PREPARED BY SNOHOMISH COUNTY
DEPARTMENT OF PUBLIC WORKS

GEOTECHNICAL REPORT

JIM CREEK BRIDGE #42

RC #1360 SN 3969
SNOHOMISH COUNTY
DIRECTOR OF PUBLIC WORKS
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EXECUTIVE SUMMARY

Based on our geotechnical investigation, the proposed Jim Creek Bridge #42 replacement project may be constructed as planned, provided that the geotechnical recommendations given in this report are followed. If adjustments or changes to the design are made, a reevaluation of the geotechnical recommendations should be made.

According to preliminary design concepts the proposed new Jim Creek Bridge will be constructed on the existing bridge alignment offset slightly to the east. bridge alignment. The existing bridge will remain in place until half of the new bridge is complete. The new bridge will be about 26 feet longer than the existing bridge, allowing the north and south abutments to be set back farther from Jim Creek. Construction of the new bridge, while maintaining one lane of traffic on the old bridge will require construction staging and shoring or construction of temporary retaining walls. This is necessary to permit excavation of the existing south approach embankment and construction of the footing / stemwall. Permanent retaining walls will need to be constructed along portions of the approach embankments in order to limit impacts on adjacent wetlands, riparian areas and right of way limits.

A brief summary of the geotechnical considerations affecting this project are listed below:

A) Subsurface information at the proposed bridge foundation locations was generated from 2 deep geotechnical borings. Subsurface exploration in the approach and storm water retention system areas was accomplished with a number of hand auger probes.

The shallow subsurface in the south approach area consists of fill up to 15 feet deep overlying till, other glacial sediments or sandstone bedrock. The north approach area is underlain by a few feet of fill overlying thin glacial sediments on top of the sandstone. Alluvial sand and gravel constitute the proposed storm water retention area north of the proposed bridge.

Groundwater is present at several locations across the site, including within the alluvium and glacial sediments.

B) Spread footings bearing on sandstone bedrock are the preferred foundation choice for the Jim Creek Bridge replacement structure.

C) Temporary shoring or the use of temporary structural earth walls (SEWs) will be necessary to maintain traffic at the south approach during the construction of the footing. Traditional shoring methods using sheet piling will not be possible because of the boulders, till and sandstone in the subsurface.

D) Retaining walls or reinforced fill will support the approach embankments in order to limit encroachment on wetlands and to minimize right of way acquisition.

E) Storm water will be treated through biofiltration and infiltrated through the permeable alluvial materials present at the north end of the site.

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1.0 INTRODUCTION

A geotechnical investigation for the Jim Creek Bridge #42 replacement project has been completed. Deep geotechnical borings were advanced at each of the proposed bridge abutments in order to supply subsurface information for foundation design and general earthwork. Preliminary bearing capacities for specified footing dimensions were calculated from the soil strength data but will need to be adjusted if footings of different dimensions are used. Shallow hand auger probes were excavated in areas proposed for stormwater infiltration in order to determine soil texture and influence of groundwater. Geotechnical recommendations for foundation design, stormwater infiltration design and earthwork are included in the following report.

The existing bridge is a 85-foot long, spandrel filled arch constructed of concrete and gravel borrow fill built in 1913. Construction drawings indicate that the arches are founded on bedrock. The bedrock is overlain by 4 to 10 feet of unspecified material. The proposed bridge will be aligned asymmetrically on the existing alignment in order to allow the existing bridge to be utilized while constructing the first lane of the new bridge. The new bridge centerline will be shifted up to 8 feet to the east. The traffic will be shifted to the new single lane bridge to allow the old bridge to be demolished. The second half of the new bridge can then be constructed.

Preliminary pier loading information calculated by Snohomish County's Bridge Group was used to help dimension the foundation types using the Load and Resistance Factor Design (LRFD) method.

The purpose of the geotechnical investigation was to evaluate the existing subsurface conditions near the north and south abutment locations in order to develop preliminary recommendations to be incorporated into the design and construction of the proposed new bridge and associated earthwork. Based on our understanding of the project scope and the results of the preliminary investigation, this report will address the following pertinent design considerations: preliminary foundation design recommendations, alignment preparation recommendations, pavement section, fill placement, erosion control, retaining structures or embankment reinforcement methods, storm water pond design, and seismic considerations.

The scope of our investigation included a visual reconnaissance of the proposed alignment and the adjacent lands, the advancement of two deep geotechnical borings near the proposed abutment locations and the excavation of a series of hand borings along the the approach areas and in prospective storm water infiltration areas. The investigation also included a review of geologic maps, aerial photographs and other pertinent information from our files. The following documents and materials were used in the process of researching this project:

1) Surficial Geologic Map of the Arlington East Quadrangle, Snohomish County, Washington 1985, U.S. Geological Survey MF-1739.

2) Soil Survey of the Snohomish County Area, Washington, U.S. Soil Conservation Service, dated July 1983.

3) Historic Aerial Photography, Snohomish County Department of Public Works, Records Section.

4) The Groundwater System and Ground-water Quality in Western Snohomish County, Washington, USGS Water-Resources Investigations Report 96-4312, 1997

2.0 PROJECT DESCRIPTION

The Jim Creek Bridge #42 replacement project will involve the construction of a new bridge at approximately the same location as the existing bridge. The bridge is located about 2 miles east of Arlington, Washington on Jordan Road. In the County road system, the bridge is at milepost MP 3.4 from the Arlington Heights Road. Jordan Road is an asphalt-paved, 2-lane, rural collector. The bridge crosses Jim Creek several hundred feet upstream from its confluence with the South Fork Stillaguamish River. The project location has been plotted on a vicinity map in Appendix A of this report.

The proposed bridge will be a 112-foot long, single span, pre-stressed concrete bulb-tee girder structure. The bridge will be built to a width of 43 feet in order to accommodate two 12-foot lanes with 8-foot shoulders and railings. The new bridge abutments are planned to extend about

13 feet beyond the existing north and south abutments. The bridge height will be raised by several feet. The approach embankments will be retained or constructed as structural earth walls. New construction will taper back to original at a distance of about 500 feet to the north and 400 feet to the south.

Stormwater treatment and infiltration are planned along the widened approach embankments north and south of the bridge. Native glacially derived outwash soils will accept excess stormwater from the road for infiltration.

3.0 DATA COMPILATION

3.1 Site Description

The proposed bridge replacement project is located in the SE ¼, SE ¼ Section 7, 31 N., R.6 E., Willamette Meridian. The terrain surrounding the bridge is hilly and reflects a glaciated landscape in the Cascade foothills physiographic province. Relief across the entire site is about 45 feet. Slopes are gentle to flat north of the bridge and gentle to moderate south of the bridge. The slopes of the Jim Creek ravine are quite steep, ranging up to 75% and having a relief of 24 feet.

3.2 Geologic Setting

The project area crosses several geologic unit boundaries as the road descends from the till plateau highlands in the south to the Stillaguamish River floodplain to the north. The highlands south of the bridge has a strong northwest to southeast fluted ground fabric indicating the direction of movement of the continental glaciers during the Fraser Glaciation. The geology was mapped by the USGS on the ***Geologic Map of the Arlington East Quadrangle, Snohomish County Washington, MF-1739.*** A representation of that mapping as compiled by the department of Natural Resources is included as Appendix B of this report. Vashon Till (Qvt), comprised of a dense mixture of cobbles, gravel, sand and silt, crops out at the south end of the project. Approaching the ravine the road crosses into the Tertiary Sandstone (Ts). North of the Jim Creek ravine the road crosses the boundary into the Older Alluvium (Qoal) then the Younger Alluvium (Qyal) that forms the flat-lying floodplain. In the proposed location of the new bridge pier, sandstone overlain by a thin till cap can be anticipated.

The Soil Conservation Service in the ***Soil Survey of Snohomish County Area*** has mapped the local soils at the site. The variability of the soil types reflects their geologic origins as glacial or alluvial. The ***Everett Gravelly Sandy Loam*** composes the floodplain north of the bridge. This low-lying unit is excessively drained and highly permeable. Another alluvial or outwash derived soil has been mapped south of the bridge as ***Ragnar Fine Sandy Loam.*** This unit also has high permeability especially below the top 2 feet. Rimming the ravine and in the upland to the south of the project site is the till derived ***Tokul Winston Gravelly Loam.*** This unit is exposed on steep slopes and has moderately low permeability.

3.3 Subsurface Conditions

Deep subsurface conditions near the two proposed bridge abutment locations were explored using a WSDOT-supplied drill rig. The drilling method used mud rotary technology combined with a 4-inch diameter casing advancer. This wire-line capability allowed the tricone bit to be extracted at intervals for standard penetration resistance (SPT) testing and sample collection using 2-inch diameter spilt spoon samplers. Sampling was conducted at 2.5 foot intervals in the top 15 feet of each boring then the interval was lengthened to the standard 5 feet. SPT testing was performed in accordance with ASTM D-1586-84. Upon encountering bedrock the drilling method was switched to diamond coring. Core runs were described and logged in the field and Rock Quality Determinations (RQD) made.

Shallow subsurface conditions in the approach areas were explored by excavating several test pits and attempted hollow-stem auger borings. The borings attempted at the south approach hit refusal on cobbles and boulders at a depth of only a few feet. Backhoe testpits were then utilized to explore the north and south approach areas.

Subsurface conditions for stormwater treatment and control were explored using a hand auger. The device consists of a 3" diameter bucket auger on an extendable rod that is twisted into the ground by hand. Areas proposed for storm water infiltration and retaining walls were explored with the hand auger. This type of exploration has difficulty in large gravel and below the water table and none of the hand augers for this study exceeded 10 feet. More detailed information on the subject of the storm water system is contained in section **4.14 Retention Pond**.

An engineering geologist logged all borings and hand auger explorations in the field. The soils were described using the *Unified Soils Classification System*, a copy of which is included as

Appendix D of this report. Representations of the field logs are presented as Appendix E of this report. Selected samples collected from specified depths were stored in airtight containers and transported to our laboratory for analysis. Sieve analyses was performed on selected samples for the purpose of estimating storm water infiltration rates.

Boring, test pit and probe sites were located in the field using a tape and compass while elevations were interpolated from the site topographic survey. The exploration locations have been plotted on the site plan in Appendix C.

North Abutment - Boring B-1 was located 12' north of the north abutment along the edge of the southbound lane. The boring encountered weathered till consisting of loose to medium dense silty sand with some gravel. Sandstone was intersected at a depth of 9 feet. The sandstone is coarse-grained and somewhat weathered and is interbedded with conglomerate and siltstone. The boring was terminated at a depth of 40 feet. The coarse-grained sandstone and conglomerate rock types predominate through the section explored had much better recovery than the siltstone. RQD for the conglomerate and sandstone range from 50% to 65%. Recovery of siltstone was so poor that no RQD could be assigned to it.

South Abutment - Boring B-2 was located 14 feet south of the south abutment along the edge of the southbound lane. The boring encountered fill consisting of wet, loose, gravelly silty sand overlying medium dense sand at 13 feet. Bedrock was encountered at about 16 feet. Coarse-grained sandstone and conglomerate occur interbedded in similar proportions and the fine-grained siltstone is less abundant. The RQD of the coarse-grained materials ranges from 50% to 98% while no recovery through the siltstone section was made. Boring B-2 was terminated at a depth of 45 feet.

Most of the hand auger probes were advanced in the stormwater management areas. A more in-depth discussion of materials encountered in the probes is presented in section ***4.15 Stormwater Infiltration.*** Probes P-3 and P-4 were advanced through the terrace and were positioned more than 350 feet north of the north bridge abutment. These probes encountered clean sand alluvium beneath a thin silt layer. The static water level in this area is about 9 feet below the surface and was intersected in one of the probes. Probes P-5 and P-6 were advanced farther north in a prospective stormwater infiltration area. Probe P-7 was advanced west of Jordan Road in another prospective stormwater infiltration area.

Hand auger probes P-1 and P-2 were advanced on either side of the road about 200 feet south of the south bridge abutment. Probe P-1 was located on the alluvial plain of the small creek south of Jim Creek and was advanced through 4 feet of silt before encountering sand. The ground water here appears to be under minor artesian pressure. The static water level at P-1 is about 2.5 feet below the surface.

Probe P-2 was situated on the hillside on the opposite side of the road from P-1 and was terminated in very dense till (Qvt) at a depth of 6.9 feet. Minor ground water was perched on the till at a depth of 6.5 feet.

Geologic materials encountered in the borings and probes can be correlated to the material types mapped by the US Geological Survey. The sand encountered in probes P-3 and P-4 are convincingly alluvial in origin and likely is a member of the Older Alluvium unit (Qoal). The fine-grained materials underlying the flat marshy area east of the south approach may be correlative with the Older Alluvium or may be Recessional Outwash (Qvr). Both of those units overlie the till encountered in probe P-2, which in turn overlies the Tertiary Sandstone (Ts) that the deeper borings are terminated in.

A geologic cross-section along the proposed bridge alignment showing the distribution of the geologic units is shown in Appendix F.

3.4 Groundwater

Groundwater was encountered in at least 2 aquifers beneath the site. Groundwater found in the shallow subsurface south of the bridge is perched on the till encountered in probes P-2. It may also be perched on the silt deposit found in probes P-1 during the wet months. The other aquifer is the Older Alluvium found in the northern portion of the project. The alluvium/outwash sand beneath the silt in probe P-1 southeast of the bridge may also be part of this aquifer. At the location of Probe P-1 the groundwater is in a confined coondition under a couple of feet of pressure.

The publication ***The Groundwater System and Ground-water Quality in Western Snohomish County, Washington***, USGS Water-Resources Investigations Report 96-4312, 1997, indicates the lower Jim Creek area as occupying a zone of discharge from the Vashon Advance Outwash aquifer that composes the lower levels of the Arlington Heights Plateau.

More discussion of groundwaterelevations and permeabilities of geologic material is contained in section ***4.15 Storm Water Facilities*** of the ***Geotechnical Recommenadations*** chapter of this report.

4.0 GEOTECHNICAL RECOMMENDATIONS

4.1 Assumptions

The geotechnical recommendations presented in this report are based on the following assumptions and observations:

1) The new bridge will be constructed along the centerline of the existing bridge one lane at a time in order to allow traffic access.
2) The new bridge will be constructed as a single 110-foot span of prestressed concrete bulb tee girders.
3) The proposed bridge will be 42.4 feet wide, about 23 feet wider than the existing bridge.
4) The elevation of the bridge deck will be raised by about 5 feet over the existing bridge deck.
5) Proposed unfactored dead load plus live load is 863.6 kips for each abutment.
6) Stormwater will be treated and infiltrated on-site through an assortment of infiltration galleries and rain gardens.
7) The approaches will be widened to fit the new bridge width extending about 500 to the north and 200 feet to the south of the abutments and the abutments will be shifted 10 feet to the south to make them equidistant to the river.

The following recommendations are based on the above listed assumptions. If changes are made to the assumptions, the Geotechnical Group should be notified so that additional or alternate recommendations can be made.

4.2 Seasonal Working Conditions

Earthwork should be performed during the dry season, otherwise added expense should be anticipated to remove and replace moisture disturbed materials. The chances for successful earthwork will improve late in the dry season as the soil moisture levels decrease. If excessive moisture conditions in the soils develop during earthwork operations, the Geotechnical Group should be consulted for additional recommendations.

4.3 Alignment Preparation

According to preliminary design concepts the proposed new Jim Creek Bridge #42 will be constructed along the existing bridge centerline. The south approach will be constructed along the existing centerline but the north approach will deviate from the existing centerline in order to open up the radii of 2 curves. Additional right of way is required on both sides of both approaches. The proposed abutment locations for the new bridge will be set back from the existing abutments and will shift 10 feet to the south to make them equidistant to the river. Earth fill approach embankments for the south approach will be confined by 4 to 8-foot high retaining walls or structural earth walls (SEW). The existing roadway sections about 500 feet north and 200 feet south of the project will taper to the new bridge approaches. Gravel shoulder added to the taper will graduate to 6-foot wide asphalt shoulders on the approaches and the bridge. Guardrails will be provided where necessary and a curb and bridge railing will be constructed on the bridge.

Preparation of the subgrade should consist of clearing and grubbing of all areas to receive embankment fill. This process should also include removal of unsuitable materials to undisturbed, competent, native material. Hand auger probes in the approach areas did not reveal significant accumulations of unsuitable material, however localized deposits may exist. The existing embankments may remain but the edges of which should be cut back and benched to allow proper compaction of the new embankment fill. Once benched the existing embankment areas should be proof-rolled to reveal any soft areas. Soft areas should be over-excavated and replaced with spalls to establish a stable base then brought to grade using compacted gravel borrow.

The gravel borrow embankment should be constructed in lifts against the benches cut into the existing embankment. If fill slopes steeper than 2:1 (H:V) are necessary additional design work

and the use of geogrid reinforcing will need to be incorporated. For details on fill placement and the recommended material gradation refer to section **4.5 Structural Fill.**

Clearing, grubbing and overexcavation procedures should be performed in accordance with the applicable sections of the *WSDOT Standard Specifications* Section 2-01, Clearing, Grubbing, and Roadside Cleanup. Disposal of unsuitable materials and debris should be in accordance with *WSDOT Standard Specifications* Section 2-03.3(7), Disposal of Surplus Materials and in accordance with the guidelines and specifications in Snohomish County's Special Provision 030307.GR2 *"Disposal of Surplus Materials."*

The existing bridge structure consists of concrete and reinforcing steel. These materials should be salvaged for recycling and not landfilled.

4.4 Cuts and Excavations

Earthwork for the Jim Creek Bridge #42 project will not require large cuts or fills but the widening and raising of the bridge approaches will require some earthwork. Minor excavations will be necessary to create the storm water infiltration facilities at the north and south ends of the project. These excavations are anticipated to encounter silty sand and sand and should be well above the groundwater table. More detailed information on the retention pond is included in section **4.15 Storm Water Facilities**

All cuts should conform to *Washington Department of Labor and Industries Safety Standards for Construction Work*, Chapter 296-155 WAC part N. All temporary cuts greater than 4 feet in height or depth should be laid back to stable inclinations no steeper than 1:1 (H:V). Exposed temporary cut faces should be protected from erosion with plastic. Permanent cuts and fills should be sloped to inclinations of 2:1 (H:V) or flatter and protected from erosion by spreading straw on bare soils and hydroseeding or other approved methods.

In areas where spatial constraints will not allow for stable slope inclinations to be achieved, shoring or retaining structures will be required. Further discussion on the design of retaining structures is presented in section **4.11 Retaining Structures**.

4.5 Structural Fill

Structural fill should be used in any area requiring fill within the roadway or shoulder area. Structural fill should be a well-graded, organic-free, granular, free-draining material. Acceptable material should meet the requirements of *WSDOT Standard Specifications* Section 9-03.14 Gravel Borrow with a slight modification. The gradation should be as follows:

SIEVE SIZE	PERCENT PASSING BY WEIGHT
4 inch (100mm)	100
No.4 (4.75mm)	50 to 80
No.40 (.425mm)	30 max.
No. 200 (.075mm)	5.0 max.

On-site materials may be used for structural fill only if they meet this specification. We do not anticipate that significant volumes of material generated from site cuts will meet the gradation requirements. Some material excavated from the stormwater infiltration facility north of the bridge may be suitable for use as gravel borrow, although the upper materials appear to be mostly sand and silty sand.

All structural fill should be placed in accordance with *WSDOT Standard Specifications* Section 2-03.3(14)B, Earth Embankment Construction and be compacted in accordance with *WSDOT Standard Specifications* Section 2-03.3(14)I Embankments at Bridge and Trestle Ends. Fills planned for sloping areas greater than 5:1 (H:V) should be constructed only on properly benched and keyed soils, as specified in *WSDOT Standard Specifications* Section 2-03.3(14), Embankment Construction. Compaction testing of embankment fill in order to monitor conformance with recommended specifications should be performed in accordance with WSDOT Standard Specification Section 2-03.3(14)D, Compaction and Control Tests.

4.6 Culvert, Storm Sewer and Drainage Placement

All ditches constructed in connection with this project should meet *WSDOT Standard Specifications* Section 2-10, Ditch and Channel Excavation. The ditches should be shaped with 2:1 (H:V) or flatter sidewall slopes and should be hydroseeded soon after completion to prevent erosion. Hydroseeding should comply with applicable sections of *WSDOT Standard Specifications* Section 8-01, Erosion Control. If the ditch soils are not suitable for use on the project, they should be disposed, as described in section **4.3 Alignment Preparation**.

Storm sewer and culvert excavations should be performed in compliance with *WSDOT Standard Specifications* Section 2-09, Structure Excavation. The depth of excavation, removal of unsuitable base materials and disposal of those materials should be performed in accordance with sections 4.3 and 4.4 above.

4.7 Pavement Section

Based on performance of pavement sections on similar soils in the county, we recommend a minimum pavement section consisting of the following:

0.35' HMA CL ½" PG 64-22.
0.50' Asphalt Treated Base or HMA CL 1"

The asphalt concrete should be placed in 2 lifts. Each lift should be a minimum of 0.17 foot thick. The road subgrade should be prepared in accordance with *WSDOT Standard Specifications* Section 2-06, Subgrade Preparation. The sub-base and any necessary embankment fill should be placed and compacted as described in section 4.5 of this report.

4.8 Preliminary Foundation Considerations

Based on our understanding of the project concepts and pier loading information developed by the County's Bridge Group preliminary geotechnical recommendations for the design of the Jim Creek Bridge #42 foundations have been developed. Site conditions and lack of a scour threat favors

shallow spread footing foundations. Deep foundations are much more expensive and are not necessary at this location.

The County, as well as Washinton State Department of Transportation (WSDOT), have chosen to adopt the new Load and Resistance Factor Design (LRFD) specification for new bridge foundation design. The following information is presented in accordance with those design specifications and the stated capacities have not been factored. Instead, the bearing resistance is taken by factors for the strength, service and extreme states to arrive at allowable capacities.

Bridge #42 foundation recommendations were made using the following load assumptions:

Unfactored Pier Loads

Pier	Dead Load	Live Load	Total Load / Pier
Pier 1 or 2	550.0 kips	313.6 kips	863.6 kips

Strength limit state:

Bearing Resistance $Ø_b$=0.45 (footing on rock)

Sliding Resistance $Ø_t$=0.80 (cast concrete on sand)

Passive Earth Pressure component of sliding resistance $Ø_{ep}$=0.5

Service limit state:

Presumptive Bearing Resistance on Weathered Rock = 20 ksf

Extreme limit state:

Eccentricity of loading not to exceed 0.33 of footing dimension, B or L, for γ EQ=0.00

Eccentricity of loading not to exceed 0.40 of footing dimension, B or L, for γ EQ =1.00

There are no materials within the zone of influence of the proposed bridge foundations that are susceptible to liquefaction.

The seismic acceleration coefficient for the area around Jim Creek bridge #42 is 0.33g, according to the USGS seismic acceleration map for Washington State.

footing design and construction

We recommend that the spread footing foundations be placed on the sandstone / conglomerate bedrock beneath the shallow fill and weathered soils. The bedrock will provide adequate support for the footing with a minimal amount of settlement. The rock was rated using the *Rock Mass Rating* (RMR) method, a system that takes into account the compressive strength, fracture density, condition and orientation of the fracture surfaces, and presence of groundwater. The evaluation resulted in an assignment of RMR=71, which classifies as "good rock" in that system. The unconfined compression strength of the sandstone is about 560 psi, based on 2 core samples. The samples had lost some of their initial strength through dessication and possible freezing of the interstitial water before the testing could be performed.

Spread footing foundations should bear on clean sandstone / conglomerate bedrock. The top of the rock surface should be encountered at a depth of about 9 feet at the north pier beneath a veneer of weathered till. The south pier bedrock surface should be encountered at a depth of about 16 feet beneath about 14 feet of embankment fill overlying a few feet of sand. The footing areas should be cut to horizontal surfaces or slightly outward sloping surfaces. The slope cannot exceed 6H:1V. The sedimentary bedrock at this location is poorly indurated and is rippable with an excavator or dozer. If the bedrock slopes steeply benching may be performed to decrease the amount of excavated rock. The streamside edge of the excavation should be keyed 0.5 foot into the sandstone.

Footings bearing on bedrock may be designed using presumptive bearing resistance values at the service limit state, in accordance with AASHTO LRFD Bridge Design Specifications 10.6.2.6. This method is based on geologic knowledge of the foundation rock and site conditions. For weathered or broken bedrock of any kind the recommended bearing resistance is 20ksf. This value cannot exceed the unconfined compressive strength of the local rock or nominal resistance of concrete, which it does not. Another method proposed by Kulhawy and Carter (1988) utilizes

the uniaxial compressive strength of the rock and factors in the effect of fracture spacing and depth. That method results in a allowable bearing pressure of about 40 ksf but because of uncertainties about the fracture density of the sandstone in the foundation area we are recommending staying with the lower pressure value.

Settlement of footings cast on fair to good bedrock can be assumed to be less than 0.5".

wing walls and retaining walls

Foundations for the wingwalls may be designed as spread footings and should bear on firm native soils or well compacted structural fill. Eighteen inches of crushed rock or railroad ballast should be placed beneath the footing areas and extend one foot beyond the edge of the footing. The footings should be designed for a maximum allowable soil bearing capacity of 2000 psf.

Lateral earth pressures of fill material under a static condition behind the walls may be modeled using an equivalent fluid pressure of 30 pcf. WSDOT requires the use of 45 pcf equivalent fluid pressure at the junction of the wall and abutment and extending out the wall a distance equal to the height of the wall. For traffic loads, we recommend that the above earth pressures be increased to reflect an additional 2 feet of surcharge above the highest adjacent grade.

Drainage behind the wing walls must be provided. A minimum of 18 inches of washed drain gravel should be placed directly behind the wall to reduce hydrostatic forces. The drain material should meet the gradation for #67 coarse concrete aggregate, as found in *WSDOT Standard Specifications* 9-03.1(4)C. Alternatively, materials meeting the *WSDOT Standard Specification 9-03.12(4) Gravel Backfill for Drains.* Weep holes or perforated pipe should be provided to allow for drainage from behind the wall. Weep holes should be installed no more than one foot above surrounding ground surface and should be spaced no less than 4 feet apart. If perforated pipe is used, it should be a minimum of 4 inches in diameter.

Placement of the drain material behind the wall should proceed in lifts not to exceed 6 inches in thickness. A temporary physical barrier may be used to facilitate the placement of and provide separation of the drain material and backfill material. Each lift of drain material and backfill material within 3 feet of the back of the wall should be compacted simultaneously. Compactive

effort for this material shall be provided by at least two passes of a vibratory compactor approved by the engineer.

The overall stability of the wall should be checked using a rcsistance factor Φ=0.75.

4.9 Rockeries

If rockeries are chosen for use on this site they should be designed and constructed in accordance with the guidelines published by Associated Rockery Contractors. We normally recommend a maximum rockery height of 6 feet against a stable cut slope and 4 feet against an embankment without the use of reinforcing materials. Rockeries should be considered as cosmetic or for erosion control purposes and are not intended to perform as retaining structures. The rockery should be constructed on firm native soils, compacted structural fill or a one foot thick mat of quarry spalls.

4.10 Retaining Structures

Retaining walls or reinforced fill slopes may be desirable to support the approach embankments of the proposed bridge in order to reduce impact on riparian zone buffers or to keep fill slopes within the right of way. The design report indicates that the approaches will be raised about 5 feet in order to meet the elevated deck of the proposed bridge. The southwest approach embankment may be provided with 3:1 (H:V) sideslopes in order to allow stormwater filtering and infiltration. A creek crossing near Sta. 6+70 may necessitate construction of headwalls for the cross culvert in order to maintain the existing culvert length and to reduce impact on the attendant riparian zone buffer.

We suggest utilizing vegetated-face structural earth walls (SEWs) or steep, vegetated reinforced slopes along the segments of the approach embankments where the footprint needs to be narrowed. These walls are economical and can withstand some settlement without affecting their structural integrity. Alternatively, the SEW amy be faced with a Lock-Block or similar concrete block in order to provide a hard surfaced wall.

All retaining structures should bear on firm native soils. Subsurface exploration revealed the existence of fill materials at the bridge approaches. The fill depth at the south approach is up too

15 feet deep but shallows rapidly towards the south. For SEWs and reinforced fill slopes the existing embankment should be excavated to native material within the required embedment length of the geogrid. The old embankment should be cut to horizontal benches to permit proper placement and compaction of the new structural fill of the SEW.

The reinforcing mechanism for SEWs and reinforced slopes is provided by uniaxial geogrid mats between compacted structural fill layers. The strength of the geogrid and embedment lengths are dependent on soil strength height and surcharge pressures. For this project we evaluated wall heights of 6, 8, 10 and 12 feet. The design parameters used in the MSE wall design included a 10:1 (V:H) batter angle on a Lock-Block or equivalent face, level backfill, 125 psf traffic load, and backfill meeting the requirements in section ***4.5 Structural Fill*** of this report.

Construction of the wall after base preparation should begin with the placement of the first layer of geogrid. Grids should always be placed with the machine direction perpendicular to the wall face. The bottom course of concrete blocks should be keyed half a foot below adjacent outside grade and on top of the grid. The geogrid embedment length and strength are listed on the table below. For simplicity the contractor may choose to use the heavy (LTDS=1457) geogrid for walls 9 to 12 feet high and mid-weight grid (LTDS=927) for walls 5 to 8 feet high. Geogrids should be placed at the base of every block for a vertical spacing of 2.5 feet. No intermediated grids are necessary for this wall design. Geogrids should overlap each other by 1 foot along their lengths.

The geogrid reinforcement shall be sized and spaced according to the following schedule:

Wall Height	Geogrid Strength T_{al} (lb/ft)	Geogrid Length (measured in feet from back of wall)	Number of Layers
6	795	8	3
8	927	8	4
10	1127	8.5	4
12	1457	8.75	5

Manufacturer supplied recommendations and requirements pertaining to geogrid handling and installation shall be adhered to by the contractor.

4.11 Erosion Control

Suitable erosion control should be installed and maintained during earthwork operations on the site, particularly because of the site's proximity to Jim Creek and the South Fork Stillaguamish River. **Best Management Practices**, as outlined in the Washington Department of Ecology's *Storm water Management Manual for the Puget Sound Basin* should be applied. Erosion control should also comply with *WSDOT Standard Specifications* Section 8-01 Erosion Control. In addition, drainage ways should be protected from siltation by filter fences, straw bale filters or settling ponds. Runoff should not be allowed to flow uncontrolled over site slopes. Permanent erosion control should be installed as soon as possible after completion of work in the area. Hydroseeding and placement of excelsior or jute matting should be performed in accordance with methods described in the above referenced standard specification.

4.12 Liquefaction Potential

Liquefaction refers to a phenomenon in which a saturated, cohesionless soil loses strength during an earthquake allowing it to move vertically and/or laterally. Obviously, structures founded on liquefying soils could be damaged or displaced during an earthquake. The potential for liquefaction is dependent of several factors:

seismic- intensity and duration of ground motion
soil moisture- only saturated soils will liquefy
soil grain size- fine sands and coarse silts are most prone to liquefaction
relative density- liquefiable materials must be loose enough to allow the movement or rearrangement of grains

The Jim Creek Bridge #42 is located in seismically susceptible region with historic occurrences of earthquakes that have resulted in localized areas of liquefaction. Most instances of documented liquefaction in the region occurred in river valleys where normally consolidated alluvial soils and high groundwater table were present. As previously stated the foundation areas of the proposed Jim Creek Bridge #42 are outside of the area where liquefaction can be expected. The bridge footings will bear on sandstone bedrock. The alluvial sand north of the bridge and including the

stromwater infiltration area is moderately susceptible to liquefaction during strong shaking events. The groundwater in this area is about 9 feet below the surface and occupies a coarse sand and gravel deposit. Gravel is normally not expected to liquefy during seismic events. Interbedded finer sands and silt may liquefy and cause local areas of minor subsidence.

4.13 Seismic Considerations

We understand that the seismic design of this bridge is to be performed in accordance with the ***AASHTO LRFD Bridge Design Specifications 4th*** Edition 2008. The seismic specifications were developed based on the principles that small to moderate earthquakes should be resisted within the elastic range of the structure without significant damage. Large earthquakes should not result in the collapse of the structure and damage that does occur should be readily detectable and accessible for inspection and repair. The AASHTO specifications require geotechnical input for the following criteria:

- *Selection of an earthquake acceleration coefficient(A).*
- *Determination of a site coefficient (S) based on a generalized soil profile type.*
- *Addressing the liquefaction potential of the foundation bearing soil.*
- *Development of foundation and abutment criteria.*
- *Determination of the proximity of USGS active faults to the site and site specific analysis if faults lie closer than 6 miles.*

Revised AASHTO design criteria for bridge design were adopted by WSDOT in the 2008 Bridge Design Manual. The recommendations were based on the USGS 2002 probabilistic ground motion analysis that predicts peak ground accelerations (PGA) for the region. The predictions consider recurrence interval and percent chance of recurrence. AASHTO and WSDOT have selected a 7% probability of exceedance in 75 years, or about a 1000 year recurrence interval for non-critical structures. New seismic acceleration maps and software developed by the USGS provide design accelerations for locations and coordinates. Interactive maps and detailed regional seismic data are available at http://earthquake.usgs.gov/hazmaps/. Inputting the longitude for the site (-122.07) and the latitude (48.18), the USGS ground motion prediction resulted in a PGA= 0.33g.

The soil type and depth to bedrock at a particular site affects the earthquake related motion characteristics of damping. AASHTO specifications account for the influence of site specific soils by assigning a site coefficient soil profile type. We recommend that the site be classified as soil profile type 1. Soil profile type 1 is assigned to sites underlain by shallow soils overlying bedrock. Our subsurface exploration did encounter bedrock at depths of 9 and 16 feet at the north and south abutment locations, respectively and will provide support for the bridge foundations. Soil type I corresponds to a site coefficient S = 1.0. These coefficients are independent of structure type and are considered to be appropriate for bridges as well as other structures.

Liquefaction potential of the site soils was previously discussed in section 4.12.

Proximity to mapped active faults is also a consideration of the AASHTO specifications. If an active fault mapped by the USGS is located within 6 miles of the site, a site specific analysis should be performed. The Jim Creek Bridge #42 does not have any mapped faults within this range. The nearest mapped fault of concern is the Devils Mountain fault about 6 miles northeast of the site. This fault is regionally extensive and has been traced for about 74 miles but the slip rate is less than 0.2 mm/yr. Significant deformation associated with this fault occurred around 130,000 years ago and may not be considered truly active. The Strawberry Point and Utsalady faults are located about 12 miles northwest of the site and cross Whidbey Island. These faults have slip rates of about .2 to 1 mm/yr. Strawberry Point fault deformation dates back to <130,000 years and Utsalady is <15,000 years. The Southern Whidbey fault is located about 12.5 miles south of the site and has a traceable length of about 38 miles. The slip rate on this fault is about 0.2 to1.0 mm/yr. with deformation <15,000 years. We believe that the potential for deformation or ground rupture at the Jim Creek bridge site due to earthquakes along any of the listed faults is low to moderate because of the distance from the faults, the history of movement along those faults and the relatively high strength of the foundation materials.

Lateral earth pressures on walls increase during an earthquake. The design method presented in the appendix of section 11 of the ***AASHTO LRFD Bridge Design Specifications 4th*** Edition for yielding and non-yielding walls is based on the Mononobe - Okabe analysis. This method assumes that the wall is free to yield outwardly at the top equal to an amount of 10A in inches, where"A" is the acceleration coefficient. As previously stated, this value is equal to 0.33 for the this particular location. This means that the wall should be allowed to yield 3.3 inches. Allowing the wall to yield significantly reduces the dynamic lateral loading as compared to a wall where no displacement is permitted.

For a yielding wall, the horizontal acceleration coefficient (K_h) used in the analysis is 0.5A. Under these conditions we recommend that a dynamic rectangular pressure increment be added to the static equivalent fluid pressure stated in section 4.8 in the form of 4H in psf, where H is the height of the wall in feet.

A non-yielding wall or a wall constrained by its stiffness or other structural constraints, will experience significantly higher lateral earth pressures during an earthquake. The horizontal acceleration coefficient (K_h) for non-yielding conditions is 1.5A. Under these conditions we recommend that a dynamic rectangular pressure increment be added to the static equivalent fluid pressure in the form of 12H in psf.

4.14 Storm Water Facilities

The subsurface conditions in the proposed storm water infiltration areas were explored with hand auger probes. A number of samples were collected at various depths and submitted to the laboratory for gradation analysis. The grain size distributions can be used to size the infiltration ponds and design water quality treatment. The grain size analyses are contained in Appendix G of this report.

south approach

The proposed infiltration areas along the east and west sides of the southern bridge approach are underlain by fine-grained materials not suited for disposal of large volumes of storm water. The geologic units anticipated in this area, as mapped by the US Geological Survey are the Vashon Till (Qvt)overlying the Tertiary Sedimentary bedrock (Ts). The Soil Conservation Service (SCS) includes this area as the Ragnar Fine Sandy Loam (58) and the Tokul Winston Gravelly Loam (77) soil types.

Probe P-1 was advanced at the east toe of the embankment about 32' right of Sta. 5+80. It encountered 4 feet of silt overlying wet sand. Ground water was about 2.5 feet below ground surface in April 2007 and in a confined condition. This location is adjacent to a low wet area that bounds a small creek. The origin of this material is probably alluvial and may represent a fine-

grained facies of the Older Alluvium (Qoal) that is mapped to the southwest and northeast of this location.

Probe P-2 was advanced about 21' left of Sta. 6+00. It encountered forest duff, topsoil and about 3 feet of silty sand before encountering iron-stained, fine-grained, silty sand. Dense, gravelly, silty sand of the Vashon Till (Qvt) geologic unit was intersected at about 6.5 feet. Ground water was perched on the till and had a static level about 6.5 feet below ground surface in April 2007.

Soil samples from the south approach probes were not submitted for grain size analysis because the D10 particle size is smaller than that allowed by the DOE 2005 ***Storm water Management Manual for Western Washington*** for infiltration rates based on textural classification. A sample of fine-grained, silty sand collected at a depth of 4.5 feet from boring B-2 had 45% passing the #200 screen.

north approach

Materials underlying the north approach area are clearly of a different origin than the ones of the south approach and are much better suited for storm water infiltration. The geologic unit mapped in this area by the US Geological Survey is the Older Alluvium (Qoal). The SCS has identified the Tokul Winton Gravelly Loam (77) along the north bank of Jim Creek and the Everett Gravelly Sandy Loam (17) to the north of that. The proposed retention facility is located on a broad alluvial terrace landform with very little relief.

The proposed infiltration pond site east of Jordan Road between Sta. 12+75 and 14+75 is suitable for the proposed infiltration pond. Data generated from handauger probes P-3, P-3A and P-4 indicate that the shallow subsurface is composed of stratified silty sand with higher permeability sand and gravel occurring at greater depths. Similar conditions favorable for infiltration are found along the west side of Jordan Road adjacent to the proposed pond site, as indicated by the data from handauger probe P-7. The watertable elevation in this area is about Elev. 96, while the ground elevations range between Elev. 106 and 104. We recommend that the interior pond slopes be constructed to 3:1 (H:V) inclinations. The pond bottom should be established at Elev. 101 or higher in order to maintain a 5-foot separation from the watertable. The target strata for the pond bottom is clean fine- or medium-grained sand. If the silty sand bed described in probe P-3 is exposed on the floor, it should be over-excavated and replaced with cleaner sand excavated from the northern portion of the pond.

An alternate infiltration site explored at the north end of the same parcel revealed conditions not well suited to infiltration. Hand auger probes P-5 and P-6 encountered quite permeable sand and gravel but also intersected the watertable at a shallow depth. The watertable at this location is at about Elev. 99, leaving only about 4 feet of separation between it and the ground surface.

Probe P-3 was advanced about 35' right of Sta. 10+90 in the southern region of the proposed retention facility at the south end of the Soper parcel. The probe encountered silty sand and fine- to medium-grained sand with minor interbeds of silt in the top 6 feet. The texture coarsened wih depth to clean medium- to coarse-grained sand becoming gravelly to a depth of 10 feet. Ground water stabilized at a depth of 9 feet in April 2007. Probe 3A was located at nearly the same location in May 2008 and was terminated in wet gravel at a depth of 9.7 feet and still above the watertable. It did not encounter the silty sand found in probe P-3. The wet season watertable elevation can be assumed to be Elev. 96.5 at this location.

Probe P-4 was located near the north end of the proposed retention pond at the south end of the Soper parcel about 26' right of Sta. 12+75. This exploration also intersected silty sand to a depth of 3.5 feet then fine- to medium-grained sand. Clean medium- to coarse-grained sand beds were intersected at 5 feet and continued to 8.3 feet where large gravel stopped the auger. No ground water was encountered in this probe but moisture levels were very moist at the bottom of the hole.

Probes P-5 and P-6 were located in an alternate proposed pond site at the north end of the Soper parcel in May 2008. These probes were advanced towards the south and north ends of the alternate pond location, respectively. P-5 encountered shallow groundwater in stratified sand and gravel a few feet below the surface at about Elev. 99. Probe P-6 encountered similar stratified sand and gravel deposits but was terminated on large gravel at a depth of about 3.7 feet before the watertable was intersected.

Probe P-7 was located on the Gardner parcel west of Jordan Road approximately adjacent to the original proposed pond location. This location was explored as another alternate pond site. The probe was advanced through silty sand in the top few feet before crossing into fine-grained silty sand and sand to a depth of about 10 feet. Iron-stained gravel was encountered at 10 feet causing a termination of the probe. The watertable was not intersected at this location but it is lower than Elev. 96.

Samples were collected at several intervals in probes P-3 and P-4 and submitted for grain size analysis. In general, the sand textures become coarser-grained below depths of 3 to 6 feet and will have higher infiltration rates than the finer-grained counterparts at shallower depths. The grain size analyses have been plotted and are included in Appendix G of this report. Correspondence regarding stomwater information is included in Appendix H.

The DOE 2005 ***Storm water Management Manual for Western Washington*** allows estimation of infiltration rates based on textural analysis. Long-term infiltration rates presented in V.III table 3.8 are based on the D_{10} of ASTM gradations. The following table presents the long-term infiltration rates of materials sampled at various depths based on their D_{10} from the grain size analysis.

Estimated Long-term Infiltration Rates*

Exploration # / approach	Material	Depth	Infiltration Rate
P-3	Silty fine **SAND**	1.3'	NA
P-3	Fine **SAND** with silt	4.0'	1.5"/hr.
P-3	Fine **SAND**	6.9'	2.5"/hr.
P-3	Medium **SAND** with gravel	9.5'	8.3"/hr.
P-4	Fine silty **SAND**	4'	0.8"/hr.
P-4	Medium **SAND** with gravel	6'	7.2"/hr.
B-2	Silty fine **SAND**	4'	NA

* DOE 2005 ***Storm water Management Manual for Western Washington*** V.III Table 3.8

The manual requires that for design long-term infiltration rates in stratified deposits the rate for the slowest infiltrating material be used. If higher rates are needed, excavation to lower higher permeability strata can be performed and the pond bottom backfilled with pea gravel or other free-draining material. The higher permeability materials occur below a depth of about 7 feet.

For sufficient treatment of pollutants in the runoff the cation exchange capacity (CEC) of the soil should exceed 5 milliequivalents/ 100g. This capacity is available in loamy sands having long-

term infiltration rates lower than 2"/hr. The materials in the shallow portions (<6') of the infiltration area and along the ditch bottoms have rates less than 2.0"/hr. and should adequately perform the treatment function. Because these materials do not extend to the minimum of 6 feet below the bottom of the infiltration pond, additional treatment should be provided. This can be accomplished by routing the runoff through a biofiltration swale lined with a soil/ compost blend upstream of the pond. Alternatively, the pond bottom can be over-excavated to Elev. 98 and 2 feet of amended soil placed as a liner. The amended soil shoud be compacted to 85-90%. The engineered soil should have a compost content of 30% and CEC of 5 milliequivalents/ 100g, verified by a testing laboratory. The The amended soil meeting the proposed gradation and mixture will have an infiltration rate of 2"/hr. and the pond size will need to be adjusted accordingly. The gradation for engineered amended soils should meet the following specification:

Engineered Amended Soil Gradation

SIEVE SIZE	PERCENT PASSING BY WEIGHT
4 inch (100mm)	100
No.4 (4.75mm)	50 to 80
No.40 (.425mm)	30 max.
No. 200 (.075mm)	5.0 max.

Materials excavated from depths 6 to 9 feet in the proposed pond area will likely meet this gradation and can be used if blended with a sufficient amount of compost.

In summary, subsurface conditions in the south approach area are not conducive to infiltrate large volumes of stormwater. The soils here are till-derived or otherwise too fine-grained to provide rapid infiltration. Conversely, the geologic materials north of the north approach were deposited in a fluvial environment are quite free-draining below a depth of about 7 feet. The pond should be lined with a 2-foot depth of amended soils for treatment purposes. Infiltration rates throught prepared pond bottom will be 2"/hr. No sole source aquifer or well head protection area is located within the project boundaries.

If the location of the proposed infiltration facilities changes from the limits stated during the preliminary design we should perform more subsurface investigation and grain size analyses in the new locations. We can perform more detailed analyses and small scale infiltration testing later in the design phase if necessary.

4.15 Approach Embankment Shoring

In order to provide access to traffic during the construction of the new bridge, careful planning of the foundation excavation and construction will be necessary. The existing embankment will need to be supported and probably widened in order to allow a single lane of traffic. Cuts up to 15 feet high into the side of the existing approach embankment will be necessary in order to construct the first half of the the new south pier. The construction contractor will be responsible for designing a method to support a portion of the existing embankment or otherwise provide access to traffic.

Subsurface conditions at the south approach were explored with back hoe test pits and attempted auger borings. The borings hit refusal at depths less than 5 feet in large cobbles and boulders. Test pit TP-1 was excavated along the western side of the south approach and encountered cobbly gravel with boulders beneath several feet of fill. Heavy groundwater seepage and subsequent caving at 5.5 feet forced the abandonment of the test pit at about 6.5 feet. Test pit TP-2 was excavated on the opposite side of the road and encountered 5 feet of sandy fill overlying about 1.5 feet of cobbles and boulders then very dense till. Moderate seepage in test pit TP-2 was noted at a depth of 5 feet. Although it was not encountered in the test pits, the sandstone bedrock lies beneath the till.

The north approach area was explored with a test pit located northwest of the existing northwest corner of the north bridge abutment. Test pit TP-3 penetrated about 6 feet of silty sand and a foot of gravel before encountering sandstone. Light seepage was noted at 3 feet.

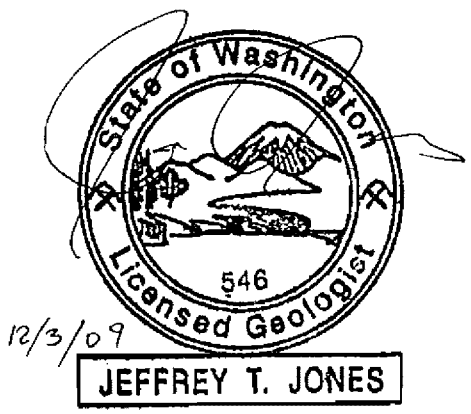
Shoring designs to support a single lane of the approach embankment should consider the geologic conditions present at this site. Driven piling that requires penetration into the till or sandstone will not be possible. H-pile designs will require that holes be drilled and supported during the placement of the piles and concrete. Heavy seepage may be encountered in excavations down to the top of the till or sandstone surface and may merit localized dewatering.

LIMITATIONS

The following special notes are recommended to be included in the contract documents:

1. The work performed and presented herein has been conducted using generally accepted engineering practices. This information is intended for the use of Snohomish County and its designated consultants. Its presentation in the plans or elsewhere is for the purpose of providing intended users access to the same information available to the County and is not intended as a substitute for personal investigation, independent interpretation or judgment by the contractor.

2. The observed soil conditions described in this study are as observed at the time of the field exploration. Actual soil conditions encountered at the time of construction may vary from those described in this report.



Jeffrey T. Jones
Engineering Geologist

Dale E. Topham, P.E.
Geotechnical Engineer

December 3, 2009

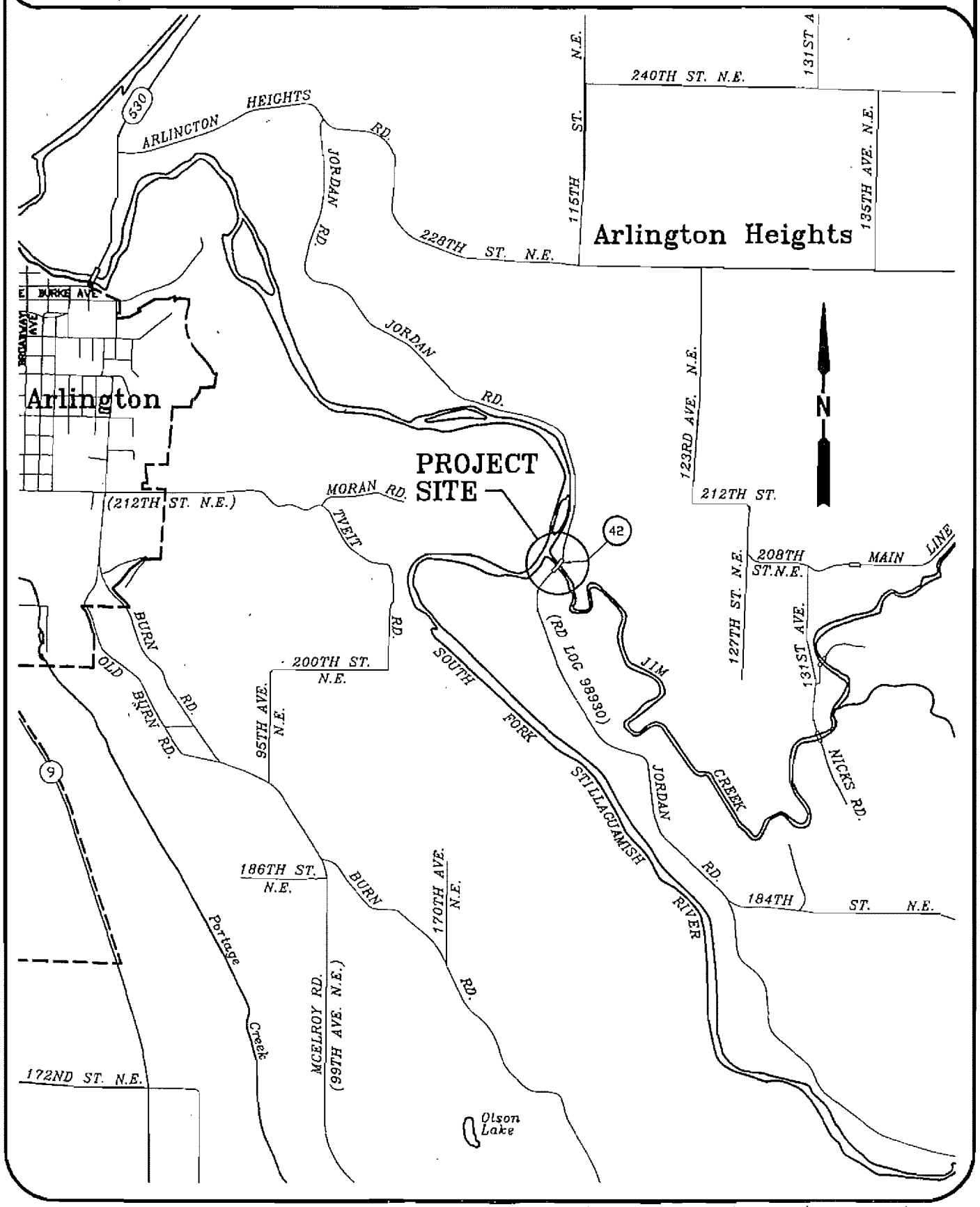
appendix a

VICINITY MAP

VICINITY MAP

JIM CREEK BRIDGE #42 JORDAN ROAD
RC #1360

SNOHOMISH COUNTY PUBLIC WORKS



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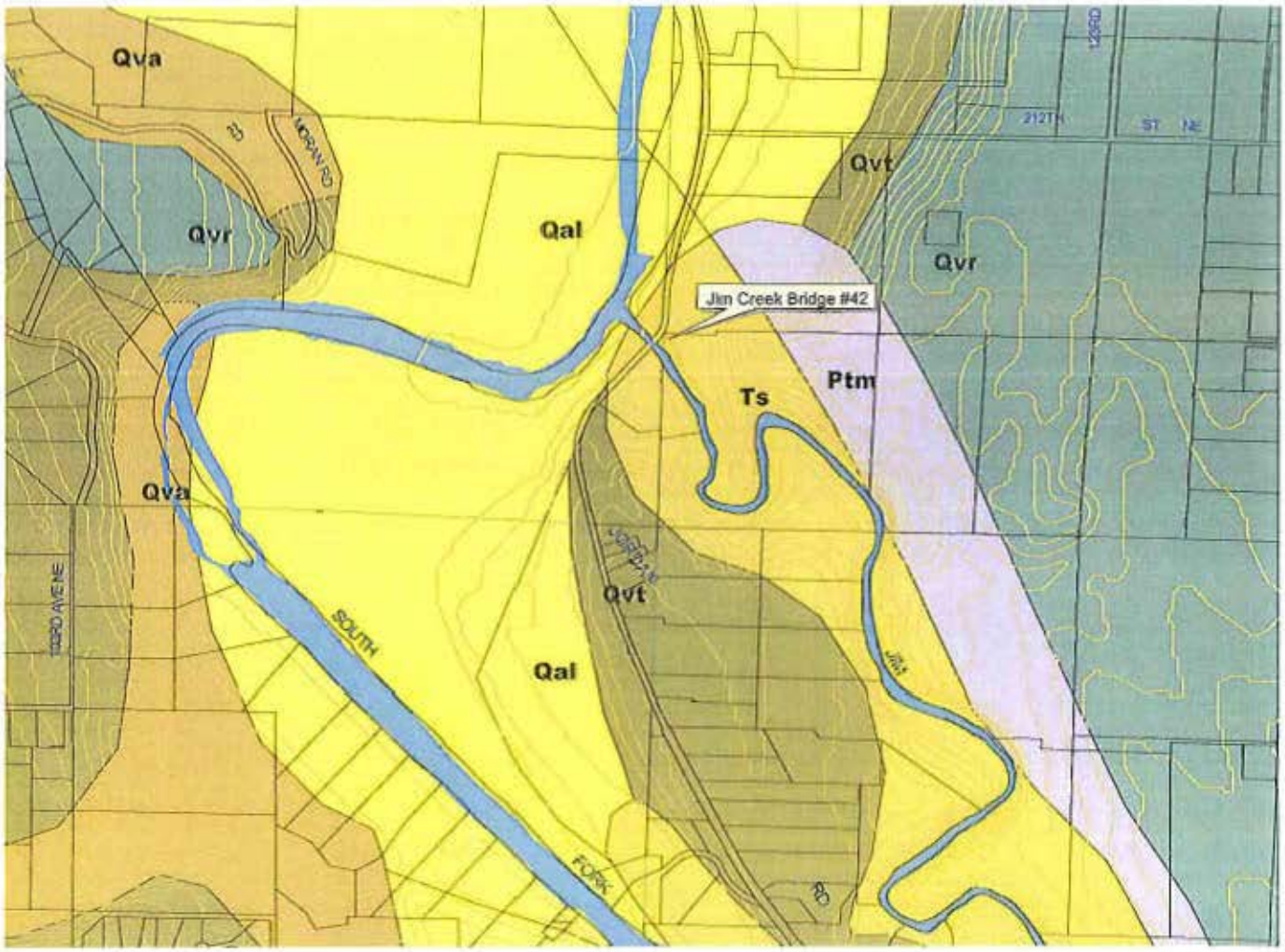
appendix b

GEOLOGIC MAP

GEOLOGIC MAP

JIM CREEK BRIDGE #42



0.4 0 0.4 0.8 Miles

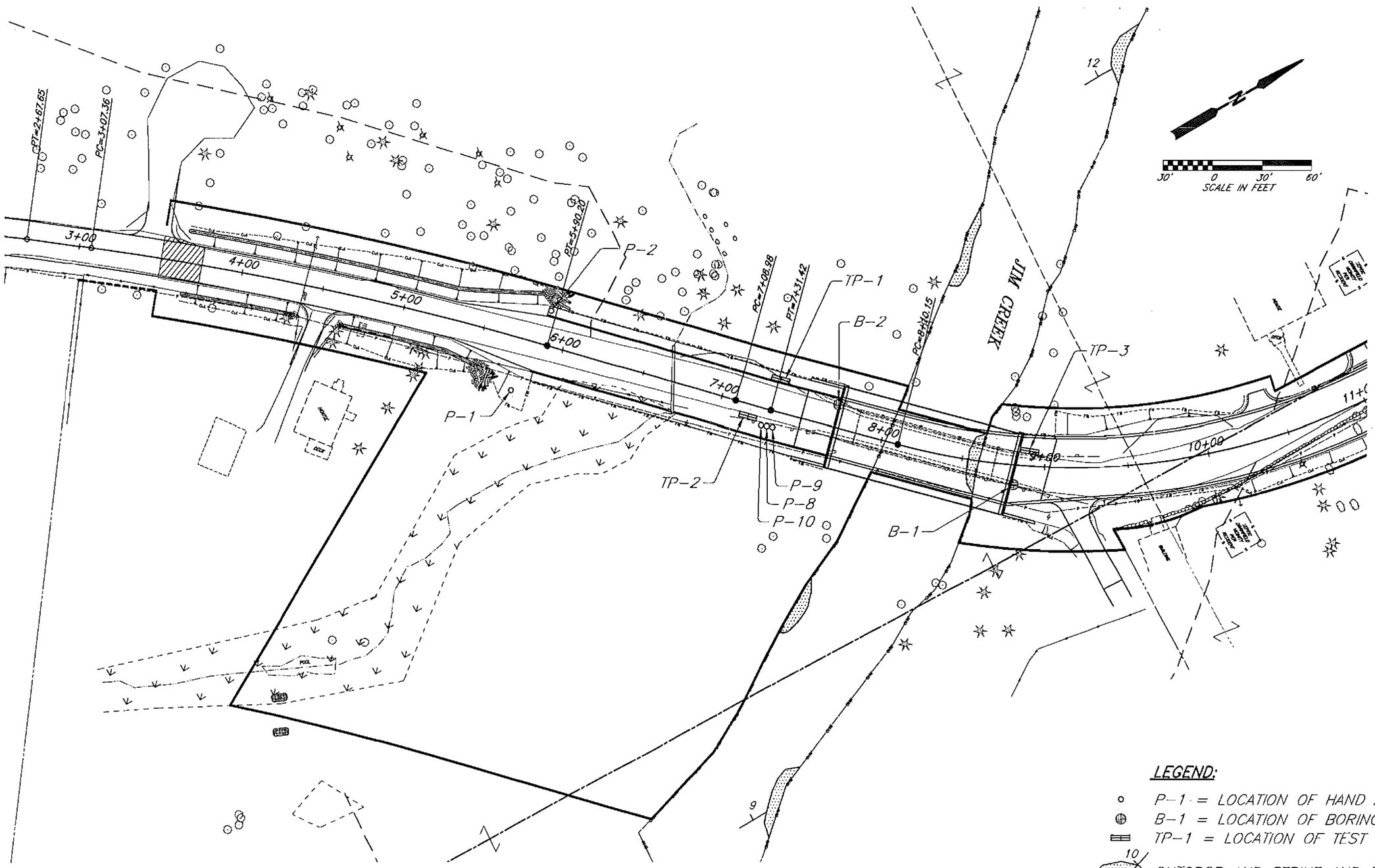
- Parcels (Assessor)
- 20 ft. intervalContours.shp
- Sections
- County.shp
- Townships
- Text Road Names (~11K Scale)

Majorwtrbdy.shp

- Bays
- Flats (tidal
- Lake or pond
- Sand or gravel in open water
- Stream
- Cities.shp

appendix c

SITE PLAN



LEGEND:

- P-1 = LOCATION OF HAND AUGER PROBE
- B-1 = LOCATION OF BORING
- TP-1 = LOCATION OF TEST PIT
- 10 OUTCROP AND STRIKE AND DIP OF BEDDING

PLAN CHECK	BY	DATE					
			DATE	NO.	REVISION	BY	

Last Saved By: spwmfh Aug 11, 2009 – 1:07pm

REGION NO.	STATE	FED. AID PROJ. NO.	SURVEY NO.
10	WASH.		3969 06-0000

DESIGNED BY: MMA | DRAWN BY: MFH

FIELD BOOK(S): | UPI#

APPROVED FOR CONSTRUCTION

OWEN B. CARTER, P.E.
SNOHOMISH COUNTY ENGINEER

DATE APPROVED:

SNOHOMISH COUNTY DEPARTMENT OF PUBLIC WORKS

FUNDING NO. RC #7333 RC #1360

JIM CREEK BRIDGE #42 REPLACEMENT

GEOTECHNICAL REPORT MAP

REFERENCE SHEET NO.

SHEET 1 OF 2 SHEETS

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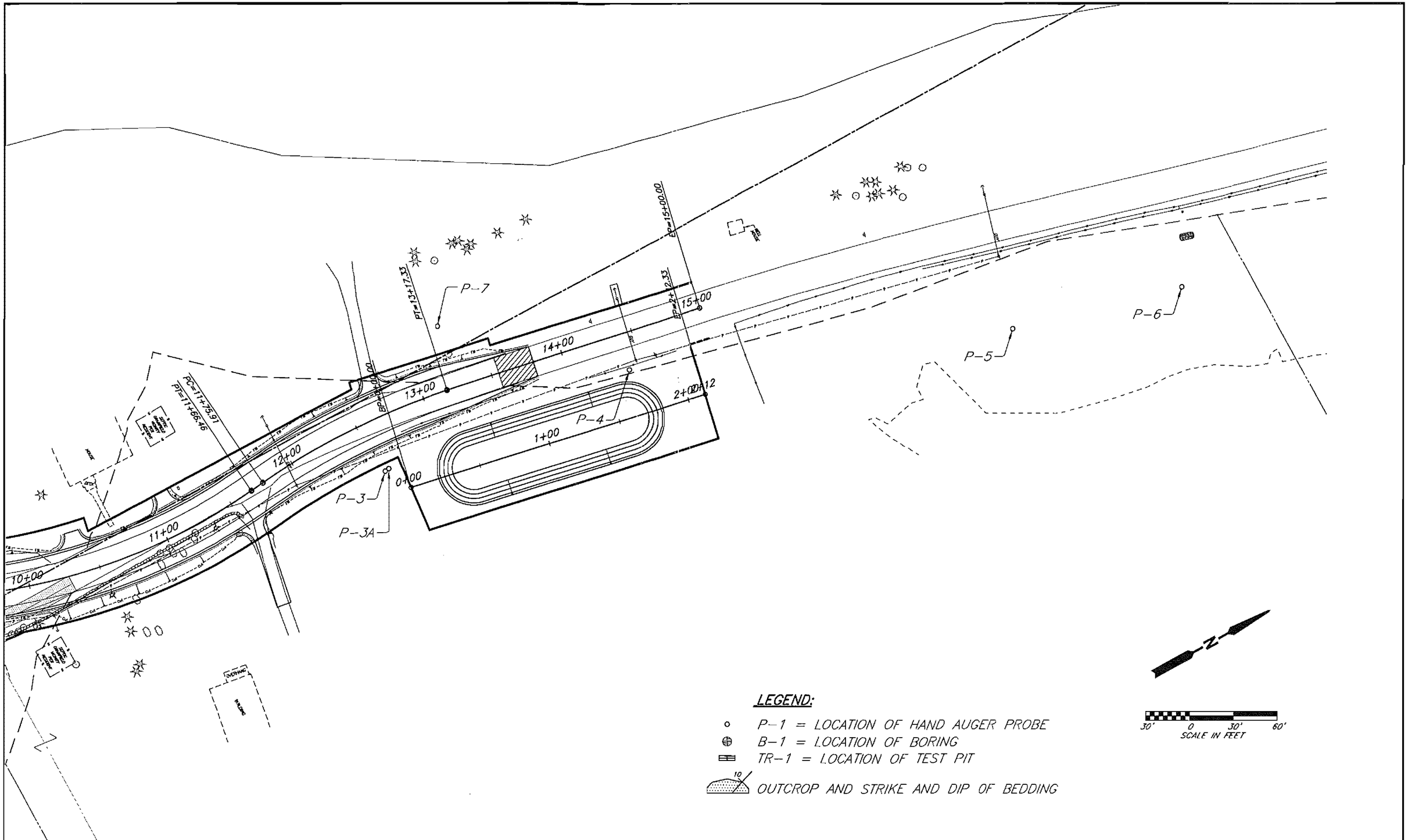

PLAN CHECK	BY	DATE	DATE	NO.	REVISION	BY

Last Saved By: spwmfh Aug 11, 2009 - 1:01pm

REGION NO.	STATE	FED. AID PROJ. NO.	SURVEY NO.
10	WASH.		3969 06-0000

DESIGNED BY: MMA
DRAWN BY: MFH
FIELD BOOK(S):
UPI#

APPROVED FOR CONSTRUCTION

OWEN B. CARTER, P.E.
SNOHOMISH COUNTY ENGINEER

DATE APPROVED:

SNOHOMISH COUNTY DEPARTMENT OF PUBLIC WORKS

FUNDING NO. RC #7333 RC #1360

JIM CREEK BRIDGE #42 REPLACEMENT

GEOTECH REPORT MAP

REFERENCE SHEET NO.

SHEET 2 OF 2 SHEETS

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appendix d

UNIFIED SOILS CLASSIFICATION SYSTEM

UNIFIED SOIL CLASSIFICATION SYSTEM

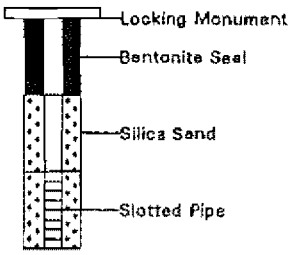
MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS		GW	WELL-GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES		GM	SILTY GRAVELS OR GRAVEL- SAND-SILT MIXTURES
				GC	CLAYEY GRAVELS OR GRAVEL-SAND-CLAY MIXTURES
	SAND AND SANDY SOILS	CLEAN SANDS		SW	WELL-GRADED SANDS OR GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS OR GRAVELLY SANDS, LITTLE OR NO FINES
		SANDS WITH FINES		SM	SILTY SANDS OR SAND-SILT MIXTURES
				SC	CLAYEY SANDS OR SAND-CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS Liquid limit less than 50			ML	INORGANIC SILTS AND VERY FINE SANDS,ROCK FLOUR,SILTY OR CLAYEY FINE SANDSOR 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 FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH -PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO -HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT OR OTHER HIGHLY ORGANIC SOILS
TOPSOIL / FILL					TOPSOIL OR FILL, MAY CONTAIN ABUNDANT ORGANIC MATERIAL

LOG SYMBOL LEGEND

- SPT sample interval
- Shelby tube sample interval
- Modified California sample interval
- Cutting sample
- Core
- Water level

- Ts Torvane reading (TSF)
- Vs Vane reading (TSF)
- MC Moisture content
- LL Liquid limit
- PI Plasticity index

WELL SYMBOLS



Snohomish County Public Works

KEY CHART

appendix e

EXPLORATION LOGS

PROJECT:	Jim Creek Bridge #42 Jordan Road	BORING NO.:	B-1
		DRILLER:	WSDOT
		DRILL TYPE:	CME850
C.R.P.#:	Rc 1360	DATE: 12/14/06	AUGER TYPE: casing advancer
STATION:		OFFSET:	FLUID: bentonite mud
GEO./ENGR.:	jjones	ELEV.: 109+-	TOTAL DEPTH: 39.50

SAMPLE INTERVAL	SAMPLE #	SPT BLOWS/FT	STRATA	DEPTH	SOIL DESCRIPTION	NOTES
					SILTY SAND; Reddish brown, some gravel, trace charcoal, trace root fibers, very moist, loose to medium dense. (weathered till, SM)	RQD
	1	3/2/2		5		
	2	50:3"				
	3					
	4			10	**SANDSTONE**; Greenish gray lithic arenite, coarse-grained to conglomeratic, vertical and horizontal iron-stained joints, moderately weak to moderately strong rock. Tan clay altered zone at 12'.	60%
	5			15	**CONGLOMERATE**; Gray, small subangular pebble to 1" minus rounded gravel, friable to 16 feet, horizontal fractures.	66% lost water at 16.5' to 22'
	6			20	**SILTSTONE**; Dark gray, weathered.	0% no recovery
	7			25	**CONGLOMERATE**; Gray, 3/4" rounded gravel, grain-supported, interbedded sandstone, friable, moderately weak rock.	52%
					SANDSTONE; Gray, lithic arenite, fine- to medium-grained, horizontal 2 mm coal seams, interbedded conglomeratic sandstone, friable, moderately strong rock.	

NOTES: **Located 12'N. of north abutment, 5' E. of west guardrail.**

PROJECT:	Jim Creek Bridge #42 Jordan Road	BORING NO.:	B-1
		DRILLER:	WSDOT
		DRILL TYPE:	CME850
C.R.P.#:	Rc 1360	DATE: 12/14/06	AUGER TYPE: casing advancer
STATION:		OFFSET:	FLUID: bentonite mud
GEO./ENGR.:	jjones	ELEV.: 109+-	TOTAL DEPTH: 39.50

SAMPLE INTERVAL	SAMPLE #	SPT BLOWS/FT	STRATA	DEPTH	SOIL DESCRIPTION	NOTES
	8			–		55%
				35–	Zone of very weak, fractured and weathered sandstone	40%
				–		
					TD=40 feet	

NOTES: Located 12'N. of north abutment, 5' E. of west guardrail.

Snohomish County Public Works

TEST BORING LOG B-1

PROJECT:	Jim Creek Bridge #42 Jordan Road	BORING NO.:	B-2
		DRILLER:	WSDOT
		DRILL TYPE:	CME850
C.R.P.#:	Rc 1360	AUGER TYPE:	casing advancer
STATION:		FLUID:	bentonite mud
GEO./ENGR.:	jjones	TOTAL DEPTH:	45.00

SAMPLE INTERVAL	SAMPLE #	SPT BLOWS/FT	STRATA	DEPTH	SOIL DESCRIPTION	NOTES
					FILL; Reddish brown gravelly silty sand, trace charcoal and root fibers, very moist to wet, loose. (SM / Fill)	RQD
	1	2/1/2		5		
	2	0/1/2				lost water 7-9'
	3	0/1/1		10		
	4	0/1/5				
	5	3/8/6		15	**SAND**; Brown-gray, medium- to coarse-grained, interbedded gravelly sand with 3/4" minus angular gravel, moist, medium dense. (SP) Orange iron-stained grit and coarse-grained sand at 15'.	gravelly drilling
	6 7	50:3"			**SANDSTONE**; Gray, weathered lithic arenite, coarse-grained grading to grit, interbedded small pebble conglomerate with subangular metamorphic gravel, vertical and horizontal iron-stained joints.	53%
	8			20	**CONGLOMERATE**; Gray, 1" minus subangular gravel, grain-supported in muddy matrix, moderately strong rock.	48%
	9			25	Tan clay altered zones at 26'	83%

NOTES: **Located 14' S. of south abutment, 5' E. of west wingwall.**

PROJECT:	Jim Creek Bridge #42 Jordan Road	BORING NO.:	B-2
		DRILLER:	WSDOT
		DRILL TYPE:	CME850
C.R.P.#:	Rc 1360	DATE: 12/14/06	AUGER TYPE: casing advancer
STATION:		OFFSET:	FLUID: bentonite mud
GEO./ENGR.:	jjones	ELEV.: 109+-	TOTAL DEPTH: 45.00

SAMPLE INTERVAL	SAMPLE #	SPT	BLOWS/FT	STRATA	DEPTH	SOIL DESCRIPTION	NOTES
	10						98%
	11				35	SILTSTONE; Dark gray, weak rock	fast drilling 35-41', very poor recovery 0%
	12				40	SANDSTONE; Gray lithic arenite, fine- to medium-grained, interbedded silstone, subhorizontal bedding. some clay altered zones.	98%
					45	TD=45 feet	

NOTES: Located 14' S. of south abutment, 5' E. of west wingwall.

Snohomish County Public Works

TEST BORING LOG B-2

PROBE LOGS

PROBE LOG P-1

Notes	Depth (feet)	Description
Located 100' S. of x-culvert inlet, 32' E. of CL.		**SILT**; Dark brown, sandy, organic-rich, trace gravel, moist, loose. (topsoil)
	2	**SILT**; Tan, slightly iron-stained, sandy, nonplastic, wet, loose. (ML)
	4	**SAND**; Brown, medium- to coarse-grained, trace gravel, wet. (SP)
		SAND; Gray with some iron-staining, fine- to medium-grained, porly graded, saturated, medium dense. (SP)
		TD= 5.1 feet

PROBE LOG P-2

Notes	Depth (feet)	Description
Located 90'S.of x-culvert outfall, 21' W. of CL.		**DUFF**; Dark reddish brown, decomposed wood debris, needles and leaves.
	2	**TOPSOIL**; Dark-brown, organic-rich, some charcoal, moist, loose. (topsoil)
		SAND; Orange-tan, silty, occasional gravel, some roots, some charcoal, damp, loose. (SM)
	4	tree roots
		SILTY SAND; Olive, mottled, very fine-grained, grades to sandy silt, very moist, medium dense. (SM)
	6	Heavily iron-stained, wet, medium dense.
		SILTY SAND; Olive, gravelly, moist, dense. (Till)
		TD=6.9 feet

Snohomish County
Public Works

PROJECT: Jim Creek Bridge #42

NUMBER: Rc 1360
DATE: 4/12/07

PROBE LOGS

PROBE LOG P-3

Notes	Depth (feet)	Description
Located 110' N. of driveway, 35' E. of CL, Elev 105.5.		TOPSOIL
		SAND; Brown-tan, sligtly silty to silty, fine- to coarse-grained, well graded, moist, medium dense. (SW)
	2	
		SILT; Tan, mottled, moist. (ML)
	4	SAND; Brown-olive, very fine-grained some silt, moist, medium dense. (SP)
		becomes medium- to coarse-grained
	6	SILTY SAND; Tan-olive, very fine-grained, bedded, very moist to wet. (SM)
		SAND; Gray, medium-grained, clean, moist. (SP)
	8	SAND; Gray-brown, gravelly, well graded, medium- to coarse-grained, very moist, medium dense. (SW)
	∇	
		becomes wet
	10	
		TD=10.0 feet

PROBE LOG P-3A

Notes	Depth (feet)	Description
Located 100'north of Soper driveway, 35' east of CL, Elev. 105.5.		FILL; Mixed topsoil, sand, silt.
	2	
		SILT; Tan, sandy, very moist, loose. (ML)
	4	SILTY SAND; Olive, mottled, fine-grained, very moist to wet, loose. (SM)
		SAND; Gray-brown, some iron-stained zones, clean, poorly graded, very moist, medium dense. (SP)
	6	Interbed of sandy SILT, iron-stained.
		Interbed of GRAVEL, iron-stained
	8	
		Iron-stained, gravelly sand
		GRAVEL;Gray, poorly graded, wet, medium dense. (GP)
		TD= 9.7 feet; terminated on gravel above S.W.L.

Snohomish County Public Works

PROJECT: Jim Creek Bridge #42

NUMBER: Rc 1360
DATE: 4/12/07

PROBE LOGS

PROBE LOG P-4

Notes	Depth (feet)	Description
Located 280' N. of 20819 driveway, 26' E. of CL, Elev. 104.	2	TOPSOIL SILTY SAND; Orange-brown, fine-grained, poorly graded, moist, loose. (SM)
	4	SAND; Olive, fine- to medium-grained, poorly graded, trace silt, moist, medium dense. (SP)
	6 8	SAND; Olive, medium- to coarse-grained, well graded, moist, medium dense. (SW) Becomes iron-stained, gravelly and very moist. TD=8.3 feet, no seepage or caving.

PROBE LOG P-5

Notes	Depth (feet)	Description
Located 50'S.70E. of PP#XJ79, Elev.102.7.		TOPSOIL
		SAND; Brown-olive, gravelly, 1" minus round 15%, poorly graded, moist, medium dense. (SP)
	2	GRAVEL; Brown-olive, sandy, 2" minus round, poorly graded, moist, medium dense. (GP) 4" minus subround gravel
	▽	SAND; Olive-brown, gravelly, 1.5" minus round, poorly graded, very moist, medium dense. (SP) Becomes wet, very gravelly TD=3.8 feet, SWL=3.6

Snohomish County
Public Works

PROJECT: Jim Creek Bridge #42

NUMBER: Rc 1360
DATE: 4/24/07

PROBE LOGS

PROBE LOG P-6

Notes	Depth (feet)	Description
Located 120' N.20E. of P-5, Elev. 104.4.	2	TOPSOIL SILTY SAND; Reddish brown, root fibers, trace gravel, moist, medium dense. (SM) SAND; Olive, medium-grained, poorly graded, moist, medium dense. (SP) Gravelly sand, 3/8" minus GRAVEL; Gray, 1" minus, sandy, moist, medium dense. (GP) 3" minus gravel TD=3.7 feet; terminated on large gravel.

PROBE LOG P-7

Notes	Depth (feet)	Description
Located 200' north of Soper driveway, 50' west of CL, Elev. 106.	2 4 6 8 10	TOPSOIL; Brown sandy silt, moist. SILTY SAND; Tan, moist, loose. SILTY SAND; Gray-green, some granules and small gravel, damp to moist, medium dense. (SM) SAND; Olive, fine-grained, some silty sections, some iron-stained horizons, moist. (SP/SM) SAND; Olive, clean, fine-grained, poorly graded, moist, medium dense. (SP) GRAVEL; Iron-stained, granules to small gravel, very moist, medium dense. (GP) TD= 10.3' ; terminated on gravel.

Snohomish County Public Works

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NUMBER: Rc 1360
DATE: 5/9/08

PROBE LOGS

PROBE LOG P-9

Notes	Depth (feet)	Description
Located 38' south of SE abutment, 5' east of eop		TOPSOIL
		SILTY SAND; Tan, trace charcoal, trace gravel, moist. (SM)
	2	
		SAND; brown, medium- to coarse-grained, clean, moist. (SP)
		Boulder at 3' TD=3.0 feet; terminated on boulder

Snohomish County
Public Works

PROJECT: Jim Creek Bridge #42

NUMBER: Rc 1360
DATE: 3/18/09

PROBE LOGS

PROBE LOG P- 10

Notes	Depth (feet)	Description
Located 44' south of SE abutment, 5' east of eop.	2	TOPSOIL SILTY SAND; Tan-brown, interbedded silt, roots, moist. (SM) SAND; Olive, medium- to coarse-grained. (SP) GRAVEL; Olive -brown, 2" minus, moist to wet. (GW) Boulder at 3.75' TD=3.75'; Terminated on boulder at 3.75 feet.

PROBE LOG P-8

Notes	Depth (feet)	Description
Located 41' south of SE abutment, 4' east of east eop.	2	TOPSOIL SILTY SAND; Yellow-tan, fine-grained, poorly graded, moist, medium dense. (SM) SAND; Olive, slightly iron-stained, fine-grained, some silt, very moist, medium dense. (SP) Boulder at 2.5 TD=2.5 feet; terminated on boulder

Snohomish County
Public Works

PROJECT: Jim Creek Bridge #42

NUMBER: Rc 1360
DATE: 3/18/09

TEST PIT LOGS

TEST PIT LOG TP-1

Notes	Depth (feet)	Description
Located 40-60 feet south of SW abutment, behind Jersey barrier end.	2 4 6	**SILTY SAND**; Brown-tan, some gravel. SM, Fill) **GRAVEL**; Reddish brown, coarse sand, cobbles and boulders. (GW) Orange iron-stained layer, wet. TD=6.5 feet; terminated because of heavy seepage at 5.5 feet and severe caving.

TEST PIT LOG TP-2

Notes	Depth (feet)	Description
Located 34-44 feet south of SE abutment, 3' back of guardrail.	2 4 6 8	**SILTY SAND**; Brown-tan, some gravel, occasional cobble. (SM, Fill) **COBBLES**; and boulders, wet. **SILTY SAND**; Gray, some gravel, moist, very dense. (SM, Till) TD=8.5 feet; moderate seepage at 5 feet.

Snohomish County Public Works

PROJECT: Jim Creek Bridge #42

NUMBER: Rc 1360
DATE: 4/2/09

TEST PIT LOGS

TEST PIT LOG TP-3

Notes	Depth (feet)	Description
Located 26' north of NW abutment, behind guardrail.		**SILTY SAND**; Tan, some gravel, some cobbles, very moist, medium dense. (SM)
	2	
	4	Root zone, locallized seepage.
	6	**GRAVEL**; Brown, clean, some coarse-grained sand, wet, medium dense. (GP)
		SANDSTONE; Reddish brown, arkosic arenite, weathered. TD=7.3 feet; light seepage at 3 feet.

Snohomish County Public Works

PROJECT: Jim Creek Bridge #42

NUMBER: Rc 1360
DATE: 4/2/09

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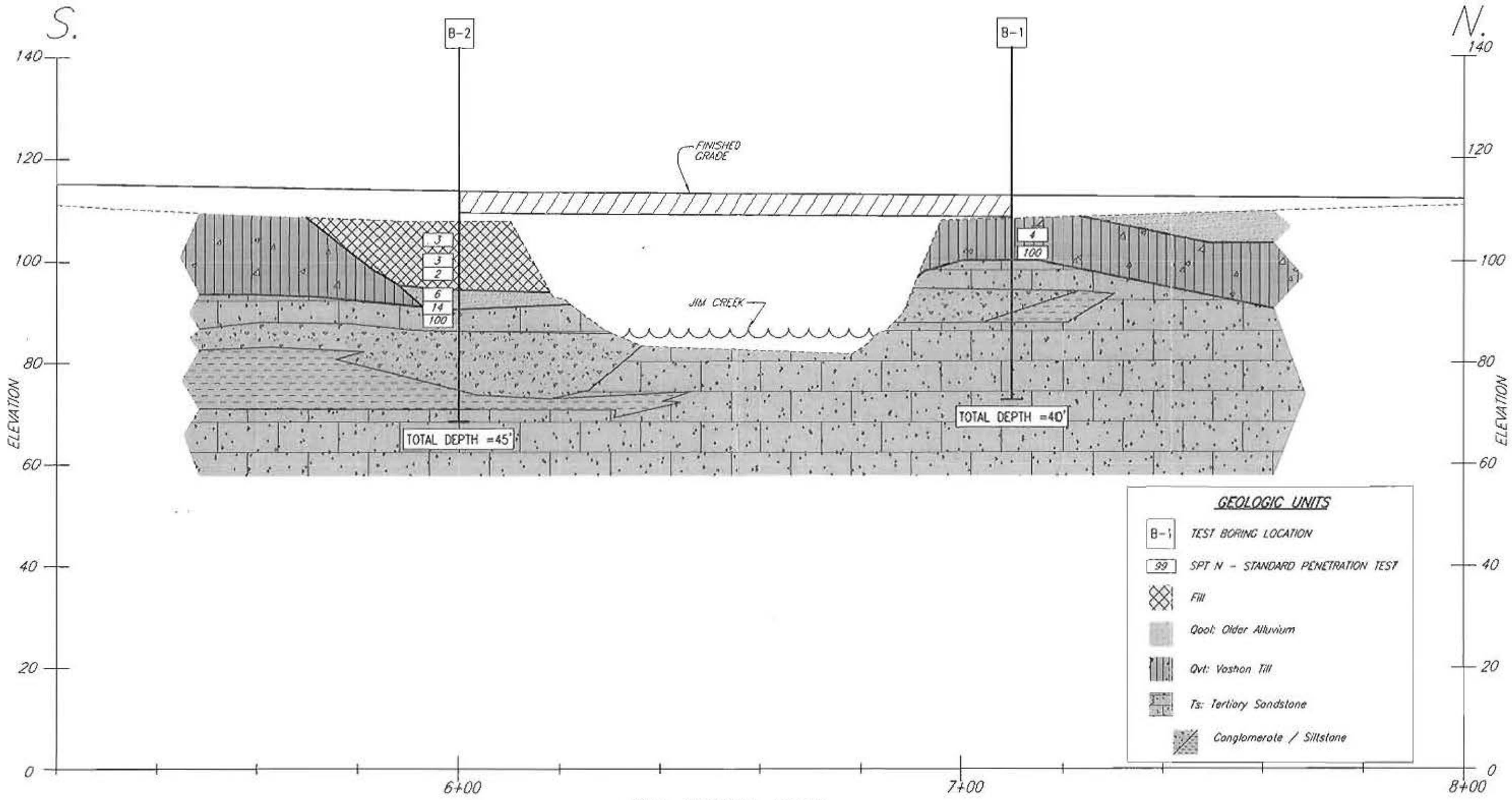
GEOLOGIC CROSS-SECTION



SOIL BORING LOGS

SCALE: 1"= 20' Horiz. & Vert.

DATE	NO.	REVISION	BY	DES.	SURV.	R/W	CONST

Last Saved By: spamth Mar 05, 2008 - 10:11am

REGION NO.	STATE	FED. AID PROJ. NO.	SURVEY NO.
10	WASH.		3969

DESIGNED BY: JEFF JONES
DRAWN BY: MFH
FIELD BOOK(S):
UPI # UPI 07-0054

APPROVED FOR CONSTRUCTION

OWEN B. CARTER, P.E.
SNOHOMISH COUNTY ENGINEER

DATE APPROVED:

SNOHOMISH COUNTY DEPARTMENT OF PUBLIC WORKS

FUNDING NO. RC1360

JIM CREEK BRIDGE #42
REPLACEMENT

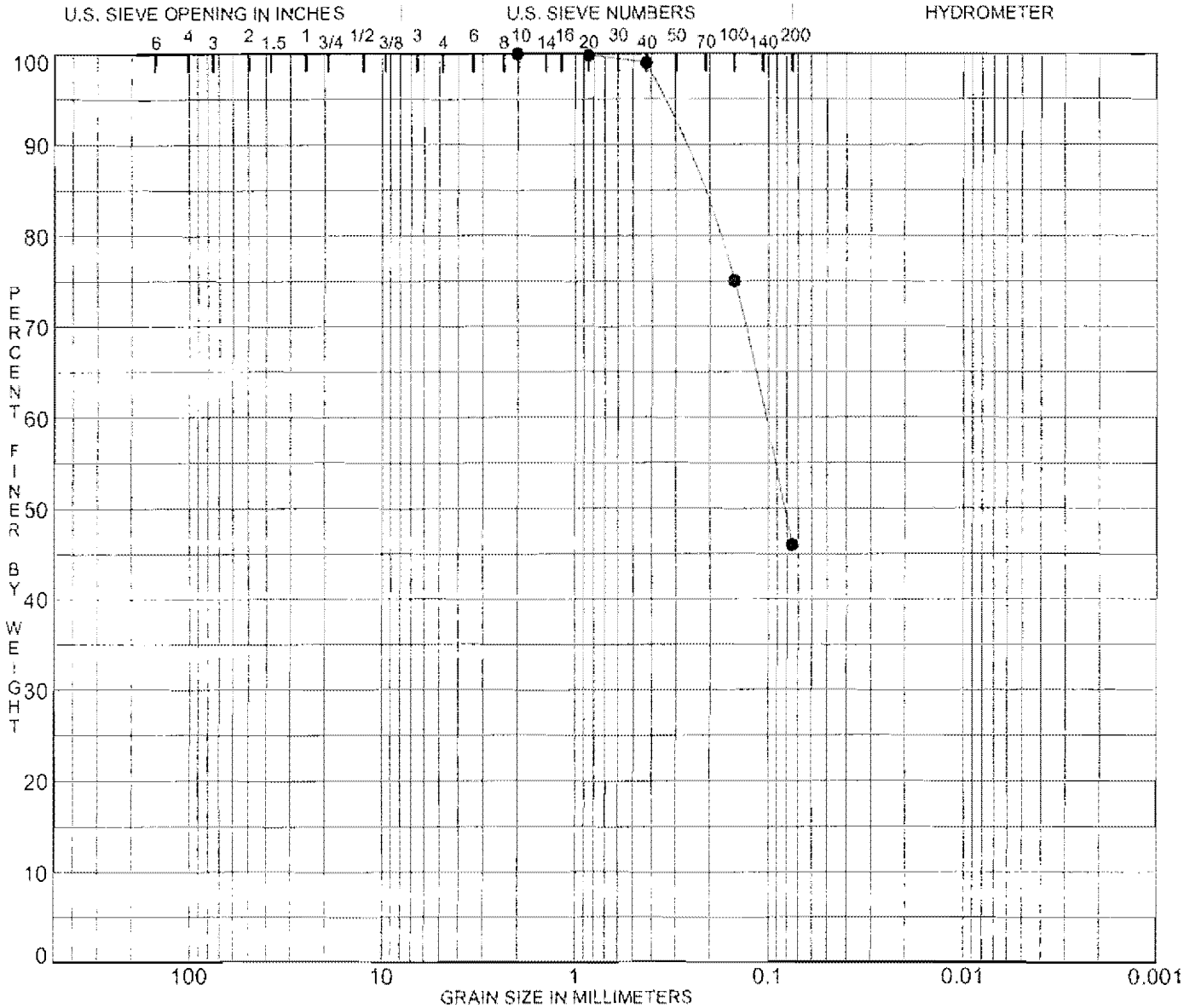
GEOLOGIC SECTION

REFERENCE SHEET NO.

SHEET 1 OF 1 SHEETS

appendix g

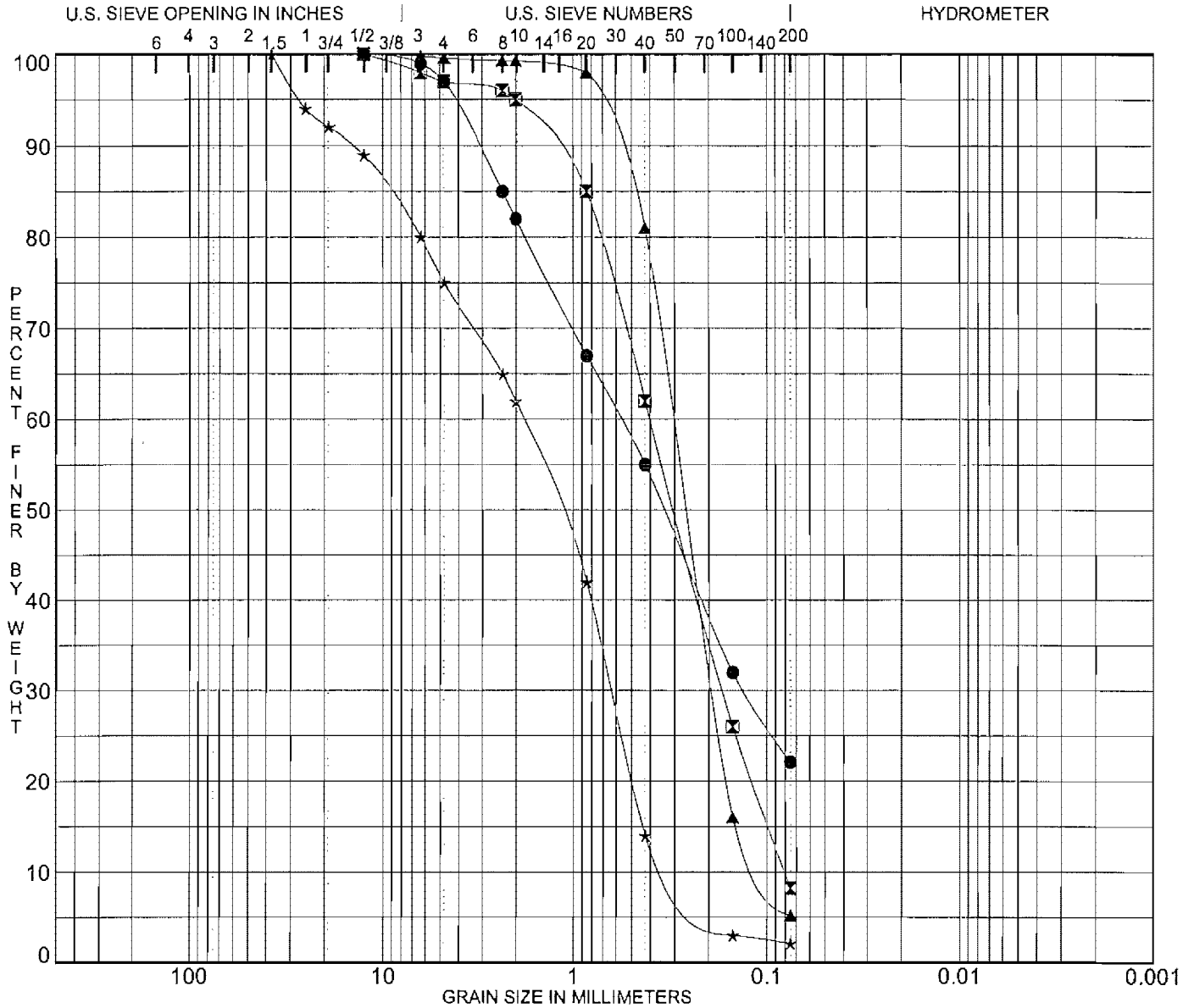
GRAIN SIZE ANALYSIS



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	P-2	4.5	Silty very fine-grained SAND						

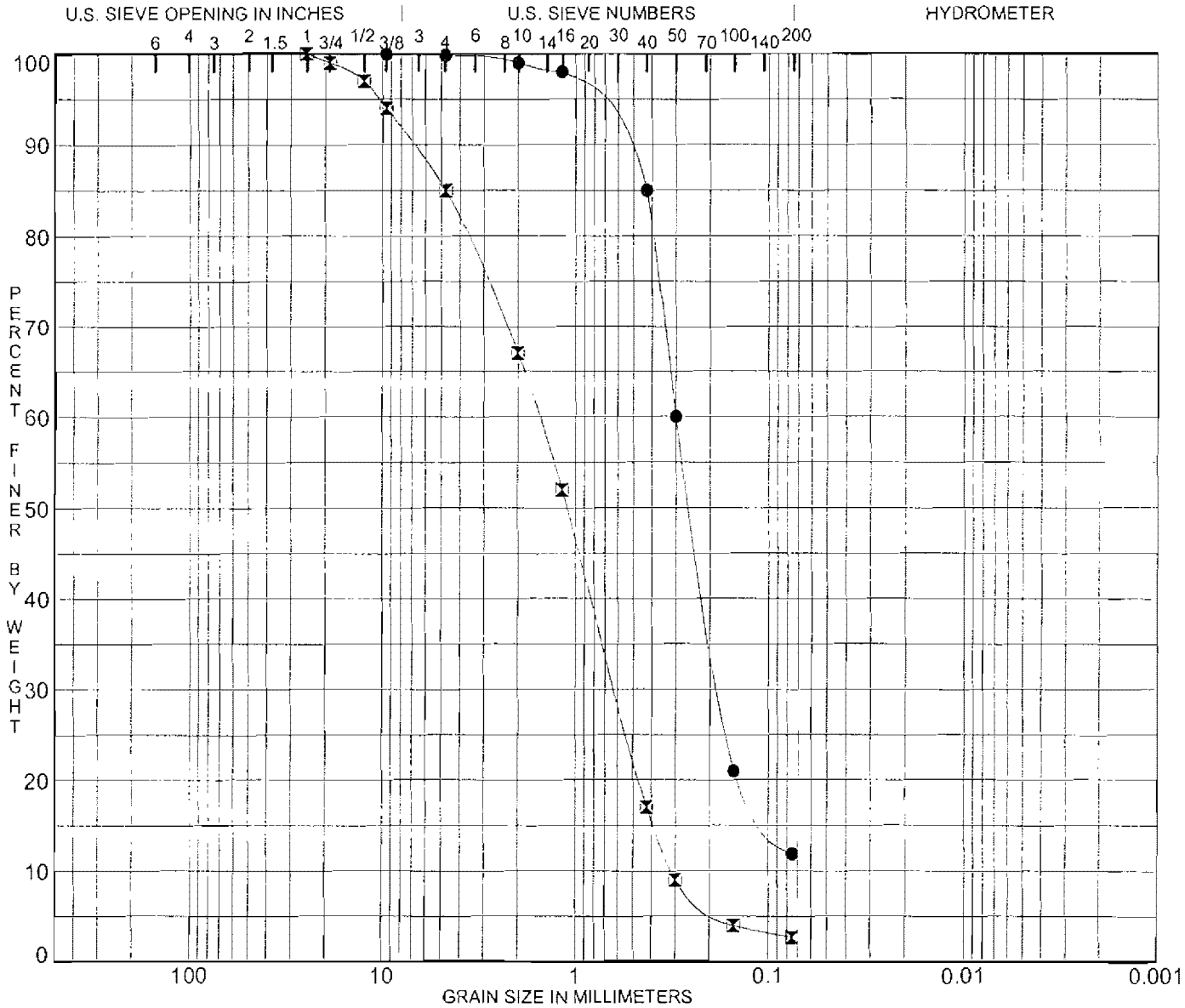
	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	P-2	4.5	2.00	0.10			0.0	54.0	46.0	



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	P-3	1.3	Silty fine-grained SAND						
⊠	P-3	4.0	Fine-grained SAND with silt					0.88	5.0
▲	P-3	6.9	Medium-grained SAND trace silt					1.14	3.0
★	P-3	9.5	POORLY GRADED SAND with GRAVEL SP					0.75	6.3

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	P-3	1.3	12.50	0.57	0.130		3.0	74.9	22.1	
⊠	P-3	4.0	12.50	0.40	0.168	0.0804	3.0	88.8	8.2	
▲	P-3	6.9	12.50	0.30	0.188	0.1021	0.4	94.4	5.2	
★	P-3	9.5	37.50	1.84	0.632	0.2910	25.0	72.9	2.1	



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	P-4	4.0	Fine-grained silty SAND					1.59	4.6
⧗	P-4	6.0	POORLY GRADED SAND with GRAVEL SP					0.79	5.0

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	P-4	4.0	9.50	0.30	0.176		0.2	87.9	11.9	
⧗	P-4	6.0	25.00	1.56	0.621	0.3134	15.0	82.3	2.7	