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April 30, 2018

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Geotechnical Engineering Evaluation - REVISED
Swan Creek Pipeline Crossing
East 68th Street & East Pipeline Road
Tacoma, Washington
NGA Project No. 1012417

Dear Mr. Faulkner:

We are pleased to submit the attached report titled **"Geotechnical Engineering Evaluation – Swan Creek Pipeline Crossing – East 68th Street & East Pipeline Road – Tacoma, Washington."** This report summarizes the existing surface and subsurface conditions within the site and provides recommendations for the proposed site improvements. Our services were completed in general accordance with our proposal dated October 2, 2017 and signed by you on January 23, 2018.

The proposed project consists of replacing supports for a water conveyance line with a new bridge that will also carry the recreational trail across Swan Creek and to remove the culverts and embankment fill from the channel. We explored subsurface conditions within the vicinity of the bridge location on March 10, 2017 with two auger-drilled soil borings to a depth of approximately 51 feet below the ground surface. The borings were located on the existing roadway embankment crossing Swan Creek and encountered 15 to 20 feet of fill soil. The underlying native soils consisted of dense to very dense gray-brown to gray fine to coarse sand with varying amounts of silt and gravel that we interpreted as native glacial soils to the depths explored.

It is our opinion that the planned site development is feasible from a geotechnical standpoint, provided that our recommendations are incorporated into the design and construction of this project. In the attached report, we have included recommendations for earthwork, foundation support, and erosion control.

We appreciate the opportunity to provide service to you on this project. Please contact us if you have any questions regarding this report or require further information.

Sincerely,

Nelson Geotechnical Associates, Inc.

Khaled M. Shawish, PE
Principal

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Geotechnical Engineering Evaluation - REVISED
Swan Creek Pipeline Crossing
East 68th Street & East Pipeline Road
Tacoma, Washington

INTRODUCTION

This report presents the results of our geotechnical engineering investigation and evaluation for the proposed Tacoma Water Pipeline Crossing of Swan Creek, located at East 68th Street & East Pipeline Road within an upland plateau south of the Puyallup River Valley in Tacoma, Washington, as shown on the Vicinity Map in Figure 1. Generally, the pipeline is buried below ground but is exposed as it crosses the creek. A gravel-surfaced access/maintenance road, which also serves as a recreational trail, parallels the pipeline within the right-of-way corridor. Swan Creek currently passes under the access road through two concrete pipes. The configuration of the passage creates a barrier to fish migration and the City of Tacoma would like to remove the barrier. The proposed project consists of replacing the existing pipeline supports with a new bridge that will also carry the recreational trail across Swan Creek and to remove the culverts and embankment fill from the channel.

SCOPE

Nelson Geotechnical Associates, Inc. (NGA) has been requested to perform a geotechnical engineering evaluation of the proposed pipeline crossing and provide our opinions and recommendations for foundation support. The purpose of this study and report is to present our findings, opinions, and conclusions regarding existing geotechnical and geologic conditions at the site, and to provide recommendations for the planned development. Based on our understanding of the proposed project and existing site conditions, the specific services provided by NGA were to:

- Review existing soils and geologic maps of the area.
- Examine any existing documents that may be available such as previous geotechnical reports, construction monitoring reports, project drawings, etc.
- Reconnoiter the property to determine existing surface conditions and to search for and identify indicators of potential geologic hazards.
- Map the conditions on the slope and assess current slope stability.
- Determine if a geologic hazard, as defined by the applicable building code, exists on or near the property.
- Explore the site subsurface soil and groundwater conditions with soil borings using a truck-mounted drill rig equipped with hollow-stem augers. The drill rig and crew were subcontracted by NGA.
- Provide recommendations for site preparation, earthwork, temporary excavation slopes, site drainage and erosion control during construction.

- Develop geotechnical engineering design recommendations for pile foundation design, including seismic design parameters.
- Document the results of our findings, conclusions, and recommendations in a written geotechnical report.

For our use in preparing this report, we were provided with a 70 percent in progress drawing prepared by Cardno Entrix, titled "Access, Staging, & TESC Plan – Pipeline Site, Swan Creek Pipe Protection Project," dated June 2014, an undated Proposed Road/Pipeline Profile drawing, and an undated Proposed Stream Profile drawing.

SITE CONDITIONS

Surface Conditions

NGA visited the site on October 16, 2017 to reconnoiter the site and plan our subsurface exploration program. Based on our observations and review of available topographic mapping, the site covers a small ravine containing Swan Creek where an existing City of Tacoma water pipeline crosses the ravine, as shown on the Site Plan in Figure 2. Topography in the area is generally level to mildly rolling with localized steeper slopes from the Swan Creek ravine. The access road slopes downward from both the north and south, forming a low spot over the creek crossing. Both approaches appear to have been constructed by excavating into the crest of the upland plateau and it is possible the spoils from these cuts were used in construction of the embankment fill.

Development in the area ranges from farms to compact residential subdivisions but is generally characterized as semi-rural residential. The slopes forming the Swan Creek ravine are moderately forested with Douglas-fir and Western Red cedar, and with Big leaf Maple and other deciduous trees of varying maturity, shrubs, and bushes. Wild grasses, blackberry vines and various herbaceous species blanket the access corridor outside of the gravel-surfaced road.

The pipeline is supported on four concrete bents at a 30-foot spacing for a total span of approximately 150 feet where it crosses the ravine. A single-lane, gravel-surfaced access/maintenance road, which also serves as a recreational trail, parallels the pipeline within the right-of-way corridor. Swan Creek currently passes under the access road through two concrete pipes, as shown in Figure 2. The inlets and outlets of the two pipes are protected with concrete bulkheads, wing walls, and aprons. The outlet apron and wing walls form an expanding spillway to reduce flow velocities but the creek channel is heavily eroded below the trailing edge of the spillway. The drop pool has near-vertical banks, with some undercutting below the spillway, and is approximately 4 to 6 feet below the spillway elevation. Compact, stratified sandy gravel and gravelly sand are exposed under the spillway and on the adjacent creek banks. A minor amount of water was dripping from this exposure; no other surface water was observed within the area.

Subsurface Conditions

Explorations: The subsurface conditions within the site were explored on January 25, 2018 by drilling two soils borings to a depth of approximately 51 feet below the ground surface. Along the pipeline alignment, boring B-1 was located approximately midway between the north abutment and the northern-most pipe bent and offset southwest to the shoulder of the access road. Boring B-2 was similarly located midway between the south abutment and the southern-most pipe bent and offset about midway between the pipeline and the eastern shoulder of the access road. The exploration locations are shown in Figure 2.

The borings were advanced with a truck-mounted drill rig equipped with hollow-stem augers. A geologist from NGA monitored the explorations, examined the soils and geologic conditions encountered, obtained samples of the different soil types, and maintained logs of the explorations. The soils were visually classified in general accordance with the Unified Soil Classification System, presented in Figure 3. The logs of the borings are presented as Figures 4 and 5. We present a brief summary of the subsurface conditions in the following paragraphs. For a more detailed description of the subsurface conditions, the exploration logs should be reviewed.

Beneath a surficial layer of dark brown silty sand with gravel, both borings encountered fill composed of gray-brown silty sand and gray-brown gravelly sand with some silt. The fill extended to a depth of approximately 20 feet in boring B-1 and 15 feet in boring B-2. Scattered roots and other organic matter were encountered in both borings and organic-rich soils were present in the samples collected at 5 feet above the base of the fill (at depths of 15 and 10 feet in B-1 and B-2, respectively) in both borings. The fill was moist and medium dense to very dense in boring B-1 and loose in boring B-2. Underlying the fill, the borings encountered gray-brown silty fine to coarse sand and gray-brown gravelly fine to coarse sand with varying amounts of silt. These materials were very dense and moist to wet except at a depth of about 42 feet in boring B-2 where the soil was siltier and only dense.

It should be noted that the standard soil sampling method limits the maximum particle size to approximately 1-3/8 inch, biasing sample composition toward the finer soil particles. Also, the presence of large gravel, cobbles, or boulders ahead of the sampler can increase the resistance as the obstruction is pushed aside or broken down, increasing the apparent relative density.

Hydrogeologic Conditions

Groundwater was encountered in the borings located at depths of 35 and 15 feet below the ground surface in borings B-1 and B-2, respectively. In B-1, the water was encountered near the upper boundary of a cleaner (less fine grains) layer of gravelly fine to coarse sand than the silty sand overlying it. In B-2, the water was present near the contact between the fill soil and the underlying gravelly fine to coarse sand.

The depth of water in Swan Creek varies seasonally, dropping (and disappearing underground) during the drier and warmer summer and early fall seasons before rebounding with late fall and winter storms. Reportedly, the creek has a rapid response to storm events, quickly increasing and decreasing in depth and velocity. Undermining of the spillway apron and downstream erosion of the lower channel banks are consistent with this behavior. Based on soil characteristics and the local geology, we expect that subsurface water is present in the soils adjacent to the creek year-round.

SEISMIC CONDITIONS

NGA completed an analysis to develop a site-specific seismic design response spectrum for the subject site. Our analysis included a review of literature regarding historical and prehistorical seismic activity in the Pacific Northwest, assessing the magnitude and location of potential seismogenic source zones, and considering the effect of site geologic conditions on potential ground shaking. The soil profile used in our analysis was based on the soils encountered in our site exploration and previously published literature. Required soil parameters such as unit weight, friction angle, and shear wave velocity were estimated from within generally-agreed-upon ranges as published in the literature.

Our analysis was directed at determining a response spectrum for ground motion that has a 10-percent chance of being exceeded in 50 years and 100 years, with viscous damping of 5, 7, and 10 percent. The results of our analysis are presented on the attached spectra curves are shown on Figure 6.

Regional Geology

The site is located in the southeastern part of the Puget Sound Basin (also known as the Puget Lowland) geologic province. The basin is an elongated topographic and structural depression bordered by the Olympic Mountains to the west and the Cascade Mountains to the east. Low-rolling relief with some deeply-cut ravines and broad valleys characterize the basin topography.

The Puget Sound area has been subjected to several major glaciations during the Pleistocene Epoch (two million to ten thousand years ago) that filled the basin to significant depths with complex sequences of glacial and nonglacial sediments. Throughout much of the basin, bedrock is deeply buried by Pleistocene and Holocene sediments and bedrock outcrops are present at only scattered locations. The most recent advance (Vashon Stade of the Frasier Glaciation) covered the area with an ice sheet estimated at 3,000 feet. Subsequent erosion, transport, and redeposition of these and other sediments make the geologic setting even more complicated.

Site Geology

The site is located near the northern extent of an upland plateau that forms the western side of the Puyallup River Valley. The plateau is relatively broad, narrowing toward the north with an elevation of approximately 400 feet above sea level.

The geologic units for this area are shown on the Geologic Map of the City of Tacoma, Pierce County, Washington by Mackey Smith, (WaDGER, 1977). The youngest soil deposit at the site is mapped as Continental Glacial Till (Qgt) throughout. Till soils are heterogeneous mixtures of sand, silt, and gravel, cobbles, and boulders that is compact with some cementing. The next youngest deposit is Advance Continental Glacial Outwash (Qga) mapped as exposed within the Swan Creek ravine. Advance Outwash is described as glaciofluvial sand and gravel and lacustrine clay, silt, and sand deposited as the glaciers advanced. In local, isolated areas, nonglacial sediments and deposits are also present. Exposed soils observed along the creek banks and the soils encountered in our explorations consisted of gravelly, fine to coarse sand with varying amounts of silt that are generally consistent with Advance Outwash soils.

Below the Advance Outwash soils, the stratigraphy is less certain, composed of a variety of sands, silts, and gravels. These deeper deposits are expected to be pre-Vashon in age. The Map Showing Depth to Bedrock of the Tacoma and Part of the Centralia 30' x 60' Quadrangles, Washington by Jane M. Buchanan-Banks and Donley S. Collins (USGS 1994) shows that the depth to bedrock exceeds 2,000 feet at the site. The closest bedrock outcrops are located in the foothills of the Cascade Mountains, approximately 16 miles southeast of the site.

Regional Tectonic Setting

Tectonic stresses in the western United States are mostly associated with the slow movement of large, lithospheric plates. As these plates collide, diverge, or grind past each other, geologic processes, such as earthquakes and faulting, mountain building, or volcanism, relieve the stresses created.

The major lithospheric plates in the Pacific Northwest are the continental North American plate and the Pacific plate underlying the Pacific Ocean. Between these two plates are several smaller plates; the Explorer plate, west of Vancouver Island; the Juan de Fuca plate, west of Washington and Oregon; and the Gorda plate, west of northern California. The tectonics and seismicity of the area is the result of ongoing, oblique, relative northeastward subduction of the Juan de Fuca plate beneath the North American plate. New material created along a spreading ridge between the Pacific and Juan de Fuca plates forces the Juan de Fuca plate to the northeast, towards the North American plate. The Juan de Fuca plate thrusts beneath (subducts) the North American plate, to lie as a great tilted slab beneath the entire western portion of Washington and Oregon. The interface between the two plates is known as the Cascadia Subduction Zone (CSZ).

The movement of these plates directly affects the seismic activity of the area. Deep earthquakes (greater than 20 miles) are associated with the subducting slab, while shallow earthquakes (less than 20 miles) are attributed to crustal subsidence in combination with regional compressive stresses. The northeast movement of the Juan de Fuca plate as it spreads toward the North American plate mostly controls the seismicity in the Pacific Northwest.

The regional faulting and structural trends in the Puget Sound Basin are greatly complicated by the masking of bedrock by the unconsolidated Pleistocene and pre-Pleistocene sediments. In limited areas where bedrock is exposed, numerous faults and folds have been mapped. In the general Seattle area, the known bedrock structure consists of a series of east-west to northwest trending anticlines and synclines.

Seismicity

The project site is located in a moderately active tectonic province that has been subjected to numerous earthquakes of low to moderate strength, and occasionally to strong shocks during the brief historical record in the Pacific Northwest. Based on their understanding of the regional tectonic framework and historical seismicity, seismologists have classified earthquake sources into three broad seismogenic zones; 1) a shallow continental crust zone; 2) a deep subcrustal zone in the Juan de Fuca Plate; and 3) an intraplate or subduction zone.

Shallow Continental Crust Zone: It had generally been thought that a magnitude 6.0 event was the maximum possible from the shallow crustal zone. The majority of historical events have occurred in the crustal zone at depths of 20 miles or less, although most of the events are relatively mild and not felt by humans. There are several shallow faults or inferred faults in the region that may be capable of generating significant earthquakes, but it is uncertain which, if any, of these faults are active at this time.

Recent research has suggested that this seismogenic source zone may be capable of producing events of greater magnitudes. Of particular concern recently is the Seattle Fault, which is now considered to be active. The Seattle Fault is actually a zone of several thrust or reverse faults located between Bremerton and the south end of Lake Sammamish. The precise location, lateral geometry, displacement history, and slip rates of the faults are poorly defined and have been the subject of recent study by the United States Geological Survey (USGS) and others. It has been suggested that the large thickness of Quaternary material in the Puget Sound Basin, along with other geologic evidence, indicates possible large Quaternary offsets that may have occurred during a large (greater than magnitude 7.0) earthquake on the Seattle Fault about 900 A.D.

The maximum magnitude event along the Seattle Fault has been estimated from 6.5 to 7.1, although seismologists have recently calculated that earthquakes with magnitudes of 7.6 to 7.7 would be possible if the total surface area (as modeled) of the Seattle Fault were to rupture during one event. The magnitude of ground shaking from an earthquake on the Seattle Fault would be significantly greater near the City of Seattle than at this site because of the relative distances.

Juan De Fuca Plate Deep Subcrustal Zone: The depth to the subducted Juan de Fuca Plate and the overlying North American Plate obscures the second source zone. Earthquakes within this zone are associated with the release of extensional stress within the subducted slab. They have occurred most commonly beneath the area between Olympia and Seattle. All of the large historic earthquakes that have occurred in the Puget Sound region have been determined to be relatively deep with foci greater than 30 miles below the ground surface.

Major historic earthquakes in the region had focal depths in excess of 30 miles and are thought to have occurred in the deep, subcrustal zone. The three largest historic earthquakes consist of the magnitude 7.1 Olympia earthquake of April 13, 1949, the magnitude 6.5 Seattle-Tacoma earthquake of April 29, 1965, and the recent magnitude 6.8 Nisqually earthquake of February 28, 2001. All three created ground shaking with maximum intensity levels of VII to VIII (modified Mercalli earthquake intensity scale) in localized areas. The maximum expected earthquake from this zone is estimated to be approximately 7.5.

Subduction Zone: The third source area is the Cascadia subduction zone (CSZ), between the Juan de Fuca and North American plates. The CSZ is located about 120 to 240 miles offshore from Washington, Oregon, and northern California. Recent interest has been focused on the CSZ as a structure capable of generating very large (greater than magnitude 8.0), shallow earthquakes. Recently discovered geologic evidence suggests that coastal estuaries underwent rapid subsidence at various times over the last 2,000 years, with the most recent event occurring about 300 years ago. Great subduction earthquakes have been postulated as the cause of this subsidence.

No large subduction earthquakes have occurred in this zone during historical times. A lack of data makes it difficult to define potential earthquake magnitudes, rupture lengths, and recurrence rates for the CSZ. The maximum credible magnitude has been estimated at 8.0 to 9.0 with a recurrence interval of estimates of 300 to 1,000 years. The lack of historical earthquakes in CSZ may indicate that either the tectonic plates are uncoupled and subducting aseismically, or that the Juan de Fuca and North American plates are locked and storing energy to be released in future great earthquakes.

The expected rupture zone for a great subduction earthquake is generally considered to be from the Pacific Coast to about 125 miles offshore. Because the epicentral distance from an offshore subduction earthquake would be on the order of 120 to 245 miles from the site, much of the damaging motions would be attenuated prior to reaching the site. Thus, peak ground acceleration at the site is expected to be more severe during a large near-field earthquake than from a greater far-field earthquake. However, the duration of a great far-field earthquake could be much greater.

Seismic Risk Study

There are two basic methods for assessing seismic risk at a specific site; deterministic and probabilistic. In the deterministic method, the risk is evaluated based on a specific seismic event from a specified source. Typically, the event used is the one that creates the worst-case ground motions at the site, and the method is often used in the design of sensitive structures such as hospitals, highway bridges, and nuclear power plants. The method requires knowing (or assuming) the characteristics of the various seismic sources, including potential magnitude and distance between the site and the fault. Because the closest source may not produce the most intense shaking, each source zone is evaluated to determine which one produces the most significant event. Once the event has been selected, the ground shaking parameters from that event are determined. One of the main disadvantages to the deterministic method is that it provides no information on the likelihood of occurrence of the controlling earthquake or the level of shaking that might be expected in a given time span.

There are no known or suspected faults located in the vicinity of the project site. The closest known fault is the Tacoma fault zone that runs west-east across the north end of the Key Peninsula, with traces trending southeast through Commencement Bay, Federal Way, and Auburn. At its closest point, one trace of the fault zone is located approximately 4 to 5 miles north of the site. Researchers have estimated that most recent prehistoric deformation in this source occurred less than 15,000 years ago but there are no earthquakes in the historical record. The estimated maximum magnitude is estimated between 6.5 and 6.9 and the slip rate is estimated at about 0.25 millimeter per year.

In areas such as the Puget Sound, where active faults, such as the Seattle and Tacoma fault zones, are poorly identified, geologic structure is obscured by deep overburden, and the historical record is limited, the probabilistic method is more appropriate because these uncertainties can be incorporated into the analysis. The probabilistic method essentially combines multiple deterministic analyses using seismic activity originating from several source zones, and incorporates probability functions and statistical analyses to generate estimates of seismic risk at the project site over a specified time period. Seismic provisions in current building codes are based on this method.

IBC Seismic Design Parameters: The 2015 International Building Code (IBC) seismic design section provides a generalized basis for seismic design of structures. Because dense to very dense glacial soils were encountered at relatively shallow depths, site conditions best fit the IBC description for Site Class C. Table 1 below provides seismic design parameters for the site that are in conformance with the 2015 IBC, which specifies a design earthquake having a 2 percent probability of occurrence in 50 years (return interval of 2,475 years), and the 2008 USGS seismic hazard maps. Spectral response acceleration values were obtained from the USGS Earthquake Hazards Program Interpolated Probabilistic Ground Motion website (2008 data) for the project latitude and longitude.

Table 1 – 2015 IBC Seismic Design Parameters

Site Class	Spectral Acceleration at 0.2 sec. (g) S_S	Spectral Acceleration at 1.0 sec. (g) S_1	Site Coefficients		Design Spectral Response Parameters	
			F_a	F_v	S_{DS}	S_{D1}
C	1.275	0.495	1.000	1.305	0.850	0.431

Site-Specific Parameters: NGA used the probabilistic method for our site-specific seismic analysis. Qualitative geologic and geophysical evaluations of seismicity have been conducted to provide a conceptual framework to interpret the statistical distribution of historic earthquakes, define seismo-tectonic provinces, and assess the physical limits on the location and magnitude of the maximum earthquake in the region. Seismic motions are computed from a statistical evaluations of felt and instrumentally-recorded historical earthquake data. Motions calculated in this manner are in general agreement with recent published seismic hazard evaluations performed in the region. Statistical distributions of historical earthquakes in time and space, combined with accepted attenuation characteristics for the region, forms the basis for the seismic motion assessment presented in this report.

The computer program ProShake 2.0, was used to evaluate seismicity at the site. The program computes soil responses to vertical propagation of shear waves originating on the bedrock surface through the soil column. The bedrock earthquake data are supplied by the user. Because there is limited ground motion data from seismic events in the Puget Sound region, data recorded during earthquakes at other locations were used as input motions in our analysis. Actual ground motion data for the input earthquakes were scaled using peak ground acceleration values derived from the 2015 IBC results. Attenuations of the motions through the soil profile are based on properties for each layer of soil present at the site. The program includes several standardized soil types with properties obtained from field testing and analyses by researchers but the user is able to modify the properties to better correlate with actual site conditions.

Horizontal ground spectra for acceleration, velocity, and displacement for four representative earthquake motions scaled to a peak ground acceleration of 0.3g and using viscous damping factors of 5 percent are shown on Figure 6. There are two motions for each earthquake, one at bedrock/bottom soil layer (L4) and one at the surface (L1). Attenuation/amplification of the motion is relatively minor due to the high density of the glacial and non-glacial deposits interpreted to underlie the site which are generally modelled as soft rock. It is our opinion that the dense soils that underlie the site have a low potential for amplification of ground motion.

An acceleration spectrum recommended for design is included in Figure 6 along with the spectrum for the estimated maximum credible earthquake response (MCER). Vertical ground motion spectra can be conservatively estimated to be two-thirds of the corresponding horizontal ground motion spectra.

Other potential hazards associated with seismic activity include liquefaction and landsliding. Liquefaction is caused by a rise in pore pressures in a loose, fine sand deposit beneath the groundwater table. These conditions were not encountered in our explorations. It is our opinion that the medium dense and better native soils interpreted to underlie the site have a low potential for liquefaction or amplification of ground motion.

CONCLUSIONS AND RECOMMENDATIONS

General

It is our opinion that the planned bridge development is feasible from a geotechnical standpoint, provided that our recommendations are incorporated into the design and construction of this project. Although supporting the new pipeline bridge on shallow spread footings may be possible from a strictly bearing pressure point of view, NGA recommends that the bridge abutments be supported on deep foundations to better resist static and dynamic lateral loadings and to reduce the potential risk of erosion-induced loss of support. The deep foundations may consist of driven steel piles or cast-in-place reinforced concrete piers installed in pre-drilled shafts. The driven piles may be H-section or round pipe. For the concrete piers, we anticipate that near-surface soils will be loose to medium dense and that temporary steel casings may be needed to keep shafts open. Also, drilling agitation below the water level may cause some caving in the surrounding soils and it may be necessary to support soils at depth with casing or drilling fluid.

Although 15 to 20 feet of undocumented fill was encountered in the borings, we expect that the thickness of surficial topsoil, random fill, and/or disturbed native soil along the pipeline alignment will generally be less than 3 feet, based on our observations of surface conditions. Fill or disturbed native soil is likely to be thicker immediately adjacent to and uphill of the existing bulkheads. Temporary cuts in these soils will need to be sloped back or otherwise supported during construction. More detailed recommendations and discussions of other issues are presented in the following subsections.

Sediment and Erosion Control

The Soil Survey of Pierce County Area, Washington, by the Soil Conservation Service (SCS), was reviewed to determine the erosion hazard of the native on-site soils. The upland soils at this site are mapped as Kapowsin gravelly ashy loam, 0 to 6 percent slopes, originating from volcanic ash mixed with glacial drift over dense glaciomarine deposits. The soils within the Swan Creek ravine are mapped as Xerochrepts, 45 to 70 percent slopes, derived from sandy and gravelly outwash and/or glacial till. The erosion hazard is rated as slight for the Kapowsin ashy loam and severe for the Xerochrept, primarily due to the difference in surface gradients.

The majority of the on-site soils are considered as having a slight to low potential for erosion when exposed but the actual potential for erosion will be dependent on site grading, rainfall patterns, and soil and water management. Best Management Practices (BMPs) should be used to control erosion. Areas disturbed during construction should be protected from rainfall or surface run-on. Erosion control measures may include covering disturbed areas with plastic sheeting before storm events and diverting surface water away from construction areas. Silt fences and/or straw bales should be erected downslope of disturbed areas to control sediment-laden water from flowing off the site. We recommend that earthwork be performed in the drier summer season when surface water in Swan Creek is not present or at a minimal level to reduce the risks of erosion.

Construction requirements should include provisions for protecting disturbed areas after the work is completed. Ground surfaces should be reseeded and re-planted as soon as practical and the vegetation should be maintained until it is established. If re-planting is delayed for suitable weather conditions, straw, jute matting, or other temporary measures may be needed to protect the ground in the interim.

Site Preparation and Grading

After erosion control measures are implemented, site preparation should consist of stripping areas to be used during construction of all organic matter and organic-laden topsoil. Based on our observations, we anticipate that a minimum stripping depth of 12 inches will be needed to remove the bulk of organic-laden soils but greater depths should be anticipated. Excavated soils should be removed from the site or stockpiled for re-use in landscaped areas, if needed. Soils intended for re-use as structural fill should be kept free of organic matter and debris and should be managed to bring or maintain moisture content near the optimum point for compaction.

After the organic-laden topsoil has been removed, grading to create level pads at the new pier/abutment locations may begin. Sloping ground to receive fill should be excavated to create one or more “steps” consisting of level benches and vertical cuts before placing the fill. Each bench should be sized to accommodate the width of the excavating and compacting equipment but be no less than 4 feet wide. The height of each step should be equal to or less than the width of the bench below.

Exposed ground surfaces should be compacted to a firm and non-yielding condition. Areas observed to pump or weave during compaction should be over-excavated and replaced with properly compacted structural fill or rock spalls. Much of the existing embankment fill soils should be suitable for re-use as structural fill. Siltier sands and gravels will only be re-usable if the moisture content can be maintained near the optimum point for compaction. Organic-laden fill soils encountered deep within the embankment in both borings should not be used as structural fill. NGA should be retained to evaluate the suitability of all on-site and imported materials for use as structural fill material during construction.

We recommend that construction take place during the drier summer and early autumn months when stream levels are lowest, if possible. If construction takes place during the rainy season, additional expenses and delays should be expected due to higher water levels and wet conditions.

Temporary and Permanent Slopes

Temporary excavation stability is a function of many factors, including the type and consistency of soils, depth of the cut, surcharge loads adjacent to the excavation, length of time a cut remains open, and the presence of surface water or groundwater. It is exceedingly difficult under these variable conditions to estimate a stable, temporary, cut slope angle. Therefore, it should be the responsibility of the contractor to maintain safe slope configurations since they are continuously at the job site, able to observe the soil and groundwater conditions encountered and able to monitor the nature and condition of the cut slopes.

The following information is provided solely for the benefit of the owner and other design consultants and should not be construed to imply that Nelson Geotechnical Associates, Inc. assumes responsibility for job site safety. Job site safety is the sole responsibility of the project contractor.

For planning purposes, we recommend that all temporary cuts be no steeper than 1½ Horizontal to 1 Vertical (1½H:1V). If significant groundwater seepage or surface water flows are encountered, we expect that flatter inclinations may be necessary.

We recommend that cut slopes be protected from erosion. Measures taken may include covering cut slopes with plastic sheeting and diverting surface runoff away from the top of cut slopes. Excavated material should not be stockpiled any closer than 10 feet from the top of the cuts. We do not recommend vertical slopes for cuts deeper than 4 feet if worker access is necessary. We recommend that cut slope heights and inclinations conform to WISHA/OSHA standards. If the above inclinations cannot be met due to site constraints and/or worker access issues, we recommend that shoring be considered for temporary cuts. We are available to provide specific recommendations for temporary shoring, if needed, once the design has been finalized.

NGA recommends that permanent cut and fill slopes be graded no steeper than 3H:1V where possible. This recommendation is not intended to preclude constructing slopes steeper than 3H:1V where needed to recreate ravine slopes mimicking current conditions above and below existing embankment. However, it should be noted that the bottom 3 to 4 feet of the native ravine slopes are near vertical and composed of bare mineral soils indicating that the slopes are being eroded during high flows. Erosion at the bottom of slopes should be considered when designing the site restoration plans. Permanent slopes should be planted and the vegetative cover should be maintained until it is established.

Shallow Foundations

The new pipeline bridge may be supported on shallow spread footings although NGA recommends that bridge abutments be supported on deep foundations to better resist static and dynamic lateral loadings and to reduce the potential risk of erosion-induced loss of support. Shallow footings should bear on medium dense or better silty sand and gravel. Based on anticipated conditions and to properly embed the footings, we recommend that the foundation subgrade elevation be at least 4 feet below the existing ground surface.

Where less dense soils are encountered at footing bearing elevation, the subgrade should be over-excavated to expose suitable bearing soil. The over-excavation may be filled with 2- to 4-inch rock spalls, or the footing may be extended down to the native bearing soils. If footings are supported on structural fill, the fill zone should extend outside the edges of the footing a distance equal to one half of the depth of the over-excavation below the bottom of the footing. Subgrade soils should be supplemented as needed with a thin (less than about 6 inches) layer ¾-inch minus crushed aggregate to fill voids and establish a level and uniform bearing surface during fine grading.

If any structures are located within about 10 feet from the edge of the creek channel, NGA recommends that the structure subgrade be located at least 2 feet below the Ordinary High Water level (OHW). If the abutments are placed further back from the channel, NGA recommends that the bottom of the structures be located at least 2 feet below an imaginary line extending upward from the toe of the main creek channel at an inclination of 3 horizontal to 1 vertical (3H:1V), if possible. It may be advisable to cover the slope between the structure and the creek channel with cobble and boulder-sized rock or other durable material to resist erosion and slope raveling, especially if the permanent slopes are steeper than 3H:1V.

For abutments supported on the medium dense or better native soils, on structural fill, or on rock spalls extending to competent native material, and constructed as outlined above, we recommend an allowable design bearing pressure of not more than 2,500 pounds per square foot (psf) be used for design. Bearing soil should be evaluated by a representative of NGA. We should be consulted if higher bearing pressures are needed. Current IBC guidelines should be used when considering increased allowable bearing pressure for short-term transitory wind or seismic loads. Potential foundation settlement using the recommended allowable bearing pressure is estimated to be less than 1-inch total, based on our experience with similar projects.

Lateral loads may be resisted by friction on the base of the abutment and passive resistance against the subsurface portions of the abutment. A coefficient of friction of 0.35 may be used to calculate the base friction and should be applied to the vertical dead load only. Passive resistance may be calculated as a triangular equivalent fluid pressure distribution. An equivalent fluid density of 250 pounds per cubic foot (pcf) should be used for passive resistance design for a level ground surface adjacent to the footing. This level surface should extend a distance equal to at least three times the depth of the abutment. These

recommended values incorporate safety factors of 1.5 and 2.0 applied to the estimated ultimate values for frictional and passive resistance, respectively. To achieve this value of passive resistance, structural fill should be used as backfill against the front of the abutment. We recommend that the upper one foot of soil be neglected when calculating the passive resistance.

Deep Foundations

We recommend that the pipe bridge be supported on deep foundations. The deep foundations may consist of driven steel piles or cast-in-place reinforced concrete piers installed in pre-drilled shafts.

Driven Piles: Driven piles may be structural W or H steel beams or round pipe. Pipe piles should be driven open-ended to aid in penetrating through the gravel and cobbles in the subsurface soils. It may be advisable to use driving shoes on the end of the piles to also facilitate penetration through the granular material.

For preliminary design, we recommend that the piles be at least 20 feet deep and should be embedded a minimum of 10 feet into the competent native granular soils to develop adequate load capacities. Based on our evaluation, NGA recommends a design value of 1,800 pounds per square foot (psf) for skin friction. For steel beams, the skin area may be calculated assuming a circular section that circumscribes the steel member. An allowable end bearing capacity may be calculated using a pressure of 6,000 psf applied over a circular area with a diameter equal to the actual diameter for pipe piles or the equivalent diameter that circumscribes the steel member. Lateral resistance may be calculated based on an equivalent fluid density of 250 pounds per cubic foot (pcf) applied across two pile diameters. To account for possible loose fill or native soil at the surface, we recommend that the upper 4 feet of embedment be neglected for the purpose of calculating the lateral resistance.

In addition to the minimum recommended embedment into the native material, the piles should also be driven to a refusal criteria to provide for proper end bearing capacity. The refusal criterion should be developed based on the size of the pile and the rated energy of the hammer used to install the piles. We should be retained to review final plans, monitor pile installation, and evaluate pile refusal.

Drilled Piers: It is possible that drilled shafts may be advanced using open-hole drilling techniques but we believe there is a potential for loose near-surface soils to slough excessively, for gravelly soils to cave into the shaft, or for running (flowing) sand conditions to develop below the water level. We recommend that the contractor be prepared to insert temporary steel casings to keep shafts open. Concrete should be readily available on site at the time of drilling; open holes should not be left open for any extended period of time, as sloughing and caving soils may hamper pier installation.

For preliminary design, we recommend that the piers be at least 20 feet deep with a minimum embedment of 10 feet into competent native granular soils to provide adequate end bearing and friction capacities. Based on our evaluation, NGA recommends a design value of 1,800 pounds per square foot (psf) for skin friction and 6,000 psf for end bearing. Lateral resistance on the piers could be calculated based on an equivalent fluid density of 250 pounds per cubic foot (pcf) applied across two pile diameters. To account for possible loose fill or native soil at the surface, we recommend that the upper 4 feet of embedment be neglected for the purpose of calculating the lateral resistance.

The actual volume of the concrete pumped into each pile should be recorded and compared with the theoretical volume of the pile. Piles with a ratio of actual to theoretical volume less than 1.1 would be re-drilled. We should be retained to review pile design and observe pile installation. These recommended values incorporate a safety factor of two.

Abutments, Bulkheads, and Wing Walls

The lateral earth pressure acting on bridge abutments, bulkheads, and wing walls is dependent on the nature and density of the soil behind the structure, the amount of lateral movement which can occur as backfill is placed and the inclination of the backfill. Walls that are free to yield at least one-thousandth of the height of the wall are in an "active" condition. Walls restrained from movement by stiffness or bracing are in an "at rest" condition. Active earth pressure and at rest earth pressure can be calculated based on equivalent fluid density. Equivalent fluid densities for active and at rest earth pressure of 40 pcf and 60 pcf, respectively, may be used for design for a 3H:1V backslope. These values assume that relatively granular on-site soils are used for backfill, and that the backfill is drained. The preceding values do not include the effects of other surcharges, such as due to foundation loads, traffic or other surface loads. Surcharge effects should be considered where appropriate. These lateral pressures may be resisted by friction at the base of the wall and passive resistance against the foundation and front face of the wall using the values presented in the **Foundation Support** subsection above. For walls located on sloping ground, the foundation should be embedded a minimum of 2 feet and the top 2 feet of embedment should be ignored in passive pressure calculations.

All backfill should be well compacted. Care should be taken to prevent the buildup of excess lateral soil pressures due to over-compaction of the backfill. This can be accomplished by placing wall backfill in maximum 8-inch loose lifts and compacting with small hand-operated compactors. Permanent drainage should be installed as recommended in the **Site Drainage** subsection below.

Structural Fill

General: Fill placed beneath foundations, surrounding bridge abutments, behind walls, and for road/trail subgrade should be placed as structural fill. Structural fill, by definition, is placed in accordance with prescribed methods and standards, and is monitored by an experienced geotechnical professional or soils technician. Field monitoring procedures would include the performance of a representative number of in-place density tests to document the attainment of the desired degree of relative compaction. The area to receive the fill should be suitably prepared as described in the **Site Preparation and Grading** subsection of this report prior to beginning fill placement.

Sloping areas to receive structural fill should be benched prior to fill placement to key the fill into the slope. The benches should be level, with a minimum width of 4 feet, and be constructed by cutting into the existing sloping ground.

Materials: Structural fill should consist of a good quality, granular soil, free of organics and other deleterious material, and be well graded to a maximum size of about 3 inches. All-weather structural fill should contain no more than 5 percent fines (soil finer than U.S. No. 200 sieve, based on that fraction passing the U.S. 3/4-inch sieve). The use of some of the on-site soils as structural fill should be feasible, but will be dependent on moisture content of the material at the time construction takes place. We should be retained to evaluate proposed structural fill material prior to placement.

Fill Placement: Following subgrade preparation, placement of structural fill may proceed. All fill placement should be accomplished in uniform lifts up to 8 inches thick. Each lift should be spread evenly and be thoroughly compacted prior to placement of subsequent lifts.

All structural fill underlying foundations and roadway subgrade should be compacted to a minimum of 95 percent of its maximum dry density. Maximum dry density, in this report, refers to that density as determined by the ASTM D-1557 Compaction Test procedure. The moisture content of the soils to be compacted should be within about ± 2 percent of optimum so that a readily compactable condition exists. It may be necessary to over-excavate and remove wet soils in cases where drying to a compactable condition is not feasible. All compaction should be accomplished by equipment of a type and size sufficient to attain the desired degree of compaction.

Site Drainage

Surface Drainage: The finished ground surface should be graded such that runoff is directed to an appropriate surface water management system. Water should not be allowed to collect in any areas where footings or roadways are to be constructed. Final site grades should allow for drainage away from the bridge structure and running surface of the access road. Where finish grades are relatively level, we suggest

that the ground surface be sloped at a minimum gradient of 3 percent, for a distance of at least 10 feet away from structures and roadways.

Subsurface Drainage and Dewatering: Excavations extending below a depth of approximately 15 feet may encounter groundwater. The deeper the excavation, the more unlikely it will be to control seepage and maintain reasonably dry conditions. Dewatering deep foundation shafts may cause the shaft wall to collapse or lead to running sand conditions. The contractor should be prepared to work in flooded conditions or to install temporary casing or shoring in excavations extending below the groundwater surface.

Backfill surrounding bridge abutments and behind walls should consist of relatively free-draining, granular material extending to the surface. On-site soils with little to no fine material may also be acceptable but should be examined and approved by NGA before being used. A footing drain should be installed along the back of each abutment or wall and it should consist of a minimum 18-inch-wide zone of very clean (less than 3 percent fines), granular material placed at the base of the abutment or wall and wrapped in a filter fabric. Washed rock is an acceptable drain material. Footing drain pipes drains are not required if weep holes are installed through the abutment or wall. Weep holes should be installed as low as practical.

USE OF THIS REPORT

NGA has prepared this report for the City of Tacoma and their agents, for use in the planning and design of the new bridge on this site only. Our scope does not include services related to construction safety precautions and our recommendations are not intended to direct the contractors' methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design. There are possible variations in subsurface conditions between the explorations and also with time. Our report, conclusions, and interpretations should not be construed as a warranty of subsurface conditions. A contingency for unanticipated conditions should be included in the budget and schedule.

We recommend that NGA be retained to provide monitoring and consultation services during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether or not construction activities comply with contract plans and specifications. We should be contacted a minimum of one week prior to construction activities and could attend pre-construction meetings if requested.

Within the limitations of scope, schedule, and budget, our services have been performed in accordance with generally accepted geotechnical engineering practices in effect in this area at the time this report was prepared. No other warranty, express or implied, is made. Our observations, findings, and opinions are a means to identify and reduce the inherent risks to the owner.

o-o-o

It has been a pleasure to provide service to you on this project. If you have any questions or require further information, please call.

Sincerely,

NELSON GEOTECHNICAL ASSOCIATES, INC.

Timothy D. Huntting, PE
Senior Engineer

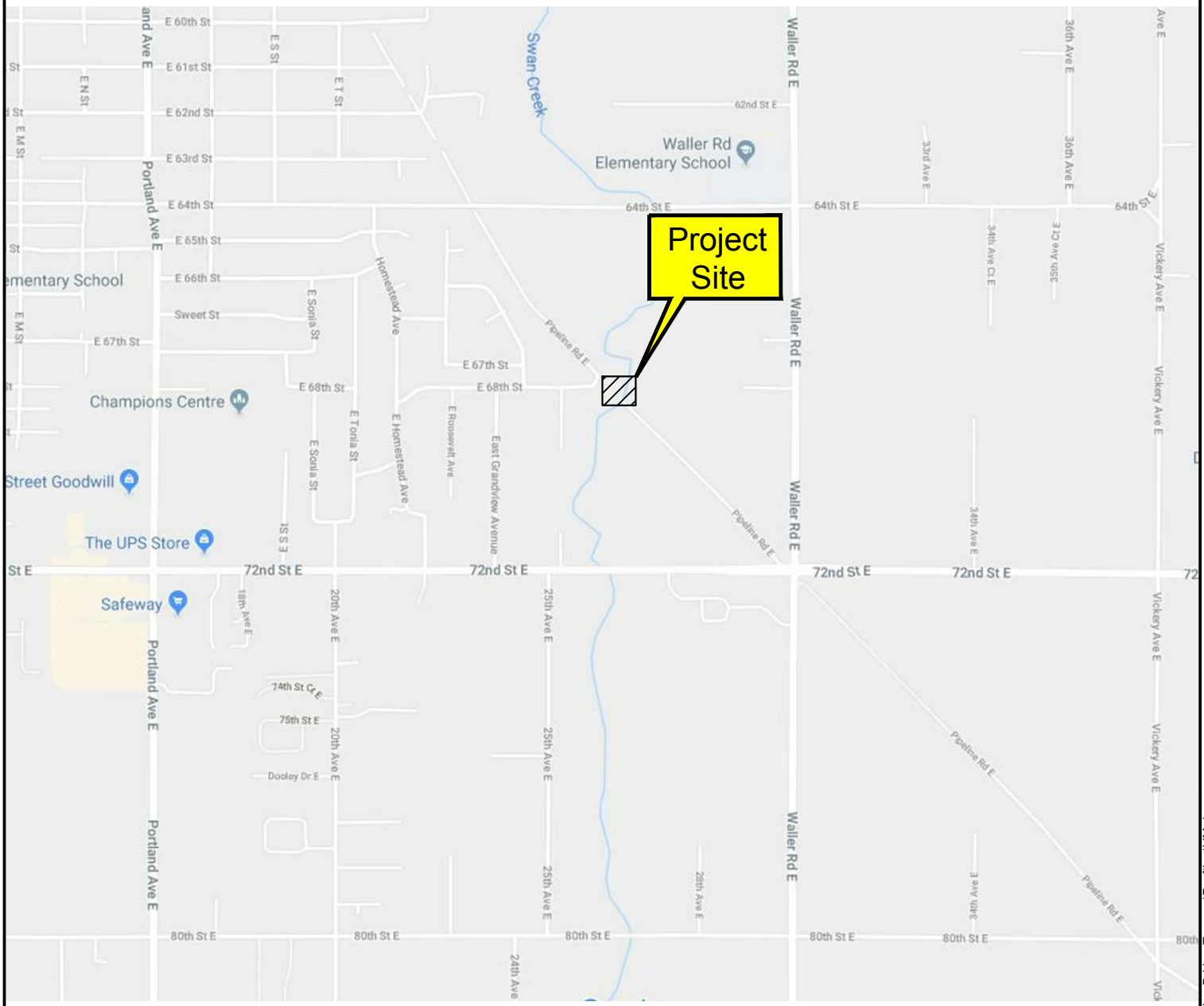
Khaled M. Shawish, PE
Principal

TDH:KMS:dy

Six Figures Attached

VICINITY MAP

Not to Scale



Tacoma, WA

Project Number 1012417	Swan Creek Pipeline Crossing Vicinity Map	NELSON GEOTECHNICAL ASSOCIATES, INC. GEOTECHNICAL ENGINEERS & GEOLOGISTS	No.	Date	Revision	By	CK
Figure 1			1	2/8/18	Original	DPN	TDH

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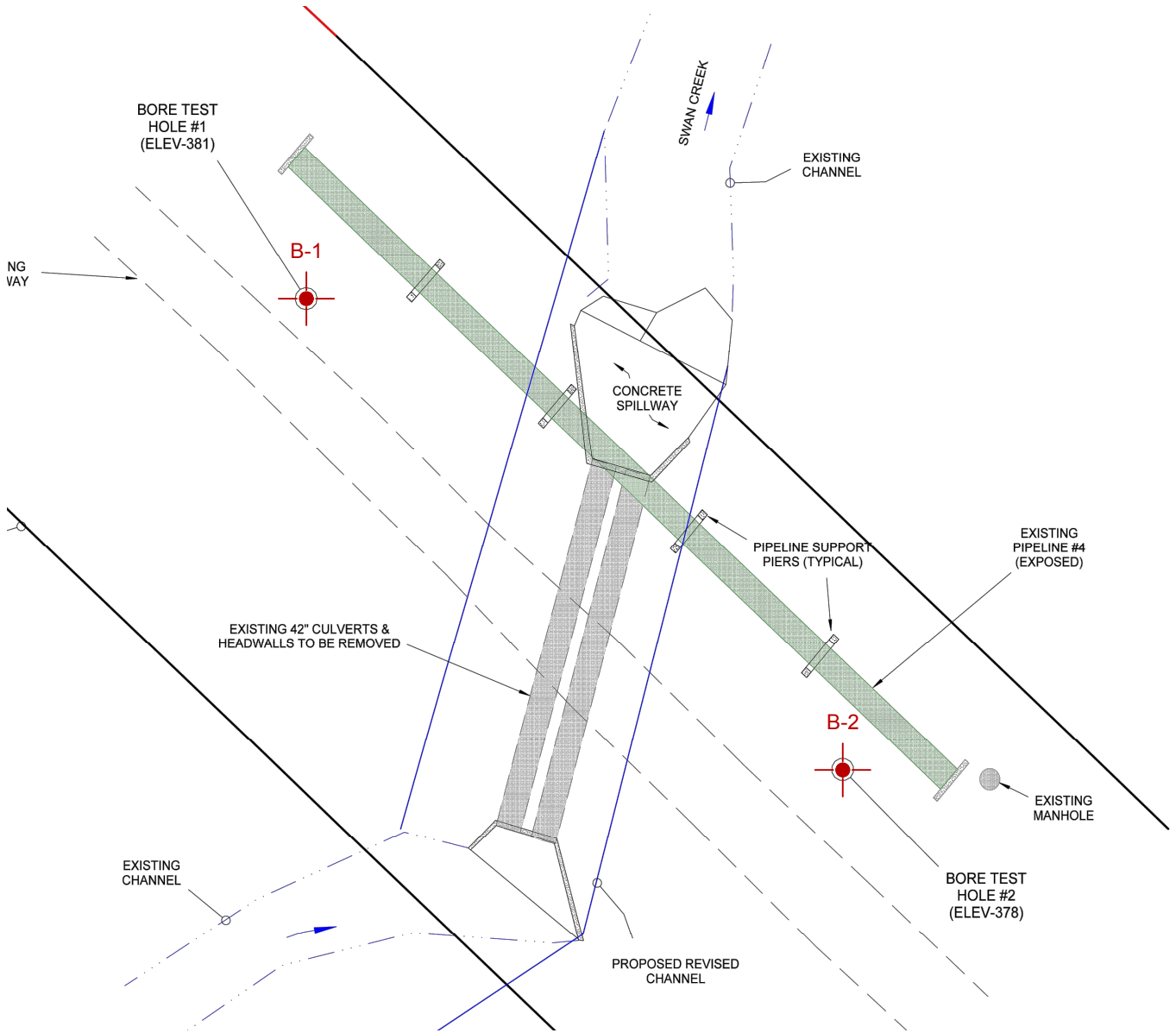
Site Plan



Scale: 1 inch = 30 feet

LEGEND

B-1

Number and approximate location of boring

Reference: Site plan based on a plan dated January 25, 2018 titled "Swan Creek Fish Passage Improvement Project," prepared by Tacoma Public Utilities Water Division.

Project Number 1012417

Figure 2

Swan Creek Pipeline Crossing Site Plan

NELSON GEOTECHNICAL ASSOCIATES, INC.

GEOTECHNICAL ENGINEERS & GEOLOGISTS

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UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE - GRAINED SOILS MORE THAN 50 % RETAINED ON NO. 200 SIEVE	GRAVEL MORE THAN 50 % OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVEL	GW	WELL-GRADED, FINE TO COARSE GRAVEL
			GP	POORLY-GRADED GRAVEL
		GRAVEL WITH FINES	GM	SILTY GRAVEL
			GC	CLAYEY GRAVEL
	SAND MORE THAN 50 % OF COARSE FRACTION PASSES NO. 4 SIEVE	CLEAN SAND	SW	WELL-GRADED SAND, FINE TO COARSE SAND
			SP	POORLY GRADED SAND
		SAND WITH FINES	SM	SILTY SAND
			SC	CLAYEY SAND
FINE - GRAINED SOILS MORE THAN 50 % PASSES NO. 200 SIEVE	SILT AND CLAY LIQUID LIMIT LESS THAN 50 %	INORGANIC	ML	SILT
			CL	CLAY
		ORGANIC	OL	ORGANIC SILT, ORGANIC CLAY
	SILT AND CLAY LIQUID LIMIT 50 % OR MORE	INORGANIC	MH	SILT OF HIGH PLASTICITY, ELASTIC SILT
			CH	CLAY OF HIGH PLASTICITY, FAT CLAY
		ORGANIC	OH	ORGANIC CLAY, ORGANIC SILT
HIGHLY ORGANIC SOILS			PT	PEAT

NOTES:

1) Field classification is based on visual examination of soil in general accordance with ASTM D 2488-93.

2) Soil classification using laboratory tests is based on ASTM D 2488-93.

3) Descriptions of soil density or consistency are based on interpretation of blowcount data, visual appearance of soils, and/or test data.

SOIL MOISTURE MODIFIERS:

Dry - Absence of moisture, dusty, dry to the touch

Moist - Damp, but no visible water.

Wet - Visible free water or saturated, usually soil is obtained from below water table

Project Number 1012417	Swan Creek Pipeline Crossing Soil Classification Chart	NELSON GEOTECHNICAL ASSOCIATES, INC. NGA GEOTECHNICAL ENGINEERS & GEOLOGISTS	No.	Date	Revision	By	CK
Figure 3		Woodinville Office: 17311-135th Ave. NE, A-500, Woodinville, WA 98072, (425) 486-1669 / Fax: 481-2510 — www.nelsongeotech.com — East Wenatchee Office: 5526 Industry Lane, #2, East Wenatchee, WA 98802, (509) 665-7696 / Fax: 665-7692	1	2/8/18	Original	DPN	TDH

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BORING LOG

B-1

Approximate Ground Surface Elevation: ??

Soil Profile: Description	Graphic Log	Group Symbol	Sample Data: Blow Count	Sample Location (depth in feet)	Penetration Resistance (Blows/foot - ●) / Moisture Content (Percent - ■)	Laboratory Testing	Piezometer Installation - Ground Water Data (Depth in Feet)
No recovery			27	5			
Gravelly, fine to coarse sand with silt and trace roots (very dense, moist) (**FILL**)			66	10			
-interbedded with dark brown, organic rich, silty fine to medium sand			50-6	15			
Gray-brown, silty fine to coarse sand with gravel, trace iron-oxide staining, and roots, (very dense, moist)			50-4	20			
-with iron-oxide staining, no roots		SM	50-5	25			

Penetration Resistance (Blows/foot - ●): 10 20 30 40 50 50+

Moisture Content (Percent - ■): 10 20 30 40 50 50+

LEGEND

- Depth Driven and Amount Recovered with 2-inch O.D. Split-Spoon Sampler
- Depth Driven and Amount Recovered with 3-inch Shelby Tube Sampler
- Solid PVC Pipe
- Slotted PVC Pipe
- Monument/ Cap to Piezometer
- ✱ Liquid Limit
- \+ Plastic Limit
- Concrete
- Bentonite
- Native Soil
- Silica Sand
- Water Level

M	Moisture Content
A	Atterberg Limits
G	Grain-size Analysis
DS	Direct Shear
PP	Pocket Penetrometer Readings, tons/ft
P	Sample Pushed
T	Triaxial

NOTE: Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis and judgement. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

Project Number	Swan Creek Pipeline Crossing Boring Log	Nelson Geotechnical Associates, Inc.
1012417		Geotechnical Engineers & Geologists
Figure 4		Woodinville Office: 17311-135th Ave. NE, A-500, Woodinville, WA 98072, (425) 486-1669 / Fax: 481-2510
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No.	Date	Revision	By	CK
1	2/8/18	Original	DPN	TDH

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BORING LOG

B-1 (cont.)

Logged by: ABR on 1/25/18

Soil Profile: Description	Graphic Log	Group Symbol	Sample Data: Blow Count	Sample Location (depth in feet)	Penetration Resistance (Blows/foot - ●) 10 20 30 40 50 50+ / Moisture Content (Percent - ■) 10 20 30 40 50 50+	Laboratory Testing	Piezometer Installation - Ground Water Data (Depth in Feet)
Gray-brown, silty fine to coarse sand with gravel and iron-oxide staining (very dense, moist)		SM	50-3				
Gray, gravelly fine to coarse sand with silt and iron-oxide weathering (very dense, wet)			50-3				
		SP-SM	50-3				
-becomes fine to coarse sand with silty and gravel			50-6				
Gray, silty fine to coarse sand with gravel (very dense, moist)		SM	54-8				
Boring terminated below existing grade at 51.4 feet on 1/25/18. Groundwater seepage was encountered at 35.0 feet during drilling							

35 40 45 50 55

LEGEND

- Depth Driven and Amount Recovered with 2-inch O.D. Split-Spoon Sampler
- Depth Driven and Amount Recovered with 3-inch Shelby Tube Sampler
- Solid PVC Pipe
- Slotted PVC Pipe
- Monument/ Cap to Piezometer
- * Liquid Limit
- + Plastic Limit
- Concrete
- Bentonite
- Native Soil
- Silica Sand
- Water Level
- M Moisture Content
- A Atterberg Limits
- G Grain-size Analysis
- DS Direct Shear
- PP Pocket Penetrometer Readings, tons/ft
- P Sample Pushed
- T Triaxial

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BORING LOG

B-2

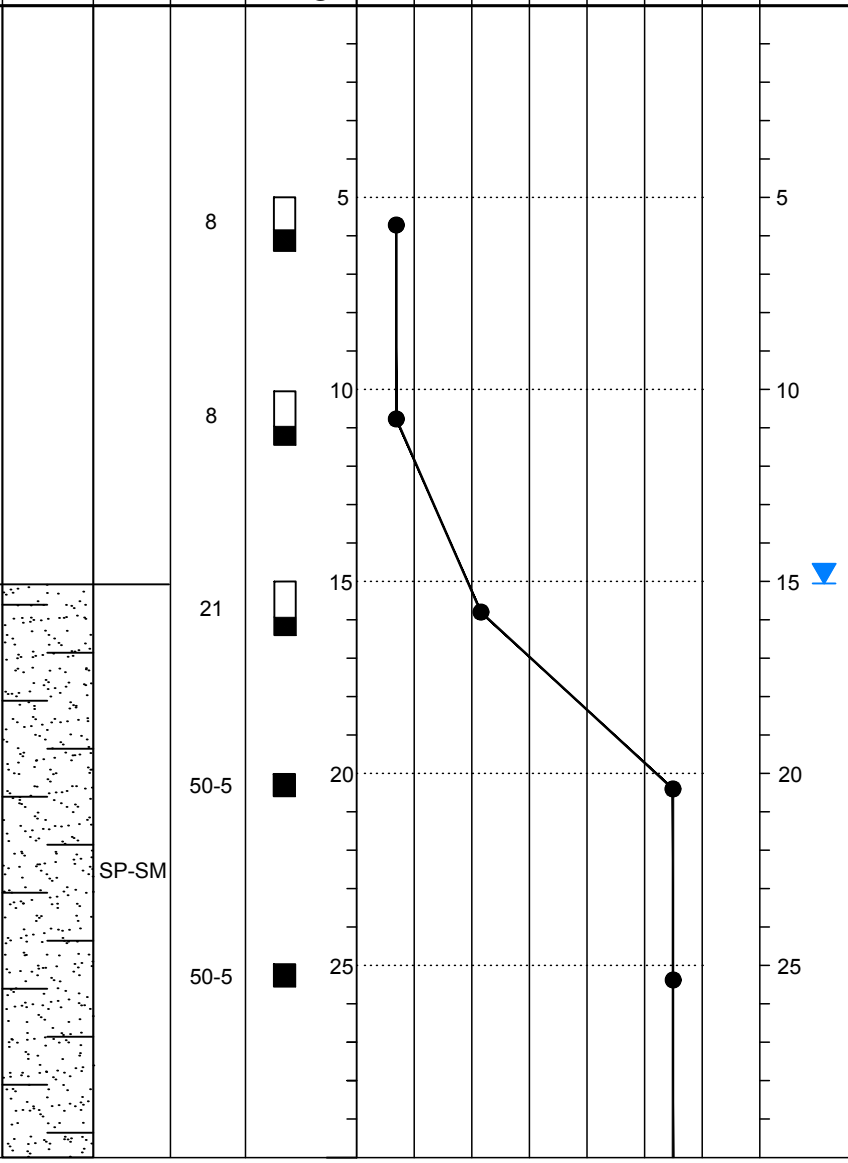
Approximate Ground Surface Elevation: ??

Soil Profile: Description	Graphic Log	Group Symbol	Sample Data: Blow Count	Sample Location (depth in feet)
Dark brown, silty fine to medium sand with gravel, roots, and organics (loose, moist) (**FILL**)			8	
-becomes organic rich			8	
Gray-brown, gravelly fine to coarse sand with silt, gravel, and trace iron-oxide staining (medium dense, wet)			21	
-becomes gray, fine to coarse sand with silt, gravel, and trace iron-oxide weathering, very dense		SP-SM	50-5	
-becomes gravelly, fine to coarse sand with silt and trace iron-oxide weathering			50-5	

Penetration Resistance (Blows/foot - ●) / Moisture Content (Percent - ■): 10 20 30 40 50 50+

Laboratory Testing

Piezometer Installation - Ground Water Data (Depth in Feet)



Logged by: ABR on 1/24/18

LEGEND

- Depth Driven and Amount Recovered with 2-inch O.D. Split-Spoon Sampler
- Depth Driven and Amount Recovered with 3-inch Shelby Tube Sampler
- Solid PVC Pipe
- Slotted PVC Pipe
- Monument/ Cap to Piezometer
- * Liquid Limit
- + Plastic Limit
- Concrete
- Bentonite
- Native Soil
- Silica Sand
- Water Level

M	Moisture Content
A	Atterberg Limits
G	Grain-size Analysis
DS	Direct Shear
PP	Pocket Penetrometer Readings, tons/ft
P	Sample Pushed
T	Triaxial

NOTE: Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis and judgement. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

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BORING LOG

B-2 (cont.)

Logged by: ABR on 1/24/18

Soil Profile: Description	Graphic Log	Group Symbol	Sample Data: Blow Count	Sample Location (depth in feet)	Penetration Resistance (Blows/foot - ●) / Moisture Content (Percent - ■)	Laboratory Testing	Piezometer Installation - Ground Water Data (Depth in Feet)
Gray, gravelly fine to coarse sand with silt and iron-oxide weathering (very dense, moist)		SP-SM	72				
Gray, silty fine to coarse sand with gravel and iron-oxide staining (very dense, moist)		SM	50-5				
-becomes silty, fine to medium sand with gravel, dense			38				
Gray, gravelly fine to coarse sand with silt and iron-oxide weathering (very dense, moist)		SP-SM	50-6				
-becomes gray-brown			50-4				
Boring terminated below existing grade at 51.0 feet on 1/24/18. Groundwater seepage was encountered at 15.0 feet during drilling							

Penetration Resistance / Moisture Content: 10 20 30 40 50 50+

Depth: 35 40 45 50 55

LEGEND

- Depth Driven and Amount Recovered with 2-inch O.D. Split-Spoon Sampler
- Depth Driven and Amount Recovered with 3-inch Shelby Tube Sampler
- Solid PVC Pipe
- Slotted PVC Pipe
- Monument/ Cap to Piezometer
- * Liquid Limit
- + Plastic Limit
- Concrete
- Bentonite
- Native Soil
- Silica Sand
- Water Level

M	Moisture Content
A	Atterberg Limits
G	Grain-size Analysis
DS	Direct Shear
PP	Pocket Penetrometer Readings, tons/ft
P	Sample Pushed
T	Triaxial

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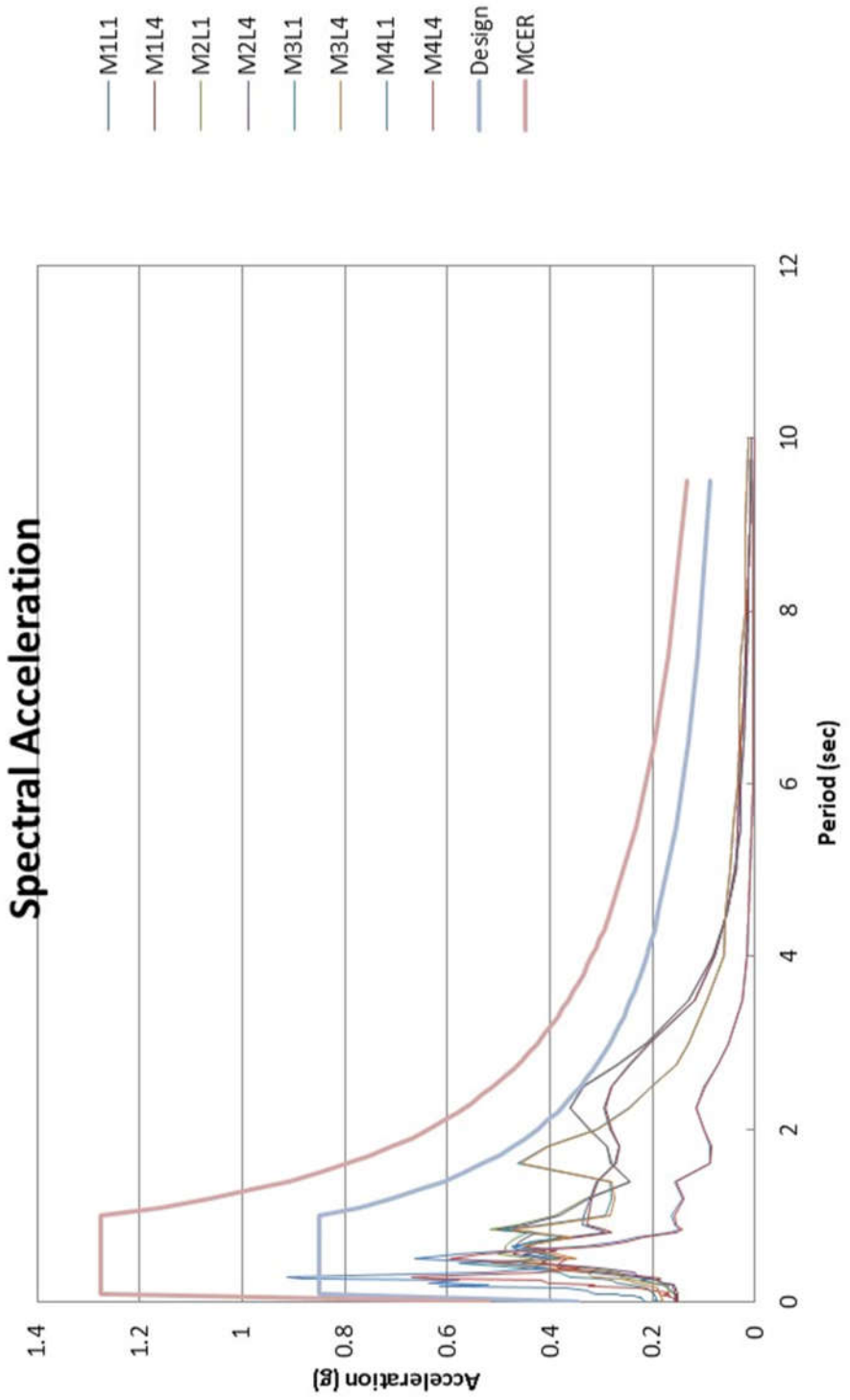

| Project Number **1012417** | **Swan Creek Pipeline Crossing Spectral Acceleration** | NGA NELSON GEOTECHNICAL ASSOCIATES, INC. GEOTECHNICAL ENGINEERS & GEOLOGISTS
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No.	Date	Revision	By	CK
1	4/6/18	Original	TDH	KMS

Figure 6