Appendix C

Draft Geotechnical Report

Draft GEOTECHNICAL REPORT
Mill Creek
Fish Passage Project
Chelan County, Washington

PROJECT NO. 14-095
July 9, 2019

Prepared for:

Natural Systems Design

CHELAN COUNTY
Natural Resources Department

PanGEO INCORPORATED

Geotechnical & Earthquake Engineering Consultants

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DRAFT GEOTECHNICAL REPORT
MILL CREEK FISH PASSAGE PROJECT
MT. HOME RANCH ROAD
CHELAN COUNTY, WASHINGTON

1.0 PROJECT DESCRIPTION

Chelan County plans to replace an existing concrete box culvert that conveys Mill Creek beneath Mountain Home Ranch Road with a new high radius arch structural plate culvert. The proposed culvert is expected to be approximately 50 feet in length, 20¾ feet in width, and 12 feet in height, with at least 2½ feet of soil cover. The proposed culvert will be constructed beneath the existing Mountain Home Ranch Road alignment using cut-and-cover construction methods. A temporary detour for traffic on Mountain Home Ranch Road is planned on the upstream side of the existing culvert. Our understanding of the project is based on our review of 30% design plans prepared by Natural Systems Design, dated June 2019.

2.0 SITE DESCRIPTION

The following site description is based in part on a visual reconnaissance of the project area on April 23, 2014. The project site is located in the southern portion of Chelan County, Washington, approximately 4 miles south of the intersection of U.S. Highway 2 and State Route 97 (Figure 1, Vicinity Map). From its summit at Blewett Pass, State Route 97 (SR-97) eventually descends into the canyon of Peshastin Creek and follows it northward on its way to the confluence with the Wenatchee River. Mountain Home Ranch Road branches west off SR-97 approximately 0.1 mile north of the existing Mill Creek culvert location, which is in turn approximately 200 feet upstream of the confluence with Peshastin Creek. The site is located in the SE ¼ of Section 6, Township 23N, Range 18E.

The topography around the existing culvert location is variable and appears to be disturbed from its original, natural condition by the construction of Mountain Home Ranch Road and / or other land modification. Mounds of boulders are abundant in the area, some of them many feet in diameter. Off the alluvial valley floors of Mill Creek and Peshastin Creek, the topography rapidly becomes very steep with bedrock outcrops and cliffs visible in many locations. The streambed of Mill Creek is also relatively steep and consists mostly of a series of boulder pools alternating with small cascades and riffles. The Mill Creek drainage is in the semi-arid region on the eastern (lee) side of the Stuart Range of the Cascade Mountains. The hillsides support a growth of conifers, while the valley bottoms have both coniferous and deciduous trees with generally arid-climate brush understory and grasses. Several rural residential structures and out-buildings are located on the flood plain of Peshastin Creek. The nearest of these is located north of Mill Creek and west of Mountain Home Ranch Road and is readily visible from the existing culvert location. Agriculture and animal husbandry are also common in the area.

Both communication and electrical lines are supported above ground on timber poles on the west side of Mountain Home Ranch Road. It appears that the proposed temporary detour alignment may conflict with the pole on the south side of Mill Creek and may require temporary re-location of the pole for construction of the detour.

3.0 GEOLOGY

The project site is located in the southern portion of Chelan County, on the west side of the Peshastin Creek watershed, a north-draining tributary of the Wenatchee River. Mill Creek, a tributary of Peshastin Creek, drains a watershed with headwaters in the Stuart Range west of the project site. According to both Tabor et al. (1987) and Whetten (1980), the valley bottoms of both Mill Creek and Peshastin Creek contain Quaternary-age alluvium, with the surrounding valley walls consisting of sedimentary sandstone, shale and conglomerate of the Chumstick Formation. Whetten (1980) maps a right lateral strike-slip fault following the general north-south trend of Peshastin Creek, indicating that the canyon has cut along the lineament of the fault trace. The fault is mapped as hidden below the valley alluvium with the exception of an exposure of the fault trace in the Chumstick Formation on the eastern canyon wall of Peshastin Creek, roughly opposite the project location. The fault is not currently thought to be active (WSDOT, 2013).

The abundant boulders in the immediate vicinity of the project site are, for the most part, not derived from the Chumstick Formation, as their lithologies are igneous and metamorphic (not sedimentary) in nature. The parent material of these boulders appears to be the Stuart batholith and surrounding rock to the west of the site. This implies considerable transport distance.

4.0 FIELD EXPLORATION

The subsurface exploration program consisted of drilling one test boring as near as practical to the likely location of the southern abutment of the new proposed culvert as shown on the Site and Exploration Plans (Figures 2A and 2B). The test boring was drilled using a BK-81 truck-mounted drill rig owned and operated by Holocene Drilling of Puyallup, Washington, under a subcontract to PanGEO. The test boring was designated BH-1-14 and was advanced to a maximum depth of 25¾-feet below the ground surface on April 28, 2014 using mud rotary drilling methods.

Standard Penetration Tests (SPT) were performed at 5-foot depth intervals using a 2-inch diameter split-spoon sampler. The sampler was driven into the soil a distance of 18-inches below the bottom of the auger using a 140-pound auto-trip safety hammer falling a distance of 30 inches for each strike, in general accordance with ASTM D-1586, Standard Test Method for Penetration Test and Split Barrel Sampling of Soils. The number of blows required for each 6-inch increment of sampler penetration was recorded. The number of blows required to achieve the last 12-inches of sample penetration is defined as the SPT N-value.

The boring was logged by a geotechnical engineer from PanGEO. The soil samples were described using the system outlined in Figure A-1 in Appendix A. A summary boring log is included as Figure A-2. The stratigraphic contacts indicated on the boring log represent the approximate depth to boundaries between soil units. Actual transitions between soil units may be more gradual or occur at different elevations. The descriptions of groundwater conditions and depths are likewise approximate.

5.0 SUBSURFACE CONDITIONS

5.1 SOILS

The site soils, as encountered in the test boring mainly consisted of basalt gravel in a silt and clay matrix, which is not consistent with the sandstone bedrock of the Chumstick Formation mapped near the project site. It is also important to note that the drilling depth and sampling were limited by the large diameter material (cobbles and boulders) encountered in the test boring. The standard penetration test (SPT) blowcounts obtained are likely affected by the gravels, cobbles, and boulders encountered during drilling and therefore may not be representative of the relative density as the blowcounts are likely overstated.

The following is a generalized description of the soils encountered in our test borings. A generalized subsurface profile is included in Figure 3 of this report.

Unit I, Fill / Re-worked Alluvium (Hf): Mountain Home Ranch Road bed was underlain by very dense, dark grey, fine to coarse grained basalt gravel with dark brown silt and clay matrix. In general, the material appeared to consist mainly of basalt with medium to high plasticity fines and scattered quartz. This layer is interpreted as fill or re-worked alluvium which appears to be disturbed from its original, natural condition by the construction of Mountain Home Ranch Road and / or dredge or placer mine workings in the alluvial valley bottoms, or both. This unit extended to a depth of roughly 12- to 13-feet below the surface at the location of test boring BH-1-14.

Unit IIa, Colluvium / Mass Wasting Deposit (Hc /Hls): Underlying the fill / re-worked alluvium, test boring BH-1-14 encountered medium dense, dark grey, clayey basalt gravel with a grey and tan clay and silt matrix. This soil unit was characterized by its medium to highly plastic fines, which appeared weathered with white and reddish mottles. This layer is interpreted as Colluvium / Mass Wasting deposits that were most likely deposited as a result of a large, prehistoric mass wasting event. This unit is approximately 6- to 7-feet thick.

Unit IIb, Colluvium / Mass Wasting Deposit (Hc /Hls): Underlying the clayey basalt gravel, we encountered very dense, dark grey, basalt gravels, cobbles, and bounders with some fines. This soil unit was characterized by its very dense / massive state, soil cuttings and extremely difficult drilling action. This is the deepest soil unit encountered in our test boring due to practical refusal on a very large basalt boulder, or possibly bedrock, at approximately 26 feet below the ground surface.

5.2 GROUNDWATER

Groundwater appeared to be perched above the clayey gravel layer, which roughly coincided with the thalweg of the creek. It should be noted that the groundwater level could not be accurately measured due to the wet (mud) drilling method and coarse grained nature of the soils. Groundwater is expected to fluctuate closely with the flow levels in the creek.

6.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

6.1 Site Seismicity

The subject site is located along the eastern margin of the Cascade Range where it joins with the Columbia Plateau Basalt province. This area is not as seismically active as is the area west of the Cascades, but does experience seismic activity. The project area is situated in an area that may be associated with the Yakima Fold Belt, or an extension of Stuart Range. These folds began to develop originally in the late Miocene and deformation may continue into the present day. Seismicity on the Columbia Plateau tends to be generally shallow and associated with thrust faults along the north limbs of the anticlinal structures.

Seismicity in the fold belt is generally limited to micro-earthquake swarms that may contain up to 100 individual events in a limited time frame. These occur at shallow depths, normally 3 to 5 kilometers (Tillson, 1989). These events rarely exceed 3.5 in magnitude. Concentrations of swarms have occurred in the area of the Saddle Mountains on the north margin of the Pasco Basin, and in the Walla Walla area. One of the most active areas for shallow earthquake swarms is along the north side of the Whiskey Dick/Frenchman Hills anticline, located approximately 50 miles southeast of the project area. This structure is apparently truncated by the southeast trending cross-structure of the Naneum Ridge anticline.

The largest historical earthquake recorded to date in Washington, with a magnitude of approximately 7.3, occurred on December 14, 1972 in the northern Cascade Mountains. Some recent thinking suggests that this event may have taken place on a postulated Chelan Seismic Zone (Crider and others, 2003), which is located about 45 miles northwest of the Colockum drainage.

6.1.1 Seismic Design Parameters

For seismic design, an acceleration coefficient of 0.17g is recommended per the current acceleration map in AASHTO (2017). The recommended acceleration coefficient is based on expected ground motion at the project site that has a 7 percent probability of exceedance in a 75-year period (approximately 1000-year return period).

Design response spectra presented in AASHTO (2017) are considered appropriate for seismic design of the proposed culvert. A horizontal response spectral acceleration coefficient at a period of 0.2 seconds (S_S) is 0.40. The horizontal response spectral acceleration coefficient at a period of 1.0 seconds (S_1) is 0.14.

The soils at the site are considered Site Class C, with associated site factors F_{pga}, F_a and F_v equal to 1.20, 1.20 and 1.66, respectively. The sites are therefore in Seismic Performance Zone 2.

6.1.2 Liquefaction Potential

Simplified screening was used to assess the liquefaction susceptibility of the site soils in accordance with 6.4.2.1 of the Geotechnical Design Manual (WSDOT, 2012b). Based on our analyses, liquefaction is not expected to develop at the site under the design earthquake conditions due to the sufficiently high SPT-blowcounts in the alluvial valley deposits and the relatively low peak ground acceleration of the design event. Therefore, no special design considerations are recommended regarding liquefaction.

6.2 Lateral and Vertical Earth Pressures

We understand that the headwalls will be constructed on both ends of the proposed culvert. The headwalls may consist of cast-in-place concrete wall, or gravity walls using precast concrete blocks. The headwalls may be supported conventional footings. The footings should be embedded sufficiently deep to mitigate the risk of erosion and scouring. Recommendations for footings outlined in Sections 6.3 and 6.4 of this report are also applicable for headwall footings.

If a joint is provided at the culvert headwall so that the headwall wall is free to deflect slightly, active pressures can be used in design of the headwalls. An equivalent fluid pressure of 35 pounds per cubic foot (pcf) may be used to calculate lateral earth pressures on the abutments. This equivalent fluid pressure does not include live load surcharge. A lateral earth pressure coefficient, K_A, of 0.28 may be used to calculate the lateral load due to surcharge.

If culvert headwalls are fixed against lateral deflection, at-rest pressures will be appropriate for design. An equivalent, at-rest fluid pressure of 55 pcf may be used to calculate at-rest passive earth pressures on the abutments. This equivalent fluid pressure does not include live load surcharge. An at-rest lateral earth pressure coefficient, K_O, of 0.44 may be used to calculate the lateral load due to surcharge.

The culvert structure should also be designed for vertical surcharges such as soil cover and vehicle/traffic loads. The weight of the soil cover should be estimated based on a soil unit weight of 130 pounds per cubic foot (pcf). In addition, a minimum uniform vertical pressure of 250 psf should be included to account for the traffic loads.

The headwalls should also be designed for traffic surcharge. A uniform lateral pressure of 80 psf is considered adequate to account for the traffic loads for wall design.

The seismic earth pressure is computed according to the Mononobe-Okabe method described in the LRFD Bridge Design Specifications (AASHTO, 2017). The walls are assumed free to move and to develop the active earth pressure conditions during a seismic event. For this project we recommend that the seismic earth pressure increment be taken as 6H psf, where H is the height of the soil behind the structure. The seismic earth pressure increment is in addition to the active static earth pressure, and is in a trapezoidal distribution, applied at 0.6H from the bottom of the pressure distribution.

The above lateral earth pressures assume that the new structures are backfilled with good quality, granular material such as Gravel Backfill for Pipe Zone Bedding, Gravel Borrow or Gravel Backfill for Walls per the *Standard Specifications* (WSDOT, 2018) within 5 feet of the structures. Beyond 5 feet of the proposed structures, the backfill may consist of Select Borrow or on-site soils, provided that the on-site soils can be properly compacted to meet the project specifications.

6.3 Lateral Resistance

Resistance to lateral loads on the spread footings may be resisted by passive earth pressure developed against the embedded portion of the foundation system and by frictional resistance between the bottom of the foundation and the supporting subgrade soils. The recommended values in Table 1 are considered nominal values. The base friction coefficient assumes concrete cast directly against soil or directly atop a rat slab. The passive pressure assumes foundations are backfilled with properly compacted structural fill.

Table 1
Spread Footing Sliding Resistance

Load Case	Base Friction Coefficient	Base Friction Resistance Factor	Passive Pressure	Passive Resistance Factor
Service	0.65	1.0	200 psf	1.0
Strength	0.65	0.8	200 psf	0.8
Extreme	0.65	1.0	200 psf	1.0

6.4 Bearing Resistance

It is our understanding that new spread footings would bear approximately at the thalweg elevation of Mill Creek. Based on this depth of bearing, the foundations may be proportioned using the nominal bearing resistances provided in Figures 4A and 4B.

The nominal bearing resistance at the service limit state was developed to limit the foundation settlement to less than ½-inch and 1-inch in Figures 4a and 4b, respectively. Differential settlement between the two headwalls is not expected to exceed the total settlement. All settlement is expected to occur rapidly, as loads are applied.

We recommend that at least one foot of well compacted, coarse bedding material be placed below the footings as a levelling course, and to provide a firm uniform support surface for the structure, and to provide a firm working surface.

Spring Constant for Spread Footings **-** Recommended parameters for computing spring constants for spread footing foundations are shown in Table 2, below. The shear modulus may be linearly interpolated for intermediate strain values.

Table 2
Recommended Spread Footing Spring Constants

Strain	G (ksf)	ν
0.02%	1500	0.35
0.2%	500	0.35

6.6 Uplift Resistance

The proposed fish passage structure is an open-bottom culvert. Therefore, buoyancy (uplift) forces are expected to be self-relieving. As such, design considerations for the uplift resistance of the culvert are not required.

6.7 CORROSION POTENTIAL

Corrosion potential and the impact to proposed metal structures has been evaluated by the structure designers in accordance with Chapter 8 of the Hydraulics Manual (WSDOT, 2019). Section 8-2.3.3 of the manual indicates the aluminum structural plate culverts can be used anywhere in the state, regardless of corrosion zone.

The current design calls for a high radius aluminum arch structural plate culvert bearing on concrete footings with concrete stem walls. As such, the proposed structure design meets the requirements for design for Corrosion Zone III.

In addition, the proposed fish passage structure is planned to bear in colluvium soils below existing fills along the entire structure. The fill soils will be removed during the excavation and the structure will be backfilled on the sides and immediately above the culvert with Gravel Backfill for Pipe Zone Bedding (Article 9-03.12(1), WSDOT, 2018). Therefore, it is our opinion that no special corrosion protection design considerations are necessary for the proposed fish passage structure.

6.8 EARTHWORKS

6.8.1 Culvert and Headwall Backfill

It is our understanding that, within 5 feet of the culvert and headwalls, the backfill will consists of imported, free draining granular material, such as Gravel Backfill for Pipe Zone Bedding (Article 9-03.12(1), WSDOT, 2018). Away from the culvert and headwalls, the fill may consist of Select Borrow or on-site soils, provided that the on-site soils can be adequately compacted to meet the project specifications.

The backfill should be moisture conditioned to within about 3 percent of optimum moisture content, placed in loose, horizontal lifts less than about a foot in thickness, and systematically compacted to a dense and relatively unyielding condition and to at least 95 percent of the maximum dry density, as determined using test method ASTM D-1557 (Modified Proctor). Within 5 feet of the walls, the backfill should be compacted with hand-operated equipment to at least 90 percent of the maximum dry density. Inadequate compaction of the backfill may lead to significant differential settlement of the pavement at the soil-culvert transition.

6.8.2 Permanent Cut and Fill Slopes

We recommend that permanent cut and fill slopes, where applicable, be constructed no steeper than 2H:1V (horizontal:vertical). For fill slopes constructed at 2H:1V or flatter, comprised of structural fill soils placed and compacted as recommended in this report, we anticipate that adequate factors of safety against global failure will be maintained.

Prior to placing compacted fill against an existing slope, all loose/soft soils must first be removed from the slope face. In addition, adequate benching must be maintained for existing slopes with angles steeper than 3H:1V. The removal of loose surficial soils is typically accomplished during the benching process, as fill placement progresses upwards. Each bench should be at least 6 feet wide and may be about 2 to 3 feet high.

Measures should be taken to prevent surficial instability and/or erosion. For a permanent fill slope, this can be accomplished by conscientious compaction of the embankment fills all the way out to the slope face, by maintaining adequate drainage, and by planting the slope face as soon as possible following construction. To achieve the specified relative compaction at the slope face, it may be necessary to overbuild the slopes several feet, and then trim back to design finish grade. In our experience, compaction of slope faces by "track-walking" is generally not as effective.

7.0 CONSTRUCTION CONSIDERATIONS

We anticipate earthwork operations will consist of removing the existing culvert, excavating to achieve the replacement culvert subgrade elevation, installing the culvert and backfilling to restore the road profile.

7.1 TEMPORARY EXCAVATION SLOPES

The inclination of temporary excavation slopes is dependent on many variables, including the depth of the excavation, the soil type and density, the presence of groundwater seepage, construction timing, weather conditions, and surcharge loads from adjacent structures, soil stockpiles, roads and equipment. Because of the many variables involved, the inclination of temporary excavation slopes should be evaluated during construction, as the actual soil conditions are exposed.

Temporary excavations should be performed in accordance with Part N of WAC (Washington Administrative Code) 296-155. The contractor is responsible for maintaining safe excavation slopes and/or shoring. For preliminary planning purposes, the temporary excavations may be sloped as steep as 1½H:1V. During wet weather, the cut slopes may need to be flattened to reduce potential erosion. In areas where excessive sloughing, groundwater seepage or unstable soils conditions are encountered, a shallower temporary slope inclination or shoring may be needed.

We anticipate the maximum depth of excavations for the culvert installation to be about 18 feet deep. To limit the areal extent of the temporary excavation, the contractor may install an excavation shoring system to support the excavation. The selection and design of the excavation shoring system should be the responsibility of the contractor.

The contractors should be aware that large cobbles and boulders are often present in colluvium and mass waiting deposits and may be encountered during excavation and/or installation of excavation shoring system.

7.2 Dewatering

Groundwater will likely be encountered in excavations for the construction of the culvert and headwalls. Due to the coarse grained nature of the alluvial soils expected within the depths of excavation for the new structure, groundwater inflow may be significant. Large trash pumps or similar may be needed in order to control groundwater inflows. Groundwater seepage is also expected to be strongly dependent on the seasonal flow in Mill Creek.

7.3 Material Reuse

The soils underlying the site have a high fines content. These soils are moderately to highly moisture sensitive, and will become disturbed and soft when exposed to inclement weather conditions. In our opinion, the site soils probably should not be used in wet weather conditions. However, if the on-site soils can be properly moisture conditioned and compacted to meet the project specifications, the on-site soils may be re-used to backfill areas that are located at least 5 feet away from the proposed culvert and headwalls.

7.4 Subgrade Protection

Rat slabs (unreinforced concrete mats) may facilitate forming and construction of the new cast-in-place footing. Rat slabs may be used without alteration of the foundation design parameters provided in this report.

7.5 Obstructions

Both natural and man-made obstructions are potentially present in the subsurface and may consist of cobbles or boulders or wood debris in the alluvial deposits, as well as the existing culvert and its appurtenances. The Contractor should be prepared to remove or clear obstructions if encountered during new abutment construction. The contract special provisions should alert the contractor to the potential presence of boulders or wood debris and the possible need to remove obstructions during foundation excavations.

8.0 ADDITIONAL SERVICES

PanGEO should review the final project plans and specifications to confirm that our geotechnical recommendations were properly incorporated into the contract documents.

9.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

PanGEO, Inc. (PanGEO) prepared this report for Natural Systems Design and Chelan County. The recommendations contained in this report are based on a site reconnaissance, a subsurface exploration program, review of pertinent subsurface information, and our understanding of the project.

Variations in soil conditions may exist between the locations of the explorations and the actual conditions underlying the site. The nature and extent of soil variations may not be evident until construction occurs. If any soil conditions are encountered at the site that are different from those described in this report, PanGEO should be immediately notified to review the applicability of the recommendations presented herein. Additionally, PanGEO should also be notified to review the applicability of these recommendations if there are any changes in the project scope.

This report may be used only by the client and for the purposes stated, within a reasonable time from its issuance. Land use, site conditions (both off and on-site), or other factors including advances in our understanding of applied science, may change over time and could materially affect our findings. Therefore, this report should not be relied upon after 36 months from its issuance. PanGEO should be notified if the project is delayed by more than 36 months from the date of this report so that the applicability of the conclusions and recommendations presented herein may be evaluated considering the time lapse.

Within the limitations of scope, schedule and budget, PanGEO engages in the practice of geotechnical engineering and endeavors to perform its services in accordance with generally accepted professional principles and practices at the time this report and/or its contents was prepared. No warranty, express or implied, is made. The scope of PanGEO's work did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous or toxic substances in the soil, surface water or groundwater at this site. PanGEO does not practice or consult in the field of safety engineering. PanGEO does not direct the contractor's operations, and cannot be held responsible for the safety of personnel other than our own on the site; the safety of others is the responsibility of the contractor.

It is the client's responsibility to see that all parties to this project, including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety. The use of information contained in this report for bidding purposes shall be at the contractor's sole option and risk. Any party other than the client who wishes to use this report shall notify PanGEO of such intended use and for permission to copy this report. Based on the intended use of the report, PanGEO may require that additional work be performed and that an updated report be reissued. Noncompliance with any of these requirements will release PanGEO from any liability resulting from the use this report.

PanGEO is pleased to support the design team and Chelan County with geotechnical engineering recommendations. If you have any questions regarding this report, please call (206) 262-0370.

Sincerely,

PanGEO, Inc.

DRAFT **DRAFT**

Nicholas Weikel, E.I.T.
Project Geotechnical Engineer

Siew L. Tan, P.E.
Principal Geotechnical Engineer

10.0 REFERENCES

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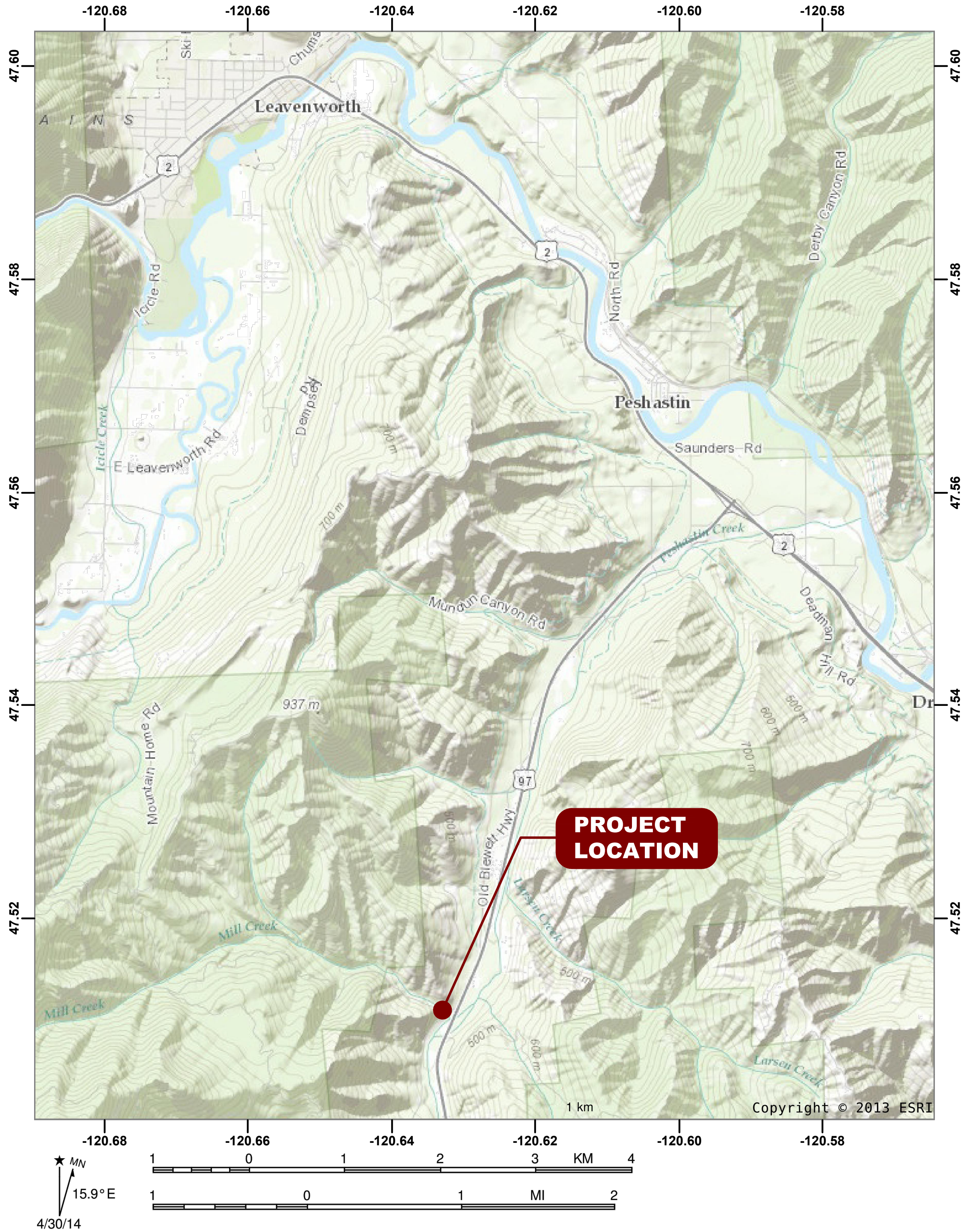
FIGURES



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DATE: 7/8/2019

CHECKED BY: SLT

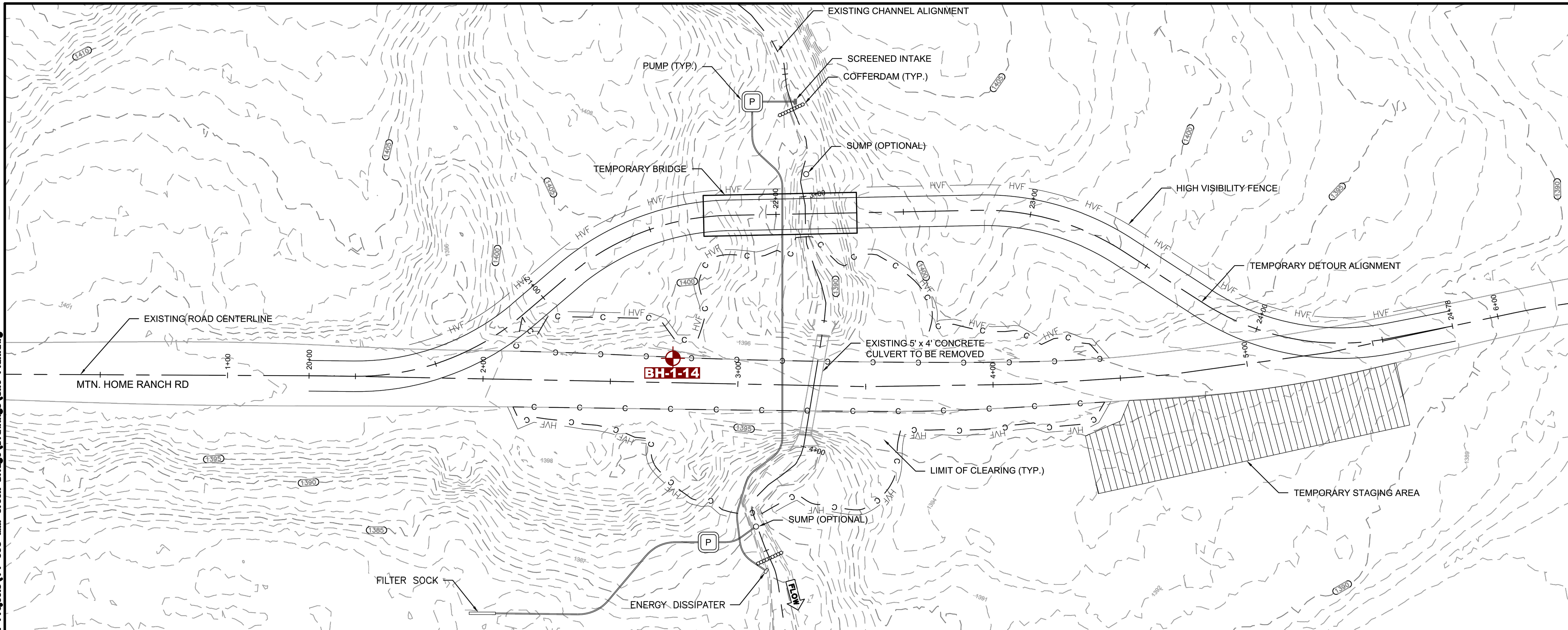
DRAWN BY: NTW

PanGEO INCORPORATED

Mill Creek
Fish Passage Project
Chelan County, Washington

VICINITY MAP

PROJECT NO.	FIGURE NO.
14-095	1



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DATE: 7/8/2019

CHECKED BY: SLT

DRAWN BY: NTW

LEGEND

BH-# Approximate Soil Boring by PanGEO, Inc. (2014 - Appendix A)

NOTES

1. Base map and topography derived from 30% Design Plan Set provided by Natural Systems Design on 6/19/2019.
2. Location of boring are approximate and based on the relative locations of known site features.
3. Features are provided for relative information only and are not a substitution for field survey.
4. Vertical Datum: NAVD 88

20' 0 20' 40'
scale in feet
Scale: 1" = 40'

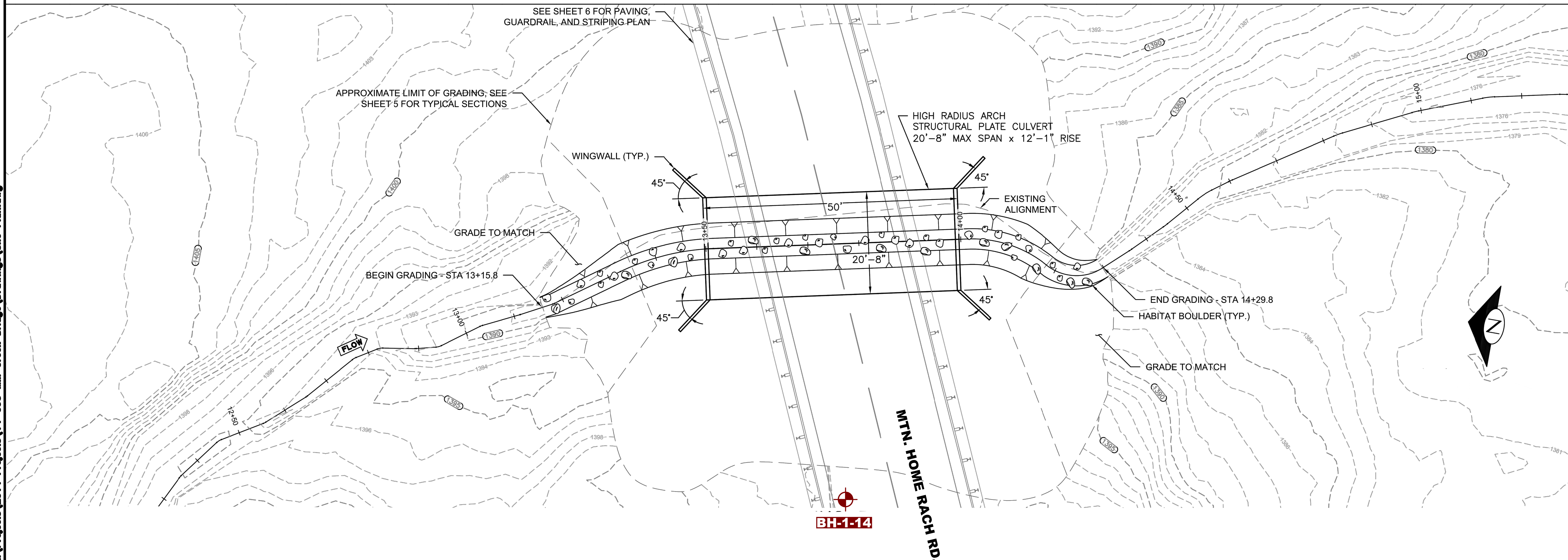
PanGEO INCORPORATED

Mill Creek
Fish Passage Project
Chelan County, Washington

Site and Exploration Plan
Existing Condition

PROJECT NO. 14-095

FIGURE NO. 2A



LEGEND

BH-# Approximate Soil Boring by PanGEO, Inc. (2014 - Appendix A)

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3. Features are provided for relative information only and are not a substitution for field survey.
4. Vertical Datum: NAVD 88

10' 0 10' 20'
scale in feet
Scale: 1" = 20'

PanGEO INCORPORATED

Mill Creek Fish Passage Project Chelan County, Washington	Site and Exploration Plan Proposed Improvements	
	PROJECT NO. 14-095	FIGURE NO. 2B

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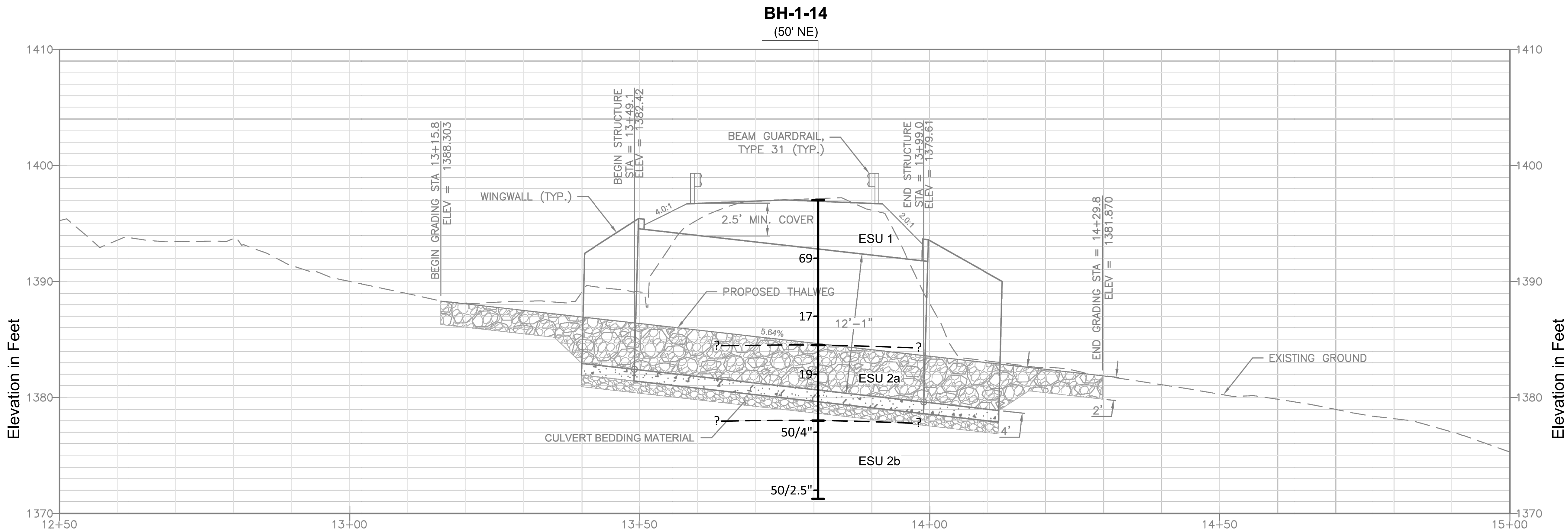

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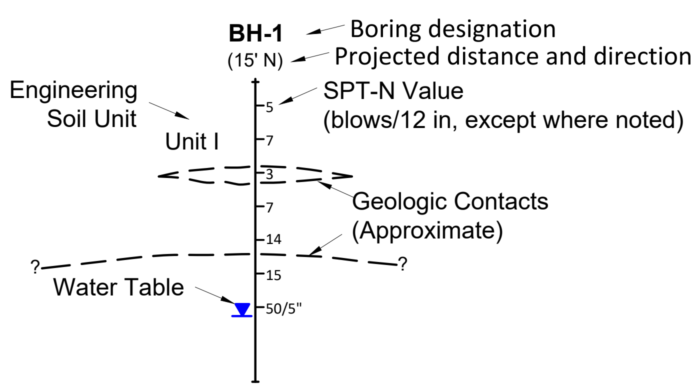

NOTES

1. Profile and topography derived from 30% Design Plan Set provided by Natural Systems Design on 6/19/2019.
2. Location of explorations were located on the current base mapping relative to existing and known site features and are therefore considered approximate and not a substitute for the accuracy of field surveys. Refer to Figure 2 for approximate boring locations.
3. Summary borings logs are presented in Appendix A.
4. All elevations referenced to North American Vertical Datum of 1988 (NAVD 88).
5. Geologic contacts between borings are inferred.

ENGINEERING SOIL UNITS

See text of report for soil unit descriptions.

ESU 1 **Fill / Re-worked Allubium (Hf)**: Very dense, dark grey, fine to coarse grained basalt gravel with dark brown silt and clay matrix. In general, the material appeared to consist mainly of basalt with medium to high plasticity fines and scattered quartz.

ESU 2a **Colluvium / Mass Wasting Deposit (Hc / Hls)**: Medium dense, dark grey, clayey basalt gravel with a grey and tan clay and silt matrix. This soil unit was characterized by its medium to highly plastic fines, which appeared weathered with white and reddish mottles

ESU 2b **Colluvium / Mass Wasting Deposit (Hc / Hls)**: Very dense, dark grey, basalt gravels, cobbles, and bounders with some fines. This soil unit was characterized by its very dense / massive state, soil cuttings and extremely difficult drilling action. This is the deepest soil unit encountered in our test boring due to practical refusal on a very large basalt boulder, or possibly bedrock, at approximately 26 feet below the ground surface

H. Scale: 1" = 10'
5 0 5 10
10 0 10 20
V. Scale: 1" = 20'

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Mill Creek
Fish Passage Project
Chelan County, Washington

GENERALIZED
SUBSURFACE PROFILE
CULVERT ALIGNMENT

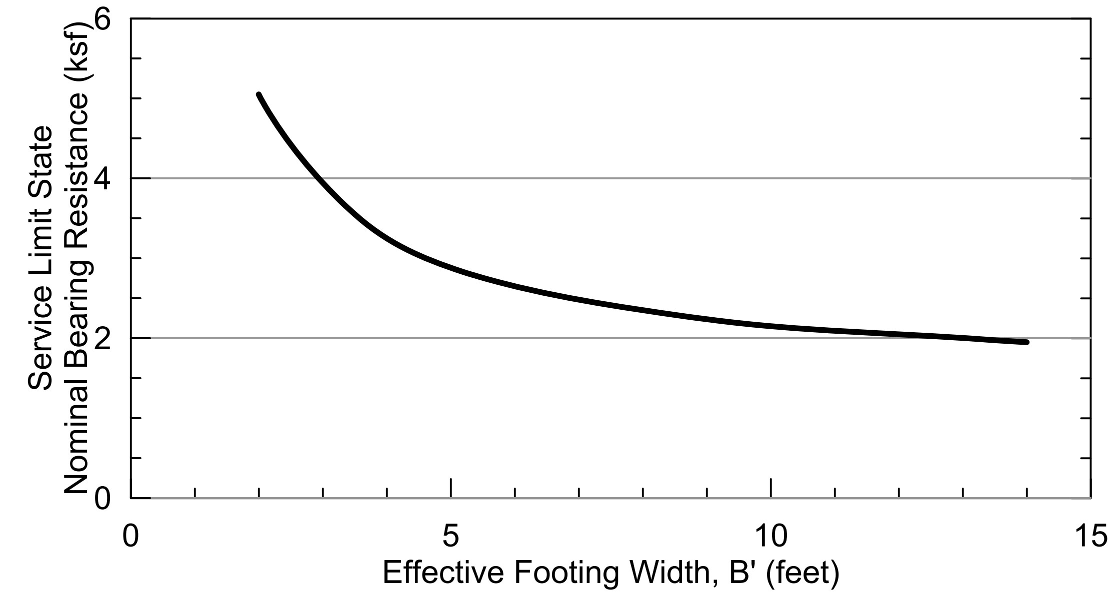
PROJECT NO.	FIGURE NO.
14-095	3

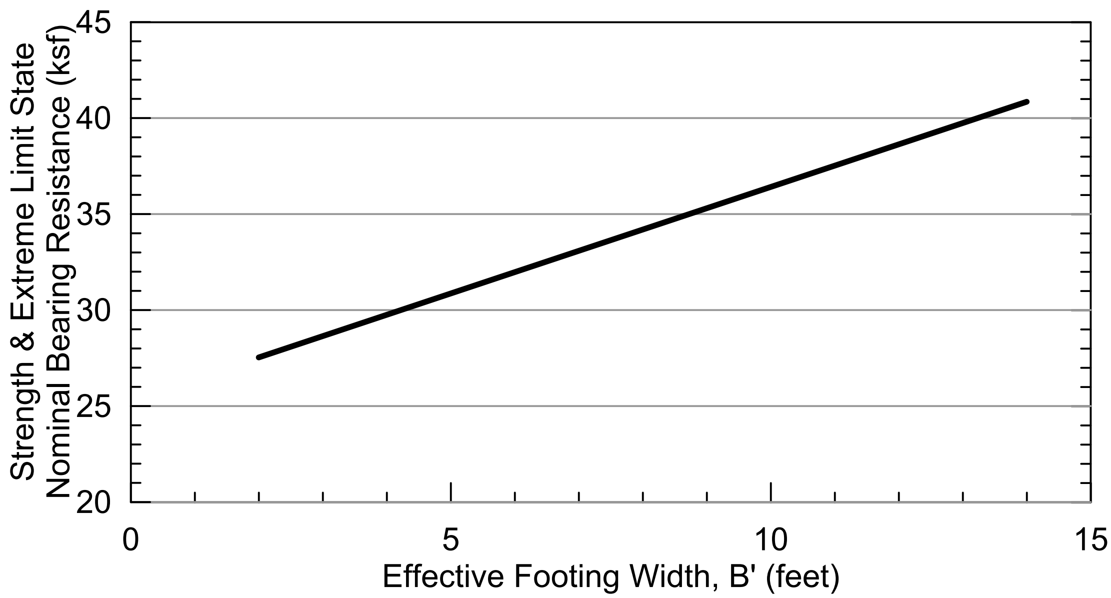
Z:\Projects\2014 Projects\14-095 Mill Creek Bridge\Drawings\Subsurface.dwg

DATE: 7/8/2019

CHECKED BY: SLT

DRAWN BY: NTW

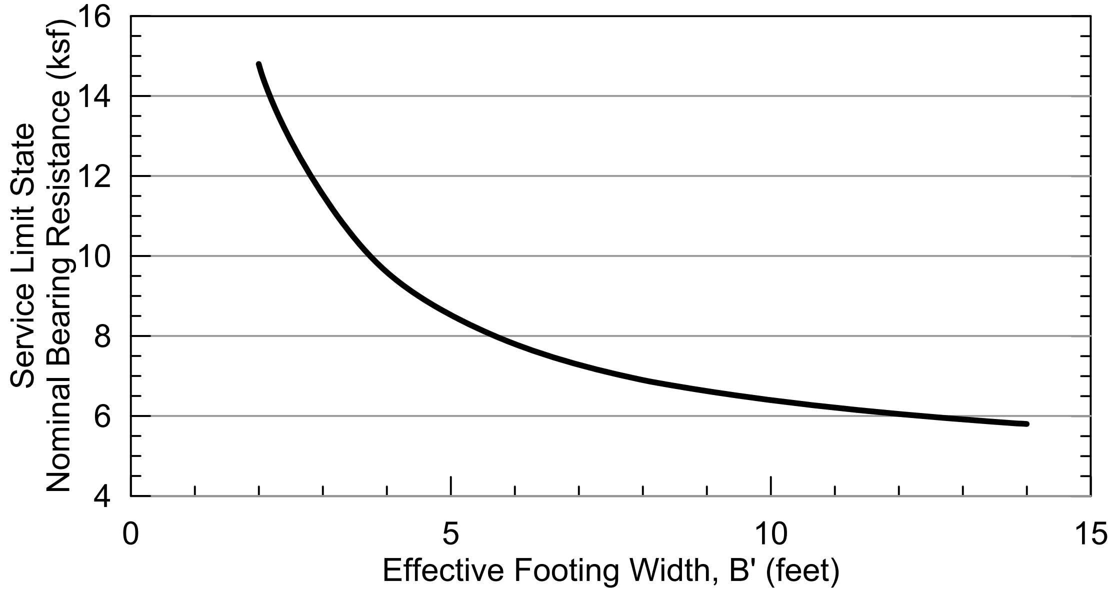


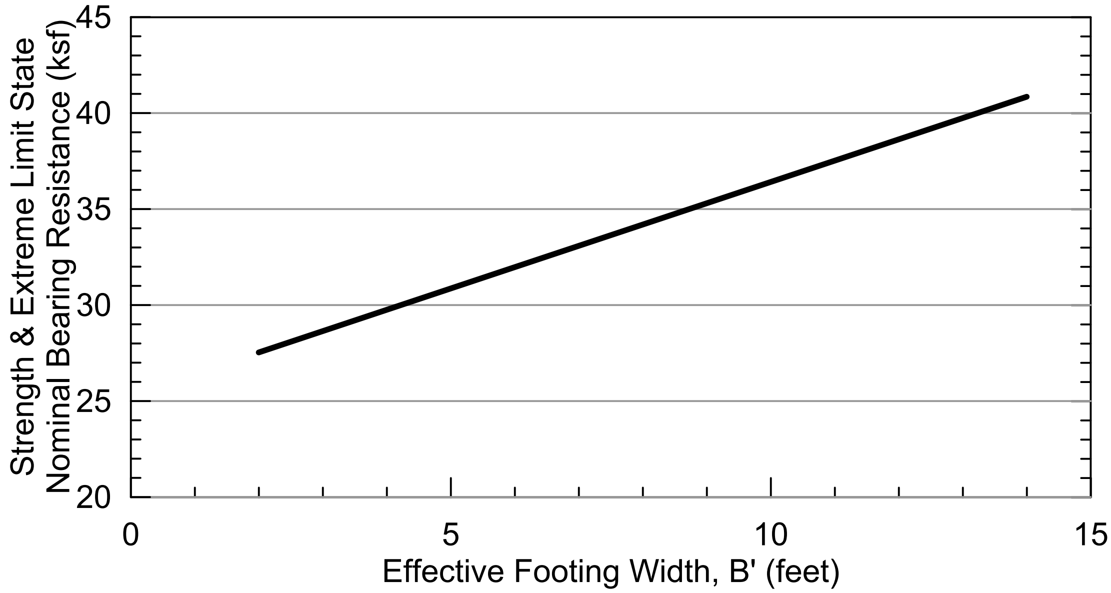


Notes:
1) Service limit state nominal resistance developed for settlement of 1/2-inch or less.
2) Resistance factor for strength limit state load combinations may be taken as $\varphi_b = 0.45$ per AASHTO LRFD 10.5.5.2.2.
3) Resistance factor for extreme limit state load combinations may be taken as 1.0.

14-095_SpreadFtgLRFD.grf w/14-095 B vs Q.xls 7/9/19 (12:39) NTW

PanGEO INCORPORATED	**Mill Creek** **Fish Passage Project** **Chelan County, Washington**	**NOMINAL BEARING RESISTANCE** **SPREAD FOOTINGS** **(Settlement Limited to 1/2-inch)**
		Project No. **14-095** / Figure No. **4A**





Notes:

1) Service limit state nominal resistance developed for settlement of 1-inch or less.
2) Resistance factor for strength limit state load combinations may be taken as $\varphi_b = 0.45$ per AASHTO LRFD 10.5.5.2.2.
3) Resistance factor for extreme limit state load combinations may be taken as 1.0.

14-095_SpreadFtgLRFD.grf w/14-095 B vs Q.xls 7/9/19 (12:40) NTW

PanGEO INCORPORATED	Mill Creek Fish Passage Project Chelan County, Washington	**NOMINAL BEARING RESISTANCE SPREAD FOOTINGS (Settlement Limited to 1-inch)**	
		Project No. **14-095**	Figure No. **4B**

APPENDIX A

FIELD EXPLORATIONS

APPENDIX A: FIELD EXPLORATIONS

Appendix A contains a summary of exploration methods and borehole log presenting the factual and interpretive results of our exploratory drilling program on the subject site. The descriptions of the materials encountered in the subsurface explorations are based on the soil samples extracted from the boring. The sample descriptions are augmented by observation of the drilling action and drill cuttings brought to the surface during field operations. The paragraphs below describe the field operations and sampling procedures used during the geotechnical field explorations.

FIELD EXPLORATIONS

The subsurface exploration program consisted of drilling one test boring as near as practical to the likely location of the southern abutment of the new proposed culvert as shown on the Site and Exploration Plan (Figures 2A & 2B). The test boring was drilled using a BK-81 truck-mounted drill rig owned and operated by Holocene Drilling of Puyallup, Washington, under a subcontract to PanGEO. The test boring was designated BH-1-14 and was advanced to a maximum depth of 25¾ feet below the ground surface on April 28, 2014 using mud rotary drilling methods.

The drilling was performed near the northern edge of the driving surface. Two representatives from Chelan County provided traffic control along Mountain Home Ranch Road. The location was chosen based on accessibility, and to avoid impacts to overhead and underground utilities. The boring met effective refusal on a large basalt boulder or possibly bedrock, short of the planned depth of 40 feet.

A representative of PanGEO logged the test boring. Soil samples were collected from selected intervals in the boring. The location of the boring was measured from existing site features and should be considered approximate.

SAMPLING METHODS

Standard penetration tests were taken at 5-foot depth intervals, starting at 5 feet below ground surface and continuing to the bottom of the boring. The number of blows to drive the sampler each 6 inches over an 18-inch interval was recorded and indicated on the boring log. The number of blows to drive the sampler the final 12 inches is termed the SPT resistance, or N-value, and is used to evaluate the strength and consistency/relative density of the soil. The hammer used to perform SPT sampling was an automatic trip-release mechanism, which generally delivers a higher energy than a “standard” hammer equipped with a rope and cathead mechanism. The efficiency of the hammer mechanism is considered when evaluating the liquefaction potential of a soil. The SPT N-values reported on the borehole logs are field values, and are therefore not corrected for hammer efficiency, overburden stress or rod lengths.

An engineer from PanGEO was present throughout the field exploration program to observe the boring, assist in sampling, and to prepare a descriptive log of the exploration. Soils were classified in general accordance with the guidelines shown on Figure A-1. A summary boring log is included as Figure A-2. The stratigraphic contacts shown on the summary log represents the approximate boundaries between soil types; actual stratigraphic contacts encountered at other locations in the field may differ from the contact elevations shown on the logs, and may be gradual rather than abrupt. The soil and groundwater conditions depicted are only for the specific date and locations reported, and therefore, are not necessarily representative of other locations and times.

RELATIVE DENSITY / CONSISTENCY

SAND / GRAVEL			SILT / CLAY		
Density	SPT N-values	Approx. Relative Density (%)	Consistency	SPT N-values	Approx. Undrained Shear Strength (psf)
Very Loose	<4	<15	Very Soft	<2	<250
Loose	4 to 10	15 - 35	Soft	2 to 4	250 - 500
Med. Dense	10 to 30	35 - 65	Med. Stiff	4 to 8	500 - 1000
Dense	30 to 50	65 - 85	Stiff	8 to 15	1000 - 2000
Very Dense	>50	85 - 100	Very Stiff	15 to 30	2000 - 4000
			Hard	>30	>4000

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS		GROUP DESCRIPTIONS	
Gravel 50% or more of the coarse fraction retained on the #4 sieve. Use dual symbols (eg. GP-GM) for 5% to 12% fines.	GRAVEL (<5% fines)	GW	Well-graded GRAVEL
		GP	Poorly-graded GRAVEL
	GRAVEL (>12% fines)	GM	Silty GRAVEL
		GC	Clayey GRAVEL
Sand 50% or more of the coarse fraction passing the #4 sieve. Use dual symbols (eg. SP-SM) for 5% to 12% fines.	SAND (<5% fines)	SW	Well-graded SAND
		SP	Poorly-graded SAND
	SAND (>12% fines)	SM	Silty SAND
		SC	Clayey SAND
Silt and Clay 50%or more passing #200 sieve	Liquid Limit < 50	ML	SILT
		CL	Lean CLAY
		OL	Organic SILT or CLAY
	Liquid Limit > 50	MH	Elastic SILT
		CH	Fat CLAY
		OH	Organic SILT or CLAY
Highly Organic Soils		PT	PEAT

Notes:

1. Soil exploration logs contain material descriptions based on visual observation and field tests using a system modified from the Uniform Soil Classification System (USCS). Where necessary laboratory tests have been conducted (as noted in the "Other Tests" column), unit descriptions may include a classification. Please refer to the discussions in the report text for a more complete description of the subsurface conditions.

2. The graphic symbols given above are not inclusive of all symbols that may appear on the borehole logs. Other symbols may be used where field observations indicated mixed soil constituents or dual constituent materials.

DESCRIPTIONS OF SOIL STRUCTURES

Layered:	Units of material distinguished by color and/or composition from material units above and below	**Fissured:**	Breaks along defined planes
Laminated:	Layers of soil typically 0.05 to 1mm thick, max. 1 cm	**Slickensided:**	Fracture planes that are polished or glossy
Lens:	Layer of soil that pinches out laterally	**Blocky:**	Angular soil lumps that resist breakdown
Interlayered:	Alternating layers of differing soil material	**Disrupted:**	Soil that is broken and mixed
Pocket:	Erratic, discontinuous deposit of limited extent	**Scattered:**	Less than one per foot
Homogeneous:	Soil with uniform color and composition throughout	**Numerous:**	More than one per foot
		BCN:	Angle between bedding plane and a plane normal to core axis

COMPONENT DEFINITIONS

COMPONENT	SIZE / SIEVE RANGE	COMPONENT	SIZE / SIEVE RANGE
Boulder:	> 12 inches	Sand	
Cobbles:	3 to 12 inches	Coarse Sand:	#4 to #10 sieve (4.5 to 2.0 mm)
Gravel		Medium Sand:	#10 to #40 sieve (2.0 to 0.42 mm)
Coarse Gravel:	3 to 3/4 inches	Fine Sand:	#40 to #200 sieve (0.42 to 0.074 mm)
Fine Gravel:	3/4 inches to #4 sieve	Silt	0.074 to 0.002 mm
		Clay	<0.002 mm

TEST SYMBOLS

for In Situ and Laboratory Tests listed in "Other Tests" column.

ATT	Atterberg Limit Test
Comp	Compaction Tests
Con	Consolidation
DD	Dry Density
DS	Direct Shear
%F	Fines Content
GS	Grain Size
Perm	Permeability
PP	Pocket Penetrometer
R	R-value
SG	Specific Gravity
TV	Torvane
TXC	Triaxial Compression
UCC	Unconfined Compression

SYMBOLS

Sample/In Situ test types and intervals

- 2-inch OD Split Spoon, SPT (140-lb. hammer, 30" drop)
- 3.25-inch OD Spilt Spoon (300-lb hammer, 30" drop)
- Non-standard penetration test (see boring log for details)
- Thin wall (Shelby) tube
- Grab
- Rock core
- Vane Shear

MONITORING WELL

- Groundwater Level at time of drilling (ATD)
- Static Groundwater Level
- Cement / Concrete Seal
- Bentonite grout / seal
- Silica sand backfill
- Slotted tip
- Slough
- Bottom of Boring

MOISTURE CONTENT

Dry	Dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water

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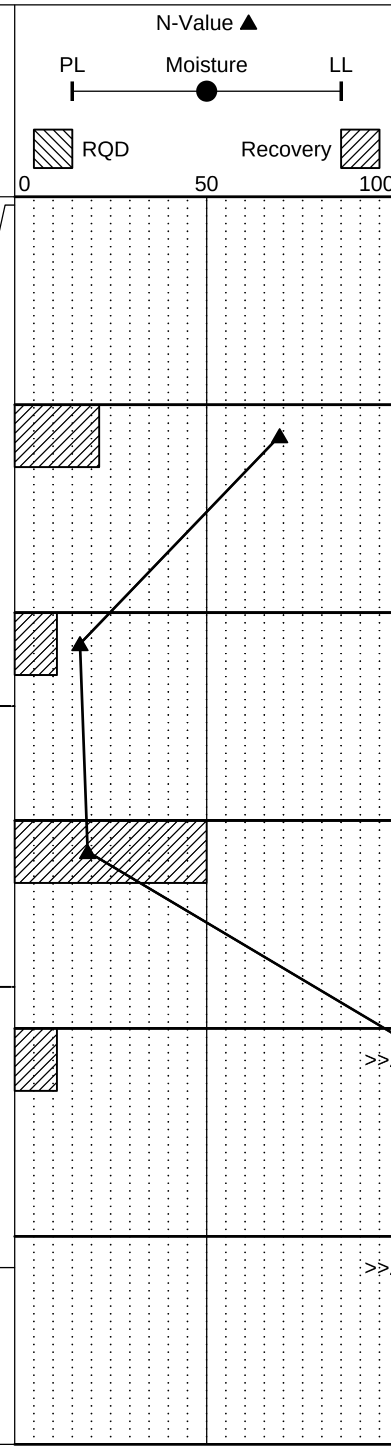
Terms and Symbols for Boring and Test Pit Logs

Figure A-1

Project:	Mill Creek Bridge	Surface Elevation:	~1397 ft
Job Number:	14-095	Top of Casing Elev.:	N/A
Location:	Chelan County, Washington	Drilling Method:	BK-81, Mud Rotary
Coordinates:	Northing: , Easting:	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0						2.5-inch layer of ASPHALT. Observed abundant large boulders and cobbles in the vicinity of the project area.
						Very dense, dark grey, fine to coarse grained basalt GRAVEL with dark brown silt and clay matrix: angular, medium to high plastic fines, scattered quartz [FILL / RE-WORKED ALLUVIUM].
						- Observed severe rig chatter and audible grinding on gravels between 0 to 5.5 ft BGS.
5	S-1		20 40 29			- Driller reports smoother drilling action below 5.5 ft BGS.
10	S-2		8 9 8			- Becomes medium dense with minor amount of dark brown silt and clay.
						Medium dense, dark grey, clayey fine grained basalt GRAVEL with grey and tan clay and silt: wet, angular to sub-angular, weathered, white and reddish mottles, medium to high plastic fines, disrupted [COLLUVIUM / MASS WASTING DEPOSIT].
15	S-3		10 11 8			
						Very dense, dark grey, basalt GRAVEL, COBBLES and BOULDERS with some fines: massive [COLLUVIUM / MASS WASTING DEPOSIT].
20	S-4		31 50/4			- Observed severe rig chatter and audible grinding on gravels below 19 ft BGS.
						- Drilling became extremely difficult below 24 ft BGS.
25	S-5		31 50/2.5			- Refusal on basalt bedrock or large boulder.
						Boring terminated at about 25.75 feet below the ground surface. Groundwater appears to be perched above the clayey gravel layer.
30						



Completion Depth:	25.0ft	Remarks:
Date Borehole Started:	4/28/14	
Date Borehole Completed:	4/28/14	
Logged By:	N. Weikel	
Drilling Company:	Holocene Drilling	

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LOG OF TEST BORING BH-1-14

Figure A-2

The stratification lines represent approximate boundaries. The transition may be gradual.