12 feet bgs (Elevation 302.5 feet) at this location. Therefore, we recommend that the base of the footing excavation be evaluated by the field geotechnical engineer prior to construction of the foundation. At the other boring locations, very dense glacial till was encountered at 1 to 3 feet bgs. Therefore we recommend that the location of the bridge footing not be located directly over the B-1 area.

Due to the high bearing pressure, if fill soils are encountered at the abutment location we recommend that they be overexcavated and replaced with structural fill consisting of crushed rock compacted to 98 percent of the maximum dry density (MDD) in accordance with ASTM D 1557 or CDF/lean concrete. The subgrade should be dense to very dense. Loose/soft, organic or other unsuitable soils encountered at the excavation subgrade may require overexcavation or stabilization as directed by the field geotechnical engineer.

Shallow Foundation Design

We anticipate that the abutment foundations will extend the entire width of the bridge. The long, continuous abutment footings founded on suitably dense soils will provide adequate support for the proposed bridge. As subsequently described, the glacial till will be very difficult to excavate. Because of the relatively remote location, it may be cost effective to construct the bridge foundations using precast foundation elements.

The footing should be embedded such that the outside edge of the footing is a minimum of 2 feet horizontally from the back of any rip-rap slope, which may be required for erosion protection, extending down to the creek. This value assumes suitable rip-rap protection as discussed in our separate report. We recommend that footings bearing on suitably dense soils be designed using an allowable soil bearing pressure of 6,000 psf for dead-plus-long-term-live loads. The allowable soil bearing pressures may be increased by up to one-third for wind and seismic loads.

Settlement Potential

We estimate the total and differential settlement of shallow spread footings founded on the soils described above to be less than ½ inch. We estimate that settlement will occur rapidly, generally as loads are applied.

Abutment Retaining Wall and Lateral Soil Pressures

Lateral soil pressures acting on the abutment retaining and wing walls will depend on the nature and density of soil behind the wall, amount of lateral wall movement which occurs as backfill is placed, and the inclination of the backfill surface. For walls free to yield at the top at least one thousandth of the wall height (i.e., wall height times 0.001), soil pressures will be less than if movement is restrained. We recommend that walls free to yield at the top and supporting horizontal backfill be designed using an equivalent fluid density of 35 pounds per cubic foot (pcf). We recommend using a uniform traffic surcharge pressure of 250 psf where traffic will be within 10 feet of the wall. We also recommend a uniformly distributed seismic surcharge of 7H psf (H = height of wall) be applied to the wall. Alternatively, the seismic loading could be calculated using a K_{ae} equal to 0.295. Lateral pressure resulting from traffic and seismic surcharge loading is additive to lateral soil pressures computed as recommended above.

The recommended equivalent fluid density presented above is based on the assumption that fill behind the walls is placed and compacted as recommended herein. Overcompaction of fill placed directly behind retaining walls should be avoided. We recommend use of hand-operated compaction equipment and maximum 6-inch loose lift thickness when compacting fill within about 5 feet of abutment walls. Compaction should be in the range of 90 to 92 percent of the MDD.

Assuming that some scour is likely, it is reasonable to assume very small passive soil pressure on the water side of the bridge abutments. The armor rock is the only material that can be relied on to remain in place after scour has occurred and provide the lateral earth pressure. Due to the typical inclination of the armor rock (1.5H:1V [Horizontal:Vertical]), an allowable passive resistance on the face of the abutment wall and foundation can be computed using an equivalent fluid density of 70 pcf (triangular distribution from the ground surface to base of the retaining wall) for structural fill or medium dense native fill. Frictional resistance may be evaluated using 0.42 for the coefficient of base friction against the footings. The recommended passive equivalent fluid density value and coefficient of friction include a factor of safety of 1.5.

Drainage

Drainage systems should be constructed to collect water and prevent the buildup of hydrostatic pressure against abutment retaining walls. We recommend these drainage systems include a zone of free-draining backfill that has a minimum of 3 feet in width against the back of the wall. Free draining backfill should conform approximately to Standard Specification 9-03.12(2), "Gravel Backfill for Walls." Material conforming to Washington State Department of Transportation (WSDOT) 9-03.9(3) Crushed Surfacing, Base Course, also may be used for free draining backfill provided a fines content of less than 3 percent is specified. The free draining backfill zone should extend for the full height of the wall. The backfill zone should be drained with either weep holes at the base of the wall or with a drainpipe. If weepholes are used, provisions should be incorporated to prevent migration of the backfill through the weepholes. The drainpipe should consist of a perforated rigid, smooth walled pipe with a minimum diameter of 4 inches and should be placed along the base of the wall within the free draining backfill, extending the entire wall length. The drainpipe should be metal or rigid PVC pipe and be sloped to drain by gravity. Discharge should be routed properly to reduce erosion potential.

Earthwork

General

Excavations will extend through any roadway fill and into native medium dense to very dense silty sand with gravel and occasional cobbles (glacial till). Any new channels will also be excavated primarily into the very dense till based on our field reconnaissance. The till is very dense and based on the drilling action will not be practical to excavate with regular backhoes. Cobbles and boulders will likely be encountered. **Therefore, we suggest that the contractor plan on using large horsepower tracked excavators to excavate the very dense glacial till.**

Glacial till typically contains a significant percentage of fines (silt and clay) and is moisture-sensitive. When the moisture content is more than a few percent above the optimum moisture content, these soils can become muddy and unstable, and operation of equipment on these soils can be difficult. Wet weather construction is generally not recommended for these soils without the use of admixtures to control moisture content. These soils typically meet the criteria for "Common Borrow." Relatively low infiltration rates (less than 0.25 inch per hour) are typically appropriate in glacial till because of the high fines content and density.

Temporary Excavations

All excavations and other construction activities must be completed in accordance with applicable city, state and federal safety standards. Regardless of the soil type encountered in the excavation shoring, trench boxes or sloped sidewalls will be required for excavations deeper than 4 feet under Washington State Administrative Code (WAC) 296-155, Part N. We expect that most of the trench excavations will be made as open cuts in conjunction with the use of a trench box and/or sloped sidewalls for shielding workers. For planning purposes only, the dense glacial till soil found on site is classified as “Type A” soil, and the fill is classified as “Type C” soil. The regulations allow temporary slopes for this condition up to 0.75:H1V and 1.5H:1V respectively.

The above regulations assume that surface loads such as construction equipment and storage loads will be kept a sufficient distance away from the top of the cut so that the stability of the excavation is not affected. In order to maintain the stability of the cut flatter slopes and/or shoring will be necessary for those portions of the excavations which are subjected to significant seepage. Temporary slopes in wet/saturated sand will be susceptible to sloughing, raveling and "running" conditions. It should be expected that unsupported cut slopes will experience some sloughing and raveling if exposed to surface water. Berms, hay bales or other provisions should be installed along the top of the excavation to intercept surface runoff to reduce the potential for sloughing and erosion of cut slopes during wet weather.

In our opinion, the contractor will be in the best position to observe subsurface conditions continuously throughout the construction process and to respond to the variable soil and groundwater conditions. Construction site safety is generally the responsibility of the contractor, who also is solely responsible for the means, methods, and sequencing of the construction operations and choices regarding temporary excavations and shoring. We are providing this information only as a service to our client. Under no circumstances should the information provided below be interpreted to mean that GeoEngineers, Inc. is assuming responsibility for construction site safety or the contractors’ activities; such responsibility is not being implied and should not be inferred.

Structural Fill

GENERAL

We anticipate that the use of structural fill on the site will be limited to backfilling against abutment walls, around footing excavations and the approach embankments. All fill placed on the site should be placed and compacted as structural fill. All structural fill material should be free of organic matter, debris, and other deleterious material. The maximum particle size diameter for structural fill should be the lesser of either 6 inches or one half of the loose lift thickness.

As the amount of fines (material passing the U.S. No. 200 sieve) increases in a soil, it becomes more sensitive to small changes in moisture content and during wet conditions, adequate compaction becomes more difficult to achieve. Generally, soils containing more than about 5 percent fines by weight cannot be properly compacted when the moisture content is more than a few percent from optimum.

The fill should be placed in horizontal lifts not exceeding 12 inches in loose thickness or that necessary to obtain the specified compaction with the equipment used. Each lift must be thoroughly and uniformly compacted. We recommend that any structural fill placed on the site be compacted to at least 95 percent of the MDD as determined by the ASTM D 1557 test procedure. As previously stated, structural fill is not desirable below the footings because of the high allowable bearing pressure. If necessary, crushed rock (WSDOT 9-03.9(3) Crushed Surfacing, Base Course) could be used and extend 1 foot beyond the edge of the footing and down to the undisturbed dense glacial till. The crushed rock should be compacted to at least 98 percent of the MDD.

Sufficient earthwork monitoring and a sufficient number of in-place density tests should be performed to evaluate fill placement and compaction operations and to confirm that the required compaction is being achieved.

SUITABILITY OF ON-SITE SOIL

The on-site soils include fill and native soils consisting of silty sand and gravel with sand and silt. Cobbles and boulders were observed in our explorations. Use of these soils will require segregation of the oversized material prior to placement. The silty materials are moisture-sensitive and can be difficult to compact to 95 percent of the MDD, particularly during periods of wet weather. At the time of our explorations, the moisture content of the materials was near or below the optimum moisture content for compaction and may require moisture conditioning to achieve recommended compaction. It is our opinion that the on-site material is generally suitable for use as structural backfill during periods of dry weather.

SELECT IMPORT FILL

To reduce extra costs and delays during construction, we suggest that imported soil could be used during periods of wet weather. Select import fill should conform to the recommendations provided in the "General" section above. We recommend using a select import fill consisting of sand and gravel with a fines content of less than 5 percent base on that portion passing the ¾-inch sieve and at least 30 percent gravel (retained on the U.S. No. 4 sieve).

LIMITATIONS

We have prepared this report for the exclusive use of Skagit River Systems Cooperative, TranTech Engineering LLC, and their authorized agents for the proposed Illabot Creek Bridge Relocation project near Rockport, 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, express 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 appendix titled Report Limitations and Guidelines for Use for additional information pertaining to use of this report.

FIGURES

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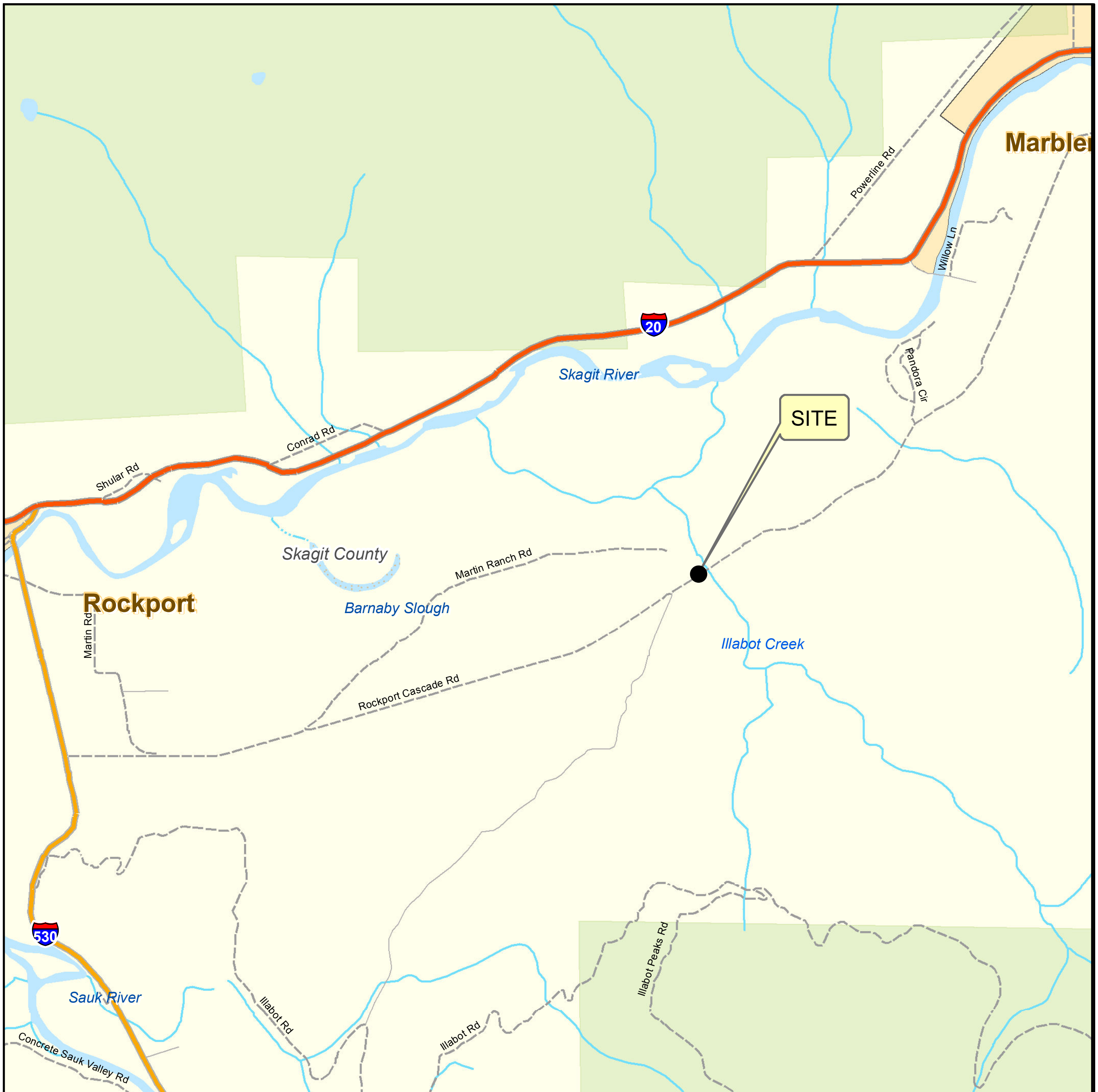

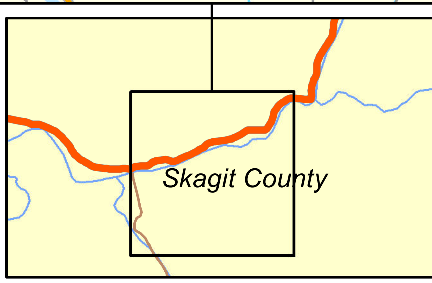

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Feet

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. can not 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.
3. It is unlawful to copy or reproduce all or any part thereof, whether for personal use or resale, without permission.

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

Illabot Creek Project
Skagit County, Washington

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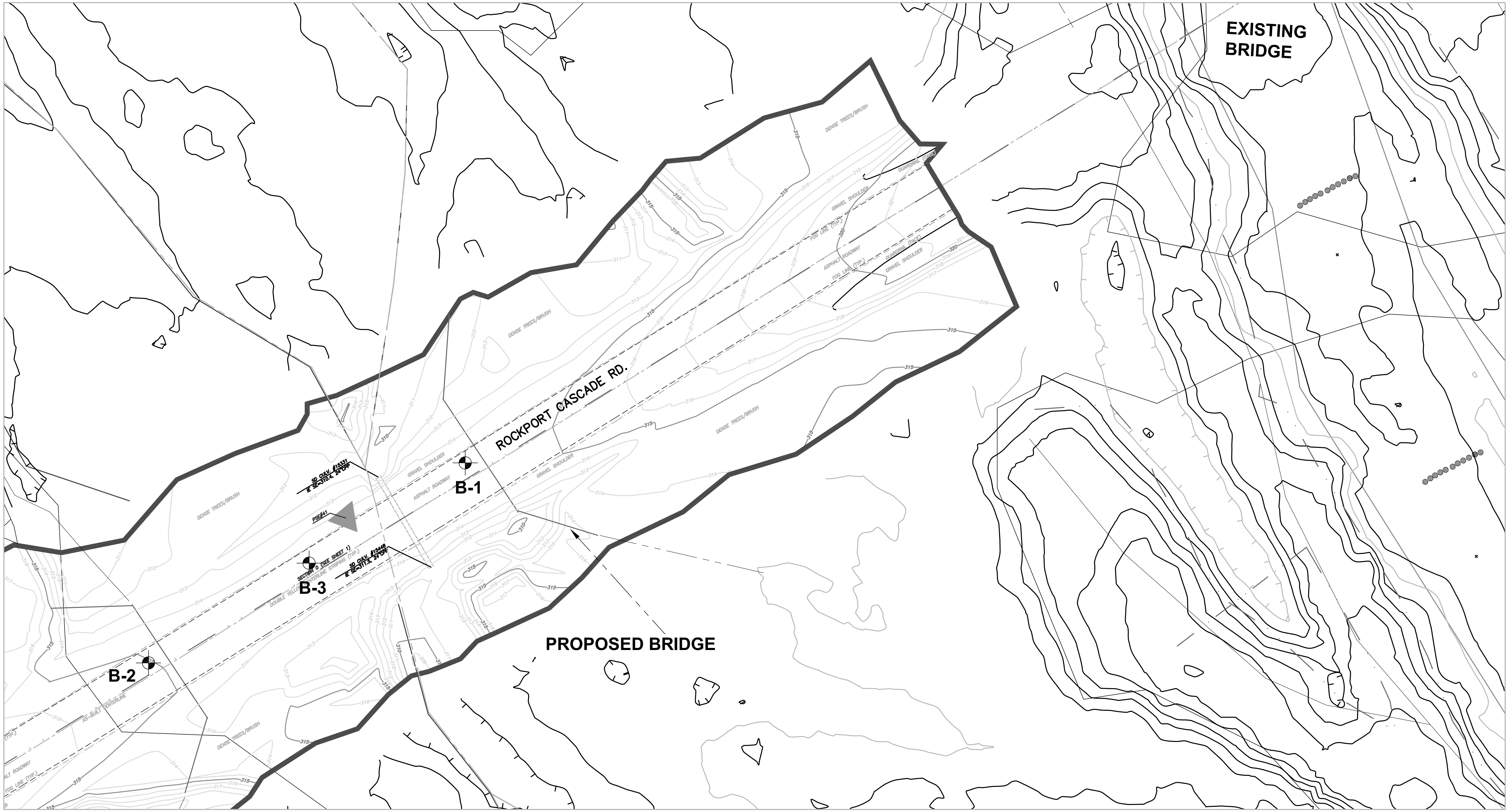
Figure 1

Office: BELL Path: P:\11\1112900400\GIS\1112900400 VM F1.mxd AJH:IMS 05/25/2011



Legend

B-1 = Boring number and approximate location

N
W E
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40 0 40
Feet

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. can not 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 Drawing provided by Resource Consultants Inc., Redmond, Washington

Site and Exploration Plan

Illabot Creek Project
Skagit County, Washington

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Figure 2

BELL P:\11\011129-004\00\AutoCAD\1112900400 SP F2.dwg AJH:ims 05/25/11

APPENDICES

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APPENDIX A
Field Explorations and Laboratory Testing

APPENDIX A
FIELD EXPLORATIONS AND LABORATORY TESTING

Field Explorations

Subsurface conditions at the site were explored by completing three geotechnical borings on April 26, 2011. The borings were completed using an M-55 track-mounted drill rig subcontracted to GeoEngineers, Inc. The approximate locations of the explorations are shown in the Site and Exploration Plan, Figure 2. The locations of the borings were determined by pacing and taping; therefore, the location shown on Figure 2 should be considered approximate. The elevations shown on the boring logs were determined by interpolating the contour information on the site plan and should be considered accurate to the degree implied by the method used.

Disturbed soils samples were obtained using Standard Penetration Test (SPT) methodology with the standard split spoon sampler in the borings. The samples were placed in plastic bags to maintain the moisture content and transported back to our laboratory for analysis and testing.

The borings were continuously monitored by a geotechnical engineer 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. Soils encountered were classified visually in general accordance with ASTM D-2488-90, which is described in Figure A-1. An explanation of our boring log symbols is also shown on Figure A-1.

The logs of the borings are presented in Figure A-2 through A-4. The exploration logs are based on our interpretation of the field and laboratory data and indicate the various types of soils encountered. It also indicates the depths at which these soils or their characteristics change, although the change might actually be gradual. If the change occurred between samples in the boring, it was interpreted.

The borings were planned to be terminated at approximately 30 feet below ground surface (bgs). Boring B-2 encountered refusal at 10 feet bgs and B-3 encountered refusal at 8 feet bgs during original drilling. We moved B-2 6 feet east and encountered refusal at approximately 20 feet bgs; we moved B-3 15 feet east and encountered refusal at approximately 9.5 feet bgs.

Laboratory Testing

General

Soil samples obtained from the explorations were transported to our 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, dry density, and percent fines. The tests were performed in general accordance with test methods of the American Society for Testing and Materials (ASTM) or other applicable procedures.

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 at the depths at which the samples were obtained.

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, classified in general accordance with the Unified Soil Classification System (USCS), and are presented in Figure A-5.

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

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.

ADDITIONAL MATERIAL SYMBOLS

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

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

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
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

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FIGURE A-1

Drilled	Start 4/26/2011 End 4/26/2011	Total Depth (ft) 30.5	Logged By AJH Checked By AJH	Driller Boretec, Inc.	Drilling Method Hollow-stem Auger
Surface Elevation (ft) Vertical Datum	314.5	Hammer Data	140 lb hammer Two wraps on cathead	Drilling Equipment	M55 Track-mounted Drill Rig
Latitude Longitude		System Datum	Geographic	Groundwater	Date Measured / Depth to Water (ft) / Elevation (ft): Undetermined
Notes:					

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	6 inches of asphalt concrete and chipseal			
								SP-SM	Gray fine to coarse sand with silt and gravel (dense, moist) (fill)			
		6	40		1					3		
310								SM	Gray silty fine to coarse sand with gravel (medium dense, moist) (fill or reworked native)			
	5	6	28		2							
305												
	10	18	18		3				- log encountered			Wood
								SM	Gray silty fine to coarse sand with gravel and occasional cobbles and boulders (dense, moist) (glacial till)			
300												
	15	6	50/6"		4					9		Blowcount overstated
295												
	20	12	44		5					7		

Note: See Figure A-1 for explanation of symbols.

Log of Boring B-1

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
290													
	25		11	50/5"		6					6		
285													
	30		3	50/5"		7					5		

Note: See Figure A-1 for explanation of symbols.

Log of Boring B-1 (continued)

GEOENGINEERS

Project: Illabot Creek Bridge Design
Project Location: Rockport, Washington
Project Number: 11129-004-00

Drilled	Start: 4/26/2011	End: 4/26/2011	Total Depth (ft)	20	Logged By: AJH / Checked By: AJH	Driller	Boretec, Inc.	Drilling Method	Hollow-stem Auger

Surface Elevation (ft) / Vertical Datum	315.0	Hammer Data	140 lb hammer / Two wraps on cathead
Drilling Equipment	M55 Track-mounted Drill Rig		
Latitude / Longitude		System / Datum	Geographic
Notes:			

Groundwater		
Date Measured	Depth to Water (ft)	Elevation (ft)
		Undetermined

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
315	0						AC	4 inches of asphalt concrete			
							SP-SM	Brown fine to coarse sand with silt and gravel (dense, moist) (fill)			
		12	35	1a 1b							
							SM	Gray silty fine to coarse sand with gravel, occasional cobbles and boulders (very dense, moist) (glacial till)	6		
310	5	0	50/2"	2 No recovery; SA					3		Sampled cuttings
305	10	3	50/3"	3a 3b				- moved 6 feet east			
300	15	12	50/6"	4					3		
295	20	1	50/2"	5							
								Boring terminated due to refusal at 20 feet			

Note: See Figure A-1 for explanation of symbols.

Log of Boring B-2

GEOENGINEERS

Project: Illabot Creek Bridge Design
Project Location: Rockport, Washington
Project Number: 11129-004-00

Figure A-3
Sheet 1 of 1

Drilled — Start: 4/26/2011, End: 4/26/2011	Total Depth (ft): 9.5	Logged By: AJH Checked By: AJH	Driller: Boretec, Inc.	Drilling Method: Hollow-stem Auger
Surface Elevation (ft) / Vertical Datum: 314.0	Hammer Data: 140 lb hammer, Two wraps on cathead		Drilling Equipment: M55 Track-mounted Drill Rig	
Latitude / Longitude	System / Datum: Geographic		Groundwater — Date Measured / Depth to Water (ft) / Elevation (ft): Undetermined	
Notes:				

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	6½ inches of asphalt concrete			
									SM	Brown silty fine to coarse sand with gravel (dense, moist)			
									SM	Gray silty fine to coarse sand with gravel, occasional cobbles and boulders (dense, moist) (glacial till)			
			12	55		1					3		
310													
	5		18	78		2 / SA					3		
										- refusal at 8 feet - moved 15 feet east			
305													
			3	50/3"		3					2		

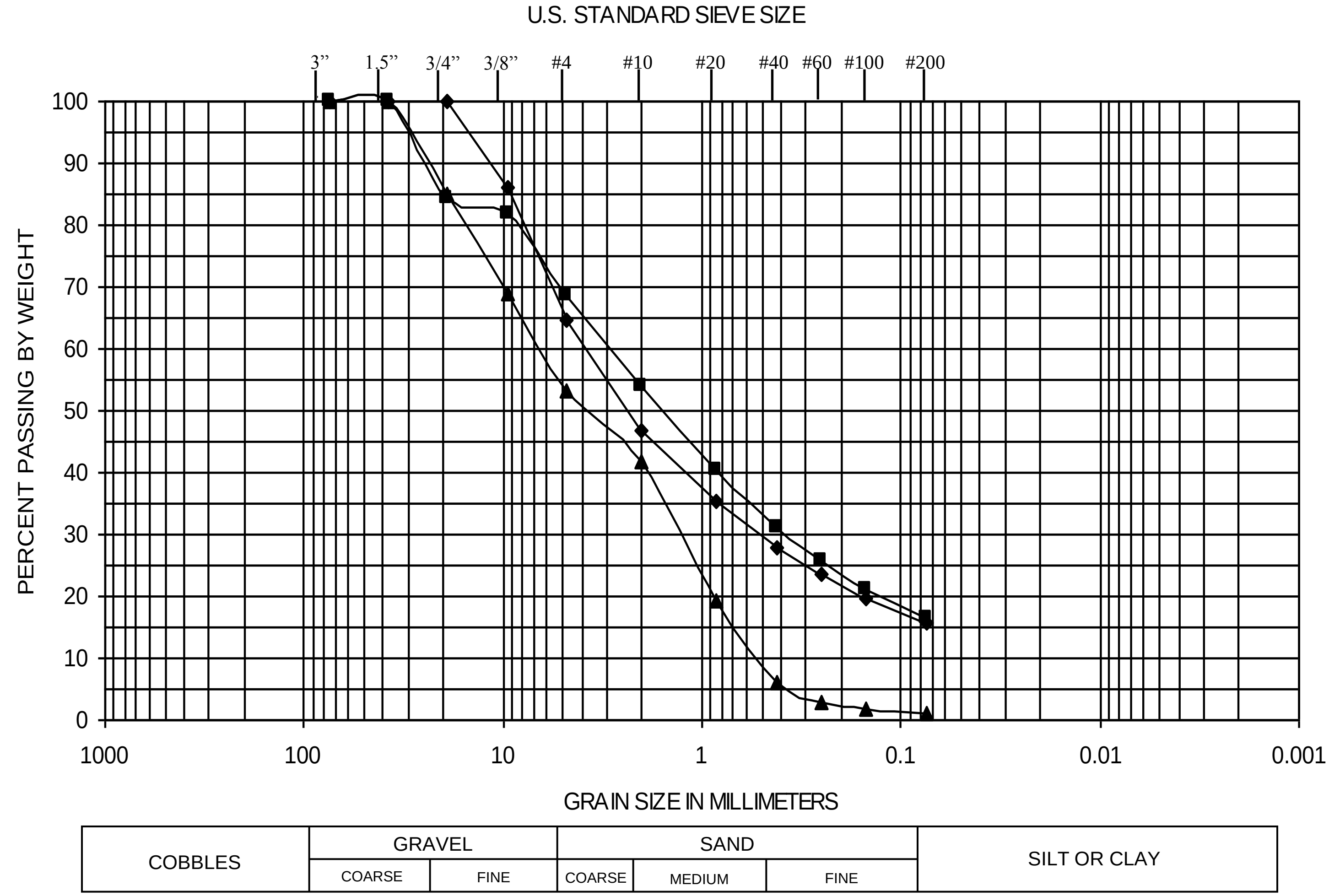
Boring terminated due to refusal at 9.5 feet on a rock

Note: See Figure A-1 for explanation of symbols.

Log of Boring B-3

GeoEngineers

Project: Illabot Creek Bridge Design
Project Location: Rockport, Washington
Project Number: 11129-004-00



Symbol	Exploration Number	Sample Depth (feet)	Soil Classification
♦	B-2	5- 10	Gray silty fine to coarse sand with gravel
■	B-3	5	Gray silty fine to coarse sand with gravel
▲	Riverbed	Surface	Gray medium to coarse sand with gravel

Sieve Analysis Results

Illabot Creek Bridge Relocation
Rockport, Washington

GEOENGINEERS

Figure A-5

APPENDIX B
Report Limitations and Guidelines for Use

APPENDIX B.
REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

This appendix 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 Skagit River Systems Cooperative, TranTech Engineering LLC., and their authorized agents. This report may be made available to other members of the design team. 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 Illabot Creek Bridge Relocation project near Rockport, 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.

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

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.

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, and 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.

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