Seattle Public Utilities
Geotechnical Engineering

MEMORANDUM

SEAN LEE CARAWAY STATE OF WASHINGTON 38046 REGISTERED PROFESSIONAL ENGINEER

Date: February 20, 2013

To: Holly McCracken
SPU Project Management and Engineering Division

From: Sean Caraway, P.E.
Hilja K. Welsh
SPU Geotechnical Engineering

Subject: GEOTECHNICAL RECOMMENDATIONS, THORNTON CREEK CULVERT REPAIR PROJECT, KING COUNTY, WASHINGTON

INTRODUCTION

In accordance with your request, Seattle Public Utilities (SPU) Geotechnical Engineering performed subsurface explorations for the Thornton Creek Culvert Repair Project located in King County, Washington. This draft memorandum presents the results of our subsurface investigations, geotechnical design parameters and project recommendations that have been developed based on discussions with the project team and review of current and existing information and conceptual plans.

PROJECT LOCATION AND DESCRIPTION

The Thornton Creek Culvert Repair Project site is located on NE 93rd Street approximately 150-feet east of Sand Point Way NE and adjacent to Matthews Beach Park, approximately 1000-feet upstream of the confluence with Lake Washington (see Figure 1). Thornton Creek passes through the Thornton Creek culvert in a northwest to southeast direction on its way to the Lake Washington shoreline. The culvert extends under the only road access to Matthews Beach Park and 51st Avenue NE. The 8-foot wide three sided box culvert, believed to have been constructed in the early 1930s, consists of a concrete wingwall (running east-west), inlet and outlet walls, culvert walls and bulkhead walls supported by shallow footings. The bottom of the culvert is the natural creek bottom.

In 2012, Osborn Consulting, Inc. (OCI), along with CivilTech Engineering (CTE), investigated the existing culvert condition, performed a risk assessment and identified structural repair alternatives and prepared a technical memorandum with their findings. OCI observed structural damage and shifting of the culvert walls caused by various levels of undermining of footings that exist along the length of the culvert, predominantly at the outlet and inlet locations and on the eastern side of the culvert. As a result, OCI recommended structural repairs to extend the life of the culvert.

AUTHORIZATION AND SCOPE OF WORK

Our work was requested and authorized by Holly McCracken, on behalf of the SPU Project Management and Engineering Division. Our scope of work included:

- Reviewing readily available geotechnical/geologic information for the project site and vicinity;
- Conducting a geotechnical exploration program including the completion of three borings;
- Performing laboratory testing and engineering analyses to develop geotechnical recommendations for the Thornton Creek Culvert Repair Project as presented herein; and,
- Preparing this geotechnical memorandum, which summarizes our investigations, conclusions and recommendations.

SUBSURFACE EXPLORATIONS AND TESTING

EXPLORATIONS

As part of this study, we completed three subsurface explorations on both sides of the box culvert to characterize soil types. Our field explorations consisted of three borings (B-1 through B-3) using hollow stem auger drilling techniques conducted on November 26, 2012. The borings were drilled by Geologic Drilling, Inc. of Spokane, Washington, under contract to the SPU Geotechnical Engineering. The borings reached depths of between 25.8 and 31.5 feet below ground surface (bgs). The exploration locations are shown on Figure 1. Summary logs of the subsurface explorations are included as Figures 3 through 5. Figure 2 is a key to the terms and symbols used on the logs.

An SPU Geotechnical Engineering representative was present throughout the field exploration program to observe the explorations, procure soil samples, and prepare descriptive logs of the explorations. Soil samples were obtained from all borings at 2.5-foot and 5-foot depth intervals using the Standard Penetration Test (SPT, ASTM D-1586). The 2.0-inch outside diameter (OD) SPT sampler was driven into the soil a distance of 18 inches using a 140-pound drive hammer falling a distance of 30 inches. A rope and cathead system was used to operate the hammer and drive the sampler. Recorded blows for each 6 inches of sampler penetration (blow counts) are shown on the summary logs in this appendix. The blow counts provide a qualitative measure of the relative density of cohesionless soil, or the relative consistency of fine-grained soils. Soils were classified in general accordance with ASTM D-2488 *Standard Practice for Description and Identification of Soils (Visual-Manual Procedure).*

Geotechnical laboratory tests were performed on selected samples collected during the field exploration. We conducted natural moisture content and grain size determination in general accordance with appropriate ASTM test standards on select samples collected during the drilling, which helps to determine index and engineering properties of the soil units encountered. We performed six laboratory tests in accordance with ASTM D1140 - 00(2006) Standard Test Methods for Amount of Material in Soils Finer than No. 200 (75-µm) Sieve to determine the silt or fine content to assist in our liquefaction analyses.

The test results are graphically indicated at the appropriate sample depth on the summary logs (Figures 3 through 5).

SITE CONDITIONS

SURFACE CONDITIONS AND ENVIRONMENTALLY CRITICAL AREAS

The development in the Thornton Creek culvert project area is generally mixed use combining private residential buildings to the south of NE 93rd Street, City of Seattle Parks Department property to the northeast of the culvert and the King County Wastewater Treatment Division Matthews Park Pumping Station to the north. The 8-foot wide culvert runs below the asphalt paved NE 93rd Street for a length of approximately 35 feet. At the locations of the three borings the asphalt ranged in thickness from 2.5 to 6 inches. At the time of drilling, the depth from top of asphalt to the bottom of Thornton Creek at the inlet was approximately 8 feet. The topography in the project area is relatively flat and according to the City of Seattle GIS the surface elevation is approximately 26 feet (NAVD88 datum). The existing underground utilities along NE 93rd Street in the project area include a distribution water main that runs parallel to and under NE 93rd St and active water service lines feeding perpendicularly from the water main.

The Seattle Department of Planning and Development has designated multiple sections within the project area as environmentally critical areas (ECAs). The ECAs are defined in the Seattle Municipal Code, Chapter 25.09. Mapped ECAs, at the project site, include wildlife habitat, wetland and riparian corridor.

SUBSURFACE CONDITIONS

The general geologic condition of Seattle is a result of glacial and non-glacial activity that occurred in the area over the course of millions of years. The non-glacial Holocene period began at the end of the last glacial activity about 10,000 years ago (10 ka) and continues to the present. The most recent and extensive glacial activity in the Puget Sound area was the Vashon Stade of the Fraser glaciation (18-10 ka). Preceding the Fraser glaciation, was the Olympia nonglacial and pre-Olympia glacial interval (70-18 ka) (Troost, et al., 2005).

Review of a geologic map (Troost, et al., 2005) indicates that the project site is underlain primarily by Holocene lake (Ql) and alluvial (Qal) deposits and pre-Olympia glacial deposits (Qpog).

Holocene lake deposit soils, also called lacustrine deposits, are typically silt and clay with local sand layers, peat and other organic sediments that are deposited in slow flowing water. Alluvium is generally sand, silt, gravel and cobbles deposited by running water and may contain trace organics. Pre-Olympia glacial deposits consist of silt, sand, gravel and till of glacial origin. The following sections present descriptions of the deposits encountered in our explorations in the order of stratigraphic sequence, with the youngest unit described first, followed by a description of groundwater conditions.

Fill

Coarse gravel fill was encountered below the asphalt surface in all soil borings and ranges in thickness from 1.5 to 4 feet based on driller observation. The fill consists of medium dense, silty sandy gravel (GM)[1]. Natural moisture content or grain size determination was not performed in the base coarse material. The depth of cover (fill) over the culvert appears to be 2 feet or less. The relatively shallow depth of fill, in combination with the preferred mitigation option indicated that geotechnical laboratory testing of the fill material would have limited significance in the design and installation recommendations. The preferred H-pile mitigation will include deeper elements for support of the existing walls, which will completely penetrate the shallow fill layer for embedment in hard or very dense native soil.

Holocene Lake Deposit

Lacustrine soils underlie the fill in all borings and is approximately 10.5 to 16 feet thick. Soils are very soft to medium stiff, moist to wet, silt, sandy silt, sandy silt with trace gravel and organic silt (ML, OL), and very loose to loose sand and silty sand (SP, SM). Wood fragments and other organics were found in almost all samples. N-values obtained for this unit range from 2 to 13, with an average of 5. Moisture content ranged from 13.3 to 139.5 percent, and an average of 53 percent.

Alluvium

Alluvial soils underlie the fill and lake deposits in all borings and is approximately 10 to 19.5 feet thick. Soils are medium dense to very dense, moist to wet, silty sand, sand with silt and sand with silt and gravel (SM, SP-SM, SW-SM) with stiff to very stiff sandy silt (ML) encountered in borings B-101 and B-102. N-values obtained for this unit range from 12 to 50/4", with an average of 36. Moisture content ranged from 15 to 39 percent, and an average of 23 percent.

Pre-Olympia Glacial Deposits

The pre-Olympia glacial deposit was observed in soil boring B-103 at a depth of 24 feet bgs, and is the oldest unit encountered during this exploration. We interpret these soils to be glacial and consist of moist, hard sandy silt with fine gravel (ML). The N-value for the unit was 50/4". The

[1] Soil classification in accordance with the Unified Soil Classification System (USCS).

natural water content of the one sample collected from the pre-Olympia glacial deposit was 13 percent.

Groundwater

Groundwater was encountered during our investigation in all three soil borings. At the time of drilling, depths, to groundwater, ranged from 6 to 15 feet bgs, within the borings. We would expect the groundwater table to fluctuate throughout the year and be at its highest during the late winter and spring seasons and its lowest during the late summer and early fall seasons.

SEISMIC SETTING

The Puget Sound area is known to be seismically active. The seismic hazard in the area comes primarily from three sources: subduction zone, intraslab or Benioff zone, and shallow crustal earthquakes. Subduction zone earthquakes occur when the interface between the North American tectonic plate and the subducting Juan de Fuca plate ruptures. These events are likely to have magnitudes of up to 9, but the distance to the rupture zone would reduce the intensity of shaking at the project site. Shaking during these events could last over one minute in duration. Intraslab events occur due to tensional rupture within the subducting Juan de Fuca plate at depths of 45 to 60 kilometers. This is the source of our largest historical earthquakes that have affected the project site and has the potential for magnitude 7.5 events.

Shallow crustal earthquakes occur on shallow faults within the Seattle area due to tectonic stresses. Several minor earthquakes occur in the area each year, most of which are not even felt. However, some of the shallow faults are capable of producing significant, damaging earthquakes. Perhaps the most notable of these faults is the Seattle Fault. Recent research indicates that this fault is capable of producing an earthquake with a magnitude 7.5 or higher, which, given the shallow depth and proximity to the Seattle urban area, could produce intense shaking at the project site. Current understanding of the structure of the Seattle Fault zone indicates that the fault consists of a blind thrust underlying a faulted roof complex. Several subparallel backthrusts are located within the roof complex, and have been considered splays of the Seattle fault. The project site is located approximately 13 kilometers north of the Seattle Fault Zone (SFZ).

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

Based on the results of our geotechnical investigations and analyses, it is our opinion that the proposed culvert repair project is feasible, within the limitations presented below, provided the recommendations of this report are incorporated in design and construction.

It is our understanding that the following three repair options are being considered to repair structure damage and limit additional wall settlement and/or lateral movement along the east side of the culvert:

- Option 1: Install steel struts between the culvert side walls.
- Option 2: Install steel H-piles on the east side of the east culvert wall. Shafts would be drilled for the upper portion of the pile to allow installation of thru-bolts which will pass through the concrete side wall of the culvert. The shafts will then be backfilled with concrete and the H-piles will provide deeper foundation support
- Option 3: Extend the existing culvert footing and install a deadman anchor at three locations along the culvert.

We understand that Option 2 is the most viable method for repair; therefore this memorandum addresses the key geotechnical issues only for this option. The key geotechnical issues for this project relate to foundation support for the existing Thornton Creek Culvert, as well as other design and construction considerations, which include seismic design considerations, drilled shaft construction for installation of the H-piles, construction vibrations and site preparation and earthwork, which are discussed below.

SEISMIC CONSIDERATIONS

This section discusses seismic hazards including site response, fault rupture, liquefaction and lateral spreading.

Design Response Spectrum

The seismic design for the mitigation construction will be in accordance with the 2009 Seattle Building Code (SBC), which incorporates the International Building Code. Computation of forces used for seismic design for this code is based on seismological input and site soil response factors. Ground motions considered for evaluation using these guidelines are defined as motions with approximately 2 percent probability of exceedance in 50 years or about a 2,475-year return period.

The seismological inputs are short-period spectral acceleration (S_s) and spectral acceleration at 1-second period (S_1) taken from approved National Seismic Hazard Mapping Project (USGS, 2009) spectral response acceleration contour map for Class B sites. Sites classified as Class B are defined as firm rock having a shear-wave velocity between 2,500 and 5,000 feet per second (fps) in the top 100 feet. The mapped S_s and S_1 values in the vicinity of the project are 1.23 g and 0.42 g, respectively.

The 2009 SBC expresses the effects of site-specific subsurface conditions on the ground motion response in terms of the “site class” for the site. The “site class” represents the density or stiffness of the soil profile underlying the site and is used to account for the seismic response of the soil profile. The “site class” can be correlated to the average standard penetration resistance (N-value) in the upper 100 feet of the soil profile. Based on available existing information, this site can be classified as Site Class D. The site coefficients for Site Class D and seismic design parameters for the project site are shown in Table 1. These seismic design parameters were developed without regard to liquefaction potential. See the Liquefaction and Lateral Spreading section below for discussion on liquefaction hazard.

Table 1 – Seismic Design Parameters, Site Class D

Seismic Design Parameter	Short Period (g)	Long Period (g)
Mapped Spectral Acceleration	$S_s = 1.23$	$S_1 = 0.42$
Site Coefficients	$F_a = 1.01$	$F_v = 1.58$
Maximum Considered Earthquake Spectral Response Acceleration	$S_{MS} = 1.24$	$S_{M1} = 0.67$
Design Spectral Response Acceleration	$S_{DS} = 0.83$	$S_{D1} = 0.44$

Liquefaction and Lateral Spreading

As part of our seismic analysis, we analyzed the potential of the soil within the project site to liquefy. Liquefaction is a momentary loss of some portion of soil shear strength during a seismic event. During a seismic event, the loose soil particles tend to densify. This occurs in a short amount of time and the water between the soil grains does not have sufficient time to drain. The result is the water between the soil grains experiences excess pore pressures. This causes a reduction in the effective stress within the soil mass and the result is a reduction, and sometimes total loss, of shear strength. Primary factors controlling the development of liquefaction include intensity and duration of strong ground motion, characteristics of the subsurface soil, in-situ stress conditions and the depth to groundwater. Soil types most prone to liquefaction are loose, saturated and relatively cohesionless, such as wet, clean sands and gravels. Liquefaction is not limited to clean granular soils. Fine grained soils, such as low plasticity silts, can also be susceptible to liquefaction during seismic shaking.

According to the current subsurface explorations (B-101 and B-102), on the south side of NE 93rd Street, the culvert is generally surrounded by areas of less susceptible cohesive soils in the upper 15 feet. The primarily cohesionless or low cohesion soils, below 15 feet are generally too dense to liquefy during a seismic event and the depth to groundwater in these boring locations generally reduces liquefaction potential. The factor of safety against liquefaction (FS) for the soils in B-101 and B-102, are generally at values of 1.0 to 1.5. Localized pockets may be susceptible to liquefaction during a seismic event. The soils encountered in boring B-3, from 6 to 14 feet bgs, have a FS of less than 1.0 against liquefaction. This appears to be due, primarily to a shallower groundwater level (at 6 feet bgs) and a relatively loose soil profile in the upper 15 feet.

Because liquefaction would occur within localized soil pockets above the embedment depth of the H-piles and given the relatively flat topography of the site, in our opinion, liquefaction and lateral spreading effects at the project should not be a concern for design, provided the H-piles are adequately embedded into the competent bearing stratum.

H-PILE RECOMMENDATIONS

This section provides necessary design parameters based on our subsurface investigations and analyses for design and installation of steel H-piles on the east side of the east culvert wall.

We anticipate that predominately lake deposits and alluvium will be encountered from the surface to approximately 25 feet (or deeper) bgs. The alluvial soils, encountered below about 20 feet are generally medium dense to very dense and should provide adequate support for the anticipated structural loads. Pre-Olympia deposits may be encountered below 25 to 30 feet. For design purposes, the engineering properties of the anticipated soil units are summarized in Table 2. These properties are primarily based on SPU Geotechnical Engineering experience in similar geologic settings within the City of Seattle.

Table 2 – Summary of Soil Engineering Properties for Thornton Creek Culvert Soil Units

Anticipated Soil Unit	Moist Unit Weight, γ (pcf)	Saturated Unit Weight, γ (pcf)	Effective Strength Parameters	
			Friction Angle, ϕ (deg.)	Cohesion, c (psf)
Very Loose Silty Sand to very Soft Silt (Lake Deposit)	90	105	28	0
Medium Dense to Very Dense Silty Sand, Stiff to Very Stiff Sandy Silt (Alluvium)	120	125	34	0
Hard Silt (Pre-Olympia)	125	130	38	300

Lateral Resistance

Earthquakes and unbalanced earth loads will subject the proposed structures to lateral forces. Lateral loading can be resisted by a combination of passive soil resistance and friction along the base of the structure. Passive soil resistance can be determined using an equivalent fluid weight of 240 pounds per cubic foot (pcf) for saturated lake deposit soils, 385 pcf for saturated alluvial soils and 480 pcf for underlying Pre-Olympia soil. Base friction can be determined using a coefficient of friction of 0.4 for concrete placed directly on competent alluvial soils and a coefficient of friction of 0.5 for concrete placed on competent Pre-Olympia soil. The coefficient of friction should be used in conjunction with the normal load adjusted for the uplift pressure due to groundwater. These are ultimate values, and do not include a safety factor.

Static and Seismic Lateral Earth Pressures

While there are no proposed new retaining walls for any of the improvement options, we are presenting lateral pressures for project background information as well as earth pressures against the existing culvert walls. We assume lake deposits and alluvium will be present adjacent to the H-piles and culvert walls, although the deepest portions of the H-piles may be surrounded by hard or very dense, glacially consolidated soils. If the H-piles or walls will be designed to allow deflection at the top of at least 0.1 percent of the wall height, the walls can be designed for the active condition. If the H-piles or walls are to be fixed or restrained, against such movement, at-rest conditions should be used. The active and at-rest equivalent fluid unit weights do not include surcharge loads from roadways, construction equipment, stockpiled materials and/or adjacent structure foundations.

Table 3 – Static Lateral Earth Pressures for Thornton Creek Culvert Soil Units

Anticipated Soil Unit	Approximate Depth Below Ground Surface (ft)	Active Pressure (psf/ft)	At-Rest Pressure (psf/ft)
Very Loose Silty Sand to very Soft Silt (Lake Deposit)	0 to 15	40	60
Medium Dense to Very Dense Silty Sand, Stiff to Very Stiff Sandy Silt (Alluvium)	15 to 25	34	54
Hard Silt (Pre-Olympia)	Below 25	30	48

Seismic shaking would induce additional earth pressures on the proposed structures. These values depend on the size of the design earthquake, and assume active earth pressure conditions. Table 4 lists our recommended seismic earth pressure surcharges for the design level event, with a horizontal coefficient based on one half of the peak ground acceleration (1/2*PGA = 0.17 g, where g is the acceleration of gravity). Since the height of the retained soil is less than 10 feet, the potential seismic surcharges can be assumed to be uniformly distributed over the depth of the retained soil.

Table 4 – Seismic Lateral Earth Pressures

Loading Condition	Design Level Earthquake 2,475-Year Return Period
Yielding (Active)	7H
Non-Yielding (At-Rest)	18H

Note: Lateral seismic surcharge values in psf where H is the height of the retained soil in feet.

H-Pile Design Recommendations

American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor (LRFD) Bridge Design Specifications (2010) was used for calculating the H-pile drilled shaft design parameters. Nominal axial unit capacities for the piles, within the described soil units, are presented in Table 5, in units of kips per square foot. The nominal axial resistance of cohesionless soils is determined by the O'Neill and Reese β-method (2010 AASHTO LRFD Bridge Design Specifications (Equation 10.8.3.5.2b-1)). The nominal tip resistance, q_p, in ksf, for drilled shafts in cohesionless soils by the O'Neil and Reese (1999) method shall be taken as for $N_{60} \leq 50$, q_p, $=1.2\ N_{60}$. (Equation 10.8.3.5.2c-1). Resistance factors of 0.55 and 0.50 should be applied to the ultimate side friction and end bearing values, respectively (Table 10.5.52.4-1 – Resistance Factors for Geotechnical Resistance of Drilled Shafts).

Table 5 – Nominal Axial Unit Pile Capacities for Thornton Creek Culvert Soil Units

Anticipated Soil Unit	Approximate Depth Below Ground Surface (ft)	End Bearing (ksf)	Side Friction (ksf)
Very Loose Silty Sand to very Soft Silt (Lake Deposit)	0 to 15	Not Recommended	Not Recommended
Medium Dense to Dense Silty Sand, Stiff and Very Stiff Sandy Silt (Alluvium)	15 to 25	15	1.1
Very Dense Silty Sand (Alluvium) or Hard Silt (Pre-Olympia)	Below 25	50	1.5

Pile embedment should be determined by evaluating different contributing factors. The final pile embedment depths will be determined based on structural requirements. We recommend a minimum embedment depth of 5 feet into the competent, glacially consolidated, Pre-Olympia aged soils or very dense alluvial soils (depth of contact between dense alluvial soils and Pre-Olympia soils may vary). These minimum embedment depth criteria should lead to minimum total pile depths of about 30 feet below the existing ground surface. The recommended minimum embedment should provide for adequate pile penetration below the isolated zones of potentially liquefiable soils.

Actual lateral loading is not available at this time. We do not anticipate that lateral loading will be a primary loading component for the H-piles. A certain level of lateral resistance from the H-piles may be used to help support the existing culvert walls and associated wing walls due to the structural distress, which is already evident from lateral movement of these features.

Table 6 presents lateral parameters for design of the backfilled H-pile shaft foundations, in accordance with the LPILE^{Plus} software from Ensoft, Inc.

Table 6 – L-PILE^{Plus} Parameters

Description of Stratum	Approx. Depths (feet)	Effective Unit Weight (pci)	Cohesion (psi)	Angle of Internal Friction (degrees)	K (pci)	Strain at 50% Max. Stress
Very Loose to Loose Silty Sand (Lake Deposit)	0 to 6	0.061	0	28	25	N/A
Very Loose to Loose Silty Sand (Lake Deposit)	6 to 15	0.025	0	28	20	N/A
Medium Dense to Dense Silty Sand (Alluvium)	15 to 25	0.036	0	34	60	N/A
Very Dense Silty Sand and Hard Silt (Alluvium and Pre-Olympia)	Below 25	0.039	0	38	125	N/A

pci = pounds per cubic inch
psi = pounds per square inch

Scour

It appears that the loss of foundation support and the lateral wall movement that have been observed are primarily the result of scour from channelized creek flow in the areas of greatest observed distress. Higher flows are observed during heavy rain events and these events likely exacerbate the loss of support beneath the existing structures. With the recommended depths of installation of the H-piles, for the preferred repair option, it is likely that the individual pile elements will help to reduce the scour at these locations, however scour will likely continue within sections between the piles.

CONSTRUCTION VIBRATION CONSIDERATIONS

Per our review of the Basis of Design Memorandum by CTE, dated February 4, 2013, we understand that the H-piles will be installed with pre-drilling to the full depth of the piles. The primary purpose for this proposed installation technique is to reduce potential adverse affects from construction vibrations.

The vibrations, with regard to limits and compliance measurements for major and cosmetic structural damages, are normally evaluated in terms of peak particle velocity. The peak particle velocity is attenuated (decays) due to geometric spreading and material damping, which means the effect of the vibrations becomes less with distance away from the source and depends on the material that the vibrations are traveling through. Very loose soils are present in the upper 15 feet of the soil profile at the site. The effects of the vibrations may be greater in these loose soils, as compared to the deeper more dense soils. The culvert repair work will take place relatively near existing residential structures, therefore measures that can be taken to reduce vibrations, created by the repair work, will be beneficial to the surrounding area. The Basis of Design Memorandum indicates that the H-piles will be installed at approximate 8-feet centers, along the entire length of the east wall. Final plans for pile installation locations and/or distances from these locations to nearby residential structures are not yet available. We recommend that preliminary H-pile installation plans be reviewed and potential construction staging for the installation be discussed with the contractor, once selected.

Generally, we expect that the distances of the existing residential structures from the areas of the proposed mitigation work, will be greater than 30 feet. Damage to the residential structures from the proposed repair activities is considered unlikely. We recommend that the planned drilling of the full pile depth be completed prior to installing the H-piles to reduce construction vibrations. It should be noted that vibration levels that can be perceived by building occupants may be well below levels that will actually induce damage to the structures. We recommend that construction vibrations be monitored throughout the H-pile installation process to document the peak particle velocities. If peak particle velocity values approach levels that could be damaging to nearby structures, the construction work should be halted to determine the best approach for completion and maintaining vibration levels below those which could be damaging.

The following is a list of building categories and Table 7, below the list, presents general construction vibration limits for the defined building categories. The terms cosmetic damage are normally defined as minor structural damages to buildings, such as cracks in wall or ceiling plaster and misalignment of windows. The vibration criteria limits for cosmetic damage are derived from the Federal Transit Administration's Transit Noise and Vibration Impact Assessment Manual, the Acoustical Society of America (*American National Standard: Guide to the Evaluation of Human Exposure to Vibration in Buildings*, ANSI S2.71) and the International Organization for Standardization (*Evaluation of Human Exposure to Whole-Body Vibration in Buildings (1 – 80 Hz)*, ISO-2361-2, 1989). The terms major structural damages are normally defined as damages to buildings rendering them unsafe for human occupancy. The vibration criteria limits for major structural damage are derived from recommendations made by the U.S. Bureau of Mines.

Category I Building consists of reinforced concrete and steel (without plaster) structures, such as industrial type buildings, bridges, retaining walls, masts, unburied pipelines and underground structures, such as tunnels, caverns and galleries (lined and unlined).

Category II Building is a structure with concrete floors and basement walls, above grade concrete walls, brick walls or ashlar masonry walls, ashlar retaining walls, buried pipelines and underground structures, such as tunnels, caverns and galleries with masonry lining.

Category III Building is a structure with concrete basement floors and walls, above grade masonry walls and timber joist floors.

Category IV Building is a structure that is sensitive or vulnerable and worth preserving (often deemed as a historic structure).

Table 7 – General Construction Vibration Limits at Building Locations

Building Category	**Continuous or Steady State Vibration Sources**		**Transient or Impact Vibration Sources**	
	Frequency (Hz)	**Max. Peak Particle Velocity (In/s)**	**Frequency (Hz)**	**Max. Peak Particle Velocity (In/s)**
I	10 – 30	0.5	10 - 60	1.2
	30 – 60	0.5 – 0.7	60 – 90	1.2 – 1.6
II	10 – 30	0.3	10 – 60	0.7
	30 – 60	0.3 – 0.5	60 – 90	0.7 – 1.0
III	10 – 30	0.2	10 – 60	0.5
	30 – 60	0.2 – 0.3	60 – 90	0.5 – 0.7
IV	10 – 30	0.12	10 – 60	0.3
	30 – 60	0.12 – 0.2	60 – 90	0.3 – 0.5

Hz = hertz or cycles per second

In/s = inches per second

DRILLED SHAFT CONSTRUCTION AND H-PILE INSTALLATION

Groundwater was encountered at relatively shallow depths within the borings (as shallow as 6 feet below ground surface). The presence of groundwater in combination with predominantly low cohesion granular soils may be conducive to caving in the drilled shaft holes. The Basis of Design Memorandum indicates that casing should be used to complete the drilled shaft installation. The casing will help to maintain the integrity of the drilled shaft holes during placement of the shaft concrete. The casing should also prevent water from Thornton Creek from entering the work area and should prevent construction material from escaping into Thornton Creek. Shaft excavations should be maintained in a dry condition for placement of H-piles and lean mix and/or concrete. The drilled shaft holes should be de-watered prior to the placement of concrete or the concrete should be placed by a tremie starting from the bottom of the hole and working upward to displace the water with the fresh concrete. The casing should be seated in the bearing stratum and water and loose soil should be removed, prior to beginning the design penetration into the bearing layer. If casing is used and is to be removed, care should be taken to maintain an adequate head of plastic lean mix or concrete within the casing during extraction, so that loose soils are not permitted to cave into open excavation areas. Completion of each H-pile installation and shaft construction should be accomplished within an 8 hour work day and preferably as rapidly as possible to reduce the chance for deterioration of the bearing surfaces. The allowable capacity recommendations presented in this memorandum are based on proper construction techniques.

All of the shaft installations should be inspected by a representative of SPU Geotechnical Engineering to verify that the recommended embedment in the bearing stratum has been achieved. Special inspection should include monitoring pile hole drilling, observation of encountered soil units, and observation of the pile and lean mix/concrete installation.

SITE PREPARATION AND EARTHWORK

The proposed construction of the Thornton Creek Culvert Repair Project structures may require relatively limited site preparation and earthwork activities, however large volumes of soil excavation and/or filling are not currently planned. We understand that undertaking site preparation and earthwork activities that require disturbance within the creek are not desired at this time due to the potential for a prolonged permitting process. The following sections present recommendations for the completion of possible limited earthwork activities for the preferred repair option.

Excavations

We expect that grading work will be limited to areas outside the creek and to those areas where such work is needed only to support the installation of the H-piles and backfilling of the drilled shaft holes. We have not yet reviewed any project grading or excavation plans.

The contractor is responsible for excavation safety, and should follow the requirements of WAC 296-155 and Sections 2-04 and 2-07 of the *City of Seattle Standard Specifications* (City of Seattle, 2011). All excavations, especially those over 4 feet deep, should be appropriately sloped or shored. For planning purposes, the site soils can be classified as Type C, indicating a maximum temporary excavation slope of 1.5 horizontal to 1 vertical (1.5H:1V). The soil type should be verified by a qualified representative of the contractor during construction.

Excavations that are not sloped should be shored. Two general types of shoring systems are possible: safety systems and support systems.

Safety systems, such as trench boxes, protect workers from caving soil, but do not necessarily prevent movement of the adjacent soil. They are appropriate for situations in which settlement-sensitive structures or utilities are not present within the zone of influence of the excavation, as shown in Figure A, below.

Where settlement sensitive structures or utilities are within the zone of influence as shown below, additional analysis should be done to determine both the amount of movement that is expected at the location of the structure or utility, as well as the amount of movement that is acceptable for the individual structure or utility. In the event that the movement anticipated at the location of the structure or utility is unacceptable, the project plans and specifications should require laterally supported trench shoring as required in Section 2-07.3(3), *Support Systems* in the City of Seattle *Standard Specifications*. This type of shoring restricts the movement of the sides of the trenches.

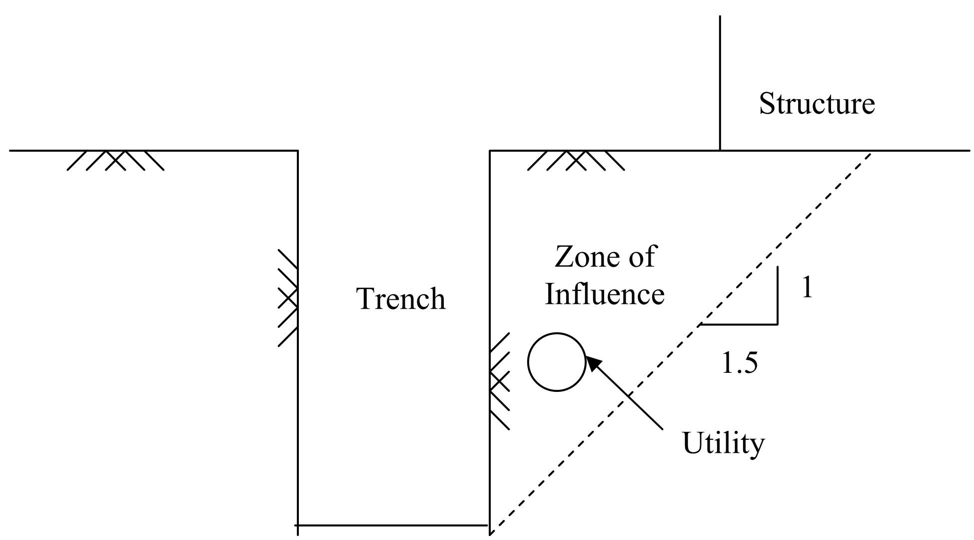


Figure A – Zone of Influence for Excavations

Support systems protect workers and prevent movement of the adjacent soil. They should be used where movement of the ground next to an excavation would damage utilities. Examples of support systems include soldier pile walls, sheet pile walls, and soil nail walls. These systems are often contractor-designed. If a shoring system will be required, the design should account for the depth and extent of the excavation, soil properties, groundwater, and equipment loads.

Groundwater was encountered during the drilling and sampling operations. We expect that groundwater could impact the drilled shaft holes for the H-pile installation and other excavations extending below about 6 feet beneath the ground surface. The contractor should anticipate groundwater seepage and determine systems to adequately control the groundwater. If de-watering systems are needed, this should be the responsibility of the contractor for design and installation. De-watering systems should be designed by a licensed engineer or geologist in the state of Washington.

Structural Fill

We do not anticipate that significant fill sections will be placed. Shallow fill zones could be needed to complete the proposed repair work. In general, the existing on-site near surface soil (within approximately the upper 15 feet) is not suitable for backfill due to the presence of organics, isolated debris (primarily wood debris) and moisture sensitivity. Imported backfill material should meet the requirements of City of Seattle Mineral Aggregate Type 17 (Section 9-03.16, *Mineral Aggregate Chart* in the *Standard Specifications*). Backfill should be placed and compacted as described in Section 2-11 of the *Standard Specifications*. Structural fill for general grading and backfill above and/or behind structures (primarily utility trenches and retaining walls) should be placed and compacted in lifts to a minimum of 95 percent of its maximum dry density as determined by ASTM Test Method D1557. Care should be taken when compacting near retaining walls to avoid damage. This may require smaller hand-held compaction equipment and thinner lifts within about 3 feet of retaining walls. The general backfill, as described in this section, should be placed in a dry excavation.

Compacted, imported materials meeting the specifications of Type 2 and Type 17 may be assumed as having an in-place moist unit weight between 125 and 140 pcf, depending on the compaction effort and the source of the material.

If the repair work can take place during the normal dry season, which typically occurs between June and October, all aspects of the construction will likely be easier to complete. Any excavation and fill placement, if needed, during wet weather may slow the progress of the final repair work.

If earthwork is to be performed or fill is to be placed in wet weather or under wet conditions when control of soil moisture content is not possible, the following recommendations should apply:

- Earthwork should be accomplished in small sections to minimize exposure to wet weather. Excavations or the removal of unsuitable soil should be followed immediately by the placement and compaction of a suitable thickness of clean structural fill, as described below. The size of construction equipment used may have to be limited to prevent soil disturbance;

- Material used as trench backfill should consist of clean, granular soil, of which not more than 5 percent by dry weight passes the U.S. Standard No. 200 sieve, based on wet sieving the fraction passing the ¾ inch sieve. The fines should be non-plastic;
- The ground surface in the construction area should be sloped and sealed with a smooth drum roller to promote rapid runoff of precipitation, to prevent surface water from flowing into excavations, and to prevent ponding of water;
- No soil should be left uncompacted so it can absorb water. Soils that become too wet for compaction should be removed and replaced with clean granular materials; and

Excavation and placement of fill should be observed on a full time basis by a person experienced in wet weather earthwork to verify that all unsuitable materials are removed and suitable compaction and site drainage is achieved.

Permanent Cut and Fill Slopes

Permanent cut and fill slopes at the site should be limited to no steeper than 2H:1V. Permanent cut and/or fill slopes inclined at 3H:1V will be easier to vegetate and maintain. Where fill is to be placed on an existing slope, the fill should be keyed and benched into the existing slope. The fill slope keyway (at the toe) and system of benches above should be excavated into competent, native soil and should be observed by the geotechnical engineer's representative, prior to commencing with fill placement. Fill slopes should be filled and compacted beyond the final configuration and then be trimmed to grade.

CONSTRUCTION DRAINAGE AND EROSION CONSIDERATIONS

Surface runoff and erosion at the site can be controlled during construction by careful grading practices and observance of best management practices (BMPs). Such practices typically include the construction of shallow, upgrade perimeter ditches or low earthen berms, and the use of temporary sumps to collect runoff. Silt fences and other features, if needed, should be installed to reduce the possibility of sediment being eroded from site slopes and entering the stormwater system or surface waters. Erosion during construction can be minimized by judicious use of the described erosion control devices, as well as other measures. If used, these devices should be in place prior to construction and remain in place throughout construction.

Stripping of vegetation from slope surfaces should be limited to the greatest extent possible. Erosion and sedimentation of exposed soils can also be reduced by quickly re-vegetating exposed areas of soil, and by staging construction such that large areas of the project site are not denuded and exposed at the same time. Areas of exposed soil requiring immediate and/or temporary protection against exposure should be covered with mulch, erosion control netting and/or blankets. Soils should not be left exposed to wet weather if these materials are not being worked. Soil stockpiles should be completely covered with plastic sheets for protection during wet weather. Storm drain inlets should be protected from eroded sediments. Construction traffic should not be permitted to track sediment from the site onto adjacent roadways and parking areas.

Permanent erosion control measures should be implemented to reduce the potential for future erosion events. Denuded areas should be mulched and/or planted with approved vegetation.

UNDERGROUND UTILITIES

Trench Subgrades

The native and fill soil encountered throughout the site should generally provide suitable support for underground utilities, provided subgrades remain in an undisturbed condition and any pipes or structures are bedded as described in the following section. A smooth-bladed excavator bucket should be used to excavate to the subgrade elevation and foot traffic on the subgrade minimized to reduce the amount of disturbance to the subgrade. A layer of bedding material or gravel may be used to protect the subgrade once it is exposed.

If unsuitable subgrade conditions are encountered at the time of construction, the subgrade should be evaluated and the course of action determined by the Geotechnical Engineer-of-Record. Typical courses of action may include overexcavation and replacement with structural fill, stabilization with quarry spalls, or use of geosynthetics.

Bedding

Bedding is material placed at the bottom of the trench to provide uniform support along the bottom of a buried utility. Bedding material and placement procedures should meet the appropriate requirements and criteria of the current *City of Seattle Standard Specifications*, depending on the utility in question. In areas where a trench box is used, the bedding material should be placed before the trench box is advanced. Bedding material disturbed by movement of trench boxes should be recompacted prior to final backfilling. Care should be taken not to disturb the utility as the trench box is advanced.

Trench backfill will be placed on top of the bedding. Refer to the backfill recommendations in the *Structural Fill* section of this memorandum.

PAVEMENTS

It is our understanding that for the H-pile option, NE 93rd Street will have to be repaved with asphalt concrete, and be primarily utilized by light automobiles and pick-up trucks, with occasional use by heavy maintenance or delivery vehicles. Pedestrian paths constructed of varying materials may be present around the perimeter of the site.

In our opinion, these pavements may be designed as flexible pavement, assuming an effective subgrade resilient modulus, M_R of 6,000 pounds per square inch (psi) when placed on existing fill. This recommendation is based on our interpretation of surficial ground conditions at the site and the guidelines of the *American Association of State Highway and Transportation Officials*

Guide for the Design of Pavement Structures (AASHTO, 1993). The effective resilient modulus takes into account the regional climate of Seattle.

CLOSURE

This draft geotechnical memorandum is intended to provide information and recommendations to support preliminary engineering activities for this project. The conclusions and interpretations presented in this report should not be construed as a warranty of the subsurface conditions.

We recommend that an experienced geotechnical engineer from SPU Geotechnical Engineering review the Project Manual to verify that our recommendations have been interpreted and implemented as intended. Recommendations for design changes will be provided should conditions revealed during construction differ from those anticipated, and to verify that the geotechnical aspects of construction comply with the Project Manual.

If you have any questions, please do not hesitate to contact us: Hilja Welsh 206-854-8790, or Sean Caraway: 206-615-1547.

REFERENCES

AASHTO (2010) *LRFD Bridge Design Specifications Fifth Edition, Part 1, Sections 1-5.*

ASTM (2012). *American Society of Testing Materials Annual Book of Standards, Vol. 4.08*, West Conshohocken, PA.

City of Seattle, Department of Planning and Development. *2009 Seattle Building Code*. Seattle, WA

Troost, K.G., Booth, D.B., Wisher, A.P., and Shimel, S.A. (2005). *The Geologic Map of Seattle - U. S. Geological Survey Open file report 2005-1252, scale 1:24,000.*

Osborn Consulting, Inc. *NE 93rd Culvert Condition Assessment Technical Memorandum*dated July 25, 2012.

USGS (2009). *National Seismic Hazard Mapping Project, 2002 Data*. October 2009. Denver Colorado.

Legend

- Boring Locations
- Thornton Creek Culvert
- Arterial Streets
- Residential Streets
- Pavement Edge
- LIDAR Topography 2ft

Thornton Creek
Culvert Repair Project
C312066

Site And Exploration Map

Seattle, Washington

FIGURE 1

February 2013

0 12.5 25 50 75
Feet

NAVD88 W E N S

Seattle Public Utilities
Geotechnical Engineering

UNIFIED SOIL CLASSIFICATION SYSTEM - ASTM D2488

MAJOR DIVISION			GROUP SYMBOL	LETTER SYMBOL	GROUP NAME
COARSE GRAINED SOILS CONTAINS LESS THAN 50% FINES	**GRAVEL AND GRAVELLY SOILS** MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVEL WITH ≤ 5% FINES		GW	Well-graded GRAVEL
				GW	Well-graded GRAVEL WITH SAND
				GP	Poorly graded GRAVEL
				GP	Poorly graded GRAVEL WITH SAND
		GRAVEL WITH BETWEEN 5% AND 15% FINES		GW-GM	Well-graded GRAVEL WITH SILT
				GW-GC	Well-graded GRAVEL WITH CLAY
				GP-GM	Poorly graded GRAVEL WITH SILT
				GP-GC	Poorly graded GRAVEL WITH CLAY
		GRAVEL WITH ≥ 15% FINES		GM	SILTY GRAVEL
				GC	CLAYEY GRAVEL
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	SAND WITH ≤ 5% FINES		SW	Well-graded SAND
				SW	Well-graded SAND WITH GRAVEL
				SP	Poorly graded SAND
				SP	Poorly graded SAND WITH GRAVEL
		SAND WITH BETWEEN 5% AND 15% FINES		SW-SM	Well-graded SAND WITH SILT
				SW-SC	Well-graded SAND WITH CLAY
				SP-SM	Poorly graded SAND WITH SILT
				SP-SC	Poorly graded SAND WITH CLAY
		SAND WITH ≥ 15% FINES		SM	SILTY SAND
				SC	CLAYEY SAND
FINE GRAINED SOILS CONTAINS MORE THAN 50% FINES	SILT AND CLAY	LIQUID LIMIT LESS THAN 50		ML	Inorganic SILT, low plasticity
				ML	Inorganic SILT WITH SAND, low plasticity
				CL	Lean inorganic CLAY, low plasticity
				CL	Lean inorganic CLAY WITH SAND, low plasticity
				OL	ORGANIC SILT, low plasticity
		LIQUID LIMIT GREATER THAN 50		MH	Elastic inorganic SILT, moderate to high plasticity
				CH	Fat inorganic CLAY, moderate to high plasticity
				OH	ORGANIC SILT or CLAY, moderate to high plasticity
HIGHLY ORGANIC SOILS				PT	PEAT soils with high organic contents
TOPSOIL				TP	TOPSOIL

NOTES:
1) Sample descriptions are based on visual field and laboratory observations using classification methods of ASTM D2488. Where laboratory data are available, classifications are in accordance with ASTM D2487.
2) Solid lines between soil descriptions indicate change in interpreted geologic unit. Dashed lines indicate stratigraphic change within the unit.
3) Fines are material passing the U.S. Std. #200 Sieve.

BORING LOG KEY :

SAMPLING METHOD

	2-inch OD SPT Split Spoon Sample with 140-lb hammer falling 30 inches (ASTM D1586).
	No Recovery.
	Shelby Tube Sample (ASTM D1587).
	3-inch OD Split Spoon Sample (California Sampler) with 300-lb hammer falling 30-inches.
	Grab Sample.
	Non Standard (As noted on log).
	Core Run.

LABORATORY TEST

AL	Atterberg Limits
FC	Fines Content
GSD	Grain Size Distribution (Sieve and/or Hydrometer)
ENV	Environmental Testing
SG	Specific Gravity
MD	Moisture Density Relationship
C	Consolidation
UCS	Unconfined Compression Strength
Perm	Hydraulic Conductivity Test
PP	Pocket Penetrometer
TV	Torvane
DS	Direct Shear
O	Organic

COMPONENT DEFINITIONS

COMPONENT	SIZE RANGE
Boulders	Larger than 12 in
Cobbles	3 in to 12 in
Gravel	3 in to No. 4 (4.75 mm)
Coarse gravel	3 in to 3/4 in
Fine gravel	3/4 in to No. 4 (4.75 mm)
Sand	No. 4 (4.75 mm) to No. 200 (0.075 mm)
Coarse Sand	No. 4 (4.75 mm) to No. 10 (2.00 mm)
Medium Sand	No. 10 (2.00 mm) to No. 40 (0.425 mm)
Fine Sand	No. 40 (0.425 mm) to No. 200 (0.075 mm)
Silt and Clay	Smaller than No. 200 (0.075 mm)

COMPONENT PROPORTIONS

DESCRIPTIVE TERMS	RANGE OF PROPORTION
Trace	Less than 5%
Few	5 - 15%
Some	15 - 30%

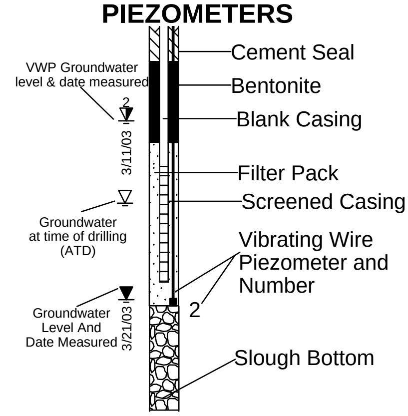
MOISTURE CONTENT

DRY	Absence of moisture, dusty, dry to the touch
MOIST	No visible water, near optimum moisture content.
WET	Visible free water,usually soil is below water table.
SATURATED	Water content prevents soil from retaining structure.

RELATIVE DENSITY OR CONSISTENCY VERSUS SPT N - VALUE

COHESIONLESS SOILS			COHESIVE SOILS		
Density	N (blows/ft)	Approximate Relative Density	Consistency	N (blows/ft)	Approximate Undrained Shear Strength (psf)
Very Loose	0 to 4	0 - 15	Very Soft	0 to 2	< 250
Loose	4 to 10	15 - 35	Soft	2 to 4	250 - 500
Medium Dense	10 to 30	35 - 65	Medium Stiff	4 to 8	500 - 1000
Dense	30 to 50	65 - 85	Stiff	8 to 15	1000 - 2000
Very Dense	over 50	85 - 100	Very Stiff	15 to 30	2000 - 4000
			Hard	over 30	> 4000

PIEZOMETERS



SOIL STRATIFICATION AND STRUCTURE

STRATA	DESCRIPTION
Parting	Less than 1/16 inch thick
Seam	1/16 to 1/2 inch thick
Layer	1/2 to 12 inch thick
Pockets	Inclusions < 1 inch thick
Occasional	< 1 occurrance per foot
Scattered	> 1, < 10 occurrance per foot
Numerous	> 10 occurrances per foot

STRUCTURE	DESCRIPTION
Laminated	Alternating layers of varying material or color with layers less than 1/4 inch thick; note thickness
Stratified	Alternating layers of varying material or color with layers > or = 1/4 inch thick; note thickness
Fissured	Breaks along definite planes of fracture with little resistance to fracturing
Slickensided	Fracture planes appear polished or glossy, sometimes striated
Blocky	Cohesive soil that can be broken down into small angular lumps which resist further breakdown
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay; note thickness
Homogenous	Same color throughout
Dilatent	Water appears quickly on the surface of the specimen during shaking and disappears quickly upon squeezing

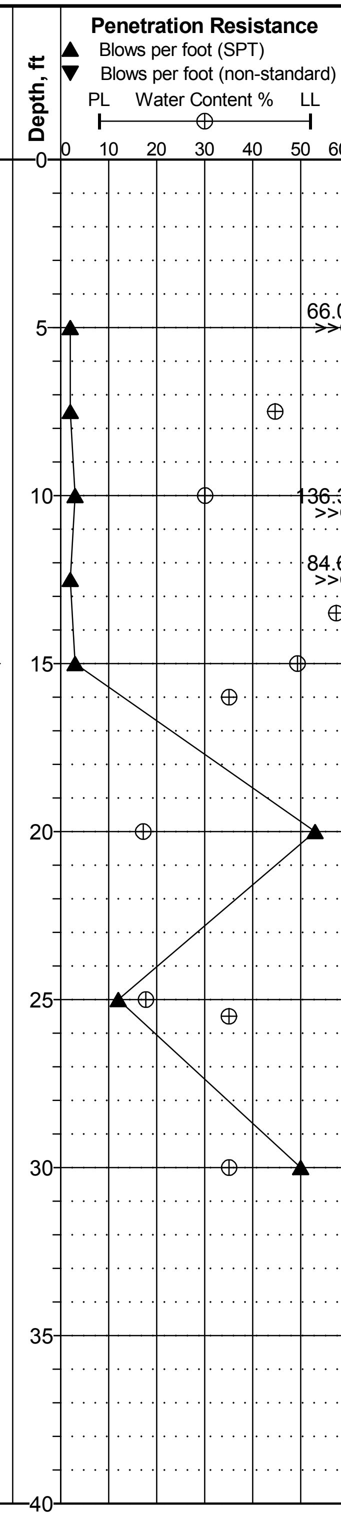
SOIL DESCRIPTION

Surface Elevation: 26 NAVD 88

Soil Description	USCS	Samples	Blows/6"	Recovery, %	Lab tests
Surface is asphalt (2.5 inches).	ASPH				
FILL (Driller notes coarse gravel to approximately 1.5 feet below ground surface (bgs)).	GM				
LAKE DEPOSIT (Ql) Very loose, dark brown, SILTY fine to medium SAND; moist; some organics (wood).	SM	1	2,1,1	100	
Very soft, grayish brown, SILT, trace to few fine sand; moist to wet; trace organics (fine roots and wood), slow dilatency.	ML	2	1,1,1	100	
Very loose, gray, SILTY fine SAND; moist; trace organics (wood).	SM	3a	2,1,2	100	
Soft, dark brown, ORGANIC SILT WITH fine SAND; moist; (3 inch piece of wood).	OL	3b 4a	 1,1,1	100 100	
Very soft to soft, grayish brown, SILT WITH fine to medium SAND, trace fine gravel; moist to wet; trace organics (wood), slow dilatency. Becomes wet and gray.	ML SM	4b 5a 5b	 1,2,1 	100 100 100	
Very loose, grayish brown, SILTY fine to medium SAND, few fine gravel, trace coarse sand; moist to wet; trace organics. (Driller notes gravelly drilling at 17.5 feet.)	SW	6	12,28,25	100	FC
Alluvium (Qal) Very dense, gray, SAND, few fine gravel and trace silt; moist to wet.					
Medium dense, gray, fine to medium SAND WITH SILT, trace fine gravel; moist.	SP-SM	7a	5,5,7	100	
Stiff, gray, SANDY SILT, few clay, trace gravel; moist; interbedded fine to medium sand seams. Becomes hard.	ML	7b 8a	 4,20,30	100 100	
Very dense, gray, SILTY SAND WITH GRAVEL; saturated. (Blow counts may be overstated due to gravels.)	SM	8b		100	

Boring completed at 31.5 feet below ground surface (bgs). Groundwater encountered at approximately 15 feet bgs. Boring backfilled with cuttings and bentonite chips, and the surface restored with asphalt.

Ground Water: ∇ 15 ft



Date Completed: 11/26/2012
Driller: Geologic Drill, Inc.
Equipment: MT52 Track Rig
Drilling Method: 2-1/4 inch ID HSA
Hammer System: Rope & Cathead

Approximate Location: Eastbound lane of NE 93rd St, 4 ft N of the south roadway edge and 50 ft E of the light pole on the southside of NE 93rd St. (N: 257147.395 E: 1285285.792)

Thornton Creek Culvert Repair Project
Seattle, WA

Seattle Public Utilites
Geotechnical Engineering

LOG OF BORING B-101

C312006 FIGURE 3

Logged by: HKW Reviewed by: CAN Sheet 1 of 1

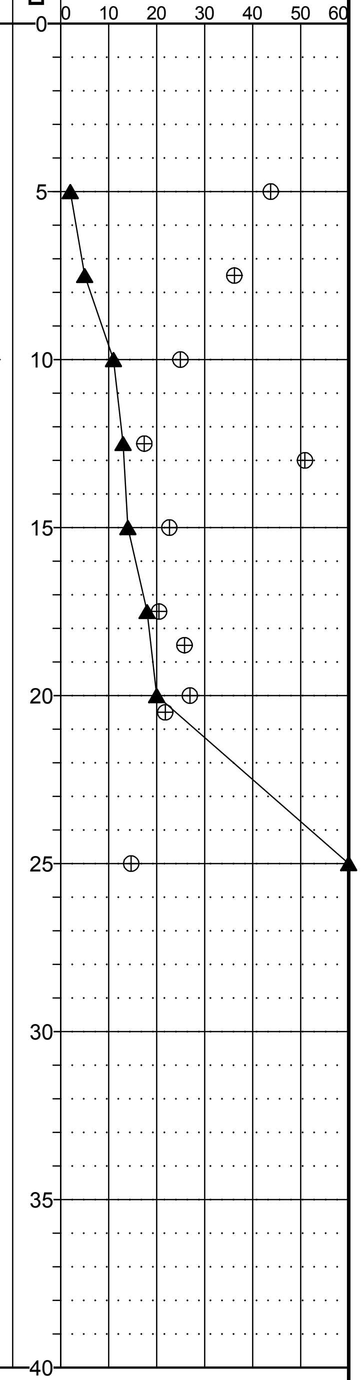
LOG OF BORING 2/1/11 THORNTON CREEK CULVERT REPAIR 2012.GPJ UPDATE_DATA_TEMPLATE_(7-20-09).GDT 2/20/13

SOIL DESCRIPTION

Surface Elevation: 26 NAVD 88

Soil Description	Depth, ft	USCS	Samples	Blows/6"	Recovery, %	Lab tests	Ground Water
Surface is asphalt (2.5 inches).	0	ASPH					
FILL (Driller notes coarse gravel to approximately 4 feet.)		GM					
LAKE DEPOSIT (Ql) Very soft to soft, gray fine SANDY SILT; moist; trace organics (wood).	5	ML	1	1,1,1	100		
Loose, gray, SILTY fine to medium SAND; moist; trace organics (wood).		SM	2	1,2,3	67		
Medium dense, gray, fine to medium SAND, few coarse sand and trace silt; moist to wet; few organics (wood).	10	SP	3 4a	3,4,7 4,4,9	100 100	FC	▽
Stiff, dark brown, ORGANIC SILT WITH SAND, few fine gravel; moist.		OL	4b		100		
Alluvium (Qal) Medium dense, gray, SILTY fine SAND, trace gravel and coarse sand; saturated.	15	SM	5 6a	4,6,8 9,9,9	100 100	FC	
Very stiff, blue gray, SANDY SILT; moist; trace organics, bedded.		ML	6b		100		
Medium dense, gray, SILTY fine to medium SAND, trace fine gravel; saturated; seated pockets blue gray silt.	20	SM	7a 7b	8,11,9	100 100		
Very dense, gray, SAND WITH SILT AND GRAVEL; wet. (Severe heaving and caving starting at 25 feet during advancement to 30 feet, no sample obtained.)	25	SW-SM	8	28,24,36	100		
Boring terminated at 30 feet below ground surface (bgs) due to heaving conditions. Groundwater encountered at approximately 10 feet bgs. Boring backfilled with cuttings and bentonite chips, and the surface restored with asphalt.	30						
	35						
	40						

Penetration Resistance
▲ Blows per foot (SPT)
▼ Blows per foot (non-standard)
PL Water Content % LL



Date Completed: 11/26/2012
Driller: Geologic Drill, Inc.
Equipment: MT52 Track Rig
Drilling Method: 2-1/4 inch ID HSA
Hammer System: Rope & Cathead

Approximate Location: Eastbound lane of NE 93rd St, 4 ft N of the south roadway edge and 61 ft W of the CL of 49th Ave NE. (N: 257151.457 E: 1285322.355)

Thornton Creek Culvert Repair Project
Seattle, WA

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LOG OF BORING B-102

C312006 **FIGURE 4**

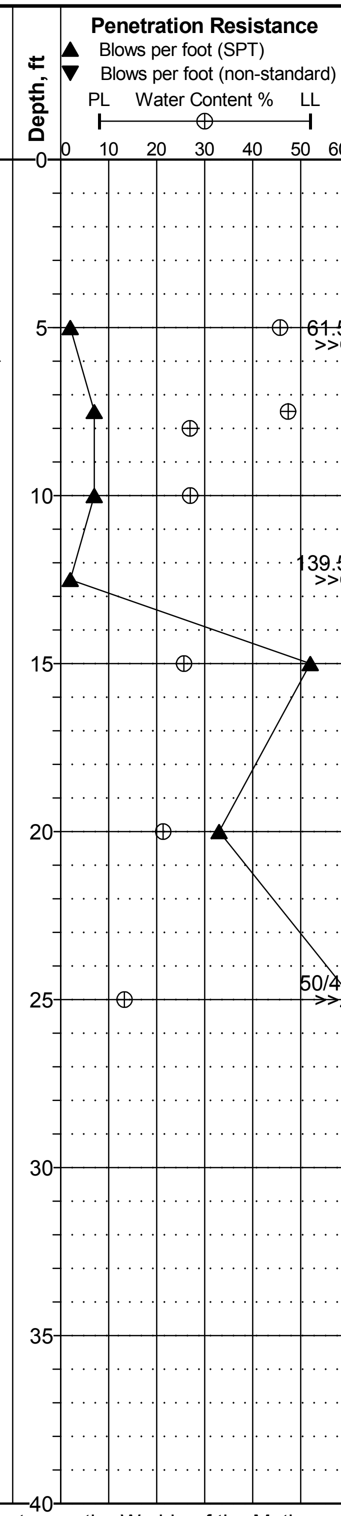
LOG OF BORING 2/1/11 THORNTON CREEK CULVERT REPAIR 2012.GPJ UPDATE_DATA_TEMPLATE_(7-20-09).GDT 2/20/13

SOIL DESCRIPTION

Surface Elevation: 26 NAVD 88

Soil Description	Depth, ft	USCS	Samples	Blows/6"	Recovery, %	Lab tests	Ground Water
Surface is asphalt (6 inches).	0	ASPH					
FILL (Driller notes coarse gravel to approximately 2 feet.)		GM					
LAKE DEPOSIT (Ql) Very loose, dark brown, SILTY SAND, trace fine gravel; wet to saturated; some organics (wood).		SM	1a	1,1,1	100		
Very soft, dark brown, ORGANIC SILT, few to trace fine sand; moist to wet.	5	OL	1b		100		∇ (approx. 6 ft)
Medium stiff, brownish gray, fine SANDY SILT; moist; trace organics (wood).		ML	2a	3,4,3	100		
Loose, gray, fine to medium SAND; moist; trace organics (wood), scattered rust staining.		SP	2b		100		
	10		3	3,3,4	100	FC	
Very soft to soft, black, ORGANIC SILT WITH SAND, trace fine gravel; moist.		OL	4	1,1,1	100		
Alluvium (Qal) Very dense, gray, SILTY fine to medium SAND, trace fine gravel; saturated. (Driller notes gravelly drilling at approximately 14 feet.)	15	SM	5	30,40,12	100	FC	
Becomes dense, moist to wet.	20		6	11,17,16	100		
(Driller notes very difficult drilling at approximately 24 feet.)		ML					
Pre-Olympia Glacial Deposit (Qpog) Hard, gray, fine SANDY SILT, few fine gravel, trace coarse sand; moist.	25		7	28,50/4"	100		

Boring completed at 25.8 feet below ground surface (bgs). Groundwater encountered at approximately 6 feet bgs. Boring backfilled with cuttings and bentonite chips, and the surface restored with asphalt.



Date Completed: 11/26/2012
Driller: Geologic Drill, Inc.
Equipment: MT52 Track Rig
Drilling Method: 2-1/4 inch ID HSA
Hammer System: Rope & Cathead

Approximate Location: B-103 is located on the W side of the Mathews Park Pumping Station driveway off of NE 93rd St, 3.5 ft E of the guardrail and 11.5 ft N of the northside of NE 93rd St. (N: 257174.582 E: 1285307.042)

Thornton Creek Culvert Repair Project
Seattle, WA

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LOG OF BORING B-103

C312006 **FIGURE 5**