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GEOTECHNICAL ENGINEERING STUDY
CHIEF JOSEPH DAM BRIDGE REPLACEMENT
DOUGLAS COUNTY, WASHINGTON

1.0 INTRODUCTION

1.1 Purpose

The purpose of this report is to provide subsurface data, site and geologic conditions, seismic design criteria, and foundation recommendations for the Chief Joseph Dam bridge replacement. We conducted our services in general accordance with our May 29, 2013 scope of services.

Shannon & Wilson prepared this report for KPFF Consulting Engineers (KPFF) and Douglas County. Do not use or rely upon this report for other locations or purposes without the written consent of Shannon & Wilson, Inc.

1.2 Scope of Services

To prepare this report, Shannon & Wilson:

- Drilled, logged, and sampled two borings;
- Reviewed geologic maps for the alignment;
- Reviewed available geotechnical information at the project site and vicinity;
- Reviewed driving records of piles driven in 2003 for the intermediate trestle piers of the existing bridge; and
- Performed engineering analyses based on our subsurface explorations.

2.0 PROJECT LOCATION AND DESCRIPTION

The project site is located on Pearl Hill Road, southwest of Chief Joseph Dam, in the Bridgeport Washington area (Figure 1). An existing, 130-foot long truss span with five short approach spans crosses the Foster Creek valley at the site. The valley is approximately 40 to 50 feet deep with steep side slopes, approximately 1.5 horizontal to 1 vertical (1.5H:1V) to 2.5H:1V inclinations.

In 2002, Hart-Crowser (HC) completed a geotechnical study at the site for reconstruction of the short approach spans. The new approach spans were supported on driven pipe piles.
We understand current plans are for replacement the entire bridge structure. The bridge alignment will remain the same, however, the road and bridge elevations may change. The new structure will be a single span, post-tension concrete girder structure. The span length will be approximately 240 feet with abutments and wing walls at each end.

In order to permit widening of Pearl Hill Road west of the bridge, we understand that the west approach along Pearl Hill Road may be elevated.

### 3.0 EXPLORATIONS

Shannon & Wilson completed drilling explorations at the bridge site on July 18, 2013. In the field, we located our explorations by measuring/estimating from existing site features, such as bridge corners and recorded the boring locations using a handheld GPS unit. Douglas County also surveyed the boring locations. We show the exploration locations in Figure 2 (Site and Exploration Plan).

Our subcontractor (Haz-Tech Drilling of Meridian, Idaho) drilled the borings with a CME 75, truck-mounted drill rig, using approximately 3¼-inch inside diameter, 4-inch outside diameter (O.D.), HW casing advancer and diamond rock coring methods. We obtained disturbed soil samples of the overburden soils at approximately 2½- to 5-foot intervals using a 2.0-inch O.D. Standard Penetration Test (SPT) sampler. Where we encountered large gravels, cobbles, and boulders, we used a 2.5-inch O.D., and a 3.0-inch O.D. samplers in an attempt to achieve better sample recovery. All penetration tests were performed in general accordance with ASTM International (ASTM) D 1586 test procedures. Samplers were driven up to 18 inches (three 6-inch increments) below the casing with an automatic hammer, weighing 140 pounds and free-falling 30 inches.

A Shannon & Wilson engineer observed and logged the explorations, directed the sampling, and obtained samples for manual-visual classification and laboratory testing. Our field representative placed drive samples into labeled ziplock bags for laboratory identification. We recorded the number of blows required to advance the split-spoon through each 6-inch increment. The SPT resistance, or N-value, is defined as the number of blows required to drive the SPT sampler from 6 to 18 inches below the drill casing. The SPT N-value is reported as the number of blows per 1-foot of penetration. When 50 blows are required to achieve penetration of 6 inches or less, we halt testing and record the number of blows with the corresponding penetration. The SPT N value provides an indication of the relative density, or consistency, of the soil and is plotted on the boring logs. The N-value for oversized split spoon samplers can be correlated with the SPT
N-value. Some of the N-values may be elevated due to testing in large gravels, cobbles, and boulders and may not be reliable.

During the coring, we recorded the core recovery and Rock Quality Designation (RQD) for each run. We then labeled and placed the recovered core in core boxes. The core recovery rate and RQD provide an indication of the rock competency and data for empirical correlations to develop design parameters. The rock recovery and RQD is presented on the boring logs in Appendix A. Photographs of the rock core are also attached in Appendix A.

We estimated soil strata boundaries in the field based on the drill action and disturbed samples (i.e., SPT drive samples, drill cuttings, etc.), as appropriate. The subsurface conditions are known only at exploration locations on the dates explored and should be considered approximate. Actual subsurface conditions may vary between explorations and within the general vicinity of the proposed improvements.

4.0 LABORATORY TESTING

We completed the following laboratory tests on selected representative soil samples obtained from the exploratory borings.

- Moisture Content (ASTM D 2216)
- Unconfined Compressive Test (ASTM D 7142-C)

Laboratory test results are attached in Appendix B.

5.0 GEOLOGY AND SUBSURFACE CONDITIONS

5.1 General Geology

In 1988, the U.S. Army Corp of Engineers (USACE) produced a report for the Chief Joseph Dam on the Columbia River (Eckerlin and Bailey, 1988). The report provides geologic description for the general area including the mouth of the Foster Creek valley.

The project area lies near the boundary of the Okanogan-Selkirk Highlands and the Columbia Plateau physiographic provinces. The bedrock in the lower valley and the Okanogan Highlands are old metamorphic rocks intruded by granitic rocks. Glacial outwash, lacustrine silts, and till overlie the bedrock. Some of the outwash deposits are coarse (sand, gravel, cobbles and boulders), having been deposited by very high energy flow during the Missoula Floods. The present river has incised the glacial sediments and locally flows on the granitic bedrock, creating
a terraced inner valley within a wider, older valley. Columbia River Basalt overlies the glacial deposits on the upper slopes, south of the Columbia River.

5.2 Site Geology

The south bank of the Columbia River contains bedrock that generally rises above the river channel. The Chief Joseph Dam Bridge spans an abandoned tributary canyon known as Foster Creek. The Foster Creek valley was likely formed from the tributary stream cutting through the glacial deposits creating a relatively steep-sided valley. The glacial deposits consist of glacial till, morainal materials, and large quantities of glaciofluvial and glaciolacustrine sediments. The glacial till typically consist of compacted, non-stratified, silty, sandy gravel. Morainal deposits consist of moderately compacted gravel with some sand and silt. Glaciofluvial deposits typically include stratified silty, sandy, gravel, but may also include sandy gravel, openwork gravel, and boulder beds. Such deposits contains little to no fine-grained soil in their matrices. Open-work texture is common. Glaciolacustrine deposits are stratified silt and clay. The existing ground surface is a mixture of these units.

The bedrock is a collection of hard, crystalline rocks that include granodiorite, granodiorite gneiss, dark schistose granodiorite, hornblende granodiorite, and lamprophyre. Granodiorite is the predominant rock type. Granodiorite is a medium- to coarse- grained, hard, rock that is light to dark gray. Granodiorite gneiss is a medium- to coarse- grained, hard, rock that is light to dark gray and typically exhibits a banded structure. Schistose granodiorite is a fine- to medium-grained, soft to hard, rock that is dark in color and contains up to 50 percent biotite or hornblende. Hornblende granodiorite is a hard, medium to dark in color, rock that contains randomly oriented, medium- to coarse- grained crystals of hornblende. Lamprophyre is a very hard, dark green rock that is typically found in dikes.

5.3 Subsurface Conditions

Boring B-1 encountered approximately 26½ feet of interbedded silty, sandy, gravel with some cobbles and silty, gravelly, sand with cobbles. The gravel and cobbles are typically subrounded to subangular and estimated to range in size from 1- to 10- inch diameter. SPT blow counts range from 24 to greater than 100. SPT blow counts are likely elevated due to the oversized gravels and cobbles that are present. Based on the blow counts in the sandy layers, the unit is likely medium dense. The boring encountered open-work gravel, cobbles, and boulders from 26½ to 65½ feet. The gravel, cobbles, and boulders are sub angular to angular and estimated to range in size from 1 to 48 inches in diameter. The drill encountered some thin soil layering and
several large void areas; represented by the near-zero penetration resistance. The driller measured a void near the 40-foot depth that is approximately 4 feet thick. We cored the large boulders and cobbles at approximately 47 feet using HQ diamond bit rock coring methods. The coring process provided greater recovery than SPT sampling. The drill encountered granodiorite bedrock below approximately 65½ feet. The rock is light to dark gray, medium to high strength with little to no weathering. RQD values in the granodiorite ranged from 58 to 93 percent, indicating fair to good quality.

Boring B-2 encountered approximately 6½ feet of silty, sandy, gravel with some cobbles. The gravel and cobbles are typically subrounded to subangular and estimated to range in size from 1- to 10- inch diameter. SPT blow counts range from 24 to greater than 100. SPT blow counts are likely elevated due to the oversized gravels and cobbles that are present. Based on the lower blow counts in two SPT samples, the deposit is medium dense. The boring typically encountered sandy, gravel, cobbles, and boulders with trace silt from 6½ to 14 feet. We cored the large boulders and cobble at approximately 14 feet using HQ diamond bit rock coring methods. Subrounded to subangular, open-work gravel, cobbles, and boulders, estimated to range in size from 1- to 24- inch diameter, were encountered from 14 to 40 feet. The boring typically encountered sandy gravel, cobbles, and boulders with trace silt from approximately 40 to 58 feet. The boulders are sub angular to angular, the gravel and cobbles are subrounded to subangular. The coring process provided greater recovery than SPT sampling. The drill encountered granodiorite bedrock below approximately 58 feet. The rock is light to dark gray, medium to high strength with little to no weathering. RQD values in the granodiorite typically ranged from 66 to 95 percent, indicating fair to good quality.

Borings completed by Shannon & Wilson, pile driving records from Hart Crowser, and USCOE explorations indicate that the bedrock surface is highly irregular, containing some steep gradients on the rock surface. Additionally, boulders, some of which are as hard as the bedrock, may result in refusal for driven piles at variable elevations.

Please see the boring logs in Appendix A for detailed subsurface conditions.

5.4 Groundwater

We did not observe groundwater during the drilling process. However, the drilling methods used to advance the boring required water circulation. Therefore, groundwater may exist.
6.0 PEARL HILL ROAD SLOPE RECONNAISSANCE

We performed a visual reconnaissance of the major slopes south of Pearl Hill Road, west of the Foster Creek valley. We understand that the purpose for our reconnaissance was to determine the potential hazards associated with widening the existing roadway and the possibility of placing a retaining wall along the south side of the Pearl Hill Road alignment directly west of the proposed bridge replacement project. Discussions with Douglas County personnel at the site further indicate that the Pearl Hill Road grade west of the bridge may be increased to allow for a wider road alignment without the need for retaining walls.

We observed, photographed, researched, and measured the existing slopes south of Pearl Hill Road. We collected general topographic data of the slopes using the USGS Quadrangle for Chief Joseph Dam, Washington. The exposed slopes typically show glacial outwash, lacustrine silts, and till with some coarser sand, gravel, cobble and boulder outwash deposits from Foster Creek to approximately 650 feet west of the creek. The existing roadway, approximately 650 to 1,250 feet west of Foster Creek, is cut into granodiorite type rocks. These rocks are similar to those encountered during the exploration phase of our work.

The exposed, north and east facing slopes near the Foster Creek valley are relatively steep, 1.5H:1V on the north and 1.25H:1V on the east. The slope heights, within approximately 650 feet of Foster creek, range from approximately 80 to 100 feet. The east slopes show signs of recent, surficial, isolated slides, approximately 500 to 700 feet southwest of the west abutment. While onsite, recent rainstorms in the area released several minor rock slides consisting of 5 to 25 gravel to boulder sized rocks sloughing down the east facing slopes. No rock slides were observed on the north facing slopes during our reconnaissance. The slopes flatten approximately 650 to 1,250 feet west of Foster creek. The rock cut slopes are near vertical and range from 3 to 15 feet tall. The exposed rock is relatively sound with some fracturing and little weathering.

7.0 SITE SEISMICITY

7.1 Seismic Hazards

Earthquake-induced geologic hazards typically include landsliding, fault rupture, settlement, and liquefaction phenomena and their associated effects (loss of shear strength, bearing capacity failures, loss of lateral support, ground oscillations, lateral spreading, etc.).

Liquefaction typically occurs when loose to medium dense, granular, saturated soils are subjected to ground shaking. Based on the relatively dense granular at shallow depths and
cobbles and boulders at deep elevation, we consider the potential for liquefaction at the site to be low. Based on the subsurface condition of boring logs B-1 and B-2, we anticipate the potential for slope instability at the site during a seismic event is low. There is no evidence of known fault on the site location and the potential for fault rupture in the site is low.

7.2 Seismic Design Criteria

We understand that the Chief Josef Dam Bridge will be designed based on American Association of State Highway and Transportation Officials Load Resistance Factor Design (LRFD) Bridge Design Specifications (AASHTO, 2012). Seismic parameters and design spectrum in AASHTO 2012 is based on U.S. Geological Survey (USGS) 2002 Probabilistic Seismic Hazard Analysis (PSHA). AASHTO uses 7 percent probability of exceedance earthquake event in 75 years (975-year return period earthquake event) for seismic design. The estimated seismic design parameters are presented below:

- Peak Ground Acceleration (PGA): 0.14g
- Short-period spectral acceleration (S_s): 0.32g
- Long-period spectral acceleration (S_1): 0.10g

The spectral values of USGS 2002 PSHA are provided for soft rock site condition and AASHTO 2012 requires adjusting the spectral values for site condition at the site. The site soil response factors in AASHTO (2012) are based on determination of the Site Class from subsurface condition. Based on evaluation of the geologic units and Standard Penetration Test (SPT) blow counts data in boring logs B-1 and B-2, it was our opinion that the site condition at the boring location is corresponding to Site Class C. Site Class C is characterized by soils that within 100 feet of the ground surface have an average shear wave velocity between 1,200 and 2,500 feet per second or an average SPT blow count above 50 (AASHTO, 2012).

We calculated the design spectrum for the site using the spectral values (i.e., PGA, S_s, and S_1) and Site Class C site factors. The standard design spectrum was calculated at each period in accordance with AASHTO (2012) and plotted in Figure 3. The site factor and adjusted spectral values for soil condition used for design spectrum are also presented in this figure.
8.0 FOUNDATION SUPPORT

8.1 General

As discussed in Section 5.3, borings B-1 and B-2 encountered sand, gravel, cobbles, and boulders over granodiorite bedrock. Open-work gravel, cobbles, and boulders were encountered from 26½ to 65½ feet in boring B-1 and from 14 to 40 feet in boring B-2. Several large void areas were encountered during drilling. The granodiorite bedrock was encountered below approximate depth of 65½ feet and 58 feet in borings B-1 and B-2, respectively. However, our review of pile driving records from Hart Crowser, and USCOE explorations indicate that the top of bedrock is highly irregular, containing some steep gradients on the rock surface.

Because of the presence of open-work gravel, cobbles, and boulders and the potential presence of voids, foundations supporting the replacement bridge should bear in the underlying bedrock. Several foundation options were evaluated for support of the replacement bridge. These options included spread footings, drilled shafts, micropiles, and driven, steel H-piles. Considering access limitations onto valley slopes, installation difficulties into the cobbles and boulders underlying the site, and cost, we propose driven, steel H-piles as the most feasible foundation option.

8.2 Driven Pile Design

We understand the replacement bridge, in keeping with our recommendations, will be designed using driven, steel H-piles. We understand that HP 18x204 piles will be used to support the replacement bridge. These piles should be driven to refusal on the underlying bedrock. Per Section 10.7.3.2.3 of AASHTO LRFD (2012), the nominal resistance of piles bearing on hard rock is controlled by the structural limit state. For piles bearing on hard rock, group efficiency (reduction) factors are not required.

Some piles may encounter refusal driving conditions on the boulders underlying the site before reaching bedrock. In addition, because voids may be present beneath the boulders, we recommend driving approximately 20 percent more piles than required to support the new bridge abutments.

Because of the potential highly irregular bedrock surface, the actual pile lengths may vary during installation. Project specifications should consider this variability.
8.3 Estimated Settlement

Assuming pile design and installation in accordance with our recommendations contained herein, we estimate total settlements will be less than 1-inch, with differential settlement between abutments approximately one-half of estimated total settlement.

8.4 Lateral Resistance

We anticipate lateral loads acting on the structure resulting from wind, seismic forces, unbalanced earth pressures, and other loading may be resisted by the lateral resistance provided by the piles and the passive earth pressure against the pile caps. The frictional resistance developed between the sides of the pile caps and surrounding soils may also provide resistance to lateral loads. Lateral pile resistance is presented below.

8.4.1 Pile Lateral Resistance

The lateral load resistance of driven piles is a complex soil-structure interaction problem that takes into account pile stiffness and the substratum’s varying resistance as the pile deflects laterally. The lateral resistance developed by the pile foundations depends on the subsurface conditions encountered, pile type, spacing, the degree of fixity at the pile top, and allowable pile deflections. We provide LPILE® geotechnical parameters for piles supporting the abutment, based on subsurface conditions encountered, for assessment of lateral pile deflections at the west abutment in Table 1, and Table 2 for the east abutment.

<table>
<thead>
<tr>
<th>Soil Layer</th>
<th>Top of Layer Elevation (feet)</th>
<th>Effective Unit Weight, (\gamma') (lb/ft³)</th>
<th>Internal Friction Angle, (\phi) (degrees)</th>
<th>Uniaxial Compressive Strength (lb/in²)</th>
<th>p-y Modulus, (k) (lb/in³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Medium dense, poorly graded Gravel/Cobbles</td>
<td>876</td>
<td>125</td>
<td>34</td>
<td>-</td>
<td>100</td>
</tr>
<tr>
<td>Medium-dense to dense Sand with gravel</td>
<td>868.5</td>
<td>130</td>
<td>36</td>
<td>-</td>
<td>110</td>
</tr>
<tr>
<td>Medium-dense to dense Gravel with sand</td>
<td>863.5</td>
<td>130</td>
<td>36</td>
<td>-</td>
<td>110</td>
</tr>
<tr>
<td>Medium-dense to dense Sand with gravel/cobbles</td>
<td>860</td>
<td>125</td>
<td>34</td>
<td>-</td>
<td>100</td>
</tr>
<tr>
<td>Soil Layer</td>
<td>Top of Layer Elevation (feet)</td>
<td>Effective Unit Weight, γ' (lb/ft³)</td>
<td>Internal Friction Angle, φ (degrees)</td>
<td>Uniaxial Compressive Strength (lb/in²)</td>
<td>p-y Modulus, k (lb/in³)</td>
</tr>
<tr>
<td>---------------------------------------------------------------------------</td>
<td>-------------------------------</td>
<td>-----------------------------------</td>
<td>-------------------------------------</td>
<td>---------------------------------------</td>
<td>------------------------</td>
</tr>
<tr>
<td>Medium-dense, poorly graded Gravel with cobbles and boulders</td>
<td>850</td>
<td>125</td>
<td>32</td>
<td>-</td>
<td>40</td>
</tr>
<tr>
<td>Medium-dense, Gravel with sand</td>
<td>821</td>
<td>130</td>
<td>36</td>
<td>-</td>
<td>120</td>
</tr>
<tr>
<td>Medium to high strength, Granodiorite (bedrock)</td>
<td>811</td>
<td>165</td>
<td>_</td>
<td>15,000</td>
<td>_</td>
</tr>
</tbody>
</table>

Notes:  
1 lb/ft³ = pounds per cubic feet  
1 lb/in³ = pounds per cubic inch

TABLE 2
BORING B-2 (EAST ABUTMENT) GEOTECHNICAL INPUT PARAMETERS FOR LATERAL PILE RESISTANCE ANALYSIS USING LPILE®

<table>
<thead>
<tr>
<th>Soil Layer</th>
<th>Top of Layer Elevation (feet)</th>
<th>Effective Unit Weight, γ' (lb/ft³)</th>
<th>Internal Friction Angle, φ (degrees)</th>
<th>Uniaxial Compressive Strength (UCS) (lb/in²)</th>
<th>p-y Modulus, k (lb/in³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Medium-dense to dense Gravel with sand</td>
<td>898</td>
<td>125</td>
<td>34</td>
<td>-</td>
<td>100</td>
</tr>
<tr>
<td>Medium-dense Gravel, Cobbles, Boulders</td>
<td>891.5</td>
<td>125</td>
<td>34</td>
<td>-</td>
<td>100</td>
</tr>
<tr>
<td>Medium-dense, poorly graded Gravel, Cobbles, Boulders</td>
<td>884</td>
<td>125</td>
<td>34</td>
<td>-</td>
<td>100</td>
</tr>
<tr>
<td>Medium-dense, sandy Gravel, Cobbles, Boulders</td>
<td>858</td>
<td>125</td>
<td>33</td>
<td>-</td>
<td>90</td>
</tr>
<tr>
<td>Medium to high strength, Granodiorite (Bedrock)</td>
<td>840</td>
<td>165</td>
<td>_</td>
<td>15,000</td>
<td>_</td>
</tr>
</tbody>
</table>

Notes:  
1 lb/ft³ = pounds per cubic feet  
1 lb/in³ = pounds per cubic inch  
See Appendix A for range of measures UCS values on selected core samples

We assume that the piles will experience refusal, and will have limited penetration into the underlying bedrock.
The parameters presented in the above tables consider a level ground surface and no ground water. Ground surfaces sloping away from the piles will result in a reduced lateral capacity. The designer should account for sloping ground surface conditions.

To account for group effects, the recommended soil parameters in the tables used for the LPILE analyses of the driven pile foundation should be adjusted using the P-multipliers summarized in Figure 4. These efficiency factors should be used in lateral resistance analyses of driven pile groups.

Scouring around a bridge foundation could potentially affect primarily the lateral capacities of these foundations. Scour potential at the project site should be considered in design of the piles supporting the replacement bridge.

8.4.2 Cap Lateral Resistance

Passive earth pressure may be included to provide resistance to the pile cap away from and parallel to the channel. Under static conditions, an equivalent fluid pressure, $\gamma_p$, of 350 pounds per cubic foot (pcf) may be used to calculate the lateral resistance. This value assumes a level ground surface. For ground sloping at 3H:1V, an $\gamma_p$ of 175 pcf may be used. In this calculation, assume the pile cap backfill consists of compacted, free-draining, granular material. These recommended $\gamma_p$ values include a factor-of-safety of 1.5 to limit soil displacements. Ignore passive resistance at the pile cap if the soils providing resistance may be removed at any time in the future. We further recommend that a frictional coefficient of 0.4 be used to estimate the frictional resistance between the sides of the pile caps and the surrounding soils.

9.0 ABUTMENT AND WINGWALL DESIGN

9.1 Subdrainage

We recommend providing suitable drainage for abutments and wingwalls through granular backfill material and a base subdrain system in accordance with WSDOT Standard Specification Section 6-02.3(22), Drainage of Substructure. Weep holes are typically incorporated into the subdrain system for wall structures that exceed 10 feet tall. The weep holes should be covered with a geotextile. Free draining backfill should comply with WSDOT Standard Plans and the WSDOT Design Manual.
9.2 Backfill Material and Compaction

The abutment and wingwall backfill material should use granular wall backfill conforming to WSDOT Standard Specifications, Section 9-03.12(2), *Gravel Backfill for Walls*. The backfill should be placed and compacted on proof-rolled or probed subgrade determined to be suitable by the geotechnical engineer or the engineer's representative.

All structural fill should be placed in layers and compacted to at least 95 percent of its modified Proctor maximum dry density as measured by ASTM D 1557, and to a dense, unyielding condition. Where more settlement can be tolerated the density requirement may be reduced to 92 percent.

The backfill material should be placed in loose lifts of 4 to 8 inches and carefully worked around structures by means of shoveling, vibration, or other approved procedures. In general, the loose lift thickness should not exceed 8 inches for heavy equipment compactors and 4 inches for hand-operated mechanical compactors. Heavy compaction equipment should not be allowed closer than 3 feet to the abutment or wingwalls to prevent high lateral earth pressures causing wall yielding and/or damage. Backfill compaction within 3 feet of the wall should be accomplished with a low-weight compactor such as a hand-operated vibratory plate compactor. All compacted surfaces should be sloped to drain to prevent ponding. Structural fill operations should be observed and evaluated by the geotechnical engineer or the engineer's representative.

9.3 Lateral Earth Pressures

We developed lateral earth pressure models for the abutments and wingwalls based on the design information and the assuming that the recommendations presented in Section 9.2 of this report are followed. In Table 3 below, we provide lateral earth pressures for design of abutment and/or wingwall structures. The static lateral earth pressure acting on walls consists of two components: earth and surcharge pressures. We assume the backfill behind structural walls is adequately drained to avoid saturation and introduction of hydrostatic pressures.

For calculation of active pressures, we assume that the wall can deflect in order to develop an active condition. Use at-rest pressures for restrained or rigid-braced walls. The values below do not include a safety factor.
TABLE 3
LATERAL EARTH PRESSURES

<table>
<thead>
<tr>
<th>Wall Condition</th>
<th>Drained Equivalent Fluid Weight (pcf/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>At-Rest</td>
<td>55</td>
</tr>
<tr>
<td>Active</td>
<td>35</td>
</tr>
</tbody>
</table>

Note: pcf/ft = pounds per cubic foot per embedment foot

Lateral earth pressures due to surcharge should be added to the recommended lateral earth pressures presented in the following sections when appropriate. Recommended lateral pressures due to surcharge loads are presented in Figure 5. Unless included as a surcharge load in the design and construction of the shoring/excavation system, excavated material, construction material stockpiles, or equipment and vehicle traffic should be placed and routed, respectively, away from the top edge of the shoring system, no closer than a distance equal to the depth of excavation.

9.4 Lateral Resistances

The abutment and wingwall lateral resistance may be provided through passive earth pressures and frictional forces, as appropriate. Please see Section 8.4.2 of this report.

10.0 EARTHWORK AND CONSTRUCTION RECOMMENDATIONS

Shannon & Wilson, Inc. should review the plans and specifications for general conformance with our geotechnical recommendations prior to finalizing.

10.1 Pile Driving

The HP 18x204 piles supporting the replacement bridge will be installed through cobbles and boulders and should be driven to refusal on the hard bedrock underlying the project site. Refusal conditions could be encountered on cobbles and boulders before reaching the bedrock. Refusal conditions are defined as driving resistance, under continuous driving, of 20 blows per inch. We recommend the HP 18x2041 piles be equipped with high strength steel tip.

An air-, steam-, or diesel-powered hammer may be used for driving the proposed piles. In order to achieve the required nominal resistance of the HP 18x204 piles and based on the results of Wave Equation Analyses for Pile driving (WEAP), we recommend that the hammer has a minimum manufacture-rated energy of 154 foot-kips and a ram weight of 13 kips. We also
recommend that the hammer allows variable energy settings and it be operated at lower energy setting to seat the pile on the bedrock. The pile would then be driven to refusal at higher energy settings. All pile-driving equipment should be designed, constructed, and maintained in a manner suitable for the work to be accomplished for this project. If, in the opinion of the Owner, the driving equipment is inadequate or deficient, the Owner may direct that it be removed from the job site. All costs for re-mobilizing, removing, or replacing such equipment should be at the Contractor’s expense. The Contractor should furnish the manufacturer’s specifications and catalog for the hammer proposed. As a minimum, the Contractor should furnish the information required on the Pile and Driving Equipment Data Sheet, shown on Table 4, seven days in advance of the scheduled pile driving.

To prevent pile damage caused by excessive stresses during driving, we recommend that a test pile program be undertaken to measure driving stresses, establish driving criteria for production piles, adjust the pile-driving equipment if required, and alter the pile installation techniques. The test pile program should consist of driving indicator piles and performing dynamic pile tests using a Pile-Driving Analyzer (PDA). We recommend that at least one production pile per abutment be driven as an indicator pile. During the indicator pile driving, we recommend that dynamic measurements using a Goble pile-driving analyzer be taken and Case Pile Wave Analysis Program (CAPWAP) analyses be performed. Based on our experience, dynamic pile tests are one of the most cost-effective methods for determining the total ultimate capacities and load distributions of the piles.

All pile driving should be monitored by taking a continuous driving record of each pile. For this purpose, the Contractor should be required to mark the pile in 1-foot increments. During re-drive, if required, additional 1-inch increments between the 1-foot marks would be required. The pile-driving record should be complete. The form should have spaces to record hammer stroke (diesel hammers), blows per foot, time, date, reasons for delays, and other pertinent information. In addition, the record should include tip elevation, driving criteria, and initials of inspectors making final acceptance of the pile. The pile-driving records should be reviewed on a daily basis. For this purpose, we recommend that an experienced and qualified geotechnical engineer familiar with the subsurface conditions of the project site be assigned to assist in construction monitoring.

It has often been difficult in the past to estimate the energy delivered by diesel hammers. The Saximeter, developed by Pile Dynamic, Inc., can be used to record hammer strokes and provide an estimate of the driving energy of diesel hammers. If the Contractor selects a diesel hammer, we recommend that a Saximeter be used during pile driving.
10.2 Earthwork

Strip the existing asphalt structural section, topsoil, fill, and any other deleterious materials within the bridge foundation areas, and all areas to receive structural fill. The strippings are not suitable for use in engineered fill.

Prior to abutment construction or backfill placement on cut ground surfaces, remove loose soil and debris. Moisture condition the upper 12 inches of the native subgrade to within 2 percent of optimum, then compact to a minimum in-place dry density of 95 percent of the maximum laboratory dry density, as determined by ASTM International Designation: D 1557, Laboratory Compaction Characteristics of Soil Using Modified Effort.

Fill should be free of debris, organic material, and any other deleterious material. If import material is required, we recommend using a well-graded, 2-inch minus, pit-run sand and gravel with less than 5 percent fines, or crushed rock for structural fill, except where noted. Shannon & Wilson should review and approve material for import prior to transporting to the site. The on-site soils are typically moisture sensitive and often difficult to compact during wet weather conditions.

Moisture-condition fill material to within 2 percent of optimum and place in 8-inch-thick maximum, horizontal, loose lifts. Compact the fill to a minimum 95 percent of ASTM D 1557. Only hand-operated compaction equipment should be allowed within 3 feet of below-grade structures.

10.3 Excavations/Slopes

Based on our explorations, we characterize the site soils as Occupational Safety and Health Administration Type C with maximum temporary slopes of 1.5H:1V. The contractor is responsible for the safety of all temporary excavations based on exposed ground conditions.

Construct permanent cut and fill slopes at 2H:1V, or less, and protect from both wind and water erosion. Erosion protection may consist of a vegetative cover or a minimum 3-inch layer of coarse concrete aggregate conforming to the requirements of Washington State Department of Transportation (WSDOT) Specification 9-03.1(4)c, “Concrete Aggregate AASHTO Grading No. 57” (WSDOT, 2012).
10.4 Construction Observation

Variations in soil conditions are possible at the site and may be encountered during construction. Shannon and Wilson should be retained to provide construction observation services during the earthwork, excavation, and foundation phases of the project. Construction observation allows the geotechnical engineer to observe the actual soil conditions exposed in the excavations, determine if the proposed design is compatible with the design recommendations, and if the conditions encountered at the site are consistent with those observed during the geotechnical study. Construction observation is conducted to reduce the potential for problems arising during and after construction. However, in all cases, the contractor is responsible for the quality and completeness of their work and for adhering to the plans, specifications, and recommendations on which their work is based.

Shannon & Wilson should be retained to review the construction plans for the proposed structure modifications, and to provide construction observation services during site grading and foundation construction.

11.0 LIMITATIONS

The analyses, conclusions, and recommendations contained in this report are based upon site conditions as they presently exist. We further assume that the site explorations are representative of the subsurface conditions throughout the site; i.e., site conditions are not significantly different from those disclosed by the field explorations and observations.

If subsurface conditions different from those encountered in the field explorations are observed or appear to be present beneath the excavations during construction, we should be advised at once so that we can review these conditions and reconsider our recommendations, where necessary. If there is a substantial lapse of time between the submission of this report and the start of construction at the site, if conditions have changed because of natural forces or construction at the site, or if the design or loading configurations change, we recommend that we review this report to determine the applicability of the conclusions and recommendations concerning the time lapse or changed conditions contained in this report.

This report was prepared for the exclusive use of KPFF and Douglas County, in the design and construction of the Chief Joseph Dam Bridge Replacement in Douglas County, Washington. Variations from the structure types or locations discussed in this report should be analyzed by Shannon & Wilson, Inc. to assess the potential geotechnical impacts of those variations on the foundation recommendations included in this report.
The scope of services did not include any environmental assessment or evaluation regarding the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater, or air, on or below the site, or for the evaluation of disposal of contaminated soils or groundwater, should any be encountered.

As an integral part of this report, we have prepared the attached “Important Information about Your Geotechnical Report” (Appendix C) to help you more clearly understand its use and limitations.

SHANNON & WILSON, INC.

Hisham J. Sarieddine, P.E.
Senior Associate

LJR:HJS:JW/ljr:hjs
12.0 REFERENCES


Pile Dynamics, Inc. (PDI), 2010, GRLWEAP; Wave equation analysis of pile driving: Cleveland, Ohio, Pile Dynamics, Inc.


**TABLE 4**

**PILE AND DRIVING EQUIPMENT DATA SHEET**

<table>
<thead>
<tr>
<th>Section</th>
<th>Information</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Contract No.</strong></td>
<td>____________________________</td>
</tr>
<tr>
<td><strong>Structure Name and/or No.</strong></td>
<td>____________________________</td>
</tr>
<tr>
<td><strong>Project</strong></td>
<td>____________________________</td>
</tr>
<tr>
<td><strong>Pile Driving Contractor or Subcontractor</strong></td>
<td>____________________________</td>
</tr>
<tr>
<td><strong>County</strong></td>
<td>____________________________</td>
</tr>
</tbody>
</table>

**HAMMER**

| Manufacturer | ____________________________ |
| Model | ____________________________ |
| Type | ____________________________ |
| Serial No. | ____________________________ |
| Rated Energy | ____________________________ |
| @ | ____________________________ |
| Length of Stroke | ____________________________ |
| Explosive Force | ____________________________ |

**HAMMER COMPONENTS**

| RAM | ____________________________ |
| Ram Weight | ____________________________ |
| Ram Length | ____________________________ |
| Ram Cross Sectional Area | ____________________________ |

| ANVIL | ____________________________ |
| Anvil Weight | ____________________________ |

| CAPBLOCK | ____________________________ |
| Material | ____________________________ |
| Area | ____________________________ |
| Thickness | ____________________________ |
| Modulus of Elasticity – E | ____________________________ (psi) |
| Coefficient of Restitution – e | ____________________________ |

| PILE CAP | ____________________________ |
| Helmet | ____________________________ |
| Bonnet | ____________________________ |
| Anvil Block | ____________________________ |
| Drivehead | ____________________________ |
| Total Weight | ____________________________ |

| CUSHION | ____________________________ |
| Cushion Material | ____________________________ |
| Area | ____________________________ |
| Thickness | ____________________________ |
| Modulus of Elasticity – E | ____________________________ (psi) |
| Coefficient of Restitution – e | ____________________________ |

| PILE | ____________________________ |
| Type | ____________________________ |
| Pile Size | ____________________________ |
| Length (in leads): | ____________________________ |
| Diameter: | ____________________________ |
| Wall Thickness: | ____________________________ |
| Taper: | ____________________________ |
| Material: | ____________________________ |
| Weight/Ft.: | ____________________________ |
| Design Pile Capacity: | ____________________________ |
| Description of Splice: | ____________________________ |
| Tip Treatment Description: | ____________________________ |
NOTE
Map adapted from 1:24,000 USGS topographic map of Chief Joseph Dam, WA (1968) and Bridgeport, WA (1980).

Chief Joseph Dam Bridge Replacement
Douglas County, Washington

VICINITY MAP

October 2013
22-1-03021-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 1
Response Spectrum Parameters

- PGA = 0.14 g's
- $S_0 = 0.32$ g's
- $S_p = 0.10$ g's
- $A_s = 0.17$ g's
- $S_{DS} = 0.39$ g's
- $S_{D1} = 0.18$ g's
- $F_{PGA} = 1.20$
- $F_s = 1.20$
- $F_{a} = 1.70$
- $T_D = 0.09$ sec.
- $T_S = 0.46$ sec.

NOTES

1. Response spectrum parameters are defined in section 3.10.4 of the code.

2. Seismic ground shaking hazard in the AASHTO LRFD Bridge Design 2012 are based on 7% probability of exceedance in 75 years (975-year return period).
<table>
<thead>
<tr>
<th>Group Type</th>
<th>Shaft Spacing (Dia.)</th>
<th>Loading Type-1 Row 1</th>
<th>Loading Type-2 Row 1</th>
<th>Loading Type-2 Row 2</th>
<th>Loading Type-2 Row 3 and higher</th>
</tr>
</thead>
<tbody>
<tr>
<td>Groups with 1 Row</td>
<td>2.0D</td>
<td>0.54</td>
<td>1.00</td>
<td>0.33</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>3.0D</td>
<td>0.80</td>
<td>1.00</td>
<td>0.50</td>
<td>0.38</td>
</tr>
<tr>
<td></td>
<td>5.0D</td>
<td>1.00</td>
<td>1.00</td>
<td>0.65</td>
<td>0.70</td>
</tr>
</tbody>
</table>

**Note:**
Linear interpolation can be used to calculate the P-Multipliers for the shaft spacings which are not listed above.

<table>
<thead>
<tr>
<th>Group Type</th>
<th>Shaft Spacing (Dia.)</th>
<th>Loading Type-3 Row 1</th>
<th>Loading Type-3 Row 2</th>
<th>Loading Type-3 Row 3 and higher</th>
</tr>
</thead>
<tbody>
<tr>
<td>Groups with 2 or more Rows</td>
<td>2.0D</td>
<td>0.53</td>
<td>0.27</td>
<td>0.20</td>
</tr>
<tr>
<td></td>
<td>3.0D</td>
<td>0.80</td>
<td>0.40</td>
<td>0.30</td>
</tr>
<tr>
<td></td>
<td>5.0D</td>
<td>1.00</td>
<td>0.85</td>
<td>0.70</td>
</tr>
</tbody>
</table>
A) LATERAL PRESSURE DUE TO POINT LOAD
i.e. SMALL ISOLATED FOOTING OR WHEEL LOAD
(NAVFAC DM 7.2, 1966)

\[ \sigma_{b} = \frac{Q}{2H} \left( 0.16 + \frac{n^{2}}{n^{2} + n^{2}} \right) \text{ (psf)} \]

For \( m \leq 0.4 \):
\[ n_{b} = 0.20 \]
\[ \frac{Q}{H} \]
\[ \frac{1}{0.16 + n^{2}} \]
\[ \text{(psf)} \]

For \( m > 0.4 \):
\[ n_{b} = 1.77 \]
\[ \frac{Q}{H} \]
\[ \frac{1}{0.16 + n^{2}} \]
\[ \text{(psf)} \]

\[ \sigma_{ex} = n_{b} \cos 2\left(1.10\right) \text{ (psf)} \]

B) LATERAL PRESSURE DUE TO LINE LOAD
i.e. NARROW CONTINUOUS FOOTING PARALLEL TO WALL
(NAVFAC DM 7.02, 1966)

\[ \sigma_{L} = \frac{n}{H} \left( 0.16 + n^{2} \right) \text{ (psf)} \]

For \( m \leq 0.4 \):
\[ n_{L} = 0.20 \]
\[ \frac{n}{H} \]
\[ \frac{1}{0.16 + n^{2}} \]
\[ \text{(psf)} \]

For \( m > 0.4 \):
\[ n_{L} = 1.28 \]
\[ \frac{n}{H} \]
\[ \frac{1}{0.16 + n^{2}} \]
\[ \text{(psf)} \]

\[ \sigma_{ex} = \sigma_{L} \cos 2\left(1.10\right) \text{ (psf)} \]

C) LATERAL PRESSURE DUE TO STRIP LOAD
(derived from Fang, Foundation Engineering Handbook, 1961)

\[ \sigma_{ex} = \frac{Q}{2H} \left( 0.16 + \frac{n^{2}}{n^{2} + n^{2}} \right) \text{ (psf)} \]

\[ \sigma_{ex} = \frac{Q}{2H} \left( 0.16 + \frac{n^{2}}{n^{2} + n^{2}} \right) \text{ (psf)} \]

D) LATERAL PRESSURE DUE TO EARTH BERM OR UNIFORM SURCHARGE
(derived from Poulos and Davis, Elastic Solutions for Soil and Rock Mechanics, 1974; and Terzaghi and Peck, Soil Mechanics in Engineering Practice, 1967)

E) LATERAL PRESSURE DUE TO ADJACENT FOOTING
(see Notes 5 and 6)
(derived from NAVFAC DM 7.02, 1966 and Sandhu, Earth Pressure on Walls Due to Surcharge, 1974)

**NOTES**

1. Figures are not drawn to scale.
2. Applicable surcharge pressures should be added to appropriate permanent wall lateral earth and water pressure.
3. If point or line loads are close to the back of the wall such that \( m \leq 0.4 \), it may be more appropriate to model the actual load distribution (i.e. Detail E) or use more rigorous analysis methods.
4. We recommend using an active K value equal to 0.33 and a wall unit weight \( \gamma \) value equal to 125 pounds per cubic foot.
5. The stress is estimated on the back of the wall at the center of the length, L, of loading.
6. The estimated stress is based on a Poisson’s ratio of 0.3.
APPENDIX A

EXPLORATORY BORING LOGS
AND CORE PHOTOGRAPHS
## SOIL CLASSIFICATION CHART

<table>
<thead>
<tr>
<th>MAJOR DIVISIONS</th>
<th>SYMBOLS</th>
<th>TYPICAL DESCRIPTIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td><strong>GRAPH</strong></td>
<td><strong>LETTER</strong></td>
</tr>
<tr>
<td><strong>COARSE GRAINED SOILS</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gravel and gravelly soils</td>
<td>CLEAN GRAVELS (LITTLE OR NO FINES)</td>
<td>GW</td>
</tr>
<tr>
<td></td>
<td>CLEAN GRAVELS</td>
<td>GP</td>
</tr>
<tr>
<td></td>
<td>GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)</td>
<td>GM</td>
</tr>
<tr>
<td></td>
<td>CLAYEY GRAVELS, GRAVEL - SAND CLAY MIXTURES</td>
<td>GC</td>
</tr>
<tr>
<td></td>
<td>SAND AND SANDY SOILS</td>
<td>SW</td>
</tr>
<tr>
<td></td>
<td>SANDS WITH FINES (LITTLE OR NO FINES)</td>
<td>SP</td>
</tr>
<tr>
<td></td>
<td>CLAYEY SANDS, SAND - CLAY MIXTURES</td>
<td>SM</td>
</tr>
<tr>
<td></td>
<td>SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)</td>
<td>SC</td>
</tr>
<tr>
<td></td>
<td>INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY</td>
<td>ML</td>
</tr>
<tr>
<td></td>
<td>INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRANULAR CLAY, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS</td>
<td>CL</td>
</tr>
<tr>
<td></td>
<td>ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY</td>
<td>OL</td>
</tr>
<tr>
<td></td>
<td>INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS</td>
<td>MH</td>
</tr>
<tr>
<td></td>
<td>INORGANIC CLAYS OF HIGH PLASTICITY</td>
<td>CH</td>
</tr>
<tr>
<td></td>
<td>ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS</td>
<td>OH</td>
</tr>
<tr>
<td></td>
<td>PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS</td>
<td>PT</td>
</tr>
</tbody>
</table>

**NOTE:** Dual symbols are used to indicate borderline soil classifications.

### NOTES

1. Dual Symbols (symbols separated by a hyphen, i.e., SP-SM, slightly silty fine SAND) are used for soils with between 5% and 12% fines or when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart.

2. Borderline symbols (Symbols separated by a slash, i.e., CL/ML, silty CLAY/clayey SILT; GW/SW, sandy GRAVEL/gravelly SAND) indicates that the soil may fall into one of the two possible basic groups.
SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Medium dense, brown/tan, poorly graded GRAVEL/COBBLES with sand (GP); moist; 50% subrounded to subangular cobbles, subrounded to rounded gravels; 40% sand; 10% silt, non-plastic.

Medium dense to dense, brown, poorly graded SAND with gravel (SP); moist; 40% subrounded to subangular, coarse gravel; 50% fine to medium sand; 10% silt, non-plastic.

Medium dense, brown/gray/tan, poorly graded GRAVEL, cobbles, and boulders (GP); moist; 90% cobbles, and boulders, maximum dimension approximately 48 inches; 5% fine to medium, angular sand; 5% silt, non-plastic; measured voids 48-inch at 40 feet, boulders primarily consist of granodiorite, no water circulation as the drilling progressed. Used rock core methods from 47 to 58 feet. Drill action indicates voids between the cobbles and boulders.

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The stratification lines represent the approximate boundaries between soil types, and the transition may be graded.
5. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
Medium dense, brown/gray, poorly graded GRAVEL with sand (GP); moist; 50% subrounded to rounded, coarse gravel; 40% fine to medium, angular sand; 10% silt, non-plastic; SPT at 60 feet, Core at 65 feet.

Dark, schistose granodiorite, medium to high strength, gray, medium grained, smooth, closely to moderately close spacing, randomly oriented joint; non-weathered to slightly weathered, brown secondary mineralization and staining in the joints.

Granodiorite, high to very high strength, white/gray, medium to coarse grained, moderately close spaced, randomly oriented jointing, non-weathered, white secondary mineralization and deposits in the joints.

Bottom of Boring 76.5 ft.

The drillers experienced poor/no water circulation during drilling as a result of voids. Driller used 5,000 gallons of water.

NOTES
1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The stratification lines represent the approximate boundaries between soil types, and the transition may be graded.
5. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
SOIL DESCRIPTION
Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Medium dense to dense, brown/gray, poorly graded GRAVEL with sand, COBBLES (GP); moist; trace to few silt; subrounded to subangular gravel and cobbles from 1 to 6 inch diameter; fine to medium sand; few silts.

Medium dense, gray/white, poorly graded GRAVEL, COBBLES, and BOULDERS (GP); moist; little sand, few silt; subangular to angular cobbles and boulders, from 3 to 24 inches diameter; subrounded to rounded gravel, from 1 to 3 inch diameter; fine to medium sand; non-plastic silt.

Medium dense, white/brown/gray, poorly graded GRAVEL, COBBLES, and BOULDERS (GP); moist; trace sand and silt; subangular to angular BOULDERS, from 12 to 48 inch diameter; subangular to subrounded cobbles, from 6 to 12 inch diameter; subrounded to rounded gravel, from 1 to 3 inch diameter; fine to medium, angular sand; non-plastic silt.

Driller reports voids between the cobbles and boulders from 15 to 25 feet.

Medium dense, brown/gray/white, sandy GRAVEL, COBBLES, and BOULDERS (GP); moist; trace silt; subangular to angular boulders, from 12 to 24 inch diameter; subrounded to rounded cobbles from 6 to 12 inch diameter; fine to medium, angular sand; non-plastic silt.

NOTES
1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
5. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
Granodiorite, medium to high strength, white/gray, medium to coarse grained, closely spaced, randomly oriented jointing, non-weathered to slightly weathered, white/brown secondary mineralization in the joints.

Dark schistose granodiorite, medium to high strength, gray/dark gray, medium grained, smooth, close to moderately close spaced, randomly oriented jointing, non-weathered to slightly weathered, white secondary mineralization in the joints, grain size increased with depth.

Granodiorite, medium to high strength, white/gray, medium to coarse grained, moderately close spaced, randomly oriented jointing, non-weathered, white secondary mineralization in the joints.

Bottom of Boring 71.5 ft.

The driller experienced no water circulation during drilling as a result of voids. Driller used 3,500 gallons of water.

### SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

### PENETRATION RESISTANCE

- **Hammer Wt. & Drop:** 140 lbs / 30 inches
- **Depth, ft.**
  - 0
  - 20
  - 40
  - 60
  - 80
  - 100

### NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
5. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
Boring B-1 Core Sample

FIG. NO. A-4

22-1-03021-001
APPENDIX B
LABORATORY TESTING RESULTS
**UNCONFINED COMPRESSION TEST REPORT – GRANITE CORES**

**PROJECT:** Chief Joseph Dam - #22-1-03021-001  
**DATE TESTED:** 7/29/13  
**CLIENT:** Shannon & Wilson  
**JOB NUMBER:** 13-176  
**CONTRACTOR:** Shannon & Wilson  
**WORK ORDER NUMBER:** 7706  
**INSPECTOR:** SB

<table>
<thead>
<tr>
<th>Lab #</th>
<th>Location</th>
<th>Length</th>
<th>Diameter</th>
<th>Radius</th>
<th>Area</th>
<th>Load (lbs)</th>
<th>Compressive Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7706-1</td>
<td>B-2, 65'-65.66'</td>
<td>4.78</td>
<td>2.39</td>
<td>1.195</td>
<td>4.486</td>
<td>83345</td>
<td>18580</td>
</tr>
<tr>
<td>7706-2</td>
<td>B-1, 68'-68.5'</td>
<td>4.80</td>
<td>2.40</td>
<td>1.200</td>
<td>4.524</td>
<td>52755</td>
<td>11660</td>
</tr>
<tr>
<td>7706-3</td>
<td>B-1, 75.33'-76'</td>
<td>4.80</td>
<td>2.46</td>
<td>1.230</td>
<td>4.753</td>
<td>80455</td>
<td>16930</td>
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<tr>
<td>7706-4</td>
<td>B-2, 60.5'-61.75'</td>
<td>4.78</td>
<td>2.39</td>
<td>1.195</td>
<td>4.486</td>
<td>58925</td>
<td>13130</td>
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<tr>
<td>7706-5</td>
<td>B-2, 24.5'-25'</td>
<td>4.80</td>
<td>2.40</td>
<td>1.200</td>
<td>4.524</td>
<td>75495</td>
<td>16690</td>
</tr>
</tbody>
</table>

Note: Cores were received in an unsealed bag, and after drying there was no moisture content measurable in rock cores.

REVIEWED BY:

X [Signature]

General E-mail: general@baeresting.com

AN EQUAL OPPORTUNITY EMPLOYER
UNIT WEIGHT TEST REPORT – ROCK CORES

PROJECT: Chief Joseph Dam Bridge #22-1-03021-001   DATE TESTED: 8/8/13
CLIENT: Shannon & Wilson   JOB NUMBER: 13-176
CONTRACTOR: Shannon & Wilson   WORK ORDER NUMBER: 7731
INSPECTOR: SRB

<table>
<thead>
<tr>
<th>Lab #</th>
<th>Location</th>
<th>Length</th>
<th>Diameter</th>
<th>Radius</th>
<th>Unit Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>7731-1</td>
<td>B-2, R-3, D 20’-20.5’</td>
<td>4.689”</td>
<td>2.391”</td>
<td>1.196”</td>
<td>165.760 pcf</td>
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<td>7731-2</td>
<td>B-2, R-12, D 69’-70’</td>
<td>4.733”</td>
<td>2.389”</td>
<td>1.195”</td>
<td>165.822 pcf</td>
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<tr>
<td>7731-3</td>
<td>B-1, R-6, D 70’-71’</td>
<td>4.773”</td>
<td>2.398”</td>
<td>1.199”</td>
<td>166.383 pcf</td>
</tr>
</tbody>
</table>

Reviewed by:

X R. Baer

General E-mail: general@baertesting.com

AN EQUAL OPPORTUNITY EMPLOYER
APPENDIX C

IMPORTANT INFORMATION ABOUT YOUR GEO TECHNICAL REPORT
Important Information About Your Geotechnical/Environmental Report

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used: (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors, which were considered in the development of the report, have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.
A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based on interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland.