DRAFT GEOTECHNICAL REPORT
HALLER SOUTH GEOTECHNICAL INVESTIGATION
ARLINGTON, WASHINGTON

HWA Project No. 2023-185-21

March 1, 2024

Prepared for:

City of Arlington

GEOSCIENCES INC.

March 1, 2024
HWA Project No. 2023-185-21

City of Arlington Public Works
154 W. Cox Ave
Arlington, Washington 98223

Attention: James Kelly, P.E.

Subject: DRAFT GEOTECHNICAL REPORT
Haller South Geotechnical Investigation
Arlington, Washington

Mr. Kelly:

Attached is our draft geotechnical report for the proposed improvements for the Haller South Geotechnical Investigation project in Arlington, Washington. This draft geotechnical report includes the results of our field explorations and our engineering analyses for design and construction of the proposed well house structures, support structures, and stairs improvements.

We appreciate the opportunity to provide geotechnical engineering services on this project. Please call if you have any questions or comments concerning this report, or if we may be of further service.

Sincerely,

HWA GEOSCIENCES INC.

Donald Huling, P.E.
Geotechnical Engineer, Principal

Sean Schlitt, P.E.
Geotechnical Engineer

Enclosure: Draft Geotechnical Report

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1.0 INTRODUCTION

1.1 GENERAL

This draft report presents the results of a geotechnical engineering study performed by HWA GeoSciences Inc. (HWA) in support of the Haller South Geotechnical Investigation at Haller Park, located at 1100 West Avenue in Arlington, Washington. The purpose of this study was to evaluate soil and groundwater conditions in the vicinity of the project and to provide recommendations regarding design and construction for the proposed improvements.

Our scope of work included drilling two (2) geotechnical borings, geotechnical laboratory testing of representative soil samples, geotechnical engineering analysis, and preparation of this draft geotechnical report.

1.2 PROJECT DESCRIPTION

We understand that the City of Arlington is proposing to construct two well houses and support structures near the existing well house structures at the northern end of Haller Park, and a set of stairs to access the Centennial Trail from the southern end of Haller Park. The project location is indicated on the Site and Vicinity Map, Figure 1.

2.0 FIELD INVESTIGATION

2.1 GEOTECHNICAL BORINGS

HWA completed a field subsurface investigation program that consisted of drilling two (2) exploratory borings to depths of approximately 29.5 to 37 feet below ground surface (bgs) using hollow stem drilling methods. The exploratory borings, designated HWA-1 and HWA-2, were drilled on January 19, 2024, by Holocene Drilling of Puyallup, Washington, under subcontract to HWA, using a Diedrich D50 tracked drill rig. Exploration HWA-1 was drilled in support of the design of the well house and support structures. Exploration HWA-2 was drilled in support of design of the stairs to access the Centennial Trail. The exploration locations completed for this project are shown in our Site and Exploration Plan, Figure 2.

Soil samples were collected within the exploratory borings at 2.5- to 5-foot depth intervals per Standard Penetration Test (SPT) sampling methods, which consisted of using a 2-inch outside diameter, split-spoon sampler driven with a 140-pound auto-hammer. During the test, each sample was obtained by driving the sampler up to 18 inches into the soil with the hammer freefalling 30 inches per stroke. The number of blows required for each 6 inches of penetration was recorded. The standard penetration resistance of the soil was calculated as the number of blows required for the final 12 inches of penetration. If a total of 50 blows was recorded within a single 6-inch interval, the test was terminated, and the blow count was recorded as 50 blows/number of inches of penetration. This resistance provides an indication of the relative density of granular soils and the relative consistency of cohesive soils. At the completion of the boreholes, they were backfilled with bentonite, per Department of Ecology requirements.

The borings were completed under the full-time observation of a geologist from HWA, who collected pertinent information as the exploration was advanced, including soil sample depths, stratigraphy, soil engineering characteristics, and ground water occurrence. Soils were classified in general accordance with the classification system described in Figure A-1, which also provides a key to the exploration log symbols. The boring logs are presented on Figures A-2 and A-3.

The stratigraphic contacts shown on the individual logs represent the approximate boundaries between soil types; actual transitions may be more gradual. The soil and ground water conditions depicted are only for the specific dates and locations reported, and therefore, are not necessarily representative of other locations and times.

2.2 LABORATORY TESTING

Laboratory tests were conducted at HWA's Bothell, Washington, laboratory on selected samples from the explorations to characterize relevant engineering and index parameters. The tests included visual classification, natural moisture content determination, and grain size distribution analyses using the washed sieve method. All tests were conducted in general accordance with appropriate ASTM International (ASTM) standards. Testing is discussed in further detail in Appendix B. The test results are presented in Appendix B, and displayed on the exploration logs in Appendix A, as appropriate.

2.3 EXPLORATIONS BY OTHERS

The following describe previous explorations performed by others at and near the project site that are relevant to our geotechnical investigation. Select logs from these explorations are included in Appendix C.

2.3.1 Pacific Groundwater Group (PGG), 2021

In October 2020, Pacific Groundwater Group conducted an exploration and testing program for the City of Arlington to evaluate development of additional municipal supply wells (PGG, 2021). PGG drilled, installed, and tested five (5) test wells between three different sites in Arlington to evaluate the hydrogeologic conditions. Of these explorations, two (designated TW-1 and TW-2) were drilled near the existing well house location and are relevant to this study. Explorations TW-1 and TW-2 were drilled to depths of 50 feet and 49.5 feet bgs, respectively, with a track-mounted sonic rig and were each completed as test wells. Exploration logs indicate continuous sampling without any tests to indicate density or consistency of material.

2.3.2 Geotest, 2018

In 2018, Geotest conducted an exploration program for geotechnical engineering evaluation of improvements for Haller Park in Arlington, Washington (Geotest, 2018). This program included excavating five (5) test pits and advancing three (3) dynamic cone penetrometer (DCP) explorations. All of these explorations are relevant to this study and are distributed around the park. Most notable is TP-4, located west of the proposed well house structures and support structures, and DCP-2, DCP-3, and TP-2, which are the closest explorations to the proposed stairs. Test pits were advanced in January 2018 to refusal at depts of 10.5 to 11.5 feet bgs using a rubber-tired backhoe. Grab samples were collected from the sidewall of the test pit. DCP explorations were advanced in February 2018 to depths of 10.25 to 15.25 feet bgs using a pointed steel rod advanced with repeated blows of a 35-pound drop hammer. The number of hammer blows required to advance the DCP rod 10 cm (approximately 4 inches) was recorded during continuous advancement.

2.3.3 GeoEngineers, 2008

In 2008, GeoEngineers conducted geotechnical engineering study for improvements at the City of Arlington Wastewater Treatment Plant (WWTP) south of Haller Park (GeoEngineers, 2008). This exploration program included advancing four (4) geotechnical borings. One boring, B-4, is relevant to this project. In July 2007, B-4 was advanced to 19 feet bgs using a truck-mounted drill rig equipped with hollow stem augers. Exploration logs indicate that samples were obtained at 2.5- to 5-foot depth intervals by means of the Dames & Moore (D&M) sampler driven by a 300-pound hammer dropped 30 inches using a rope and cathead.

2.3.4 Robert M. Pride, Inc., 1999

In 1999, Robert M. Pride, Inc. completed a geotechnical design report for a proposed water treatment building at the Arlington WWTP, south of Haller Park (Pride, 1999). The report recommendations were based on an exploration program advanced by Pride in 1996 for additions at the WWTP. The 1996 exploration program included drilling five (5) borings. One boring, B-1, is relevant to this project. In February 1996, B-1 was advanced to refusal at 7.5 feet bgs. The exploration log indicates that samples were obtained at approximately 5-foot depth intervals by means of the SPT).

3.0 SITE CONDITIONS

3.1 SITE TOPOGRAPHY

Haller Park is located at the northern tip of Arlington, east of Highway 9 and south adjacent to where the North Fork and South Fork of the Stillaguamish combine into the Stillaguamish River. The park is generally flat, with a gradual slope from an elevation of about 74 feet at the south end of the park to 69 feet near the existing well houses on the north side of the park. About 40 feet north of the new well is the crest of the riverbank, which drops down approximately 15 to 20 feet to the river.

3.2 GENERAL GEOLOGIC CONDITIONS

The project is located within the Puget Lowland. The Puget Lowland has repeatedly been occupied by a portion of the continental glaciers that developed during the ice ages of the Quaternary period. During at least four periods, portions of the ice sheet advanced south from British Columbia into the lowlands of western Washington. The southern extent of these glacial advances was near Tenino, Washington. Each major advance included numerous local advances and retreats, and each advance and retreat resulted in its own sequence of erosion and deposition of glacial lacustrine, outwash, till, and drift deposits.

Between and following these glacial advances, sediments from the Olympic and Cascade Mountains accumulated in the Puget Lowland. As the most recent glacier retreated, it uncovered a sculpted landscape of elongated, north-south trending hills and valleys between the Cascade and Olympic Mountain ranges. This landscape is composed of a complex sequence of glacial and interglacial deposits. Since the last glaciation, alluvial deposits, such as those from the Stillaguamish River, have been deposited in some of the sculpted landscape. The Stillaguamish watershed, from which the Stillaguamish River draws its headwaters, originates in the foothills of the northern Cascades.

Geological information for this site was obtained from the *Geologic Map of the Arlington West 7.5 Minute Quadrangle, Snohomish County, Washington* (Minard, 1985). According to this geologic map, the near-surface deposits in the vicinity of the project are identified as Marysville Sand recessional outwash in the southern half of the park and younger Quaternary alluvium in the northern half of the park. Also mapped nearby is glacial till in the hillslope of the South Fork Stillaguamish River, just to the east of the park.

- Marysville sand deposits generally consist of well-drained, stratified to massive outwash sand, some fine gravel, and some areas of silt and clay. The Marysville Sand member unit is at least 65 feet thick according to the geologic map.

- The outwash sediments were deposited by meltwater flowing south from the stagnating and receding Vashon Glacier.

- Alluvium is deposited by rivers and streams, and consists of stratified clays, silts, sands and gravels, with grain size dependent on stream energy and distance from active channels. Fine-grained soils are typically indicative of flood deposits. Alluvium deposits range in thickness from approximately 2 to 20 feet according to the geologic map.

- Glacial till was deposited as an unsorted mixture of materials that were overridden by the advancing glacier, resulting in compaction to a very dense state by the pressure of the ice sheet.

3.3 SITE SOIL CONDITIONS

Based on our subsurface explorations, the site is underlain by fill, alluvium, ice-contact drift, and sandstone. Ice-contact drift and sandstone are not indicated on the geologic map at the site; however, the map indicates sedimentary rock along the river upstream of the site, and the ice-contact deposits observed were similar to glacial till units mapped nearby. Subsurface conditions for the two areas are described separately below due to differing geology encountered in explorations. Further descriptions of soils encountered in our explorations are presented below in order of deposition, beginning with the most recently deposited. The exploration logs in Appendix A provide more detail of subsurface conditions observed at specific locations and depths.

3.3.1 Well House Locations

- **Fill** – Fill was encountered in TW-1 and TW-2 (PGG, 2021) and HWA-1 to a depth of approximately 8 to 9 feet bgs. The fill generally consisted of medium dense, olive brown to brown, slightly silty to silty sand with variable amounts of gravel to silty, sandy gravels with cobbles and few scattered pieces of wood. Exploration TP-4 (Geotest, 2018), located west of the proposed well house structures and support structures had approximately 3 feet of fill generally consisting of silty, sandy gravel with scattered organics.

- **Alluvium** – Alluvium was encountered in TW-1 and TW-2 (PGG, 2021) and HWA-1 at about 8 to 9 feet bgs. HWA-1 terminated in alluvium. TW-1 and TW-2 consisted of alluvium until approximately 46 to 49.5 feet bgs, respectively. The alluvium generally consisted of distinct layers, including a very loose to medium dense, silty to clean sand with variable amounts of gravel to about 30 feet bgs. Medium dense gravels were observed beginning between 30 and 35 feet bgs, until about 35 to 43 feet bgs, respectively. Clean sand to silt was observed beneath the gravel layers, with some gravel or cobbles in the coarse-grained material. It should be noted that blow counts in HWA-1 may be overstated on gravels at the 35-foot sample. Some of the alluvium may be associated with recessional outwash. HWA-1 reached auger refusal at about 37 feet bgs, likely on a cobble or a boulder. Alluvium or possible levee deposits were observed in

TP-4 (Geotest, 2018) from about 3 feet to the bottom of the exploration, and generally consisted of very sandy silt.

- **Bedrock/Sandstone** – Explorations TW-1 and TW-2 (PGG, 2021) encountered bedrock at approximately 46 to 49.5 feet bgs, respectively. In TW-1, bedrock was identified as gray, fine sandstone. Exploration TW-2 terminated on bedrock based on drill action; however, no bedrock cuttings were observed. The PGG (2021) report describes historical Well 1R, located as an existing structure on Figure 2, as encountering bedrock at approximately 35 feet; however, no log is provided in the report. Well 1R is located further south of the proposed improvements than TW-1 and TW-2.

3.3.2 Stairs Location

- **Fill** – Fill was encountered to a depth of approximately 12.5 feet bgs in HWA-2 near the base of the stairs to about 9 feet in TP-2 (Geotest, 2018) to the north of the proposed stairs. The fill generally consisted of very loose to very dense sands with variable amounts of silt and gravel, and slightly silty, sandy gravel. Blow counts were likely overstated on gravels. The fill at HWA-2 was substantially more gravelly than at the well house locations, and likely includes material previously used as fill under old Highway 9. Fill along the slope of the proposed stairs generally consisted of 2 to 3 feet of very loose material along the side of the slope, as indicated by T-probe penetrations, which is further discussed in Section 3.5 below.

- **Alluvium** – Exploration TP-2 (Geotest, 2018), located north of the proposed stair location, encountered alluvium at about 9 feet bgs, and terminated in this unit. The alluvium generally consisted of slightly silty, sandy gravel with cobbles. Material was generally very loose to medium dense as indicated by nearby DCP-2 and DCP-3. The bottom two readings of DCP-3 indicate very dense material; however, blow counts may have been overstated on gravels or cobbles indicated by material observed near that depth at TP-2.

- **Ice-Contact Drift** – Ice-Contact Drift was encountered in HWA-2 at approximately 12.5 feet bgs. The unit generally consisted of medium dense to dense or very stiff to hard, gray to olive brown, slightly gravelly, very silty sand to very sandy silt with few instances of red-brown staining. The ice contact drift contained portions of weathered and un-weathered sandstone that were red-brown to gray in color and alternated with the ice-contact material. This indicates an ice contact that involved plucking from the underlying bedrock.

- **Sandstone** – Sandstone was encountered at about 18 feet bgs in HWA-2, and generally consisted of very dense, light gray to gray, silty, fine sand of weak to moderate cementation. The exploration terminated in this unit. Pieces of sandstone were observed in the ice-contact drift unit above.

3.4 GROUNDWATER

Near the proposed locations of the well houses and support structures, groundwater and saturated soil conditions were observed in TW-1 and TW-2 (PGG, 2021) and HWA-1 at depths of approximately 18 to 20.5 feet bgs. These explorations are near the Stillaguamish River and closely reflect the elevation of the river. For design of the well house structures and support structures, the contractor should anticipate groundwater levels that closely reflect the elevation of the river.

Further from the river, including at the proposed stair location, minimal groundwater was encountered in explorations, but generally consisted of minimal perched water at depths of 10 to 12 feet. Perched groundwater was observed in HWA-2 and TP-5 (Geotest, 2018). Further south, groundwater was observed at a similar depth of 11 feet in B-4 (GeoEngineers, 2008); however, the exploration did not indicate if the water was perched. About 1 foot of perched groundwater was indicated by wet cuttings at HWA-2 near the base of the proposed stairs, at about 12 feet bgs at the time of drilling. For design of the stairs, the contractor should anticipate some perched water conditions near the base of the stairs. High groundwater elevations may vary with wet-weather conditions following the completion of a wet season.

3.5 STAIR SLOPE RECONNAISSANCE

Slope conditions near the proposed location of the stairs at the south end of Haller Park were observed on January 10, 2024, by an HWA geologist. The proposed stairs will go from the park up to the Centennial Trail, located east of the park. The slope is defined by an increase in elevation of about 20 feet over about 40 horizontal feet, an approximately 2H:1V (horizontal:vertical) slope. The slope begins north of east Haller Ave, gradually increasing in height towards the Stillaguamish River to the north. A few portions of the slope show shallow raveling up to 1.5 feet where foot traffic has created paths through the surface fill material.

The HWA geologist used a ½-inch diameter steel T-probe to probe the profile of the slope approximately 44 linear feet from the Centennial Trail to the foot path at the base of the slope in the park. We observed relatively shallow penetrations of 2 to 8 inches by the flat at the top of the slope and 2 to 5 inches near the base of the slope from the lower foot path to approximately 10.5 feet east to the break in slope. In the steeper portion of the slope, penetrations of about 22 to 36 inches were observed. This suggests that loose soils are present along the steep slope to a depth of 22 to 36 inches.

No groundwater seepage was observed; however, a small ditch at the base of the slope indicates a potential location for water accumulation during stormwater events. The slope was vegetated with grasses and a few smaller trees and bushes.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 GENERAL

Based on our field exploration program and site reconnaissance, the project site is underlain by fill, saturated alluvium, and glacially consolidated ice-contact stratified drift (ICSD) soils. A sandstone contact was also encountered at the southern end of the park and is anticipated below the ICSD deposits below the depth of our exploration at this location. Additionally, up to 2.5 feet of very loose colluvium was observed at the surface near the proposed location of the stair structure.

The saturated coarse-grained alluvium is expected to liquefy during the design earthquake and could experience settlement and/or lateral displacement as a result. Therefore, foundations for the well houses and support structures should consider the loads and deformations that could occur as a result of liquefaction-induced settlements. Liquefaction-induced settlement is not anticipated to occur at the proposed stair structure.

Slope stability analyses were conducted for the existing slope at the northern end of the project site leading down to the Stillaguamish River. This slope is anticipated to be stable in static loading conditions but may fail in response to a design level seismic event. However, the failure surfaces are only anticipated to manifest up to 25 feet to the south of the crest of the slope. Based on provided plans, we do not anticipate the proposed well house structures and support structures will be located within this 25-foot setback. Therefore, we do not anticipate that these structures will be impacted by slope instability.

Slope stability analyses were also conducted for the proposed stair structure. Similar to the northern slope, this slope is anticipated to be stable in static loading conditions but may fail in response to a design level seismic event. We anticipate that removal of the near-surface colluvium extending to a depth of 2.5 feet and placement of the stair structure foundation at this grade will provide suitable slope stabilization.

The proposed well house improvements will consist of two above ground well house structures and a series of support structures. These structures can be founded on the existing fill soils and can be placed on spread footings or mat foundations. We anticipate these soils will provide an allowable bearing capacity of 1,500 psf.

The proposed stair structure will be installed at the southeastern corner of the site leading up to an existing Centennial Trail. Based on our site reconnaissance, we anticipate this stair structure can be founded on the existing fill soils, assuming it is bedded a minimum of 2.5 feet (30 inches) below the existing slope surface.

It is our understanding that a new blowoff line, water line, and electrical conduit will be installed as part of the proposed improvements. Based on our subsurface explorations, we anticipate that the existing fill soils are suitable to support the new pipes.

4.2 SEISMIC CONSIDERATIONS

4.2.1 Design Parameters

The contribution of potential earthquake-induced ground motion from known sources is included in the probabilistic ground motion maps developed by the U.S. Geological Survey (USGS). Design seismic site characterization and design recommendations based on USGS mapping and analysis are implemented in the 2018 *International Building Code* (*IBC*). As part of this code, the design of structures must consider dynamic forces resulting from seismic events. These forces are dependent upon the magnitude of the earthquake event as well as the properties of the soils that underlie the site.

Earthquake loading for the proposed well house structures and support structures was developed in accordance with Chapter 11: Seismic Design Criteria of the ASCE 7-16: *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE, 2017). Per ASCE 7-16, the selection of seismic design parameters is based on the Maximum Considered Earthquake (MCE), which corresponds to an event with a 2% probability of exceedance in 50 years (a return period of 2,475 years). The mapped seismic design parameters for this site were obtained using the Applied Technology Council Seismic Hazard webtool. This tool incorporates the probabilistic seismic hazard maps developed by the USGS for the ASCE 7-16 and utilizes the site parameters based on the 2014 *Updates to the National Hazard Maps* (Peterson et al., 2014).

As part of the procedure to evaluate seismic forces, the 2018 IBC requires the evaluation of the Seismic Site Class, which categorizes the site based upon the characteristics of the subsurface profile 100 feet below the proposed foundation. The Site Class can then be determined in accordance with Section 20.3 of ASCE 7-16. Based on the obtained SPT blow counts noted in our boring logs and extrapolated to a depth of 100 feet, the site appears to be represented by different site classes depending upon which portion of the site you are on.

In the areas where the new well house structures, support structures, and stairs structure are planned; the site classifies as Site Class “D” (stiff soil). In Section 11.4.8 of ASCE 7-16, Supplement 3, it is stated that a site-specific ground motions hazard analysis is required for Site Class D where values of S_1 are greater than or equal to 0.2. This would necessitate performing a site-specific ground motions hazard analysis on this site, based on the determined value of S_1 for this site. However, the exception in section 11.4.8 of ASCE 7-16 Supplement 3, allows for the determination of seismic design parameters without performing a site-specific ground motions hazard analysis. Provided the value of parameter S_{M1} determined by Equation 11.4-2, of ASCE 7-16, is increased by 50% for all applications of S_{D1} in the standard. Based on this, the values of S_{M1} and S_{D1} have been increased by 50% in Table 1.

The associated probabilistic ground acceleration values and site coefficients for the general site area were obtained from the Applied Technology Council Seismic Hazard Maps. The risk-targeted seismic values and coefficient for this portion of the site are presented in Table 1.

Table 1: Ground Motion Values, Site Class D*

Period (sec)	**Mapped MCE Spectral Response Acceleration (g)**		**Site Coefficient**		**Adjusted MCE Spectral Response Acceleration (g)**		**Design Spectral Response Acceleration (g)**		**Transition Point**	**Period (sec)**
0.0	PGA	0.439	F_{pga}	1.161	PGA_M	0.510	-	-	T_0	0.1904
0.2	S_s	1.033	F_a	1.087	S_{Ms}	1.123	S_{Ds}	0.748	T_S	0.952
1.0	S_1	0.369	F_v	2.897	S_{M1}	1.069	S_{D1}	0.713	T_L	6

Notes: **2% Probability of Exceedance in 50 years for Latitude 48.203117° and Longitude -122.128503°*
PGA = Peak Ground Acceleration
F_{PGA} = Site PGA coefficient
PGA_M = Site Modified PGA
S_S = Short period (0.2 second) Mapped Spectral Acceleration
S_1 = 1.0 second period Mapped Spectral Acceleration
S_{MS} = Spectral Response adjusted for site class effects for short period = $F_a \cdot S_S$
S_{M1} = Spectral Response adjusted for site class effects for 1-second period = $F_v \cdot S_1$
S_{DS} = Design Spectral Response Acceleration for short period = $2/3 \cdot S_{MS}$
S_{D1} = Design Spectral Response Acceleration for 1-second period =$2/3 \cdot S_{M1}$
F_a = Short Period Site Coefficients
F_v = Long Period Site Coefficients
$T_0 = 0.2 \cdot S_{D1}/S_{Ds}$
$T_s = S_{D1}/S_{Ds}$
T_L = Long Period Transition period

Based on Tables 11.6-1 and 11.6-2 (of ASCE 7-16), the Seismic Design Category conditions is "D."

If the Site Class, as determined from the intended building use and the IBC, is interpreted to be C, D, E or F, the code requires an assessment of slope stability, liquefaction potential, and surface rupture due to faulting or lateral spreading. Detailed evaluations of these factors were beyond the scope of this study. However, the following sections present a qualitative *assessment* of these issues considering the site class, the subsurface soil properties, the groundwater elevation, and probabilistic ground motions.

4.2.2 Near Fault Ground Motion Considerations

As required by the ASCE 7-16 near fault effects should be considered for projects that are within 6 miles of a known fault. The project site is located about 9.5 miles south of the Darrington-Devils Mountain Fault. Therefore, near-fault effects were not considered for design analyses of the proposed improvements.

4.2.3 Liquefaction Susceptibility

Liquefaction is a temporary loss of soil shear strength due to earthquake shaking. Loose, saturated, cohesionless soils are the most susceptible to earthquake-induced liquefaction; however, experience and research has shown that certain silts and low-plasticity clays are also susceptible. Primary factors controlling the development of liquefaction include the intensity and duration of strong ground motions, the characteristics of subsurface soils, in-situ stress conditions and the depth to groundwater. Based on the WSDOT *GDM* (WSDOT, 2022), the liquefaction susceptibility of the soils at the project site was determined utilizing the simplified procedure originally developed by Seed and Idriss (1971) and updated by Youd et al. (2001) and Idriss and Boulanger (2004, 2006).

The simplified procedure is a semi-empirical approach which compares the cyclic resistance ratio (CRR) required to initiate liquefaction of the material to the cyclic shear stress ratio (CSR) induced by the design earthquake. The factor of safety relative to liquefaction is the ratio of the CRR to the CSR; where this ratio is computed to be less than 1, the analysis would indicate that liquefaction is likely to occur during the design earthquake. The CRR is primarily dependent on soil density, with the current practice being to base it on the SPT N-value, corrected for energy consideration, fines content, and earthquake magnitude. CSR is generally determined by the formulation developed by Seed and Idriss (1971) and relates equivalent shear stress caused in the soil at any depth to the effective stress at that depth and the peak ground acceleration at the surface.

Our analyses indicate that the saturated, loose to medium dense, alluvial deposits observed near the well house structures and support structures are potentially liquefiable during the design earthquake. Where identified, the effects of liquefaction should be considered and accounted for in the design of proposed improvements. The dense fill and ICSD deposits encountered at the proposed stair location are not identified as liquefiable; therefore, liquefaction is not anticipated to occur at that location.

4.2.4 Liquefaction-Induced Settlement Analysis

For liquefaction susceptible soil deposits, excess pore water pressure builds up during the earthquake excitation, leading to loss of strength, termed as liquefaction. After the shaking stops, excess pore water pressures dissipate toward a zone where water pressure is relatively lower, usually the ground surface. The dissipation is accompanied by a reconsolidation of the loose sand (Ishihara and Yoshimine, 1992 & Tokimatsu and Seed, 1987). The reconsolidation is manifested at the ground surface as vertical settlement, usually termed as liquefaction-induced settlement or seismic settlement.

The potential for liquefaction-induced settlement was evaluated within each boring. The methodologies used to estimate the magnitude of liquefaction-included settlement were developed by Idriss and Boulanger (2008) and are generally based on the relationship between cyclic stress ratio, corrected SPT blow counts, and volumetric strain. Using these methods, liquefaction-induced settlement within the alluvial soils at the proposed well house structures and support structures is estimated to be between 2 and 4 inches. This settlement is expected to be differential in nature and could result in damage to the proposed improvements founded above or within potentially liquefiable materials. We expect that during the design earthquake, these anticipated differential settlements will result in deformation of the proposed improvements.

In general, mitigation of the onset of liquefaction is expensive and not implemented unless the onset of liquefaction is expected to result in a threat to life safety. Although we expect that liquefaction-induced settlement could result in damage to the proposed improvements, we do not expect the damage to result in a threat to life safety. Therefore, we do not recommend implementation of measures to prevent liquefaction from occurring. Rather, we recommend that the anticipated liquefaction-induced settlement be considered when designing proposed improvements and repairs of proposed improvements.

4.2.5 Post Liquefaction Residual Shear Strengths

Upon initiation of liquefaction, the shear strength of the liquefiable soils will be reduced to a residual shear strength while the excess pore pressure within the soil dissipates. For this project, residual shear strengths were estimated using a weighted average of the results of the Tokimatsu and Seed (1987), Seed and Harder (1990), Olson and Stark (2002), Idriss and Boulanger (2006) and Kramer and Wang (2007) relationships. The residual shear strengths were assigned as reduced friction angle materials and are estimated as a function of the equivalent clean sand SPT value, $(N_1)_{60cs}$, the potential for void redistribution, and the initial effective overburden stress. The residual shear strengths were then used to evaluate the potential for liquefaction-induced slope failure through the proposed well house structures and support structures and the adjacent slope to the north.

4.3 SLOPE STABILITY NEAR PROPOSED WELL HOUSE STRUCTURE

The proposed well house structures and support structures are positioned approximately 30 feet from the crest of a steep slope leading down to the Stillaguamish River. Slope stability analysis of the existing slopes was performed along Slope Stability Section A-A' shown on Figure 2. Analyses were performed both for failures of the slope directly adjacent to the river as well as any slope failures that manifest beneath the proposed wellhouse structures. We note that the location of Slope Stability Section A-A' represents the location that HWA anticipates the proposed well house structures and support structures will be constructed. If that location differs from what is shown, HWA should be notified to revise slope stability analyses at this location.

Global slope stability was evaluated using limit equilibrium methods for three scenarios: (1) static loading, (2) pseudo-static earthquake loading, and (3) post liquefaction earthquake loading. In the pseudo-static earthquake loading analysis, a constant horizontal acceleration of 0.255g was applied to the slope.

Limit equilibrium analyses were performed using the computer program SLIDE2 9.023 to calculate the global factor of safety with respect to potential deep-seated failure surfaces. The factor of safety computed is the ratio of the summation of the driving forces to the summation of the resisting forces. Where the factor of safety is less than 1.0, instability is predicted. For global slope stability design, the minimum acceptable factors of safety under static loading conditions are commonly taken as 1.5 for slopes supporting structures or walls. For slopes adjacent to structures or minor walls, where slope instability would have a lesser effect in terms of safety considerations, the factor of safety may be taken as 1.3. The minimum acceptable factor of safety for the pseudo-static case is 1.1. A summary of the results of our analysis is provided in the following sections and the slope stability analysis models are provided in Appendix D.

4.3.1 Static Slope Stability

The stability of Slope Stability Section A-A' under static loading conditions was evaluated with Spencer's method and GME/Morgenstern-Price method using circular failure planes. Our analysis showed that the slope geometry of the existing slope was marginally stable under the static loading condition with a factor of safety of 1.27, as shown on Figure D-1. This indicates that the existing slope directly near the crest is marginally stable under static loading conditions and can be expected to continue to undergo slow and episodic movements. However, slope stability failure surfaces extending to the proposed improvements showed that the slope geometry was stable under the static loading condition with a factor of safety of 2.32, as shown on Figure D-2.

4.3.2 Pseudo-Static Slope Stability

Under pseudo-static earthquake loading, the existing slope near the crest was found to be unstable with a factor of safety of 0.80, as shown on Figure D-3. Therefore, in its current state, the existing slope close to the river is expected to experience slope instability due to seismic shaking under the design seismic event. However, under pseudo-static earthquake loading, a slope failure extending back to the proposed well house structures and support structures was found to be stable with a factor of safety of 1.1, as shown on Figure D-4. Our analysis indicates that the pseudo-static slope failures could manifest at the surface as far as 25 feet from the slope crest, under pseudo-static loading.

Mitigating the expected pseudo-static slope failures would require construction of significant soldier pile and lagging walls along the crest of the adjacent slope. It is our understanding that construction of a soldier pile wall along the crest of the adjacent steep slope is not within the scope of this project. Therefore, we anticipate that the existing slope will be allowed to fail in response to a seismic event.

Based on slope stability analysis, we recommend that all proposed improvements not be placed at least 25 feet of the crest of the existing slope. This will ensure that the proposed improvements will not reduce the stability of the slope and expected slope displacements during a design seismic event will not intersect with the proposed improvement foundations.

4.3.3 Post Liquefaction Slope Stability

Potentially liquefiable soils are present below the project site. Our analyses suggest that the slope in the vicinity of Slope Stability Section A-A' possesses a factor of safety of approximately 1.7 under the post liquefaction condition, as shown on Figure D-5. This indicates that post liquefaction-induced flow sliding is not expected. Depending on the point at which liquefaction occurs during the strong shaking, liquefaction-induced slope displacements could occur in the form of lateral spreading. However, given the anticipated factor of safety, we do not anticipate lateral spreading to be a concern for the proposed improvements.

4.4 SLOPE STABILITY NEAR PROPOSED STAIRS

The proposed stair structure is positioned perpendicular to an existing trail that is placed at the top of a 17-foot-tall embankment. Slope stability analysis of the existing slopes was performed along Slope Stability Section B-B,' shown on Figure 2. We note that the location of Slope Stability Section B-B' represents the location that HWA anticipates the proposed stairs will be constructed. If that location differs from what is shown, HWA should be notified to revise slope stability analyses at this location.

Global slope stability was evaluated using limit equilibrium methods for two scenarios: (1) static loading and (2) pseudo-static earthquake loading. In the pseudo-static earthquake loading analysis, a constant horizontal acceleration of 0.255g was applied to the slope. As discussed in Section 4.2.3, we do not anticipate the occurrence of liquefaction at this location. Therefore, post liquefaction slope stability analyses were not performed.

Limit equilibrium analyses were performed using the computer program SLIDE2 9.023 to calculate the global factor of safety with respect to potential deep-seated failure surfaces. The factor of safety computed is the ratio of the summation of the driving forces to the summation of the resisting forces. Where the factor of safety is less than 1.0, instability is predicted. For global slope stability design, the minimum acceptable factors of safety under static loading conditions are commonly taken as 1.5 for slopes supporting structures or walls. For slopes adjacent to structures or minor walls where slope instability would have a lesser effect in terms of safety considerations, the factor of safety may be taken as 1.3. The minimum acceptable factor of safety for the pseudo-static case is 1.1. A summary of the results of our analysis is provided in the following sections and the slope stability analysis models are provided in Appendix D.

4.4.1 Static Slope Stability

The stability of Slope Stability Section B-B' under static loading conditions was evaluated with Spencer's method and GME/Morgenstern-Price method using circular failure planes. Our analysis showed that the geometry of the existing slope along the trail embankment was marginally stable under the static loading condition with a factor of safety of 1.05, as shown on Figure D-6. This indicates that the existing slope, directly near the crest of the slope, is marginally stable under static loading conditions and can be expected to continue to undergo slow and episodic movements. Upon introduction of the stair structure with embedment depths of 30 inches, our analysis showed that the slope geometry of the existing slope increased under the static loading condition to a factor of safety of 1.70, as shown on Figure D-7, suggesting that the proposed improvements will increase the near surface slope instability.

4.4.2 Pseudo-Static Slope Stability

Under pseudo-static earthquake loading, the existing slope of the trail embankment was found to be unstable with a factor of safety of 0.65, as shown on Figure D-8. Therefore, in its current state, the existing embankment is expected to experience slope instability due to seismic shaking under the design seismic event. Upon introduction of the stair structure with embedment depths of 30 inches, our analysis showed that the slope geometry of the existing slope increased under the pseudo-static loading condition to a factor of safety of 1.1, as shown on Figure D-9, suggesting that the proposed improvements will increase the near surface slope instability.

4.5 WELL HOUSE FOUNDATION DESIGN

4.5.1 Spread Footing Foundation Recommendations

We understand that the proposed well house structures and support structures will be founded on spread footings. We recommend an allowable bearing pressure of 1,200 pounds per square foot (psf) for foundations that are bearing directly on compacted native soils within the upper 7 feet. If excavations extend deeper than 7 feet bgs, proposed foundations will bear on loose alluvial deposits that will not support the proposed spread footings. HWA should be notified if foundation grades are anticipated to extend greater than 7 feet bgs so we can provide suitable foundation recommendations. For short-term wind and seismic loading conditions, this allowable bearing pressure may be increased by 33 percent to resist increased pressures on the edge of the foundation due to over-turning moments.

4.5.2 Foundation Resistance to Lateral Loading

Earthquake and unbalanced earth loads will likely subject the proposed structure foundations to lateral forces. Horizontal forces can be resisted partially or completely by frictional forces developed between the base of the foundation and the underlying crushed rock. The total shearing resistance between the foundation footprint and the soil should be taken as the normal force (i.e., the sum of all vertical forces [dead load plus real live load]) times the coefficient of friction between the soil and the base of the footing. We recommend assuming an ultimate coefficient of friction value of 0.5 may be used for lateral movement. A factor of safety of 1.5 should be used when computing the frictional resistance.

4.5.3 Mat Foundation

If required, the proposed well house structures and support structures can be founded on a mat foundation. The mat foundation should bear at least 18 inches below the lowest adjacent finished grade for frost protection considerations. We recommend an allowable bearing pressure of 500 psf for foundations that are bearing directly on compacted native soils. For short-term wind and seismic loading conditions, this allowable bearing pressure may be increased by 33 percent to resist increased pressures on the edge of the foundation due to over-turning moments. For slabs designed as a beam on an elastic foundation, a static modulus of subgrade reaction of 150 pounds per cubic inch (pci) may be used for subgrade soils prepared as recommended.

4.5.4 Foundation Construction Considerations

We recommend that the foundation subgrade for both spread or mat foundations be prepared by over-excavating a minimum of 1 foot of native soil from below the proposed foundation grade. This over-excavation should extend across the entire structure footprint and at least 1-foot outside the perimeter of the proposed foundation. Once these soils are over-excavated, the exposed subgrade soils should be compacted to a dense and unyielding condition. After the subgrade soils are firmly compacted, the over-excavated area should be backfilled with compacted structural fill. We recommend that all slabs be constructed on a 1-foot-thick (minimum) pad of compacted Crushed Surfacing Base Course (CSBC), meeting the requirements of Section 9-03.9(3) of the WSDOT *Standard Specifications* (WSDOT, 2024). CSBC placed below slabs should be compacted to a dense and unyielding condition and at least 95% of its maximum density, as determined using test method ASTM D 1557 (Modified Proctor).

4.5.5 Foundation Design Settlement

Elastic settlement is the vertical component of soil compression under static loading. These settlements occur relatively rapidly upon the application of load and are dependent on the elastic properties of the soils and/or rock underlying the proposed foundation. Loose to medium dense soils generally undergo larger elastic settlements than harder materials such as rock. Using the computer program Settle3 (Rocscience, 2023), HWA modeled the anticipated elastic settlement under the load of the proposed well house structures and support structures.

The site is underlain by medium dense sand and gravel fill soils underlain by very loose to medium dense sandy alluvial deposits. Based on the foundation layout provided by Brown and Caldwell, we estimate that foundations designed and constructed in accordance with the recommendations herein will experience post-construction immediate settlements (non-earthquake) of less than 1 inch. Differential settlements after site improvements for static settlements are anticipated to be less than ½ inch along a 40-foot span. Differential settlements following liquefaction are anticipated to be of the order of 2 to 4 inches over that span. Total and differential settlements of these magnitudes are usually considered tolerable for the anticipated construction; however, the tolerance of the proposed structure to the predicted total and differential settlements should be confirmed by the structural engineer.

4.5.6 Drainage

Subsurface drains consisting of 4-inch-minimum-diameter perforated pipe, should be installed around the perimeter of building foundations and should be founded at or below the level of the bottom of the footing. Washed drain rock should be placed such that a minimum of 6 inches of drain rock envelops the drainpipe around its perimeter. Roof drain downspouts should not be hydraulically connected to the footing drains, to prevent potential clogging and flooding of the drain system. All drains should be sloped to drain and tightlined to appropriate storm drainage outlets.

4.6 PROPOSED STAIR STRUCTURE

We understand that the proposed stair structure will be founded directly on native soils. We anticipate that the stair structure will be constructed by benching of the trail embankment to limit the amount of excavation required. We recommend that benches be limited to a maximum of 4 feet in height. Based on our slope stability analysis presented in Section 4.4, we anticipate that the stair structure will need to be founded at least 30 inches below the lowest finished grade for each bench to ensure suitable slope stability.

4.6.1 Stair Structure Design Settlement

Elastic settlement is the vertical component of soil compression under static loading. These settlements occur relatively rapidly upon the application of load and are dependent on the elastic properties of the soils and/or rock underlying the proposed foundation. Loose to medium-dense soils generally undergo larger elastic settlements than harder materials such as rock. Using the computer program Settle3 (Rocscience, 2023), HWA modeled the anticipated elastic settlement under the load of the proposed stair structure.

The site is underlain by medium dense sand and gravel fill soils underlain by very loose to medium dense sandy alluvial deposits. Based on the foundation layout provided by Brown and Caldwell, we estimate that foundations designed and constructed in accordance with the recommendations herein will experience post-construction immediate settlements (non-earthquake) of less than 0.25 inch. As discussed in Section 4.2.3, we do not anticipate the onset of liquefaction-induced settlement at this location. Differential settlements after site improvements for static settlements are anticipated to be less than 0.25 inch along a 40-foot span. Total and differential settlements of these magnitudes are usually considered tolerable for the anticipated construction; however, the tolerance of the proposed structure to the predicted total and differential settlements should be confirmed by the structural engineer.

4.6.2 Subgrade Preparation

Based on the findings of our slope reconnaissance, we encountered loose, near surface colluvial deposits extending to a depth of approximately 2.5 feet bgs. Therefore, we recommend that the stair structure subgrade be prepared by excavating a minimum of 2.5 feet of native soil to found the proposed stair structure. This excavation should extend across the entire structure footprint and at least 1 foot outside the perimeter of the proposed foundation. Once these soils are excavated, the exposed subgrade soils should be compacted to a dense and unyielding condition.

After the subgrade soils are firmly compacted, the excavated area should be backfilled with compacted structural fill. We recommend that all foundations be constructed on a 0.5-foot-thick (minimum) pad of compacted Crushed Surfacing Base Course (CSBC), meeting the requirements of Section 9-03.9(3) of the WSDOT *Standard Specifications* (WSDOT, 2024). CSBC placed below slabs should be compacted to a dense and unyielding condition and at least 95% of its maximum density, as determined using test method ASTM D 1557 (Modified Proctor).

4.7 UTILITIES

It is our understanding that a new 6-inch utility line for the well house blow-off, a new 6-inch water line, and electrical conduit will be installed leading to the new well house structures. These utilities are anticipated to be installed with a depth of cover of approximately 3 feet. Therefore, we anticipate excavations up to 5 feet will be required for placement of the proposed utilities. Design and construction considerations for proposed utilities are provided below.

4.7.1 Temporary Shoring and Sloped Excavations

Trench excavations for the proposed utilities can be accomplished with conventional excavation equipment such as backhoes and trackhoes. Trench excavation should be made with a smooth edge (toothless) bucket or a bucket with a plate welded over the teeth to minimize disturbance to the pipeline subgrade. It should be noted that loose alluvial deposits are anticipated to be encountered at a depth of approximately 7 to 10 feet bgs; if trench excavations are anticipated to be deeper than 5 feet, temporary shoring will likely be required. HWA should be notified to provide recommendations if deeper excavations are anticipated.

Given the adjacency to existing structures, sloped trench excavations may not be an acceptable means of temporary shoring for the installation of utilities. Trench support can be achieved using a trench box, augmented as necessary with steel sheets and struts. Caving of the sidewalls may occur, and temporary shoring may be necessary to limit the extent of the excavations. The design of the temporary shoring systems should be the responsibility of the contractor.

4.7.2 Utility Line Trenching Obstructions

While not encountered during our explorations, excavations may encounter scattered cobbles and boulders. The project specifications and cost estimate should account for the potential for encountering and removing obstructions during utility installation.

4.7.3 Utility Line Trench Caving

The near surface coarse-grained fill soils are expected to be medium dense in nature and may slough during excavation if not properly shored. Trench sidewall caving can result in undermining adjacent pavements, utilities, and other structures. Some level of sidewall caving may occur during utility trenching, and mitigation measures should be implemented to avoid damage to adjacent pavements, utilities, and structures. It will be important that the contractor's temporary shoring provides sufficient lateral support for the native soils to avoid settlement and ground disturbance extending away from the trench excavations.

We recommend that the project specifications clearly state that reconstruction or compaction of soils adjacent to utility trenches that are disturbed by utility construction is the responsibility of the contractor and is incidental to the utility installation bid items.

The contractor should be responsible for control of ground and surface water and should employ sloping, slope protection, ditching, sumps, dewatering, and other measures, as necessary, to prevent sloughing of soils and heave of the bottom of the excavation.

4.7.4 Utility Pipe Bedding Recommendations

The bottom of utility trenches should be free of debris and standing water. If subgrade soils are disturbed, the disturbed materials should be removed down to undisturbed soils and replaced with properly placed and compacted bedding material. To minimize trench subgrade disturbance during excavation, the excavator should use a smooth-edged bucket rather than a toothed bucket. If poor quality soils are encountered, the geotechnical engineer should be notified to evaluate the conditions and provide recommendations.

Once the utility trench subgrade soils are properly prepared, pipe zone bedding should be placed. Pipe bedding should consist of Gravel Backfill for Pipe Zone Bedding, meeting the requirements of Section 9-03.12(3) of the WSDOT *Standard Specifications* (WSDOT, 2024).

Pipe bedding should provide a firm uniform cradle for support of the pipes. A minimum 12-inch thickness of bedding material beneath the pipe should be provided. Prior to installation of the pipe, the pipe bedding should be shaped to fit the lower part of the pipe exterior with reasonable closeness to provide uniform support along the pipe. Pipe bedding material should be used as pipe zone backfill and placed in layers and tamped around the pipe to obtain complete contact. To protect the pipe, bedding material should extend at least 12 inches above the top of the pipe.

4.7.5 Trench Backfill Recommendations

The near-surface fill soils are expected to be suitable for reuse as trench backfill material. We do not anticipate excavations to extend to depths that will encounter the native alluvial deposits; however, this material is also expected to be suitable for reuse as trench backfill.

Where imported backfill material is required, trench backfill should consist of imported, clean, free-draining, granular soils free from organic matter or other deleterious materials. Such materials should be less than 4 inches in maximum particle dimension, with less than 7 percent fines (portion passing the U. S. Standard No. 200 sieve), as specified for Gravel Borrow in Section 9-03.14(1) of the WSDOT *Standard Specifications* (WSDOT, 2024). The fine-grained portion of structural fill soils should be non-plastic.

4.7.6 Trench Backfill Placement and Compaction

Proper preparation, placement, and compaction of the trench backfill is extremely important to limit future settlement of the ground surface around structures and along trenches. Given the depth of the proposed trench, failure to achieve proper compaction could result in significant settlement on the order of several inches, resulting in distress to pavements, utilities, and other structures along the trench.

Trench backfill should be uniformly moisture conditioned to within about 3 percent of optimum moisture content prior to placement in the trench. Properly prepared backfill should be placed in successive layers with the minimum cover to be determined based on the pipe material utilized, and the following layers not exceeding 12 inches in loose thickness with each layer being compacted in a systematic manner using appropriately sized compaction equipment to achieve at least 95 percent of the maximum dry density as determined using ASTM D-1557. Smaller loose lifts may be necessary to achieve compaction where handheld compaction equipment such as jumping jacks, hoe-packs, or plate compactors are used. The contractor should develop compaction methods that consistently produce adequate compaction levels.

Full-time observation and testing of trench backfill by a representative of the geotechnical engineer is recommended to help the contractor achieve proper backfill preparation and uniform moisture conditioning, loose lift thickness control, and application of appropriate compactive effort.

During placement of the initial lifts, the trench backfill material should not be bulldozed into the trench or dropped directly on the pipe. Heavy vibratory equipment should not be permitted to operate directly over the pipe until at least 2 feet of backfill has been placed over the pipe.

4.8 GENERAL EARTHWORK

4.8.1 Subgrade Preparation

In all areas of proposed improvements, subgrade preparation should begin with the removal of any existing pavement, topsoil, debris and vegetation. Using a smooth (toothless) bucket, the soils should be excavated to the design elevation. Based on the results of the subsurface explorations, medium dense, silty sand soils are typically expected. The exposed subgrade soils should be thoroughly compacted and proof-rolled using a fully loaded dump truck, or similar heavy equipment, under the observation of the geotechnical engineer, or qualified earthworks inspector. If soft or otherwise unsuitable soils with deleterious material are encountered, they should be over-excavated per the geotechnical engineer and backfilled using Structural Fill per the recommendations in the following section.

4.8.2 Structural Fill

We anticipate that the onsite native soils can be reused as structural fill for this project. However, due to the depositional nature of the alluvial deposits, the composition may vary from those encountered in our explorations. If the native soils are observed to not meet the requirements provided below, imported structural fill may be required. Structural fill should consist of imported clean, free-draining, granular soils free from organic matter or other deleterious materials. Such materials should be less than 4 inches in maximum particle dimension, with less than 7 percent fines (portion passing the U.S. Standard No. 200 sieve), as specified for Gravel Borrow in Section 9-03.14(1) of the 2024 WSDOT *Standard Specifications*. The fine-grained portion of structural fill soils should be non-plastic.

4.8.3 Compaction

Structural fill soils should be moisture conditioned and compacted to the requirements specified in Section 2-03.3(14), Method C, of the 2024 WSDOT *Standard Specifications*, except that maximum dry densities should be obtained using ASTM D 1557 (Modified Proctor). Achievement of proper density of a compacted fill depends on the size and type of compaction equipment, the number of passes, thickness of the layer being compacted, and soil moisture-density properties. In areas where limited space restricts the use of heavy equipment, smaller equipment can be used, but the soil must be placed in thin enough layers to achieve the required relative compaction.

4.8.4 Temporary Excavation

We expect that excavations completed onsite can be accomplished with conventional excavating equipment such as trackhoes. Maintenance of safe working conditions, including temporary excavation stability, is the responsibility of the contractor. In accordance with Part N of Washington Administrative Code (WAC) 296-155, latest revisions, all temporary cuts in excess of 4 feet in height must be either sloped or shored prior to entry by personnel.

Fill and alluvium soils are generally classified as Type C Soils per WAC 296-155. Where no temporary shoring is used, excavations in Type C Soil should be sloped no steeper than 1.5H:1V. Flatter slopes will be required where ground water seepage exists.

The contractor should be responsible for control of ground and surface water and should employ sloping, slope protection, ditching, sumps, dewatering, and other measures as necessary to prevent sloughing of soils.

4.8.5 Wet Weather Earthwork

During periods of wet weather, even the most permeable soils can become difficult to work and compact. We anticipate considerable variability in the fines content of the native soils. Soils with higher fines contents will be difficult to work and compact when wet. If fill is to be placed or earthwork is to be performed in wet weather or under wet conditions, the following recommendations apply:

- Earthwork should be performed in small areas to minimize exposure to wet weather. Excavation of unsuitable and/or softened soil should be followed promptly by placement and compaction of clean structural fill. The size and type of construction equipment used may need to be limited to prevent soil disturbance. Under some circumstances, it may be necessary to excavate soils with a backhoe to minimize subgrade disturbance caused by equipment traffic.

- Material used as excavation backfill in wet weather should consist of clean granular soil with less than 5 percent passing the U.S. No. 200 sieve, based on wet sieving the fraction passing the ¾-inch sieve. The fines should be non-plastic. It should be noted this is an additional restriction on the structural fill materials specified.

- The ground surface within the construction area should be graded to promote surface water run-off and to prevent ponding.

- Within the construction area, the ground surface should be sealed on completion of each shift by a smooth drum vibratory roller, or equivalent, and under no circumstances should soil be left uncompacted and exposed to moisture infiltration.

- Excavation and placement of backfill materials should be monitored by a geotechnical engineer experienced in wet weather earthwork to determine that the work is being accomplished in accordance with the project specifications and the recommendations contained herein.

5.0 CONDITIONS AND LIMITATIONS

We have prepared this draft geotechnical report for the City of Arlington for use in design for this project. The conclusions and interpretations presented in this report should not be construed as our warranty of subsurface conditions at the site. Experience has shown that soil and ground water conditions can vary significantly over small distances and with time. Inconsistent conditions can occur between explorations that may not be detected by a geotechnical study of this scope and nature. If, during future site operations, subsurface conditions are encountered which vary appreciably from those described herein, HWA should be notified for review of the recommendations of this report, and revision of such if necessary.

Within the limitations of scope, schedule and budget, HWA attempted to execute these services in accordance with generally accepted professional principles and practices in the fields of geotechnical engineering and engineering geology in the area at the time the report was prepared. No warranty, express or implied, is made.

HWA does not practice or consult in the field of safety engineering. We do not direct the contractor's operations and cannot be responsible for the safety of personnel other than our own on the site. As such, the safety of others is the responsibility of the contractor. The contractor should notify the owner if any of the recommended actions presented herein are considered unsafe.

We appreciate the opportunity to provide geotechnical services on this project. Should you have any questions or comments, or if we may be of further service, please do not hesitate to call.

Sincerely,

HWA GEOSCIENCES INC.

Donald Huling, P.E.
Geotechnical Engineer, Principal

Sean Schlitt, P.E.
Geotechnical Engineer

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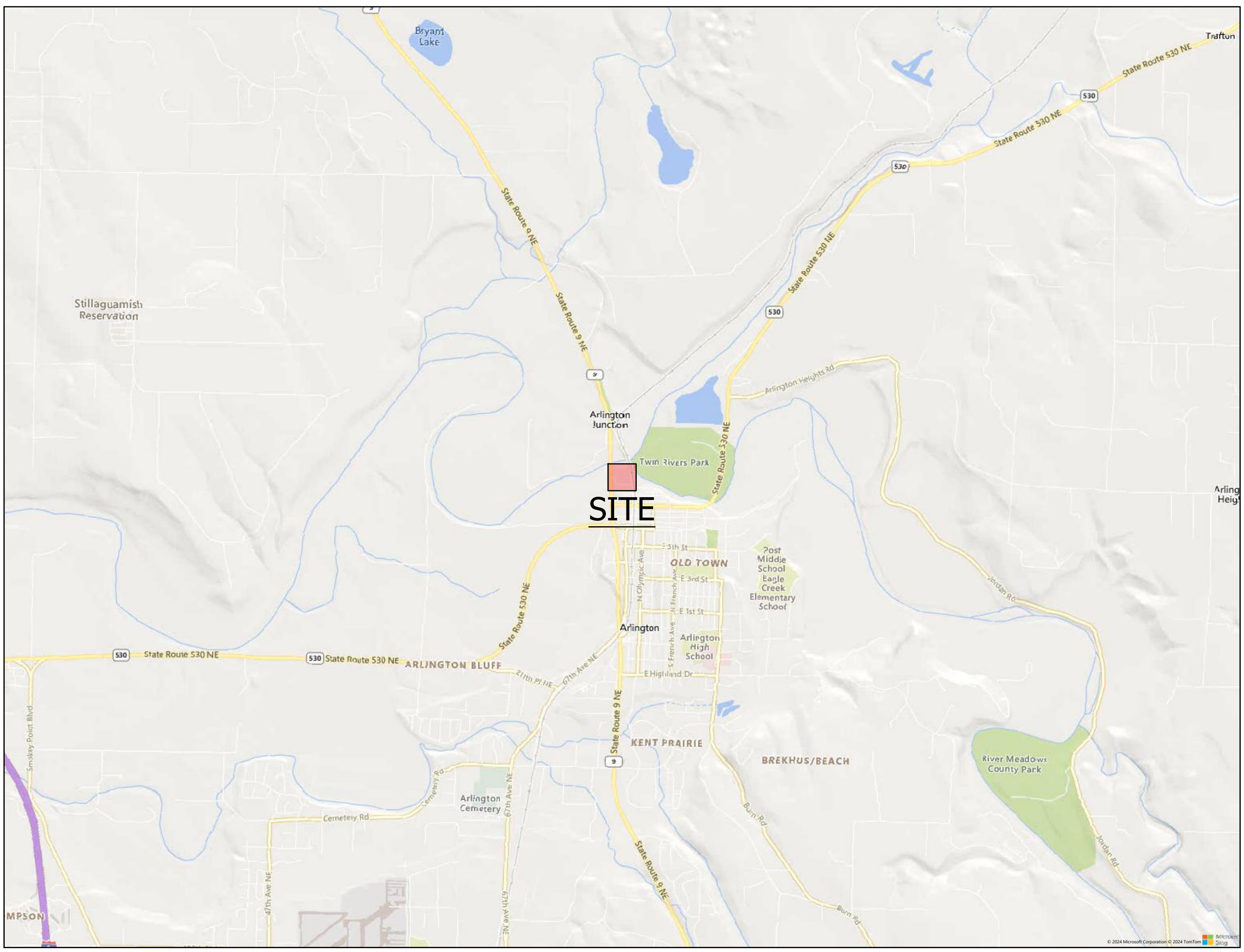
SITE MAP

0 250' 500' 750' 1000'

SCALE: 1" = 500'

N



VICINITY MAP

0 2500' 5000' 7500' 10000'

SCALE: 1" = 5000'

N

HWA GEOSCIENCES INC. DBE/MWBE

SITE AND VICINITY MAP

HALLER SOUTH GEOTECHNICAL INVESTIGATION
ARLINGTON, WASHINGTON

FIGURE NO.:	
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CF	AHF
PROJECT #	
2023-185-21	

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ARLINGTON WELLFIELD

Scale: 1" = 100'-0"

EXPLORATION LEGEND

- HWA-1 BOREHOLE DESIGNATION AND APPROXIMATE LOCATION (HWA, 2024)
- TW-1 BOREHOLE DESIGNATION AND APPROXIMATE LOCATION (PGG, 2021)
- TP-1 EXPLORATION DESIGNATION AND APPROXIMATE LOCATION (GEOTEST, 2018)
- B-1 BOREHOLE DESIGNATION AND APPROXIMATE LOCATION (GEOENGINEERS, 2008)
- B-1 BOREHOLE DESIGNATION AND APPROXIMATE LOCATION (PRIDE, 1999)

A A' SLOPE STABILITY SECTIONS

N

0 50 100 150 200

SCALE: 1" = 100'

BASE MAP PROVIDED BY: BING AND CITY OF ARLINGTON 01.26.2024

HWA GEOSCIENCES INC. DBE/MWBE

HALLER SOUTH
GEOTECHNICAL INVESTIGATION
ARLINGTON, WASHINGTON

SITE & EXPLORATION PLAN

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CF	2
CHECK BY:	PROJECT NO.:
AHF/DJH	2023-185-21

C:\USERS\CFRY\DESKTOP\2023-185-21 HALLER SOUTH GEOTECHNICAL INVESTIGATION\2023-185-21 HALLER SOUTH GEOTECHNICAL INVESTIGATION.DWG <2> Plotted: 3/1/2024 1:06 PM

APPENDIX A

FIELD INVESTIGATION

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

USCS SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP DESCRIPTIONS	
Coarse Grained Soils More than 50% Retained on No. 200 Sieve Size	Gravel and Gravelly Soils More than 50% of Coarse Fraction Retained on No. 4 Sieve	Clean Gravel (little or no fines)	GW	Well-graded GRAVEL
			GP	Poorly-graded GRAVEL
		Gravel with Fines (appreciable amount of fines)	GM	Silty GRAVEL
			GC	Clayey GRAVEL
	Sand and Sandy Soils 50% or More of Coarse Fraction Passing No. 4 Sieve	Clean Sand (little or no fines)	SW	Well-graded SAND
			SP	Poorly-graded SAND
		Sand with Fines (appreciable amount of fines)	SM	Silty SAND
			SC	Clayey SAND
Fine Grained Soils 50% or More Passing No. 200 Sieve Size	Silt and Clay	Liquid Limit Less than 50%	ML	SILT
			CL	Lean CLAY
			OL	Organic SILT/Organic CLAY
	Silt and Clay	Liquid Limit 50% or More	MH	Elastic SILT
			CH	Fat CLAY
			OH	Organic SILT/Organic CLAY
Highly Organic Soils			PT	PEAT

TEST SYMBOLS

%F	Percent Fines
AL	Atterberg Limits: PL = Plastic Limit, LL = Liquid Limit
CBR	California Bearing Ratio
CN	Consolidation
DD	Dry Density (pcf)
DS	Direct Shear
GS	Grain Size Distribution
K	Permeability
MD	Moisture/Density Relationship (Proctor)
MR	Resilient Modulus
OC	Organic Content
pH	pH of Soils
PID	Photoionization Device Reading
PP	Pocket Penetrometer (Approx. Comp. Strength, tsf)
Res.	Resistivity
SG	Specific Gravity
CD	Consolidated Drained Triaxial
CU	Consolidated Undrained Triaxial
UU	Unconsolidated Undrained Triaxial
TV	Torvane (Approx. Shear Strength, tsf)
UC	Unconfined Compression

SAMPLE TYPE SYMBOLS

- 2.0" OD Split Spoon (SPT) (140 lb. hammer with 30 in. drop)
- Shelby Tube
- Non-standard Penetration Test (3.0" OD Split Spoon with Brass Rings)
- Small Bag Sample
- Large Bag (Bulk) Sample
- Core Run
- 3-1/4" OD Split Spoon

GROUNDWATER SYMBOLS

- ∇ Groundwater Level (measured at time of drilling)
- ▼ Groundwater Level (measured in well or open hole after water level stabilized)

COMPONENT DEFINITIONS

COMPONENT	SIZE RANGE
Boulders	Larger than 12 in
Cobbles	3 in to 12 in
Gravel	3 in to No 4 (4.5mm)
Coarse gravel	3 in to 3/4 in
Fine gravel	3/4 in to No 4 (4.5mm)
Sand	No. 4 (4.5 mm) to No. 200 (0.074 mm)
Coarse sand	No. 4 (4.5 mm) to No. 10 (2.0 mm)
Medium sand	No. 10 (2.0 mm) to No. 40 (0.42 mm)
Fine sand	No. 40 (0.42 mm) to No. 200 (0.074 mm)
Silt and Clay	Smaller than No. 200 (0.074mm)

COMPONENT PROPORTIONS

PROPORTION RANGE	DESCRIPTIVE TERMS
< 5%	Clean
5 - 12%	Slightly (Clayey, Silty, Sandy)
12 - 30%	Clayey, Silty, Sandy, Gravelly
30 - 50%	Very (Clayey, Silty, Sandy, Gravelly)

Components are arranged in order of increasing quantities.

NOTES: Soil classifications presented on exploration logs are based on visual and laboratory observation. Soil descriptions are presented in the following general order:

Density/consistency, color, modifier (if any) GROUP NAME, additions to group name (if any), moisture content. Proportion, gradation, and angularity of constituents, additional comments. (GEOLOGIC INTERPRETATION)

Please refer to the discussion in the report text as well as the exploration logs for a more complete description of subsurface conditions.

MOISTURE CONTENT

DRY	Absence of moisture, dusty, dry to the touch.
MOIST	Damp but no visible water.
WET	Visible free water, usually soil is below water table.

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Arlington, Washington

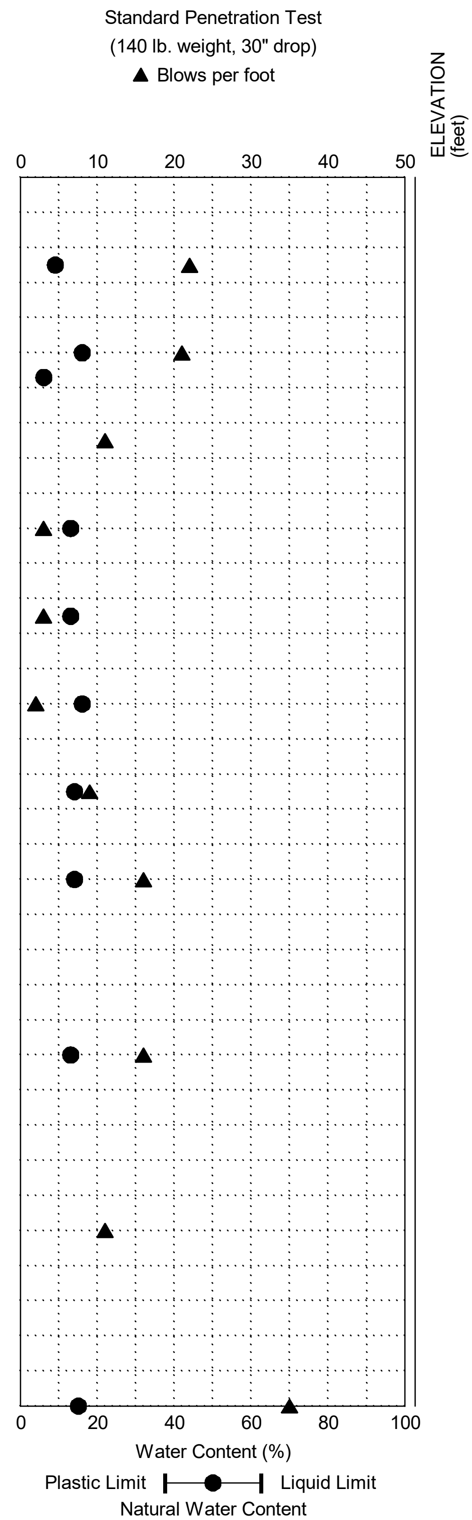
LEGEND OF TERMS AND SYMBOLS USED ON EXPLORATION LOGS

PROJECT NO.: 2023-185 FIGURE: A-1

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: HSA w/3.25" ID, Diedrich D70
SAMPLING METHOD: SPT w/ Autohammer
LOCATION: See Figure 2

DATE STARTED: 1/19/2024
DATE COMPLETED: 1/19/2024
LOGGED BY: A. Heinze Fry

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
0	GP	Gray, pea GRAVEL, moist. **(PLAYGROUND GRAVEL)**				
	SP SM	Medium dense, olive brown, slightly silty, very gravelly, fine to coarse SAND, moist. 2-inches silty fine sand, wet on top. **(FILL)** Gravelly drilling.	S-1	9-11-11	GS	
5	SM	Medium dense, olive brown and gray, silty fine SAND, moist.	S-2a	9-11-10		
	GM	Medium dense, olive brown, silty, sandy, crushed and angular GRAVEL, moist.	S-2b			
		No recovery. Auger cuttings look similar.	S-3	10-6-5		
10	SM	Driller notes softer drilling conditions. Very loose, olive brown, silty, fine SAND, moist. **(ALLUVIUM)**	S-4	3-2-1	GS	
		Few gray laminations with higher silt content. Little yellow-brown coloring in bottom 6 inches.	S-5	2-2-1		
15		2-inch layer gray, very silty sand, at about 15.6 feet. Trace wood.	S-6	1-1-1		
	SP	Loose, dark olive brown, gravelly, fine to medium SAND, wet. Fine, subrounded to rounded gravel. Trace silt.	S-7	5-4-5	GS	▽
20		Becomes medium dense. Increased silt content in tip. Couple coarse gravels.	S-8	6-7-9		
25		Becomes very gravelly; sand component becomes fine to coarse. Driller notes about 6 inches heave - begins to add water. Gravelly drill action at about 26 feet.	S-9	5-6-10		
30	GP	Medium dense, dark olive brown, sandy, fine, subrounded to rounded GRAVEL, wet. Low recovery.	S-10	4-5-6		
35						



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

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Arlington, Washington

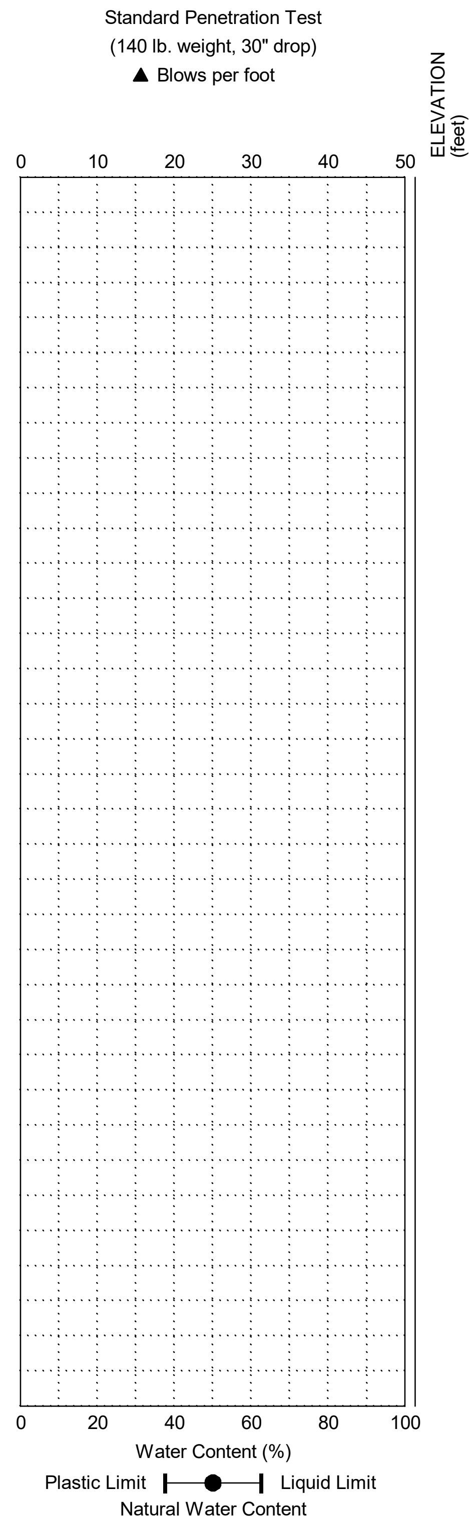
BORING:
HWA-1

PROJECT NO.: 2023-185 FIGURE: A-2

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: HSA w/3.25" ID, Diedrich D70
SAMPLING METHOD: SPT w/ Autohammer
LOCATION: See Figure 2

DATE STARTED: 1/19/2024
DATE COMPLETED: 1/19/2024
LOGGED BY: A. Heinze Fry

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
35		SP	Dense, dark olive brown, gravelly, fine to coarse SAND, wet. Blow counts potentially overstated from gravels. Very gravelly or cobbly drilling from 36 to 37 feet.		S-11	39-21-14		
			HWA-1 terminated at 37 feet below ground surface due to auger refusal. Groundwater was encountered encountered at about 18 feet during exploration. Borehole backfilled with 3/8" bentonite chips.					
40								
45								
50								
55								
60								
65								
70								



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

Haller South Geotechnical Investigation
Arlington, Washington

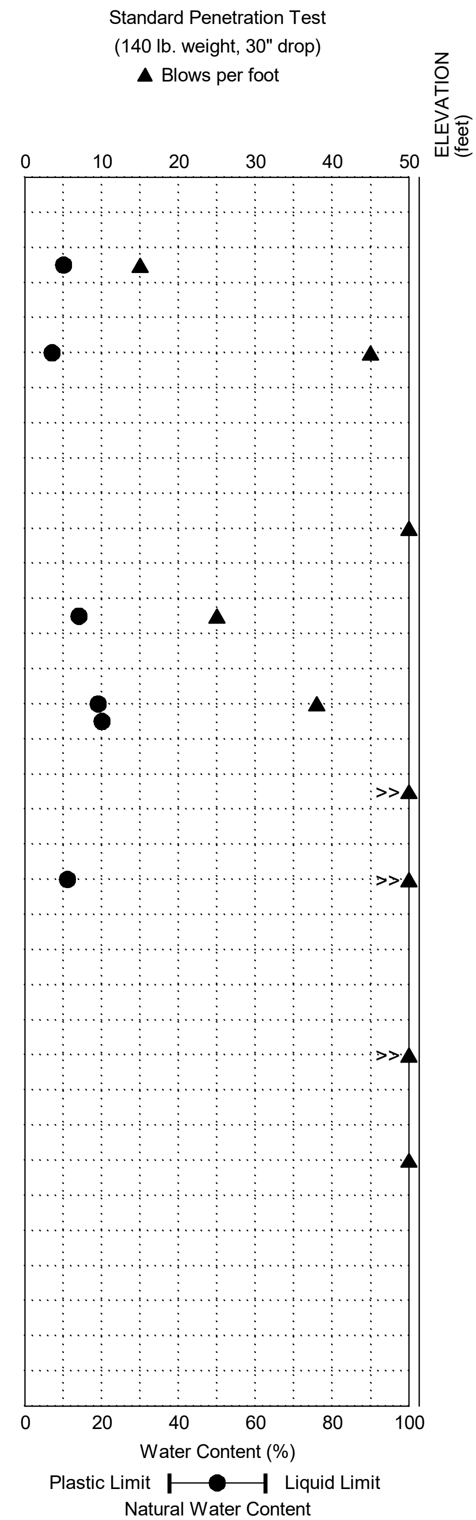
BORING:
HWA-1

PROJECT NO.: 2023-185 FIGURE: A-2

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD:
SAMPLING METHOD: SPT w/ Autohammer
LOCATION: See Figure 2

DATE STARTED: 1/19/2024
DATE COMPLETED: 1/19/2024
LOGGED BY: A. Heinze Fry

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
0	SM	Grass with brown, very silty fine SAND, moist. **(TOPSOIL)**				
		Medium dense, brown, slightly gravelly, silty, fine SAND, moist. **(FILL)** No recovery. Sample from auger cuttings.	S-1	7-6-9		
5	SP SM	Very dense, yellow-brown to red-brown, slightly silty, very gravelly, poorly graded SAND, moist. Blow counts likely overstated on gravel.	S-2	7-15-30	GS	
	GP GM	Very gravelly drilling from about 5 to 12 feet bgs. Very gravelly drilling coarse gravels ascending auger flights. Gravels on auger flights finining slightly near 10 feet bgs.				
10		Very dense, gray, slgihtly silty, sandy, fine to coarse and fractured GRAVEL, moist. Auger cuttings becoming wet while drilling to 12.5 feet.	S-3	15-21-29		▽
	SM	Medium dense, gray, slightly gravelly, very silty, fine to medium SAND, moist. 1-inch red-brown staining on top.	S-4a	8-8-17		
	SM	**(ICE-CONTACT DRIFT)**	S-4b			
15	ML	Medium dense, olive brown with olive green, gravelly, very silty SAND, moist. Reddish brown pockets.	S-5a	11-13-25		
	SM	Driller begins adding water.	S-5b			
		Hard, gray with some olive brown, slightly gravelly, fine to medium sandy SILT, moist.	S-6a	18-23-50/2"		
	SM	Dense, yellow-brown, fine, silty SAND, moist. Few coarse gravel pieces. Some weathered gray fine sandstone.	S-6b			
20		Becomes olive brown to brown.	S-7	21-50/5"		
		Very dense, light gray, silty, fine SANDSTONE, moist to dry. Drilling becomes hard and chattery. **(SANDSTONE)**				
25			S-8	22-19-40		
		Auger refusal at 28 feet bgs. About 9 inches of auger-ground material based on blow counts.	S-9	7-17-33		
30		HWA-2 terminated at 29.5 feet below ground surface due to auger refusal. Perched groundwater was encountered encountered at about 12 feet during exploration. Borehole backfilled with 3/8" bentonite chips.				
35						



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

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Haller South Geotechnical Investigation
Arlington, Washington

BORING:
HWA-2

PROJECT NO.: 2023-185

FIGURE: A-3

APPENDIX B

LABORATORY INVESTIGATION

APPENDIX B

LABORATORY INVESTIGATION

Representative soil samples obtained from our explorations were placed in plastic bags to prevent loss of moisture and transported to our Bothell, Washington, laboratory for further examination and testing. Laboratory tests were conducted on selected soil samples to characterize relevant engineering and index properties of the site soils. Laboratory testing was conducted as described below. A Summary of Material Properties is provided on Figure B-1.

Moisture Content of Soil: The moisture content of selected soil samples (percent by dry mass) was determined in general accordance with ASTM D 2216. The results are shown at the sampled intervals on the appropriate summary logs in Appendix A and on the Summary of Material Properties provided on Figure B-1.

Particle Size Analysis of Soils: Selected samples were tested to determine the particle (grain) size distribution of material in general accordance with ASTM D 6913 and ASTM D 7928/D 6913. The results are summarized on the attached Particle Size Analysis of Soils report, Figures B-2 and B-3, which also provide information regarding the classification of the sample, and the moisture content at the time of testing.

EXPLORATION DESIGNATION	TOP DEPTH (feet)	BOTTOM DEPTH (feet)	MOISTURE CONTENT (%)	ORGANIC CONTENT (%)	SPECIFIC GRAVITY	ATTERBERG LIMITS (%)			% GRAVEL	% SAND	% FINES	ASTM SOIL CLASSIFICATION	SAMPLE DESCRIPTION
						LL	PL	PI					
HWA-1,S-1	2.5	4.0	8.5						30.5	58.9	10.6	SP-SM	Dark olive-brown, poorly graded SAND with silt and gravel
HWA-1,S-2a	5.0	5.7	15.8									SM	Very dark grayish-brown, silty SAND
HWA-1,S-2b	5.7	6.5	5.7									GM	Very dark grayish-brown, silty GRAVEL with sand
HWA-1,S-4	10.0	11.5	12.8						1.8	81.9	16.3	SM	Dark grayish-brown, silty SAND
HWA-1,S-5	12.5	14.0	13.3									SM	Olive-brown, silty SAND
HWA-1,S-6	15.0	16.5	16.0									SM	Dark olive-brown, silty SAND
HWA-1,S-7	17.5	19.0	14.0						28.3	68.6	3.1	SP	Dark grayish-brown, poorly graded SAND with gravel
HWA-1,S-8	20.0	21.5	14.0									SP	Dark gray, poorly graded SAND with gravel
HWA-1,S-9	25.0	26.5	12.5									SP	Very dark gray, poorly graded SAND with gravel
HWA-1,S-11	35.0	36.5	14.7									SP	Very dark gray, poorly graded SAND with gravel
HWA-2,S-1	2.5	4.0	10.1									SM	Dark olive-brown, silty SAND
HWA-2,S-2	5.0	6.5	6.9						43.1	46.1	10.8	SP-SM	Dark olive-brown, poorly graded SAND with silt and gravel
HWA-2,S-4a	12.5	13.7	14.2									SM	Dark grayish-brown, silty SAND
HWA-2,S-5a	15.0	15.5	19.3									ML	Dark gray, SILT with sand
HWA-2,S-5b	15.5	16.5	19.9									SM	Brown, silty SAND
HWA-2,S-7	20.0	20.9	11.1									SM	Very dark gray, silty SAND

Notes: 1. This table summarizes information presented elsewhere in the report and should be used in conjunction with the report test, other graphs and tables, and the exploration logs.
2. The soil classifications in this table are based on ASTM D2487 and D2488 as applicable.

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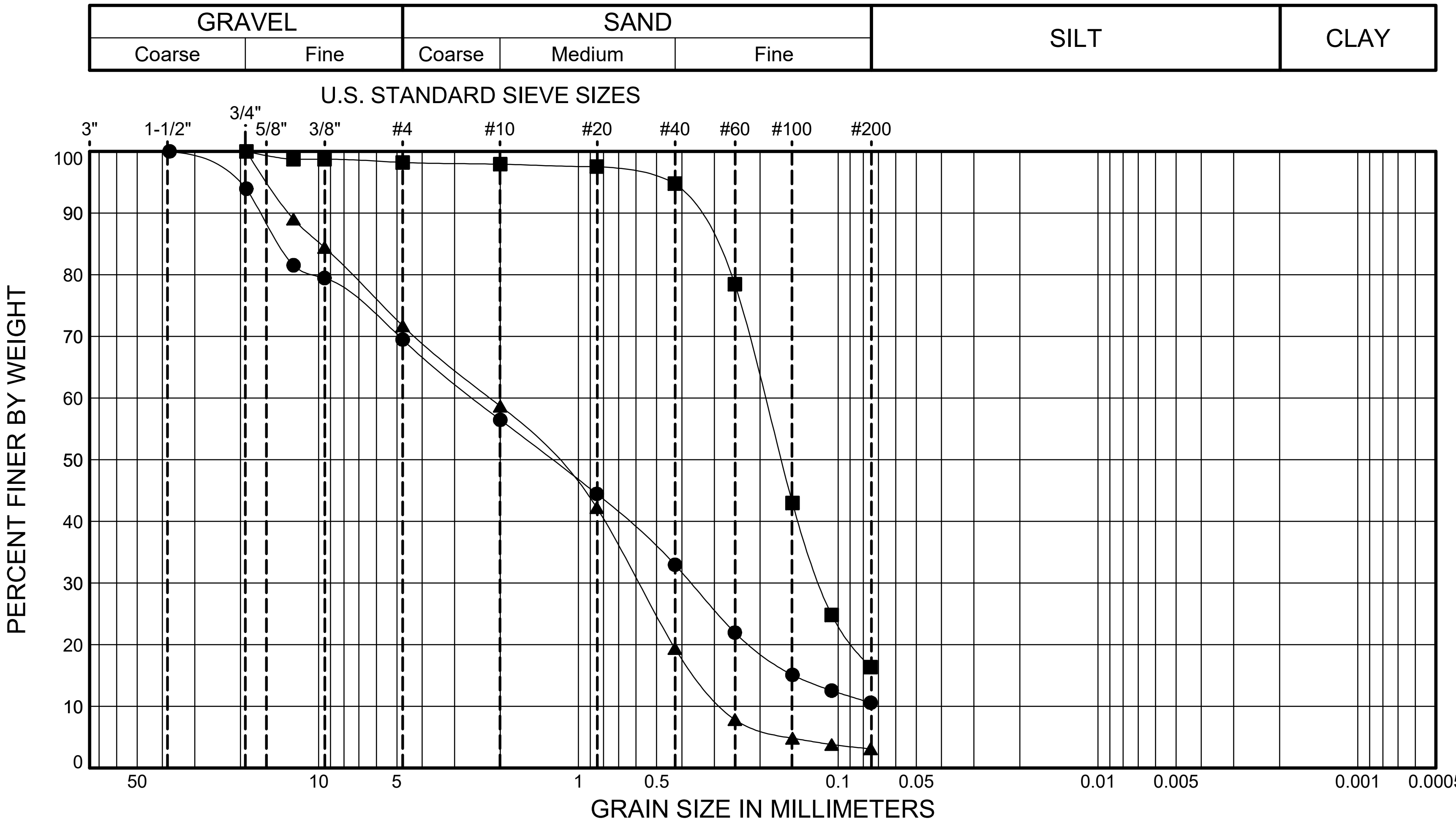
Haller South Geotechnical Investigation
Arlington, Washington

SUMMARY OF
MATERIAL PROPERTIES

PAGE: 1 of 1

PROJECT NO.: 2023-185 FIGURE: B-1



SYMBOL	SAMPLE		DEPTH (ft.)	CLASSIFICATION OF SOIL- ASTM D2487 Group Symbol and Name	% MC	LL	PL	PI	Gravel %	Sand %	Fines %
●	HWA-1	S-1	2.5 - 4.0	(SP-SM) Dark olive-brown, poorly graded SAND with silt and gravel	9				30.5	58.9	10.6
■	HWA-1	S-4	10.0 - 11.5	(SM) Dark grayish-brown, silty SAND	13				1.8	81.9	16.3
▲	HWA-1	S-7	17.5 - 19.0	(SP) Dark grayish-brown, poorly graded SAND with gravel	14				28.3	68.6	3.1

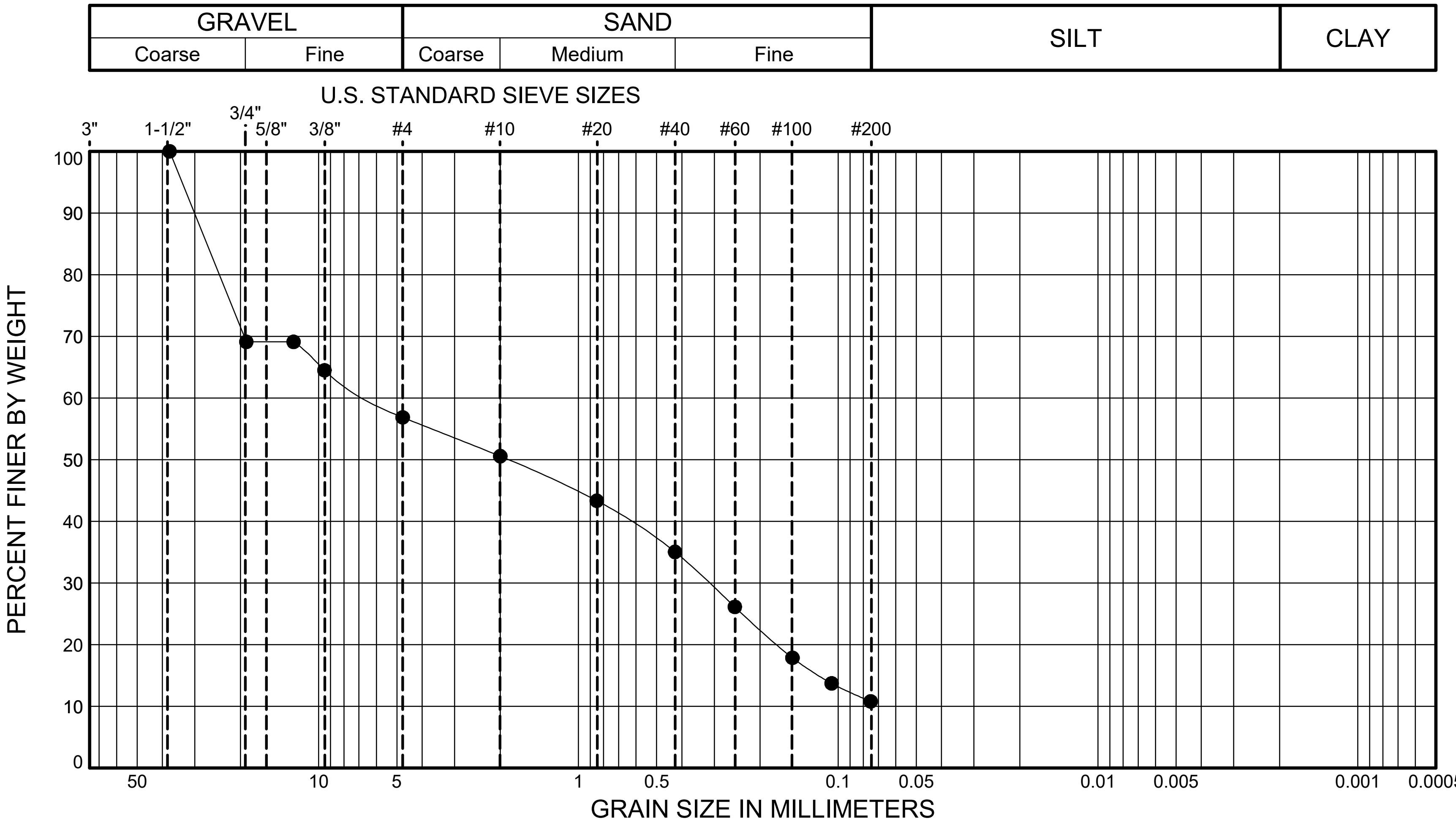
HWA GeoSciences Inc.

Arlington Wellfield
Arlington, Washington

PARTICLE-SIZE ANALYSIS OF SOILS
METHOD ASTM D6913

PROJECT NO.: 2023-185 FIGURE: B-2

HWAGRSZ 2023-185.GPJ 1/29/24



SYMBOL	SAMPLE		DEPTH (ft.)	CLASSIFICATION OF SOIL- ASTM D2487 Group Symbol and Name	% MC	LL	PL	PI	Gravel %	Sand %	Fines %
●	HWA-2	S-2	5.0 - 6.5	(SP-SM) Dark olive-brown, poorly graded SAND with silt and gravel	7				43.1	46.1	10.8

HWA GEOSCIENCES INC.

Arlington Wellfield
Arlington, Washington

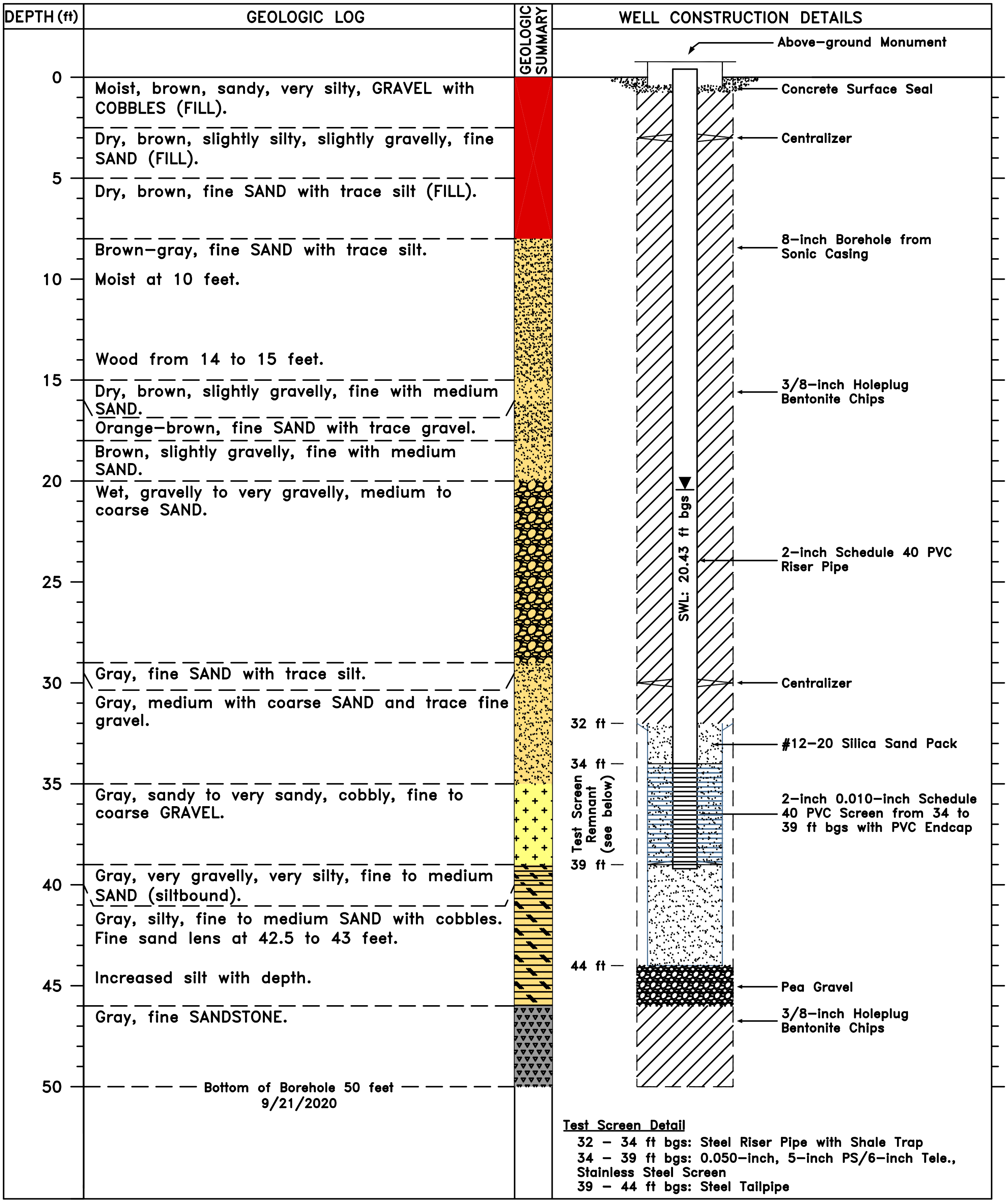
PARTICLE-SIZE ANALYSIS
OF SOILS
METHOD ASTM D6913

PROJECT NO.: 2023-185 FIGURE: B-3

APPENDIX C

EXPLORATIONS BY OTHERS

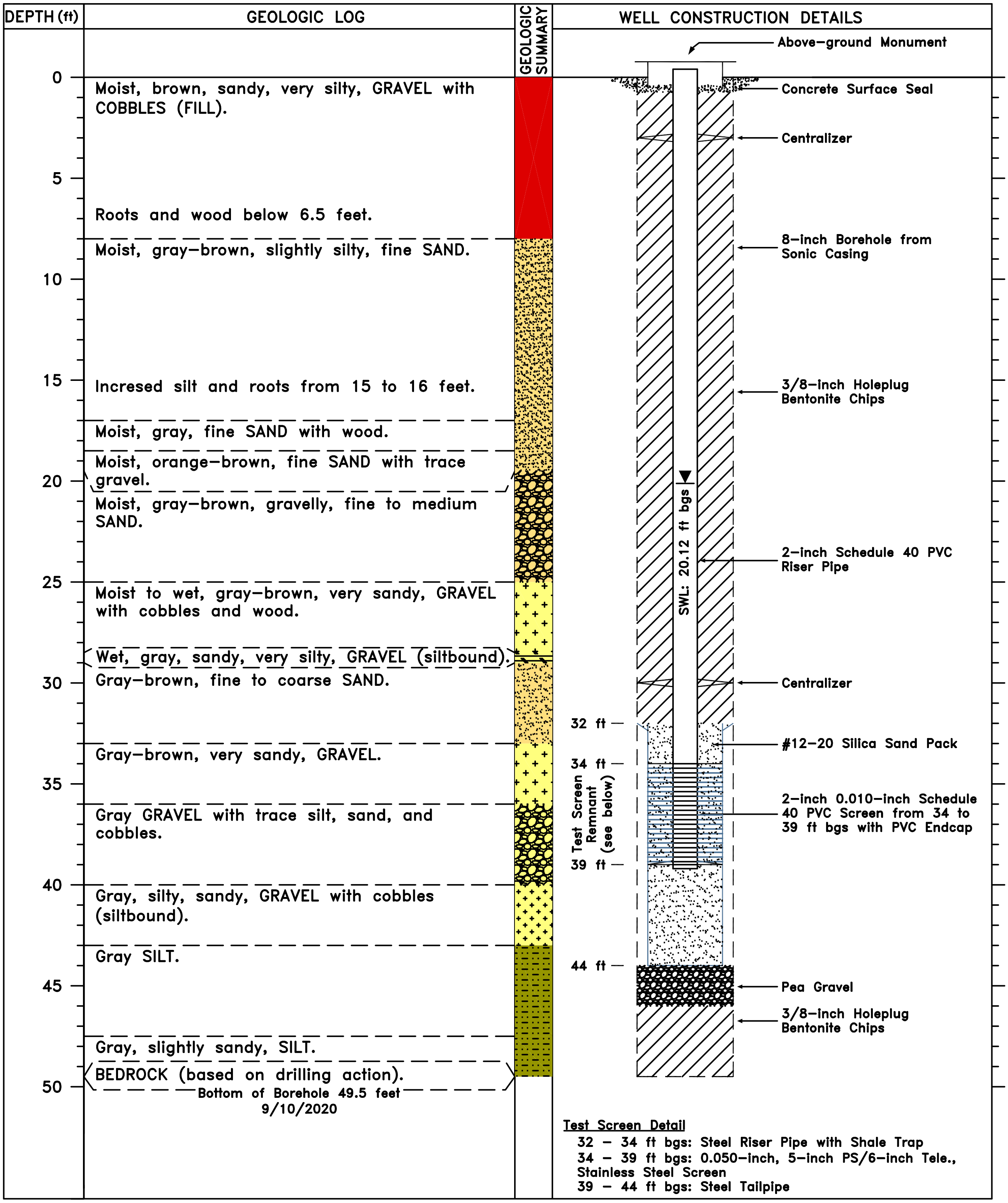
Figure 5-1 Geologic Log and As Built for Arlington TW-1



PROJECT NAME: Arlington Water Source Expansion
WELL INDENTIFICATION NUMBER: TW–1
LOCATION: SE ¼ NW ¼ Sec. 02, T.31N., R.05E.
CONSULTING FIRM: Pacific Groundwater Group
REPRESENTATIVE: Inger Jackson
WDOE NO: BND–054

MEASURING POINT ELEVATION: XXXXX feet NAVD88
MEASURING POINT DESCRIPTION: Top of 2–inch Casing
WATER LEVEL DATE: 10/1/2020
START CARD NO.: RE19802
DRILLING METHOD: Sonic
DRILLER: Ben Johnson
FIRM: Holt Services

Figure 5-2. Geologic Log and As Built for Arlington TW-2



PROJECT NAME: Arlington Water Source Expansion
WELL INDENTIFICATION NUMBER: TW-2
LOCATION: SE ¼ NW ¼ Sec. 02, T.31N., R.05E.
CONSULTING FIRM: Pacific Groundwater Group
REPRESENTATIVE: Joe Morrice
WDOE NO: BND-053

MEASURING POINT ELEVATION: XXXXX feet NAVD88
MEASURING POINT DESCRIPTION: Top of 2-inch Casing
WATER LEVEL DATE: 10/1/2020
START CARD NO.: RE19802
DRILLING METHOD: Sonic
DRILLER: Ben Johnson
FIRM: Holt Services

Soil Classification System

MAJOR DIVISIONS			GRAPHIC SYMBOL	USCS LETTER SYMBOL	TYPICAL DESCRIPTIONS[1][2]
COARSE-GRAINED SOIL (More than 50% of material is larger than No. 200 sieve size)	GRAVEL AND GRAVELLY SOIL (More than 50% of coarse fraction retained on No. 4 sieve)	CLEAN GRAVEL (Little or no fines)		GW	Well-graded gravel; gravel/sand mixture(s); little or no fines
				GP	Poorly graded gravel; gravel/sand mixture(s); little or no fines
		GRAVEL WITH FINES (Appreciable amount of fines)		GM	Silty gravel; gravel/sand/silt mixture(s)
				GC	Clayey gravel; gravel/sand/clay mixture(s)
	SAND AND SANDY SOIL (More than 50% of coarse fraction passed through No. 4 sieve)	CLEAN SAND (Little or no fines)		SW	Well-graded sand; gravelly sand; little or no fines
				SP	Poorly graded sand; gravelly sand; little or no fines
		SAND WITH FINES (Appreciable amount of fines)		SM	Silty sand; sand/silt mixture(s)
				SC	Clayey sand; sand/clay mixture(s)
FINE-GRAINED SOIL (More than 50% of material is smaller than No. 200 sieve size)	SILT AND CLAY (Liquid limit less than 50)			ML	Inorganic silt and very fine sand; rock flour; silty or clayey fine sand or clayey silt with slight plasticity
				CL	Inorganic clay of low to medium plasticity; gravelly clay; sandy clay; silty clay; lean clay
				OL	Organic silt; organic, silty clay of low plasticity
	SILT AND CLAY (Liquid limit greater than 50)			MH	Inorganic silt; micaceous or diatomaceous fine sand
				CH	Inorganic clay of high plasticity; fat clay
				OH	Organic clay of medium to high plasticity; organic silt
	HIGHLY ORGANIC SOIL			PT	Peat; humus; swamp soil with high organic content

OTHER MATERIALS	GRAPHIC SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
PAVEMENT		AC or PC	Asphalt concrete pavement or Portland cement pavement
ROCK		RK	Rock (See Rock Classification)
WOOD		WD	Wood, lumber, wood chips
DEBRIS		DB	Construction debris, garbage

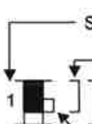
Notes: 1. Soil descriptions are based on the general approach presented in the *Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)*, as outlined in ASTM D 2488. Where laboratory index testing has been conducted, soil classifications are based on the *Standard Test Method for Classification of Soils for Engineering Purposes*, as outlined in ASTM D 2487.

2. Soil description terminology is based on visual estimates (in the absence of laboratory test data) of the percentages of each soil type and is defined as follows:

Primary Constituent: > 50% - "GRAVEL," "SAND," "SILT," "CLAY," etc.
Secondary Constituents: > 30% and ≤ 50% - "very gravelly," "very sandy," "very silty," etc.
> 12% and ≤ 30% - "gravelly," "sandy," "silty," etc.
Additional Constituents: > 5% and ≤ 12% - "slightly gravelly," "slightly sandy," "slightly silty," etc.
≤ 5% - "trace gravel," "trace sand," "trace silt," etc., or not noted.

Drilling and Sampling Key

SAMPLE NUMBER & INTERVAL



Sample Identification Number
Recovery Depth Interval
Sample Depth Interval
Portion of Sample Retained for Archive or Analysis

SAMPLER TYPE

Code	Description
a	3.25-inch O.D., 2.42-inch I.D. Split Spoon
b	2.00-inch O.D., 1.50-inch I.D. Split Spoon
c	Shelby Tube
d	Grab Sample
e	Other - See text if applicable
1	300-lb Hammer, 30-inch Drop
2	140-lb Hammer, 30-inch Drop
3	Pushed
4	Other - See text if applicable

Groundwater

ATD — Approximate water elevation at time of drilling (ATD) or on date noted. Groundwater levels can fluctuate due to precipitation, seasonal conditions, and other factors.

Field and Lab Test Data

Code	Description
PP = 1.0	Pocket Penetrometer, tsf
TV = 0.5	Torvane, tsf
PID = 100	Photoionization Detector VOC screening, ppm
W = 10	Moisture Content, %
D = 120	Dry Density, pcf
-200 = 60	Material smaller than No. 200 sieve, %
GS	Grain Size - See separate figure for data
AL	Atterberg Limits - See separate figure for data
GT	Other Geotechnical Testing
CA	Chemical Analysis

GEOTEST

Haller Park Improvements
1100 West Avenue
Arlington, Washington

Soil Classification System and Key

Figure 3

TP-1

Excavation Method: Rubber-tired Backhoe
Ground Elevation (ft): ~73
Excavated By: City of Arlington

Depth (ft)	Sample Number & Interval	Sampler Type	Test Data	USCS Symbol	Soil Profile	Groundwater
0				SM/OH	Topsoil/Sod	Groundwater not encountered.
				SM	Medium dense, tan, moist, silty, gravelly, SAND with trace organics (Fill)	
2				SM/OH	Relict Topsoil	
				SM	Medium dense, grey, moist, slightly silty, gravelly, SAND (Fill)	
4				SM/OH	Relict Topsoil	
6			W = 5 GS	GP	Loose to medium dense, tan/brown, moist, slightly silty, sandy, GRAVEL, cobble sized grains, slightly oxidized (Alluvium) Becomes medium dense at ~10' BGS	

Test Pit Completed 01/08/18

TP-2

Excavation Method: Rubber-tired Backhoe
Ground Elevation (ft): ~74
Excavated By: City of Arlington

Depth (ft)	Sample Number & Interval	Sampler Type	Test Data	USCS Symbol	Soil Profile	Groundwater
0				SM/OH	Topsoil	Groundwater not encountered.
				SM	Medium dense, brown, moist, slightly silty, slightly gravelly, SAND with trace woody debris (Fill)	
3.5				SM/OH	Relict Topsoil	
4				SM	Medium dense, brown, moist, silty, gravelly, SAND with scattered organics and manmade debris (Fill) Becomes very loose to loose at ~4' BGS	
9				GM	Medium dense to dense, tan/brown, moist, slightly silty, sandy, GRAVEL, cobble sized grains, slightly oxidized (Alluvium)	

Test Pit Completed 01/08/18

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

GEOTEST

Haller Park Improvements
1100 West Avenue
Arlington, Washington

Log of Test Pits

Figure 4 (1 of 3)

TP-3

SAMPLE DATA			SOIL PROFILE			GROUNDWATER
Depth (ft)	Sample Number & Interval	Sampler Type	Test Data	Graphic Symbol	USCS Symbol	Excavation Method: Rubber-tired Backhoe Ground Elevation (ft): ~71 Excavated By: City of Arlington	
0					SM/OH	Topsoil	
0–9			W = 6 GS		GP/GM	Medium dense, grey to brown, slightly silty, sandy, GRAVEL with trace organics, cobbles, stratified (Fill) Becomes loose at ~2' BGS Strata dipping towards the north Metal debris in sidewall throughout depth Becomes loose to medium dense at ~9' BGS	Groundwater not encountered.
9–11					GP	Loose to Medium dense, tan/brown, moist, slightly silty, sandy, GRAVEL, cobble sized grains, slightly oxidized (Alluvium)	

Test Pit Completed 01/08/18
Total Depth of Test Pit = 11.0 ft.

TP-4

SAMPLE DATA			SOIL PROFILE			GROUNDWATER
Depth (ft)	Sample Number & Interval	Sampler Type	Test Data	Graphic Symbol	USCS Symbol	Excavation Method: Rubber-tired Backhoe Ground Elevation (ft): ~70 Excavated By: City of Arlington	
0					SM/OH	Topsoil	
0–3					GM	Medium dense, brown ,damp, silty, sandy, GRAVEL with scattered organics and debris (Fill)	Groundwater not encountered.
3–11.5			W = 24 GS		ML	Medium stiff, dark grey, moist, very sandy, low plasticity SILT with trace organics, homogenous (Alluvium - Possible Levee Deposits)	

Test Pit Completed 01/08/18

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

TP-5

SAMPLE DATA				SOIL PROFILE			GROUNDWATER
Depth (ft)	Sample Number & Interval	Sampler Type	Test Data	Graphic Symbol	USCS Symbol	Excavation Method: Rubber-tired Backhoe Ground Elevation (ft): ~70 Excavated By: City of Arlington	
0					SM/OH	Topsoil/Sod	
					GP	Loose, brown, wet, sandy, GRAVEL, fine gravel particles (Fill - Pea gravel/Drain rock)	Slight groundwater seepage encountered at 1.0 ft.
2					SM	Loose to medium dense, black/brown, moist, gravelly, silty, Sand with numerous organics, scattered debris, and organic odor (Fill)	
4							
6						Steam coming from excavated material	
8						Metal debris at ~7.5' BGS	
						Piece of concrete sidewalk at ~9' BGS	
10					ML/CL	Soft, blue, moist, slightly silty, gravelly, CLAY/SILT (Possible Alluvium or Fill)	Slight groundwater seepage encountered at 10.0 ft.
12							

Test Pit Completed 01/08/18
Total Depth of Test Pit = 11.0 ft.

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

17-0857 2/6/18 X:\0-PROJECTS GEO\00000-PROJECTS 2017-GEO\FULL GEO EVALUATIONS\CITY OF ARLINGTON - 17-0857 - HALLER PARK IMPROVEMENTS\GINT\17-0857 - HALLER PARK.GPJ TEST PIT LOG

GEOTEST	Haller Park Improvements 1100 West Avenue Arlington, Washington	Log of Test Pits	Figure 6 (3 of 3)

WILDCAT DYNAMIC CONE LOG

GeoTest Services, Inc.
741 Marine Drive
Bellingham, WA 98225

PROJECT NUMBER: 17-0857
DATE STARTED: 02-02-2018
DATE COMPLETED: 02-02-2018

HOLE #: DCP-1
CREW: TG/DM
PROJECT: Haller Park Improvements
ADDRESS: Haller Park
LOCATION: Arlington, WA

SURFACE ELEVATION: ~71'
WATER ON COMPLETION: Not Determined
HAMMER WEIGHT: 35 lbs.
CONE AREA: 10 sq. cm

DEPTH	BLOWS PER 10 cm	RESISTANCE Kg/cm²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY SAND & SILT	CLAY
-	7	31.1	•••••••••	8	LOOSE	MEDIUM STIFF
-	5	22.2	••••••	6	LOOSE	MEDIUM STIFF
- 1 ft	9	40.0	•••••••••••	11	MEDIUM DENSE	STIFF
-	15	66.6	•••••••••••••••••••	19	MEDIUM DENSE	VERY STIFF
-	13	57.7	••••••••••••••••	16	MEDIUM DENSE	VERY STIFF
- 2 ft	10	44.4	••••••••••••	12	MEDIUM DENSE	STIFF
-	9	40.0	•••••••••••	11	MEDIUM DENSE	STIFF
-	5	22.2	••••••	6	LOOSE	MEDIUM STIFF
- 3 ft	5	22.2	••••••	6	LOOSE	MEDIUM STIFF
- 1 m	6	26.6	•••••••	7	LOOSE	MEDIUM STIFF
-	5	19.3	•••••	5	LOOSE	MEDIUM STIFF
- 4 ft	5	19.3	•••••	5	LOOSE	MEDIUM STIFF
-	5	19.3	•••••	5	LOOSE	MEDIUM STIFF
-	5	19.3	•••••	5	LOOSE	MEDIUM STIFF
- 5 ft	5	19.3	•••••	5	LOOSE	MEDIUM STIFF
-	5	19.3	•••••	5	LOOSE	MEDIUM STIFF
-	7	27.0	•••••••	7	LOOSE	MEDIUM STIFF
- 6 ft	6	23.2	••••••	6	LOOSE	MEDIUM STIFF
-	6	23.2	••••••	6	LOOSE	MEDIUM STIFF
- 2 m	5	19.3	•••••	5	LOOSE	MEDIUM STIFF
- 7 ft	4	13.7	•••	3	VERY LOOSE	SOFT
-	5	17.1	••••	4	VERY LOOSE	SOFT
-	6	20.5	•••••	5	LOOSE	MEDIUM STIFF
- 8 ft	10	34.2	•••••••••	9	LOOSE	STIFF
-	9	30.8	••••••••	8	LOOSE	MEDIUM STIFF
-	9	30.8	••••••••	8	LOOSE	MEDIUM STIFF
- 9 ft	10	34.2	•••••••••	9	LOOSE	STIFF
-	11	37.6	••••••••••	10	LOOSE	STIFF
-	11	37.6	••••••••••	10	LOOSE	STIFF
- 3 m 10 ft	9	30.8	••••••••	8	LOOSE	MEDIUM STIFF
-	9	27.5	•••••••	7	LOOSE	MEDIUM STIFF
-	7	21.4	••••••	6	LOOSE	MEDIUM STIFF
-	6	18.4	•••••	5	LOOSE	MEDIUM STIFF
- 11 ft	6	18.4	•••••	5	LOOSE	MEDIUM STIFF
-	6	18.4	•••••	5	LOOSE	MEDIUM STIFF
-	9	27.5	•••••••	7	LOOSE	MEDIUM STIFF
- 12 ft	7	21.4	••••••	6	LOOSE	MEDIUM STIFF
-	7	21.4	••••••	6	LOOSE	MEDIUM STIFF
-	9	27.5	•••••••	7	LOOSE	MEDIUM STIFF
- 4 m 13 ft	39	119.3	••••••••••••••••••••••••••••••••••	-	DENSE	HARD

HOLE #: DCP-1

WILDCAT DYNAMIC CONE LOG

PROJECT: Haller Park Improvements

PROJECT NUMBER: 17-0857

DEPTH	BLOWS PER 10 cm	RESISTANCE Kg/cm²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY SAND & SILT	TESTED CONSISTENCY CLAY
	9	24.9	•••••••	7	LOOSE	MEDIUM STIFF
	5	13.9	••••	3	VERY LOOSE	SOFT
14 ft	4	11.1	•••	3	VERY LOOSE	SOFT
	6	16.6	••••	4	VERY LOOSE	SOFT
	7	19.4	•••••	5	LOOSE	MEDIUM STIFF
15 ft	8	22.2	••••••	6	LOOSE	MEDIUM STIFF
	8	22.2	••••••	6	LOOSE	MEDIUM STIFF
16 ft						
5 m						
17 ft						
18 ft						
19 ft						
6 m						
20 ft						
21 ft						
22 ft						
7 m 23 ft						
24 ft						
25 ft						
26 ft						
8 m						
27 ft						
28 ft						
29 ft						
9 m						

WILDCAT DYNAMIC CONE LOG

GeoTest Services, Inc.
741 Marine Drive
Bellingham, WA 98225

PROJECT NUMBER: 17-0857
DATE STARTED: 02-02-2018
DATE COMPLETED: 02-02-2018

HOLE #: DCP-2
CREW: TG/DM
PROJECT: Haller Park Improvements
ADDRESS: Haller Park
LOCATION: Arlington, WA

SURFACE ELEVATION: ~70'
WATER ON COMPLETION: Not Determined
HAMMER WEIGHT: 35 lbs.
CONE AREA: 10 sq. cm

DEPTH	BLOWS PER 10 cm	RESISTANCE Kg/cm²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY SAND & SILT	TESTED CONSISTENCY CLAY
-	14	62.2	••••••••••••••••••	17	MEDIUM DENSE	VERY STIFF
-	33	146.5	••	-	DENSE	HARD
- 1 ft	40	177.6	•••	-	DENSE	HARD
-	30	133.2	••••••••••••••••••••••••••••••••••••••	-	DENSE	HARD
-	48	213.1	•••	-	VERY DENSE	HARD
- 2 ft	115	510.6	•••	-	VERY DENSE	HARD
-	9	40.0	•••••••••••	11	MEDIUM DENSE	STIFF
-	5	22.2	••••••	6	LOOSE	MEDIUM STIFF
- 3 ft	4	17.8	•••••	5	LOOSE	MEDIUM STIFF
- 1 m	2	8.9	••	2	VERY LOOSE	SOFT
-	2	7.7	••	2	VERY LOOSE	SOFT
- 4 ft	7	27.0	•••••••	7	LOOSE	MEDIUM STIFF
-	8	30.9	••••••••	8	LOOSE	MEDIUM STIFF
-	13	50.2	••••••••••••••	14	MEDIUM DENSE	STIFF
- 5 ft	8	30.9	••••••••	8	LOOSE	MEDIUM STIFF
-	8	30.9	••••••••	8	LOOSE	MEDIUM STIFF
-	6	23.2	••••••	6	LOOSE	MEDIUM STIFF
- 6 ft	12	46.3	•••••••••••••	13	MEDIUM DENSE	STIFF
-	20	77.2	••••••••••••••••••••••	22	MEDIUM DENSE	VERY STIFF
- 2 m	10	38.6	•••••••••••	11	MEDIUM DENSE	STIFF
- 7 ft	9	30.8	••••••••	8	LOOSE	MEDIUM STIFF
-	8	27.4	•••••••	7	LOOSE	MEDIUM STIFF
-	9	30.8	••••••••	8	LOOSE	MEDIUM STIFF
- 8 ft	13	44.5	••••••••••••	12	MEDIUM DENSE	STIFF
-	50	171.0	•••	-	DENSE	HARD
-	50	171.0	•••	-	DENSE	HARD
- 9 ft	30	102.6	•••••••••••••••••••••••••••••	-	MEDIUM DENSE	VERY STIFF
-	13	44.5	••••••••••••	12	MEDIUM DENSE	STIFF
-	5	17.1	••••	4	VERY LOOSE	SOFT
- 3 m 10 ft	4	13.7	•••	3	VERY LOOSE	SOFT
-	4	12.2	•••	3	VERY LOOSE	SOFT
-	5	15.3	••••	4	VERY LOOSE	SOFT
-	7	21.4	••••••	6	LOOSE	MEDIUM STIFF
- 11 ft	6	18.4	•••••	5	LOOSE	MEDIUM STIFF
-	6	18.4	•••••	5	LOOSE	MEDIUM STIFF
-	8	24.5	•••••••	6	LOOSE	MEDIUM STIFF
- 12 ft	6	18.4	•••••	5	LOOSE	MEDIUM STIFF
-	7	21.4	••••••	6	LOOSE	MEDIUM STIFF
-	11	33.7	•••••••••	9	LOOSE	STIFF
- 4 m 13 ft	12	36.7	••••••••••	10	LOOSE	STIFF

HOLE #: DCP-2

WILDCAT DYNAMIC CONE LOG

PROJECT: Haller Park Improvements

PROJECT NUMBER: 17-0857

DEPTH	BLOWS PER 10 cm	RESISTANCE Kg/cm²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY SAND & SILT	CLAY
-	10	27.7	••••••••	7	LOOSE	MEDIUM STIFF
-	10	27.7	••••••••	7	LOOSE	MEDIUM STIFF
- 14 ft						
-						
-						
- 15 ft						
-						
-						
- 16 ft						
- 5 m						
-						
- 17 ft						
-						
-						
- 18 ft						
-						
-						
- 19 ft						
-						
- 6 m						
- 20 ft						
-						
-						
- 21 ft						
-						
-						
- 22 ft						
-						
-						
- 7 m 23 ft						
-						
-						
- 24 ft						
-						
-						
- 25 ft						
-						
-						
- 26 ft						
- 8 m						
-						
- 27 ft						
-						
-						
- 28 ft						
-						
-						
- 29 ft						
-						
- 9 m						

WILDCAT DYNAMIC CONE LOG

GeoTest Services, Inc.
741 Marine Drive
Bellingham, WA 98225

PROJECT NUMBER: 17-0857
DATE STARTED: 02-02-2018
DATE COMPLETED: 02-02-2018

HOLE #: DCP-3
CREW: TG/DM
PROJECT: Haller Park Improvements
ADDRESS: Haller Park
LOCATION: Arlington, WA

SURFACE ELEVATION: ~74'
WATER ON COMPLETION: Not Determined
HAMMER WEIGHT: 35 lbs.
CONE AREA: 10 sq. cm

DEPTH	BLOWS PER 10 cm	RESISTANCE Kg/cm²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY SAND & SILT	CLAY
-	3	13.3	•••	3	VERY LOOSE	SOFT
-	20	88.8	•••••••••••••••••••••••••	25	MEDIUM DENSE	VERY STIFF
- 1 ft	23	102.1	•••••••••••••••••••••••••••••	-	MEDIUM DENSE	VERY STIFF
-	30	133.2	••••••••••••••••••••••••••••••••••••••	-	DENSE	HARD
-	21	93.2	•••••••••••••••••••••••••••	-	MEDIUM DENSE	VERY STIFF
- 2 ft	13	57.7	••••••••••••••••	16	MEDIUM DENSE	VERY STIFF
-	11	48.8	••••••••••••••	13	MEDIUM DENSE	STIFF
-	12	53.3	•••••••••••••••	15	MEDIUM DENSE	STIFF
- 3 ft	11	48.8	••••••••••••••	13	MEDIUM DENSE	STIFF
- 1 m	13	57.7	••••••••••••••••	16	MEDIUM DENSE	VERY STIFF
-	15	57.9	••••••••••••••••	16	MEDIUM DENSE	VERY STIFF
- 4 ft	15	57.9	••••••••••••••••	16	MEDIUM DENSE	VERY STIFF
-	8	30.9	••••••••	8	LOOSE	MEDIUM STIFF
-	3	11.6	•••	3	VERY LOOSE	SOFT
- 5 ft	2	7.7	••	2	VERY LOOSE	SOFT
-	1	3.9	•	1	VERY LOOSE	VERY SOFT
-	0	0.0		0	VERY LOOSE	VERY SOFT
- 6 ft	0	0.0		0	VERY LOOSE	VERY SOFT
-	0	0.0		0	VERY LOOSE	VERY SOFT
- 2 m	0	0.0		0	VERY LOOSE	VERY SOFT
- 7 ft	0	0.0		0	VERY LOOSE	VERY SOFT
-	1	3.4		0	VERY LOOSE	VERY SOFT
-	1	3.4		0	VERY LOOSE	VERY SOFT
- 8 ft	1	3.4		0	VERY LOOSE	VERY SOFT
-	2	6.8	•	1	VERY LOOSE	VERY SOFT
-	7	23.9	••••••	6	LOOSE	MEDIUM STIFF
- 9 ft	3	10.3	••	2	VERY LOOSE	SOFT
-	4	13.7	•••	3	VERY LOOSE	SOFT
-	13	44.5	••••••••••••	12	MEDIUM DENSE	STIFF
- 3 m 10 ft	60	205.2	•••	-	VERY DENSE	HARD
-	60	183.6	•••	-	VERY DENSE	HARD
-						
-						
- 11 ft						
-						
-						
- 12 ft						
-						
-						
- 4 m 13 ft						

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% RETAINED ON NO. 200 SIEVE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS
				SP	POORLY-GRADED SANDS, GRAVELLY SAND
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% PASSING NO. 200 SIEVE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS, ROCK FLOUR, CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS AND SILTS OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: Multiple symbols are used to indicate borderline or dual soil classifications

ADDITIONAL MATERIAL SYMBOLS

SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
	CC	Cement Concrete
	AC	Asphalt Concrete
	CR	Crushed Rock/ Quarry Spalls
	TS	Topsoil/ Forest Duff/Sod

Measured groundwater level in exploration, well, or piezometer

Groundwater observed at time of exploration

Perched water observed at time of exploration

Measured free product in well or piezometer

Stratigraphic Contact

Distinct contact between soil strata or geologic units

Gradual change between soil strata or geologic units

Approximate location of soil strata change within a geologic soil unit

Sampler Symbol Descriptions

2.4-inch I.D. split barrel

Standard Penetration Test (SPT)

Shelby tube

Piston

Direct-Push

Bulk or grab

Blowcount is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted). See exploration log for hammer weight and drop.

A "P" indicates sampler pushed using the weight of the drill rig.

Laboratory / Field Tests

%F	Percent fines
AL	Atterberg limits
CA	Chemical analysis
CP	Laboratory compaction test
CS	Consolidation test
DS	Direct shear
HA	Hydrometer analysis
MC	Moisture content
MD	Moisture content and dry density
OC	Organic content
PM	Permeability or hydraulic conductivity
PP	Pocket penetrometer
SA	Sieve analysis
TX	Triaxial compression
UC	Unconfined compression
VS	Vane shear

Sheen Classification

NS	No Visible Sheen
SS	Slight Sheen
MS	Moderate Sheen
HS	Heavy Sheen
NT	Not Tested

NOTE: The reader must refer to the discussion in the report text and the logs of explorations for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the specific exploration locations and at the time the explorations were made; they are not warranted to be representative of subsurface conditions at other locations or times.

Date(s) Drilled	07/17/07	Logged By	BHC	Checked By	DCO
Drilling Contractor	Boart Longyear	Drilling Method	Hollow-stem Auger	Sampling Methods	D&M
Auger Data	4" ID	Hammer Data	300 lb hammer/30 in drop Rope and Cathead	Drilling Equipment	B-59 Truck
Total Depth (ft)	19	Surface Elevation (ft)		Groundwater Level (ft. bgs)	11
Vertical Datum		Datum/ System		Easting(x): Northing(y):	

Depth feet	SAMPLES Interval	Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft^3	OTHER TESTS AND NOTES
0							CR	10 inches crushed rock			
							SP-SM	Brown fine to coarse sand with silt and gravel (medium dense, moist) (fill)			
		6	15	1					5		SA
							SM	Brown silty fine to medium sand with gravel (dense to very dense, moist)			
5		6	84	2							
							GM	Brown-gray silty gravel with sand (dense to very dense, moist)			
		6	84	3							
10					▼						
		0	50/2"	4							
							SP-SM	Gray-brown fine sand with silt (dense, wet)			
15											
		6	37	5					20	108	
20											

Note: See Figure A-1 for explanation of symbols.

LOG OF BORING B-4

GeoEngineers

Project: City of Arlington WWTP Improvements
Project Location: Arlington, Washington
Project Number: 5430-004-00 Task 100

Figure A-5
Sheet 1 of 1

LOG OF BORING NO. B-1

Sheet 1 of 1

Date drilled 2/7/96 Sampler / Driving Weight 140 lb 30" Drop Elevation (ft) 73

This log is part of the report prepared by Yonemitsu Geological Services (YGS) for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered.

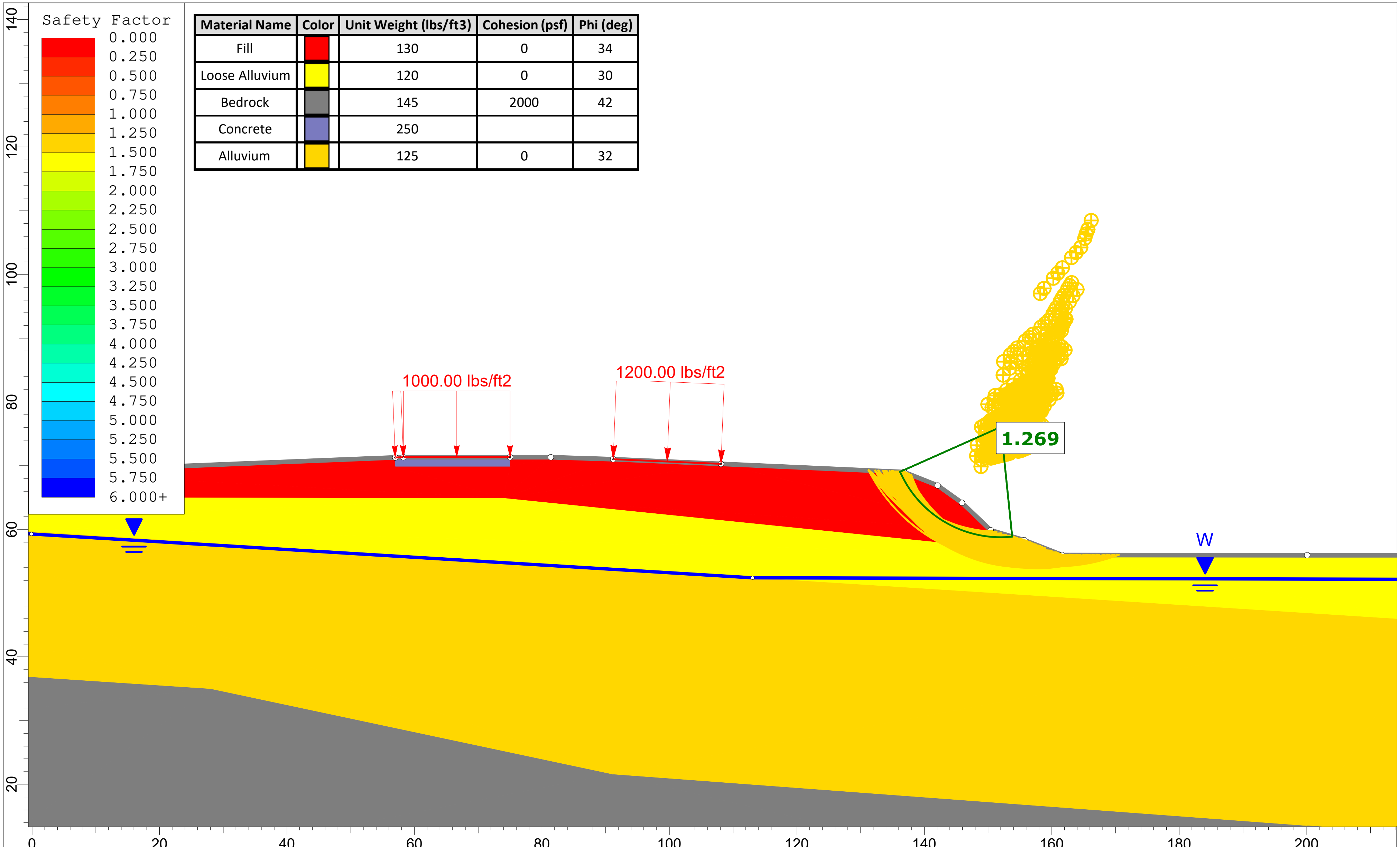
Depth, ft	Elevation	Samples	Blows/6"	Graphic Symbol	DESCRIPTION	Observation Well	Dry density pcf	Moisture Content, %	Other tests
					Fill				
					2-inch layer of base course gravel				
	70		3 4 5		SILTY SAND WITH GRAVEL; dark brown, fine to medium, trace organics; medium dense, moist				
5					**Older Alluvium Deposits**				
					SILTY SAND WITH GRAVEL; gray-brown, fine to medium, trace cobbles up to 8" diameter, scattered boulders estimated by drill action 1-foot diameter; dense, moist				
	65		73/6"		Cobbles and Boulders; boulders estimated to be 2-foot diameter				
10					Drilled to refusal to a depth of 7.5 feet. Boring was restarted two other locations and drilled to refusal. Bottom of boring at depth 8.0 feet; completed and backfilled with bentonite chips on 2/7/96. No groundwater encountered during drilling.				
	60								
15									
	55								
20									
	50								
25									
	45								

Arlington Wastewater Treatment Facility
Arlington, Washington
For Earth Tech

Project No. 5014112.001

APPENDIX D

SLOPE STABILITY ANALYSIS MODELS

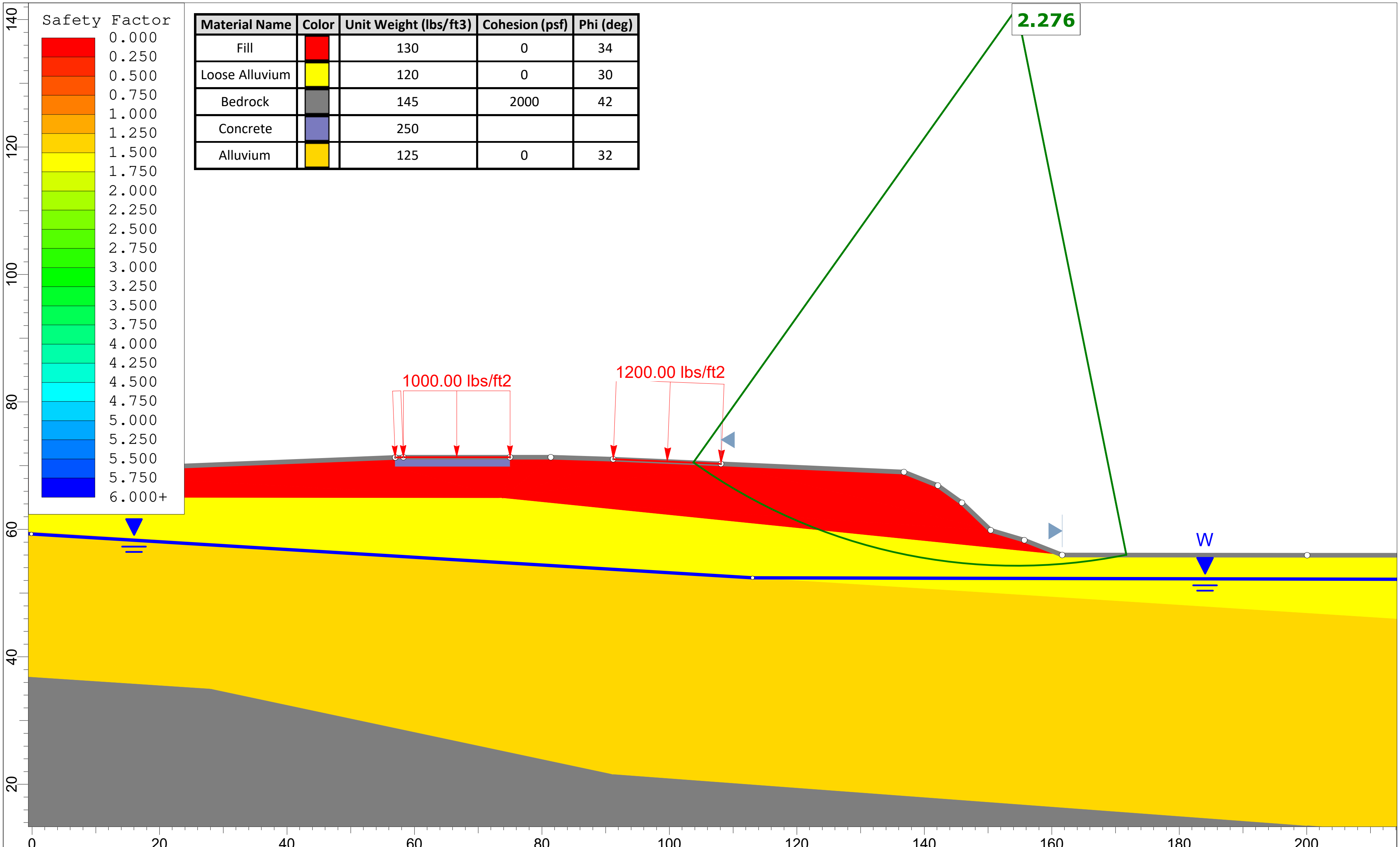


Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Fill		130	0	34
Loose Alluvium		120	0	30
Bedrock		145	2000	42
Concrete		250		
Alluvium		125	0	32

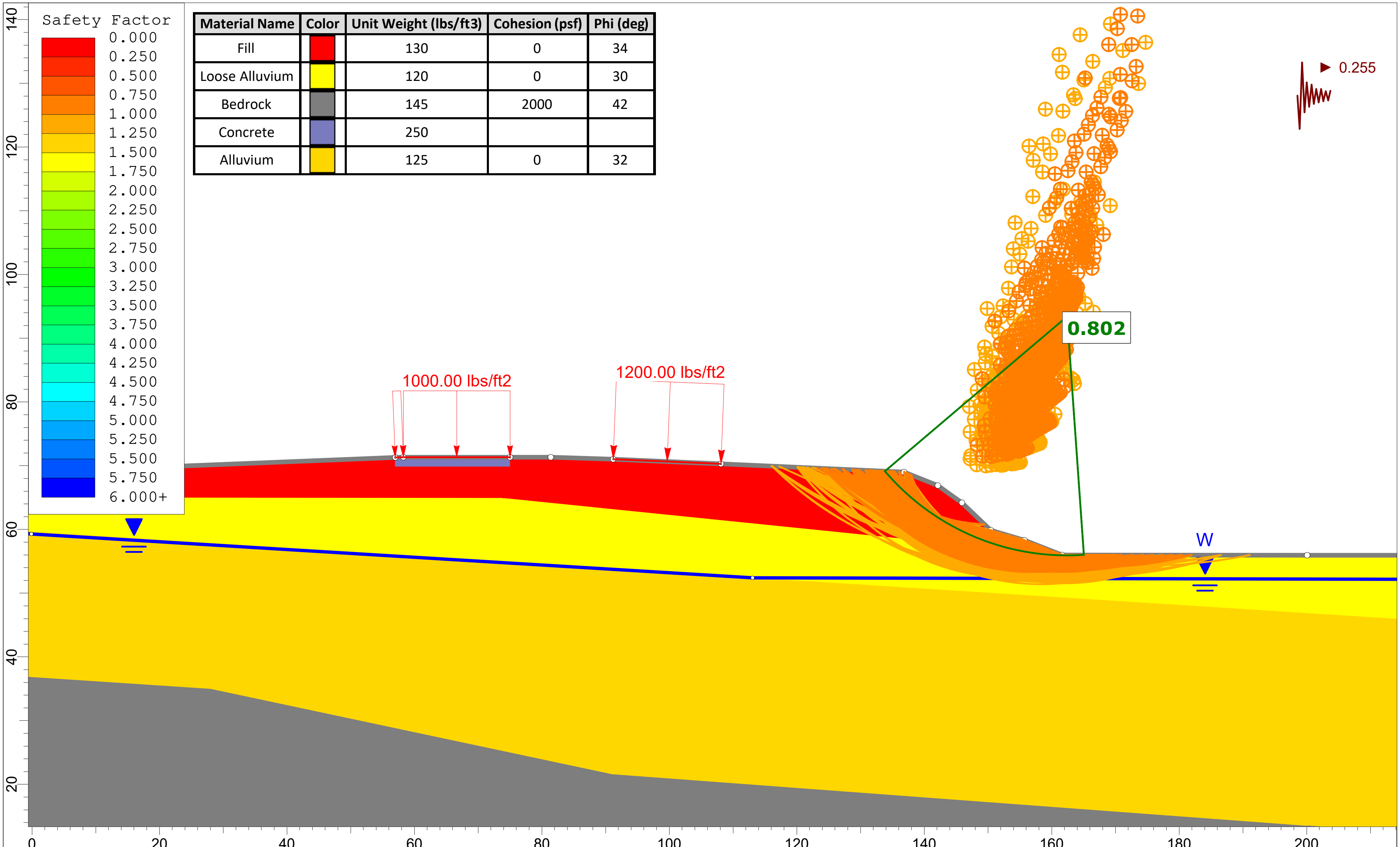
HWA GEOSCIENCES INC.
DBE/MWBE

SLIDEINTERPRET 9.023

Project	Haller South Geotechnical Investigation
Analysis Description	Global Stability - Proposed Conditions - Well House Location
Drawn By	AHF/SKS
Scale	1:250
Company	HWA GeoSciences
Date	2/15/2024, 1:16:01 PM
Loading Scenario	Figure D-1 - Static Analysis



HWA GEOSCIENCES INC. DBE/MWBE	*Project* Haller South Geotechnical Investigation		
	Analysis Description Global Stability - Existing Conditions - Well House Location		
	Drawn By AHF/SKS	*Scale* 1:250	*Company* HWA GeoSciences
SLIDEINTERPRET 9.023	*Date* 2/15/2024, 1:16:01 PM		*Loading Scenario* Figure D-2 - Static Analysis

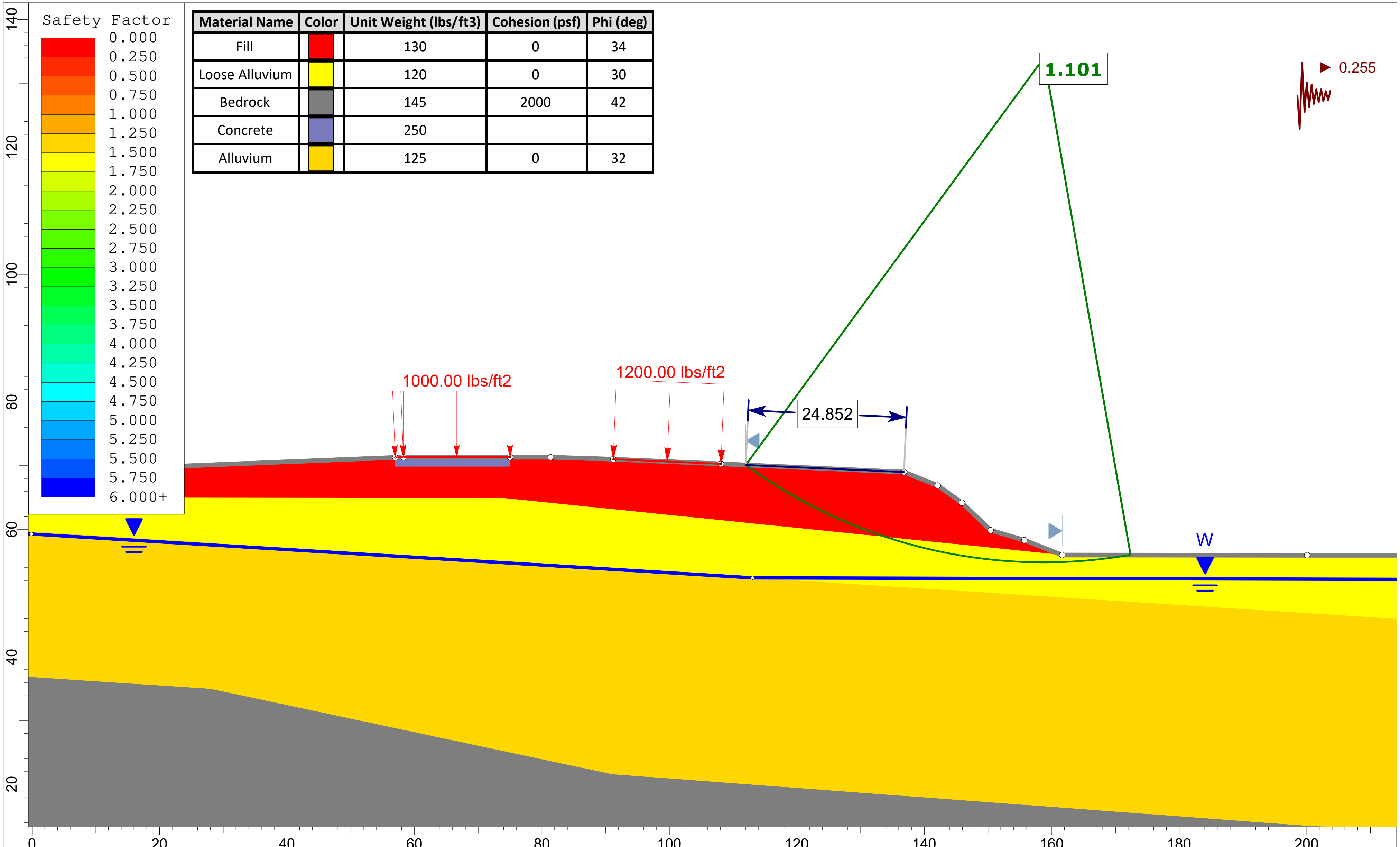


Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Fill		130	0	34
Loose Alluvium		120	0	30
Bedrock		145	2000	42
Concrete		250		
Alluvium		125	0	32

HWA GEOSCIENCES INC.
DBE/MWBE

SLIDEINTERPRET 9.023

Project	Haller South Geotechnical Investigation		
Analysis Description	Global Stability - Proposed Conditions - Well House Location		
Drawn By AHF/SKS	*Scale* 1:250	*Company*	HWA GeoSciences
Date 2/15/2024, 1:16:01 PM		*Loading Scenario*	Figure D-3 - Pseudostatic Analysis

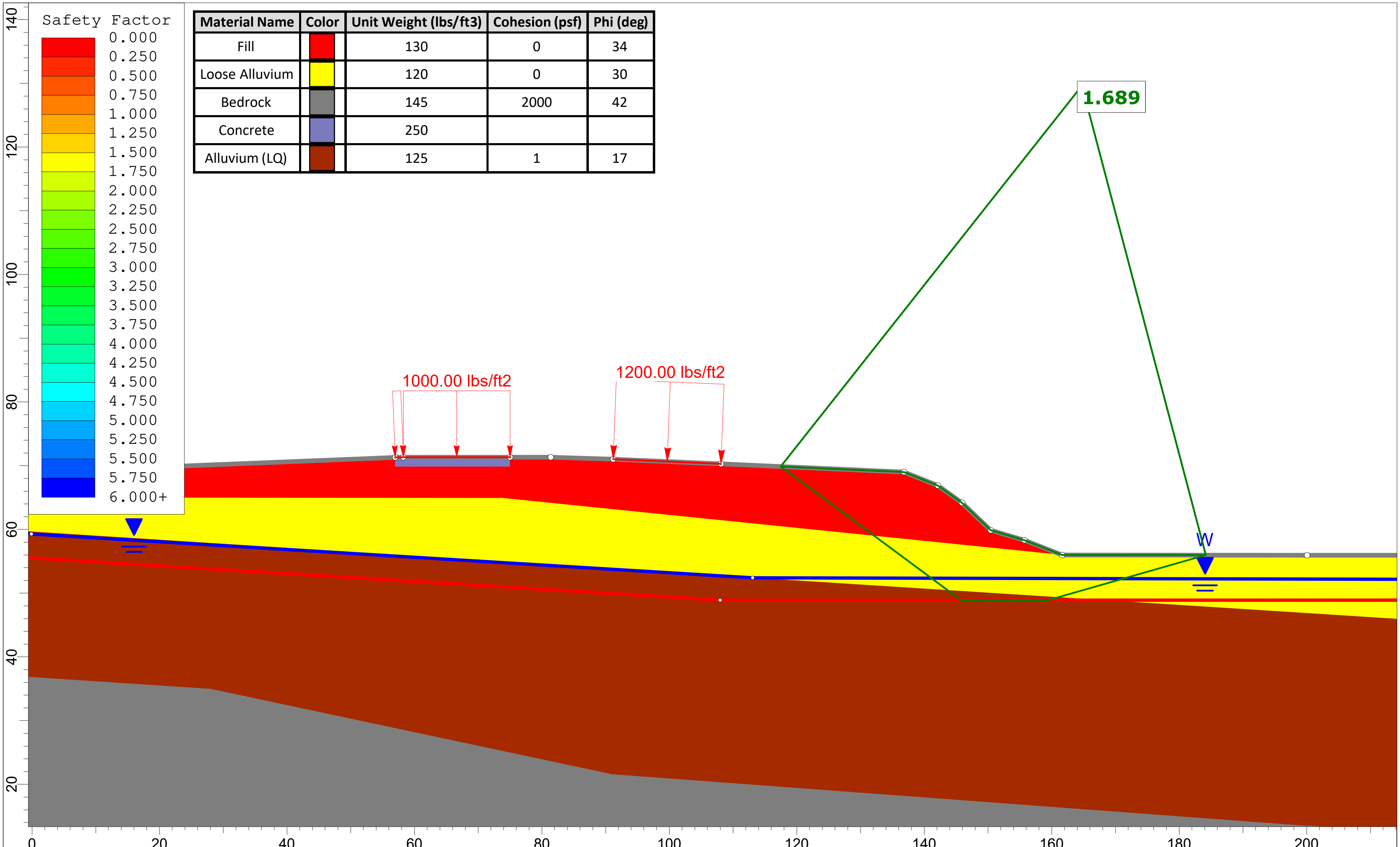


Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Fill		130	0	34
Loose Alluvium		120	0	30
Bedrock		145	2000	42
Concrete		250		
Alluvium		125	0	32

HWA GEOSCIENCES INC.
DBE/MWBE

Project	Haller South Geotechnical Investigation		
Analysis Description	Global Stability - Proposed Conditions - Well House Location		
Drawn By AHF/SKS	*Scale* 1:250	*Company*	HWA GeoSciences
Date	2/15/2024, 1:16:01 PM	*Loading Scenario*	Figure D-4 - Pseudostatic Analysis

SLIDEINTERPRET 9.023



Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Fill		130	0	34
Loose Alluvium		120	0	30
Bedrock		145	2000	42
Concrete		250		
Alluvium (LQ)		125	1	17

HWA GEOSCIENCES INC.
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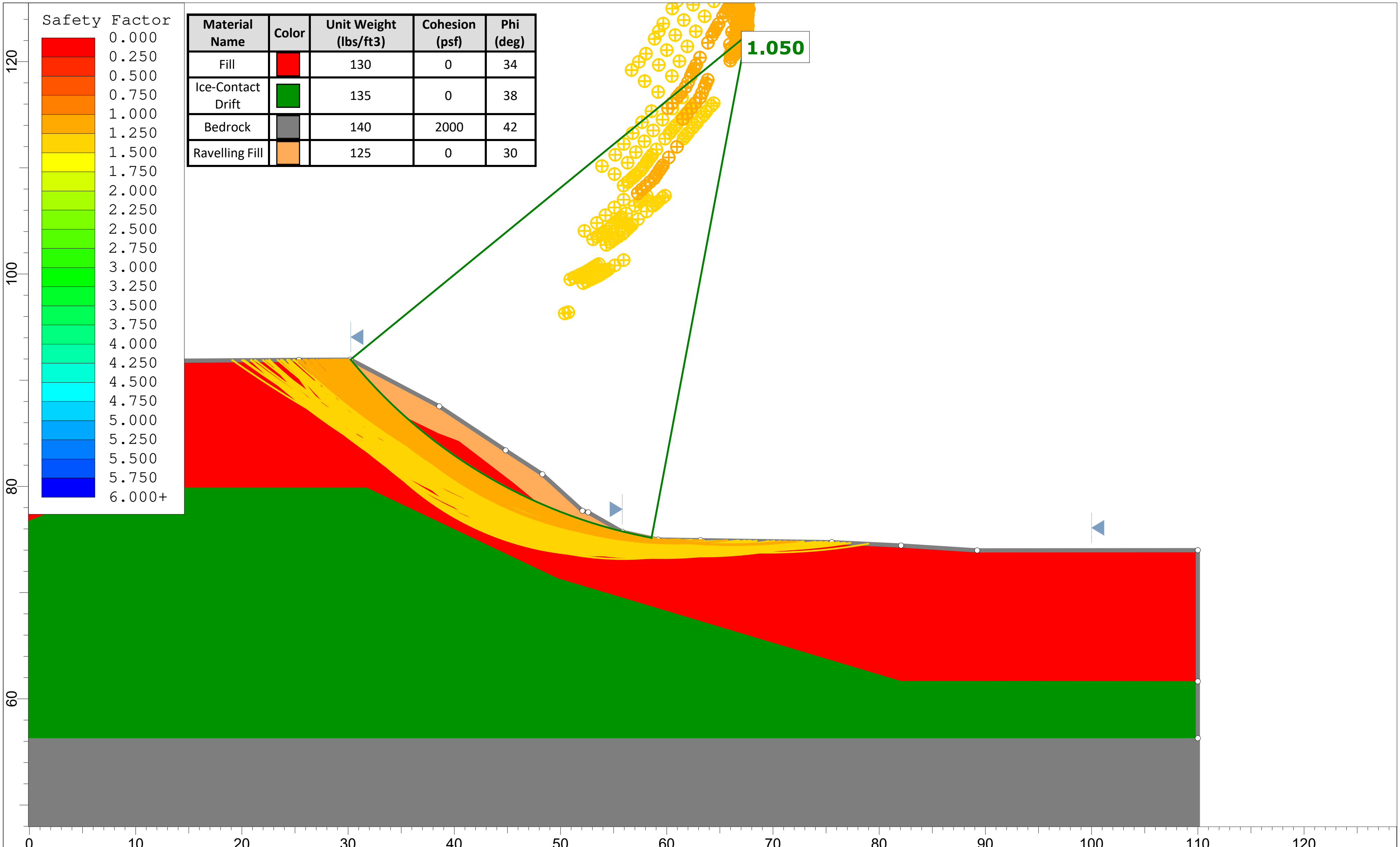
SLIDEINTERPRET 9.023

Project	Haller South Geotechnical Investigation
Analysis Description	Global Stability - Proposed Conditions - Well House Location
Drawn By	AHF/SKS
Scale	1:250
Company	HWA GeoSciences
Date	2/15/2024, 1:16:01 PM
Loading Scenario	Figure D-5 - Post Liquefaction Analysis



Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Fill		130	0	34
Ice-Contact Drift		135	0	38
Bedrock		140	2000	42
Ravelling Fill		125	0	30

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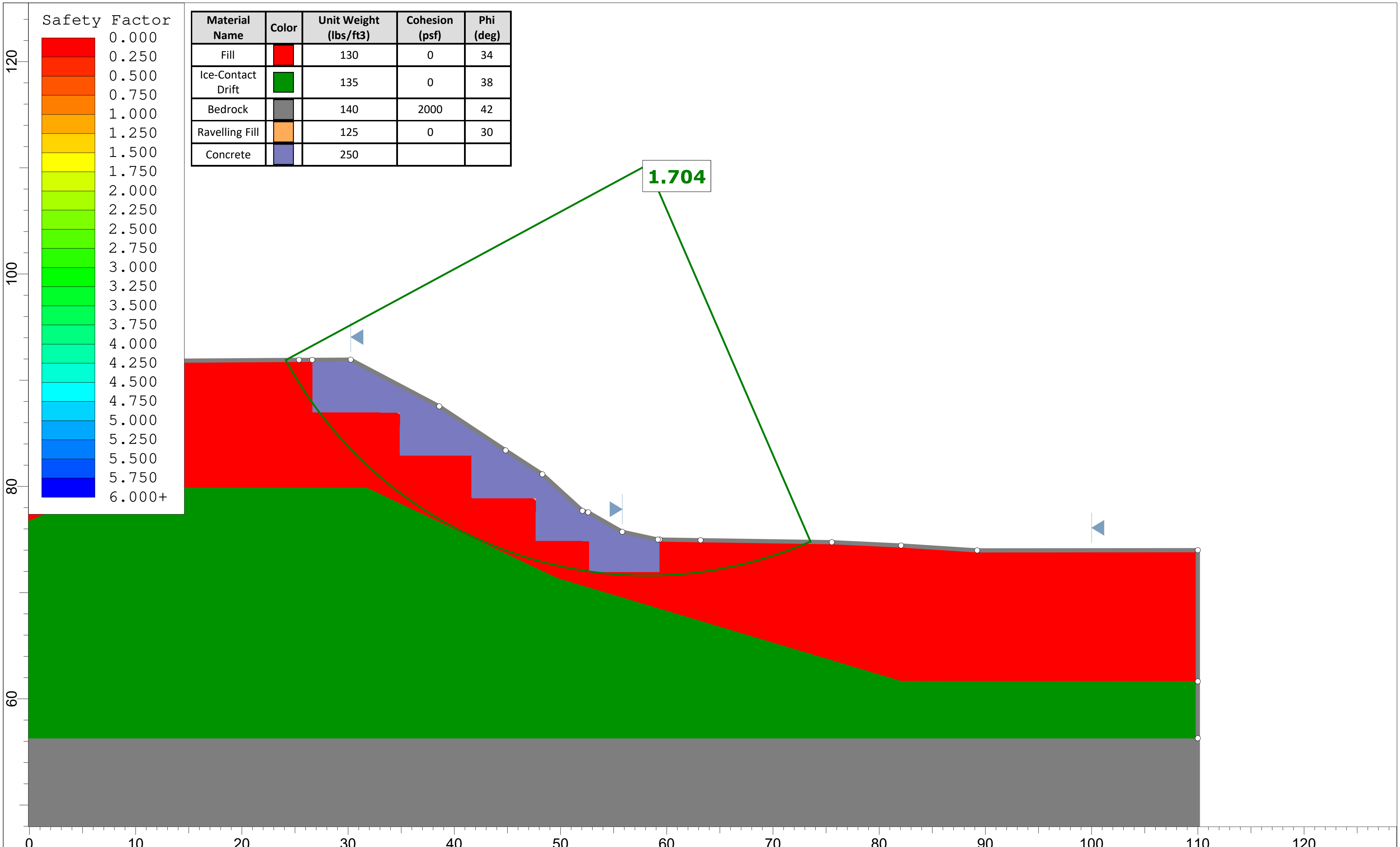
Project	Haller South Geotechnical Investigation
Analysis Description	Global Stability - Existing Conditions - Stairs Location
Drawn By	AHF/SKS
Scale	1:150
Company	HWA GeoSciences
Date	2/15/2024, 1:16:01 PM
Loading Scenario	Figure D-6 - Static Analysis

SLIDEINTERPRET 9.023

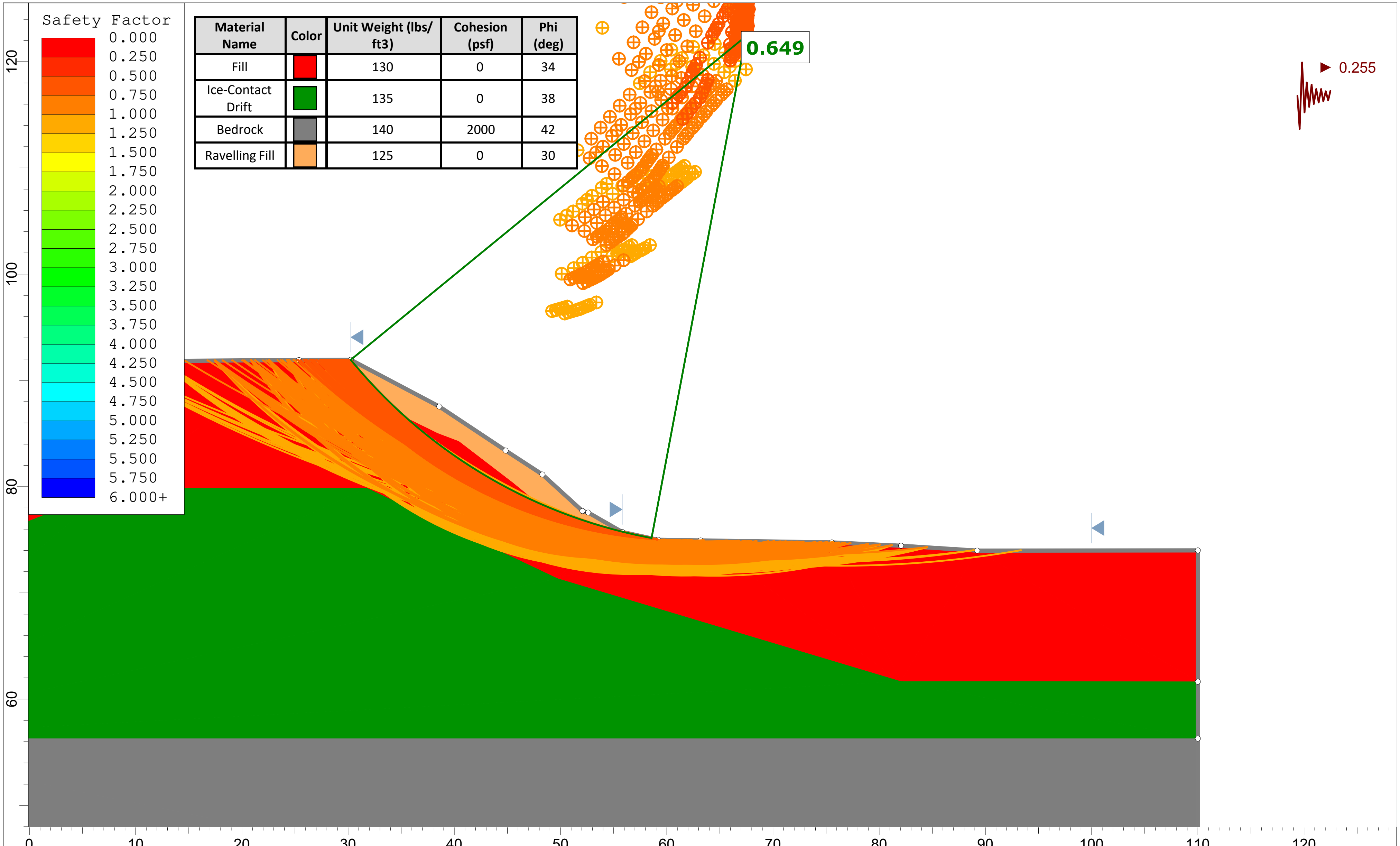


Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Fill		130	0	34
Ice-Contact Drift		135	0	38
Bedrock		140	2000	42
Ravelling Fill		125	0	30
Concrete		250		

HWA GEOSCIENCES INC.
DBE/MWBE

Project	Haller South Geotechnical Investigation		
Analysis Description	Global Stability - Proposed Conditions - Stairs Location		
Drawn By AHF/SKS	*Scale* 1:150	*Company*	HWA GeoSciences
Date	2/15/2024, 1:16:01 PM	*Loading Scenario*	Figure D-7 - Static Analysis

SLIDEINTERPRET 9.023

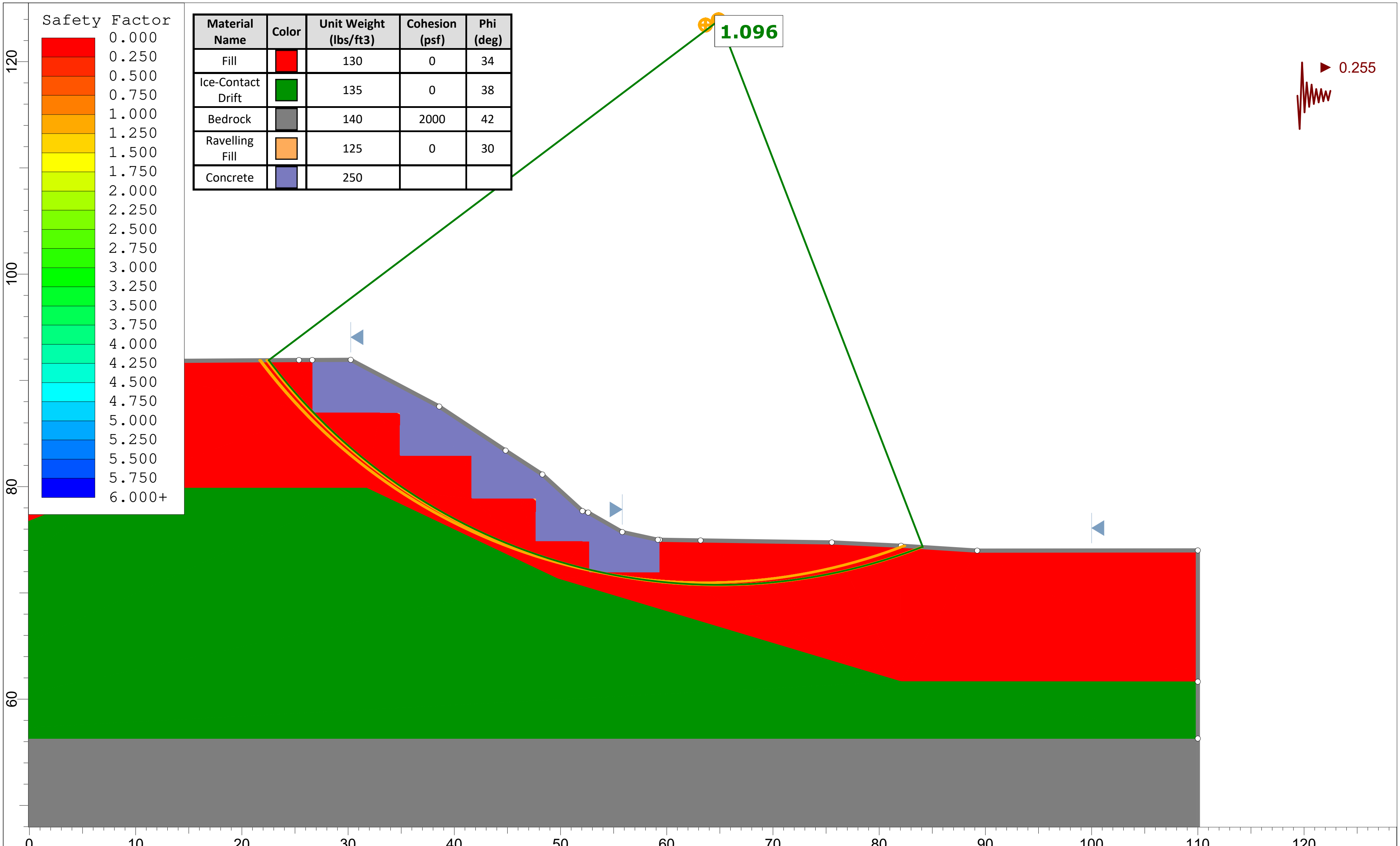


Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Fill		130	0	34
Ice-Contact Drift		135	0	38
Bedrock		140	2000	42
Ravelling Fill		125	0	30

HWA GEOSCIENCES INC.
DBE/MWBE

Project	Haller South Geotechnical Investigation		
Analysis Description	Global Stability - Existing Conditions - Stairs Location		
Drawn By AHF/SKS	*Scale* 1:150	*Company*	HWA GeoSciences
Date 2/15/2024, 1:16:01 PM		*Loading Scenario*	Figure D-8 - Pseudostatic Analysis

SLIDEINTERPRET 9.023



Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Fill		130	0	34
Ice-Contact Drift		135	0	38
Bedrock		140	2000	42
Ravelling Fill		125	0	30
Concrete		250		

HWA GEOSCIENCES INC.
DBE/MWBE

SLIDEINTERPRET 9.023

Project	Haller South Geotechnical Investigation		
Analysis Description	Global Stability - Proposed Conditions - Stairs Location		
Drawn By	AHF/SKS	*Scale* 1:150	*Company* HWA GeoSciences
Date	2/15/2024, 1:16:01 PM	*Loading Scenario*	Figure D-9 - Pseudostatic Analysis