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Geotechnical Investigation Lower Prune Hill Booster Pump Station and Reservoir Replacement

Camas, Washington

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1 INTRODUCTION

As requested, GRI completed a geotechnical investigation for the construction of a new booster pump station and replacement of the existing 0.5 MG reservoir located at the Prune Hill Reservoir site located at 600 NW 18th Loop in Camas, Washington. The general location of the project is shown on the Vicinity Map, Figure 1. Our investigation included a review of available geotechnical information and relevant geologic maps for the site, subsurface explorations, laboratory testing, and engineering studies and analyses. The report describes the work accomplished and provides our conclusions and recommendations for the design and construction of the proposed replacement reservoir and booster pump station.

2 PROJECT DESCRIPTION

We understand that the project includes the replacement of the existing 0.5 MG reservoir with a new 0.5 MG welded steel reservoir south of the existing 0.5 MG reservoir and at the southern edge of the property. The floor elevation of the new reservoir will be at elevation 435 feet. The reservoir will have an interior diameter of 65 feet and a water height of 20 feet when full. The reservoir roof will be supported by interior columns and the walls of the reservoir will be supported by a continuous ring foundation.

The new booster pump station is planned at the location of the existing 0.5 MG reservoir and will include pump cans embedded approximately 10 feet below existing site grades. The floor elevation of the new pump station will be at elevation 435.5 feet. Cuts up to 19 feet will be required in the hillside on the southern and western property boundary to accommodate the new reservoir and pump station A permanent, cantilevered soldier pile retaining wall is planned to support the new cuts along the south, west, and north side of the new reservoir and pump station. The inclination of the backslope behind the proposed retaining wall will not exceed 2H:1V. Due to site constraints, we understand that tiebacks or a soil nail wall are not being considered.

3 SITE DESCRIPTION

3.1 Site Conditions

The Lower Prune Hill Reservoir site is located on the southeastern flank of Prune Hill. The proposed improvements are planned for the southernmost portion of the site, where existing improvements include the 0.5 MG reservoir, a 5-foot to 6-foot-tall masonry block wall, telecommunications equipment, and yard piping. A 1.5 MG reservoir, a 1½H:1V (Horizontal to Vertical) ivy-covered slope, and a lawn-covered area are in the northern portion of the site. Residential developments are located to the south, west, and north of the reservoir site. A tree-covered slope, which is bisected by NW 18th Loop Road, and residential properties are located east of the site.

Site grades in the southern portion of the site and around the proposed improvements range from about elevation 430 feet to 450 feet. Grades above the proposed cantilevered retaining wall location range from near horizontal to 3H:1V. The slope to the southeast of the proposed new reservoir improvements drops about 25 feet to 40 feet to NW 18th Loop Road. The slope between the site and NW 18th Loop Road is relatively uniform at about 1½H:1V with no significant indications of slope movement. Below NW 18th Loop Road, the grades are flatter with typical inclinations in the range of 2H:1V to 2¼H:1V. Springs or seeps were not observed on the portion of the slope located between the reservoir site and NW 18th Loop Road at the time of our field-exploration program.

3.2 Geology

Based on our review of readily available geologic mapping, the Prune Hill Reservoir site is underlain by a sedimentary rock unit mapped as quaternary unnamed conglomerate (QTc). This unit consists of unconsolidated to cemented, well-rounded gravel, cobble, and boulder (i.e., conglomerate) interbedded with sandstone.

4 SUBSURFACE CONDITIONS

4.1 General

Subsurface materials and conditions at the site were investigated on September 10 and 11, 2020, with two Rotosonic borings, designated B-1 and B-2, and one mud-rotary boring, designated B-3 completed on July 7, 2021, at the approximate locations shown on the Site Plan, Figure 2. Borings B-1 and B-2 were advanced to a depth of 31.5 feet using the Rotosonic drilling method, while boring B-3 was advanced to a depth of 51.5 feet using mud-rotary drilling techniques. Logs of the borings are provided on Figures 1A through 3A. Discussion of the field-exploration and laboratory-testing programs are provided in Appendix A. The terms and symbols used to describe the soils encountered in the borings are defined in Table 1A and the attached legend. Photographs of the core samples recovered from the Rotosonic borings B-1 and B-2 are provided in Appendix B.

A soil boring was advanced in April 1971 by CH2M at the location of the existing 1.5-MG reservoir as part of the original design of this structure. The boring disclosed approximately 5 feet of clayey silt at the surface underlain by weathered conglomerate to the maximum depth explored of about 70 feet. The location of the historical boring is shown on Figure 2, and the historical boring log is included as an attachment at the end of Appendix A.

4.2 Soils

For the purpose of discussion, the materials disclosed by the borings have been grouped into the following units based on their physical characteristics and engineering properties and listed as they were encountered from the ground surface:

a. Sandy SILT to Silty SAND (Decomposed QTc Sandstone)
b. Silty GRAVEL (Decomposed QTc Conglomerate)
c. CONGLOMERATE (QTc)

The following paragraphs provide a description of the materials encountered and a discussion of the groundwater conditions at the site. A 4- to 6-inch-thick, heavily rooted zone was encountered at the ground surface in each of the explorations completed for this study.

a. Sandy SILT to Silty SAND (Decomposed QTc Sandstone)

Sandy silt and silty sand were encountered in boring B-1 to a depth of 3 feet; in boring B-2 between 6 feet and 7.5 feet, between 8.5 feet and 14 feet, and between 17 feet and 20 feet below the ground surface; and in boring B-3 to a depth of 12.5 feet. The sandy silt includes a trace of clay and has low plasticity, and the sand is fine to coarse grained. Based on SPT N-values, the relative consistency of the sandy silt is stiff to very stiff. The relative density of the silty sand is loose to medium dense.

b. Silty GRAVEL (Decomposed QTc Conglomerate)

The silty sand to sandy silt is underlain by silty gravel to the maximum depth explored, about 31.5 feet, in boring B-1 and to a depth of 40 feet in boring B-3. In boring B-2, silty gravel was encountered below the heavily rooted zone to a depth of 6 feet, between 7.5 feet and 8.5 feet, between 14 feet and 17 feet, and from 20 feet to 31.5 feet (maximum depth explored). The silty gravel unit contains variable fine- to coarse-grained sand content, ranging from a trace of sand to sandy. The unit contains cobbles, and the gravel is typically subangular. Although not observed in our explorations, boulders are commonly encountered within the decomposed conglomerate. Based on SPT N-values and modified California N*-values, the silty gravel is medium dense to very dense in density.

c. CONGLOMERATE (QTc Conglomerate)

Extremely soft to very soft (R0 to R1) conglomerate rock was encountered at a depth of 40 feet in boring B-3. The conglomerate rock is predominately decomposed and unconsolidated to poorly cemented and extends to the maximum depth explored, 51.5 feet, in this boring.

4.3 Groundwater

At the time of drilling in September 2020, groundwater was encountered at a depth of about 26.5 feet (elevation 422.5 feet) in boring B-1 and at a depth of about 29 feet in

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boring B-2 (elevation 408.5 feet). Due to the mud-rotary drilling method, direct measurement of groundwater was not possible at the time of drilling in boring B-3 in July 2021. Groundwater was encountered at a depth of about 36 feet (elevation 440 feet) on April 8, 1971, in a boring advanced as part of the design of the existing 1.5 MG reservoir. A vibrating-wire piezometer was installed in boring B-2 at the time of drilling to measure the depth to groundwater. The groundwater measurements from the vibrating-wire piezometer are inconsistent with the measurements at the time of drilling. and our experience in the project area and are being further evaluated by GRI.

The groundwater data indicate the groundwater elevation decreases with the elevation of the ground surface to the southwest and southeast of the site. We anticipate that zones of perched groundwater may be present within the decomposed conglomerate or sandstone, especially during periods of high precipitation.

5 FINDINGS AND RECOMMENDATIONS

5.1 General

The explorations completed for this investigation disclosed decomposed sandstone or conglomerate consisting of sandy silt, silty sand, or silty gravel to about elevation 410 feet. Below this depth, extremely soft to very soft (R0 to R1) conglomerate rock was encountered. Based on groundwater measurements made at the time of drilling (September 2020) and the historical geotechnical data, we anticipate that groundwater is present at depths of at least 25 feet below the ground surface in the vicinity of the proposed new reservoir, pump station, and retaining walls. We anticipate that perched groundwater conditions may approach the ground surface during periods of extended wet weather after heavy rainfall.

The primary geotechnical considerations associated with the project include the presence of moisture-sensitive, fine-grained soils; temporary excavation shoring; permanent retaining walls; and foundation support and settlement. Our conclusions and recommendations for the design and construction of the project are discussed below.

5.2 Geologically Hazardous Areas

5.2.1 General

This section of this report documents potential geological hazards at the project site with respect to reporting requirements of the Critical Areas protection guidance provided in the City of Camas Municipal Code Chapter 16.59.

5.2.2 Erosion Hazard Area

This slope located to the east of the proposed water reservoir is greater than 10 feet tall and declined at about 1.5H:1V (horizontal to vertical) or 67% and classifies as an Erosion

Hazard Area per City of Camas Municipal Code. We did not observe indications of significant erosion during our June 15, 2020, site reconnaissance or on other site visits completed during the geotechnical investigation.

5.2.3 Landslide Hazard Area

Published landslide mapping of the area was reviewed (Fiksdal, 1975; and Walsh, 1987). According to Fiksdal, the project site is in a mapped area of potential instability due to the underlying geologic conditions and physical characteristics associated with steepness and therefore classifies as a Landslide Hazard Area per the City of Camas Municipal Code. During a site reconnaissance on June 15, 2020, and on subsequent site visits, GRI did not observe obvious indications of large-scale or deep-seated landslide movement such as new ground cracking, fresh scarps, or accumulations of recent landslide debris on the project site.

As discussed in Section 5.7 of this report, slope stability modeling was completed to evaluate the impact of the proposed development on the stability of the slope. The analysis indicates that the slope has an adequate factor of safety during both sustained long-term and the level of seismic loading required by the 2018 *International Building Code* (IBC).

5.2.4 Seismic Hazard Area

Based on the conditions observed in the borings completed for this investigation, the near-surface soils at the site consist of decomposed sandstone, decomposed conglomerate, or conglomerate. As a result of their density, these materials are not considered to be susceptible to liquefaction and the risk of significant ground-shaking amplification is low. These findings are consistent with mapping provided by Palmer (2004), which indicates that the near-surface soils have a very low susceptibility to liquefaction. Additional discussion of the seismic hazards at the site, including recommended seismic design parameters are provided in Section 5.5 of this report.

5.2.5 Geological Hazards Area Conclusions

The following conclusions are based on the work completed for this evaluation:

1. The slope located east of the proposed reservoir classifies as an erosion hazard area per the City of Camas Municipal Code. However, based on our observations, the erosion risk is low provided the vegetation is maintained on the slope and that grading at the top of the slope, if completed, directs stormwater away from the top of the slope. In our opinion, the project as currently designed will not adversely affect the erosion hazard.

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2. The project site is located within a landslide hazard area; however, our site reconnaissance and engineering analysis indicates that the risk of landslide is relatively low and the proposed improvements will not significantly adversely affect the overall stability of the slope under both static and seismic loading conditions.

5.3 Earthwork

5.3.1 Site Preparation

Demolition within the limits of the new structures, structural fills, or pavement and hardscape areas should include the removal of existing structures, pavements, and underground utilities. The project site is mantled with a 4- to 6-inch-thick, heavily rooted topsoil layer. Where vegetation is present, the ground surface should be stripped to remove the surface vegetation and rooted zone. Deeper stripping and grubbing depths should be anticipated to remove stumps and roots larger than about ½ inch in diameter. Strippings will not be suitable for structural fill and should only be used in landscaped areas or removed from the site. The lateral limits of stripping and grubbing should extend at least 10 feet beyond improvement areas.

To reduce the risk of disturbing the near-surface soils during demolition and stripping and grubbing activities, we recommend using hydraulic excavators equipped with smooth-cutting edges. Excavations made during demolition, stripping, and grubbing should be backfilled with structural fill prepared in accordance with the Structural Fill section of this report.

5.3.2 Subgrade Preparation and Wet Weather Construction

Following site preparation activities and any additional excavation needed to reach the planned subgrade in areas to receive fill or other improvements, the exposed subgrade should be evaluated by a member of GRI's geotechnical engineering staff. Loose, soft, or disturbed areas should either be moisture conditioned and recompacted as structural fill (dry weather conditions only) or removed and replaced with imported structural fill. Proof rolling with a loaded dump truck or other heavy, rubber-tired vehicle may be part of the evaluation.

Near-surface soils that mantle the site consist primarily of silty gravel, silty sand, or sandy silt with considerable fines (i.e., material passing the No. 200 sieve) content. These soils are sensitive to moisture content, and during wet ground or weather conditions can be easily disturbed, rutted, and weakened by construction activities. For this reason, we recommend, if possible, all earthwork activities be accomplished during the normally dry summer and early fall months. We recommend making all excavations using large hydraulic excavators equipped with smooth-cutting edges in lieu of bulldozers to prevent softening of the subgrade soils. Also, the contractor should plan the earthwork operations so that no construction equipment, e.g., bulldozers, dump trucks, etc., traffic the exposed, moisture-sensitive soils. This will require the placement of imported granular fill for working pads and/or haul roads as the excavation progresses. If the subgrade is disturbed during construction, soft, disturbed soils should be overexcavated to firm soil and backfilled with clean, granular materials.

During wet weather or wet ground conditions, it should be anticipated haul roads or granular work pads constructed of Select Granular Fill as described in this report will be necessary to provide access and protect the subgrade from damage due to construction traffic. In our opinion, a 12-inch-thick granular work pad should be sufficient to prevent disturbance of the fine-grained sand and silt subgrade by lighter construction equipment and limited traffic by dump trucks. Haul roads and other high-density traffic areas will require at least 18 inches to 24 inches of crushed rock to prevent subgrade deterioration. Haul road requirements will be minimized if work is accomplished during the driest months of the year. The performance of haul roads can usually be improved by placing a woven geotextile fabric over the fine-grained subgrade prior to placing the rock.

5.3.3 Structural Fill

In our opinion, on-site soils free of organics, debris, and cobbles less than about 6 inches in diameter are suitable for use in structural fills. As noted above, the on-site soils contain a significant amount of silt and fine-grained sand. These silty soils are moisture sensitive and can be placed and adequately compacted only during the dry summer months when they can be moisture conditioned. For construction during the wet winter and spring months, site fills should be constructed using relatively clean granular materials.

In general, approved on-site or imported, organic-free, fine-grained sand and silty soils used to construct structural fills should be placed in 9-inch-thick lifts (loose) and compacted using medium-size (48-inch-diameter), segmented-pad rollers to a density not less than 95% of the maximum dry density determined by ASTM International (ASTM) D698. Pieces of rock and cobbles larger than about 6 inches and boulders should be removed from the fill prior to compaction. In our opinion, the moisture content of fine-grained soils at the time of compaction should be controlled to within 3% of optimum. Moisture conditioning of the on-site, fine-grained sand and silty soils will be required to achieve the recommended compaction criteria. All structural fills should extend a minimum horizontal distance of 5 feet beyond the limits of the structural improvements.

Imported granular material used to construct structural fills or work pads during wet ground or wet weather can consist of relatively clean, granular material with a maximum particle size of 4 inches and not more than about 7% passing the No. 200 sieve (washed analyses), such as sand, sand and gravel, or crushed rock. Gravel Borrow meeting the requirements of Section 9-03.14(1) of the 2022 Washington State Department of Transportation (WSDOT) *Standard Specifications* can be used for this purpose. The first lift of granular-fill material placed over a silty subgrade should be in the range of 12-inch- to 18-inch-thick (loose) and subsequent lifts should be 12-inch-thick (loose). All lifts should be compacted to at least 95% of the maximum dry density determined by ASTM D698 using a medium-weight (48-inch-diameter drum), smooth, steel-wheeled, vibratory roller. Generally, compaction should be achieved by a minimum of four passes with the roller.

5.4 Excavations

5.4.1 General

We understand that the existing 0.5 MG reservoir will be removed as part of the proposed improvements and that the base of the existing reservoir is at about elevation 432 feet. The new pump station will be located within the footprint of the existing 0.5 MG reservoir and the base of the pump station pump cans will be between elevation 425 feet to elevation 430 feet and about 10 feet below the surrounding final site grades. The 1.5 MG reservoir is located approximately 30 feet to the north of the existing 0.5 MG reservoir and the planned pump station. According to the 1971 as-built drawings, the top of the foundation elevation for the 1.5 MG reservoir is at an elevation of 431.85 feet. The new 0.5 MG reservoir will be constructed south of the existing 0.5 MG reservoir. A permanent retaining wall will be used to support the excavations necessary for the new reservoir and pump station.

The method of excavation and design of temporary shoring, trench support, and groundwater-management system are the responsibilities of the contractor. The means methods and sequencing of construction operations and site safety are also the responsibilities of the contractor. We recommend that the contractor submit an excavation and dewatering plan prepared by a Washington-registered professional engineer or hydrogeologist for review by the owner and engineer. The information provided below is for use by the owner and engineer and should not be interpreted to mean GRI is assuming responsibility for the contractor’s actions, site safety, or design.

It has been our experience that good trench excavation, shoring, and backfilling procedures will reduce, but may not eliminate, the settlement at the ground surface following backfilling.

5.4.2 Excavation and Groundwater Control

The explorations completed for this investigation encountered decomposed sandstone and decomposed conglomerate consisting of loose to medium-dense, silty sandy; stiff to very stiff sandy silt, or medium-dense to very dense, silty gravel. Cobbles were encountered within the decomposed conglomerate. We anticipate that fill of unknown composition and density/consistency surrounds the existing 0.5 MG reservoir.

Based on our experience with similar materials in the region, we anticipate that the fill, weathered sandstone, and weathered conglomerate can be excavated using conventional excavation methods, such as a large (e.g., a 75,000-pound machine with more than 270 hp) hydraulic excavator equipped with rock teeth (i.e., replacement hardened-steel points). Cobbles were encountered in the explorations and boulders and less-weathered zones of conglomerate rock and sandstone are present in the area. The contractor should be prepared to handle and excavate these materials. We recommend that the contract documents include unit pricing for removal of boulders and bedrock.

At the time of drilling, groundwater was observed at a depth of about 26.5 feet (elevation 422.5 feet) in boring B-1 and at a depth of 29 feet (elevation 408.5 feet) in boring B-2. Groundwater was encountered at about 36 feet below the original site grades (elevation 440 feet) in a boring advanced on April 8, 1971, during design of the 1.5 MG reservoir. Depending on the time of year and precipitation, it is our opinion that groundwater could rise and be encountered at shallower depths and be encountered in the deeper excavations made for the project. Furthermore, we anticipate shallow, perched-groundwater conditions may develop above the silty soils, especially during periods of wet weather.

Control of groundwater, if encountered, will depend on the soils and groundwater levels encountered in the excavation and the contractor/owner’s approach to the work. To minimize dewatering requirements, we recommend construction of the deeper structures occur during the late-summer and early-fall months when the groundwater levels are near their seasonal lows. In our opinion, perched groundwater seepage entering from the sides of the shored excavations can be managed by pumping from sumps in the bottom of the excavation.

To provide a level and firm surface to place the foundations and facilitate any necessary dewatering, if required, we recommend placing a minimum-1-foot thickness of free-draining base course at the bottom of the excavation. All soft or loose material present in the bottom of the excavation should be removed prior to placement of the base course and the prepared subgrade should be observed by GRI. The base-course material should consist of clean, open-graded, angular, crushed rock with a maximum size of about 2.5 inches and containing less than 2% passing the No. 200 sieve (washed analysis). Permeable ballast material meeting the requirements of Section 9-03.9(2) of the 2022 WSDOT *Standard Specifications* can be used for this purpose. Base-course material should be placed in a maximum of 12-inch-thick lifts and compacted until well keyed. The open-

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graded base-course material may need to be capped with about 3 inches to 6 inches of well-compacted, 1½- or ¾-inch-minus, crushed rock to serve as a leveling course and choke off the surface of the coarser-graded stabilization material to facilitate placement of the wet-well base. If the subgrade consists of sand or silt, a woven geotextile fabric meeting the requirements for soil stabilization in Table 3 of Section 9-33.2 of the 2022 WSDOT *Standard Specifications* should be placed over the subgrade prior to placing the stabilization material.

5.4.3 Temporary Excavation Slopes and Shoring

5.4.3.1 Temporary Excavation Slopes

Temporary excavations may be necessary to demolish the existing 0.5 MG reservoir and construct the new pump station. Temporary excavations will likely encounter fill or decomposed sandstone or conglomerate. In our opinion, the fill should be classified as Type C soil according to current Occupational Safety and Health Administration (OSHA) regulations, while the decomposed sandstone and conglomerate would classify as Type B soil. If groundwater seepage is present, all soil within the excavation depth would be classified as Type C soil. Per OSHA regulations, the maximum temporary excavation slope in Type B soils is 1H:1V, and the maximum temporary excavation slope in Type C soils is 1½H:1V. Construction equipment, vehicle parking, material lay down, etc., should not be allowed within 10 feet of the top of slopes.

Depending on the actual conditions encountered, flatter slopes may be necessary to reduce the risk of instability, particularly if groundwater is encountered. If groundwater seepage is encountered, a blanket of relatively clean, well-graded, 2- to 4-inch-minus crushed rock placed against the slopes may be required to reduce the risk of running soils and sloughing. The required thickness of the granular blanket should be evaluated based on the actual conditions but could be in the range of 1 foot to 2 feet.

Additional measures that should be implemented to reduce the risk of localized failures of temporary slopes include (1) using woven geotextile fabric or plastic sheeting to protect the exposed cut slopes from surface erosion; (2) providing positive drainage away from the tops and bottoms of the cut slopes; (3) constructing and backfilling walls as soon as practical after completing the excavation; and (4) periodically monitoring the area around the top of the excavation for evidence of ground cracking. It must be emphasized that following these recommendations does not guarantee sloughing or movement of the temporary cut slopes will not occur; however, the measures should serve to reduce the risk of major slope failures. It should be realized, however, that blocks of ground and/or localized slumps may tend to move into the excavation during construction. In our opinion, all temporary excavation slopes should be periodically observed by a qualified geotechnical engineer.

5.4.3.2 Shoring Criteria

We anticipate engineered shoring systems will be used for temporary excavation support in areas where existing infrastructure is present and/or site access constraints do not permit the use of open-cut excavations. It is common practice in the region to use shoring systems consisting of soldier pile and lagging, either cantilevered or with tieback anchors, or potentially a soil nail wall. The use of tiebacks or soil nail walls may not be feasible due to the proximity of property lines or other features behind the proposed wall.

The design of temporary shoring systems depends on the total magnitude of forces that the system is designed to resist and the tolerable yielding of the system and the surrounding ground. The pattern and intensity of the lateral earth pressures on the shoring wall will be governed by the height of the wall, soil type, the degree to which the walls are structurally supported, surcharge loads behind the wall, and whether the walls are drained. The lateral earth pressure diagram on Figure 3 can be used for the design of a cantilevered shoring system with a backslope of up to 1½H:1V or flatter. Cantilevered shoring wall systems are typically feasible where the retained height is relatively small and where the shoring can be allowed to yield somewhat into the excavation during construction and that settlement behind the wall system can be tolerated. The lateral earth pressure criteria on Figure 4 can be used for tieback shoring with a backslope of about 1½H:1V or flatter. Tieback shoring is typically required for taller walls or in areas where minimizing settlement behind the walls is important, such as where an existing structure, road, or other critical infrastructure element is present.

If a soldier pile and lagging wall are used, we anticipate the soldier piles will consist of steel H-pile sections placed into drilled shafts backfilled with either controlled density fill (CDF) or pumpable lean concrete. The subsurface explorations completed at the site encountered silty sand and sandy silt (decomposed sandstone), silty gravel (decomposed conglomerate), or predominately decomposed conglomerate. Based on the conditions observed in our explorations, cobbles and potentially boulders, as well as more cemented zones and less decomposed zones of conglomerate rock, should be anticipated within a predominately decomposed conglomerate unit. Groundwater was encountered at about elevation 408.5 feet in boring B-2 and may be encountered during the construction of the soldier piles.

Caving conditions may occur during the construction of the soldier piles, which may require the use of temporary casing. In addition, the contractor should anticipate that different tooling may be required to advance shafts through the gravel and cobble material, more cemented and less decomposed conglomerate rock, and to remove boulders. Although not observed in our explorations, open-work zones of gravel and cobbles are often encountered in the conglomerate unit. Therefore, the possibility of CDF or pumpable lean concrete loss should be anticipated during the installation of the shafts. Upon completion of drilling and setting the steel section, the temporary casing should be withdrawn as the CDF or pumpable lean concrete is placed; however, the top of the CDF or pumpable lean concrete should be maintained at least 5 feet above the bottom of the casing. We recommend placing the CDF or pumpable lean concrete using tremie methods. The bottom of the tremie pipe should be maintained at least 4 feet below the top of the CDF or pumpable lean concrete. The soldier pile specifications should require that the contractor assume that grout take will be at least 120% of the theoretical volume of the drilled shaft.

We recommend that all tieback anchors for a tieback soldier pile shoring system (if feasible) develop their pull-out resistance beyond a no-load zone defined by a plane that extends a horizontal distance equal to H/4 (where H is the height of the wall) or 5 feet (whichever is greatest) from the bottom of the excavation into the retained earth and then upwards at an angle of 30° from vertical. The no-load zone is presented graphically on Figure 4. Verification tests should be completed for at least one anchor per level. Verification anchor tests should be conducted to at least 200% of the design anchor load. The results of the tests will be used to review and revise, if necessary, the anchor design criteria. In addition, each production anchor should be proof tested to at least 133% of the design load for temporary anchors. The temporary shoring contractor and designer should have a proven record of successful shoring and tieback installations in similar materials.

If shoring is required, we recommend the following monitoring and performance provisions be included in the project specifications.

1. Horizontal movement of the shoring system in the vicinity of adjacent streets or property lines should be accurately measured and recorded at each stage of the excavation by the project surveyor or contractor's surveyor. Horizontal movement should be measured at the top and at each intermediate bracing level, on at least every second soldier pile. Settlement of the ground surface near adjacent streets should be monitored at a minimum spacing of 25 feet along the curb line closest to the excavation.

2. Horizontal movement of the shoring system should not exceed ½ inch toward the excavation.

3. Lagging should be installed, and any voids backfilled using controlled-density fill, if necessary, as the excavation proceeds.

4. The excavation should not extend more than about 3 feet below a bracing level until the tiebacks, lagging, and backfill at that level are in place.

5. The excavation for cantilever shoring should not extend more than 3 feet below the depth of lagging.

5.4.4 Backfill and Compaction Criteria

Backfill placed in utility-trench excavations and the annulus between the embedded structures and the excavation sides should consist of sand, sand and gravel, or crushed rock with a maximum size of up to 1½ inches and not more than 10% passing the No. 200 sieve (washed analysis). An example of a material that satisfies this requirement is Gravel Backfill for Pipe Zone Bedding meeting the requirements of Section 9-03.12(3) of the 2022 WSDOT Standard Specifications. The granular material should be placed in lifts and compacted to at least 95% of the maximum dry density determined by ASTM D698. Lift thicknesses should be no thicker than 8 inches for hand-operated equipment and 12 inches for trackhoe-mounted vibratory compactors (hoepack). The groundwater level should be maintained at least 2 feet below the backfill surface while the excavation is being backfilled. Flooding or jetting the backfill with water to achieve the recommended compaction should not be permitted.

Compaction techniques can significantly affect the actual lateral earth pressure. Overcompaction of the backfill behind cast-in-place concrete walls should be avoided. We recommend compacting backfill within 5 feet of concrete walls to at least 95% of the maximum dry density determined by ASTM D698 using hand-operated, vibratory-plate compactors. Heavy compactors and large pieces of construction equipment should not operate within 5 feet of any of the concrete walls to avoid the buildup of excessive lateral pressures.

5.5 Seismic Considerations

5.5.1 General

We understand the project will be designed using both the American Water Works Association document AWWA D100-11, *Welded Carbon Steel Tanks for Water Storage* and the 2018 IBC. Both the AWWA Standard D100-11 and the 2018 IBC are based on the American Society of Civil Engineers (ASCE) 7-16 document, titled *Minimum Design Loads for Buildings and Other Structures*.

The IBC design methodology uses two spectral response parameters, S_S and S_1, corresponding to periods of 0.2 second and 1.0 second, to develop the Risk-Targeted Maximum Considered Earthquake (MCE_R) response spectrum. The spectral response parameters were obtained from the U.S. Geological Survey Hazard Response Spectra Curves for the coordinates of 45.5919° N latitude and 122.4154° W longitude. Based on soil characteristics, the soil column at the site would be classified as IBC Site Class D. The S_S and S_1 parameters identified for the site are 0.82 and 0.35 g, respectively. These spectral response parameters are adjusted for Site Class with the 0.2- and 1-second period site coefficients, F_a and F_v, based on the soil profile in the upper 100 feet. This spectrum is designated the MCE_R-level spectrum. The design-level response spectrum is calculated as two-thirds of the Site Class-adjusted MCE_R-level spectrum.

We recommend using the code-based 0.2- and 1-second period site coefficients, F_a and F_v, for Site Class D to estimate the ground surface MCE_R spectrum. The F_a and F_v factors are 1.17 and 1.95, respectively. The spectral values are generally based on a damping ratio of 5%. To evaluate water sloshing within the tank at a damping ratio of 0.5%, the design spectrum for Site Class D can be multiplied by a factor of 1.5. The code-based MCE_R and design response spectra values are tabulated below.

Table 5-1: 2018 IBC SEISMIC DESIGN RECOMMENDATIONS, 5% DAMPING

Seismic Variable	Recommended Value
Site Class	D
MCE_R 0.2-Sec Period Spectral Response Acceleration, S_{MS}	0.96 g
MCE_R 1.0-Sec Period Spectral Response Acceleration, S_{M1}	0.69 g
Design 0.2-Sec Period Spectral Response Accelerations, S_{DS}	0.64 g
Design 0.2-Sec Period Spectral Response Accelerations, S_{D1}	0.46 g

5.5.2 Other Seismic Considerations

In our opinion, the potential for earthquake-induced fault rupture at the ground surface is low unless occurring on a previously unknown or unmapped fault. Based on the location of the site and the grain size and stiffness of the soil beneath the site, it is our opinion the risk for liquefaction and liquefaction-induced lateral spreading, settlement, and subsidence is low. The risk of tsunamis or seiches at the site is absent. Additional discussion regarding the static and seismic stability of the slope located southeast of the reservoir is provided in Section 5.7 of this report.

5.6 Structures

5.6.1 Reservoir Foundation Support

We understand that foundation support of the reservoir will be provided by a conventional concrete ring-type continuous footing and center interior spread footings and that the maximum gravity ring-type foundation loads will be on the order of 4,000 pounds per foot and that the maximum gravity interior spread footing loads will be on the order of 6,000 pounds. In our opinion, foundation support for the reservoir can be provided using these foundation types assuming the structure can tolerate some settlement as described in the settlement section below.

Footings should be established at a minimum depth of 18 inches below the lowest adjacent finished grade and the width of the footings should not be less than 24 inches. To provide uniform foundation support and to facilitate foundation drainage, we recommend the subgrade for the tank floor and footings and extending 5 feet beyond the tank footprint be overexcavated a minimum depth of 24 inches and backfilled with drain rock. The foundation subgrade should be evaluated by a qualified geotechnical engineer prior to placing the drainage layer. Any soft areas should be overexcavated to firm soil and backfilled with crushed rock.

The drain rock should consist of a well-graded angular crushed rock with a maximum size of 1½ inches and less than 2% passing the No. 200 sieve (washed analysis). Material meeting the requirements for Gravel Backfill for Drains in Section 9-03.12(4) of the of the 2022 WSDOT *Standard Specifications* can be used for this purpose. The drainage layer should be provided with rigid 4-inch-diameter perforated drainage pipes designed for the imposed loads of the reservoir or construction traffic, whichever is greater. The drainage layer may be capped with 3 inches to 6 inches of Crushed Surfacing Top Course, meeting the requirements of Section 9-03.9(3) of the 2022 WSDOT *Standard Specifications* to facilitate compaction of the drain rock and limit contamination from construction activities prior to constructing the floor slab. All fill placed beneath the tank should be compacted to at least 95% of the maximum dry density as determined by ASTM D698.

For reservoir subgrade prepared as discussed above, spread footings can be designed to impose an allowable soil bearing pressure of 2,500 pounds per square foot (psf). This value applies to the total of dead load plus frequently and/or permanently applied live loads and can be increased by one-third for the total of dead, live, wind, and seismic loads. The allowable soil bearing pressure is a net value and applies to the structural loads imposed by the tank structure and the load on the roof. The gross footing bearing pressure, including the water load and structural loads, will be less than about 4,000 psf. The allowable bearing pressure includes a factor of safety of at least 3 on the estimated ultimate bearing pressure.

The total settlement of the continuous ring footings and interior spread footings due to wall and roof loads is estimated to be on the order of ½ inch. Total settlement in the middle of the tank after filling with water is estimated to be in the range of 1 inch to 2 inches. Settlement at the edge of the tank is estimated to be ½ to ⅔ of the settlement in the middle of the tank. Some differential settlement around the perimeter should be anticipated due to variations in the soil properties. We anticipate that differential

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settlement around the perimeter of the tank will be less than 1 inch. In our opinion, the differential settlement will be gradual and can be estimated to be a linear change across the diameter of the tank, i.e., no abrupt differential is anticipated over short distances. The majority of the tank floor and footing settlement will occur rapidly as the tank is filled with water.

Lateral loads (seismic, soil, etc.) can be resisted partially or completely by frictional forces developed between the base of footings or tank bottom and underlying crushed rock. The total frictional resistance between the tank and the underlying material is the normal force times the coefficient of friction between the crushed rock and the base of the footing and reservoir. We recommend ultimate values for the coefficient of friction of 0.50 and 0.40 for cast-in-place concrete and steel, respectively, placed over a minimum of 12 inches of crushed rock fill. If additional lateral resistance is required, passive earth pressure against the perimeter footing and the walls of the tank can be computed on the basis of an equivalent fluid having a unit weight of 250 pounds per cubic foot (pcf). This passive earth pressure assumes the backfill for the footings is placed as granular structural fill and does not slope downward away from the tank.

The embedded reservoir walls must be fully drained. The drainage system should consist of a minimum 2-foot-wide zone of free-draining granular material, such as Gravel Backfill for Drains as described in Section 9-03.12(4) of the 2022 WSDOT *Standard Specifications*. A minimum 4-inch-diameter rigid, perforated drainpipe should be provided near the bottom of the reservoir foundation. A non-woven geotextile, meeting the requirements for Moderate Survivability, in Table 1 of Section 9-33.2(1) of the 2022 WSDOT *Standard Specifications* is recommended between the free draining backfill and general site fill to reduce the risk of contamination of the wall drain system.

5.6.2 Booster Pump Station and Generator Pad Mat Foundations

Based on information provided by the team, the booster pump station will be supported by a 20-foot-wide and 36-foot-long mat foundation with thickened edges or an inverted T-stem wall. The average sustained bearing pressure (dead plus real live loads) on the mat foundation subgrade is estimated to be less than 300 pounds per square foot. A generator, weighing approximately 45,000 pounds, is planned to the west of the booster pump station. The generator will be supported by a 215-square-foot mat foundation.

We anticipate that the mat foundations for the new booster pump station and generator will be established in decomposed conglomerate or decomposed sandstone or on structural fill placed on these materials. To provide uniform support, the mat foundation should be underlain by a minimum of 6 inches of well-graded, crushed rock with a maximum particle size of 1½ inches and containing less than 8% passing the No. 200 sieve (washed analysis). Crushed Surfacing Base Course meeting the requirements of Section 9-03.9(3) of the 2022 WSDOT *Standard Specifications* meets these criteria and can be used to provide uniform mat foundation support. The crushed rock should be compacted to at least 95% of the maximum dry density as determined by ASTM D698.

For frost protection, the bottom of the thickened edges of the mat foundation should be embedded at least 12 inches below adjacent site grades. For the loads provided above, we estimate that settlement of the booster pump station and generator mat foundation will in the range of ¼ inch to ½ inch, with differential settlement across the length of the mat foundation on the order of half of the total settlement. It is anticipated that the settlement described above will occur during construction and as the loads are applied to the mat foundation. For evaluating point or short-term loads on the mat, a subgrade modulus of 150 pci can be considered.

Recommendations for resistance to lateral loads are provided in Section 5.5.1 of this report.

5.6.3 Pump Can Design Considerations

We anticipate that the base of the pump cans will be established in decomposed conglomerate or decomposed sandstone or on structural fill placed on these materials. The foundation subgrade for the pump cans should be prepared in accordance with Section 5.3.2 of this report. Pump can foundations established in accordance with the above criteria can be designed to impose an allowable bearing pressure of 3,000 psf. This value applies to the total of all dead plus frequently or permanently applied live loads and can be increased by one-third for the total of all loads: dead, live, and wind or seismic. We estimate the total settlement of the wet-well facility during static loads will be less than 1 inch and this settlement will occur rapidly as the wet well is installed and backfilled.

The walls of the below-grade structures (e.g., utility access holes, wet wells, and vaults) should be considered rigid and non-yielding for design purposes. We recommend lateral earth pressures be evaluated on the basis of an equivalent fluid having a unit weight of 90 pcf. This value assumes the groundwater level could rise to near the ground surface and the surrounding ground is level. This value does not include the influence of additional surface surcharge loads. Additional lateral loading induced by surcharge loads should be evaluated in accordance with the criteria shown on Figure 5.

We recommend designing below-grade structures to resist the full hydrostatic uplift pressure. The uplift force is computed by multiplying the submerged volume of the structure by the unit weight of water (62.4 pcf). Common methods used to resist the uplift force include increasing the thickness of the walls and/or base or extending the base slab beyond the sidewalls of the structure.

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Only the compacted backfill directly over the extended base slab should be considered an additional load to resist the uplift force. The effective unit weight of the submerged backfill should be evaluated using a buoyant unit weight of 60 pcf. This assumes the backfill consists of imported granular material.

5.7 Slope Stability Analysis

Slope stability analyses were completed to evaluate the potential risk of local slope instability affecting the proposed reservoir. The cross-section of the slope that was used to develop the slope stability model is oriented in a generally northwest-southeast direction through the center of the planned reservoir. At this location, the reservoir is setback approximately 30 feet from the edge of the 1½H:1V cut slope down to NW 18th Loop. The slope stability analysis was completed using a generalized limit equilibrium (GLE) analysis with the assistance of the Slide2 software developed by Rocscience, Inc. of Toronto, Ontario, Canada. The basic input for the models included the existing topography and proposed grading provided to GRI by Murraysmith, subsurface profiles disclosed by the subsurface investigations completed by GRI, correlations of soil strengths to N-values obtained during drilling, and our experience with similar soils. In our analyses, groundwater was assumed to be present at about elevation 420 feet beneath the proposed reservoir and about 20 feet below the ground surface along the slope to the southeast of the proposed reservoir.

Factors of safety against sliding were computed using Spencer's Method of Slices, which satisfies both force and moment equilibrium while assuming the resultant of interslice forces are of constant orientation throughout the sliding mass. The computed factor of safety is defined as the ratio of the forces (or moments) tending to resist sliding to the forces (or moments) tending to cause sliding within the slope. Computed factors of safety less than 1.0 indicate instability or incipient slope movement. Slopes supporting critical structures are typically designed to have an estimated factor of safety of at least 1.5 under static and 1.1 under seismic loading conditions. A horizontal pseudo-static coefficient, $k_{h,}$ of 0.23g was used to model seismic inertial loads. In our slope stability model, a uniform surcharge load of 1,500 psf was used to model the weight of the water within the replacement reservoir. Uniform surcharge load of 250 psf and 125 psf were used to model the weight of vehicular traffic around the perimeter of the reservoir and along NW 18th Loop for static and seismic loading conditions, respectively. Figures 6 and 7 show the groundwater level and locations/boundaries of soil units and associated physical properties used in our slope stability models and the minimum factor of safety for a potential failure surface that impacts the proposed tank.

The analyses indicate that potential failure surfaces, which extend back to the reservoir, have a factor of safety of at least 1.5 under static loading conditions and 1.1 under seismic loading conditions. In this regard, it is our opinion that the risk of a deep-seated failure impacting the new reservoir is low. The slope stability analyses indicate that the shallow surface of the slope is over-steepened and has a low factor and that there is a potential for surficial sloughing or raveling of the slope. Our observations indicate the slope has performed relatively well over its life and no obvious indication of sloughing was observed. However, we recommend setting back all critical yard piping and other utilities a minimum of 10 feet away from the crest of the slope.

Based on the conditions observed in the explorations completed for this study and the proximity of the site to the steep slopes to the west, it is our opinion that infiltration of significant quantities of groundwater will result in a decreased factor of safety. In this regard, stormwater infiltration is not recommended for this project.

5.8 Retaining Walls

5.8.1 General

A permanent soldier pile wall is planned near the perimeter of the proposed 0.5 MG reservoir and booster pump station. It will begin on the south side of the reservoir, extend northward to the west of both proposed structures, and end to the north of the proposed pump station. The soldier pile wall will be up to 19 feet tall and will be cantilevered. Slopes behind the proposed reservoir may be inclined up to 2H:1V. The project may also include shorter modular blocks or mechanically stabilized earth fill walls as necessary.

5.8.2 Cantilevered Soldier Pile Wall

A lateral earth pressure diagram for the design of the permanent cantilevered soldier pile wall is provided on Figure 8 for walls with level backslope, walls with backslopes of about 2H:1V, and for walls with backslopes of 3H:1V. The lateral earth pressure diagram assumes that groundwater is at about elevation 420 feet at the location of the wall.

The lateral earth pressure diagram includes active earth pressures, uniform surcharge earth pressures, dynamic lateral earth pressure increment, and passive earth pressures. The dynamic lateral earth pressure increment should be added to the static lateral earth pressure for design load cases, including seismic. The soldier pile wall may be subjected to the influence of surcharge loading, and the wall should be designed to accommodate this additional horizontal pressure. It is typical to accommodate traffic and typical construction equipment loading with a uniform vertical surcharge pressure, q_s, of 250 psf, for static loading conditions. Non-uniform surcharge loads, such as from soil stockpiles or construction equipment, can be estimated using the criteria on Figure 5. Transient surcharge loads, such as wheel loading, do not need to be included in the seismic-loading case. The active earth pressure and surcharge lateral earth pressures should be applied

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over the width of the wall in the portion of the wall that is lagged and over the soldier pile drilled-shaft diameter where lagging is not used between soldier piles.

The passive earth pressure provided on Figure 8 assumes that the ground surface in front of the wall is flat and has been reduced by a factor of safety of 1.5. The passive earth pressure should be applied over two pile soldier pile diameters or the spacing of the soldier piles, whichever is less.

We recommend installing permanent drainage behind the lagged portion of the wall to reduce the risk of perched hydrostatic groundwater developing. Typical drainage systems for similar applications have consisted of 16-inch-wide drainage panels spaced about every 6 feet to 8 feet along the embedded wall or between each set of soldier piles. The drainage strips should extend to the base of the wall fascia, where any water would be collected in a perforated plastic pipe and discharged away from the wall.

Additional discussion regarding the construction of soldier pile walls is provided in Section 5.3.3.2 of this report.

5.8.3 Modular Block Walls

5.8.3.1 General

Design lateral earth pressures for embedded walls will depend on the drainage condition behind the wall and the ability of the wall to yield. We recommend a drainage system be provided behind the wall. Modular block or mechanically stabilized earth walls that can yield or rotate slightly away from the backfill can be designed using active earth pressures.

5.8.3.2 Foundation Design

The base of all modular-block or mechanically stabilized earth walls should be embedded a minimum of 1 foot below adjacent site grades and founded on firm, on-site soil, or structural fill placed above these on-site materials. Excavation for the walls should be made with excavators equipped with a smooth-edged bucket and the wall subgrade should be evaluated by a member of GRI's geotechnical engineering staff. If soft soils are encountered at the base of the excavation, it will be necessary to overexcavate and replace the unsuitable materials with well-graded, crushed rock, such as Crushed Surfacing Base Course meeting the requirements of Section 9-03.9(3) of the 2022 WSDOT *Standard Specifications*. All prepared foundation-bearing surfaces should be free of loose soil and water. The modular block or the facing units of mechanically stabilized earth walls should be founded on a minimum-6-inch thickness of compacted crushed rock to provide uniform support.

Provided the subgrade is prepared as described above, retaining walls can be designed on the basis of an allowable bearing pressure of 2,000 psf. The total settlement of the modular-block or mechanically stabilized earth retaining walls are estimated to be less than 1 inch.

5.8.3.3 Lateral Earth Pressures

Modular-block or mechanically stabilized earth retaining walls free to yield and for drained conditions can be designed using an equivalent fluid unit weight of 35 pcf for level backfill and 50 pcf for slopes inclined at 2H:1V or flatter. Additional lateral pressures due to surcharge loading in the backfill area, such as vehicle or construction traffic or soil stockpiles, can be estimated using the guidelines provided on Figure 5. The dynamic lateral earth pressure increment for yielding walls can be estimated using an equivalent fluid unit weight of 6 pcf and 23 pcf for walls with level backslopes and walls with backslopes inclined at 2H:1V. The dynamic lateral earth pressure increment should be added to the static lateral earth pressure. Transient surcharge loads, such as wheel loads, do not need to be included in the seismic-loading case.

If the internal design of the retaining wall is completed using a wall-design software program, the following soil parameters in Table 5-1 can be used for the design of modular-block walls and mechanically stabilized earth walls, assuming on-site soils are used to raise site grades and backfill behind the wall and this material is compacted as structural fill. A peak horizontal ground acceleration of 0.45g can be used for evaluating seismic loading. Lateral earth pressures due to surcharge loading should be considered, as discussed above.

Table 5-2: MODULAR BLOCK OR MECHANICALLY STABILIZED EARTH WALL SOIL DESIGN PARAMETERS

Soil Property	Wall Backfill	Retained Soil	Foundation Soil
Unit Weight, pcf	130	125	125
Friction Angle	36	35	35
Cohesion, psf	0	0	0

5.8.3.4 Resistance to Lateral Loads

Lateral loads (seismic, soil, etc.) can be resisted partially or completely by frictional forces developed between the base of the wall foundation and underlying crushed rock. Assuming a minimum-6-inch-thick leveling course of compacted crushed-rock fill placed over foundation subgrade, we recommend an ultimate value for the coefficient of friction of 0.35 for precast concrete block facing elements and a coefficient of friction of 0.50 for gabion basket facing elements. If additional lateral resistance is required, passive earth pressure against the embedded portion of the wall can be computed on the basis of an equivalent fluid having a unit weight of 250 pcf. This passive earth pressure assumes the backfill for the footings is placed as granular structural fill and does not slope downward away from the retaining wall.

5.8.3.5 Wall Backfill and Compaction Criteria

The use of on-site soils for wall backfill will only be practical during periods of dry weather or dry conditions when the moisture content of the on-site soils can be maintained near optimum. Furthermore, it will be necessary to screen gravels, cobbles, and boulder materials greater than about 2 inches if the on-site soils will be used for backfill in the reinforced zone of mechanically stabilized earth walls. If used, an imported backfill for modular-block walls should consist of Gravel Backfill for Walls as described in Section 9-03.12(2) of the 2022 WSDOT *Standard Specifications*. Imported backfill for mechanically stabilized earth walls, if used, should consist of Gravel Borrow for Structural Earth Wall as described in Section 9-03.14(4) of the 2022 WSDOT *Standard Specifications*. Wall backfill should be compacted to at least 95% of the maximum dry density determined by ASTM D698. Heavy compactors and large pieces of construction equipment should not operate within 5 feet of any backs of modular-block- or mechanically stabilized earth wall-facing units to avoid the buildup of excessive lateral pressures. Compaction close to the backs of modular-block- or mechanically stabilized earth wall-facing units should be accomplished using hand-operated vibratory-plate compactors.

Drainage of the wall backfill is an essential element of wall design. Drainage requirements depend on the type of backfill used. If on-site soil is used as backfill, we recommend a full-height drainage blanket at the back of the mechanically stabilized earth wall-reinforcement zone, a drainage blanket at the base of the wall-reinforcement zone, and a vertical drainage blanket between the backfill and the wall's facing units. Figure 9 shows the recommended drainage for a mechanically stabilized earth wall constructed of on-site soils. The drainage blankets behind the reinforced zone and the facing units should be a minimum of 18 inches wide and extend the full height of the wall. The drainage blanket at the base of the wall should be at least 12 inches thick. All drainage blankets behind and under the wall should be interconnected with each other and consist of open-graded, angular, crushed rock with a maximum size of 1 inch and not more than about 2% passing the No. 200 sieve (washed analysis). Crushed rock meeting the gradation requirements for Gravel Backfill for Drains in Section 9-03.12(4) of the 2022 WSDOT *Standard Specifications* is suitable for this purpose. A minimum-4-inch-diameter perforated drainpipe should be placed at the bottom of the drainage blanket located behind the zone of reinforcement and at the bottom of the drainage blanket behind the wall's facing units. The perforated drainpipe should be surrounded by a minimum of 12 inches of open-graded, angular, crushed rock encapsulated with non-woven geotextile fabric, such as Mirafi 160N, meeting the requirements for moderate survivability in Section 9-33.2 of the 2022 WSDOT *Standard Specifications*. If imported granular backfill is used for wall construction, only the drainpipe system behind the reinforcement zone is required. For modular-block walls, a full-height drainage blanket should be placed behind the modular blocks as described above.

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6 DESIGN REVIEW AND CONSTRUCTION SERVICES

We welcome the opportunity to review and discuss construction plans and specifications for this project as they are being developed. In addition, GRI should be retained to review all geotechnical-related portions of the plans and specifications to evaluate whether they are in conformance with the recommendations provided in our report. To observe compliance with the intent of our recommendations, the design concepts, and the plans and specifications, it is our opinion all construction operations dealing with earthwork, retaining walls, foundations, and pile installations should be observed by a GRI representative. Our construction-phase services will allow for timely design changes if site conditions are encountered that are different from those described in our report. If we do not have the opportunity to confirm our interpretations, assumptions, and analyses during construction, we cannot be responsible for the application of our recommendations to subsurface conditions different from those described in this report.

7 LIMITATIONS

This report has been prepared to aid the project team in the design of this project. The scope is limited to the specific project and location described within this report. Our project description represents our understanding of the significant aspects of the project relevant to earthwork and design and construction of the new booster pump station and replacement reservoir. In the event any changes in the design and location of the project elements as outlined in this report are planned, we should be given the opportunity to review the changes and modify or reaffirm the conclusions and recommendations of this report in writing.

The conclusions and recommendations in this report are based on the data obtained from the subsurface explorations at the locations shown on Figure 2 and other sources of information discussed in this report. In the performance of subsurface investigations, specific information is obtained at specific locations at specific times. However, it is acknowledged variations in subsurface conditions may exist between exploration locations. This report does not reflect variations that may occur between these explorations. The nature and extent of variation may not become evident until construction. If during construction, subsurface conditions differ from those encountered in the explorations, we should be advised at once so we can observe and review these conditions and reconsider our recommendations where necessary.

Submitted for GRI,

Matthew S. Shanahan, PE
Principal

Brian A. Bennetts, PE
Senior Engineer

This document has been submitted electronically.

8 REFERENCES

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Palmer, S. P., Magsino, S. L, Poelstra, J. L., and Niggeman, R. A., 2004, Alternative liquefaction susceptibility map of Clark County, Washington, based on Swanson's groundwater model: State Department of Natural Resources, Division of Geology and Earth Resources.

Phillips, W.M., 1987, Geologic map of the Vancouver quadrangle, Washington and Oregon: Washington Department of Natural Resources, Division of Earth Resources Open File Report 87-10.

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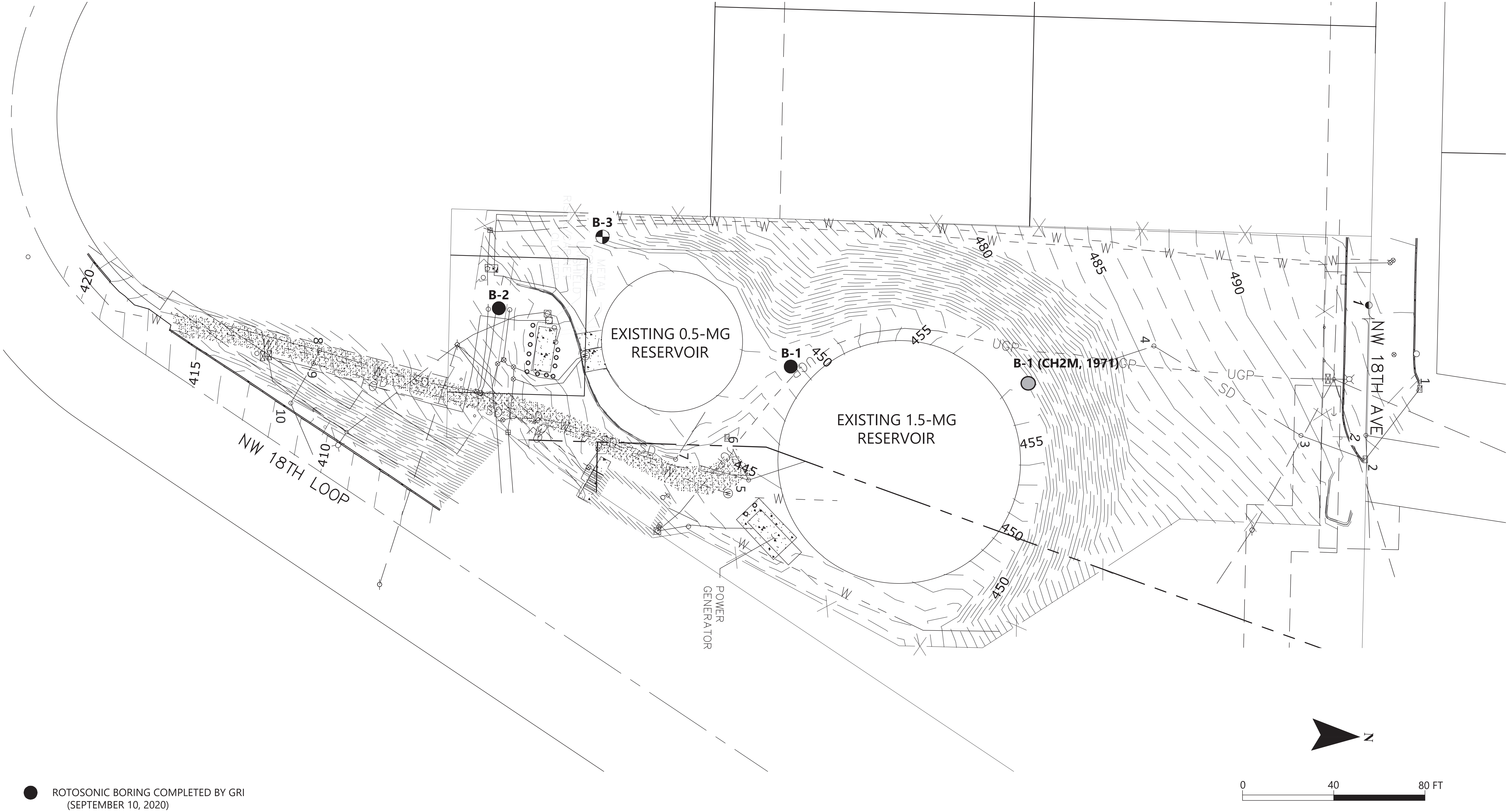
MURRAYSMITH
LOWER PRUNE HILL BOOSTER PUMP
STATION IMPROVEMENTS

VICINITY MAP

OCT. 2021 JOB NO. W1277 FIG. 1

Exhibit 10 SPRV22-06



● ROTOSONIC BORING COMPLETED BY GRI
(SEPTEMBER 10, 2020)

◐ MUD ROTARY BORING COMPLETED BY GRI
(JULY 7, 2021)

○ BORING COMPLETED BY CH2M
(APRIL 1971)

SITE PLAN FROM FILE BY CITY OF CAMAS & MURRAYSMITH, 2020

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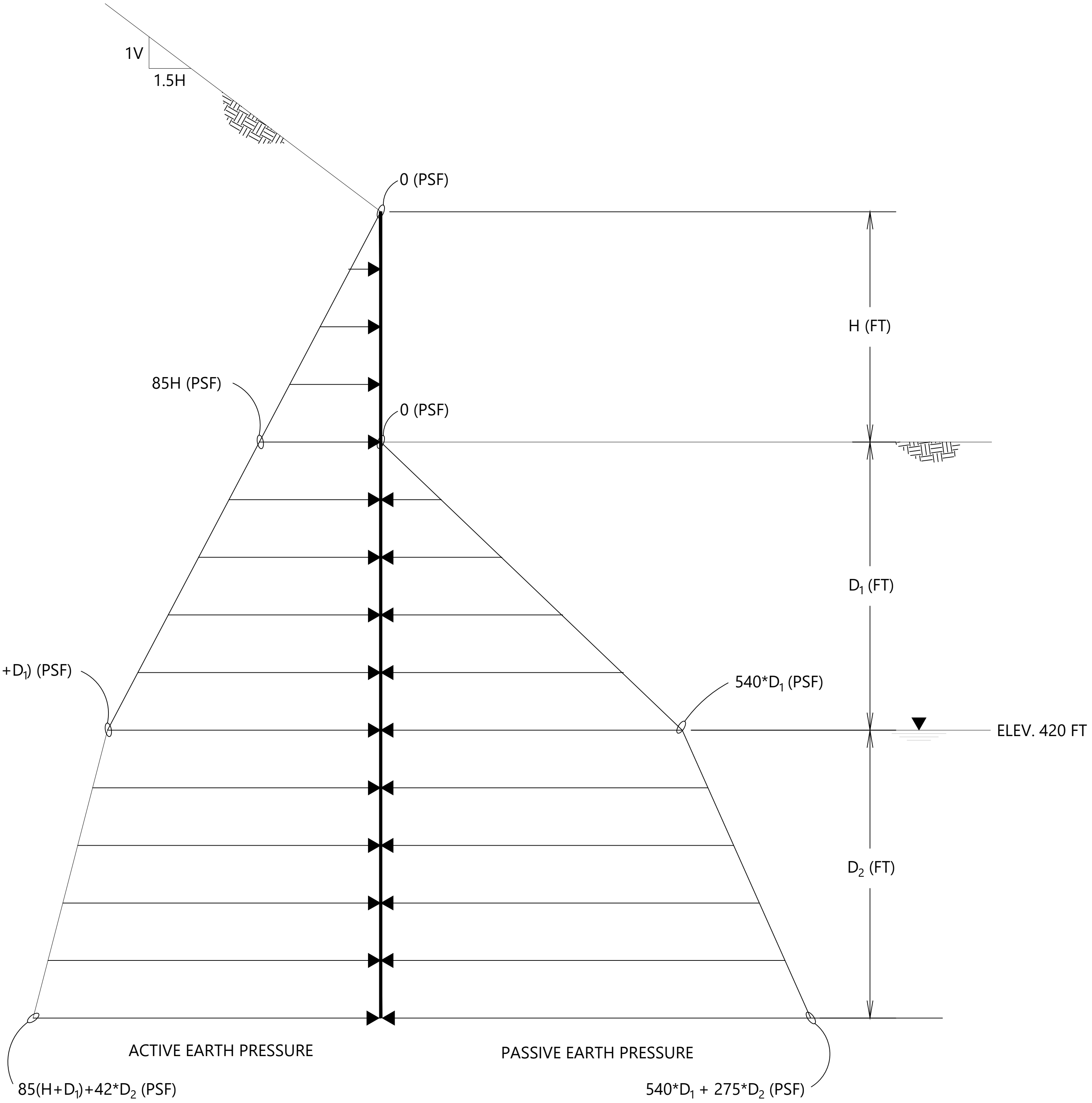
SITE PLAN

OCT. 2021 JOB NO. W1277 FIG. 2

Exhibit 10 SPRV22-06



NOTES:

1) LATERAL EARTH PRESSURES DIAGRAM IS FOR A TEMPORARY CANTILEVERED SOLDIER PILE SHORING WALL WITH A $1\frac{1}{2}$H:1V BACKSLOPE.

2) ACTIVE EARTH PRESSURE CAN BE ASSUMED TO ACT OVER THE ENTIRE EXPOSED WALL AREA AND OVER THE WIDTH OF THE SOLDIER PILE BELOW THE LAGGING.

3) THE DESIGN GROUNDWATER TABLE AT THE WALL LOCATION IS ASSUMED TO BE AT ELEVATION 420 FEET. THE DISTANCE D_1 IS THE DISTANCE BETWEEN THE GROUND SURFACE AND ELEVATION 420 FEET. THE DISTANCE D_2 IS THE DISTANCE BETWEEN THE BOTTOM OF THE PILE AND ELEVATION 420 FEET.

4) THE PASSIVE EARTH PRESSURE SHOULD BE ASSUMED TO ACT OVER TWO SOLDIER PILE DIAMETERS OR THE SOLDIER PILE SPACING, WHICHEVER IS LESS.

5) DRAWING NOT TO SCALE.

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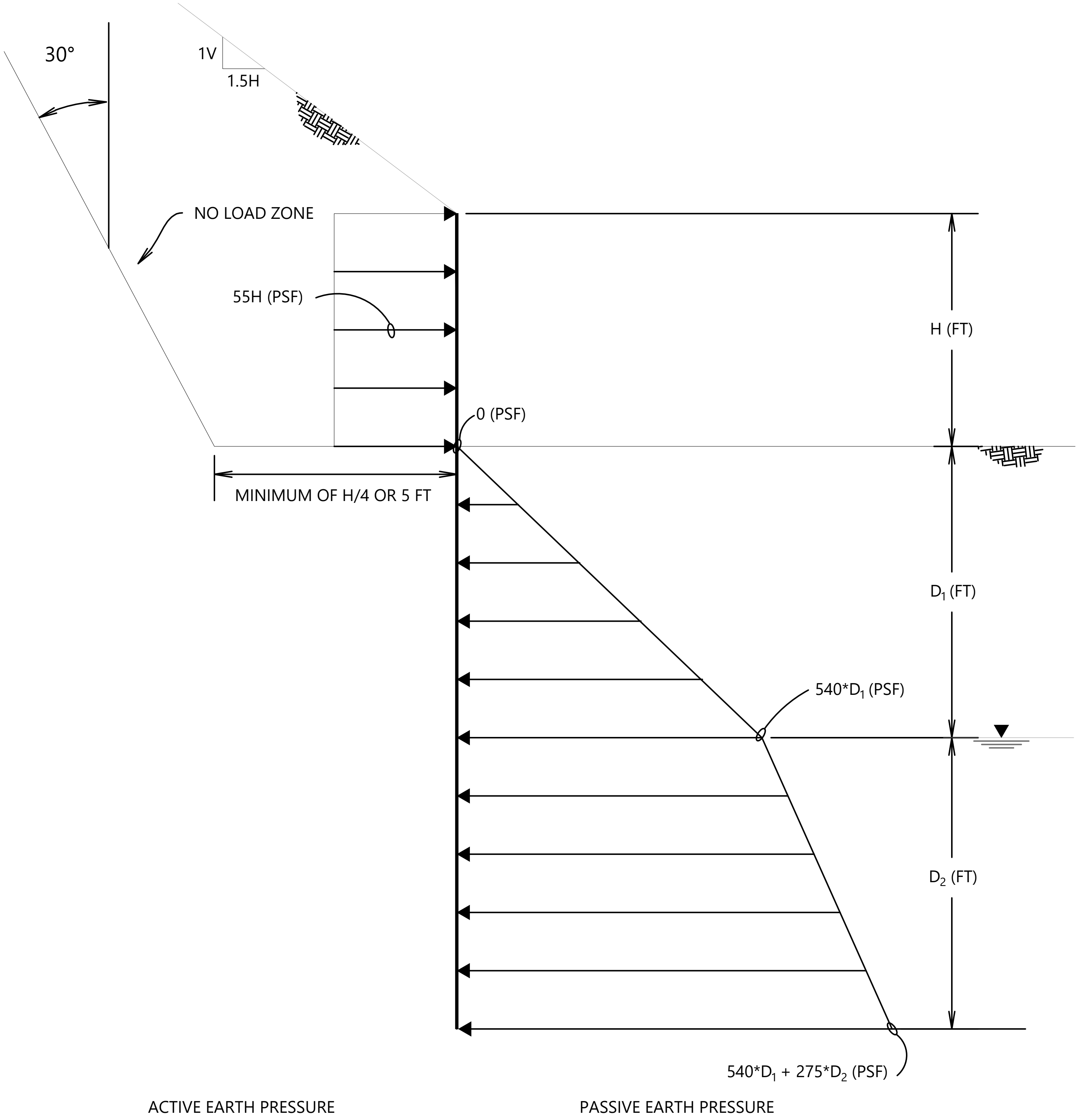
LATERAL EARTH PRESSURES FOR CANTILEVERED SHORING

OCT. 2021 JOB NO. W1277 FIG. 3

Exhibit 10 SPRV22-06



NOTES:

1) LATERAL EARTH PRESSURES DIAGRAM IS FOR A TEMPORARY BRACED SOLDIER PILE SHORING WALL WITH A $1\frac{1}{2}$ H:1V BACKSLOPE.

2) ACTIVE EARTH PRESSURE CAN BE ASSUMED TO ACT OVER THE ENTIRE EXPOSED WALL AREA AND OVER THE WIDTH OF THE SOLDIER PILE BELOW THE LAGGING.

3) THE DESIGN GROUNDWATER TABLE AT THE WALL LOCATION IS ASSUMED TO BE AT ELEVATION 420 FEET. THE DISTANCE D1 IS THE DISTANCE BETWEEN THE GROUND SURFACE AND ELEVATION 420 FEET. THE DISTANCE D2 IS THE DISTANCE BETWEEN THE BOTTOM OF THE PILE AND ELEVATION 420 FEET.

4) THE PASSIVE EARTH PRESSURE SHOULD BE ASSUMED TO ACT OVER TWO SOLDIER PILE DIAMETERS OR THE SOLDIER PILE SPACING, WHICHEVER IS LESS.

5) SOLDIER PILES SHOULD EXTEND AT LEAST 8 FEET BELOW THE LOWEST ADJACENT EXCAVATION LEVEL.

6) DRAWING NOT TO SCALE.

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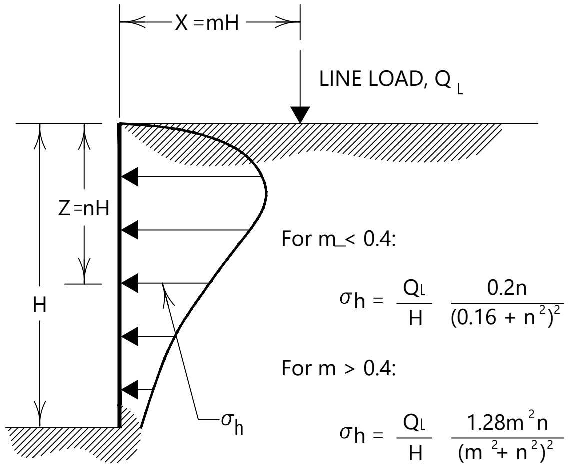
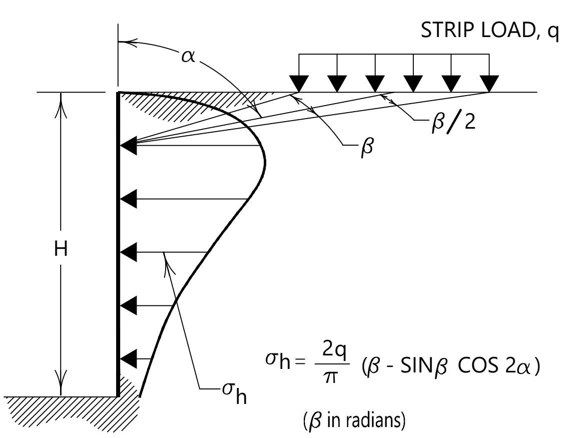
LATERAL EARTH PRESSURE FOR BRACED SHORING

OCT. 2021 JOB NO. W1277 FIG. 4

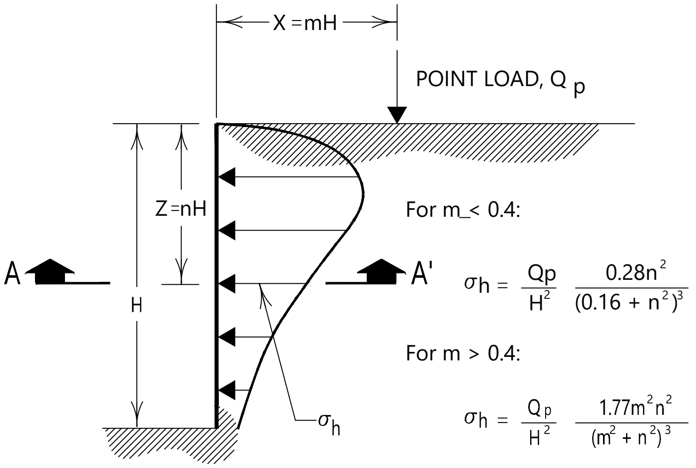
Exhibit 10 SPRV22-06

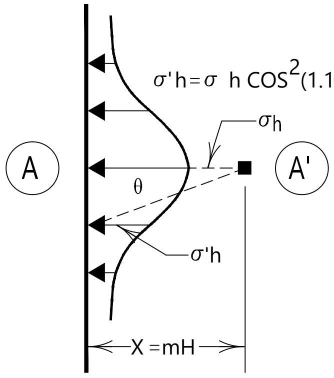


LINE LOAD PARALLEL TO WALL



STRIP LOAD PARALLEL TO WALL





DISTRIBUTION OF HORIZONTAL PRESSURES

VERTICAL POINT LOAD

NOTES:

1. THESE GUIDELINES APPLY TO RIGID WALLS WITH POISSON'S RATIO ASSUMED TO BE 0.5 FOR BACKFILL MATERIALS.
2. LATERAL PRESSURES FROM ANY COMBINATION OF ABOVE LOADS MAY BE DETERMINED BY THE PRINCIPLE OF SUPERPOSITION.

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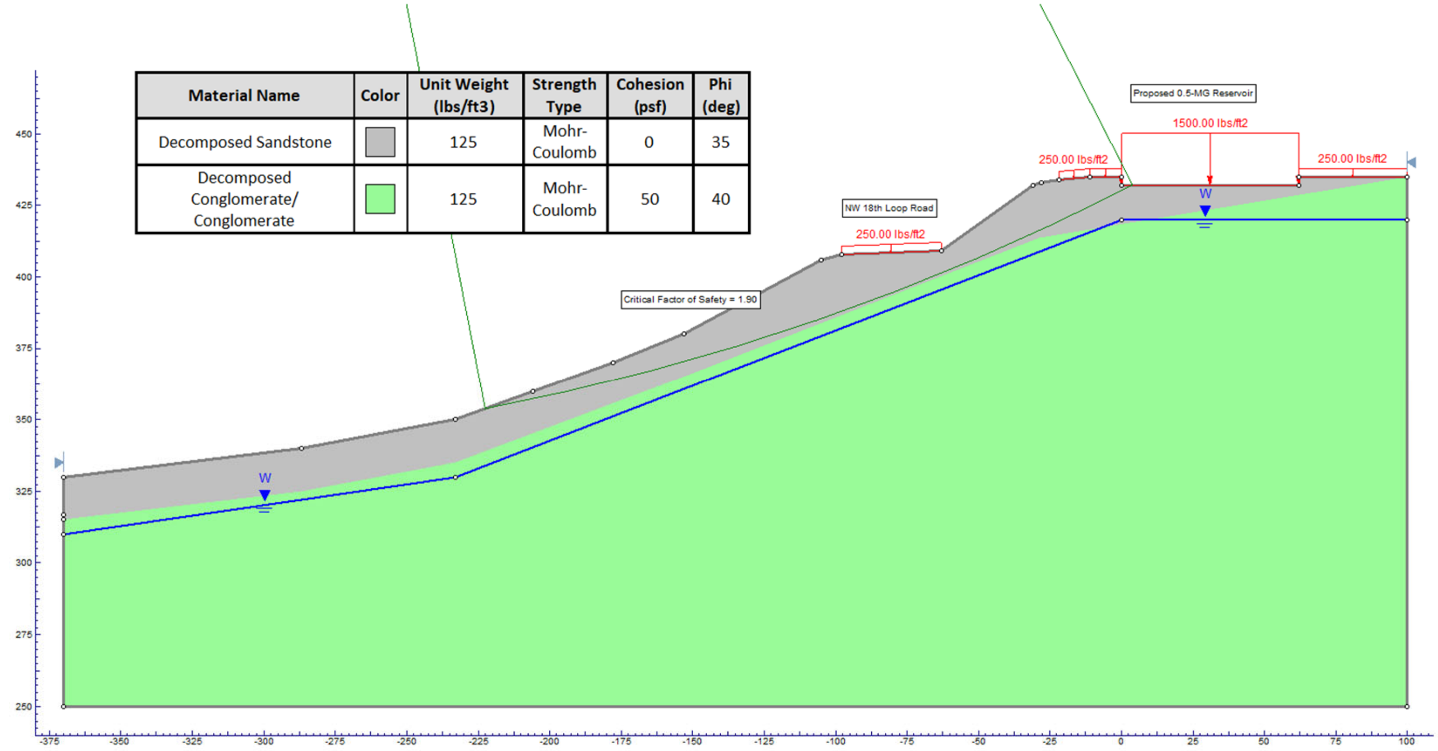
LOWER PRUNE HILL BOOSTER PUMP
STATION IMPROVEMENTS

SURCHARGE-INDUCED LATERAL PRESSURE

OCT. 2021 JOB NO. W1277 FIG. 5

Exhibit 10 SPRV22-06



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Decomposed Sandstone	▨	125	Mohr-Coulomb	0	35
Decomposed Conglomerate/ Conglomerate	🟩	125	Mohr-Coulomb	50	40

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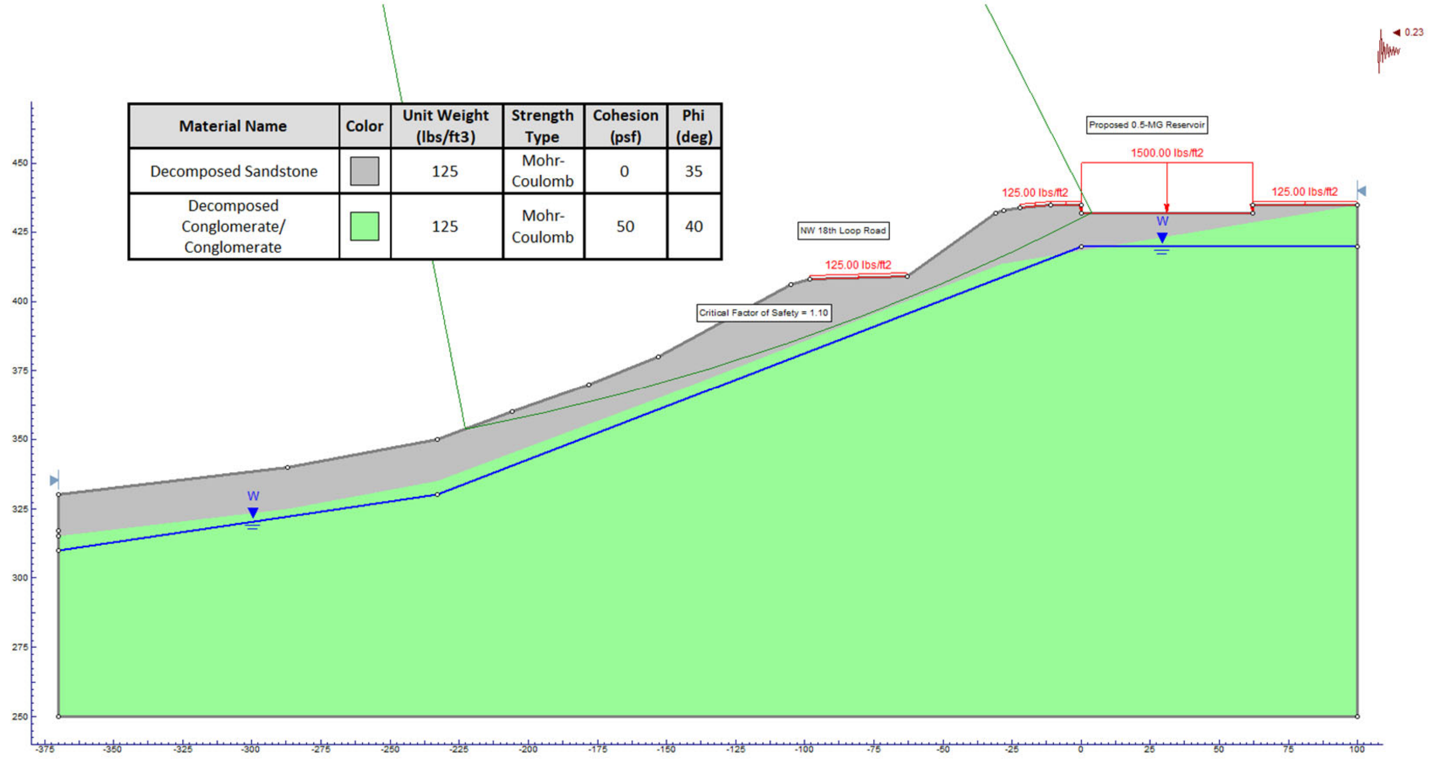
STATIC SLOPE STABILITY RESULTS

OCT. 2021 JOB NO. W1277 FIG. 6

Exhibit 10 SPRV22-06



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Decomposed Sandstone	▨	125	Mohr-Coulomb	0	35
Decomposed Conglomerate/ Conglomerate	🟩	125	Mohr-Coulomb	50	40

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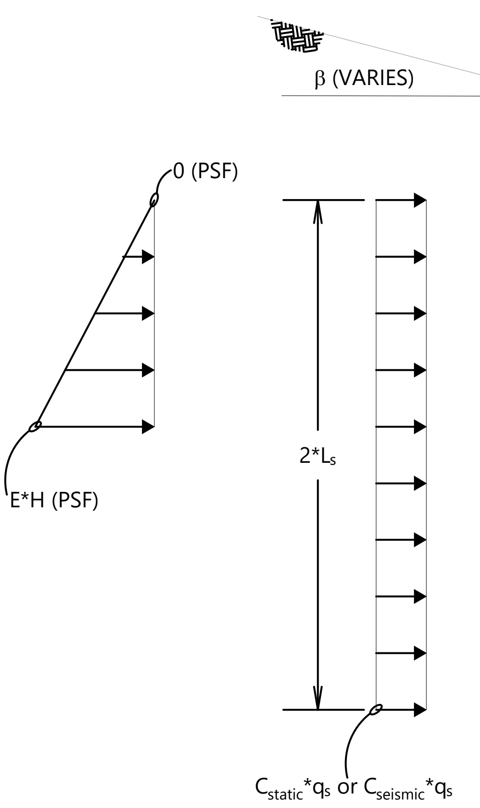
SEISMIC SLOPE STABILITY RESULTS

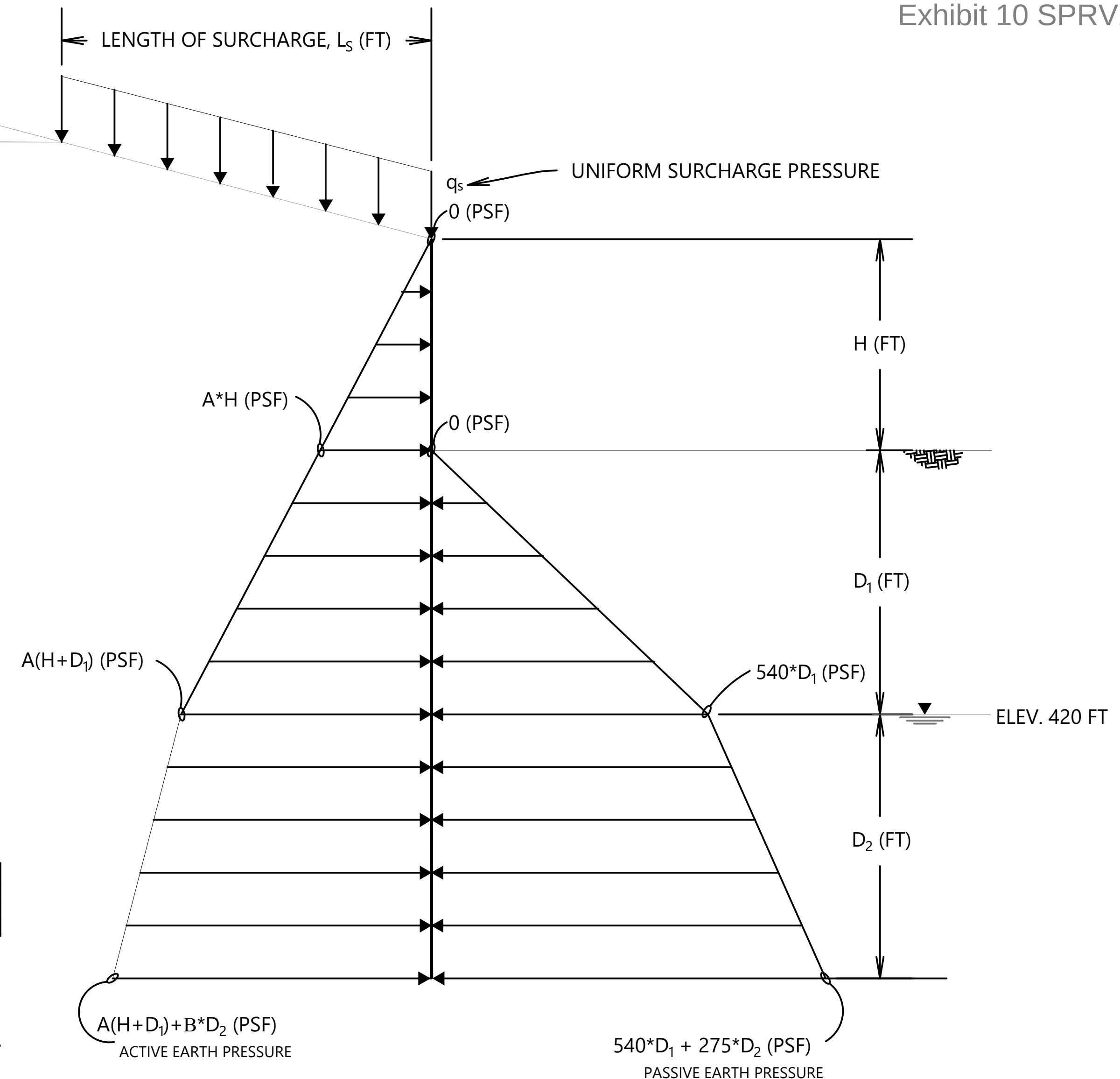
OCT. 2021 JOB NO. W1277 FIG. 7

Exhibit 10 SPRV22-06



SEISMIC EARTH PRESSURE

UNIFORM SURCHARGE PRESSURE



ACTIVE EARTH PRESSURE

PASSIVE EARTH PRESSURE

SLOPE, β	A	B	C_{STATIC}	$C_{SEISMIC}$	E
3H:1V, β=18.9°	42	20	0.34	0.43	11
2H:1V, β=26.6°	49	23	0.39	0.58	23
HORIZONTAL, β=0°	34	16	0.27	0.32	6

NOTES:

1) LATERAL EARTH PRESSURES DIAGRAM IS FOR A CANTILEVERED SOLDIER PILE WALL.

2) THE ACTIVE EARTH PRESSURE, SEISMIC EARTH PRESSURE, AND UNIFORM SURCHARGE PRESSURE ARE DEPENDENT ON THE SLOPE BEHIND THE WALL. REFER TO TABLE FOR VALUES OF COEFFICIENT A, B, C, AND E FOR LEVEL BACKSLOPES, SLOPES INCLINED AT 2H:1V, AND FOR SLOPES INCLINED AT 3H:1V. TWO VALUES ARE PROVIDED FOR C - C_{STATIC} FOR EVALUATING STATIC LOADING CONDITIONS AND $C_{SEISMIC}$ FOR LOAD CASES INCLUDING SEISMIC LOADING.

3) ADDITIONAL SURCHARGE PRESSURES DUE TO NON-UNIFORM LOADS, IF ANY, SHOULD BE INCLUDED IN THE DESIGN USING THE GUIDELINES PROVIDED ON FIGURE 5.

4) ACTIVE, SURCHARGE-INDUCED LATERAL PRESSURES, AND SEISMIC EARTH PRESSES CAN BE ASSUMED TO ACT OVER THE ENTIRE EXPOSED WALL AREA AND OVER THE WIDTH OF THE SOLDIER PILE BELOW THE LAGGING.

5) THE DESIGN GROUNDWATER TABLE AT THE WALL LOCATION IS ASSUMED TO BE AT ELEVATION 420 FT. THE DISTANCE D_1 IS THE DISTANCE BETWEEN THE GROUND SURFACE AND ELEVATION 420 FT. THE DISTANCE D_2 IS THE DISTANCE BETWEEN THE BOTTOM OF THE PILE AND ELEVATION 420 FT.

6) THE PASSIVE EARTH PRESSURE SHOULD BE ASSUMED TO ACT OVER TWO SOLDIER PILE DIAMETERS OR THE SOLDIER PILE SPACING, WHICHEVER IS LESS.

7) DRAWING NOT TO SCALE.

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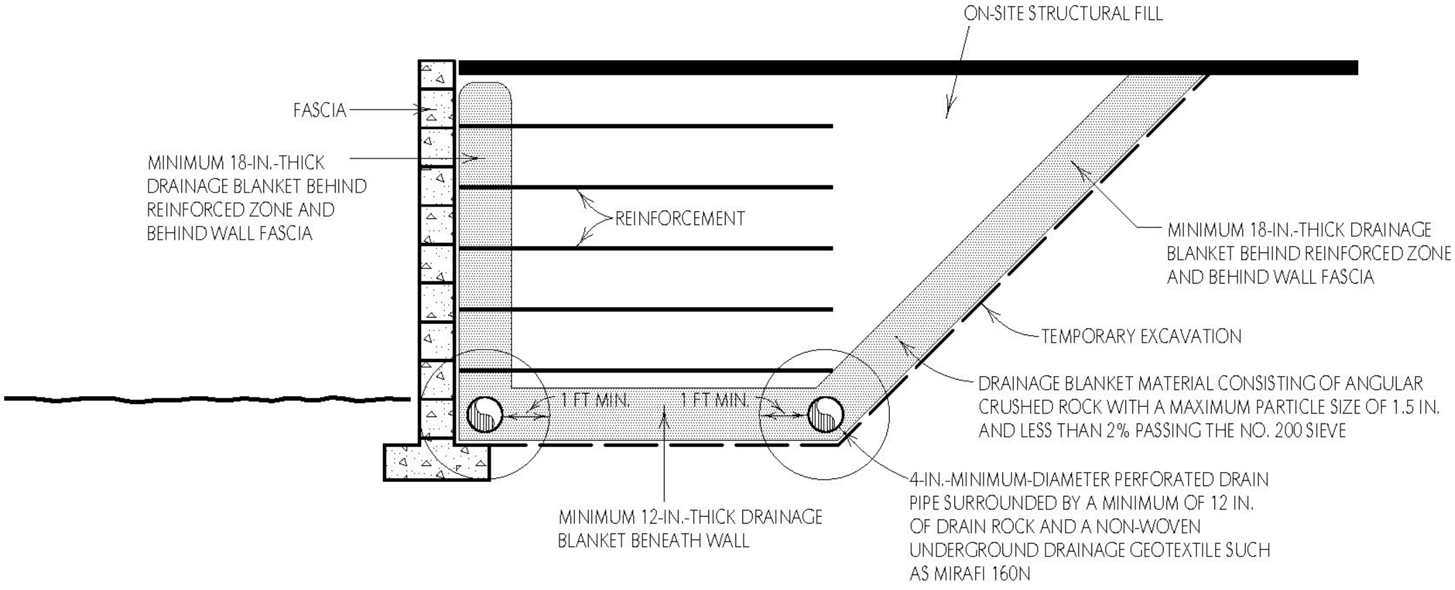
PERMANENT WALL LATERAL EARTH PRESSURE DIAGRAM

OCT. 2021 JOB NO. W1277 FIG. 8

Exhibit 10 SPRV22-06



NO SCALE

GRI

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STATION IMPROVEMENTS

TYPICAL DRAINAGE FOR MSE WALL CONSTRUCTED WITH ON-SITE SOILS

OCT. 2021 JOB NO. W1277 FIG. 9

Exhibit 10 SPRV22-06

DRAFT

APPENDIX A

Field Explorations and Laboratory Testing

DRAFT

APPENDIX A

FIELD EXPLORATIONS AND LABORATORY TESTING

A.1 FIELD EXPLORATIONS

A.1.1 General

Subsurface materials and conditions at the site were evaluated between September 10, 2020, and July 7, 2021. The field program included two Rotosonic borings and one mud-rotary boring. The approximate locations of the explorations are shown on Figure 2. An experienced member of the GRI's staff directed the explorations and maintained a log of the materials and conditions disclosed during the work.

A.1.2 Machine-Drilled Borings

Borings B-1 and B-2 were completed on September 10 and 11, 2020, to a depth of 31.5 feet using Rotosonic drilling techniques and a track-mounted Boart Longyear LS 250 MiniSonic drill rig provided and operated by Cascade Drilling, Inc. of Clackamas, Oregon. Continuous, 6-inch-diameter runs were obtained from the Rotosonic borings in flexible plastic tubing. The plastic tubing was opened in the field for visual classifications, and photographs were taken of each of the runs. Selected samples were returned to our laboratory for further examination in our laboratory. The photographs of the runs from borings B-1 and B-2 are provided at the end of this Appendix. In addition, the Standard Penetration Test (SPT) was conducted at 3- to 5-foot intervals of depth during the advancement of the boring. This test consists of driving a standard split-spoon sampler into the soil a distance of 18 inches using a 140-pound hammer dropped 30 inches. The number of blows required to drive the sampler the last 12 inches is known as the standard penetration resistance, or SPT N-value. SPT N-values provides a measure of the relative density of granular soils, such as sand, and the relative consistency, or stiffness, of cohesive soils, such as silt. The split-spoon samples were carefully examined in the field, and representative portions were saved in airtight jars.

Boring B-3 was completed on July 7, 2021, to a depth of 51.5 feet using mud-rotary drilling techniques and a track-mounted Mobile Drill B-57 drill rig provided and operated by Holt Drilling, Inc. of Vancouver, Washington. Disturbed samples were obtained from the boring at 2.5-foot intervals of depth in the upper 15 feet and 5-foot intervals below this depth. Disturbed soil samples were obtained using either a standard split spoon sampler or a California-modified sampler (CMS) split-spoon sampler with an outside diameter of 3 inches. The CMS sampler was used at selected depths to collect more representative sample of the soil than is possible with the smaller 2-inch Standard Penetration Test sampler. An approximation of standard penetration test (SPT) N-values from N*-value can be made by multiplying N*-value by a factor of 0.7. Samples obtained from the boring were placed in airtight jars and returned to our laboratory for further classification and testing.

Logs of the machine-drilled borings discussed above are provided on Figures 1A through 3A. Each log presents a descriptive summary of the various types of materials encountered in the boring and notes the depths at which the materials and/or characteristics of the materials change. To the right of the descriptive summary, the numbers and types of samples taken during the drilling operation are indicated. Farther to the right, N- and N*-values are shown graphically along with the natural moisture contents, Atterberg limits, and percentage of material passing the No. 200 sieve. The terms and symbols used to describe the soils encountered in the borings are defined in Table 1A and the attached legend.

A.1.3 Instrumentation

An RST Instruments Model VW2100 vibrating-wire piezometer was installed at a depth of about 28 feet (elevation 409.5 feet) in boring B-2. The piezometer is equipped with an RST Model DT2011B single-channel data logger programmed to record data at regular intervals. At the time of installation, the piezometer was saturated with water, taped to a 1-inch-outside-diameter polyvinyl chloride grout pipe in an inverted position to maintain saturation and inserted into the open borehole to the desired depth. The boring was then filled with cement-bentonite grout near the ground surface. The performance of the piezometer was verified before installation and immediately after insertion to design depth. The installation is equipped with a steel monument casing that was cement grouted into the borehole collar to protect the data logger and readout cables from vehicle traffic and the elements. The data logger is being downloaded periodically to evaluate the data.

A.2 LABORATORY TESTING

A.2.1 General

The samples obtained from the borings were examined in our laboratory, where the physical characteristics of the samples were noted, and the field classifications modified where necessary. At the time of classification, the natural moisture content of each sample was determined. Additional testing included Atterberg limits and grain size testing. A summary of the laboratory test results is provided in Table 2A. The following sections describe the testing program in more detail.

A.2.2 Natural Moisture Content

Natural moisture content determinations were made in conformance with ASTM International (ASTM) D2216. The results are summarized on Figures 1A through 3A and in Table 2A.

Exhibit 10 SPRV22-06

A.2.3 Atterberg Limits

Atterberg-limits testing was conducted on a select soil sample in conformance with ASTM D4318. The test results are summarized on the boring log, Figure 2A; the Plasticity Chart, Figure 4A, and in Table 2A.

A.2.4 Grain-Size Analysis

Washed-sieve grain-size analyses were performed on selected soil samples to evaluate the percentage of material passing the No. 200 sieve. The test is performed by taking a sample of known dry weight and washing it over a No. 200 sieve. The material retained on the sieve is oven-dried and weighed. The percentage of material passing the No. 200 sieve is then calculated. The results are summarized on Figures 1A through 3A and in Table 2A.

Dry sieve analyses were completed on selected samples in substantial conformance with ASTM D6913-04. The test is performed by taking a sample of known dry weight and washing it over a No. 200 sieve. The material retained on the sieve is oven-dried and weighed, and the percentage of material passing the No. 200 sieve is calculated. The soil retained on the No. 200 sieve is then screened through a series of sieves of various sizes using a sieve shaker. The weight of each sieve is measured prior to and after the test. The weight of the sample retained on each sieve is recorded and expressed as a percentage of the total sample weight. The test results are shown on Figures 5A through 6A.

Exhibit 10 SPRV22-06

Table 1A

GUIDELINES FOR CLASSIFICATION OF SOIL

Description of Relative Density for Granular Soil

Relative Density	Standard Penetration Resistance, (N-values) blows/ft
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	over 50

Description of Consistency for Fine-Grained (Cohesive) Soils

Consistency	Standard Penetration Resistance (N-values), blows/ft	Torvane or Undrained Shear Strength, tsf
Very Soft	0 - 2	less than 0.125
Soft	2 - 4	0.125 - 0.25
Medium Stiff	4 - 8	0.25 - 0.50
Stiff	8 - 15	0.50 - 1.0
Very Stiff	15 - 30	1.0 - 2.0
Hard	over 30	over 2.0

Grain-Size Classification
Boulders: >12 in.
Cobbles: 3-12 in.
Gravel: ¼ - ¾ in. (fine) ¾ - 3 in. (coarse)
Sand: No. 200 - No. 40 sieve (fine) No. 40 - No. 10 sieve (medium) No. 10 - No. 4 sieve (coarse)
Silt/Clay: Pass No. 200 sieve

Modifier for Subclassification		
	Primary Constituent SAND or GRAVEL	Primary Constituent SILT or CLAY
Adjective	Percentage of Other Material (By Weight)	
trace:	5 - 15 (sand, gravel)	5 - 15 (sand, gravel)
some:	15 - 30 (sand, gravel)	15 - 30 (sand, gravel)
sandy, gravelly:	30 - 50 (sand, gravel)	30 - 50 (sand, gravel)
trace:	<5 (silt, clay)	*Relationship of clay and silt determined by plasticity index test*
some:	5 - 12 (silt, clay)	
silty, clayey:	12 - 50 (silt, clay)	

Table 2A

SUMMARY OF LABORATORY RESULTS

Sample Information						Atterberg Limits			
Location	Sample	Depth, ft	Elevation, ft	Moisture Content, %	Dry Unit Weight, pcf	Liquid Limit, %	Plasticity Index, %	Fines Content, %	Soil Type
B-1	G-1	1.0	--	18	--	--	--	46	Silty SAND
	G-2	4.5	--	30	--	--	--	--	Clayey GRAVEL
	S-1	5.0	--	31	--	--	--	--	Clayey GRAVEL
	G-3	7.5	--	13	--	--	--	18	Clayey GRAVEL
	G-4	8.5	--	17	--	--	--	--	Clayey GRAVEL
	S-2	10.0	--	34	--	--	--	--	Clayey GRAVEL
	G-5	13.5	--	32	--	--	--	31	Clayey GRAVEL
	S-3	15.0	--	25	--	--	--	--	Clayey GRAVEL
	S-4	20.0	--	26	--	--	--	--	Clayey GRAVEL
	G-8	23.0	--	20	--	--	--	27	Clayey GRAVEL
	S-5	25.0	--	33	--	--	--	--	Clayey GRAVEL
	G-9	28.5	--	37	--	--	--	--	Clayey GRAVEL
	S-6	30.0	--	33	--	--	--	29	Gravelly SAND
B-2	G-2	6.0	--	20	--	--	--	47	Silty SAND
	S-1	7.0	--	35	--	--	--	--	Silty SAND
	G-3	9.0	--	32	--	--	--	55	Sandy CLAY
	S-2	10.0	--	38	--	43	11	54	Sandy CLAY
	G-4	14.0	--	34	--	--	--	--	Clayey GRAVEL
	S-3	15.0	--	29	--	--	--	--	Clayey GRAVEL
	G-5	19.5	--	35	--	--	--	52	Sandy CLAY
	S-4	20.0	--	32	--	--	--	21	Clayey SAND
	G-6	22.0	--	27	--	--	--	37	Clayey SAND
	G-7	24.0	--	31	--	--	--	--	Clayey GRAVEL
	S-5	25.0	--	22	--	--	--	--	Clayey GRAVEL
	G-8	27.0	--	26	--	--	--	29	Clayey GRAVEL
	S-6	30.0	--	39	--	--	--	29	Clayey GRAVEL

Exhibit 10 SPRV22-06

BORING AND TEST PIT LOG LEGEND

SOIL SYMBOLS

Symbol	Typical Description
⁘	LANDSCAPE MATERIALS
▩	FILL
◌	GRAVEL; clean to some silt, clay, and sand
◌	Sandy GRAVEL; clean to some silt and clay
◌	Silty GRAVEL; up to some clay and sand
◌	Clayey GRAVEL; up to some silt and sand
░	SAND; clean to some silt, clay, and gravel
░	Gravelly SAND; clean to some silt and clay
░	Silty SAND; up to some clay and gravel
░	Clayey SAND; up to some silt and gravel
▥	SILT; up to some clay, sand, and gravel
▥	Gravelly SILT; up to some clay and sand
▥	Sandy SILT; up to some clay and gravel
▥	Clayey SILT; up to some sand and gravel
▨	CLAY; up to some silt, sand, and gravel
▨	Gravelly CLAY; up to some silt and sand
▨	Sandy CLAY; up to some silt and gravel
▨	Silty CLAY; up to some sand and gravel
≋	PEAT

BEDROCK SYMBOLS

Symbol	Typical Description
⊞	BASALT
▤	MUDSTONE
☰	SILTSTONE
⋯	SANDSTONE

SURFACE MATERIAL SYMBOLS

Symbol	Typical Description
■	Asphalt concrete PAVEMENT
▒	Portland cement concrete PAVEMENT
◌	Crushed rock BASE COURSE

SAMPLER SYMBOLS

Symbol	Sampler Description
Ⅰ	2.0 in. O.D. split-spoon sampler and Standard Penetration Test with recovery (ASTM D1586)
▮	Shelby tube sampler with recovery (ASTM D1587)
▮	3.0 in. O.D. split-spoon sampler with recovery (ASTM D3550)
⊠	Grab Sample
▯	Rock core sample interval
▮	Sonic core sample interval
▯	Push probe sample interval

INSTALLATION SYMBOLS

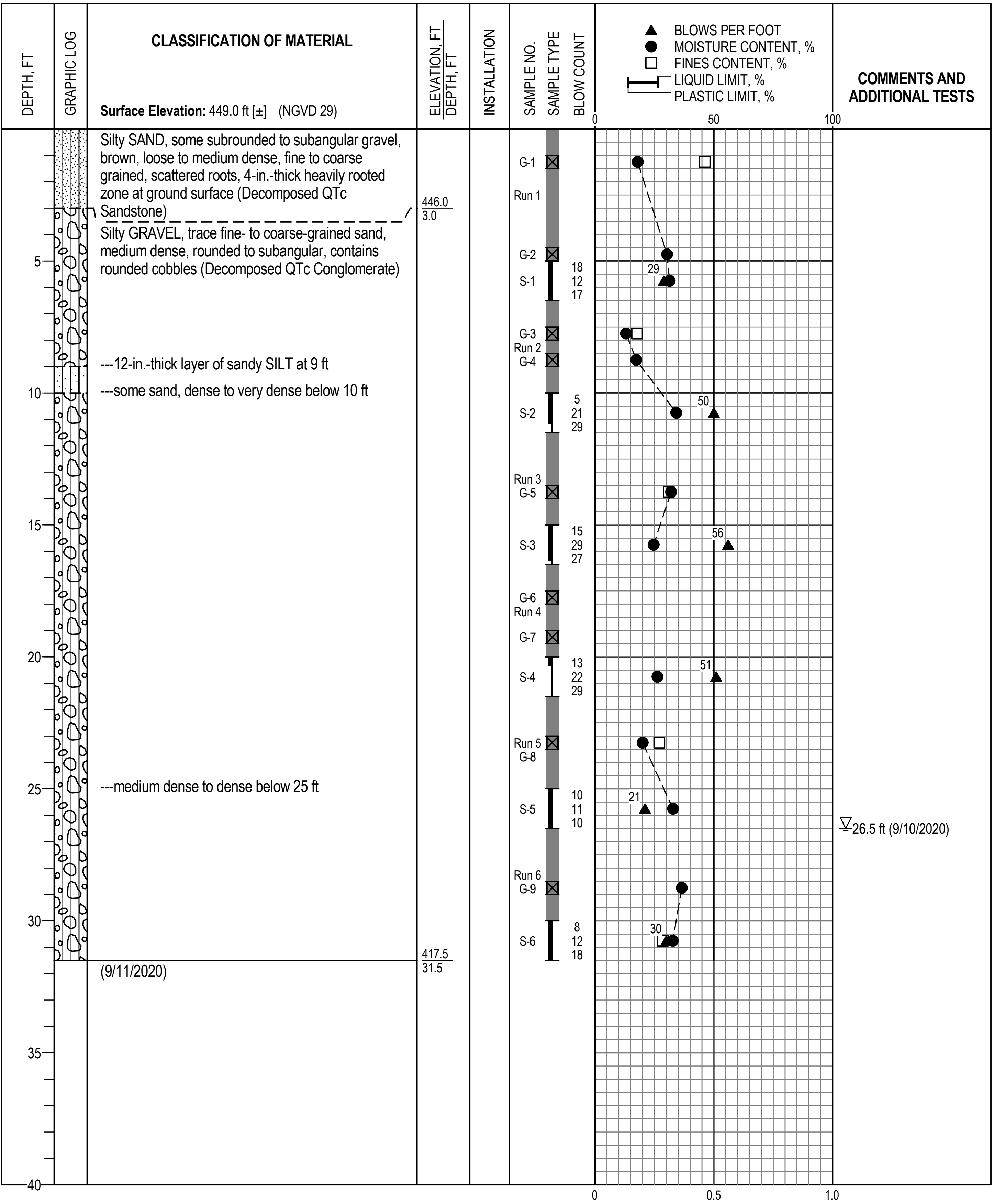
Symbol	Symbol Description
◫	Flush-mount monument set in concrete
◫	Concrete, well casing shown where applicable
▨	Bentonite seal, well casing shown if applicable
⁘	Filter pack, machine-slotted well casing shown where applicable
◫	Grout, vibrating-wire transducer cable shown where applicable
P	Vibrating-wire pressure transducer
◫	1-in.-diameter solid PVC
┆	1-in.-diameter hand-slotted PVC
◫	Grout, inclinometer casing shown where applicable

FIELD MEASUREMENTS

Symbol	Typical Description
▽	Groundwater level during drilling and date measured
▼	Groundwater level after drilling and date measured
▨	Rock/sonic core or push probe recovery (%)
▨	Rock quality designation (RQD, %)

BORING AND TEST PIT LOG LEGEND GRI DATA TEMPLATE.GDT 9/25/20

Exhibit 10 SPRV22-06



CLASSIFICATION OF MATERIAL

Surface Elevation: 449.0 ft [±] (NGVD 29)

Silty SAND, some subrounded to subangular gravel, brown, loose to medium dense, fine to coarse grained, scattered roots, 4-in.-thick heavily rooted zone at ground surface (Decomposed QTc Sandstone)

446.0 / 3.0

Silty GRAVEL, trace fine- to coarse-grained sand, medium dense, rounded to subangular, contains rounded cobbles (Decomposed QTc Conglomerate)

---12-in.-thick layer of sandy SILT at 9 ft

---some sand, dense to very dense below 10 ft

---medium dense to dense below 25 ft

417.5 / 31.5

(9/11/2020)

SAMPLE NO.	BLOW COUNT	BLOWS PER FOOT
G-1		
Run 1		
G-2		
S-1	18 12 17	29
G-3		
Run 2		
G-4		
S-2	5 21 29	50
Run 3		
G-5		
S-3	15 29 27	56
G-6		
Run 4		
G-7		
S-4	13 22 29	51
Run 5		
G-8		
S-5	10 11 10	21
Run 6		
G-9		
S-6	8 12 18	30

COMMENTS AND ADDITIONAL TESTS

∇26.5 ft (9/10/2020)

GRI BORING LOG (LAT/LONG) GRI DATA TEMPLATE.GDT 10/27/21

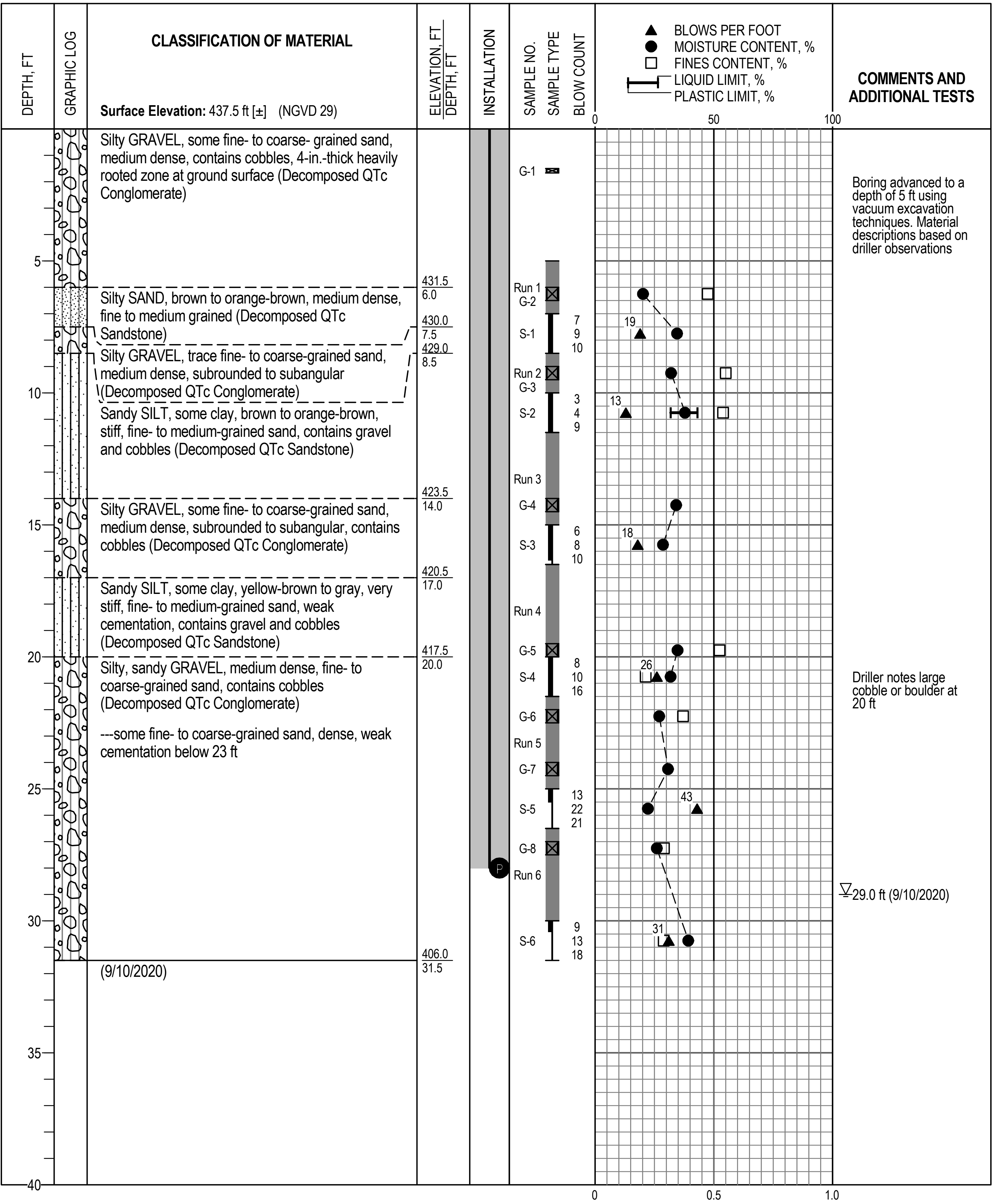
Logged By: G. Martin
Drilled by: Cascade Drilling, LP
Date Started: 9/10/20
Coordinates: 45.5919° N -122.4154° W (WGS 84)
Drilling Method: Roto Sonic
Equipment: Boart Longyear LS 250 MiniSonic
Hole Diameter: 6 in.
Note: See Legend for Explanation of Symbols
Hammer Type: Auto Hammer
Weight: 140 lb
Drop: 30 in.
Energy Ratio: Not Available

GRI

BORING B-1

OCT. 2021 JOB NO. W1277 FIG. 1A

Exhibit 10 SPRV22-06



CLASSIFICATION OF MATERIAL

Surface Elevation: 437.5 ft [±] (NGVD 29)

Silty GRAVEL, some fine- to coarse- grained sand, medium dense, contains cobbles, 4-in.-thick heavily rooted zone at ground surface (Decomposed QTc Conglomerate)		
Silty SAND, brown to orange-brown, medium dense, fine to medium grained (Decomposed QTc Sandstone)	431.5	6.0
Silty GRAVEL, trace fine- to coarse-grained sand, medium dense, subrounded to subangular (Decomposed QTc Conglomerate)	430.0	7.5
Sandy SILT, some clay, brown to orange-brown, stiff, fine- to medium-grained sand, contains gravel and cobbles (Decomposed QTc Sandstone)	429.0	8.5
Silty GRAVEL, some fine- to coarse-grained sand, medium dense, subrounded to subangular, contains cobbles (Decomposed QTc Conglomerate)	423.5	14.0
Sandy SILT, some clay, yellow-brown to gray, very stiff, fine- to medium-grained sand, weak cementation, contains gravel and cobbles (Decomposed QTc Sandstone)	420.5	17.0
Silty, sandy GRAVEL, medium dense, fine- to coarse-grained sand, contains cobbles (Decomposed QTc Conglomerate) ---some fine- to coarse-grained sand, dense, weak cementation below 23 ft	417.5	20.0
(9/10/2020)	406.0	31.5

G-1	
Run 1 G-2	
S-1	7 9 10
Run 2 G-3	
S-2	3 4 9
Run 3	
G-4	
S-3	6 8 10
Run 4	
G-5	
S-4	8 10 16
G-6	
Run 5	
G-7	
S-5	13 22 21
G-8	
Run 6	
S-6	9 13 18

COMMENTS AND ADDITIONAL TESTS

Boring advanced to a depth of 5 ft using vacuum excavation techniques. Material descriptions based on driller observations

Driller notes large cobble or boulder at 20 ft

∇29.0 ft (9/10/2020)

◆ TORVANE SHEAR STRENGTH, TSF
■ UNDRAINED SHEAR STRENGTH, TSF

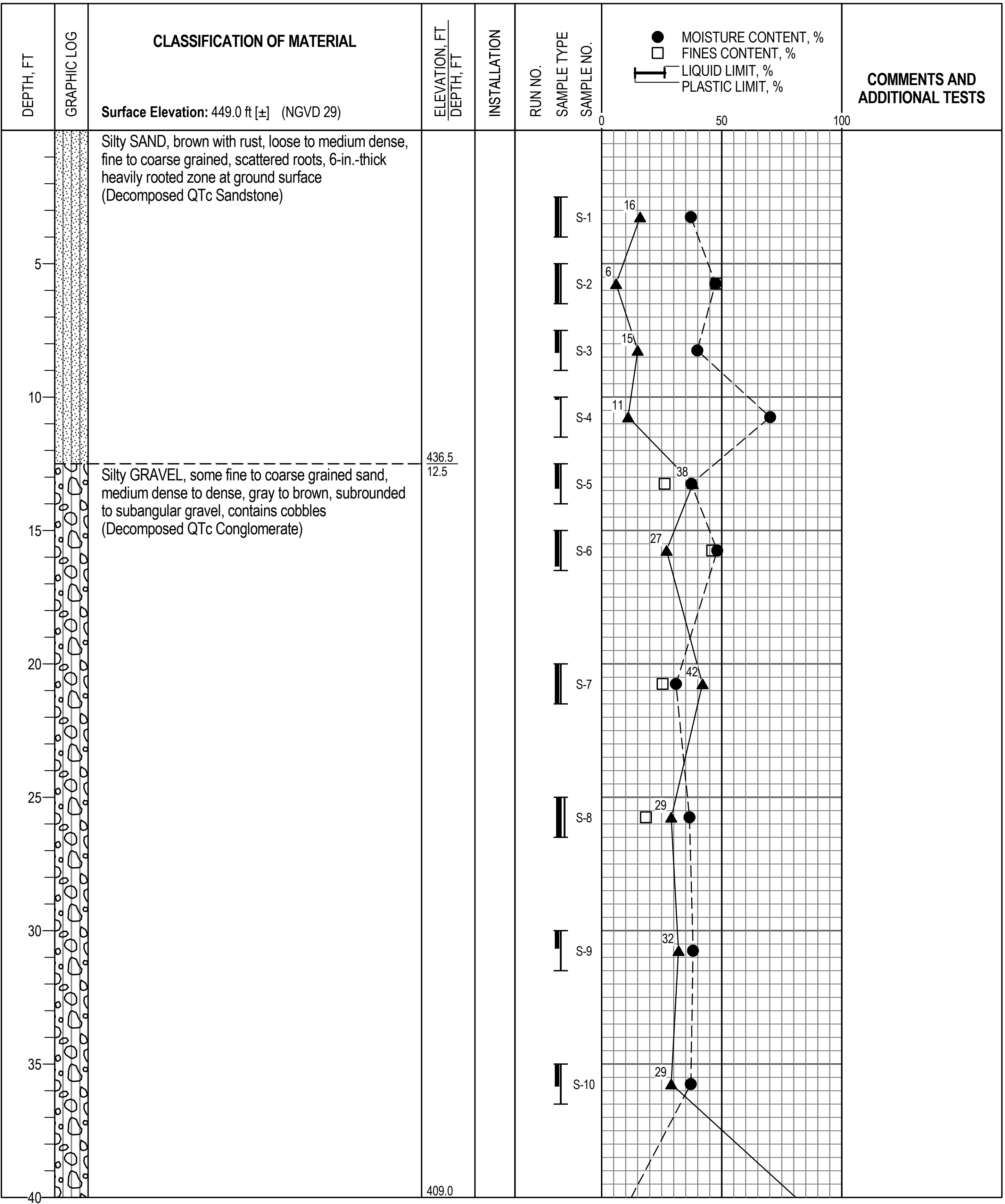
Logged By: G. Martin
Drilled by: Cascade Drilling, LP
Date Started: 9/10/20
Coordinates: 45.5916° N -122.4155° W (WGS 84)
Drilling Method: Roto Sonic
Equipment: Boart Longyear LS 250 MiniSonic
Hole Diameter: 6 in.
Note: See Legend for Explanation of Symbols
Hammer Type: Auto Hammer
Weight: 140 lb
Drop: 30 in.
Energy Ratio: Not Available

GRI

BORING B-2

OCT. 2021 JOB NO. W1277 FIG. 2A

Exhibit 10 SPRV22-06



CLASSIFICATION OF MATERIAL

Surface Elevation: 449.0 ft [±] (NGVD 29)

COMMENTS AND ADDITIONAL TESTS

Silty SAND, brown with rust, loose to medium dense, fine to coarse grained, scattered roots, 6-in.-thick heavily rooted zone at ground surface (Decomposed QTc Sandstone)

Silty GRAVEL, some fine to coarse grained sand, medium dense to dense, gray to brown, subrounded to subangular gravel, contains cobbles (Decomposed QTc Conglomerate)

(CONTINUED NEXT PAGE)

Logged By: D. Schade
Drilled by: Holt Services, Inc.
Date Started: 7/7/21
Coordinates: 45.5917° N -122.4157° W (WGS 84)
Drilling Method: Mud Rotary
Equipment: Mobile B-57 Track-Mounted Drill Rig
Hole Diameter: 4.5 in.
Note: See Legend for Explanation of Symbols
Hammer Type: Auto Hammer
Weight: 140 lb
Drop: 30 in.
Energy Ratio: Not Available

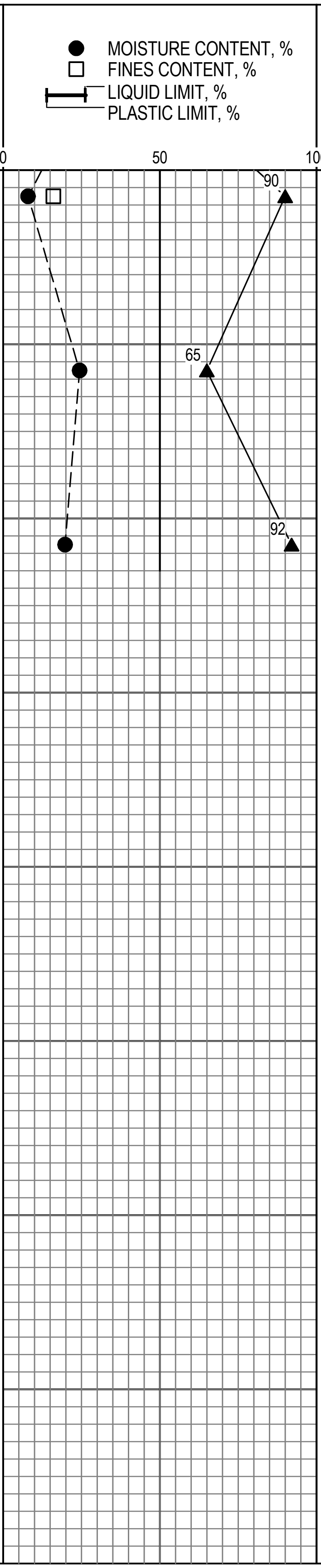
GRI

BORING B-3

OCT. 2021 JOB NO. W1277 FIG. 3A

Exhibit 10 SPRV22-06

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL **Surface Elevation:** 449.0 ft [±] (NGVD 29)	ELEVATION, FT / DEPTH, FT	INSTALLATION	RUN NO.	SAMPLE TYPE	SAMPLE NO.	COMMENTS AND ADDITIONAL TESTS
40	+ + +	CONGLOMERATE, gray and brown, predominately decomposed, extremely soft to very soft (R0 to R1), unconsolidated to poorly cemented (Quaternary unnamed conglomerate - QTc)	40.0				S-11 S-12 S-13	
51.5		(7/7/2021)	397.5 51.5					



Depth, ft: 45, 50, 55, 60, 65, 70, 75, 80

GRI SONIC BORING (LAT/LONG) GRI DATA TEMPLATE.GDT 10/27/21

GRI

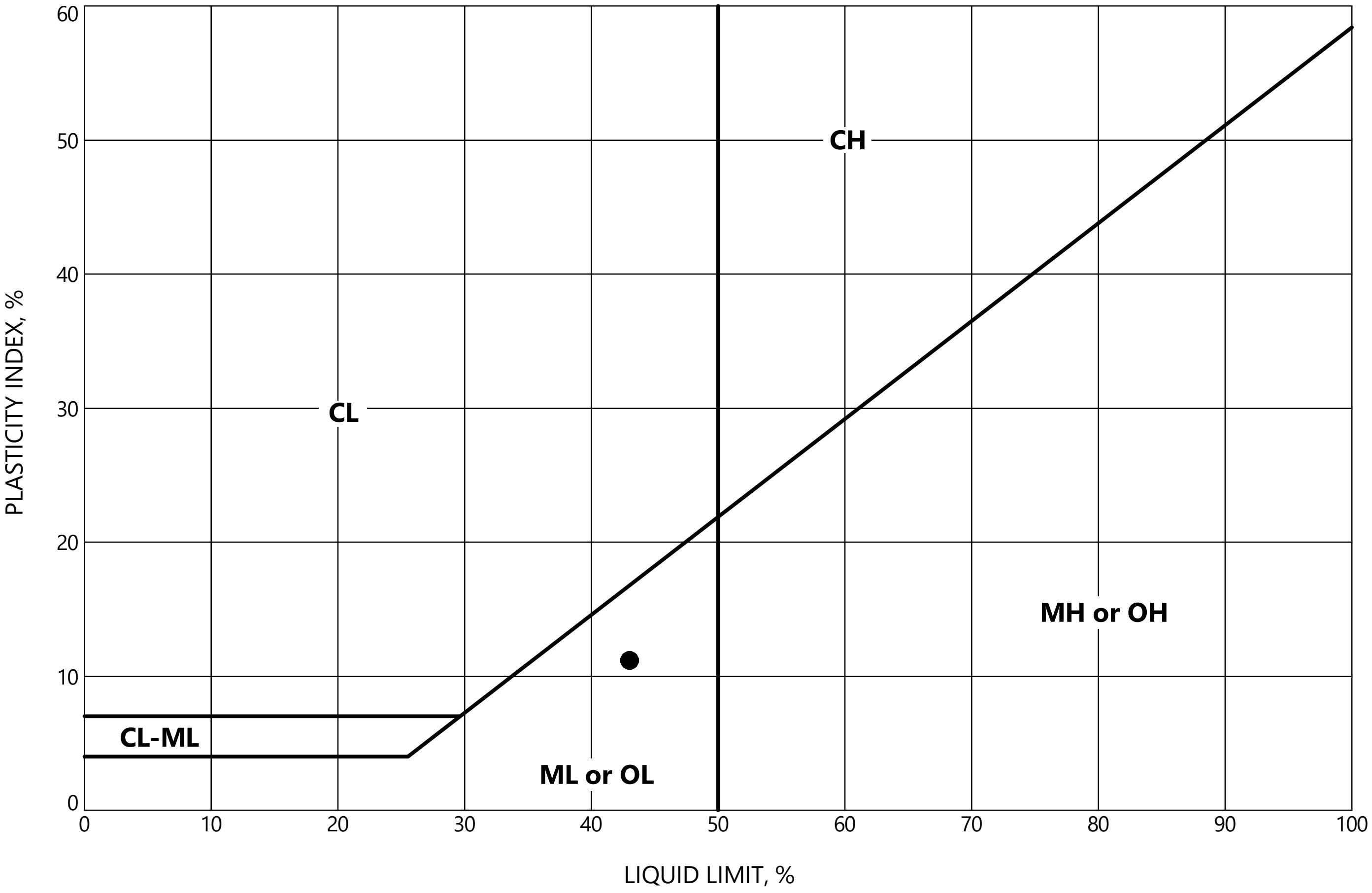
BORING B-3

OCT. 2021 JOB NO. W1277 FIG. 3A

Exhibit 10 SPRV22-06

GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE-GRAINED SOIL GROUPS
OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
ML	INORGANIC CLAYEY SILTS TO VERY FINE SANDS OF SLIGHT PLASTICITY
CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY

GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE-GRAINED SOIL GROUPS
OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
MH	INORGANIC SILTS AND CLAYEY SILT
CH	INORGANIC CLAYS OF HIGH PLASTICITY



	Location	Sample	Depth, ft	Classification	LL	PL	PI	MC, %
●	B-2	S-2	10.0	Sandy SILT, some clay, brown to orange-brown, fine- to medium-grained sand	43	32	11	38

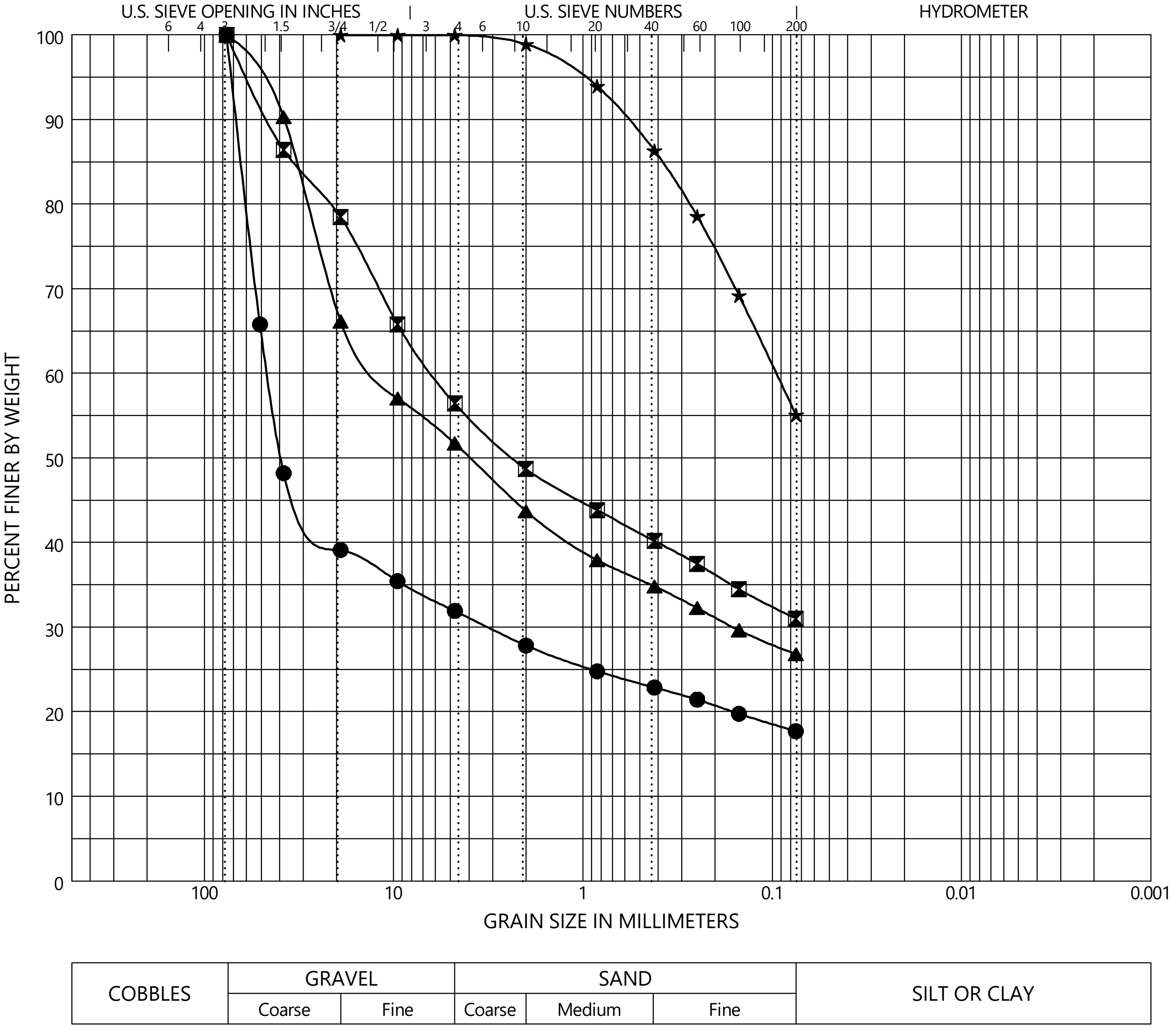
ATTERBERG-PLASTICITY 1 PER PAGE GRI DATA TEMPLATE.GDT 10/7/21

GRI

PLASTICITY CHART

OCT. 2021 JOB NO. W1277 FIG. 4A

Exhibit 10 SPRV22-06



	Location	Sample	Depth, ft	Classification	Gravel, %	Sand, %	Fines, %
●	B-1	G-3	7.5	Silty GRAVEL, trace fine- to coarse-grained sand, trace to some silt	66.8	14.2	17.7
⧗	B-1	G-5	13.5	Silty GRAVEL, some fine- to coarse-grained sand	43.3	25.4	31.0
▲	B-1	G-8	23.0	Silty GRAVEL, some fine- to coarse-grained sand	48.1	24.9	26.8
★	B-2	G-3	9.0	Sandy SILT, some clay, fine- to medium-grained sand	0.0	44.9	55.1

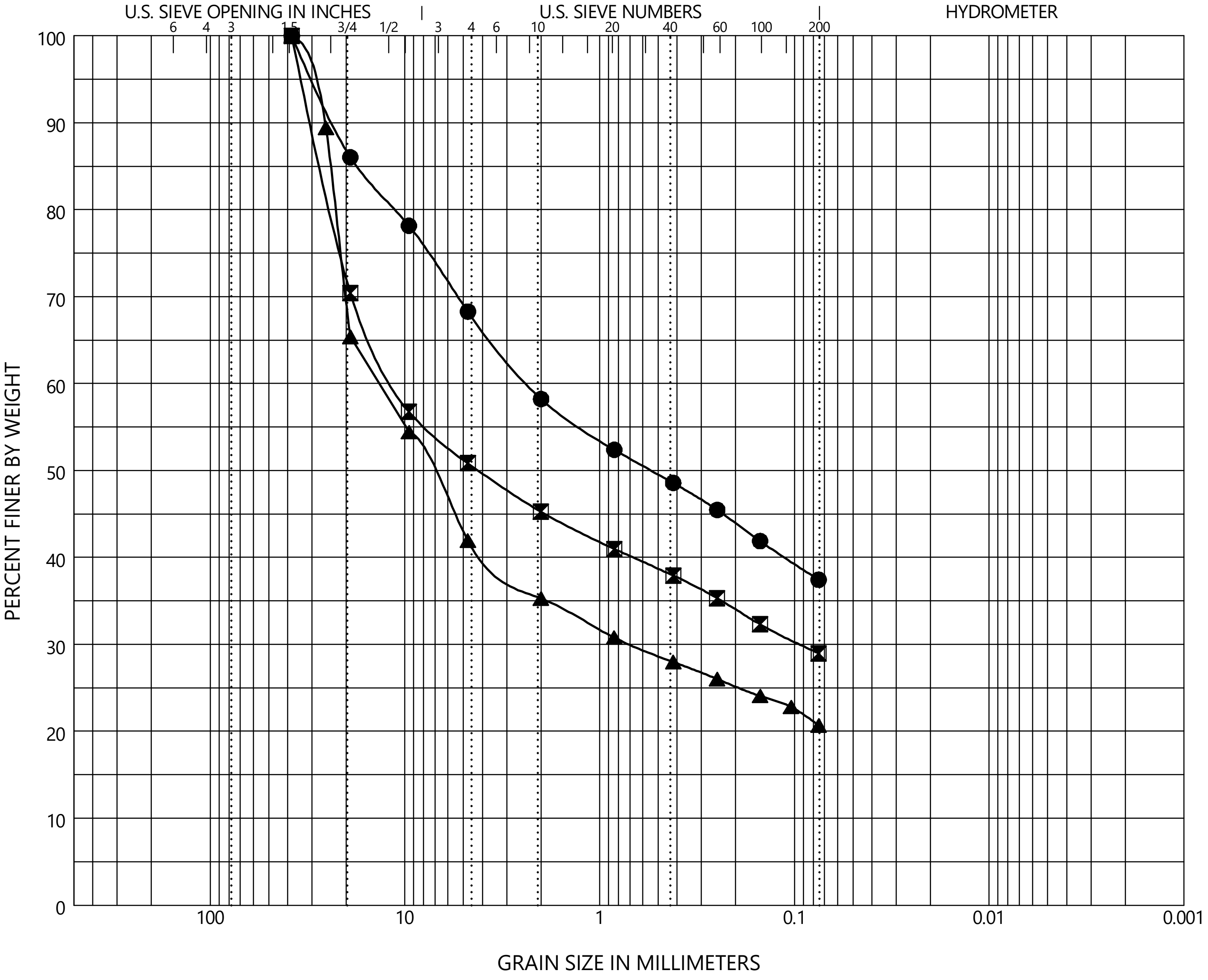
GRI

GRAIN SIZE DISTRIBUTION

OCT. 2021 JOB NO. W1277 FIG. 5A

GRAIN SIZE GRI - 4 PER PAGE GRI DATA TEMPLATE.GDT 10/7/21

Exhibit 10 SPRV22-06



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

	Location	Sample	Depth, ft	Classification	Gravel, %	Sand, %	Fines, %
●	B-2	G-6	22.0	Silty, sandy GRAVEL, fine- to coarse-grained sand	31.7	30.9	37.4
⧗	B-2	G-8	27.0	Silty GRAVEL, some fine- to coarse-grained sand	49.2	21.9	29.0
▲	B-3	S-8	25.0	Silty GRAVEL, some fine- to coarse-grained sand	58.1	21.3	20.7

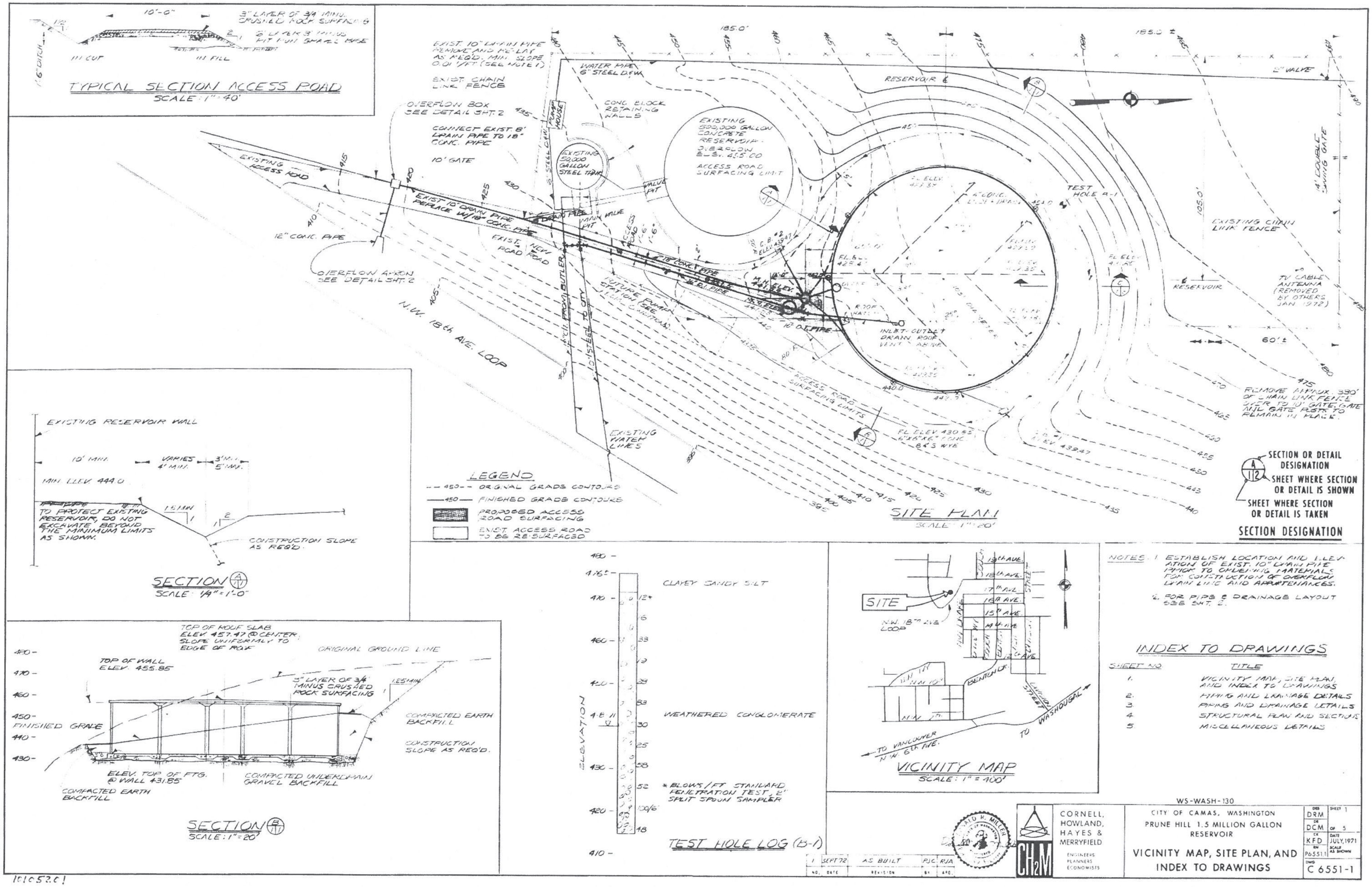
GRAIN SIZE GRI - 3 PER PAGE GRI DATA TEMPLATE.GDT 10/7/21

GRI

GRAIN SIZE DISTRIBUTION

OCT. 2021 JOB NO. W1277 FIG. 6A

Exhibit 10 SPRV22-06



NOTES: 1. ESTABLISH LOCATION AND ELEVATION OF EXIST. 10" DRAIN PIPE PRIOR TO ORDERING MATERIALS FOR CONSTRUCTION OF OVERFLOW DRAIN LINE AND APPURTENANCES.

2. FOR PIPE & DRAINAGE LAYOUT SEE SHT. 2.

INDEX TO DRAWINGS

SHEET NO.	TITLE
1.	VICINITY MAP, SITE PLAN, AND INDEX TO DRAWINGS
2.	PIPING AND DRAINAGE DETAILS
3.	PIPING AND DRAINAGE DETAILS
4.	STRUCTURAL PLAN AND SECTIONS
5.	MISCELLANEOUS DETAILS

NO.	DATE	REVISION	BY	APD.
1	SEPT 72	AS BUILT	PJC	RJA

DONALD R. MILLER

CORNELL,
HOWLAND,
HAYES &
MERRYFIELD
ENGINEERS
PLANNERS
ECONOMISTS

WS-WASH-130

CITY OF CAMAS, WASHINGTON
PRUNE HILL 1.5 MILLION GALLON
RESERVOIR

VICINITY MAP, SITE PLAN, AND
INDEX TO DRAWINGS

DES DRM	SHEET 1
DR DCM	OF 5
CK KFD	DATE JULY, 1971
EN P6551.1	SCALE AS SHOWN
DWG C 6551-1	

101052.01

Exhibit 10 SPRV22-06

DRAFT

APPENDIX B

Rotosonic Core Photos

Exhibit 10 SPRV22-06

GRI

MURRAYSMITH
LOWER PRUNE HILL BOOSTER PUMP
STATION IMPROVEMENTS

BORING B-1 CORE PHOTOS

OCT. 2021 JOB NO. W1277 FIG. 1B

Exhibit 10 SPRV22-06

GRI

MURRAYSMITH
LOWER PRUNE HILL BOOSTER PUMP
STATION IMPROVEMENTS

BORING B-1 CORE PHOTOS

OCT. 2021 JOB NO. W1277 FIG. 2B

Exhibit 10 SPRV22-06

MURRAYSMITH
LOWER PRUNE HILL BOOSTER PUMP
STATION IMPROVEMENTS

BORING B-1 CORE PHOTOS

OCT. 2021 JOB NO. W1277 FIG. 3B

Exhibit 10 SPRV22-06

GRI

MURRAYSMITH
LOWER PRUNE HILL BOOSTER PUMP
STATION IMPROVEMENTS

BORING B-2 CORE PHOTOS

OCT. 2021 JOB NO. W1277 FIG. 4B

Exhibit 10 SPRV22-06

MURRAYSMITH
LOWER PRUNE HILL BOOSTER PUMP
STATION IMPROVEMENTS

BORING B-2 CORE PHOTOS

OCT. 2021 JOB NO. W1277 FIG. 5B

Exhibit 10 SPRV22-06

GRI

MURRAYSMITH
LOWER PRUNE HILL BOOSTER PUMP
STATION IMPROVEMENTS

BORING B-2 CORE PHOTOS

OCT. 2021 JOB NO. W1277 FIG. 6B