GEOTECHNICAL ENGINEERING REPORT

PROPOSED BARNABY CREEK BRIDGE

Colville Confederated Tribes Indian Reservation
Ferry County, Washington

Prepared for:
Water & Natural Resource Group
P.O. Box 28755
Spokane, Washington 99228

Prepared by:
Associated Earth Sciences, Inc.
911 5th Avenue, Suite 100
Kirkland, Washington 98033
425-827-7701
Fax: 425-827-5424

April 13, 2005
Project No. KE05150A

I. PROJECT AND SITE CONDITIONS

1.0 INTRODUCTION

This report presents the results of our geotechnical engineering study for the proposed Barnaby Creek Bridge located on the fee land owned by Mr. and Mrs. Abell, Colville Confederated Tribes (CCT) Indian Reservation, Ferry County, Washington. The general location of the site is shown on the Vicinity Map, Figure 1, included with this report. Information regarding site development is based on verbal information provided to us. No details for site development were available to us as of the date of this report. Therefore, recommendations presented herein are preliminary.

Site reconnaissance and exploration was conducted by representatives of the Water & Natural Resource Group (WNR Group), the results of which have been provided to us for use as a basis on which to prepare recommendations presented in this report. No site visit or subsurface exploration has been conducted by Associated Earth Sciences, Inc. (AESI). The site description and geologic interpretation have been incorporated into the text of this report. Logs of exploration pits are presented in Appendix A. Site photos are included as Appendix B.

1.1 Purpose and Scope

The purpose of this study was to provide geotechnical design recommendations to be utilized in the design of the Barnaby Creek Bridge project. As requested, geotechnical engineering studies were completed based on information provided to us to establish recommendations for the bridge foundation including minimum footing width and depth, foundation soil bearing pressure, site preparation, lateral earth pressure, base friction, and drainage considerations. This report summarizes our recommendations based on our present understanding of the project. We recommend that our office be allowed to review the finalized project plans and specifications once they become available to verify that our geotechnical recommendations adequately address the project design.

1.2 Authorization

Authorization to proceed with this study was granted by Mr. Eugene St.Godard, R.G., L.Hg. of the WNR Group and Mr. Kris Ray of the CCT Indian Reservation. Our study was accomplished in general accordance with our agreement dated January 3, 2005. This report has been prepared for the exclusive use of the WNR Group, the CCT, and their agents for specific application to this project. Within the limitations of scope, schedule, and budget, our services have been performed in accordance with generally accepted geotechnical engineering practices in effect in this area at the time our report was prepared. No other warranty, express or implied, is made. Our observations, findings, and opinions are a means to identify and reduce the inherent risks to the owner.

2.0 PROJECT AND SITE DESCRIPTION

This report is based on verbal information, site description, subsurface data, photographs, and an undated, untitled map provided by the WNR Group. It is our understanding that the proposed development will include construction of a bridge across Barnaby Creek. The proposed bridge will cross Barnaby Creek at approximately the same location as an existing wood bridge. The following site description was provided by Mr. Eugene St.Godard of the WNR Group.

2.1 Barnaby Creek Bridge - Site Description

The subject site is located in the northeast ¼ of the southwest ¼ of Section 15, Township 34 North, Range 36 East within the Barnaby Creek drainage. The site is located approximately 1½ miles west of the confluence of Barnaby Creek and the Columbia River (Lake Roosevelt) and is located within the boundaries of the CCT Indian Reservation. However, the site is located on fee lands within the Reservation. The site is located within a steep incised section of the Barnaby Creek drainage forested with cedar and tamarack with some pine and fir. The general area is currently utilized as timberland with sporadic residential housing.

Access to the proposed bridge site is via a dirt logging road from the north valley wall. An existing wood bridge is currently located at the proposed bridge location, which was constructed from planks placed on boulder footings. The bridge deck is currently located approximately 4 feet above the stream.

The valley walls vary in slopes from 20 to 60 degrees. Soils within the drainage consists of colluvium from the bedrock outcrops and glacial deposits near the top of the slopes. Barnaby Creek is approximately 15 to 18 feet wide at the proposed bridge site with the bank width estimated at 35 to 40 feet. The stream gradient is approximately 1 to 2 percent. The stream is incised in a fairly developed channel. The streambed consists mostly of sands, gravels, and cobbles.

3.0 SUBSURFACE EXPLORATION

Subsurface exploration was conducted by the WNR Group on December 17, 2004. The field study included excavating two exploration pits to gain surface and subsurface information about the site. The various types of sediments, as well as the depths where characteristics of the sediments changed, are indicated on the exploration logs presented in Appendix A. The depths indicated on the logs where conditions changed may represent gradational variations between sediment types in the field.

The conclusions and recommendations presented in this report are based on information presented on the exploration logs, photographs, site description, and other information provided to us. Because of the nature of exploratory work below ground, extrapolation of subsurface conditions between field explorations is necessary. It should be noted that differing subsurface conditions may sometimes be present due to the random nature of deposition and the alteration of topography by past grading and/or filling. The nature and extent of any variations between the field explorations may not become fully evident until construction. If variations are observed at that time, it may be necessary to re-evaluate specific recommendations in this report and make appropriate changes.

3.1 Exploration Pits

The exploration pits were completed by Shifflet & Sons with a rubber-tire, Case 580K backhoe. The exploration pits were observed and logged by a hydrogeologist from the WNR Group. The exploration logs presented in Appendix A are unedited copies of those presented to us.

4.0 SUBSURFACE CONDITIONS

Subsurface conditions at the project site were inferred from the results of the field explorations accomplished for this study and information provided to us. A copy of the exploration logs is presented in Appendix A. The field soil logs indicate that the project site is mantled by a relatively thin (0.5- to 1-foot) veneer of loose soil underlain by a medium dense to dense mixture of silt, sand, gravel, and cobble with boulders. This soil unit is interpreted as "River Alluvium".

4.1 Soil Conditions

Fill/Topsoil

A brown, damp, organic, silty sand in a generally loose condition was encountered to a depth of 1 foot in the exploration pits excavated for this study. Because of the relatively loose condition, the fill/topsoil is not considered suitable for foundation support.

Alluvium

Underlying the fill/topsoil was a medium dense to dense, moist to saturated, brown to dark brown mixture of sand, gravel, cobble, and boulders to a depth of between 5.5 and 7 feet below existing ground surface. Medium dense to very dense silt with gravel was encountered below the granular sediments. These sediments are considered to be alluvial, stream deposit sediments. This material, where in a medium dense or dense condition, will provide suitable foundation support for the proposed Barnaby Creek Bridge.

4.2 Hydrology

Ground water seepage was encountered at a depth of approximately 3.5 feet in exploration pit TP-2. In general, the soils below the observed seepage appeared to be above their optimum moisture contents at the time of exploration. The quantity of flow from excavations that exposes seepage will depend on the soil grain size, topography, and season. Ground water conditions should be expected to vary in response to changes in precipitation, land usage, and other factors.

II. DESIGN RECOMMENDATIONS

5.0 INTRODUCTION

Exploration indicates that, from a geotechnical standpoint, the proposed Barnaby Creek Bridge project is compatible with this site provided the recommendations contained herein are properly followed. The foundation bearing stratum (medium dense to dense alluvial sediments or structural fill) was encountered within 6 inches to 1.5 feet of the existing ground surface within the two exploration pits excavated. The bridge footings should be embedded in the bearing soils a specified distance to provide erosion scour protection. The soils encountered within the zone of seepage were generally above their optimum moisture content for compaction, thus their reuse as structural fill during all but the driest times of the year will be difficult.

Conventional foundations may be used with normal site preparation procedures where suitable foundation bearing soils are encountered near foundation elevation as anticipated within the vicinity of the bridge. The following report sections present our specific geotechnical site development recommendations.

6.0 SITE PREPARATION

In our opinion, stable construction slopes should be the responsibility of the contractor and should be determined during construction. For estimating purposes, however, we anticipate that temporary, unsupported cut slopes in the existing alluvial stratum can be made at a slope of 1.5H:1V (Horizontal:Vertical). This slope angle assumes that ground water seepage is not strong enough to reduce slope stability and that surface water is not allowed to flow across the temporary slope faces. If ground or surface water is present when the temporary excavation slopes are exposed, flatter slope angles will be required. As is typical with earthwork operations, some sloughing and raveling may occur and cut slopes may have to be adjusted in the field. In addition, WISHA/OSHA regulations should be followed at all times. We should be allowed to review excavation cut slopes greater than 4 feet in height.

The on-site soils below a depth of approximately 5 feet contain a high percentage of fine-grained material, which makes them moisture-sensitive and subject to disturbance when wet. The contractor must use care during site preparation and excavation operations so that the underlying soils are not softened. If disturbance occurs, the softened soils should be removed and the area brought to grade with structural fill.

7.0 STRUCTURAL FILL

Structural fill will be necessary to backfill the foundations and abutments for the bridge. All references to structural fill in this report refer to subgrade preparation, fill type, and placement and compaction of materials as discussed in this section. If a percentage of compaction is specified under another section of this report, the value given in that section should be used.

After stripping, excavation, and any required overexcavation have been performed to the satisfaction of the geotechnical engineer/engineering geologist, the exposed ground should be recompacted to 90 percent of the modified Proctor maximum density using American Society for Testing and Materials (ASTM):D 1557 as the standard or to a firm, unyielding condition. If the subgrade contains too much moisture, adequate recompaction may be difficult or impossible to obtain and should probably not be attempted. In lieu of recompaction, the area to receive fill should be blanketed with washed rock or quarry spalls to act as a capillary break between the new fill and the wet subgrade. Where the exposed ground remains soft and further overexcavation is impractical, placement of an engineering stabilization fabric may be necessary to prevent contamination of the free-draining layer by silt migration from below.

After recompaction of the exposed ground is tested and approved or a free-draining rock course is laid, structural fill may be placed to attain desired grades. Structural fill is defined as non-organic soil, acceptable to the geotechnical engineer, placed in maximum 8-inch loose lifts with each lift being compacted to 95 percent of the modified Proctor maximum density using ASTM:D 1557 as the standard. In the case of roadway and utility trench filling, the backfill should be placed and compacted in accordance with current local or county codes and standards. The top of the compacted fill should extend horizontally outward a minimum distance of 3 feet beyond footings or roadway edges before sloping down at an angle of 2H:1V.

The contractor should note that any proposed fill soils must be evaluated by AESI prior to their use in fills. This would require that we have a sample of the material 72 hours in advance of filling activities to perform a Proctor test and determine its field compaction standard. Soils in which the amount of fine-grained material (smaller than the No. 200 sieve) is greater than approximately 5 percent (measured on the minus No. 4 sieve size) should be considered moisture-sensitive. The siltier materials are acceptable for use as fill provided they are placed and compacted at a moisture content that allows for the minimum specified compaction presented in this report. Reuse of silty soils during wet site or weather conditions is expected to be difficult or impossible due to the high silt content and moisture sensitivity. Use of moisture-sensitive soil in structural fills should be limited to favorable dry weather conditions. A portion of the existing fill soil on-site was observed to have elevated moisture content and is not considered suitable for use in structural fill applications.

Free-draining fill can be used in wetter site and weather conditions under a wider range of moisture conditions than the on-site soils. Free-draining fill consists of non-organic soil with the amount of fine-grained material limited to 5 percent by weight when measured on the minus No. 4 sieve fraction and containing at least 25 percent greater than the No. 4 sieve.

We recommend that during construction, traffic across exposed site soils should be kept to a minimum during and after storm events until the surface drains. When the siltier materials are wet, it may result in disturbance of the otherwise firm stratum requiring removal and replacement of disturbed soils. If wet weather construction is expected, construction access and staging areas should be protected as described in the *Site Preparation* section of this report.

The stripped/overexcavated subgrade should be inspected by a qualified geotechnical professional who should also be present during placement of structural fill to observe the work and perform a representative number of in-place density tests. In this way, the adequacy of the earthwork may be evaluated as filling progresses and any problem areas may be corrected at that time. It is important to understand that taking random compaction tests on a part-time basis will not assure uniformity or acceptable performance of a fill.

8.0 FOUNDATIONS

The bridge foundation should extend below the lowest anticipated erosion/scour depth. Based on our experience with similar sites, a foundation system consisting of a 2-foot-thick (min) compacted quarry spall mat (2- to 4-inch spalls) beneath a conventional footing is recommended. The quarry spall mat should extend beyond the footing a distance of at least 2 feet in all directions. Spread footings may be used for support of the proposed Barnaby Creek Bridge when founded on the recommended rock base placed on suitable medium dense to dense alluvium or structural fill placed as previously discussed. We recommend that an allowable foundation soil bearing pressure of 3,000 pounds per square foot (psf) be utilized for design purposes, including both dead and live loads. An increase of one-third may be used for short-term seismic loading. Footings should be buried at least 2 feet into the surrounding soil for frost protection. All footings must penetrate to the prescribed bearing stratum and no footing should be founded in or above loose or organic soils.

It should be noted that the area bounded by lines extending downward at 1H:1V from any footing must not intersect another footing or intersect a filled area which has not been compacted to at least 95 percent of ASTM:D 1557. In addition, a 1.5H:1V line extending down from any footing must not daylight because sloughing or raveling may eventually undermine the footing. Thus, footings should not be placed near the edge of steps or cuts in the bearing soils.

Anticipated settlement of footings founded as described above should be on the order of ¾ inch or less. However, disturbed soil not removed from footing excavations prior to footing placement could result in increased settlements. All footing areas should be inspected by a qualified geotechnical professional prior to placing concrete to verify that the design bearing capacity of the soils has been attained and that construction conforms to the recommendations contained in this report. The regulatory agencies may also require such inspections.

9.0 LATERAL WALL PRESSURES

Walls that are free to yield laterally at least 0.1 percent of their height should be designed using "active" equivalent fluid pressures. Fully restrained, rigid walls that cannot yield should be designed using "at-rest" equivalent fluid pressures. If any portion of the structure cannot be drained (due to high water elevation), it should be designed for the undrained values presented below. The following Table 1 provides appropriate active, at-rest, and passive equivalent fluid values (and associated base friction coefficients) for the anticipated wall design conditions.

Table 1

Soil Type/ Drainage	Backslope Conditions (Horizontal:Vertical)	Active Equivalent Fluid (pcf)*	At-Rest Equivalent Fluid (pcf)*	Traffic Surcharge Equivalent Soil Height (feet)	Passive Equivalent Fluid (pcf)*	Allowable Friction Coefficient
Medium dense to dense native strata/ Drained	Horizontal	30	50	2	350	0.35
Structural fill/ Drained	Horizontal	35	55	2	300	0.30
Native or structural fill/Undrained	Horizontal	80	90	2	150	0.30

* pcf = pounds per cubic foot

All backfill behind walls or around foundation units should be placed as per our recommendations for structural fill and as described in this section of the report. Where cast-in-place retaining walls face structural fill, the backfill should consist of on-site or imported granular fill compacted to 90 percent of ASTM:D 1557. A higher degree of compaction is not recommended as this will increase the pressure acting on the walls. A lower compaction may result in settlement of pavement or other improvements placed above the walls. Thus, the compaction level is critical and must be tested by a qualified geotechnical representative during placement. Surcharges from adjacent footings, heavy construction equipment, or sloping ground must be added to the above values. Footing drains as discussed under the section on *Drainage Considerations* should be provided for all retaining walls where designed for drained conditions presented in Table 1.

Lateral loads can be resisted by friction between the foundation and the native soil, supporting structural fill, or by passive earth pressure acting on the buried portions of the foundations. The foundations must be backfilled with structural fill and compacted to at least 95 percent of the maximum dry density to achieve the passive resistance shown in Table 1 for footings cast against structural fill. The presented base friction values are allowable and include a safety factor of at least 1.5.

10.0 DRAINAGE CONSIDERATIONS

It is probable that ground water or seepage may be encountered at shallow depths in the foundation excavations. Therefore, the contractor should be prepared to provide temporary water collection, storage, and disposal as necessary prior to site work and during construction. It is anticipated that dewatering of the foundation excavation can be accomplished by pumping from open sump(s).

Where feasible, all retaining walls should be provided with a drain at the base elevation. Drains should consist of rigid, perforated, polyvinyl chloride (PVC) pipe surrounded by washed pea gravel. The level of the perforations in the pipe should be set as low as possible to allow gravity flow, and the drain collectors should be constructed with sufficient gradient to allow gravity discharge away from the structure. Any undrained portion of the wall below the drain elevation should be designed using the undrained parameters as provided in Table 1. All retaining walls should be lined with a minimum, 12-inch-thick, washed gravel blanket provided to within 1 foot of the top of wall and which ties into the footing drain. Surface runoff should not discharge into the footing drain system, but should be handled by a separate, rigid, tightline drain. Grades adjacent to walls should be sloped to promote surface drainage away from the structure.

11.0 PROJECT DESIGN AND RECOMMENDATIONS FOR FURTHER STUDY

We are available to provide additional geotechnical consultation and exploration where the project design changes from that upon which this report is based. We recommend that AESI perform a geotechnical review of the plans prior to final design completion. In this way, our earthwork and foundation recommendations may be properly interpreted and implemented in the design. We are also available to provide geotechnical engineering and monitoring services during construction. The integrity of the foundations depends on proper site preparation and construction procedures. In addition, engineering decisions may have to be made in the field in the event that variations in subsurface conditions become apparent. Construction monitoring services are not part of this current scope of work. If these services are desired, please let us know and we will prepare a cost proposal.

We have enjoyed working with you on this study and are confident that these recommendations will aid in the successful completion of your project. If you should have any questions or require further assistance, please do not hesitate to call.

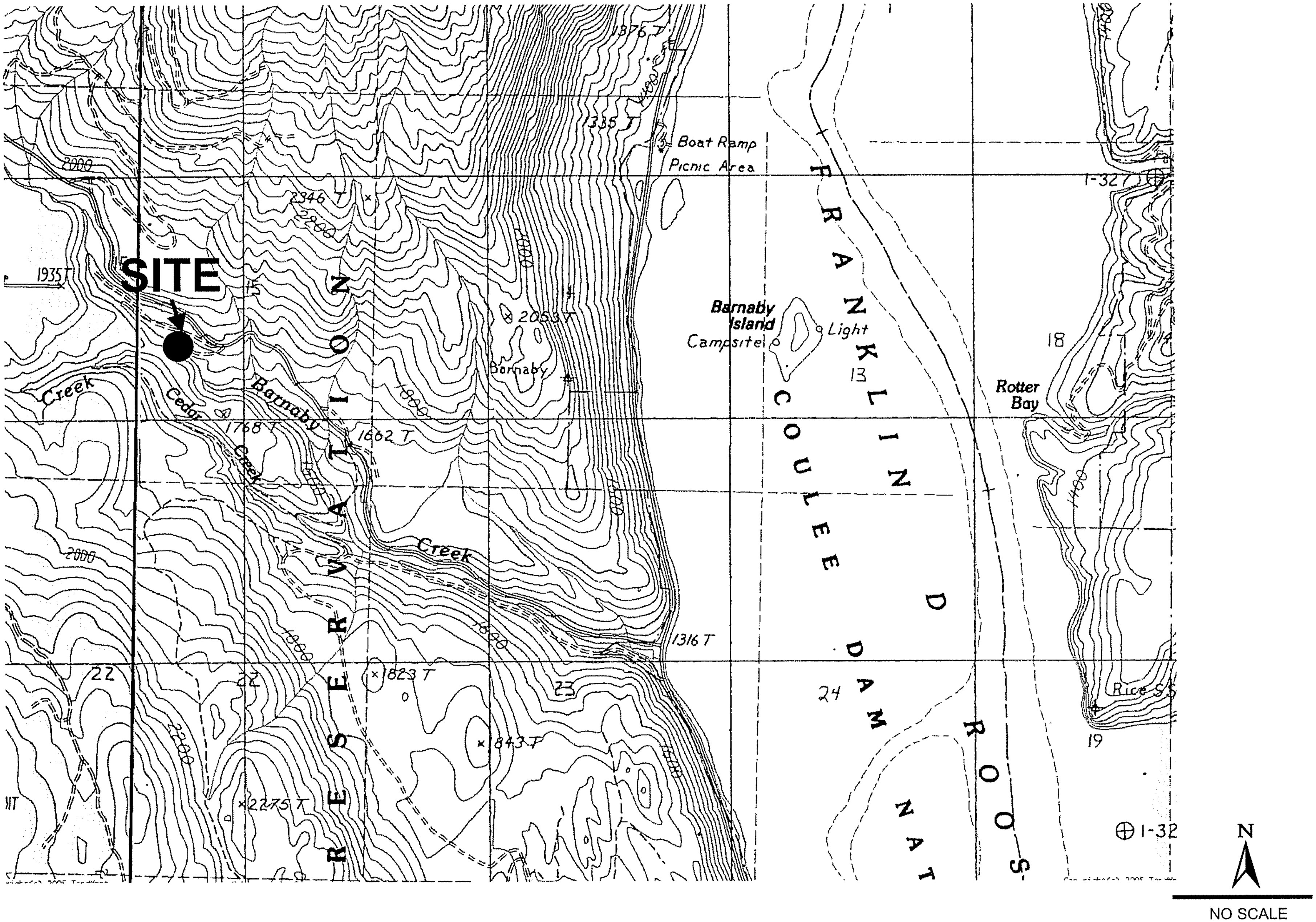
Sincerely,
ASSOCIATED EARTH SCIENCES, INC.
Kirkland, Washington

BRUCE BLYTON
STATE OF WASHINGTON
REGISTERED
PROFESSIONAL ENGINEER
4/14/05
EXPIRES 5/3/06

Maire Thornton, P.E.
Senior Project Engineer

Bruce L. Blyton, P.E.
Principal Engineer

Attachments: Figure 1: Vicinity Map
Appendix A: Exploration Logs
Appendix B: Site Photos



Associated Earth Sciences, Inc.

VICINITY MAP
BARNABY CREEK
FERRY COUNTY, WASHINGTON

FIGURE 1

DATE 04/05

PROJ. NO. KE05150A

05150 Barnaby Creek\05150 Vicinity.cdr

APPENDIX A

Exploration Logs

TEST PIT LOGS

TP-1: Located on south side of creek where center-line of new bridge is located.
Note: depth measurements taken at southern TP wall.

Depth (ft)	Soil Description
0 – 1.0	Loose to Medium Dense, Brown, damp, silty SAND with organics and old wood debris (from old bridge). (FILL)
1.0 – 4.0	Dense, brown, moist, cobbly Sand (med-crse) with trace silt, some boulders.
4.0 – 5.5	Dense to Very Dense, brown, moist to wet, cobbly coarse SAND, with some boulders 1 to 2 feet in diameter (see photo 3).
5.5 – 7.0	Same as above, boulders becoming more abundant. Much harder to excavate.
7.0 – 7.5	Dense to Very Dense, dark gray, damp, SILT with trace gravel and sand.
7.5	Very Dense, dark gray, damp, Silt with some gravel and trace sand (HardPan).

TP-2: Located on north side of existing Bridge approximately 10-feet from structure.
Note: depth measurements taken from west side of exploration.

Depth (ft)	Soil Description
0 – 0.5	Loose to Medium Dense, Brown, damp, silty SAND with (Topsoil)
0.5 – 2.5	Medium Dense, moist, brown SAND with some silt and gravel. Roots extending down to 1.5 feet.
2.5 – 5.0	Medium Dense to dense, wet, dark brown, gravelly SAND, with cobbles. Gravels increase with depth and becomes a sandy Gravel. Water seeping into excavation below 3.5 feet. Sidewalls caving below 4 feet.
5.0 – 5.5	Dense, saturated, sandy GRAVEL with some cobbles.
5.5 – 7.5	Medium dense to dense, moist, tannish SILT with sand and gravel grading to a wet to saturated, brown to tan, dense, silty Sand with gravel and cobbles. Some thin gravelly sand lenses which are more saturated. Excavation becoming "sloppy" and caving below 4- feet once water begins seeping from sidewalls.

APPENDIX B

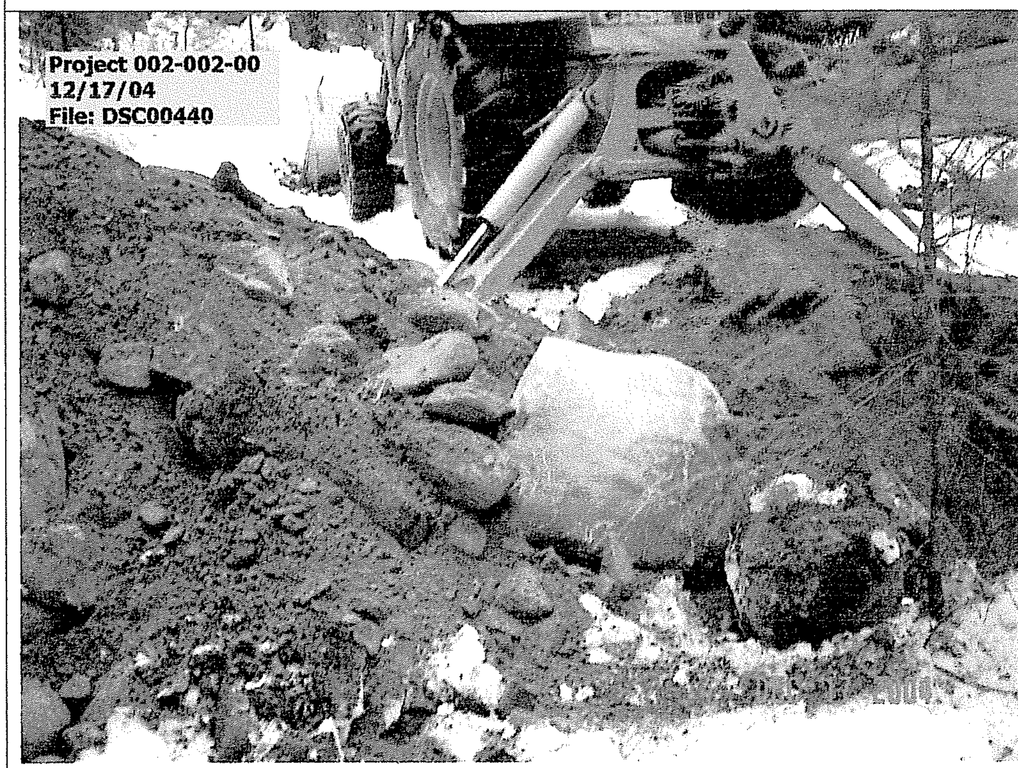
Site Photos

Colville Confederated Tribes
Barnaby Creek Bridge Design
Preliminary Site Visit
Project No. 002-002-00
Photo Log – Test Pit 1

Photograph 1. View looking to southeast from existing bridge. Explorations completed with Case 580 backhoe. Test Pit 1 location.

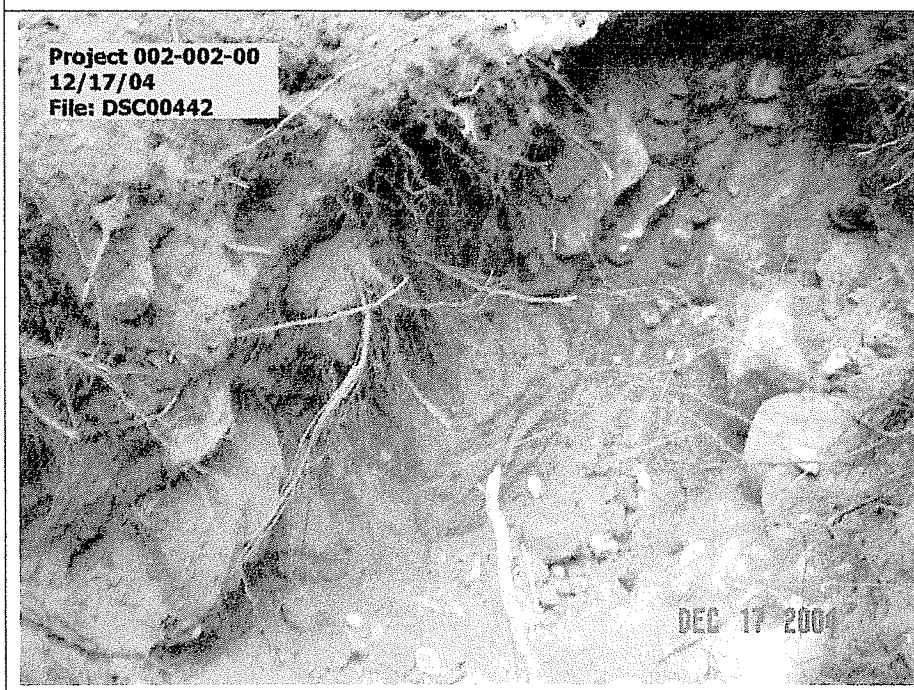
Photograph 2. Excavated material from upper 4 feet of test pit exploration. Soil consisted of a dense, brown, moist, cobbly Sand (med-crse) with trace silt.

Photograph 3. Photo looking at soil excavated from a depth of approximately 4 to 5-1/2 feet in Test Pit 1. Large 1 to 2 foot cobbles became more abundant at approximately 4-feet.

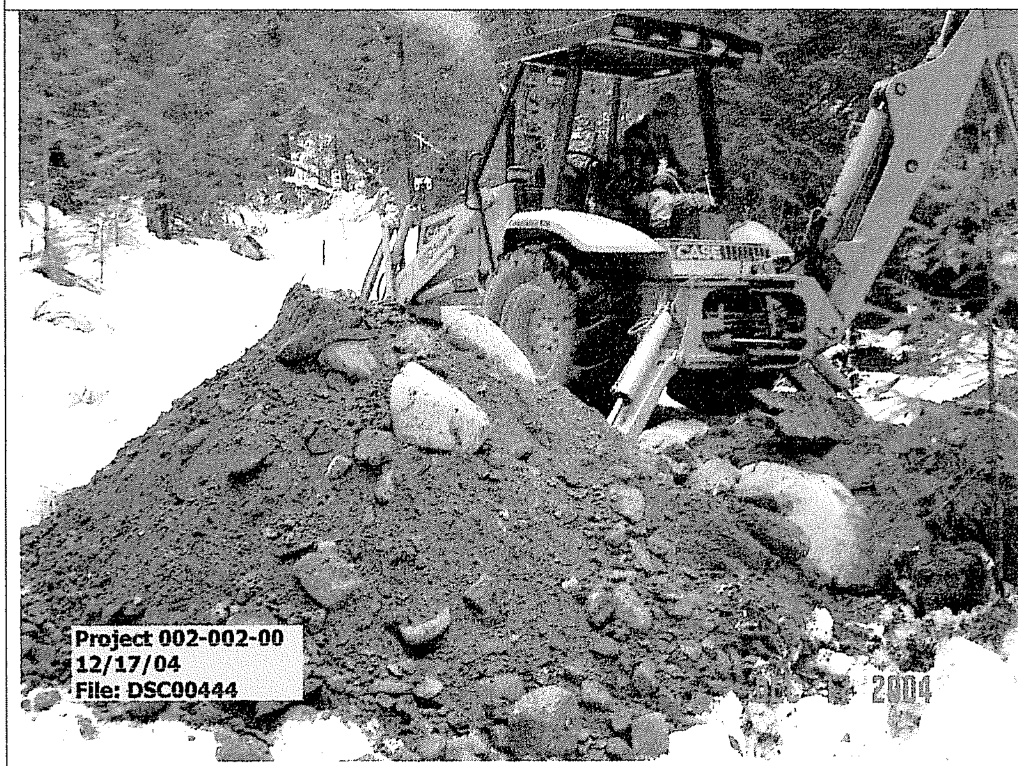


Photograph 4. Photo of boulders encountered at 5-1/2 feet in Test Pit 1.

Photograph 5. Photo looking into Test Pit 1. Front wall is to a depth of 7-feet, at which depth the dark gray hardpan was encountered. Refusal was a t a depth of 7.5 feet below grade in the dark gray, damp, very dense, Silt with some gravel and trace sand. Large boulders are at a depth of 4 to 5.5 feet below grade.



Photograph 6. View of Test Pit 1. Refusal at 7.5-feet below grade.

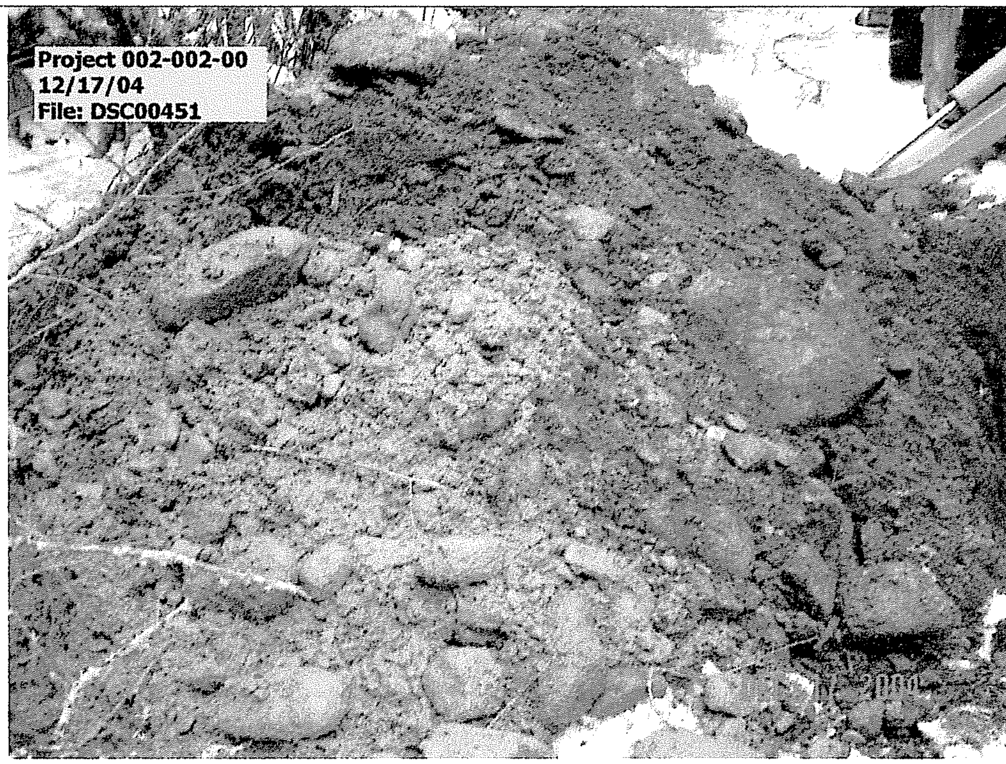
	Photograph 7. View of excavated material from a depth of 5.5 to 7-feet below grade in Test Pit 1.
	Photograph 8. View of excavated material from Test Pit 1.

Colville Confederated Tribes
Barnaby Creek Bridge Design
Preliminary Site Visit
Project No. 002-002-00
Photo Log – Test Pit 2

Photograph 1. View looking into Test Pit 2. Dense medium Sand from 0.5 to 2.5 feet. Dense gravelly Sand with some cobbles from 2.5 to 5.5 feet. Photo is taken at Test Pit depth of 4-feet where small seeps were encountered on left side of Test Pit.

Photograph 2. Excavated material from upper 6 feet of test pit exploration #2. Soil below 4 feet consisted of a Dense gravelly Sand to sandy Gravel with cobbles. Soil became saturated below 5-feet as shown in front left area of photo.

Photograph 3. Photo looking at soil excavated from a depth of approximately 5.5 to 7.5 feet in Test Pit 2. Soil consisted of a wet to saturated, brown to tan, dense, silty Sand with gravel and cobbles.



Photograph 4. Photo of soils encountered at 5.5 to 7.5 feet in Test Pit 2.

Photograph 5. Photo looking into Test Pit 2. Front wall is to a depth of 8.5-feet. Exploration began caving due to saturated soils. Seeps were occurring at approximately 4 feet in left bottom corner of picture. Notice soils below 7 feet are saturated.