GEOTECHNICAL REPORT
DEADHORSE CANYON SEWER
SEATTLE, WASHINGTON

Work Authorization No.: E314059
December 2014

Seattle Public Utilities
Geotechnical Engineering

707 South Plummer Street
Seattle, Washington 98134

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DRAFT GEOTECHNICAL REPORT
DEADHORSE CANYON SEWER
SEATTLE, WASHINGTON

1.0 INTRODUCTION

1.1 GENERAL

Seattle Public Utilities (SPU) Geotechnical Engineering was retained to complete a geotechnical engineering study for the Deadhorse Canyon Sewer in the Lakeridge neighborhood of Seattle, Washington. The project location is shown on Figure 1.

Our understanding of the project is based on our discussions with the project team. We understand that a 10-inch diameter sanitary sewer line is located in the east slope of Deadhorse Canyon in Lakeridge Park. The sewer is located beneath and follows the alignment of the Lakeridge Park recreational trail and is between 3 feet and 10 feet below ground surface (bgs). An approximately 100 foot long section of the slope has subsided up to 1 foot. Although there is no evidence that the current slope movement has affected the sewer line, there is concern that additional movement could cause damage.

We understand that SPU's priority is to provide stability and support to the sewer line, and repair portions of the slope only if needed. To assist in project planning and development we completed a geotechnical investigation that included reviewing historical documents, completing a slope reconnaissance, conducting a subsurface investigation, and performing geotechnical laboratory tests on select soil samples. We performed static and seismic stability analyses to determine the relative stability of the slope and the vertical and lateral extents of instability as well as possible causes of slope instability.

1.2 AUTHORIZATION AND SCOPE OF WORK

Our work was requested and authorized by Betsy Lyons of the SPU Capital Portfolio Management 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 hollow stem auger (hsa) soil borings;

- Performing laboratory testing and engineering analyses to develop geotechnical recommendations presented in this report;

- Performing static and seismic stability analyses; and,

- Preparing this report summarizing our investigations, conclusions and recommendations.

2.0 METHODOLOGY

2.1 Field Investigation

SPU Geotechnical Engineering personnel conducted subsurface explorations on September 25 and 26, 2014 by completing three soil borings (B-101 through B-103). Appendix A describes our investigation in detail. Boring locations are shown on Figure 1. The summary logs of the subsurface explorations are included as Figures A-2 through A-4. Figure A-1 is a key to the terms and symbols used on the logs.

2.2 Geotechnical Laboratory Testing

We conducted laboratory tests to determine the natural moisture content and Atterberg Limits of select samples collected during the drilling. Test results are graphically indicated at the appropriate sample depth on the summary logs (Figures A-2 through A-4). A detailed description of the test methods and results are presented in Appendix B.

3.0 SITE DESCRIPTION

3.1 Site History

We understand that the sewer line was installed and the recreational trail was constructed along the same alignment above the sewer in 1997. Movement of the slope was first noted by the Seattle Parks and Recreation Department (Parks) in 2009. A member of the volunteer group that assists with maintenance of the trail indicates that up to 6 inches of movement occurred after a period of heavy rain during the winter of 2013 and 2014.

3.2 Surface Conditions

We completed a limited slope stability reconnaissance of an approximately 100 foot long section of the west slope of Deadhorse Canyon in July 2014. The purpose of the reconnaissance was to observe current conditions within the project area and select the critical section for subsurface explorations and subsequent slope stability analyses. We examined the slope in the project area looking for signs of instability including seepage, erosion, tension cracks, and over-steepened slopes. During the reconnaissance we did not find any indications that the slide was caused by human activity.

The project area is located on a mildly divergent slope that would tend to disperse surface runoff across the slope rather than funneling it towards the project area. We did not observe evidence of surficial erosion or ponded water along the trail or on the slope.

In general, the slope is heavily vegetated with mature conifers and deciduous trees up to 24 inches in diameter at breast height and low growing brush. The trees are generally straight with only a few showing typical signs of slope instability (e.g., pistol-butted or leaning trunks).

An approximately 100-foot-long tension crack and vertical scarp with a maximum height of 1 foot was observed generally along the 7 foot wide aggregate surfaced recreational trail that traverses the park. The scarp was not visible in the vegetated area downslope of the trail.

Taylor Creek, which flows from south to north through the canyon, is located at the toe of the slope. Within the slide area, the creek is relatively straight and does not show signs of incision or down-cutting and does not appear to have a significant effect on stability of the slope. The ground surface near the toe was generally moist and uneven, with bare soil visible in some locations.

We selected the area with the highest slope and greatest signs of movement as the critical section, and surveyed a cross-section. Figure 1 shows the location of the critical section A-A'. Our subsurface explorations were completed along this critical section.

Taylor Creek is located at elevation 155 feet, the trail and scarp are located at elevation 183 feet, and the top of slope is located at elevation 260 feet. These approximate ground surface elevations were determined based on LIDAR topography of the area, and are referenced to the NAVD88 datum.

The project site is located within two Environmental Critical Areas (ECAs) as mapped by the Seattle Department of Planning and Development (DPD): Steep Slopes > 40% and Potential Slide Area. The toe of the slope and Taylor Creek are located within two additional DPD ECAs, Wetlands and Riparian Corridors.

3.3 General Geology

The general geologic condition of the Puget Sound region is a result of glacial and non-glacial activity that occurred over the course of millions of years. Review of a geologic map (Troost, et al, 2005) indicates that the project alignment is underlain by Pre-Olympia non-glacial deposits. Pre-Olympia non-glacial deposits consist mainly of very dense/hard sand, gravel, silt and clay.

The subsurface explorations completed for this study generally agree with the mapped geology. Descriptions of the soil deposits encountered within our explorations are provided below.

3.4 Site Subsurface Conditions

Based on our exploration program and review of existing subsurface data, we have characterized the subsurface conditions as described in the following text. Descriptions of the soil deposits encountered in our explorations are provided below in the order of stratigraphic sequence, with the youngest unit described first, followed by a description of groundwater conditions.

3.4.1 Silt

Medium stiff to stiff silt with sand (ML) [1] was encountered from the ground surface to a depth of between 4 to 6 feet. Moisture contents within the silt ranged from 11 to 24 percent with an average value of 18 percent. The N-values[2] for this unit ranged from 7 to 11 with an average value of 9. The silt is characterized by low to moderate shear strength, moderate compressibility and low to moderate permeability.

3.4.2 Silty Sand

A three foot thick layer of loose silty sand (SM) was encountered beneath the silt in boring B-103. The moisture content within the silty sand was 30 percent and the N-value was 7. In general, the silty sand is characterized by low shear strength, moderate compressibility and moderate to high permeability.

3.4.3 Silt to Silty Clay

Medium stiff to hard silt (ML) to silty clay (CL) was encountered beneath the silt in borings B-101 and B-102 and beneath the silty sand in B-103. The thickness of the silt to silty clay layer ranged from 10 to 13 feet. Boring B-101 was terminated in this unit 16 feet bgs. Moisture contents within the silt to silty clay ranged from 21 to 33 percent with an average value of 27 percent. The N-values for this unit ranged from 6 to 68 with an average value of 24. In general, the silt to silty clay is characterized by low to moderate shear strength, moderate compressibility and low permeability. The silt to silty clay is the predominant unit in the active slide area.

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

[2] N-values are defined as the number of blows required to drive a 2.0-inch outside diameter sampler one foot, using a 140-pound hammer falling a distance of 30 inches. Refer to ASTM D-1586 (ASTM, 2007)

3.4.4 Very Dense Silty Sand with Gravel (Glacial Till)

Glacial till was encountered beneath the silt to silty clay in borings B-102 and B-103. The glacial till consists of very dense, silty sand (SM) to silty gravel with sand (GM). Borings B-102 and B-103 were terminated in this unit 18 and 25 feet bgs, respectively. Moisture contents within the till were 12. The N-values for this unit ranged from 80 to 100 with an average value of 93. In general, the glacial till is characterized by high shear strength, low compressibility and low permeability.

3.4.5 Groundwater

Groundwater was not encountered during our investigation. 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.

3.5 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, although shaking 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 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 7.0 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 (SFZ) 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 3 kilometers south of the SFZ.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 Slope Stability Analyses

Slope stability analyses were conducted using a simplified computer model of the subsurface section shown on Figure 2. The surface of the profile is based on City of Seattle GIS data and our slope reconnaissance. The soil borings from our current study were used to interpret the subsurface information for the cross-section. The subsurface conditions west of boring B-103 and east of boring B-101 are estimated.

The analyses were conducted using the computer program SLOPE/W Version 7.11 (Geo-Studio, 2007). The SLOPE/W program uses limit equilibrium methods to perform slope stability computations based on the modeled slope conditions, and calculates a factor-of-safety against slope failure, FS, defined as:

$$FS = s/\tau$$

where s is the available shear strength of the soil and τ is the shear stress required for "just-stable" equilibrium. A "just-stable" equilibrium condition would result in a FS of 1.0, while an unstable condition would result in a FS less than 1.0. Slopes are considered to be stable under static and dynamic loading conditions if the minimum calculated FS is equal to or greater than 1.3 and 1.1, respectively.

We performed an initial analysis using intact shear strength properties for all soil layers. The initial properties were based on our experience in similar geologic settings and correlations of field tests and laboratory soil classification test results with engineering properties. However, because the slope exhibits signs of failure, we assigned residual strength properties to the Silt to Silty Clay layers. The residual friction angle of this material was varied until the global slope FS was near unity. The soil engineering properties based on the back analysis are provided in Table 1.

Table 1 – Summary of Soil Engineering Properties Used in Slope Stability Analyses

Soil Unit	Total Unit Weight (pcf)	Effective Strength Parameters	
		Friction Angle (deg.)	Cohesion (psf)
Medium Stiff to Stiff Silt with Sand	120	30	0
Loose Silty Sand	120	30	0
Medium Stiff Silt to Silty Clay (Active Slide Area Material)	115	18	0
Very Stiff Silt to Silty Clay (Active Slide Area Material)	120	18	0
Very Dense Silty Sand with Gravel (Glacial Till)	140	40	0

In order to achieve a FS of unity (i.e. on the verge of failure), the residual friction angle of the active slide area was lowered to 18 degrees. This correlates reasonably well with published relationships (Mesri and Shahien, 2003) based on Plasticity Index (PI). The average PI of the pre-sheared materials is 17 percent. Depending on effective normal stress level, the anticipated residual friction angle ranges from about 18 to 28 degrees.

Although we did not observe seeps in our site reconnaissance or encounter groundwater in our subsurface explorations, the history of the slide suggests that movement coincides with rainfall events. The lack of groundwater during our explorations may be explained by the time of year the explorations were completed and may not reflect typical wet season conditions. As a result, we completed an analysis that included groundwater perched above the silty clay and extending up to the ground surface.

Analyses were completed for both static and dynamic (seismic) load cases. For the seismic case, a pseudostatic analysis was performed, using a seismic coefficient, k_h, equal to 0.21g, in accordance with Section 1803.5.11 of the Seattle Building Code (2012).

For both the static and seismic loading conditions we analyzed the slope and estimated the existing FS. Then we included a vertical reinforcing element in the model that contributes additional shear resistance. The reinforcing element extended from the ground surface down to the glacial till layer. We iteratively increased the shear resistance of the reinforcing element until the FS exceeded 1.3 and 1.1 for the static and seismic loading conditions, respectively. Graphical representations of the results are presented on Figures 3 through 7. The results are summarized below.

4.1.1 Slope Stability Analyses Results

The results of our slope stability analyses are summarized in Table 2.

Table 2 –Existing Factors of Safety and Required Shear Resistance for Stabilization

Loading Condition	Existing FS	Target FS	Required Shear Element Resistance in Kips per Lineal Foot of Slide Width
Static	1.01	1.30	5
Static w/ Ground Water	0.96	1.30	5.5
Seismic	0.60	1.10	17
Seismic w/ Ground Water	0.57	1.10	18

The tabulated FS reflect the most critical slip surfaces (i.e., the minimum FS) found by the computer model for each mode of failure and the loading conditions described above. The critical slip surfaces identified during the analysis are shown graphically on Figures 3 through 7.

In general, if the sewer is located to the east of the failure surface, there is a high probability that the sewer has been or will be deformed and ultimately damaged as a result of previous and future slope movement. In addition, if the sewer is located near (within 5 feet) the failure surface there is a moderate probability that the sewer will be damaged by future slope movement. From our discussions with the Project Team, we understand that the sewer is located under the trail and between 3 feet and 10 feet bgs. The slope stability analyses results indicate that part of this possible sewer location zone is intersected by the estimated failure surface, and more than half of the zone is within 5 feet of the failure surface.

Based on the slope stability analyses results and the approximate sewer location there is a moderate to high probability that the sewer could be affected by future slope movement. However, because the exact location of the sewer is not known, there is some uncertainty as to whether the sewer will be affected by slope movement. As a result, we recommend that the sewer be surveyed to determine if there are sags or other misalignments within the landslide area. If the survey shows that the sewer has deformed within the landslide area it will affirm that the sewer is indeed within or near the failure surface and there is a high probability that the sewer will be affected by further slope movement. However, if the survey shows no signs that the sewer has deformed, there is still a reasonable probability that the sewer could be affected by further slope movement.

4.1.2 Potential Slope Stabilization Measures

In this section we provide several conceptual options for stabilizing the slope and protecting the sewer. Final design recommendations can be provided once the preferred stabilization measure is selected. The slope stabilization measures should be applied over the length of the visible slide area (100 feet) plus a minimum of 10 to 20 feet beyond the visible extents. In general, the each of the following slope stabilization measures is a long-term solution with a design service life of at least 50 years.

- **Groundwater Control:** Construct a trench drain upslope of the failure to intercept groundwater and route it away from the slide area. The trench drain would need to extend to a minimum depth of 10 feet and up to 15 feet bgs. The drain would keep the failed area in a relatively dry condition which based on our analyses we anticipate to have a FS of approximately 1.0. This is still lower than the target slope stability FS under static loading conditions; as a result, it is not recommended as the sole slope stabilization measure but may be used in conjunction with other stabilization measures to provide the most cost-effective solution.

- **Slope Grading:** Re-grade the slope to unload the head of the slide. However, because the existing slope is not overly steep, this option would likely require significant grading to obtain a stable configuration. In addition, the sewer line limits the ability to re-grade the slope.

- **Rock Buttress:** Place a rock berm or buttress at the toe of the landslide. The increased weight of material at the toe acts as a counterforce that resists failure. We estimate that a minimum of 550 cubic yards of rip rap would be needed to provide sufficient counterforce. The rock buttress would have a minimum thickness of 5 feet and extend approximately 20 feet up the slope from the toe at Taylor Creek. Construction of the rock buttress would require vegetation removal and limited excavation within the Taylor Creek riparian corridor. In addition, a temporary access road would need to be constructed to deliver the rip rap and other buttress materials to the site and to the toe of the slope.

- **Structural Reinforcement:** Install structural reinforcing elements along the length of the slope failure in a row approximately mid-way between the head and toe of the slide. The structural elements would be advanced across the failure plane into the stable glacial till material to provide passive shear capacity, disrupt the slide plane, and reinforce the soil mass. We determined that an increase in shear strength of 5.5 and 18 kips per lineal foot was required to provide the target FS for the static and seismic cases, respectively.

  A preliminary design chart (Armour 1997) indicates that 6-inch-diameter micropiles spaced approximately 2-foot on-center should provide the required shear resistance.

Micropiles are small diameter (less than 12 inches) drilled and grouted elements that are typically reinforced using either a drill casing or high strength reinforcing bar.

Micropile drill rigs, which are available with tracks that are less than 5 feet wide, are well suited to limited access installations at sites like the Deadhorse Canyon. We anticipate that the reinforcing elements and grout can be delivered to the site using small equipment traveling along the existing trail system; however, a contractor who specializes in micropile construction should be consulted to verify if this is possible. Some clearing would be required along the slope prior to construction of the micropiles.

4.1.3 Conclusions

The slope is presently in a “just-stable” condition. The site history suggests that the slope has been moving primarily during the rainy winter seasons for at least the past five years. We anticipate that this pattern of slope movement will continue. The magnitude of slope movement is likely tied to the amount of rainfall, i.e., a wetter year will yield greater slope movement. In addition, an extended period of heavy rainfall may lead to catastrophic failure of the slope, which may or may not affect the sewer.

The slope stability results indicate that the sewer may be located within or close enough to the mass of soil that is moving that it may be affected by the movement. We recommend that SPU conduct additional investigations to help determine if the sewer is being affected by slope movement and therefore if slope stabilization is necessary.

5.0 LIMITATIONS AND ADDITIONAL SERVICES

This report was prepared accordance with generally accepted professional principles and practices in the field of geotechnical engineering at the time the report was prepared. The scope of our work did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous or toxic substances in the soil, surface water, or groundwater at this site. However, we did not encounter apparent indications of contamination in our explorations

This geotechnical report 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 SPU Geotechnical Engineering be retained to review the plans and specifications and verify that our recommendations have been interpreted and implemented as intended. Sufficient geotechnical monitoring, testing, and consultation should be provided during construction to confirm that the conditions encountered are consistent with those indicated by explorations and to verify that the geotechnical aspects of construction comply with the contract plans and specifications. Recommendations for design changes will be provided should conditions revealed during construction differ from those anticipated.

We appreciate the opportunity to be of service.

Sincerely,

SPU GEOTECHNICAL ENGINEERING

Megan Higgins, P.E.
Geotechnical Engineer

6.0 REFERENCES

Armour, T.A., 1997. Design Methodology: Micropiles for Slope Stabilization and Earth Retention. *Proceedings of First International Workshop on Micropiles*. pp 330-411.

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Mesri, G. and Shahien, A.M., 2003. Residual Shear Strength Mobilized in First-Time Slope Failures. *Journal of Geotechnical and Geoenvironmental Engineering*. Vol. 129 No. 1, pp 12-31.

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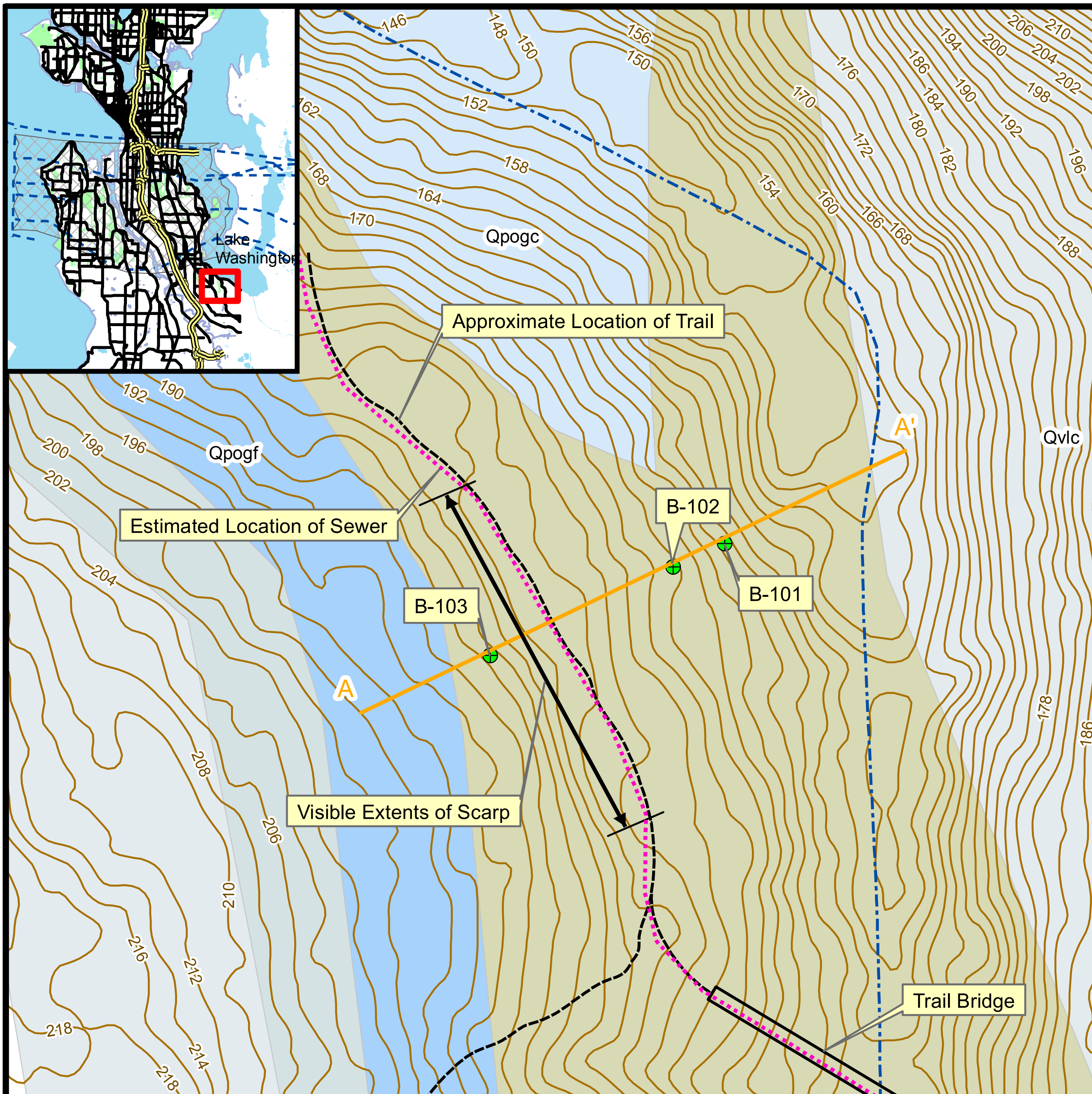

Legend

- SPU Exploration Location
- Stream
- 2 ft LIDAR Contours
- Qvlc - Lawton Clay member of the Vashon Drift
- Qpogc - Pre-Olympia coarse-grained glacial deposits
- Qpogf - Pre-Olympia fine-grained glacial deposits
- Qpogd - Pre-Olympia glacial diamict
- Qpon - Pre-Olympia nonglacial deposits

Deadhorse Canyon Sewer
E314059

Site & Exploration Map

Seattle, Washington

FIGURE 1

DECEMBER 2014

20 10 0 20 40
Feet

NAVD88 N S W E

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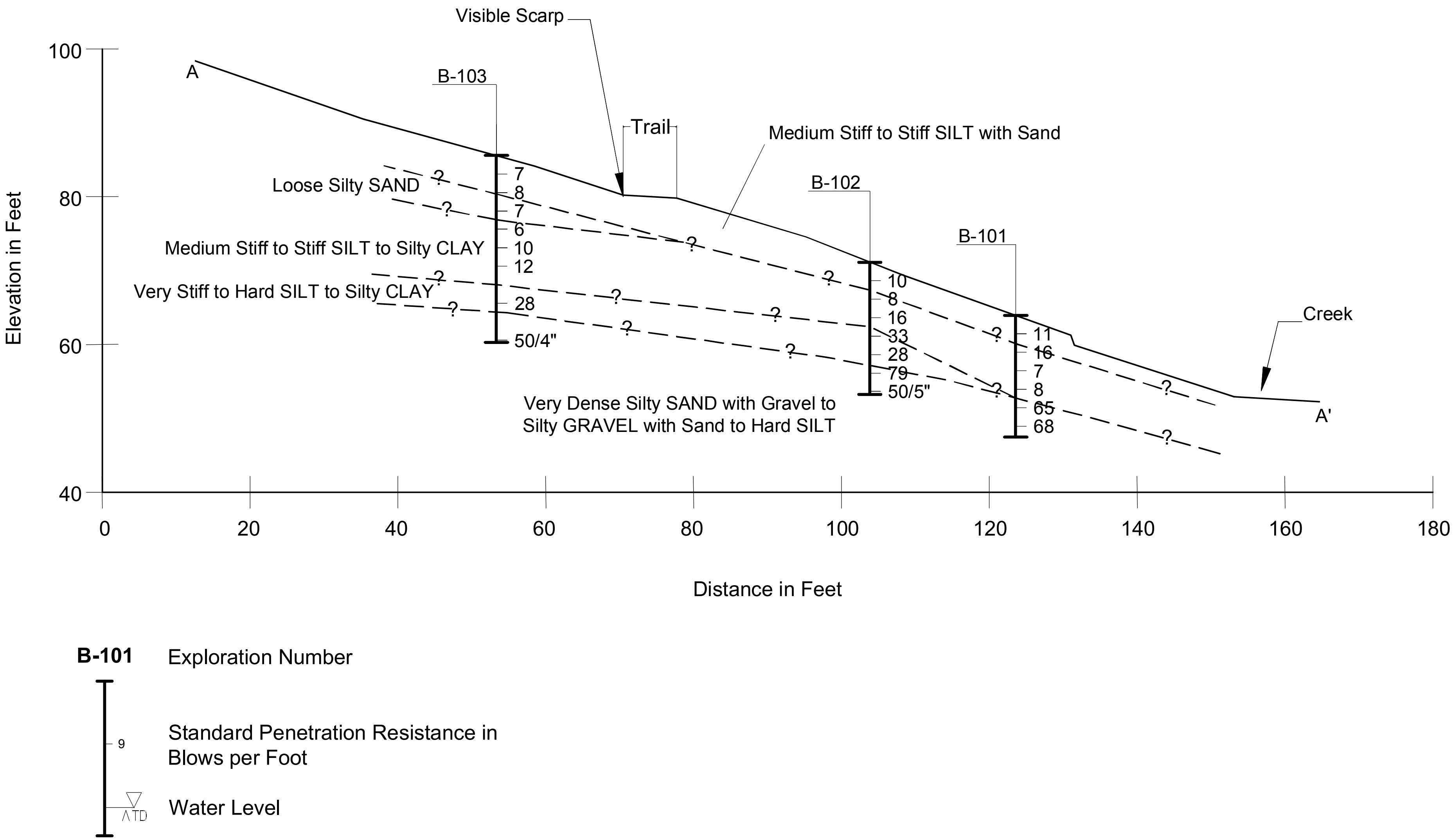

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SEATTLE, WA

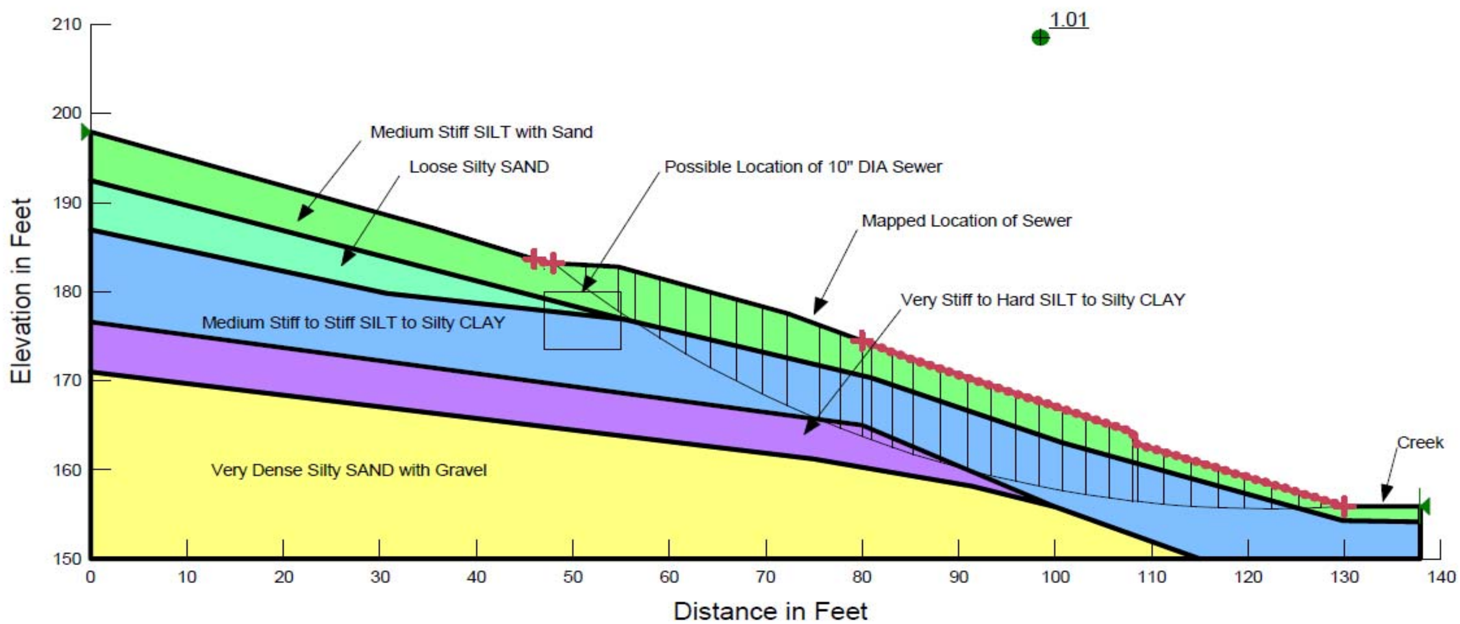
GENERALIZED SUBSURFACE PROFILE A-A'

NOVEMBER 2014

FIGURE 2

Date: 11/3/2014, Time: 6:38:31 PM
Created By: Megan Higgins

Name: Static Drained Residual Shear Strength
Method: Morgenstern-Price
FOS: 1.01



Name: Very Dense Silty SAND with Gravel Cohesion: 0 psf Phi: 40 °
Name: Medium Stiff SILT with Sand Cohesion: 0 psf Phi: 30 °
Name: Loose Silty SAND Cohesion: 0 psf Phi: 30 °
Name: Medium Stiff to Stiff SILT to Silty CLAY Cohesion: 0 psf Phi: 18 °
Name: Very Stiff to Hard SILT to Silty CLAY Cohesion: 0 psf Phi: 18 °

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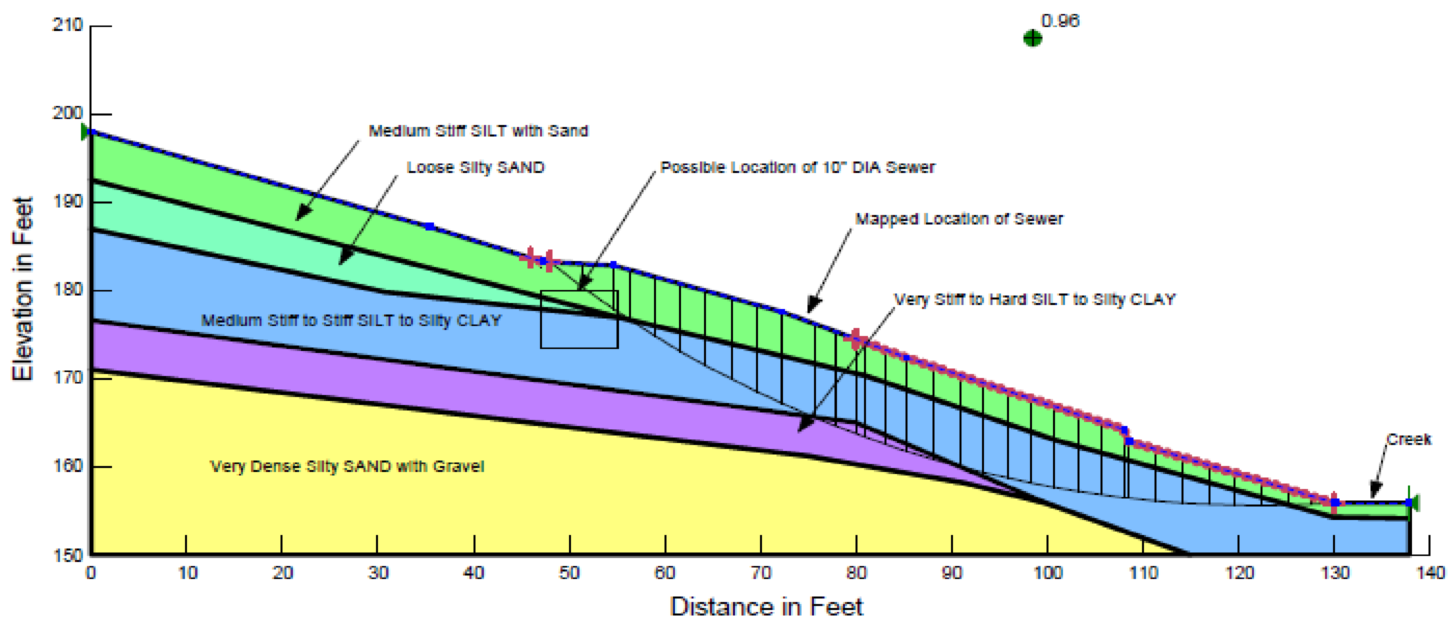
STATIC SLOPE STABILITY RESULTS
EXISTING SLOPE

DECEMBER 2014 **FIGURE 3**

File Name: Deadhorse Canyon Slide.gsz
Date: 11/3/2014, Time: 6:55:47 PM
Created By: Megan Higgins

Name: Static Drained Residual Shear Strength GW
Method: Morgenstern-Price
FOS: 0.96



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Name: Medium Stiff SILT with Sand Cohesion: 0 psf Phi: 30 °
Name: Loose Silty SAND Cohesion: 0 psf Phi: 30 °
Name: Medium Stiff to Stiff SILT to Silty CLAY Cohesion: 0 psf Phi: 18 °
Name: Very Stiff to Hard SILT to Silty CLAY Cohesion: 0 psf Phi: 18 °

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STATIC SLOPE STABILITY RESULTS
EXISTING SLOPE W/ GROUNDWATER

DECEMBER 2014 **FIGURE 4**

12/23/2014

File Name: Deadhorse Canyon Slide.gsz
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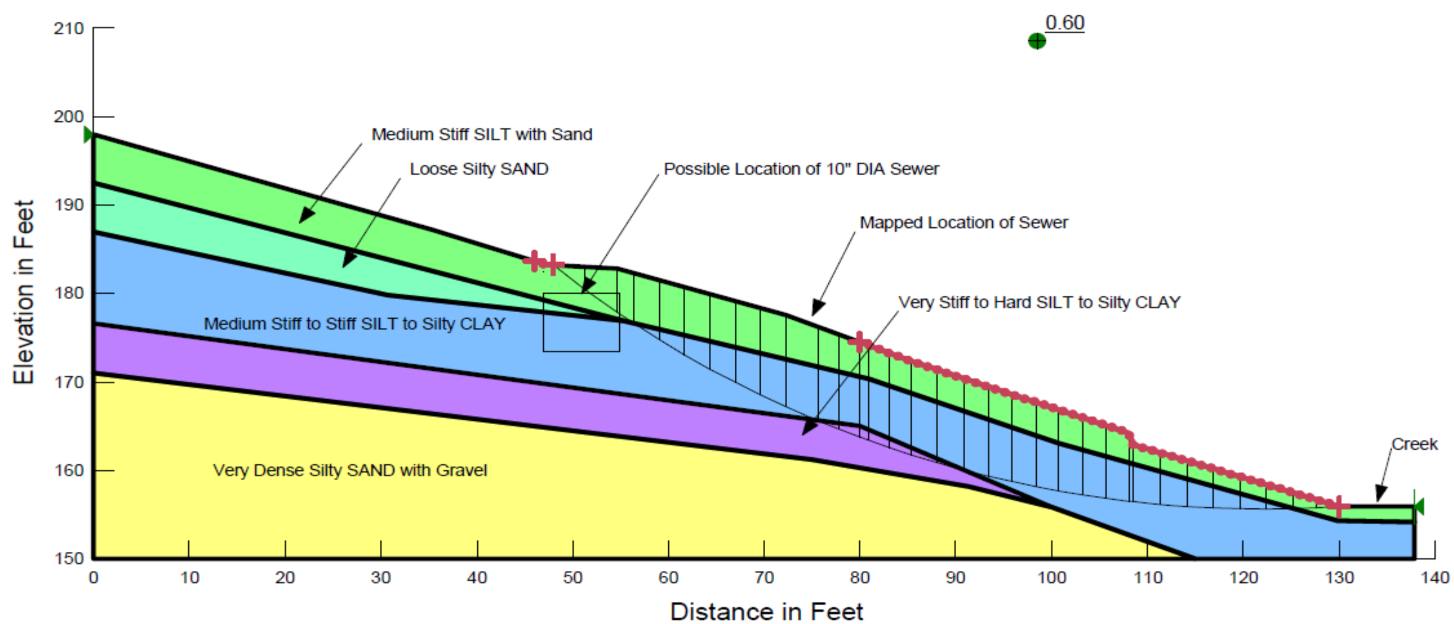
Created By: Megan Higgins

Name: Seismic Drained Residual Shear Strength
Method: Morgenstern-Price
FOS: 0.60



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Name: Loose Silty SAND Cohesion: 0 psf Phi: 30 °
Name: Medium Stiff to Stiff SILT to Silty CLAY Cohesion: 0 psf Phi: 18 °
Name: Very Stiff to Hard SILT to Silty CLAY Cohesion: 0 psf Phi: 18 °

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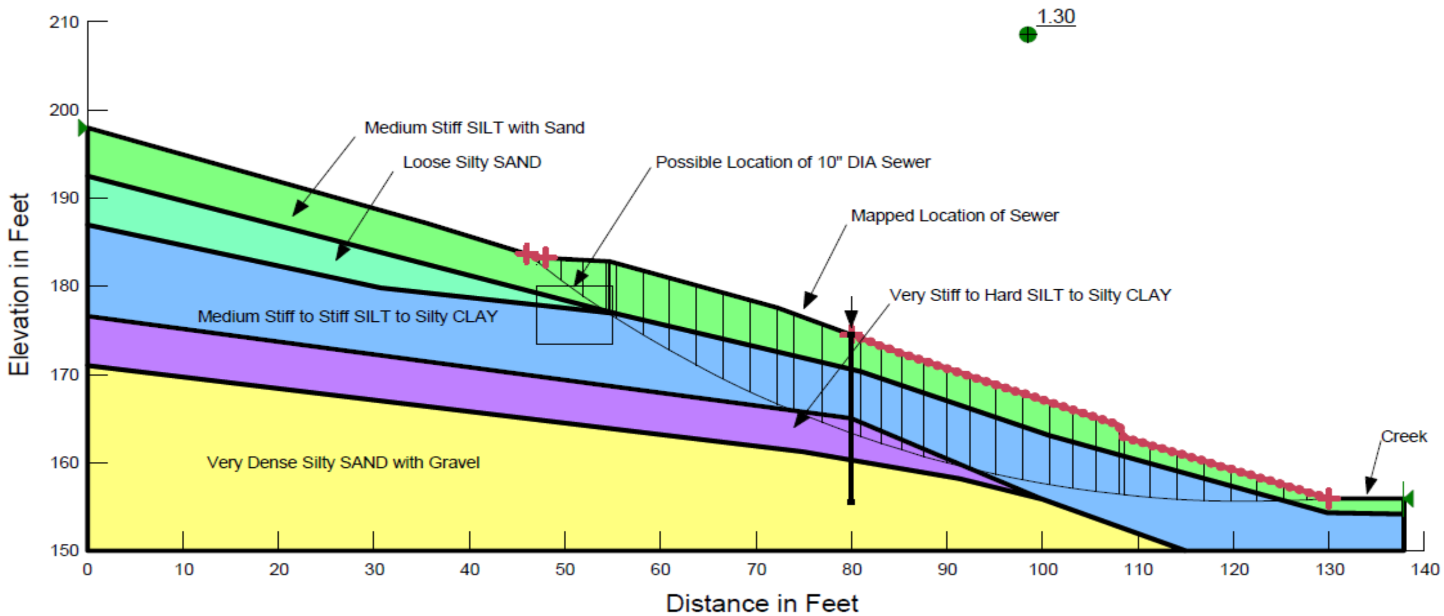
SEISMIC SLOPE STABILITY RESULTS
EXISTING SLOPE

DECEMBER 2014 **FIGURE 5**

File Name: Deadhorse Canyon Slide.gsz
Date: 11/3/2014, Time: 7:11:03 PM
Created By: Megan Higgins

Name: Static Drained Residual Shear Strength Reinforced
Method: Morgenstern-Price
FOS: 1.30



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Name: Loose Silty SAND Cohesion: 0 psf Phi: 30 °
Name: Medium Stiff to Stiff SILT to Silty CLAY Cohesion: 0 psf Phi: 18 °
Name: Very Stiff to Hard SILT to Silty CLAY Cohesion: 0 psf Phi: 18 °

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STATIC SLOPE STABILITY RESULTS
REINFORCED SLOPE

DECEMBER 2014 **FIGURE 6**

File Name: Deadhorse Canyon Slide.gsz
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Created By: Megan Higgins

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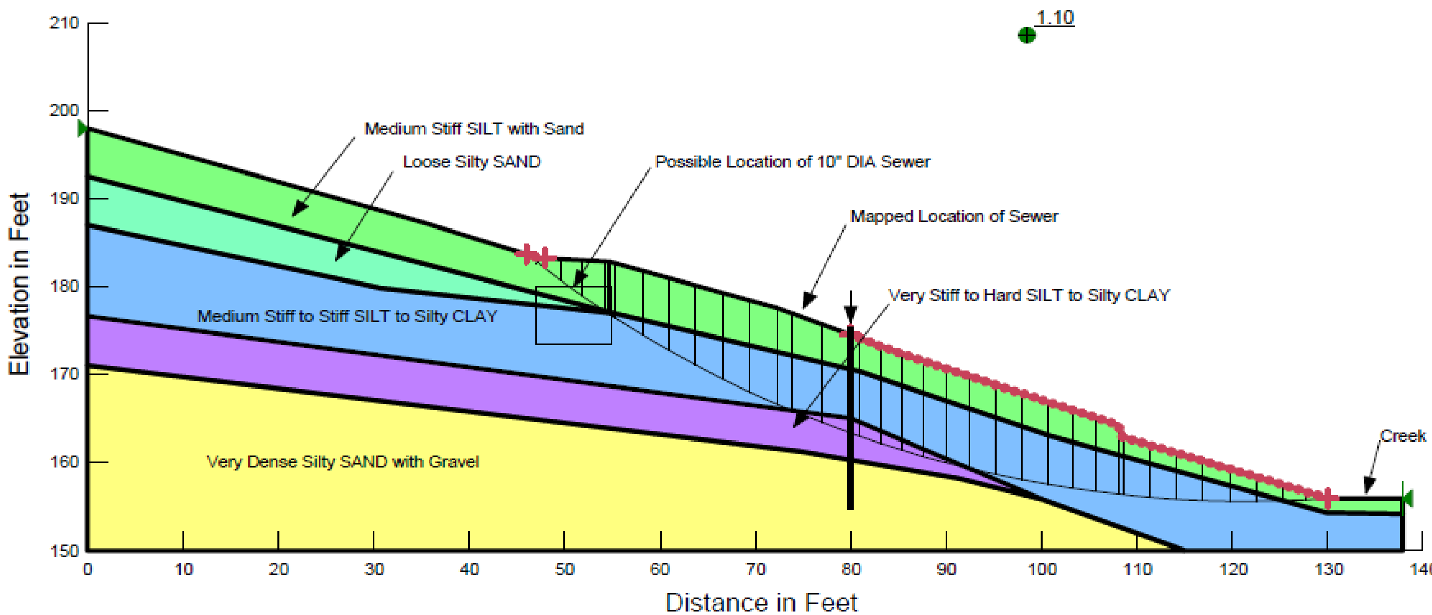
Method: Morgenstern-Price
FOS: 1.10



Name: Very Dense Silty SAND with Gravel Cohesion: 0 psf Phi: 40 °
Name: Medium Stiff SILT with Sand Cohesion: 0 psf Phi: 30 °
Name: Loose Silty SAND Cohesion: 0 psf Phi: 30 °
Name: Medium Stiff to Stiff SILT to Silty CLAY Cohesion: 0 psf Phi: 18 °
Name: Very Stiff to Hard SILT to Silty CLAY Cohesion: 0 psf Phi: 18 °

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DEADHORSE CANYON SEWER
SEATTLE, WA

SEISMIC SLOPE STABILITY RESULTS
REINFORCED SLOPE

DECEMBER 2014 **FIGURE 7**

APPENDIX A

FIELD EXPLORATION PROGRAM

APPENDIX A

FIELD EXPLORATION PROGRAM

GEOTECHNICAL SOIL BORINGS

Subsurface conditions for the current study were explored using hollow stem auger drilling techniques. Three borings, B-101 through B-103 were completed to depths ranging from 16½ to 25½ feet between September 25 and September 26, 2014. The approximate locations of the explorations are illustrated on Figures 1 and 2 in the main body of the text. The explorations were located relative to prominent features in the area. The approximate ground surface elevations at the exploration locations were determined based on LIDAR topography of the area, and are referenced to the NAVD88 datum.

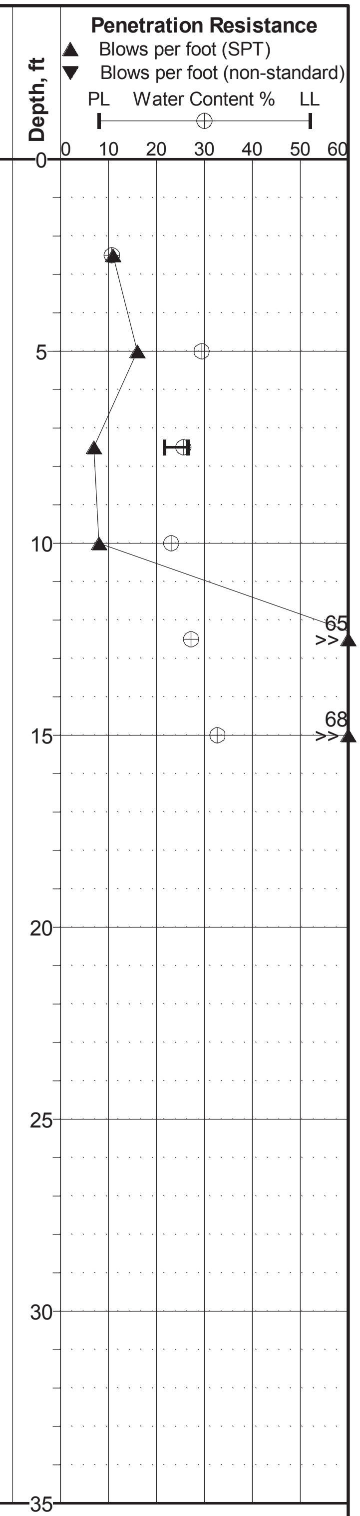
The borings were drilled by CN Drilling, Inc. of Seattle, Washington, under contract to SPU Geotechnical Engineering. An Acker Soil Mechanized hand portable drill rig, equipped with 4¼-inch OD hollow stem augers was used for the soil borings. The results of the explorations are summarized on the individual summary boring logs, which are included here as Figures A-2 through A-4. A key to the symbols and terms used on the summary logs is presented as Figure A-1.

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. Representative portions of all recovered samples were placed in sealed containers and transported to our laboratory for further observation and testing.

SOIL DESCRIPTION

Surface Elevation: 167 NAVD 88

Soil Description	Depth, ft	USCS	Samples	Blows/6"	Recovery, %	Lab tests	Ground Water
Stiff, dark brown, SILT WITH fine to medium SAND, trace coarse sand; moist; some organics (rootlets). Becomes light brown with trace organics and fine gravel.	0	ML	1	4,5,6	60		
Very stiff, gray, SILTY CLAY, trace fine to medium sand; moist. Becomes brown, trace to few coarse sand and fine gravel.	5	CL-ML	2	4,7,9	53		
Becomes medium stiff and gray.			3	2,3,4	47	AL	
Becomes medium stiff to stiff with trace fine sand, scattered rust stained pockets and trace organics.	10		4	1,3,5	100		
Hard, gray, CLAYEY SILT, trace fine sand; moist; occassional fine sand partings. Trace coarse sand and rust staining throughout.		ML	5	9,17,48	80		
Hard, gray, SILT, trace fine sand; moist. Pinkish SILT WITH SAND in tip of sampler.	15	ML	6	17,25,43	100		
Boring completed at 16.5 feet below ground surface. Groundwater not encountered. Boring backfilled with bentonite chips and surface restored with soil plug.	20, 25, 30, 35						



Date Completed: 9/25/2014
Driller: CN Drilling
Equipment: Acker
Drilling Method: 4-1/2 inch OD HSA
Hammer System: Cathead

Approximate Location: 100 feet N of large bridge crossing the bridge, 52 feet E of center of trail and 23 feet W of creek. (N: 187235.8 E: 1291095)

Deadhorse Canyon Sewer
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LOG OF BORING B-101

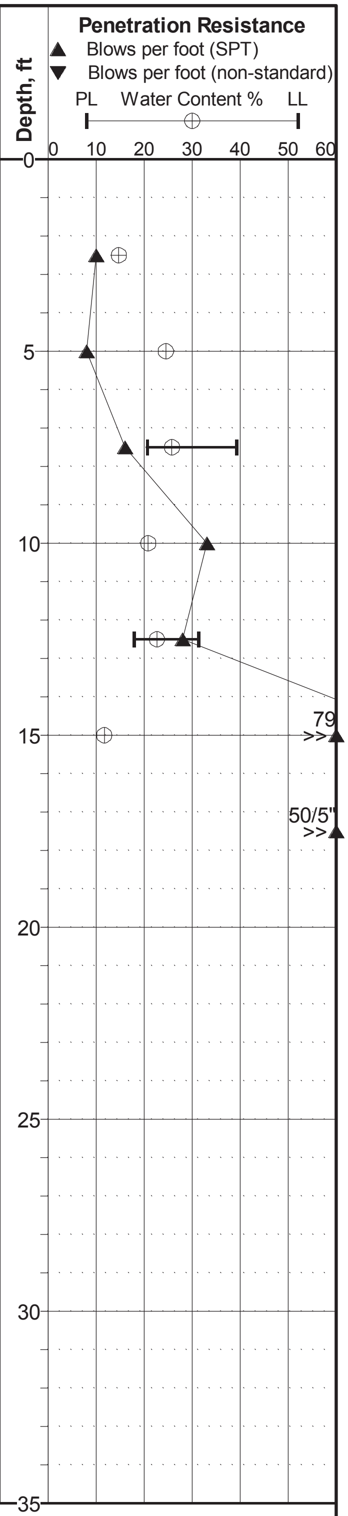
E314059 FIGURE A-2

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

SOIL DESCRIPTION

Surface Elevation: 174 NAVD 88

Soil Description	Depth, ft	USCS	Samples	Blows/6"	Recovery, %	Lab tests	Ground Water
Stiff, dark brown, SILT WITH fine to medium SAND, trace coarse sand; moist; some organics (rootlets). Becomes tan with trace fine gravel.	0	ML	1	5,5,5	100		
Medium stiff to stiff, gray, SILT, trace to few fine sand, trace fine gravel; moist; interbedded layer light gray fine sand.	5	ML	2	3,4,4	60		
Very stiff, tannish gray, SILTY CLAY, trace sand and fine gravel; moist; scattered rust mottling.		CL	3	3,5,11	100	AL	
Becomes hard, light brown with occassional fine sand partings.	10		4	9,12,21	100		
Becomes very stiff, gray with rust pockets and occassional black silt pockets.			5	5,11,17	100	AL	
GLACIAL TILL Very dense, tan, SILTY fine to medium SAND, few coarse sand and gravel; moist; rust mottling throughout.	15	SM	6	19,37,42	100		
			7	50/5"	100		
Boring completed at 17.9 feet below ground surface. Groundwater not encountered. Boring backfilled with bentonite chips and surface restored with soil plug.	20						
	25						
	30						
	35						



Date Completed: 9/25/2014
Driller: CN Drilling
Equipment: Acker
Drilling Method: 4-1/2 inch OD HSA
Hammer System: Cathead

Approximate Location: 100 feet N of large bridge crossing the bridge, 21 feet W of B-101 and 30 feet E of center of trail. (N: 187229.3 E: 1291080.8)

Deadhorse Canyon Sewer
Seattle, Washington

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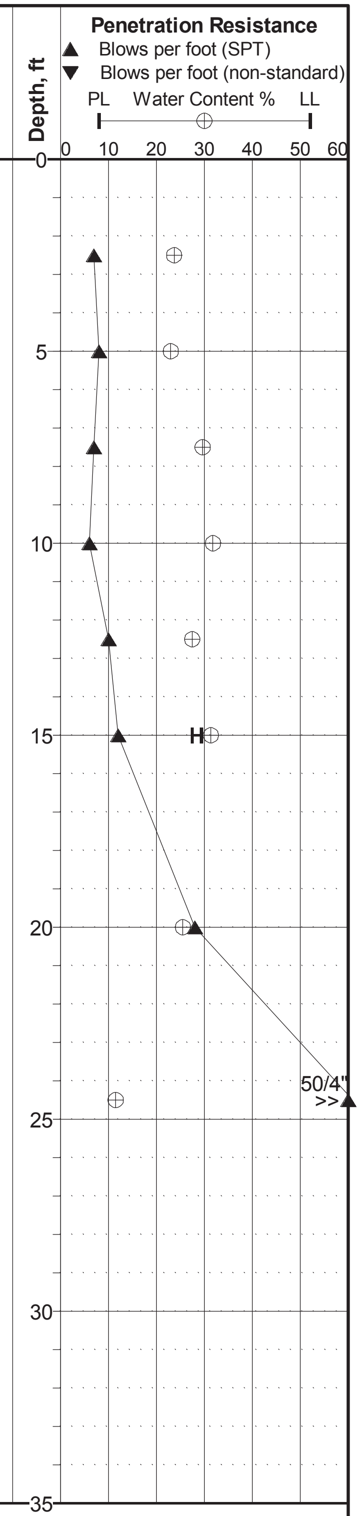
LOG OF BORING B-102

E314059 — FIGURE A-3

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

SOIL DESCRIPTION Surface Elevation: 189 NAVD 88	Depth, ft	Symbol	USCS	Samples	Blows/6"	Recovery, %	Lab tests	Ground Water
Medium stiff, dark brown, SILT, trace coarse sand; moist; some organics (rootlets). Becomes tan with trace fine sand and rust mottling throughout. Trace organics.	0		ML	1	2,3,4	100		
Becomes medium stiff to stiff and gray.	5			2	2,3,5	100		
Loose, tan to light gray, SILTY fine SAND; moist; interbedded silt layers, rust mottling throughout.			SM	3	2,3,4	100		
Medium stiff, gray, SILT, trace to few sand; moist; rust mottling throughout. Silty sand in tip of sampler	10		ML	4	1,2,4	100		
				5	3,3,7	100		
	15			6	5,6,6	100	AL	
	20			7	5,11,17	100		
GLACIAL TILL Very dense, gray, SILTY GRAVEL WITH SAND; dry to moist; oxidation and cemented gravels.			GM	8	29,50/4"	100		
Boring completed at 25.3 feet below ground surface. Groundwater not encountered. Boring backfilled with bentonite chips and surface restored with soil plug.	25							
	30							
	35							



Date Completed: 9/25/2014
Driller: CN Drilling
Equipment: Acker
Drilling Method: 4-1/2 inch OD HSA
Hammer System: Cathead

Approximate Location: 100 feet N of large bridge crossing the creek, 21.5 feet W of center of trail and 54.5 feet from B-102. Approximately 35 feet E of large cedar. (N: 187205.6 E: 1291030.4)

Deadhorse Canyon Sewer
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LOG OF BORING B-103

E314059 **FIGURE A-4**

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

APPENDIX B

LABORATORY TESTING PROGRAM

APPENDIX B

LABORATORY TESTING PROGRAM

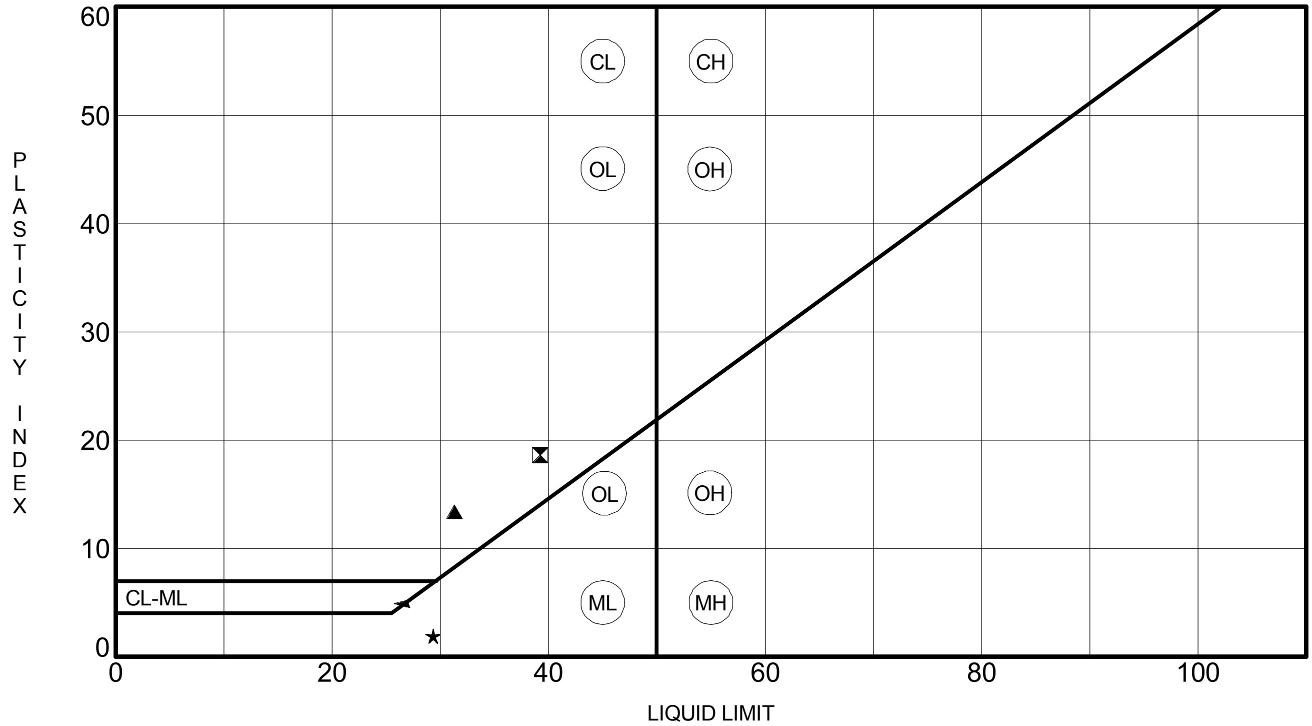
SPU Geotechnical Engineering representatives performed laboratory tests on selected soil samples collected during our field investigation. The laboratory tests were conducted in general accordance with appropriate ASTM test methods. The test procedures and test results are discussed below.

Natural Water Content

Natural water content determinations were made on selected soil samples in general accordance with ASTM D-2216, *Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass*. Test results are graphically indicated at the appropriate sample depth on the summary logs in Appendix A.

Atterberg Limit Determination

Atterberg Limits testing of four samples was performed. The testing was conducted in accordance with ASTM D 4318 *Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils*. The results of the testing are shown on Figure B-1 at the end of this appendix. The soil samples tested are indicated on the summary logs.



	Sample ID	Depth	LL	PL	PI	Fines	Classification
◄	B-101, 3	7.5	26	22	4		Gray, SILTY CLAY, trace to few sand and fine gravel (CL-ML).
⧗	B-102, 3	7.5	39	21	18		Gray, SILTY CLAY, trace to few sand and fine gravel (CL).
▲	B-102, 5	12.5	31	18	13		Gray, SILTY CLAY, trace to few sand and fine gravel (CL).
★	B-103, 6	15.0	29	27	2		Gray, SILT, trace to few sand (ML).

US ATTERBERG LIMITS DEADHORSE CANYON.GPJ DATA TEMPLATE (7-21-11).GDT 11/25/14

Seattle Public Utilites
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WA No.: E314059

ATTERBERG LIMITS RESULTS

Deadhorse Canyon Sewer
Seattle, Washington

FIGURE: B-1