Geotechnical Report

Green Hill School Athletic Facility

Chehalis, Washington

Prepared for

DRL Group

June 16, 2020
19461-00

HARTCROWSER

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1.0 INTRODUCTION

Hart Crowser is pleased to present this report to DRL Group summarizing the results of our field explorations and engineering analysis completed for the proposed athletic facility at Green Hill School (GHS) in Chehalis, Washington. Our work was completed in general accordance with our agreement dated February 28, 2019 and the consulting services amendment dated February 13, 2020.

The project consists of development of a playfield as well as a building for Wellness and Activities, which will include an indoor pool and other amenities. The Building is anticipated to be a single “tall” story with plan dimensions of about 130 by 300 feet. We understand the building will be steel framed with masonry façade and will have maximum column loads and wall loads of up to 175 kips and 3 kips per foot, respectively. We understand that the planned finished floor elevation is 188.67 feet (NAVD 88).

This report contains the results of our analysis and provides recommendations for design and construction of the proposed development. The first section of this report provides an overview of the project information discussed in the text. The main body of the report presents our geotechnical engineering findings and recommendations in detail.

Figures are presented at the end of the text. The location of the site is shown on Figure 1. The site exploration plan is shown on Figure 2. Supporting information is provided in the appendices. Appendix A contains the logs of our soil borings and test pits (TP). Appendix B contains the results of our laboratory testing.

2.0 SCOPE OF SERVICES

The purpose of our work was to evaluate subsurface conditions at the site and to develop geotechnical design recommendations and construction guidelines for the proposed project. Our scope of work was outlined in our proposal dated April 22, 2020, and we generally completed the following tasks.

- Reviewed relevant, readily available geologic maps that cover the site vicinity to evaluate geologic hazards and regional soil mapping.
- Conducted field explorations consisting of the following:
    - Advancing three soil borings, designated B-1, B-2, and B-3, to depths of 35 feet, 50 feet, and 25 feet below the existing ground surface (bgs), respectively.
    - Installing open standpipe monitoring wells in two of the soil borings (B-1 and B-3).
    - Excavating eight test pits to depths ranging between 7 and 14 feet bgs.

- Conducted engineering analysis to develop geotechnical design recommendations for foundations, slabs, pavements, infiltration and seismic design criteria.
- Prepared this report which contains the following information:
    - A site plan showing the locations of the explorations;
    - Logs of the borings and test pits, including the results of all field and lab testing;
    - Summary of subsurface conditions, including the impacts of those conditions on project development;
    - Estimates of the drainage characteristics of the near-surface soils;
    - Seismic design parameters per UBC;
    - Assessment of seismic hazards at the site, including the potential for seismically induced liquefaction and anticipated associated subsidence;
    - Recommendations for design of shallow foundations for the building, including allowable bearing pressures, minimum footing dimension, depth of burial, and minimum widths;
    - Estimates of total and differential settlement;
    - Assessment of general infiltration characteristics of the near-surface site soils based on grain size characteristics;
    - Recommendations for building drainage provisions and drainage considerations of a below-grade pool structure;
    - Recommendations for selection, placement, and compaction of structural fill, including an assessment of the suitability of on-site soils for reuse as fill;
    - Geotechnical recommendations for design of utilities; and
    - Geotechnical recommendations for design of pavements;
- Provided geotechnical project management and support services.

3.0 SITE CONDITIONS

3.1 Surface Conditions

The proposed project area consists of a relatively flat open area within the larger GHS campus that contains a soccer field, baseball diamond, and a few paved paths. The site of the proposed building is roughly coincident with the soccer field currently on the site, while the other features of the proposed development roughly occupy the remainder of the open space to the west of the soccer field. The open area is generally flanked by one- to two-story buildings, which occupy most of the remainder of the GHS campus.

Site grades are relatively level, but somewhat irregular, within the proposed project area. In approximately area of the proposed building (current soccer field), elevations range from approximately 190 feet above mean sea level (MSL) along the east side to approximately 189 feet MSL along the west side. Elevations within the remainder of the project site generally range from approximately 186 feet near the north end to 193 feet MSL near the south end. However, localized areas of higher or lower elevations are present.

3.2 Geologic and Soil Mapping

3.2.1 Geologic Mapping

The geology of the site is mapped as “Modified Land” (fill), described as rubble of northern sourced cobbles and sand, locally sourced and redistributed to modify topography (Sadowski et al. 2018). Underlying the modified land deposits, the mapping indicates the GHS campus is underlain by older alluvial (terrace) deposits to the east and fine-grained alluvial deposits to the west, with the contact between the two trending roughly northwest-southeast and cutting through roughly the center of the GHS campus. The more recent deposits are mapped as overlying the Eocene Lincoln Creek Formation at depth.

The older alluvial deposits are described as terrace deposits consisting of pebbles, cobbles, sand, silt, clay, and boulders in varying amounts. They are described as light tannish gray to dark brown, fresh to lightly weathered, except where streams have incorporated older deposits; typically, well rounded and well sorted, and not compacted or cemented (Sadowski et al. 2018). The fine-grained alluvial deposits are described as overbank material generally consisting of tannish gray to light brown, fresh to lightly weathered, not compacted or cemented, silt to very fine sand. The fine-grained alluvial deposits are described as generally thin and underlain by recent alluvial deposits ranging from gravel to clay. The Lincoln Creek formation is described as moderately to poorly lithified siltstone to very fine sandstone.

3.2.2 Soils Mapping

Soils within the project area mapped primarily as Lacamas silt loam, 0 to 3 percent slopes (USDA 2020). The Lacamas soils are described as silt loam to 17 inches bgs, silty clay to 27 inches bgs, and clay to 60 inches bgs occurring on flood plains and terraces. They are poorly drained with an estimated depth to water of approximately 12 to 18 inches and very low hydraulic conductivity (approximately 0 inches per hour) in the most restrictive layer.

3.3 Previous Studies

Previous explorations completed toward the west end of the GHS campus (nearby, but outside of the current project area) generally encountered mixed fill overlying native clay, sand, silty sand, gravel, and silty gravel (Creative Engineering Options 2006; GeoEngineers 2011). The fill is generally described as loose to medium dense/soft to medium stiff sand, silty sand, and clay, as well as occasional debris (brick fragments, concrete/asphalt rubble, and charcoal) extending to approximately 4 to 10 feet bgs. The native soils are generally described as up to approximately 6 feet of medium stiff lean to fat clay overlying loose to very dense sand, silty sand, gravel, and silty gravel. The granular soils extended to the base of the explorations, approximately 36.5 feet bgs. Groundwater was encountered in these explorations between approximately 6 and 11 feet bgs.

3.4 Subsurface Conditions

3.4.1 General

Soil conditions interpreted from geologic maps, previous subsurface studies at the site, and our explorations, in conjunction with soil properties inferred from field observations and laboratory tests, formed the basis for the conclusions and recommendations provided in this report.

We completed field explorations at the site by advancing three borings (designated B-1 through B-3) to depths between approximately 26.5 and 51.5 feet bgs. In addition to the borings, we excavated eight test pits (designated TP-1 through TP-8) to depths between approximately 6 and 14 feet bgs. Two groundwater monitoring wells, MW-1 and MW-2, were installed at the locations of B-1 and B-3, respectively. The locations of the explorations are shown on Figure 2.

Appendix A describes our field exploration procedures and presents field data and logs. Appendix B describes our laboratory testing procedures and results.

Based on the results of borings, test pits, and visual field and laboratory observations of the site soils, the site is generally blanketed by approximately 5 to 8 inches of topsoil and sod. Deposits of fill, and/or possible fill, were observed in all our explorations and extended between approximately 2.5 and 8 feet bgs. Underlying the surficial fill and clay soils, native soils generally consisted of medium dense to very dense clayey gravels with sand and silty sand extending to approximately 51.5 feet bgs, the deepest depth explored.

Detailed descriptions of the soils encountered are provided below.

3.4.2 Topsoil

We encountered topsoil/sod in all our explorations. The thickness of the topsoil ranged from approximately 5-inches thick in TP-1 to approximately 8-inches thick in TP-4, TP-5, and TP-7.

3.4.3 Surficial Fill and Clay Soils

All our explorations encountered material interpreted as fill and/or possible fill below the topsoil. Immediately below the topsoil, the fill materials consisted of generally loose to occasionally medium dense sand, sand with silt, silty sand, poorly graded gravel with sand, poorly graded gravel with silt and sand, and silty/clayey gravel. The fill contained debris including brick, concrete, rebar, wire, plastic, and charcoal. In TP-6, the debris included large concrete blocks that were many feet in length. In TP-5, the fill immediately below the topsoil consisted of clay with sand that contained shattered glass and charcoal, and in TP-8 we encountered minor brick debris in lean clay at approximately 8 feet bgs.

In borings B-1 through B-3, and test pits TP-1, TP-2, TP-3, and TP-5, we encountered fine-grained soils interpreted as possible fill based on the deep debris found in TP-8 and softer soil horizons found at depth in the fine-grained soils. In TP-7 the fine-grained material was interpreted as native because of a buried topsoil mat observed at approximately 5 feet above the clay.

The fine-grained soils consisted of lean to fat clay. Standard penetration test (SPT) N-values within the clay soils were generally 3 blows per foot (bpf) in samples taken at 2.5 feet bgs indicating a generally soft consistency. Moisture contents in the clay soils ranged from approximately 23 to 39 percent. Three Atterberg limits tests conducted on the fine-grained soils yielded plastic limits ranging from approximately 22 to 26 percent, liquid limits ranging from approximately 34 to 68 percent, and plasticity indices ranging from approximately 12 to 42 to percent. These limits indicate that the fine-grained soils on the site range from lean to fat clay.

3.4.4 Older Alluvium (Terrace Deposits)

In all of our borings and most of the test pit explorations (TP-1 through TP-5, and TP-8), we encountered clayey gravel with sand, silty sand, and poorly graded gravel with silt and sand beneath the surficial fill and clay soils. In our test pit explorations, the gravels within the upper approximately 5 to 10 SPT N-values in these materials in the upper portion of the formation, from approximately 5 to 10 feet bgs ranged from 14 to 31 bpf, indicating a generally medium dense relative density. Below approximately 15 feet bgs, the SPT N-values in this material ranged from approximately 33 to greater than 50 bpf indicating a generally dense to very dense relative density. The sample from approximately 50 feet bgs in boring B-2, was laminated silty sand with only fine sand and may represent the top of the underlying Lincoln Creek formation.

Moisture contents in the older alluvial deposits ranged from approximately to 11 to 57.5 percent. The highest moisture contents came from wet samples of silty sand from our test pit explorations where minor to moderate seepage was observed. Fines content analyses on six samples of the clayey gravels with sand from between approximately 5 and 10 feet bgs yielded fines contents of between approximately 19 and 37 percent. Fines content analyses on two samples of silty sand from between approximately 10 and 13 feet bgs yielded a fines content of approximately 15 percent. One Atterberg limits test conducted on the portion of a gravel sample from 7.5 feet bgs in boring B-1 yielded a plastic limit of 26 percent and a liquid limit of 50 percent indicating that the fines fraction of the gravelly soils is generally clayey.

One grain size analysis conducted on a sample from approximately 7 feet bgs from TP-2 yielded approximately 26 percent fines, 39 percent sand, and 35 percent gravel. However, prior to the test, the sample was observed to have cobbles and a high percentage of gravel that slacked during the test process. Therefore, we consider this sample to be gravel, and also indicate that many of the gravels/cobbles are highly weathered, have minor cementation, and/or the potential for slaking.

3.4.5 Groundwater

Mud rotary drilling techniques do not allow for direct measurements of groundwater levels at the time of drilling. However, we encountered minor to moderate seepage in our test pit excavations between approximately 9.5 and 13 feet bgs. Additionally, water levels in the two monitoring wells were between approximately 4.5 and 6 feet bgs at the time of our departure and following manual bailing. For this project, we recommend using a design groundwater elevation of 4 feet bgs. This corresponds to an approximate elevation of 184.6 feet (NAVD 88).

Signs of groundwater (e.g., mottling) were observed in samples above the measured water levels; therefore, seasonal high groundwater levels may be slightly higher than those identified at the time of our explorations.

3.5 Geologic and Seismic Hazards

3.5.1 Seismic Design Parameters

The 2018 International Building Code (IBC) and associated *Minimum Design Loads for Buildings and Other Structures* (American Society of Civil Engineers [ASCE] 7-16) will be adopted in Washington on November 1, 2020. As such, if the development package is submitted after this date, design parameters from the most current code will be needed. Therefore, we have provided parameters from the current state of Washington code (based on 2015 IBC and ASCE 7-10) for submittals prior to November 1, 2020, and parameters from the most recent code for submittals after November 1, 2020.

We evaluated potential seismic shaking at the site using data obtained from the U.S. Geological Survey (USGS) Seismic Design Maps (USGS 2018). The expected peak bedrock acceleration having a 2 percent probability of exceedance in 50 years (2,475-year return period) is 0.494g for the ASCE 7-10 code and 0.517g for the ASCE 7-16 code. This value represents the peak acceleration on bedrock beneath the site and does not account for ground motion amplification due to site-specific effects. The peak ground acceleration (PGA) is determined by applying a site class factor to the peak bedrock acceleration. The PGA accounting for site amplification is PGA_M = 0.497g for ASCE 7-10 and PGA_M = 0.568g for ASCE 7-16. Refer to Section 3.5.2 Site Classification for a discussion of ground motion amplification.

We obtained a deaggregation of the seismic sources contributing to the expected peak bedrock acceleration shown above from the USGS Unified Hazard Tool (USGS 2018). Seismic sources contributing to this potential ground shaking include the shallow crustal faults and the Cascadia Subduction Zone (CSZ) megathrust and intraplate sources. The data indicated that the “mean source” for shaking at the site at all potential periods of interest (0.0 to 2.0) is a magnitude 7.7 earthquake with an epicenter approximately 58.5 kilometers from the site for the ASCE 7-10 code and a magnitude 7.9 earthquake with an epicenter approximately 53.6 kilometers from the site for the ASCE 7-16 code.

3.5.2 Site Classification

The “Site Class” is a designation used to quantify ground motion amplification. The classification is based on the stiffness of the upper 100 feet of a site, as evaluated with SPT or shear wave velocity data. For our analysis, SPT N-values were extrapolated from the bases of our borings to a depth of 100 feet. Based on our analysis of SPT N-values, the site soils are estimated to have a shear wave velocity profile consistent with **Site Class D**, without regard for liquefaction potential.

Our analyses have identified that a liquefaction hazard is present at the site. The IBC indicates that sites where a liquefaction hazard is identified should be represented as **Site Class F** and a site-specific ground response analysis be completed to determine the response spectrum for design, unless the building period is less than 0.5 second. We understand that proposed development will consist of lightweight, one-story, wood- or steel-framed structures that are assumed to fundamental periods of less than 0.5 second, so **Site Class D** is allowed per the code. Refer to Section 4.3 Seismic Design of this report for additional discussion regarding the recommended site class value for design of structures.

3.5.3 Liquefaction

Liquefaction is a phenomenon caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles, resulting in the sudden loss of shear strength in the soil. Granular soils, which rely on interparticle friction for strength, are susceptible to liquefaction until the excess pore pressures can dissipate. Sand boils and flows observed at the ground surface after an earthquake are the result of excess pore pressures dissipating upwards, carrying soil particles with the draining water. In general, loose, saturated sand soils with low silt and clay contents are the most susceptible to liquefaction. Silty soils with low plasticity are moderately susceptible to liquefaction and softening under relatively higher levels of ground shaking. For any soil type, the soil must be saturated for liquefaction to occur.

We performed site-specific liquefaction potential analysis on the soils underlying the site using procedures outlined in Idriss and Boulanger (2014). The analysis was conducted using the data from our soil borings. We completed the liquefaction hazard analysis using the site class adjusted Maximum Considered Earthquake Geometric Mean PGA (PGA_M) from both the ASCE 7-10 and ASCE 7-16 codes. We used the PGA_M and associated earthquake magnitude from each respective code in our analysis. We also assumed that the groundwater level was 5 feet bgs.

Based on our analysis, the saturated sandy soils below the groundwater table appear susceptible to liquefaction. The analysis indicates that liquefaction-induced ground settlement of approximately less than 1 inch will likely occur. We note the maximum depth of our explorations was approximately 50 feet bgs and potentially liquefiable soils could extend deeper; however, based on the relative density of the soils encountered at that depth and based on our knowledge of the regional geology, we determined that the soil below 50 feet bgs is not liquefiable. In general, we would consider such ground settlement to have the potential to cause differential settlement approximately half the total ground settlement (0.5 inches on average).

3.5.4 Earthquake-Induced Landsliding/Lateral Spreading

Based on the gentle slope gradients at the site and surrounding areas, it is our opinion the potential for earthquake-induced landsliding and lateral spreading is low.

3.5.5 Fault Rupture

The potential impacts of fault rupture include abrupt, large, differential ground movements and associated damage to structures that might straddle a fault, such as a bridge abutment or retaining wall. The USGS maintains information on faults and associated folds in the United States that are believed to be sources of magnitude 6 or higher earthquakes during the Quaternary period (USGS, 2019). Based on our review of the USGS Interactive Fault Map, the closest faults to the site are part of the Willapa Bay fault zone (45 miles west). Due to the distance between our site and the nearest mapped faults, the risk of rupture is low.

4.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

4.1 Foundation Support Recommendations

4.1.1 General

Section 12.13.9 of the IBC states that sites where the potential for soil strength loss, due to liquefaction, exists must be designed to accommodate the effects of liquefaction unless there is negligible risk of lateral spreading, no bearing capacity loss, and differential settlements of site soils or improved site soils do not exceed one fourth of the differential settlement threshold specified in Table 12.13-3. The site soils at the proposed athletic facility meet the exception requirements; therefore, the proposed buildings may be supported by conventional spread footings overlying compacted structural backfill following suitable depths of overexcavation of the near surface soils, although the system should be capable of accommodating the anticipated settlement.

The design philosophy behind the IBC is that a building will not collapse during a design-level earthquake. However, cosmetic and functional distress will occur, and even structural distress is likely to result, potentially rendering the structure unusable until repaired or replaced. If these performance criteria are not acceptable, we should be notified so we can modify our recommendations.

The following recommendations are based on the assumption that maximum structural loads will be no greater than 175 kips for column footings and 3 kips per linear foot for continuous wall footings. If structural loads are greater, then we should be contacted to verify that our recommendations are appropriate.

4.1.2 Dimensions and Design Criteria

Isolated column footings and strip footings should be at least 24- and 18-inches wide, respectively. The bottom of perimeter footings should extend at least 18 inches below the lowest adjacent exterior grade, while interior footings should extend at least 12 inches below the base of the floor slab. The footings may be sized assuming a maximum allowable bearing pressure of 2,000 pounds per square foot (psf). This value may be increased by one-third for short-term, non-seismic loads (e.g., wind loads). No increase should be assumed for seismic loading conditions. The above bearing pressure values represent net bearing pressures; the weight of the footings and overlying backfill can be ignored in calculating footing sizes.

As mentioned previously, there is approximately 3 to 8 feet of soft and loose fill overlying the site. We would anticipate about 2 feet or more of overexcavation below footings will be necessary to achieve the recommended bearing pressure. The actual depth of overexcavation is best determined in the field during construction. Therefore, contract documents should be prepared in a manner that allows for variable amounts of overexcavation and backfill, depending on the conditions encountered. For budgeting purposes, we would recommend an initial amount of overexcavation below all footings of 3 feet and

18 inches below slabs-on-grade. Overexcavation should be performed as described on Figure 3. Backfill material should be consistent with material described in Section 7.4.2 of this report.

4.1.3 Lateral Resistance

Lateral loads on footings can be resisted by passive earth pressures on the sides of footings and by friction on bearing surfaces. We recommend that passive earth pressures be calculated using an equivalent fluid density of 250 pounds per cubic foot (pcf). We recommend using a friction coefficient 0.55 for foundations on aggregate base subgrade. The passive earth pressure and friction components may be combined, provided the passive component does not exceed two-thirds of the total. The lateral resistance values do not include safety factors.

4.1.4 Settlement

Footings that bear on new structural fill should experience “static” settlement of less than 1 inch, with differential settlement of less than half that value over a 50-foot span. As previously noted, overall seismically induced ground settlement on the order of 1 inch may occur in addition to the static settlement. Differential seismic settlement over a 50-foot span is estimated to be on the order of 1/2 inch. A total differential settlement, including static and seismic settlement, over a 50-foot span is estimated to be about 1 inch or less.

4.1.5 Foundation Subgrade Preparation

Footings may bear on structural fill that is placed and compacted as recommended herein. Prior to the placement of reinforcing steel in the footing excavations, loose or disturbed soils should be removed. If water infiltrates and pools in the excavation, the water, along with any disturbed soil, should be removed before placing the reinforcing steel. We recommend that contract documents be prepared in such a manner that the contractor is required to choose means and methods that will avoid disturbance of excavated surfaces.

We recommend that Hart Crowser observe all foundation excavations before placement of aggregate base to determine that bearing surfaces have been adequately prepared and that the soil conditions are consistent with those observed during our explorations.

4.2 Building Floor Slabs

Satisfactory subgrade support for building floor slabs supporting up to 175 psf areal loading can be obtained from a building floor slab on a minimum of 12 inches of sand and gravel structural fill prepared in conformance with Section 7.0 Earthwork Recommendations of this report. A minimum 6-inch-thick layer of clean aggregate base should be placed over the structural fill to assist as a capillary break. Aggregate base material placed directly below the slab should be 3/4 to 1 inch maximum size and have less than 5 percent fines.

Flooring manufacturers often require vapor barriers to protect flooring and flooring adhesives. Many flooring manufacturers will warrant their product only if a vapor barrier is installed according to their recommendations. Selection and design of an appropriate vapor barrier, if needed, should be based on discussions among members of the design team. Slabs should be reinforced according to their proposed use and per the structural engineer's recommendations.

4.3 Seismic Design

We have provided design parameters for both the current 2015 IBC and future 2018 IBC. We obtained the seismic hazard from the National Seismic Hazard Maps (USGS 2016) for Latitude 46.6507 and Longitude -122.9588 for the 2,475-year return period. The parameters provided in Tables 1 and 2 are appropriate for code-level seismic design.

Table 1 – Seismic Design Parameters 2015 IBC (ASCE 7-10)

Parameter	Value
Site Class	D
Spectral Response Acceleration, S_s	1.145 g
Spectral Response Acceleration, S_1	0.498 g
Site Coefficient, F_a	1.042
Site Coefficient, F_v	1.502
Spectral Response Acceleration (Short Period), S_{DS}	0.795 g
Spectral Response Acceleration (1-Second Period), S_{D1}	0.499 g
Mapped MCE_G peak ground acceleration, PGA	0.494
PGA Site Coefficient, F_{PGA}	1.006
Maximum Considered Earthquake Geometric Mean PGA, PGA_M	0.497 g

Table 2 – Seismic Design Parameters 2018 IBC (ASCE 7-16)

Parameter	Value
Site Class	D
Spectral Response Acceleration, S_s	1.17 g
Spectral Response Acceleration, S_1	0.483 g
Site Coefficient, F_a	1.032
Site Coefficient, F_v	1.817
Spectral Response Acceleration (Short Period), S_{DS}	0.805 g
Spectral Response Acceleration (1-Second Period), S_{D1}	0.585 g
Unfactored Peak Ground Acceleration, PGA	0.517 g
Site Coefficient, F_{PGA}	1.1
Maximum Considered Earthquake Geometric Mean PGA, PGA_M	0.568 g

Notes:

a. Per ASCE 7-16 Section 11.4.8, Site Class D sites with S_1 greater than or equal to 0.6g; Site Class E sites with S_s greater than or equal to 1.0g; or Site Class D or E sites with S_1 greater than or equal to 0.2g shall have a site-specific ground motion hazard analysis performed in accordance with Section 21.2 unless Exceptions are taken per Section 11.4.8.

b. Per Exception 2 of ASCE 7-16, Section 11.4.8, structures on Site Class D sites with S_1 greater than or equal to 0.2g, a ground motion hazard analysis is not required provided the value of the seismic response coefficient C_s is determined by Eq. (12.8-2) for values of $T \le 1.5T_s$ and taken as equal to 1.5 times the value computed in accordance with either Eq. (12.8-3) for $T_L \ge T > T_S$ or Eq. (12.8-4) for $T > T_L$.

As discussed previously, our findings indicate there is a potential for the site to be affected by liquefaction; therefore, a Site Class F is required by the IBC. However, in accordance with ASCE 7-10 (ASCE/SEI 2010), Site Class F soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils, may be classified without regard for liquefaction, provided the structures under design will have a fundamental period of vibration equal to or less than 0.5 second or if the liquefaction hazard has been properly mitigated. The structural engineer should verify the building fundamental period is below 0.5 second.

5.0 DRAINAGE DESIGN RECOMMENDATIONS

5.1 Temporary Drainage

During mass grading at the site, the contractor should be made responsible for temporary drainage of surface water as necessary to prevent standing water and/or erosion at the working surface. During rough and finished grading of the building site, the contractor should keep all footing excavations and building pads free of water.

5.2 Surface Drainage

The finished ground surface around buildings should be sloped away from their foundations at a minimum 2 percent gradient for a distance of at least 5 feet. Downspouts or roof scuppers should discharge into a storm drain system that carries the collected water to the existing regional stormwater system. They should not be attached to wall or footing drains. Trapped planter areas should not be created adjacent to buildings without providing means for positive drainage (i.e., swales or catch basins).

5.3 Infiltration Characteristics of Site Soils

Surficial fill soils are primarily fine-grained clay soils as such we anticipate the infiltration rate into theses soils to be low. As mentioned previously, these surficial soils are approximately 3 to 8 feet in thickness. The underlying soils consists of medium dense to dense sands and gravels. We determined the infiltration rate of onsite native soils using equations based on grain size distribution in accordance with the Stormwater Management Manual for Western Washington Section V-5.4. Using the equation developed by Massman, we determined a design infiltration rate of approximately 1.2 inches per hour. Even though the native soils appear to have an infiltration rate suitable for the design of infiltration systems, due to the design water table of 5 feet bgs, and the low permeability of the surficial fill soils, it is our opinion the use of infiltration systems is not feasible at this site.

If stormwater detention systems are proposed, then the use of closed or lined systems will be required. These systems or liners will need to be designed to resistant buoyancy forces. For design of stormwater detention systems, the groundwater level should be assumed as shallow as 2 feet below existing grade.

5.4 Pool Design

The pool shell walls should be designed to resist an at-rest soil pressure of 55 pcf acting as an equivalent fluid weight. This is assuming structural backfill in accordance with Section 7.4 of this report will be placed around the pool perimeter. We recommend a minimum 12-inch-thick layer of drain rock be placed along the base of the pool excavation and along the pool walls. The filter layer of drain rock must be wrapped in a filter fabric in accordance with Table 2 from Section 9-33.2(1) of the WSDOT Standard Specifications, in order to prevent the migration of fines.

We recommend providing hydrostatic relief to the pool by one of two methods. The first method involves installing a series of hydrostatic pressure relief valves along the bottom of the pool. The second method would require the construction of a sump beneath the pool and installing a pump sump. The sump pump could then be used to drain the drainage layer beneath the pool during maintenance periods when the pool is empty. If this approach is used, the drainage layer below the pool should include 4-inch perforated drainpipe at 25 feet on centers in addition to a perimeter drain.

The decking around the pool will consist of concrete slabs-on-grade. They should be constructed in a manner consistent with recommendations provided in Section 4.2 Building Floor Slabs of this report. We recommend that decking be structurally isolated from the pool and spa shells and the skimmer.

The pool floor should be designed in accordance with Section 4.2 Building Floor Slabs of this report. The boring logs indicate soft fill soils to a depth of 5 feet bgs in the vicinity of the planned pool. As such, we do not expect a significant amount of overexcavation; however, soft soils encountered in the pool footprint should be removed to the more competent native sands and gravels. Given the close proximity of the pool bottom to the water table, it is anticipated that some dewatering in accordance with Section 7.3.3 Dewatering of this report will be required such that the bottom of the excavation is not disturbed. The pool will need to be underlain by a drainage system including perforated cross drains in accordance with Section 5.5 Subsurface Drainage of this report to prevent heave of the pool when the pool is emptied for maintenance or other reasons.

In lieu of providing hydrostatic pressure relief, the structural engineering may provide a concrete section at the bottom of the pool that will be thick enough to resist hydrostatic pressures. We recommend using a design groundwater elevation of 184.6 feet (NAVD 88).

Once the final pool design is complete, we should be allowed to review and modify our recommendations as necessary.

5.5 Subsurface Drainage

We estimate that the seasonal high groundwater table may rise to within 4 feet of the existing ground surface. As such, we recommend installing a perimeter footing and subslab drainage system at the proposed buildings. Additionally, if trapped planters or adverse grades are created adjacent to buildings, then the use of footing drains is even more important.

The footing drainage system should consist of a filter fabric-wrapped, drain rock-filled trench that extends at least 12 inches below the lowest adjacent grade (i.e., crawlspace or slab subgrade elevation). A perforated pipe should be placed at the base to collect water that gathers in the drain rock. The drain rock and filter fabric should meet specifications outlined in Section 7.4 Structural Fill and Backfill.

The subslab drainage systems should consist of a minimum 8-inch layer of drain rock beneath the entire slab. The drain rock should be underlain by a geotextile filter fabric. We recommend using 4-inch perforated collector pipes embedded within the drain rock layer with a spacing no greater than 30 feet on center.

The discharge for subsurface drainage systems should not be tied directly into the stormwater drainage system, unless mechanisms are installed to prevent backflow. The use of a sump pump may be required.

5.6 Bioretention Planters

We understand the new drainage system will include bioretention planters. Information concerning the bioretention planters was provided by the DRL group via email on June 12, 2020. Based on our review of the provided information, the planters are a drainage swale with slopes of 3H:1V or flatter with an approximate 8-foot base. The planters consist of 2 inches of mulch on top of 18 inches of Biosoil along the side slopes. The base cross section consists of 2 inches of mulch on top of 18 inches of Biosoil on top of 12 inches of drain rock on top of an 8-inch underdrain. We understand the design groundwater elevation is approximately even with the base of the bioretention planter (elevation 184.6 feet NAVD) at the critical cross section.

We recommend placing an impermeable liner along the base of the bioretention planters' excavation prior to placing drain rock and Biosoil, to prevent the flow of groundwater into the bioretention planter. The impermeable liner must meet the strength requirements of Table V-1.6 of the Stormwater Management Manual for Western Washington (Ecology 2019). We have reviewed the information provided by DRL and we have determined that the bioretention planters are not at risk of failure from failure from the buoyant forces from the groundwater. If the design of the bioretention planters changes from that provided, we must be allowed to review the new design and adjust our recommendations as necessary.

The drain rock must meet the requirements of section 7.4.6 of this report.

6.0 PAVEMENT DESIGN AND CONSIDERATIONS

6.1 General

Our pavement design recommendations include options for flexible Asphaltic Concrete (AC) and rigid Portland Cement Concrete (PCC) pavement. Our design thicknesses assume that new pavements will be supported by new structural fill placed and compacted per Section 7.0 Earthwork Recommendations of this report. It is our understanding that the pavement sections will be primarily used by pedestrians, maintenance vehicles, and consistent patrols from security vehicles.

6.2 Pavement Sections

The PCC and AC pavement sections in Table 3 are minimum recommended material thicknesses. If the anticipated site traffic is different than noted above, then the recommended sections should be reevaluated.

Table 3 – PCC and AC Pavement Sections

Pavement Type	AC Thickness (inches)	Aggregate Base Thickness (inches)
PCC Pavement	6	4
AC Pavement	3	6

Due to the presence of soft surficial clay soils, we recommend that an additional 18 inches of existing fill be removed and replaced with Stabilization Material in accordance with Section 7.4 of this report.

6.3 Pavement Materials

6.3.1 Flexible AC

Flexible AC should be 1/2-inch hot mix asphalt in conformance with the specifications provided in Washington State Department of Transportation (WSDOT) Standard Specifications (WSS) 5 04 – Hot Mix Asphalt and WSS 9 03.8 – Aggregates for Hot Mix Asphalt (WSDOT 2018). The asphalt cement binder should be PG 64-22 Performance Grade Asphalt Cement, according to WSS 9-02.1(4) – Performance Graded Asphalt Binder. The AC should be placed with a minimum lift thickness of 1.5 inches and maximum thickness of 3 inches and be compacted to at least 91 percent of Rice Density of the mix, as determined in accordance with American Society for Testing and Materials (ASTM) D 2041.

6.3.2 Rigid PCC

Rigid PCC pavement should meet the specifications provided in WSS 5 05 – Cement Concrete Pavement. The PCC should have a minimum compressive strength of 4,000 pounds per square inch (psi) and nominal maximum aggregate size of 1.5 inches. The PCC should be constructed with a maximum joint spacing of 15 feet. The slabs should be interlocked at contraction joints (e.g., continuous slab with no dowels). However, dowels should be used at construction and expansion joints.

6.3.3 Aggregate Base

Imported granular material used as base aggregate (base rock) for conventional pavements should meet the criteria specified in Section 7.4 Structural Fill and Backfill of this report. The base aggregate should be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D 1557.

6.3.4 Soil Subgrade

The pavement design assumes the soil subgrade consists of previously placed engineered fill with a resilient modulus of 5,000 psi. This assumes that subgrade has been moisture conditioned and compacted in conformance with Section 7.0 Earthworks Recommendations of this report.

7.0 EARTHWORK RECOMMENDATIONS

7.1 General

Based on available information, we anticipate that earthwork will generally consist of light mass grading and excavation and backfilling for utilities and foundations. We recommend that earthwork activities be conducted in accordance with the WSS (WSDOT 2018).

7.2 Site Preparation

7.2.1 Clearing and Grubbing

Initial site preparation and earthwork operations will include clearing and grubbing, stripping, and grading to establish subgrade elevation for improvements. We estimate the depth of material to be stripped is between 4 and 8 inches (average 6 inches). Actual stripping depths should be based on field observations at the time of construction. Stripped material should be transported off-site for disposal or stockpiled for use in landscaped areas.

Trees and their root balls should be grubbed out to the depth of significant roots, which could exceed 3 to 5 feet bgs for the tall trees. Depending on the methods used to remove the root balls, considerable disturbance and loosening of the subgrade could occur during site grubbing. We recommend that soil disturbed during grubbing operations be removed to expose firm, undisturbed subgrade. The resulting excavations should be backfilled with compacted structural fill.

7.2.2 Demolition

Demolition should include complete removal of existing site improvements within areas to receive new pavements, buildings, or engineered fill. Underground utility lines or vaults encountered in areas of new improvements should be completely removed or grouted full if left in place. Any existing concrete structures should be removed if located beneath the proposed building or pavement areas.

Voids resulting from removal of pavements, sidewalks, etc. or loose soil in utility lines should be backfilled with compacted structural fill, as discussed in Section 7.4 Structural Fill and Backfill of this report. The bases of such excavations should be completed to a firm subgrade before filling, and their sides configured to allow for uniform compaction at the edges of the excavations.

Materials generated during demolition of existing improvements should be transported off site for disposal or stockpiled in areas designated by the owner. In general, these materials will not be suitable for reuse as engineered fill. However, asphalt, concrete, and base rock materials may be crushed and recycled for use as general fill. Such recycled materials should meet the specifications for imported granular material, as described in Section 7.4 Structural Fill and Backfill of this report.

7.2.3 Subgrade Preparation and Evaluation

Following stripping, demolition, site preparation, and rough grading, the suitability of the subgrade should be evaluated by proof rolling with a fully loaded dump truck or similar heavy rubber-tired construction equipment to identify any remaining soft, loose, or unsuitable areas. The proof roll should be conducted prior to placing new fill. Proof rolling should be observed by a representative of Hart Crowser who would evaluate the suitability of the subgrade and identify areas of yielding that are indicative of soft or loose soil. During wet weather or when the exposed subgrade is wet or unsuitable for proof rolling, the prepared subgrade should be evaluated by observing excavation activity and probing with a steel foundation probe. Observations and probing should be performed by Hart Crowser.

If soft or loose zones are identified during proof rolling or probing, these areas should be excavated to the extent indicated by Hart Crowser and replaced with structural fill.

If site preparation activities cause excessive subgrade disturbance, replacement with imported structural fill may be necessary. Disturbance to the subgrade should be expected if site preparation and earthwork are conducted during periods of excessive wet weather and/or when the moisture content of the surficial soil exceeds optimum.

7.2.4 Wet Soil/Wet Weather Construction

The near-surface site soils generally consist of fat to lean clay. These materials will have a moderate susceptibility to becoming disturbed when they are wet or heavily trafficked. If not carefully executed, site preparation, utility trench work, and pavement construction can create extensive soft areas, and significant repair costs can result. Earthwork planning should include considerations for minimizing subgrade disturbance.

One method for minimizing subgrade disturbance during construction is through the use of temporary haul roads and staging areas. Based on our experience, between 12 and 18 inches of imported granular material is generally required to construct staging areas and haul roads that will support typical construction traffic. However, the actual thickness will depend on the contractor’s means and methods, and accordingly, should be the contractor’s responsibility. Additionally, a geotextile fabric may be placed as a barrier between the subgrade and imported granular material in areas of repeated construction traffic to provide separation between the imported rock and native soils. The imported granular material and geotextile fabric should meet the specifications in Section 7.4 Structural Fill and Backfill of this report.

7.3 Excavation

7.3.1 General Excavation

Site soils are generally soft/loose within expected excavation depths. However, denser sand and gravel soils may be encountered in excavations that are 5 feet or greater. It is our opinion that conventional earthmoving equipment in proper working condition should be capable of making necessary general excavations for utilities, footings, and other earthwork. The earthwork contractor should be responsible for providing equipment and following procedures as needed to excavate the site soils, as described in this report. Permanent slope excavations should have a maximum gradient of 2 horizontal to 1 vertical (2H:1V).

7.3.2 Temporary Excavation Stability

Due to the granular nature of the site soils, even shallow excavations will have a high susceptibility to sloughing, raveling, or caving. Open excavation techniques may be used for temporary excavations above the groundwater table. For planning purposes only, we expect that cut slopes may be excavated at an angle of 1H:1V or flatter. However, because of the variables involved, actual slope angles required for stability in temporary cut areas can only be estimated before construction. We recommend that stability of the temporary slopes used for construction be the responsibility of the contractor, since the contractor is in control of the construction operation and is continuously at the site to observe the nature and condition of the subsurface.

All temporary soil cuts associated with site excavations should be adequately sloped back to prevent sloughing and collapse, in accordance with Department of Occupational Safety and Health (DOSH) Chapter 296-155 Washington Administrative Code (WAC) Part N Excavation, Trenching, and Shoring Occupational Safety and Health Administration (OSHA) guidelines.

The stability and safety of cut slopes depend on a number of factors, including:

- Type and density of the soil;
- Presence and amount of any seepage;
- Depth of cut;
- Proximity and magnitude of the cut to any surcharge loads, such as stockpiled material, traffic loads, or structures;
- Duration of the open excavation; and
- Care and methods used by the contractor.

According to DOSH guidelines, we interpret the existing site soils as Type C.

It is the responsibility of the contractor to ensure the excavation is properly sloped or braced for worker protection, in accordance with DOSH guidelines. To assist with this effort, for planning purposes only, we make the following recommendations regarding temporary excavation slopes.

- Protect the slope from erosion with plastic sheeting for the duration of the excavation to minimize surface erosion and raveling.
- Limit the maximum duration of open excavation to the shortest time period practicable.
- Place no surcharge loads (equipment, materials, etc.) within 10 feet of the top of any excavation or slope.

More restrictive requirements may apply, depending on specific site conditions, which should be continuously assessed by the contractor.

If temporary sloping is not feasible due to site spatial constraints, excavations could be supported by internally braced shoring systems, such as a trench box or other temporary shoring. There are a variety of options available. We recommend the contractor be responsible for selecting the type of shoring system to use. We note that box shoring is a safety feature used to protect workers and does not prevent caving. If the excavations are left open for extended periods of time, caving of the sidewalls may occur. The presence of caved material will limit the ability to properly backfill and compact the trenches. The voids between the box shoring and the sidewalls of the trenches should be properly filled with sand or gravel before caving occurs.

7.3.3 Dewatering

Groundwater is expected to be encountered at approximately 5 feet bgs. Construction of utilities and other improvements that extend below groundwater levels will require dewatering and shoring programs capable of adapting to varied soil and groundwater conditions. We anticipate that water will have a low to moderate flow rate, although zones of sandy soils may present rapid water flow. Significant dewatering efforts may be required for the pool installation. The contractor shall be prepared to provide shoring and dewatering systems that are capable of adapting to varied soil and groundwater conditions. In addition to safety considerations, running soil, caving, or other loss of ground will increase backfill volumes and can result in damage to adjacent structures or utilities.

Due to low to moderate seepage observed while excavating test pits, the use of pumping from sumps within excavations is expected to be feasible for trench dewatering and dewatering of the area below the planned pool.

We anticipate that the base of excavations will be soft and/or unstable if groundwater is present or within a few feet of the base of the trenches. If that is the case, we recommend placing stabilization material at the base of excavations. Stabilization material should be placed to a minimum thickness of 12 inches, or as needed to provide an adequate working surface and should meet the criteria discussed in Section 7.4 Structural Fill and Backfill of this report. The use of a geotextile separation fabric may be necessary below stabilization material to help prevent the stabilization material from pushing into the unstable base materials.

7.4 Structural Fill and Backfill

Structural fill should be considered to include subgrade soils beneath buildings, foundations, slabs, and pavements and in other areas intended to support structures or within the influence zone of structures.

Fill should only be placed over a subgrade that has been prepared in conformance with the prior sections of this report. A variety of material may be used as structural fill at the site. However, all material used as structural fill should be free of organic matter or other unsuitable materials and should meet specifications provided in the WSS (WSDOT 2018). A brief characterization of some of the acceptable materials and our recommendations for their use as structural fill are provided below. All materials should be placed and compacted in lifts with maximum uncompacted thicknesses and relative densities, as recommended in the tables that follow.

7.4.1 On-Site Soils

Due to the moist, soft nature of the on-site near-surface fill soils, we recommend that these *in situ* soils not be used as structural fill, unless extended periods of hot, dry weather are forecast, which would allow for extensive moisture conditioning (e.g., drying) of the soils and the subgrade. Topsoil and organic-rich soils are also not suitable for structural fill.

On-site, near-surface soils that might be used for fills generally consist of clayey sand and gravel. These soils are sensitive to moisture and will require significant moisture conditioning before they can be used. If properly moisture conditioned (i.e., dried) this material may be used as structural fill, provided that debris, organic materials, and particles over 6 inches in diameter are removed and it otherwise meets the specifications provided in WSS 9 03.14(3) – Common Borrow.

7.4.2 Imported Select Structural Fill

Imported granular material used as structural fill should be pit or quarry run rock, crushed rock, or crushed gravel and sand and should meet the specifications provided in WSS 9 03.9(1) – Ballast, WSS 9 03.14(1) – Gravel Borrow, or WSS 9 03.14(2) – Select Borrow. However, the imported granular material should also have a maximum size of 2 inches, be angular and fairly well graded between coarse and fine material, have less than 5 percent by dry weight passing the U.S. Standard No. 200 mesh sieve, and have at least two mechanically fractured faces.

7.4.3 Aggregate Base

Imported granular material used as aggregate base (base rock) beneath pavements should be clean, crushed rock or crushed gravel and sand that is fairly well graded between coarse and fine. The base aggregate should meet the specifications provided in WSS 9 03.9 – Aggregates for Ballast and Crushed Surfacing, depending upon application. For use beneath general building slabs, the base rock should also meet the gradation of WSS 9 03.9(3) – Crushed Surfacing for "Base Course," although should have less than 5 percent by dry weight passing a U.S. Standard No. 200 mesh sieve.

For use beneath pavements or footings, the aggregate base should have a maximum particle size of 1 or 1.5 inches, while for use beneath buildings or sidewalk slabs should have a maximum particle size of 0.75 or 1 inch.

7.4.4 Trench Backfill

Trench backfill placed beneath, adjacent to, and for at least 12 inches above utility lines (i.e., the pipe zone) should consist of well graded granular material with a maximum particle size of 1 inch and should meet the specifications provided in WSS 9 03.12(3) – Gravel Backfill for Pipe Zone Bedding and the pipe manufacturer.

Within pavement and slab subgrades, the remainder of the trench backfill up to the subgrade elevation can consist of the above 1-inch material or of granular material with a maximum particle size of 2.5 inches, less than 10 percent by dry weight passing the U.S. Standard No. 200 mesh sieve, and meeting the specifications provided in WSS 9 03.19 – Bank Run Gravel for Trench Backfill.

7.4.5 Stabilization Material

Imported material that is placed as a stabilization layer for haul roads or staging areas should consist of a clean, angular, crushed rock, such as ballast or quarry spalls. The material should have a maximum particle size of 4 inches, a nominal size between 2 and 4 inches, less than 5 percent by dry weight passing the U.S. Standard No. 4 mesh sieve, and at least two mechanically fractured faces. The material should be free of organic matter and other deleterious material.

Material meeting the gradations of WSS 9-03.9(2) – Shoulder Ballast, WSS 9 03.12(1)B – Gravel Backfill for Foundations (Class B), WSS 9-03.12(5) – Gravel Backfill for Drains, WSS 9-13.1(2) – Light Loose Riprap, WSS 9-03.12(5) – Gravel Backfill for Drywells, or WSS 9-13.6 – Quarry Spalls is generally acceptable for use. Stabilization material should be placed in lifts between 12 and 18 inches thick and be compacted to a well-keyed condition with a smooth drum roller without using vibratory action.

Stabilization material should be separated from the base of soft or fine-grained subgrades with a layer of subgrade geotextile that meets the specifications provided in WSDOT SS 9-33.2(1) Table 3 – Geotextile for Separation or Soil Stabilization. The geotextile should be installed in conformance with the specifications provided in WSS 2-12 – Construction Geosynthetic.

7.4.6 Drain Rock

Drain rock used for subsurface drainage systems should meet the specifications provided in WSS 9 03.12(4) – Gravel Backfill for Drains. The drain rock should be wrapped in a geotextile fabric that meets the specifications provided in WSS 9 33.2 for drainage geotextiles. The geotextile should be installed in conformance with the specifications provided in WSS 2 12 – Construction Geosynthetic.

7.5 Fill Placement and Compaction

Structural fill should be placed and compacted in accordance with the following guidelines.

- Place fill and backfill on a prepared subgrade that consists of firm, inorganic native soils or approved structural fill.

- Place fill or backfill in uniform horizontal lifts with a thickness appropriate for the material type and compaction equipment. Table 4, below, provides general guidance for lift thicknesses.

Table 4 – Guidelines for Uncompacted Lift Thickness

Compaction Equipment	Guidelines for Uncompacted Lift Thickness (inches)		
	On-Site Soil	Granular and Crushed Rock Maximum Particle Size ≤ 1½ inch	Crushed Rock Maximum Particle Size > 1½ inch
Plate Compactors and Jumping Jacks	4 – 8	4 – 8	Not Recommended
Rubber-Tire Equipment	6 – 8	10 – 12	6 – 8
Light Roller	8 – 10	10 – 12	8 – 10
Heavy Roller	10 – 12	12 – 18	12 – 16
Hoe Pack Equipment	12 – 16	18 – 24	12 – 16

Note:
The above table is based on our experience and is intended to serve as a guideline. The information provided in this table should not be included in the project specifications.

- Use appropriate operating procedures to attain uniform coverage of the area being compacted.
- Place fill at a moisture content within approximately 3 percent of optimum as determined in accordance with ASTM D 1557. Moisture condition fill soil to achieve uniform moisture content within the specified range before compacting. Compact fill to the percent of maximum dry densities as noted in Table 5.
- Do not place, spread, or compact fill soils during freezing or unfavorable weather conditions. Frozen or disturbed lifts should be removed or properly recompacted prior to placement of subsequent lifts of fill soils.

Table 5 – Fill Compaction Criteria

Fill Type	Percent of Maximum Dry Density Determined in Accordance with ASTM D 1557		
	0 – 2 Feet Below Subgrade	>2 Feet Below Subgrade	Pipe Bedding and Pipe Zone
Mass Fill: fine-grained soils	92	90	-----
Mass Fill: granular materials	95	92	-----
Aggregate Base	95	95	-----
Trench Backfill	95	92	90
Nonstructural Trench Backfill	90	88	-----
Nonstructural Zones	90	88	90

Note:
“Nonstructural” areas are only located in landscaping zones, where the potential for localized trench settlement is acceptable to the owner.

During structural fill placement and compaction, a sufficient number of in-place density tests should be completed by Hart Crowser to verify that the specified degree of compaction is being achieved. For structural fill with more than 30 percent retained on the 3/4-inch sieve, Hart Crowser should visually verify proper compaction with a proof roll or other methods.

8.0 UTILITY CONSTRUCTION CONSIDERATIONS

In general, we recommend that utility trench cut design be the contractor's responsibility. For shallow trench excavations less than 4 feet deep, open cutting is not prohibited. Temporary shoring may be necessary if deeper excavation is required for utility placement or if the soils are unstable. The contractor should verify the condition of the side slopes during construction and lay back trench cuts as necessary to conform to current standards of practice. We can provide additional recommendations, as required.

8.1.1 Utility Bedding and Trench Backfill

For bedding and trench backfill materials, all minimum dry densities recommended are a percentage of the modified Proctor maximum dry density, as determined by the ASTM D1557 test procedure. We recommend the following for bedding and trench backfill materials:

- Use at least 6 inches of bedding for all pipe utilities, consisting of well-graded sand and gravel with less than 3 percent material passing the U.S. No. 200 mesh sieve based on the minus 3/4-inch fraction. Bedding material should be compacted to a firm non-yielding condition.

- The recommended bedding materials can be used as backfill around the pipe utilities (pipe zone backfill). Extend pipe zone backfill to at least the top of the utility pipe.

- For bedding material beneath manholes, use 6 inches of imported structural fill (or acceptable on-site material) that consists of well-graded sand and gravel with less than 3 percent material passing the U.S. No. 200 mesh sieve based on the minus 3/4-inch fraction. Compact the bedding material to 90 percent.

- Provide a firm, non-yielding, and stable subgrade for excavations for underground structures.

- Evaluate utilities that extend below the groundwater table for the potential to float out of the ground during high groundwater levels.

Deeper utilities may require dewatering well points to obtain a suitable working base. The contractor may elect to place a geotextile fabric at the base of the excavation to help create a suitable working surface.

9.0 CONSTRUCTION OBSERVATIONS

Satisfactory foundation and earthwork performance depends to a large degree on quality of construction. Sufficient monitoring of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. Subsurface conditions observed during construction should be compared with those encountered during subsurface explorations. Recognition of changed conditions often requires experience; therefore, Hart Crowser or their representative should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated.

We recommend that Hart Crowser be retained to monitor construction at the site to confirm that subsurface conditions are consistent with the site explorations and to confirm that the intent of project plans and specifications relating to earthwork, foundation, and pavement construction are being met. In particular, we recommend the foundation and building subgrades, infiltration system subgrade, pavement subgrade, and compaction of structural fill and aggregate bases be observed and/or tested by Hart Crowser.

10.0 LIMITATION

We have prepared this report for the exclusive use of Covenant Real Estate Group and their authorized agents for the proposed Green Hill School Athletic Facility in Chehalis, Washington. Our work was completed in general accordance with our Services Agreement dated February 28, 2019. Our report is intended to provide our opinion of geotechnical parameters for design and construction of the proposed project based on exploration locations that are believed to be representative of site conditions. However, conditions can vary significantly between exploration locations and our conclusions should not be construed as a warranty or guarantee of subsurface conditions or future site performance.

Within the limitations of scope, schedule, and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering in this area at the time this report was prepared. No warranty, express or implied, should be understood.

Any electronic form, facsimile, or hard copy of the original document (email, text, table, and/or figure), if provided, and any attachments are only a copy of the original document. The original document is stored by Hart Crowser and will serve as the official document of record.

11.0 REFERENCES

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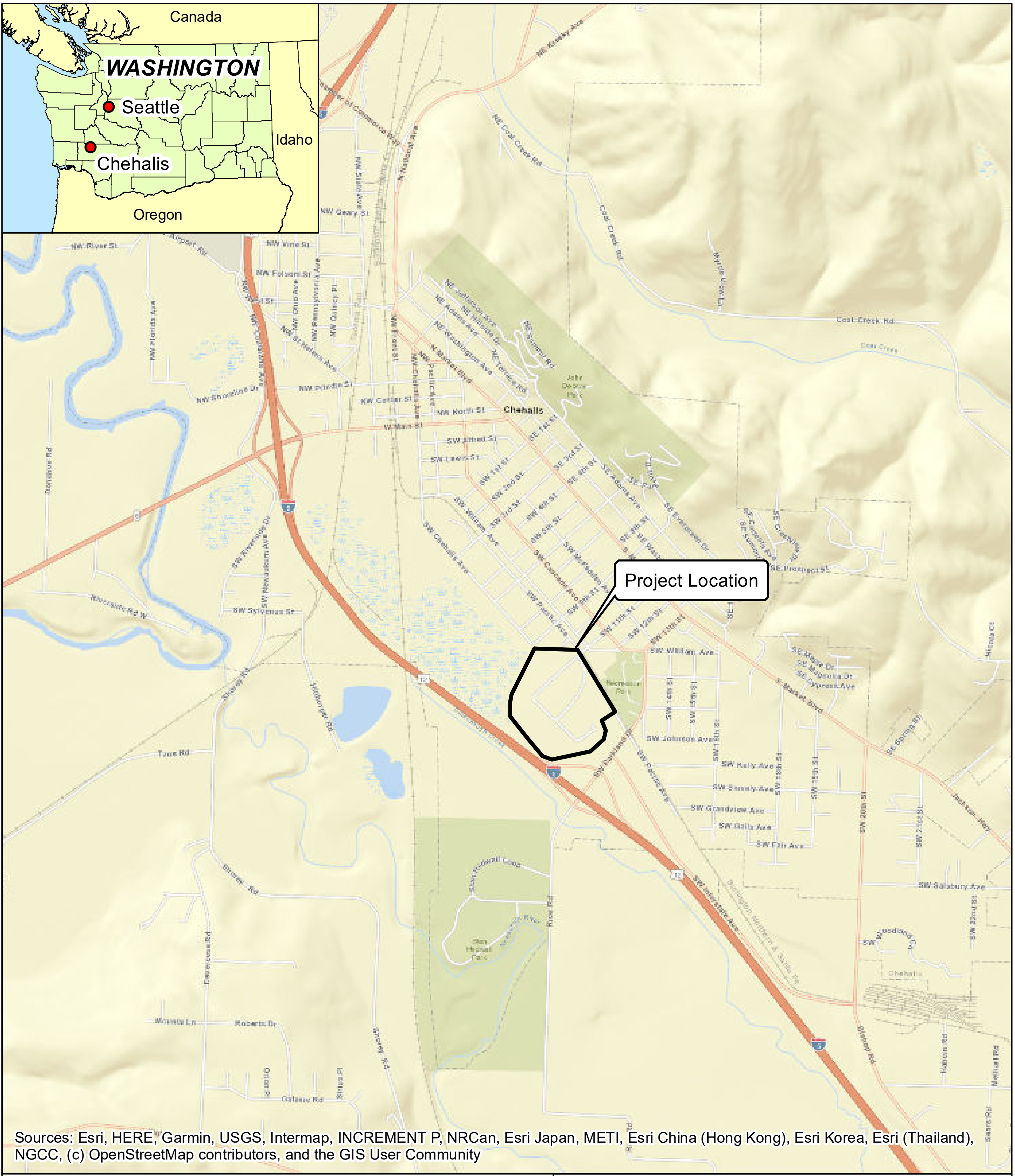

Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community

0 1,000 2,000 4,000
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Note: Feature locations are approximate.

N

Green Hill School Athletic Facility
Chehalis, Washington

Vicinity Map

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Figure
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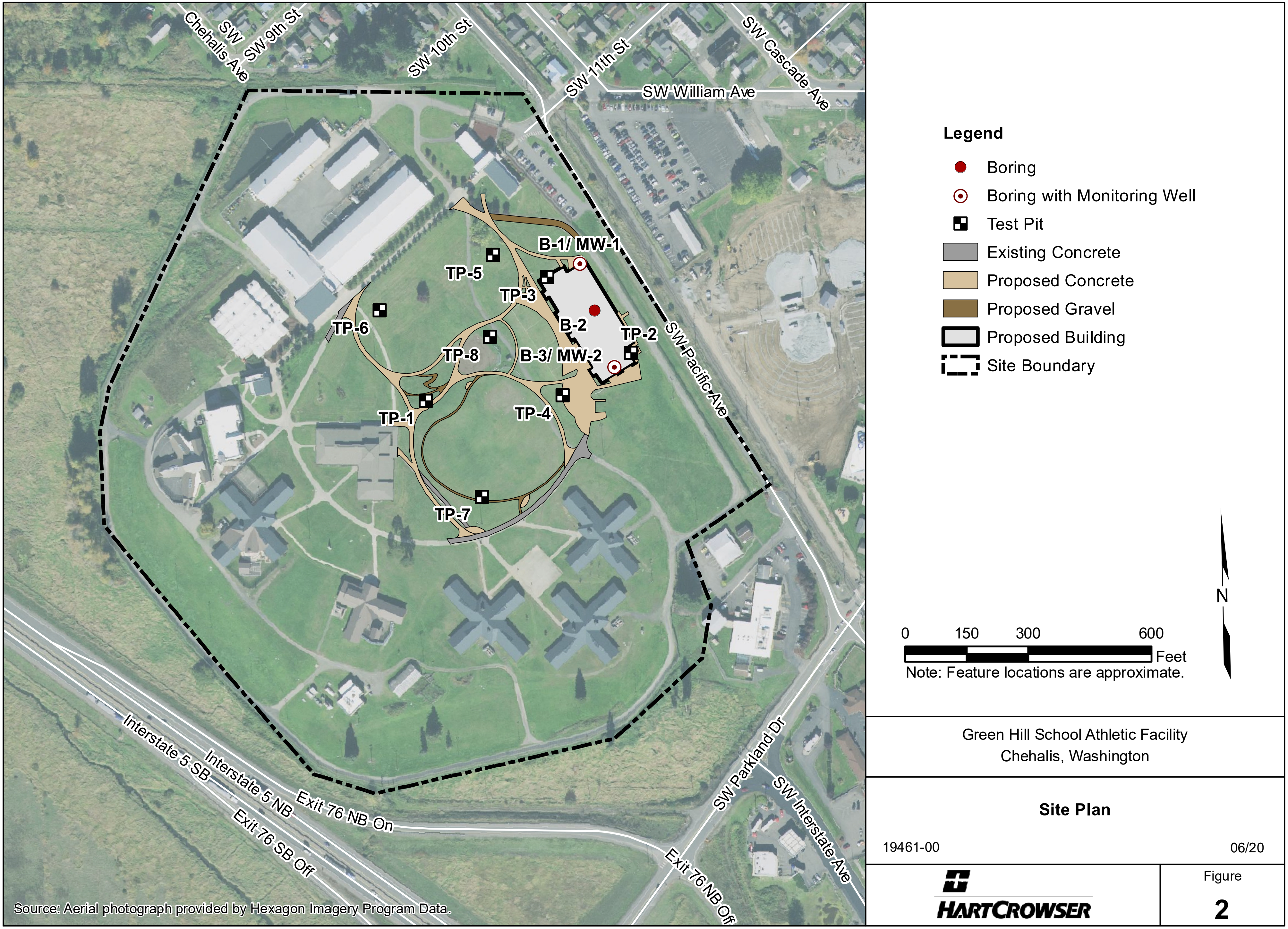

Note: Feature locations are approximate.

Source: Aerial photograph provided by Hexagon Imagery Program Data.

Green Hill School Athletic Facility
Chehalis, Washington

Site Plan

19461-00

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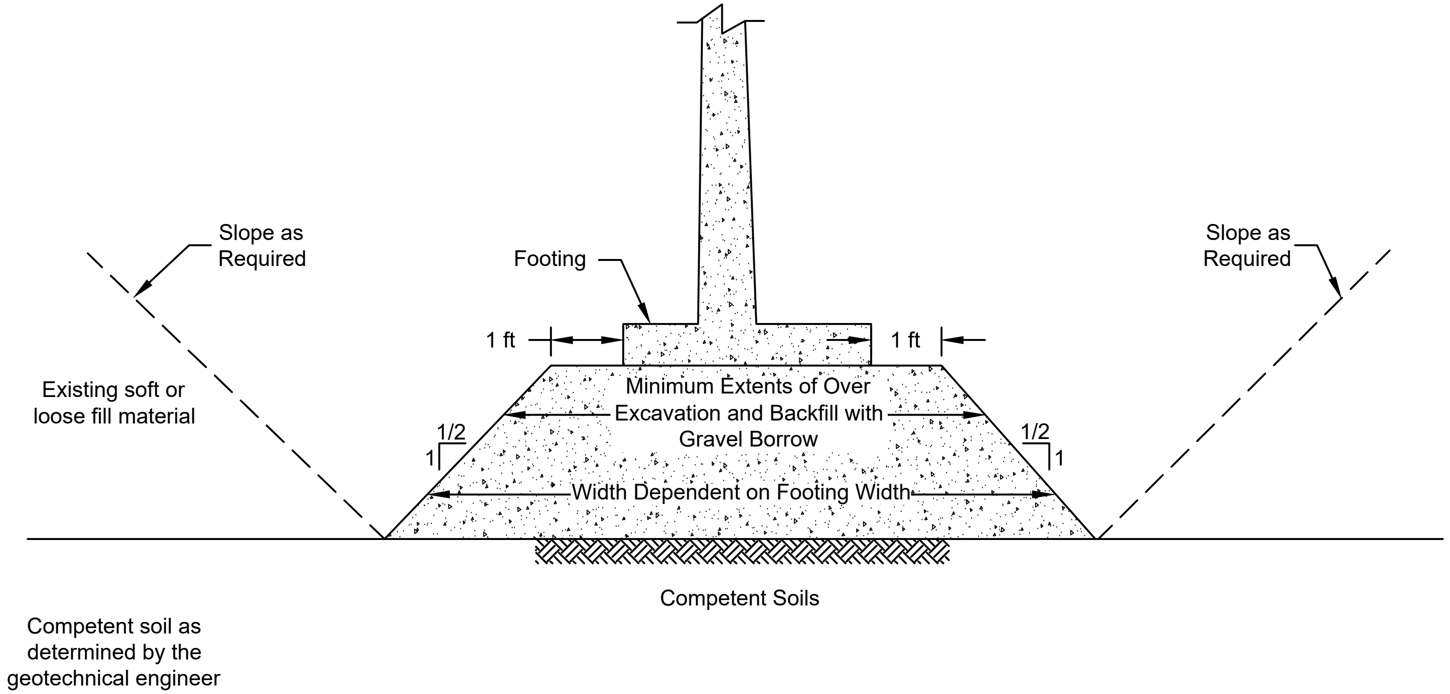
Figure

2



NOTES

1) Depth of overexcavation may vary depending on soil conditions.
2) Temporary slopes will be needed for excavations greater than 4 feet bgs.
3) Gravel borrow must be placed and compacted in accordance with the Geotechnical report.

NOT TO SCALE

Green Hill School Athletic Facility
Chehalis, Washington

Footing Overexcavation Details

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Figure
3

APPENDIX A
Field Explorations

APPENDIX A

Field Explorations

General

We evaluated subsurface conditions at the site by advancing three geotechnical borings, eight test pits, and two monitoring wells. The explorations were coordinated by a geologist on our staff, who classified the various soil units encountered, obtained representative soil samples for geotechnical testing, observed and recorded groundwater conditions, and maintained a detailed log of each boring and test pit. Logs of the geotechnical borings and test pits are included in this appendix. Results of the laboratory testing are indicated on the exploration logs and are included in Appendix B.

Materials encountered in the explorations were classified in the field in general accordance with American Society for Testing and Materials (ASTM) Standard Practice D 2488 “Standard Practice for the Classification of Soils (Visual-Manual Procedure).” Disturbed split spoon samples and relatively undisturbed tube samples were collected from the borings. Disturbed (“grab”) samples were collected from sidewalls or excavation spoils during test pit explorations. Sampling intervals are shown on the exploration log included in this appendix.

The exploration logs in this appendix show our interpretation of the exploration, sampling, and testing data. The logs indicate the depth where the soils change. Note that the change may be gradual. In the field, we classified the samples taken from the explorations according to the methods presented on the *Figure A-1 - Key to Exploration Logs*. This figure also provides a legend explaining the symbols and abbreviations used in the logs.

The approximate locations of the explorations are shown on Figure 2 of the report. Explorations were located in the field using a hand-held, mapping-grade, Trimble GPS unit with a horizontal accuracy of approximately 1 to 3 feet.

Geotechnical Borings

Three geotechnical borings were advanced between April 28 and April 30, 2020, using mud-rotary drilling methods with a track-mounted CME-850 drill rig operated by Western States Soil Conservation, Inc. of Hubbard, Oregon. The borings created an initial hole approximately 3.875 inches in diameter. Borings B-1 and B-3 had subsequent installations of monitoring wells and were widened to approximately 6 inches in diameter. Boring B-2 was backfilled to approximately 10 feet below ground surface (bgs) with a cement-bentonite grout then with bentonite chips up to the ground surface in accordance with state of Washington regulations. Monitoring wells in B-1 and B-3 were constructed and backfilled, as described below in the *Monitoring Wells* section of this appendix. The logs of the borings are included in this appendix.

Soil Sampling Procedures

Soil samples were obtained from the borings using the following methods.

- Sampling using a SPT sampler was completed in general conformance with ASTM Test Method D 1586 "Standard Method for Penetration Test and Split-Barrel Sampling of Soils." The sampler was driven with a 140-pound auto-trip hammer falling 30 inches. The sampler was driven a total distance of 18 inches or until refusal criteria was met (greater than 50 blows per 6 inches). The number of blows required to drive the samplers the final 12 inches (the "N" value) is recorded on the exploration logs, unless otherwise noted. All soil samples were placed into watertight bags and delivered to Hart Crowser's laboratory for subsequent classification and testing.

- We also performed sampling with a split-barrel, 3-inch outer-diameter, 2.4-inch inner-diameter modified California sampler. The sampler was also driven with a 140-pound hammer falling 30 inches. The number of blows required to drive the sampler the last 12 inches was correlated to SPT blow counts (N-values), using a Burmister (1948) correction of 64 percent. The corrected blow counts are plotted on the boring logs at their respective sample depths. Disturbed samples were obtained from the split barrel and placed into watertight plastic bags and delivered to Hart Crowser's laboratory for subsequent classification and testing.

- Relatively undisturbed samples were obtained using a thin-walled Shelby tube sampler in general conformance with ASTM Test Method D1587 "Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes." The sampler is driven using the hydraulic down-pressure of the drill rig mast.

Monitoring Wells

Two monitoring wells, MW-1 and MW-2, were installed in borings B-1 and B-3, respectively, to allow long-term groundwater elevation monitoring. The wells consist of a 4-inch-long PVC end cap threaded onto a 2-inch-diameter PVC riser pipe with 2-inch-diameter slotted screened pipe. MW-1 was screened from approximately 34 to 24 feet bgs and MW-2 was screened from approximately 24 to 14 feet bgs. Silica sand was used to fill the annulus surrounding the PVC pipe over the screened length and was extended to approximately 1 to 1.5 feet above the top of the screen. The sand was followed by hydrated bentonite chips from the top of sand in each well, approximately 23 and 13 feet, respectively, to approximately 1 foot bgs. The well head is protected by a surface-mounted monument cast into concrete from approximately 1 foot bgs to the surface.

Test Pits

Eight test pit explorations, designated TP-1 through TP-8, were performed on May 1, 2020. Test pit explorations were completed using a tracked excavator operated by Rivers Edge Environmental Services of Black Diamond, Washington. The explorations were continuously observed by a geologist on our staff, and detailed field logs of the test pits were prepared. Disturbed ("grab") samples were collected from sidewalls or excavation spoils during test pit explorations. Sampling intervals are shown on the exploration logs included in this appendix. The logs are presented at the end of this appendix.

Sample Description

Identification of soils in this report is based on visual field and laboratory observations which include density/consistency, moisture condition, grain size, and plasticity estimates and should not be construed to imply field nor laboratory testing unless presented herein. ASTM D 2488 visual-manual identification methods were used as a guide. Where laboratory testing confirmed visual-manual identifications, then ASTM D 2487 was used to classify the soils.

Relative Density/Consistency

Soil density/consistency in borings is related primarily to the standard penetration resistance (N). Soil density/consistency in test pits and probes is estimated based on visual observation and is presented parenthetically on the logs.

SAND or GRAVEL Relative Density	N (Blows/Foot)	SILT or CLAY Consistency	N (Blows/Foot)
Very loose	0 to 4	Very soft	0 to 1
Loose	5 to 10	Soft	2 to 4
Medium dense	11 to 30	Medium stiff	5 to 8
Dense	31 to 50	Stiff	9 to 15
Very dense	>50	Very stiff	16 to 30
		Hard	>30

Moisture

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

USCS Soil Classification Chart (ASTM D 2487)

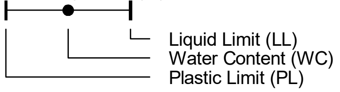
Major Divisions			Symbols Graph	Symbols USCS	Typical Descriptions
Coarse Grained Soils More than 50% of Material Retained on No. 200 Sieve	Gravel and Gravelly Soils More than 50% of Coarse Fraction Retained on No. 4 Sieve	Clean Gravels *(<5% fines)*		GW	Well-Graded Gravel; Well-Graded Gravel with Sand
				GP	Poorly Graded Gravel; Poorly Graded Gravel with Sand
		Gravels *(5-12% fines)*		GW-GM	Well-Graded Gravel with Silt; Well-Graded Gravel with Silt and Sand
				GW-GC	Well-Graded Gravel with Clay; Well-Graded Gravel with Clay and Sand
				GP-GM	Poorly Graded Gravel with Silt; Poorly Graded Gravel with Silt and Sand
				GP-GC	Poorly Graded Gravel with Clay; Poorly Graded Gravel with Clay and Sand
		Gravels with Fines *(>12% fines)*		GM	Silty Gravel; Silty Gravel with Sand
				GC	Clayey Gravel; Clayey Gravel with Sand
	Sand and Sandy Soils More than 50% of Coarse Fraction Passing No. 4 Sieve	Sands with few Fines *(<5% fines)*		SW	Well-Graded Sand; Well-Graded Sand with Gravel
				SP	Poorly Graded Sand; Poorly Graded Sand with Gravel
		Sands *(5-12% fines)*		SW-SM	Well-Graded Sand with Silt Well-Graded Sand with Silt and Gravel
				SW-SC	Well-Graded Sand with Clay; Well-Graded Sand with Clay and Gravel
				SP-SM	Poorly Graded Sand with Silt; Poorly Graded Sand with Silt and Gravel
				SP-SC	Poorly Graded Sand with Clay; Poorly Graded Sand with Clay and Gravel
		Sands with Fines *(>12% fines)*		SM	Silty Sand; Silty Sand with Gravel
				SC	Clayey Sand; Clayey Sand with Gravel
Fine Grained Soils More than 50% of Material Passing No. 200 Sieve	Silts			ML	Silt; Silt with Sand or Gravel; Sandy or Gravelly Silt
				MH	Elastic Silt; Elastic Silt with Sand or Gravel; Sandy or Gravelly Elastic Silt
	Silty Clay *(based on Atterberg Limits)*			CL-ML	Silty Clay; Silty Clay with Sand or Gravel; Gravelly or Sandy Silty Clay
	Clays			CL	Lean Clay; Lean Clay with Sand or Gravel; Sandy or Gravelly Lean Clay
				CH	Fat Clay; Fat Clay with Sand or Gravel; Sandy or Gravelly Fat Clay
	Organics			OL/OH	Organic Soil; Organic Soil with Sand or Gravel; Sandy or Gravelly Organic Soil
Highly Organic *(>50% organic material)*				PT	Peat - Decomposing Vegetation - Fibrous to Amorphous Texture

Minor Constituents

	Estimated Percentage
Sand, Gravel	
Trace	<5
Few	5 - 15
Cobbles, Boulders	
Trace	<5
Few	5 - 10
Little	15 - 25
Some	30 - 45

Soil Test Symbols

%F	Percent Passing No. 200 Sieve
AL	Atterberg Limits (%)



CA	Chemical Analysis
CAUC	Consolidated Anisotropic Undrained Compression
CAUE	Consolidated Anisotropic Undrained Extension
CBR	California Bearing Ratio
CIDC	Consolidated Drained Isotropic Triaxial Compression
CIUC	Consolidated Isotropic Undrained Compression
CK0DC	Consolidated Drained k0 Triaxial Compression
CK0DSS	Consolidated k0 Undrained Direct Simple Shear
CK0UC	Consolidated k0 Undrained Compression
CK0UE	Consolidated k0 Undrained Extension
CRSCN	Constant Rate of Strain Consolidation
DS	Direct Shear
DSS	Direct Simple Shear
DT	In Situ Density
GS	Grain Size Classification
HYD	Hydrometer
ILCN	Incremental Load Consolidation
K0CN	k0 Consolidation
kc	Constant Head Permeability
kf	Falling Head Permeability
MD	Moisture Density Relationship
OC	Organic Content
OT	Tests by Others
P	Pressuremeter
PID	Photoionization Detector Reading
PP	Pocket Penetrometer
SG	Specific Gravity
TRS	Torsional Ring Shear
TV	Torvane
UC	Unconfined Compression
UUC	Unconsolidated Undrained Triaxial Compression
VS	Vane Shear
WC	Water Content (%)

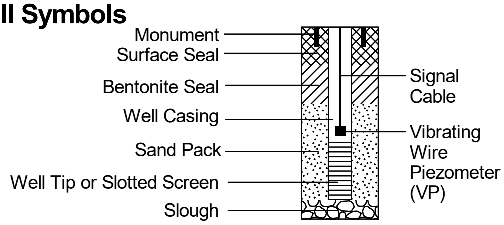
Groundwater Indicators

▽	Groundwater Level on Date or At Time of Drilling (ATD)
▼	Groundwater Level on Date Measured in Piezometer
♀	Groundwater Seepage (Test Pits)

Sample Symbols

☒ 1.5" I.D. Split Spoon	■ Rock Core Run	☒ Grab
◩ 3.25" O.D. Split Spoon	☐ Sonic Core	▥ Cuttings
◧ Modified California Sampler	◫ Thin-walled Sampler	▥ Push Probe

Well Symbols



HARTCROWSER

Project: Green Hill School Athletic Facility
Location: Chehalis, Washington
Project No.: 19461-00

Key to Exploration Logs

Figure **A-1**
Sheet **1 of 1**

Date Started: 4/28/20
Date Completed: 4/28/20
Logged by: R. Rosenberg
Checked by: D. Knapp
Location: Lat: 46.651020 Long: -122.959001 (WGS 84)
Ground Surface Elevation: 188.6 feet (NAVD 88)
Comments: Well Tag ID: BJC 769

Drilling Contractor/Crew: Western States Soil Conservation, Inc. / Jeff Christman
Drilling Method: Mud Rotary
Rig Model/Type: CME-850 XR / Track-mounted drill rig
Hammer Type: Auto-hammer
Hammer Weight (pounds): 140
Hammer Drop Height (inches): 30
Measured Hammer Efficiency (%): 80.4
Hole Diameter: 6 inches
Casing Diameter: ID: 2 inches
Total Depth: 35.2 feet
Depth to Groundwater: 4.45 feet

Blow Count	Recovery	Length (inches)	Number / Tests
1 1 2	12in.	18	S-1 / WC
	14in.	14	S-2
10 7 7	6in.	18	S-3 / GS, WC
6 7 7	9in.	18	S-4 / AL, GS, WC
11 15 16	10in.	18	S-5
15 20 20	12in.	18	S-6 / WC
43 40 24	12in.	18	S-7
37 50	10in.	12	S-8 / WC
50	5in.	5	S-9
50	2in.	2	S-10 / WC

Material Description

Topsoil (6-inch thick)

FAT CLAY (CH), soft, moist, light gray with orange-brown mottling. [FILL]

CLAYEY GRAVEL WITH SAND (GC), medium dense, moist, red-orange-brown to tan, fine to coarse subangular to rounded gravel, oxidized, likely few or with cobbles based on drill action and adjacent test pits. [OLDER ALLUVIUM]

becomes sandier

grades to dense

grades to moist to wet, red-brown to orange

grades to very dense, dark red to orange

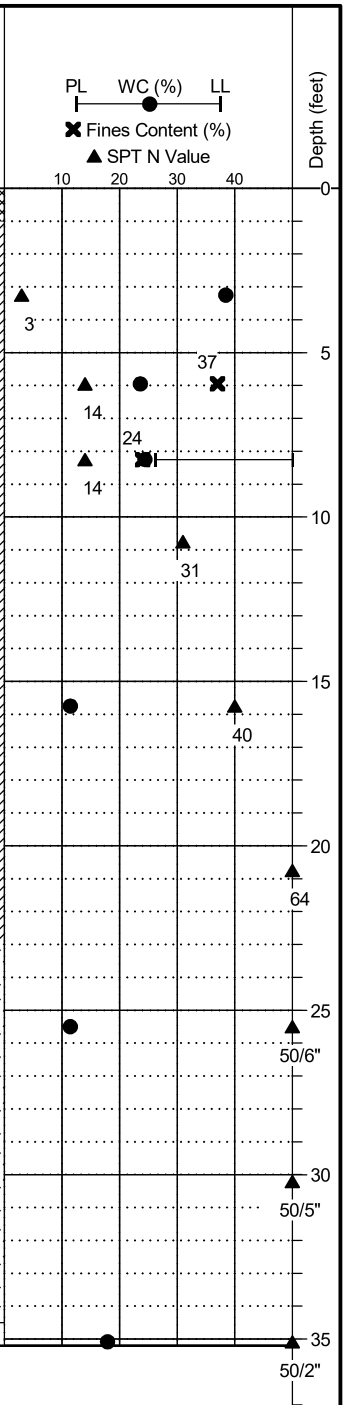
becomes sandier

POORLY GRADED GRAVEL WITH CLAY AND SAND (GP-GC), very dense, moist to wet, red-orange to black, fine to coarse subangular to rounded gravel, oxidized.

CLAYEY SAND WITH GRAVEL (SC), very dense, moist to wet, olive, fine to coarse sand, fine subangular to rounded gravel.

Bottom of Borehole at 35.2 feet.

Water Level: ⊻ 4/28/2020



General Notes:
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Material stratum lines are interpretive and actual changes may be gradual. Solid lines indicate distinct contacts and dashed lines indicate gradual or approximate contacts.
3. USCS designations are based on visual-manual identification (ASTM D 2488), unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling/excavation (ATD) or for date specified. Level may vary with time.
5. Location and ground surface elevations are approximate.

HARTCROWSER

Project: Green Hill School Athletic Facility
Location: Chehalis, Washington
Project No.: 19461-00

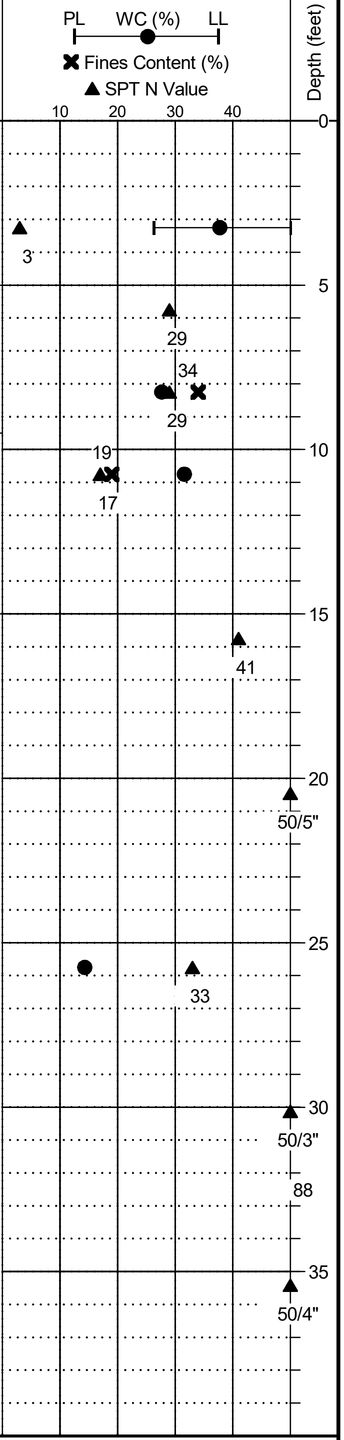
Boring and Monitoring Well Log
B-1/MW-1

Figure **A-2**
Sheet **1 of 1**

Date Started: 4/29/20
Date Completed: 4/29/20
Logged by: R. Rosenberg
Checked by: D. Knapp
Location: Lat: 46.650706 Long: -122.958853 (WGS 84)
Ground Surface Elevation: 188.8 feet (NAVD 88)
Comments: Blow counts for >1.5" split spoon adjusted to approximate SPT N-values (see report text).

Drilling Contractor/Crew: Western States Soil Conservation, Inc. / Jeff Christman
Drilling Method: Mud Rotary
Rig Model/Type: CME-850 XR / Track-mounted drill rig
Hammer Type: Auto-hammer
Hammer Weight (pounds): 140
Hammer Drop Height (inches): 30
Measured Hammer Efficiency (%): 80.4
Hole Diameter: 3.875 inches
Casing Diameter: NA
Total Depth: 51.5 feet
Depth to Groundwater: Not Identified

Elevation (feet)	Depth (feet)	Sample Data: Blow Count	Type	Recovery	Length (inches)	Number Tests	Graphic Log	Material Description
	0							Topsoil (6-inch thick)
								FAT CLAY (CH), trace fine to medium sand, soft, moist, light green-gray with orange-brown mottling, scattered rootlets. [FILL]
185		0 1 2		18in.	18	S-1 AL, WC		
	5	12 13 16		14in.	18	S-2		CLAYEY GRAVEL WITH SAND (GC), medium dense, moist, gray to orange-yellow-brown, fine to coarse subangular to rounded gravel, oxidized, likely few or with cobbles based on drill action and adjacent test pits. [OLDER ALLUVIUM]
180		3 10 19		10in.	18	S-3 GS, WC		
	10	16 9 8		10in.	18	S-4 GS, WC		SILTY SAND (SM), medium dense, moist to wet, dark red-brown, fine to medium sand, trace coarse sand.
175								minor chatter 13 to 15'
	15	15 20 21		8in.	18	S-5		CLAYEY GRAVEL WITH SAND (GC), dense, moist to wet, red-brown to yellow-brown, fine to coarse subangular to rounded gravel.
170								
	20	27 50		8in.	11	S-6		grades to very dense
165								
	25	17 18 15		12in.	18	S-7 WC		grades to dense
160								
	30	50 4 38 50		3in. 12in.	3 18	S-8 S-8re		grades to very dense
155								
	35	3 50		9in.	10	S-9		grades to moist to wet, olive
150								



General Notes:
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Material stratum lines are interpretive and actual changes may be gradual. Solid lines indicate distinct contacts and dashed lines indicate gradual or approximate contacts.
3. USCS designations are based on visual-manual identification (ASTM D 2488), unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling/excavation (ATD) or for date specified. Level may vary with time.
5. Location and ground surface elevations are approximate.

HARTCROWSER

Project: Green Hill School Athletic Facility
Location: Chehalis, Washington
Project No.: 19461-00

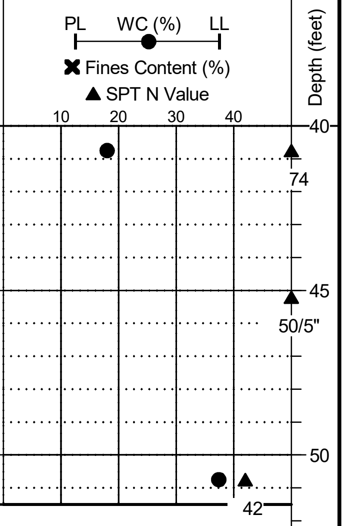
Boring Log
B-2

Figure **A-3**
Sheet **1 of 2**

Date Started: 4/29/20
Date Completed: 4/29/20
Logged by: R. Rosenberg
Checked by: D. Knapp
Location: Lat: 46.650706 Long: -122.958853 (WGS 84)
Ground Surface Elevation: 188.8 feet (NAVD 88)
Comments: Blow counts for >1.5" split spoon adjusted to approximate SPT N-values (see report text).

Drilling Contractor/Crew: Western States Soil Conservation, Inc. / Jeff Christman
Drilling Method: Mud Rotary
Rig Model/Type: CME-850 XR / Track-mounted drill rig
Hammer Type: Auto-hammer
Hammer Weight (pounds): 140
Hammer Drop Height (inches): 30
Measured Hammer Efficiency (%): 80.4
Hole Diameter: 3.875 inches
Casing Diameter: NA
Total Depth: 51.5 feet
Depth to Groundwater: Not Identified

Elevation (feet)	Depth (feet)	Sample Data: Blow Count	Type	Recovery	Length (inches)	Number / Tests	Graphic Log	Material Description	Depth (feet)
	40	14 30 44	☒	12in.	18	S-10 WC		CLAYEY SAND WITH GRAVEL (SC), very dense, moist to wet, olive, mostly fine subangular to rounded gravel.	40
145									
	45	50	☒	5in.	5	S-11		CLAYEY GRAVEL WITH SAND (GC), very dense, moist to wet, fine to coarse subangular to rounded gravel.	45
140									
	50	11 16 26	☒	18in.	18	S-12 WC		SILTY SAND (SM), dense, moist, dark green-gray, fine sand, laminated. [COMPLETELY DECOMPOSED LINCOLN CREEK FORMATION?]	50
								Bottom of Borehole at 51.5 feet.	
135									
	55								55
130									
	60								60
125									
	65								65
120									
	70								70
115									
	75								75
110									



General Notes:
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Material stratum lines are interpretive and actual changes may be gradual. Solid lines indicate distinct contacts and dashed lines indicate gradual or approximate contacts.
3. USCS designations are based on visual-manual identification (ASTM D 2488), unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling/excavation (ATD) or for date specified. Level may vary with time.
5. Location and ground surface elevations are approximate.

HARTCROWSER

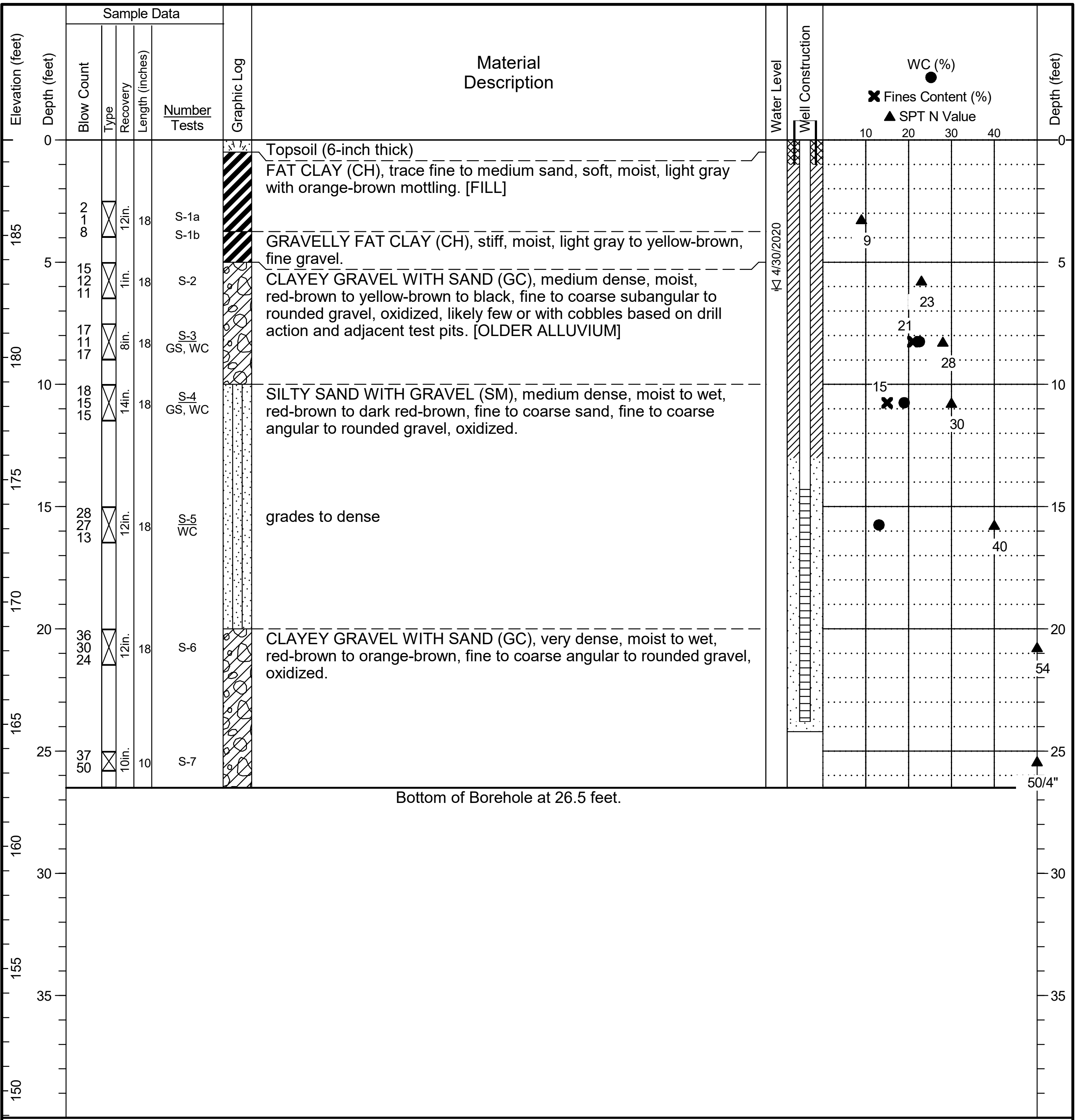
Project: Green Hill School Athletic Facility
Location: Chehalis, Washington
Project No.: 19461-00

Boring Log
B-2

Figure **A-3**
Sheet **2 of 2**

Date Started: 4/29/20
Date Completed: 4/30/20
Logged by: R. Rosenberg
Checked by: D. Knapp
Location: Lat: 46.650332 Long: -122.958652 (WGS 84)
Ground Surface Elevation: 188.9 feet (NAVD 88)
Comments: Well Tag ID: BJC 770

Drilling Contractor/Crew: Western States Soil Conservation, Inc. / Jeff Christman
Drilling Method: Mud Rotary
Rig Model/Type: CME-850 XR / Track-mounted drill rig
Hammer Type: Auto-hammer
Hammer Weight (pounds): 140
Hammer Drop Height (inches): 30
Measured Hammer Efficiency (%): 80.4
Hole Diameter: 3.875 inches
Casing Diameter: ID: 2 inches
Total Depth: 26.5 feet
Depth to Groundwater: 6.09 feet



Sample Data					Material Description
Blow Count	Type	Recovery	Length (inches)	Number Tests	
					Topsoil (6-inch thick)
2 1 8	☒	12in.	18	S-1a S-1b	FAT CLAY (CH), trace fine to medium sand, soft, moist, light gray with orange-brown mottling. [FILL]
					GRAVELLY FAT CLAY (CH), stiff, moist, light gray to yellow-brown, fine gravel.
15 12 11	☒	1in.	18	S-2	CLAYEY GRAVEL WITH SAND (GC), medium dense, moist, red-brown to yellow-brown to black, fine to coarse subangular to rounded gravel, oxidized, likely few or with cobbles based on drill action and adjacent test pits. [OLDER ALLUVIUM]
17 11 17	☒	8in.	18	S-3 GS, WC	
18 15 15	☒	14in.	18	S-4 GS, WC	SILTY SAND WITH GRAVEL (SM), medium dense, moist to wet, red-brown to dark red-brown, fine to coarse sand, fine to coarse angular to rounded gravel, oxidized.
28 27 13	☒	12in.	18	S-5 WC	grades to dense
36 30 24	☒	12in.	18	S-6	CLAYEY GRAVEL WITH SAND (GC), very dense, moist to wet, red-brown to orange-brown, fine to coarse angular to rounded gravel, oxidized.
37 50	☒	10in.	10	S-7	

Bottom of Borehole at 26.5 feet.

General Notes:
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Material stratum lines are interpretive and actual changes may be gradual. Solid lines indicate distinct contacts and dashed lines indicate gradual or approximate contacts.
3. USCS designations are based on visual-manual identification (ASTM D 2488), unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling/excavation (ATD) or for date specified. Level may vary with time.
5. Location and ground surface elevations are approximate.

HARTCROWSER

Project: Green Hill School Athletic Facility
Location: Chehalis, Washington
Project No.: 19461-00

Boring and Monitoring Well Log
B-3/MW-2

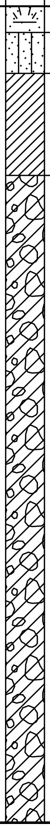
Figure **A-4**
Sheet **1 of 1**

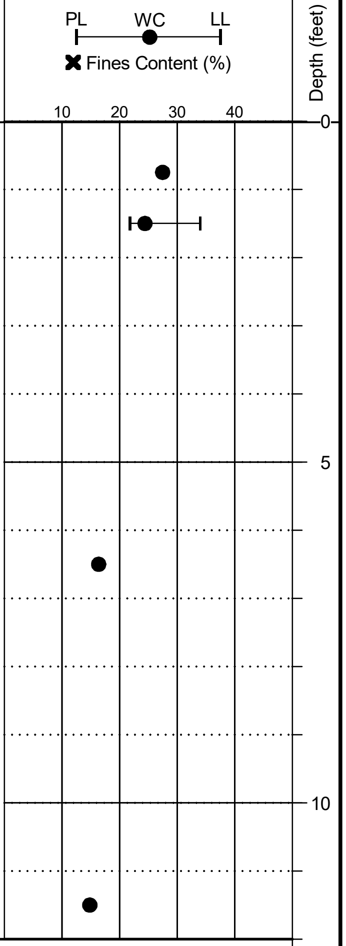
HC BORING LOG - F:\GINT\HC_LIBRARY.GLB - 5/26/20 11:36 - \\SEAFS\PROJECTS\NOTEBOOKS\1946100_GREEN_HILL_SCHOOL_ATHLETIC_FACILITY\FIELD DATA\PERM_GINT FILES\1946100_EXPLORATIONS.GPJ - danielknapp

Date Started: 5/1/20
Date Completed: 5/1/20
Contractor/Crew: Rivers Edge Environmental Services / Robert McMeyer
Logged by: R. Rosenberg
Checked by: D. Knapp
Rig Model/Type: Volvo 160 / Excavator
Location: Lat: 46.650088 Long: -122.960475 (WGS 84)
Total Depth: 12 feet
Depth to Seepage: 11 feet
Ground Surface Elevation: 185.4 feet (NAVD 88)
Comments:

Elevation (feet)	Depth (feet)	Sample Data Number / Tests	Material Description	Water Level
185	0		Topsoil (5-inch thick)	
		S-1 WC	SILTY SAND (SM), (loose), moist, red-brown, fine sand, occasional rootlets. [FILL]	
		S-2 AL, GS, WC	LEAN CLAY (CL), (medium stiff to stiff), moist, brown, scattered rootlets.	
		S-3	CLAYEY GRAVEL WITH SAND (GC), (medium dense), moist, light-gray to orange mottled, fine to coarse subangular to rounded gravel, oxidized. [OLDER ALLUVIUM]	
180	5			
		S-4 WC	grades to with cobbles	
		S-5	grades to slightly denser, no cobbles, moist to wet, gray-brown	
175	10			
			minor seepage	5/1/2020
		S-6 WC	grades to wet, red-brown	
170	15			

Bottom of Test Pit at 12.0 feet.





General Notes:
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Material stratum lines are interpretive and actual changes may be gradual. Solid lines indicate distinct contacts and dashed lines indicate gradual or approximate contacts.
3. USCS designations are based on visual-manual identification (ASTM D 2488), unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling/excavation (ATD) or for date specified. Level may vary with time.
5. Location and ground surface elevations are approximate.

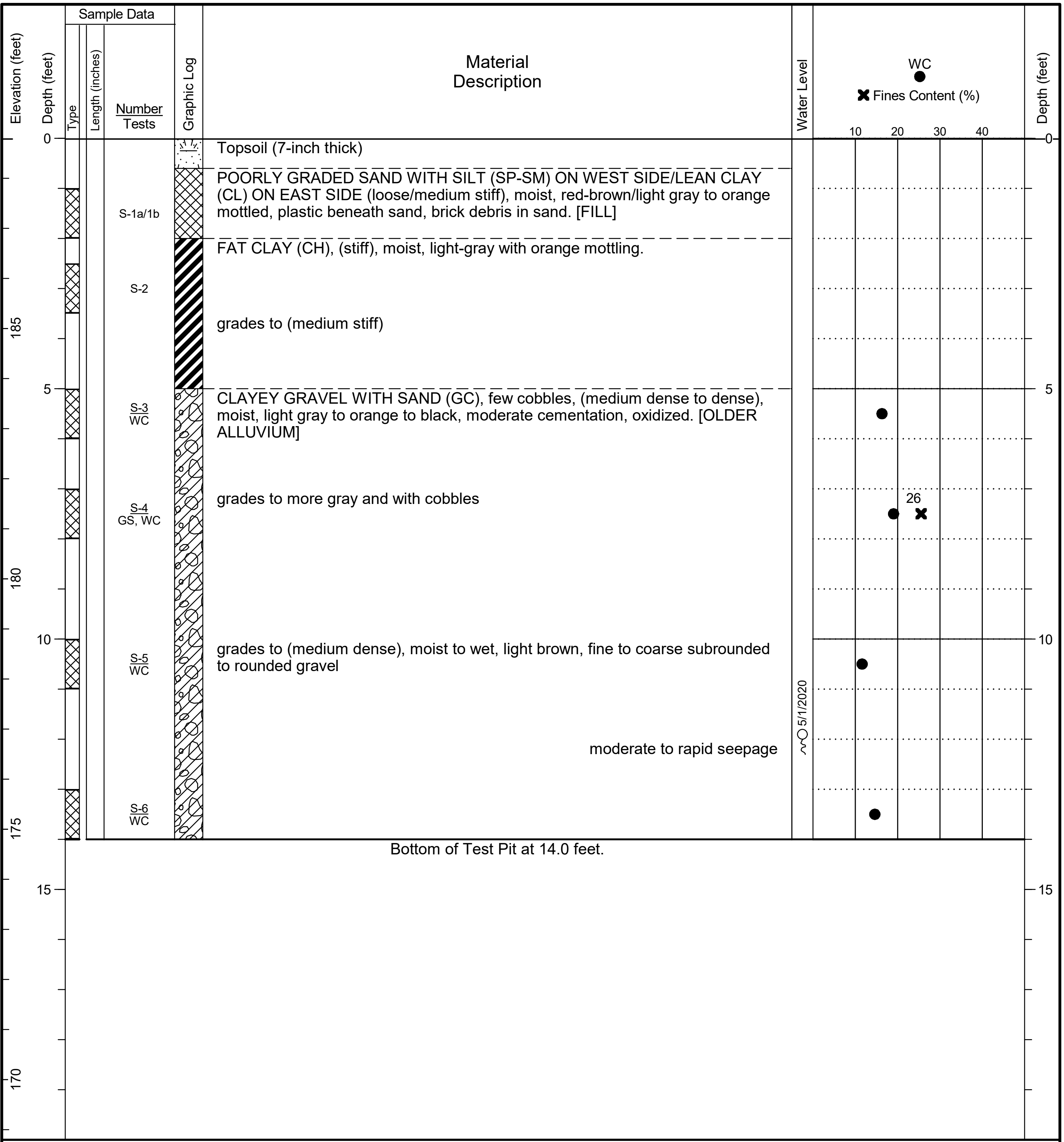
HARTCROWSER

Project: Green Hill School Athletic Facility
Location: Chehalis, Washington
Project No.: 19461-00

Test Pit Log
TP-1

Figure **A-5**
Sheet **1 of 1**

Date Started: 5/1/20
Date Completed: 5/1/20
Contractor/Crew: Rivers Edge Environmental Services / Robert McMeyer
Logged by: R. Rosenberg
Checked by: D. Knapp
Rig Model/Type: Volvo 160 / Excavator
Location: Lat: 46.650424 Long: -122.958494 (WGS 84)
Total Depth: 14 feet
Depth to Seepage: 12 feet
Ground Surface Elevation: 188.8 feet (NAVD 88)
Comments:



Material Description

Topsoil (7-inch thick)

POORLY GRADED SAND WITH SILT (SP-SM) ON WEST SIDE/LEAN CLAY (CL) ON EAST SIDE (loose/medium stiff), moist, red-brown/light gray to orange mottled, plastic beneath sand, brick debris in sand. [FILL]

FAT CLAY (CH), (stiff), moist, light-gray with orange mottling.

grades to (medium stiff)

CLAYEY GRAVEL WITH SAND (GC), few cobbles, (medium dense to dense), moist, light gray to orange to black, moderate cementation, oxidized. [OLDER ALLUVIUM]

grades to more gray and with cobbles

grades to (medium dense), moist to wet, light brown, fine to coarse subrounded to rounded gravel

moderate to rapid seepage

Bottom of Test Pit at 14.0 feet.

General Notes:
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Material stratum lines are interpretive and actual changes may be gradual. Solid lines indicate distinct contacts and dashed lines indicate gradual or approximate contacts.
3. USCS designations are based on visual-manual identification (ASTM D 2488), unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling/excavation (ATD) or for date specified. Level may vary with time.
5. Location and ground surface elevations are approximate.

HARTCROWSER

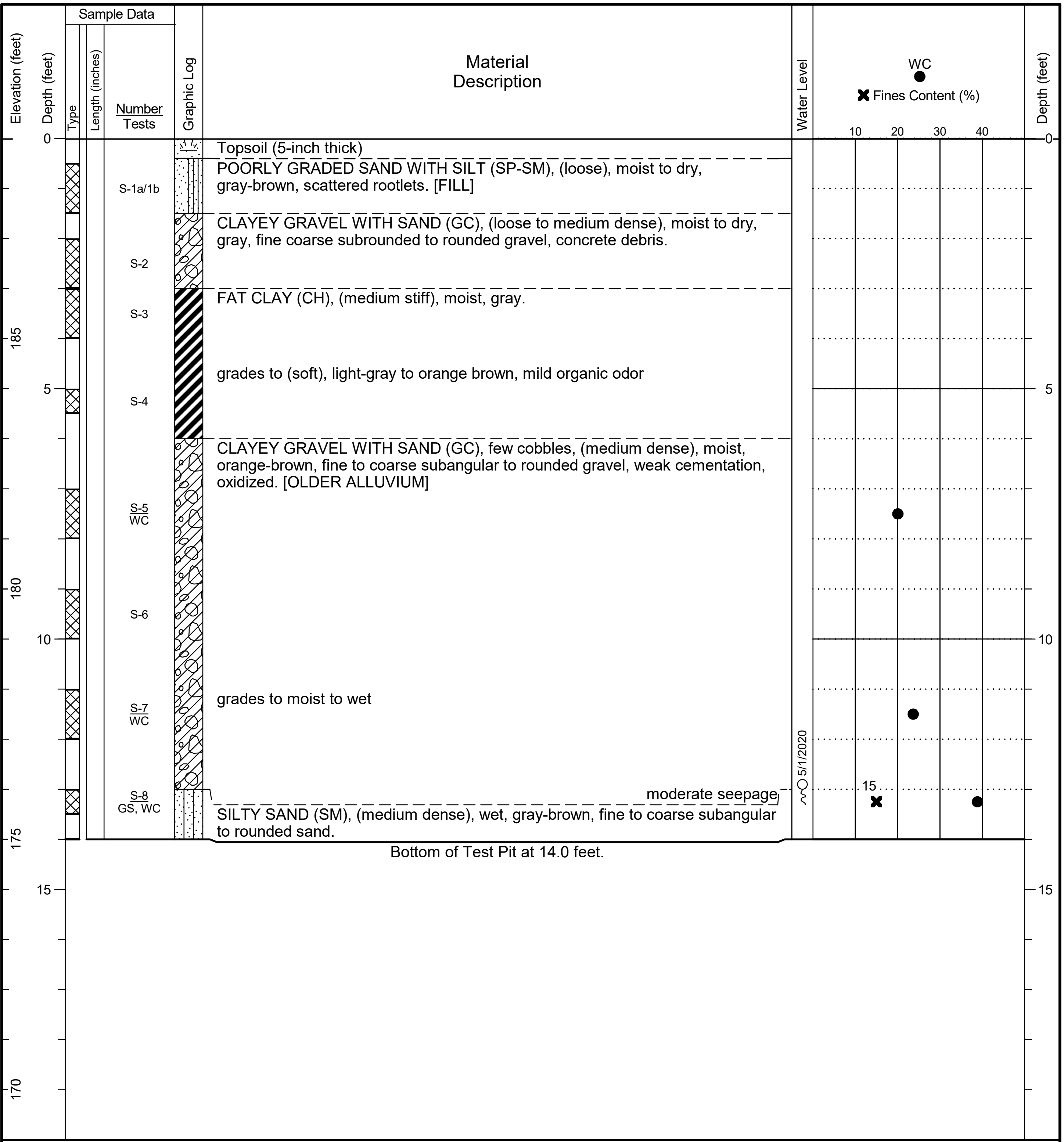
Project: Green Hill School Athletic Facility
Location: Chehalis, Washington
Project No.: 19461-00

Test Pit Log
TP-2

Figure **A-6**
Sheet **1 of 1**

HC TEST PIT - F:\GINT\HC_LIBRARY.GLB - 5/26/20 11:40 - \\SEAFS\PROJECTS\NOTEBOOKS\1946100_GREEN_HILL_SCHOOL_ATHLETIC_FACILITY\FIELD DATA\PERM_GINT FILES\1946100_EXPLORATIONS.GPJ - danielknapp

Date Started: 5/1/20
Date Completed: 5/1/20
Contractor/Crew: Rivers Edge Environmental Services / Robert McMeyer
Logged by: R. Rosenberg
Checked by: D. Knapp
Rig Model/Type: Volvo 160 / Excavator
Location: Lat: 46.650923 Long: -122.959312 (WGS 84)
Total Depth: 14 feet
Depth to Seepage: 13 feet
Ground Surface Elevation: 189.0 feet (NAVD 88)
Comments:



Topsoil (5-inch thick)

POORLY GRADED SAND WITH SILT (SP-SM), (loose), moist to dry, gray-brown, scattered rootlets. [FILL]

CLAYEY GRAVEL WITH SAND (GC), (loose to medium dense), moist to dry, gray, fine coarse subrounded to rounded gravel, concrete debris.

FAT CLAY (CH), (medium stiff), moist, gray.

grades to (soft), light-gray to orange brown, mild organic odor

CLAYEY GRAVEL WITH SAND (GC), few cobbles, (medium dense), moist, orange-brown, fine to coarse subangular to rounded gravel, weak cementation, oxidized. [OLDER ALLUVIUM]

grades to moist to wet

moderate seepage

SILTY SAND (SM), (medium dense), wet, gray-brown, fine to coarse subangular to rounded sand.

Bottom of Test Pit at 14.0 feet.

General Notes:
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Material stratum lines are interpretive and actual changes may be gradual. Solid lines indicate distinct contacts and dashed lines indicate gradual or approximate contacts.
3. USCS designations are based on visual-manual identification (ASTM D 2488), unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling/excavation (ATD) or for date specified. Level may vary with time.
5. Location and ground surface elevations are approximate.

HARTCROWSER

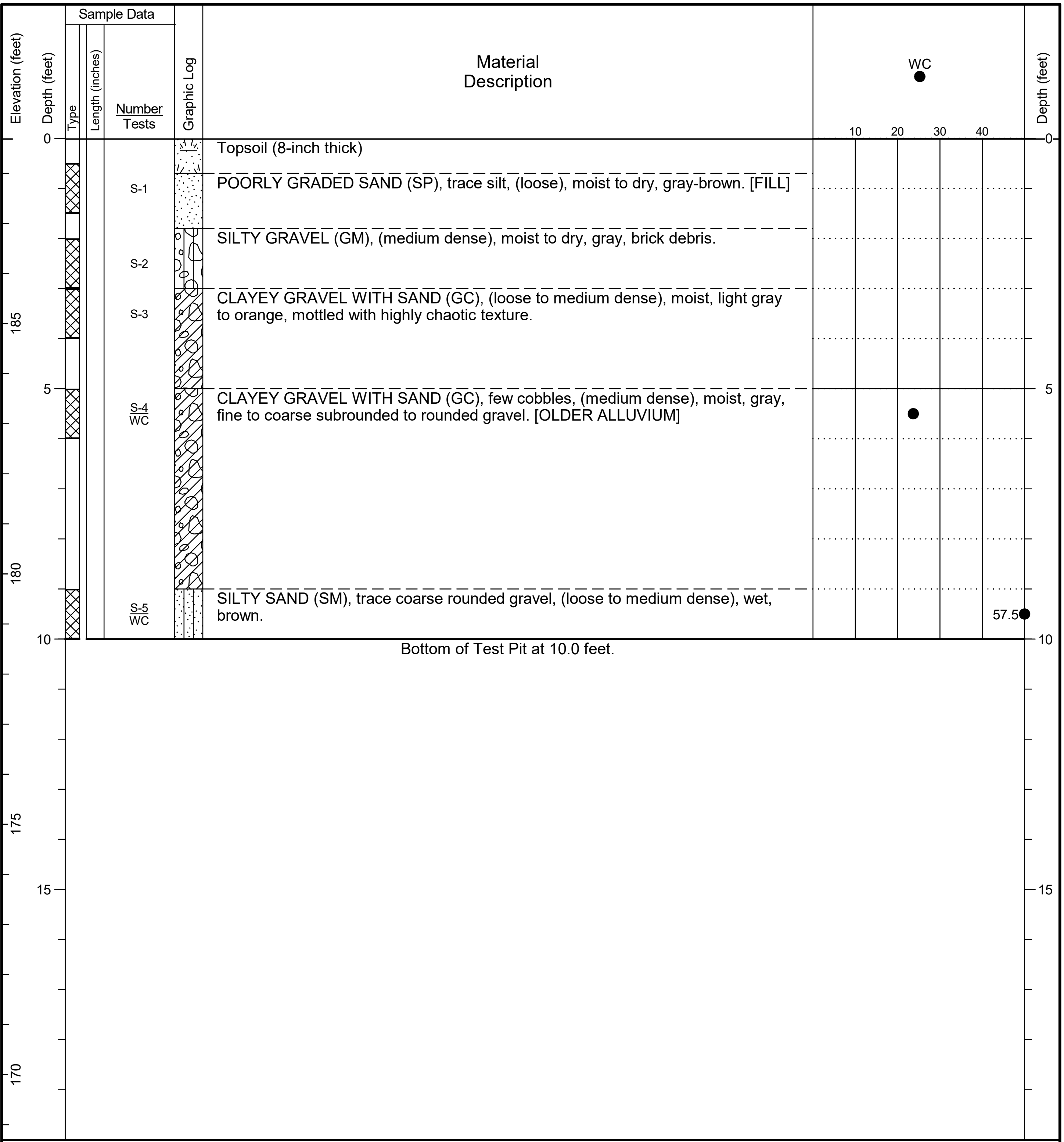
Project: Green Hill School Athletic Facility
Location: Chehalis, Washington
Project No.: 19461-00

Test Pit Log
TP-3

Figure **A-7**
Sheet **1 of 1**

HC TEST PIT - F:\GINT\HC_LIBRARY.GLB - 5/26/20 11:41 - \\SEAFS\PROJECTS\NOTEBOOKS\1946100_GREEN_HILL_SCHOOL_ATHLETIC_FACILITY\FIELD DATA\PERM_GINT FILES\1946100_EXPLORATIONS.GPJ - danielknapp

Date Started: 5/1/20 Date Completed: 5/1/20 Contractor/Crew: Rivers Edge Environmental Services / Robert McMeyer
Logged by: R. Rosenberg Checked by: D. Knapp Rig Model/Type: Volvo 160 / Excavator
Location: Lat: 46.650133 Long: -122.959155 (WGS 84) Total Depth: 10 feet Depth to Seepage: Not Encountered
Ground Surface Elevation: 188.7 feet (NAVD 88)
Comments:



Number Tests	Material Description	WC
	Topsoil (8-inch thick)	
S-1	POORLY GRADED SAND (SP), trace silt, (loose), moist to dry, gray-brown. [FILL]	
S-2	SILTY GRAVEL (GM), (medium dense), moist to dry, gray, brick debris.	
S-3	CLAYEY GRAVEL WITH SAND (GC), (loose to medium dense), moist, light gray to orange, mottled with highly chaotic texture.	
S-4 WC	CLAYEY GRAVEL WITH SAND (GC), few cobbles, (medium dense), moist, gray, fine to coarse subrounded to rounded gravel. [OLDER ALLUVIUM]	●
S-5 WC	SILTY SAND (SM), trace coarse rounded gravel, (loose to medium dense), wet, brown.	57.5 ●

Bottom of Test Pit at 10.0 feet.

General Notes:
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Material stratum lines are interpretive and actual changes may be gradual. Solid lines indicate distinct contacts and dashed lines indicate gradual or approximate contacts.
3. USCS designations are based on visual-manual identification (ASTM D 2488), unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling/excavation (ATD) or for date specified. Level may vary with time.
5. Location and ground surface elevations are approximate.

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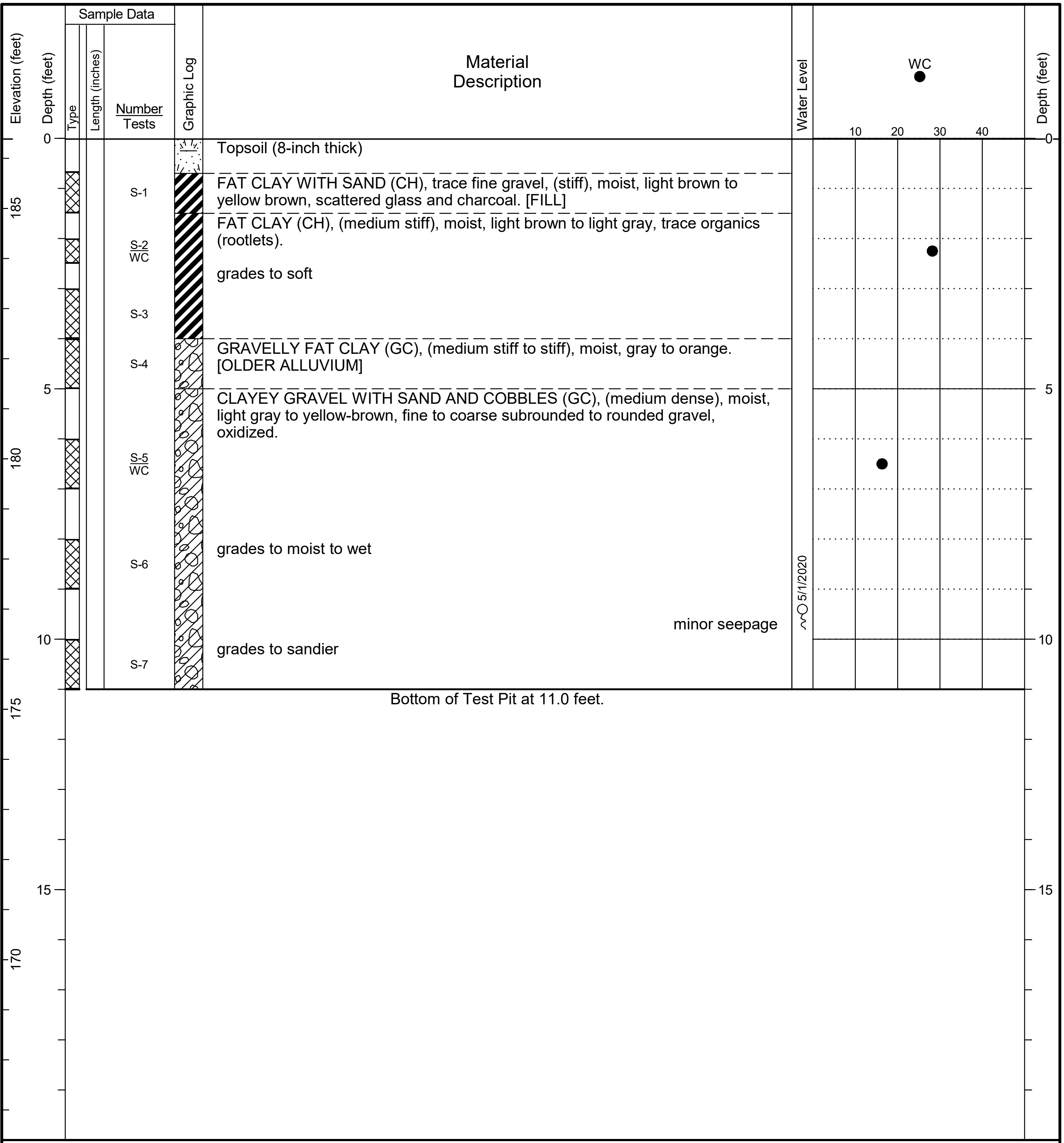
HARTCROWSER

Project: Green Hill School Athletic Facility
Location: Chehalis, Washington
Project No.: 19461-00

Test Pit Log
TP-4

Figure **A-8**
Sheet **1 of 1**

Date Started: 5/1/20 Date Completed: 5/1/20 Contractor/Crew: Rivers Edge Environmental Services / Robert McMeyer
Logged by: R. Rosenberg Checked by: D. Knapp Rig Model/Type: Volvo 160 / Excavator
Location: Lat: 46.651066 Long: -122.959842 (WGS 84) Total Depth: 11 feet Depth to Seepage: 9.5 feet
Ground Surface Elevation: 186.4 feet (NAVD 88)
Comments:



Material Description

Topsoil (8-inch thick)

FAT CLAY WITH SAND (CH), trace fine gravel, (stiff), moist, light brown to yellow brown, scattered glass and charcoal. [FILL]

FAT CLAY (CH), (medium stiff), moist, light brown to light gray, trace organics (rootlets).

grades to soft

GRAVELLY FAT CLAY (GC), (medium stiff to stiff), moist, gray to orange. [OLDER ALLUVIUM]

CLAYEY GRAVEL WITH SAND AND COBBLES (GC), (medium dense), moist, light gray to yellow-brown, fine to coarse subrounded to rounded gravel, oxidized.

grades to moist to wet

minor seepage

grades to sandier

Bottom of Test Pit at 11.0 feet.

General Notes:
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Material stratum lines are interpretive and actual changes may be gradual. Solid lines indicate distinct contacts and dashed lines indicate gradual or approximate contacts.
3. USCS designations are based on visual-manual identification (ASTM D 2488), unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling/excavation (ATD) or for date specified. Level may vary with time.
5. Location and ground surface elevations are approximate.

HARTCROWSER

Project: Green Hill School Athletic Facility
Location: Chehalis, Washington
Project No.: 19461-00

Test Pit Log
TP-5

Figure **A-9**
Sheet **1 of 1**

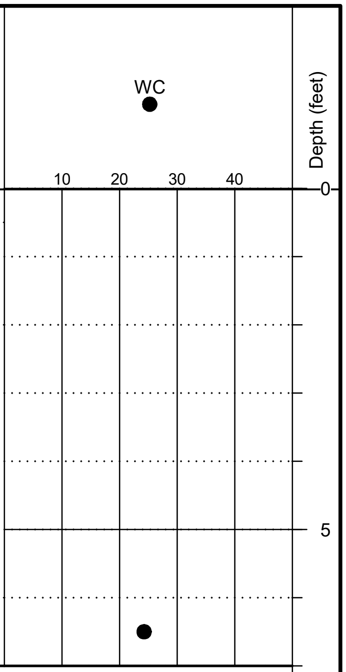
HC TEST PIT - F:\GINT\HC_LIBRARY.GLB - 5/26/20 11:42 - \\SEAFS\PROJECTS\NOTEBOOKS\1946100_GREEN_HILL_SCHOOL_ATHLETIC_FACILITY\FIELD DATA\PERM_GINT FILES\1946100_EXPLORATIONS.GPJ - danielknapp

Date Started: 5/1/20 Date Completed: 5/1/20
Logged by: R. Rosenberg Checked by: D. Knapp
Location: Lat: 46.650689 Long: -122.960935 (WGS 84)
Ground Surface Elevation: 187.9 feet (NAVD 88)
Comments:

Contractor/Crew: Rivers Edge Environmental Services / Robert McMeyer
Rig Model/Type: Volvo 160 / Excavator
Total Depth: 7 feet Depth to Seepage: Not Encountered

Elevation (feet)	Depth (feet)	Sample Data: Number Tests	Material Description
	0		Topsoil (6-inch thick)
		S-1	POORLY GRADED GRAVEL WITH SAND (GP-GM), (loose), moist, gray to gray-brown, fine to coarse rounded gravel. [FILL]
185		S-2	POORLY GRADED GRAVEL WITH SILT AND SAND (GP-GM), (loose), moist, brown, fine to coarse gravel, large concrete blocks and rebar.
	5	S-3 WC	LEAN CLAY (CL), (soft), moist, gray.

Bottom of Test Pit at 7.0 feet.



Depth scale: 0, 5, 10, 15; Elevation scale: 185, 180, 175, 170

General Notes:
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Material stratum lines are interpretive and actual changes may be gradual. Solid lines indicate distinct contacts and dashed lines indicate gradual or approximate contacts.
3. USCS designations are based on visual-manual identification (ASTM D 2488), unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling/excavation (ATD) or for date specified. Level may vary with time.
5. Location and ground surface elevations are approximate.

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HARTCROWSER

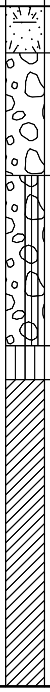
Project: Green Hill School Athletic Facility
Location: Chehalis, Washington
Project No.: 19461-00

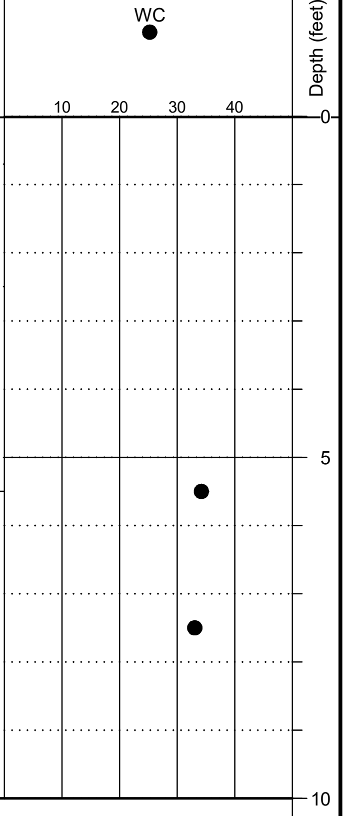
Test Pit Log
TP-6

Figure **A-10**
Sheet **1 of 1**

Date Started: 5/1/20 Date Completed: 5/1/20 Contractor/Crew: Rivers Edge Environmental Services / Robert McMeyer
Logged by: R. Rosenberg Checked by: D. Knapp Rig Model/Type: Volvo 160 / Excavator
Location: Lat: 46.649452 Long: -122.959921 (WGS 84) Total Depth: 10 feet Depth to Seepage: Not Encountered
Ground Surface Elevation: 193.3 feet (NAVD 88)
Comments:

Elevation (feet)	Depth (feet)	Sample Data Number Tests	Material Description
	0		Topsoil (8-inch thick)
		S-1	POORLY GRADED GRAVEL WITH SAND (GP), (loose), dry to moist, gray-brown, fine to coarse gravel, plastic debris. [FILL]
190		S-2	POORLY GRADED GRAVEL WITH SILT AND SAND (GP-GM), (loose), moist, gray, fine to coarse gravel, brick and wire.
	5	S-3 WC	SILT WITH SAND (ML), (soft), moist, brown, grassy organic mat. [BURIED TOPSOIL]
			LEAN CLAY (CL), trace fine sand, (soft), moist, gray. [OLDER ALLUVIUM]
		S-4 WC	
185			
		S-5	grades to light gray, higher plasticity
	10		Bottom of Test Pit at 10.0 feet.
180			
	15		
175			





General Notes:
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Material stratum lines are interpretive and actual changes may be gradual. Solid lines indicate distinct contacts and dashed lines indicate gradual or approximate contacts.
3. USCS designations are based on visual-manual identification (ASTM D 2488), unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling/excavation (ATD) or for date specified. Level may vary with time.
5. Location and ground surface elevations are approximate.

HARTCROWSER

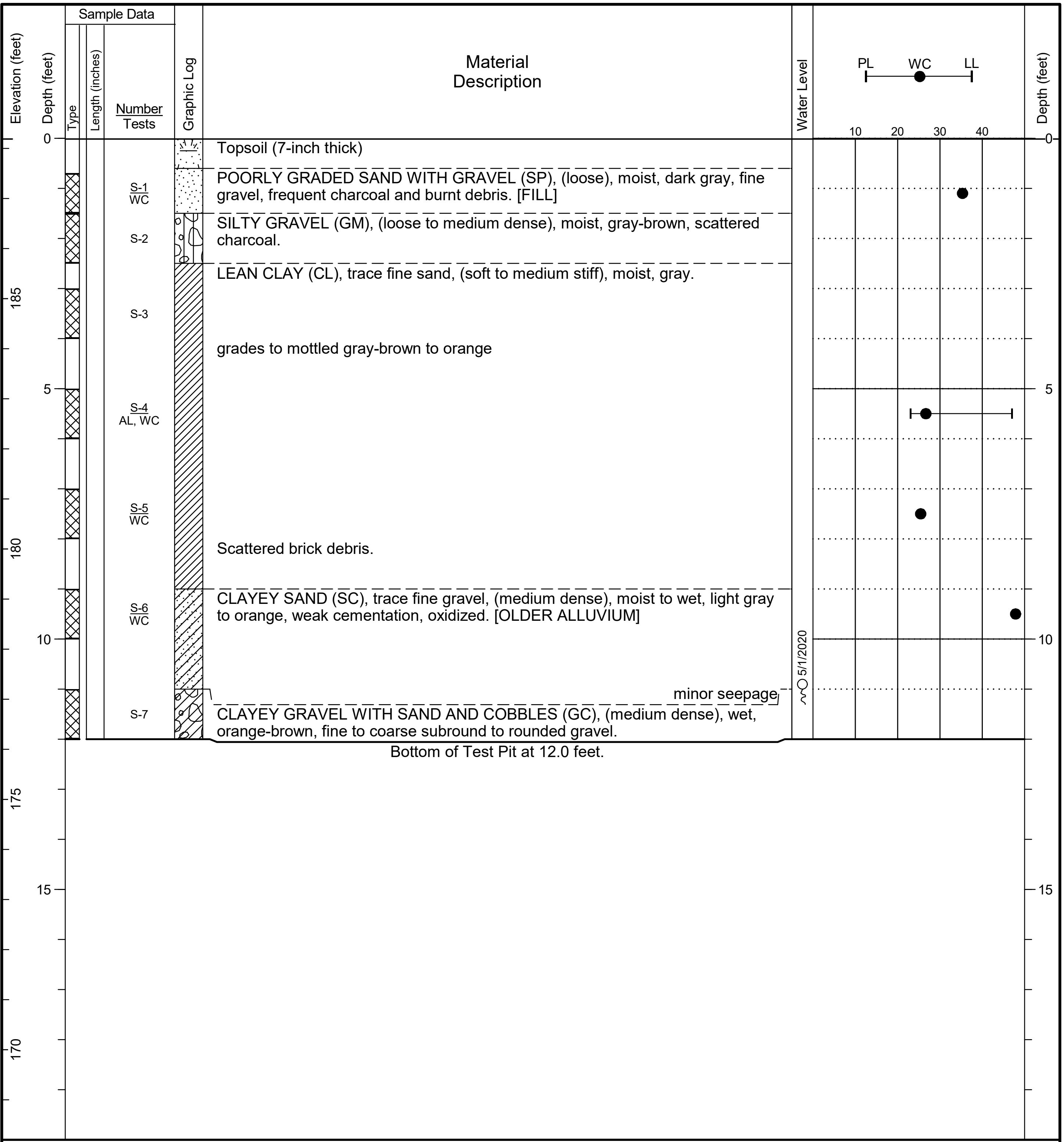
Project: Green Hill School Athletic Facility
Location: Chehalis, Washington
Project No.: 19461-00

Test Pit Log
TP-7

Figure **A-11**
Sheet **1 of 1**

HC TEST PIT - F:\GINT\HC_LIBRARY.GLB - 5/26/20 11:42 - \\SEAFS\PROJECTS\NOTEBOOKS\1946100_GREEN_HILL_SCHOOL_ATHLETIC_FACILITY\FIELD DATA\PERM_GINT FILES\1946100_EXPLORATIONS.GPJ - danielknapp

Date Started: 5/1/20
Date Completed: 5/1/20
Contractor/Crew: Rivers Edge Environmental Services / Robert McMeyer
Logged by: R. Rosenberg
Checked by: D. Knapp
Rig Model/Type: Volvo 160 / Excavator
Location: Lat: 46.650519 Long: -122.959864 (WGS 84)
Total Depth: 12 feet
Depth to Seepage: 11 feet
Ground Surface Elevation: 188.2 feet (NAVD 88)
Comments:



Number Tests	Material Description
	Topsoil (7-inch thick)
S-1 WC	POORLY GRADED SAND WITH GRAVEL (SP), (loose), moist, dark gray, fine gravel, frequent charcoal and burnt debris. [FILL]
S-2	SILTY GRAVEL (GM), (loose to medium dense), moist, gray-brown, scattered charcoal.
S-3	LEAN CLAY (CL), trace fine sand, (soft to medium stiff), moist, gray.
	grades to mottled gray-brown to orange
S-4 AL, WC	
S-5 WC	
	Scattered brick debris.
S-6 WC	CLAYEY SAND (SC), trace fine gravel, (medium dense), moist to wet, light gray to orange, weak cementation, oxidized. [OLDER ALLUVIUM]
	minor seepage
S-7	CLAYEY GRAVEL WITH SAND AND COBBLES (GC), (medium dense), wet, orange-brown, fine to coarse subround to rounded gravel.

Bottom of Test Pit at 12.0 feet.

General Notes:
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Material stratum lines are interpretive and actual changes may be gradual. Solid lines indicate distinct contacts and dashed lines indicate gradual or approximate contacts.
3. USCS designations are based on visual-manual identification (ASTM D 2488), unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling/excavation (ATD) or for date specified. Level may vary with time.
5. Location and ground surface elevations are approximate.

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Project: Green Hill School Athletic Facility
Location: Chehalis, Washington
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Test Pit Log
TP-8

Figure **A-12**
Sheet **1 of 1**

HC TEST PIT - F:\GINT\HC_LIBRARY.GLB - 5/26/20 11:43 - \\SEAFS\PROJECTS\NOTEBOOKS\1946100_GREEN_HILL_SCHOOL_ATHLETIC_FACILITY\FIELD DATA\PERM_GINT FILES\1946100_EXPLORATIONS.GPJ - danielknapp

APPENDIX B

Laboratory Testing

APPENDIX B

Laboratory Testing

General

Soil samples obtained from the explorations were transported to our laboratory in our office in Portland, Oregon and evaluated to confirm or modify field classifications, as well as to assess engineering properties of the soils encountered. Representative samples were selected for laboratory testing. The tests were performed in general accordance with the test methods of the ASTM or other applicable procedures. A summary of the test results is included as Figure B-1.

Visual Classifications

Soil samples obtained from the explorations were visually classified in the field and in our geotechnical laboratory based on the Unified Soil Classification System (USCS) and ASTM classification methods. ASTM Test Method D 2488 was used to classify soils using visual and manual methods. ASTM Test Method D 2487 was used to classify soils based on laboratory test results.

Laboratory Test Results

Moisture Content

Moisture contents of samples were obtained in general accordance with ASTM Test Method D 2216. The results of the moisture content tests completed on samples from the explorations are presented on the exploration logs included in Appendix A and on Figure B-1 in this appendix.

Percent Fines

Fines content analyses were performed to determine the percentage of soils finer than the U.S. No. 200 mesh sieve—the boundary between sand size particles and silt size particles. The tests were performed in general accordance with ASTM Test Method D 1140. The test results are indicated on the exploration logs included in Appendix A and on Figure B-1 in this appendix.

Grain Size Distribution

Sieve analysis tests were performed to determine the quantitative distribution of particle sizes in the sample. The tests were performed in general accordance with ASTM D 6913. The percentages of “fines” sand, and gravel from the test results are indicated on Figure B-1 in this appendix. The full test results are shown on Figure B-3 in this appendix.

Atterberg Limits Testing

Atterberg limits (liquid limit, plastic limit, and plasticity index) were obtained in general accordance with ASTM Test Method D 4318. The results of the Atterberg limits test is presented on the exploration logs included in Appendix A, summarized on Figure B-1 in this appendix, and shown in detail on Figure B-2 in this appendix.

Exploration	Sample ID	Depth	Water Content (%)	Dry Density (pcf)	Fines (%)	Sand (%)	Gravel (%)	Liquid Limit	Plastic Limit	Plasticity Index	Organic Content (%)	Pocket Pen (tsf)	Torvane (tsf)	
B-1/MW-1	S-1	2.5	38.4											
B-1/MW-1	S-3	5.2	23.6		37									
B-1/MW-1	S-4	7.5	24.4		24			50	26	24				
B-1/MW-1	S-6	15.0	11.5											
B-1/MW-1	S-8	25.0	11.4											
B-1/MW-1	S-10	35.0	17.9											
B-2	S-1	2.5	37.8					68	26	42				
B-2	S-3	7.5	27.7		34									
B-2	S-4	10.0	31.6		19									
B-2	S-7	25.0	14.3											
B-2	S-10	40.0	18.0											
B-2	S-12	50.0	37.4											
B-3/MW-2	S-3	7.5	22.5		21									
B-3/MW-2	S-4	10.0	19.0		15									
B-3/MW-2	S-5	15.0	13.1											
TP-1	S-1	0.5	27.5											
TP-1	S-2	1.0	24.4					34	22	12				
TP-1	S-4	6.0	16.4											
TP-1	S-6	11.0	14.8											
TP-2	S-3	5.0	16.3											
TP-2	S-4	7.0	19.0		26	39	35							
TP-2	S-5	10.0	11.6											
TP-2	S-6	13.0	14.6											
TP-3	S-5	7.0	20.1											
TP-3	S-7	11.0	23.7											
TP-3	S-8	13.0	38.9		15									
TP-4	S-4	5.0	23.7											
TP-4	S-5	9.0	57.5											
TP-5	S-2	2.0	28.2											
TP-5	S-5	6.0	16.3											
TP-6	S-3	6.0	24.3											
TP-7	S-3	5.0	34.2											
TP-7	S-4	7.0	33.1											
TP-8	S-1	0.7	35.3											
TP-8	S-4	5.0	26.7					47	23	24				
TP-8	S-5	7.0	25.5											
TP-8	S-6	9.0	47.9											

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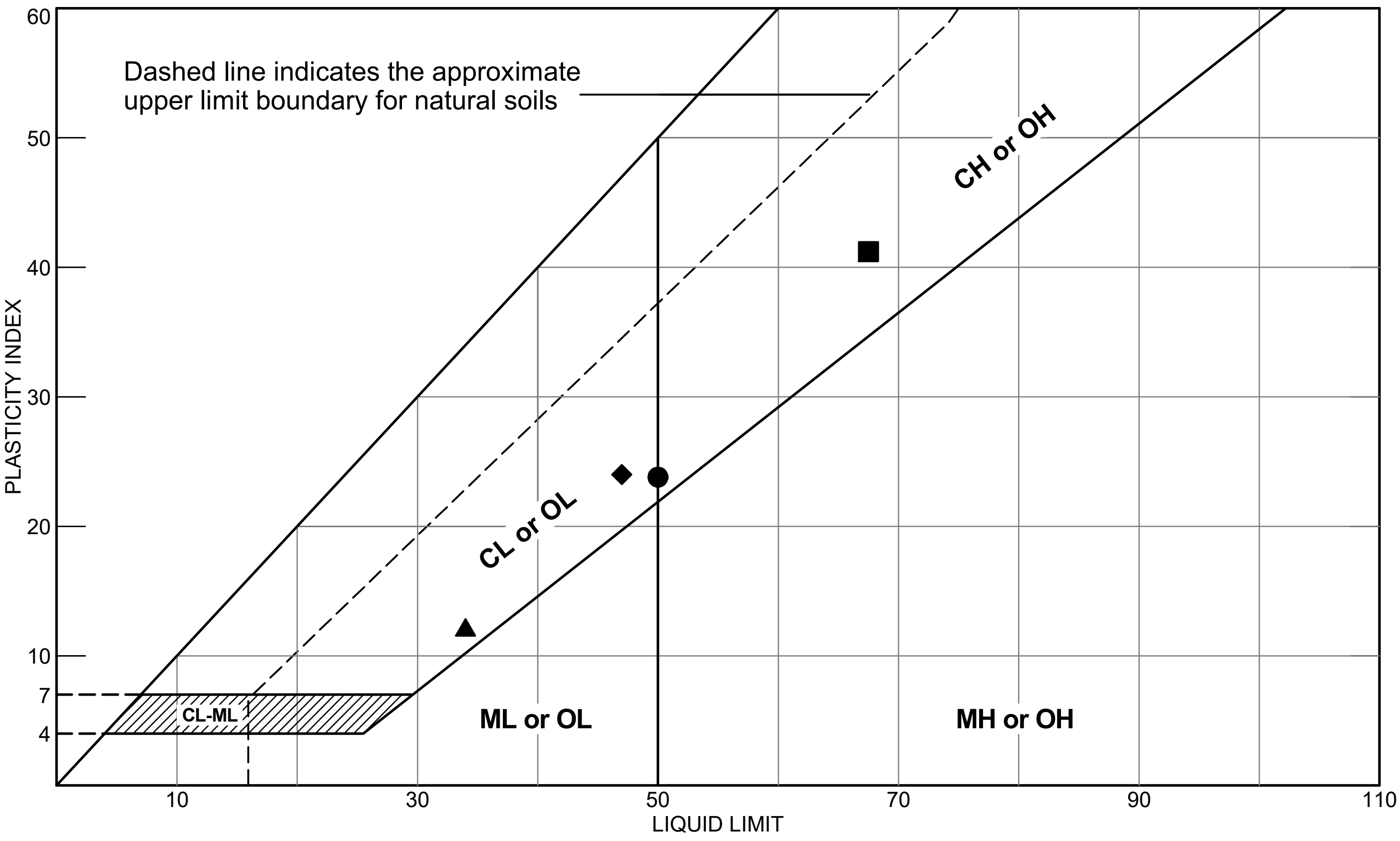
Project: Green Hill School Athletic Facility
Location: Chehalis, Washington
Project No.: 19461-00

Summary of Laboratory Results

Figure **B-1**
Sheet **1 of 1**



Location and Description	LL	PL	PI	-#200	MC%	USCS
● **Source:** B-1/MW-1 **Sample No.:** S-4 **Depth:** 7.5 to 9.0 CLAYEY GRAVEL WITH SAND	50	26	24	24	24	GC
■ **Source:** B-2 **Sample No.:** S-1 **Depth:** 2.5 to 4.0 FAT CLAY	68	26	42	NT	38	CH
▲ **Source:** TP-1 **Sample No.:** S-2 **Depth:** 1.0 to 2.0 LEAN CLAY	34	22	12	NT	24	CL
◆ **Source:** TP-8 **Sample No.:** S-4 **Depth:** 5.0 to 6.0 LEAN CLAY	47	23	24	NT	27	CL

Remarks:

● Test performed only on the material passing the No. 40 sieve; moderate plasticity
■ High plasticity
▲ Low to moderate plasticity
◆ Moderate plasticity

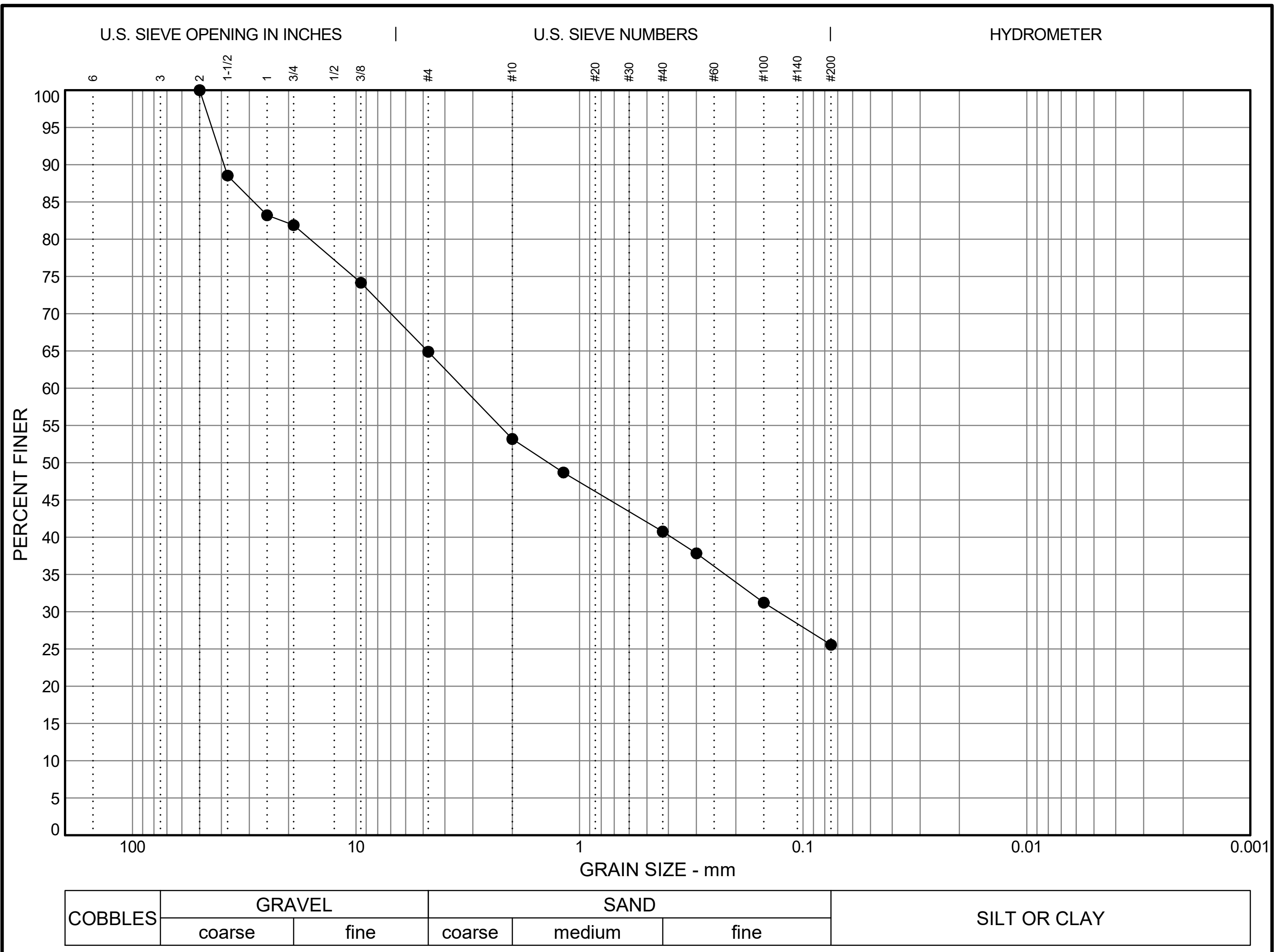
HARTCROWSER

Project: Green Hill School Athletic Facility
Location: Chehalis, Washington
Project No.: 19461-00

Liquid Limit, Plastic Limit, and Plasticity Index

Figure **B-2**
Sheet **1 of 1**

HC ATTERBERG LIMITS - F:\GINT\HC_LIBRARY.GLB - 5/26/20 11:55 - \\SEAFS\PROJECTS\NOTEBOOKS\1946100_GREEN_HILL_SCHOOL_ATHLETIC_FACILITY\FIELD DATA\PERM_GINT FILES\1946100_EXPLORATIONS.GPJ - danielknapp



Location and Description	% Cobbles	% Gravel	% Sand	% Silt	% Clay	MC%	USCS
● **Source:** TP-2 **Sample No.:** S-4 **Depth:** 7.0 to 8.0 CLAYEY GRAVEL WITH SAND	0.0	35.1	39.3	25.6		19	GC

☒	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			28.647	3.310	1.376	0.129				

Remarks:

● Large highly weathered cobbles slaked during the test, therefore we identify this soil as a gravel not a sand.

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Project: Green Hill School Athletic Facility
Location: Chehalis, Washington
Project No.: 19461-00

Particle-Size Analysis

Figure **B-3**
Sheet **1 of 1**