GEOTECHNICAL ENGINEERING REPORT

Cascade Orchard Irrigation Company Improvement Project
Leavenworth, Washington

Prepared for: Anchor QEA

Project No. 170700 • September 17, 2021

Aspect
CONSULTING

earth + water

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Aspect Consulting, LLC

9/17/2021

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1 Introduction and Project Description

This geotechnical engineering report (report) provides Aspect Consulting, LLC's (Aspect) geotechnical design and construction recommendations for the Cascade Orchards Icicle Creek Flow Restoration (Project). This report is intended to support Project design, planning and cost estimating efforts being led by Anchor QEA (Anchor). The Project is located near Leavenworth, Washington (Site), as shown on Figure 1.

From Anchor, we understand the Project is planned to consist of replacing the existing Cascade Orchard Irrigation Company (COIC) irrigation delivery system. The existing irrigation system consists of a gravity-fed, unlined canal (COIC canal) and buried non-pressurized pipe system. The replacement irrigation system is proposed to consist of a pressurized pipe system supplied through a relocated intake and pump station. The Project will result in shifting surface water diversions from Icicle Creek (creek) downstream from the existing COIC intake upstream of the Leavenworth National Fish Hatchery (hatchery) to a new intake and pump station infrastructure. The new intake and pump station infrastructure will be located on the left bank of the creek near the east end of Shore Street, just upstream of the creek's confluence with the Wenatchee River.

Pressurized delivery pipes will include a main line that will extend from the new intake and pump station infrastructure at the end of Shore Street, and then north and south within the existing COIC canal alignment. New laterals will extend to the east from the main line along public roadways including:

- Cemetery Road
- Fish Hatchery Road
- East Leavenworth Road
- Prowell Street and
- Wilson Street

The locations of the proposed pressurized main pipeline, laterals pipelines, and pump station infrastructure are shown on Figures 2 and 3.

The pressurized main line and lateral pipelines are proposed to range in size from 6 to 18 inches in diameter and consist of high-density polyethylene (HDPE) material. The invert of the main pipeline (along existing COIC canal alignment) is proposed to be bedded and set less than 12 inches below the COIC canal invert and covered with at least 30 inches of fill—backfilling the COIC canal in the process. The lateral pipelines are also proposed to be bedded and buried with fill. Where possible, existing culverts under driveways and roads are planned to be utilized to carry the new pipelines.

The new pump station infrastructure is planned to consist of a buried settling basin and pump station/wet well (wet well) at about 26 feet below existing grade near the northwest bank of the creek (Figure 3). A 30-inch-diameter HDPE pipe with a cone screen (intake pipe) is planned to extend east from the settling basin into the creek.

We understand the Project will not be designed for seismic hazards due to the relatively low risk of interrupting a critical service in the event the new infrastructure is damaged and out of service.

To develop geotechnical recommendations, Aspect completed a desk study review of geologic maps and well logs, a field reconnaissance along the proposed main line and laterals, and subsurface geotechnical explorations at the pump station infrastructure area. We completed this work based on our scope of work and sub-consultant agreements dated December 27, 2017, and November 16, 2020. The results and recommendations are presented below.

2 Site Conditions

The following sections present descriptions of Site conditions developed from desk-study review of publicly available maps and data, a Site reconnaissance, and subsurface explorations.

2.1 Desk Study Review

The surficial soils across the Site are primarily mapped as quaternary alluvium (Qa) described as gravel, sand, and silt in channels and underlying floodplains of the Wenatchee River and tributaries, and valley sidewall colluvium, which includes cobbles and boulders.

Pre-Tertiary rocks (pTu) are mapped along the west side of the COIC canal/main pipeline alignment, and as an outcrop just south of Prowell Street. The Pre-Tertiary rocks are reported to consists of schist, granite, quartz, and serpentinite (DNR, 2021).

We reviewed logs of water wells (well logs) completed by others and drilled at/near the Site and within the mapped alluvium geology. The well logs were obtained from Washington State Department of Ecology (Ecology) online well log database (Ecology, 2020). The well logs generally report a relatively thick sequence of clay, silt, sand, gravel, cobble, and boulder mixtures interpreted to be alluvium.

The alluvium is reportedly underlain by granite bedrock (a subunit of pre-Tertiary bedrock) at depths as shallow as 27 feet below ground surface (bgs) near Prowell Street, but otherwise deeper than 50 feet bgs or not encountered at all at other areas of the Site.

2.2 Site Reconnaissance

We completed a Site reconnaissance on April 26, 2018, by walking along the base of the COIC canal/main line and lateral alignments, and on November 11 and December 8, 2020, at the pump station infrastructure area. The COIC canal had been empty of irrigation water for many months prior to the Site reconnaissance. Our observations and interpretations are described below.

2.2.1 Main Line Along COIC Canal

Surficial soil conditions along the majority of the existing COIC canal/main pipeline alignment were observed to consist of up to several inches of organic-laden topsoil with rootmass, overlying mixtures of silt and sand with variable gravel, cobble, and boulder content (ML, SM, and SP-SM)[1] content. In our opinion, these materials are generally consistent with the mapped alluvium geology.

We completed hand probing with a ½-inch diameter steel rod at approximately 15- to 20-foot distance intervals along the COIC canal base. We were typically able to hand probe 12 inches or more below the COIC canal base ground surface over the majority of the main line alignment, except for an approximately 800-foot-long section between

[1] Soil Classification per the United Soil Classification System (USCS). Refer to ASTM D2488.

about Cemetery Road and Fish Hatchery Road, and a 1,300-foot-long section between Fish Hatchery Road to just beyond East Leavenworth Road (Figure 2).

In these two sections, hand probe depths were commonly limited to less than 6 to 12 inches and met refusal on hard material interpreted to be rock(s). We also observed large boulders (greater than 5 feet in diameter) near or abutting the west side of the COIC canal/main line alignment in some areas of these two sections, consistent with the mapped valley sidewall colluvium (a subunit of alluvium) and/or Pre-Tertiary bedrock geology. In our opinion, the rock-like material these sections (whether directly observed or inferred by hand probing) are likely nested colluvial boulders originating from the valley sidewall, or Pre-Tertiary bedrock (such as granite).

2.2.2 Laterals

We observed exposed soils along the unpaved roadway shoulders of the laterals generally consist of mixtures of silt and sand with variable gravel, cobble, and boulder content (ML, SM, and SP-SM) content. We interpret these materials are alluvium or fill likely derived from alluvium.

We observed several rockeries constructed on private property near the laterals. We suspect the rockeries were built from boulders segregated from on-Site alluvium excavated for development. The boulders were typically observed to range in size from 1 to 5 feet in diameter, and less typically larger than 5 feet in diameter.

We observed utility manhole covers along the laterals, indicating manhole structures and utility pipes have been excavated, installed, and backfilled along the lateral alignments in the past.

2.2.3 Pump Station Infrastructure Area

We observed surface soil conditions in the pump station infrastructure area (Figure 3) were generally similar to those described for the laterals, except fill was not observed.

The exposed shoreline of the creek near the proposed intake location was observed to consist of silty sand with gravel (SM) moist, brown covered grass, shrubs and a veneer of cobbles and boulder presumably placed for scour/erosion protection. The shoreline was observed to be sloped at angles ranging from 2H:1V to 3H:1V (horizontal:vertical), with relief of about 6 to 7 feet down to the edges of the gently sloped creek channel.

2.3 Subsurface Explorations

We completed subsurface explorations at the pump station infrastructure area that consisted of a machine-drilled soil boring (AB-01), installation of one groundwater level monitoring piezometer (monitoring well; AMW-01) within a second boring, and three test pit excavations (ATP-01 through ATP-03). See Figure 3 for the locations of the explorations. Of note, the proposed location of the pump station infrastructure was shifted slightly to the south (to the location shown on Figure 3) by Anchor after completion of the explorations.

Descriptions of the soils encountered in the explorations, as well as the depths where characteristics of the soils changed, are indicated on the exploration logs presented in Appendix A of this report.

Definitions of the terminology and symbols used on the logs are provided in an Exploration Log Key in Appendix A.

2.3.1 Soil Borings

On November 18, 2020, Western States Soil Conservation, Inc. (WSSC), under full-time observation and direction of an Aspect geologist, used mud rotary drilling methods to advance two soil borings (AB-01 and AMW-01) near the proposed location of pump station infrastructure elements. Soil borings AB-01 and AMW-01 were completed to total depths of 36.5 feet and 41.5 feet bgs, respectively.

Soil samples were collected during the drilling at the discretion of the Aspect geologist from a 2-inch-diameter split spoon sampler utilizing the Standard Penetration Test (SPT) in general accordance with methods described in ASTM D1586, as well as a non-standard 3-inch-diameter split spoon sampler ('non-SPT'; where indicated on the boring logs) to increase sample recovery in gravel soils. Soil samples were identified by the Aspect geologist in general accordance with methods in ASTM D2488, *Standard Practice for Description and Identification of Soils* (Visual-Manual Procedure).

Descriptions of soils encountered in the soil borings, as well as the depths where characteristics of the soils changed, are shown on exploration logs in Appendix A. Selected soil samples were submitted to a geotechnical laboratory for testing (described in Section 2.3.4).

Upon completion of drilling and sampling, soil boring AB-01 was backfilled with bentonite chips to the ground surface, compliant with the provisions in Washington Administrative Code (WAC) Section 173-160 (WAC, 2020a). Soil boring AMW-01 was completed as a monitoring well as described in the follow section.

2.3.2 Monitoring Well

Under Aspect's direction, WSSC installed a 2-inch-diameter polyvinylchloride (PVC) monitoring well within the AMW-01 boring. The monitoring well was installed per the exploration completion notes shown on the exploration logs in Appendix A with methods compliant with the provisions in WAC Section 173-160 (WAC, 2020a). The well was completed at the surface with an 8-inch-diameter flush-mount monument set into concrete.

After the monitoring well was installed, Aspect developed the well using a pump and bailer to remove sediment and turbid water prior to slug testing (described below in Section 2.3.2.1). During development of AMW-01, it was determined that the well screen had become clogged by drilling mud.

2.3.2.1 Slug Testing

Aspect performed slug testing in the monitoring well to estimate the hydraulic conductivity of the water-bearing soils around the screened zone between 30 and 40 feet bgs. In general, slug testing was completed by inserting a solid rod (slug) of known volume into the monitoring well AMW-01 casing to induce a rapid change in the water level. The induced change in water level simulated near-instantaneous addition and removal of water, creating a rising and then falling head condition, respectively. Given that AMW-01 was completed with mud rotary methods to prevent soil heave during drilling, we strongly suspect that development of the monitoring well may have been compromised by residual drilling mud remaining in the sand filter pack and the surrounding water bearing soil.

Based on our field observations, slug test results from AMW-01 are likely not representative of the hydraulic conductivity of the surrounding soils. Because of this, the slug testing results are not included as part of this report.

2.3.3 Test Pits

On December 8, 2020, Leavenworth Electric & Excavation, Inc, under full-time observation and direction of an Aspect geologist used a tracked excavator (John Deere 160LC) to excavate three test pits designated as ATP-01 through ATP-03 at the locations shown on Figure 3.

ATP-01, APT-02, and ATP-03 were excavated to 14.0, 15.0, and 15.0 feet bgs, respectively. Bulk soil samples were collected during excavation at the discretion of the Aspect geologist and identified the soil in accordance with ASTM D2488 (Visual-Manual Procedures). After completion, the test pits were backfilled with excavated material and tamped into place with the excavator bucket.

Descriptions of the soils encountered in the test pits, as well as the depths where characteristics of the soils changed, are indicated on the exploration logs presented in Appendix A of this report.

2.3.4 Geotechnical Laboratory Testing

Select soil samples collected from the explorations were submitted to a geotechnical testing laboratory to complete natural moisture content and grain size distribution (with and without hydrometer analysis) as indicated on the exploration logs in Appendix A. The results of the geotechnical laboratory testing are shown in Appendix B.

2.4 Stratigraphy

Stratigraphy in subsurface explorations completed at the pump station infrastructure area was observed to generally consist of a 6- to 12-inch-thick layer of topsoil consisting of silty sand (SM); very loose or dense (due to being frozen), moist, brown or dark brown; with variable grass and rootmass overlying alluvium deposits, consistent with the mapped surficial geology. The alluvium was generally divided into three layers as described below:

- **Upper Silty Sand and Sandy Silt (SM and ML) –** Very loose to medium dense, moist or wet; brown or gray; non-plastic silt (Photograph 1). This material was observed to extend 9 to 10 feet bgs equal to about Elevation 1,113 feet[2] in AB-01, Elevation 1,110 feet in AMW-01, and Elevation 1,109 feet in ATP-01 through ATP-03.
- **Gravel with Sand and Cobbles (GW) –** Medium dense to dense, wet, gray brown (Photograph 2); this material was observed below the Upper Silty Sand and Sandy Silt and extended to about 23 feet bgs and about Elevation 1,100 feet in AB-01, and 25 feet bgs and about Elevation 1,093 ft in AMW-01. This material

[2] All elevations are determined using North American Vertical Datum of 1988 (NAVD88).

was also observed in ATP-01 through ATP-03 below the Upper Silty Sand and Sandy Silt to total exploration depths of 14 to 15 feet bgs. We interpret some of the SPT and non-SPT blow counts in the gravel with sand and cobbles layer may be overstated at some depths due to the gravel and cobbles content.

- **Lower Silty Sand and Sandy Silt (SM and ML) –** Medium dense, wet, brown or gray; non-plastic silt. This material was observed below the Gravel with Sand and Cobbles in both AB-01 and AMW-01 to the total exploration depths of 36 and 42 feet bgs, respectively.

Photograph 1. Upper Silty Sand and Sandy Silt Alluvium

Photograph 2. Gravel with Sand and Cobbles Alluvium

Subsurface explorations were not completed along the main pipeline and laterals under our scope of work. From desk-study review and Site reconnaissance, our observations are generally inferred to be similar to those at the pump station infrastructure area, with one exception. Of exception, some areas along the main pipeline alignment described in detail in Section 2.2.1 are expected to consist of shallow nested boulders and/or bedrock underlying a thin layer of topsoil.

In addition, fill may be present from past grading and utility installation along the laterals. We anticipate the fill will generally be of similar composition to the native alluvium soils but might include other imported materials.

2.5 Groundwater

The reviewed well logs (Ecology, 2020) report static groundwater levels are more than 20 feet bgs at/near the main pipeline and lateral alignments. Static groundwater level or seepage was not observed along the main pipeline and lateral alignments during the Site reconnaissance completed in April 2018. We observed several areas of ponded water in localized low-lying areas of the COIC canal base measuring less than about 10 feet long, 2 to 4 few feet wide, and less than 6 inches deep. We suspect the ponded water is a result of accumulated precipitation and/or irrigation runoff in the COIC canal. Seepage and perched and ponded groundwater at the main pipeline and lateral alignments is anticipated to vary seasonally with weather and irrigation practices, especially with filling and emptying of the COIC canal.

At the pump station infrastructure area, the at-time of drilling/excavation (ATD) groundwater levels in the explorations ranged from 7.5 to 10.5 feet bgs (approximately Elevation 1,111.5 to 1,108.5 feet). The static groundwater level was measured in monitoring well AMW-01 and was 10.0 feet bgs (1,111 feet) on December 8, 2020. In general, the observed/measured groundwater level was roughly equal to the nearby creek level at the time of observations/measurement on November 18 and December 8, 2020.

Abundant groundwater seepage was observed to occur near the top of the gravel with sand and cobbles (GW) alluvium in all three of the test pits explorations.

Groundwater levels, including seepage, are expected to vary seasonally with changes in weather, snowmelt, and water level in the creek. We understand from Anchor the water level in the creek ranges from about Elevation 1,105 feet during late-summer to as high as Elevation 1,119 feet (approximately equal to Site ground surface elevation) during the design 100-year flood event. We recommend measuring groundwater levels in AMW-01 before final design and construction to better understand and correlate Site groundwater levels compared to creek levels.

We recommend assuming groundwater levels at the proposed pump station area fluctuate in concert with and generally match the creek surface water elevation. In addition, we recommend the pump station infrastructure buried structures be designed assuming the static groundwater level will rise up to the ground surface elevation at some time during the design life.

3 Geotechnical Design and Construction Recommendations

3.1 Main Line and Lateral Pipes

In our opinion, excavation, installation, and backfilling of new pressurized sections of pipelines to relatively shallow depths at the proposed main pipeline (less than about 12 inches bgs below COIC canal base) and lateral alignments (less than about 5 feet bgs) is feasible from a geotechnical perspective. Sections of the pipe installation will require bedrock and/or boulder excavation, potentially requiring blasting or mechanical rock breaking. The following sections provide preliminary geotechnical design and construction recommendations to support 30% design and cost estimating of main line and lateral pipes.

3.1.1 Excavation and Temporary Slopes

We anticipate most materials encountered for excavation down to about 12 inches below the COIC canal base existing ground surface to main pipeline invert, and as deep as 5 feet to lateral pipeline inverts, will consist of a layer of topsoil overlying native alluvium or fill consisting of mixtures of silt, sand, gravel, cobble, and boulders, with some exceptions.

Of exception, we anticipate main line pipe excavations deeper than about 6 to 12 inches along an 800-foot long (approximate) section between about Cemetery Road and Fish Hatchery Road, and a 1,300-foot-long (approximate) section between Fish Hatchery Road to just beyond East Leavenworth Road (Figure 2) will likely encounter shallow, nested boulders and/or bedrock. Based on the geologic setting, this condition might also exist in other localized areas, particularly near mapped bedrock outcrops along the west edge of the COIC canal and near Prowell Street, and/or seemingly random locations anywhere along the main line and lateral alignments.

In general, we anticipate the contractor can excavate along the main pipeline and lateral alignments using standard excavation equipment such as large, tracked excavators, except where boulders and/or bedrock are encountered. In these areas, we expect standard excavation equipment will need to be supplemented by rock/boulder chipping and splitting excavator attachments, demolition grout, and/or drilled-in, targeted explosive charges.

Maintenance of safe working conditions, including temporary excavation stability, is the responsibility of the contractor. All temporary cuts in excess of 4 feet in height that are not protected by trench boxes or otherwise shored, should be sloped in accordance with Part N of WAC 296-155 (WAC, 2020b) for worker safety. For planning purposes, using guidance provided by the WAC, and our observations, we suspect the soils along the main line and lateral alignments are “Type C” with a maximum allowable temporary slope inclination of 1.5H:1V.

With time and the presence of seepage and/or precipitation, the stability of temporary unsupported cut slopes can be significantly reduced. Therefore, all temporary slopes should be protected from erosion by installing a surface water diversion ditch or berm at the top of the slope if precipitation is expected.

In addition, the contractor should monitor the stability of the temporary cut slopes and adjust the construction schedule and slope inclination accordingly. Vibrations created by traffic and construction equipment may cause caving and raveling of the temporary slopes. In such an event, lateral support for the temporary slopes should be provided by the contractor.

3.1.2 Dewatering

We do not anticipate the static groundwater level will be encountered for main line and lateral pipelines excavations and assume irrigation water in the COIC canal will be shut off before and during construction. We expect localized perched groundwater or sidewall seepage will be encountered in some localized areas of excavation, especially at the main pipeline alignment if excavation occurs during or within a few weeks of water ponded in the COIC canal from irrigation, snowmelt, and/or rain that has not drained out.

We recommend the contractor be prepared to dewater seepage in excavations with sumps and pumps or other means, and/or modify pipe installation means and methods accordingly (i.e., working toward the upgradient direction).

3.1.3 Pipe Subgrade Support

We anticipate most subgrade materials encountered at the main and lateral pipeline inverts will consist of very loose to medium dense native alluvium or fill consisting of mixtures of silt, sand, gravel with variable cobble and boulder content, or nested boulders and/or bedrock. We estimate the bearing pressure exerted on the subgrade soil by the new pipe (full of water) will be relatively low and generally less than the pressure exerted by the volume of soil removed.

We recommend pipeline invert subgrade (below the pipe bedding) consist of relatively firm and unyielding soil and/or rock subgrade that is free of relatively sharp or pronounced edges from underlying cobbles and boulders. All pipe should be underlain by at least 6 inches of compacted Gravel Backfill for Pipe Zone Bedding as defined by the Washington State Department of Transportation (WSDOT) Standard Specifications Section 9-03.12(3) (WSDOT, 2021), or in accordance with pipe manufacturer recommendations.

Pipe invert subgrade that is observed to be soft, muddy, or contain abundant organic material such as rootmass or peat should be subexcavated and removed to a maximum depth of 1 foot below planned subgrade elevation. If such conditions persist deeper than 1 foot below planned subgrade, we recommend placing and tamping angular ballast or quarry spall material into the exposed surface to a create a relatively firm and unyielding surface.

Open-graded void space at the ballast or quarry spall surface should be choked with Crushed Surfacing Base Coarse (CSBC) as defined by WSDOT Standard Specifications Section 9-03.9(3), or pipe bedding material with relatively high gravel content (at least 50-percent by mass). Alternatively, the ballast or quarry spalls material can be covered with a geotextile separator fabric prior to placing required thickness of pipe bedding material. The geotextile should be non-woven and generally meet the requirements as defined by WSDOT Standard Specifications 9-33-1, Table 3 for “Separation” and “Non-Woven” material.

3.1.4 Pipe Bedding, Cover and Excavation Backfill Materials

We recommend pipe be bedded and covered by at least 6 inches of compacted Gravel for Pipe Zone Bedding material as defined by WSDOT Standard Specifications Section 9-03.12(3), or in accordance with pipe manufacturer recommendations.

Beneath nonroad areas, we recommend material above the pipe cover used to backfill the excavation to ground surface consist of compacted material generally meeting the requirements for Common Borrow as defined by WSDOT Standard Specifications Section 9-03.14(3). We estimate the majority of on-Site and/or excavated materials (those not containing organics such as topsoil, or clay) can likely serve as borrow material for this purpose.

Beneath road areas, we recommend backfill and pavement section be placed and compacted in general accordance with applicable Chelan County Standard Road Cut Repair and Requirements (Chelan County, 2016) requirements.

Pipe bedding, cover and excavation backfill should be placed and compacted in general accordance with the methods described in WSDOT Standard Specifications 2-03.3(14) C, Method C (WSDOT, 2021) with some exceptions.

Of exception, it is our experience that nuclear densometer field compaction testing of materials containing more than 25 percent or more (by volume) of gravel or particles greater than 4 inches in diameter cannot be applied with a reasonable degree of accuracy. In this situation, we recommend a given amount of effort be used to compact the material as described by WSDOT Standard Specification for Rock Embankment Construction 2-03.3(14)A (WSDOT, 2021).

3.2 Pump Station Infrastructure

In our opinion, design and construction of the pump station infrastructure is feasible from a geotechnical perspective and will require a contractor-designed combination of temporary slopes and shoring, groundwater cutoff and/or active dewatering during construction. The following sections provide general assumptions and geotechnical engineering recommendations and considerations to support design and cost estimating of pump station infrastructure.

3.2.1 General Type, Size, and Location Assumptions

Based on the 60% Design Drawings (Anchor, 2021), the pump station infrastructure is proposed to consist of a buried settling basin and pump station/wet well (wet well) with an intake pipe located at the left bank of the creek to draw creek water (approximate locations shown on Figure 3).

The approximate footprint (in plan) of the settling basin and wet/well is planned to be rectangular in shape with a length of about 54 feet (parallel to the shoreline) and a width varying from 14 to 22 feet (Figure 3). The top of the settling basin and wet well slab is planned at Elevation 1,095.7 feet (approximately 24 feet below existing grade) resulting in estimated excavation depth of approximately 26 feet deep.

A 30-inch-diameter HDPE intake pipe is planned to extend from just above the base of the settling basin slab and extend about 100 linear feet toward and into the creek (to the east). A flow control manhole is proposed to be located approximately mid-length along the intake pipe at approximate elevation 1,096 feet (approximately 24 feet bgs).

3.2.2 Excavation

Based on the Site subsurface explorations data, we expect excavation for the pump station infrastructure as deep as 26 feet bgs will extend through and expose silty sand, sandy silt and gravel with sand and cobble (SM, ML, GW) alluvium described in Section 2.4. We expect these materials can generally be excavated with standard excavation equipment such as large, tracked, long-reach excavators, and/or clamshell buckets suspended from a mobile crane. We expect variable cobble and boulder content, possibly requiring chipping or splitting to remove, will be encountered in the alluvium.

We expect excavation for the pump station infrastructure will extend below Site groundwater level and require a combination of temporary slopes and shoring, groundwater cutoff and/or active dewatering during construction.

3.2.2.1 Temporary Slopes

Maintenance of safe working conditions, including temporary excavation stability, is the responsibility of the contractor. All temporary cuts in excess of 4 feet in height that are not protected by trench boxes or otherwise shored, should be sloped in accordance with Part N of Washington Administrative Code (WAC) 296-155 for worker safety. Based on the Site soils, it is our opinion that temporary slopes should have a maximum slope of 1.5H:1V above the groundwater level. Shallower slopes, especially beneath the groundwater level, will likely need to be considered in accordance with Part N WAC 296-155.

With time and the presence of seepage and/or precipitation, the stability of temporary unsupported cut slopes can be significantly reduced.

In addition, the contractor should monitor the stability of the temporary cut slopes and adjust the construction schedule and slope inclination accordingly. Vibrations created by traffic and construction equipment may cause caving and raveling of the temporary slopes. In such an event, lateral support for the temporary slopes should be provided by the contractor.

3.2.2.2 Temporary Shoring

We assume the contractor will be responsible for designing the temporary shoring in accordance with Part N of the WAC 296-155 (WAC, 2020b). In our opinion, dewatering open-cut excavations using a vacuum well-pump system or multiple wells without shoring and groundwater cutoff will be cost prohibitive and impractical because of the anticipated high inflow rates through for the gravel with sand and cobbles alluvium into the excavation. Therefore, we recommend excavations extending below groundwater be supported with shoring that can function to cutoff groundwater within the highly permeable gravel with sand and cobbles alluvium observed between 9 and 25 feet bgs.

We consider feasible temporary shoring methods to consist of, but not be limited to, a cofferdam/cell system created from interlocking driven sheet piles or secant pile shoring, which consists of a row of drilled shafts filled with structural concrete and reinforced with steel rebar cages. We estimate secant piles to be prohibitively expensive and that the excavation will likely be shored utilizing driven sheet piles. Given the depth of the excavation, internal bracing consisting of wales and struts is expected to be needed to support shoring.

In our opinion, driving of sheet piles at the Site will likely encounter obstructive cobbles and possibly boulders in some areas, requiring dislodgment and/or breaking using an excavator (pre-trenching), a large driven steel H-pile (pre-spudding), and/or percussive air rotary drilling equipment (pre-drilling). We recommend shallow pre-trenching be considered along the entire proposed shoring alignment to clear the shoring alignment of near-surface obstructions, debris, and other obstructions (if any). Deeper obstructions could be removed from the shoring alignment on an ad-hoc basis via pre-drilling and/or pre-spudding.

We recommend the active earth pressure diagram shown on Figure 4 be used for basis of preliminary design and cost estimating of temporary shoring assuming that the excavation backslope is relatively flat and that the shoring is supported by internal bracing. Seismic loading is not included on Figure 4 due to the temporary nature of the shoring. Detailed surcharge loading such as construction equipment and geometry variations in retained soil backslope should be evaluated by the shoring designer on a case-by-case basis.

It should be clear in the contract documents that the Contractor is responsible for designing the temporary shoring. The geotechnical recommendations presented in this report were developed using assumptions that may or may not be applicable for the Contractor’s operations and shoring installation methods. It is the contractor’s responsibility to verify the provided lateral earth pressures and surcharge pressures are appropriate for their operation, excavation/shoring configuration (i.e., level or sloping ground above shoring) or use the soil and groundwater conditions appropriate for the Project area develop their final design.

We envision temporary shoring for excavation and installation of the intake pipe extending out from the base of the stilling basin and into the creek could be accomplished using large aggregate or water-filled bags (often referred to as ‘super sacks’ and an ‘Aquadam’) to create a coffer dam/cell or other in-water shoring techniques during a period of low creek flow.

Alternatively, internally braces steel sheet pile coffer cells could be utilized to allow for the excavation to occur in the dry with pre-trenching and pre-spudding considerations described above for the settling basin and wet well excavation.

3.2.2.3 Dewatering

We assume the contractor will be responsible for designing groundwater cutoff and dewatering system. In general, the groundwater cutoff and dewatered must be designed and constructed to adequately control groundwater levels inside excavations/shoring to allow for safe construction, subgrade preparation and maintenance of base stability to prevent heave.

Significant dewatering effort is anticipated for the pump station infrastructure excavation planned to extend to depths of about 26 feet bgs. The static groundwater level elevation is expected to be highly connected to the creek level elevation and is likely to fluctuate throughout the year. Table 1 below summarizes the variation of creek level at the site throughout the year (Anchor, 2021) and estimated depth of excavation below groundwater level.

Table 1. Icicle Creek Surface Water Elevations

Creek Water Surface Condition	Approximate/Estimated Groundwater Level Elevation[1] (feet)	Approximate/Estimated Excavation Depth Below Groundwater Level[1] (feet)
Design 100-Year Flood Event	1,119.7	26.0
Extreme High Late-Spring	1,113.2	19.5
Typical Late-Summer	1,107.0	13.3
Extreme Low Late-Summer	1,105.4	11.7

Notes:

1. Approximate/estimated groundwater levels assume groundwater level at excavation area is approximate to the creek level.

Base heave from unbalanced hydrostatic pressures inside and outside temporary shoring and groundwater cutoff system is a major design and construction consideration for excavations extending below groundwater level. We recommend groundwater levels be kept at least 2 feet below the base of the excavations, or deeper as the Contractor’s shoring design indicates, to prevent heave or “quick” condition. In addition, we recommend that the temporary shoring utilized consist of relatively water-tight sheet piles which are embedded a minimum of 10 feet below than the base of excavation, or deeper as the Contractor’s shoring design indicates. Utilizing this type of shoring/cutoff system will partially hydraulically isolate the excavation from the high permeability alluvium deposits from the excavation.

With the expectation that the excavation will be hydraulically isolated from the surrounding aquifer by the sheet pile shoring/cutoff, we recommend the Contractor consider internal pumping wells or vacuum well point system, combined with sumps and pumps at the excavation base. We assume that a water collection pipe would be installed around the perimeter of the shoring, and groundwater removed from the excavation would be treated to meet water quality requirement prior to discharge, presumably to the creek.

Pumping wells can effectively dewater large areas in permeable soils and may produce large amounts of water. Dewatering pumping wells typically consist of 6- to 12-inch casing installed in 8- to 36-inch boreholes extending below the excavation base. Screen designs and filter packs are specified based on the texture of the water-bearing zone. Submersible pumps are generally used.

Vacuum well point systems generally provide more localized dewatering control than pumping wells and are installed 6 to 8 feet on center along the perimeter of the shoring/cutoff wall depths below the base of excavation. The well points are connected to a vacuum head system with maximum vacuum/pumping depth of about 15 feet deep below the header.

The estimated hydraulic conductivity of the alluvium observed in explorations at the Site was estimated using correlations to grain size distribution results and porosity assumptions (Carrier, 2003). These estimations should be considered approximate, with localized variability in alluvium expected to affect the hydraulic conductivity. A summary of our estimations of the horizontal hydraulic conductivity (expressed in units of centimeter per second (cm/sec) of the alluvium is shown below:

- Silty sand and sandy silty (SM, ML): between $3*10^{-3}$ and $5*10^{-4}$ cm/sec
- Gravel with sand and cobbles (GW): between $3*10^{-2}$ cm/sec and $9*10^{-3}$ cm/sec

The vertical hydraulic conductivity of granular materials is typically between 2 and 10 times lower than horizontal hydraulic conductivity in naturally deposited soils.

3.2.3 Lateral Pressures on Permanent Walls

The recommended permanent earth pressure diagram for the pump station infrastructure permanent walls is presented on Figure 5. We assume that the permanent walls will be fixed or restrained against lateral rotation resulting in the ‘at-rest’ earth pressure conditions with level finished grade above the walls.

In addition, we recommend the Contractor should consider a range of water levels both inside and outside the permanent walls when computing unbalanced hydrostatic pressures. We recommend considering that the static ground water level will rise to the ground surface during a large flood event. Lateral surcharge pressures from surface loading should be considered for final design when the Project layout/loading is better defined.

3.2.4 Vertical Foundation Support

Based on the results of the completed subsurface exploration program, the alluvium described in Section 2.4 will be encountered at the foundation/slab elevation of the pump station infrastructure.

Provided base heave of the foundation subgrade is controlled by properly designed and constructed excavation, shoring and dewatering, it is our opinion that alluvium is generally suitable foundation subgrade if properly prepared.

We recommend all foundation bearing surfaces be prepared by exposing alluvium soils and clearing them of disturbed, muddy, or softened soil. Subsequently we recommend the alluvium subgrade be overlain by a 12-inch-thick layer of compacted, imported gravel (leveling pad) meeting the requirements for Class A Gravel Backfill for Foundations as defined by WSDOT Standard Specifications Section 9-03.12(1)A. The leveling pad material should be compacted to a firm and unyielding condition. We recommend all bearing surfaces be observed by an Aspect geotechnical engineer or geologist prior to placing the leveling pad to verify that appropriate bearing subgrade material has been exposed and properly prepared.

Any unsuitable subgrade soils should be subexcavated to expose relatively firm and unyielding subgrade and replaced with material meeting the requirements of Class A Gravel Backfill for Foundations compacted to a relatively firm and unyielding condition.

Foundations prepared in accordance with the recommendations above can be designed for a maximum allowable (net) bearing pressure of 2,000 pounds per square foot (psf). The recommended maximum allowable bearing pressures may be increased by 1/3 for short-term transient loading conditions.

We estimate total settlement of infrastructure foundations will be less than about 1 inch and with differential settlement across the foundation of less than about 1/2 inch. We anticipate that the majority of the estimated settlement will occur during construction as the loads are applied. The structures may also exhibit some of the anticipated settlements after construction as the structures are initially loaded with water and pumped dry. This cyclic loading may result in minor structure rebound and re-settlement.

3.2.5 Buoyancy

Submerged structures and pipes will be subjected to upward buoyancy forces when the water level around the structure/pipe is higher than the water level inside the structure/pipe. Groundwater level across the Site should be assumed at the ground surface for the calculation of uplift pressures (assuming large flood event conditions).

The submerged structures and intake pipe should be designed to resist this upward force and to prevent possible heave and cracking of the structure base. This can typically be accomplished by increasing the weight of the structure, constructing an extended foundation base, structurally tying the structure to the temporary shoring that is left in place, or a group of uplift anchors (such as helical piles or micropiles) that extend below the base. We recommend a minimum long-term factor of safety against buoyancy uplift of 1.3 in accordance with U.S. Army Corps of Engineers (USACE) *Design of Concrete Structures Guidelines* (USACE, 2005). See Figures 6 and 7 for guidance in determining buoyancy uplift resistance for extended foundation base, and permanent shoring wall alternatives, respectively. In our opinion, uplift anchors to mitigate buoyance may be more expensive than the other alternatives. We are available to assist the design team with further evaluation of the uplift anchor alternative (anchor/pile type, size/diameter, and location/depth) if needed.

Alternatively, buoyancy could be mitigated by placing weep holes in base and/or walls of the submerged structures thereby preventing development of an unbalanced water level condition inside and outside the structure. This alternative could result in maintenance challenges when groundwater levels exceed the weep hole elevations and the structures is full of water.

3.2.6 Lateral Foundation Support

Lateral forces can be resisted by a combination of sliding resistance of its foundation on the underlying soil and passive earth pressure against the buried portions of the structure. For use in design, an allowable coefficient of friction of 0.55 may be assumed along the interface between the base of the foundation and underlying gravel leveling pad.

An allowable passive earth pressure value of 300 and 150 pounds per cubic foot (pcf) may be assumed above and below Site groundwater level, respectively, provided foundations are cast directly against relatively undisturbed native soils or structural fill that is relatively level. The recommended coefficient of friction and passive earth pressure values include a safety factor of about 1.5.

3.3 Permanent Slopes and Erosion Control

At the time of this report, it is not well understood or defined the exact proximity of structures to the creek shoreline, or proposed grading at the shoreline. In general, we recommend permanent slopes be graded flatter than 2H:1V and be comprised of alluvium or properly compacted structural fill.

Permanent slopes should be covered with erosion and/or scour resistant materials such as hydroseeding or rip rap suitable to slope location and environment. (i.e., shoreline exposed to flow).

Geotechnical consideration or permanent slope stability and erosion control should be revisited during final design to verify adequate long-term stability and erosion/scour protection.

3.4 Structural Fill

Structural fill around the pump station infrastructure may generally consist of on-Site granular sand or gravel materials generally meeting the requirements for WSDOT Common Borrow (9-03.14(3)), except maximum particle size shall be limited to 6 inches in diameter. We estimate the majority of on-Site and/or excavated materials (those not containing organics (topsoil) or clay) can likely serve as borrow material for this purpose if taken from above the Site groundwater level, or allowed to dry in windrowed piles.

The fines content of the on-Site material is high enough that it may be difficult to properly compact if the moisture content is more than 3-percent above or below optimum moisture for compaction, or during wet weather. If needed, we recommend imported material to be used as structural fill around structures consist of material generally meeting the requirements of WSDOT Gravel Borrow (9-03.14(1)). Structural fill around the pump station infrastructure should be compacted in accordance with the methods described in WSDOT Standard Specifications 2-03.3(14) C, Method C, or Rock Embankment Construction 2-03.3(14)A (WSDOT, 2021) as applicable.

Structural fill below foundations as leveling pad or to replace unsuitable subgrade material should meet the requirements for Class A Gravel Backfill for Foundations as defined by WSDOT Standard Specifications Section 9-03.12(1)A and be compacted to a firm and unyielding condition.

References

Anchor QEA, LLC, 2021, Cascade Orchard Irrigation Company Improvement Project, Revised 30% Design, Phase III: Intake and Pumping Facilities, Dated March 2021.

ASTM International (ASTM), 2018, 2018 Annual Book of ASTM Standards, West Conshohocken, Pennsylvania.

Carrier III, W. D., 2003, Goodbye, Hazen; Hello, Kozeny-Carman, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 129, No. 11, pages 1054-1056.

Chelan County, 2016, Road Cut Repair Standards and Requirements, Online at: https://www.codepublishing.com/WA/ChelanCounty/html/Chelco15/Chelco1530.html

U.S. Army Corps of Engineers (USACE), 2005, Stability Analysis of Concrete Structures, Publication EM 1110-2-2100, prepared by the USACE, December 1, 2005.

Washington Administrative Code (WAC), 2020a, Chapter 173-160, Minimum Standards for Construction and Maintenance of Wells, Online at: https://app.leg.wa.gov/wac/default.aspx?cite=173-160

Washington Administrative Code (WAC), 2020b, Chapter 296-155, WAC Construction Work, Part N – Excavation, Trenching, and Shoring, Online at: https://www.lni.wa.gov/safety-health/safety-rules/chapter-pdfs/WAC296-155.pdf

Washington State Department of Ecology (Ecology), 2020, Washington State Well Log Viewer, Website accessed January 2020: https://fortress.wa.gov/ecy/waterresources/map/WCLSWebMap/.

Washington Department of Natural Resources Division of Geology and Earth Resources (DNR), 2021, Washington Geologic Information Portal, 2021, online at: https://geologyportal.dnr.wa.gov/ accessed March 2021.

Washington State Department of Transportation (WSDOT), 2021, Standard Specifications for Road, Bridge and Municipal Construction, Document M 41-10.

Limitations

Work for this project was performed for Anchor QEA (Client), and this report was prepared as a preliminary work product to provide geotechnical information to the design team to support preliminary design and cost estimating. A more detailed geotechnical report will be provided under separate contract and scope of work after more details of the project features have been defined.

This report was prepared consistent with recognized standards of professionals in the same locality and involving similar conditions, at the time the work was performed. No other warranty, expressed or implied, is made by Aspect Consulting, LLC (Aspect).

Recommendations presented herein are based on our interpretation of site conditions, geotechnical engineering calculations, and judgment in accordance with our mutually agreed-upon scope of work. Our recommendations are unique and specific to the project, site, and Client. Application of this report for any purpose other than the project should be done only after consultation with Aspect.

Variations may exist between the soil and groundwater conditions reported and those actually underlying the site. The nature and extent of such soil variations may change over time and may not be evident before construction begins. If any soil conditions are encountered at the site that are different from those described in this report, Aspect should be notified immediately to review the applicability of our recommendations.

Risks are inherent with any site involving slopes and no recommendations, geologic analysis, or engineering design can assure slope stability. Our observations, findings, and opinions are a means to identify and reduce the inherent risks to the Client.

It is the Client's responsibility to see that all parties to this project, including the designer, contractor, subcontractors, and agents, are made aware of this report in its entirety. At the time of this report, design plans and construction methods have not been finalized, and the recommendations presented herein are based on preliminary project information. If project developments result in changes from the preliminary project information, Aspect should be contacted to determine if our recommendations contained in this report should be revised and/or expanded upon.

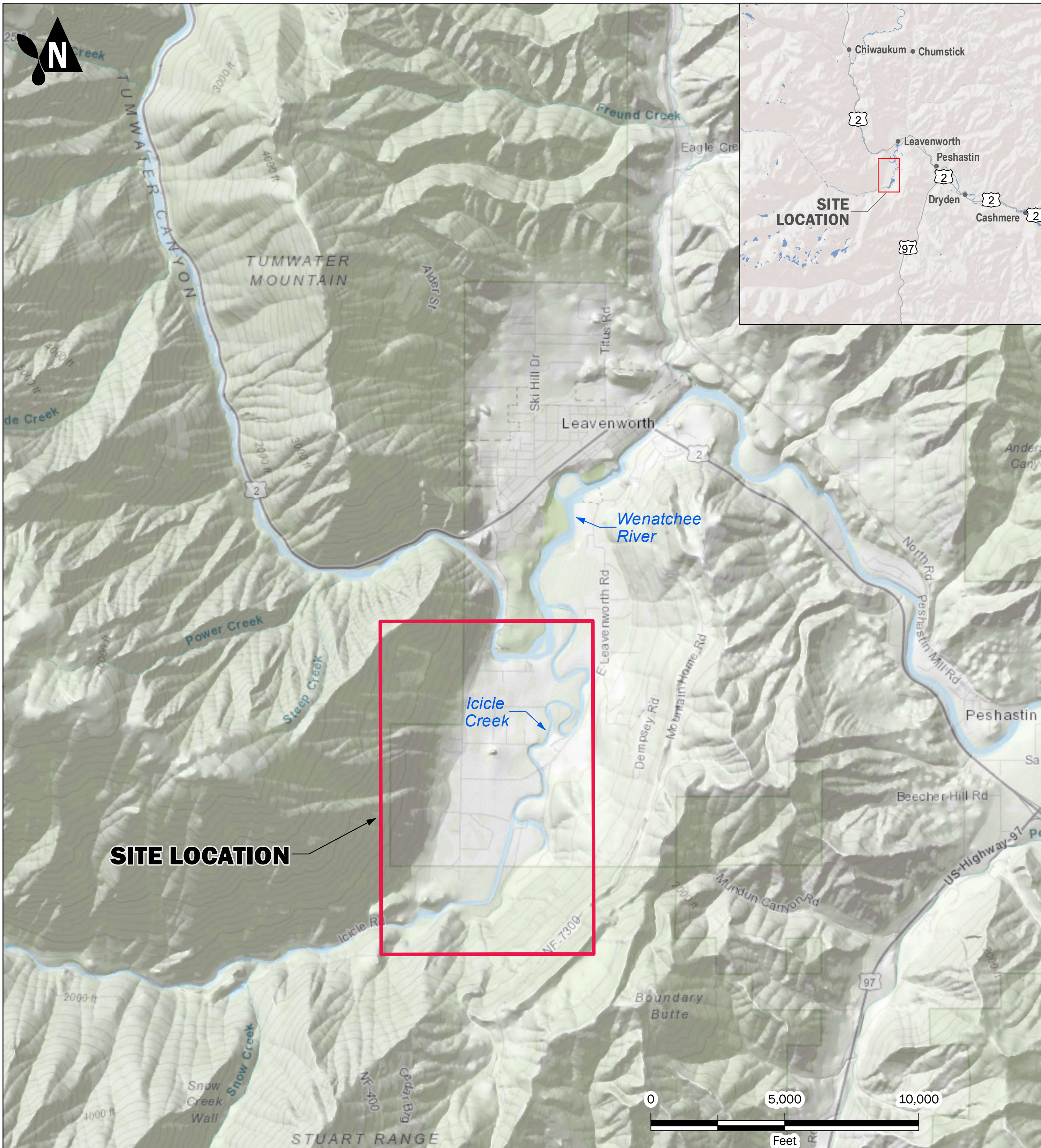
The scope of work does not include services related to construction safety precautions. Site safety is typically the responsibility of the contractor, and our recommendations are not intended to direct the contractor’s site safety methods, techniques, sequences, or procedures. The scope of our work also does not include the assessment of environmental characteristics, particularly those involving potentially hazardous substances in soil or groundwater.

All reports prepared by Aspect for the Client apply only to the services described in the Agreement(s) with the Client. Any use or reuse by any party other than the Client is at the sole risk of that party, and without liability to Aspect. Aspect’s original files/reports shall govern in the event of any dispute regarding the content of electronic documents furnished to others.

Please refer to Appendix C titled "Report Limitations and Guidelines for Use" for additional information governing the use of this report.

We appreciate the opportunity to perform these services. If you have any questions, please call Nick Szot, PE, Associate Geotechnical Engineer, at 509.888.7218.

FIGURES



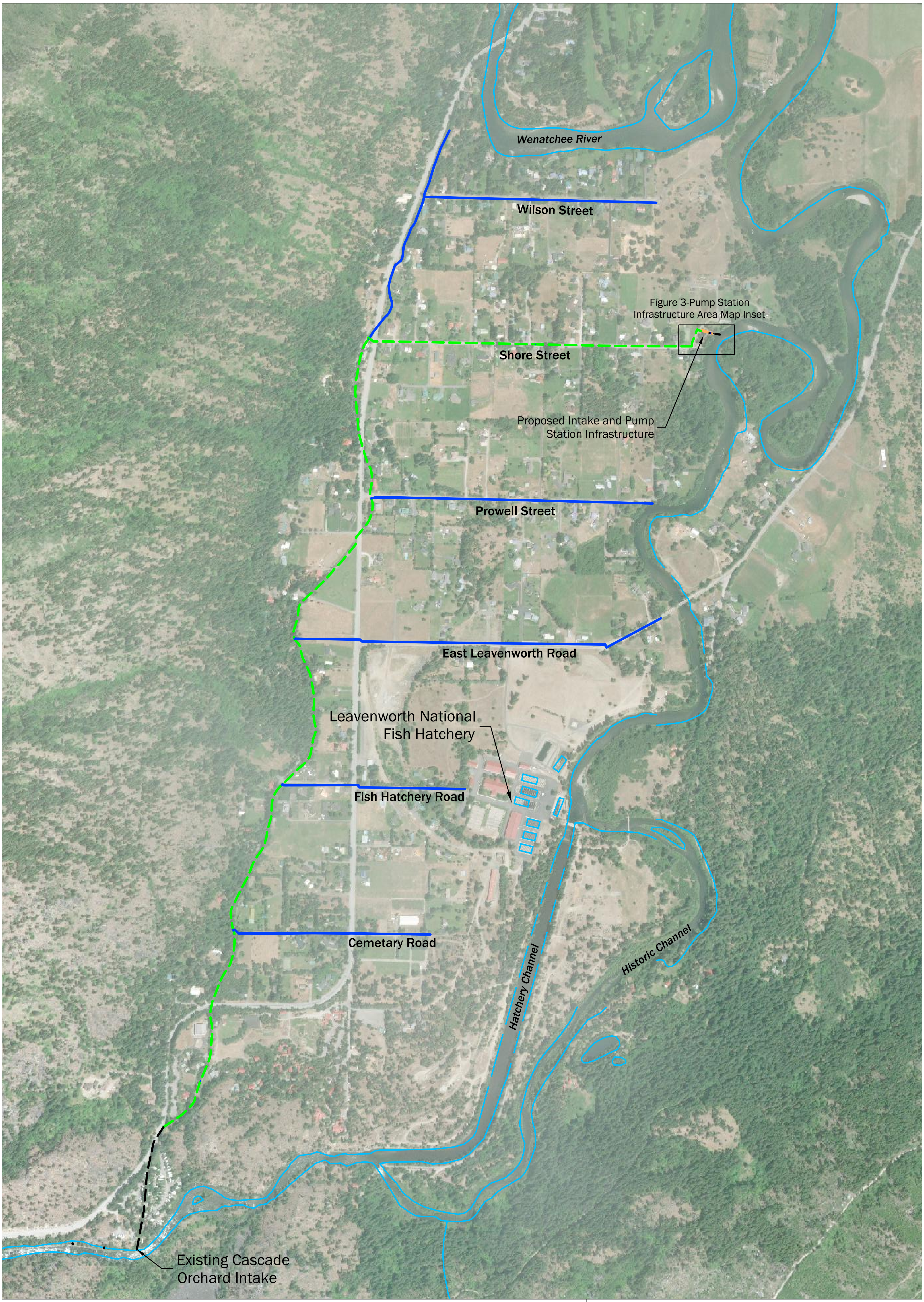
Site Location Map

Geotechnical Engineering Report
Cascade Orchard Irrigation Company Improvement Project
Leavenworth, Washington

Aspect CONSULTING	SEP-2021	BY: NCS / SCC	FIGURE NO. 1
	PROJECT NO. 170700	REVISED BY: - - -	

GIS Path: Q:_GeoTech\170700 Cascade Orchards Icicle Creek Flow Restoration\2018-11 Geotechnical Investigation\GIS\Site Location Map.mxd || Coordinate System: NAD 1983 StatePlane Washington North FIPS 4601 Feet || Date Saved: 2021-03-19 || User: trullen || Print Date: 2021-03-19

Basemap Layer Credits || Esri, HERE, Garmin, (c) OpenStreetMap contributors, and the GIS user community
Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), (c) OpenStreetMap contributors, and the GIS User Community



Legend

- Proposed Pressurized Main Pipeline Alignment
- Proposed Pressurized Pipeline Lateral Alignments
- Existing Cascade Orchard Intake Gravity Pipe - Proposed to be Abandoned

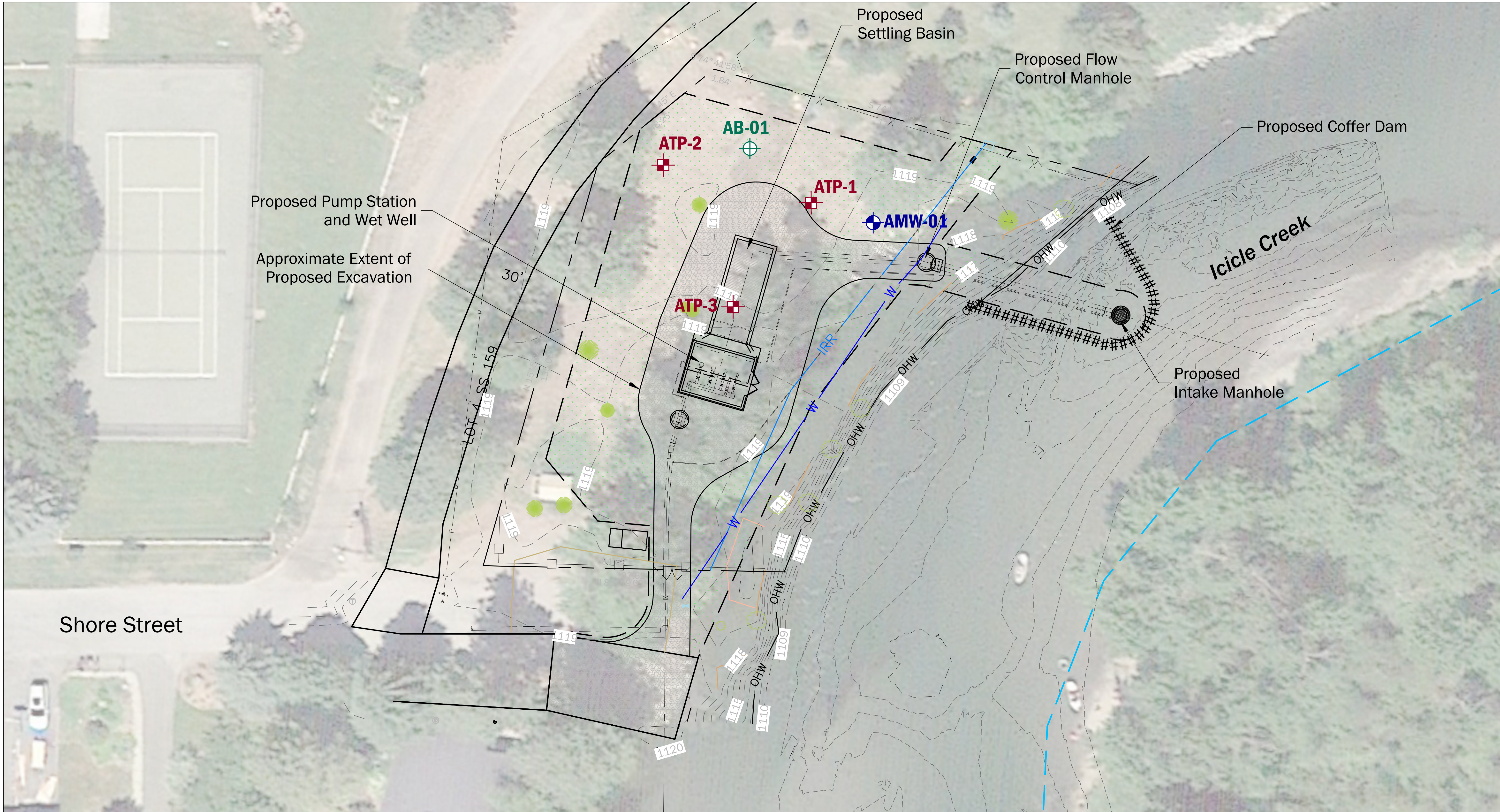
0 800 1600 Feet

Project Site Map

Geotechnical Engineering Report
Cascade Orchard Irrigation Company Improvement Project
Leavenworth, Washington

Aspect CONSULTING	Sep-2021	BY: NCS/SCC	FIGURE NO.
	PROJECT NO. 170700	REVISED BY: CMV	2

CAD Path: Q:_GeoTech\170700 Cascade Orchards Icicle Creek Flow Restoration\2021-09 Figure edits\170700-02.dwg 02 Project Site Map || Date Saved: Sep 15, 2021 12:36pm || User: cvanslyke



Legend

- Boring Location
- Test Pit Location
- Monitoring Well Location

References: Aerial imagery provided by Google Earth, 2021. Topography and bathymetry provided by Anchor QEA, March 2021.

0 30 60 Feet

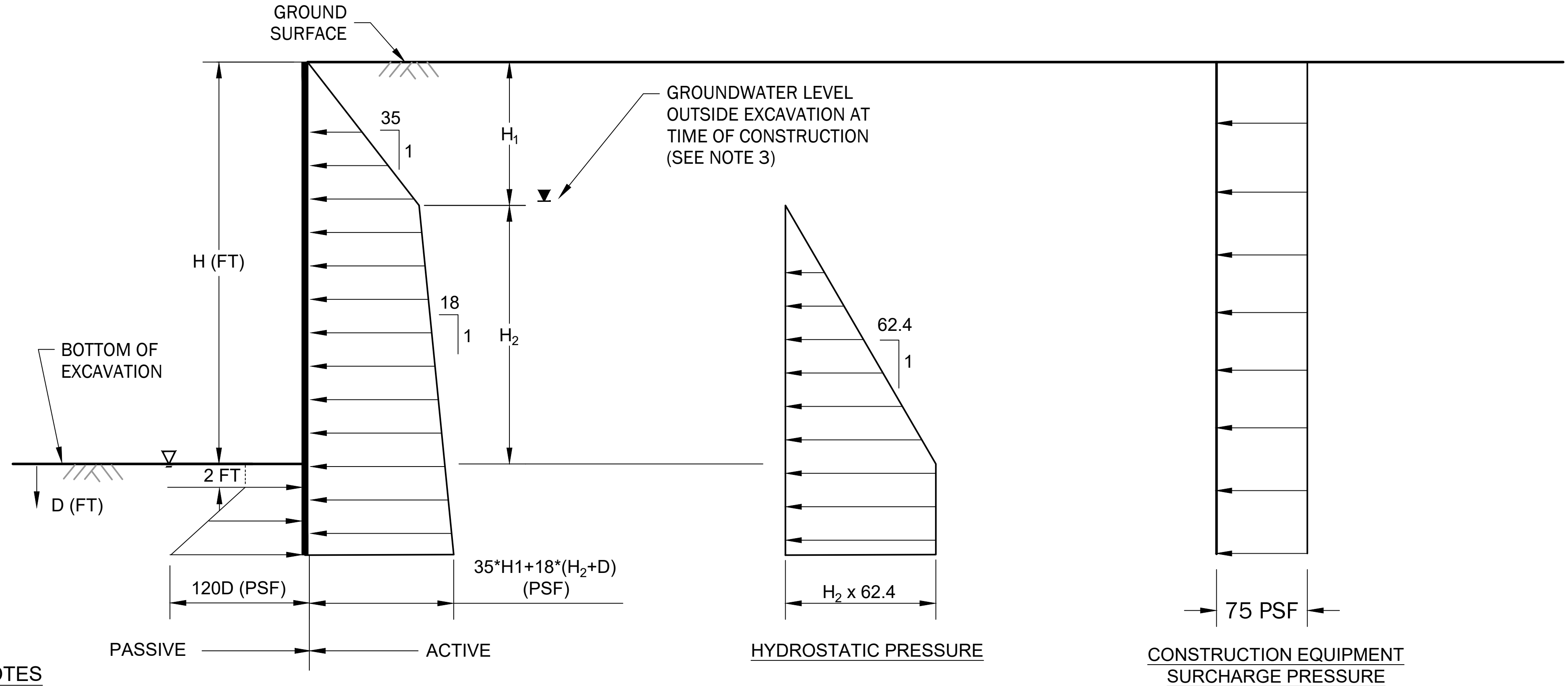
Pump Station Infrastructure Area Map

Geotechnical Engineering Report
Cascade Orchard Irrigation Company Improvement Project
Leavenworth, Washington

Aspect CONSULTING

Sep-2021	BY: RPK/CMV	FIGURE NO. 3	
PROJECT NO. 170700	REVISED BY: -		

CAD Path: Q:_GeoTech\170700 Cascade Orchards Icicle Creek Flow Restoration\2021-09 Figure edits\170700-03.dwg 03 Pump Station Site Map || Date Saved: Sep 15, 2021 12:36pm || User: cvanslyke



NOTES

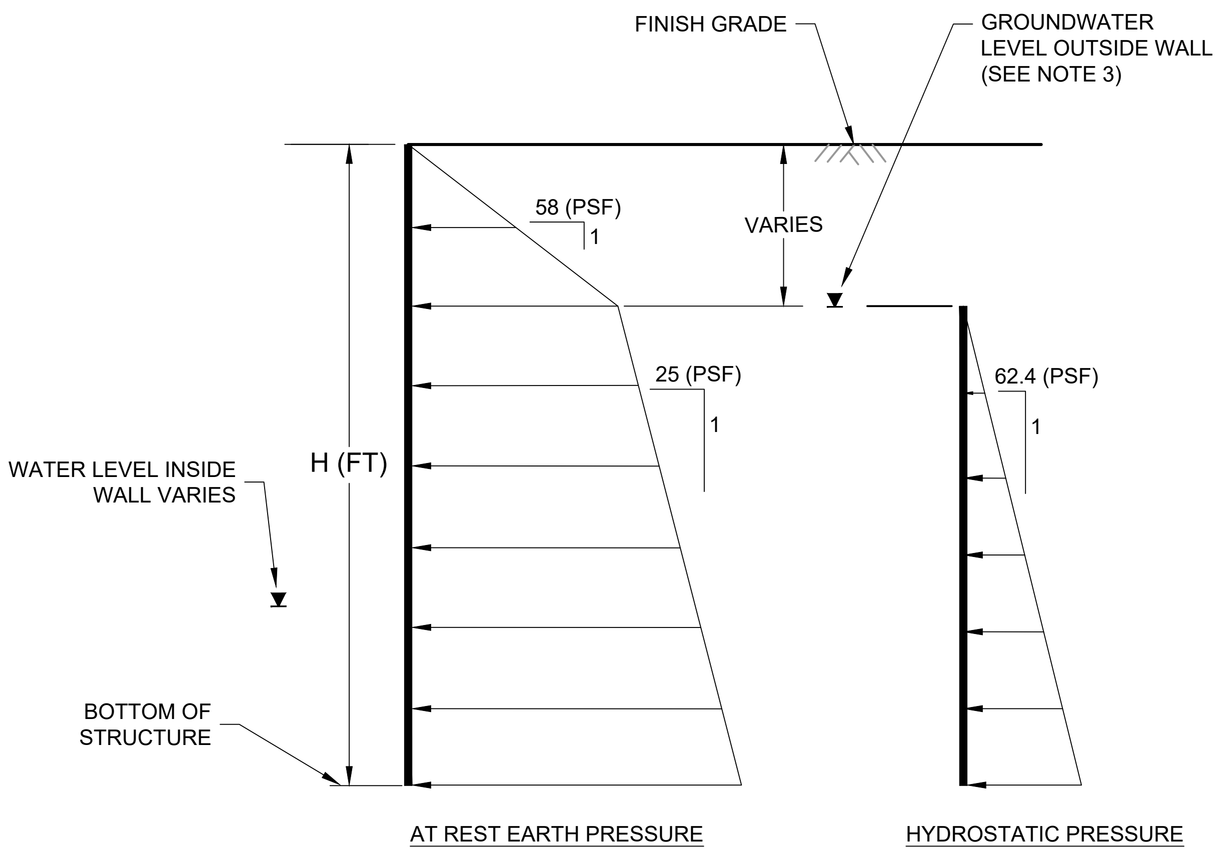
1. EARTH PRESSURES ARE IN POUNDS PER SQUARE FOOT (PSF), LINEAR DIMENSIONS IN FEET (FT).
2. AN EXCAVATION DEPTH, H, OF UP TO ABOUT 25 FT BGS, AND LEVEL GROUND CONDITIONS, ARE ASSUMED.
3. GROUNDWATER LEVELS ARE EXPECTED TO FLUCTUATE DUE TO WEATHER, CREEK WATER LEVEL AND OTHER FACTORS. GROUNDWATER LEVEL SHOULD BE FURTHER EVALUATED WITH ADDITIONAL MONITORING WELL(S) GROUNDWATER LEVEL MEASUREMENTS AND REVIEW OF CREEK HYDROGRAPH PRIOR TO CONSTRUCTION.
4. GROUNDWATER LEVEL WITHIN THE EXCAVATION IS ASSUMED AT THE BOTTOM OF THE EXCAVATION, BY DEWATERING.
5. EARTH PRESSURES ASSUME EXCAVATION AND SHORING METHODS WILL RESULT IN THE ACTIVE EARTH PRESSURE CONDITIONS. ACTIVE EARTH PRESSURES ASSUME THE THE SHORING IS SUPPORTED WITH MULTIPLE LEVELS OF INTERNAL BRACING.
6. UNBALANCED HYDROSTATIC LATERAL PRESSURE ASSUMED TO ACT AT 62.4 PCF AGAINST SHORING.
7. PASSIVE RESISTANCE SHOULD BE NEGLECTED WITHIN 2 FT OF THE BASE OF THE EXCAVATION.
8. A FACTOR OF SAFETY OF 1.5 HAS BEEN ADDED TO THE PASSIVE EARTH PRESSURE IN DIAGRAM. VALUES ARE ALLOWABLE.
9. REFER TO TEXT FOR ADDITIONAL TEMPORARY SHORING CONSIDERATIONS AND DETAILS.
10. CONSTRUCTION EQUIPMENT SURCHARGE PRESSURE ASSUMES A TYPICAL UNIFORM VERTICAL LOAD OF 250 PSF. IF HIGHER POINT LOADS ARE ANTICIPATED DURNING CONSTRUCTION, I.E., CRANE LOADS, THE SHORING DESIGNER SHOULD REVISE THE EARTH PRESSURE DIAGRAM ACCORDINGLY.
11. EARTH PRESSURE DIAGRAM ASSUMES THAT EXCAVATION IS SHORED FROM THE GROUND SURFACE. IF THE EXCAVATION IS DESIGNED USING A COMBINATION OF SLOPING AND SHORING, THE SHORING DESIGNER SHOULD REVISE THE EARTH PRESSURE DIAGRAM ACCORDINGLY.

NOT TO SCALE

Earth Pressure Diagram for Temporary Shoring

Geotechnical Engineering Report
Cascade Orchard Irrigation Company Improvement Project
Leavenworth, Washington

Aspect CONSULTING	Sep-2021	BY: NCS/SCC	FIGURE NO. 4
	PROJECT NO. 170700	REV BY: CMV/SCC	



NOT TO SCALE

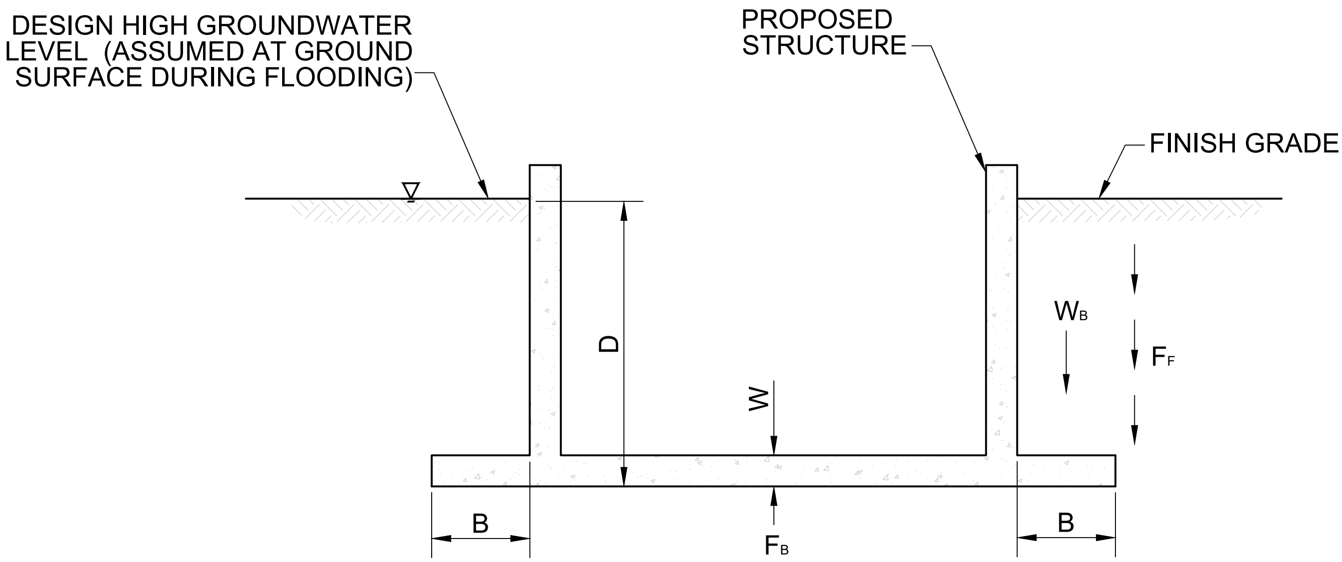
NOTES

1. EARTH PRESSURES ARE IN POUNDS PER SQUARE FOOT (PSF), LINEAR DIMENSIONS IN FEET (FT).
2. A PERMANENT WALL HEIGHT, H, OF UP TO ABOUT 25 FT BGS AND LEVEL GROUND CONDITIONS ARE ASSUMED.
3. RECOMMEND ASSUMING HIGH GROUNDWATER LEVEL CONDITION IS EQUAL TO GROUND SURFACE FOR PRELIMINARY DESIGN. GROUNDWATER LEVELS ARE EXPECTED TO FLUCTUATE WITH WEATHER, CREEK WATER LEVEL, AND OTHER FACTORS. GROUNDWATER LEVEL SHOULD BE FURTHER EVALUATED WITH ADDITIONAL MONITORING WELL(S) GROUNDWATER LEVEL MEASUREMENTS, AND REVIEW OF CREEK HYDROGRAPH.
4. GROUNDWATER LEVEL INSIDE THE PERMANENT WALL IS ASSUMED TO VARY WITH INFRASTRUCTURE OPERATION.
5. EARTH PRESSURES ASSUME "AT-REST" CONDITIONS.
6. UNBALANCED HYDROSTATIC LATERAL PRESSURE ASSUMED TO ACT AT 62.4 PCF AGAINST PERMANENT WALLS.
7. REFER TO TEXT FOR ADDITIONAL PERMANENT WALL CONSIDERATIONS AND DETAILS.

Earth Pressure Diagram
for Permanent Walls

Geotechnical Engineering Report
Cascade Orchard Irrigation Company Improvement Project
Leavenworth, Washington

Aspect CONSULTING	Sep-2021	BY: NCS/SCC	FIGURE NO.
	PROJECT NO. 170700	REV BY: CMV/SCC	5



SYMBOL

B = WIDTH OF EXTENDED BASE IN FEET (IF REQUIRED)

W = STRUCTURAL WEIGHT IN POUNDS

W_B = SOIL WEIGHT ABOVE EXTENDED BASE IN POUNDS
= VOLUME OF SOIL ABOVE EXTENDED BASE X SOIL UNIT WEIGHT

F_B = BUOYANT FORCE IN POUNDS
= UNIT WEIGHT OF WATER X VOLUME OF STRUCTURE BELOW DESIGN GROUNDWATER LEVEL

F_F = SOIL FRICTION IN POUNDS PER FOOT
= $6D^2$

L = PERIMETER AROUND BASE IN FEET

ASSUMPTIONS

WATER UNIT WEIGHT = 63 PCF

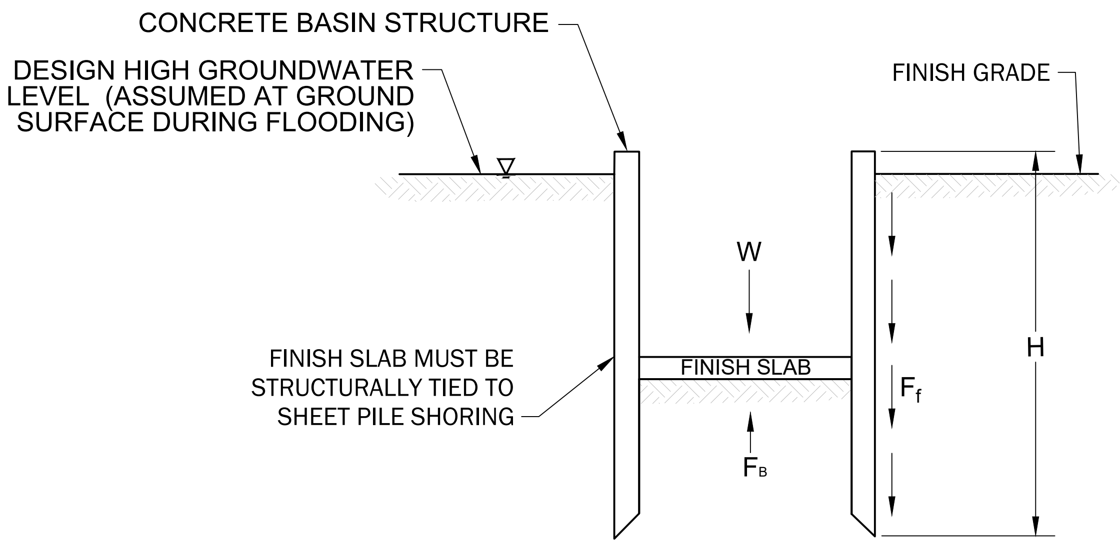
SOIL UNIT WEIGHT = 125 PCF ABOVE DESIGN HIGH GROUNDWATER LEVEL
62 PCF BELOW DESIGN HIGH GROUNDWATER LEVEL

$$\text{FACTOR OF SAFETY AGAINST UPLIFT} = \frac{W + W_B + F_F L}{F_B}$$

Expanded Foundation Base Uplift Resistance

Geotechnical Engineering Report
Cascade Orchard Irrigation Company Improvement Project
Leavenworth, Washington

Aspect CONSULTING	Sep-2021	BY: RPK/CMV	FIGURE NO.
	PROJECT NO. 170700	REV BY: -	6



SYMBOL

W = STRUCTURAL WEIGHT IN KIPS

F_B = BUOYANT FORCE IN POUNDS
= UNIT WEIGHT OF WATER x VOLUME OF STRUCTURE BELOW DESIGN GROUNDWATER LEVEL

L = PERIMETER AROUND BASE OF BASIN WALLS IN FEET

F_f = SOIL FRICTION IN POUNDS PER FOOT
= $2.5H^2$

ASSUMPTIONS

SOIL UNIT WEIGHT = 125 PCF ABOVE DESIGN HIGH GROUNDWATER LEVEL
62 PCF BELOW DESIGN HIGH GROUNDWATER LEVEL

NOTES

$$\text{FACTOR OF SAFETY} = \frac{W + F_f L}{F_B}$$

Permanent Shoring Wall Uplift Resistance

Geotechnical Engineering Report
Cascade Orchard Irrigation Company Improvement Project
Leavenworth, Washington

Aspect CONSULTING	Sep-2021	BY: RPK/CMV	FIGURE NO.
	PROJECT NO. 170700	REV BY: -	7

APPENDIX A

Subsurface Exploration Logs

Major Divisions		Fines	Group Symbol	Group Name
Coarse-Grained Soils - More than 50%[1] Retained on No. 200 Sieve	Gravels - More than 50%[1] of Coarse Fraction Retained on No. 4 Sieve	≤5% Fines	GW	Well-graded GRAVEL Well-graded GRAVEL WITH SAND
			GP	Poorly-graded GRAVEL Poorly-graded GRAVEL WITH SAND
		≥15% Fines	GM	SILTY GRAVEL SILTY GRAVEL WITH SAND
			GC	CLAYEY GRAVEL CLAYEY GRAVEL WITH SAND
	Sands - 50%[1] or More of Coarse Fraction Passes No. 4 Sieve	≤5% Fines	SW	Well-graded SAND Well-graded SAND WITH GRAVEL
			SP	Poorly-graded SAND Poorly-graded SAND WITH GRAVEL
		≥15% Fines	SM	SILTY SAND SILTY SAND WITH GRAVEL
			SC	CLAYEY SAND CLAYEY SAND WITH GRAVEL
Fine-Grained Soils - 50%[1] or More Passes No. 200 Sieve	Silts and Clays Liquid Limit Less than 50%		ML	SILT SANDY or GRAVELLY SILT SILT WITH SAND SILT WITH GRAVEL
			CL	LEAN CLAY SANDY or GRAVELLY LEAN CLAY LEAN CLAY WITH SAND LEAN CLAY WITH GRAVEL
			OL	ORGANIC SILT SANDY or GRAVELLY ORGANIC SILT ORGANIC SILT WITH SAND ORGANIC SILT WITH GRAVEL
	Silts and Clays Liquid Limit 50% or More		MH	ELASTIC SILT SANDY or GRAVELLY ELASTIC SILT ELASTIC SILT WITH SAND ELASTIC SILT WITH GRAVEL
			CH	FAT CLAY SANDY or GRAVELLY FAT CLAY FAT CLAY WITH SAND FAT CLAY WITH GRAVEL
			OH	ORGANIC CLAY SANDY or GRAVELLY ORGANIC CLAY ORGANIC CLAY WITH SAND ORGANIC CLAY WITH GRAVEL
Highly Organic Soils			PT	PEAT and other mostly organic soils

"WITH SILT" or "WITH CLAY" means 5 to 15% silt and clay, denoted by a "-" in the group name; e.g., SP-SM • "SILTY" or "CLAYEY" means >15% silt and clay • "WITH SAND" or "WITH GRAVEL" means 15 to 30% sand and gravel. • "SANDY" or "GRAVELLY" means >30% sand and gravel. • "Well-graded" means approximately equal amounts of fine to coarse grain sizes • "Poorly graded" means unequal amounts of grain sizes • Group names separated by "/" means soil contains layers of the two soil types; e.g., SM/ML.

Soils were described and identified in the field in general accordance with the methods described in ASTM D2488. Where indicated in the log, soils were classified using ASTM D2487 or other laboratory tests as appropriate. Refer to the report accompanying these exploration logs for details.

1. Estimated or measured percentage by dry weight
2. (SPT) Standard Penetration Test (ASTM D1586)
3. Determined by SPT, DCPT (ASTM STP399) or other field methods. See report text for details.

GEOTECHNICAL LAB TESTS

MC	=	Natural Moisture Content
PS	=	Particle Size Distribution
FC	=	Fines Content (% < 0.075 mm)
GH	=	Hydrometer Test
AL	=	Atterberg Limits
C	=	Consolidation Test
Str	=	Strength Test
OC	=	Organic Content (% Loss by Ignition)
Comp	=	Proctor Test
K	=	Hydraulic Conductivity Test
SG	=	Specific Gravity Test

CHEMICAL LAB TESTS

Organic Chemicals

BTEX	=	Benzene, Toluene, Ethylbenzene, Xylenes
TPH-Dx	=	Diesel and Oil-Range Petroleum Hydrocarbons
TPH-G	=	Gasoline-Range Petroleum Hydrocarbons
VOCs	=	Volatile Organic Compounds
SVOCs	=	Semi-Volatile Organic Compounds
PAHs	=	Polycyclic Aromatic Hydrocarbon Compounds
PCBs	=	Polychlorinated Biphenyls

Metals

RCRA8	=	As, Ba, Cd, Cr, Pb, Hg, Se, Ag, (d = dissolved, t = total)
MTCA5	=	As, Cd, Cr, Hg, Pb (d = dissolved, t = total)
PP-13	=	Ag, As, Be, Cd, Cr, Cu, Hg, Ni, Pb, Sb, Se, Tl, Zn (d=dissolved, t=total)

FIELD TESTS

PID	=	Photoionization Detector
Sheen	=	Oil Sheen Test
SPT[2]	=	Standard Penetration Test
NSPT	=	Non-Standard Penetration Test
DCPT	=	Dynamic Cone Penetration Test

COMPONENT DEFINITIONS

Descriptive Term		***Size Range and Sieve Number***
Boulders	=	Larger than 12 inches
Cobbles	=	3 inches to 12 inches
Coarse Gravel	=	3 inches to 3/4 inches
Fine Gravel	=	3/4 inches to No. 4 (4.75 mm)
Coarse Sand	=	No. 4 (4.75 mm) to No. 10 (2.00 mm)
Medium Sand	=	No. 10 (2.00 mm) to No. 40 (0.425 mm)
Fine Sand	=	No. 40 (0.425 mm) to No. 200 (0.075 mm)
Silt and Clay	=	Smaller than No. 200 (0.075 mm)

ESTIMATED[1] PERCENTAGE

% by Weight		***Modifier***	***% by Weight***		***Modifier***
<1	=	Subtrace	15 to 25	=	Little
1 to <5	=	Trace	30 to 45	=	Some
5 to 10	=	Few	>50	=	Mostly

MOISTURE CONTENT

Dry	=	Absence of moisture, dusty, dry to the touch
Slightly Moist	=	Perceptible moisture
Moist	=	Damp but no visible water
Very Moist	=	Water visible but not free draining
Wet	=	Visible free water, usually from below water table

RELATIVE DENSITY

Non-Cohesive or Coarse-Grained Soils

Density[3]		***SPT[2] Blows/Foot***	***Penetration with 1/2" Diameter Rod***
Very Loose	=	0 to 4	≥ 2'
Loose	=	5 to 10	1' to 2'
Medium Dense	=	11 to 30	3" to 1'
Dense	=	31 to 50	1" to 3"
Very Dense	=	> 50	< 1"

CONSISTENCY

Cohesive or Fine-Grained Soils

Consistency[3]		***SPT[2] Blows/Foot***	***Manual Test***
Very Soft	=	0 to 1	Penetrated >1" easily by thumb. Extrudes between thumb & fingers.
Soft	=	2 to 4	Penetrated 1/4" to 1" easily by thumb. Easily molded.
Medium Stiff	=	5 to 8	Penetrated >1/4" with effort by thumb. Molded with strong pressure.
Stiff	=	9 to 15	Indented ~1/4" with effort by thumb.
Very Stiff	=	16 to 30	Indented easily by thumbnail.
Hard	=	> 30	Indented with difficulty by thumbnail.

GEOLOGIC CONTACTS

Observed and Distinct | Observed and Gradual | Inferred

Aspect CONSULTING

Exploration Log Key

Aspect CONSULTING	Cascade Orchards Icicle Creek - 170700			Geotechnical Exploration Log	
	Project Address & Site Specific Location			*Coordinates (Lat,Lon WGS84)*	*Exploration Number*
	12463 Shore Street, Leavenworth, WA 98826, See Figure 2			47.5702, -120.6671 (est)	**AB-01**
Contractor	*Equipment*	*Sampling Method*		*Ground Surface Elev. (NAVD88)*	
WSSC	ATV CME 550	Autohammer; 140 lb hammer; 30" drop		1123' (est)	
Operator	*Exploration Method(s)*	*Work Start/Completion Dates*		*Top of Casing Elev. (NAVD88)*	*Depth to Water (Below GS)*
Alex	4.875" Tricone Mud Rotary	11/18/2020		NA	7.5' (ATD)

Depth (feet)	Elev. (feet)	Exploration Notes and Completion Details	Sample Type/ID	Blows/foot ▲ Water Content (%) ●	Blows/6"	Tests	Description	Depth (ft)
0	1123	Exploration capped with cuttings.					**TOPSOIL** SILTY SAND (SM); very loose, moist, dark brown; fine to medium sand; few organics, grass, and rootlets.	
							ALLUVIUM SANDY SILT (ML); very loose, moist, dark brown; nonplastic; fine to medium sand; trace fine to coarse, subrounded to rounded gravel; trace woody organics.	
	1120		S1		2, 1, 2			
5			S2		2, 2, 2			5
	1115	▽ 11/18/2020 Water observed in sampler.	S3		2, 2, 1	GH, MC FC=61.4%	Becomes wet.	
10		Drill chatter from 10 to 23-feet bgs.	S4		17, 18, 20		GRAVEL WITH SAND AND COBBLES (GW); medium dense to dense, wet, gray brown; medium to coarse sand; fine to coarse, subrounded to rounded gravel; rounded cobbles.	10
	1110		S5			Blows (non-SPT) =27,32,27		
15		Exploration backfilled with 3/8-inch bentonite chips.	S6			Blows (non-SPT) =30,25,30 GH, MC FC=3.3%		15
	1105							
20			S7			Blows (non-SPT) =14,15,20		20
	1100	Driller notes smooth drilling conditions.					SANDY SILT (ML); medium dense, wet, dark brown; nonplastic; fine to medium sand; horizontial stratifications of 0.5 to 4-inch thick layers of silty sand and sandy silt.	

Legend

Sample Type: ○ No Soil Sample Recovery; ◪ Split Barrel 2" X 1.375" (SPT); ▮ Split Barrel 3" X 2.375" (Mod Cal)

Plastic Limit ├──┤ Liquid Limit

Water Level: ▽ Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: JBM
Approved by: RPK

Exploration Log AB-01

Sheet 1 of 2

Cascade Orchards Icicle Creek - 170700

Geotechnical Exploration Log

Project Address & Site Specific Location	Coordinates (Lat,Lon WGS84)	Exploration Number
12463 Shore Street, Leavenworth, WA 98826, See Figure 2	47.5702, -120.6671 (est)	AB-01

Contractor	Equipment	Sampling Method	Ground Surface Elev. (NAVD88)
WSSC	ATV CME 550	Autohammer; 140 lb hammer; 30" drop	1123' (est)

Operator	Exploration Method(s)	Work Start/Completion Dates	Top of Casing Elev. (NAVD88)	Depth to Water (Below GS)
Alex	4.875" Tricone Mud Rotary	11/18/2020	NA	7.5' (ATD)

Depth (feet)	Elev. (feet)	Exploration Notes and Completion Details	Sample Type/ID	Blows/foot ▲ Water Content (%)●	Blows/6"	Tests	Material Type	Description
			S8		15, 11, 11			SANDY SILT (ML); medium dense, wet, dark brown; nonplastic; fine to medium sand; horizontial stratifications of 0.5 to 4-inch thick layers of silty sand and sandy silt. (continued)
	1095							
30		Exploration backfilled with 3/8-inch bentonite chips.	S9		7, 8, 7			
			S10			Blows (non-SPT) =7,5,10		
	1090							
								SILTY SAND (SM); medium dense, wet, dark grey; fine to medium sand; horizontial stratifications of 1 to 4-inch thick layers of silty sand and sand.
35			S11			Blows (non-SPT) =11,13,12		
								Bottom of exploration at 36.5 ft. bgs.
	1085							
40								
	1080							
45								
	1075							

Legend

Sample Type:
- No Soil Sample Recovery
- Split Barrel 2" X 1.375" (SPT)
- Split Barrel 3" X 2.375" (Mod Cal)

Plastic Limit ⊢—⊣ Liquid Limit

Water Level:
- ▽ Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: JBM
Approved by: RPK

Exploration Log AB-01

Aspect CONSULTING	Cascade Orchards Icicle Creek - 170700			Geotechnical Exploration Log	
	Project Address & Site Specific Location			*Coordinates (Lat,Lon WGS84)*	*Exploration Number*
	12463 Shore Street, Leavenworth, WA 98826, See Figure 2			47.5701, -120.6669 (est)	**AMW-01**
Contractor	*Equipment*	*Sampling Method*		*Ground Surface Elev. (NAVD88)*	
WSSC	ATV CME 550	Autohammer; 140 lb hammer; 30" drop		1119' (est)	Ecology Well Tag No. BJC 773
Operator	*Exploration Method(s)*	*Work Start/Completion Dates*		*Top of Casing Elev. (NAVD88)*	*Depth to Water (Below GS)*
Alex	4.875" Tricone Mud Rotary	11/18/2020		1121' (est)	10' (Static)

Depth (feet)	Elev. (feet)	Exploration Notes and Completion Details	Sample Type/ID	Blows/foot ▲ Water Content (%) ● 0 10 20 30 40 50	Blows/6"	Tests	Material Type	Description	Depth (ft)
0		8-inch flush-mount steel monument set in concrete.						**TOPSOIL** SILTY SAND (SM); very loose, moist, dark brown; fine to medium sand; few organics, grass, and rootlets.	
								ALLUVIUM SAND WITH SILT AND GRAVEL (SP-SM); loose, moist, brown; fine to medium sand; fine to coarse, subrounded to rounded gravel; trace woody organics.	
			S1		5, 3, 4				
	1115								
5		2-inch-diameter PVC pipe set in 3/8-inch bentonite chips.	S2		3, 3, 3				5
		▽ 11/18/2020 Water observed in sampler.	S3		4, 4, 9			Becomes medium dense, wet, and dark brown.	
	1110	Driller chatter from 9 to 27-feet bgs.						GRAVEL WITH SAND AND COBBLES (GW); medium dense to dense, wet, gray brown; medium to coarse sand; fine to coarse, subrounded to rounded gravel; rounded cobbles.	
10		▼ 12/8/2020	S4		8, 5, 13				10
			S5			Blows (non-SPT) =25,12,13			
	1105								
15			S6			Blows (non-SPT) =20,44,23			15
		▼ 11/18/2020 Water level rising after installation.							
	1100								
20			S7			Blows (non-SPT) =27,22,38 GH, MC FC=5.1%			20
	1095								

Legend

Plastic Limit ├──┤ Liquid Limit

Sample Type:
- ◯ No Soil Sample Recovery
- ◩ Split Barrel 2" X 1.375" (SPT)
- ▮ Split Barrel 3" X 2.375" (Mod Cal)

Water Level:
- ▼ Static Water Level
- ▽ Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: JBM
Approved by: RPK

Exploration Log AMW-01

Sheet 1 of 2

Cascade Orchards Icicle Creek - 170700

Geotechnical Exploration Log

Project Address & Site Specific Location	Coordinates (Lat,Lon WGS84)	Exploration Number
12463 Shore Street, Leavenworth, WA 98826, See Figure 2	47.5701, -120.6669 (est)	**AMW-01**

Contractor	Equipment	Sampling Method	Ground Surface Elev. (NAVD88)	Ecology Well Tag No.
WSSC	ATV CME 550	Autohammer; 140 lb hammer; 30" drop	1119' (est)	BJC 773

Operator	Exploration Method(s)	Work Start/Completion Dates	Top of Casing Elev. (NAVD88)	Depth to Water (Below GS)
Alex	4.875" Tricone Mud Rotary	11/18/2020	1121' (est)	10' (Static)

Depth (feet)	Elev. (feet)	Exploration Notes and Completion Details	Sample Type/ID	Blows/6"	Tests	Description
25			S8/S9		Blows (non-SPT) =21,10,12	SANDY SILT (ML); medium dense, wet, dark brown; nonplastic; fine sand.
		2-inch-diameter PVC pipe set in sand.				
28						SILTY SAND (SM); medium dense, wet, brown to dark grey; fine sand; trace fine, subrounded gravel
	1090					
30			S10	5 7 9	GH, MC FC=23.1%	
		2-inch-diameter slotted PVC pipe set in sand.				
	1085					
35			S11	6 8 6		Becomes without gravel and stratified with 0.5 to 4-inch thick layers of silty sand and sand.
	1080					
40			S12	8 8 8		Becomes dark gray.
41.5						Bottom of exploration at 41.5 ft. bgs.
	1075					
45						
	1070					

Blows/foot ▲ Water Content (%) ● 0 10 20 30 40 50

Legend

Sample Type:
- No Soil Sample Recovery
- Split Barrel 2" X 1.375" (SPT)
- Split Barrel 3" X 2.375" (Mod Cal)

Plastic Limit ⊢—⊣ Liquid Limit

Water Level:
- Static Water Level
- Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: JBM
Approved by: RPK

Exploration Log AMW-01

Aspect CONSULTING

Cascade Orchards Icicle Creek - 170700

Geotechnical Exploration Log

Project Address & Site Specific Location			Coordinates (Lat,Lon WGS84)	Exploration Number
12463 Shore Street, Leavenworth, WA 98826, See Figure 2			47.5701, -120.6669 (est)	ATP-01
Contractor	**Equipment**	**Sampling Method**	**Ground Surface Elev. (NAVD88)**	
LE&E	Deere 160LC	Grab	1119' (est)	
Operator	**Exploration Method(s)**	**Work Start/Completion Dates**	**Top of Casing Elev. (NAVD88)**	**Depth to Water (Below GS)**
Rick Youngers	Excavator	12/8/2020	NA	10' (ATD)

Depth (feet)	Elev. (feet)	Exploration Notes and Completion Details	Sample Type/ID	Blows/foot ▲ Water Content (%)● 0 10 20 30 40 50	Blows/6"	Tests	Material Type	Description	Depth (ft)
								TOPSOIL SILTY SAND (SM); dense (frozen), moist, brown; fine sand; few grass roots.	
1	1118							**ALLUVIUM** SILTY SAND (SM); medium dense, moist, brown; fine to coarse sand; trace gravel	1
2	1117								2
3	1116							Between 3 and 6 ft bgs, a few 6-inch-thick beds of sand trace silt	3
4	1115	Test pit backfilled with spoils and tamped with excavator bucket							4
5	1114								5
6	1113								6
7	1112								7
8	1111								8
9	1110							GRAVEL WITH SAND AND COBBLES (GW); loose, wet, brown; coarse sand; fine to coarse gravel; cobbles to 10 inch diameter	9
10	1109	∇ 12/8/2021							10
11	1108								11
12	1107	11 to 14 ft bgs: side walls caving							12
13	1106							Terminated due to combination of excavator reach and caving sidewalls	13
14	1105							Bottom of exploration at 14 ft. bgs.	14
15	1104								15
16	1103								16
17	1102								17
18	1101								18
19	1100								19
20	1099								20
21	1098								21
22	1097								22
23	1096								23
24	1095								24

Legend

Sample Type

Plastic Limit ├──┤ Liquid Limit

Water Level: ∇ Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: MvA
Approved by: RPK

Exploration Log ATP-01

Sheet 1 of 1

Aspect CONSULTING	Cascade Orchards Icicle Creek - 170700			Geotechnical Exploration Log	
	Project Address & Site Specific Location			*Coordinates (Lat,Lon WGS84)*	*Exploration Number*
	12463 Shore Street, Leavenworth, WA 98826, See Figure 2			47.5702, -120.6671 (est)	**ATP-02**
Contractor	*Equipment*	*Sampling Method*		*Ground Surface Elev. (NAVD88)*	
LE&E	Deere 160LC	Grab		1119' (est)	
Operator	*Exploration Method(s)*	*Work Start/Completion Dates*		*Top of Casing Elev. (NAVD88)*	*Depth to Water (Below GS)*
Rick Youngers	Excavator	12/8/2020		NA	10.5' (ATD)

Depth (feet)	Elev. (feet)	Exploration Notes and Completion Details	Sample Type/ID	Blows/foot ▲ Water Content (%)● 0 10 20 30 40 50	Blows/6"	Tests	Material Type	Description	Depth (ft)
								TOPSOIL SILTY SAND (SM); dense (frozen), moist, brown; fine sand; few grass roots.	
1	1118							**ALLUVIUM**	1
2	1117							SILTY SAND (SM); medium dense, moist, brown; fine to coarse sand; trace gravel	2
3	1116								3
4	1115	Test pit backfilled with spoils and tamped with excavator bucket						Between 3 and 6 ft bgs, a fews 6-inch-thick beds of sand trace silt	4
5	1114								5
6	1113								6
7	1112								7
8	1111								8
9	1110								9
10	1109	∇ 12/8/2020						GRAVEL WITH SAND AND COBBLES (GW); loose, wet, brown; coarse sand; fine to coarse gravel; cobbles to 10 inch diameter	10
11	1108								11
12	1107	11 to 15 ft bgs: side walls caving							12
13	1106								13
14	1105							Terminated due to combination of excavator reach and caving sidewalls	14
15	1104							Bottom of exploration at 15 ft. bgs.	15
16	1103								16
17	1102								17
18	1101								18
19	1100								19
20	1099								20
21	1098								21
22	1097								22
23	1096								23
24	1095								24

Legend

Sample Type

Plastic Limit ├──┤ Liquid Limit

Water Level: ∇ Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: MvA
Approved by: RPK

Exploration Log ATP-02

Sheet 1 of 1

Aspect CONSULTING	Cascade Orchards Icicle Creek - 170700			Geotechnical Exploration Log	
	Project Address & Site Specific Location			Coordinates (Lat,Lon WGS84)	Exploration Number
	12463 Shore Street, Leavenworth, WA 98826, See Figure 2			47.5701, -120.6670 (est)	ATP-03
Contractor	Equipment	Sampling Method		Ground Surface Elev. (NAVD88)	
LE&E	Deere 160LC	Grab		1119' (est)	
Operator	Exploration Method(s)	Work Start/Completion Dates		Top of Casing Elev. (NAVD88)	Depth to Water (Below GS)
Rick Youngers	Excavator	12/8/2020		NA	11' (ATD)

Depth (feet)	Elev. (feet)	Exploration Notes and Completion Details	Sample Type/ID	Blows/foot ▲ Water Content (%)● 0 10 20 30 40 50	Blows/6"	Tests	Material Type	Description	Depth (ft)
								TOPSOIL SILTY SAND (SM); dense (frozen), moist, brown; fine sand; few grass roots.	
1	1118								1
								ALLUVIUM SILTY SAND (SM); medium dense, moist, brown; fine to coarse sand; trace gravel	
2	1117								2
3	1116								3
4	1115	Test pit backfilled with spoils and tamped with excavator bucket						Between 3 and 6 ft bgs, a few 6-inch-thick beds of sand trace silt	4
5	1114								5
6	1113							Sidewalls stay upright, no caving	6
7	1112								7
8	1111								8
9	1110								9
10	1109							GRAVEL WITH SAND AND COBBLES (GW); loose, wet, brown; coarse sand; fine to coarse gravel; cobbles to 7 inch diameter	10
11	1108	∇ 12/8/2020							11
12	1107	11 to 15 ft bgs: side walls caving							12
13	1106								13
14	1105							Terminated due to combination of excavator reach and caving sidewalls	14
15	1104							Bottom of exploration at 15 ft. bgs.	15
16	1103								16
17	1102								17
18	1101								18
19	1100								19
20	1099								20
21	1098								21
22	1097								22
23	1096								23
24	1095								24

Legend

Sample Type

Plastic Limit ⊢—⊣ Liquid Limit

Water Level: ∇ Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: MvA
Approved by: RPK

Exploration Log ATP-03

Sheet 1 of 1

NEW STANDARD EXPLORATION LOG TEMPLATE P:\GINTW\PROJECTS\170700 - CASCADE ORCHARDS IRRIGATION 2020.GPJ March 18, 2021

Review Stage:FINAL

APPENDIX B

Geotechnical Laboratory Testing Results

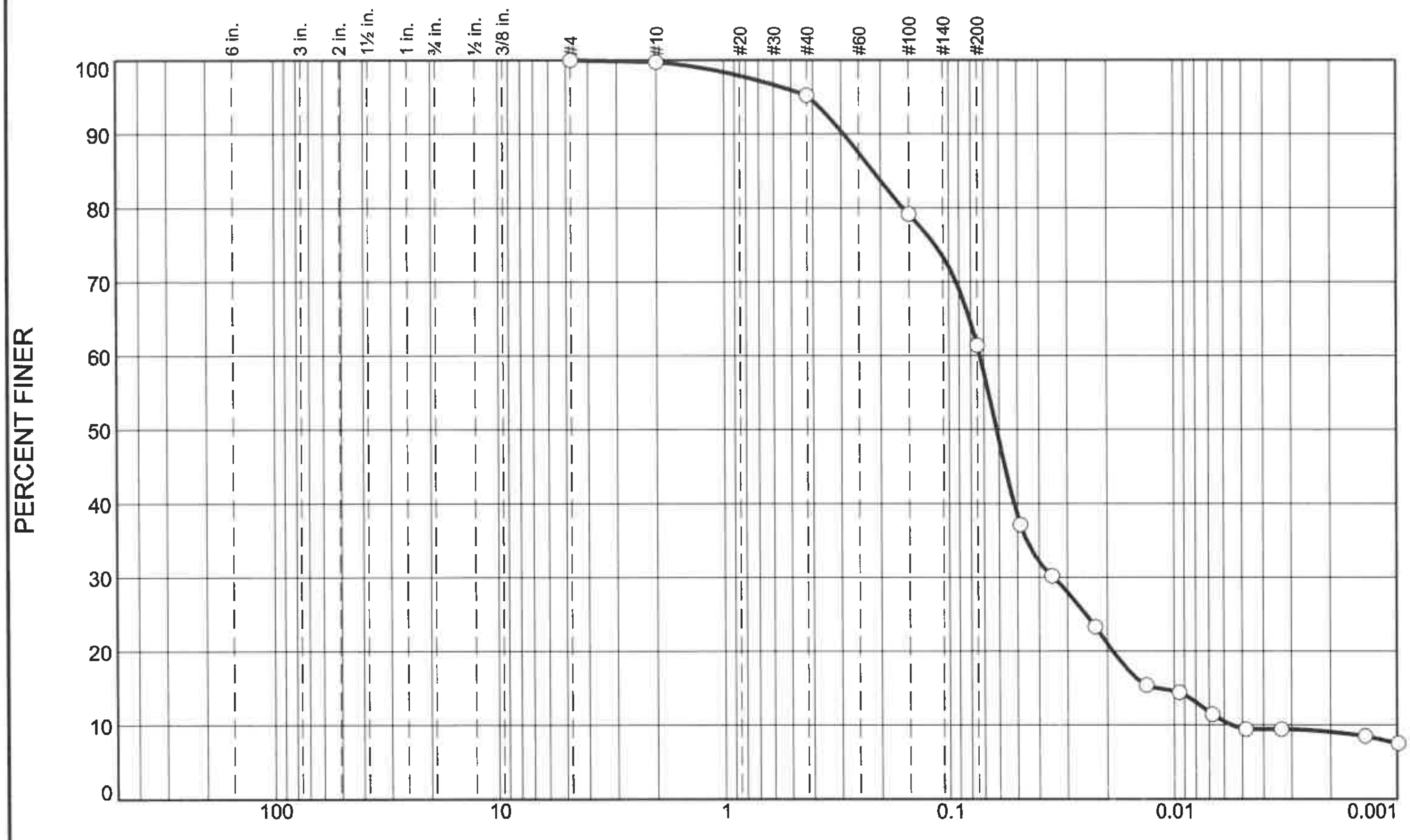
Geotechnical Laboratory Tests

Geotechnical laboratory tests were conducted on selected soil samples collected during drilling of the soil borings. Samples were sent to Materials Testing & Consulting, Inc. for determination of moisture content, grain size distribution, and Atterberg limits (plasticity).

- Moisture content was determined by ASTM D2216, Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass.
- Grain-size analyses was completed in accordance with ASTM D6913, Standard Test Methods for Particle-Size Distribution (Gradation) of Soil Using Sieve Analysis, as well as ASTM D7928, Standard Test Method for Particle-Size Distribution (Gradation) of Fine-Grained Soils Using the Sedimentation (Hydrometer) Analysis.

The soil samples tested are indicated graphically on the subsurface exploration logs in Appendix A. results of the tests are provided in the attached data sheets here in Appendix B.

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.2	4.6	33.8	51.8	9.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.8		
#40	95.2		
#100	79.1		
#200	61.4		

* (no specification provided)

Material Description

Sandy silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2918 D_{85}= 0.2159 D_{60}= 0.0730
D_{50}= 0.0617 D_{30}= 0.0345 D_{15}= 0.0122
D_{10}= 0.0055 C_u= 13.18 C_c= 2.94

Classification

USCS= ML AASHTO=

Remarks

MC=41.5%

Location: AB-01
Sample Number: S-3 **Depth:** 7.5 **Date:** 1/1/2021

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Cascade Orchards
Project No: 08-175/170700 **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

1/1/2021

Client: Aspect Consulting
Project: Cascade Orchards
Project Number: 08-175/170700
Location: AB-01
Depth: 7.5 **Sample Number:** S-3
Material Description: Sandy silt
Date: 1/1/2021
USCS Classification: ML
Testing Remarks: MC=41.5%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 127.00
Tare Wt. = 13.00
Minus #200 from wash = 54.0%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
260.90	13.00	0.00	#4	0.00	100.0
			#10	0.60	99.8
			#40	11.80	95.2
			#100	51.70	79.1
			#200	95.70	61.4

Hydrometer Test Data

Hydrometer test uses material passing #10
Percent passing #10 based upon complete sample = 99.8
Weight of hydrometer sample =50
Table of composite correction values:

Temp., deg. C:	17.8	18.2	19.4
Comp. corr.:	-4.5	-4.3	-4.0

Meniscus correction only = 0.0
Specific gravity of solids = 2.7
Hydrometer type = 152H
Hydrometer effective depth equation: L = 16.294964 - 0.164 x Rm

Elapsed Time (min.)	Temp. (deg. C.)	Actual Reading	Corrected Reading	K	Rm	Eff. Depth	Diameter (mm.)	Percent Finer
1.00	18.5	23.0	18.8	0.0137	23.0	12.5	0.0485	37.1
2.00	18.5	19.5	15.3	0.0137	19.5	13.1	0.0351	30.2
5.00	18.5	16.0	11.8	0.0137	16.0	13.7	0.0227	23.3
15.00	18.5	12.0	7.8	0.0137	12.0	14.3	0.0134	15.4
30.00	18.5	11.5	7.3	0.0137	11.5	14.4	0.0095	14.4
60.00	18.5	10.0	5.8	0.0137	10.0	14.7	0.0068	11.5
120.00	18.4	9.0	4.8	0.0137	9.0	14.8	0.0048	9.5
250.00	18.4	9.0	4.8	0.0137	9.0	14.8	0.0033	9.5
1440.00	18.5	8.5	4.3	0.0137	8.5	14.9	0.0014	8.5
2880.00	18.5	8.0	3.8	0.0137	8.0	15.0	0.0010	7.5

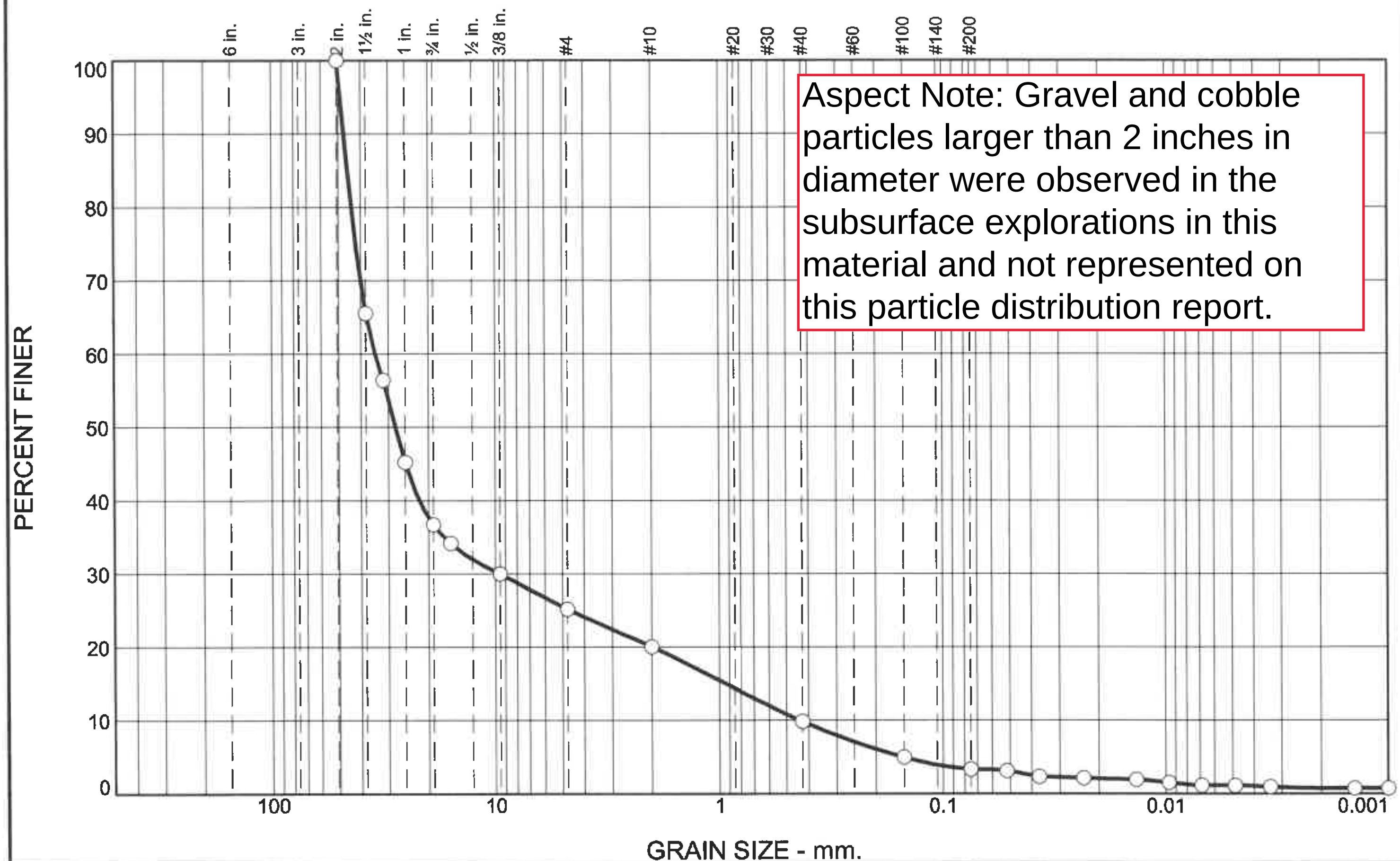
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	0.0	0.0	0.2	4.6	33.8	38.6	51.8	9.6	61.4

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
	0.0055	0.0122	0.0189	0.0345	0.0517	0.0617	0.0730	0.1588	0.2159	0.2918	0.4162

Fineness Modulus	C_u	C_c
0.35	13.18	2.94

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	63.3	11.5	5.1	10.3	6.5	2.2	1.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2"	100.0		
1.5"	65.5		
1 1/4"	56.4		
1"	45.2		
3/4"	36.7		
5/8"	34.1		
3/8"	30.0		
#4	25.2		
#10	20.1		
#40	9.8		
#100	5.0		
#200	3.3		

* (no specification provided)

Material Description

Gravel with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 47.3280 D_{85}= 45.6193 D_{60}= 34.5658
D_{50}= 27.9526 D_{30}= 9.5210 D_{15}= 0.9345
D_{10}= 0.4360 C_u= 79.28 C_c= 6.02

Classification

USCS= GP AASHTO=

Remarks

MC=5.1%

Location: AB-01
Sample Number: S-6 **Depth:** 15 **Date:** 1/1/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Cascade Orchards

Project No: 08-175/170700 **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

1/1/2021

Client: Aspect Consulting
Project: Cascade Orchards
Project Number: 08-175/170700
Location: AB-01
Depth: 15 **Sample Number:** S-6
Material Description: Gravel with sand
Date: 1/1/2020
USCS Classification: GP
Testing Remarks: MC=5.1%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 695.20
Tare Wt. = 12.70
Minus #200 from wash = 2.9%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
715.30	12.70	0.00	2"	0.00	100.0
			1.5"	242.30	65.5
			1 1/4"	306.30	56.4
			1"	384.90	45.2
			3/4"	444.50	36.7
			5/8"	462.70	34.1
			3/8"	491.80	30.0
			#4	525.70	25.2
			#10	561.70	20.1
			#40	633.40	9.8
			#100	667.60	5.0
			#200	679.10	3.3

Hydrometer Test Data

Hydrometer test uses material passing #10
Percent passing #10 based upon complete sample = 20.1
Weight of hydrometer sample =50
Table of composite correction values:

Temp., deg. C:	17.8	18.2	19.4
Comp. corr.:	-4.5	-4.3	-4.0

Meniscus correction only = 0.0
Specific gravity of solids = 2.7
Hydrometer type = 152H
Hydrometer effective depth equation: L = 16.294964 - 0.164 x Rm

Elapsed Time (min.)	Temp. (deg. C.)	Actual Reading	Corrected Reading	K	Rm	Eff. Depth	Diameter (mm.)	Percent Finer
1.00	18.7	12.0	7.9	0.0137	12.0	14.3	0.0517	3.1
2.00	18.7	10.0	5.9	0.0137	10.0	14.7	0.0370	2.3
5.00	18.7	9.5	5.4	0.0137	9.5	14.7	0.0235	2.1
15.00	18.7	9.0	4.9	0.0137	9.0	14.8	0.0136	1.9
30.00	18.7	8.0	3.9	0.0137	8.0	15.0	0.0097	1.5
60.00	18.6	7.0	2.8	0.0137	7.0	15.1	0.0069	1.1
120.00	18.5	7.0	2.8	0.0137	7.0	15.1	0.0049	1.1
250.00	18.5	6.5	2.3	0.0137	6.5	15.2	0.0034	0.9
1440.00	18.5	6.0	1.8	0.0137	6.0	15.3	0.0014	0.7
2880.00	18.5	6.0	1.8	0.0137	6.0	15.3	0.0010	0.7

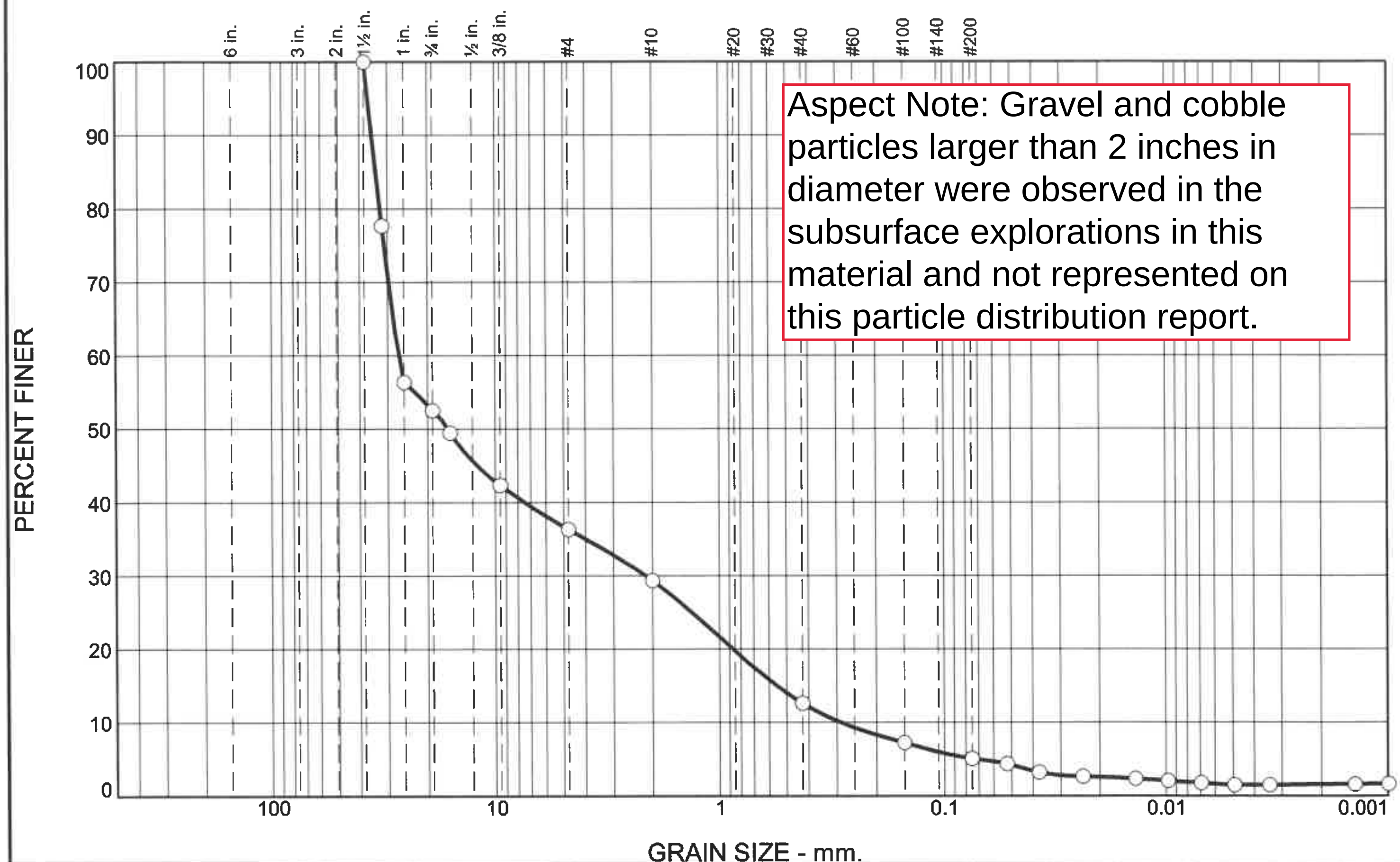
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	63.3	11.5	74.8	5.1	10.3	6.5	21.9	2.2	1.1	3.3

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.1508	0.4360	0.9345	1.9824	9.5210	22.0015	27.9526	34.5658	43.8882	45.6193	47.3280	49.0459

Fineness Modulus	C_u	C_c
6.80	79.28	6.02

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	47.5	16.2	7.0	16.7	7.5	3.6	1.5

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1.5"	100.0		
1 1/4"	77.7		
1"	56.4		
3/4"	52.5		
5/8"	49.5		
3/8"	42.4		
#4	36.3		
#10	29.3		
#40	12.6		
#100	7.2		
#200	5.1		

* (no specification provided)

Material Description

Gravel with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 35.1449 D_{85}= 33.7403 D_{60}= 26.8017
D_{50}= 16.2687 D_{30}= 2.1455 D_{15}= 0.5517
D_{10}= 0.2865 C_u= 93.53 C_c= 0.60

Classification

USCS= GP AASHTO=

Remarks

MC=6.5%

Location: AMW-01
Sample Number: S-7 **Depth:** 20 **Date:** 1/1/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Cascade Orchards
Project No: 08-175/170700 **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

1/1/2021

Client: Aspect Consulting
Project: Cascade Orchards
Project Number: 08-175/170700
Location: AMW-01
Depth: 20
Sample Number: S-7
Material Description: Gravel with sand
Date: 1/1/2020
USCS Classification: GP
Testing Remarks: MC=6.5%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 705.50
Tare Wt. = 12.70
Minus #200 from wash = 4.1%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
735.00	12.70	0.00	1.5"	0.00	100.0
			1 1/4"	161.30	77.7
			1"	315.10	56.4
			3/4"	342.80	52.5
			5/8"	364.80	49.5
			3/8"	416.40	42.4
			#4	460.00	36.3
			#10	510.40	29.3
			#40	631.10	12.6
			#100	670.10	7.2
			#200	685.70	5.1

Hydrometer Test Data

Hydrometer test uses material passing #10
Percent passing #10 based upon complete sample = 29.3
Weight of hydrometer sample =50
Table of composite correction values:

Temp., deg. C:	17.8	18.2	19.4
Comp. corr.:	-4.5	-4.3	-4.0

Meniscus correction only = 0.0
Specific gravity of solids = 2.7
Hydrometer type = 152H
Hydrometer effective depth equation: L = 16.294964 - 0.164 **x Rm**

Elapsed Time (min.)	Temp. (deg. C.)	Actual Reading	Corrected Reading	K	Rm	Eff. Depth	Diameter (mm.)	Percent Finer
1.00	17.8	12.0	7.5	0.0138	12.0	14.3	0.0523	4.4
2.00	17.8	10.0	5.5	0.0138	10.0	14.7	0.0374	3.2
5.00	17.8	9.0	4.5	0.0138	9.0	14.8	0.0238	2.6
15.00	17.8	8.5	4.0	0.0138	8.5	14.9	0.0138	2.3
30.00	17.7	8.0	3.5	0.0138	8.0	15.0	0.0098	2.0
60.00	17.7	7.5	3.0	0.0138	7.5	15.1	0.0069	1.7
120.00	17.7	7.0	2.5	0.0138	7.0	15.1	0.0049	1.5
250.00	17.6	7.0	2.5	0.0139	7.0	15.1	0.0034	1.5
1440.00	18.2	7.0	2.8	0.0138	7.0	15.1	0.0014	1.6
2880.00	18.5	7.0	2.8	0.0137	7.0	15.1	0.0010	1.6

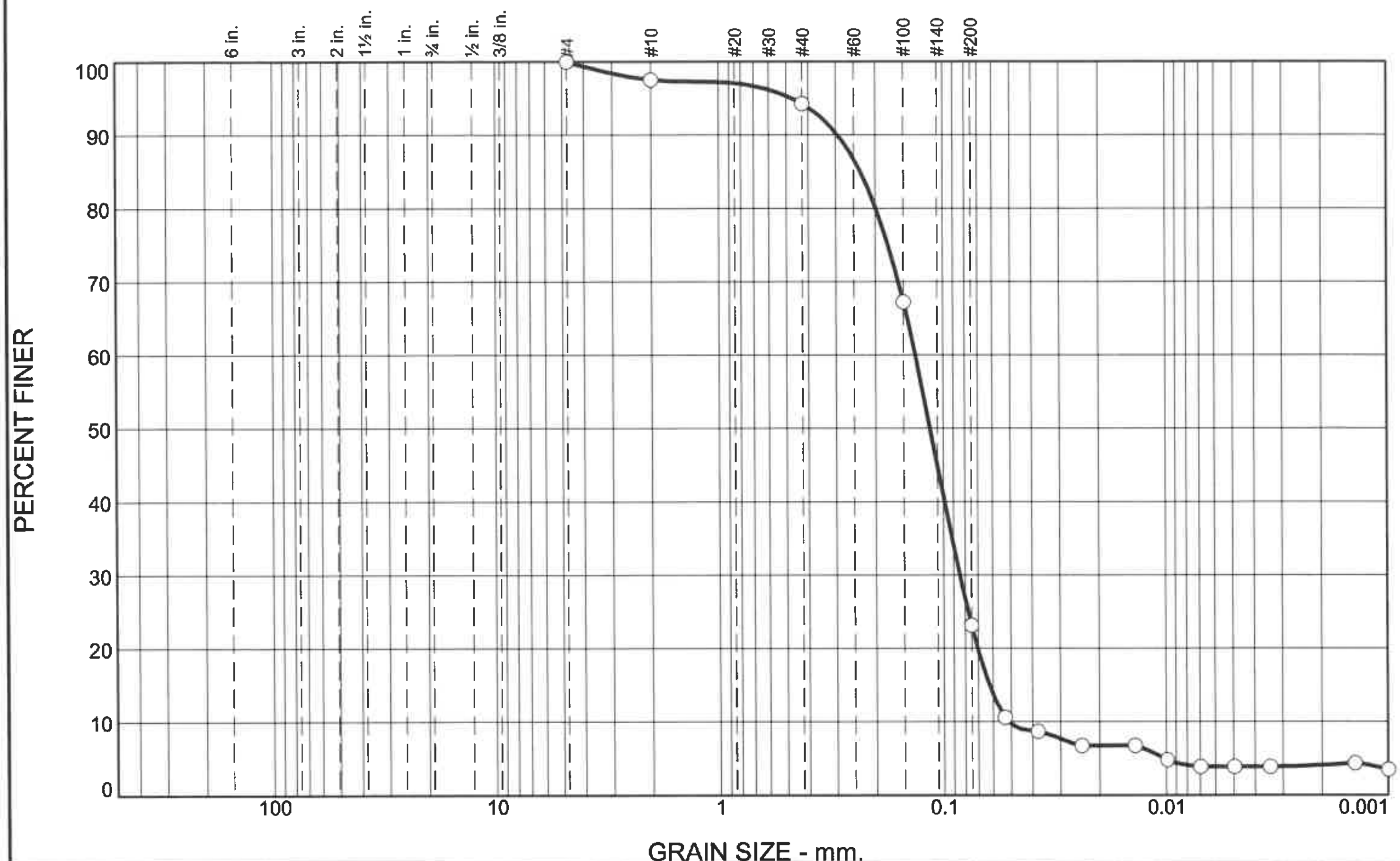
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	47.5	16.2	63.7	7.0	16.7	7.5	31.2	3.6	1.5	5.1

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.0724	0.2865	0.5517	0.8698	2.1455	7.4398	16.2687	26.8017	32.3751	33.7403	35.1449	36.5956

Fineness Modulus	C_u	C_c
5.81	93.53	0.60

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	2.5	3.2	71.2	19.2	3.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	97.5		
#40	94.3		
#100	67.2		
#200	23.1		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2945 D_{85}= 0.2337 D_{60}= 0.1329
D_{50}= 0.1142 D_{30}= 0.0845 D_{15}= 0.0625
D_{10}= 0.0511 C_u= 2.60 C_c= 1.05

Classification

USCS= SM AASHTO=

Remarks

MC=26.9%

Location: AMW-01
Sample Number: S-10 **Depth:** 30 **Date:** 1/1/2021

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Cascade Orchards

Project No: 08-175/170700 **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

1/1/2021

Client: Aspect Consulting
Project: Cascade Orchards
Project Number: 08-175/170700
Location: AMW-01
Depth: 30 **Sample Number:** S-10
Material Description: Silty sand
Date: 1/1/2021
USCS Classification: SM
Testing Remarks: MC=26.9%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 186.90
Tare Wt. = 12.60
Minus #200 from wash = 16.0%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
220.00	12.60	0.00	#4	0.00	100.0
			#10	5.10	97.5
			#40	11.90	94.3
			#100	68.00	67.2
			#200	159.40	23.1

Hydrometer Test Data

Hydrometer test uses material passing #10
Percent passing #10 based upon complete sample = 97.5
Weight of hydrometer sample =50
Table of composite correction values:

Temp., deg. C:	17.8	18.2	19.4
Comp. corr.:	-4.5	-4.3	-4.0

Meniscus correction only = 0.0
Specific gravity of solids = 2.7
Hydrometer type = 152H
Hydrometer effective depth equation: L = 16.294964 - 0.164 **x Rm**

Elapsed Time (min.)	Temp. (deg. C.)	Actual Reading	Corrected Reading	K	Rm	Eff. Depth	Diameter (mm.)	Percent Finer
1.00	17.5	10.0	5.5	0.0139	10.0	14.7	0.0531	10.6
2.00	17.5	9.0	4.5	0.0139	9.0	14.8	0.0378	8.7
5.00	17.5	8.0	3.5	0.0139	8.0	15.0	0.0240	6.8
15.00	17.5	8.0	3.5	0.0139	8.0	15.0	0.0139	6.8
30.00	17.5	7.0	2.5	0.0139	7.0	15.1	0.0099	4.8
60.00	17.5	6.5	2.0	0.0139	6.5	15.2	0.0070	3.9
120.00	17.4	6.5	2.0	0.0139	6.5	15.2	0.0049	3.9
250.00	17.4	6.5	2.0	0.0139	6.5	15.2	0.0034	3.9
1440.00	18.2	6.5	2.3	0.0138	6.5	15.2	0.0014	4.3
2880.00	18.5	6.0	1.8	0.0137	6.0	15.3	0.0010	3.5

Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	0.0	0.0	2.5	3.2	71.2	76.9	19.2	3.9	23.1

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.0102	0.0511	0.0625	0.0705	0.0845	0.0986	0.1142	0.1329	0.1991	0.2337	0.2945	0.4718

Fineness Modulus	C_u	C_c
0.51	2.60	1.05

APPENDIX C

Report Limitations and Guidelines for Use

REPORT LIMITATIONS AND GUIDELINES FOR USE

Geoscience is Not Exact

The geoscience practices (geotechnical engineering, geology, and environmental science) are far less exact than other engineering and natural science disciplines. It is important to recognize this limitation in evaluating the content of the report. If you are unclear how these "Report Limitations and Guidelines for Use" apply to your project or property, you should contact Aspect Consulting, LLC (Aspect).

This Report and Project-Specific Factors

Aspect's services are designed to meet the specific needs of our clients. Aspect has performed the services in general accordance with our agreement (the Agreement) with the Client (defined under the Limitations section of this project's work product). This report has been prepared for the exclusive use of the Client. This report should not be applied for any purpose or project except the purpose described in the Agreement.

Aspect considered many unique, project-specific factors when establishing the Scope of Work for this project and report. You should not rely on this report if it was:

- Not prepared for you;
- Not prepared for the specific purpose identified in the Agreement;
- Not prepared for the specific subject property assessed; or
- Completed before important changes occurred concerning the subject property, project, or governmental regulatory actions.

If changes are made to the project or subject property after the date of this report, Aspect should be retained to assess the impact of the changes with respect to the conclusions contained in the report.

Reliance Conditions for Third Parties

This report was prepared for the exclusive use of the Client. No other party may rely on the product of our services unless we agree in advance to such reliance in writing. This is to provide our firm with reasonable protection against liability claims by third parties with whom there would otherwise be no contractual limitations. Within the limitations of scope, schedule, and budget, our services have been executed in accordance with our Agreement with the Client and recognized geoscience practices in the same locality and involving similar conditions at the time this report was prepared.

Property Conditions Change Over Time

This report is based on conditions that existed at the time the study was performed. The findings and conclusions of this report may be affected by the passage of time, by events such as a change in property use or occupancy, or by natural events, such as floods, earthquakes, slope instability, or groundwater fluctuations. If any of the described events may have occurred following the issuance of the report, you should contact Aspect so that we may evaluate whether changed conditions affect the continued reliability or applicability of our conclusions and recommendations.

Geotechnical, Geologic, and Environmental Reports Are Not Interchangeable

The equipment, techniques, and personnel used to perform a geotechnical or geologic study differ significantly from those used to perform an environmental study and vice versa. For that reason, a geotechnical engineering or geologic report does not usually address any environmental findings, conclusions, or recommendations (e.g., about the likelihood of encountering underground storage tanks or regulated contaminants). Similarly, environmental reports are not used to address geotechnical or geologic concerns regarding the subject property.

We appreciate the opportunity to perform these services. If you have any questions, please contact the Aspect Project Manager for this project.