GEOTECHNICAL DESIGN REPORT

Jan Road Neighborhood Improvements Design Project

Prepared for: Tetra Tech, Inc. and King County Department of Natural Resources and Parks, Water, and Land Resources Division, River and Floodplain Management Section

Project No. 190175-2000 • February 7, 2022 DRAFT

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Aspect CONSULTING

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1 Introduction

This Geotechnical Design Report (GDR) presents the results of Aspect Consulting, LLC's (Aspect) geotechnical investigation and design recommendations supporting the design being prepared by Tetra Tech, Inc. (Tetra Tech) and the King County River and Floodplain Management Section (County) for the Jan Road Neighborhood Improvements Design (Project). The Project area is located along the Cedar River (river) between Renton and Maple Valley, Washington, approximately 1.3 miles northwest of the Highway 18 and State Route 169 (SR 169) interchange (Site; Figure 1). The Site extends from approximately River Miles (RM) 12.8 to RM 13.4.

This report supersedes Aspect's Preliminary Geotechnical Characterization Report (PGCR), dated October 29, 2019. This GDR report summarizes the results of the completed field investigations and laboratory testing and presents Aspect's final geotechnical engineering analyses and design considerations for the Project.

1.1 Project Description and Background

The Site contains two County flood protection facilities: the Jan Road Levee (levee) and the Cedar River Trail Site 7 Revetment (CRT 7 revetment). A description of these facilities is provided below:

- **Jan Road Levee** (levee): The facility is located along the right bank of the Cedar River (right bank) and is approximately 1,400 feet in length (Photo 1 below). The levee is not designed to contain a 100-year flood event and is not tied to high ground at either end. It has been overtopped and/or circumvented in moderate flood events in recent history, resulting in flooding of the SE 19th Pl/221st Ave SE/228th Ave SE (access road), and isolation of up to 16 single-family homes in the neighborhood behind the levee.

Photo 1: Jan Road Levee at RM 13.3, Looking Southeast

- **Cedar River Trail Site 7 Revetment** (CRT 7 revetment): This facility is located along the left bank of the Cedar River (left bank; Photo 2 below), and protects the Cedar River Trail (trail) and, further west, SR 169. The facility is approximately 500 feet in length. County records and observation have confirmed that this facility has experienced erosion and scour effects from the river.

Photo 2: CRT 7 Revetment at RM 13.15, Looking Northwest

The primary Project goals are to reduce flood-related risks to people, property, and infrastructure while improving riparian and aquatic habitat within the Project area. This will be accomplished by removing the existing Jan Road levee, constructing a new setback levee on the right bank, excavating a new side channel on the right bank floodplain, and constructing engineered log jam structures (ELJs) throughout the side channel and floodplains to improve aquatic habitat. Core objectives in support of these primary goals include the following:

- Design a system that minimizes long-term maintenance needs and associated costs.
- Reduce overall flood risk at the Site by increasing capacity for conveyance of flood flows through levee modifications and river side-channel construction within the adjacent floodplain, as well as roadway and drainage improvements.
- Set the new flood protection system back from the river, where feasible, to improve riverine and riparian processes, function, and habitat.
- Reconnect a minimum of approximately 5 acres of floodplain to provide flood conveyance and encourage natural river processes, as part of the Cedar River 2017 Large Wood Modifications Mitigation Plan (King County, 2017).

Approximately 1,200 linear feet of the existing levee will be removed, which will include establishing a finished grade level with the ground surface beyond the landside levee toe and removing riprap armoring to approximately 2 feet below the riverbed. Approximately 2,300 linear feet of a new setback levee will be constructed to current engineering

standards. The setback levee will typically extend between 4 and 6 feet above the existing ground surface to provide adequate flood protection and a minimum of 1.0 feet of freeboard for the 1 percent annual chance flood (ACF), also known as the 100-year flood event. Scour protection measures integrating large wood will be constructed along the toe of the levee embankment as a biorevetment and within the floodplain as engineered log jams (ELJs).

A new side channel will be excavated within the right bank floodplain in the area opened up by the removal of the existing levee. The side channel will be approximately 1,100 feet long and will help to reconnect the floodplain with the Cedar River, convey flood flows, and improve instream and riparian area habitat. The side channel will extend to depths of 4 to 8 feet below the existing ground surface within the floodplain.

An existing culvert under SE 197th Place will be replaced with a larger, four-sided, precast concrete box culvert structure located just north of the upstream end of the new setback levee. The box culvert will be approximately 16 feet wide and 30 feet long, with angled wingwalls to achieve grade control around the culvert. The culvert will help convey overbank flows from Taylor Creek. We assume that design and construction of the culvert will be in accordance with the current American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specifications (BDS) (AASHTO, 2020) and/or the Washington State Department of Transportation (WSDOT) Bridge Design Manual (BDM) (WSDOT, 2020).

Tetra Tech is responsible for the hydrology and hydraulic evaluations, levee removal and setback design, side channel design (except for the wood structures), scour countermeasure design (including the biorevetment), and design of the roadway and drainage facilities. The County is responsible for the design of the large wood structures (except for the biorevetments).

The Site, topography, existing flood protection facilities, new setback levee alignment, proposed side channel, and other key Project features are shown on Figure 2.

1.2 Scope of Work

Aspect's scope of work builds upon the investigative work completed for the preliminary phase of the Project by the County to facilitate and support of the final design, and included:

- Additional subsurface investigations, including drilled borings, monitoring well installations, and test pits
- Geotechnical laboratory testing on selected soil samples
- Slug testing at the monitoring well locations.
- Identification of setback levee critical sections for analysis
- Evaluation of seepage pressures and vertical hydraulic exit gradients through the landside toe of the setback levee
- Estimating levee settlement
- Evaluating global stability of the setback levee for the following conditions
    - End of construction
    - Steady-state seepage
    - Sudden drawdown

  - Seismic
- Evaluating liquefaction susceptibility and associated potential settlement during the maximum design earthquake (MDE) and operating basis earthquake (OBE) scenarios
- Setback levee design recommendations based on the outcomes of the above explorations, testing, and analyses
- Timber pile evaluations, including lateral resistance, uplift capacity, and drivability
- Geotechnical design recommendations for the new box culvert
- Geotechnical design support for other Project features, including roadway embankments, pavement, utilities, habitat excavations, and scour protection
- Other geotechnical recommendations and considerations, including groundwater and soil management, temporary cut slope and excavation stability, design considerations for temporary construction haul roads, monitoring well decommission, and general earthwork recommendations for construction

1.3 Design Criteria

We evaluated the Project elements in general accordance with guidance provided in the following United States Army Corps of Engineers (USACE), AASHTO, WSDOT and timber pile design manuals:

- Engineering Manual (EM) 1110-1-1904 Settlement Analysis (USACE, 1990)
- EM 1110-2-1913 Design and Construction of Levees (USACE, 2000)
- EM 1110-2-1902 Slope Stability (USACE, 2003)
- Engineering Technical Letter (ETL) 1110-2-569 Design Guidance for Levee Underseepage (USACE, 2005)
- EM 1110-2-6053 Earthquake Design and Evaluation of Concrete Hydraulic Structures (USACE, 2007), as referenced by the Engineering Regulation (ER) 1110-2-1806 Earthquake Design and Evaluation for Civil Works Projects (USACE, 2016)
- Engineering Pamphlet (EP) 1110-2-18 Guidelines for Landscape Planting and Vegetation Management at Levees, Floodwalls, Embankment Dams, and Appurtenant Structures (USACE, 2019)
- AASHTO LRFD Bridge Design Specifications (AASHTO, 2020)
- WSDOT BDM (WSDOT, 2020)
- ASD/LRFD Manual for Engineered Wood Construction (AWC, 2012) and associated National Design Specification (NDS) Supplement: Design Values for Wood Construction 2018 Edition (AWC, 2018)
- Timber Pile Design and Construction Manual,(AWPI, 2016)

The design details for the various Project elements used in our analyses were developed based on the communications with the Project team (Tetra Tech, County) and the draft 100 percent design drawings (Tetra Tech, 2021). The levee design heights were determined based on the 1 percent annual probability of exceedance flood, or 100-year flood, accounting for future projected floodplain roughness conditions 50 years

post-construction, plus a minimum of 1.0 feet of freeboard to account for uncertainty in the hydrologic and hydraulic analyses and future Site and river conditions.

2 Site Description

This section provides a summary of the Site, land use history, surface conditions, and geologic history, and provides context for the engineering geologic units used for design of the Project elements. Our summary and interpretations are based on review of existing information collected by others in the vicinity of the Site, geologic mapping of the area, and our surface observations at the Site.

2.1 Geography and Land Use

The Cedar River meanders through the Site, generally traveling from south (upstream) to north (downstream). At approximately RM 13.37, a generally east-west flowing creek (Taylor Creek) intersects with the Cedar River. The river makes a sharp turn to the west from approximately RM 13.4 to RM 13.3, and another sharp turn to the northeast at approximately RM 13.15, where it encounters the CRT 7 revetment.

The western bank (left bank) of the Cedar River is occupied by heavily vegetated floodplain from approximately RM 13.3 to 13.15, where it intersects with the Cedar River Trail, an asphalt-paved bike and pedestrian trail built atop a historical railroad grade. The eastern bank (right bank) of the river consists of several residential parcels and open space, accessed by a private, gravel-surfaced road named SE 197th Place / 221st Ave SE / 228th Ave SE.

The cultural resources risk report written for this Project describes the effect of human presence on the landscape – in particular, how construction of dams, revetments, and artificial channels have reduced flood flows and decreased the width of the river channel (CRC, 2019). The nearest dam to the Site is the Landsburg Diversion Dam, which is located approximately 5 miles southeast (upstream) of the Site, and was built in 1901 (City of Seattle, 2019). A larger dam on the Cedar River is the Cedar Falls/Masonry Dam, which is located approximately 14 miles east (upstream) of the Site, and was built in 1902 (HAER, 1968).

2.2 Jan Road Levee

The Jan Road Levee extends along the right bank of the Cedar River, from approximately RM 13.18 to RM 13.37. According to the archaeologic study performed for this Project (CRC, 2019), the levee was built by King County Public Works in the 1960s after a large flood affected the Site in 1959. The USACE has recorded several inspections and repairs on the levee over time and have provided them upon request; additionally, King County has recorded inspections of the levee over time. Copies of the available records and a summary are included in Appendix C.

The topography of the floodplain is generally flat or gently sloping downward towards the west and northwest, with the levee crossing through the floodplain generally in a southerly to northerly direction. The levee typically slopes up from the river at a 1.5H:1V (horizontal:vertical) slope to a height of 8 to 12 feet above the river channel (Photo 1). The levee top is approximately 15 to 20 feet wide, and then slopes down on the landward side at an approximately 1.5H:1V slope. The levee height ranges from approximately 3 to

9.5 feet above the floodplain elevation (landward side of the levee), with an average height of approximately 5 feet above the floodplain. The tallest point of the levee (about 9.5 feet high) was observed near RM 13.2. The waterward side of the levee is surfaced by riprap revetment starting from the Taylor Creek-Cedar River confluence at approximately RM 13.37 and continuing downstream to approximately RM 13.18. Based on review of construction plans for the existing levee (County, 1962; County, 1965), the riprap revetment extends to about 1 foot below the riverbed and is about 0.5 feet thick at the levee top and 2 feet thick at its base.

2.3 CRT 7 Revetment

The CRT 7 revetment extends along the Cedar River left bank from approximately RM 12.8 to RM 13.13. At this location, the river flows almost directly perpendicular (west) to the revetment before being redirected by the revetment towards the northwest and parallel to the CRT 7 revetment alignment. Information related to the timing and details of the original construction of the revetment were not available at the time of this report.

The revetment, which protects the trail and SR 169, underwent repairs in 1998 for erosion and scour that occurred during a flood event and has been monitored by King County since then. Recent inspection reports from the County indicate that the revetment was damaged from a flood event in January 2009 and has since experienced oversteepening and voids in the riprap revetment due to ongoing scour/erosion.

The CRT 7 revetment typically slopes up from the river channel at a 1.5H:1V slope to a height of 20 to 25 feet. During a site recon on August 15, 2019, Aspect observed some oversteepened areas at the toe of the revetment where its slope approaches vertical (Photo 3). The revetment slopes up toward the CRT, which is approximately 20 feet wide. East of the trail surface is a small drainage swale and, beyond that, the SR 169 roadway.

Photo 3: CRT 7 Revetment at RM 13.1, Oversteepened Soil Along Revetment Toe (Looking Southwest)

2.4 Vegetation

Site vegetation typically consists of a variety of deciduous and conifer trees, thick blackberry brambles, and tall grass and weeds. Trees along both the left and right bank of the Cedar River are typically straight and vertical. Some trees near the base of the CRT 7 revetment are leaning towards the river. Landscaping along the trail typically consists of short grass. East of the river, several wide paths have been cut through the floodplain vegetation to provide access to the levee and river.

2.5 Drainage

The Jan Road neighborhood along the right bank floodplain becomes inundated at about the 5 percent annual probability of exceedance flood, initiating from Taylor Creek at the upstream end of the Site with flows spilling over SE 197th Place and passing through a 12-inch-diameter, concrete culvert beneath the roadway. During non-flood conditions, the existing culvert connects a wetland area north of the roadway with the floodplain on the south side of roadway.

Central to the right bank floodplain, there are two existing ponds. These ponds extend about 10 to 15 feet below the floodplain.

2.6 Geological Setting

The geology of the Site vicinity is characterized by Cedar River alluvium deposited within a sequence of Fraser and pre-Fraser glacial deposits overlying Tertiary bedrock of Renton Formation at depth (Figure A below; Booth, 1995). Quaternary glacial deposits in the Site vicinity show a record of multiple ice advances through the region. Drainage of a regional glacial lake caused rapid erosion, creating the Cedar River valley within the thick, complex sequence of glacial and nonglacial deposits. The present-day Cedar River valley is characterized by the steep sideslopes, often mantled by colluvium from episodic landslides and soil creep (Booth, 1995).

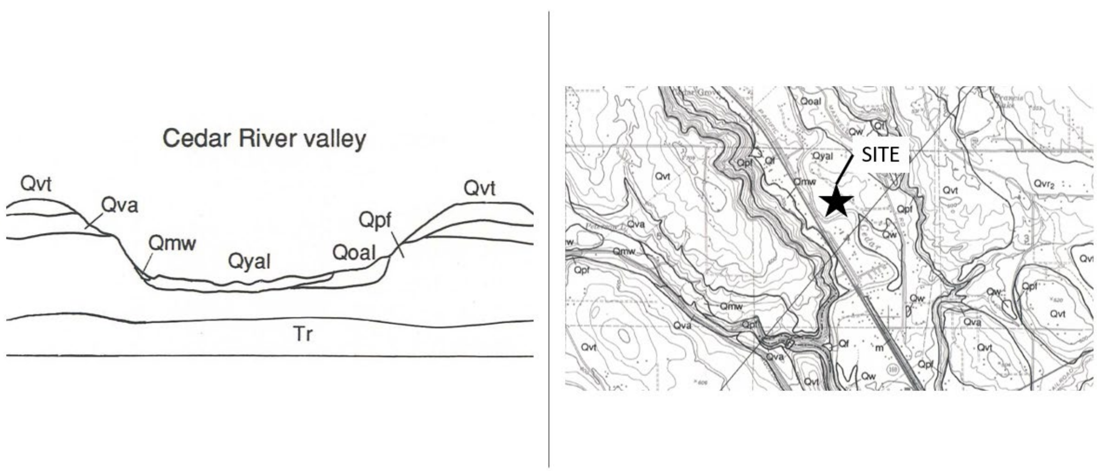

Figure A: Site Geologic Map and Stratigraphy (Booth, 1995)

The relevant geologic units at and near the Site and depositional history includes the following units (from oldest to youngest):

Regional Bedrock – Renton Formation: Sedimentary bedrock is not exposed at the surface in the immediate vicinity of the Site; however, there are some exposures within the Cedar River valley both to the north (downstream) and south (upstream) of Site (Booth, 1995; Schuster et al., 2015). Cross-sectional analysis indicates that the Renton Formation Tertiary sandstone (Tr) bedrock underlies the pre-Fraser glacial deposits in the Site vicinity (Booth, 1995). Renton Formation bedrock consists of nonmarine sandstone and claystone with abundant coal beds.

Pre-Fraser Glacial Deposits: Older (pre-Fraser) glacial deposits (Qpf) are exposed along the lower sideslopes of the Cedar River valley in the vicinity of the Site and are anticipated to directly underly the alluvium of the Cedar River (Booth, 1995). These deposits are described as a sequence of multiple thin deposits, generally too small to show in map scale. These deposits include sand and gravel, lacustrine sediments containing local peat layers, and diamict composed of silty matrix and rounded gravel clasts. The Pre-Fraser glacial deposits are commonly weakly oxidized to oxidized in appearance.

Advance Glacial Outwash (Fraser): Upper slopes of the Cedar River valley sidewall are composed of Fraser-age Advance Outwash (Qva; Booth, 1995). Advance outwash is described as well-bedded sand and gravel. These deposits are often susceptible to water erosion, resulting in multiple large ravines and gullies in the Site vicinity.

Glacial Till (Fraser): Upland areas above the Cedar River valley are capped with Fraser-age till (Qvt; Booth 1995). Glacial till in the region is described as compact diamict containing subrounded to well-rounded clasts. The till ranges from few feet to a few tens of feet thick.

Older Alluvium (Cedar River): Older alluvial deposits are present on the east side of the Cedar River Valley in Site vicinity (Qoal; Booth, 1995). These deposits consist of moderately sorted cobble gravels, pebbly sand, and sandy silt. Deposits are at a higher elevation than the younger alluvium of the Cedar River, though are otherwise indistinguishable.

Younger Alluvium (Cedar River): Alluvial sediments deposited by the present-day Cedar River make up the surficial geology in the immediate Site vicinity (Qyal; Booth 1995). Younger alluvium deposits consist of cobbles, pebbly sand, and sandy silt.

Alluvial Fan Deposits: Alluvial fan deposits are present in the Site vicinity where streams emerge from confining valleys into the Cedar River valley and reduced gradients cause stream sediment loads to be deposited (Qf; Booth, 1995). Alluvial fan deposits consist of boulders, cobbles, and sand.

Mass Wasting Deposits: Deposits of colluvium or landslide debris are common along the bottoms of the inner walls of the Cedar River valley (Qmw; Booth 1995). Landslides are common where advance outwash is in contact with the less permeable Pre-Fraser glacial deposits and a steep slope is present. Mass wasting deposits include a mixture of sediments from the overlying glacial deposits and are of varying thicknesses.

Wetland Deposits: Wetland deposits are present along the right bank of the Cedar River in the Site vicinity (Qw; Booth, 1995). These deposits include peat and alluvium and are typically poorly drained and seasonally wet. Wetlands are present at the approximate

location of the Taylor Creek-Cedar River confluence, northwest of approximately RM 13.35.

The Site geology is generally characterized as younger alluvium deposits (Qyal), with isolated areas of wetland deposits (Qw) along the Cedar River right bank. The younger alluvium deposits are typically described as cobbles, gravel, pebbly sand, and sandy silt; wetland deposits are typically described as peat and alluvium which grade into the younger alluvium deposits unit (Booth, 1995).Wetland deposits are mapped along the river right bank in the area north of approximately RM 13.35, the approximate location of the Taylor Creek-Cedar River confluence.

Based on previous explorations (discussed in Section 3) and geologic mapping and cross-sectional analysis (Booth, 1995), we anticipate the Site to be underlain by alluvial deposits (Qyal, Qoal) with thickness on the order of 20 to 30 feet and overlying pre-Fraser glacial deposits (Qpf). Renton Formation sedimentary bedrock (Tr) is anticipated to underly the pre-Fraser glacial deposits at unknown depths.

2.7 Geologic Hazards

The King County PGDR (King County, 2020) presents a review of the King County iMap tool and Washington State Department of Natural Resources (DNR) Geologic Information Portal (DNR, 2020) to determine the geologic hazards present at the Site. These geologic hazards are reiterated and added to below.

Seismic Hazard: Alluvium deposits in the areas surrounding the Cedar River are mapped with moderate to high susceptibility of liquefaction. Liquefaction susceptibility generally becomes low in the areas mapped as glacially consolidated sediments along the river valley sidewalls to the east and west (Palmer et al., 2007).

Potential Landslide / Steep Slope Hazard: The riverbank, including the CRT 7 revetment slope and areas of the levee, are mapped as steep slopes. Isolated areas along the river valley sidewalls are mapped as existing landslide deposits (King County, 2019a; WGS, 2020).

Erosion Hazard: The left and right bank of the Cedar River are mapped as susceptible to erosion in the Site vicinity from approximately RM 13.1 to 13.4 (left bank) and from approximately RM 13.4 to RM 13.9 (right bank) (King County, 2019a).

Channel Migration Hazard: Areas throughout the Site and immediately surrounding the trail are mapped as moderately to severely susceptible to channel migration (King County, 2019a).

Surficial Fault Rupture: A trace of an active, east-west trending thrust fault zone (Seattle Fault zone) is located approximately 3.6 miles north-northeast of the Site (WGS, 2020). This fault zone consists of multiple strands of southerly dipping shallow thrust faults and northerly dipping backthrust planes that extended to the ground surface in the late Holocene (Pratt et al., 2015). However, due to the suspected long recurrence interval and the distance of the Site to the mapped fault zone, the potential for surficial ground rupture at the Site is considered low.

3 Subsurface Conditions

Subsurface conditions at the Site were inferred from the completed field investigation, our review of previous explorations nearby, a review of applicable geologic literature, local geologic experience, and geotechnical laboratory testing.

3.1 Subsurface Explorations

Aspect completed a total of 15 subsurface explorations at the Site between October 2020 and January 2021. The explorations included:

- Five drilled soil borings, designated AB-01 through AB-05, advanced using sonic drilling techniques to depths between approximately 21 to 41 feet below ground surface (bgs).
- Three drilled soil borings equipped with groundwater monitoring well installations, designated AMW-01 and AMW-03, advanced using sonic drilling techniques to depths of approximately 60 feet bgs.
- Seven test pits, designated ATP-01 through ATP-07, advanced using an excavator to depths between approximately 5 and 14 feet bgs.

The locations of Aspect's explorations are shown on Figure 2. Exploration logs created by Aspect for the Site and more detailed descriptions of the exploration methodologies are provided in Appendix A. Representative photographs of the subsurface exploration program are shown below in Photographs 4 and 5 below.

Photo 4: Sonic drill positioned at AB-05

Photo 5: Excavation of test pit at ATP-02

3.2 Laboratory Testing

Aspect completed moisture content, fines content, organic content, particle-size distribution, and moisture-density characteristics (compaction) analyses on select soil samples obtained from our explorations to inform our geotechnical analyses.

Detailed descriptions of the laboratory tests and results are presented in Appendix B. The results of the tests were incorporated into the exploration logs in Appendix A.

3.3 Review of Existing Data

Aspect reviewed several sources of existing subsurface exploration data as part of our investigation, including:

- Data from 6 drilled soil borings and 12 excavated test pits at the Site collected by King County Roads Services Division in May 2019 and March 2020 during the preliminary geotechnical investigation stage of this Project (County, 2020).
- Two preliminary shallow hand auger explorations that Aspect advanced with hand tools in March 2020 for the Project (Aspect, 2020a).
- Data from one drilled soil boring and three excavated test pits collected by Aspect in October 2020 upstream of the Site along the left bank of the river for the separate, but related, Rutledge-Johnson Project (Aspect, 2020c).

The data from these explorations are included in Appendix C and summarized in Table 1 below.

Table 1. Previous Explorations

Exploration ID	General Location	Exploration Equipment	Depth to Water (ft)	Final Depth (ft)	Comments
B-1	Left Bank – CRT 7 revetment, just south of revetment	Hollow-Stem Auger and Mud Rotary Drill Rig	22.0	51.5	B-1 log is a composite of 3 boring attempts. See PGDR for details
B-2	Left Bank – CRT 7 revetment, just north of revetment	Sonic Drill Rig	21.0	70.0	Boring terminated due to heave
B-3	Left Bank – CRT 7 revetment, north of revetment	Sonic Drill Rig	21.0	60.3	Boring terminated due to heave
B-4	Right Bank – Floodplain northeast of levee	Hollow-Stem Auger Drill Rig	7.5	21.5	-
B-5	Right Bank – Floodplain north of levee	Hollow-Stem Auger Drill Rig	7.5	21.5	Monitoring well installed, well screen between 5 and 10 feet below ground surface
B-6	Right Bank – Floodplain north of levee	Hollow-Stem Auger Drill Rig	7.5	21.5	-
TP-1	Right Bank – Jan Rd Levee embankment and levee toe	Backhoe Excavator	n/a	8.0	2 adjacent pits denoted '-Levee' and '-Toe'
TP-2	Right Bank – Jan Rd Levee embankment and levee toe	Backhoe Excavator	4.0 ft below levee toe	8.0	2 adjacent pits denoted '-Levee' and '-Toe'
TP-3	Right Bank – Jan Rd Levee embankment and levee toe	Backhoe Excavator	6.5 ft below levee toe	7.0	2 adjacent pits denoted '-Levee' and '-Toe'
TP-4	Right Bank – Jan Rd Levee embankment and levee toe	Backhoe Excavator	n/a	6.0	2 adjacent pits denoted '-Levee' and '-Toe'
TP-5	Right Bank – Floodplain north of levee	Backhoe Excavator	7.0	7.5	-
TP-6	Right Bank – Floodplain north of levee	Backhoe Excavator	4.8	5.0	-
TP-7	Right Bank – Floodplain north of levee	Backhoe Excavator	4.8	5.5	-

Exploration ID	General Location	Exploration Equipment	Depth to Water (ft)	Final Depth (ft)	Comments
TP-8	Right Bank – Adjacent to SE 197th Pl	Backhoe Excavator	-	8.0	-
TP-9	Right Bank – Adjacent to SE 197th Pl	Backhoe Excavator	7.8	8.0	-
TP-10	Right Bank – Adjacent to SE 197th Pl	Backhoe Excavator	-	8.0	-
TP-11	Right Bank – Adjacent to Taylor Creek	Backhoe Excavator	5.3	5.3	-
TP-12	Right Bank – Adjacent to Taylor Creek	Backhoe Excavator	5.0	5.0	-
AHA-01	Right Bank – Near RM 13.5	Hand Tools	0.5	6.0	-
AHA-02	Right Bank – Near RM 13.5	Hand Tools	-	3.0	-
RJB-01	Left Bank - Top of Rutledge-Johnson levee near RM 13.45	Sonic Drill Rig	10.2	31.50	-
RJTP-01	Left Bank – Southeast of former side channel	Backhoe Excavator	3.5	7.00	-
RJTP-02	Left Bank – Northeast of former side channel	Backhoe Excavator	3.3	6.10	-
RJTP-03	Left Bank – Southwest of former side channel	Backhoe Excavator	3.3	5.25	-

We also reviewed a stability evaluation and risk assessment previously prepared by Aspect (Aspect, 2020b) for the CRT 7 revetment, as well as levee and revetment repair records from the County and USACE and record drawings of the levee (County, 1962; County, 1965). This data informed the subsurface explorations completed as part of this geotechnical assessment and provided further context of subsurface conditions at and near the Site. These documents are also included in Appendix C.

3.4 Stratigraphy

The results of our subsurface explorations correlate well with the existing geologic mapping and the previous explorations near the Site. The primary soil units observed in the subsurface explorations were CRT 7 revetment fill, levee embankment fill, alluvial deposits (sand and gravel facies), and pre-Fraser deposits. Within the primary alluvial unit, which dominates the shallow subsurface at the Site, we have distinguished between

sand facies and gravel facies to better characterize the subsurface conditions for our geotechnical engineering analyses.

Stratigraphically (from top to bottom), the fill associated with the CRT 7 revetment and levee embankment was encountered at the surface (where applicable), typically following by sand facies of the alluvial deposits, then gravel facies of the alluvial deposits, then pre-Fraser deposits at depth. These units are described in more detail below, and presented visually on Figures 3 and 4 – Geologic Cross Section, along with the proposed levee profile.

3.4.1 Cedar River Trail 7 (CRT 7) Revetment Fill

Soil interpreted to be fill was encountered by the County Roads Division under the trail in borings B-1 through B-3 to depths of approximately 20 feet bgs. The fill soils typically consist of fine to medium sand and silty sand (SP, SM)[1], fine and coarse gravel (GP), and cobbles. The fill was likely placed during the construction of the railroad grade and/or revetment that underlies the trail alignment; however, fill typically has a jumbled and unstructured appearance which is similar to the appearance of landslide deposits. Landslide or mass wasting deposits are not mapped at the revetment location, but are mapped along the lower portion of the slope immediately west of SR 169; it is possible that a portion of the fill noted in borings B-1 through B-3 consists of landslide deposits.

The standard penetration test blow count values ("N-values") ranged from 5 to 18 blow per foot (bpf), with an average N-value of 9 bpf, indicating the revetment fill was placed in a loose to medium dense configuration. No groundwater was encountered within the revetment fill.

Based on soil type and density, the revetment fill can be assumed to possess moderate shear strength, low to moderate compressibility, low to moderate moisture sensitivity due to its variable fines content, no susceptibility to liquefaction (no groundwater was encountered in this unit), and high permeability.

3.4.2 Levee Embankment Fill

Fill soils were encountered by the County Roads Division at the ground surface of the existing Jan Road levee embankment in test pits TP-1 Levee through TP-4 Levee to depths of approximately 4 to 8 feet bgs. The fill soils typically consist of a relatively thin (less than 0.5 feet) layer of sand or silty sand (SP-SM, SM) over fine and coarse gravel (GP, GW) and cobbles that appeared to have been sourced from the Cedar River channel and/or nearby floodplain. Soil density tests were not completed within the levee embankment, but the log descriptions refer to them as being "loosely deposited" County, 2020). No groundwater was encountered within the levee embankment fill.

Based on soil type and the reported qualitative observations of soil density, the levee embankment fill can generally be assumed to possess moderate shear strength, low compressibility, low moisture sensitivity due to its low fines content, and high permeability.

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[1]Soils are classified per the Unified Soil Classification System (USCS) in general accordance with the ASTM International (ASTM) Method D2488 *Standard Practice of Description and Identification of Soils*.

3.4.3 Alluvial Deposits – Sand Facies

Sand facies of the alluvial deposits were encountered in ATP-01, ATP-03 through ATP-05, ATP-07, B-4 through B-6, TP-1 Toe through TP-4 Toe, TP-5, TP-7 through TP-12, AB-01, and AB-03 through AB-05. In general, sand facies of the alluvial deposits were encountered below the existing levee embankment fill or near the existing grade (just below a layer of surficial topsoil) to approximately 2 to 5 feet bgs. Exceptions to this pattern were observed in AB-04, ATP-02, and ATP-07. In AB-04, on the left bank, sand facies were encountered below gravel facies from approximately 8 to 12 feet bgs, indicating a more dynamic alluvial depositional environment in this location. Similar depositional patterns were observed at ATP-02 and ATP-07, where thin layers of sand facies were encountered from approximately 3 to 4 feet bgs. In general, the sand facies are not laterally continuous over the Site; they were encountered only in the isolated areas shown on Figures 3 and 4.

The sand facies generally consisted of loose to medium dense, moist to wet, brown to gray sand with varying proportions of silt and gravel (SW-SM, SP, SM) and non-plastic sandy silt (ML). In AB-01, AB-05, and ATP-03 through ATP-07, organics were observed in the sand facies, including roots and fragments of woody debris.

The sand facies of the alluvial deposits can be expected to exhibit moderate shear strength characteristics, high permeability, moderate compressibility, and low to moderate moisture sensitivity.

3.4.4 Alluvial Deposits – Gravel Facies

Gravel facies of the alluvial deposits were encountered in every exploration at the Site. Where the explorations encountered fill from the levee embankment, fill from the CRT 7 revetment, or sand facies of the alluvial deposits, the gravel facies were encountered immediately beneath the fill or sand facies. Elsewhere, gravel facies were encountered near the existing grade, just below a layer of surficial topsoil. The gravel facies of the alluvial deposits extended to depths of between approximately 10 to 19 feet bgs in the drilled soil borings completed by Aspect. The excavated test pits generally terminated in gravel facies of the alluvial deposits at the bottom of the explorations between approximately 5 to 14 feet bgs.

Gravel facies of the alluvial deposits generally consisted of medium dense to very dense, dry to wet, brown to gray gravel with varying proportions of silt, sand, cobbles, and boulders (GP, GW, GP-GM, GW-GM). We expect that cobbles and boulders were underrepresented in the samples from the drilled soil borings, due to limitations on sample size from the core barrel diameter. In the excavated test pits, cobbles were observed at ATP-01 and ATP 04 through ATP-07, while boulders were observed at ATP-02 and ATP-03. In addition, we observed roots in the gravel facies to approximately 2 feet bgs at ATP-01, ATP-02, and ATP-07. The density of the gravel facies tended to increase with depth.

We expect the gravel facies of the alluvial deposits to exhibit moderate to high shear strength characteristics, high permeability, low to moderate compressibility, and low moisture sensitivity.

3.4.5 Pre-Fraser Deposits

Soils interpreted to be pre-Fraser deposits were encountered in all of the drilled soil borings from the bottom of the alluvial deposits to the bottom of the explorations between approximately 20 to 70 feet bgs. The pre-Fraser deposits consisted generally of dense to very dense, moist to wet sand with varying proportions of silt and gravel (SP, SP-SM, SW, SW-SM, SM) and gravel with sand (GP). We observed a consistent, characteristic color change in the pre-Fraser deposits, where the deposits transitioned from red-brown to dark gray (i.e., oxidized to unoxidized) over the shallowest 10 to 20 feet of the deposit.

At AMW-01, the pre-Fraser deposits graded from a silty sand (SM) to a hard, moist, gray silt (ML) near the bottom of the exploration at approximately 60 feet bgs. Trace amounts of wood debris were observed in the pre-Fraser deposits at AB-01 and AMW-03. During drilling, the pre-Fraser deposits tended to heave into the sonic core barrel, which may have resulted in understated blow counts.

We expect the pre-Fraser deposits to exhibit high shear strength characteristics, low to moderate permeability, low compressibility, and low moisture sensitivity.

3.5 Groundwater Conditions

In general, relatively shallow groundwater is present throughout the Site and .appears to be in hydraulic continuity with the Cedar River. The depth to groundwater at the time of drilling or test pit excavation was noted for each exploration and is shown on the logs (Appendices A and C) and the geologic cross section (Figures 3 and 4).

Groundwater monitoring wells were installed in drilled soil borings B-5 and AMW-01 through AMW-03. The County monitored groundwater conditions at each location during the time periods indicated in Table 2 below. The groundwater or groundwater head fluctuations and trends are presented graphically in Appendix D along with corresponding precipitation and river stage data. The inferred average groundwater level is also shown on the geologic cross section (Figures 3 and 4).

Table 2. Summary of Groundwater Monitoring Installations

Exploration ID	General Location	Monitoring Time Period
B-5	Right Bank – Floodplain north of existing levee	7/19/2019 to 9/8/2021
AMW-01	Right Bank – Setback Levee STA 25+20	3/4/2021 to 9/8/2021
AMW-02	Right Bank – Setback Levee STA 14+75	3/4/2021 to 9/8/2021
AMW-03	Right Bank – Setback Levee STA 6+40	5/3/2021 to 9/8/2021

3.6 Slug Testing

Aspect performed slug testing in monitoring wells AMW-01, AMW-02, and AMW-03 to estimate the hydraulic conductivity of the soils within the screened zone in each well. A slug test produces a rapid change in water level within a well and measures the rate of return to the static water level (SWL). This rate of water level change in the well is used to calculate the hydraulic conductivity (K) of the water bearing zone.

A slug rod (a solid cylinder of known volume) was used to displace water in the tested wells. The slug test is performed in two phases: the falling head test and the rising head

test. During the falling head test, the slug rod was rapidly submerged in the water column, causing the water level to rapidly rise before falling back to the SWL over time. Upon completion of the falling head test, a rising head test was performed by quickly removing the slug rod from the water column, causing the water level to fall before rising back to the SWL over time. To test the results for repeatability and dependency on hydraulic head, at least three slug tests were performed in each well with two slug rods of different lengths.

The water levels in the wells during testing were measured using a vented pressure transducer (5 or 15 pounds per square inch [psi] range) and collected electronically on a data logger set to record measurements on a nearly continuous time interval (1 second or less). Manual depth to water measurements were also collected using an electronic tape to verify water levels. After reviewing the slug test data, an appropriate analysis method was selected to estimate the hydraulic conductivity: Bouwer & Rice (1976) for moderate K values, and Butler & Garnett (2000) for high K values.

3.6.1 Slug Testing Results

Based on our analyses, K estimates ranged from 12 to 350 feet per day (ft/day) for a total geometric average of 44 ft/day. Repeated test results were consistent at each well and consistent with field observations during the tests. However, average K values at AMW-03 (250 ft/day) were an order of magnitude higher than AMW-01 (13 ft/day) and AMW-02 (14 ft/day). Slug test parameters and results are summarized in Table 3 (attached).

4 Geotechnical Engineering Analyses

Our geotechnical engineering analyses for the Project included reduction and organization of the subsurface exploration and laboratory testing data; development of soil engineering properties; determination of critical levee section locations and geometry, with settlement, stability, seepage, and liquefaction evaluations at the appropriate critical sections; and development of culvert and timber pile design recommendations. The following sections describe the basis of our methodologies, key assumptions, and results.

Subsurface exploration data by Aspect, geotechnical laboratory testing results, compiled historical data related to the Site and existing structures, and groundwater data collected through continuous monitoring are included in Appendices A through D. Detailed analysis descriptions and outputs related to settlement, stability, seepage, liquefaction, timber pile evaluations, and culvert design are included in Appendices E through J.

4.1 Soil Engineering Properties

We developed soil engineering and hydrogeologic properties based on the results of our completed subsurface explorations, previous exploration data by others, lab test results, slug testing results, empirical correlations with standard penetration test (SPT) blow counts, empirical formulas for estimating hydraulic conductivity, back calculations of existing conditions, literature review, and our experience with the local geology.

For levee embankment materials, we assumed a zoned levee section consisting of a relatively low permeability core (Levee Core Fill) surrounded by a shell of select fill (Select Fill Type 1). Based on our understanding of the Project design, we have assumed the material for the Levee Core Fill will be imported. The majority of the Select Fill Type 1 will either be derived from on-Site excavations related to removal of the existing levee, floodplain modifications and side channel creation, and/or imported from the nearby pits, which will result in relatively granular (sand and gravel) material. On-Site derived material will also be used for backfill of portions of the revetment along the waterward toe of the levee and below the anticipated groundwater table (Ballast Fill). Based on these anticipated materials, we developed representative engineering properties for use in our analyses.

To account for the range and variability of the native soil and the materials anticipated for re-use and import on the Project, we also varied the engineering properties used in our analyses for sensitivity scenarios and to help verify the assumed engineering properties.

Specific references utilized in the development of the soil engineering and hydrogeologic properties include the WSDOT Geotechnical Design Manual (GDM; WSDOT, 2021), the Navy Facilities Engineering Command (NAVFAC) Design Manual 7.1 (NAVFAC, 1986), USACE EM 1110-2-1904 *Settlement* (USACE, 1990), USACE 1110-2-1901 *Seepage Analysis and Control for Dams* (USACE, 1993), and Massman (2003).

The soil engineering and hydrogeologic properties used in our geotechnical engineering analyses are shown on Table 4 (attached).

4.2 Levee Analysis Section Locations and Geometry

We analyzed settlement, seepage, slope stability, and liquefaction at five critical sections along the proposed setback levee alignment. The analyzed sections are shown on Figure 2 and were chosen based on critical locations accounting for a combination of factors, including proposed levee height, underlying geology, groundwater conditions, and predicted scour/revetment configuration. The section locations analyzed are detailed in Table 5 below.

Table 5. Levee Analysis Section Locations and Details

Section	Stationing(1)	Levee Height(2)	Riverward 100-yr(3) WSE(4)(5)	Landward 100-yr(6) WSE	Mean Annual WSE	Scour Elevation(7)
A-A'	3+50	5.8	257.2	255.5	251.5	244.2
B-B'	12+37.30	3.9	259.4	-	253.4	245.3
C-C'	16+25	6.3	263.4	-	251.5	246.3
D-D'	19+00	5.7	264.0	-	251.5	240.9
E-E'	24+00	4.3	267.6	-	258.0	245.4

Notes:

1) Stationing per draft 100 percent Project drawings and as shown on Figure 2.
2) Levee height is calculated as top of levee (at centerline) to base of levee core fill.
3) 1% annual probability of exceedance flood.
4) WSE = water surface elevation, yr = year.
5) WSE information provided by Tetra Tech (Tetra Tech, 2021).
6) Where elevation is noted, minor flooding of landward side of levee was predicted by hydraulic modeling. Where no elevations are noted, WSE was predicted at or below the ground surface by hydraulic modeling.
7) Long-term elevations at riverside levee toe due to scour or projected future channel migration. Scour elevations provided by Tetra Tech (Tetra Tech, 2021).

Ground surface elevations, topography, and levee geometry were provided by Tetra Tech and are based on the proposed Project grading and survey data collected for the Project, and are detailed in the draft 100 percent drawings (Tetra Tech, 2021). The levee typical section geometry assumed in our analyses are shown below on Figures B through F.

The setback levee at Section B-B' intersects an existing pond, which will be partially filled prior to building the setback levee, leaving a swale at the landward toe of the levee. The levee core at this section was also designed with straight sides, as opposed to the "T-shape" seen in the typical levee section. These conditions led to unique stability and seepage conditions that were assessed by Aspect. We included an additional typical levee section figure at Section B-B' to illustrate the geometry of the setback levee at this location.

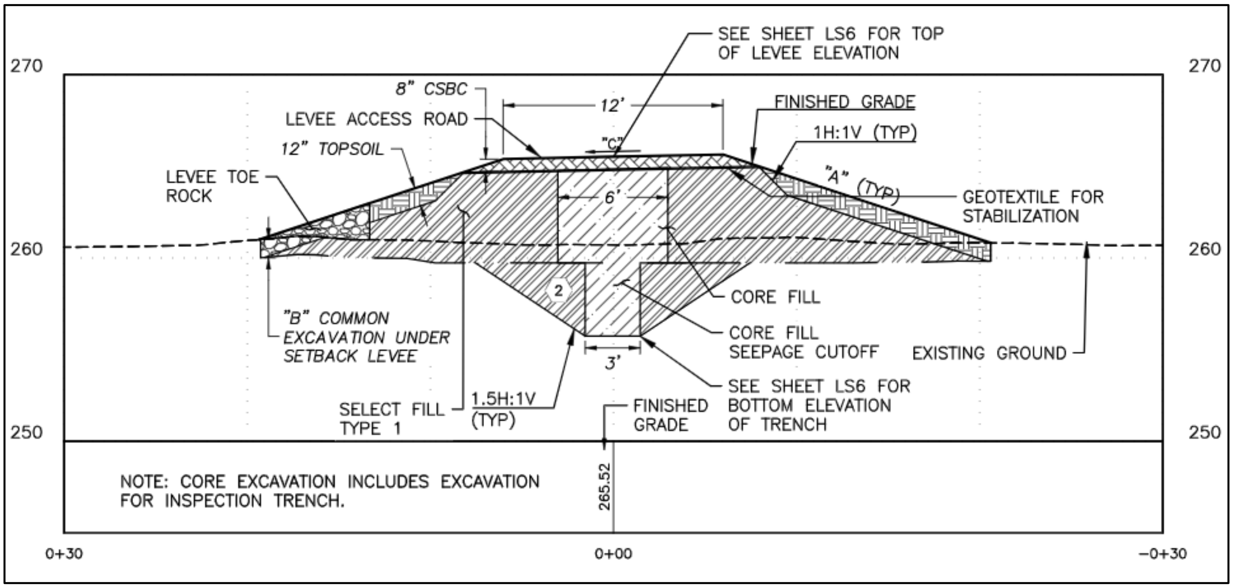


Figure B. Typical Levee Section (Tetra Tech, 2021)

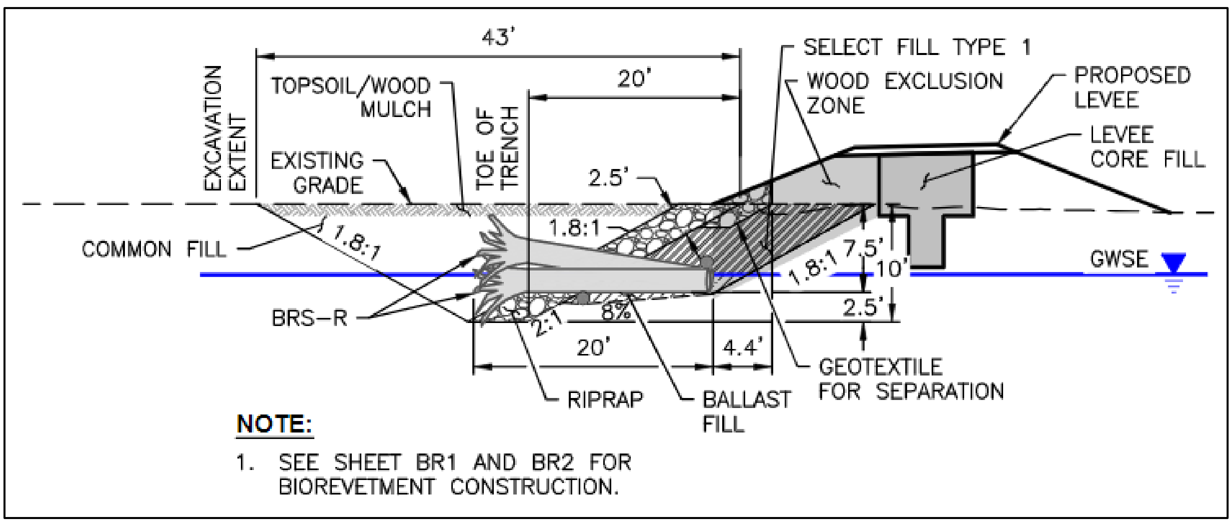


Figure C. Typical Levee and Biorevetment Geometry, Section A-A'

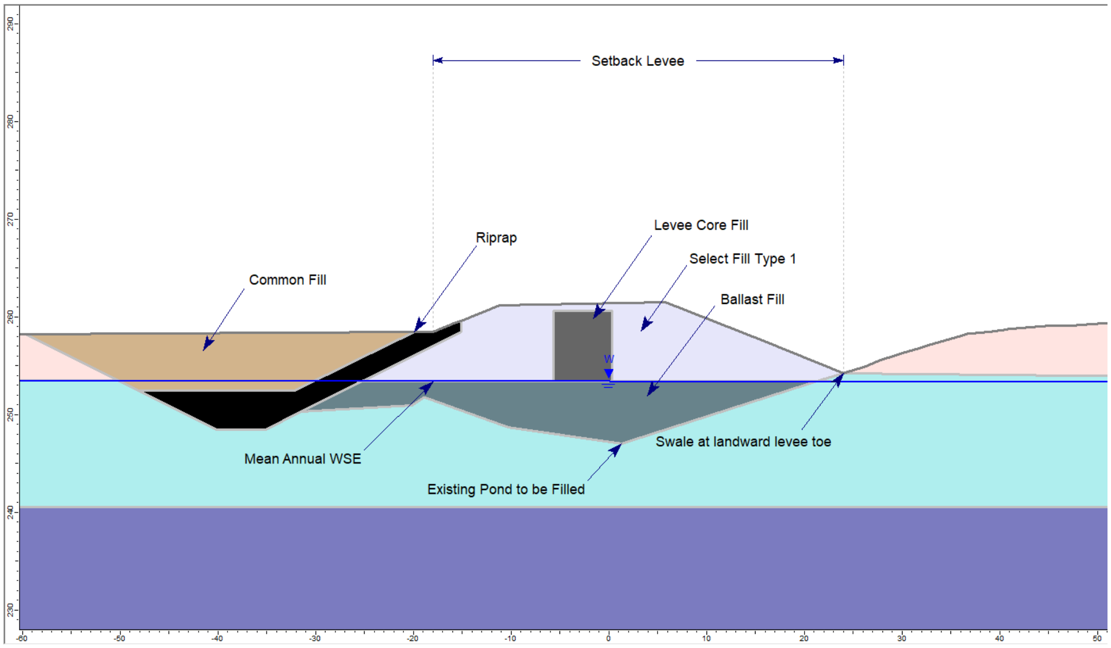
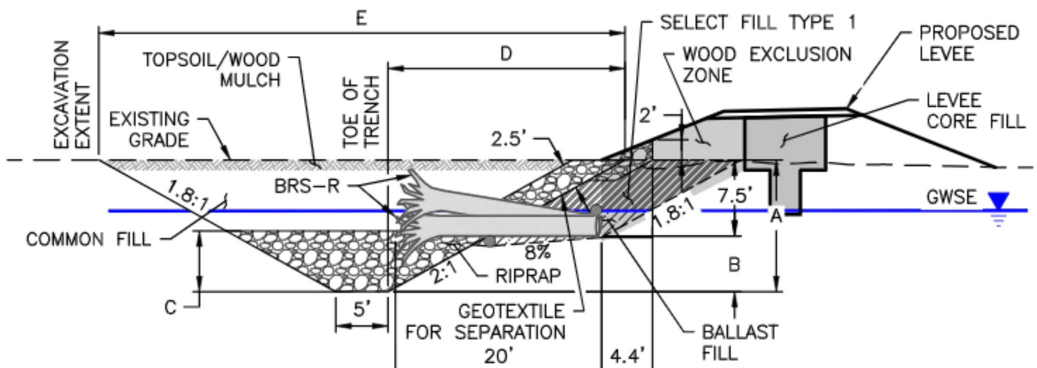


Figure D. Typical Biorevetment Geometry, Section B-B'



LS STATION	A	B	C	D	E
11+75 TO 12+70	10.0	2.5	4.0	20.0	43.0
15+85 TO 16+65	10.0	2.5	4.0	20.0	43.0
16+65 TO 20+60	10.0	2.5	6.6	20.0	43.0
20+60 TO 23+00	13.0	5.5	6.0	26.0	54.4
25+25 TO 28+10	13.0	5.5	6.0	26.0	54.4

NOTE:

1. SEE SHEET BR1 AND BR2 FOR BRS–R CONSTRUCTION.

Figure E. Typical Biorevetment Geometry, Sections C-C' through D-D'

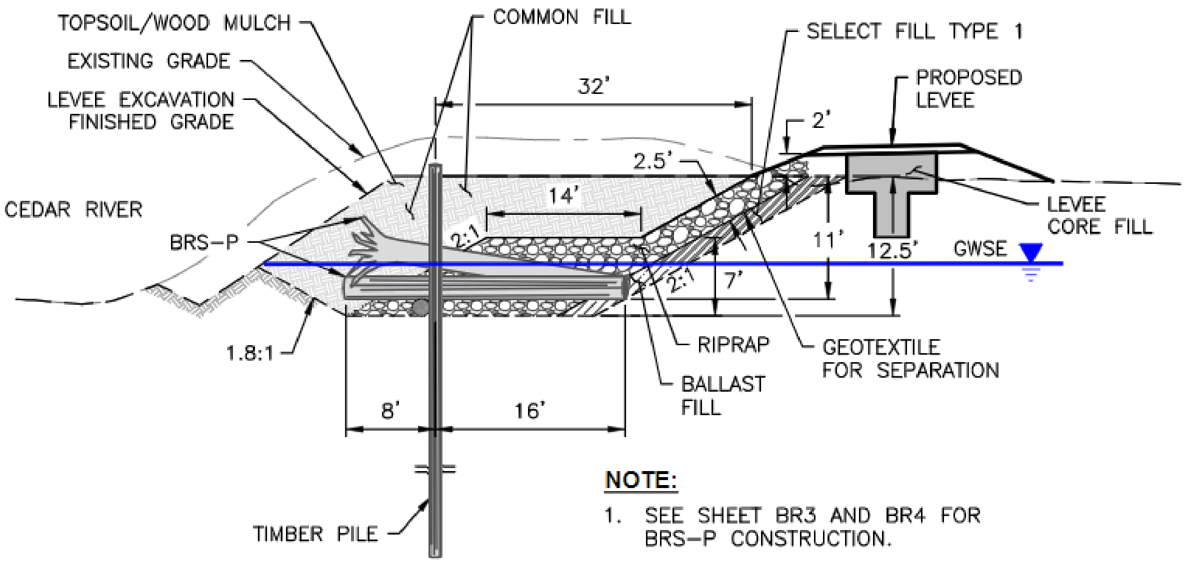


Figure F. Typical Biorevetment Geometry, Section E-E'

4.3 Levee Settlement

Construction of the new setback levee embankment will result in surcharge pressures on the existing Site soils and will trigger settlement along the proposed setback levee alignment. Calculation of total soil settlement is divided into three components: immediate settlement, consolidation settlement, and secondary compression. Each of these are summarized below.

- Immediate settlement is primarily elastic and occurs rapidly, typically during the construction process, in both nonplastic fine-grained and coarse-grained soils. Immediate settlement is expected to be less than 0.5 inches along the setback levee alignment and will occur as the levee fill is placed.
- Consolidation settlement is a time-dependent process that occurs in saturated, fine-grained soils as excess porewater pressures dissipate after a load is placed on the soil. The magnitude of consolidation settlement depends on the compressibility and *in situ* stress state of the soil deposit, the magnitude of the applied load, and the soil deposit's stress history. Due to the lack of saturated, fine-grained soil at the Site, consolidation settlement is not anticipated.
- Secondary compression is a time-dependent process that occurs after excess porewater pressures dissipate through consolidation settlement, over long periods of time that range from years to decades. Due to the lack of saturated, fine-grained soil at the Site, significant secondary compression is not anticipated.

Our explorations and analyses indicate the settlement hazard at the Site is low, with immediate settlement of less than about 0.5 inches expected along the setback levee alignment that will occur as the levee fill is placed and no long-term implications to the levee after construction. Therefore, we do not recommend any specific settlement mitigation for the Project.

Details of our settlement analysis methodology and results are presented in Appendix E, and an example output from our settlement analysis is presented in Figure G.

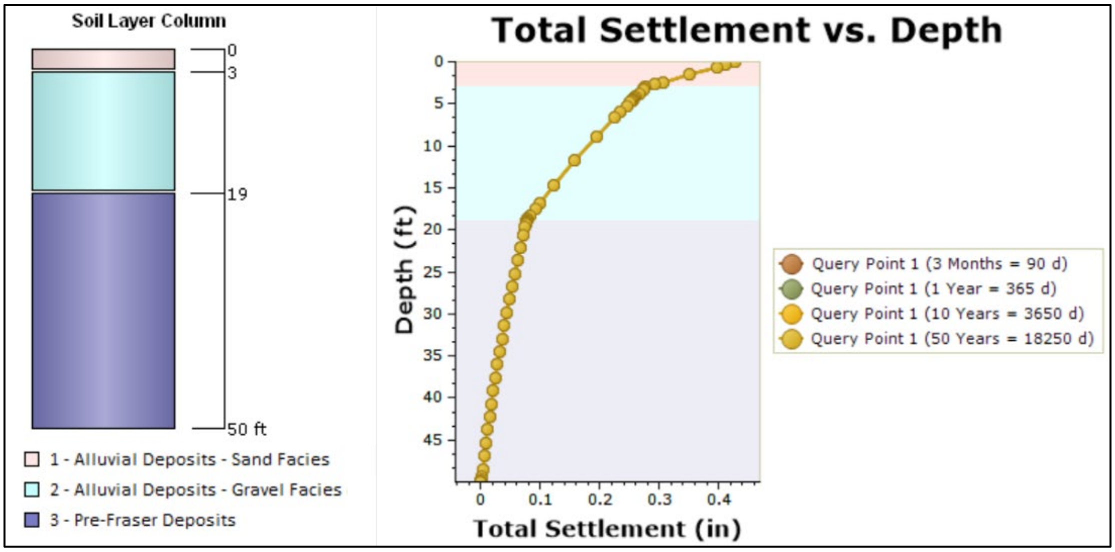


Figure G. Example Results from Settlement Analysis at Station 16+25

4.4 Geotechnical Earthquake Engineering

Following the procedures outlined in USACE Engineering Manual (EM) 1110-2-6053 *Earthquake Design and Evaluation of Concrete Hydraulic Structures* (USACE, 2007), we determined seismic parameters for design presented in Table 6 below. The USACE requires design for an Operating Basis Earthquake (OBE) and Maximum Design Earthquake (MDE):

- The OBE is level of ground motion for which the levee is able to maintain operation with little to no damage. It corresponds to a 50-percent probability of exceedance over a project service life of 100 years, which corresponds to a 144-year return period event.
- The MDE is the maximum level of ground motion under which the levee should not experience catastrophic failure. It corresponds to a 10-percent probability of exceedance over a project life of 100 years, which corresponds to a 1000-year return period event.

Each design earthquake scenario has an associated moment magnitude (M_w) and effective peak ground acceleration (PGA). The U.S. Geological Survey (USGS) has completed probabilistic ground motion studies and design maps for Washington (USGS, 2014). The design-level earthquake parameters are presented in Table 6 below.

Table 6. Design-Level Earthquake Parameters

Earthquake	Seismic Event Return Period (years)	Peak Ground Acceleration (g)	Earthquake Magnitude	Mean Source-to-Site Distance (km)
OBE	144	0.21	7.10	68.04
MDE	1,000	0.41	7.11	66.92

Notes:

1) OBE = Operating Basis Earthquake; MDE = Maximum Design Earthquake
2) Based on the latitude and longitude of the Site: 47.426601°, -122.051578°
3) Values taken from the USGS Unified Hazard Tool (USGS, 2014)

We completed seismic evaluations for the Project considering both the OBE and MDE, including pseudo-static seismic slope stability (described in Appendix G) and liquefaction (described in Appendix H).

4.5 Levee Seepage

Seepage through and/or beneath the levee embankment can cause several undesirable levee performance and integrity issues including:

- Sand boils
- Reduced soil shear strength and stability due to seepage forces
- Excessive seepage through the levee embankment
- Internal erosion or soil piping of fine sand and nonplastic silt from within the levee embankment

To account for these issues, the USACE guidelines recommend a maximum vertical exit gradient at/near the landside toe of the levee embankment of 0.5 (dimensionless. In addition, excessive levee seepage for the Project can be defined as more than 200 gallons per minute (gpm) over 100 linear feet of embankment (USACE, 2005).

To evaluate seepage through the proposed setback levee embankment, we performed steady-state seepage analyses using the finite element analysis groundwater module within the computer program SLIDE (Rocscience, 2017). We conducted seepage analyses at each of the critical sections identified in Table 5 (Section 4.2). We considered the critical case for seepage: the 100-year flood condition in long-term (fully scoured) conditions. Water surfaces for the 100-year flood conditions were provided by Tetra Tech (Tetra Tech, 2021). Using SLIDE, we modeled flow directions through and beneath the levee embankment and measured vertical hydraulic gradients and discharge potential at the landward levee toe.

The results of our seepage analyses are summarized in Table 7 below. Detailed discussion of analysis methodologies, assumptions, design criteria, and analysis outputs is included in Appendix F.

Table 7. Seepage Analysis Results, 100-Year Flood Scenario

Section	Stationing(1)	Vertical Hydraulic Gradient (unitless)(2)	Discharge near the levee toe per 100 linear feet of embankment (gpm)(3)
A-A'	3+50	< 0.1	9
B-B'	12+37.30	0.3	48
C-C'	16+25	0.3	10
D-D'	19+00	0.1	< 5
E-E'	24+00	0.4	9

Notes:

1) Stationing per draft 100 percent Project drawings and as shown on Figure 2.
2) Maximum vertical hydraulic gradient anticipated during the 100-year flood WSE. Modeled with long-term (scoured) levee geometry. Measured at/near the landward toe of the levee embankment.
3) Total discharge anticipated during the 100-year flood WSE. Modeled with long-term (scoured) levee geometry. Measured at/near the landward toe of the levee embankment. gpm = gallons per minute.

Our steady-state seepage analyses indicate that seepage passing below the levee embankment is likely to manifest in excessive vertical exit gradients or excessive seepage at the critical sections. Accordingly, we recommended extending the low permeability core below the base of the levee embankment a minimum of 5 feet or equivalent to the height of the levee (whichever is less) to provide additional shallow underseepage cutoff and to reduce the vertical exit gradients at the landward toe of the levee to meet the Project design and USACE criteria. This recommendation has been incorporated into the draft 100 percent Project design plans (Tetra Tech, 2021).

Our seepage sensitivity analyses (described in Appendix F) indicate the Levee Core Fill may have a horizontal hydraulic conductivity ranging between 2.9×10^{-4} feet per second (ft/s) and 2.9×10^{-8} ft/s. Our seepage sensitivity analyses also indicate that the Select Fill Type 1 used for the levee embankment beyond the levee core fill should be carefully controlled to limit its horizontal hydraulic conductivity to a range between approximately 1×10^{-2} ft/s and 4×10^{-4} ft/s. The gradations of the Levee Core Fill and Select Fill Type 1, described in Sections 6.5.3 and 6.5.4, respectively, are anticipated to result in a horizontal hydraulic conductivity within this range.

The Project documents should require the Contractor to submit material samples proposed for the levee embankment materials and their respective gradations for review by the Project Representative prior to levee construction and in the event of any significant changes in the materials during construction.

4.6 Slope Stability

In accordance with USACE guidance (USACE, 2000), we evaluated four analysis cases for slope stability at the critical sections of the setback levee: end-of-construction, steady-state seepage, sudden drawdown, and seismic. The minimum required factor of safety (FS) for each analysis case is shown in Table 8 below.

Our results indicate that the levee will be sufficiently stable on both the waterward and landward sides in most analysis cases. In the MDE seismic scenario, our results indicated the waterward side of the levee will be marginally stable (FS just under or equal to 1.0).

We interpret these results to signify that some damage to the levee should be anticipated in the MDE scenario, for which we anticipate maintenance-level repairs will be necessary. We do not anticipate catastrophic failure of the levee in the MDE seismic scenario.

The results of our slope stability analyses are presented in Table 8 below. Detailed methodology, key assumptions, results, and graphical outputs of our slope stability analyses are included in Appendix G.

Table 8. Slope Stability Analysis Results

Case	Water Level(1)(2)	Minimum Required FS(1)	Waterward /Landward	FS Resulting at the Critical Section(4) (Reference Figure)(5)				
				A-A'	B-B'	C-C'	D-D'	E-E'
End of Construction	Mean Annual WSE	1.3	Waterward Landward	2.6 (G-3) 2.4 (G-4)	2.9 (G-15) 1.9 (G-16)	2.5 (G-27) 2.3 (G-28)	2.6 (G-39) 2.4 (G-40)	2.5 (G-51) 2.1 (G-52)
Steady-State Seepage	100-Yr Flood(3)	1.4	Waterward Landward	1.8 (G-5) 2.1 (G-6)	1.8 (G-17) 1.8 (G-18)	1.8 (G-29) 2.1 (G-30)	1.7 (G-41) 2.1 (G-42)	1.7 (G-53) 2.0 (G-54)
Sudden Drawdown	100-Yr Flood(3) -> Mean Annual WSE	1.1	Waterward Landward	1.7 (G-7) 2.4 (G-8)	1.7 (G-19) 1.9 (G-20)	1.7 (G-31) 2.3 (G-32)	1.6 (G-43) 2.4 (G-44)	1.5 (G-55) 2.1 (G-56)
Seismic, OBE	Mean Annual WSE	1.1	Waterward Landward	1.2 (G-9) 1.7 (G-10)	1.2 (G-21) 1.4 (G-22)	1.3 (G-33) 1.7 (G-34)	1.2 (G-45) 1.7 (G-46)	1.1 (G-57) 1.6 (G-58)
Seismic, MD	Mean Annual WSE	1.0	Waterward Landward	1.0 (G-11) 1.3 (G-12)	**0.9** (G-23) 1.1 (G-24)	1.0 (G-35) 1.3 (G-36)	**0.9** (G-47) 1.3 (G-48)	**0.9** (G-59) 1.2 (G-60)

Notes:
1) WSE = water surface elevation, yr = year; FS = factor of safety.
2) WSE information provided by Tetra Tech (Tetra Tech, 2021).
3) 1% annual probability of exceedance flood.
4) Minimum factor of safety affecting the levee crest or beyond. **Bold** values indicate a factor of safety less than the minimum required FS.
5) Model setups are shown on in Appendix G - Figures G-1 and G-2 (Section A-A'), Figures G-13 and G-14 (Section B-B'), Figures G-25 and G-26 (Section C-C'), Figures G-37 and G-38 (Section D-D'), and Figures G-49 and G-50 (Section E-E').

4.7 Liquefaction

Typically, levees are not designed or constructed to mitigate for seismic-induced settlement and/or lateral spreading resulting from liquefaction because the probability of coinciding earthquake and flood events is very low (USACE, 2000). However, adoption of this approach requires more active inspections and repairs where the levee owner conducts post-seismic inspections of the levee and completes required repairs.

To aid in understanding the magnitude, distribution, and probability of significant levee damage related to liquefaction, we conducted analyses at a series of locations along the setback levee alignment for the OBE and MDE.

The results of our liquefaction analyses indicate that the probability of liquefaction along the setback levee alignment is very low to none with respect to the OBE and MDE.

Appendix H presents the details of our analyses and graphical results including key inputs and factors of safety against liquefaction.

4.8 Engineered Log Jam (ELJ) and Biorevetment Timber Piles

Several wood structures are proposed within the river, side channels, and floodplain. We understand four types of engineered log jam (ELJ) structures (designated ELJ Types 1, 2, 3, and 4) and a biorevetment on the waterward side of the setback levee between Stations 23+00 and 25+25 will be timber pile-supported. These structures will be subject to drag-induced lateral forces and uplift forces due to buoyancy and hydraulic flow-induced lift that will be resisted by the timber support piles. In support of the timber pile design, we evaluated pile uplift capacity, lateral capacity, and drivability. The pile capacity analyses informed minimum pile embedment and minimum number of piles per structure used in design. These analyses were performed for typical subsurface conditions anticipated at the Site and typical structures given their differing pile geometry and loading conditions (lateral and uplift capacity).

Based on an iterative design process with King County and Tetra Tech, we recommend three pile types be used for design:

- 50-foot-long piles with tapered diameters ranging from 18 inches at the butt to 14 inches at the tip (to be used for ELJ Type 2 structures)
- 38-foot-long piles with tapered diameters ranging from 18 inches at the butt to 14 inches at the tip (to be used for biorevetment structures)
- 35-foot-long piles with tapered diameters ranging from 16 inches at the butt to 12 inches at the tip (to be used for ELJ Types 1, 3, and 5)

Detailed methodology, key assumptions, results, and graphical outputs of our ELJ pile analyses are included in Appendix I. Our recommendations for design and construction are summarized in Sections 5.3 and 6.7.

4.9 Risk Informed Design

A risk-informed design approach does not replace the need for traditional deterministic analysis and criteria, but rather informs where designs should be up-scaled (e.g., use more conservative inputs and/or use a factor of safety higher than the minimum) or downscaled (e.g., use more aggressive/realistic inputs and/or use a factor of safety lower than the minimum). Supplementing traditional deterministic design with risk-informed design should provide a reasonable level of assurance that the levee will perform adequately over the full range of loading and reasonably account for variation in design assumptions.

From a geotechnical perspective, risk-informed design considerations undertaken for the Project include:

- Sensitivity analyses related to the assumed engineering properties of the levee fill and underlying soil units at the critical levee sections

- Seismic stability and liquefaction analyses considering the OBE and MDE
- Assessment of sensitivity to predicted scour and wood recruitment for the ELJ support piles

The results of these risk-informed design considerations are described in Appendices E through I and add context to the design of the levee, ELJs, level of reliability, and risk.

5 Geotechnical Engineering Conclusions and Recommendations

Based on the subsurface explorations, laboratory testing, review of existing data, and our geotechnical engineering analyses, we have developed the following design conclusions and recommendations for the Project.

5.1 Levee Embankment Design

The following subsections present our understanding of setback levee embankment elements and our design recommendations.

5.1.1 Levee Geometry

The crest width of the levee embankment will be 12 feet (typically) and will be gravel-surfaced for maintenance vehicle access. The top of levee, as defined for the Project, is taken at the base of the gravel surfacing. The levee crest will have a 2 percent side slope to avoid water ponding on the levee crest. Levee embankment side slopes of 2.5H:1V or flatter are suitable for the Site and proposed conditions.

5.1.2 Levee Fill

The proposed levee will be a zoned levee embankment consisting of a levee core, select levee fill zone, topsoil shell, and levee crest surfacing. The levee embankment material will be dependent on the determined feasible sources for the material, and their suitability with respect to the material requirements of each zone.

Based on the outcomes of our geotechnical engineering analyses, our recommendations the levee embankment are as follows.

5.1.2.1 Levee Core

We recommend the main levee core section extend from the top of levee to the proposed base of levee and be a minimum of 6 feet wide. For seepage mitigation, we primarily recommend an extended levee core that is a minimum 3 feet wide and extends below the base of levee a minimum of 5 feet or equivalent to the height of the levee (whichever is less) to provide additional shallow underseepage cutoff and to reduce the vertical exit gradients at the landward toe of the levee to meet the Project design criteria.

One unique area of the setback levee is between Stations 12+00 and 13+00, where an existing pond will be filled beneath the proposed levee embankment. In this location, we recommend a minimum core width of 6 feet, and we recommend the extended levee core also be a minimum width of 6 feet and extend uniformly down to the groundwater level (top of the Ballast Fill).

The hydraulic conductivity of the compacted levee core fill should range between approximately $2.9\text{x}10^{-4}$ ft/s and $2.9\text{x}10^{-8}$ ft/s. The levee core should be comprised of levee core fill as described in Section 6.5.3.

5.1.2.2 Select Fill

We recommend Select Fill Type 1 be used for levee embankment fill outside of the levee core. We recommend a maximum sideslope for the Select Fill Type 1 of 2.5H:1V when used within the levee embankment. The hydraulic conductivity of the compacted levee embankment fill outside of the levee core should range between approximately 1×10^{-2} ft/s and 4×10^{-4} ft/s. Gradation, material, and compaction requirements for Select Fill Type 1 are included in Section 6.5.4.

5.1.2.3 Levee Topsoil Shell

The levee topsoil shell may be imported or derived from on-Site borrow; we anticipate that topsoil stripped from the setback levee alignment may be partially stockpiled and reused, with the remainder required imported from off-Site. We recommend a topsoil thickness of 12 inches.

5.1.2.4 Crushed Surfacing Base Course

For surfacing the levee crest, we recommend a minimum section of 8 inches of imported Crushed Surfacing Base Course (CSBC). To promote a finer surface for users and/or pavement constructability, the upper 4 inches may consist of imported Crushed Surfacing Top Course (CSTC).

5.1.3 Inspection Trench

An inspection trench is recommended in USACE guidance to verify levee subgrade conditions, check for buried utilities, and to confirm that adverse seepage conditions are not present beneath the setback levee (old drainage features, animal burrows, concentrated organics/logs, or other debris). The inspection trench should be 6 feet deep or equal to the height of the levee embankment, (whichever is less) and approximately 3 feet wide. We recommend the inspection trench be co-located with the extended levee core fill described in Section 5.1.2.1.

Based on our observations of shallow subsurface conditions and groundwater levels during dry summer months (when the levee will likely be constructed), we anticipate groundwater will not be encountered during excavation, backfill, and compaction of the inspection trench/extended levee core.

The subsurface conditions revealed by the inspection trench should be carefully evaluated and any zones of unsuitable foundation soil should be removed or appropriately mitigated at the discretion of the Project Representative. The inspection trench should be backfilled with levee core fill meeting the material requirements and compaction rates described in Section 5.1.2.1.

5.1.4 Woody Vegetation-Free Zone

We recommend incorporating a woody vegetation-free zone on both sides of the levee in accordance with USACE EP 1110-2-18 (USACE, 2019). Woody vegetation is defined as trees, shrubs, vines, and other woody vegetation that can create structural and seepage instabilities in a levee and/or prevent adequate access for inspections, maintenance, and flood-fighting activities. The woody vegetation-free zone should extend a minimum of 15 feet horizontally from the toe of the levee embankment. Native, perennial grasses that can withstand regular mowing are allowed within the woody vegetation-free zone for ground cover and erosion protection.

5.1.5 Biorevetment Scour Protection

Due to the variable scour/channel migration potential along the proposed setback levee alignment, biorevetment scour protection is required along the majority of the proposed alignment except for between approximately Stations 12+70 and 14+10. The biorevetment scour protection will typically consist of key logs set into a ballast zone surrounded by riprap scour protection rock. A launchable riprap scour toe and vertical timber piles will be applied where needed; locations where a launchable riprap scour toe and vertical timber piles are necessary have been determined by Tetra Tech. The riprap portion of the biorevetment has been designed by Tetra Tech (2021) based on flow velocities, scour characteristics, and topography near the setback levee.

Illustrations of the typical biorevetment geometry are presented on Figures B through F. Our recommendations for the geotechnical elements of the biorevetment scour protection are as follows:

5.1.5.1 Wood Exclusion Zone

To mitigate future subsidence and potential instability of the levee core given horizontal drag loads on the key logs, we recommend a wood exclusion zone below the horizontal extents of the levee embankment that is bounded by an imaginary line extending at a 2H:1V slope down from the bottom edge of the levee core.

5.1.5.2 Biorevetment Ballast Fill

We recommend biorevetment ballast zone fill placed above the groundwater table be imported or derived from on-Site borrow, and follow the material specifications and compaction rates for Select Fill Type 1 as described in Section 5.1.2.2.

For ballast zone fill placed below the groundwater table (Ballast Fill) shall consist of well-graded soil free of organic and deleterious material, and meet the Unified Soil Classification System (USCS) soil type classification of GP-GM, GW-GM, GP, or GW with the gradation characteristics and compaction rates for Ballast Fill as described in Section 6.5.5.

5.2 Geosynthetics

We recommend the use of geosynthetics at key locations and material interfaces to provide subgrade stabilization and mitigate piping of fines, sand, and small gravel from the levee embankment/foundation into the void spaces of the ballast and riprap zones. Our recommendations are summarized below:

- We recommend a geosynthetic for soil separation be wrapped around the ballast zone with the exception of where the key logs are anchored (for constructability).
- We recommend a geosynthetic for soil stabilization at the interface between the top of the levee core and the crushed surfacing at the levee crest.
- We recommend a geosynthetic for soil stabilization be wrapped around the culvert rock fill pad.

Geosynthetic material should be woven and meet the requirements of WSDOT Standard Specification 9-33.2(1) – Table 3 (Geotextile for Separation or Soil Stabilization; WSDOT, 2021).

5.3 ELJ and Biorevetment Timber Piles

Our recommendations for pile dimensions, pile lateral capacity, allowable uplift capacity, minimum pile lengths, and minimum pile count per ELJ and biorevetment types are presented in Table I-1 in Appendix I. Table 9 below presents a summary of our design recommendations resulting from our analyses:

Table 9. Summary of Design Recommendations – ELJ and Biorevetment Timber Piles

Structure	Recommended Minimum Pile Length (ft)	Recommended Minimum Pile Embedment[(1)]
ELJ Type 1	50	23
ELJ Type 2	35	22
ELJ Type 3	35	25
ELJ Type 4	35	19
Biorevetment	38	22

Note: (1) feet below anticipated scour elevation

Detailed methodology, design considerations, and tabular outputs of our results are presented in Appendix I.

5.3.1 ELJ Ballast

Ballast materials are required for the ELJs. Unlike the biorevetments, which are in close proximity and influence the new setback levee, the ELJs are located further out in the floodplain. As such, the material and compaction requirements for ballast for the ELJs may be reduced. We anticipate that on-Site materials can be reused for the ELJ ballast fill and recommend Select Fill Type 2 be placed with an allowable lift thickness of 12 to 24 inches and required degree of compaction of approximately 80 percent of the maximum dry density (MDD) as determined by ASTM International (ASTM) D 1557 (Modified Proctor).

5.4 Culvert Design

The replacement culvert beneath SE 197th Place will consist of a four-sided, concrete, box culvert structure that is approximately 16 feet wide and 30 feet long with angled wingwalls to achieve grade control around the culvert. Due to a span width of less than 20 feet, AASHTO and WSDOT design methodologies (AASHTO, 2020; WSDOT, 2020; WSDOT, 2021) do not require consideration of seismic loading and effects. A summary of key geotechnical conclusions and recommendations for the design and construction of the culvert are listed below and described in more detail in the following sections.

- The dense alluvial deposits that underlie the crossing location are a suitable bearing layer for culvert and wingwall grade-supported foundations.
- The concrete footings supporting the culvert and wingwalls should be placed directly atop a 12-inch-thick rock fill pad consisting of Permeable Ballast as

specified in Section 9-03.9(2) of the WSDOT Standard Specifications (WSDOT, 2021), overlying the existing alluvial deposits. The rock fill pad is intended to provide relatively uniform support and a stable working surface directly beneath the footings. The rock fill pad should extend at least 2 feet in all directions (horizontally) beyond the edges of the culvert footings and at least 1 foot in all directions (horizontally) beyond the edges of the wingwall footings, and should be wrapped in a woven geotextile for soil stabilization meeting the requirements of WSDOT Standard Specification 9-33.2(1) – Table 3.

- The base of the culvert footings should be founded below the design scour level and/or protected by appropriate scour protection measures.
- The existing fill and alluvial deposits are likely suitable for use as structural fill around the new culvert and below the new road surface, provided they can be screened to meet the requirements for Select Fill Type 2 and pending approval by the Project Representative at the time of construction. Imported fill will be required for the bearing pads below culvert and wingwall foundations and pavement base course.

Detailed design recommendations, calculations, and considerations for the culvert are included in Appendix J.

5.5 Pavement Sections

We anticipate standard County pavement sections will be used for new pavement on the Project. In our opinion, a pavement section consisting of a 4-inch-thick layer of hot mix asphalt overlying an 8.5-inch-thick layer of CSBC is suitable.

6 Construction Recommendations

Based on our subsurface explorations, laboratory testing, review of existing data, and geotechnical engineering analyses, we have developed the following construction recommendations for the Project.

6.1 General Earthwork and Soil Management

Based on the subsurface exploration data across the Site and our understanding of the Project, it is our opinion that the Contractor should be able to complete planned excavations and earthwork activity with relatively standard construction equipment, although our regional experience, the depositional history of the alluvial deposits in the floodplain, and our subsurface explorations indicate that oversized materials, such as buried stumps, logs, cobbles, and boulders, may be present in excavations near the ground surface at the Site.

We anticipate construction work will largely take place during the dry summer months. During this period, groundwater should be anticipated below about 5 feet bgs. The Contractor should anticipate wet excavations below this depth and soil conditions that may not support excavation equipment. We recommend maintaining working platforms for equipment a minimum of 3 feet above the anticipated groundwater level and strategically planning earthwork activities to allow for elevated working platforms and access/haul routes.

6.2 Excavations

The Project will include primary excavations for the removal of the existing levee, establishing the new setback levee footprint, installing scour protection, and creating a side channel. Saturated soil conditions and elevated soil moisture contents should be anticipated below the anticipated groundwater level and potentially above the anticipated groundwater level during wet weather periods.

6.2.1 Temporary Excavation 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.

6.2.1.1 Temporary Excavations Above Groundwater

In general, the near-surface soils across the Project area classify as Occupational Safety and Health Administration (OSHA) Soil Classification Type B. Temporary excavation side slopes within the shallow alluvial deposits are anticipated to stand as steep as 1H:1V above the groundwater table, up to a maximum height of 20 feet. The cut-slope inclinations estimated above are for planning purposes only and are applicable to excavations without inflowing groundwater or stormwater.

6.2.1.2 Temporary Excavations Below Groundwater

To determine appropriate slope cuts for temporary excavations below the groundwater table, we observed and evaluated the performance of a deep test pit excavation (ATP-02)

and performed slope stability analyses using the two-dimensional, finite-element modelling program SLIDE (Rocscience, 2017). Details of the analysis methodology is presented in Appendix G. Based on our observations and analysis, we anticipate that temporary excavation side slopes within the shallow alluvial deposits can stand as steep as 1.8H:1V below the groundwater table, up to a maximum height of 10 feet.

6.2.1.3 Composite Temporary Excavations Above and Below Groundwater

Composite temporary excavations that extend above and below the groundwater table are feasible with the temporary excavation slopes described above, provided a horizontal bench with a minimum width of 4 feet is incorporated just above the groundwater table. If no bench is included, the sloped excavation should be no steeper than 1.8H:1V.

The temporary excavation cut-slope recommendations here are for planning purposes only and assume short stand-up times (less than a few days). Our recommendations do not account for stockpiling of materials or equipment surcharge loads above the cuts and surface water runoff into the cuts. With time and the presence of seepage and/or precipitation, the stability of temporary unsupported cut slopes can be significantly reduced. 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 trench walls. In such an event, lateral support for the trench walls should be provided by the Contractor to prevent loss of ground support.

6.2.2 Side Channel and Existing Levee Removal Excavation

The new side channel will extend between 4 and 8 feet below the floodplain and excavations along the riverbank are required to remove the existing levee (including existing scour protection rock). The goal of these excavations is to provide for stable conditions in the short term and then allow the side channel and remnant riverbank to reach a state of natural stability in the long term. We recommend excavating the side channel side slopes and remnant riverbank at a maximum angle of 2H:1V for initial stability.

6.3 Wet Weather Earthwork and Erosion Control

Existing Site soils may prove to be difficult to handle or traverse with construction equipment during periods of wet weather. Our general recommendations relative to earthwork performed in wet weather or in wet conditions are presented below. These recommendations should be incorporated into the contract specification and should be required when earthwork is performed in wet conditions:

- Site stripping and fill placement should be accomplished in small sections to minimize exposure to wet weather. Excavation or removal of unsuitable soil should be followed promptly by placement and compaction of a suitable thickness of Select Fill Type 1 (described in Section 6.5.4). The size and type of construction equipment used may have to be limited to prevent soil disturbance.
- No soil should be left uncompacted so it can absorb water. Stockpiles of excavated soil should either be shaped and the surface compacted or be covered

with plastic sheets. Soils that become too wet should be removed and replaced with clean granular materials.

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

- The fine-grained nature of the levee core fill material (described in Section 6.5.3) means it is sensitive to moisture and may be difficult to compact in wet weather conditions. We recommend that placement and compaction of levee core fill in the levee embankment and placement and compaction of the backfill for the inspection trench be avoided during wet weather periods.

Soil erosion can be minimized by implementing these recommendations in concert with careful grading practices and the appropriate use of silt fences and/or straw bales. Surface runoff control during construction should be the responsibility of the Contractor. All collected water should be controlled and discharged in accordance with local regulations. Grading measures, slope protection, ditching, sumps, dewatering, and other measures should be employed, as necessary, to permit proper completion of the work. Permanent control of surface water should be incorporated in the final grading design. Water should not be allowed to pond immediately adjacent to the levee.

6.4 Subgrade Preparation

Based on the results of our subsurface explorations along the setback levee alignment, we estimate typical stripping depths (topsoil, sod, organics, and roots/woody debris greater than about 0.5-inch diameter) will be required as follows:

- Station 0+00 to Station 14+50: Average stripping thickness of 12 inches

- Station 14+50 to Station 26+00: Average stripping thickness of 6 inches

The estimates provided above are for planning purposes only. In isolated areas, the stripping depth and/or presence of unsuitable foundation soil may result in greater stripping depths, up to 4 feet, but it is our opinion that these areas will comprise approximately 10 percent of the levee footprint or less. Unsuitable foundation soil is defined as unstable soil that has significantly higher permeability than the surrounding subgrade soil (significant void space), and/or organic-rich material.

After stripping is completed, the exposed subgrade soils should be compacted to a firm an unyielding condition and evaluated with a roll test (proof roll) using a fully-loaded dump truck or similar equipment. The proof roll should be observed and evaluated by the Project Representative, and areas that exhibit yielding or rutting within the subgrade should be repaired by loosening, aerating, and compacting the materials in-place or by removal and replacement of the poor subgrade with a suitable fill material (typically Select Fill Type 1 along the setback levee alignment) meeting the requirements and compaction standards described in Sections 6.5 and 6.6.

6.5 Fill Materials and Requirements

A variety of fill materials are required for construction of the Project. Based on the results of our geotechnical engineering analyses, we recommend the following material requirements.

6.5.1 Reuse of On-Site Soil

We anticipate a variety of on-Site soil and materials can be suitable for reuse related to the following Project applications: Common Fill, Select Fill Types 1 and 2, Ballast Fill, and topsoil. For reuse, the on-Site soil and materials must meet (or be successfully amended to meet) the material requirements for the proposed application as described herein and be approved by the Project Representative.

The shallow Alluvial Deposits – Sand Facies above the groundwater table are likely to be suitable for reuse as Common Fill and Select Fill Types 1 and 2. Saturated Alluvial – Sand Facies sourced from below the groundwater table may be suitable for Common Fill (with some drying) and for reuse as Select Fill Types 1 and 2 (with significant drying).

The Alluvial Deposits – Gravel Facies above the groundwater table are likely to be suitable for reuse as Common Fill, Select Fill Types 1 and 2, and Ballast Fill. Saturated Alluvial – Gravel Facies sourced from below the groundwater table may be suitable for re-use as Ballast Fill and Common Fill with some drying.

The topsoil along the setback levee alignment appears to be feasible for reuse as topsoil on the levee and may be stockpiled and/or processed for this purpose. Similarly, the shallow Alluvial Deposits – Sand Facies excavated for the Project may also be suitable for reuse as topsoil with some amending and processing (e.g., mixing and supplementing the organic content).

6.5.2 Common Fill

Common Fill shall consist of inorganic soils that are capable of attaining specified compaction levels, excluding soils which contain organics, contain debris, are potentially expansive, or are USCS soil types OL, OH, CH, MH or PT. Common Fill shall be obtained from on-Site sources as approved by the Project Representative and should conform to the following:

- Maximum particle size of 6 inches
- Maximum 30-percent fines content (percent passing the #200 [0.075mm] sieve).
- Less than 3 percent organic material by volume.

6.5.3 Levee Core Fill

The levee core fill should be imported from a suitable source and consist of relatively low permeability material. Low permeability core fill shall consist of well-graded soil free of organic and deleterious materials and meet the USCS soil type classification of CL, ML, SC, or SM with the gradation characteristics shown in Table 10, below. Fines (material passing the No. 200 sieve) should have a maximum plasticity index of 40 and a maximum liquid limit of 45 percent.

Table 10. Levee Core Fill Gradation

Sieve Size	Percent Passing (by Dry Weight)
2-inch	100
No. 4	85 min.
No. 200 (0.075 mm)	30 min

Levee core fill shall be compacted to at least 90 percent of its MDD as determined by ASTM D1557 (Modified Proctor). The material shall be placed in horizontal lifts that do not exceed 12 inches in loose lift thickness.

6.5.4 Select Fill

We anticipate select fill will be used for multiple Project applications. We recommend two types of select fill to appropriately cover the range of applications:

- Select Fill Type 1 will be used for levee fill outside of the levee core and revetment ballast above the groundwater level. The gradation requirements for the Select Fill Type 1 include a smaller maximum particle size, requirements for a well-graded particle distribution, and higher allowable fines content.
- Select Fill Type 2 will be used for ELJ ballast, culvert backfill, and general structural fill. The gradation requirements for the Select Fill Type 2 include a larger maximum particle size and lower allowable fines content.

Both types of select fill shall consist of well-graded soil free of organic and deleterious material, and meet the USCS soil type classification of GM, GC, GP, GW, SM, SC, SP-SM, SW-SM, SP, SW with the gradation characteristics shown in Table 11, below. Select fill derived from on-Site borrow should be carefully screened to ensure that it meets the general gradation criteria described in Table 11 and that oversized materials and deleterious material are removed.

Table 11. Select Fill Gradation

Sieve Size	Percent Passing (by Dry Weight)	
	Select Fill Type 1	Select Fill Type 2
6-inch	100	100
3-inch	100	75-100
No. 4	20 min.	25-75
No. 40	50 max.	50 max.
No. 200 (0.075 mm)	2-30(2)	10 max.(2)
Organic Content	1 max.	3 max.

Notes: (1) Where Select Fill Type 2 is within 2 feet below a paved surface or for utility trench backfill, reduce maximum particle size to 3-inches. (2) Fines (material passing the No. 200 sieve) should have a maximum plasticity index of 40 and a maximum liquid limit of 45 percent.

Select fill shall be compacted to at least 90 percent of its MDD as determined by ASTM D1557 (Modified Proctor). Within 2 feet below pavement surfaces, the select fill shall be compacted to at least 95 percent of its MDD as determined by ASTM D1557 (Modified

Proctor). The material shall be placed in horizontal lifts that do not exceed 12 inches in loose lift thickness.

6.5.5 Ballast Fill

Ballast fill shall consist of well-graded soil free of organic and deleterious material, and meet the USCS soil type classification of GP-GM, GW-GM, GP, or GW meeting the gradation requirements in Table12 below.

Table 12. Ballast Fill

Sieve Size	Percent Passing (by Dry Weight)
6-inch	100
2-inch	65-100
No. 4	45 max.
No. 40	15 max.
No. 200 (0.075 mm)	5 max.

Ballast zone fill placed below the groundwater table shall be placed in level lifts not exceeding 9 inches thick and tamped with an excavator bucket or similar equipment to achieve a firm and unyielding condition.

6.5.6 Crushed Surfacing

Crushed surfacing for the Project should be imported and meet the requirements of Crushed Surfacing Base Course (CSBC) as described in Section 9-03.9(3) of the WSDOT Standard Specifications (WSDOT, 2021). For pavements and the levee crest surfacing, the upper zone of crushed surfacing may consist of Crushed Surfacing Top Course (CSTC) as described in Section 9-03.9(3) of the WSDOT Standard Specifications (WSDOT, 2022).

Crushed surfacing shall be compacted to at least 90 percent of its MDD as determined by ASTM D1557 (Modified Proctor). Within 2 feet below pavement surfaces, the select fill shall be compacted to at least 95 percent of its MDD as determined by ASTM D1557 (Modified Proctor). The material shall be placed in horizontal lifts that do not exceed 6 inches in loose lift thickness

6.5.7 Permeable Ballast

Permeable ballast should be imported and meet the requirements of Section 9-03.9(2) of the WSDOT Standard Specifications (WSDOT, 2021). Permeable ballast shall be placed in level lifts not exceeding 9-inches-thick and tamped with an excavator bucket or similar equipment to achieve a firm and unyielding condition.

6.6 Compaction Requirements

Compaction requirements for the various materials proposed for the Project are detailed in the preceding sections and are summarized below in Table 13.

Table 13. Material Compaction Requirements

Material	Maximum Loose Lift Thickness (in)	Compaction Criteria	Moisture Control Requirement	Refer to Report Section
Common Fill	18	80% of MDD(3)	N/A	6.5.2
Levee Core Fill	12	90% of MDD	+/- 2% of OMC	6.5.3
Select Fill Type 1	12	90% of MDD	+/- 2% of OMC	6.5.4
Select Fill Type 2	12 to 24(4)	80%(5) to 95%(6) of MDD(3)	+/- 2% of OMC	6.5.4
Ballast Fill	9	Firm and Unyielding Condition(3)	N/A	6.5.5
Crushed Surfacing	6	90% to 95%(6) of MDD	+/- 2% of OMC	6.5.6
Permeable Ballast	9	Firm and Unyielding Condition(3)	N/A	6.5.7

Notes:

1. MDD = Maximum Dry Density as determined by ASTM D 1557 (Modified Proctor).
2. OMC = Optimum Moisture Content as determined by ASTM D 1557 (Modified Proctor).
3. Compaction may be evaluated using Visual Inspection Method, when appropriate.
4. Initial lift thickness over utilities of 24 inches and lift thickness may range from 12 to 24 inches for ELJ Ballast applications.
5. Compact to 80% of MDD for ELJ Ballast applications.
6. Compact to 95% of MDD within 2 feet below pavements.
7. For utility trench backfill in unimproved areas where settlement can be tolerated, we recommend the backfill material be placed in loose lifts not exceeding 18 inches and compacted to a reasonably firm and unyielding condition.

The procedure to achieve the specified minimum relative compaction depends on the size and type of compacting equipment, the number of passes, thickness of the layer being compacted, and certain soil properties. When the size/constraints of the fill area restrict the use of heavy equipment, smaller equipment can be used, but the soil must be placed in thin enough lifts to achieve the required compaction. A sufficient number of in-place density tests should be performed as the fill is placed to verify the required relative compaction is being achieved. For certain fill types and applications, compaction may be evaluated using the “Visual Inspection Method” as detailed in the Project specifications, where observations are made during material placement and compaction that indicate the maximum lift thicknesses are not exceeded and that the material does not exhibit pumping, reaction, yielding, cracking, or other deformations during final compaction.

Generally, loosely compacted soils are a result of poor construction techniques or improper moisture content. Soils with a high percentage of silt or clay are particularly susceptible to becoming too wet, and coarse-grained materials easily become too dry, for proper compaction. Silty or clayey soils with a moisture content too high for adequate compaction should be dried, as necessary, or moisture conditioned by mixing with drier materials, or other methods.

6.7 Timber Piles

Timber piles for support of the ELJs and biorevetment should be installed in accordance with Section 6-05 of the WSDOT Standard Specifications (WSDOT, 2021). The Project contract documents should require that the Contractor provide submittals detailing the selected piles along with information regarding the respective pile driving systems and equipment that will be used for installing the piles prior to the start of construction. A wave equation analysis demonstrating that the proposed pile driving equipment is capable of driving the piles to the required minimum embedment depths without causing damage to the piles should be included in the initial submittal information.

We recommend test piles be installed and tested to confirm the assumed uplift capacities in accordance with Section 6-05.3(10) of the WSDOT Standard Specifications (WSDOT, 2021). Pile load tests should be observed and evaluated by the Project Representative. The test piles may be used as production piles provided they are considered satisfactory by the Project Representative.

6.7.1 Timber Pile Driving

Given the Site subsurface conditions and with the understanding that the majority of the timber piles for the ELJs and biorevetment will be between 30 and 40 feet long with some timber piles up to 50 feet in length that will be required for the Type 2 ELJ structure, we conducted a pile drivability analysis. Detailed methodology, design considerations, and graphical outputs of our results are presented in Appendix I.

Based on the results of our drivability analyses, we recommend the following:

- We anticipate that pile driving with a vibratory hammer will require oversized hammers that risk overstressing the timber piles. Therefore, we consider pile drivability using only vibratory hammers to be infeasible at the Site

- Our drivability results show that pile driving is marginally feasible at the Site with an impact hammer.

- Due to the presence of cobbles and boulders in the subsurface, and the associated potential for hard driving conditions, we recommend the pile locations be pre-drilled to below the specified minimum embedment depth by approximately 5 percent of the pile length to allow loose material to accumulate in the pre-bored hole prior to driving the piles. The diameter of the pre-drill should be approximately the diameter of the timber pile tip or slightly larger.

- We anticipate the Contractor will elect to use pre-drilling in conjunction with impact hammer driving to install the piles. The impact hammer used should be appropriately sized for the pile and soil conditions. We recommend an impact hammer similar to, or smaller than, an APE D 12-42. Larger impact hammers will likely be oversized for the timber piles, and use of larger impact hammers could lead to pile damage.

- The piles should have steel tips/points for protection during driving and straps to help prevent splintering during driving. The purpose of the steel tips is to protect

the piles from damage should cobbles or other obstructions migrate back into the driving alignment between the pre-drilling and pile driving process.

- If the piles cannot be driven to planned embedment depth following this process, pre-drilling with temporary steel casing may be necessary. The piles could be placed within the temporary steel casing, then the casing could be backfilled with drill spoils. The temporary casing would then be extracted and reused for subsequent pile installations. We recommend the construction budget for pile installation should have a contingency to account for the potential need for drill casing. The contract plans and/or specifications should also include language stating that piles shall be installed via drill casing if the intended embedment is not reached.
- Obstructions encountered during pile drilling/driving may cause some of the piles to be driven out-of-plumb, or to "drift" off of the design horizontal location. Also, if significant obstructions are encountered at certain locations, it may be necessary to adjust certain pile locations to avoid the obstructions. Because of this potential effect, some flexibility should be allowed in the design to enable adjustment of pile locations. Any such situations that arise during construction should be evaluated on a case-by-case by the Project Representative.
- Pile load testing may be used at the direction of the Engineer and Project Representative to assess the uplift capacities of installed piles and to determine acceptability of piles installed short of the specified minimum embedment depths where very difficult driving conditions persist.

In general, pile driving construction should follow the guidelines set forth in the Project specifications. The Project contract documents should require that the Contractor provide submittals detailing the selected piles along with information regarding the respective pile driving systems and equipment that will be used for installing the piles prior to the start of construction.

6.7.2 Pile Load Testing

Pile load testing may be used to determine if installed piles are able to provide the required uplift capacities assumed in design. This can be particularly useful when very difficult driving conditions are encountered and when piles are not able to be driven/installed to the specified minimum embedment depth. Pile load testing can also be used to assess if shorter piles can provide the minimum design capacities.

Pile load testing in the floodplain should be completed with a tensiometer and equipment capable of generating uplift loads on the order of 20,000 to 40,000 pounds based on the assumed pile uplift capacities in design. When testing, the Contractor should utilize the equipment to pull steadily and carefully to determine if the pile can meet the required uplift resistance for structure stability. The Contractor should avoid jerking the pile or ramping up tension too high to destabilize the pile. If the pile breaks or starts to pull out while testing, it will need to be pulled and a replacement pile driven in, as directed by the Engineer.

Based on the results of the pile load testing, the Engineer will assess and recommend either additional testing of piles not meeting the minimum embedment depth, or that no

additional testing is needed, and the pile may be accepted. The Project Representative and/or Engineer may recommend additional test piles within the structure.

6.7.3 Pile Driving Vibrations and Noise

The proposed pile driving will result in ground-borne vibrations that propagate through the ground by means of Rayleigh (surface) waves and secondarily by body (shear and compressional) waves and may impact adjacent structures. The amplitude of these waves diminishes with distance due to expansion of the wave front and from dissipation of energy within the soil itself (material damping). The nearest structures to the Project are single-family residences, located about 200 feet from the nearest proposed pile-supported ELJs and biorevetment. Given the distance of the structures from the proposed pile driving, it is unlikely that vibration-induced damage will occur.

Pile driving in the riverine setting of the Project will cause noise that can affect species listed in the Endangered Species Act (ESA). We understand that others on the Project design team are responsible for evaluating the pile driving impacts to ESA-listed species and identifying potential mitigation alternatives and best management practices (BMPs) in support of the environmental permits for the Project. We are available to support and evaluate mitigation measures for pile driving noise and specific pile driving-induced vibration considerations as needed.

6.8 Monitoring Well Decommissioning

Monitoring wells installed for the Project should be decommissioned at an appropriate time during construction in accordance with Washington State Department of Ecology requirements and regulations (WLS, 2008).

7 Recommendations for Continuing Geotechnical Services

We recommend that Aspect provide services during final design, procurement, and construction as follows.

7.1 Additional Design and Consultation Services

Aspect will continue to meet with the design team to advance geotechnical elements of the design process. Our geotechnical engineering conclusions and recommendations will be refined as the design process continues, and this report will be superseded by our final GDR report.

Before construction begins, we recommend that Aspect:

- Continue to meet with the design team, as needed, to address geotechnical questions that may arise throughout the remainder of the design, procurement, and construction planning process.
- Review the geotechnical elements of the Project plans to see that the geotechnical engineering recommendations are properly interpreted.
- Review and respond to any questions or comments from Project Stakeholders.

7.2 Additional Construction Services

We understand the County will retain a Construction Management (CM) Team to oversee the construction of the Project. We are available to provide geotechnical engineering services in support of the CM team during construction. The integrity of the geotechnical elements depends on proper Site preparation and construction procedures. In addition, engineering decisions may have to be made in the field if variations in subsurface conditions become apparent.

During the construction phase of the Project, we recommend that Aspect be engaged to perform the following tasks:

- Review applicable submittals.
- Provide support to the CM team during Project construction.
- Attend meetings, as needed.
- Address other geotechnical engineering considerations that may arise during construction.

The purpose of our observations is to verify compliance with design concepts and recommendations, and to allow design changes or evaluation of appropriate construction methods if subsurface conditions differ from those anticipated prior to the start of construction.

8 References

American Association of State Highway and Transportation Officials (AASHTO), 2020, LRFD Bridge Design Specifications, 9th Edition.

American Wood Council (AWC), 2017, National Design Specification Supplement: Design Values for Wood Construction, 2018 Edition, October 2017.

American Wood Council (AWC), 2012, ASD/LRFD Manual for Engineered Wood Construction, 2012 Edition, October 2012.

American Wood Preservers Institute (AWPI), 2016, Timber Pile Design and Construction Manual, September 2016.

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

Aspect Consulting, LLC (Aspect), 2020a, Memorandum to J. Scheller, PE (Tetra Tech, Inc.) from A. Holmson, PE (Aspect), H. H. Haselton, PE, PMP (Aspect) and M. Otto, EIT re: Jan Road Neighborhood Improvements – Preliminary Hand Auger Investigations. Project No. 190175-600.3. April 1, 2020.

Aspect Consulting, LLC (Aspect), 2020b, Stability Evaluation and Risk Assessment, CRT7 Revetment, Prepared for Tetra Tech, Inc., and King County Department of Natural Resources and Parks, Water and Land Resources Division, River and Floodplain Management Section, Project No. 190175-600.2 August 7, 2020, Revised September 4, 2020.

Aspect Consulting, LLC (Aspect), 2020c, Memorandum to J. Scheller, PE (Tetra Tech, Inc.) from A. Holmson, PE (Aspect) and M. Reiter, PE (Aspect) re: Rutledge-Johnson Geotechnical Exploration Summary. Project No. 190175-2100. December 18, 2020.

Booth, D.B., 1995, Surficial Geologic Map of the Maple Valley Quadrangle, King County, Washington, U.S. Geological Survey, Miscellaneous Field Studies Map MF-2297, Scale 1:24,000.

Bouwer, H. and R.C. Rice, 1976. A slug test method for determining hydraulic conductivity of unconfined aquifers with completely or partially penetrating wells, Water Resources Research, vol. 12, no. 3, pp. 423-428.

Bowles J. K. 1996, Foundation Analysis and design, fifth edition, U.S. America. : McGraw Hill Book Company.

Butler, J.J., Jr., and E.J. Garnett, 2000, Simple procedures for analysis of slug tests in formations of high hydraulic conductivity using spreadsheet and scientific graphics software, Kansas Geological Survey Open-File Rept. 2000-40, Lawrence, Ks.

City of Seattle, 2019, Landsburg Fish Passage Facilities, https://www.seattle.gov/utilities/environment-and-conservation/our-

watersheds/habitat-conservation-plan/fish-and-landsburg, Accessed September 14, 2019.

Cultural Resources Consultants (CRC), 2019, Jan Road Neighborhood Improvements Cultural Risk Assessment, Maple Valley, King County, Washington, DAHP Project #: 2019-05-03263.

Ensoft, Inc., 2019, LPILE v2019.11.08 for Windows, Analysis Program.

Historic American Engineering Record (HAER), C., Dimock, A., Morse, C. & Harvey, D. W., Shoemaker, K. B., photographer, 1968, Cedar Falls Hydroelectric Works, Cedar Falls Road, 3.5 miles south of I-90, Cedar Falls, King County, WA. Cedar Falls King County Washington, 1968, Accessed from the Library of Congress, https://www.loc.gov/item/wa0217/, September 23, 2019.

King County (County), 1962, King County Engineering Department, Flood Control Diversion Construction Drawings, Cedar River, Jan Road Revetment Extension, January 22, 1962.

King County (County), 1965, King County Engineering Department, Flood Control Diversion, Jan Road No. 2 Cedar River, Construction Drawings, August 27, 1965

King County (County), 2016a, Flood Frequency Analysis of King County Rivers with an Emphasis on the January 2009 Floods, prepared by Curtis DeGasperi, Water and Land Resources Division, Seattle, Washington.

King County (County), 2017, Mitigation Plan: Cedar River 2017 Large Wood Modifications, July 27, 2017.

King County (County), 2019a, King County iMap, Accessed September 12, 2019 from https://gismaps.kingcounty.gov/iMap/.

King County (County), 2019b, King County Hydrologic Information Center, Station Map JanRD_W1 – Jan Road Levee Well 1 Groundwater Observation Well (Recording), downloaded from https://green2.kingcounty.gov/hydrology/GaugeMetaData.aspx?G_ID=1894 on September 12, 2019.

King County (County), 2020, Preliminary Geotechnical Site Characterization Report – Revision No. 1, Jan Road Neighborhood Improvements, April 21, 2020.

Naval Facilities Engineering Command, 1986, Soil Mechanics Design Manual 7.01.

National Earthquake Hazard Mapping Program (NEHRP), 2003, NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (FEMA 450).

Massman, J., 2003, A Design Manual for Sizing Infiltration Ponds, Washington State Department of Transportation Report No. WA-RD 578.2.

Palmer, S.P., Magsino, S.L.Bilderback, E.L., Poelstra, J.L., Folger, D.S., Niggemann, R.A, (2007) Liquefaction susceptibility and site class maps of Washington State,

by county: Washington Division of Geology and Earth Resources Open File Report 2004-20.

Pile Dynamics, 2010, GRLWEAP Wave Equation Analysis program, version 2010.

Pratt, T.L., K.G. Troost, J.K. Odum, and W.J. Stephenson, 2015, Kinematics of shallow backthrusts in the Seattle fault zone, Washington State, Geosphere, v. 11, no. 6, p. 1–27, doi:10.1130/GES01179.1.

Rocscience, 2017, Slide 7.024 Analysis Program. May 2017.

Rocscience, 2019, Settle3D 4.021 Analysis Program. February 2019.

Rogers, J.D., 2006, Subsurface Exploration Using the Standard Penetration Test and the Cone Penetrometer Test, Environmental and Engineering Geoscience, May 2006.

Schuster, J.E., Cabibbo, A.A., Schilter, J.F., Hubert, I.J., 2015, Geologic Map of the Tacoma 1:100,000-scale quadrangle, Map Series 2015-03.

Tetra Tech, 2021, Jan Road Levee Setback Project Draft 100% Submittal, Cedar River, River Mile 12.9 – 13.4, 47.427 N. Lat., -122.050 W. Long.

Tetra Tech, 2022, Jan Road Levee Setback Project, Basis of Design Report, Draft 100 Percent Design, January 2022.

U.S. Army Corp of Engineers (USACE), 1990, Settlement Analysis. EM 1110-1-1904. September 30, 1990.

U.S. Army Corp of Engineers (USACE), 2000, Design and Construction of Levees. EM 1110-2-1913. April 30, 2000.

U.S. Army Corps of Engineers (USACE), 2003, Slope Stability. EM 1110-2-1902. October 31, 2003.

U.S. Army Corps of Engineers (USACE), 2005, Design Guidance for Levee Underseepage. ETL 1110-2-569. May 1, 2005.

U.S. Army Corps of Engineers (USACE), 2007, Earthquake Design and Evaluation of Concrete Hydraulic Structures, EM 1110-2-6053, May 1, 2007.

U.S. Army Corps of Engineers (USACE), 2016, Earthquake Design and Evaluation for Civil Works Projects. ER 1110-2-1806. May 31, 2016.

U.S. Army Corps of Engineers (USACE), 2019, Guidelines for Landscape Planting and Vegetation Management at Levees, Floodwalls, Embankment Dams, and Appurtenant Structures. EP 1110-2-18. May 1, 2019.

U.S. Department of Transportation Federal Highway Administration, Design and Construction of Driven Pile Foundations – Volume I and II, Publication No. FHWA-NHI-16-009, FHWA GEC 012 – Volume I and II, September 2016.

U.S. Geological Survey (USGS), 2020a, Quaternary fault and fold database for the United States, at: https://www.usgs.gov/natural-hazards/earthquake-hazards/faults.

U.S. Geological Survey (USGS), 2020b, Earthquake Hazards Program Unified Hazard Tool, at: https://earthquake.usgs.gov/hazards/interactive.

Washington Geological Survey (WGS), 2020, Landslide protocol inventory mapping--GIS data, February, 2020: Washington Geological Survey Digital Data Series 19, version 2.0, previously released January, 2019. [https://fortress.wa.gov/dnr/geologydata/publications/data_download/ger_portal_landslide_inventory.zip]

Washington State Department of Ecology (Ecology), 2021a, Abandoned Wells, https://ecology.wa.gov/Water-Shorelines/Water-supply/Wells/Information-for-drillers/Abandoned-wells, Accessed, June 29, 2021.

Washington State Department of Ecology (Ecology), 2021b, Well Construction and Licensing Tools, Well Report Viewer, Accessed June 29, 2021, https://appswr.ecology.wa.gov/wellconstruction/map/WCLSWebMap/WellLogSearchResult.aspx?imageName=00092125.pdf&region=NWRO&folder=00159&xcoord=1256623&ycoord=766411&search_scope=&result_num=10&welllogid=92125.

Washington State Department of Natural Resources (DNR), 2020, Washington Geologic Information Portal, Accessed September 9, 2020, https://geologyportal.dnr.wa.gov/.

Washington State Department of Transportation (WSDOT), 2020, Bridge Design Manual LRFD, M 23-50, September 30, 2020.

Washington State Department of Transportation (WSDOT), 2021, Standard Specifications for Road, Bridge, and Municipal Construction. M41-10.

Washington State Department of Transportation (WSDOT), 2021, Geotechnical Design Manual (GDM), M 46-03, December, 2021.

Washington State Legislature (WLS), 2008, Washington Administrative Code (WAC), Chapter 173-160, Minimum Standards for Construction and Maintenance of Wells, updated December 19, 2008.

9 Limitations

Work for this Project was performed for Tetra Tech, Inc., and the King County River and Floodplain Management Section (Client), and 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. If new information is developed, such as existing levee history and repairs, or levee performance during a significant flood event, the professional opinions, recommendations, and conclusions contained in this report may be modified by Aspect.

Levees and flood protection systems include many components like earthen embankments, filters, scour protection, and drainage elements that must work together to ensure effective performance. It is not practical to know and/or control all of the engineering properties of the components and system; therefore, inherent uncertainty about the system performance exists. Regular inspections and flood monitoring of the flood protection components and system should be performed by a qualified professional with any deficiencies mitigated appropriately.

It should be understood that some seepage through and beneath the flood protection system is normal and expected during significant flood events. Uses incompatible with this type and frequency of seepage should not be allowed in areas protected by the flood protection system. Excavations near or within the levee could compromise the system and should not be performed without adequate engineering and construction controls. Similarly, any future penetrations through or beneath the flood protection system should be assessed on a case-by-case basis by a qualified professional as penetrations can result in failures or undesirable performance if not appropriately designed.

Risks are inherent with any site involving levees and flood control structures subject to flooding and geologic hazards and no recommendations, geologic analysis, or engineering design can assure performance. 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 K 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 Andrew Holmson, PE, Senior Associate Geotechnical Engineer, (971) 865-5894.

TABLES

Table 3. Hydraulic Conductivity Estimates from Slug Tests

Project No.190175, Jan Road Neighborhood Improvements

Monitoring Well	AMW-1			AMW-2			AMW-3			
Well Depth in Feet	20.0			20.0			22.5			
Screen Length in Feet	10.0			10.0			10.0			
Depth to Screen in Feet	10.0			10.0			12.5			
Depth to Aquitard in Feet	35			20			24			
Depth to Water in Feet	5.01			3.28			4.66			
Depth to Sandpack in Feet	8.0			8.0			10.0			
Slug Displacement (Ho) in Feet	1.80	0.55	0.67	1.13	0.94	0.48	0.08	0.14	0.13	0.07
Porosity (n)	0.20			0.20			0.20			
Radius of Casing (rc) in Feet	0.08			0.08			0.08			
Radius of Borehole (rw) in Feet	0.33			0.33			0.33			
Saturated Aquifer Thickness (H) in Feet	**30.0**			**16.8**			**19.3**			
Saturated Well Thickness (Lw) in Feet	**15.0**			**16.7**			**17.8**			
Effective Radius (reff) in Feet	**0.083**			**0.083**			**0.083**			
Effective Screen Length (Le) in Feet	**10.0**			**10.0**			**10.0**			
Slug Size	*3' x1"*	*1' x1"*	*1' x1"*	*3' x1"*	*3' x1"*	*1' x1"*	*3' x1"*	*3' x1"*	*1' x1"*	*1' x1"*
Rising/Falling Head Test	*Falling*	*Rising*	*Rising*	*Falling*	*Rising*	*Falling*	*Falling*	*Rising*	*Falling*	*Rising*
Fully Submerged Sandpack	*Yes*	*Yes*	*Yes*	*Yes*	*Yes*	*Yes*	*Yes*	*Yes*	*Yes*	*Yes*
Transiently Exposed Sandpack	*No*	*No*	*No*	*No*	*No*	*No*	*No*	*No*	*No*	*No*
Transiently Exposed Screen	*No*	*No*	*No*	*No*	*No*	*No*	*No*	*No*	*No*	*No*
Partially Submerged Screen	*No*	*No*	*No*	*No*	*No*	*No*	*No*	*No*	*No*	*No*
Bouwer and Rice Analysis Parameters										
Normalized Head at t1 (y1) in Feet	**0.36**	**0.30**	**0.29**	**0.40**	**0.40**	**0.46**				
Time - t1 in Seconds	**5**	**5**	**5**	**5**	**5**	**5**	**Analyzed using Butler-Garnett**			
Normalized Head at t2 (y2) in Feet	**0.16**	**0.12**	**0.10**	**0.18**	**0.18**	**0.19**	**method for High K aquifers [b]**			
Time - t2 in Seconds	**10**	**10**	**10**	**10**	**10**	**10**				
Le/rw	**30.0**			**30.0**			**30.0**			
Calculated K in cm/sec	**4.4E-03**	**4.8E-03**	**4.8E-03**	**5.0E-03**	**4.9E-03**	**5.3E-03**	**4.4E-02**	**1.2E-01**	**1.1E-01**	**9.5E-02**
Calculated K in ft/day	**12**	**14**	**13**	**14**	**14**	**15**	**125**	**354**	**307**	**269**
Geometric Mean K in ft/day	**13**			**14**			**246**			
Geometric Mean K in ft/s	**1.53E-04**			**1.67E-04**			**2.85E-03**			
Screened Interval Soil Type	SP & SP/SM			GP w/ Sand			SP & GP			
Aquifer Geometric Mean K in ft/day	**44**									

Notes:

Data analysis by method of Bouwer and Rice (1976; 1989) or Butler-Garnett (2000).

Bold values are entered from field data and other values are calculated.

All depths are below ground surface

[a] The Bouwer and Rice A, B, and C coefficients are calculated using regression equations of Van Rooy (1988).

[b] Analyzed using methods for oscilatory water level repsonse in Butler, et al (2000).

[c] R_e/r_w is the effective radial distance over which y is dissipated, divided by the radial distance of well development.

Table 4. Soil Engineering and Hydraulic Properties

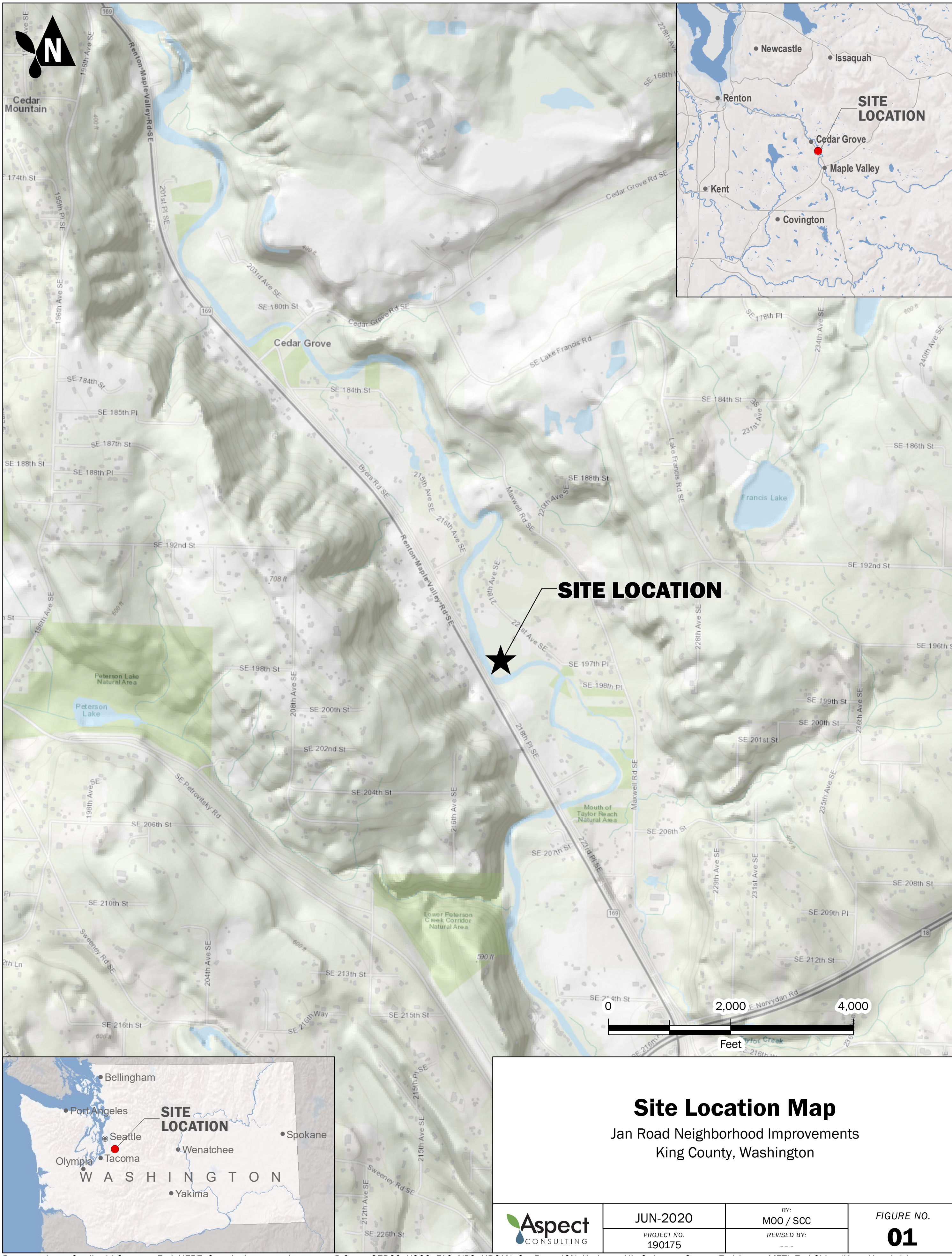
Project No. 190175, Jan Road Neighborhood Improvements

		General			Soil Parameters for Settlement Analyses							Soil Parameters for Seepage Analyses		Soil Parameters for Slope Stability Analyses			
		Soil Unit Weight		Soil Modulus, k		Consolidation Parameters[3]											
Soil Layer Number	Soil Layer	Moist, pcf[1,2]	Saturated, pcf[1,2]	Saturated, pci[1]	Elastic Modulus, ksf[1]	Cc	Cr	Cv	Cα	eo	OCR	Horizontal Hydraulic Conductivity, ft/s[1]	Vertical/Horizontal Hydraulic Conductivity Ratio (K_y/K_x)	Drained Soil Cohesion, psf[1,4]	Drained Soil Friction Angle, deg[1,4]	Undrained Shear Strength, psf[1,4]	Undrained Soil Friction Angle, deg[1,4]
1	Alluvial Deposits - Sand Facies; medium dense (SP, SW-SM, SM, ML (non-plastic))	120	125	-	160	-						2.4×10^{-3}	0.1	-	33	-	-
2	Alluvial Deposits - Gravel Facies; medium dense to dense (GW, GP, GW-GM, GP-GM)	130	135	125	540	-						1.1×10^{-3}	0.1	-	38	-	-
3	Pre-Fraser Deposits; very dense (SP, SW-SM, SP-SM, SM, ML, GP)	135	140	125	1600	-						9.4×10^{-4}	0.1	-	45	-	-
4	Levee Core Fill[(5)]	110	115	-	-	-						2.9×10^{-6}	1	100	28	500.0	-
5	Select Fill Type 1[(5)]	130	135	-	-	-						1.1×10^{-3}	1	-	34	-	-
6	Select Fill Type 2[(5)]	130	135	-	-	-						1.1×10^{-3}	1	-	36	-	-
7	Ballast Fill[(5)]	130	135									1.1×10^{-3}	1		38		
8	Common Fill[(5)]	120	125	-	-	-						1.1×10^{-3}	1	-	32	-	-
9	Riprap[(5)]	130	-	-	-	-						1.0×10^{-2}	1	-	42	-	-

Notes

1. pcf = pounds per cubic foot; ksf = kips per square foot; ft/s = feet per second; psf = pounds per square foot; deg = degrees
2. Moist unit weights were applied above the water table; saturated unit weights were applied below the water table.
3. C_c = coefficient of consolidation; C_r = Recompression Index; C_v = Time Rate of Consolidation; C_α = Coefficient of Secondary Compression; e_0 = initial void ratio; OCR = Overconsolidation Ratio. All consolidation parameters are unitless. Only applied to cohesive subsurface soils (none at this site).
4. Drained shear strength parameters were applied to all cohesionless soils in all scenarios; Undrained shear strength parameters were applied to cohesive soils in short-term loading conditions (i.e. end-of-construction, suddent drawdown, seismic)
5. Refer to Project plans and specifications for use and material requirements for fill material. Select Fill Type 2 was not used in our analyses, but typical material properties are included for reference.

FIGURES



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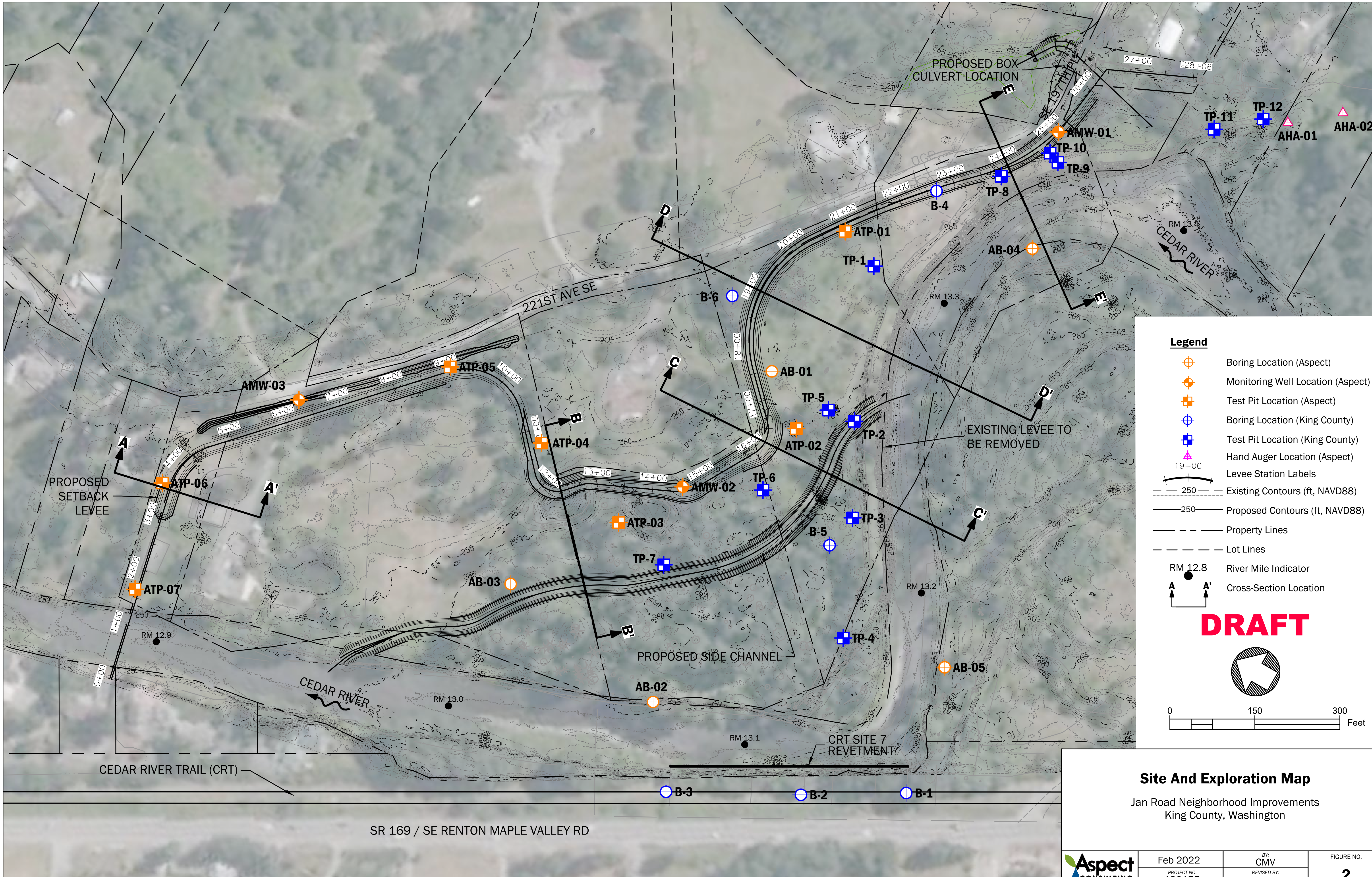
Basemap Layer Credits || Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), (c) OpenStreetMap contributors, and the GIS User Community



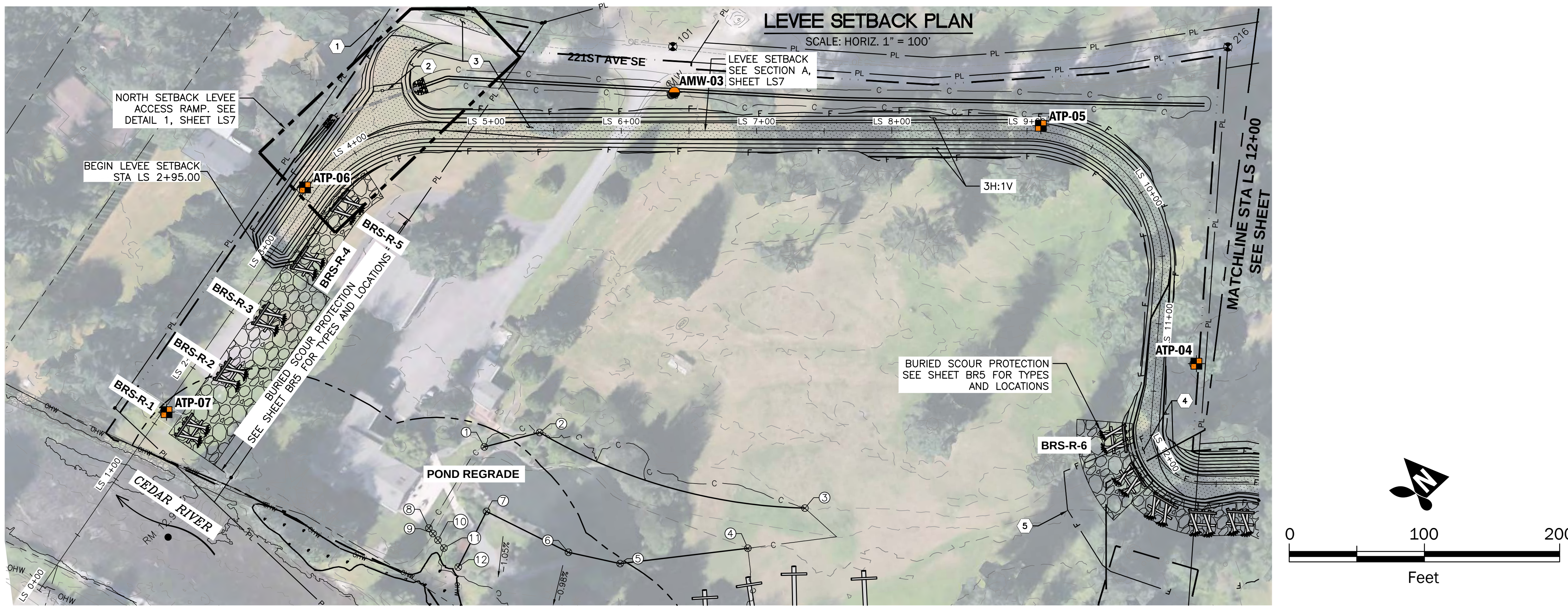
Site And Exploration Map

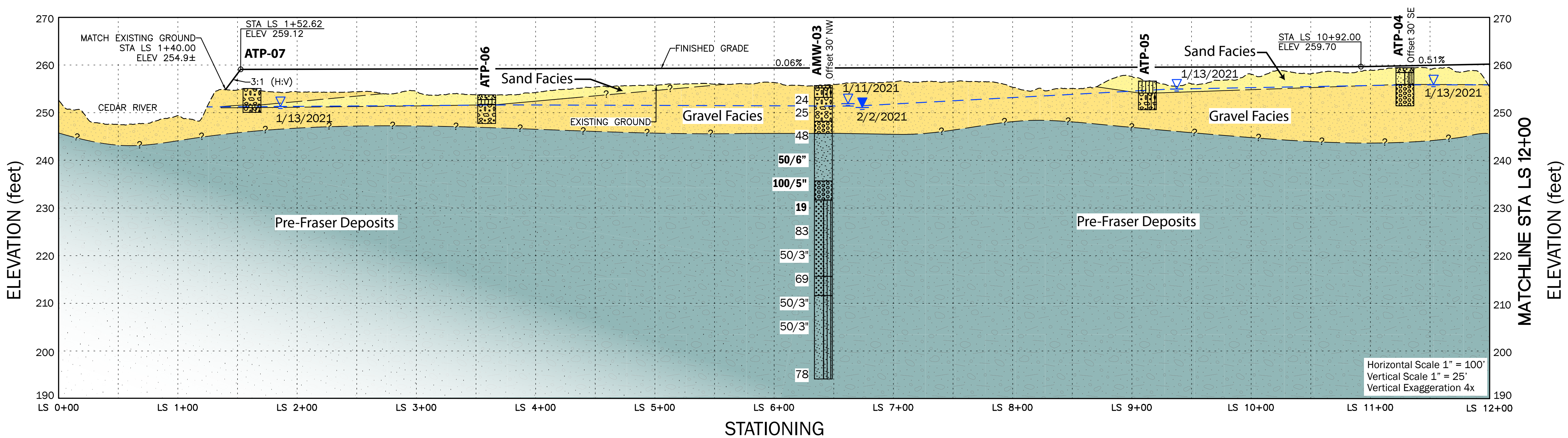
Jan Road Neighborhood Improvements
King County, Washington

Aspect CONSULTING

Feb-2022	BY: CMV	FIGURE NO.
PROJECT NO. 190175	REVISED BY: -	2

CAD Path: Q:\King County\190175 Jan Road Neighborhood Levee\2022-01 Data Memo Figures\190175-02.dwg Fig 2 || Date Saved: Feb 02, 2022 12:38pm || User: cvanslyke





LEVEE SETBACK PROFILE

Legend

- Alluvial Deposits- Sand Facies
- Alluvial Deposits- Gravel Facies
- Pre-Fraser Deposits
- Boring Location
- Monitoring Well Location
- Test Pit Location

Aspect Consulting
King County

- Contact - Known
- Contact - Inferred
- Contact - Queried
- Proposed Setback Levee Alignment Centerline and Stationing (20+00, 21+00, 22+00)
- Wet Season Groundwater Level - Inferred
- Existing Ground Surface
- Proposed Top of Setback Levee

Aspect Exploration

- Exploration ID — AB-01 (Offset 5' W) — Offset and direction
- SPT Blow Count N Value — 8
- Modified California Blow Count — **8**
- Material Type (see Appendix A)
- Water Level in Exploration at time of drilling
- Water Level in Monitoring Well, Date

Exploration By Others — B-1

DISCLAIMER: The subsurface conditions presented in this geologic cross section are conceptual. Variations may exist between the soil and groundwater conditions depicted on this figure and those actually underlying the site. Refer to the contents of this report for further context.

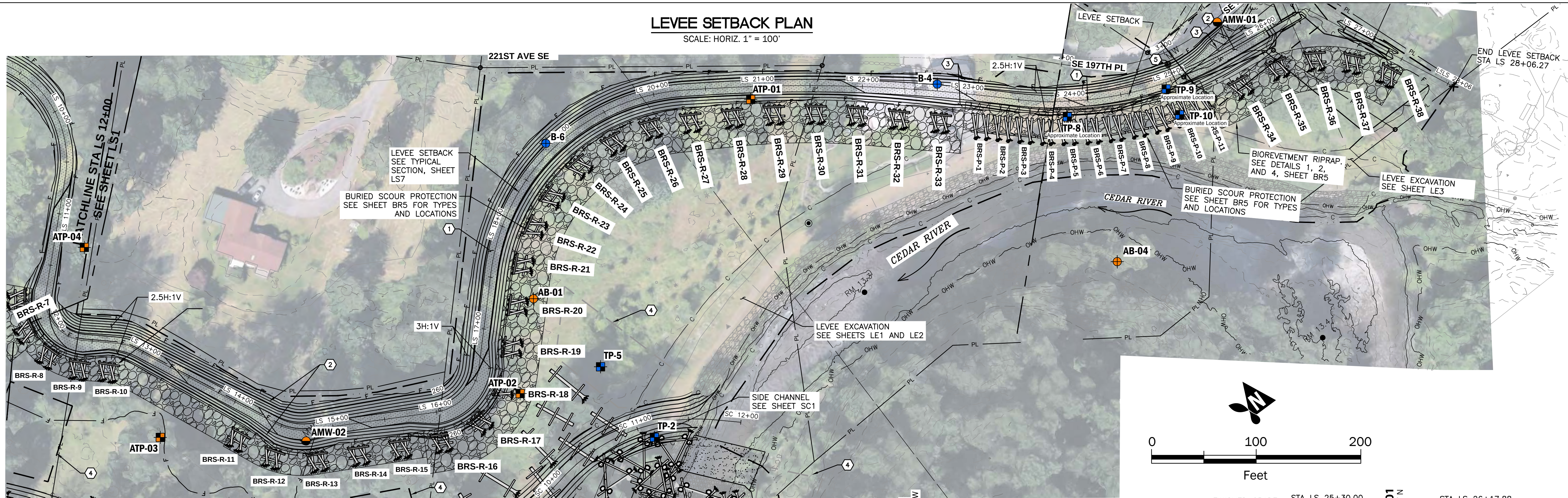
Notes:
- Cross section surface generated from CAD files provided by Tetra Tech, Inc. (Received February 2021).
- Site features are approximate.

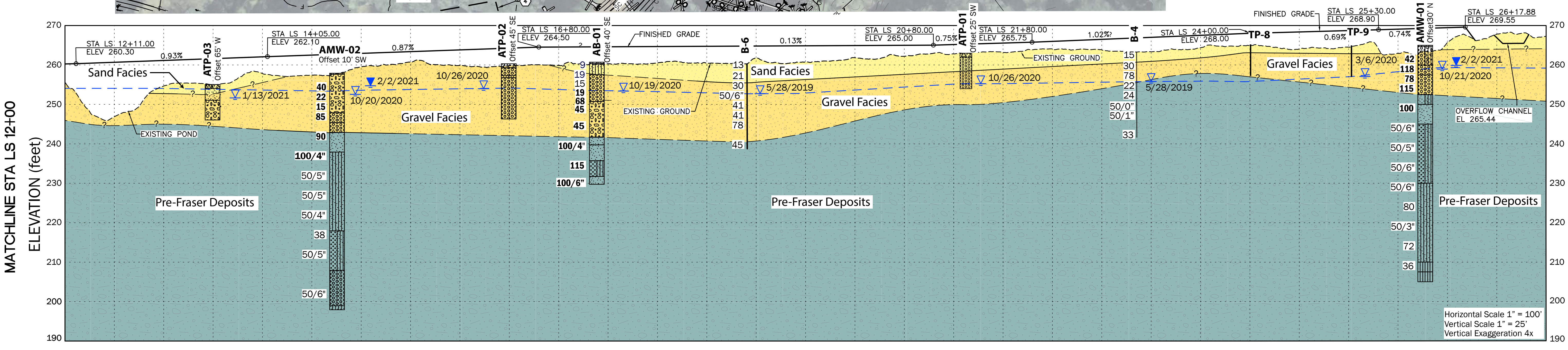
Geologic Cross Section

Jan Road Neighborhood Improvements
King County, Washington

DRAFT

Aspect CONSULTING	JAN-2022	BY: JBM	FIGURE NO.
	PROJECT NO. 190175	REVISED BY: MO/MBR/MS	03





Legend

- Alluvial Deposits-Sand Facies
- Alluvial Deposits-Gravel Facies
- Pre-Fraser Deposits
- Boring Location
- Monitoring Well Location
- Test Pit Location
- Aspect Consulting
- King County
- Contact - Known
- Contact - Inferred
- Contact - Queried
- Proposed Setback Levee Alignment Centerline and Stationing
- Wet Season Groundwater Level - Inferred
- Existing Ground Surface
- Proposed Top of Setback Levee

Aspect Exploration: Exploration ID; Offset and direction; SPT Blow Count N Value; Modified California Blow Count; Material Type (see Appendix A); Water Level in Exploration at time of drilling; Water Level in Monitoring Well, Date

Exploration By Others

DISCLAIMER: The subsurface conditions presented in this geologic cross section are conceptual. Variations may exist between the soil and groundwater conditions depicted on this figure and those actually underlying the site. Refer to the contents of this report for further context.

Notes:
- Cross section surface generated from CAD files provided by Tetra Tech, Inc. (Received February 2021).
- Site features are approximate.

Geologic Cross Section

Jan Road Neighborhood Improvements
King County, Washington

DRAFT

Aspect CONSULTING

JAN-2022	BY: JBM	FIGURE NO.
PROJECT NO. 190175	REVISED BY: MO/MBR/MS	04

APPENDIX A

Subsurface Explorations

A. Subsurface Explorations

Aspect's field exploration program consisted of eight machine-drilled borings and seven test pits. Logs are presented in this appendix and locations of the explorations are shown on Figure 2.

An Aspect engineer was present throughout the field exploration program to observe the drilling or excavation procedures, collect soil samples, and prepare descriptive logs of each exploration. Soils were classified in general accordance with ASTM International (ASTM) D2488, *Standard Practice for Description and Identification of Soils* (Visual-Manual Procedure)(ASTM, 2018). The summary exploration log represents our interpretation of the contents of the field logs. The stratigraphic contacts shown on the summary log represents the approximate boundaries between soil types; actual transitions may be more gradual. The subsurface conditions depicted are only for the specific date and location reported; therefore, are not necessarily representative of other locations and times.

A.1. Drilled Soil Borings

Aspect subcontracted with Holocene Drilling, Inc, an experienced and licensed local driller to drill eight soil borings, three of which were installed with wells (AB-01 through AB-05 and AMW-01 through AMW-03) using sonic drilling techniques. Drilling was completed over two mobilization periods: the first occurred between October 19 and 23, 2020, and the second over January 11 and 12, 2021. Drilling was completed with a track-mounted GeoProbe 8140 LC rig advancing 6-inch-diameter sonic casing, with continuous soil sampling. The borings were advanced to depths of between approximately 20 to 60 feet bgs.

Soil sampling in the drilled borings was completed continuously from the core barrel as needed, and at select depth intervals using standard penetration test methods in accordance with ASTM Method D1586, *Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils* (ASTM, 2018). The SPT method involves driving a 2-inch-outside-diameter split-barrel sampler with a 140-pound hammer free-falling from a distance of 30 inches[2]. The number of blows for each 6-inch interval is recorded, and the number of blows required to drive the sampler (2-inch-outside-diameter) the final 12 inches is known as the Standard Penetration Resistance ("N") or blow count. The resistance, or N-value, provides a measure of the relative density of granular soils or the relative consistency of cohesive soils. If a total of 50 blows are recorded for a single 6-inch interval, the test is terminated, and the blow count is recorded as 50 blows for the total inches of penetration. Samples were placed in labeled plastic jars for geologic review and laboratory analysis.

[2] Due to the presence of oversized particles, a modified California sampler (3-inch-outside-diameter) was also used with a 140-pound hammer free-falling from a distance of 30 inches. The blow counts were correlated to standard N-values using an empirical correction factor from Rogers (2006).

Upon completion, borings AB-01 through AB-05 were backfilled with hydrated 3/8-inch bentonite chips in accordance with requirements of the Washington State Department of Ecology. AMW-01 through AMW-03 were equipped with groundwater monitoring well installations consisting of 2-inch-diameter threaded Schedule 40, polyvinyl chloride (PVC) solid-wall casing and 0.020-inch (20 slot) screen for the screened intervals. A sand filter pack was installed around the screened intervals. A concrete surface seal with a metal, flush-mount monument was installed at the ground surface to protect each well.

A.2. Excavated Test Pits

Aspect completed seven excavated test pit explorations (ATP-01 through ATP-07) on October 26, 2020, and January 13, 2021. The County provided an equipment operator, as well as a John Deere 50G mini-excavator (October 26, 2020) and John Deere 85G excavator (January 13, 2021) to complete the work. The relative density/consistency of the soils was evaluated qualitatively with a 0.5-inch-diameter steel T probe[3] and observation of digging difficulty. Relative density was quantitatively assessed with Dynamic Cone Penetrometer Testing (DCPT) at various depth intervals within the test pits. The test pits were backfilled with the excavated soils.

The DCPT method involves a 15-pound steel mass falling 20 inches to strike an anvil, which drives a 1.5 inch-diameter, 45-degree cone into the soil. The number of blows required to drive the cone 1.75 inches is considered one data point. The DCPT data has been calibrated with Standard Penetration Test (SPT; ASTM Method D1586) results to provide a more refined estimate of soil relative density and consistency.

[3] The T-probe data is a measure of soil density or consistency. T-probe measurements are presented in inches, which refers to the depth to which a ½-inch-diameter steel probe can be pushed into a relatively undisturbed, flat soil surface.

Major Division	Subdivision	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

Density³		*SPT² 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

Consistency³		*SPT² 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

Jan Rd Neighborhood Improvements - 190175 — Geotechnical Exploration Log

Aspect CONSULTING

Project Address & Site Specific Location	Coordinates (Lat,Lon WGS84)	Exploration Number
Cedar River near RM 13.0 - RM 13.4, See Figure.	47.4256, -122.0486 (est)	**AB-01**

Contractor	Equipment	Sampling Method	Ground Surface Elev.(NAVD88)
Holocene Drilling, Inc.	GeoProbe 8140 LC	Autohammer; 140 lb hammer; 30" drop	260' (est)

Operator	Exploration Method(s)	Work Start/Completion Dates	Top of Casing Elev. (NAVD88)	Depth to Water (Below GS)
Cory	Sonic	10/19/2020	NA	7.5' (ATD)

Depth (feet)	Elev. (feet)	Exploration Notes and Completion Details	Sample Type/ID	Blows/6"	Tests	Description
0	260	Boring backfilled with bentonite chips	S0	2, 5, 4		**TOPSOIL** ORGANIC CLAY WITH GRAVEL (OH); Stiff, moist, dark brown; mostly non-plastic silt; some organics.
						ALLUVIAL DEPOSITS - SAND FACIES SILTY SAND (SM); Medium dense, moist, brown; fine to medium sand; trace rootlets and wood fragments.
			G1		PS, MC	
			S1	12, 12, 7		**ALLUVIAL DEPOSITS - GRAVEL FACIES** GRAVEL WITH SAND (GP); Medium dense, moist, brown; fine to coarse, subrounded to subangular gravel; fine to coarse sand; trace silt.
5	255			8, 7, 8		
		10/19/2020 Approximate (water level)	S2		Blows (non-SPT) =10,8,11	
			S3		Blows (non-SPT) =30,38,30 PS, MC	Becomes dense, wet.
10	250		S4		Blows (non-SPT) =11,15,30	GRAVEL WITH SAND (GW); Medium dense, wet, gray brown; fine to coarse, subrounded to angular gravel; medium to coarse sand; trace silt.
			G2		PS, MC	
15	245		S5		Blows (non-SPT) =17,22,23 PS, MC	
						PRE-FRASER DEPOSITS SAND WITH GRAVEL (SP); Very dense, wet, gray brown; fine to coarse sand; fine to coarse, subrounded to subangular gravel; trace silt.
20	240		S6		Blows (non-SPT) =12,100/4" PS, MC	

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

Plastic Limit ⊢—⊣ Liquid Limit

Legend

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

Water Level: Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: M. Otto
Approved by: A. Holmson

Exploration Log AB-01

Jan Rd Neighborhood Improvements - 190175			Geotechnical Exploration Log	
Project Address & Site Specific Location: Cedar River near RM 13.0 - RM 13.4, See Figure.			Coordinates (Lat,Lon WGS84): 47.4256, -122.0486 (est)	Exploration Number: AB-01
Contractor: Holocene Drilling, Inc.	Equipment: GeoProbe 8140 LC	Sampling Method: Autohammer; 140 lb hammer; 30" drop	Ground Surface Elev. (NAVD88): 261' (est)	
Operator: Cory	Exploration Method(s): Sonic	Work Start/Completion Dates: 10/19/2020	Top of Casing Elev. (NAVD88): NA	Depth to Water (Below GS): 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
								PRE-FRASER DEPOSITS SAND WITH GRAVEL (SP); Very dense, wet, gray brown; fine to coarse sand; fine to coarse, subrounded to subangular gravel; trace silt. (continued)
25	235		S7			Blows (non-SPT) =25,50,65 PS, MC		SAND WITH SILT (SW-SM); Very dense, wet, red brown; fine to coarse sand; few fine, subangular gravel; subtrace wood fragments.
			G3			PS, MC		SAND WITH GRAVEL (SP); Very dense, wet, red brown; fine to coarse sand; fine, subangular gravel; trace silt.
30	230	Driller reported heave	S8			Blows (non-SPT) =23,100/6"		
31								Bottom of exploration at 31 ft. bgs.
35	225							
40	220							

Legend

Sample Type: No Soil Sample Recovery; Split Barrel 2" X 1.375" (SPT); Grab sample; 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: M. Otto
Approved by: A. Holmson

Exploration Log AB-01

Aspect CONSULTING	Jan Rd Neighborhood Improvements - 190175			Geotechnical Exploration Log	
	Project Address & Site Specific Location			*Coordinates (Lat,Lon WGS84)*	*Exploration Number*
	Cedar River near RM 13.0 - RM 13.4, See Figure.			47.4253, -122.0511 (est)	**AB-02**
Contractor	*Equipment*	*Sampling Method*		*Ground Surface Elev. (NAVD88)*	
Holocene Drilling, Inc.	GeoProbe 8140 LC	Autohammer; 140 lb hammer; 30" drop		257' (est)	
Operator	*Exploration Method(s)*	*Work Start/Completion Dates*		*Top of Casing Elev. (NAVD88)*	*Depth to Water (Below GS)*
Cory	Sonic	10/19/2020		NA	5' (ATD)

Depth (feet)	Elev. (feet)	Exploration Notes and Completion Details	Sample Type/ID	Blows/foot ▲ Water Content (%) ●	Blows/6"	Tests	Material Type	Description	Depth (ft)
0	257							**TOPSOIL** TOPSOIL; mostly grass and rootmass.	
		Boring backfilled with bentonite chips						**ALLUVIAL DEPOSITS - GRAVEL FACIES** GRAVEL WITH SAND (GW); Medium dense, moist, gray brown; fine to coarse, subrounded to subangular gravel; fine to coarse sand; subtrace rootlets.	
	255								
			S1			Blows (non-SPT) =9,11,13			
			G1			PS, MC			
5		∇ 10/19/2020 Approximate	S2			Blows (non-SPT) =11,20,28		Becomes dense, wet.	5
	250								
			G2			PS, MC		GRAVEL WITH SAND (GP); Very dense, wet, gray brown; fine to coarse, subrounded to subangular gravel; fine to coarse sand; trace silt.	
10			S3			Blows (non-SPT) =30,38,52 PS, MC			10
	245								
15			S4			Blows (non-SPT) =18,42,37		Becomes dense, dark brown.	15
	240							**PRE-FRASER DEPOSITS** SAND (SP); Very dense, wet, brown; fine to medium sand; trace silt.	
			G3			PS, MC			
20		Driller reported heave and added water to the borehole	S5			Blows (non-SPT) =32,69,48		SILTY SAND (SM); Very dense, wet, brown; fine to coarse sand; few fine, subrounded to subangular gravel.	20

Legend

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

Plastic Limit ⊢—⊣ Liquid Limit

Water Level
- ∇ Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: M. Otto
Approved by: A. Holmson

Exploration Log AB-02

Sheet 1 of 2

Jan Rd Neighborhood Improvements - 190175

Geotechnical Exploration Log

Project Address & Site Specific Location	Coordinates (Lat,Lon WGS84)	Exploration Number
Cedar River near RM 13.0 - RM 13.4, See Figure.	47.4253, -122.0511 (est)	AB-02

Contractor	Equipment	Sampling Method	Ground Surface Elev. (NAVD88)
Holocene Drilling, Inc.	GeoProbe 8140 LC	Autohammer; 140 lb hammer; 30" drop	257' (est)

Operator	Exploration Method(s)	Work Start/Completion Dates	Top of Casing Elev. (NAVD88)	Depth to Water (Below GS)
Cory	Sonic	10/19/2020	NA	5' (ATD)

Depth (feet)	Elev. (feet)	Sample Type/ID	Blows/6"	Tests	Description
					SILTY SAND (SM); Very dense, wet, brown; fine to coarse sand; few fine, subrounded to subangular gravel. (continued)
25		S6	32 45 40	PS, MC	
	230				Bottom of exploration at 26.5 ft. bgs.
30	225				
35	220				
40	215				

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

Legend

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

Plastic Limit ├──┤ Liquid Limit

Water Level: ▽ Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: M. Otto
Approved by: A. Holmson

Exploration Log AB-02

Sheet 2 of 2

Aspect CONSULTING	Jan Rd Neighborhood Improvements - 190175		Geotechnical Exploration Log	
	Project Address & Site Specific Location		*Coordinates (Lat,Lon WGS84)*	*Exploration Number*
	Cedar River near RM 13.0 - RM 13.4, See Figure.		47.4262, -122.0509 (est)	**AB-03**
Contractor	*Equipment*	*Sampling Method*	*Ground Surface Elev. (NAVD88)*	
Holocene Drilling, Inc.	GeoProbe 8140 LC	Autohammer; 140 lb hammer; 30" drop	256' (est)	
Operator	*Exploration Method(s)*	*Work Start/Completion Dates*	*Top of Casing Elev. (NAVD88)*	*Depth to Water (Below GS)*
Cory	Sonic	1/12/2021	NA	4' (ATD)

Depth (feet)	Elev. (feet)	Exploration Notes and Completion Details	Sample Type/ID	Blows/foot ▲ Water Content (%) ●	Blows/6"	Tests	Description
0	256	Boring backfilled with bentonite chips					**TOPSOIL** TOPSOIL; Very loose, slightly moist, dark brown; mostly non-plastic silt, few fine sand; abundant organics and rootlets.
1							**ALLUVIAL DEPOSITS - SAND FACIES** SAND WITH SILT AND GRAVEL (SW-SM); Medium dense, wet, gray brown; fine to coarse sand; fine to coarse, subrounded to rounded gravel.
2.5–3.5			S1		7, 7, 8		
4		1/12/2021 Approximate (water level)					
5	251		S2			Blows (non-SPT) =28,38,30 PS, MC FC=2%	**ALLUVIAL DEPOSITS - GRAVEL FACIES** GRAVEL WITH SAND (GP); Dense, wet, gray brown; fine to coarse, subrounded to rounded gravel; fine to coarse sand; trace silt.
7.5			S3			Blows (non-SPT) =20,28,30	
10	246		S4			Blows (non-SPT) =21,34,45	Becomes very dense.
12.5			G1			PS, MC FC=1.1%	GRAVEL (GW); Very dense, wet, gray brown; fine to coarse, subrounded to rounded gravel; few fine to coarse sand; trace silt.
15	241		S5			Blows (non-SPT) =36,44,50/5"	**PRE-FRASER DEPOSITS** SAND WITH SILT AND GRAVEL (SW-SM); Very dense, very moist, dark brown; fine to coarse sand; fine, subrounded to rounded gravel.
20	236	Driller reported heave and added water to the borehole	S6			Blows (non-SPT) =32,45,50	

Legend

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

Plastic Limit ├─┤ Liquid Limit

Water Level: ∇ Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: M. Reiter
Approved by: A. Holmson

Exploration Log AB-03

Sheet 1 of 2

Project Address & Site Specific Location	Coordinates (Lat,Lon WGS84)	Exploration Number
Cedar River near RM 13.0 - RM 13.4, See Figure.	47.4262, -122.0509 (est)	**AB-03**

Contractor	Equipment	Sampling Method	Ground Surface Elev. (NAVD88)
Holocene Drilling, Inc.	GeoProbe 8140 LC	Autohammer; 140 lb hammer; 30" drop	256' (est)

Operator	Exploration Method(s)	Work Start/Completion Dates	Top of Casing Elev. (NAVD88)	Depth to Water (Below GS)
Cory	Sonic	1/12/2021	NA	4' (ATD)

Depth (feet)	Elev. (feet)	Sample Type/ID	Water Content (%) / Blows/foot	Tests	Description
					PRE-FRASER DEPOSITS SAND WITH SILT AND GRAVEL (SW-SM); Very dense, very moist, dark brown; fine to coarse sand; fine, subrounded to rounded gravel. (continued)
25		G2 (Grab)	● ~7	PS, MC FC=6.3%	
	230				
30		S7	▲ >50	Blows (non-SPT) =49,100/6"	
	225				
35		S8	● ~8; ▲ >50	Blows (non-SPT) =12,45,40 PS, MC FC=6.1%	SAND WITH SILT AND GRAVEL (SP-SM); Very dense, very moist, dark brown to dark gray; fine to coarse sand; fine to coarse subrounded to subangular gravel.
	220				
40		S9	▲ >50	Blows (non-SPT) =16,49,100/5"	Becomes dark gray.
	215				
41.5					Bottom of exploration at 41.5 ft. bgs.

Legend

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

Plastic Limit ├──┤ Liquid Limit

Water Level: ▽ Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: M. Reiter
Approved by: A. Holmson

Exploration Log AB-03

Jan Rd Neighborhood Improvements - 190175

Geotechnical Exploration Log

Aspect CONSULTING

Project Address & Site Specific Location	Coordinates (Lat,Lon WGS84)	Exploration Number
Cedar River near RM 13.0 - RM 13.4, See Figure.	47.4248, -122.0469 (est)	AB-04

Contractor	Equipment	Sampling Method	Ground Surface Elev. (NAVD88)
Holocene Drilling, Inc.	GeoProbe 8140 LC	Autohammer; 140 lb hammer; 30" drop	262' (est)

Operator	Exploration Method(s)	Work Start/Completion Dates	Top of Casing Elev. (NAVD88)	Depth to Water (Below GS)
Cory	Sonic	10/23/2020	NA	4.8' (ATD)

Depth (feet)	Elev. (feet)	Exploration Notes and Completion Details	Sample Type/ID	Blows/6"	Tests	Description
0	262	Boring backfilled with bentonite chips				**ALLUVIAL DEPOSITS - SAND FACIES** SILTY SAND (SM); Loose, moist, brown; fine sand.
0.5						**ALLUVIAL DEPOSITS - GRAVEL FACIES** GRAVEL WITH SAND, COBBLES, AND BOULDERS (GP); Medium dense, very moist, dark brown to gray; fine to coarse, subrounded to rounded gravel; fine to coarse sand; few subrounded to rounded, 4-inch cobbles and 12-inch boulders; trace silt.
2.5	260		G1 (grab)		PS, MC	
4.8		∇ 10/23/2020				
5			S1 (3" split barrel)		Blows (non-SPT) =21,15,14 PS, MC	GRAVEL WITH SILT AND SAND (GP-GM); Medium dense, wet, gray brown; fine to coarse, subrounded to rounded gravel; fine to coarse sand.
7.5	255		S2 (3" split barrel)		Blows (non-SPT) =14,7,8 PS, MC	**ALLUVIAL DEPOSITS - SAND FACIES** SAND WITH GRAVEL (SP); Loose, wet, gray brown; fine to coarse sand; fine to coarse, subrounded to rounded gravel; trace silt.
10			S3 (SPT)	17, 15, 11	PS, MC	Becomes medium dense, dark brown.
12.5	250		G2 (grab)		PS, MC	**ALLUVIAL DEPOSITS - GRAVEL FACIES** GRAVEL WITH SAND AND COBBLES (GW); Medium dense, wet, brown; fine to coarse, subrounded to rounded gravel; fine to coarse sand; few 4-inch, subrounded to rounded cobbles.
15		Driller reported heave and added water to the borehole	S4 (SPT)	25, 30, 36	PS, MC	**PRE-FRASER DEPOSITS** SAND WITH SILT AND GRAVEL (SW-SM); Very dense, wet, dark gray; fine to coarse sand; fine, subrounded gravel.
17.5	245					
20			S5 (SPT)	30, 50/5"		
21						Bottom of exploration at 21 ft. bgs.

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

Legend

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

Plastic Limit ├──┤ Liquid Limit

Water Level: ∇ Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: M. Reiter
Approved by: A. Holmson

Exploration Log AB-04

Sheet 1 of 1

Jan Rd Neighborhood Improvements - 190175

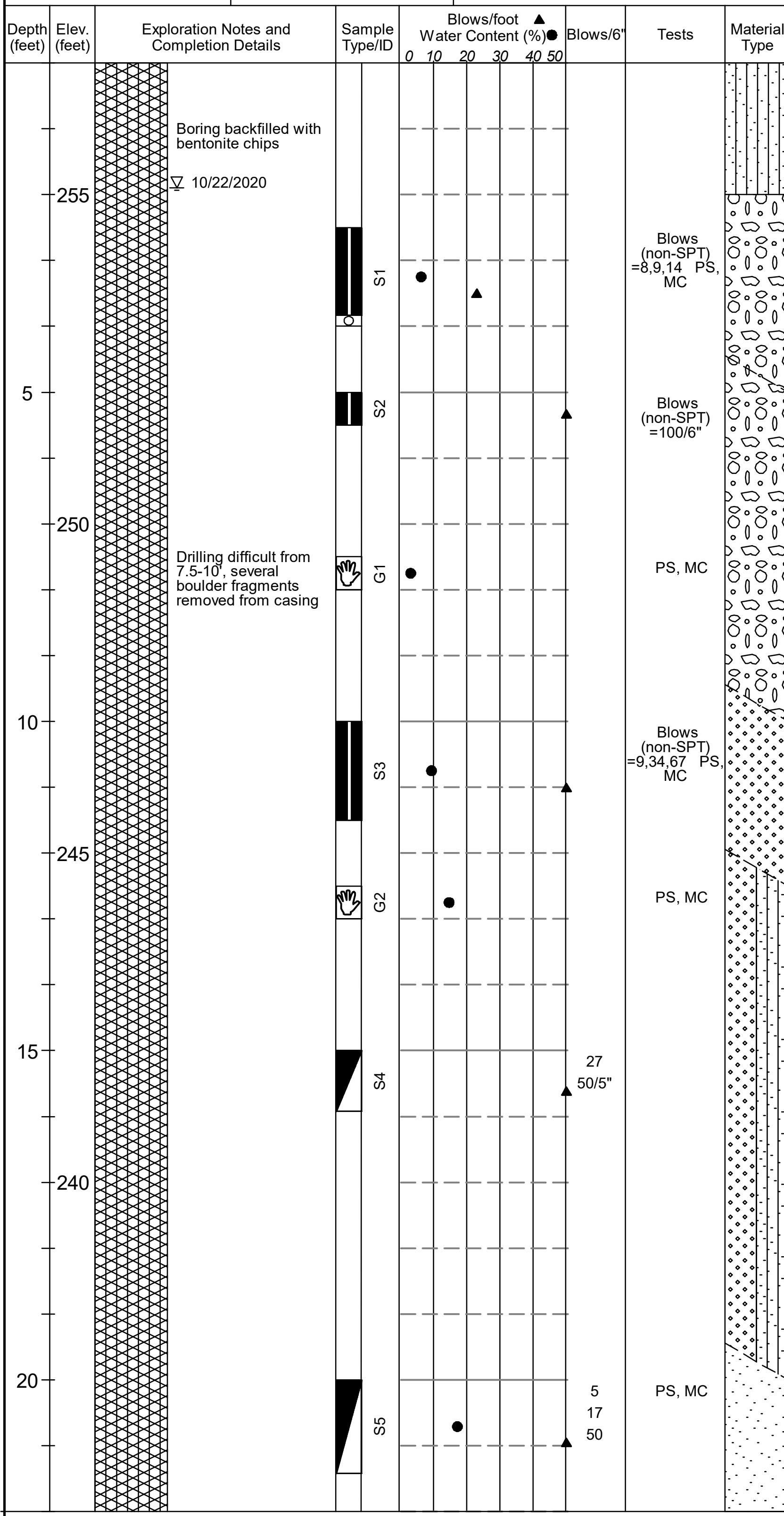
Geotechnical Exploration Log

Aspect CONSULTING

Project Address & Site Specific Location	Coordinates (Lat,Lon WGS84)	Exploration Number
Cedar River near RM 13.0 - RM 13.4, See Figure.	47.4242, -122.0498 (est)	AB-05

Contractor	Equipment	Sampling Method	Ground Surface Elev. (NAVD88)
Holocene Drilling, Inc.	GeoProbe 8140 LC	Autohammer; 140 lb hammer; 30" drop	257' (est)

Operator	Exploration Method(s)	Work Start/Completion Dates	Top of Casing Elev. (NAVD88)	Depth to Water (Below GS)
Cory	Sonic	10/22/2020	NA	1.9' (ATD)



ALLUVIAL DEPOSITS - SAND FACIES
SILTY SAND (SM); Loose, moist, brown with some red mottling; fine sand; some organics.

ALLUVIAL DEPOSITS - GRAVEL FACIES
GRAVEL WITH SAND (GW); Medium dense, very moist, gray brown; fine to coarse, subrounded to rounded gravel; medium to coarse sand.

GRAVEL WITH COBBLES AND BOULDERS (GW); Medium dense, very moist, gray brown; fine to coarse, subrounded to rounded gravel; some 6-inch, rounded to subrounded cobbles and 12-inch, rounded to subrounded boulders; little fine to coarse sand; trace silt. Blow count may be overstated due to presence of cobbles and boulders.

PRE-FRASER DEPOSITS
SAND WITH GRAVEL (SW); Very dense, wet, dark brown to gray; fine to coarse sand; fine to coarse, subrounded to rounded gravel; trace silt.

SAND WITH SILT AND GRAVEL (SW-SM); Very dense, very moist, dark brown to brown; fine to coarse sand; fine, subrounded to rounded gravel.

SAND (SP); Very dense, very moist, dark brown to brown; fine to coarse sand; few to little fine, subrounded to subangular gravel; trace silt.

Legend

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

Plastic Limit ├──┤ Liquid Limit

Water Level: Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: M. Reiter
Approved by: A. Holmson

Exploration Log AB-05

Jan Rd Neighborhood Improvements - 190175

Geotechnical Exploration Log

Aspect CONSULTING

Project Address & Site Specific Location	Coordinates (Lat,Lon WGS84)	Exploration Number
Cedar River near RM 13.0 - RM 13.4, See Figure.	47.4242, -122.0498 (est)	AB-05

Contractor	Equipment	Sampling Method	Ground Surface Elev. (NAVD88)
Holocene Drilling, Inc.	GeoProbe 8140 LC	Autohammer; 140 lb hammer; 30" drop	257' (est)

Operator	Exploration Method(s)	Work Start/Completion Dates	Top of Casing Elev. (NAVD88)	Depth to Water (Below GS)
Cory	Sonic	10/22/2020	NA	1.9' (ATD)

Depth (feet)	Elev. (feet)	Exploration Notes and Completion Details	Sample Type/ID	Blows/foot ▲ Water Content (%) ●	Blows/6"	Tests	Material Type	Description	Depth (ft)
								SAND WITH GRAVEL (SP); Very dense, very moist, dark brown to brown; fine to coarse sand; few to little fine, subrounded to subangular gravel; trace silt.	
25		Driller reported heave	S6	● ▲	44 50/4"	PS, MC		Becomes dark gray.	25
								Bottom of exploration at 25.8 ft. bgs.	
	230								
30									30
	225								
35									35
	220								
40									40
	215								

Legend

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

Plastic Limit ├──┤ Liquid Limit

Water Level: ∇ Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: M. Reiter
Approved by: A. Holmson

Exploration Log AB-05

Aspect Consulting	Jan Rd Neighborhood Improvements - 190175			Geotechnical Exploration Log	
	Project Address & Site Specific Location			*Coordinates (Lat,Lon WGS84)*	*Exploration Number*
	Cedar River near RM 13.0 - RM 13.4, See Figure.			47.4250, -122.0461 (est)	**AMW-01**
Contractor	*Equipment*	*Sampling Method*		*Ground Surface Elev. (NAVD88)*	Ecology Well Tag No. BMP853
Holocene Drilling, Inc.	GeoProbe 8140 LC	Autohammer; 140 lb hammer; 30" drop		265' (est)	
Operator	*Exploration Method(s)*	*Work Start/Completion Dates*		*Top of Casing Elev. (NAVD88)*	*Depth to Water (Below GS)*
Cory	Sonic	10/21/2020 to 10/22/2020		264' (est)	6.25' (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)
0	265	Well No. BMP853, flush monument set in concrete						**FILL** GRAVEL WITH SAND (GW); 4 inches crushed aggregate roadway surfacing.	
		2" PVC pipe set in concrete						**ALLUVIAL DEPOSITS - GRAVEL FACIES** GRAVEL WITH SILT AND SAND (GP-GM); Medium dense, dry to slightly moist, brown; fine to coarse, rounded to subrounded gravel; fine to coarse sand; trace wood fragments.	
			S1			Blows (non-SPT) =8,17,25 PS, MC			
5	260	2" PVC pipe set in bentonite chips	S2			Blows (non-SPT) =28,50,68 PS, MC		GRAVEL WITH SAND (GP); Very dense, dry to slightly moist, brown to gray; fine to coarse, subrounded to rounded gravel; fine to coarse sand; trace silt.	5
		▽ 10/21/2020 Approximate							
		2" PVC pipe set in sand	S3			Blows (non-SPT) =45,50,28 PS, MC		GRAVEL WITH SAND AND COBBLES (GP); Dense, wet, gray brown; fine to coarse, subrounded to rounded gravel; fine to coarse sand; trace silt; trace 4-inch, rounded to subrounded cobbles.	
10	255		S4			Blows (non-SPT) =22,30,85 PS, MC		Becomes with few to some 6-inch, rounded to subrounded cobbles.	10
			G1			PS, MC		**PRE-FRASER DEPOSITS** SAND WITH SILT (SP-SM); Very dense, moist, light brown to brown; fine to medium sand; few fine, subrounded gravel.	
15	250	2" slotted PVC pipe set in sand. 20 slot screen.	S5			Blows (non-SPT) =36,45,55 PS, MC		SAND (SP); Very dense, moist, light brown to brown; fine to medium sand; few fine, subrounded gravel; trace silt.	15
20	245	2' of sand placed beneath PVC pipe	S6		30 50/6"	PS, MC		SAND WITH SILT (SW-SM); Very dense, moist, gray to dark gray; fine to coarse sand; few to little fine, subrounded gravel.	20

Legend

Plastic Limit ├──┤ Liquid Limit

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

Water Level:
- ▽ Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: M. Reiter
Approved by: A. Holmson

Exploration Log AMW-01

Jan Rd Neighborhood Improvements - 190175

Geotechnical Exploration Log

Aspect CONSULTING

Project Address & Site Specific Location			Coordinates (Lat,Lon WGS84)	Exploration Number
Cedar River near RM 13.0 - RM 13.4, See Figure.			47.4250, -122.0461 (est)	**AMW-01**
Contractor	**Equipment**	**Sampling Method**	**Ground Surface Elev. (NAVD88)**	Ecology Well Tag No.
Holocene Drilling, Inc.	GeoProbe 8140 LC	Autohammer; 140 lb hammer; 30" drop	265' (est)	BMP853
Operator	**Exploration Method(s)**	**Work Start/Completion Dates**	**Top of Casing Elev. (NAVD88)**	**Depth to Water (Below GS)**
Cory	Sonic	10/21/2020 to 10/22/2020	264' (est)	6.25' (ATD)

Depth (feet)	Elev. (feet)	Exploration Notes and Completion Details	Sample Type/ID	Blows/6"	Tests	Description
		Borehole backfilled with bentonite chips				SAND WITH SILT (SW-SM); Very dense, moist, gray to dark gray; fine to coarse sand; few to little fine, subrounded gravel. (continued)
25	240	Driller reported heave and added water to the borehole	S7	10, 26, 50/5"	PS, MC	Becomes dark gray.
30	235		S8	30, 50/6"		
35	230		S9	50/6"		SILTY SAND (SM); Very dense, very moist, dark gray; fine to coarse sand; few fine, subrounded gravel.
40	225		S10	27, 30, 50	PS, MC	

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

Plastic Limit ├──┤ Liquid Limit

Legend

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

Water Level:
- Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: M. Reiter
Approved by: A. Holmson

Exploration Log AMW-01

Jan Rd Neighborhood Improvements - 190175

Geotechnical Exploration Log

Aspect CONSULTING	Project Address & Site Specific Location		Coordinates (Lat,Lon WGS84)	Exploration Number
	Cedar River near RM 13.0 - RM 13.4, See Figure.		47.4250, -122.0461 (est)	**AMW-01**
Contractor	Equipment	Sampling Method	Ground Surface Elev. (NAVD88)	
Holocene Drilling, Inc.	GeoProbe 8140 LC	Autohammer; 140 lb hammer; 30" drop	265' (est)	Ecology Well Tag No. BMP853
Operator	Exploration Method(s)	Work Start/Completion Dates	Top of Casing Elev. (NAVD88)	Depth to Water (Below GS)
Cory	Sonic	10/21/2020 to 10/22/2020	264' (est)	6.25' (ATD)

Depth (feet)	Elev. (feet)	Sample Type/ID	Blows/6"	Tests	Description
44	221				SILTY SAND (SM); Very dense, very moist, dark gray; fine to coarse sand; few fine, subrounded gravel. (continued)
45	220	S11	50/3"		
50	215	S12	40 30 42	PS, MC	
55	210	S13	20 20 16	PS, MC	SANDY SILT (ML); Dense, moist, gray; non-plastic, rapid dilatancy, low toughness, low dry strength; fine sand.
57.5					SILT (ML); Hard, moist, gray; low to medium plasticity, no dilatancy, low to medium toughness, medium dry strength; trace fine sand.
59		G2		AL, MC	
60	205				Bottom of exploration at 60 ft. bgs.
65	200				

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

Legend

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

Plastic Limit ├──┤ Liquid Limit

Water Level
- Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: M. Reiter
Approved by: A. Holmson

Exploration Log AMW-01

Jan Rd Neighborhood Improvements - 190175		Geotechnical Exploration Log	
Project Address & Site Specific Location: Cedar River near RM 13.0 - RM 13.4, See Figure.		Coordinates (Lat,Lon WGS84): 47.4257, -122.0497 (est)	Exploration Number: **AMW-02**
Contractor: Holocene Drilling, Inc.	Equipment: GeoProbe 8140 LC	Sampling Method: Autohammer; 140 lb hammer; 30" drop	Ground Surface Elev. (NAVD88): 257' (est)
			Ecology Well Tag No. BMP852
Operator: Cory	Exploration Method(s): Sonic	Work Start/Completion Dates: 10/20/2020	Top of Casing Elev. (NAVD88): 264' (est)
			Depth to Water (Below GS): 5.5' (ATD)

Depth (feet)	Elev. (feet)	Exploration Notes and Completion Details	Sample Type/ID	Blows/foot ▲ / Water Content (%) ●	Blows/6"	Tests	Description
0	257	Well No. BMP852, flush monument set in concrete					**TOPSOIL** TOPSOIL; Loose, moist, dark brown; mostly non-plastic silt; some fine to medium sand; some roots.
1–3	255	2" PVC pipe set in concrete					**ALLUVIAL DEPOSITS - GRAVEL FACIES** GRAVEL WITH COBBLES (GW); Medium dense, moist, gray brown with multicolored clasts; fine to coarse, subrounded to rounded gravel; little fine to coarse sand; trace silt; trace 4-inch, subrounded to rounded cobbles.
2.5–4			S1			Blows (non-SPT) =7,15,25	
5–6.5		∇ 10/20/2020	S2			Blows (non-SPT) =13,10,12 PS, MC	Becomes wet.
7.5–9	250	2" PVC pipe set in bentonite chips; 2" PVC pipe set in sand	S3			Blows (non-SPT) =7,6,9	Becomes loose.
10–11.5			S4			Blows (non-SPT) =13,35,50 PS, MC	GRAVEL WITH SAND (GP); Very dense, wet, gray brown; fine to coarse, subrounded to rounded gravel; fine to coarse sand; trace silt.
12.5–13	245		G1			PS, MC	GRAVEL WITH COBBLES (GP); Very dense, wet, gray brown; fine to coarse, subrounded to rounded gravel; few fine to coarse sand; trace 4-inch, subrounded to rounded cobbles; subtrace silt.
15–16.5	240	2" slotted PVC pipe set in sand. 20 slot screen.	S5			Blows (non-SPT) =23,44,46 PS, MC	**PRE-FRASER DEPOSITS** SAND (SP); Very dense, very moist, brown to red brown; fine to coarse sand; trace silt; trace fine, subrounded gravel.
20–21.3		Driller reported heave and added water to the borehole; 2' of sand placed beneath PVC pipe	S6			Blows (non-SPT) =19,67,100/4" PS, MC	SILTY SAND (SM); Very dense, very moist, dark brown; fine to coarse sand; few fine, subrounded gravel; interbedded with approx. 6" thickness of diamict at 20'.

Legend

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

Plastic Limit ⊢—⊣ Liquid Limit

Water Level: ∇ Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: M. Reiter
Approved by: A. Holmson

Exploration Log AMW-02

Jan Rd Neighborhood Improvements - 190175		Geotechnical Exploration Log	
Project Address & Site Specific Location: Cedar River near RM 13.0 - RM 13.4, See Figure.		Coordinates (Lat,Lon WGS84): 47.4257, -122.0497 (est)	Exploration Number: AMW-02
Contractor: Holocene Drilling, Inc.	Equipment: GeoProbe 8140 LC	Sampling Method: Autohammer; 140 lb hammer; 30" drop	Ground Surface Elev. (NAVD88): 257' (est)	Ecology Well Tag No. BMP852
Operator: Cory	Exploration Method(s): Sonic	Work Start/Completion Dates: 10/20/2020	Top of Casing Elev. (NAVD88): 264' (est)	Depth to Water (Below GS): 5.5' (ATD)

Depth (feet)	Elev. (feet)	Exploration Notes and Completion Details	Sample Type/ID	Blows/6"	Tests	Description
						SILTY SAND (SM); Very dense, very moist, dark brown; fine to coarse sand; few fine, subrounded gravel; interbedded with approx. 6" thickness of diamict at 20'. (continued)
25		Borehole backfilled with bentonite chips	S7	30 / 40 / 50/5"	PS, MC	Becomes moist, light brown to dark gray.
	230					
30			S8	38 / 39 / 50/5"	PS, MC	Becomes brown.
	225					
35			S9	35 / 38 / 50/4"		Becomes dark gray.
	220					
40			S10	18 / 12 / 26	PS, MC	SAND WITH SILT AND GRAVEL (SW-SM); Very dense, very moist, dark gray; fine to coarse sand; fine to coarse, subrounded to subangular gravel.
	215					

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

Legend

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

Plastic Limit ├──┤ Liquid Limit

Water Level: Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: M. Reiter
Approved by: A. Holmson

Exploration Log AMW-02

Jan Rd Neighborhood Improvements - 190175

Geotechnical Exploration Log

Aspect CONSULTING

Project Address & Site Specific Location	Coordinates (Lat,Lon WGS84)	Exploration Number
Cedar River near RM 13.0 - RM 13.4, See Figure.	47.4257, -122.0497 (est)	AMW-02

Contractor	Equipment	Sampling Method	Ground Surface Elev. (NAVD88)	Ecology Well Tag No.
Holocene Drilling, Inc.	GeoProbe 8140 LC	Autohammer; 140 lb hammer; 30" drop	257' (est)	BMP852

Operator	Exploration Method(s)	Work Start/Completion Dates	Top of Casing Elev. (NAVD88)	Depth to Water (Below GS)
Cory	Sonic	10/20/2020	264' (est)	5.5' (ATD)

Depth (feet)	Elev. (feet)	Sample Type/ID	Water Content (%)	Blows/6"	Tests	Description
44–45		S11 (Split Barrel 2" X 1.375" (SPT))		50/5"		SAND WITH SILT AND GRAVEL (SW-SM); Very dense, very moist, dark gray; fine to coarse sand; fine to coarse, subrounded to subangular gravel. (continued) Becomes moist.
50	210	G2 (Grab sample)	~8		PS, MC	GRAVEL WITH SAND (GP); Very dense, moist, dark gray; fine to coarse, rounded to subrounded gravel; fine to coarse sand; trace silt.
55	205	S12 (Split Barrel 2" X 1.375" (SPT))		28 50/6"		
59	200	G3 (Grab sample)	~17		PS, MC	SILTY SAND (SM); Very dense, moist, dark gray; fine to coarse sand; few fine, subrounded to subangular gravel.
60						Bottom of exploration at 60 ft. bgs.
65	195					

Legend

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

Plastic Limit ⊢—⊣ Liquid Limit

Water Level: ∇ Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: M. Reiter
Approved by: A. Holmson

Exploration Log AMW-02

Aspect CONSULTING

Jan Rd Neighborhood Improvements - 190175

Geotechnical Exploration Log

Project Address & Site Specific Location		Coordinates (Lat,Lon WGS84)	Exploration Number
Cedar River near RM 13.0 - RM 13.4, See Figure.		47.4275, -122.0505 (est)	**AMW-03**

Contractor	Equipment	Sampling Method	Ground Surface Elev. (NAVD88)	Ecology Well Tag No.
Holocene Drilling, Inc.	GeoProbe 8140 LC	Autohammer; 140 lb hammer; 30" drop	255' (est)	BMP917

Operator	Exploration Method(s)	Work Start/Completion Dates	Top of Casing Elev. (NAVD88)	Depth to Water (Below GS)
Cory	Sonic	1/11/2021	254.5' (est)	4' (ATD)

Depth (feet)	Elev. (feet)	Exploration Notes and Completion Details	Sample Type/ID	Blows/foot ▲ Water Content (%)●	Blows/6"	Tests	Description
0	255	Well No. BMP917, flush monument set in concrete					**TOPSOIL** TOPSOIL; Soft, slightly moist, dark brown; mostly non-plastic silt; few fine sand; abundant organics and rootlets.
		2" PVC pipe set in concrete					**ALLUVIAL DEPOSITS - GRAVEL FACIES** GRAVEL WITH SILT AND SAND (GW-GM); Medium dense, slightly moist, gray; fine to coarse, rounded to subrounded gravel; fine to coarse sand.
			S1		18 10 14	PS, MC FC=4.3%	GRAVEL WITH SAND (GW); Medium dense, wet, gray brown; fine to coarse, subrounded to rounded gravel; fine to coarse sand; trace silt.
		∇ 1/11/2021 Approximate					Becomes wet.
5	250		S2		12 11 14		
		2" PVC pipe set in bentonite chips	G1			PS, MC FC=1%	GRAVEL WITH SAND AND COBBLES (GP); Medium dense, wet, gray brown; fine to coarse, subrounded to rounded gravel; fine to coarse sand; some subrounded to rounded, 4-inch cobbles; trace silt; density inferred.
10	245		S3		13 21 27		**PRE-FRASER DEPOSITS** SAND WITH GRAVEL (SP); Dense, wet, gray; fine to coarse sand; fine to coarse, subrounded to rounded gravel; trace silt.
		2" PVC pipe set in sand	G2			PS, MC FC=2%	
15	240	2" slotted PVC pipe set in sand. 20 slot screen.	S4			Blows (non-SPT) =50/6"	
20	235		S5			Blows (non-SPT) =24,44,100/5" PS, MC FC=3.6%	GRAVEL WITH SAND (GP); Very dense, wet, dark brown; fine to coarse, subrounded to rounded gravel; fine to coarse sand; trace silt.

Legend

Sample Type:
- No Soil Sample Recovery
- Split Barrel 2" X 1.375" (SPT)
- Grab sample
- 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: M. Reiter
Approved by: A. Holmson

Exploration Log AMW-03

Jan Rd Neighborhood Improvements - 190175

Geotechnical Exploration Log

Aspect CONSULTING

Project Address & Site Specific Location			Coordinates (Lat,Lon WGS84)	Exploration Number
Cedar River near RM 13.0 - RM 13.4, See Figure.			47.4275, -122.0505 (est)	**AMW-03**
Contractor	**Equipment**	**Sampling Method**	**Ground Surface Elev. (NAVD88)**	Ecology Well Tag No. BMP917
Holocene Drilling, Inc.	GeoProbe 8140 LC	Autohammer; 140 lb hammer; 30" drop	255' (est)	
Operator	**Exploration Method(s)**	**Work Start/Completion Dates**	**Top of Casing Elev. (NAVD88)**	**Depth to Water (Below GS)**
Cory	Sonic	1/11/2021	254.5' (est)	4' (ATD)

Depth (feet)	Elev. (feet)	Exploration Notes and Completion Details	Sample Type/ID	Blows/foot ▲ Water Content (%) ●	Blows/6"	Tests	Description
		2' of sand placed beneath PVC pipe					GRAVEL WITH SAND (GP); Very dense, wet, dark brown; fine to coarse, subrounded to rounded gravel; fine to coarse sand; trace silt. (continued)
							SAND WITH SILT AND GRAVEL (SW-SM); Medium dense, very moist, dark gray; fine to medium sand; fine, subangular to rounded gravel.
25	230	Borehole backfilled with bentonite chips	S6			Blows (non-SPT) =2,2,17	Blowcounts understated due to heave.
30	225		S7		17 33 50	PS, MC FC=7.9%	Becomes very dense.
35	220		S8		36 50/3"		
40	215		S9		29 33 36	PS, MC FC=10%	SAND WITH SILT (SW-SM); Very dense, very moist, dark gray; fine to coarse sand; few fine, subangular to rounded gravel.

Plastic Limit ├──┤ Liquid Limit

Legend

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

Water Level:
- Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: M. Reiter
Approved by: A. Holmson

Exploration Log AMW-03

Sheet 2 of 3

Aspect CONSULTING	Jan Rd Neighborhood Improvements - 190175			Geotechnical Exploration Log	
	Project Address & Site Specific Location			*Coordinates (Lat,Lon WGS84)*	*Exploration Number*
	Cedar River near RM 13.0 - RM 13.4, See Figure.			47.4275, -122.0505 (est)	**AMW-03**
Contractor	*Equipment*	*Sampling Method*		*Ground Surface Elev. (NAVD88)*	Ecology Well Tag No. BMP917
Holocene Drilling, Inc.	GeoProbe 8140 LC	Autohammer; 140 lb hammer; 30" drop		255' (est)	
Operator	*Exploration Method(s)*	*Work Start/Completion Dates*		*Top of Casing Elev. (NAVD88)*	*Depth to Water (Below GS)*
Cory	Sonic	1/11/2021		254.5' (est)	4' (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)
								SAND WITH SILT (SP-SM); Medium dense, very moist, dark gray; fine to medium sand; trace fine, subangular to rounded gravel.	
45	210		S10	▲ 50/3"	35 50/3"				45
50	205		S11	▲ 50/3"	44 50/3"				50
55	200		G3	●		PS, MC FC=9.3%		Trace woody debris observed.	55
60	195		S12	▲	28 38 40				60
								Bottom of exploration at 61.5 ft. bgs.	
65	190								65

Legend

Sample Type:
- No Soil Sample Recovery
- Split Barrel 2" X 1.375" (SPT)
- Grab sample
- 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: M. Reiter
Approved by: A. Holmson

Exploration Log AMW-03

Jan Rd Neighborhood Improvements - 190175

Geotechnical Exploration Log

Aspect CONSULTING

Project Address & Site Specific Location	Coordinates (Lat,Lon WGS84)	Exploration Number
Cedar River near RM 13.0 - RM 13.4, See Figure.	47.4256, -122.0475 (est)	ATP-01

Contractor	Equipment	Sampling Method	Ground Surface Elev. (NAVD88)
King County	John Deere 50G Mini-Excavator	Grab	263' (est)

Operator	Exploration Method(s)	Work Start/Completion Dates	Top of Casing Elev. (NAVD88)	Depth to Water (Below GS)
Aaron	Backhoe or trackhoe	10/26/2020	NA	8' (Static)

Depth (feet)	Elev. (feet)	Exploration Notes and Completion Details	Sample Type/ID	Blows/foot ▲ Water Content (%)●	Blows/6"	Tests	Description	Depth (ft)
0	263					DCPT =7,9,9	**TOPSOIL** TOPSOIL; Loose, moist, brown to dark brown; mostly fine to medium sand; some silt; few fine subrounded to rounded gravel; some roots.	
1	262	Backfilled with excavated soils					**ALLUVIAL DEPOSITS - GRAVEL FACIES** GRAVEL WITH SAND AND COBBLES (GP); Medium dense, moist, gray brown; fine to coarse, subrounded to rounded gravel; fine to coarse sand; little 6-inch, subrounded to rounded cobbles; few rootlets to 2 feet bgs; trace silt.	1
2	261							2
3	260							3
4	259		Grab S1	●		DCPT =10,12,13 PS, MC		4
5	258							5
6	257							6
7	256							7
8	255	▼ 10/26/2020					Becomes wet.	8
9	254						Bottom of exploration at 9 ft. bgs.	9
10	253							10
11	252							11
12	251							12
13	250							13
14	249							14

Legend

Sample Type: Grab sample

Plastic Limit ├─┤ Liquid Limit

Water Level: ▼ Static Water Level

See Exploration Log Key for explanation of symbols

Logged by: M. Reiter
Approved by: A. Holmson

Exploration Log ATP-01

Sheet 1 of 1

Aspect CONSULTING	Jan Rd Neighborhood Improvements - 190175		Geotechnical Exploration Log	
	Project Address & Site Specific Location		*Coordinates (Lat,Lon WGS84)*	*Exploration Number*
	Cedar River near RM 13.0 - RM 13.4, See Figure.		47.4254, -122.0489 (est)	**ATP-02**
Contractor	*Equipment*	*Sampling Method*	*Ground Surface Elev. (NAVD88)*	
King County	John Deere 50G Mini-Excavator	Grab	261' (est)	
Operator	*Exploration Method(s)*	*Work Start/Completion Dates*	*Top of Casing Elev. (NAVD88)*	*Depth to Water (Below GS)*
Aaron	Backhoe or trackhoe	10/26/2020	NA	6.5' (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)
								TOPSOIL TOPSOIL; Loose, moist, dark brown; mostly fine to coarse sand; some silt; some roots; few fine to coarse, subrounded to rounded gravel.	
1	260	Backfilled with excavated soils						**ALLUVIAL DEPOSITS - GRAVEL FACIES** GRAVEL WITH SAND AND COBBLES (GP); Medium dense, moist, gray bown; fine to coarse, subrounded to rounded gravel; fine to coarse sand; little 6-inch, subrounded to rounded cobbles; few rootlets to 2 feet bgs; trace silt.	1
2	259								2
3	258					T-probe =3"		**ALLUVIAL DEPOSITS - SAND FACIES** SANDY SILT (ML); Medium dense, moist, brown; non-plastic; fine sand.	3
4	257		Grab S1	●		DCPT =16,20,22 PS, MC		**ALLUVIAL DEPOSITS - GRAVEL FACIES** GRAVEL WITH SAND, COBBLES, AND BOULDERS (GP); Medium dense, moist, gray brown; fine to coarse, subrounded to rounded gravel; fine to coarse sand; little 6-inch, subrounded to rounded cobbles and 12-inch, subrounded to rounded boulders; trace silt.	4
5	256							Becomes very moist.	5
6	255	▼ 10/26/2020						Becomes wet.	6
7	254								7
8	253								8
9	252								9
10	251								10
11	250								11
12	249								12
13	248							Material stands at approx. 1.7H:1V slope below groundwater level.	13
14	247							Bottom of exploration at 14 ft. bgs.	14

Legend

Sample Type: Grab sample

Plastic Limit ├─┤ Liquid Limit

Water Level: ▼ Static Water Level

See Exploration Log Key for explanation of symbols

Logged by: M. Reiter
Approved by: A. Holmson

Exploration Log ATP-02

Sheet 1 of 1

Aspect CONSULTING	Jan Rd Neighborhood Improvements - 190175			Geotechnical Exploration Log	
	Project Address & Site Specific Location			*Coordinates (Lat,Lon WGS84)*	*Exploration Number*
	Cedar River near RM 13.0 - RM 13.4, See Figure.			47.4259, -122.0501 (est)	**ATP-03**
Contractor	*Equipment*	*Sampling Method*		*Ground Surface Elev. (NAVD88)*	
King County	John Deere 85G Excavator	Grab		257' (est)	
Operator	*Exploration Method(s)*	*Work Start/Completion Dates*		*Top of Casing Elev. (NAVD88)*	*Depth to Water (Below GS)*
Aaron	Backhoe or trackhoe	1/13/2021		NA	3.75' (ATD)

Depth (feet)	Elev. (feet)	Exploration Notes and Completion Details	Sample Type/ID	Blows/foot ▲ Water Content (%) ●	Blows/6"	Tests	Material Type	Description	Depth (ft)
0	257	Backfilled with excavated soils	G1 (Grab)	● ~31		Manual penetration w/ thumb => 1" easily by thumb OC, MC OC=7.8%	Topsoil	**TOPSOIL** TOPSOIL; Very soft, moist, dark brown; mostly non-plastic silt; few fine sand; abundant organics, roots, and woody debris up to 3" diameter; rootlets to 2 feet bgs.	
1	256						Sand	**ALLUVIAL DEPOSITS - SAND FACIES** SAND WITH GRAVEL (SP); Medium dense, moist, gray brown; fine to medium sand; fine to coarse, subrounded to rounded gravel; abundant roots.	1
2	255		G2 (Grab)	● ~6		T-probe =6" PS, MC FC=0.3%			2
3	254	▽ 1/13/2021							3
4	253						Gravel	**ALLUVIAL DEPOSITS - GRAVEL FACIES** GRAVEL WITH SAND, COBBLES, AND BOULDERS (GW); Medium dense, wet, gray brown; fine to coarse, subrounded to rounded gravel; fine to coarse sand; some rounded to subrounded cobbles and boulders up to approximately 12".	4
5	252								5
6	251		G3 (Grab)						6
7	250								7
8	249								8
9	248							Bottom of exploration at 9 ft. bgs.	9

Blows/foot scale: 0 10 20 30 40 50

Legend

Sample Type: Grab sample

Plastic Limit ├──┤ Liquid Limit

Water Level: ▽ Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: M. Reiter
Approved by: A. Holmson

Exploration Log ATP-03

Sheet 1 of 1

Aspect CONSULTING	Jan Rd Neighborhood Improvements - 190175			Geotechnical Exploration Log	
	Project Address & Site Specific Location			*Coordinates (Lat,Lon WGS84)*	*Exploration Number*
	Cedar River near RM 13.0 - RM 13.4, See Figure.			47.4264, -122.0499 (est)	**ATP-04**
Contractor	*Equipment*	*Sampling Method*		*Ground Surface Elev. (NAVD88)*	
King County	John Deere 85G Excavator	Grab		257' (est)	
Operator	*Exploration Method(s)*	*Work Start/Completion Dates*		*Top of Casing Elev. (NAVD88)*	*Depth to Water (Below GS)*
Aaron	Backhoe or trackhoe	1/13/2021		NA	4' (ATD)

Depth (feet)	Elev. (feet)	Exploration Notes and Completion Details	Sample Type/ID	Water Content (%)	Tests	Description	Depth (ft)
0–1	257–256		G1 (Grab)	~37	Manual penetration => 1" easily by thumb OC, MC OC=6.5%	**TOPSOIL** TOPSOIL; Very soft, moist, dark brown to black; mostly non-plastic silt; few fine sand; abundant organics, roots, and woody debris up to 3" diameter.	
1	256	Backfilled with excavated soils				**ALLUVIAL DEPOSITS - SAND FACIES** SAND WITH SILT (SP-SM); Medium dense, moist, gray brown; fine to coarse sand; little fine, subrounded to rounded gravel; abundant roots and woody debris.	1
2	255						2
3	254		G2 (Grab)	~10	T-probe =8" PS, MC FC=6.3%		3
4	253	∇ 1/13/2021				**ALLUVIAL DEPOSITS - GRAVEL FACIES** GRAVEL WITH SAND AND COBBLES (GP); Medium dense, wet, gray brown; fine to coarse, subrounded to rounded gravel; fine to coarse sand; some 5", rounded to subrounded cobbles.	4
5	252						5
6	251		G3 (Grab)		PS, MP FC=1.1%		6
7	250						7
8	249					Bottom of exploration at 8 ft. bgs.	8
9	248						9

Columns also include: Blows/foot ▲, Water Content (%) ● (scale 0 10 20 30 40 50), Blows/6", Material Type.

Legend

Sample Type: Grab sample

Plastic Limit ⊢—⊣ Liquid Limit

Water Level: ∇ Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: M. Reiter
Approved by: A. Holmson

Exploration Log ATP-04

Sheet 1 of 1

Jan Rd Neighborhood Improvements - 190175

Geotechnical Exploration Log

Aspect CONSULTING

Project Address & Site Specific Location	Coordinates (Lat,Lon WGS84)	Exploration Number
Cedar River near RM 13.0 - RM 13.4, See Figure.	47.4270, -122.0498 (est)	ATP-05

Contractor	Equipment	Sampling Method	Ground Surface Elev. (NAVD88)
King County	John Deere 85G Excavator	Grab	255' (est)

Operator	Exploration Method(s)	Work Start/Completion Dates	Top of Casing Elev. (NAVD88)	Depth to Water (Below GS)
Aaron	Backhoe or trackhoe	1/13/2021	NA	2' (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	Description	Depth (ft)
							ALLUVIAL DEPOSITS - SAND FACIES SILTY SAND (SM); Loose, moist, brown; fine to medium sand; few fine, subrounded gravel; abundant roots and woody debris.	
1	254	Backfilled with excavated soils						1
			Grab G1	● (~22)		T-probe =14" FC, OC, MC OC=3.2% FC=23.3%		
2	253	∇ 1/13/2021						2
							ALLUVIAL DEPOSITS - GRAVEL FACIES GRAVEL WITH SILT, SAND, AND COBBLES (GW-GM); Medium dense, wet, gray brown; fine to coarse, subrounded to rounded gravel; fine to coarse sand; some 6", subrounded to rounded cobbles.	
3	252							3
4	251							4
5	250							5
			Grab G2			MP		
6	249						Bottom of exploration at 6 ft. bgs.	6
7	248							7
8	247							8
9	246							9

Legend

Sample Type: Grab sample

Plastic Limit ├─┤ Liquid Limit

Water Level: ∇ Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: M. Reiter
Approved by: A. Holmson

Exploration Log ATP-05

Sheet 1 of 1

Aspect CONSULTING	Jan Rd Neighborhood Improvements - 190175		Geotechnical Exploration Log	
	Project Address & Site Specific Location		*Coordinates (Lat,Lon WGS84)*	*Exploration Number*
	Cedar River near RM 13.0 - RM 13.4, See Figure.		47.4279, -122.0515 (est)	**ATP-06**
Contractor	*Equipment*	*Sampling Method*	*Ground Surface Elev. (NAVD88)*	
King County	John Deere 85G Excavator	Grab	255' (est)	
Operator	*Exploration Method(s)*	*Work Start/Completion Dates*	*Top of Casing Elev. (NAVD88)*	*Depth to Water (Below GS)*
Aaron	Backhoe or trackhoe	1/13/2021	NA	2.25' (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)
0	255					Manual penetration =1/4" to 1" easily by thumb		**TOPSOIL** TOPSOIL; Very loose, moist, dark brown; mostly non-plastic silt; few fine sand; some rootlets.	
1	254	Backfilled with excavated soils	G1 (grab)			T-probe =6"		**ALLUVIAL DEPOSITS - SAND FACIES** SILTY SAND (SM); Medium dense, moist, brown; fine to medium sand; trace rootlets.	1
2	253	▽ 1/13/2021						**ALLUVIAL DEPOSITS - GRAVEL FACIES** GRAVEL WITH SAND AND COBBLES (GW); Medium dense, wet, gray brown; fine to coarse, subrounded to rounded gravel; fine to coarse sand; some 6", rounded to subrounded cobbles.	2
3	252								3
4	251								4
5	250		G2 (grab)						5
6	249							Bottom of exploration at 6 ft. bgs.	6
7	248								7
8	247								8
9	246								9

Legend

Sample Type: Grab sample

Plastic Limit ├──┤ Liquid Limit

Water Level: ▽ Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: M. Reiter
Approved by: A. Holmson

Exploration Log ATP-06

Sheet 1 of 1

Jan Rd Neighborhood Improvements - 190175

Geotechnical Exploration Log

Aspect CONSULTING

Project Address & Site Specific Location	Coordinates (Lat,Lon WGS84)	Exploration Number
Cedar River near RM 13.0 - RM 13.4, See Figure.	47.4277, -122.0523 (est)	ATP-07

Contractor	Equipment	Sampling Method	Ground Surface Elev. (NAVD88)
King County	John Deere 85G Excavator	Grab	255' (est)

Operator	Exploration Method(s)	Work Start/Completion Dates	Top of Casing Elev. (NAVD88)	Depth to Water (Below GS)
Aaron	Backhoe or trackhoe	1/13/2021	NA	4' (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)
0–1	255–254							**TOPSOIL** TOPSOIL; Very loose, moist, dark brown; mostly non-plastic silt; few fine sand; abundant rootlets.	
1	254	Backfilled with excavated soils						**ALLUVIAL DEPOSITS - GRAVEL FACIES** GRAVEL WITH SAND (GW); Medium dense, moist, brown; fine to coarse, subrounded to rounded gravel; fine to medium sand; sand interbedded with 4"-6" seams of gravel; trace rootlets.	1
			G1 (Grab)			T-probe =3"-4" PS, MP FC=1%			
2	253								2
3	252								3
			G2 (Grab)			Manual penetration =1/4" to 1" easily by thumb		**ALLUVIAL DEPOSITS - SAND FACIES** SANDY SILT (ML); Loose, very moist, brown; non-plastic silt; fine sand; woody debris up to 3" diameter.	
4	251	▽ 1/13/2021						**ALLUVIAL DEPOSITS - GRAVEL FACIES** GRAVEL WITH SILT, SAND, AND COBBLES (GW-GM); Medium dense, wet, gray brown; fine to coarse, subrounded to rounded gravel; fine to coarse sand; some 6", subrounded to rounded cobbles.	4
5	250							Bottom of exploration at 5 ft. bgs.	5
6	249								6
7	248								7
8	247								8
9	246								9

Legend

Sample Type: Grab sample

Plastic Limit ⊢—⊣ Liquid Limit

Water Level: ▽ Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: M. Reiter
Approved by: A. Holmson

Exploration Log ATP-07

Sheet 1 of 1

APPENDIX B

Laboratory Testing Results

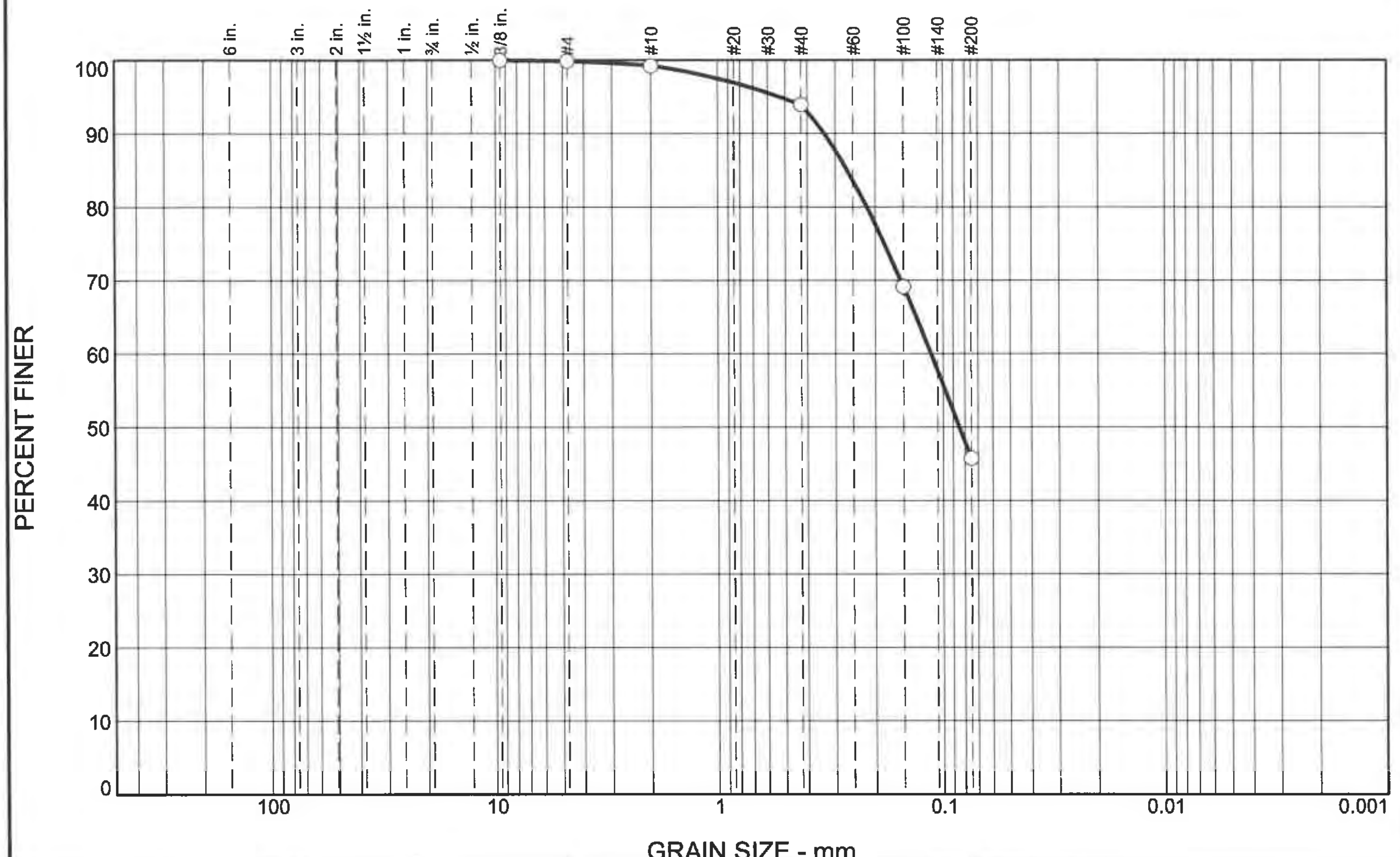
B. Geotechnical Laboratory Testing

Geotechnical laboratory tests were conducted on selected soil samples collected during the field exploration program. Laboratory testing was performed by Hayre McElroy & Associates, Inc. under subcontract to Aspect. The tests performed and the procedures followed are outlined below.

- Water content determination (MC): Water contents of select soil samples collected from the soil borings were determined in accordance with ASTM International (ASTM) D2216 (ASTM, 2018).
- Fines content determination (FC): Fines contents of select soil samples collected from the soil borings were determined in accordance with ASTM C117.
- Organic content determination (OC): Organic contents of select soil samples collected from the soil borings were determined in accordance with ASTM D2974.
- Particle-size distribution (PS): Particle-size distribution of select soil samples collected from the soil borings were determined in accordance with ASTM D6913.
- Atterberg Limits (AL): Atterberg limits of select soil samples collected from the soil borings were determined in accordance with ASTM D4318.
- Modified Proctor Test (MP): Modified proctor tests were performed on selected samples to determine the maximum dry density and optimum water correct in accordance with ASTM D 1557-91 Procedure C method. Per ASTM D4718, the oversize correction was applied to each test point to achieve both rock corrected and un-corrected values maximum dry density and optimum moisture content. At ATP-04, the analysis could not be completed on the native material due to the high gravel content. Particles over 3-inches were removed from bulk sample so the test could be completed with standard equipment.

The results of the tests are summarized here in Appendix B and incorporated into the exploration logs in Appendix A.

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.1	0.7	5.3	48.1	45.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8"	100.0		
#4	99.9		
#10	99.2		
#40	93.9		
#100	69.1		
#200	45.8		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.3360 D_{85}= 0.2659 D_{60}= 0.1134
D_{50}= 0.0847 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

MC=23.6%

Location: AB-01
Sample Number: G-1 **Depth:** 2-2.5 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AB-01
Depth: 2-2.5 **Sample Number:** G-1
Material Description: Silty sand
Date: 11/20/2020
USCS Classification: SM
Testing Remarks: MC=23.6%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 163.80
Tare Wt. = 12.60
Minus #200 from wash = 35.7%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
247.70	12.60	0.00	3/8"	0.00	100.0
			#4	0.30	99.9
			#10	1.80	99.2
			#40	14.30	93.9
			#100	72.60	69.1
			#200	127.50	45.8

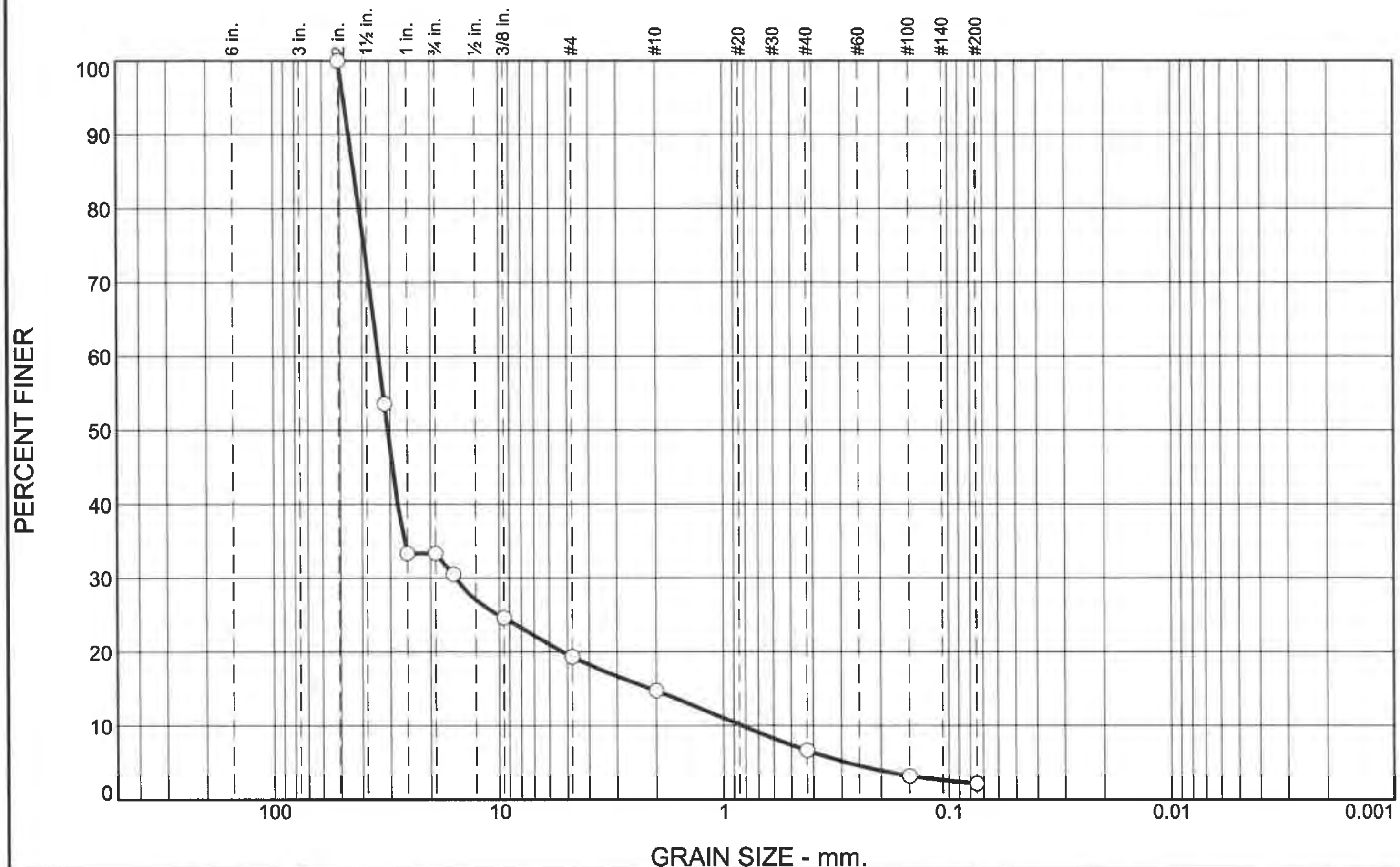
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	0.1	0.1	0.7	5.3	48.1	54.1			45.8

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
						0.0847	0.1134	0.2179	0.2659	0.3360	0.5297

Fineness Modulus
0.50

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	66.7	14.0	4.6	8.1	4.4	2.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2"	100.0		
1 1/4"	53.6		
1"	33.3		
3/4"	33.3		
5/8"	30.5		
3/8"	24.6		
#4	19.3		
#10	14.7		
#40	6.6		
#100	3.2		
#200	2.2		

* (no specification provided)

Material Description

Gravel with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 45.6541 D_{85}= 43.3051 D_{60}= 33.6945
D_{50}= 30.7230 D_{30}= 15.4520 D_{15}= 2.1084
D_{10}= 0.8260 C_u= 40.79 C_c= 8.58

Classification

USCS= GP AASHTO=

Remarks

MC=4.4%

Location: AB-01
Sample Number: S-3 **Depth:** 7.5 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road

Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AB-01
Depth: 7.5 **Sample Number:** S-3
Material Description: Gravel with sand
Date: 11/20/2020
USCS Classification: GP
Testing Remarks: MC=4.4%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 670.60
Tare Wt. = 12.50
Minus #200 from wash = 2.3%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
686.20	12.50	0.00	2"	0.00	100.0
			1 1/4"	312.50	53.6
			1"	449.10	33.3
			3/4"	449.10	33.3
			5/8"	467.90	30.5
			3/8"	507.90	24.6
			#4	543.40	19.3
			#10	574.40	14.7
			#40	628.90	6.6
			#100	652.00	3.2
			#200	659.20	2.2

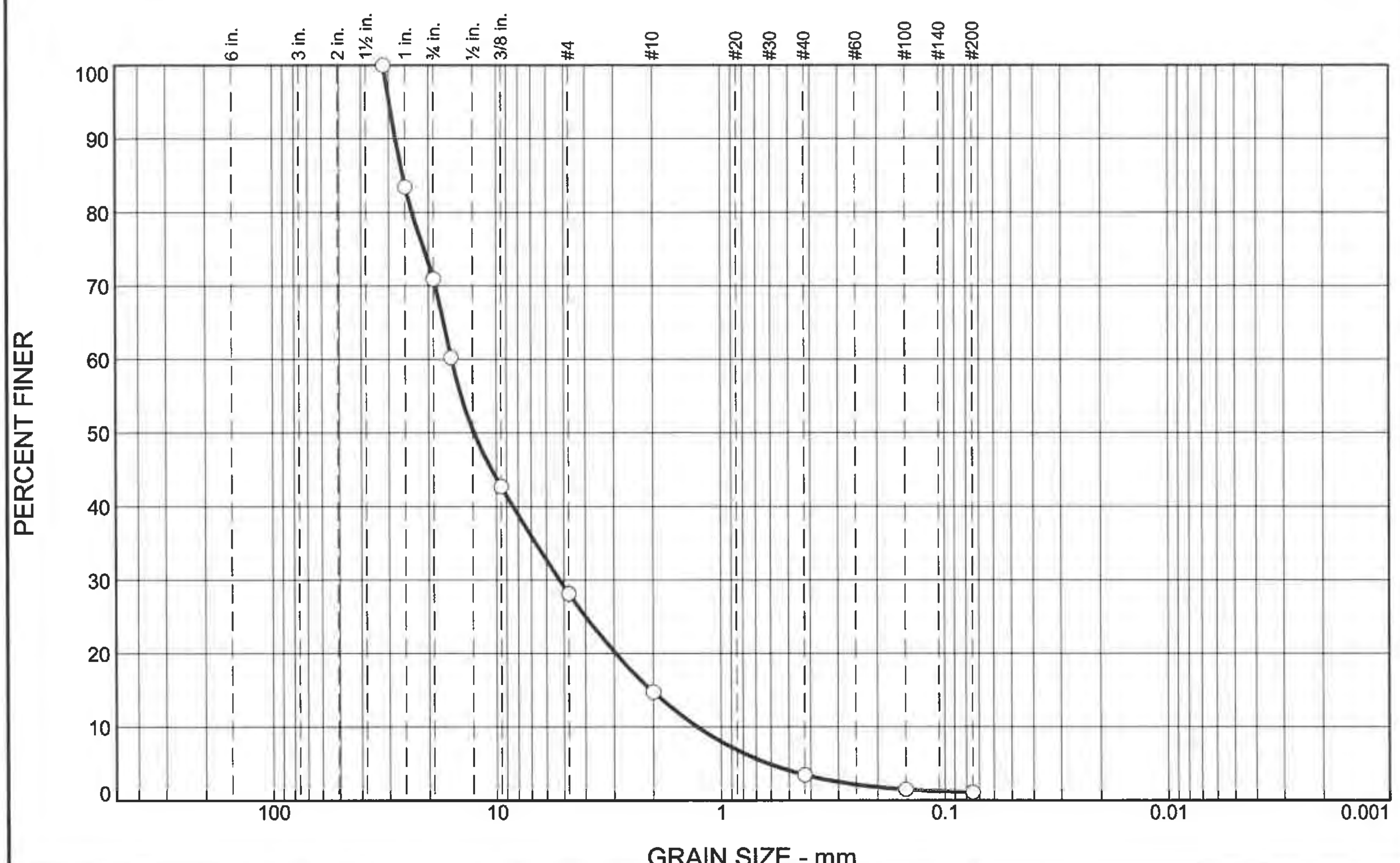
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	66.7	14.0	80.7	4.6	8.1	4.4	17.1			2.2

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.2818	0.8260	2.1084	5.2229	15.4520	27.8879	30.7230	33.6945	41.1045	43.3051	45.6541	48.1527

Fineness Modulus	C_u	C_c
7.06	40.79	8.58

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	29.1	42.8	13.4	11.2	2.4	1.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/4"	100.0		
1"	83.5		
3/4"	70.9		
5/8"	60.2		
3/8"	42.7		
#4	28.1		
#10	14.7		
#40	3.5		
#100	1.5		
#200	1.1		

* (no specification provided)

Material Description

Gravel with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 28.0246 D_{85}= 26.0650 D_{60}= 15.8110
D_{50}= 12.5685 D_{30}= 5.2401 D_{15}= 2.0417
D_{10}= 1.2722 C_u= 12.43 C_c= 1.37

Classification

USCS= GW AASHTO=

Remarks

MC=2.8%

Location: AB-01
Sample Number: G-2 **Depth:** 12.5 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road

Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/24/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AB-01
Depth: 12.5
Sample Number: G-2
Material Description: Gravel with sand
Date: 11/20/2020
USCS Classification: GW
Testing Remarks: MC=2.8%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 636.00
Tare Wt. = 12.70
Minus #200 from wash = 1.2%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
643.30	12.70	0.00	1 1/4"	0.00	100.0
			1"	104.30	83.5
			3/4"	183.20	70.9
			5/8"	250.80	60.2
			3/8"	361.20	42.7
			#4	453.30	28.1
			#10	537.60	14.7
			#40	608.50	3.5
			#100	621.20	1.5
			#200	623.70	1.1

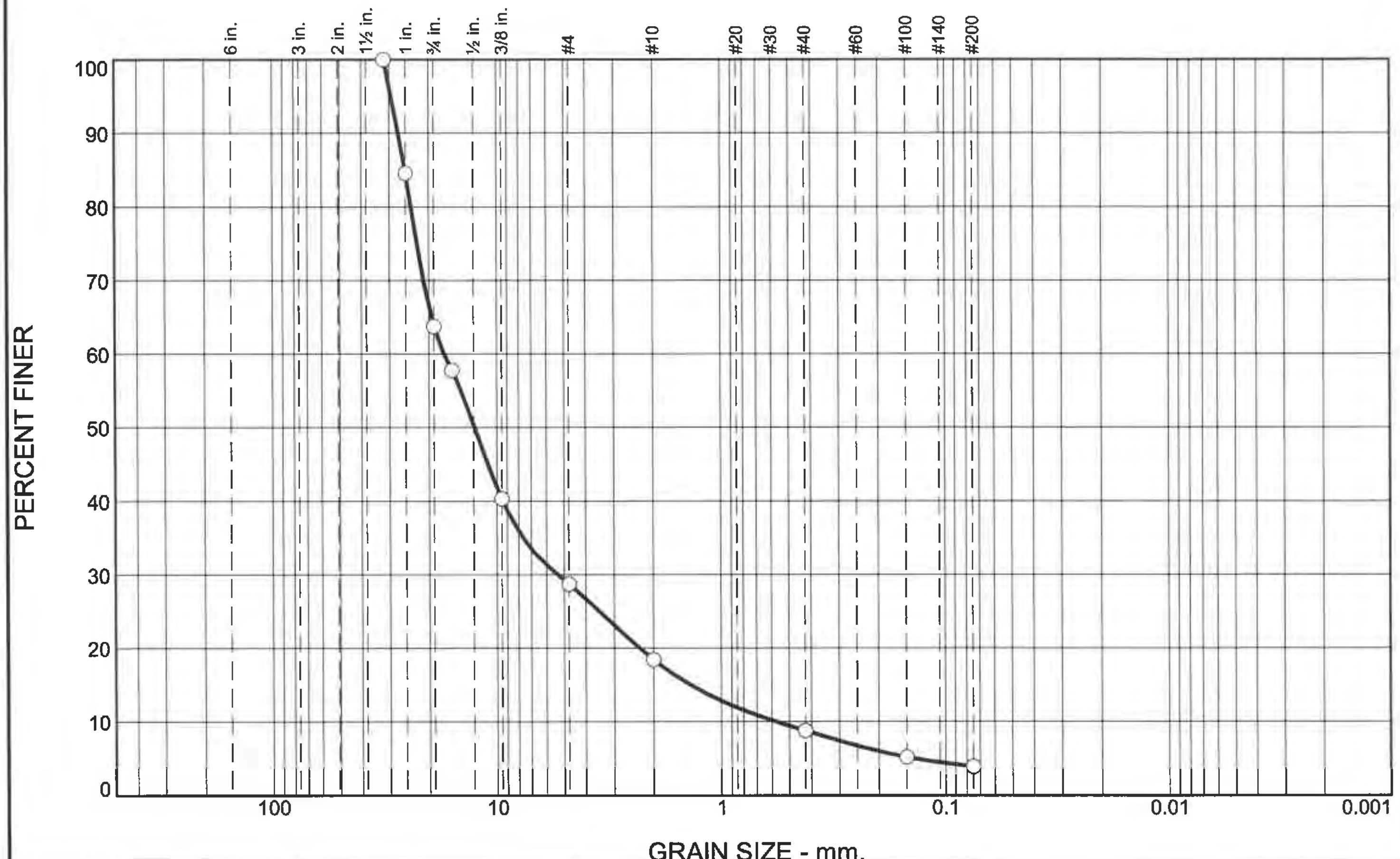
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	29.1	42.8	71.9	13.4	11.2	2.4	27.0			1.1

D_{5}	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.6104	1.2722	2.0417	2.9423	5.2401	8.4361	12.5685	15.8110	23.6916	26.0650	28.0246	29.8764

Fineness Modulus	C_u	C_c
6.23	12.43	1.37

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	36.2	35.1	10.3	9.6	4.9	3.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/4"	100.0		
1"	84.6		
3/4"	63.8		
5/8"	57.8		
3/8"	40.3		
#4	28.7		
#10	18.4		
#40	8.8		
#100	5.2		
#200	3.9		

* (no specification provided)

Material Description

Gravel with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 27.3528 D_{85}= 25.5453 D_{60}= 17.2172
D_{50}= 12.5776 D_{30}= 5.3399 D_{15}= 1.3688
D_{10}= 0.5709 C_u= 30.16 C_c= 2.90

Classification

USCS= GW AASHTO=

Remarks

MC=9.4%

Location: AB-01
Sample Number: S-5 **Depth:** 15 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AB-01
Depth: 15 **Sample Number:** S-5
Material Description: Gravel with sand
Date: 11/20/2020
USCS Classification: GW
Testing Remarks: MC=9.4%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 650.40
Tare Wt. = 15.80
Minus #200 from wash = 3.6%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
674.00	15.80	0.00	1 1/4"	0.00	100.0
			1"	101.60	84.6
			3/4"	238.50	63.8
			5/8"	277.90	57.8
			3/8"	392.80	40.3
			#4	469.30	28.7
			#10	536.90	18.4
			#40	600.40	8.8
			#100	624.10	5.2
			#200	632.60	3.9

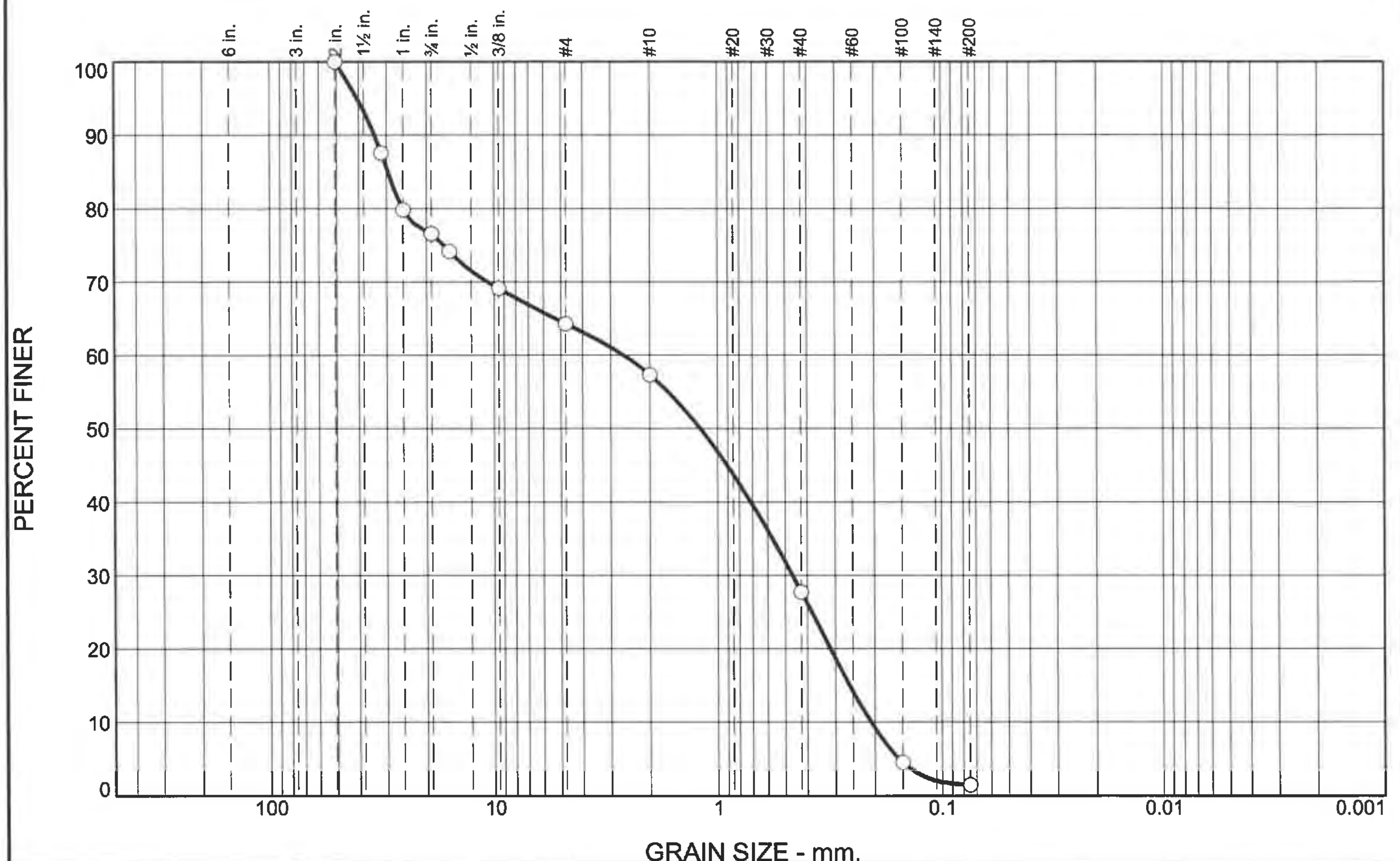
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	36.2	35.1	71.3	10.3	9.6	4.9	24.8			3.9

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.1390	0.5709	1.3688	2.3081	5.3399	9.4213	12.5776	17.2172	23.9726	25.5453	27.3528	29.4279

Fineness Modulus	C_u	C_c
6.10	30.16	2.90

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	23.4	12.3	7.0	29.6	26.2	1.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2"	100.0		
1 1/4"	87.6		
1"	79.8		
3/4"	76.6		
5/8"	74.1		
3/8"	69.1		
#4	64.3		
#10	57.3		
#40	27.7		
#100	4.5		
#200	1.5		

* (no specification provided)

Material Description

Sand with gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 34.0796 D_{85}= 29.6625 D_{60}= 2.6163
D_{50}= 1.1943 D_{30}= 0.4650 D_{15}= 0.2593
D_{10}= 0.2080 C_u= 12.58 C_c= 0.40

Classification

USCS= SP AASHTO=

Remarks

MC=17.4%

Location: AB-01
Sample Number: S-6 **Depth:** 20 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road

Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AB-01
Depth: 20
Sample Number: S-6
Material Description: Sand with gravel
Date: 11/20/2020
USCS Classification: SP
Testing Remarks: MC=17.4%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 584.20
Tare Wt. = 16.30
Minus #200 from wash = 2.0%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
595.60	16.30	0.00	2"	0.00	100.0
			1 1/4"	72.00	87.6
			1"	117.00	79.8
			3/4"	135.80	76.6
			5/8"	149.80	74.1
			3/8"	178.90	69.1
			#4	206.80	64.3
			#10	247.20	57.3
			#40	418.70	27.7
			#100	553.00	4.5
			#200	570.70	1.5

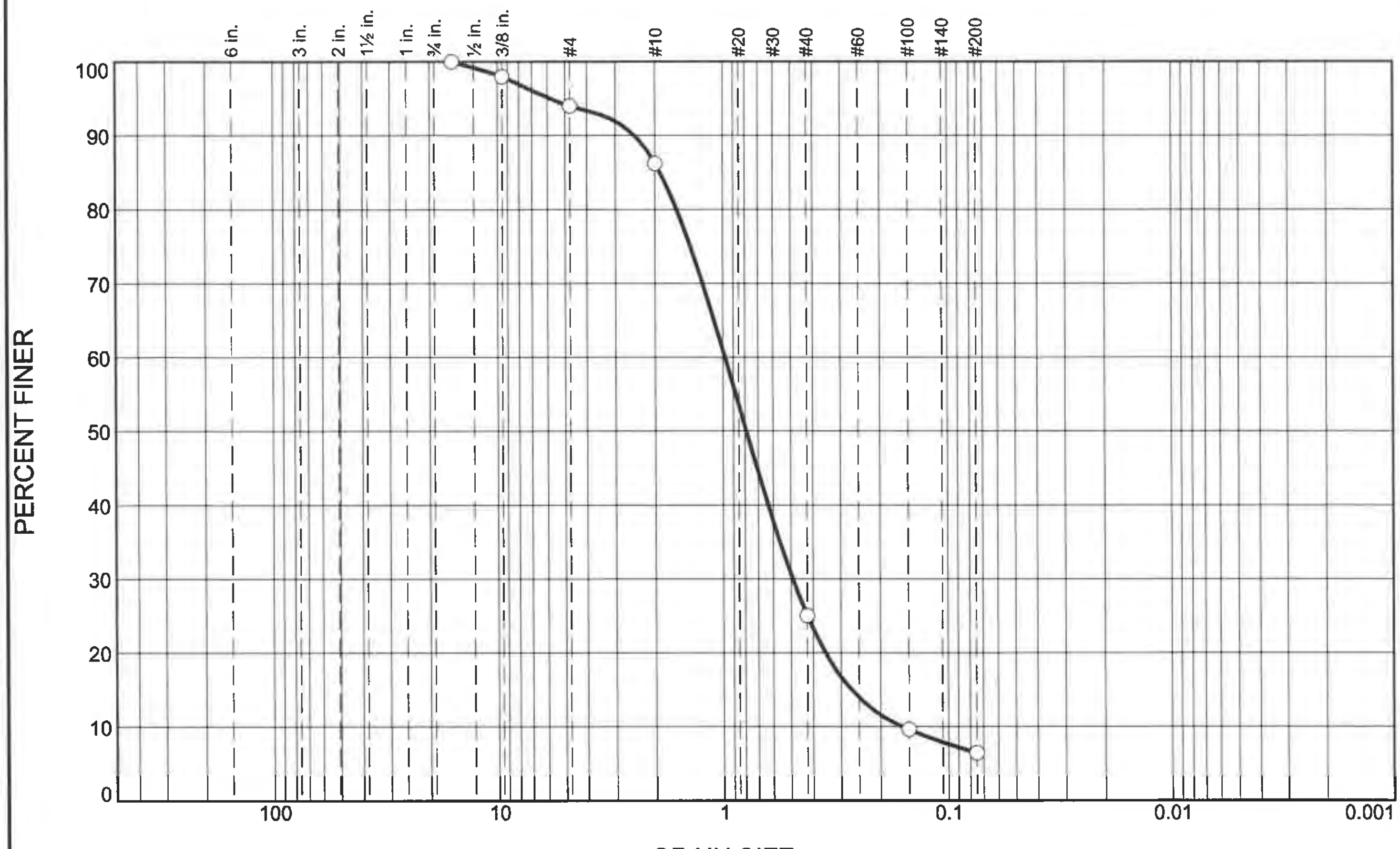
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	23.4	12.3	35.7	7.0	29.6	26.2	62.8			1.5

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.1556	0.2080	0.2593	0.3160	0.4650	0.7130	1.1943	2.6163	25.6001	29.6625	34.0796	40.6993

Fineness Modulus	C_u	C_c
4.29	12.58	0.40

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	6.0	7.8	61.1	18.6	6.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
5/8"	100.0		
3/8"	98.0		
#4	94.0		
#10	86.2		
#40	25.1		
#100	9.6		
#200	6.5		

* (no specification provided)

Material Description

Sand with silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 2.4708 D_{85}= 1.9027 D_{60}= 0.9818
D_{50}= 0.7893 D_{30}= 0.4920 D_{15}= 0.2698
D_{10}= 0.1596 C_u= 6.15 C_c= 1.54

Classification

USCS= SW-SM AASHTO=

Remarks

MC=17.4%

Location: AB-01
Sample Number: S-7 **Depth:** 25 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AB-01
Depth: 25
Sample Number: S-7
Material Description: Sand with silt
Date: 11/20/2020
USCS Classification: SW-SM
Testing Remarks: MC=17.4%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 478.60
Tare Wt. = 16.10
Minus #200 from wash = 6.9%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
513.00	16.10	0.00	5/8"	0.00	100.0
			3/8"	10.10	98.0
			#4	29.80	94.0
			#10	68.50	86.2
			#40	372.40	25.1
			#100	449.00	9.6
			#200	464.80	6.5

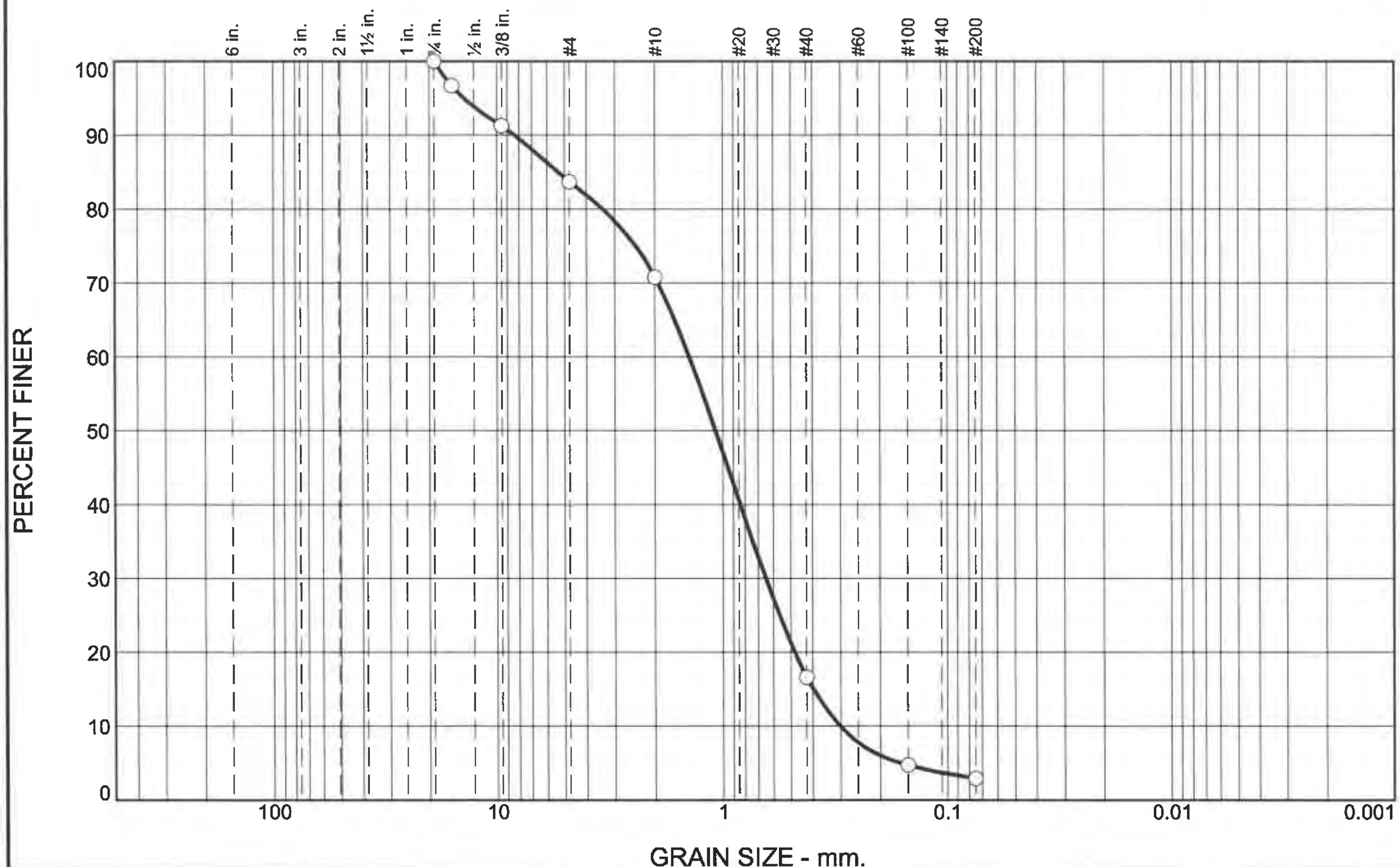
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	6.0	6.0	7.8	61.1	18.6	87.5			6.5

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
	0.1596	0.2698	0.3521	0.4920	0.6312	0.7893	0.9818	1.6071	1.9027	2.4708	5.7930

Fineness Modulus	C_u	C_c
2.86	6.15	1.54

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	16.3	12.9	54.2	13.7	2.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
5/8"	96.7		
3/8"	91.3		
#4	83.7		
#10	70.8		
#40	16.6		
#100	4.7		
#200	2.9		

* (no specification provided)

Material Description

Sand with gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 8.3521 D_{85}= 5.3351 D_{60}= 1.4100
D_{50}= 1.0808 D_{30}= 0.6477 D_{15}= 0.3970
D_{10}= 0.3017 C_u= 4.67 C_c= 0.99

Classification

USCS= SP AASHTO=

Remarks

MC=13.4%

Location: AB-01
Sample Number: G-3 **Depth:** 29 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/24/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AB-01
Depth: 29 **Sample Number:** G-3
Material Description: Sand with gravel
Date: 11/20/2020
USCS Classification: SP
Testing Remarks: MC=13.4%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 355.50
Tare Wt. = 12.60
Minus #200 from wash = 2.7%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
364.90	12.60	0.00	3/4"	0.00	100.0
			5/8"	11.60	96.7
			3/8"	30.70	91.3
			#4	57.40	83.7
			#10	103.00	70.8
			#40	293.70	16.6
			#100	335.60	4.7
			#200	342.10	2.9

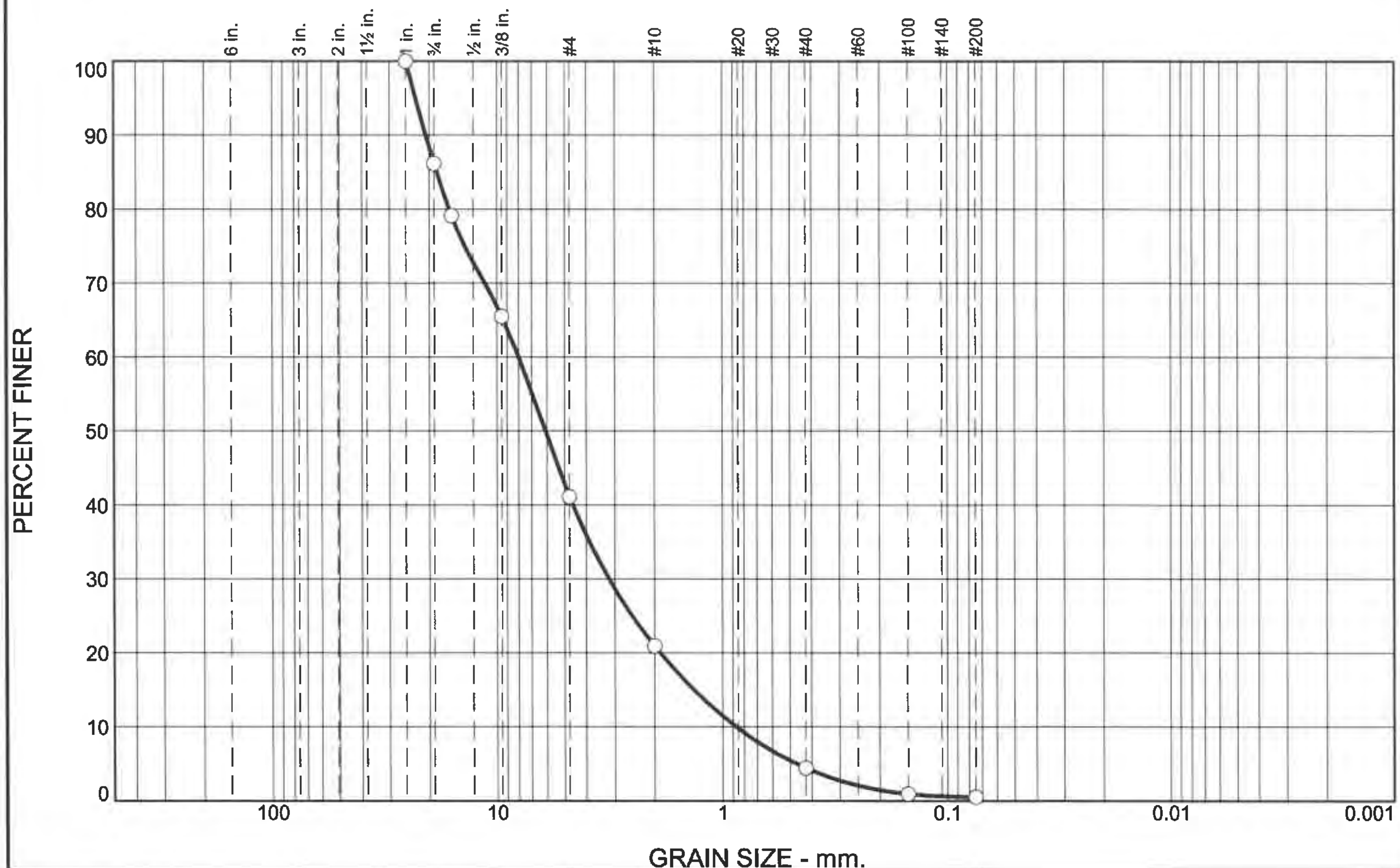
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	16.3	16.3	12.9	54.2	13.7	80.8			2.9

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.1609	0.3017	0.3970	0.4806	0.6477	0.8400	1.0808	1.4100	3.3929	5.3351	8.3521	14.0416

Fineness Modulus	C_u	C_c
3.55	4.67	0.99

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	13.8	45.1	20.1	16.5	4.0	0.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1"	100.0		
3/4"	86.2		
5/8"	79.1		
3/8"	65.5		
#4	41.1		
#10	21.0		
#40	4.5		
#100	0.9		
#200	0.5		

* (no specification provided)

Material Description

Gravel with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 20.7227 D_{85}= 18.5296 D_{60}= 8.0136
D_{50}= 6.0930 D_{30}= 3.1870 D_{15}= 1.3373
D_{10}= 0.8675 C_u= 9.24 C_c= 1.46

Classification

USCS= GW AASHTO=

Remarks

MC=2.9%

Location: AB-02
Sample Number: G-1 **Depth:** 4 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/24/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AB-02
Depth: 4 **Sample Number:** G-1
Material Description: Gravel with sand
Date: 11/20/2020
USCS Classification: GW
Testing Remarks: MC=2.9%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 430.40
Tare Wt. = 12.60
Minus #200 from wash = 0.5%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
432.50	12.60	0.00	1"	0.00	100.0
			3/4"	58.00	86.2
			5/8"	87.70	79.1
			3/8"	144.90	65.5
			#4	247.30	41.1
			#10	331.90	21.0
			#40	401.20	4.5
			#100	416.00	0.9
			#200	417.80	0.5

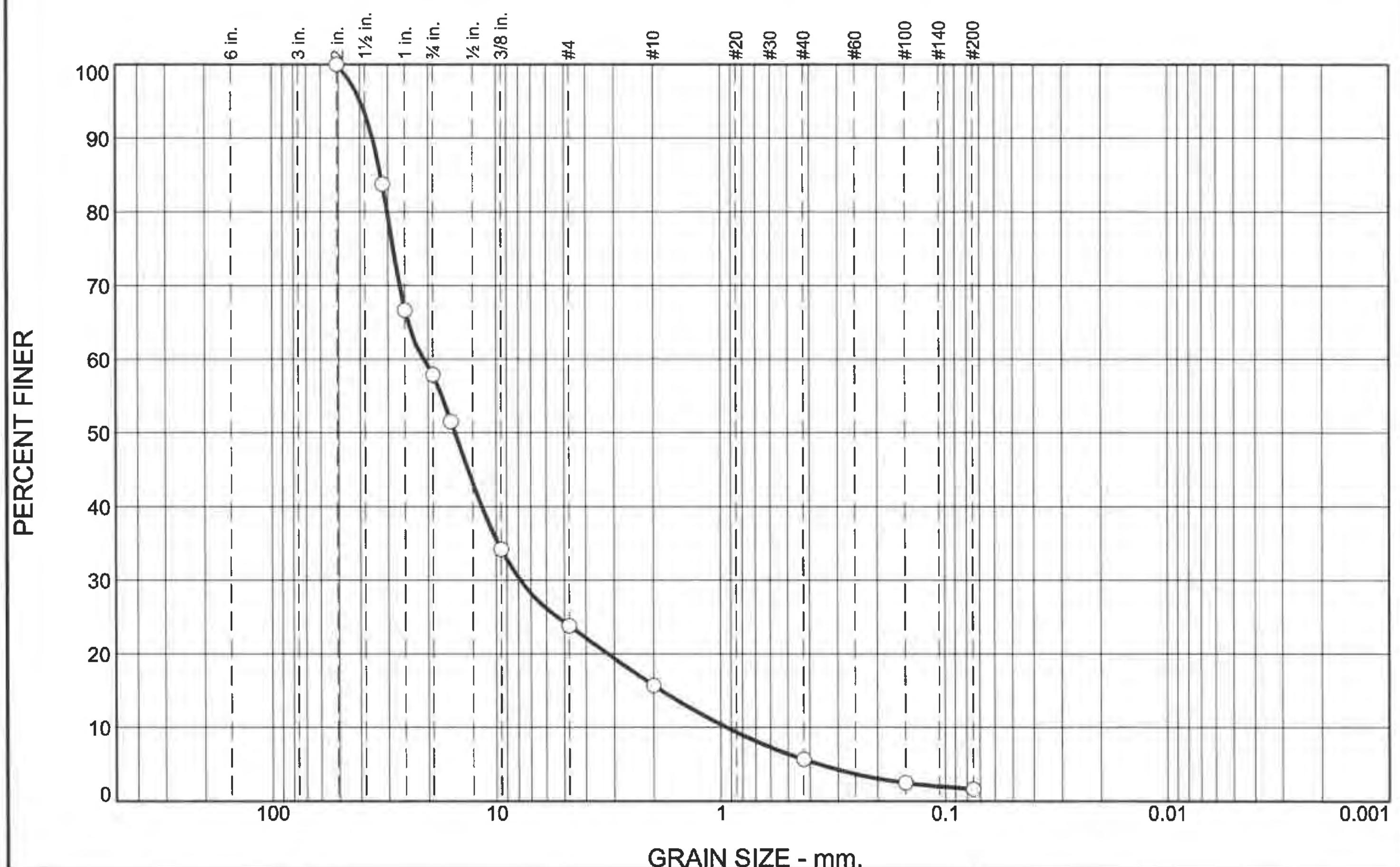
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	13.8	45.1	58.9	20.1	16.5	4.0	40.6			0.5

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.4657	0.8675	1.3373	1.8855	3.1870	4.5917	6.0930	8.0136	16.2892	18.5296	20.7227	22.9823

Fineness Modulus	C_u	C_c
5.60	9.24	1.46

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	42.1	34.1	8.1	10.0	4.1	1.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2"	100.0		
1 1/4"	83.8		
1"	66.6		
3/4"	57.9		
5/8"	51.6		
3/8"	34.2		
#4	23.8		
#10	15.7		
#40	5.7		
#100	2.5		
#200	1.6		

* (no specification provided)

Material Description

Gravel with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 35.1702 D_{85}= 32.3120 D_{60}= 20.9395
D_{50}= 15.2469 D_{30}= 7.8402 D_{15}= 1.8377
D_{10}= 0.9390 C_u= 22.30 C_c= 3.13

Classification

USCS= GP AASHTO=

Remarks

MC=3.6%

Location: AB-02
Sample Number: G-2 **Depth:** 7.5 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road

Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/24/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AB-02
Depth: 7.5 **Sample Number:** G-2
Material Description: Gravel with sand
Date: 11/20/2020
USCS Classification: GP
Testing Remarks: MC=3.6%

Sieve Test Data

Post #200 Wash Test Weights (grams): Dry Sample and Tare = 1003.60
Tare Wt. = 12.80
Minus #200 from wash = 1.7%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
1020.60	12.80	0.00	2"	0.00	100.0
			1 1/4"	163.60	83.8
			1"	336.30	66.6
			3/4"	424.40	57.9
			5/8"	488.20	51.6
			3/8"	663.10	34.2
			#4	768.00	23.8
			#10	849.40	15.7
			#40	950.70	5.7
			#100	982.80	2.5
			#200	991.20	1.6

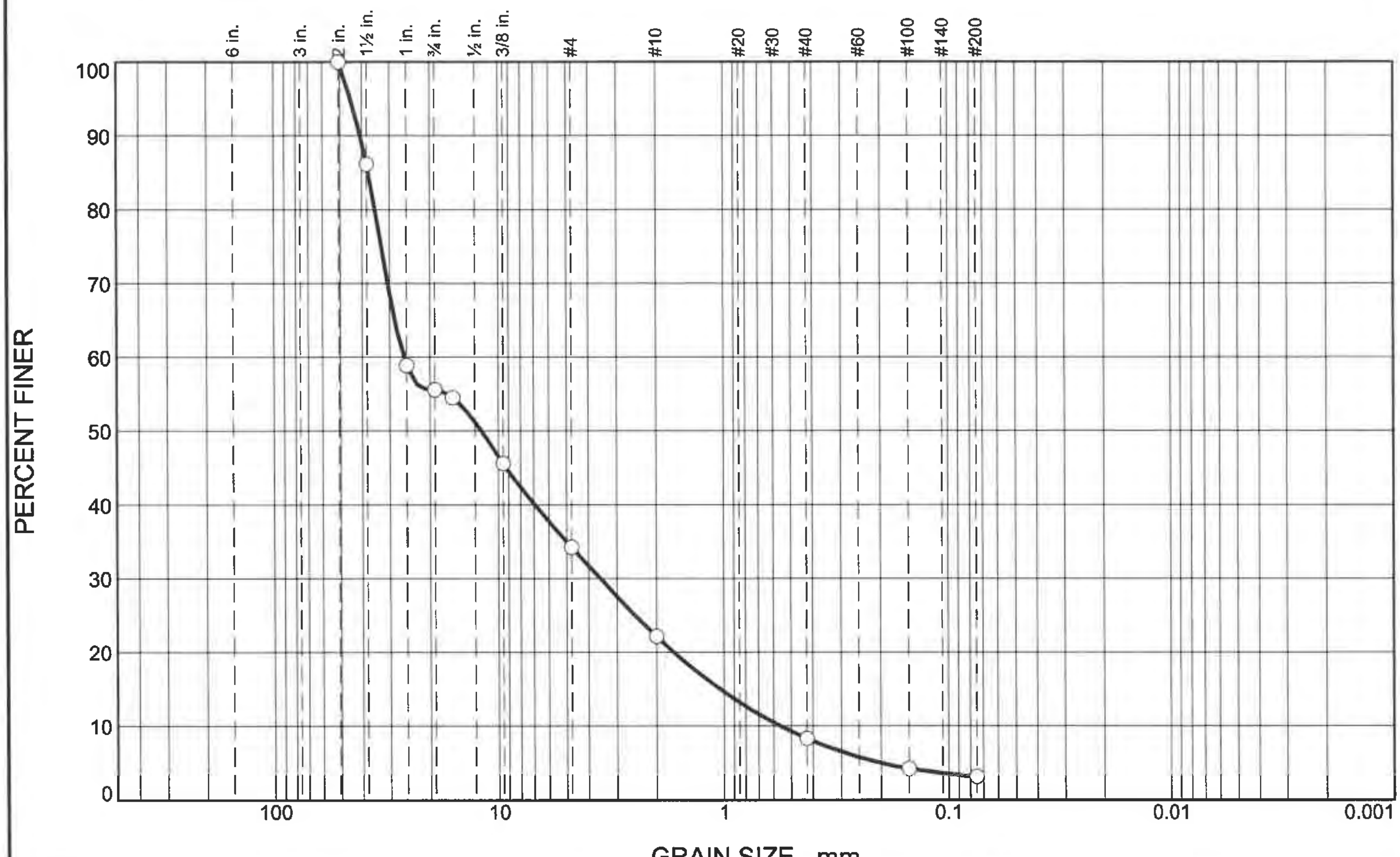
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	42.1	34.1	76.2	8.1	10.0	4.1	22.2			1.6

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.3626	0.9390	1.8377	3.2011	7.8402	11.5784	15.2469	20.9395	30.2382	32.3120	35.1702	39.8711

Fineness Modulus	C_u	C_c
6.48	22.30	3.13

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	44.4	21.4	12.1	13.8	5.2	3.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2"	100.0		
1.5"	86.1		
1"	58.9		
3/4"	55.6		
5/8"	54.5		
3/8"	45.6		
#4	34.2		
#10	22.1		
#40	8.3		
#100	4.2		
#200	3.1		

* (no specification provided)

Material Description

Gravel with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 40.6773 D_{85}= 37.4626 D_{60}= 26.0830
D_{50}= 11.9012 D_{30}= 3.5592 D_{15}= 1.0390
D_{10}= 0.5567 C_u= 46.85 C_c= 0.87

Classification

USCS= GP AASHTO=

Remarks

MC=7.4%

Location: AB-02
Sample Number: S-3 **Depth:** 10 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AB-02
Depth: 10 **Sample Number:** S-3
Material Description: Gravel with sand
Date: 11/20/2020
USCS Classification: GP
Testing Remarks: MC=7.4%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 763.80
Tare Wt. = 12.80
Minus #200 from wash = 3.2%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
789.00	12.80	0.00	2"	0.00	100.0
			1.5"	107.80	86.1
			1"	319.10	58.9
			3/4"	344.70	55.6
			5/8"	353.10	54.5
			3/8"	422.40	45.6
			#4	510.50	34.2
			#10	604.40	22.1
			#40	711.60	8.3
			#100	743.30	4.2
			#200	751.80	3.1

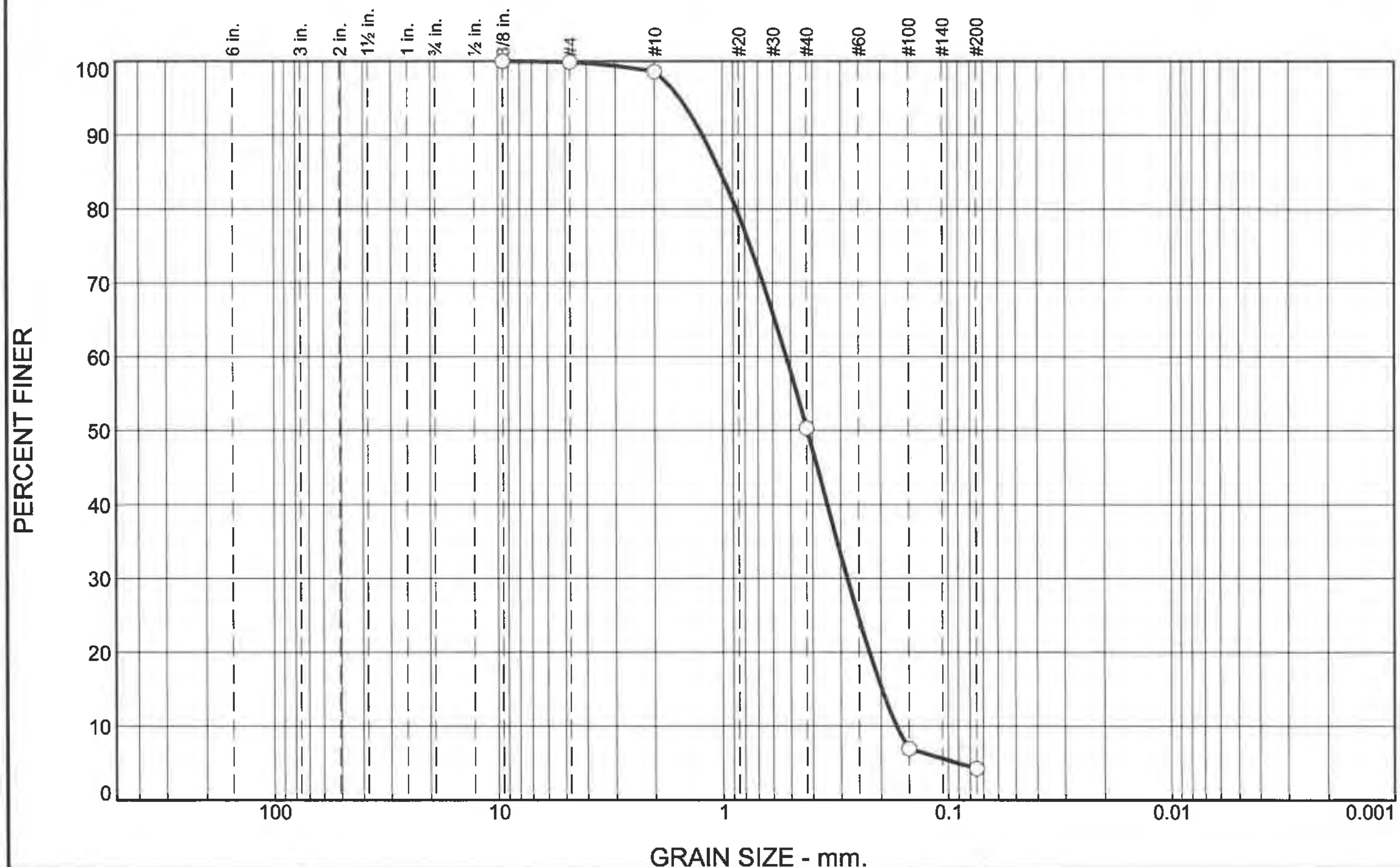
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	44.4	21.4	65.8	12.1	13.8	5.2	31.1			3.1

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.1991	0.5567	1.0390	1.6755	3.5592	6.9375	11.9012	26.0830	34.9330	37.4626	40.6773	45.0431

Fineness Modulus	C_u	C_c
6.17	46.85	0.87

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.1	1.3	48.3	46.0	4.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8"	100.0		
#4	99.9		
#10	98.6		
#40	50.3		
#100	7.0		
#200	4.3		

* (no specification provided)

Material Description

Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 1.2266 D_{85}= 1.0209 D_{60}= 0.5237
D_{50}= 0.4221 D_{30}= 0.2810 D_{15}= 0.1982
D_{10}= 0.1700 C_u= 3.08 C_c= 0.89

Classification

USCS= SP AASHTO=

Remarks

MC=29.3%

Location: AB-02
Sample Number: G-3 **Depth:** 17.5 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road

Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/24/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AB-02
Depth: 17.5 **Sample Number:** G-3
Material Description: Sand
Date: 11/20/2020
USCS Classification: SP
Testing Remarks: MC=29.3%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 305.30
Tare Wt. = 12.70
Minus #200 from wash = 4.3%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
318.50	12.70	0.00	3/8"	0.00	100.0
			#4	0.40	99.9
			#10	4.30	98.6
			#40	151.90	50.3
			#100	284.50	7.0
			#200	292.70	4.3

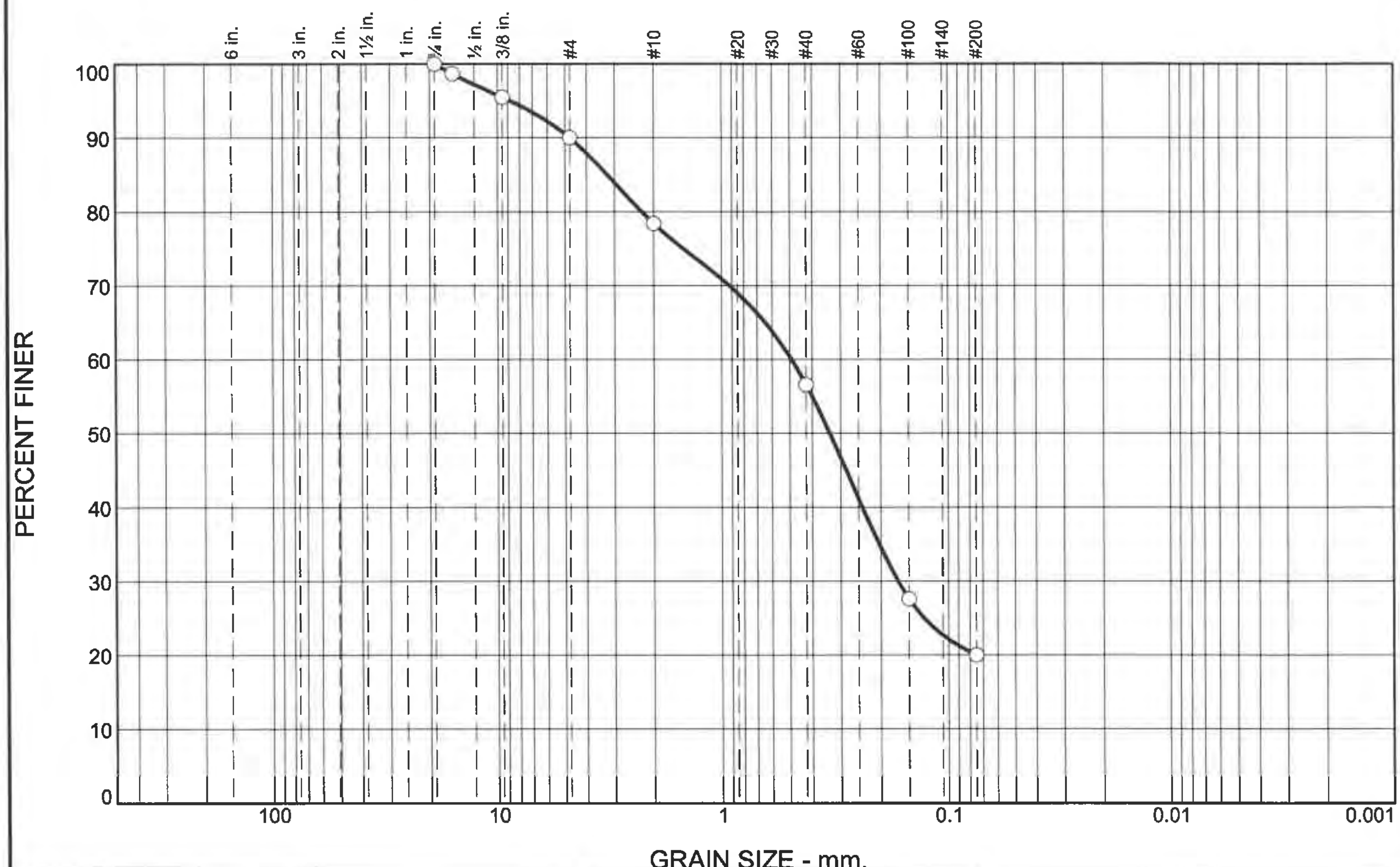
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	0.1	0.1	1.3	48.3	46.0	95.6			4.3

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.0903	0.1700	0.1982	0.2252	0.2810	0.3446	0.4221	0.5237	0.8717	1.0209	1.2266	1.5489

Fineness Modulus	C_u	C_c
2.06	3.08	0.89

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	10.0	11.6	21.8	36.6	20.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
5/8"	98.7		
3/8"	95.5		
#4	90.0		
#10	78.4		
#40	56.6		
#100	27.6		
#200	20.0		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 4.7535 D_{85}= 3.2300 D_{60}= 0.4921
D_{50}= 0.3349 D_{30}= 0.1676 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

MC=18%

Location: AB-02
Sample Number: S-6 **Depth:** 25 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AB-02
Depth: 25 **Sample Number:** S-6
Material Description: Silty sand
Date: 11/20/2020
USCS Classification: SM
Testing Remarks: MC=18%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 515.90
Tare Wt. = 15.80
Minus #200 from wash = 19.1%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
634.30	15.80	0.00	3/4"	0.00	100.0
			5/8"	8.20	98.7
			3/8"	27.80	95.5
			#4	61.90	90.0
			#10	133.60	78.4
			#40	268.50	56.6
			#100	447.70	27.6
			#200	494.90	20.0

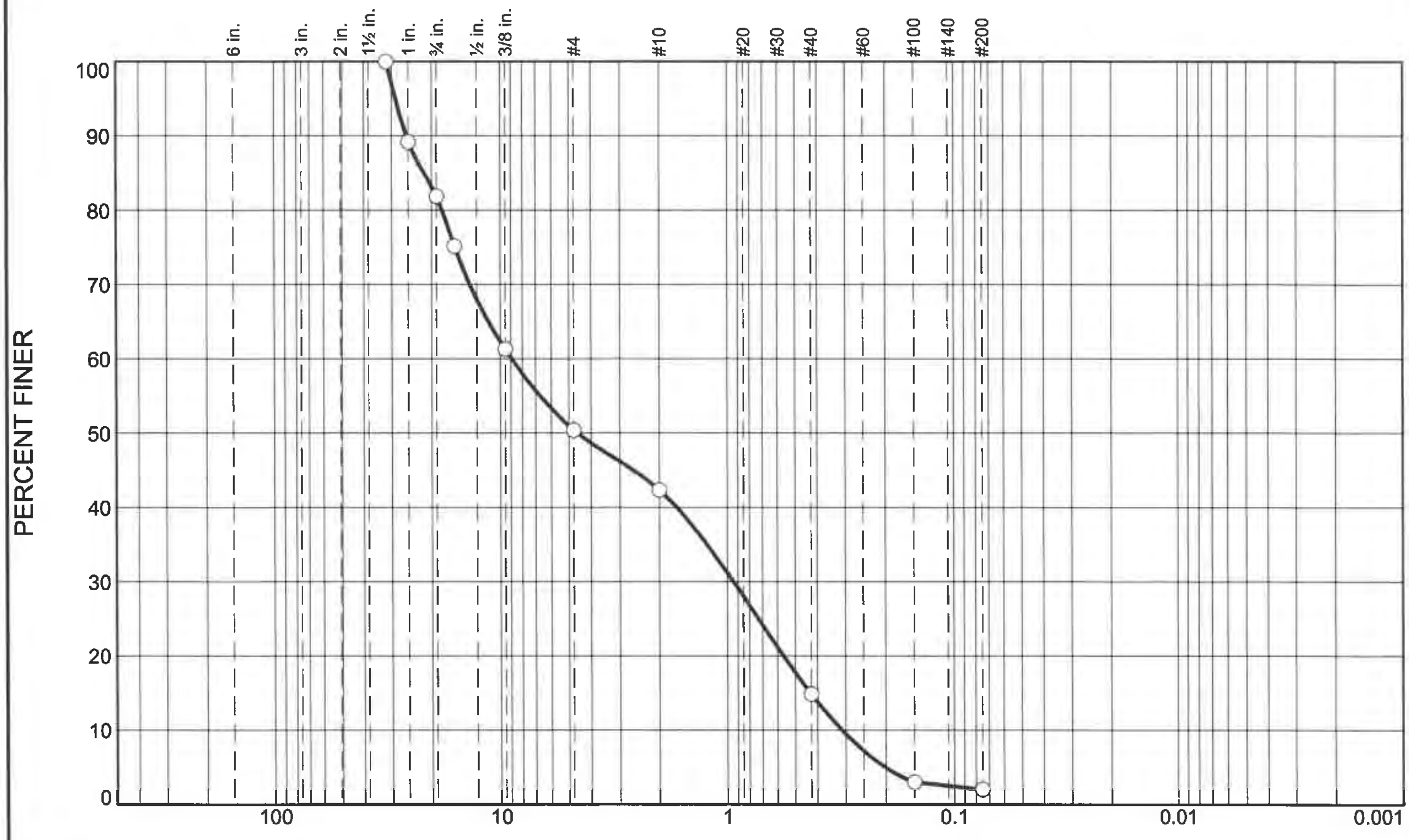
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	10.0	10.0	11.6	21.8	36.6	70.0			20.0

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
			0.0752	0.1676	0.2413	0.3349	0.4921	2.2631	3.2300	4.7535	8.7712

Fineness Modulus
2.23

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	18.1	31.5	8.1	27.4	12.9	2.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/4"	100.0		
1"	89.2		
3/4"	81.9		
5/8"	75.1		
3/8"	61.3		
#4	50.4		
#10	42.3		
#40	14.9		
#100	3.0		
#200	2.0		

* (no specification provided)

Material Description

Gravel with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 25.9629 D_{85}= 21.6359 D_{60}= 8.9062
D_{50}= 4.5851 D_{30}= 0.9416 D_{15}= 0.4283
D_{10}= 0.3107 C_u= 28.66 C_c= 0.32

Classification

USCS= GP AASHTO=

Remarks

MC=11.5%

Location: AB-03
Sample Number: S-2 **Depth:** 5 **Date:** 1/23/2021

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175 / 190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

1/23/2021

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175 / 190175-JR
Location: AB-03
Depth: 5 **Sample Number:** S-2
Material Description: Gravel with sand
Date: 1/23/2021
USCS Classification: GP
Testing Remarks: MC=11.5%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 639.60
Tare Wt. = 12.60
Minus #200 from wash = 2.0%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
652.30	12.60	0.00	1 1/4"	0.00	100.0
			1"	69.20	89.2
			3/4"	115.80	81.9
			5/8"	159.00	75.1
			3/8"	247.40	61.3
			#4	317.40	50.4
			#10	369.00	42.3
			#40	544.60	14.9
			#100	620.30	3.0
			#200	626.70	2.0

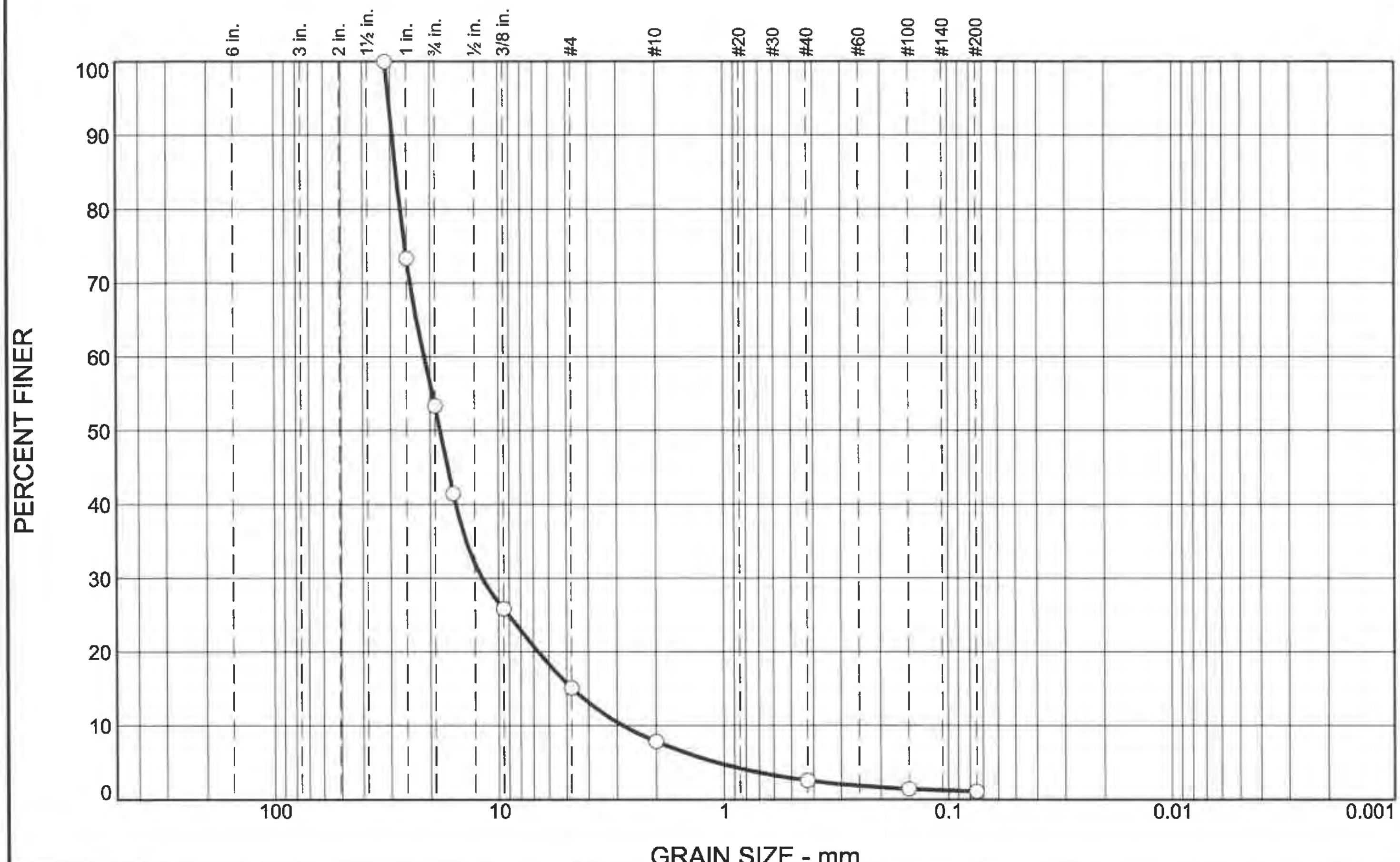
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	18.1	31.5	49.6	8.1	27.4	12.9	48.4			2.0

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.2005	0.3107	0.4283	0.5639	0.9416	1.6781	4.5851	8.9062	18.0056	21.6359	25.9629	28.9357

Fineness Modulus	C_u	C_c
4.94	28.66	0.32

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	46.6	38.3	7.2	5.3	1.5	1.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/4"	100.0		
1"	73.3		
3/4"	53.4		
5/8"	41.5		
3/8"	25.9		
#4	15.1		
#10	7.9		
#40	2.6		
#100	1.4		
#200	1.1		

* (no specification provided)

Material Description

Gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 29.4015 D_{85}= 28.2454 D_{60}= 21.2362
D_{50}= 18.1018 D_{30}= 11.8124 D_{15}= 4.7079
D_{10}= 2.8014 C_u= 7.58 C_c= 2.35

Classification

USCS= GW AASHTO=

Remarks

MC=5.1%

Location: AB-03
Sample Number: G-1 **Depth:** 12.5 **Date:** 1/23/2021

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road

Project No: 08-175 / 190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

1/23/2021

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175 / 190175-JR
Location: AB-03
Depth: 12.5 **Sample Number:** G-1
Material Description: Gravel
Date: 1/23/2021
USCS Classification: GW
Testing Remarks: MC=5.1%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 580.30
Tare Wt. = 12.50
Minus #200 from wash = 1.1%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
586.90	12.50	0.00	1 1/4"	0.00	100.0
			1"	153.20	73.3
			3/4"	267.90	53.4
			5/8"	336.10	41.5
			3/8"	425.80	25.9
			#4	487.60	15.1
			#10	529.20	7.9
			#40	559.60	2.6
			#100	566.20	1.4
			#200	568.00	1.1

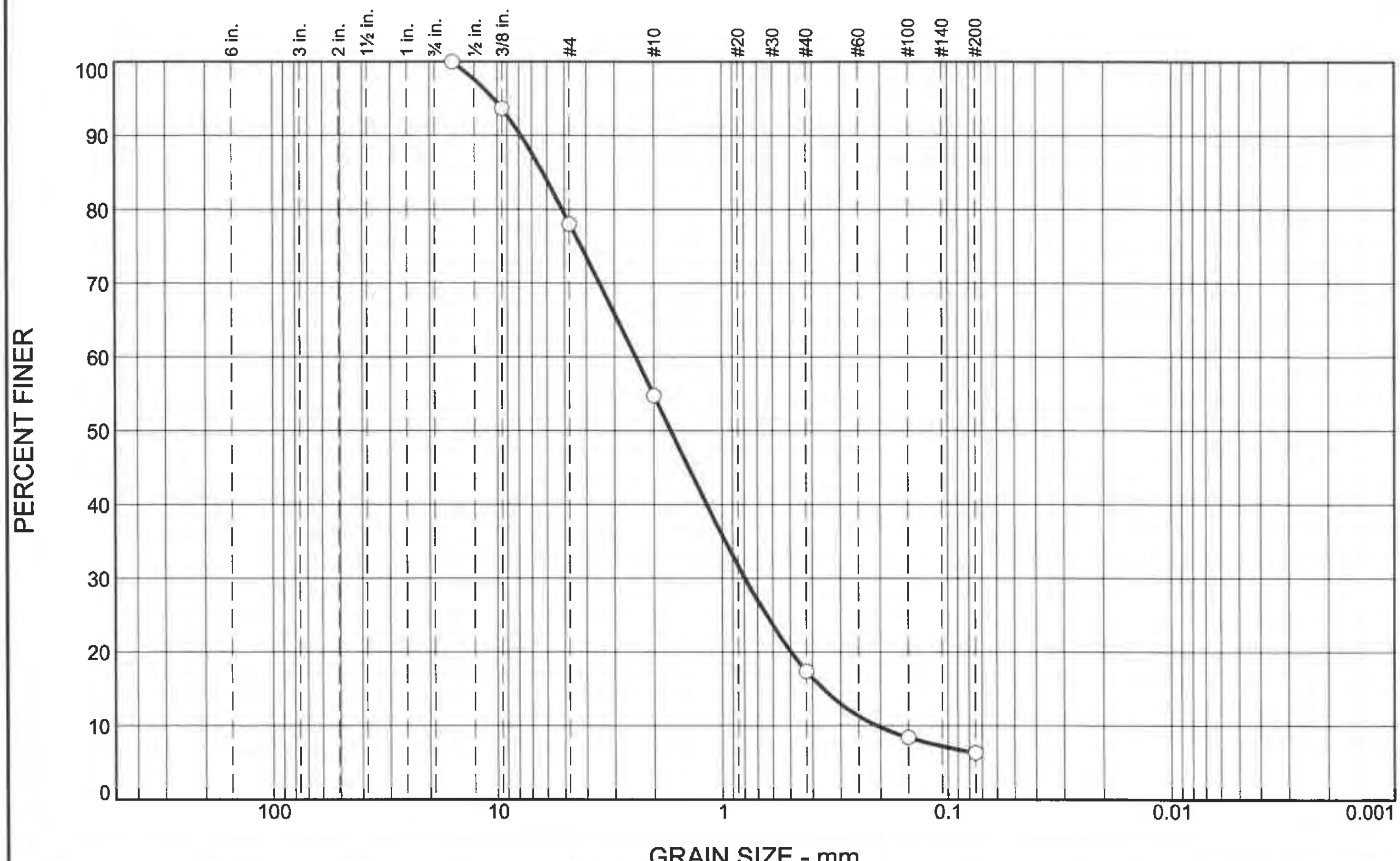
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	46.6	38.3	84.9	7.2	5.3	1.5	14.0			1.1

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
1.0679	2.8014	4.7079	6.6739	11.8124	15.4606	18.1018	21.2362	27.0697	28.2454	29.4015	30.5628

Fineness Modulus	C_u	C_c
6.85	7.58	2.35

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	22.0	23.3	37.3	11.1	6.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
5/8"	100.0		
3/8"	93.7		
#4	78.0		
#10	54.7		
#40	17.4		
#100	8.4		
#200	6.3		

* (no specification provided)

Material Description

Sand with silt and gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 7.8236 D_{85}= 6.2669 D_{60}= 2.4198
D_{50}= 1.6869 D_{30}= 0.7945 D_{15}= 0.3587
D_{10}= 0.2061 C_u= 11.74 C_c= 1.27

Classification

USCS= SW-SM AASHTO=

Remarks

MC=10.8%

Location: AB-03
Sample Number: G-2 **Depth:** 25 **Date:** 1/23/2021

Hayre McElroy & Associates, LLC
Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175 / 190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

1/23/2021

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175 / 190175-JR
Location: AB-03
Depth: 25 **Sample Number:** G-2
Material Description: Sand with silt and gravel
Date: 1/23/2021
USCS Classification: SW-SM
Testing Remarks: MC=10.8%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 486.30
Tare Wt. = 12.70
Minus #200 from wash = 6.0%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
516.70	12.70	0.00	5/8"	0.00	100.0
			3/8"	31.80	93.7
			#4	110.80	78.0
			#10	228.10	54.7
			#40	416.40	17.4
			#100	461.50	8.4
			#200	472.10	6.3

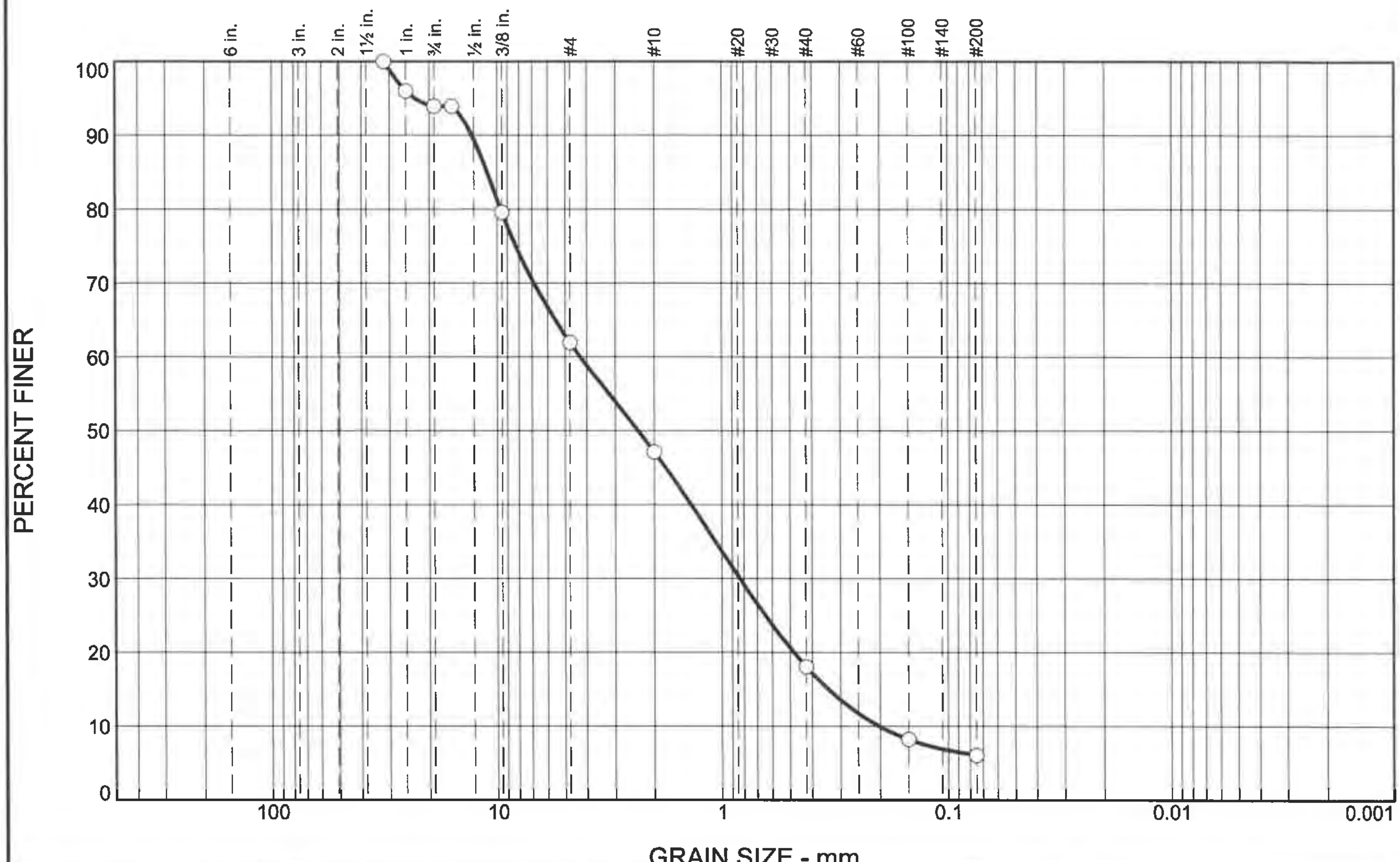
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	22.0	22.0	23.3	37.3	11.1	71.7			6.3

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
	0.2061	0.3587	0.4967	0.7945	1.1733	1.6869	2.4198	5.1294	6.2669	7.8236	10.3612

Fineness Modulus	C_u	C_c
3.84	11.74	1.27

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	6.1	32.0	14.8	29.0	12.0	6.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/4"	100.0		
1"	96.0		
3/4"	93.9		
5/8"	93.9		
3/8"	79.6		
#4	61.9		
#10	47.1		
#40	18.1		
#100	8.2		
#200	6.1		

* (no specification provided)

Material Description

Sand with silt and gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 12.9177 D_{85}= 11.0939 D_{60}= 4.2754
D_{50}= 2.3603 D_{30}= 0.8342 D_{15}= 0.3388
D_{10}= 0.2012 C_u= 21.25 C_c= 0.81

Classification

USCS= SP-SM AASHTO=

Remarks

MC=12.6%

Location: AB-03
Sample Number: S-8 **Depth:** 35 **Date:** 1/23/2021

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175 / 190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

1/23/2021

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175 / 190175-JR
Location: AB-03
Depth: 35 **Sample Number:** S-8
Material Description: Sand with silt and gravel
Date: 1/23/2021
USCS Classification: SP-SM
Testing Remarks: MC=12.6%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 532.00
Tare Wt. = 12.70
Minus #200 from wash = 6.1%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
565.60	12.70	0.00	1 1/4"	0.00	100.0
			1"	22.20	96.0
			3/4"	33.60	93.9
			5/8"	33.60	93.9
			3/8"	113.00	79.6
			#4	210.40	61.9
			#10	292.30	47.1
			#40	453.10	18.1
			#100	507.30	8.2
			#200	519.20	6.1

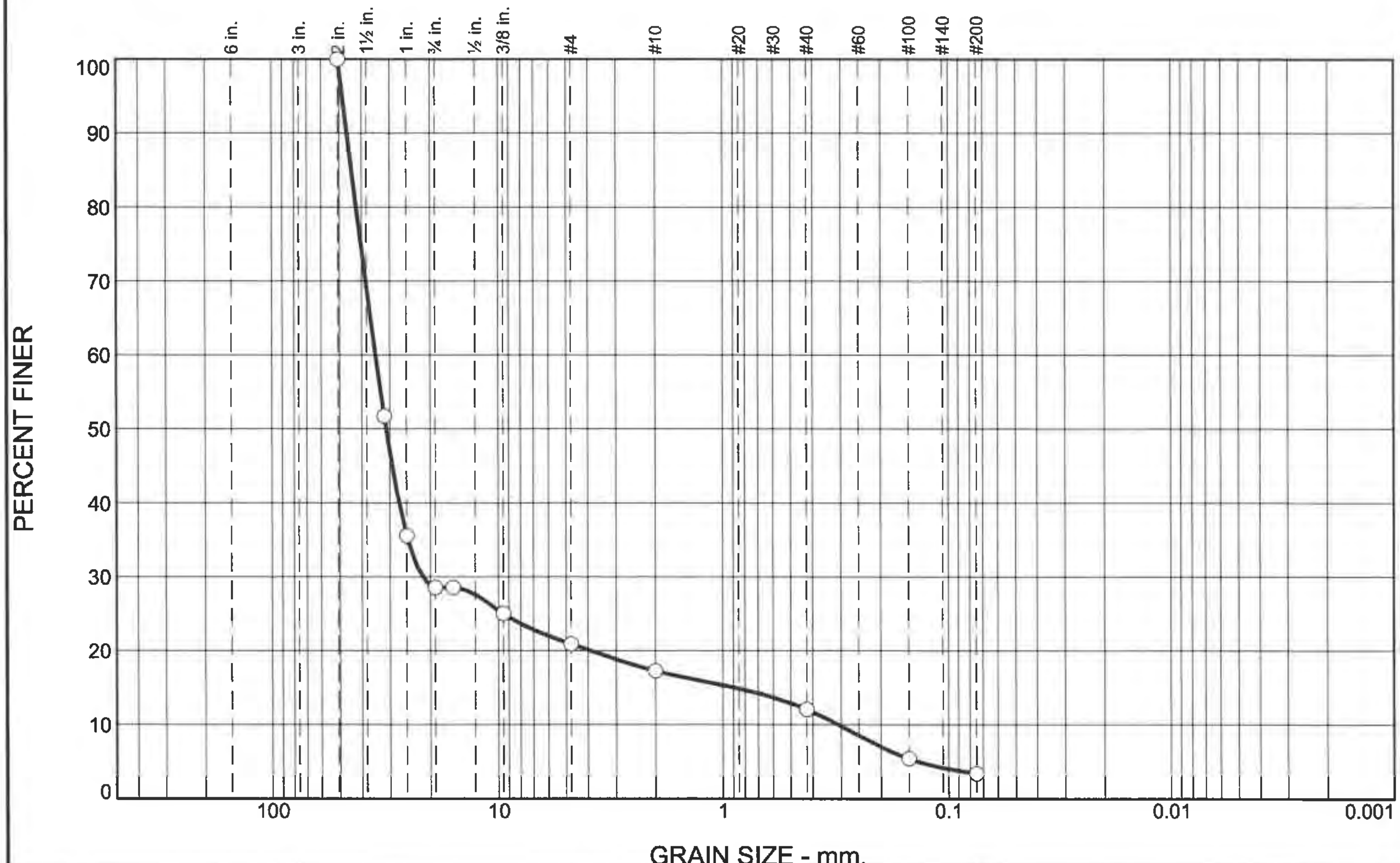
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	6.1	32.0	38.1	14.8	29.0	12.0	55.8			6.1

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
	0.2012	0.3388	0.4822	0.8342	1.3735	2.3603	4.2754	9.6462	11.0939	12.9177	23.0317

Fineness Modulus	C_u	C_c
4.32	21.25	0.81

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	71.4	7.7	3.6	5.2	8.7	3.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2"	100.0		
1 1/4"	51.7		
1"	35.6		
3/4"	28.6		
5/8"	28.6		
3/8"	25.0		
#4	20.9		
#10	17.3		
#40	12.1		
#100	5.4		
#200	3.4		

* (no specification provided)

Material Description

Gravel with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 46.3725 D_{85}= 44.2865 D_{60}= 34.7311
D_{50}= 31.1244 D_{30}= 21.6277 D_{15}= 0.8844
D_{10}= 0.3080 C_u= 112.75 C_c= 43.72

Classification

USCS= GP AASHTO=

Remarks

MC=3.6%

Location: AB-04
Sample Number: G-1 **Depth:** 2.5 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road

Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AB-04
Depth: 2.5 **Sample Number:** G-1
Material Description: Gravel with sand
Date: 11/20/2020
USCS Classification: GP
Testing Remarks: MC=3.6%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 580.30
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
597.30	12.60	0.00	2"	0.00	100.0
			1 1/4"	282.20	51.7
			1"	376.80	35.6
			3/4"	417.70	28.6
			5/8"	417.70	28.6
			3/8"	438.30	25.0
			#4	462.40	20.9
			#10	483.60	17.3
			#40	514.20	12.1
			#100	553.00	5.4
			#200	564.80	3.4

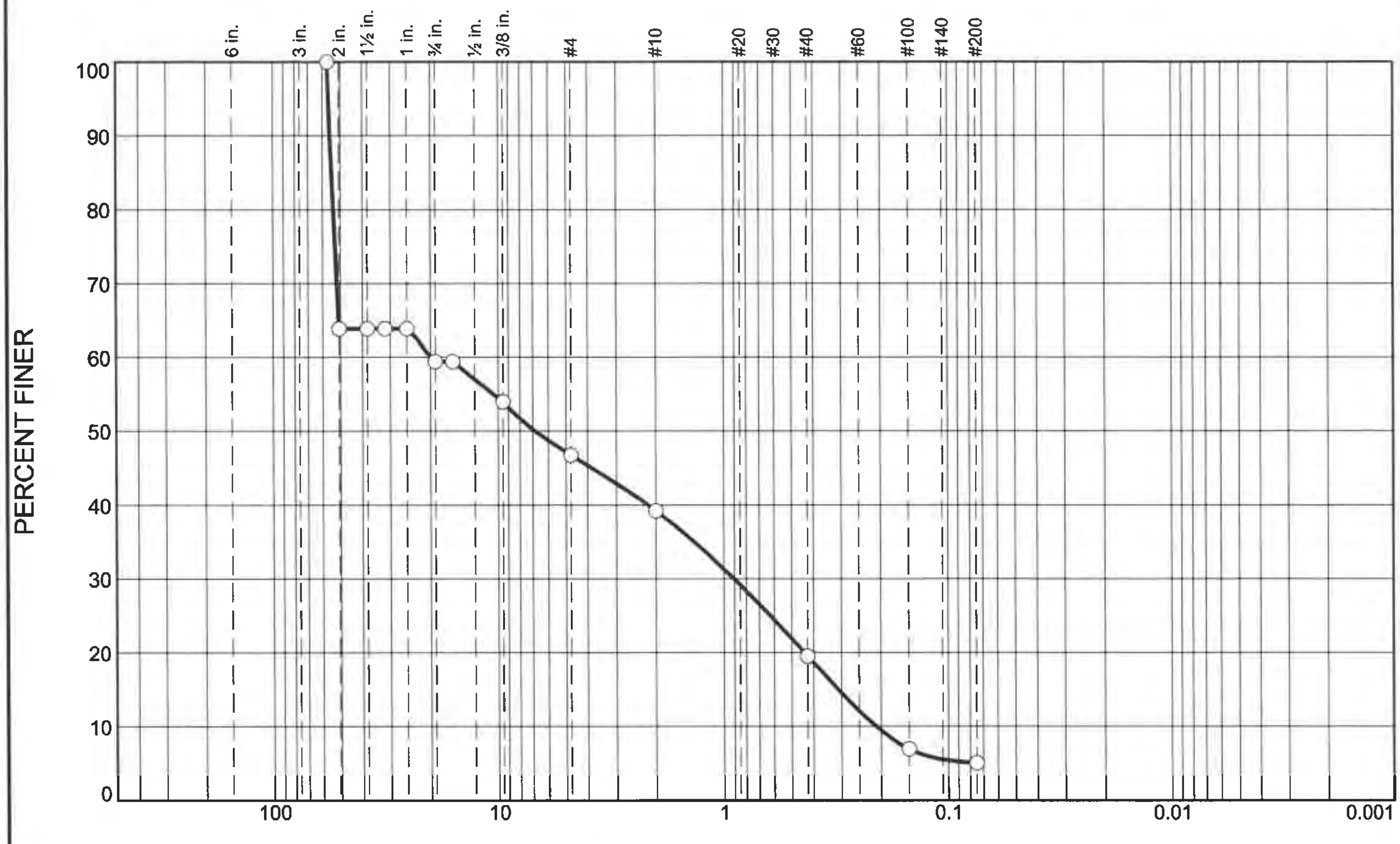
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	71.4	7.7	79.1	3.6	5.2	8.7	17.5			3.4

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.1361	0.3080	0.8844	3.8829	21.6277	27.3638	31.1244	34.7311	42.2716	44.2865	46.3725	48.5399

Fineness Modulus	C_u	C_c
6.94	112.75	43.72

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	40.6	12.6	7.6	19.7	14.4	5.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2 1/4"	100.0		
2"	63.9		
1.5"	63.9		
1 1/4"	63.9		
1"	63.9		
3/4"	59.4		
5/8"	59.4		
3/8"	54.0		
#4	46.8		
#10	39.2		
#40	19.5		
#100	7.0		
#200	5.1		

* (no specification provided)

Material Description

Gravel with silt and sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 55.5582 D_{85}= 54.7542 D_{60}= 19.9991
D_{50}= 6.8319 D_{30}= 0.9055 D_{15}= 0.3104
D_{10}= 0.2100 C_u= 95.22 C_c= 0.20

Classification

USCS= GP-GM AASHTO=

Remarks

MC=9.8%

Location: AB-04
Sample Number: S-1 **Depth:** 5 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AB-04
Depth: 5 **Sample Number:** S-1
Material Description: Gravel with silt and sand
Date: 11/20/2020
USCS Classification: GP-GM
Testing Remarks: MC=9.8%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 599.50
Tare Wt. = 12.70
Minus #200 from wash = 4.7%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
628.30	12.70	0.00	2 1/4"	0.00	100.0
			2"	222.30	63.9
			1.5"	222.30	63.9
			1 1/4"	222.30	63.9
			1"	222.30	63.9
			3/4"	249.80	59.4
			5/8"	249.80	59.4
			3/8"	283.40	54.0
			#4	327.70	46.8
			#10	374.30	39.2
			#40	495.40	19.5
			#100	572.70	7.0
			#200	584.50	5.1

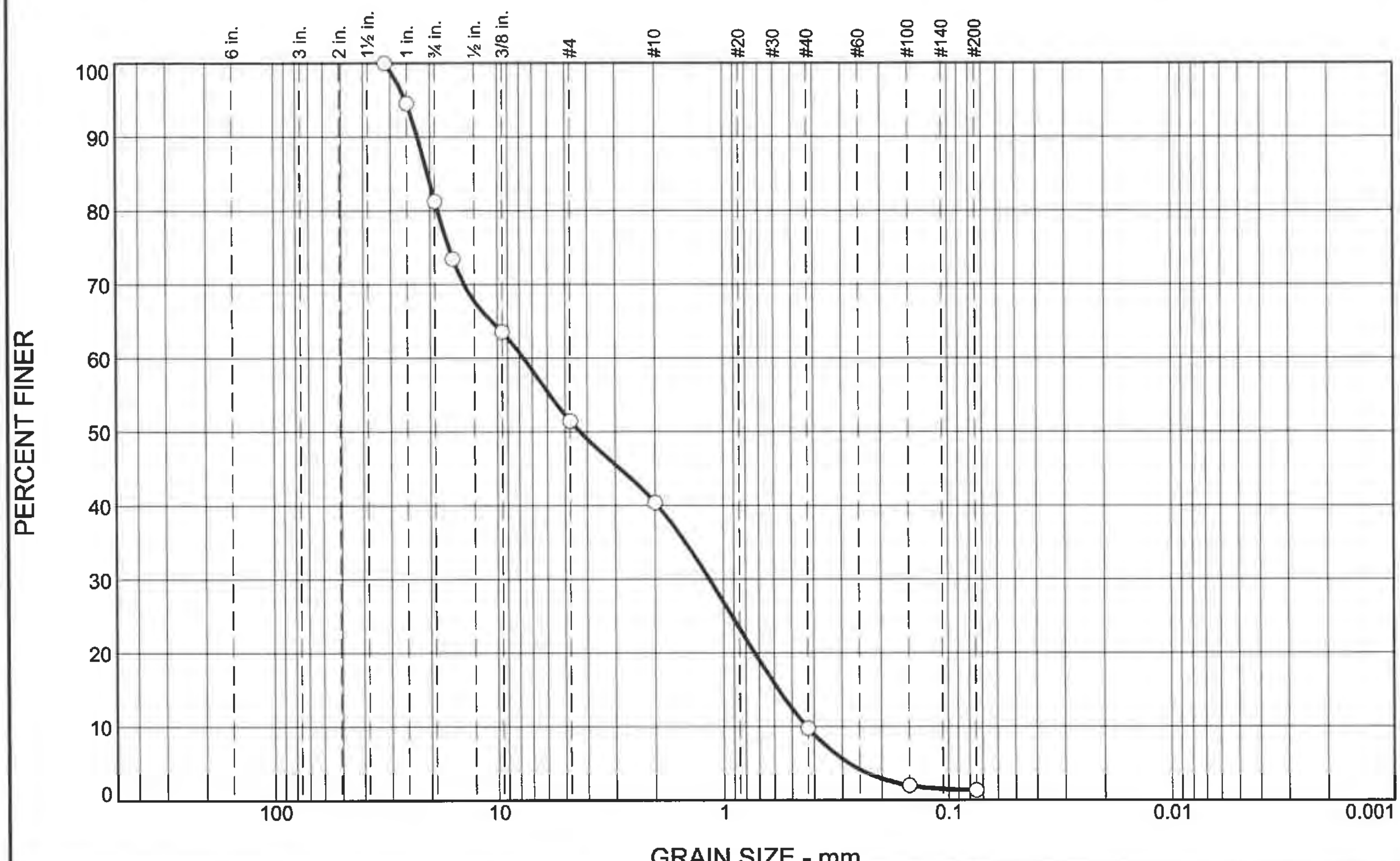
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	40.6	12.6	53.2	7.6	19.7	14.4	41.7			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}
	0.2100	0.3104	0.4391	0.9055	2.1704	6.8319	19.9991	53.9284	54.7542	55.5582	56.3534

Fineness Modulus	C_u	C_c
5.56	95.22	0.20

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	18.7	29.9	11.0	30.6	8.4	1.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/4"	100.0		
1"	94.5		
3/4"	81.3		
5/8"	73.4		
3/8"	63.6		
#4	51.4		
#10	40.4		
#40	9.8		
#100	2.0		
#200	1.4		

* (no specification provided)

Material Description

Sand with gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 22.7530 D_{85}= 20.5342 D_{60}= 7.6621
D_{50}= 4.3083 D_{30}= 1.1566 D_{15}= 0.5734
D_{10}= 0.4301 C_u= 17.81 C_c= 0.41

Classification

USCS= SP AASHTO=

Remarks

MC=11.7%

Location: AB-04
Sample Number: S-2 **Depth:** 7.5 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AB-04
Depth: 7.5
Sample Number: S-2
Material Description: Sand with gravel
Date: 11/20/2020
USCS Classification: SP
Testing Remarks: MC=11.7%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 554.60
Tare Wt. = 12.60
Minus #200 from wash = 0.9%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
559.40	12.60	0.00	1 1/4"	0.00	100.0
			1"	29.90	94.5
			3/4"	102.40	81.3
			5/8"	145.20	73.4
			3/8"	199.30	63.6
			#4	265.50	51.4
			#10	326.00	40.4
			#40	493.10	9.8
			#100	535.90	2.0
			#200	539.40	1.4

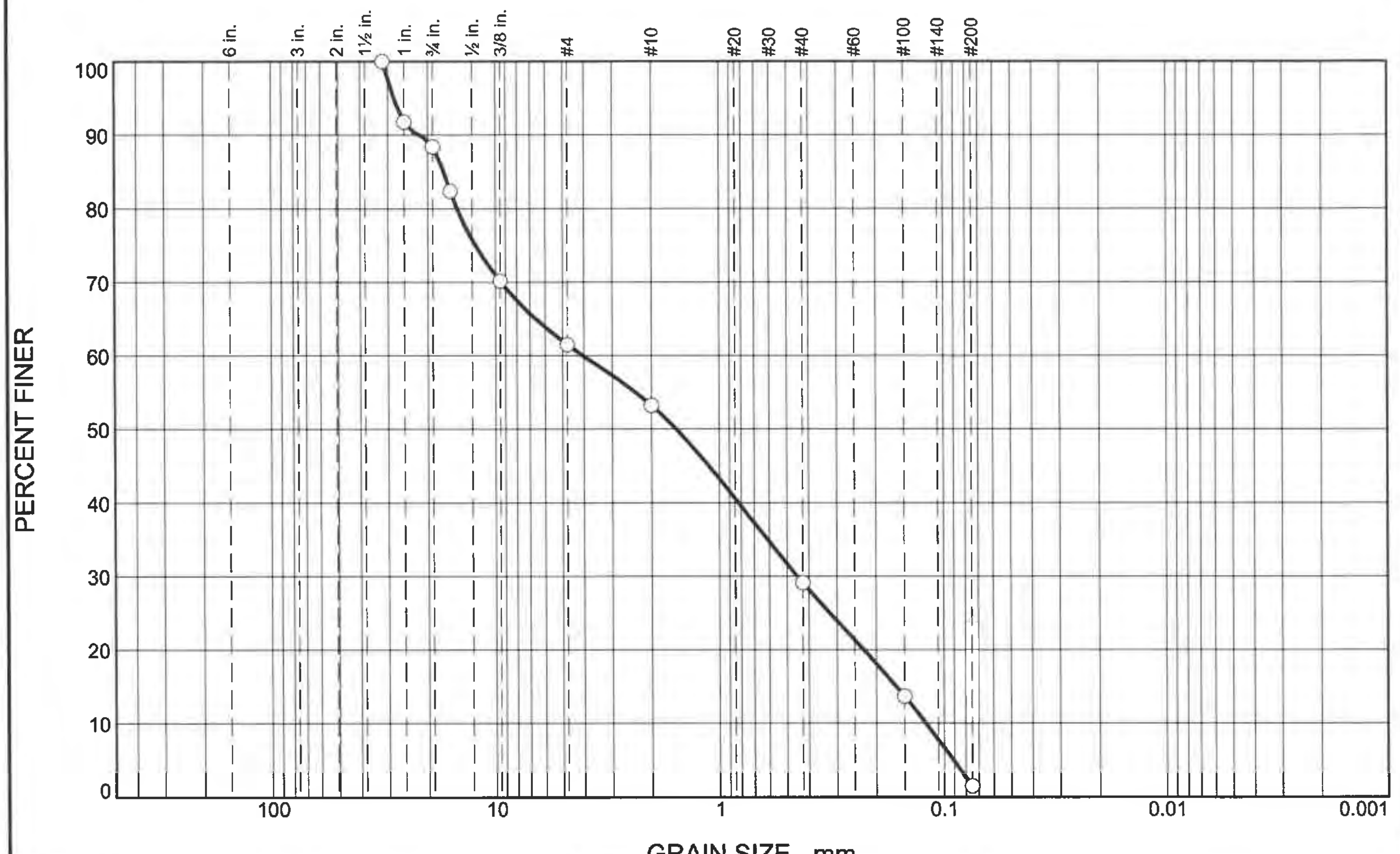
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	18.7	29.9	48.6	11.0	30.6	8.4	50.0			1.4

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.2782	0.4301	0.5734	0.7320	1.1566	1.9513	4.3083	7.6621	18.5523	20.5342	22.7530	25.7534

Fineness Modulus	C_u	C_c
5.07	17.81	0.41

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	11.6	26.9	8.3	24.1	27.6	1.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/4"	100.0		
1"	91.8		
3/4"	88.4		
5/8"	82.4		
3/8"	70.2		
#4	61.5		
#10	53.2		
#40	29.1		
#100	13.7		
#200	1.5		

* (no specification provided)

Material Description

Sand with gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 21.6344 D_{85}= 17.0533 D_{60}= 4.0452
D_{50}= 1.5548 D_{30}= 0.4500 D_{15}= 0.1630
D_{10}= 0.1203 C_u= 33.61 C_c= 0.42

Classification

USCS= SP AASHTO=

Remarks

MC=18.3%

Location: AB-04
Sample Number: S-3 **Depth:** 10 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AB-04
Depth: 10 **Sample Number:** S-3
Material Description: Sand with gravel
Date: 11/20/2020
USCS Classification: SP
Testing Remarks: MC=18.3%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 268.30
Tare Wt. = 12.90
Minus #200 from wash = 1.5%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
272.30	12.90	0.00	1 1/4"	0.00	100.0
			1"	21.30	91.8
			3/4"	30.10	88.4
			5/8"	45.70	82.4
			3/8"	77.40	70.2
			#4	99.90	61.5
			#10	121.30	53.2
			#40	183.90	29.1
			#100	223.90	13.7
			#200	255.50	1.5

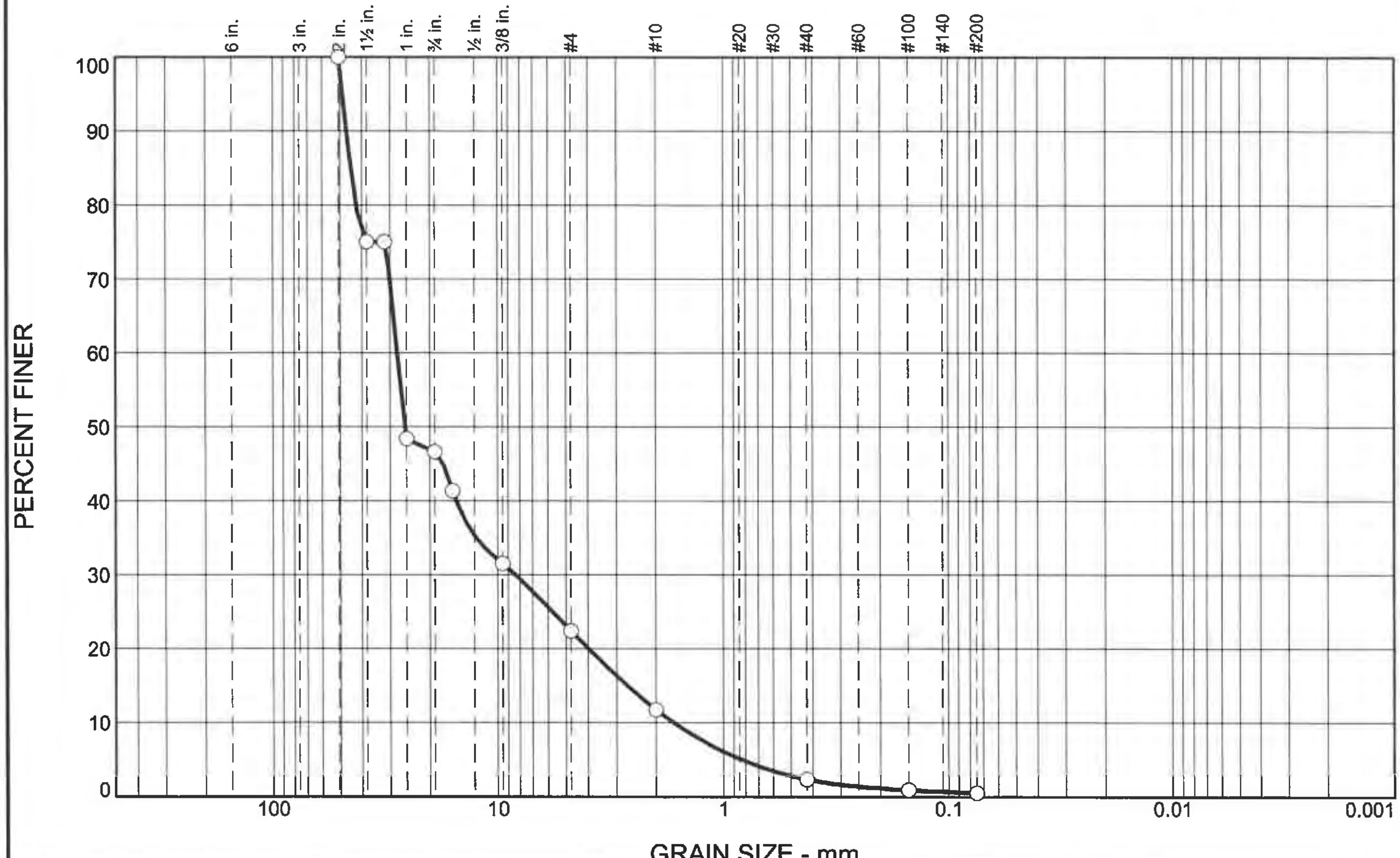
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	11.6	26.9	38.5	8.3	24.1	27.6	60.0			1.5

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.0908	0.1203	0.1630	0.2279	0.4500	0.8222	1.5548	4.0452	14.7854	17.0533	21.6344	28.2114

Fineness Modulus	C_u	C_c
4.07	33.61	0.42

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	53.4	24.2	10.6	9.4	1.9	0.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2"	100.0		
1.5"	75.0		
1 1/4"	75.0		
1"	48.4		
3/4"	46.6		
5/8"	41.3		
3/8"	31.5		
#4	22.4		
#10	11.8		
#40	2.4		
#100	0.9		
#200	0.5		

* (no specification provided)

Material Description

Gravel with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 46.8331 D_{85}= 44.7610 D_{60}= 27.8974
D_{50}= 25.8221 D_{30}= 8.3592 D_{15}= 2.6826
D_{10}= 1.6681 C_u= 16.72 C_c= 1.50

Classification

USCS= GW AASHTO=

Remarks

MC=5.2%

Location: AB-04
Sample Number: G-2 **Depth:** 12.5 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AB-04
Depth: 12.5 **Sample Number:** G-2
Material Description: Gravel with sand
Date: 11/20/2020
USCS Classification: GW
Testing Remarks: MC=5.2%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 586.60
Tare Wt. = 12.70
Minus #200 from wash = 0.7%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
590.50	12.70	0.00	2"	0.00	100.0
			1.5"	144.30	75.0
			1 1/4"	144.30	75.0
			1"	298.10	48.4
			3/4"	308.40	46.6
			5/8"	339.00	41.3
			3/8"	395.60	31.5
			#4	448.40	22.4
			#10	509.90	11.8
			#40	564.20	2.4
			#100	572.50	0.9
			#200	574.90	0.5

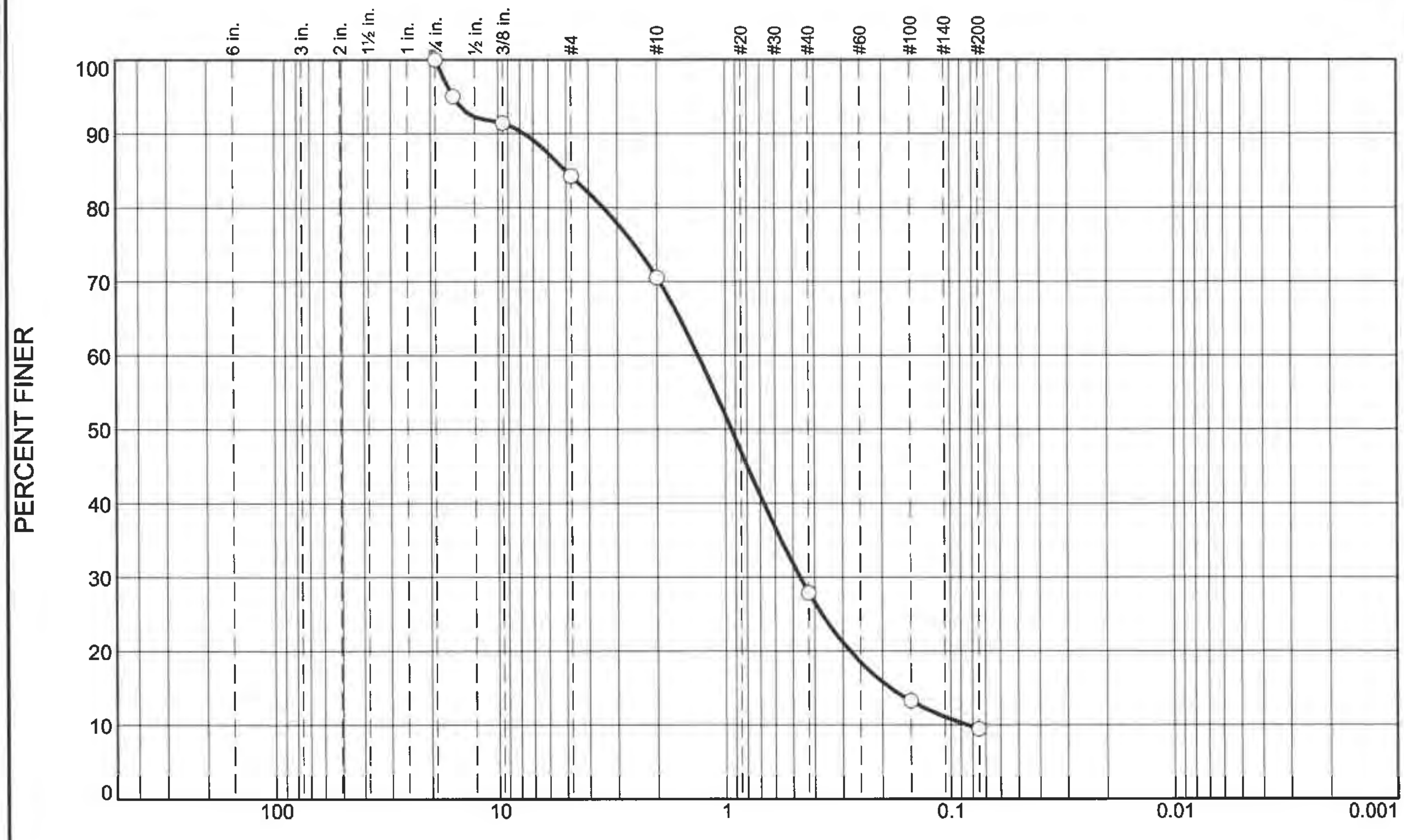
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	53.4	24.2	77.6	10.6	9.4	1.9	21.9			0.5

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.8307	1.6681	2.6826	3.9816	8.3592	15.2920	25.8221	27.8974	42.3961	44.7610	46.8331	48.8140

Fineness Modulus	C_u	C_c
6.98	16.72	1.50

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	15.7	13.8	42.6	18.4	9.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
5/8"	95.1		
3/8"	91.5		
#4	84.3		
#10	70.5		
#40	27.9		
#100	13.3		
#200	9.5		

* (no specification provided)

Material Description

Sand with silt and gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 7.6273 D_{85}= 5.0081 D_{60}= 1.3229
D_{50}= 0.9422 D_{30}= 0.4650 D_{15}= 0.1838
D_{10}= 0.0840 C_u= 15.75 C_c= 1.95

Classification

USCS= SW-SM AASHTO=

Remarks

MC=16.6%

Location: AB-04
Sample Number: S-4 **Depth:** 15 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AB-04
Depth: 15 **Sample Number:** S-4
Material Description: Sand with silt and gravel
Date: 11/20/2020
USCS Classification: SW-SM
Testing Remarks: MC=16.6%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 353.70
Tare Wt. = 12.70
Minus #200 from wash = 9.1%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
387.70	12.70	0.00	3/4"	0.00	100.0
			5/8"	18.50	95.1
			3/8"	32.00	91.5
			#4	58.90	84.3
			#10	110.60	70.5
			#40	270.40	27.9
			#100	325.20	13.3
			#200	339.40	9.5

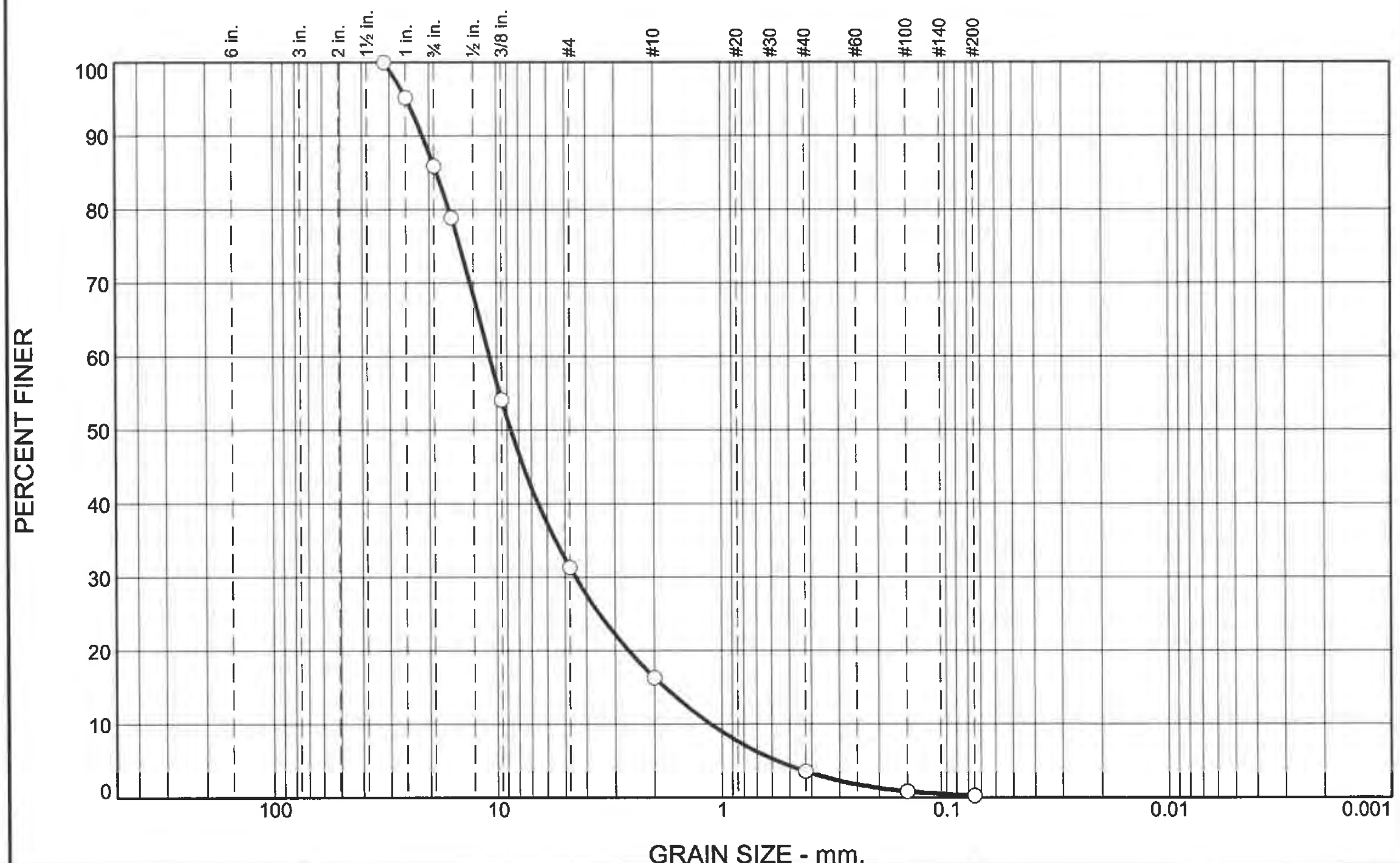
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	15.7	15.7	13.8	42.6	18.4	74.8			9.5

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
	0.0840	0.1838	0.2781	0.4650	0.6739	0.9422	1.3229	3.4585	5.0081	7.6273	15.8247

Fineness Modulus	C_u	C_c
3.23	15.75	1.95

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	14.1	54.6	15.0	12.7	3.4	0.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/4"	100.0		
1"	95.2		
3/4"	85.9		
5/8"	78.9		
3/8"	54.1		
#4	31.3		
#10	16.3		
#40	3.6		
#100	0.8		
#200	0.2		

* (no specification provided)

Material Description

Gravel with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 21.4124 D_{85}= 18.5970 D_{60}= 10.7743
D_{50}= 8.6717 D_{30}= 4.4839 D_{15}= 1.7957
D_{10}= 1.1207 C_u= 9.61 C_c= 1.67

Classification

USCS= GW AASHTO=

Remarks

MC=6.5%

Location: AB-05
Sample Number: S-1 **Depth:** 2.5 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC
Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AB-05
Depth: 2.5 **Sample Number:** S-1
Material Description: Gravel with sand
Date: 11/20/2020
USCS Classification: GW
Testing Remarks: MC=6.5%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 903.20
Tare Wt. = 193.00
Minus #200 from wash = 0.4%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
905.80	193.00	0.00	1 1/4"	0.00	100.0
			1"	34.10	95.2
			3/4"	100.70	85.9
			5/8"	150.60	78.9
			3/8"	327.30	54.1
			#4	489.60	31.3
			#10	596.30	16.3
			#40	687.40	3.6
			#100	707.10	0.8
			#200	711.20	0.2

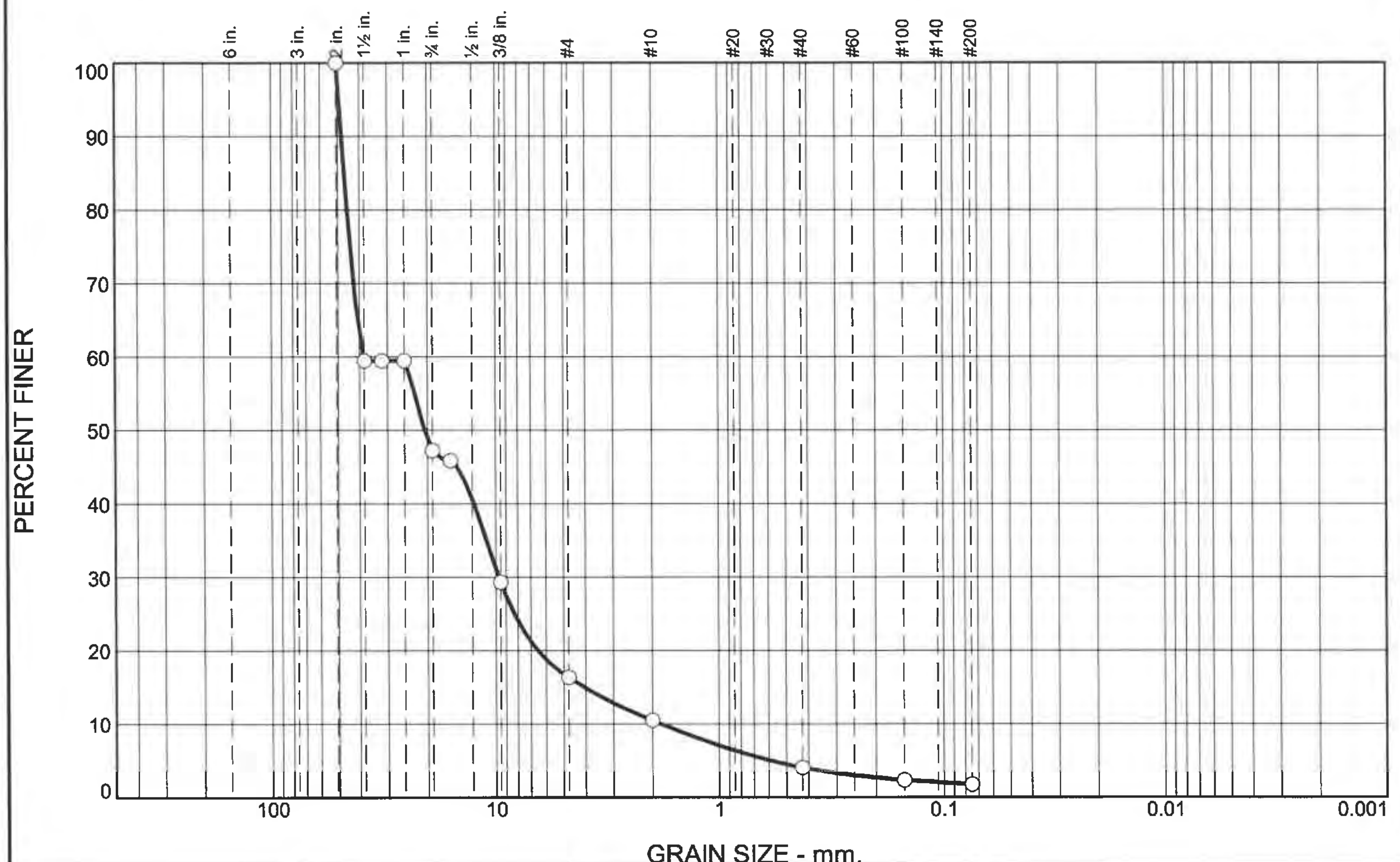
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	14.1	54.6	68.7	15.0	12.7	3.4	31.1			0.2

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.5665	1.1207	1.7957	2.6026	4.4839	6.5677	8.6717	10.7743	16.3125	18.5970	21.4124	25.1928

Fineness Modulus	C_u	C_c
5.91	9.61	1.67

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	52.7	30.9	5.9	6.4	2.4	1.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2"	100.0		
1.5"	59.5		
1 1/4"	59.5		
1"	59.5		
3/4"	47.3		
5/8"	45.9		
3/8"	29.3		
#4	16.4		
#10	10.5		
#40	4.1		
#100	2.3		
#200	1.7		

* (no specification provided)

Material Description

Gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 48.1072 D_{85}= 46.7714 D_{60}= 38.4699
D_{50}= 20.5939 D_{30}= 9.6997 D_{15}= 4.0395
D_{10}= 1.8183 C_u= 21.16 C_c= 1.35

Classification

USCS= GW AASHTO=

Remarks

MC=3.3%

Location: AB-05
Sample Number: G-1 **Depth:** 7.5 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AB-05
Depth: 7.5 **Sample Number:** G-1
Material Description: Gravel
Date: 11/20/2020
USCS Classification: GW
Testing Remarks: MC=3.3%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 685.20
Tare Wt. = 12.50
Minus #200 from wash = 1.7%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
696.60	12.50	0.00	2"	0.00	100.0
			1.5"	277.40	59.5
			1 1/4"	277.40	59.5
			1"	277.40	59.5
			3/4"	360.80	47.3
			5/8"	370.10	45.9
			3/8"	483.40	29.3
			#4	572.20	16.4
			#10	612.20	10.5
			#40	656.20	4.1
			#100	668.10	2.3
			#200	672.20	1.7

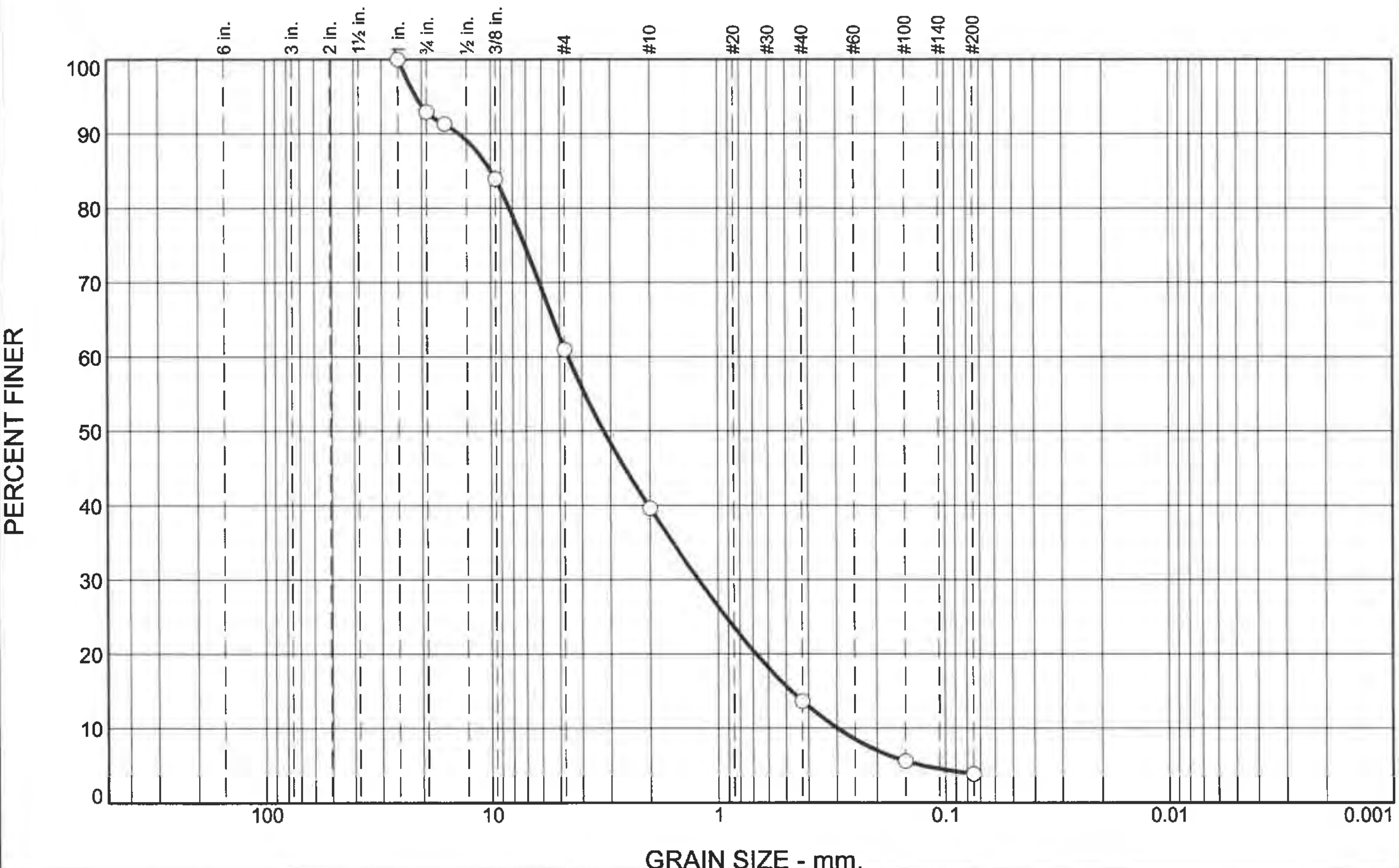
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	52.7	30.9	83.6	5.9	6.4	2.4	14.7			1.7

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.5801	1.8183	4.0395	6.5195	9.6997	12.4894	20.5939	38.4699	45.4148	46.7714	48.1072	49.4440

Fineness Modulus	C_u	C_c
7.18	21.16	1.35

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	7.1	31.9	21.4	25.9	9.8	3.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1"	100.0		
3/4"	92.9		
5/8"	91.3		
3/8"	83.9		
#4	61.0		
#10	39.6		
#40	13.7		
#100	5.6		
#200	3.9		

* (no specification provided)

Material Description

Sand with gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 13.6354 D_{85}= 9.9715 D_{60}= 4.6057
D_{50}= 3.2186 D_{30}= 1.2215 D_{15}= 0.4743
D_{10}= 0.2979 C_u= 15.46 C_c= 1.09

Classification

USCS= SW AASHTO=

Remarks

MC=9.6%

Location: AB-05
Sample Number: S-4 **Depth:** 10 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AB-05
Depth: 10 **Sample Number:** S-4
Material Description: Sand with gravel
Date: 11/20/2020
USCS Classification: SW
Testing Remarks: MC=9.6%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 522.10
Tare Wt. = 12.40
Minus #200 from wash = 4.0%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
543.10	12.40	0.00	1"	0.00	100.0
			3/4"	37.50	92.9
			5/8"	46.10	91.3
			3/8"	85.30	83.9
			#4	207.00	61.0
			#10	320.60	39.6
			#40	458.20	13.7
			#100	500.90	5.6
			#200	510.20	3.9

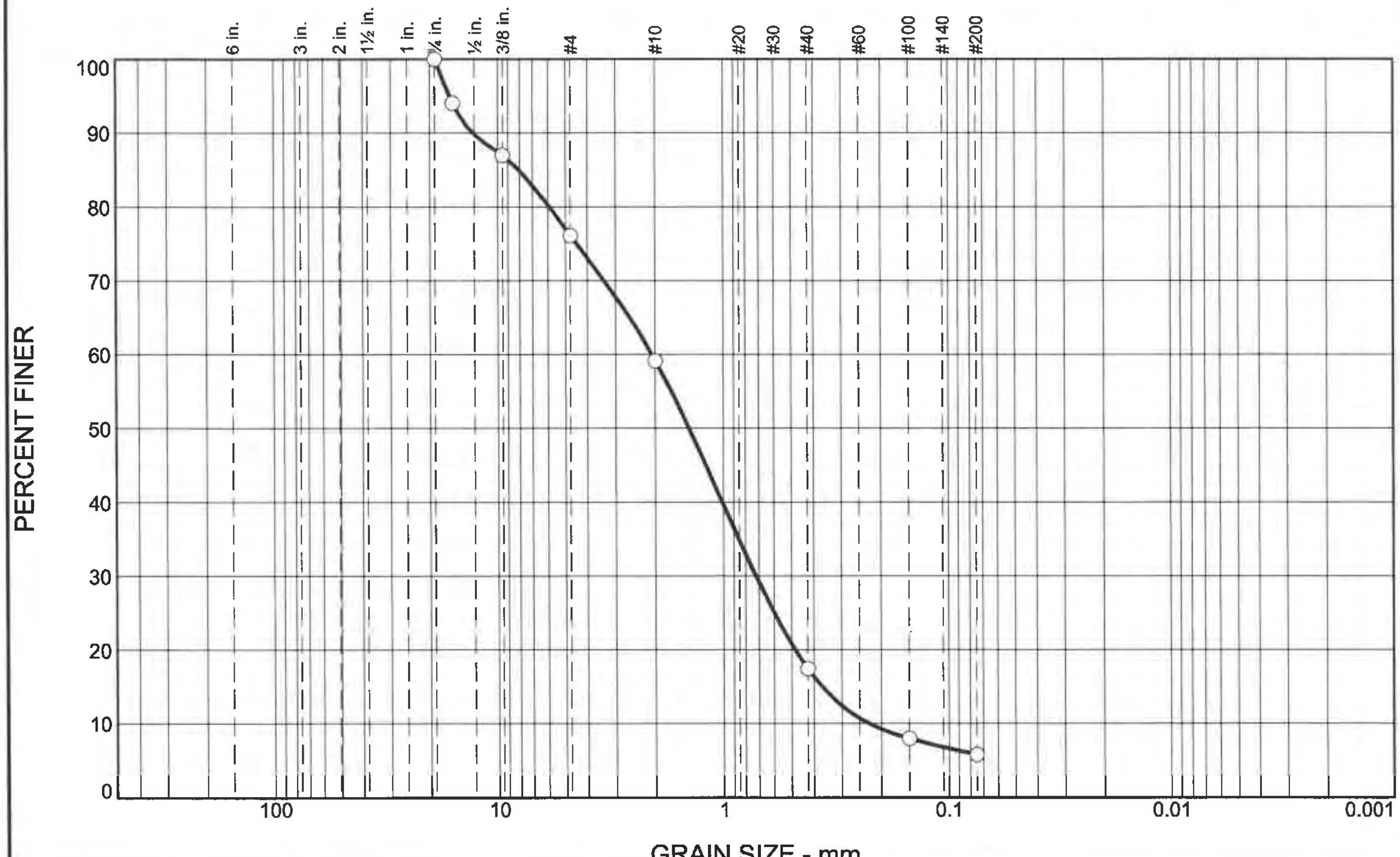
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	7.1	31.9	39.0	21.4	25.9	9.8	57.1			3.9

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.1258	0.2979	0.4743	0.6780	1.2215	2.0409	3.2186	4.6057	8.2648	9.9715	13.6354	21.1859

Fineness Modulus	C_u	C_c
4.56	15.46	1.09

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	23.9	17.0	41.7	11.6	5.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
5/8"	94.1		
3/8"	87.0		
#4	76.1		
#10	59.1		
#40	17.4		
#100	8.0		
#200	5.8		

* (no specification provided)

Material Description

Sand with silt and gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 12.8545 D_{85}= 8.0799 D_{60}= 2.0725
D_{50}= 1.4225 D_{30}= 0.7210 D_{15}= 0.3671
D_{10}= 0.2261 C_u= 9.17 C_c= 1.11

Classification

USCS= SW-SM AASHTO=

Remarks

MC=14.9%

Location: AB-05
Sample Number: G-2 **Depth:** 12.5 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC
Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AB-05
Depth: 12.5
Sample Number: G-2
Material Description: Sand with silt and gravel
Date: 11/20/2020
USCS Classification: SW-SM
Testing Remarks: MC=14.9%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 547.10
Tare Wt. = 12.50
Minus #200 from wash = 5.8%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
580.10	12.50	0.00	3/4"	0.00	100.0
			5/8"	33.70	94.1
			3/8"	73.80	87.0
			#4	135.60	76.1
			#10	231.90	59.1
			#40	468.70	17.4
			#100	522.20	8.0
			#200	534.60	5.8

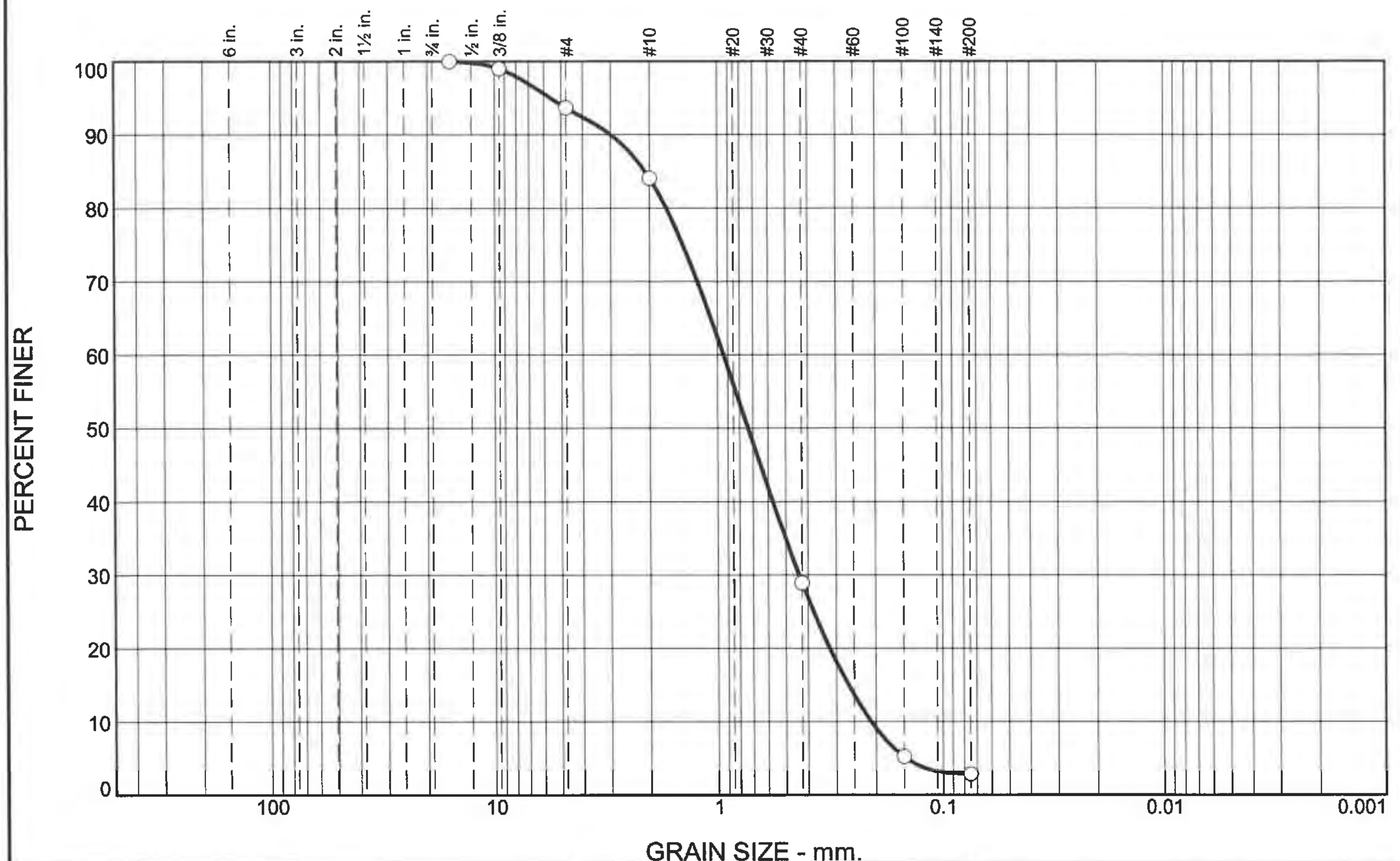
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	23.9	23.9	17.0	41.7	11.6	70.3			5.8

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
	0.2261	0.3671	0.4837	0.7210	1.0151	1.4225	2.0725	5.9081	8.0799	12.8545	16.4061

Fineness Modulus	C_u	C_c
3.84	9.17	1.11

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	6.3	9.6	55.2	26.0	2.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
5/8"	100.0		
3/8"	99.0		
#4	93.7		
#10	84.1		
#40	28.9		
#100	5.2		
#200	2.9		

* (no specification provided)

Material Description

Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 2.9828 D_{85}= 2.0941 D_{60}= 0.9426
D_{50}= 0.7343 D_{30}= 0.4382 D_{15}= 0.2653
D_{10}= 0.2099 C_u= 4.49 C_c= 0.97

Classification

USCS= SP AASHTO=

Remarks

MC=17.4%

Location: AB-05
Sample Number: S-5 **Depth:** 20 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AB-05
Depth: 20 **Sample Number:** S-5
Material Description: Sand
Date: 11/20/2020
USCS Classification: SP
Testing Remarks: MC=17.4%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 432.50
Tare Wt. = 12.50
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
445.00	12.50	0.00	5/8"	0.00	100.0
			3/8"	4.20	99.0
			#4	27.30	93.7
			#10	68.90	84.1
			#40	307.40	28.9
			#100	409.80	5.2
			#200	420.10	2.9

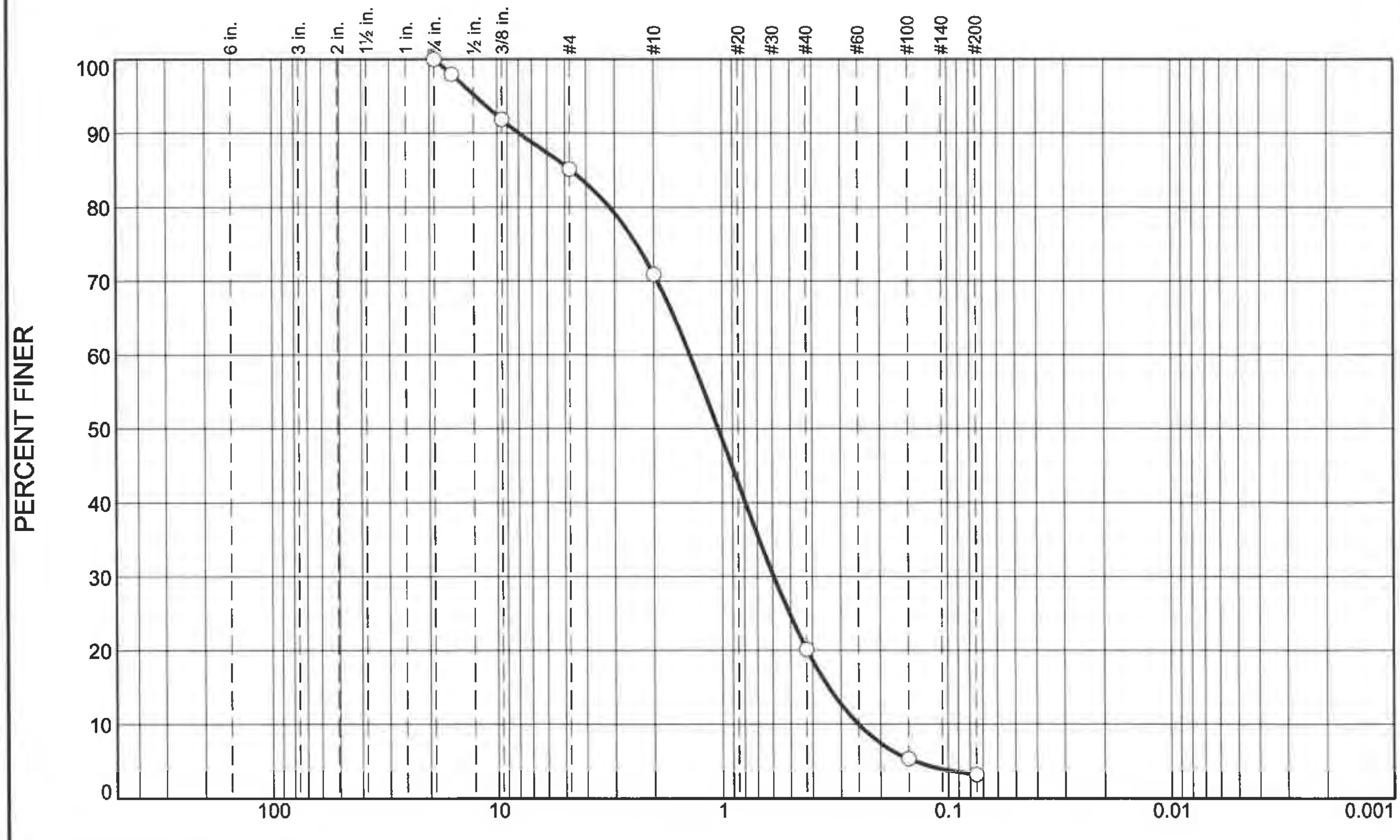
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	6.3	6.3	9.6	55.2	26.0	90.8			2.9

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.1461	0.2099	0.2653	0.3207	0.4382	0.5720	0.7343	0.9426	1.6902	2.0941	2.9828	5.5668

Fineness Modulus	C_u	C_c
2.86	4.49	0.97

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	14.9	14.2	50.8	16.9	3.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
5/8"	98.0		
3/8"	91.8		
#4	85.1		
#10	70.9		
#40	20.1		
#100	5.3		
#200	3.2		

* (no specification provided)

Material Description

Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 7.9562 D_{85}= 4.6915 D_{60}= 1.3886
D_{50}= 1.0431 D_{30}= 0.5934 D_{15}= 0.3394
D_{10}= 0.2506 C_u= 5.54 C_c= 1.01

Classification

USCS= SP AASHTO=

Remarks

MC=13.6%

Location: AB-05
Sample Number: S-6 **Depth:** 25 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road

Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AB-05
Depth: 25
Sample Number: S-6
Material Description: Sand
Date: 11/20/2020
USCS Classification: SP
Testing Remarks: MC=13.6%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 534.40
Tare Wt. = 12.50
Minus #200 from wash = 3.1%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
551.10	12.50	0.00	3/4"	0.00	100.0
			5/8"	10.90	98.0
			3/8"	43.90	91.8
			#4	80.10	85.1
			#10	156.80	70.9
			#40	430.30	20.1
			#100	509.90	5.3
			#200	521.30	3.2

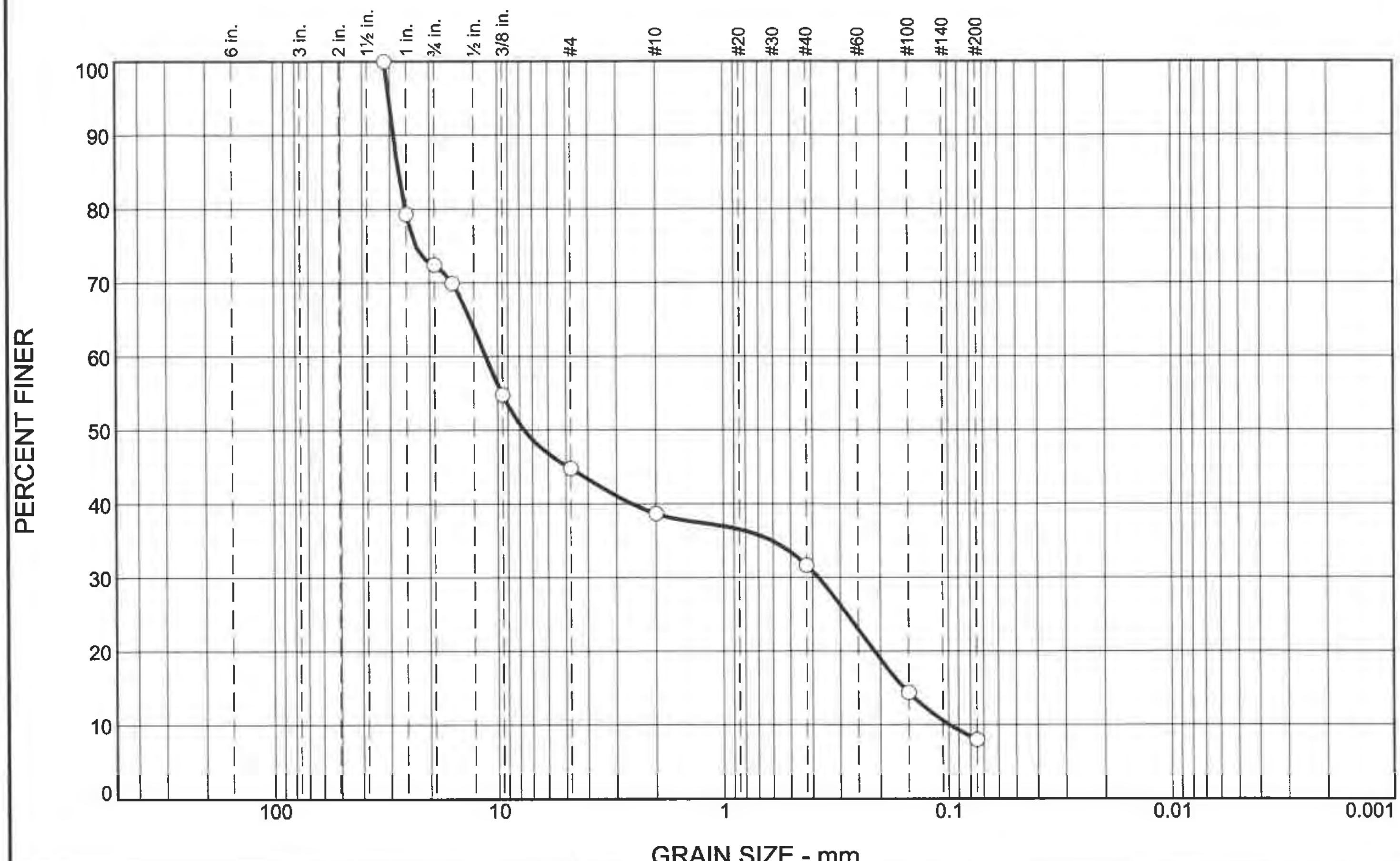
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	14.9	14.9	14.2	50.8	16.9	81.9			3.2

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.1409	0.2506	0.3394	0.4232	0.5934	0.7916	1.0431	1.3886	3.1506	4.6915	7.9562	12.4190

Fineness Modulus	C_u	C_c
3.45	5.54	1.01

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	27.6	27.6	6.2	7.0	23.6	8.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/4"	100.0		
1"	79.3		
3/4"	72.4		
5/8"	69.9		
3/8"	54.8		
#4	44.8		
#10	38.6		
#40	31.6		
#100	14.4		
#200	8.0		

* (no specification provided)

Material Description

Gravel with silt and sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 28.8759 D_{85}= 27.3833 D_{60}= 11.2925
D_{50}= 7.6329 D_{30}= 0.3772 D_{15}= 0.1567
D_{10}= 0.0984 C_u= 114.80 C_c= 0.13

Classification

USCS= GP-GM AASHTO=

Remarks

MC=6.6%

Location: AMW-01
Sample Number: S-1 **Depth:** 2.5 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC
Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AMW-01
Depth: 2.5 **Sample Number:** S-1
Material Description: Gravel with silt and sand
Date: 11/20/2020
USCS Classification: GP-GM
Testing Remarks: MC=6.6%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 376.90
Tare Wt. = 12.60
Minus #200 from wash = 5.6%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
398.40	12.60	0.00	1 1/4"	0.00	100.0
			1"	79.80	79.3
			3/4"	106.40	72.4
			5/8"	116.10	69.9
			3/8"	174.40	54.8
			#4	213.00	44.8
			#10	236.70	38.6
			#40	263.70	31.6
			#100	330.30	14.4
			#200	354.90	8.0

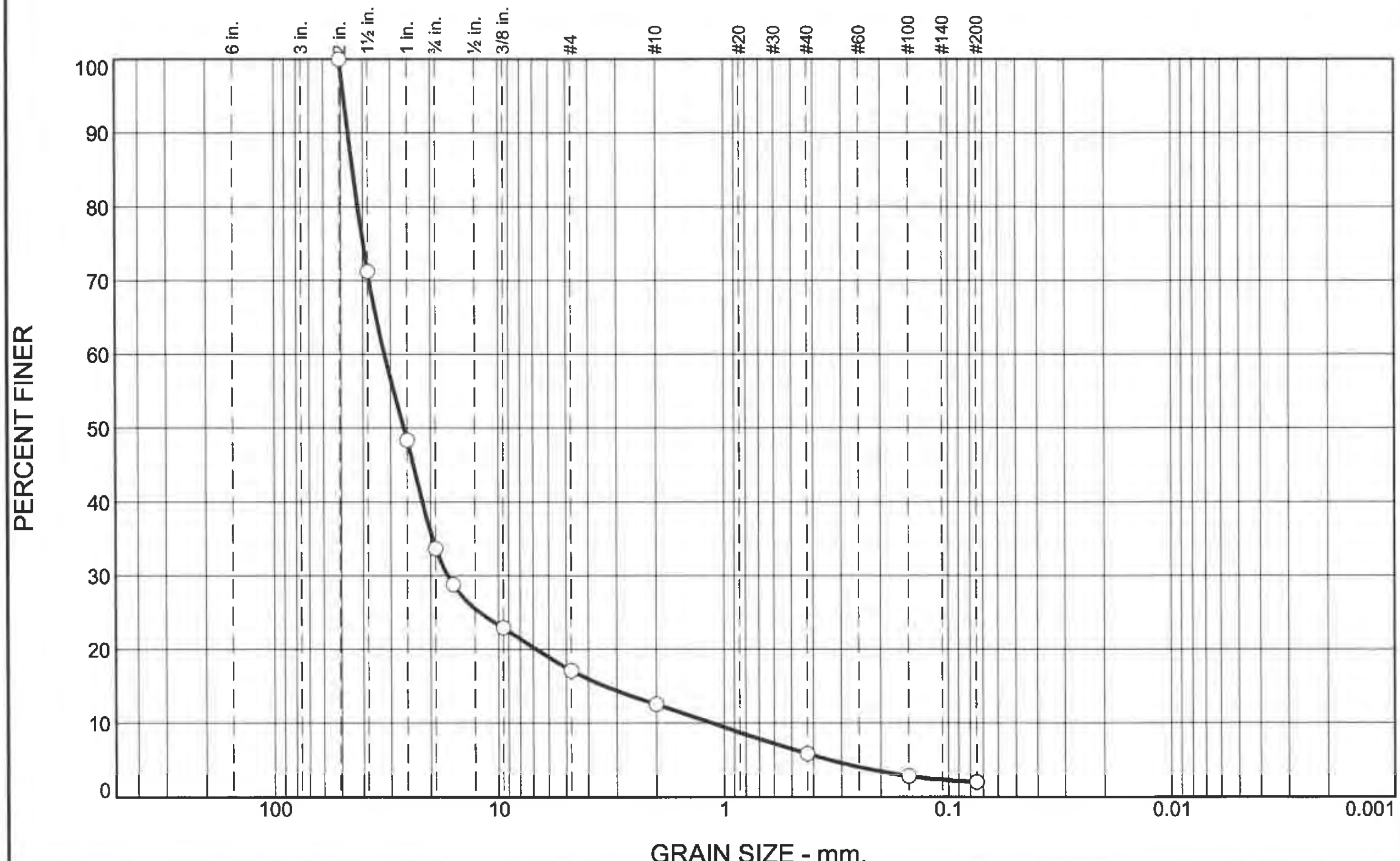
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	27.6	27.6	55.2	6.2	7.0	23.6	36.8			8.0

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
	0.0984	0.1567	0.2125	0.3772	2.5603	7.6329	11.2925	25.6735	27.3833	28.8759	30.3073

Fineness Modulus	C_u	C_c
4.76	114.80	0.13

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	66.3	16.6	4.5	6.7	3.9	2.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2"	100.0		
1.5"	71.2		
1"	48.4		
3/4"	33.7		
5/8"	28.8		
3/8"	22.9		
#4	17.1		
#10	12.6		
#40	5.9		
#100	2.8		
#200	2.0		

* (no specification provided)

Material Description

Gravel with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 46.3484 D_{85}= 44.1956 D_{60}= 32.0717
D_{50}= 26.2303 D_{30}= 16.8412 D_{15}= 3.3702
D_{10}= 1.1308 C_u= 28.36 C_c= 7.82

Classification

USCS= GP AASHTO=

Remarks

MC=1.3%

Location: AMW-01
Sample Number: S-2 **Depth:** 5 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC
Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AMW-01
Depth: 5
Sample Number: S-2
Material Description: Gravel with sand
Date: 11/20/2020
USCS Classification: GP
Testing Remarks: MC=1.3%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 657.60
Tare Wt. = 12.40
Minus #200 from wash = 0.0%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
657.60	12.40	0.00	2"	0.00	100.0
			1.5"	185.60	71.2
			1"	333.20	48.4
			3/4"	428.00	33.7
			5/8"	459.50	28.8
			3/8"	497.20	22.9
			#4	534.70	17.1
			#10	564.20	12.6
			#40	607.40	5.9
			#100	627.00	2.8
			#200	632.10	2.0

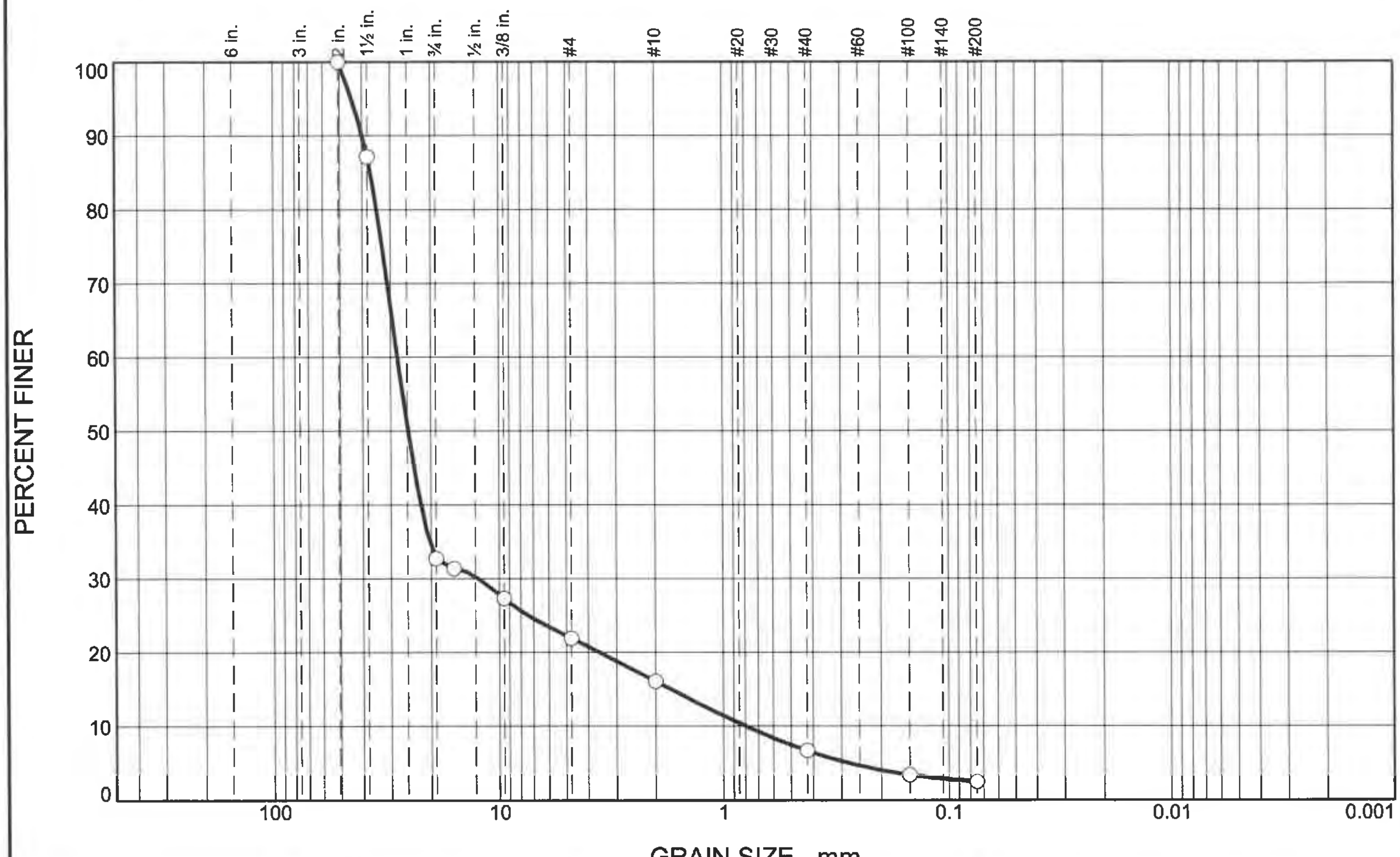
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	66.3	16.6	82.9	4.5	6.7	3.9	15.1			2.0

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.3360	1.1308	3.3702	6.7696	16.8412	21.7863	26.2303	32.0717	42.0431	44.1956	46.3484	48.5388

Fineness Modulus	C_u	C_c
7.17	28.36	7.82

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	67.3	10.8	5.8	9.4	4.2	2.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2"	100.0		
1.5"	87.2		
3/4"	32.7		
5/8"	31.4		
3/8"	27.3		
#4	21.9		
#10	16.1		
#40	6.7		
#100	3.4		
#200	2.5		

* (no specification provided)

Material Description

Gravel with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 39.9130 D_{85}= 36.9328 D_{60}= 27.9485
D_{50}= 25.1079 D_{30}= 12.3323 D_{15}= 1.7095
D_{10}= 0.7906 C_u= 35.35 C_c= 6.88

Classification

USCS= GP AASHTO=

Remarks

MC=4.4%

Location: AMW-01
Sample Number: S-3 **Depth:** 7.5 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AMW-01
Depth: 7.5 **Sample Number:** S-3
Material Description: Gravel with sand
Date: 11/20/2020
USCS Classification: GP
Testing Remarks: MC=4.4%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 679.50
Tare Wt. = 12.60
Minus #200 from wash = 2.6%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
697.10	12.60	0.00	2"	0.00	100.0
			1.5"	87.90	87.2
			3/4"	460.70	32.7
			5/8"	469.90	31.4
			3/8"	497.60	27.3
			#4	534.50	21.9
			#10	574.50	16.1
			#40	638.70	6.7
			#100	661.20	3.4
			#200	667.40	2.5

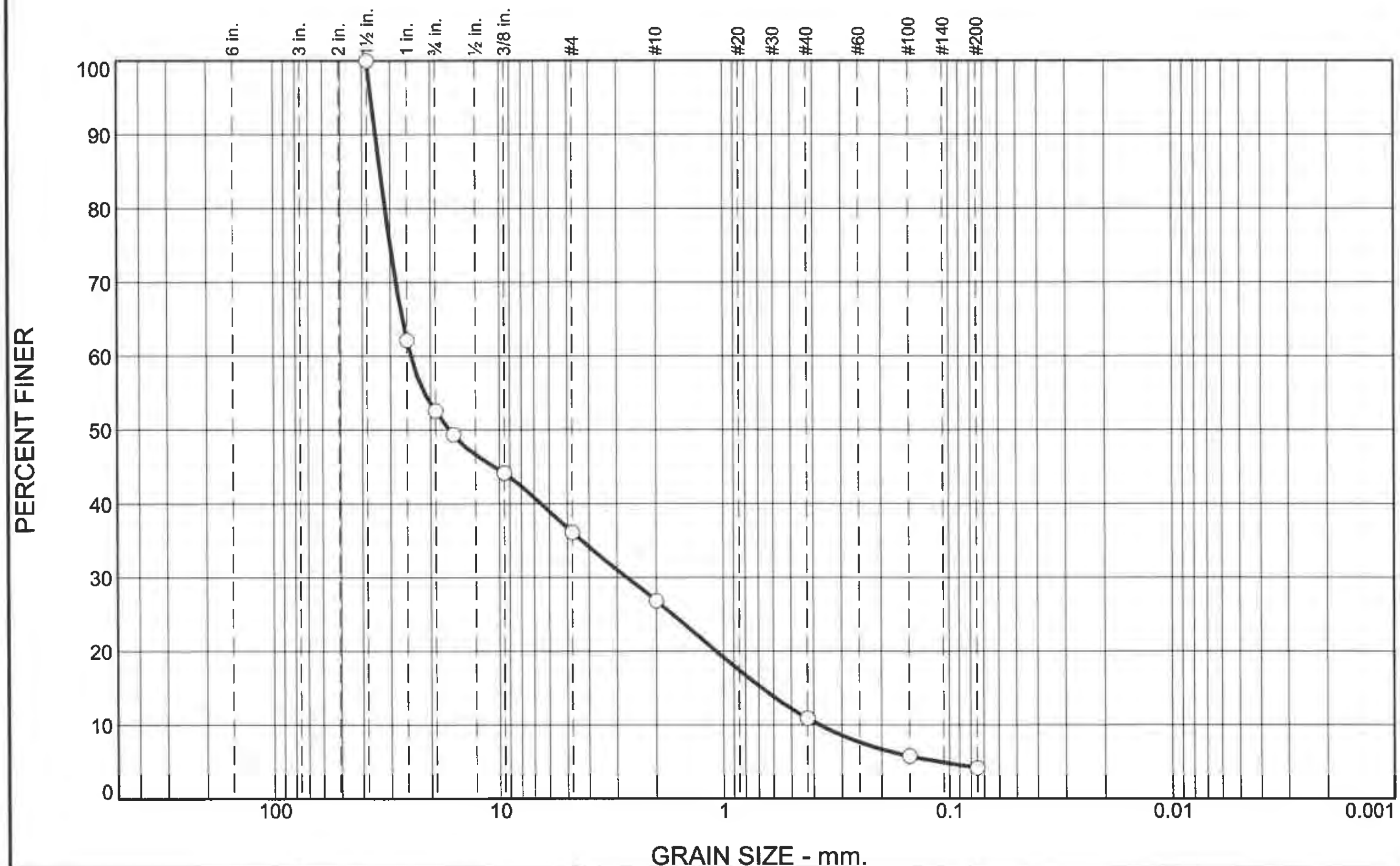
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	67.3	10.8	78.1	5.8	9.4	4.2	19.4			2.5

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.2773	0.7906	1.7095	3.5635	12.3323	22.1363	25.1079	27.9485	34.6494	36.9328	39.9130	44.2771

Fineness Modulus	C_u	C_c
6.84	35.35	6.88

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	47.4	16.5	9.3	15.9	6.7	4.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1.5"	100.0		
1"	62.1		
3/4"	52.6		
5/8"	49.3		
3/8"	44.1		
#4	36.1		
#10	26.8		
#40	10.9		
#100	5.8		
#200	4.2		

* (no specification provided)

Material Description

Gravel with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 34.7860 D_{85}= 33.1913 D_{60}= 24.4053
D_{50}= 16.5541 D_{30}= 2.7101 D_{15}= 0.6744
D_{10}= 0.3751 C_u= 65.07 C_c= 0.80

Classification

USCS= GP AASHTO=

Remarks

MC=7.3%

Location: AMW-01
Sample Number: S-4 **Depth:** 10 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC
Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AMW-01
Depth: 10 **Sample Number:** S-4
Material Description: Gravel with sand
Date: 11/20/2020
USCS Classification: GP
Testing Remarks: MC=7.3%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 528.90
Tare Wt. = 12.50
Minus #200 from wash = 4.3%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
551.90	12.50	0.00	1.5"	0.00	100.0
			1"	204.20	62.1
			3/4"	255.80	52.6
			5/8"	273.30	49.3
			3/8"	301.30	44.1
			#4	344.70	36.1
			#10	394.70	26.8
			#40	480.60	10.9
			#100	508.20	5.8
			#200	516.70	4.2

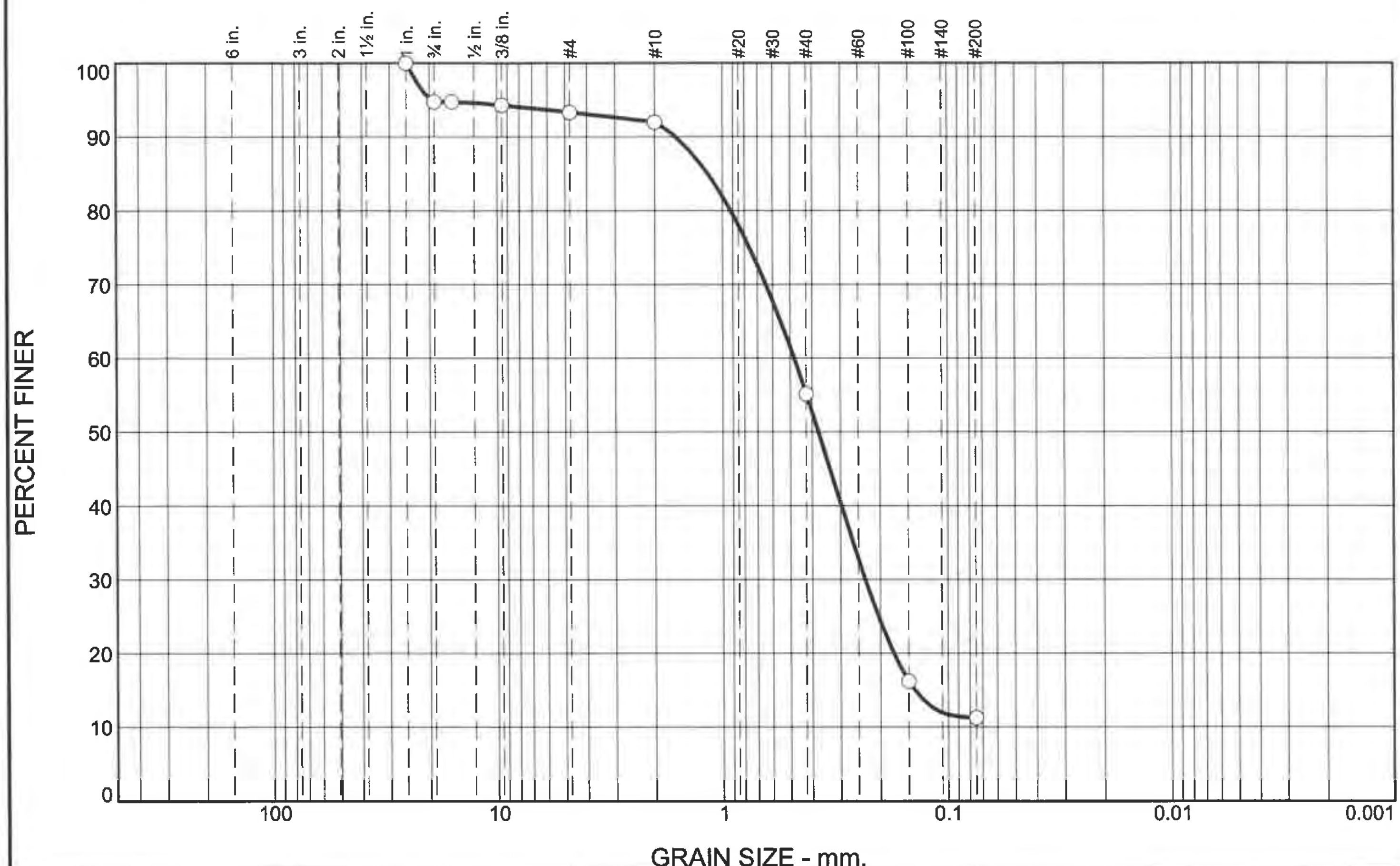
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	47.4	16.5	63.9	9.3	15.9	6.7	31.9			4.2

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.1101	0.3751	0.6744	1.0795	2.7101	6.5379	16.5541	24.4053	31.6086	33.1913	34.7860	36.4152

Fineness Modulus	C_u	C_c
5.89	65.07	0.80

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	5.2	1.5	1.3	36.9	43.9	11.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1"	100.0		
3/4"	94.8		
5/8"	94.8		
3/8"	94.3		
#4	93.3		
#10	92.0		
#40	55.1		
#100	16.2		
#200	11.2		

* (no specification provided)

Material Description

Sand with silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 1.6173 D_{85}= 1.1655 D_{60}= 0.4814
D_{50}= 0.3753 D_{30}= 0.2335 D_{15}= 0.1407
D_{10}= C_u= C_c=

Classification

USCS= SP-SM AASHTO=

Remarks

MC=20.7%

Location: AMW-01
Sample Number: G-1 **Depth:** 12.5 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AMW-01
Depth: 12.5
Sample Number: G-1
Material Description: Sand with silt
Date: 11/20/2020
USCS Classification: SP-SM
Testing Remarks: MC=20.7%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 325.70
Tare Wt. = 12.80
Minus #200 from wash = 12.7%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
371.30	12.80	0.00	1"	0.00	100.0
			3/4"	18.80	94.8
			5/8"	18.80	94.8
			3/8"	20.40	94.3
			#4	23.90	93.3
			#10	28.70	92.0
			#40	160.80	55.1
			#100	300.40	16.2
			#200	318.20	11.2

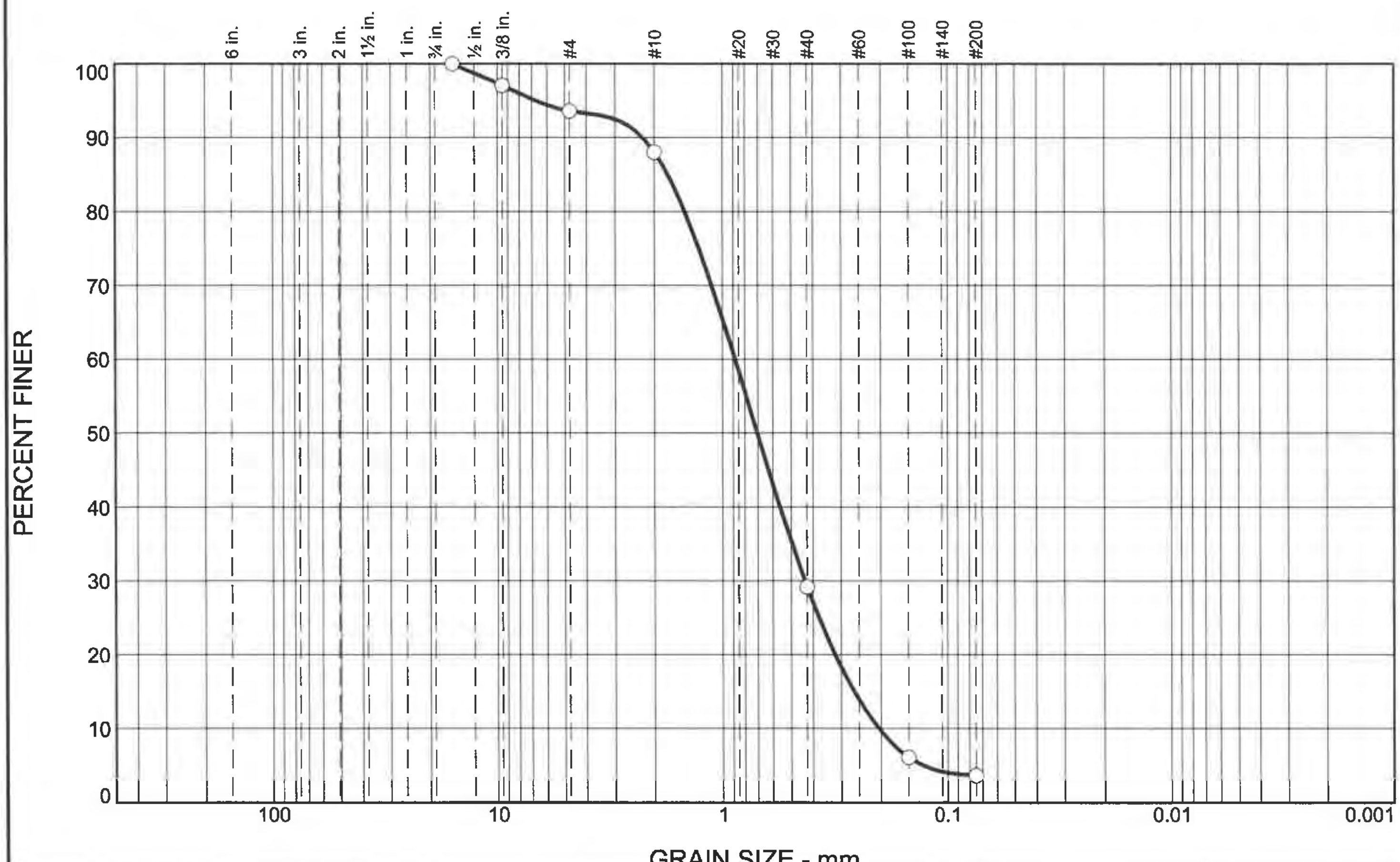
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	5.2	1.5	6.7	1.3	36.9	43.9	82.1			11.2

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
		0.1407	0.1747	0.2335	0.2973	0.3753	0.4814	0.9220	1.1655	1.6173	19.6125

Fineness Modulus
2.16

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	6.4	5.6	58.8	25.6	3.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
5/8"	100.0		
3/8"	97.1		
#4	93.6		
#10	88.0		
#40	29.2		
#100	6.1		
#200	3.6		

* (no specification provided)

Material Description

Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 2.2520 D_{85}= 1.7476 D_{60}= 0.8827
D_{50}= 0.7037 D_{30}= 0.4347 D_{15}= 0.2629
D_{10}= 0.2039 C_u= 4.33 C_c= 1.05

Classification

USCS= SP AASHTO=

Remarks

MC=21%

Location: AMW-01
Sample Number: S-5 **Depth:** 15 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AMW-01
Depth: 15 **Sample Number:** S-5
Material Description: Sand
Date: 11/20/2020
USCS Classification: SP
Testing Remarks: MC=21%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 455.20
Tare Wt. = 12.60
Minus #200 from wash = 4.5%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
476.00	12.60	0.00	5/8"	0.00	100.0
			3/8"	13.50	97.1
			#4	29.70	93.6
			#10	55.40	88.0
			#40	328.20	29.2
			#100	435.10	6.1
			#200	446.50	3.6

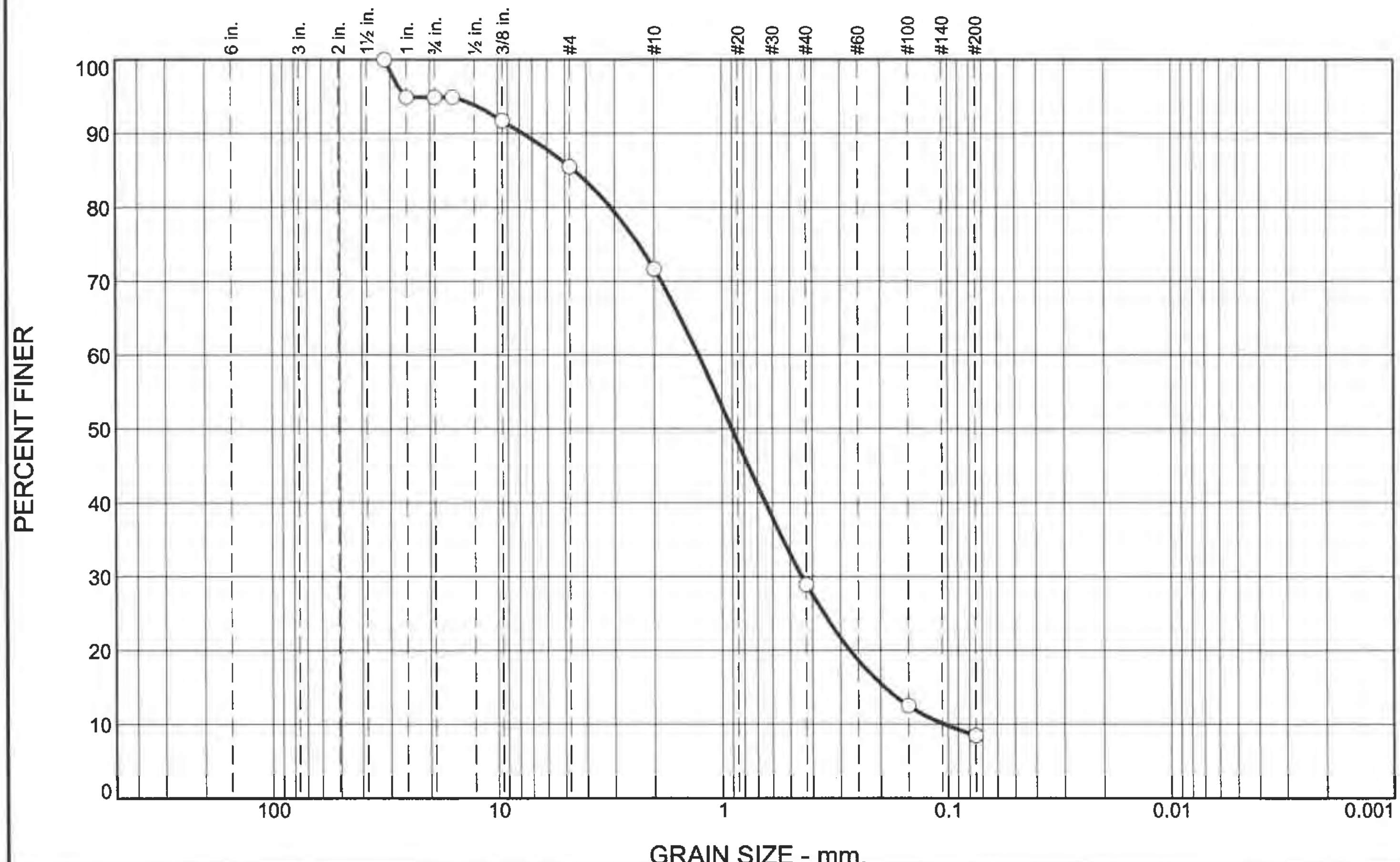
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	6.4	6.4	5.6	58.8	25.6	90.0			3.6

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.1290	0.2039	0.2629	0.3196	0.4347	0.5589	0.7037	0.8827	1.4722	1.7476	2.2520	6.6828

Fineness Modulus	C_u	C_c
2.79	4.33	1.05

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	5.1	9.4	13.9	42.7	20.4	8.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/4"	100.0		
1"	94.9		
3/4"	94.9		
5/8"	94.9		
3/8"	91.7		
#4	85.5		
#10	71.6		
#40	28.9		
#100	12.5		
#200	8.5		

* (no specification provided)

Material Description

Sand with silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 7.6830 D_{85}= 4.5413 D_{60}= 1.2745
D_{50}= 0.9064 D_{30}= 0.4445 D_{15}= 0.1916
D_{10}= 0.1033 C_u= 12.33 C_c= 1.50

Classification

USCS= SW-SM AASHTO=

Remarks

MC=12.2%

Location: AMW-01
Sample Number: S-6 **Depth:** 20 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road

Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AMW-01
Depth: 20 **Sample Number:** S-6
Material Description: Sand with silt
Date: 11/20/2020
USCS Classification: SW-SM
Testing Remarks: MC=12.2%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare** = 353.50
Tare Wt. = 12.50
Minus #200 from wash = 8.6%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
385.60	12.50	0.00	1 1/4"	0.00	100.0
			1"	18.90	94.9
			3/4"	18.90	94.9
			5/8"	18.90	94.9
			3/8"	30.80	91.7
			#4	54.10	85.5
			#10	105.80	71.6
			#40	265.20	28.9
			#100	326.40	12.5
			#200	341.40	8.5

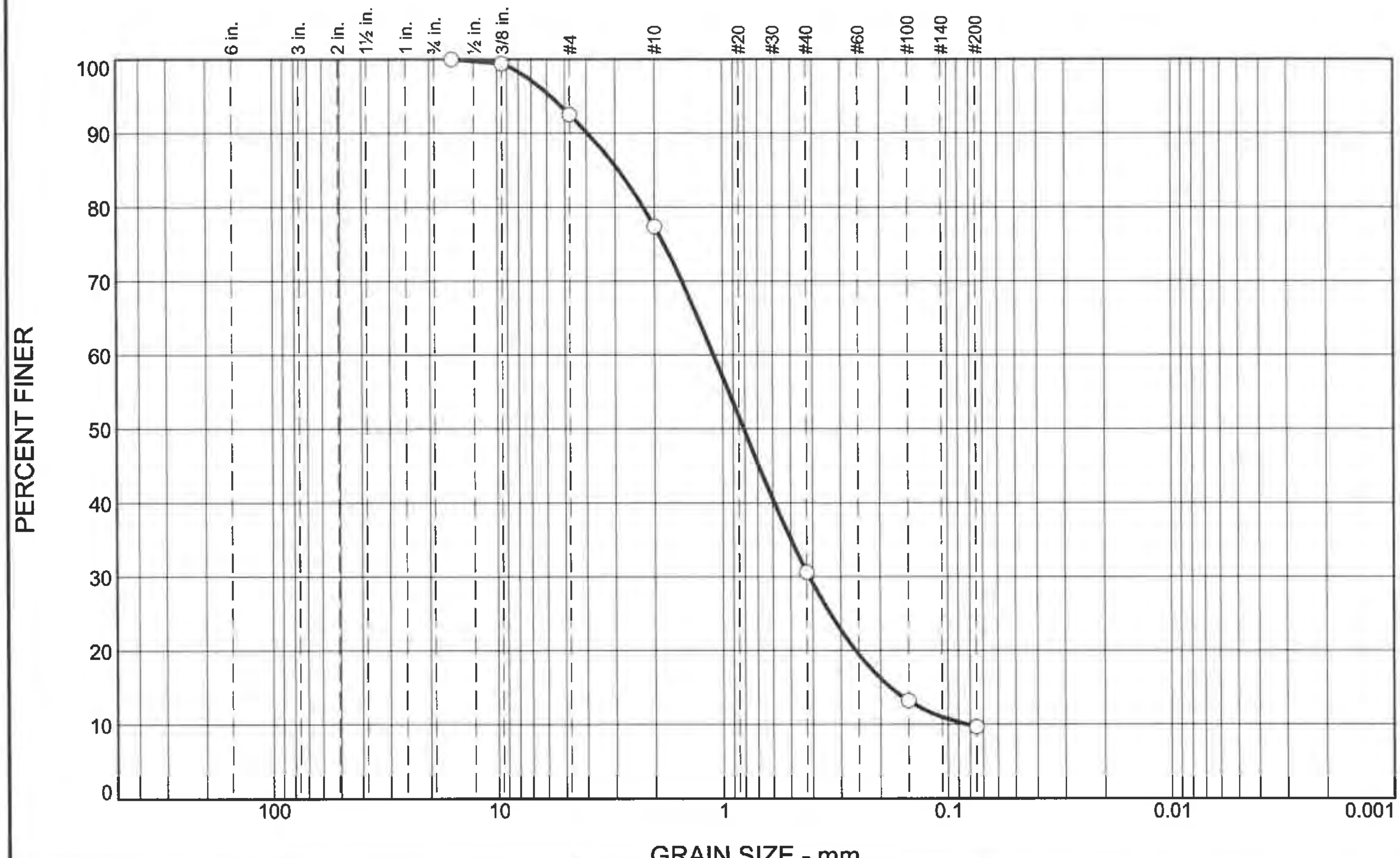
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	5.1	9.4	14.5	13.9	42.7	20.4	77.0			8.5

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
	0.1033	0.1916	0.2727	0.4445	0.6457	0.9064	1.2745	3.1174	4.5413	7.6830	25.5356

Fineness Modulus	C_u	C_c
3.23	12.33	1.50

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	7.5	15.1	46.8	20.9	9.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
5/8"	100.0		
3/8"	99.4		
#4	92.5		
#10	77.4		
#40	30.6		
#100	13.2		
#200	9.7		

* (no specification provided)

Material Description

Sand with silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 3.9529 D_{85}= 2.8885 D_{60}= 1.0987
D_{50}= 0.8087 D_{30}= 0.4149 D_{15}= 0.1799
D_{10}= 0.0822 C_u= 13.37 C_c= 1.91

Classification

USCS= SW-SM AASHTO=

Remarks

MC=15.3%

Location: AMW-01
Sample Number: S-7 **Depth:** 25 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AMW-01
Depth: 25
Sample Number: S-7
Material Description: Sand with silt
Date: 11/20/2020
USCS Classification: SW-SM
Testing Remarks: MC=15.3%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 385.50
Tare Wt. = 12.80
Minus #200 from wash = 9.4%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
424.00	12.80	0.00	5/8"	0.00	100.0
			3/8"	2.30	99.4
			#4	30.70	92.5
			#10	93.10	77.4
			#40	285.30	30.6
			#100	356.80	13.2
			#200	371.40	9.7

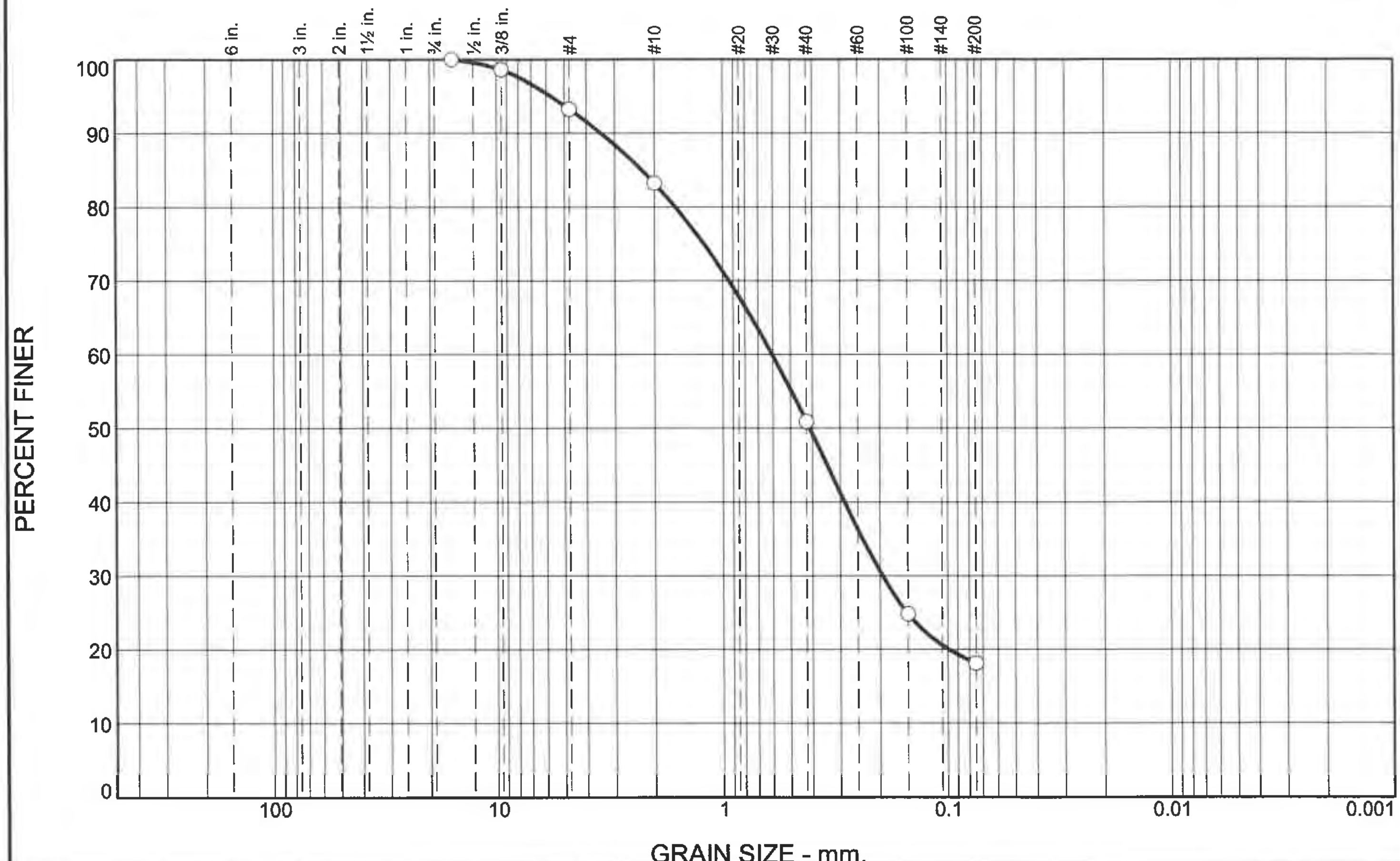
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	7.5	7.5	15.1	46.8	20.9	82.8			9.7

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
	0.0822	0.1799	0.2576	0.4149	0.5910	0.8087	1.0987	2.2427	2.8885	3.9529	5.7636

Fineness Modulus	C_u	C_c
2.88	13.37	1.91

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	6.7	10.1	32.3	32.8	18.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
5/8"	100.0		
3/8"	98.6		
#4	93.3		
#10	83.2		
#40	50.9		
#100	24.8		
#200	18.1		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 3.4436 D_{85}= 2.2728 D_{60}= 0.6043
D_{50}= 0.4110 D_{30}= 0.1949 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

MC=17.7%

Location: AMW-01
Sample Number: S-10 **Depth:** 40 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AMW-01
Depth: 40
Sample Number: S-10
Material Description: Silty sand
Date: 11/20/2020
USCS Classification: SM
Testing Remarks: MC=17.7%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 324.30
Tare Wt. = 12.70
Minus #200 from wash = 17.3%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
389.40	12.70	0.00	5/8"	0.00	100.0
			3/8"	5.10	98.6
			#4	25.30	93.3
			#10	63.20	83.2
			#40	184.90	50.9
			#100	283.20	24.8
			#200	308.40	18.1

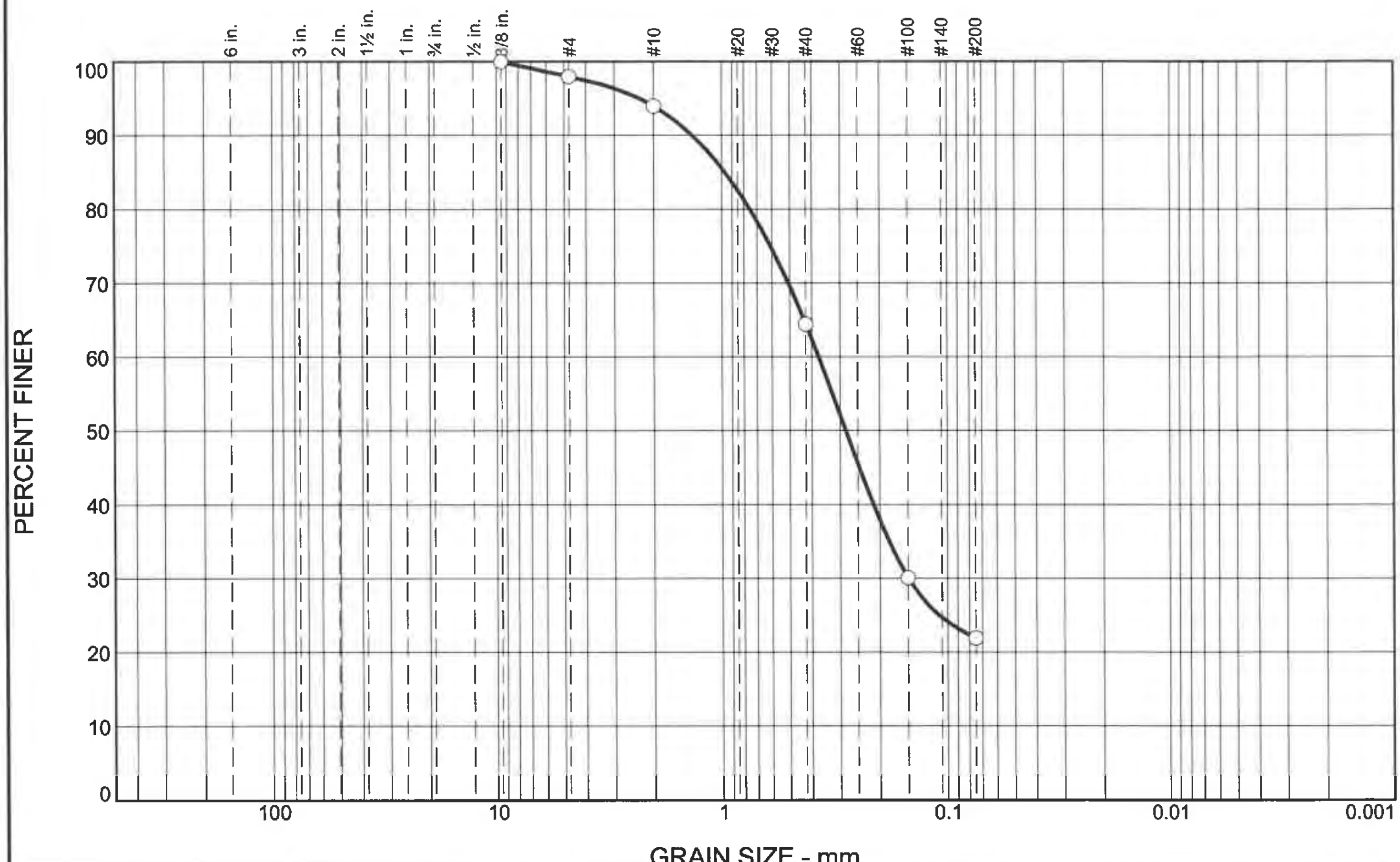
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	6.7	6.7	10.1	32.3	32.8	75.2			18.1

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
			0.0987	0.1949	0.2876	0.4110	0.6043	1.6201	2.2728	3.4436	5.7051

Fineness Modulus
2.22

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	2.0	4.0	29.6	42.5	21.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8"	100.0		
#4	98.0		
#10	94.0		
#40	64.4		
#100	30.1		
#200	21.9		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 1.3514 D_{85}= 0.9700 D_{60}= 0.3733
D_{50}= 0.2835 D_{30}= 0.1494 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

MC=19.4%

Location: AMW-01
Sample Number: S-12 **Depth:** 50 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road

Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AMW-01
Depth: 50
Sample Number: S-12
Material Description: Silty sand
Date: 11/20/2020
USCS Classification: SM
Testing Remarks: MC=19.4%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 280.80
Tare Wt. = 12.70
Minus #200 from wash = 20.2%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
348.70	12.70	0.00	3/8"	0.00	100.0
			#4	6.80	98.0
			#10	20.20	94.0
			#40	119.50	64.4
			#100	234.90	30.1
			#200	262.40	21.9

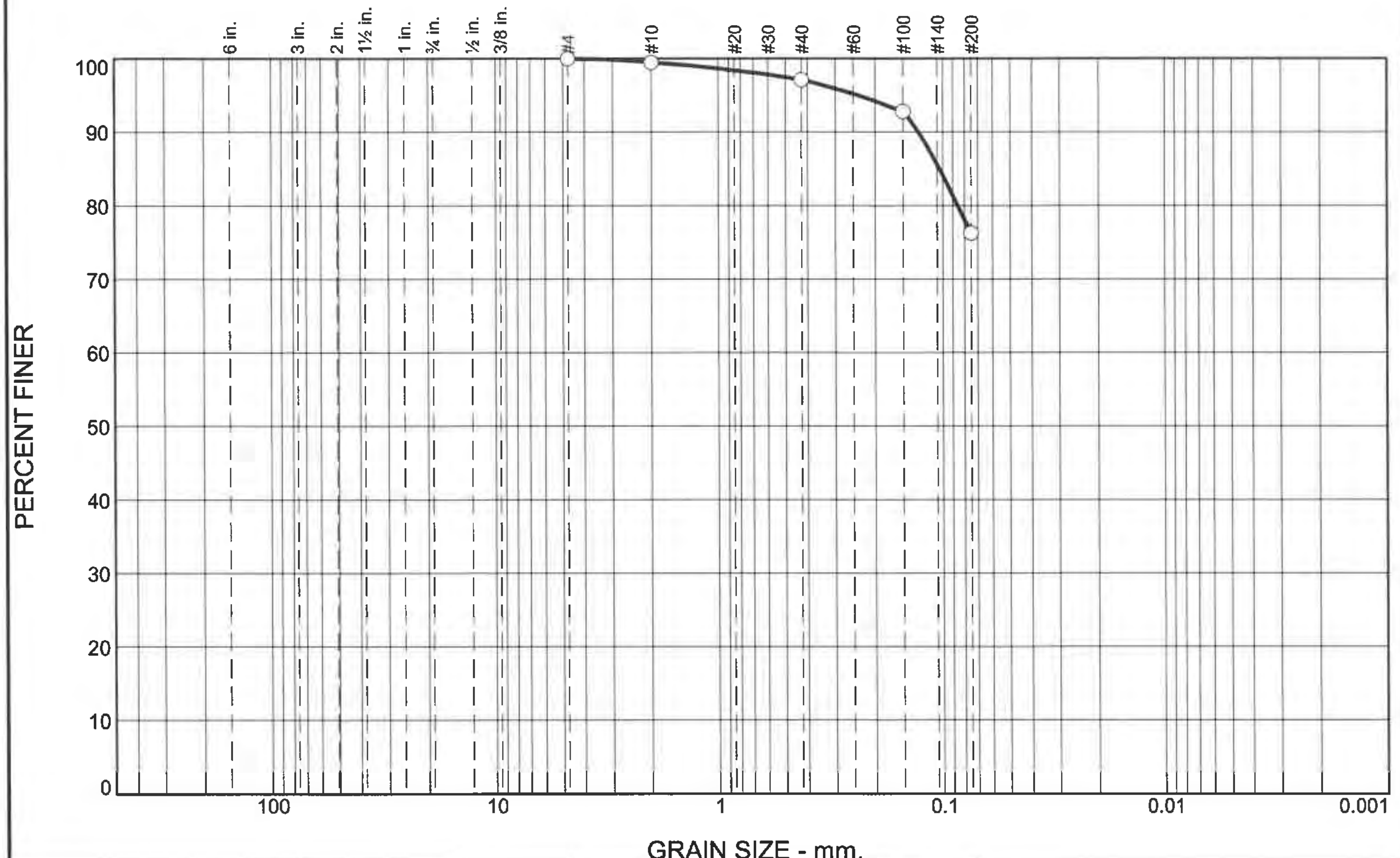
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	2.0	2.0	4.0	29.6	42.5	76.1			21.9

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
				0.1494	0.2135	0.2835	0.3733	0.7525	0.9700	1.3514	2.3071

Fineness Modulus
1.62

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.5	2.4	20.8	76.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.5		
#40	97.1		
#100	92.8		
#200	76.3		

* (no specification provided)

Material Description

Sandy silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.1285 D_{85}= 0.1035 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

MC=23.3%

Location: AMW-01
Sample Number: S-13 **Depth:** 55 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AMW-01
Depth: 55
Sample Number: S-13
Material Description: Sandy silt
Date: 11/20/2020
USCS Classification: ML
Testing Remarks: MC=23.3%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 88.90
Tare Wt. = 12.70
Minus #200 from wash = 72.6%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
291.30	12.70	0.00	#4	0.00	100.0
			#10	1.50	99.5
			#40	8.00	97.1
			#100	20.00	92.8
			#200	66.10	76.3

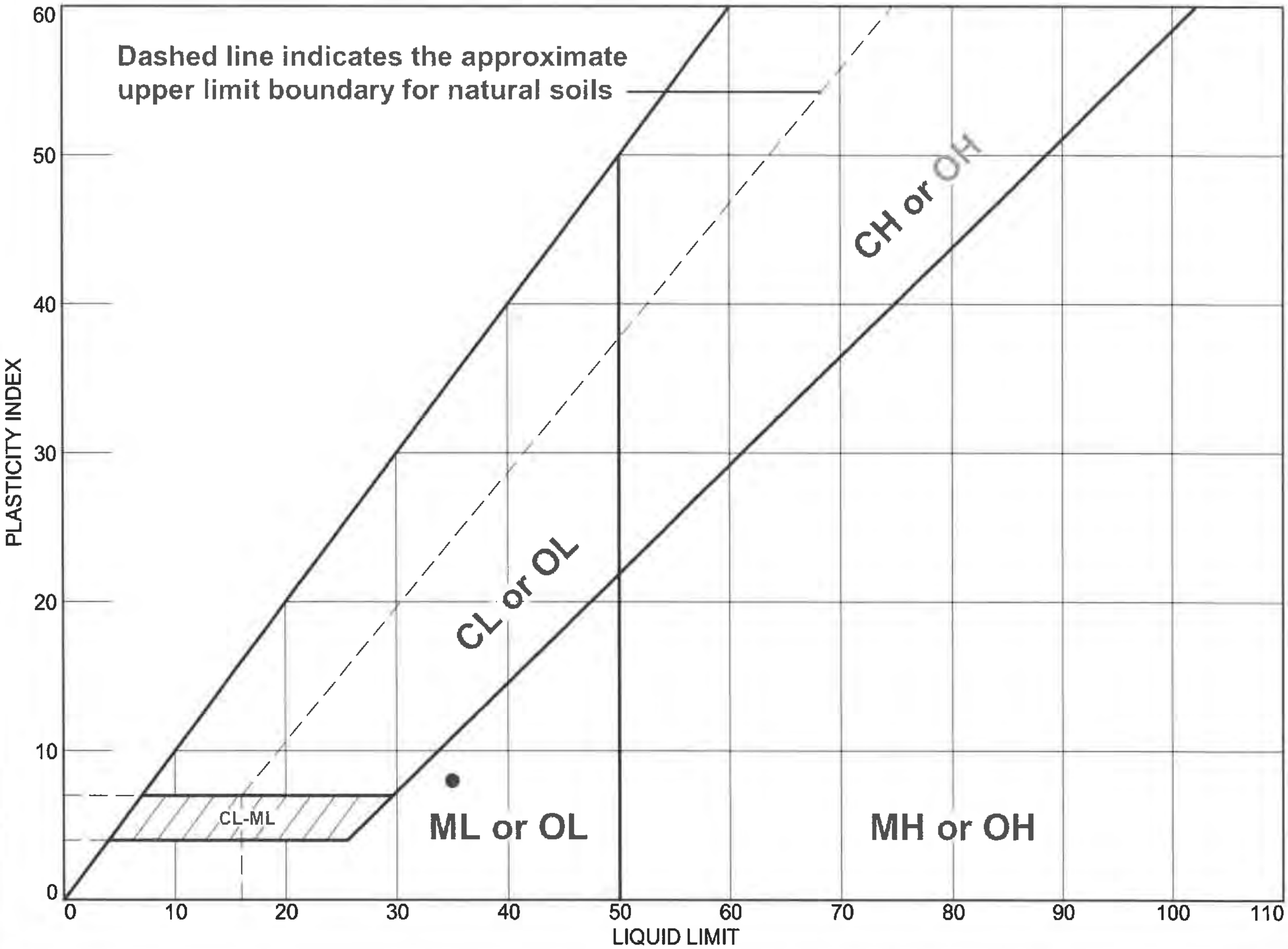
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	0.0	0.0	0.5	2.4	20.8	23.7			76.3

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
								0.0857	0.1035	0.1285	0.2379

Fineness Modulus
0.15

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Silt	35	27	8			ML

Project No. 08-175/ **Client:** Aspect Consulting

Project: Jan Road

● **Location:** AMW-01 **Depth:** 60 **Sample Number:** G-2

Remarks:

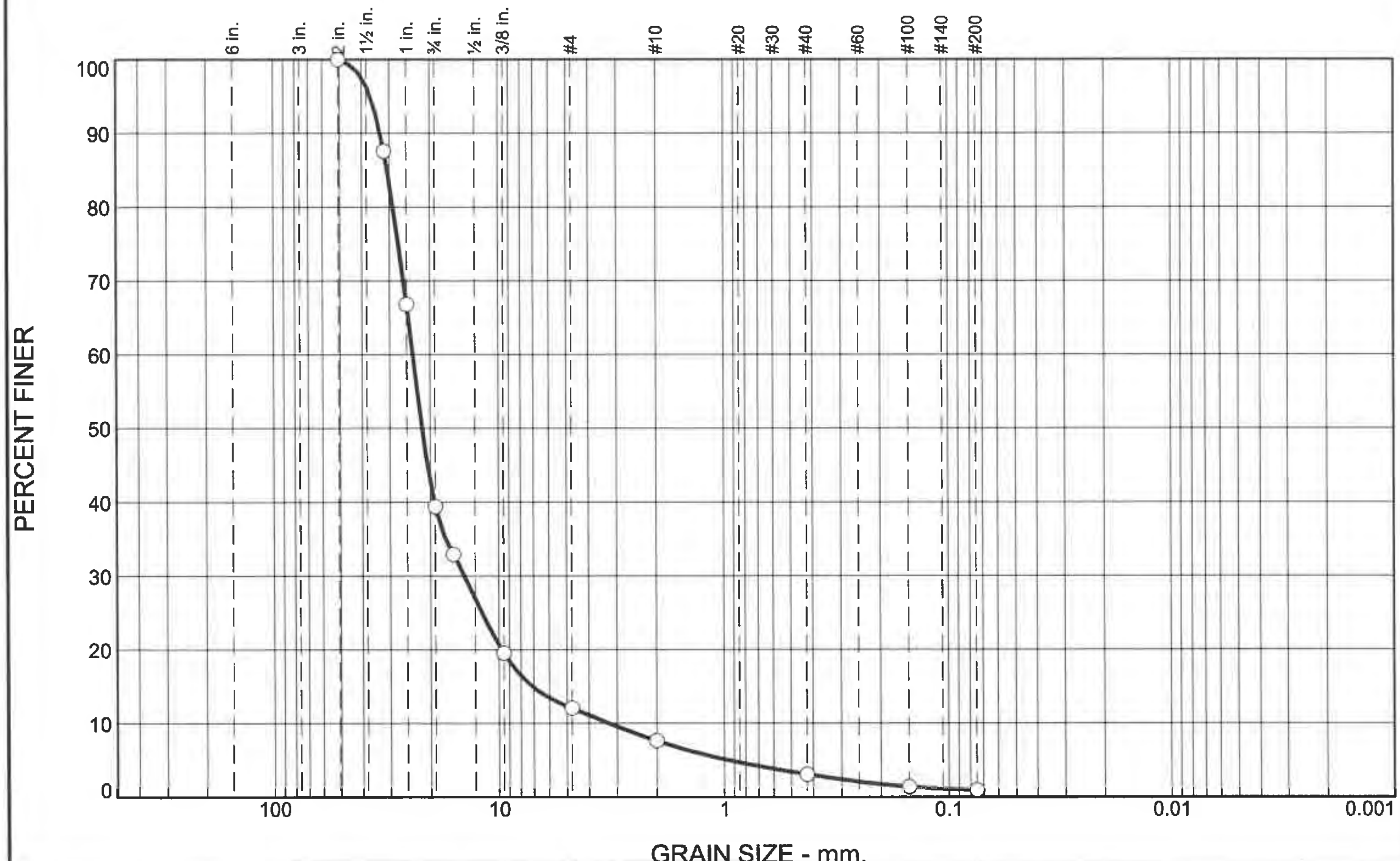
● MC=26.4%

Hayre McElroy & Associates, LLC

Redmond, WA

Figure

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	60.6	27.3	4.5	4.5	2.2	0.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2"	100.0		
1 1/4"	87.6		
1"	66.8		
3/4"	39.4		
5/8"	32.9		
3/8"	19.6		
#4	12.1		
#10	7.6		
#40	3.1		
#100	1.4		
#200	0.9		

* (no specification provided)

Material Description

Gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 32.9603 D_{85}= 30.6708 D_{60}= 23.8632
D_{50}= 21.6982 D_{30}= 14.2005 D_{15}= 7.0910
D_{10}= 3.2512 C_u= 7.34 C_c= 2.60

Classification

USCS= GW AASHTO=

Remarks

MC=2.4%

Location: AMW-02
Sample Number: S-1/S-2/S-3 **Depth:** 2.5-7.5 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC
Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/24/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AMW-02
Depth: 2.5-7.5 **Sample Number:** S-1/S-2/S-3
Material Description: Gravel
Date: 11/20/2020
USCS Classification: GW
Testing Remarks: MC=2.4%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 947.40
Tare Wt. = 12.60
Minus #200 from wash = 1.2%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
959.10	12.60	0.00	2"	0.00	100.0
			1 1/4"	117.60	87.6
			1"	314.00	66.8
			3/4"	573.40	39.4
			5/8"	634.80	32.9
			3/8"	761.30	19.6
			#4	832.30	12.1
			#10	874.20	7.6
			#40	917.40	3.1
			#100	933.50	1.4
			#200	937.60	0.9

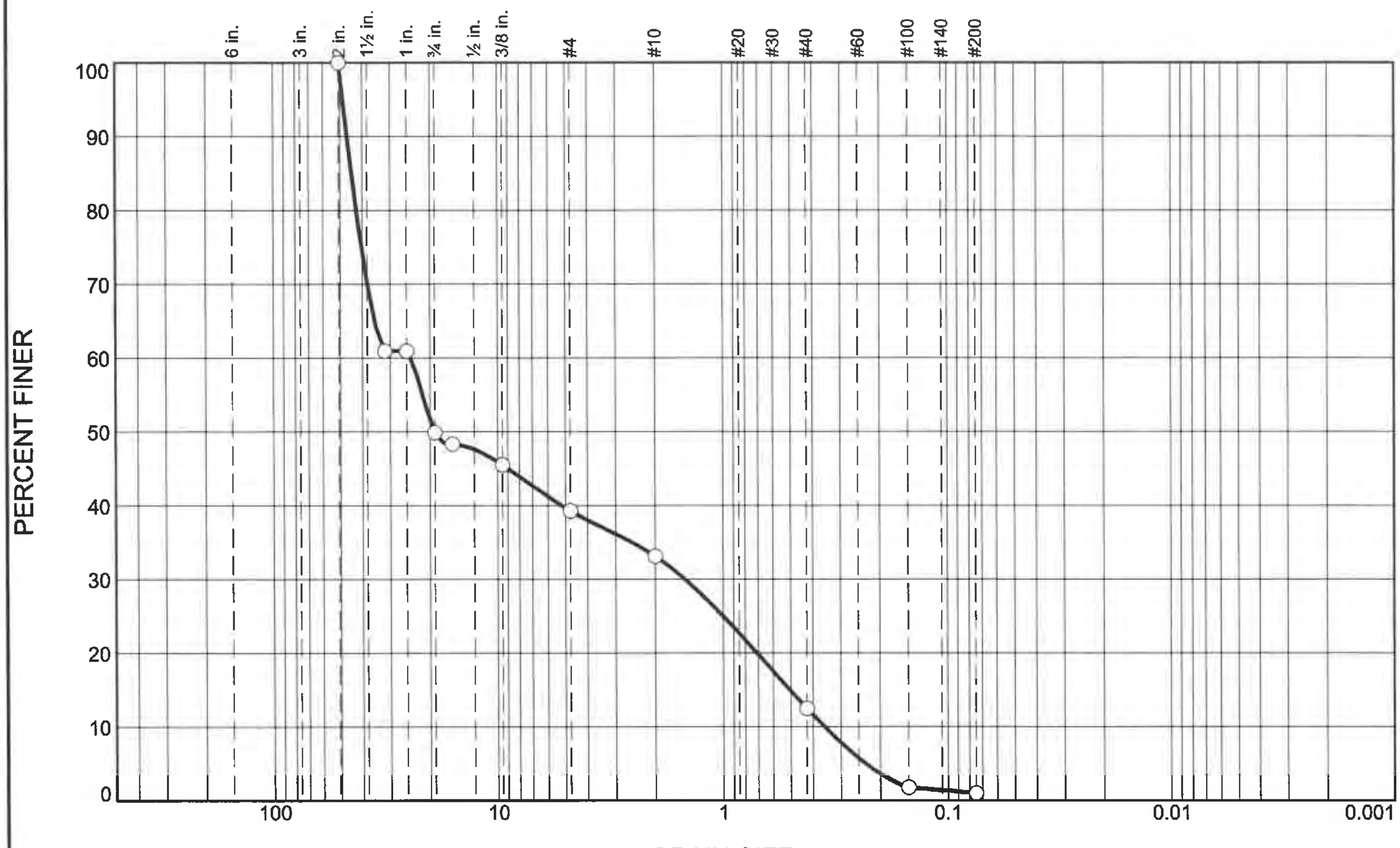
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	60.6	27.3	87.9	4.5	4.5	2.2	11.2			0.9

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.9501	3.2512	7.0910	9.7185	14.2005	19.2313	21.6982	23.8632	28.9287	30.6708	32.9603	36.6119

Fineness Modulus	C_u	C_c
7.11	7.34	2.60

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	50.1	10.6	6.2	20.6	11.5	1.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2"	100.0		
1 1/4"	60.9		
1"	60.9		
3/4"	49.9		
5/8"	48.3		
3/8"	45.5		
#4	39.3		
#10	33.1		
#40	12.5		
#100	1.7		
#200	1.0		

* (no specification provided)

Material Description

Gravel with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 46.5791 D_{85}= 44.5264 D_{60}= 24.3510
D_{50}= 19.1326 D_{30}= 1.4793 D_{15}= 0.5055
D_{10}= 0.3543 C_u= 68.74 C_c= 0.25

Classification

USCS= GP AASHTO=

Remarks

MC=10.2%

Location: AMW-02
Sample Number: S-4 **Depth:** 10 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC
Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AMW-02
Depth: 10 **Sample Number:** S-4
Material Description: Gravel with sand
Date: 11/20/2020
USCS Classification: GP
Testing Remarks: MC=10.2%

Sieve Test Data

Post #200 Wash Test Weights (grams): Dry Sample and Tare = 584.00
Tare Wt. = 12.90
Minus #200 from wash = 1.4%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
591.90	12.90	0.00	2"	0.00	100.0
			1 1/4"	226.10	60.9
			1"	226.10	60.9
			3/4"	290.20	49.9
			5/8"	299.20	48.3
			3/8"	315.50	45.5
			#4	351.60	39.3
			#10	387.20	33.1
			#40	506.60	12.5
			#100	568.90	1.7
			#200	573.30	1.0

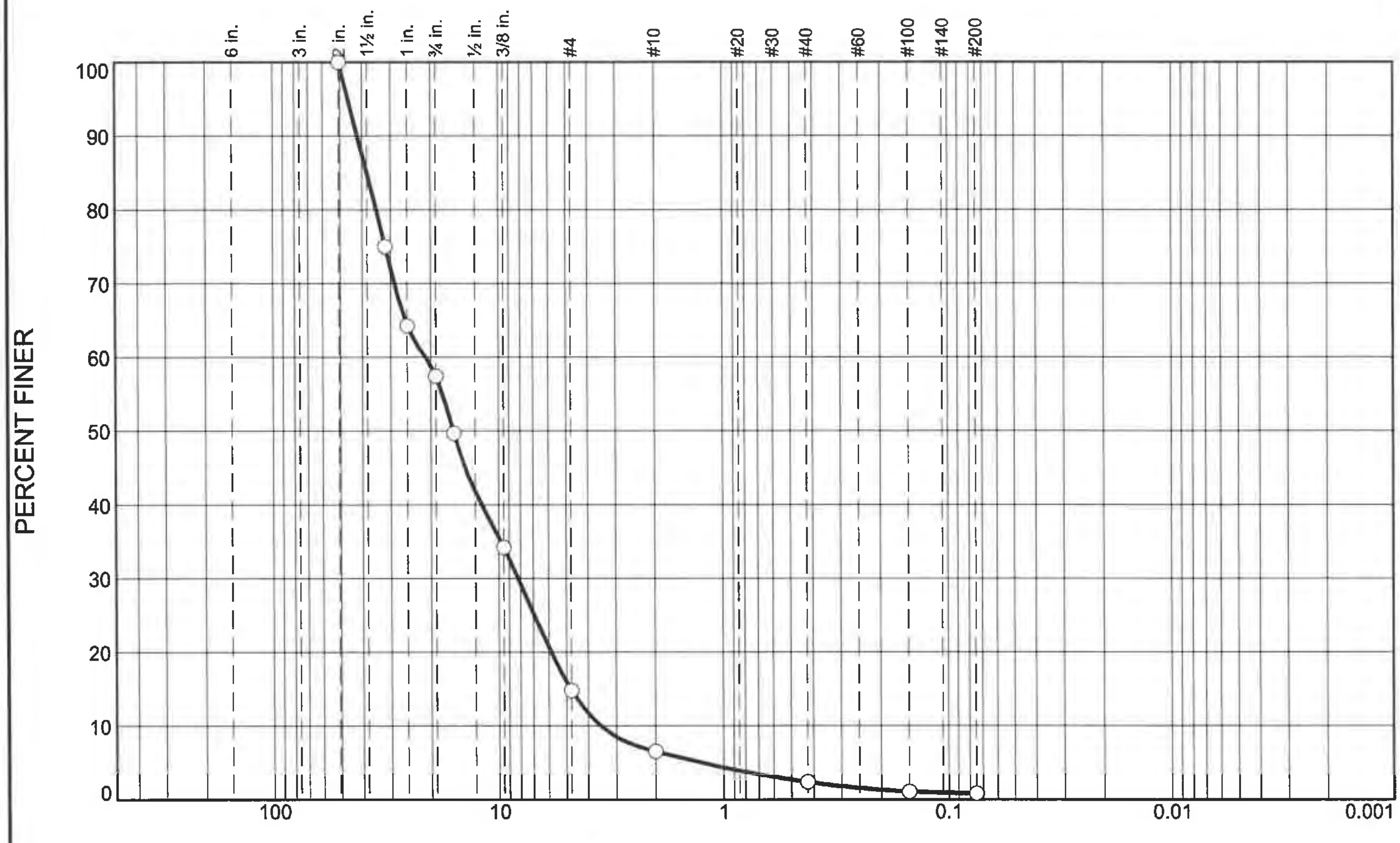
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	50.1	10.6	60.7	6.2	20.6	11.5	38.3			1.0

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.2322	0.3543	0.5055	0.7090	1.4793	5.1946	19.1326	24.3510	42.4631	44.5264	46.5791	48.6594

Fineness Modulus	C_u	C_c
6.06	68.74	0.25

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	42.5	42.7	8.2	4.2	1.6	0.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2"	100.0		
1 1/4"	75.0		
1"	64.3		
3/4"	57.5		
5/8"	49.7		
3/8"	34.2		
#4	14.8		
#10	6.6		
#40	2.4		
#100	1.1		
#200	0.8		

* (no specification provided)

Material Description

Gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 41.9730 D_{85}= 38.1910 D_{60}= 21.0503
D_{50}= 15.9957 D_{30}= 8.2142 D_{15}= 4.7916
D_{10}= 3.5101 C_u= 6.00 C_c= 0.91

Classification

USCS= GP AASHTO=

Remarks

MC=4.5%

Location: AMW-02
Sample Number: G-1 **Depth:** 12.5 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road

Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AMW-02
Depth: 12.5
Sample Number: G-1
Material Description: Gravel
Date: 11/20/2020
USCS Classification: GP
Testing Remarks: MC=4.5%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 676.30
Tare Wt. = 12.60
Minus #200 from wash = 0.9%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
682.30	12.60	0.00	2"	0.00	100.0
			1 1/4"	167.30	75.0
			1"	239.40	64.3
			3/4"	284.90	57.5
			5/8"	337.07	49.7
			3/8"	440.70	34.2
			#4	570.50	14.8
			#10	625.80	6.6
			#40	653.90	2.4
			#100	662.60	1.1
			#200	664.30	0.8

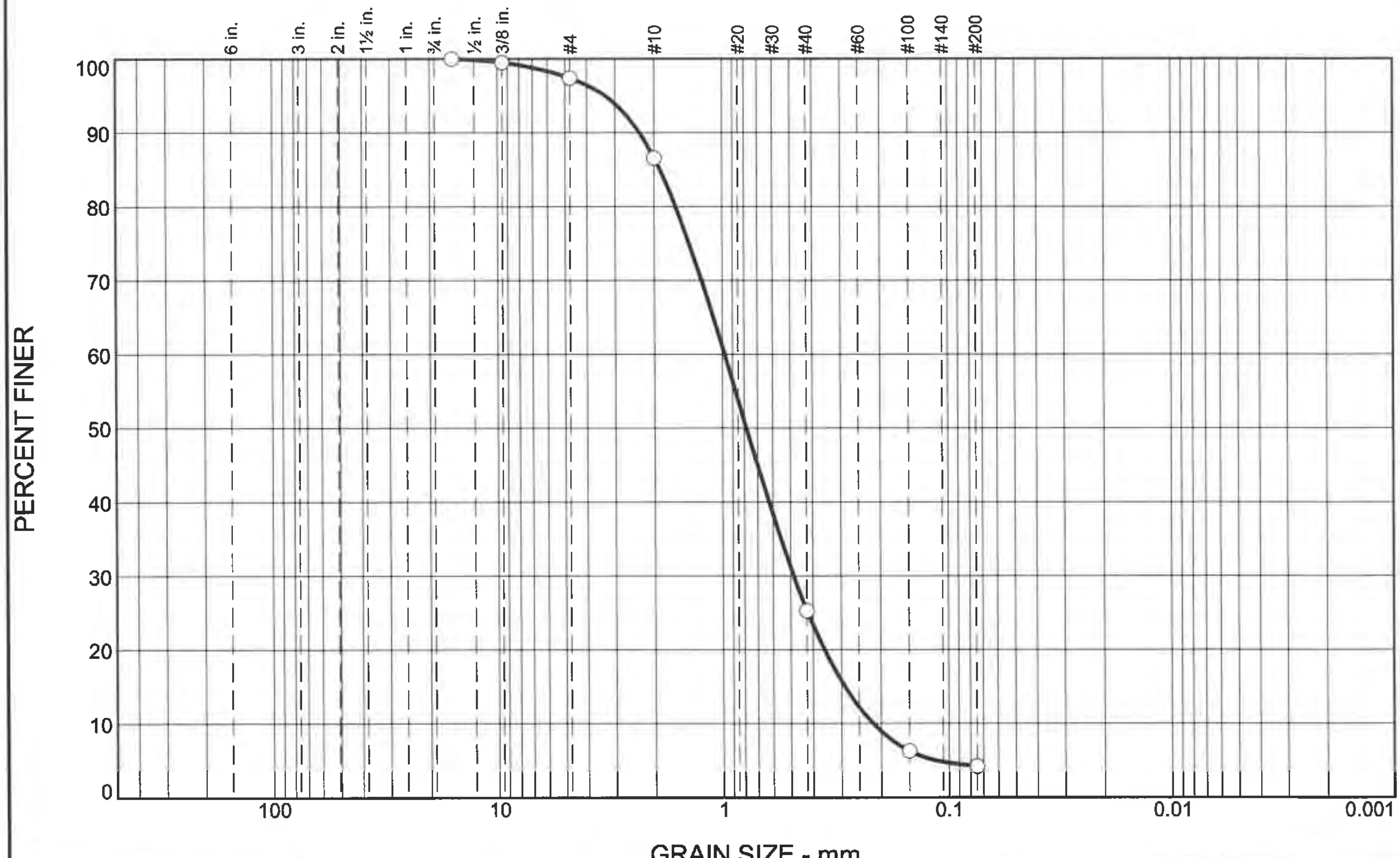
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	42.5	42.7	85.2	8.2	4.2	1.6	14.0			0.8

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
1.2397	3.5101	4.7916	5.8519	8.2142	11.9107	15.9957	21.0503	34.7903	38.1910	41.9730	46.1666

Fineness Modulus	C_u	C_c
6.91	6.00	0.91

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	2.6	10.8	61.3	21.1	4.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
5/8"	100.0		
3/8"	99.5		
#4	97.4		
#10	86.6		
#40	25.3		
#100	6.3		
#200	4.2		

* (no specification provided)

Material Description

Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 2.3241 D_{85}= 1.8870 D_{60}= 0.9796
D_{50}= 0.7833 D_{30}= 0.4861 D_{15}= 0.2897
D_{10}= 0.2169 C_u= 4.52 C_c= 1.11

Classification

USCS= SP AASHTO=

Remarks

MC=21.4%

Location: AMW-02
Sample Number: S-5 **Depth:** 15 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road

Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AMW-02
Depth: 15
Sample Number: S-5
Material Description: Sand
Date: 11/20/2020
USCS Classification: SP
Testing Remarks: MC=21.4%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 319.20
Tare Wt. = 15.80
Minus #200 from wash = 4.9%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
334.70	15.80	0.00	5/8"	0.00	100.0
			3/8"	1.60	99.5
			#4	8.30	97.4
			#10	42.80	86.6
			#40	238.30	25.3
			#100	298.80	6.3
			#200	305.40	4.2

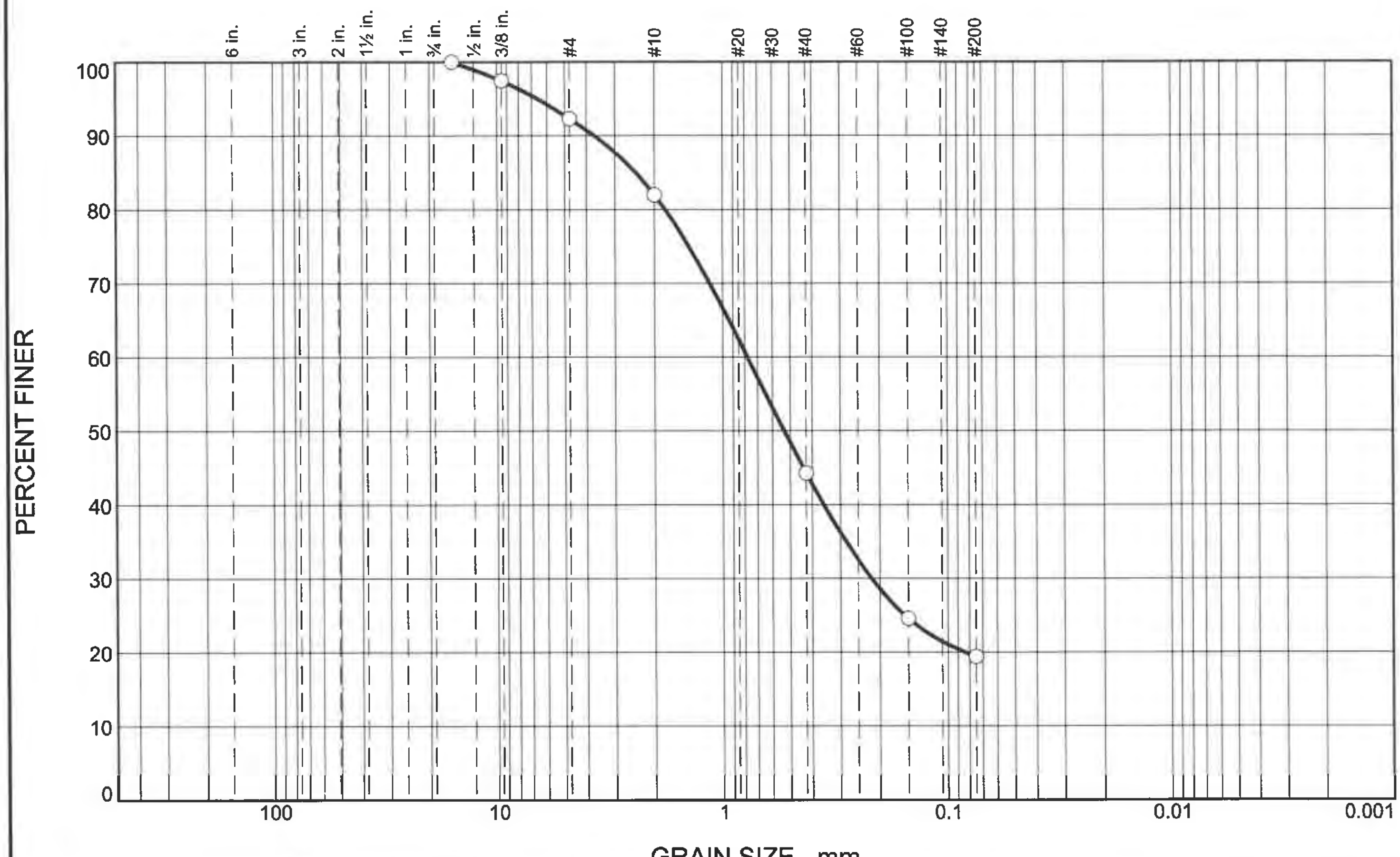
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	2.6	2.6	10.8	61.3	21.1	93.2			4.2

D_{5}	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.1142	0.2169	0.2897	0.3566	0.4861	0.6236	0.7833	0.9796	1.6093	1.8870	2.3241	3.3037

Fineness Modulus	C_u	C_c
2.84	4.52	1.11

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	7.7	10.3	37.7	24.9	19.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
5/8"	100.0		
3/8"	97.5		
#4	92.3		
#10	82.0		
#40	44.3		
#100	24.6		
#200	19.4		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 3.6896 D_{85}= 2.4233 D_{60}= 0.7766
D_{50}= 0.5326 D_{30}= 0.2178 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

MC=10.2%

Location: AMW-02
Sample Number: S-6 **Depth:** 20 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AMW-02
Depth: 20
Sample Number: S-6
Material Description: Silty sand
Date: 11/20/2020
USCS Classification: SM
Testing Remarks: MC=10.2%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 337.50
Tare Wt. = 16.20
Minus #200 from wash = 19.9%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
417.50	16.20	0.00	5/8"	0.00	100.0
			3/8"	10.10	97.5
			#4	30.80	92.3
			#10	72.40	82.0
			#40	223.70	44.3
			#100	302.60	24.6
			#200	323.40	19.4

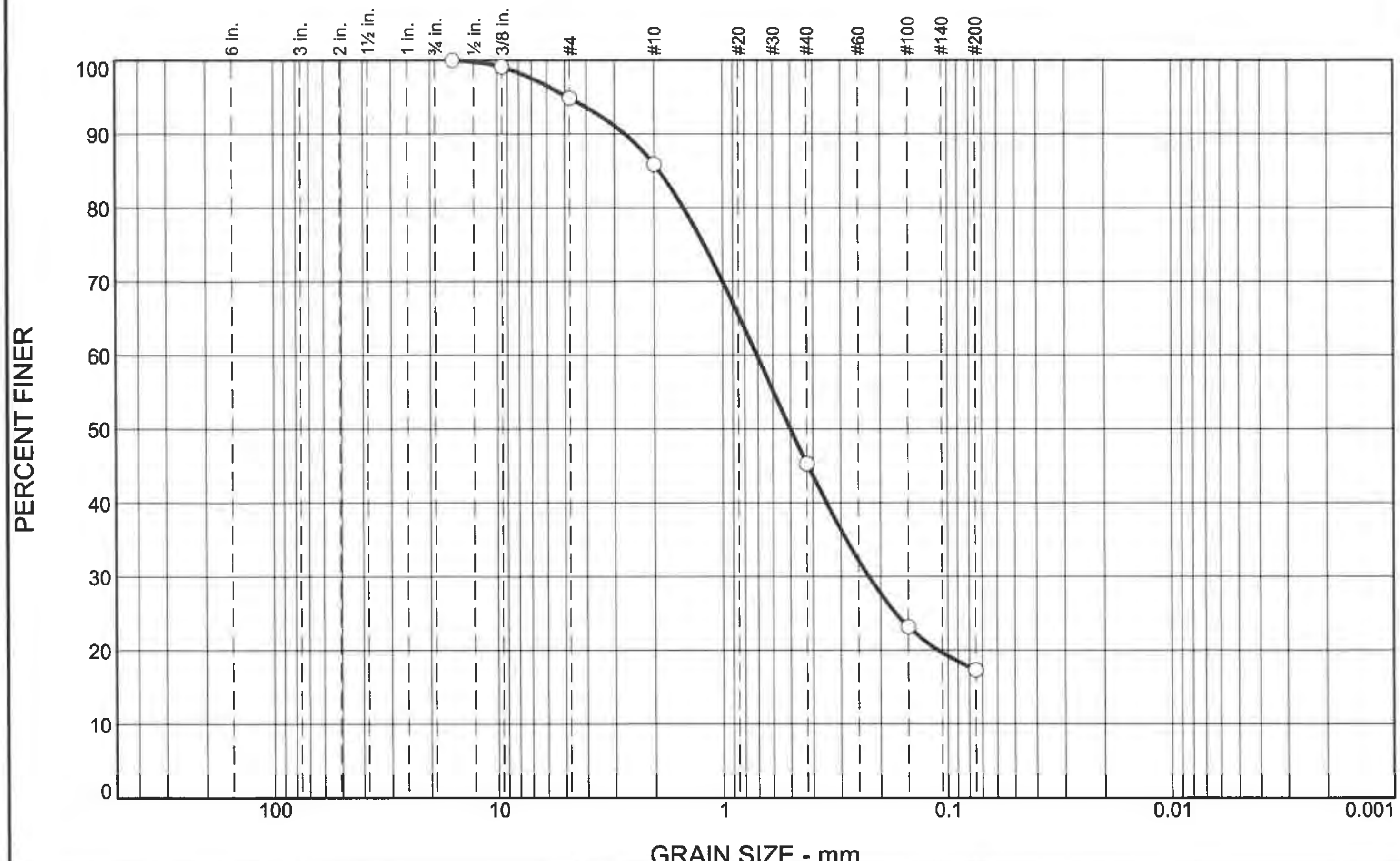
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	7.7	7.7	10.3	37.7	24.9	72.9			19.4

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
			0.0834	0.2178	0.3557	0.5326	0.7766	1.7972	2.4233	3.6896	6.5935

Fineness Modulus
2.41

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	5.1	9.0	40.6	28.0	17.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
5/8"	100.0		
3/8"	99.1		
#4	94.9		
#10	85.9		
#40	45.3		
#100	23.2		
#200	17.3		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 2.6898 D_{85}= 1.9012 D_{60}= 0.7065
D_{50}= 0.5017 D_{30}= 0.2256 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

MC=14.8%

Location: AMW-02
Sample Number: S-7 **Depth:** 25 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road

Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AMW-02
Depth: 25
Sample Number: S-7
Material Description: Silty sand
Date: 11/20/2020
USCS Classification: SM
Testing Remarks: MC=14.8%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 277.10
Tare Wt. = 15.90
Minus #200 from wash = 17.7%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
333.20	15.90	0.00	5/8"	0.00	100.0
			3/8"	2.80	99.1
			#4	16.20	94.9
			#10	44.80	85.9
			#40	173.50	45.3
			#100	243.70	23.2
			#200	262.30	17.3

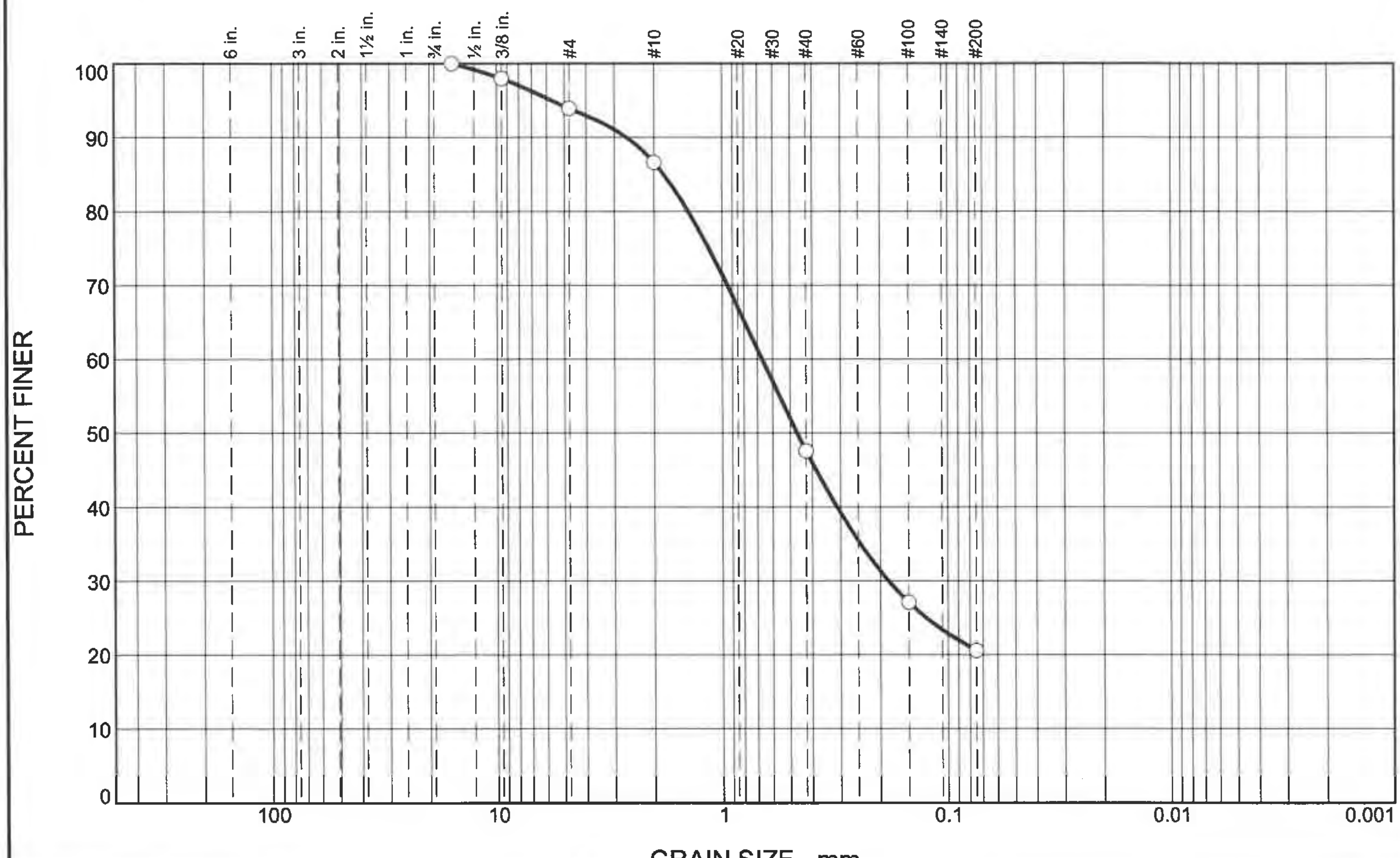
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	5.1	5.1	9.0	40.6	28.0	77.6			17.3

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
			0.1104	0.2256	0.3481	0.5017	0.7065	1.4872	1.9012	2.6898	4.8195

Fineness Modulus
2.29

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	6.1	7.3	39.0	27.0	20.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
5/8"	100.0		
3/8"	98.0		
#4	93.9		
#10	86.6		
#40	47.6		
#100	27.1		
#200	20.6		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 2.6416 D_{85}= 1.8117 D_{60}= 0.6664
D_{50}= 0.4662 D_{30}= 0.1840 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

MC=13.3%

Location: AMW-02
Sample Number: S-8 **Depth:** 30 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AMW-02
Depth: 30
Sample Number: S-8
Material Description: Silty sand
Date: 11/20/2020
USCS Classification: SM
Testing Remarks: MC=13.3%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 241.30
Tare Wt. = 12.60
Minus #200 from wash = 20.7%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
301.10	12.60	0.00	5/8"	0.00	100.0
			3/8"	5.80	98.0
			#4	17.50	93.9
			#10	38.60	86.6
			#40	151.30	47.6
			#100	210.30	27.1
			#200	229.10	20.6

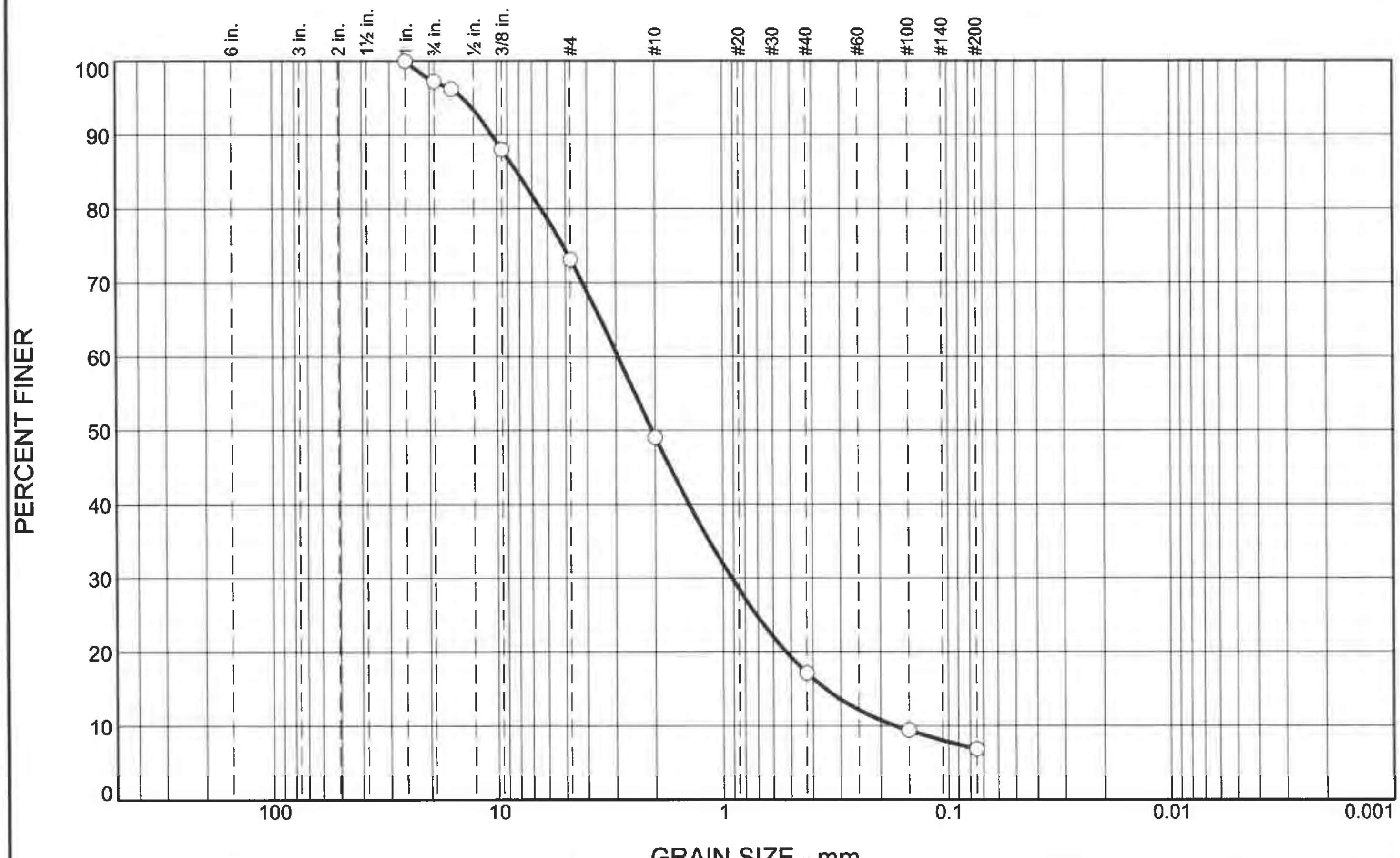
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	6.1	6.1	7.3	39.0	27.0	73.3			20.6

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
				0.1840	0.3110	0.4662	0.6664	1.4131	1.8117	2.6416	5.6787

Fineness Modulus
2.20

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	2.8	24.1	24.0	32.0	10.2	6.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1"	100.0		
3/4"	97.2		
5/8"	96.2		
3/8"	88.0		
#4	73.1		
#10	49.1		
#40	17.1		
#100	9.4		
#200	6.9		

* (no specification provided)

Material Description

Sand with silt and gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 10.5155 D_{85}= 8.1899 D_{60}= 2.9331
D_{50}= 2.0678 D_{30}= 0.9175 D_{15}= 0.3505
D_{10}= 0.1707 C_u= 17.18 C_c= 1.68

Classification

USCS= SW-SM AASHTO=

Remarks

MC=12%

Location: AMW-02
Sample Number: S-10 **Depth:** 40 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AMW-02
Depth: 40
Sample Number: S-10
Material Description: Sand with silt and gravel
Date: 11/20/2020
USCS Classification: SW-SM
Testing Remarks: MC=12%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 425.30
Tare Wt. = 15.90
Minus #200 from wash = 7.3%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
457.70	15.90	0.00	1"	0.00	100.0
			3/4"	12.30	97.2
			5/8"	16.80	96.2
			3/8"	52.90	88.0
			#4	118.80	73.1
			#10	225.00	49.1
			#40	366.10	17.1
			#100	400.20	9.4
			#200	411.50	6.9

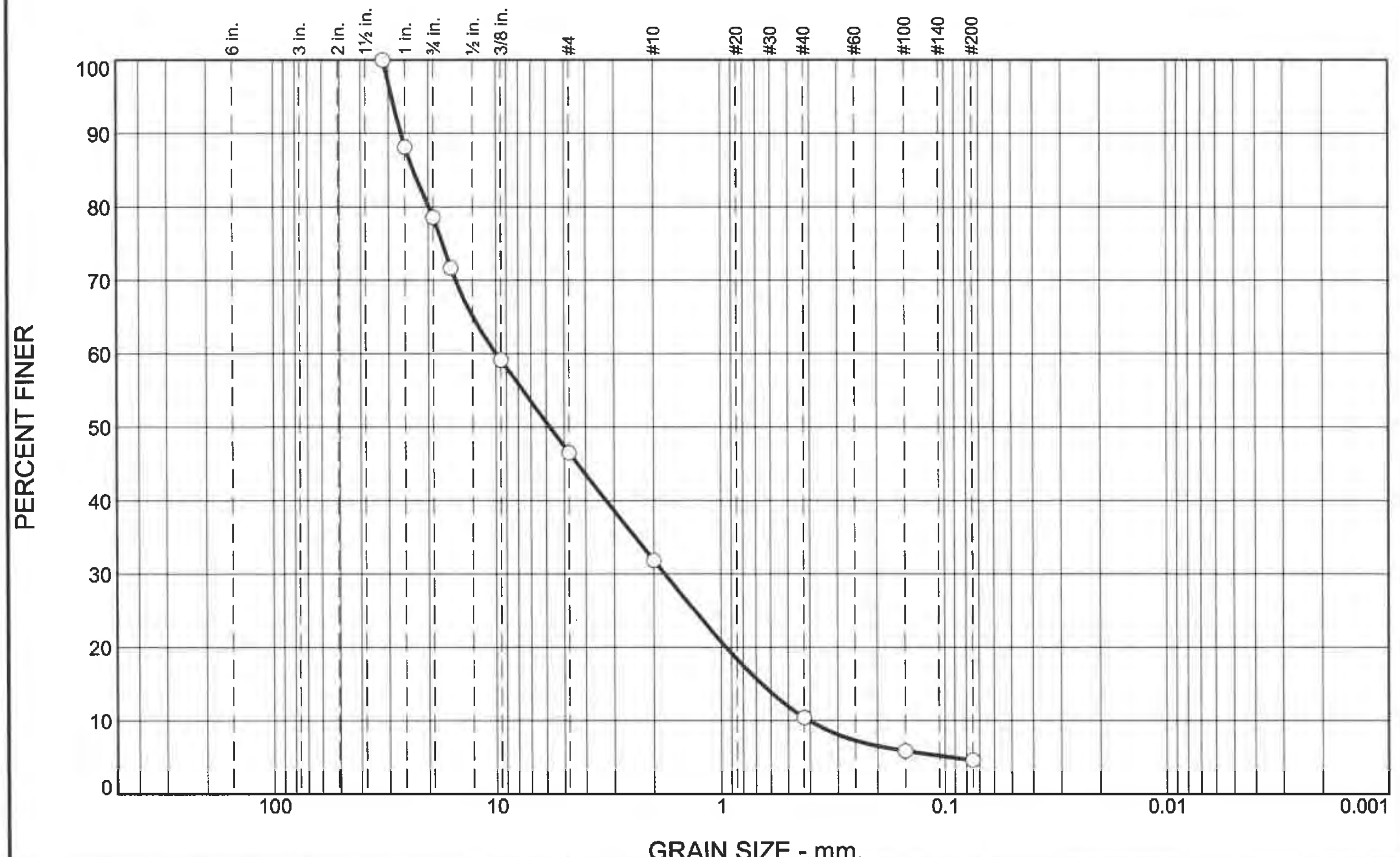
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	2.8	24.1	26.9	24.0	32.0	10.2	66.2			6.9

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
	0.1707	0.3505	0.5253	0.9175	1.4179	2.0678	2.9331	6.4248	8.1899	10.5155	14.1256

Fineness Modulus	C_u	C_c
4.07	17.18	1.68

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	21.4	32.1	14.6	21.5	5.8	4.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/4"	100.0		
1"	88.1		
3/4"	78.6		
5/8"	71.7		
3/8"	59.1		
#4	46.5		
#10	31.9		
#40	10.4		
#100	5.9		
#200	4.6		

* (no specification provided)

Material Description

Gravel with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 26.4667 D_{85}= 23.3718 D_{60}= 9.9737
D_{50}= 5.7800 D_{30}= 1.7877 D_{15}= 0.6587
D_{10}= 0.4028 C_u= 24.76 C_c= 0.80

Classification

USCS= GP AASHTO=

Remarks

MC=8.3%

Location: AMW-02
Sample Number: G-2 **Depth:** 50 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AMW-02
Depth: 50
Sample Number: G-2
Material Description: Gravel with sand
Date: 11/20/2020
USCS Classification: GP
Testing Remarks: MC=8.3%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 563.50
Tare Wt. = 12.60
Minus #200 from wash = 4.7%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
590.70	12.60	0.00	1 1/4"	0.00	100.0
			1"	68.60	88.1
			3/4"	123.80	78.6
			5/8"	163.70	71.7
			3/8"	236.20	59.1
			#4	309.20	46.5
			#10	393.90	31.9
			#40	517.80	10.4
			#100	544.10	5.9
			#200	551.30	4.6

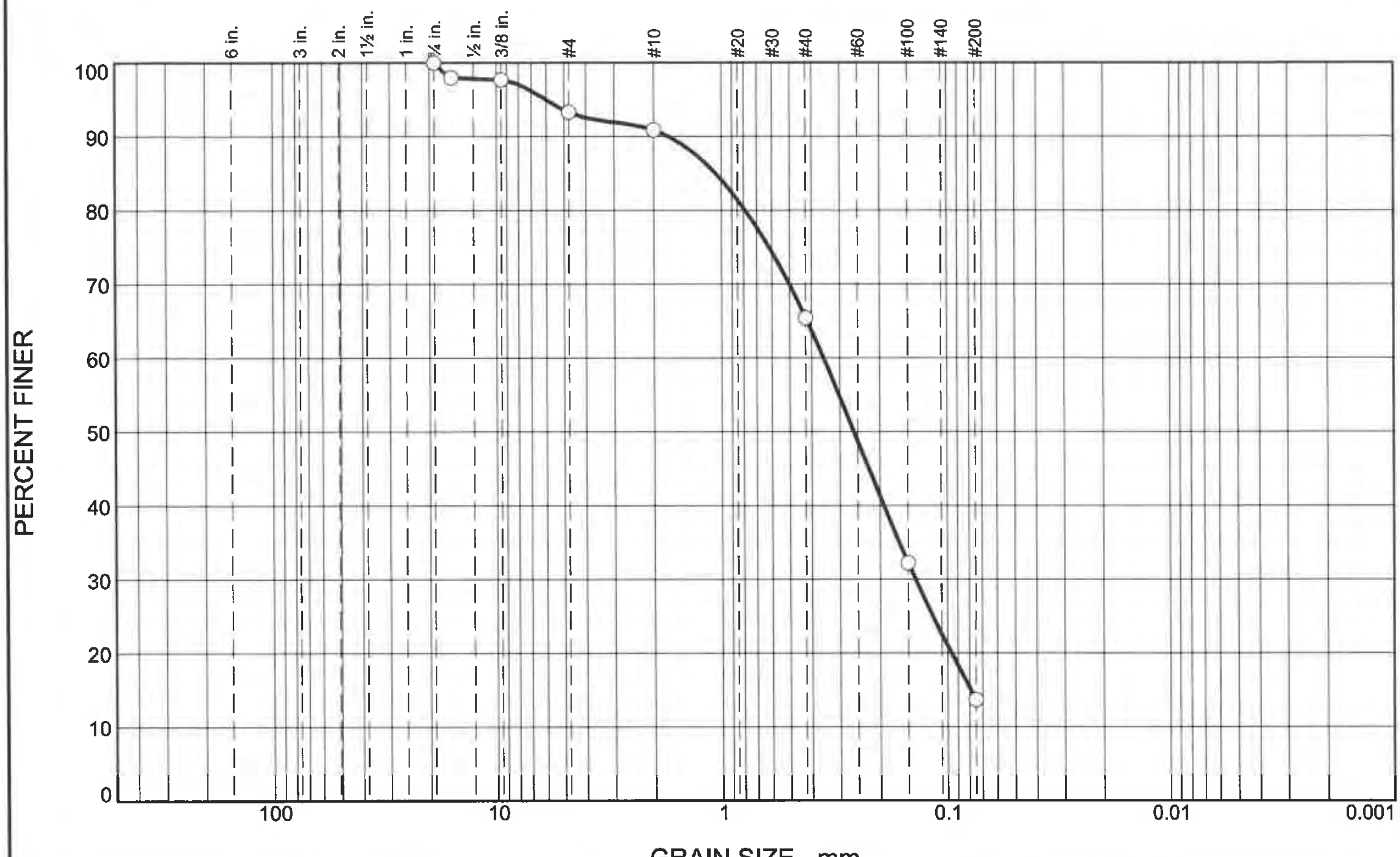
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	21.4	32.1	53.5	14.6	21.5	5.8	41.9			4.6

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.0933	0.4028	0.6587	0.9517	1.7877	3.2534	5.7800	9.9737	19.8825	23.3718	26.4667	29.1125

Fineness Modulus	C_u	C_c
5.30	24.76	0.80

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	6.7	2.4	25.5	51.7	13.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
5/8"	98.0		
3/8"	97.7		
#4	93.3		
#10	90.9		
#40	65.4		
#100	32.2		
#200	13.7		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 1.7107 D_{85}= 1.0641 D_{60}= 0.3549
D_{50}= 0.2604 D_{30}= 0.1391 D_{15}= 0.0790
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

MC=17.4%

Location: AMW-02
Sample Number: G-3 **Depth:** 59 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC
Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: AMW-02
Depth: 59 **Sample Number:** G-3
Material Description: Silty sand
Date: 11/20/2020
USCS Classification: SM
Testing Remarks: MC=17.4%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 482.90
Tare Wt. = 12.80
Minus #200 from wash = 11.2%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
542.20	12.80	0.00	3/4"	0.00	100.0
			5/8"	10.80	98.0
			3/8"	12.30	97.7
			#4	35.40	93.3
			#10	48.20	90.9
			#40	183.20	65.4
			#100	358.80	32.2
			#200	457.00	13.7

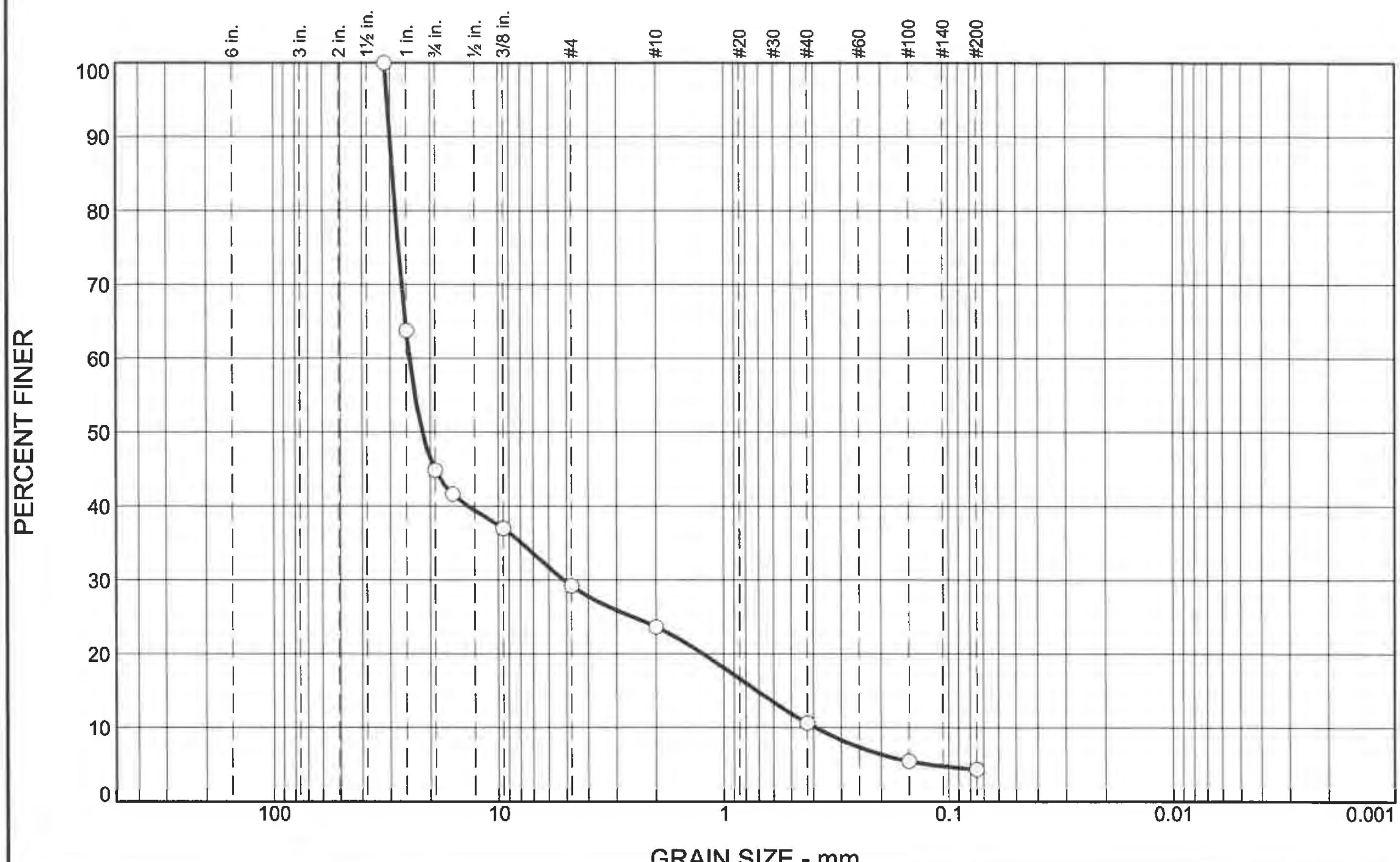
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	6.7	6.7	2.4	25.5	51.7	79.6			13.7

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
		0.0790	0.0962	0.1391	0.1923	0.2604	0.3549	0.7863	1.0641	1.7107	5.9988

Fineness Modulus
1.70

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	55.2	15.6	5.5	13.1	6.3	4.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/4"	100.0		
1"	63.8		
3/4"	44.8		
5/8"	41.6		
3/8"	37.0		
#4	29.2		
#10	23.7		
#40	10.6		
#100	5.5		
#200	4.3		

* (no specification provided)

Material Description

Gravel with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 30.0411 D_{85}= 29.2005 D_{60}= 24.5783
D_{50}= 21.6821 D_{30}= 5.1396 D_{15}= 0.7111
D_{10}= 0.3889 C_u= 63.20 C_c= 2.76

Classification

USCS= GW AASHTO=

Remarks

MC=2.5%

Location: AMW-03
Sample Number: S-1 **Depth:** 2.5 **Date:** 1/23/2021

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175 / 190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

1/23/2021

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175 / 190175-JR
Location: AMW-03
Depth: 2.5 **Sample Number:** S-1
Material Description: Gravel with sand
Date: 1/23/2021
USCS Classification: GW
Testing Remarks: MC=2.5%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 587.30
Tare Wt. = 12.80
Minus #200 from wash = 4.4%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
614.00	12.80	0.00	1 1/4"	0.00	100.0
			1"	217.80	63.8
			3/4"	331.60	44.8
			5/8"	350.90	41.6
			3/8"	378.90	37.0
			#4	425.60	29.2
			#10	459.00	23.7
			#40	537.20	10.6
			#100	568.40	5.5
			#200	575.10	4.3

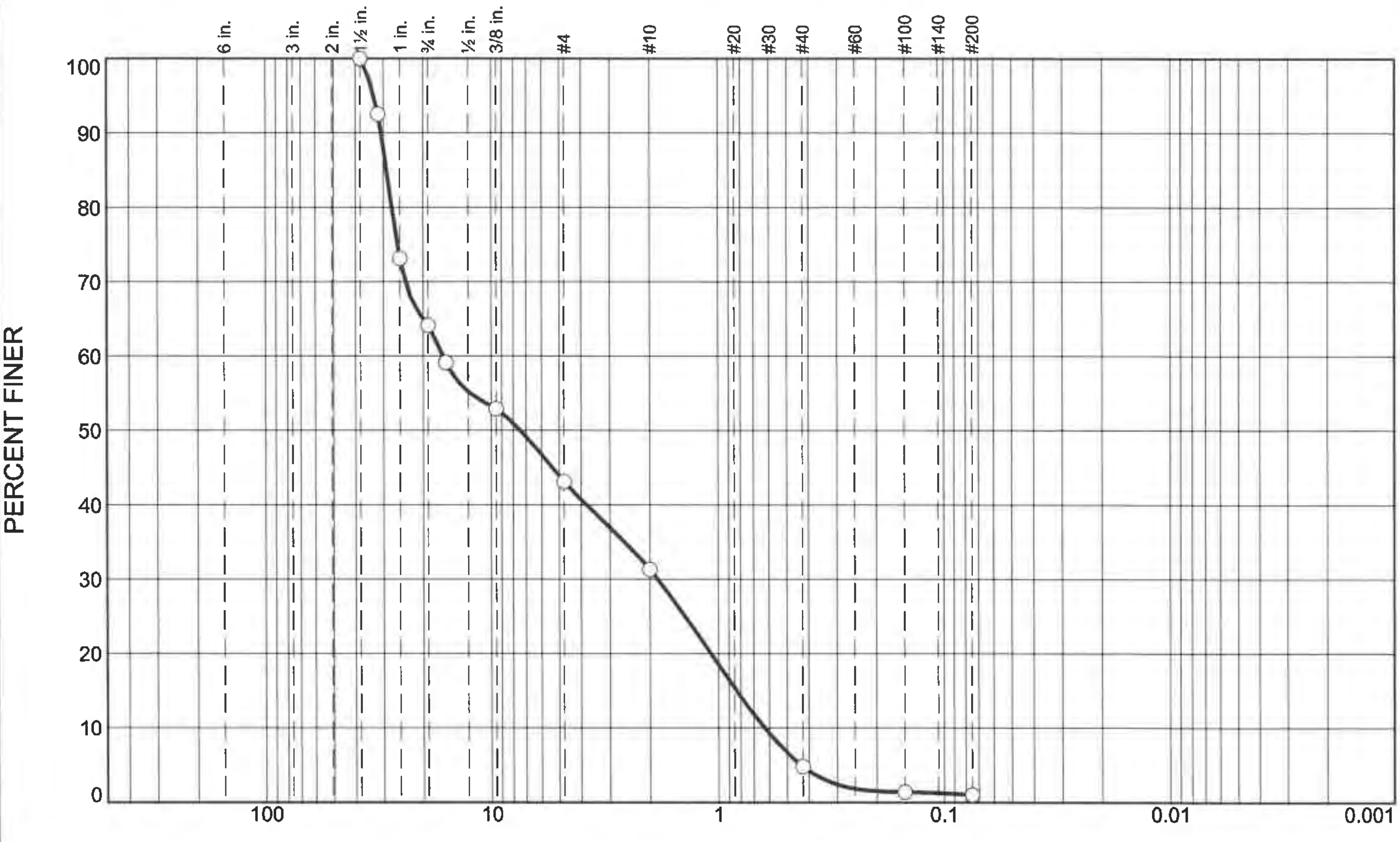
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	55.2	15.6	70.8	5.5	13.1	6.3	24.9			4.3

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.1220	0.3889	0.7111	1.2363	5.1396	13.5574	21.6821	24.5783	28.3563	29.2005	30.0411	30.8880

Fineness Modulus	C_u	C_c
6.17	63.20	2.76

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	35.9	21.0	11.8	26.5	3.8	1.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1.5"	100.0		
1 1/4"	92.5		
1"	73.1		
3/4"	64.1		
5/8"	59.1		
3/8"	52.9		
#4	43.1		
#10	31.3		
#40	4.8		
#100	1.4		
#200	1.0		

* (no specification provided)

Material Description

Gravel with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 30.7179 D_{85}= 29.0512 D_{60}= 16.3998
D_{50}= 7.4437 D_{30}= 1.8467 D_{15}= 0.8323
D_{10}= 0.6263 C_u= 26.19 C_c= 0.33

Classification

USCS= GP AASHTO=

Remarks

MC=7.9%

Location: AMW-03
Sample Number: G-1 **Depth:** 7.5 **Date:** 1/23/2021

Hayre McElroy & Associates, LLC
Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175 / 190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

1/23/2021

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175 / 190175-JR
Location: AMW-03
Depth: 7.5 **Sample Number:** G-1
Material Description: Gravel with sand
Date: 1/23/2021
USCS Classification: GP
Testing Remarks: MC=7.9%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 706.60
Tare Wt. = 13.50
Minus #200 from wash = 1.1%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
714.50	13.50	0.00	1.5"	0.00	100.0
			1 1/4"	52.40	92.5
			1"	188.30	73.1
			3/4"	251.50	64.1
			5/8"	286.60	59.1
			3/8"	330.30	52.9
			#4	398.70	43.1
			#10	481.60	31.3
			#40	667.20	4.8
			#100	691.20	1.4
			#200	693.80	1.0

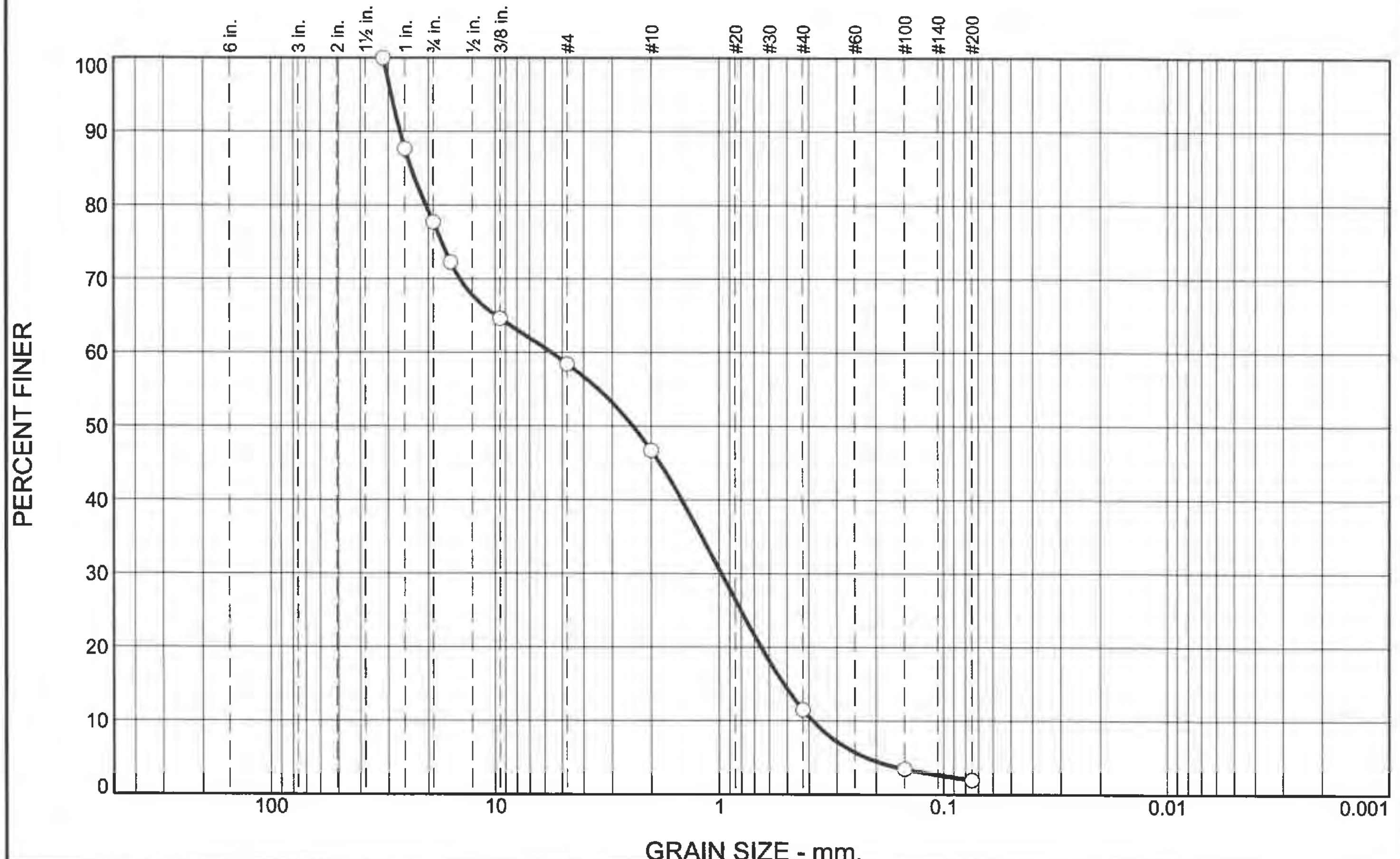
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	35.9	21.0	56.9	11.8	26.5	3.8	42.1			1.0

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.4325	0.6263	0.8323	1.0796	1.8467	3.7935	7.4437	16.3998	27.5627	29.0512	30.7179	33.0737

Fineness Modulus	C_u	C_c
5.71	26.19	0.33

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	22.3	19.3	11.7	35.2	9.5	2.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/4"	100.0		
1"	87.7		
3/4"	77.7		
5/8"	72.3		
3/8"	64.6		
#4	58.4		
#10	46.7		
#40	11.5		
#100	3.5		
#200	2.0		

* (no specification provided)

Material Description

Sand with gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 26.6515 D_{85}= 23.8407 D_{60}= 5.6084
D_{50}= 2.4083 D_{30}= 0.9803 D_{15}= 0.5162
D_{10}= 0.3831 C_u= 14.64 C_c= 0.45

Classification

USCS= SP AASHTO=

Remarks

MC=11.3%

Location: AMW-03
Sample Number: G-2 **Depth:** 12.5 **Date:** 1/23/2021

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road

Project No: 08-175 / 190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

1/23/2021

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175 / 190175-JR
Location: AMW-03
Depth: 12.5 **Sample Number:** G-2
Material Description: Sand with gravel
Date: 1/23/2021
USCS Classification: SP
Testing Remarks: MC=11.3%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 574.70
Tare Wt. = 16.40
Minus #200 from wash = 2.0%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
586.20	16.40	0.00	1 1/4"	0.00	100.0
			1"	70.30	87.7
			3/4"	126.90	77.7
			5/8"	158.00	72.3
			3/8"	201.50	64.6
			#4	236.80	58.4
			#10	303.70	46.7
			#40	504.10	11.5
			#100	549.80	3.5
			#200	558.60	2.0

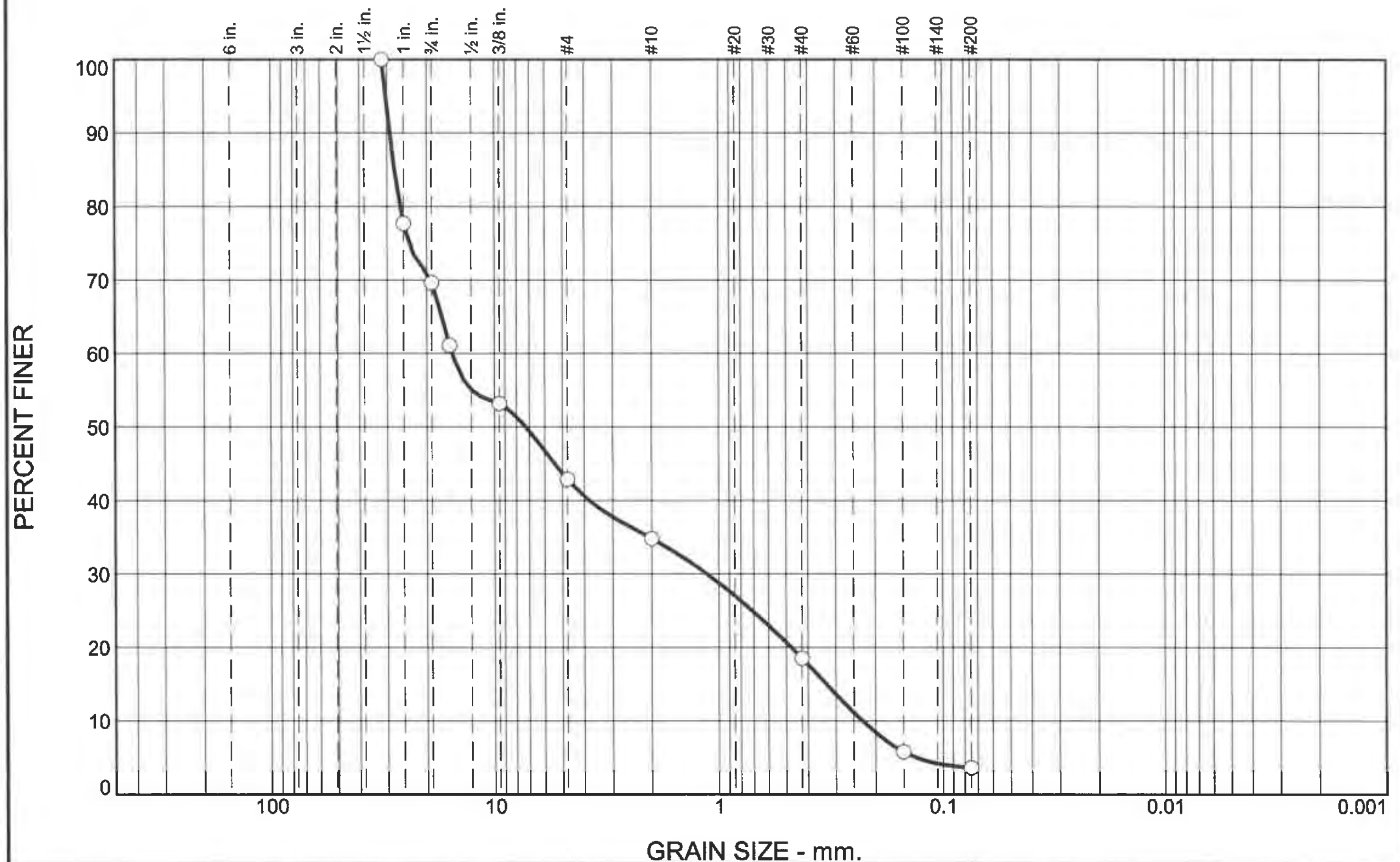
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	22.3	19.3	41.6	11.7	35.2	9.5	56.4			2.0

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.2202	0.3831	0.5162	0.6519	0.9803	1.4672	2.4083	5.6084	20.5373	23.8407	26.6515	29.1862

Fineness Modulus	C_u	C_c
4.86	14.64	0.45

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	30.4	26.7	8.1	16.3	14.9	3.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/4"	100.0		
1"	77.7		
3/4"	69.6		
5/8"	61.1		
3/8"	53.2		
#4	42.9		
#10	34.8		
#40	18.5		
#100	5.8		
#200	3.6		

* (no specification provided)

Material Description

Gravel with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 29.1269 D_{85}= 27.7693 D_{60}= 15.4641
D_{50}= 7.2623 D_{30}= 1.1349 D_{15}= 0.3309
D_{10}= 0.2281 C_u= 67.80 C_c= 0.37

Classification

USCS= GP AASHTO=

Remarks

MC=10.1%

Location: AMW-03
Sample Number: S-5 **Depth:** 20 **Date:** 1/23/2021

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175 / 190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

1/23/2021

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175 / 190175-JR
Location: AMW-03
Depth: 20 **Sample Number:** S-5
Material Description: Gravel with sand
Date: 1/23/2021
USCS Classification: GP
Testing Remarks: MC=10.1%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 605.10
Tare Wt. = 12.60
Minus #200 from wash = 3.5%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
626.50	12.60	0.00	1 1/4"	0.00	100.0
			1"	136.80	77.7
			3/4"	186.50	69.6
			5/8"	238.80	61.1
			3/8"	287.30	53.2
			#4	350.70	42.9
			#10	400.10	34.8
			#40	500.30	18.5
			#100	578.20	5.8
			#200	591.70	3.6

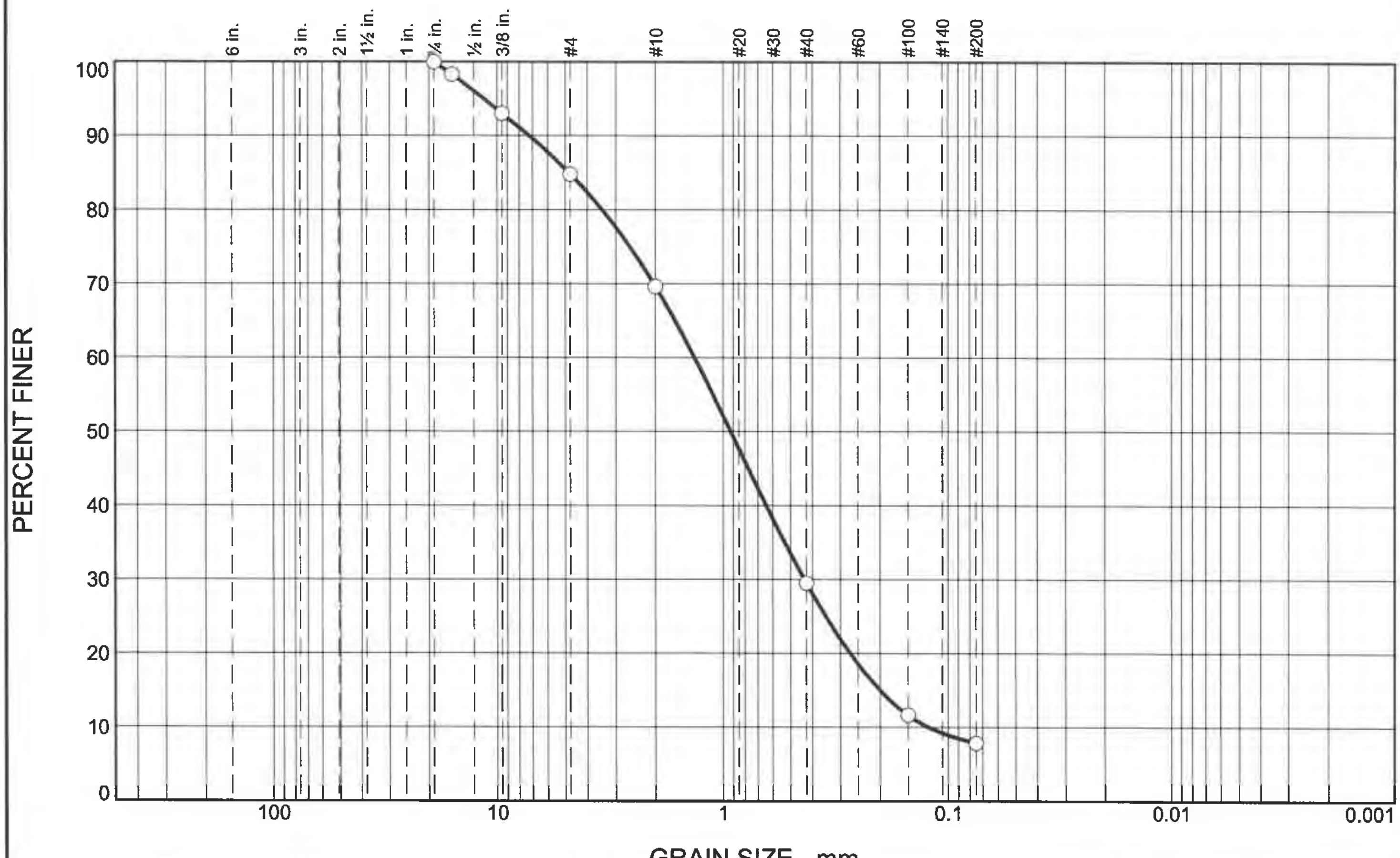
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	30.4	26.7	57.1	8.1	16.3	14.9	39.3			3.6

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.1309	0.2281	0.3309	0.4749	1.1349	3.7969	7.2623	15.4641	26.2435	27.7693	29.1269	30.4340

Fineness Modulus	C_u	C_c
5.25	67.80	0.37

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	15.2	15.2	40.1	21.6	7.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
5/8"	98.3		
3/8"	93.0		
#4	84.8		
#10	69.6		
#40	29.5		
#100	11.7		
#200	7.9		

* (no specification provided)

Material Description

Sand with silt and gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 7.2306 D_{85}= 4.8163 D_{60}= 1.3456
D_{50}= 0.9272 D_{30}= 0.4343 D_{15}= 0.1995
D_{10}= 0.1212 C_u= 11.11 C_c= 1.16

Classification

USCS= SW-SM AASHTO=

Remarks

MC=13%

Location: AMW-03
Sample Number: S-7 **Depth:** 30 **Date:** 1/23/2021

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road

Project No: 08-175 / 190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

1/23/2021

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175 / 190175-JR
Location: AMW-03
Depth: 30
Sample Number: S-7
Material Description: Sand with silt and gravel
Date: 1/23/2021
USCS Classification: SW-SM
Testing Remarks: MC=13%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 457.10
Tare Wt. = 12.50
Minus #200 from wash = 7.6%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
493.80	12.50	0.00	3/4"	0.00	100.0
			5/8"	8.10	98.3
			3/8"	33.50	93.0
			#4	73.10	84.8
			#10	146.20	69.6
			#40	339.40	29.5
			#100	425.20	11.7
			#200	443.40	7.9

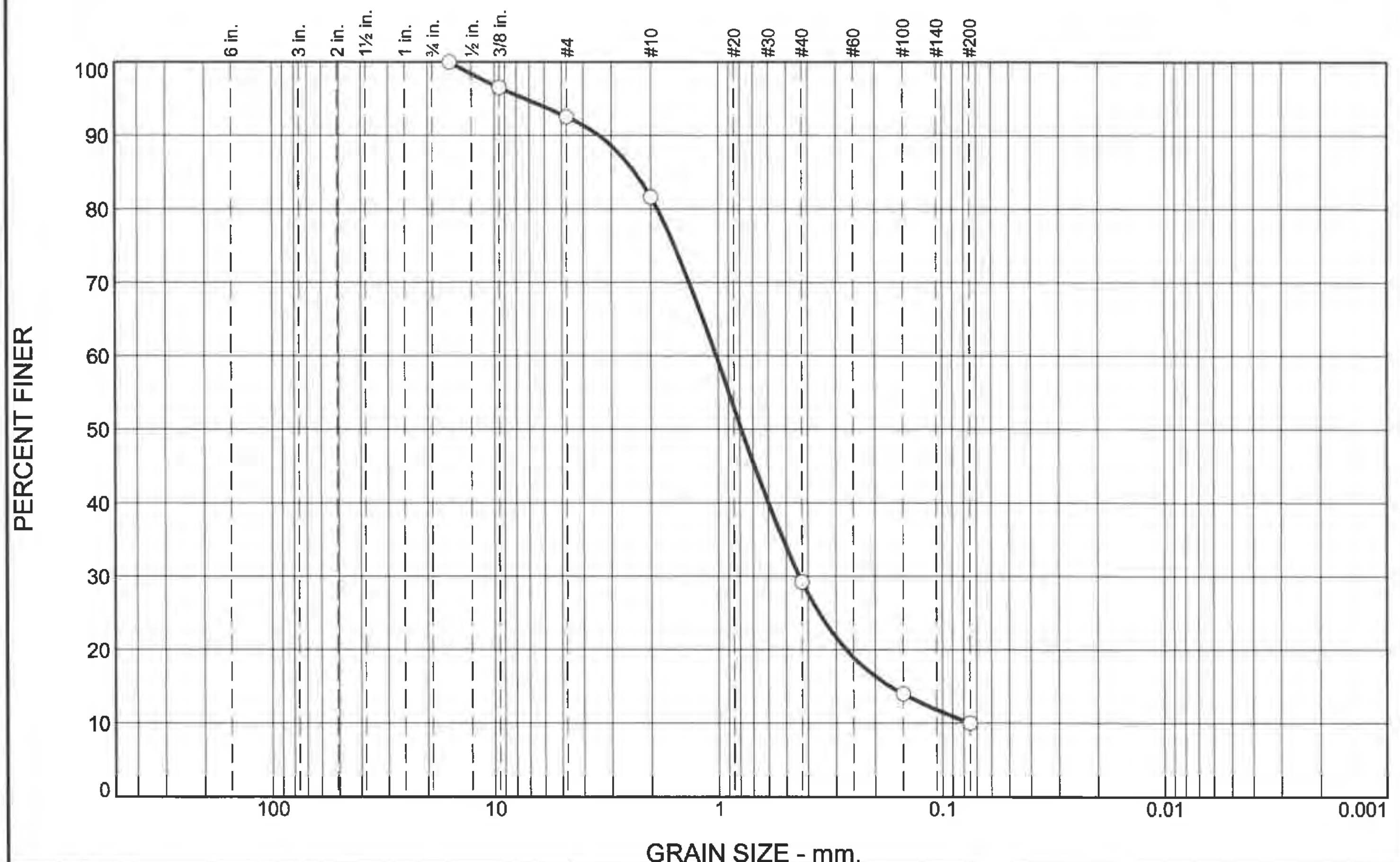
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	15.2	15.2	15.2	40.1	21.6	76.9			7.9

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
	0.1212	0.1995	0.2714	0.4343	0.6424	0.9272	1.3456	3.4407	4.8163	7.2306	11.4357

Fineness Modulus	C_u	C_c
3.21	11.11	1.16

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	7.4	11.0	52.4	19.2	10.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
5/8"	100.0		
3/8"	96.5		
#4	92.6		
#10	81.6		
#40	29.2		
#100	14.0		
#200	10.0		

* (no specification provided)

Material Description

Sand with silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 3.3965 D_{85}= 2.3523 D_{60}= 1.0244
D_{50}= 0.7880 D_{30}= 0.4376 D_{15}= 0.1709
D_{10}= C_u= C_c=

Classification

USCS= SW-SM AASHTO=

Remarks

MC=19.8%

Location: AMW-03
Sample Number: S-9 **Depth:** 40 **Date:** 1/23/2021

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175 / 190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

1/23/2021

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175 / 190175-JR
Location: AMW-03
Depth: 40
Sample Number: S-9
Material Description: Sand with silt
Date: 1/23/2021
USCS Classification: SW-SM
Testing Remarks: MC=19.8%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 493.20
Tare Wt. = 12.90
Minus #200 from wash = 9.6%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
544.30	12.90	0.00	5/8"	0.00	100.0
			3/8"	18.50	96.5
			#4	39.50	92.6
			#10	97.70	81.6
			#40	376.20	29.2
			#100	456.90	14.0
			#200	478.20	10.0

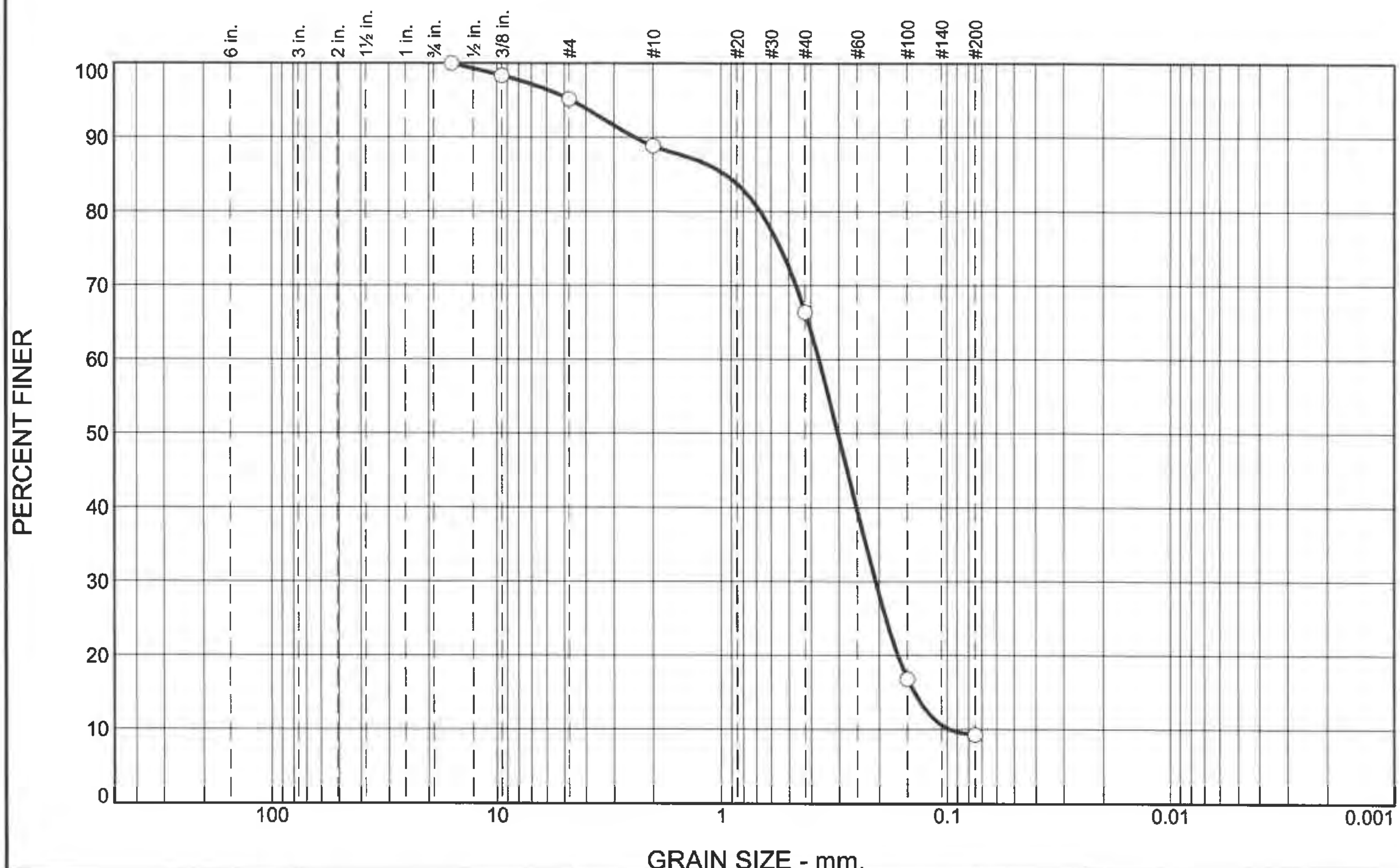
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	7.4	7.4	11.0	52.4	19.2	82.6			10.0

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
		0.1709	0.2697	0.4376	0.6003	0.7880	1.0244	1.8736	2.3523	3.3965	7.3029

Fineness Modulus
2.85

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	4.8	6.4	22.4	57.1	9.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
5/8"	100.0		
3/8"	98.4		
#4	95.2		
#10	88.8		
#40	66.4		
#100	16.8		
#200	9.3		

* (no specification provided)

Material Description

Sand with silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 2.4128 D_{85}= 0.9574 D_{60}= 0.3688
D_{50}= 0.3036 D_{30}= 0.2078 D_{15}= 0.1401
D_{10}= 0.0970 C_u= 3.80 C_c= 1.21

Classification

USCS= SP-SM AASHTO=

Remarks

MC=20.5%

Location: AMW-03
Sample Number: G-3 **Depth:** 55 **Date:** 1/23/2021

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road

Project No: 08-175 / 190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

1/23/2021

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175 / 190175-JR
Location: AMW-03
Depth: 55
Sample Number: G-3
Material Description: Sand with silt
Date: 1/23/2021
USCS Classification: SP-SM
Testing Remarks: MC=20.5%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 458.00
Tare Wt. = 12.80
Minus #200 from wash = 8.7%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
500.30	12.80	0.00	5/8"	0.00	100.0
			3/8"	8.00	98.4
			#4	23.60	95.2
			#10	54.40	88.8
			#40	163.80	66.4
			#100	405.40	16.8
			#200	442.40	9.3

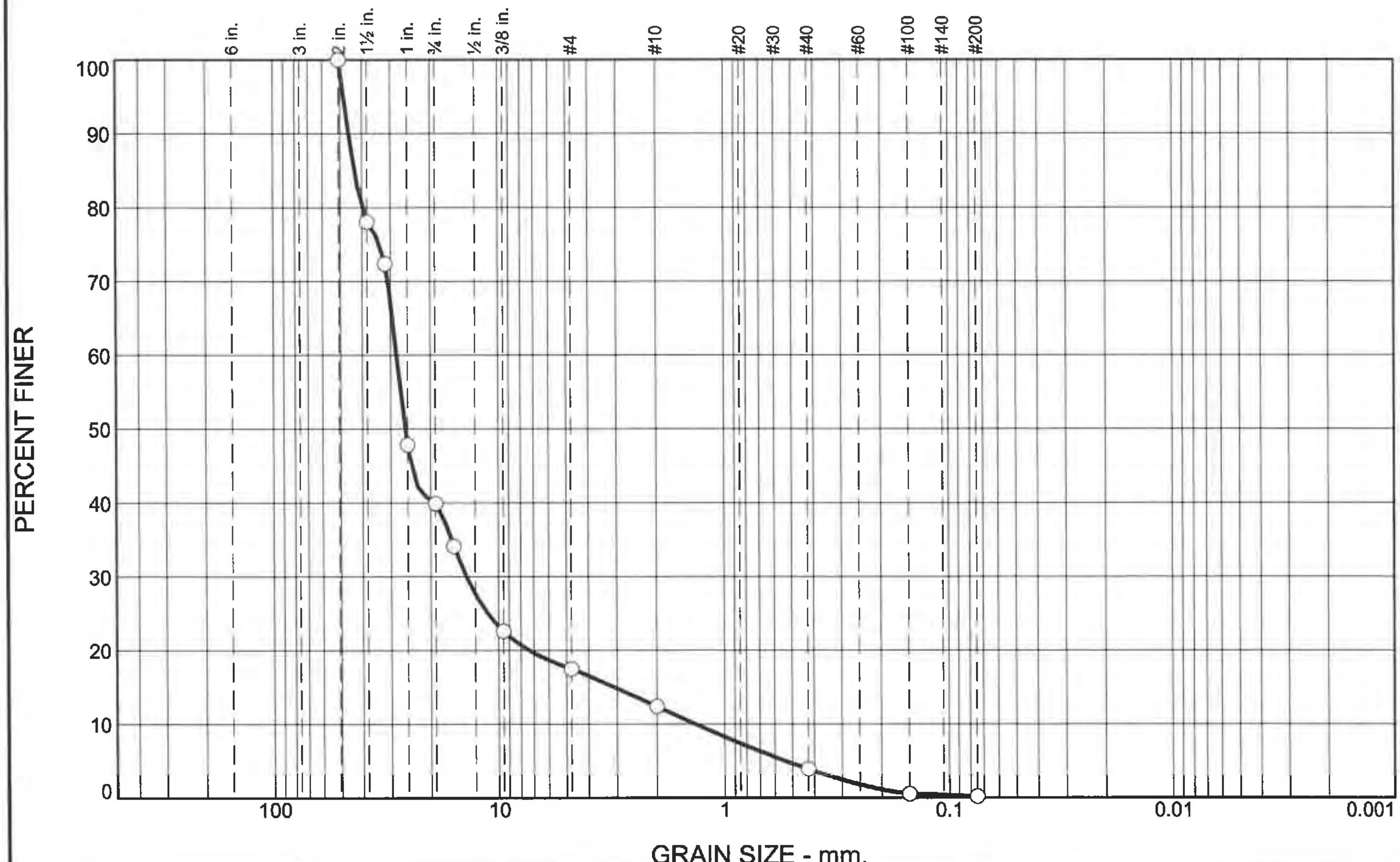
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	4.8	4.8	6.4	22.4	57.1	85.9			9.3

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
	0.0970	0.1401	0.1650	0.2078	0.2522	0.3036	0.3688	0.6700	0.9574	2.4128	4.6383

Fineness Modulus	C_u	C_c
1.86	3.80	1.21

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	60.1	22.4	5.1	8.5	3.7	0.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2"	100.0		
1.5"	78.1		
1 1/4"	72.4		
1"	47.9		
3/4"	39.9		
5/8"	34.1		
3/8"	22.6		
#4	17.5		
#10	12.4		
#40	3.9		
#100	0.5		
#200	0.2		

* (no specification provided)

Material Description

Gravel with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 45.7470 D_{85}= 43.0913 D_{60}= 28.2732
D_{50}= 25.9798 D_{30}= 13.9451 D_{15}= 3.0649
D_{10}= 1.3435 C_u= 21.04 C_c= 5.12

Classification

USCS= GP AASHTO=

Remarks

MC=1.6%

Location: ATP-01
Sample Number: S-1 **Depth:** 4 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road

Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: ATP-01
Depth: 4
Sample Number: S-1
Material Description: Gravel with sand
Date: 11/20/2020
USCS Classification: GP
Testing Remarks: MC=1.6%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 1076.80
Tare Wt. = 12.60
Minus #200 from wash = 0.2%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
1079.10	12.60	0.00	2"	0.00	100.0
			1.5"	234.00	78.1
			1 1/4"	294.60	72.4
			1"	556.10	47.9
			3/4"	641.00	39.9
			5/8"	702.60	34.1
			3/8"	825.30	22.6
			#4	880.10	17.5
			#10	934.40	12.4
			#40	1024.60	3.9
			#100	1061.00	0.5
			#200	1064.40	0.2

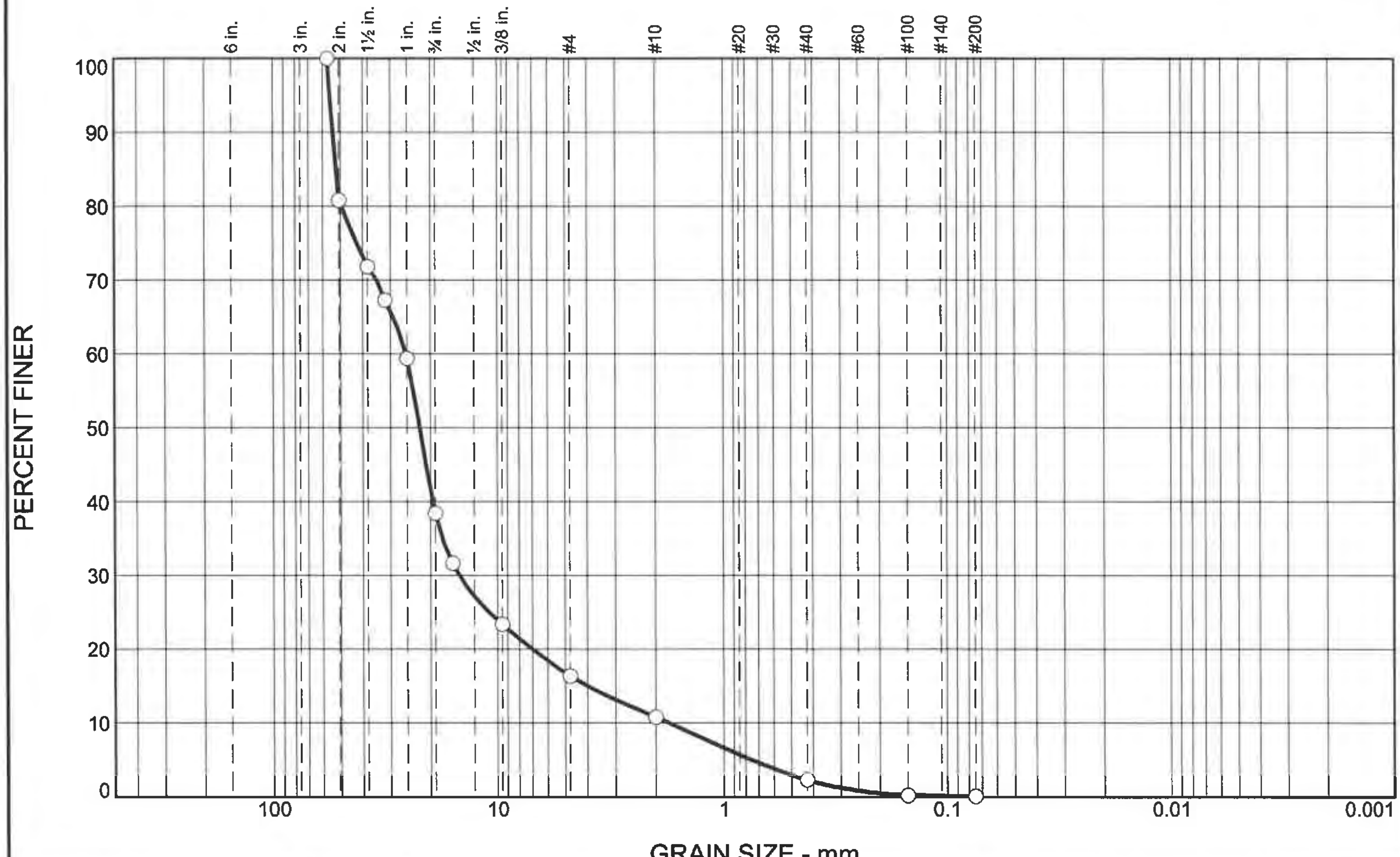
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	60.1	22.4	82.5	5.1	8.5	3.7	17.3			0.2

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.5333	1.3435	3.0649	7.3293	13.9451	19.1693	25.9798	28.2732	39.8974	43.0913	45.7470	48.2651

Fineness Modulus	C_u	C_c
7.11	21.04	5.12

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	61.6	22.0	5.6	8.5	2.2	0.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2 1/4"	100.0		
2"	80.8		
1.5"	71.8		
1 1/4"	67.3		
1"	59.4		
3/4"	38.4		
5/8"	31.6		
3/8"	23.4		
#4	16.4		
#10	10.8		
#40	2.3		
#100	0.2		
#200	0.1		

* (no specification provided)

Material Description

Gravel with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 54.0499 D_{85}= 52.3901 D_{60}= 25.6933
D_{50}= 22.2901 D_{30}= 14.7918 D_{15}= 3.9765
D_{10}= 1.7413 C_u= 14.75 C_c= 4.89

Classification

USCS= GP AASHTO=

Remarks

MC=1.3%

Location: ATP-02
Sample Number: S-1 **Depth:** 4 **Date:** 11/20/2020

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175/190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

11/22/2020

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175/190175-JR
Location: ATP-02
Depth: 4
Sample Number: S-1
Material Description: Gravel with sand
Date: 11/20/2020
USCS Classification: GP
Testing Remarks: MC=1.3%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 1216.40
Tare Wt. = 12.50
Minus #200 from wash = 0.1%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
1218.00	12.50	0.00	2 1/4"	0.00	100.0
			2"	231.20	80.8
			1.5"	339.80	71.8
			1 1/4"	394.50	67.3
			1"	489.80	59.4
			3/4"	742.50	38.4
			5/8"	824.30	31.6
			3/8"	923.40	23.4
			#4	1008.30	16.4
			#10	1075.10	10.8
			#40	1177.60	2.3
			#100	1202.60	0.2
			#200	1204.30	0.1

Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	61.6	22.0	83.6	5.6	8.5	2.2	16.3			0.1

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.7512	1.7413	3.9765	6.9970	14.7918	19.5467	22.2901	25.6933	49.6195	52.3901	54.0499	55.6090

Fineness Modulus	C_u	C_c
7.25	14.75	4.89

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ORGANIC CONTENT

ASTM D 2974

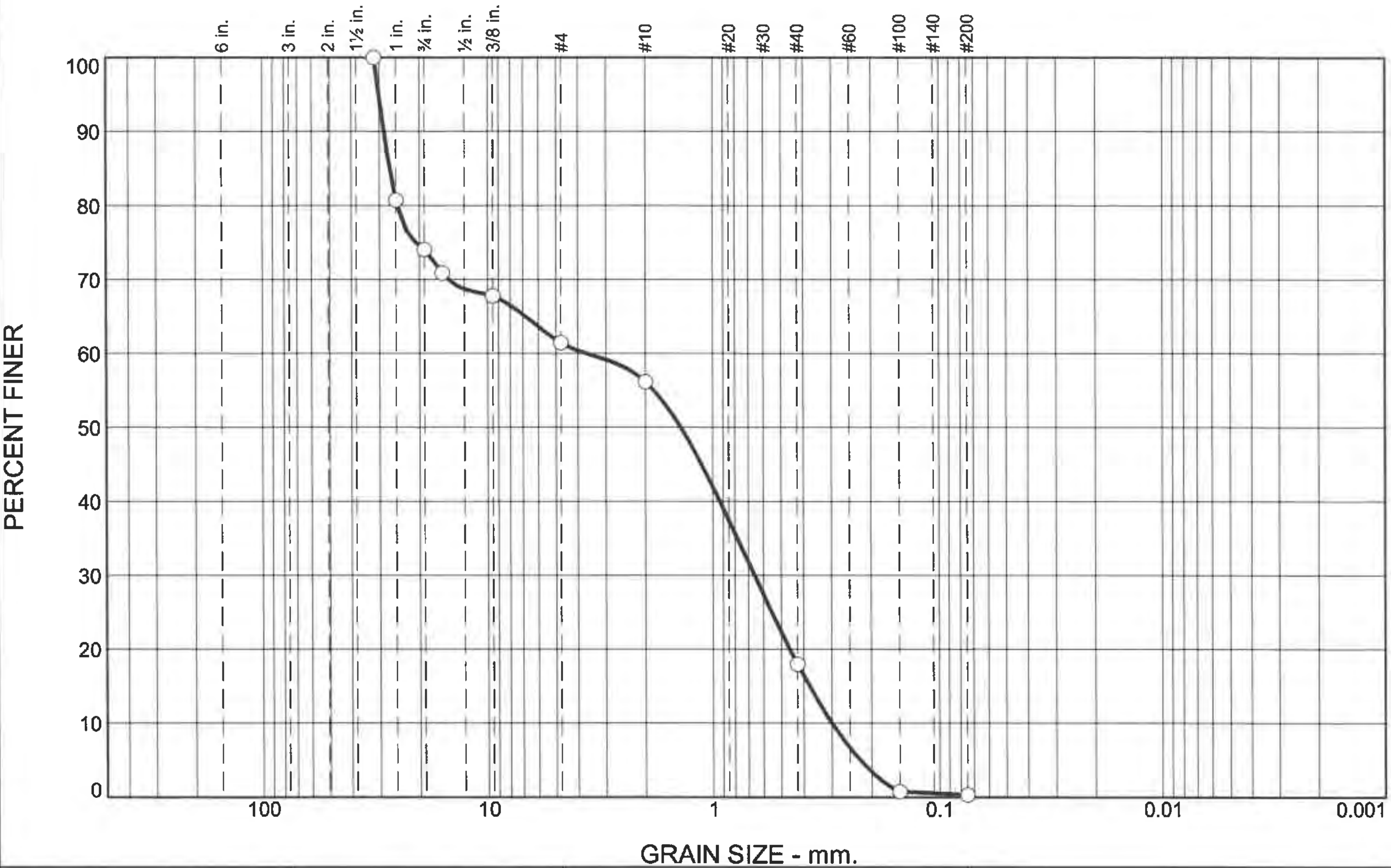
Project Name:	Jan Road
Project Number:	08-175
Date Tested:	1/30/2021
HMA Lab #	8379
Material Description:	ATP-03, G-1
Material Location:	1 ft
Testing Spec:	

Total Wet Wt + Tare	281.1 grams
Total Oven Dried Wt + Tare	216.3 grams
Wt of Tare	12.6 grams
Moisture Loss	64.8 grams
Moisture Content	**31.8 %**

BURN No.	Tare + Dry Weight
1	227.2
2	227
3	226.4
4	
5	

Total Oven Dried Wt + Tare	235.20 grams
Total Ash/Soil Wt + Tare	226.40 grams
Wt of Tare	122.80 grams
Ash Content	8.8 grams
Percentage Ash Content	**7.8 %**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	26.0	12.6	5.2	38.2	17.7	0.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/4"	100.0		
1"	80.7		
3/4"	74.0		
5/8"	70.9		
3/8"	67.8		
#4	61.4		
#10	56.2		
#40	18.0		
#100	0.8		
#200	0.3		

* (no specification provided)

Material Description

Sand with gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 28.6751 D_{85}= 27.0493 D_{60}= 3.6654
D_{50}= 1.3961 D_{30}= 0.6573 D_{15}= 0.3766
D_{10}= 0.2996 C_u= 12.23 C_c= 0.39

Classification

USCS= SP AASHTO=

Remarks

MC=6.3%

Location: ATP-03
Sample Number: G-2 **Depth:** 2 **Date:** 1/23/2021

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175 / 190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

1/23/2021

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175 / 190175-JR
Location: ATP-03
Depth: 2 **Sample Number:** G-2
Material Description: Sand with gravel
Date: 1/23/2021
USCS Classification: SP
Testing Remarks: MC=6.3%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 563.10
Tare Wt. = 12.70
Minus #200 from wash = 0.4%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
565.40	12.70	0.00	1 1/4"	0.00	100.0
			1"	106.60	80.7
			3/4"	143.60	74.0
			5/8"	161.10	70.9
			3/8"	178.20	67.8
			#4	213.10	61.4
			#10	242.30	56.2
			#40	453.40	18.0
			#100	548.40	0.8
			#200	551.00	0.3

Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	26.0	12.6	38.6	5.2	38.2	17.7	61.1			0.3

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.2246	0.2996	0.3766	0.4596	0.6573	0.9342	1.3961	3.6654	25.0682	27.0493	28.6751	30.2094

Fineness Modulus	C_u	C_c
4.55	12.23	0.39

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ORGANIC CONTENT

ASTM D 2974

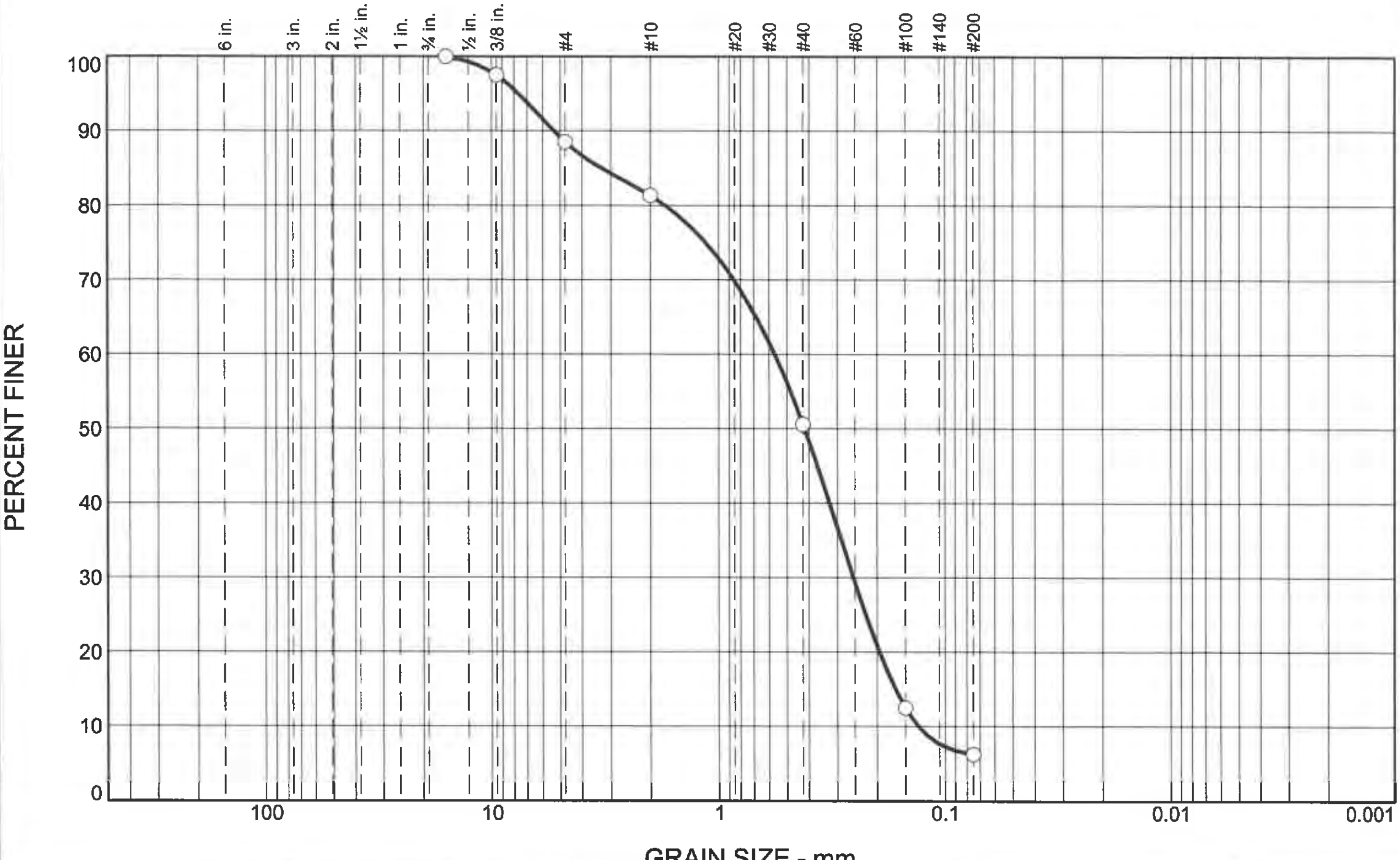
Project Name:	Jan Road
Project Number:	08-175
Date Tested:	1/30/2021
HMA Lab #	8379
Material Description:	ATP-04, G-1
Material Location:	1 ft
Testing Spec:	

Total Wet Wt + Tare	341.1	grams
Total Oven Dried Wt + Tare	250.9	grams
Wt of Tare	12.7	grams
Moisture Loss	90.2	grams
Moisture Content	**37.9**	**%**

BURN No.	Tare + Dry Weight
1	249
2	248.1
3	247.7
4	
5	

Total Oven Dried Wt + Tare	256.40	grams
Total Ash/Soil Wt + Tare	247.70	grams
Wt of Tare	122.80	grams
Ash Content	8.7	grams
Percentage Ash Content	**6.5**	**%**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	11.5	7.2	30.7	44.3	6.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
5/8"	100.0		
3/8"	97.5		
#4	88.5		
#10	81.3		
#40	50.6		
#100	12.5		
#200	6.3		

* (no specification provided)

Material Description

Sand with silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 5.3363 D_{85}= 3.2822 D_{60}= 0.5674
D_{50}= 0.4184 D_{30}= 0.2547 D_{15}= 0.1660
D_{10}= 0.1309 C_u= 4.33 C_c= 0.87

Classification

USCS= SP-SM AASHTO=

Remarks

MC=10.6%

Location: ATP-04
Sample Number: G-2 **Depth:** 3 **Date:** 1/23/2021

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175 / 190175-JR **Figure**

GRAIN SIZE DISTRIBUTION TEST DATA

1/23/2021

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175 / 190175-JR
Location: ATP-04
Depth: 3
Sample Number: G-2
Material Description: Sand with silt
Date: 1/23/2021
USCS Classification: SP-SM
Testing Remarks: MC=10.6%

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 384.30
Tare Wt. = 12.70
Minus #200 from wash = 5.6%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
406.20	12.70	0.00	5/8"	0.00	100.0
			3/8"	9.80	97.5
			#4	45.20	88.5
			#10	73.50	81.3
			#40	194.50	50.6
			#100	344.30	12.5
			#200	368.90	6.3

Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	11.5	11.5	7.2	30.7	44.3	82.2			6.3

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
	0.1309	0.1660	0.1953	0.2547	0.3248	0.4184	0.5674	1.7258	3.2822	5.3363	7.6376

Fineness Modulus	C_u	C_c
2.45	4.33	0.87

hayremcelroy & associates, llc

9644 153rd Ave NE
Redmond, WA 98052
425-869-6750

Minus No. 200 Wash

ASTM C117

Project Number: 08-175/190175-JR
Project Name: Aspect-Jan Road
Lab Number: 8379

Technician: SL
Received: 1/18/2021
Start Date: 1/19/2021
Finish Date: 1/23/2021

HMA LAB NO	Boring No	Sample Number	Depth (ft)	Tare Weight (g)	Tare+Dry Weight Before Wash (g)	Tare+Dry Weight After Wash (g)	% Retained	% PASSING
8379	ATP-05	G1	2	12.5	359.4	278.5	76.7	23.3

hayremcelroy & associates, llc

ORGANIC CONTENT

ASTM D 2974

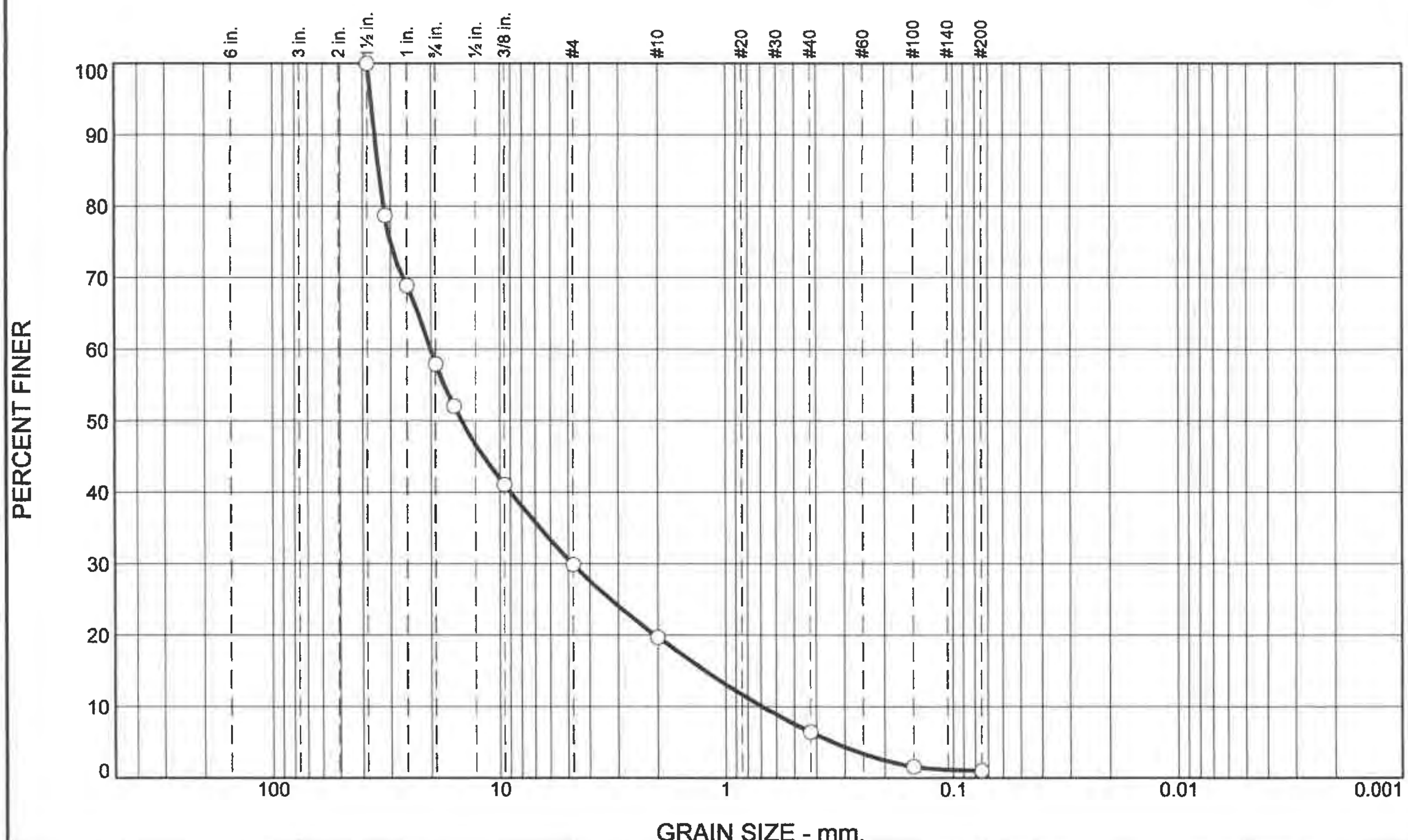
Project Name:	Jan Road
Project Number:	08-175
Date Tested:	1/30/2021
HMA Lab #	8379
Material Description:	ATP-05, G-1
Material Location:	2 ft
Testing Spec:	

Total Wet Wt + Tare	271.7	grams
Total Oven Dried Wt + Tare	224.7	grams
Wt of Tare	16.1	grams
Moisture Loss	47.0	grams
Moisture Content	**22.5**	**%**

BURN No.	Tare + Dry Weight
1	275.1
2	275
3	274.9
4	
5	

Total Oven Dried Wt + Tare	279.90	grams
Total Ash/Soil Wt + Tare	274.90	grams
Wt of Tare	122.90	grams
Ash Content	5.0	grams
Percentage Ash Content	**3.2**	**%**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	42.1	28.0	10.2	13.3	5.4	1.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1.5"	100.0		
1.25"	78.7		
1"	68.9		
3/4"	57.9		
5/8"	52.1		
3/8"	41.1		
#4	29.9		
#10	19.7		
#40	6.4		
#100	1.6		
#200	1.0		

* (no specification provided)

Material Description

Well graded GRAVEL with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 35.3009 D_{85}= 33.8492 D_{60}= 20.0982
D_{50}= 14.6975 D_{30}= 4.7984 D_{15}= 1.2335
D_{10}= 0.6922 C_u= 29.03 C_c= 1.65

Classification

USCS= GW AASHTO=

Remarks

Location: ATP-07
Sample Number: G-1 / 8379 **Depth:** 2.0 **Date:** 2-1-2021

Hayre McElroy & Associates, LLC

Redmond, WA

Client: Aspect Consulting
Project: Jan Road
Project No: 08-175 / 190175-JR **Figure**

Tested By: AD **Checked By:** JAM

GRAIN SIZE DISTRIBUTION TEST DATA

5/27/2021

Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175 / 190175-JR
Location: ATP-07
Depth: 2.0
Sample Number: G-1 / 8379
Material Description: Well graded GRAVEL with sand
Date: 2-1-2021
USCS Classification: GW
Tested by: AD
Checked by: JAM

Sieve Test Data

Post #200 Wash Test Weights (grams): **Dry Sample and Tare =** 2984.80
Tare Wt. = 169.40
Minus #200 from wash = 1.3%

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
3022.30	169.40	0.00	1.5"	0.00	100.0
			1.25"	607.40	78.7
			1"	886.60	68.9
			3/4"	1200.30	57.9
			5/8"	1367.80	52.1
			3/8"	1681.20	41.1
			#4	2001.10	29.9
			#10	2290.30	19.7
			#40	2669.40	6.4
			#100	2806.80	1.6
			#200	2823.40	1.0

Fractional Components

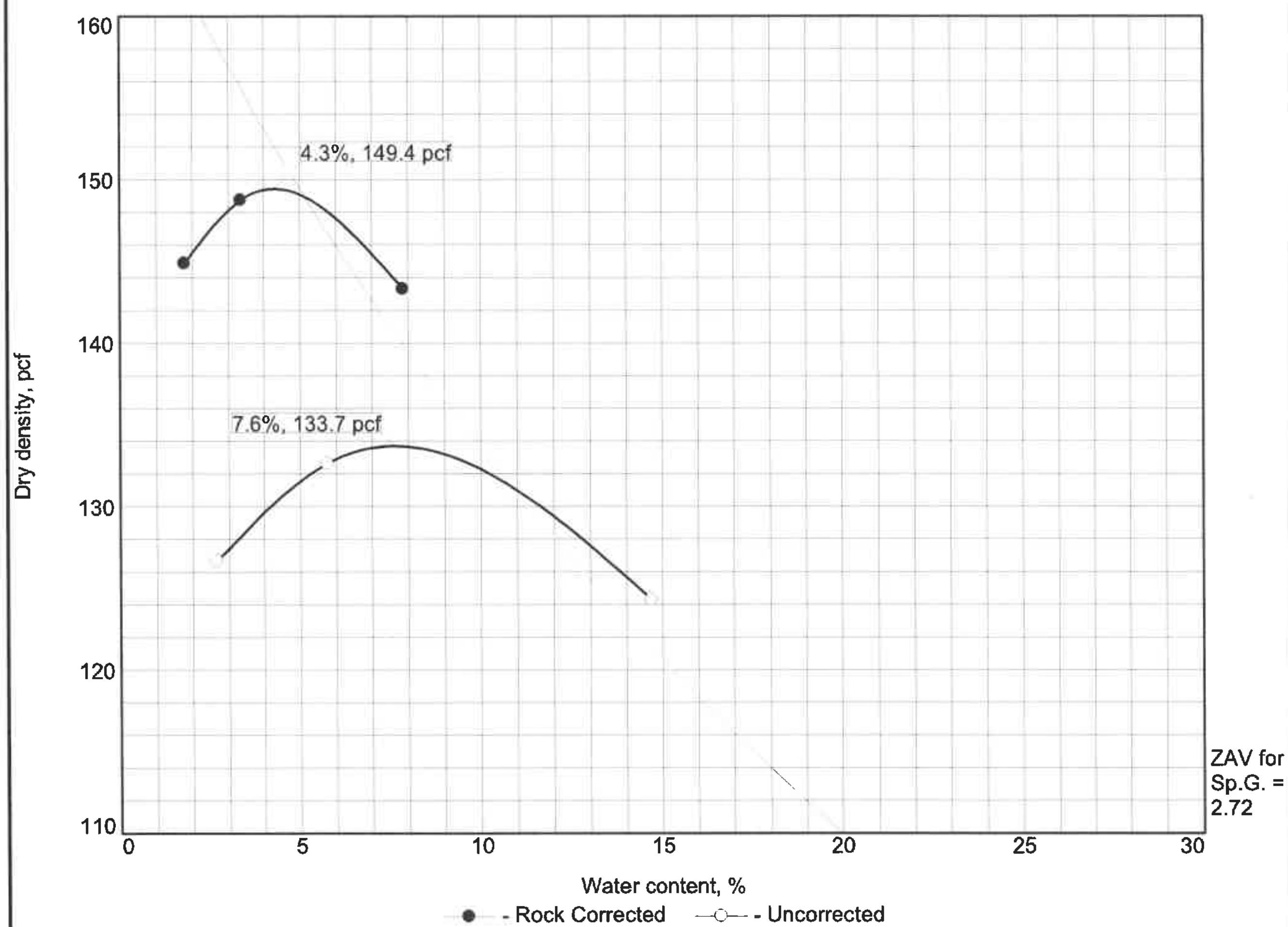
Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	42.1	28.0	70.1	10.2	13.3	5.4	28.9			1.0

D_{5}	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.3382	0.6922	1.2335	2.0555	4.7984	8.9652	14.6975	20.0982	32.2284	33.8492	35.3009	36.6979

Fineness Modulus	C_u	C_c
6.20	29.03	1.65

Hayre McElroy & Associates, LLC

COMPACTION TEST REPORT



Test specification: ASTM D 1557-91 Procedure C Modified
ASTM D4718-15 Oversize Corr. Applied to Each Test Point

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > 3/4 in.	% < No.200
	USCS	AASHTO						
6.0	GP			2.72			49.5	1.1

ROCK CORRECTED TEST RESULTS	UNCORRECTED	MATERIAL DESCRIPTION
Maximum dry density = 149.4 pcf	133.7 pcf	Poorly graded GRAVEL with sand
Optimum moisture = 4.3 %	7.6 %	

Project No. 08-175 / **Client:** Aspect Consulting
Project: Jan Road

Location: ATP-04 **Sample Number:** G-3 / 8379

Remarks:
Data represents the sample after having removed the material greater than 3" and is not representative of the native material.

Hayre McElroy & Associates, LLC

Redmond, WA

Figure

Tested By: AD **Checked By:** JAM

MOISTURE DENSITY TEST DATA

2/1/2021

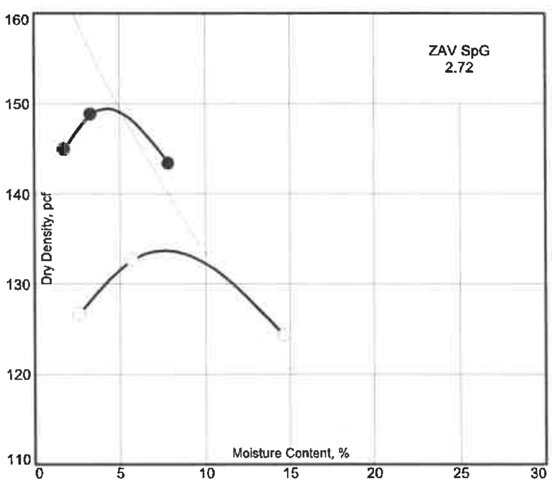
Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175 / 190175-JR
Location: ATP-04
Depth: 6.0 **Sample Number:** G-3 / 8379
Description: Poorly graded GRAVEL with sand
USCS Classification: GP
Testing Remarks: Data represents the sample after having removed the material greater than 3" and is not representative of the native material.
Tested by: AD **Checked by:** JAM

Test Data and Results

Test Specification:

Type of Test: ASTM D 1557-91 Procedure C Modified

Mold Dia: 6.00 **Hammer Wt.:** 10 lb. **Drop:** 18 in. **Layers:** five **Blows per Layer:** 56



Point No.	1	2	3
Wt. M+S	9870.2	10218.8	10296.0
Wt. M	5447.0	5447.0	5447.0
Wt. W+T	414.1	548.9	600.4
Wt. D+T	403.7	519.7	525.1
Tare	12.6	12.6	12.7
Moist.	2.7	5.8	14.7
Moist.*	1.8	3.3	7.9
Dry Den.*	144.9	148.7	143.3

Rock Corrected Results: **Max. Dry Den.=** 149.4 pcf **Opt. Moist.=** 4.3%
Uncorrected Results: **Max. Dry Den.=** 133.7 pcf **Opt. Moist.=** 7.6%

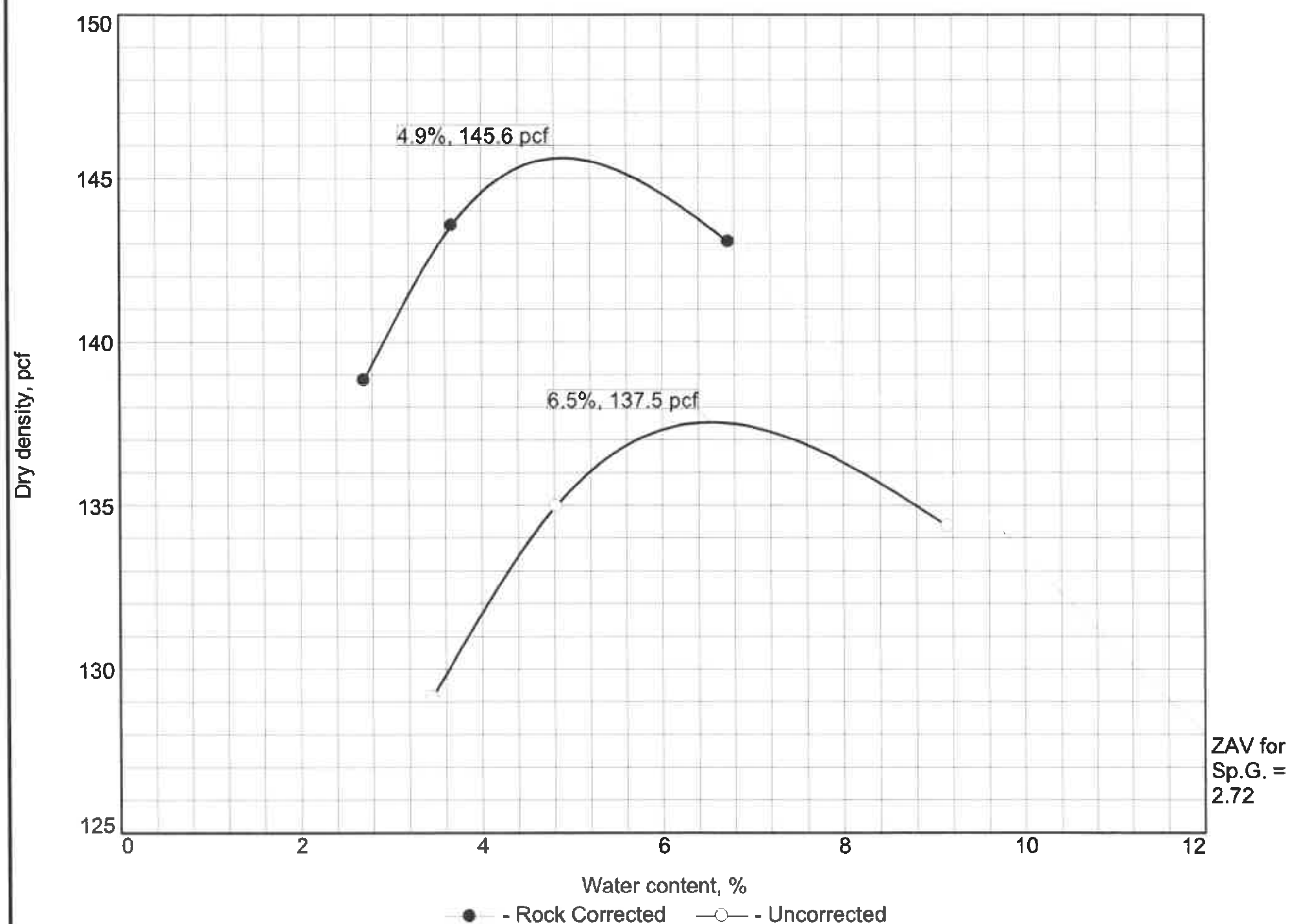
Rock Correction Data **Correction Method:** ASTM D4718-15

Percentage of Oversize Material (%> 3/4 in.): 49.5 **Bulk Specific Gravity of Oversize Material:** 2.72

Oversize Material Moisture Content: .89

***Note: the rock correction was applied to every test point's density and moisture value.**

COMPACTION TEST REPORT



Test specification: ASTM D 1557-91 Procedure C Modified
ASTM D4718-15 Oversize Corr. Applied to Each Test Point

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > 3/4 in.	% < No.200
	USCS	AASHTO						
6.0	GW-GM			2.72			29.1	9.8

ROCK CORRECTED TEST RESULTS	UNCORRECTED
Maximum dry density = 145.6 pcf	137.5 pcf
Optimum moisture = 4.9 %	6.5 %

MATERIAL DESCRIPTION

Poorly graded GRAVEL with silt and sand

Project No. 08-175 / **Client:** Aspect Consulting
Project: Jan Road

Location: ATP-05 **Sample Number:** G-2 / 8379

Hayre McElroy & Associates, LLC

Redmond, WA

Remarks:

Figure

Tested By: AD **Checked By:** JAM

MOISTURE DENSITY TEST DATA

2/1/2021

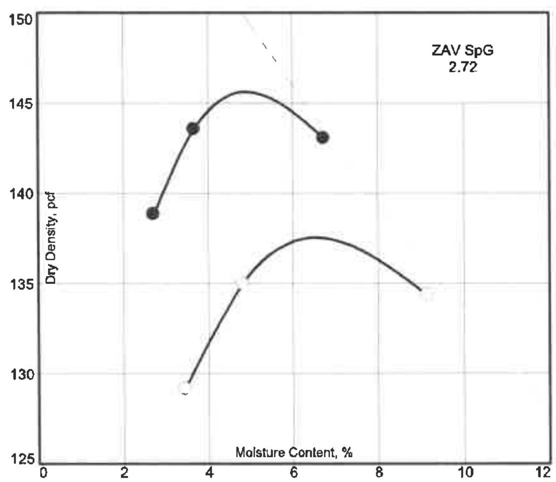
Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175 / 190175-JR
Location: ATP-05
Depth: 6.0 **Sample Number:** G-2 / 8379
Description: Poorly graded GRAVEL with silt and sand
USCS Classification: GW-GM
Tested by: AD **Checked by:** JAM

Test Data and Results For Curve 5447

Test Specification:

Type of Test: ASTM D 1557-91 Procedure C Modified

Mold Dia: 6.00 **Hammer Wt.:** 10 lb. **Drop:** 18 in. **Layers:** five **Blows per Layer:** 56



Point No.	1	2	3
Wt. M+S	9992.7	10260.6	10435.8
Wt. M	5447.0	5447.0	5447.0
Wt. W+T	431.5	532.1	489.8
Wt. D+T	417.5	508.2	450.1
Tare	12.8	12.5	15.9
Moist.	3.5	4.8	9.1
Moist.*	2.7	3.7	6.7
Dry Den.*	138.8	143.6	143.0

Rock Corrected Results: **Max. Dry Den.=** 145.6 pcf **Opt. Moist.=** 4.9%
Uncorrected Results: **Max. Dry Den.=** 137.5 pcf **Opt. Moist.=** 6.5%

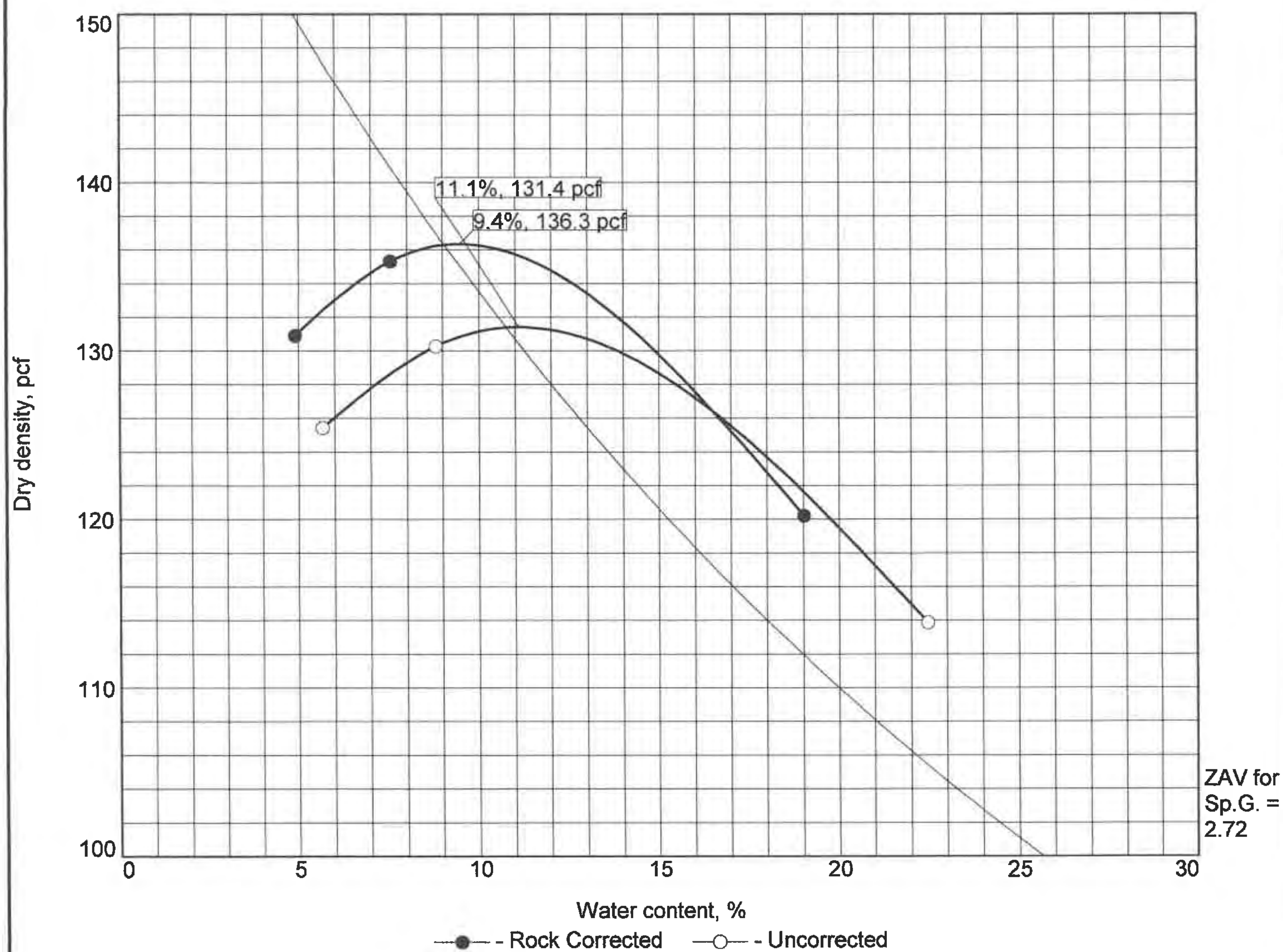
Rock Correction Data **Correction Method:** ASTM D4718-15

Percentage of Oversize Material (%> 3/4 in.): 29.1 **Bulk Specific Gravity of Oversize Material:** 2.72

Oversize Material Moisture Content: .89

***Note: the rock correction was applied to every test point's density and moisture value.**

COMPACTION TEST REPORT



Test specification: ASTM D 1557-91 Procedure C Modified
ASTM D4718-15 Oversize Corr. Applied to Each Test Point

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > 3/4 in.	% < No.200
	USCS	AASHTO						
2.0	GW			2.72			16	1.0

ROCK CORRECTED TEST RESULTS	UNCORRECTED	MATERIAL DESCRIPTION
Maximum dry density = 136.3 pcf	131.4 pcf	Well graded GRAVEL with sand
Optimum moisture = 9.4 %	11.1 %	

Project No. 08-175 / **Client:** Aspect Consulting
Project: Jan Road

○ **Location:** ATP-07 **Sample Number:** G-1 / 8379

Hayre McElroy & Associates, LLC

Redmond, WA

Remarks:

Figure

Tested By: AD **Checked By:** JAM

MOISTURE DENSITY TEST DATA

5/27/2021

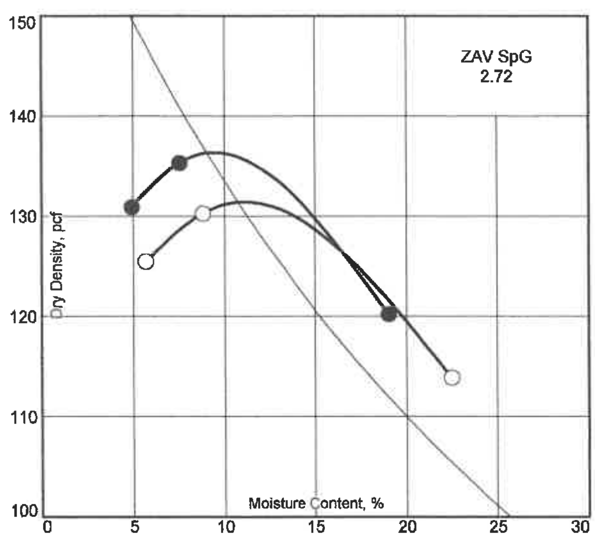
Client: Aspect Consulting
Project: Jan Road
Project Number: 08-175 / 190175-JR
Location: ATP-07
Depth: 2.0
Sample Number: G-1 / 8379
Description: Well graded GRAVEL with sand
USCS Classification: GW
Tested by: AD
Checked by: JAM

Test Data and Results

Test Specification:

Type of Test: ASTM D 1557-91 Procedure C Modified

Mold Dia: 6.00 **Hammer Wt.:** 10 lb. **Drop:** 18 in. **Layers:** five **Blows per Layer:** 56



Point No.	1	2	3
Wt. M+S	9955.7	10268.9	10191.2
Wt. M	5447.0	5447.0	5447.0
Wt. W+T	303.7	413.1	487.3
Wt. D+T	288.1	380.7	400.8
Tare	12.7	12.9	15.8
Moist.	5.7	8.8	22.5
Moist.*	4.9	7.5	19.0
Dry Den.*	130.9	135.3	120.2

Rock Corrected Results: **Max. Dry Den.=** 136.3 pcf **Opt. Moist.=** 9.4%

Uncorrected Results: **Max. Dry Den.=** 131.4 pcf **Opt. Moist.=** 11.1%

Rock Correction Data **Correction Method:** ASTM D4718-15

Percentage of Oversize Material (%> 3/4 in.): 16 **Bulk Specific Gravity of Oversize Material:** 2.72

Oversize Material Moisture Content: .89

***Note: the rock correction was applied to every test point's density and moisture value.**

APPENDIX C

Historical Data

C. Historical Data

For the Project, we reviewed historical data related to the Site and Site features and the Project. The attachments to this appendix are outlined below, and further summarized in the following sections.

- Appendix C.1: Jan Road Levee Revetment Extension (levee construction drawings), (County, 1962)
- Appendix C.2: Jan Road No. 2, Cedar River (levee construction drawings), (County, 1965)
- Appendix C.3: Jan Road Levee inspection records and repair documentation
    - Jan Road Levee inspection records by King County, 2011 – 2017
    - Jan Road Levee repair documentation by King County, 2006
    - Jan Road Levee inspection records by USACE, 1990-2011
- Appendix C.4: W1 (Henderson) Well Log (Ecology, 2021b)
- Appendix C.5: CRT7 Risk Evaluation (Aspect, 2020b)
    - Appendix C.5.A: Preliminary Geotechnical Data Report (King County, 2020)
        - Appendix C.5.A.B: Preliminary Hand Auger Investigation (Aspect, 2020a)
    - Appendix C.5.B: King County [CRT7] Revetment Inspection Records, 2011 – 2020
    - Appendix C.5.C: CRT7 Site Reconnaissance Photos
    - Appendix C.5.D: Slope Stability Analysis Outputs, Section B-B'
    - Appendix C.5.E: CRT7 Revetment Risk Assessment Workshop Meeting Minutes

C.1.1. Jan Road Levee Construction and Repair Information

We reviewed the levee construction drawings and records related to subsequent inspections by the USACE and County and repairs of the levee. A summary of these records is presented in Tables C-1. The available records are also included in this appendix.

Table C-1. Jan Road Levee Inspection and Repair History

Date	Inspecting Agency	Description of Record	Comments
1962 to 1965	King County	Levee Construction	Levee was constructed in two phases starting with the downstream portion of the levee.
January 29, 1976	USACE	Facility Inspection Report	490 linear feet of levee repair reported in USACE facility inspection report on May 30, 1990. Repair was part of PL 99 program and appears to be in response to a 1975 flood event (approximately 4 percent annual exceedance frequency, at time of inspection).
May 30, 1990	USACE	Facility Inspection Report	Riprap revetment reported to be in good shape. Overgrown vegetation on landward side reported, and insufficient level of protection reported (less than 20 percent annual exceedance frequency flood event, at time of inspection).
1998	King County	Repair	Repair to address erosion and scour from 1995 flood event (approximately 3 percent annual exceedance frequency [King County, 2016a]). No repair documentation available.
October 31, 2005	USACE	Facility Inspection Report	Levee rated "Minimally Acceptable." Overgrown vegetation on landward side reported. The levee was estimated to provide a level of protection of at least the 10 percent annual exceedance frequency flood event with sufficient freeboard.
2006	King County	Repair	Flooding events in 2006 (approximately 6 percent annual exceedance frequency [King County, 2016a]) resulted in scour along the landward side at approximately RM 13.3, resulting in property damages on King County Parcel No. 0422069027. Scour was repaired by placing quarry spalls along levee embankment (landward side).
February 15, 2011	King County	Post-flood inspection	No inspection notes available.
September 27, 2011	USACE	Facility Inspection Report	Levee rated "Unacceptable." Overgrown vegetation on the landward side and displaced riprap were reported. The levee was estimated to provide a level of protection for the 10 percent annual exceedance frequency flood event with sufficient freeboard.
October 6, 2011	King County	Facility Inspection Report	Vegetation (blackberries) noted on bank.
October 10, 2012	King County	Facility Inspection Report	Some scour noted.

Date	Inspecting Agency	Description of Record	Comments
October 9, 2014	King County	Facility Inspection Report	Minor displaced riprap revetment, but no slumping noted. Bank appeared stable.
September 22, 2017	King County	Facility Inspection Report	Displacement of riprap revetment observed. Vegetation (blackberries) noted on bank.
September 19, 2019	King County	Facility Inspection Report	Erosion and scour at upstream end. Overall facility over-steepened.

C.1.2. W1 Well Log

We also reviewed the well log for a (now inactive) deep monitoring well named Jan Road Levee Well 1 (herein referred to as W1), located in the floodplain along the right bank near RM 13.3. A copy of the well log is included in this appendix.

C.1.3. CRT 7 Risk Assessment

We completed a risk assessment the existing CRT 7 revetment in September 2020. The purpose of the assessment was to help understand the existing stability and vulnerability of the revetment as well as potential stability benefits and impacts resulting from implementation of the Project, based on the design alternatives. A copy of the risk assessment is included in this appendix.

C.1.4. King County Roads Services Division Preliminary Geotechnical Data Report

We reviewed existing subsurface exploration data at the Site collected by the King County Roads Services Division (Roads) in May 2019 during the preliminary geotechnical investigation stage of this Project. The exploration locations and logs of the Roads subsurface explorations are provided in the Preliminary Geotechnical Data Report (PGDR) in this appendix.

Roads completed seven test pit excavations (designated TP-1 through TP-7) between May 10 and 13, 2019, and six soil borings (designated B-1 through B-6) between May 20 and May 31, 2019. The explorations are summarized in Table C-2 below.

Table C-2. Previous Explorations Completed by King County Roads Services Division

Exploration ID	General Location	Exploration Equipment	Depth to Water (ft)	Final Depth (ft)	Comments
B-1	Left Bank – CRT 7 revetment, just south of revetment	Hollow-Stem Auger and Mud Rotary Drill Rig	22.0	51.5	B-1 log is a composite of 3 boring attempts. See PGDR for details
B-2	Left Bank – CRT 7 revetment, just north of revetment	Sonic Drill Rig	21.0	70.0	Boring terminated due to heave
B-3	Left Bank – CRT 7 revetment, north of revetment	Sonic Drill Rig	21.0	60.3	Boring terminated due to heave
B-4	Right Bank – Floodplain northeast of levee	Hollow-Stem Auger Drill Rig	7.5	21.5	-
B-5	Right Bank – Floodplain north of levee	Hollow-Stem Auger Drill Rig	7.5	21.5	Monitoring well installed, well screen between 5 and 10 feet below ground surface
B-6	Right Bank – Floodplain north of levee	Hollow-Stem Auger Drill Rig	7.5	21.5	-
TP-1	Right Bank – Jan Rd Levee embankment and levee toe	Backhoe Excavator	n/a	8.0	-
TP-2	Right Bank – Jan Rd Levee embankment and levee toe	Backhoe Excavator	4.0 ft below levee toe	8.0	-
TP-3	Right Bank – Jan Rd Levee embankment and levee toe	Backhoe Excavator	6.5 ft below levee toe	7.0	-
TP-4	Right Bank – Jan Rd Levee embankment and levee toe	Backhoe Excavator	n/a	6.0	-
TP-5	Right Bank – Floodplain north of levee	Backhoe Excavator	7.0	7.5	-
TP-6	Right Bank – Floodplain north of levee	Backhoe Excavator	4.8	5.0	-
TP-7	Right Bank – Floodplain north of levee	Backhoe Excavator	4.8	5.5	-

Roads performed laboratory testing (grain-size analyses) to characterize geotechnical properties on selected soil samples obtained from the explorations, with input from Aspect and Tetra Tech. Methodology and results of the laboratory tests are included in the PGDR in this appendix.

C.1.5. Aspect Hand Augers

During the preliminary phase of the Project in March 2020, we completed two shallow hand auger explorations at the upstream end of the Site near the confluence of Taylor Creek and the Cedar River. The hand auger logs and locations are detailed in the technical memorandum included in this appendix.

C.1.6. CRT 7 Revetment Repair Information

We reviewed records related to a repair along the revetment in 1998 for erosion and scour that occurred during a flood event. Recent inspection reports from the County also indicate that the revetment was damaged from a flood event in January 2009 and has since experienced oversteepening and voids in the riprap revetment due to ongoing scour/erosion. Copies of the relevant documents and records are included in this appendix.

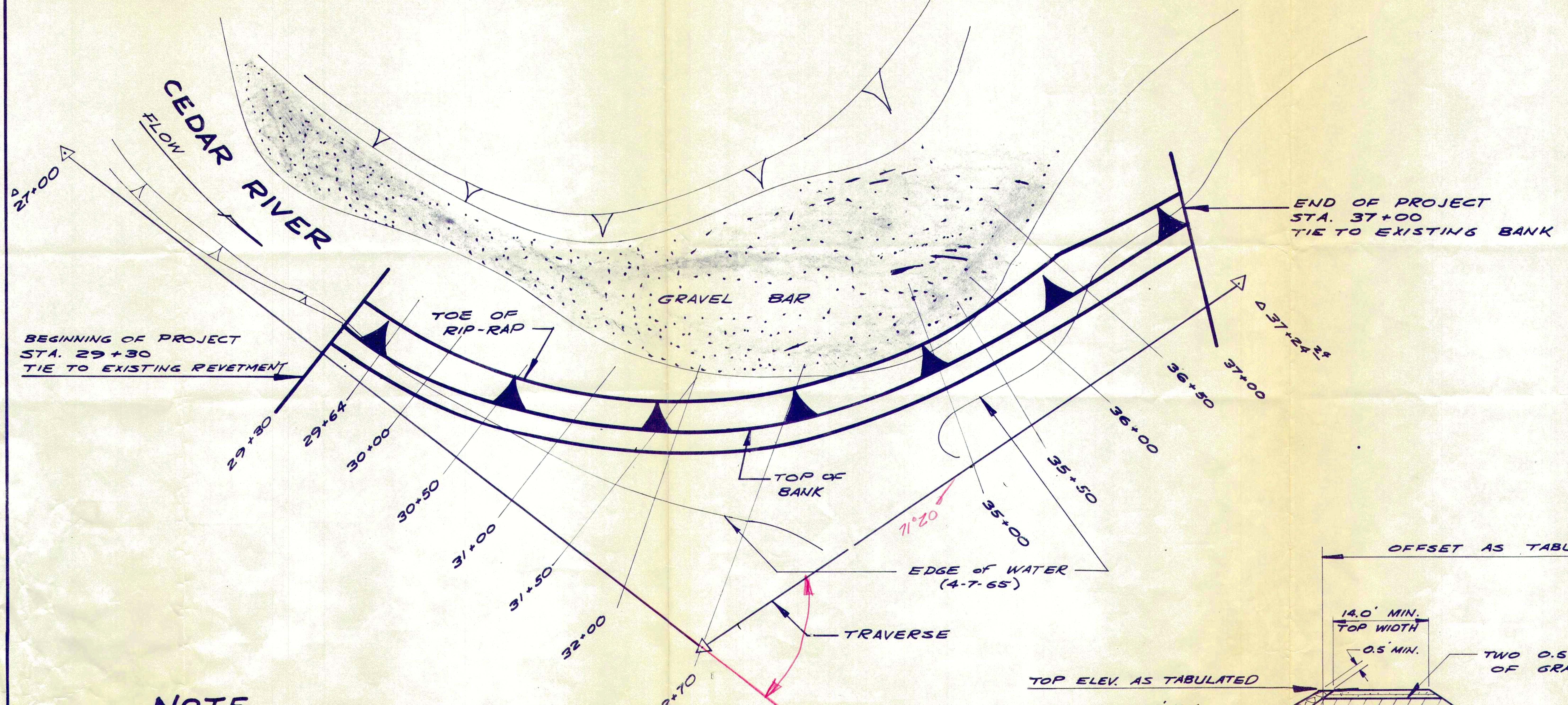
APPENDIX C.1

Jan Road Levee Revetment Extension (King County, 1962)

Reg 40 Mile Post 12.9R

APPENDIX C.2

Jan Road No. 2, Cedar River (King County, 1965)

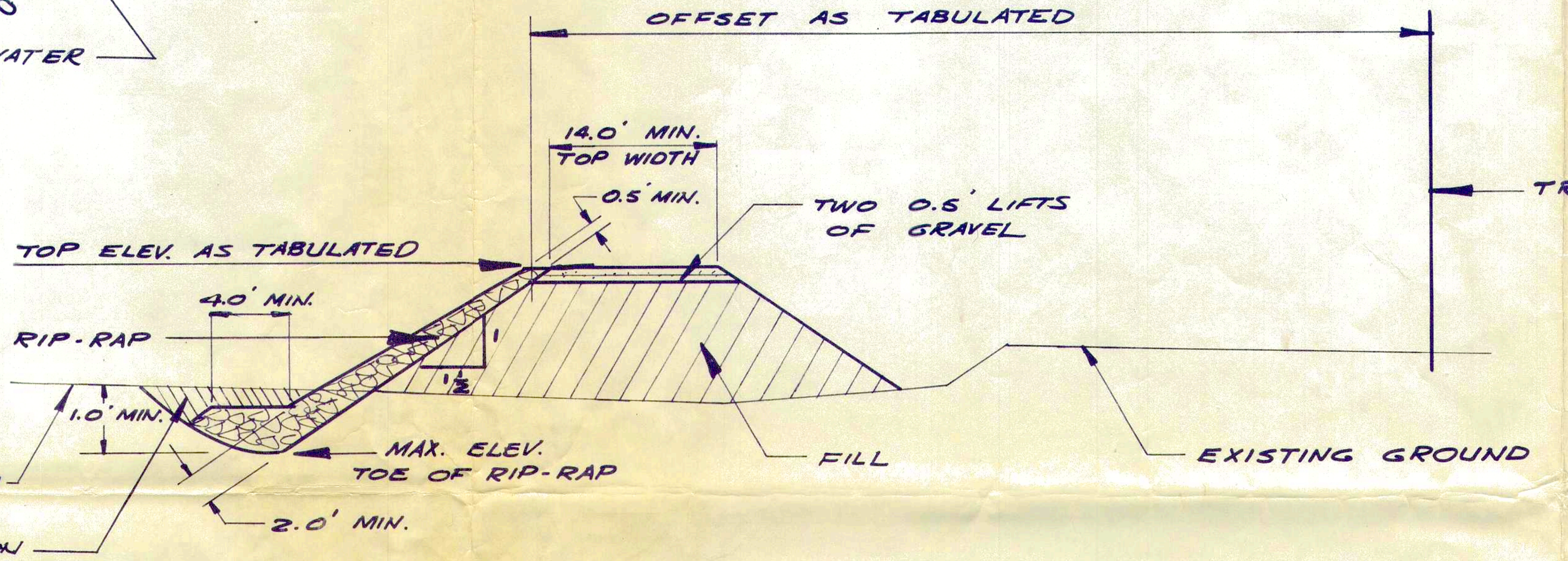


PLAN

SCALE 1" = 50'

NOTE

TOP OF EXISTING LEVEE BETWEEN STA. 26+00 & 29+30 TO BE RAISED TO ELEV. 263.0



TYPICAL SECTION

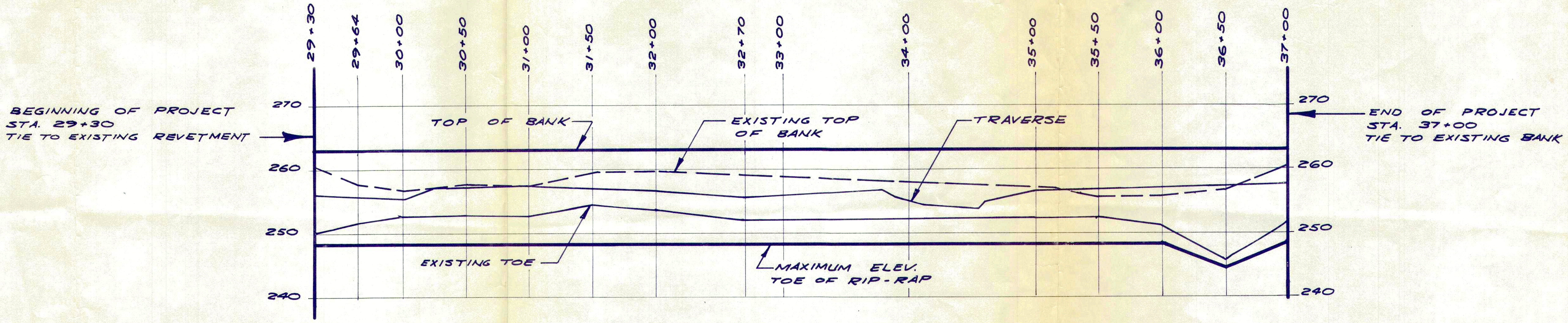
NO SCALE

CONSTRUCTION DATA

STA.	OFFSET	TOP ELEV.	MAX. ELEV. TOE OF RIP-RAP	ANGLE OFFSET TAKEN FROM TRAVERSE
29+30	TIE TO EXISTING REVETMENT	263.0	248.3	—
29+64	24.0 L			92° TO FWD. TAN.
30+00	31.0			90°
30+50	47.0			90°
31+00	69.0			90°
31+50	101.0			90°
32+00	115.0			70°40' TO BK. TAN.
32+70	167.0			36° TO FWD. TAN.
35+00	71.0			76° TO FWD. TAN.
35+50	65.0			87°24' TO FWD. TAN.
36+00	64.0		248.3	104° TO FWD. TAN.
36+50	61.0 L		244.5	114°44' TO FWD. TAN
37+00	TIE TO EXISTING BANK	263.0	248.3	56° TO BK TAN.

NOTES

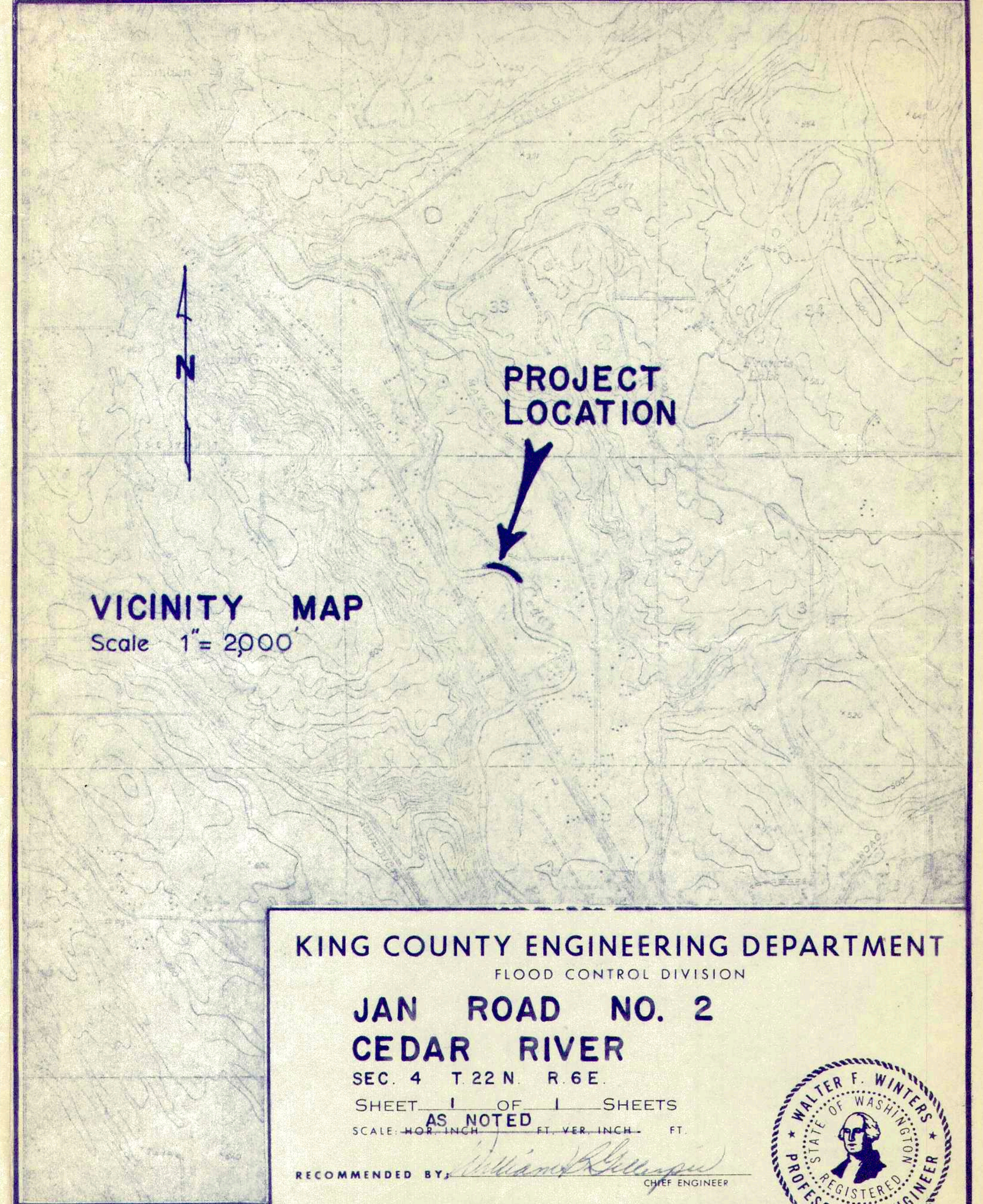
1. ALL ELEVATIONS REFER TO MEAN SEA LEVEL DATUM
2. T.B.M. "1 ELEV. 262.66 SPIKE IN 3 FT. MAPLE 40 FT. RIGHT @ STA. 27+70



PROFILE

SCALE 1" = 10' VERT. 1" = 50' HOR.

STATE FLOOD CONTROL PERMIT NO. 3-116



KING COUNTY ENGINEERING DEPARTMENT
FLOOD CONTROL DIVISION

JAN ROAD NO. 2
CEDAR RIVER

SEC. 4 T.22 N. R.6 E.

SHEET 1 OF 1 SHEETS

SCALE: AS NOTED

RECOMMENDED BY: William A. Gillespie, CHIEF ENGINEER

APPROVED BY: ______ COUNTY ENGINEER

WALTER F. WINTERS — STATE OF WASHINGTON — REGISTERED PROFESSIONAL ENGINEER

JOB NO. 500-517-65				
DATE 8-27-65	DRAWN BY JESCHKE	DESIGNED BY JESCHKE	CHECKED BY GEW	A-42-1

APPENDIX C.3

Jan Road Levee Inspection Record and Repair Documentation

- **Jan Road Levee inspection records by King County, 2011 – 2017**
- **Jan Road Levee repair documentation by King County, 2006**
- **Jan Road Levee inspection records by USACE, 1990-2011**

Jan Road Levee

Inspection Records, King County (2011 - 2017)

Rivers Facility Inventory Inspection Tickets: 12/10/2018

If you have any questions about this data, please contact **Kyle Comanor** at **206-477-4076** or email **kyle.comanor@kingcounty.gov** .

Facility Name	River Name	Inspection Date	Inspection Type	Is Partial Inspection	Inspection RM	Inspector 1	Flow Condition	Inspection Mode	Is QA/QC	Modified By	Inspection Notes
Jan Road	Cedar River	9/22/2017	Routine Inspection	No		Heckendorf, Dan	Low - toe visible	Same Bank	No	Heckendorf, Dan	Approximately RM 13.3: Portions of toe rock out in channel; small/large boulder gravel deposition on bar. Toe rock ~2-3' round, bank rock ~1-2' round Consider vegetation (blackberry) management towards upstream end of facility. ~40' length of willows extending into flow; providing good regeneration
Jan Road	Cedar River	10/9/2014	Routine Inspection	No		Garric, Craig	Low - toe visible	Same Bank	No	Walker, Eric	U/S end no visible problems or changes. Good mature willow. Front of house- weeds Tansy? No willow, bank stable. D/S from house some large rock riverward of toe, but bank appears stable. No obvious damage. Small area of potential displaced rocks but no wholsale slumping. Overall intact. No other signs of damage or failure on toe or bank.
Jan Road	Cedar River	10/10/2012	Routine Inspection	No		Faegenburg, Nancy	Low - toe visible	Same Bank	No	Faegenburg, Nancy	Tansy at U/S end, willow established at U/S end. @ house, top layer has some scour, but toe intact. Patches willow + alder Photos: looking d/s from u/s end/toe # 18-19, view D/S from house 20, view of revetment on levee face by home 21, spawning soc
Jan Road	Cedar River	10/6/2011	Routine Inspection	No		Barton, Chase	Low - toe visible	Same Bank	No	Walker, Eric	looks good- a lot of blackberries on bank.
Jan Road	Cedar River	2/16/2011	Post-flood Inspection	No		Butchart, Carolyn	OHW	Same Bank	No	Walker, Eric	

Jan Road Levee

Repair Documentation, King County (2006)

Pre-Construction Photos

Jan Road Levee

Repair Documentation, King County (2006)

Post-Construction Photos

Jan Road Levee

Inspection Records, USACE (1990 - 2011)

CEDAR RIVER
LEVEE INSPECTION

JAN CURVE – RIGHT BANK
RIVER MILE 12.5 TO RIVER MILE 12.3

King County
Cedar River, King County, State of Washington

Inspector (s) Monte Kaiser
Evangelos Angelou, P.E.
Robert Newbill, P.E.

Date 30 May 1990

Weather 46, cloudy

This report has been reviewed and approved by the following Seattle District personnel:

Signature on file___________
Ch, Hydr. Sect., H&H Br.

Signature on file___________
Ch, Soils Sect., GeoTech. Br.

Signature on file___________
Ch, Em. Mgt. Br., Ops. Div.

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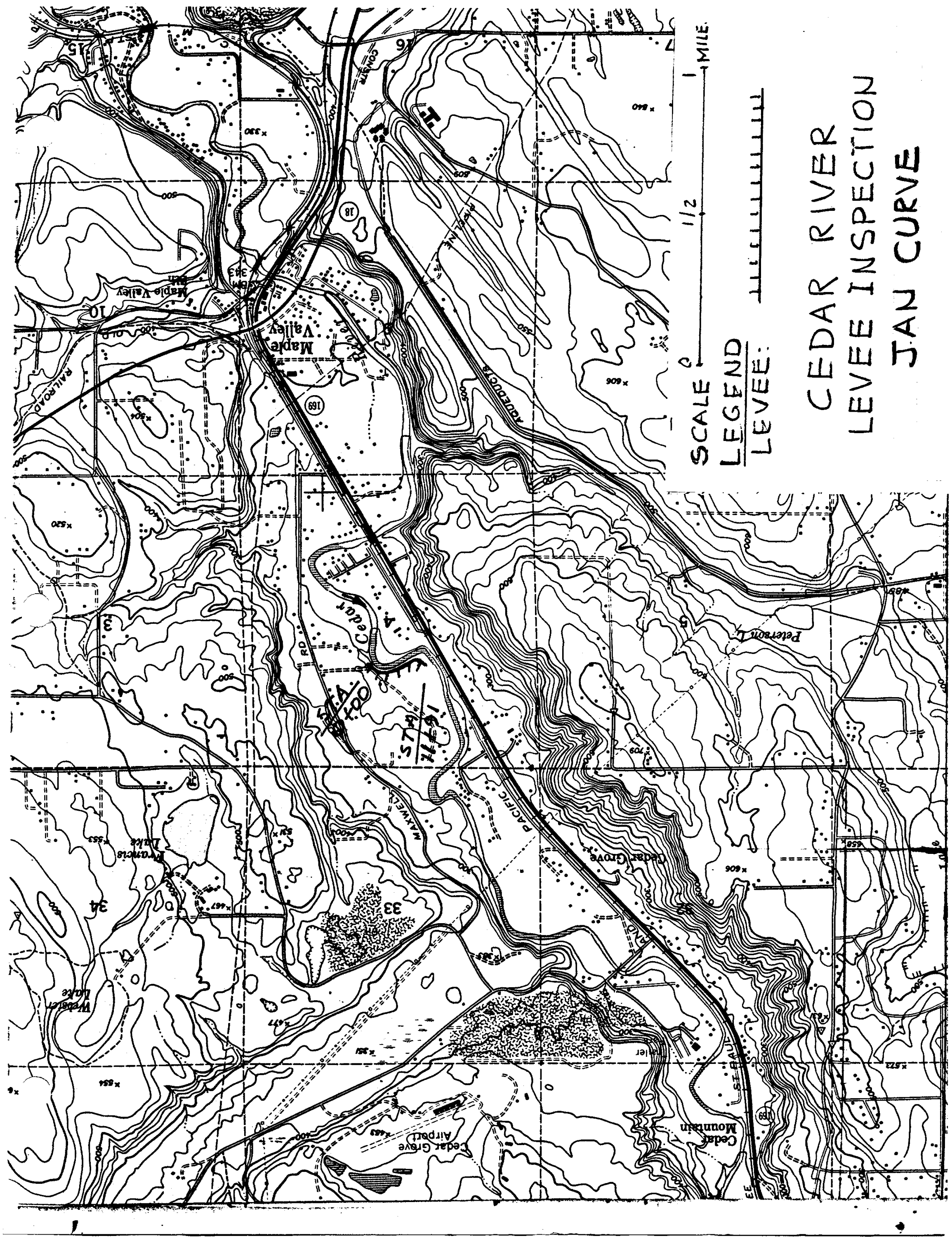
SCALE
0
1/2
1 MILE
LEGEND
LEVEE:
CEDAR RIVER
LEVEE INSPECTION
JAN CURVE

2. PROJECT INFORMATION

A) Basic Data

(1) King County of the State of Washington is the sponsor for this levee system.

Surface Water Management Division
Department of Public Works
701 Dexter-Horton Building
710 Second Avenue
Seattle, WA 98104

(2) The Project Identification is Jan Curve Levee.

(3) The levee is located on the right bank of the Cedar River from River mile 12.5 to River mile 12.3, King County, State of Washington.

(4) The Corps of Engineers has performed the following work on this levee: 490 linear feet of levee repair completed 29 January 1976 under authority PL 99. Job CED-3-75.

B) Classification:

This levee is the only means of flood control for the area behind Jan Curve Levee.

C) Project Data:

(1) Economic

(a) Total acreage protected: 79 acres.

(b) Cropping pattern: None.

(c) Land use: Residential.

(d) Value of property protected: $812,000.

(e) Facilities Protected (i.e. Homes, businesses, industry, etc.): 14 Single family homes, and 2 mobile homes.

(f) Historic flood damage – Year: 1975
Frequency of event: Approximately 25-Year
Amount: N/A

(2) Existing Condition

(a) General: Height: 6 to 8 feet.
Top width: 15 to 20 feet.
Riverward side slope: 1v: 2h.
Landward side slope: 1v: 1-1.5h.

(b) Estimated level of protection: The levee embankment provides less than 20 percent exceedence frequency flood protection with one foot of freeboard.

(c) Gage data if available: U.S.G.S. Gaging station at river mile 1.59 and at river mile 23.4.

(d) Type of materials: Compact, silty sandy gravel.

(e) Erosion protection: Riprap bank protection has been provided on most of the riverward levee slope. The riprap appears to have been placed properly and is in good condition.

(f) Interior drainage: None noted, except for the natural drainage through the downstream end of the levee.

3. CONCLUSIONS

This levee segment fails to meet the minimum eligibility criteria, for the following reasons: the levee embankment fails to meet the minimum acceptable performance standard for the level of protection, and the overgrowth of vegetation on the landward slope and portions of the riverward slope hinders detailed visual inspection.

4. RECOMMENDATIONS

In order for this levee system to meet the minimum acceptable level of protection the following improvements must be made:

A. The levee embankment must be raised to provide 20 percent exceedence frequency flood protection with a minimum of one foot of freeboard. The required levee profile is shown on Drawing E-19-1-67, sheet 3 of 3.

B. All trees greater than 2 inches in diameter and the overgrown brush must be removed from both side slopes of the levee.

In addition, the following recommendations should be followed to improve the effectiveness of the levee system:

A. An annual maintenance program should be established to keep the levee clear of overgrowth and the channel clear of debris. This annual program must also address and repair miscellaneous damages as they occur.

5. FIELD INSPECTIONS

Location/description of levee reach:

The levee extends from river mile 12.5 to river mile 12.3 along the right bank of the Cedar River. The upstream end of the levee is tied to high ground. The levee follows the channel course with little or no setback, and the downstream end is also tied to high ground. Due to the topography, flow will not get behind the levee from the downstream end during high water. The top of the levee has been graveled.

The levee height varies from 6 to 8 feet, and the top width varies from 15 to 20 feet.

The riverward side slope is about 1v: 2h, while landward slope varies from 1v: 1h to 1v: 1.5h.

Riprap has been placed on most of the riverward slope of the levee. The riprap has been evenly placed on a 1v: 2h slope and is in good condition.

The entire levee is drivable with access to the top from upstream end.

Overgrown brush covers the landward slope and portions of the riverward slope throughout this segment (See picture A).

6. EVALUATION

A. Technical Evaluation – Hydraulic Analysis

1) Stream Characteristics:
Throughout this reach, the Cedar River is a medium gradient stream capable of carrying large quantities of sediment under flood conditions. The channel bed is characterized by small cobbles and gravel, and there is gravel bar development.

2) Erosion Control:
Riprap bank protection of good quality has been provided on most of the riverward slope. The riprap has been evenly placed on a 1v: 2h slope.

3) Level of Protection:
Based on detailed hydraulic analysis for the Cedar River Flood Reduction study done in 1989, this levee provides a level of protection of less than a 20 percent exceedence frequency flood with at least one foot of freeboard (See Drawing E-19-1-67, sheet 3 of 3). Water surface profiles used in this PL-99 evaluation were developed from a numerical analysis (HEC-2 program) accomplished for that study. Top of levee data for the PL-99 evaluation is based on 1987 topography. This levee does not meet the minimum acceptable performance standard for level of protection for PL-99 eligibility.

B. Technical Evaluation – Geotechnical Analysis

1) Soil Characteristics:
The levee soils were visually classified according to the Unified Soil Classification Systems as GM, silty sandy gravel.

2) Slope Stability:
The levee embankment appears stable. No evidence of instability was observed.

3) Seepage:
Seepage through the levee embankment and foundation is not anticipated to cause instability.

C. Levee Embankment

1) Levee Depressions:
None noted.

2) Levee Surface Erosion:
None noted.

3) Animal Burrows:
None noted.

4) Unwanted Levee Growth:
Brush covers small portions of the riverward side slope, and most of the landward side slope (See picture A). The brush is not considered to be a threat to the integrity of the levee due to the over built width.

5) Encroachments:
No encroachments were noted along the top of the levee and in the channel.

D. Channel / Floodway Capacity

Capacity of the channel may be reduced slightly if large size debris accumulates in the channel.

E. Structures

None noted.

F. Culverts

None noted.

G. Pumping Plants

None noted.

H. Other

None noted.

7. FIELD PHOTOGRAPHS

Photograph

A – At upstream end looking downstream. Brush and small trees on the landward slope of the levee.

B – At station 11+00 looking downstream. Riprap bank protection.


A


B

LEVEE INSPECTION REPORT

31 October 2005

CEDAR RIVER BASIN

JAN'S CURVE LEVEE

Maple Valley, Washington

SEGMENT: Right Bank R.M. 12.5 – 12.3

PREVIOUS STATUS: ELIGIBLE
CURRENT RATING: MINIMALLY ACCEPTABLE
CURRENT RATING: MINIMALLY ACCEPTABLE

The Jan's Curve Levee was inspected on 31 October 2005 by Seattle District Corps of Engineers of employees Monte Kaiser, Jennifer West and Cathie DesJardin for the Emergency Management Branch. The inspection team was accompanied by Nancy Faegenburg and John Koon of King County. The purpose of the inspection was to evaluate the condition of the levee and determine if the levee continues to meet the eligibility requirements of the Corps' PL 84-99 Non-Federal Levee Rehabilitation Program. Previous years' continuing inspections and the initial levee inspection for this levee are stored in Seattle District's levee database.

The levee inspection team started at mid-levee where the property owner has the access gate locked, but there is access to the left. The team walked to the upstream end, began the inspection and continued approximately 1,000 feet downstream to end of the levee.

Observations:

The levee driving surface was recently mowed and brushed. There is a great deal of Japanese Knotweed that has been injected with herbicide; monitor to see if it was effective in killing the vegetation. The levee landward slope is generous just upstream and along the curve. The levee sideslopes still require brushing. Trees over 4" diameter are growing within the levee prism that require removal. All visible toe and face rock seem to be in the same condition as the last inspection. Overall the levee is in minimally acceptable condition. Homeowners along the levee typically do the vegetation maintenance.

All levees require annual vegetation maintenance. Levee vegetation maintenance is exempt from Corps of Engineers 404 Permitting requirements. The County must perform vegetation maintenance where homeowners are not. The County should continue to monitor the levee for any encroachments by property owners that may harm the integrity of the levee. Property owners should be aware that the levee surface must be clear of structures during flood season, November 1st through May 31st.

All trees over 4” diameter must be removed from the levee prism. Trees and their root systems compromise the structural integrity of levees and can cause levee failure. Trees also compromise the reliable flood protection afforded by the levee, such as during high winds keeping the soil loose, collecting debris during periods of high flow, and toppling. The Corps is available for technical advice to designate and mark sites and trees that require removal.

Recommendations:

In order to improve the effectiveness of this levee system and to meet the minimum eligibility standards for PL84-99, the following recommendations must be implemented:

A. Remove all trees over 4” diameter from the levee prism. The Corps is available to designate and mark trees.
B. Continue to brush and mow the levee top and sideslopes to allow better inspection of the levee. Monitor the injected Japanese Knotweed to note success of herbicide injection.
C. Remove furniture and obstructions from levee surface during flood season November 1 through May 31.

The levee surface is required to be accessible and drivable during high water events.

Field Photographs

Photo 1 – Jan's Curve Levee – At the curve, looking downstream. The levee face and toe require brushing and tree removal.

Photo 2 – Jan's Curve Levee – Looking upstream near mid-levee.

Photo 3 – Jan's Curve Levee –Looking downstream near the downstream end. The knotweed was recently injected with herbicide.

Levee Segment Name	Jan’s Curve
Location	Cedar River, Right Bank
Sponsor	King County
Contact	John Koon, 206-296-8062
Level of Protection	The levee is estimated to provide a level of protection at least equal to the 10 percent exceedance frequency flood event with sufficient freeboard for that event.
Approximate length	1,000 ft
Topwidth	12-18 ft
Front slope	1.5H:1V to 2H:1V
Back slope	2H:1V to 3H:1V
Vegetation	Trees and brush
Driving surface	Good sod base
Maintained	Minimally acceptable
Armored	Riprap toe
Benefits	Flood Control
Recommendations	See recommendations portion of report
Current Rating	Minimally Acceptable
Current Status	Eligible

Date of Inspection: 31 October 2005

Inspected by:

Monte Kaiser, Corps
Cathie DesJardin, Corps
Jennifer West, Corps
Nancy Faegenburg, King County
John Koon, King County

US Army Corps of Engineers®

Flood Damage Reduction System Inspection Report

Name of System: Jan's Curve Levee

Public Sponsor(s): King County

Public Sponsor Representative: John Koon

Sponsor Phone: (206)890-2562

Sponsor Email:

Corps of Engineers Inspectors: Travis Goss, Jason Villarreal   Date of Inspection: 27 Sep 2011

Inspection Report Prepared By: Jason Villarreal   Date Report Prepared: 28 Sep 2011

Internal Technical Review (for Periodic Inspections) By: Charles Ifft, P.E.   Date of ITR: 28 SEP 11

Final Approval By: Dennis Fischer, P.E.   Date Approved: 5 Oct 11

Type of Inspection:

- ☐ **Initial Eligibility Inspection**
- ☒ **Continuing Eligibility Inspection (Routine)**
- ☐ **Continuing Eligibility Inspection (Periodic)**

Overall System Rating:

- ☐ **Acceptable**
- ☐ **Minimally Acceptable**
- ☒ **Unacceptable**

Contents of this Report:

- ☐ **Instructions**
- ☐ **Initial Eligibility Inspection**
- ☒ **General Items for All Flood Control Works**
- ☒ **Levee Embankments**
- ☐ **Concrete Floodwalls**
- ☐ **Sheet Pile and Concrete I-walls**
- ☐ **Interior Drainage System**
- ☐ **Pump Stations**
- ☐ **FDR system Channels**

Note: This inspection rating represents the Corps evaluation of operations and maintenance of the flood damage reduction system and may be used in conjunction with other information for a levee certification determination for National Flood Insurance Program (NFIP) purposes if applicable. An Acceptable Corps inspection rating, alone, does not equate to a certifiable levee for the NFIP. It is recommended for levee systems currently accredited by the Federal Emergency Management Agency (FEMA) for NFIP purposes receiving a Corps Minimally Acceptable or Unacceptable rating be evaluated by the levee owner to determine the potential impacts to the certification for FEMA.

LEVEE INSPECTION REPORT

27 September 2011

JAN'S CURVE LEVEE

CEDAR RIVER BASIN

(Cedar River, Maple Valley, King County, Washington)

SEGMENT: Right Bank, River Mile 12.5 to River Mile 12.3

CURRENT RATING: **UNACCEPTABLE**
PREVIOUS RATING: **MINIMALLY ACCEPTABLE – 31 October 2005**

INSPECTION PERSONNEL:

- Travis Goss – USACE, Seattle District, Geotechnical Branch
- Jason Villarreal – USACE, Seattle District, Geotechnical Branch

BACKGROUND:

The Jan's Curve Levee was inspected on 27 September 2011 by Seattle District personnel. The following report describes the current conditions as seen on the inspection.

To reach the levee from the District Office, head south on SR-99 to I-405 N. Take the WA 169 S exit. Take a left at Cedar Grove Rd SE and a right onto SE Lake Francis Rd. Then, take a right at Maxwell Rd SE and follow this down the hill taking an eventual right onto SE 197th Pl. The downstream extent of the levee is approximately near the turn where SE 197th Pl turns into 221st Ave SE. The levee is approximately 1,190 feet long. This levee system sustained damage in 2006 and the County elected to end the rehabilitation partnership with the Corps following the repair discussions.

OBSERVATIONS:

The inspection began at the upstream end of the levee. The entire length of levee is in desperate need of vegetation maintenance. A majority of the levee length has very dense brush and overgrown grass covering both the landward and riverward slopes and the crown (see Figure 2). There are also a number of trees on both the riverward and landward slopes that exceed the maximum allowable size of 4" diameter at breast height (DBH). Near the middle section of the levee, a local homeowner has maintained the levee crown and it appears that it sees a reasonable amount of recreational use. Near a picnic table on the crown, the crown shoulder appears to be losing material (see Figure 3). The inspection team is assuming this is caused by foot traffic displacing the riprap blanket near the top of the slope and subsequent damage to the material underneath. The heavy brush covering the levee slopes made inspection of the rock blanket, and landward slope, impossible resulting in a "Not Rated" rating for the related categories.

CONCLUSIONS:

This levee continues to have issues regarding unwanted vegetation and it appears as if King County is uninterested in correcting these deficiencies. At this time, the Jan’s Curve Levee is given an UNACCEPTABLE rating and is considered INELIGIBLE for rehabilitation under Public Law 84-99.

RECOMMENDATIONS:

In order to improve the effectiveness of this levee system and become eligible for the PL 84-99 program, the inspection team recommends the following:

A. Establish/Continue regularly scheduled maintenance activities on the levee including slope maintenance and brush removal. In order to receive an Acceptable rating and become eligible, the trees and brush should be maintained according to the Seattle District Vegetation Variance from 1995. All slopes should be easily visible, and no trees should be within the levee prism in excess of 4” diameter breast-height (DBH). A Corps representative can assist in marking trees for removal if requested. In order to provide the best flood protection and the best possible flood control facility, it is suggested that all trees and brush be removed from the side slopes and 15 feet from the landward toe with grass planted on all surfaces.
B. Levee crown must be free of all vegetation and obstructions to allow at least 12 feet for vehicle access.
C. Monitor for burrowing animal activity.
D. Replace displaced rock on riverward slope and make efforts to inform the public that displacing this rock can have negative impacts on the levee integrity.

AERIAL MAP:

Figure 1 - Aerial map of Jan’s Curve Levee.

INSPECTION PHOTOGRAPHS:

Figure 2 – View of levee from downstream end looking upstream.

Figure 3 – Shoulder of crown deterioration appears to be from homeowners walking on and displacing slope rock.

Figure 4 – Trees larger than 4" DBH on both riverward and landward slopes.

SUMMARY TABLE:

Levee Segment Name	Jan’s Curve Levee
Location	Cedar River, Maple Valley, King County, Washington
Sponsor	King County
Contact Information	John Koon, 206-890-2562
Level of Protection	10 percent exceedance
Approximate Length	1,190 feet
Top Width	18 to 20 ft
Crown Surface	Sod
Riverward Slope	1.5H : 1V to 2H : 1V
Landward Slope	2H : 1V to 3H : 1V
Vegetation	Trees and dense brush
Maintained	No
Armored	Yes
Benefits	Provides flood protection to residential property.
Current Rating	Unacceptable
Current Status	Ineligible

Date of Inspection: 27 September 2011

Inspected by:
Travis Goss, USACE
Jason Villarreal, USACE

General Items for All Flood Damage Reduction Systems

For use during all inspections of all Flood Damage Reduction Systems

Rated Item	Rating	Rating Guidelines		Location/ Remarks/ Recommendations
1. Operations and Maintenance Manuals	A	**A**	Levee Owner's Manual, O&M Manuals, and/or manufacturer's operating instructions are present.	
		M	Sponsor manuals are lost or missing or out of date; however, sponsor will obtain manuals prior to next scheduled inspection.	
		U	Sponsor has not obtained lost or missing manuals identified during previous inspection.	
2. Emergency Supplies and Equipment (A or M only)	A	**A**	The sponsor maintains a stockpile of sandbags, shovels, and other flood fight supplies which will adequately supply all needs for the initial days of a flood fight. Sponsor determines required quantity of supplies after consulting with inspector.	
		M	The sponsor does not maintain an adequate supply of flood fighting materials as part of their preparedness activities.	
3. Flood Preparedness and Training (A or M only)	A	**A**	Sponsor has a written system-specific flood response plan and a solid understanding of how to operate, maintain, and staff the FDR system during a flood. Sponsor maintains a list of emergency contact information for appropriate personnel and other emergency response agencies.	
		M	The sponsor maintains a good working knowledge of flood response activities, but documentation of system-specific emergency procedures and emergency contact personnel is insufficient or out of date.	

Key: A = Acceptable. M = Minimally Acceptable; Maintenance is required. U = Unacceptable. N/A = Not Applicable. FDR = Flood Damage Reduction. NR - Not Rated

Levee Embankments

For use during Initial and Continuing Eligibility Inspections of levee systems

Rated Item	Rating	Rating Guidelines		Location/ Remarks/ Recommendations
1. Unwanted Vegetation Growth[1]	**U**	**A**	The levee has little or no unwanted vegetation (trees, bush, or undesirable weeds), except for vegetation that is properly contained and/or situated on overbuilt sections, such that the mandatory 3-foot root-free zone is preserved around the levee profile. The levee has been recently mowed. The vegetation-free zone extends 15 feet from both the landside and riverside toes of the levee to the centerline of the tree. If the levee access easement doesn't extend to the described limits, then the vegetation-free zone must be maintained to the easement limits. Reference EM 1110-2-301 or Corps policy for regional vegetation variance.	Great deal of brush and trees on slopes and crown for entire length of levee. Several trees in both riverward and landward slopes that exceed 4" DBH.
		M	Minimal vegetation growth (brush, weeds, or trees 2 inches in diameter or smaller) is present within the zones described above. This vegetation must be removed but does not currently threaten the operation or integrity of the levee.	
		U	Significant vegetation growth (brush, weeds, or any trees greater than 2 inches in diameter) is present within the zones described above and must to be removed to reestablish or ascertain levee integrity.	
2. Sod Cover	**A**	**A**	There is good coverage of sod over the levee.	
		M	Approximately 25% of the sod cover is missing or damaged over a significant portion or over significant portions of the levee embankment. This may be the result of over-grazing or feeding on the levee, unauthorized vehicular traffic, chemical or insect problems, or burning during inappropriate seasons.	
		U	Over 50% of the sod cover is missing or damaged over a significant portion or portions of the levee embankment.	
		N/A	Surface protection is provided by other means.	
3. Encroach-ments	**A**	**A**	No trash, debris, unauthorized farming activity, structures, excavations, or other obstructions present within the easement area. Encroachments have been previously reviewed by the Corps, and it was determined that they do not diminish proper functioning of the levee.	
		M	Trash, debris, unauthorized farming activity, structures, excavations, or other obstructions present, or inappropriate activities noted that should be corrected but will not inhibit operations and maintenance or emergency operations. Encroachments have not been reviewed by the Corps.	
		U	Unauthorized encroachments or inappropriate activities noted are likely to inhibit operations and maintenance, emergency operations, or negatively impact the integrity of the levee.	
4. Closure Structures (Stop Log, Earthen Closures, Gates, or Sandbag Closures) (A or U only)	**N/A**	**A**	Closure structure in good repair. Placing equipment, stoplogs, and other materials are readily available at all times. Components are clearly marked and installation instructions/ procedures readily available. Trial erections have been accomplished in accordance with the O&M Manual.	
		U	Any of the following issues is cause for this rating: Closure structure in poor condition. Parts missing or corroded. Placing equipment may not be available within the anticipated warning time. The storage vaults cannot be opened during the time of inspection. Components of closure are not clearly marked and installation instructions/ procedures are not readily available. Trial erections have not been accomplished in accordance with the O&M Manual.	
		N/A	There are no closure structures along this component of the FDR system.	

Key: A = Acceptable. M = Minimally Acceptable; Maintenance is required. U = Unacceptable. N/A = Not Applicable. FDR = Flood Damage Reduction. NR - Not Rated

[1] If there is significant growth on the levee that inhibits the inspection of animal burrows or other items, the inspection should be ended until this item is corrected.

Levee Embankments

For use during Initial and Continuing Eligibility Inspections of levee systems

Rated Item	Rating		Rating Guidelines	Location/ Remarks/ Recommendations
5. Slope Stability	**NR**	**A**	No slides, sloughs, tension cracking, slope depressions, or bulges are present.	Assessment of this rated item was not possible due to presence of excessive vegetation at the time of inspection.
		M	Minor slope stability problems that do not pose an immediate threat to the levee embankment.	
		U	Major slope stability problems (ex. deep seated sliding) identified that must be repaired to reestablish the integrity of the levee embankment.	
6. Erosion/ Bank Caving	**NR**	**A**	No erosion or bank caving is observed on the landward or riverward sides of the levee that might endanger its stability.	Assessment of this rated item was not possible due to presence of excessive vegetation at the time of inspection. Deterioration of crown shoulder near residence.
		M	There are areas where minor erosion is occurring or has occurred on or near the levee embankment, but levee integrity is not threatened.	
		U	Erosion or caving is occurring or has occurred that threatens the stability and integrity of the levee. The erosion or caving has progressed into the levee section or into the extended footprint of the levee foundation and has compromised the levee foundation stability.	
7. Settlement[1]	**A**	**A**	No observed depressions in crown. Records exist and indicate no unexplained historical changes.	
		M	Minor irregularities that do not threaten integrity of levee. Records are incomplete or inclusive.	
		U	Obvious variations in elevation over significant reaches. No records exist or records indicate that design elevation is compromised.	
8. Depressions/ Rutting	**A**	**A**	There are scattered, shallow ruts, pot holes, or other depressions on the levee that are unrelated to levee settlement. The levee crown, embankments, and access road crowns are well established and drain properly without any ponded water.	
		M	There are some infrequent minor depressions less than 6 inches deep in the levee crown, embankment, or access roads that will pond water.	
		U	There are depressions greater than 6 inches deep that will pond water.	
9. Cracking	**NR**	**A**	Minor longitudinal, transverse, or desiccation cracks with no vertical movement along the crack. No cracks extend continuously through the levee crest.	Assessment of this rated item was not possible due to presence of excessive vegetation at the time of inspection.
		M	Longitudinal and/or transverse cracks up to 6 inches in depth with no vertical movement along the crack. No cracks extend continuously through the levee crest. Longitudinal cracks are no longer then the height of the levee.	
		U	Cracks exceed 6 inches in depth. Longitudinal cracks are longer than the height of the levee and/or exhibit vertical movement along the crack. Transverse cracks extend through the entire levee width.	
10. Animal Control	**NR**	**A**	Continuous animal burrow control program in place that includes the elimination of active burrowing and the filling in of existing burrows.	Assessment of this rated item was not possible due to presence of excessive vegetation at the time of inspection.
		M	The existing animal burrow control program needs to be improved. Several burrows are present which may lead to seepage or slope stability problems, and they require immediate attention.	
		U	Animal burrow control program is not effective or is nonexistent. Significant maintenance is required to fill existing burrows, and the levee will not provide reliable flood protection until this maintenance is complete.	

Key: A = Acceptable. M = Minimally Acceptable; Maintenance is required. U = Unacceptable. N/A = Not Applicable. FDR = Flood Damage Reduction. NR - Not Rated

[1] Detailed survey elevations are normally required during Periodic Inspections, and whenever there are obvious visual settlements.

Levee Embankments

For use during Initial and Continuing Eligibility Inspections of levee systems

Rated Item	Rating	Rating Guidelines		Location/ Remarks/ Recommendations
11. Culverts/ Discharge Pipes[1] (This item includes both concrete and corrugated metal pipes.)	N/A	**A**	There are no breaks, holes, cracks in the discharge pipes/ culverts that would result in significant water leakage. The pipe shape is still essentially circular. All joints appear to be closed and the soil tight. Corrugated metal pipes, if present, are in good condition with 100% of the original coating still in place (either asphalt or galvanizing) or have been relined with appropriate material, which is still in good condition. Condition of pipes has been verified using television camera video taping or visual inspection methods within the past five years, and the report for every pipe is available for review by the inspector.	
		M	There are a small number of corrosion pinholes or cracks that could leak water and need to be repaired, but the entire length of pipe is still structurally sound and is not in danger of collapsing. Pipe shape may be ovalized in some locations but does not appear to be approaching a curvature reversal. A limited number of joints may have opened and soil loss may be beginning. Any open joints should be repaired prior to the next inspection. Corrugated metal pipes, if present, may be showing corrosion and pinholes but there are no areas with total section loss. Condition of pipes has been verified using television camera video taping or visual inspection methods within the past five years, and the report for every pipe is available for review by the inspector.	
		U	Culvert has deterioration and/or has significant leakage; it is in danger of collapsing or as already begun to collapse. Corrugated metal pipes have suffered 100% section loss in the invert. HOWEVER: Even if pipes appear to be in good condition, as judged by an external visual inspection, an Unacceptable Rating will be assigned if the condition of pipes has not been verified using television camera video taping or visual inspection methods within the past five years, and reports for all pipes are not available for review by the inspector.	
		N/A	There are no discharge pipes/ culverts.	

Key: A = Acceptable. M = Minimally Acceptable; Maintenance is required. U = Unacceptable. N/A = Not Applicable. FDR = Flood Damage Reduction. NR - Not Rated

[1] The decision on whether or not USACE inspectors should enter a pipe to perform a detailed inspection must be made at the USACE District level. This decision should be made in conjunction with the District Safety Office, as pipes may be considered confined spaces. This decision should consider the age of the pipe, the diameter of the pipe, the apparent condition of the pipe, and the length of the pipe. If a pipe is entered for the purposes of inspection, the inspector should record observations with a video camera in order that the condition of the entire pipe, including all joints, can later be assessed. Additionally, the video record provides a baseline to which future inspections can be compared.

Levee Embankments

For use during Initial and Continuing Eligibility Inspections of levee systems

Rated Item	Rating		Rating Guidelines	Location/ Remarks/ Recommendations
12. Riprap Revetments & Bank Protection	NR	A	No riprap displacement or stone degradation that could pose an immediate threat to the integrity of channel bank. Riprap intact with no woody vegetation present.	Assessment of this rated item was not possible due to presence of excessive vegetation at the time of inspection.
		M	Minor riprap displacement or stone degradation that could pose an immediate threat to the integrity of the channel bank. Unwanted vegetation must be cleared or sprayed with an appropriate herbicide.	
		U	Significant riprap displacement, exposure of bedding, or stone degradation observed. Scour activity is undercutting banks, eroding embankments, or impairing channel flows by causing turbulence or shoaling. Rock protection is hidden by dense brush, trees, or grasses.	
		N/A	There is no riprap protecting this feature of the system, or riprap is discussed in another section.	
13. Revetments other than Riprap	N/A	A	Existing revetment protection is properly maintained, undamaged, and clearly visible.	
		M	Minor revetment displacement or deterioration that does not pose an immediate threat to the integrity of the levee. Unwanted vegetation must be cleared or sprayed with an appropriate herbicide.	
		U	Significant revetment displacement, deterioration, or exposure of bedding observed. Scour activity is undercutting banks, eroding embankments, or impairing channel flows by causing turbulence or shoaling. Revetment protection is hidden by dense brush and trees.	
		N/A	There are no such revetments protecting this feature of the system.	
14. Underseepage Relief Wells/ Toe Drainage Systems	N/A	A	Toe drainage systems and pressure relief wells necessary for maintaining FDR system stability during high water functioned properly during the last flood event and no sediment is observed in horizontal system (if applicable). Nothing is observed which would indicate that the drainage systems won't function properly during the next flood, and maintenance records indicate regular cleaning. Wells have been pumped tested within the past 5 years and documentation is provided.	
		M	Toe drainage systems or pressure relief wells are damaged and may become clogged if they are not repaired. Maintenance records are incomplete or indicate irregular cleaning and pump testing.	
		U	Toe drainage systems or pressure relief wells necessary for maintaining FDR system stability during flood events have fallen into disrepair or have become clogged. No maintenance records. No documentation of the required pump testing.	
		N/A	There are no relief wells/ toe drainage systems along this component of the FDR system.	
15. Seepage	NR	A	No evidence or history of unrepaired seepage, saturated areas, or boils.	Assessment of this rated item was not possible due to presence of excessive vegetation at the time of inspection.
		M	Evidence or history of minor unrepaired seepage or small saturated areas at or beyond the landside toe but not on the landward slope of levee. No evidence of soil transport.	
		U	Evidence or history of active seepage, extensive saturated areas, or boils.	

Key: A = Acceptable. M = Minimally Acceptable; Maintenance is required. U = Unacceptable. N/A = Not Applicable. FDR = Flood Damage Reduction. NR - Not Rated

APPENDIX C.4

W1 (Henderson) Well Log

22/6E/4 F

B

WATER WELL REPORT
STATE OF WASHINGTON

Start Card No. 073814
Water Right Permit No.

(1) OWNER: Name HENDERSON, GREG Address 15256 22 AVE SW SEATTLE, WA 98166-

(2) LOCATION OF WELL: County KING - SE 1/4 NW 1/4 Sec 4 T 22 N., R 6 WM
(2a) STREET ADDRESS OF WELL (or nearest address) 218XX SE 198 PL

(3) PROPOSED USE: DOMESTIC

(4) TYPE OF WORK: Owner's Number of well (If more than one)
NEW WELL Method: ROTARY

(5) DIMENSIONS: Diameter of well 6 inches
Drilled 58 ft. Depth of completed well 58 ft.

(6) CONSTRUCTION DETAILS:
Casing installed: 6 " Dia. from 0 ft. to 53 ft.
WELDED " Dia. from ft. to ft.
" Dia. from ft. to ft.

Perforations: NO
Type of perforator used
SIZE of perforations in. by in.
perforations from ft. to ft.
perforations from ft. to ft.
perforations from ft. to ft.

Screens: YES
Manufacturer's Name JOHNSON
Type STAINLESS STEEL Model No.
Diam. 6 slot size .020 from 53 ft. to 58 ft.
Diam. slot size from ft. to ft.

Gravel packed: NO Size of gravel
Gravel placed from ft. to ft.

Surface seal: YES To what depth? 18 ft.
Material used in seal BENTONITE CLAY
Did any strata contain unusable water? YES
Type of water? SURFACE Depth of strata 16 ft.
Method of sealing strata off CASING

(7) PUMP: Manufacturer's Name
Type N/A H.P.

(8) WATER LEVELS: Land-surface elevation above mean sea level ... ft.
Static level 1/2 ft. below top of well Date 10/16/90
Artesian Pressure lbs. per square inch Date
Artesian water controlled by N/A

(9) WELL TESTS: Drawdown is amount water level is lowered below static level.
Was a pump test made? NO If yes, by whom?
Yield: gal./min with ft. drawdown after hrs.

Recovery data

Time	Water Level	Time	Water Level	Time	Water Level

Date of test / /
Bailer test gal/min. ft. drawdown after hrs.
Air test 30 gal/min. w/ stem set at 53 ft. for 1 hrs.
Artesian flow g.p.m. Date
Temperature of water Was a chemical analysis made? NO

(10) WELL LOG

Formation: Describe by color, character, size of material and structure, and show thickness of aquifers and the kind and nature of the material in each stratum penetrated, with at least one entry for each change in formation.

MATERIAL	FROM	TO
BROWN SAND & GRAVEL	0	6
BROWN CEMENTED SAND & GRAVEL	6	15
WATER BEARING SAND & GRAVEL	15	27
BLUE SILTY SAND	27	39
WATER BEARING SAND	39	58

Work started 10/15/90 Completed 10/16/90

WELL CONSTRUCTOR CERTIFICATION:
I constructed and/or accept responsibility for construction of this well, and its compliance with all Washington well construction standards. Materials used and the information reported above are true to my best knowledge and belief.

NAME NORTHWEST PUMP & DRILLING
(Person, firm, or corporation) (Type or print)

ADDRESS 3245 AUBURN WAY SOUTH

[SIGNED] RB DeCamu License No. 0097

Contractor's Registration No. NORTHPD137PQ Date 10/18/90

APPENDIX C.5

Stability Evaluation and Risk Assessment Report - CRT7 Revetment

STABILITY EVALUATION AND RISK ASSESSMENT

CRT7 Revetment

Prepared for: Tetra Tech, Inc. and King County Department of Natural Resources and Parks, Water, and Land Resources Division, River and Floodplain Management Section

Project No. 190175-600.2 • August 7, 2020 (Revised September 4, 2020) FINAL

earth + water

Aspect CONSULTING

STABILITY EVALUATION AND RISK ASSESSMENT

CRT7 Revetment

Prepared for: Tetra Tech, Inc. and King County Department of Natural Resources and Parks, Water, and Land Resources Division, River and Floodplain Management Section

Project No. 190175-600.2 • August 7, 2020 (Revised September 4, 2020) DRAFT

Aspect Consulting, LLC

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Executive Summary

The Cedar River Trail Site 7 (CRT7) Revetment (revetment) along the left bank of the Cedar River (river) between approximately river mile (RM) 12.95 and RM 13.16 is an existing King County (County) flood protection facility that protects the left bank of the river, the Cedar River Trail (trail), and Washington State Route (SR) 169 from channel migration and scour-related impacts of the river. Currently, flows are directed and concentrated at the revetment by the existing upstream Jan Road levee (levee) along the right bank of the river, leading to vulnerabilities caused by bank erosion and scour processes occurring along the revetment.

The revetment is located within the larger Jan Road Neighborhood Improvements Project (project), which aims to reduce overall flood risks to people, property, and infrastructure and improve natural river processes, function, and habitat, where feasible. Reduction in flood risks to the revetment, and the public infrastructure that it protects (i.e. trail and SR 169), is a primary consideration of the project.

Aspect Consulting, LLC (Aspect) has completed a slope stability evaluation and risk assessment to provide an understanding of the existing stability and vulnerability of the revetment as well as potential stability benefits and impacts resulting from implementation of the project, based on the current design alternatives. Upon implementation of the project, it is anticipated that hydraulic and geomorphic conditions within the project reach, including near the revetment, will change. Additionally, the adjacent and upstream Rutledge-Johnson project will change conditions at the revetment and the effects of the Rutledge-Johnson project, were considered in our evaluation to the extent possible.

The results of our analyses are summarized on attached Tables 7 through 9, with detailed model outputs for the critical section included in Appendix D. Based on our evaluation, we have concluded the following about the revetment:

- Overall, the revetment is tall, steep, and primarily composed of loose fill soils creating a baseline condition of marginal stability.

- The risk of instability within the armor rock and surface of the revetment is moderate with primary consequences including progressive failure of the revetment and potential failure of portions of the trail.

- The risk of instability affecting the trail outside of the progressive scenario described above is low to moderate with primary consequences including the sudden failure of portions or all of the trail.

- The risk of instability affecting SR 169 is low due to the very low likelihood of the influence of revetment failure extending landward far enough to impact the highway.

- The project alternatives will reduce flows, flow velocity, and shear stresses resulting in lower water levels and scour depths at the revetment on the order of 2 feet.
- Our analyses indicate, the revetment stability is not sensitive to minor reductions (~ 2 feet) in water levels and scour depths like those anticipated for the project alternatives and the adjacent Rutledge-Johnson project.
- Overall, implementation of the project alternatives will not significantly improve or reduce the stability of the revetment in the short to medium term.
- Over the long term (20+ years), channel migration could manifest several ways ranging between the following two scenarios:
    - The river could migrate into the right floodplain, encroach against the new setback levee, and direct the main river channel back toward the revetment. This will likely increase flows and shear stresses on the revetment, thereby potentially increasing the vulnerability of the revetment.
    - The river could avulse into the new primary side channel on the right floodplain and likely further reduce flows and shear stresses on the revetment, thereby potentially reducing the vulnerability of the revetment.

A full description of our observations, analyses, and conclusions regarding revetment risk is contained herein.

1 Introduction

This report presents the results of Aspect Consulting, LLC's (Aspect) stability evaluation and risk assessment of the Cedar River Trail Site 7 (CRT7) Revetment (revetment) along the left bank of the Cedar River (river) between approximately river mile (RM) 12.95 and RM 13.16 (site; Figure 1).

The revetment protects the left bank and overlying earthen embankment that supports the Cedar River Trail (trail) and Washington State Route (SR) 169 from channel migration and scour-related impacts of the river. The site topography, bathymetry, revetment extents, trail, and SR 169 are shown on Figure 2.

This report summarizes our site reconnaissance, characterization of the Site subsurface conditions, our slope stability analyses of the revetment under existing and potential future conditions related to the Jan Road Neighborhood Improvements Project (project), and our geotechnical engineering conclusions regarding the existing stability and vulnerability of the revetment as well as potential stability benefits and impacts resulting from the implementation of the project alternatives. This risk assessment and report is specific to the revetment site defined above which is located within the larger project area.

A workshop was held on June 4, 2020 to discuss the findings of our analyses and risk implications to the revetment. The workshop was attended by the members of the King County Department of Natural Resources and Parks, Water and Land Resources Division, Risk and Floodplain Management Section (County), Tetra Tech, and Watershed GeoDynamics.

1.1 Scope of Work and Authorization

Our work was performed in accordance with Task 600.2 of our subconsultant agreement with Tetra Tech, Inc. under King County Department of Natural Resources and Parks, Water and Land Resources Division, Risk and Floodplain Management Section, Contract No. E00599E19, authorized on July 29, 2019.

1.2 Project Description

The revetment is an existing County flood protection facility that protects the left bank of the river, the trail, and SR 169 from channel migration and scour-related impacts of the river. Currently, flows are directed and concentrated at the revetment by the existing levee along the right bank of the river, leading to vulnerabilities caused by bank erosion and scour processes occurring along the revetment. The trail is part of a regional multi-modal network and is located atop the revetment approximately 30 feet (horizontally) from the edge of the river at its nearest point. SR 169 is a primary transportation corridor between Maple Valley and Renton and is located behind (landward) of the revetment approximately 100 feet (horizontally) from the edge of the river at its nearest point.

The revetment is located within the larger project, which aims to reduce overall flood risks to people, property, and infrastructure and improve natural river processes, function, and habitat, where feasible. Reduction in flood risks to the revetment, and the public infrastructure that it protects (i.e. trail and SR 169), is a primary consideration of the project.

Two project alternatives are under consideration and include the following elements along the right bank of the river:

- **Alternative 1:** Elevating the majority of a private roadway (SE 197th Pl/221st Ave SE/218th Ave SE) serving the neighborhood and installation of cross culverts, removing the existing levee, constructing new setback levee, floodplain regrading with side channels, and upgrades to the existing culvert under SE 197th Pl.

- **Alternative 2:** removing the existing levee, constructing new setback levee, floodplain regrading with side channels, and upgrades to the existing culvert under SE 197th Pl.

Additionally, an adjacent project called the Rutledge-Johnson project is under concurrent development and located along the left bank of the river immediately upstream of the Jan Road Neighborhood project. The primary design elements of the Rutledge-Johnson project are not fully known but anticipated to include some amount of removal of the existing levee and side channel development through the left bank floodplain.

The project will change hydraulic and geomorphic conditions near the revetment. Accordingly, the County wishes to understand the stability and vulnerability of the revetment under existing and proposed project conditions.

2 Site Description

This section presents the site conditions, including site surface conditions, geologic setting, and subsurface conditions as informed by our site reconnaissance and review of existing data. This information provides context for the discussion of types and distribution of geologic soil units, and a basis for our geotechnical engineering analyses and conclusions.

2.1 Existing Data Review

The County provided the following documentation related to the revetment for our review:

- Revetment inspection records from 2011 to 2020.
- Damage observations from the winter of 1995/1996 that included revetment failure at approximately RM 12.95.
- Repair drawings for the damaged section of revetment at approximately RM 12.95 from 1996.
- Preliminary geotechnical data report completed by County Road Services Division including three soil borings along the revetment (King County, 2020a; Appendix A).
- Utility locating information related to the previous soil borings.

Information related to the timing and details of the original construction of the revetment was not available.

2.1.1 Revetment Records

Extensive damage of the revetment at approximately RM 12.95 was observed following flood events in November 1995 and February 1996. The damage included slumping of the revetment and riverbank adjacent to the trail. The high flows of the flood events almost completely washed away the revetment armor rock resulting in a nearly vertical slope of bare and exposed soil extending along approximately 125 linear feet of the revetment surface. Photographs from the damage appear to indicate the revetment armoring prior to the flood damage included angular rocks on the surface of the slope between 3 and 5 feet thick.

Figure 3. Revetment Damage (1996)

Repair of the damaged section of revetment was completed in 1996, and included replacing the large, 5- to 6-man, armor rocks supplemented with anchored logs with rootwads and regrading of the revetment with a mechanically stabilized earth (geogrid reinforced) 2H:1V (Horizontal:Vertical) slope.

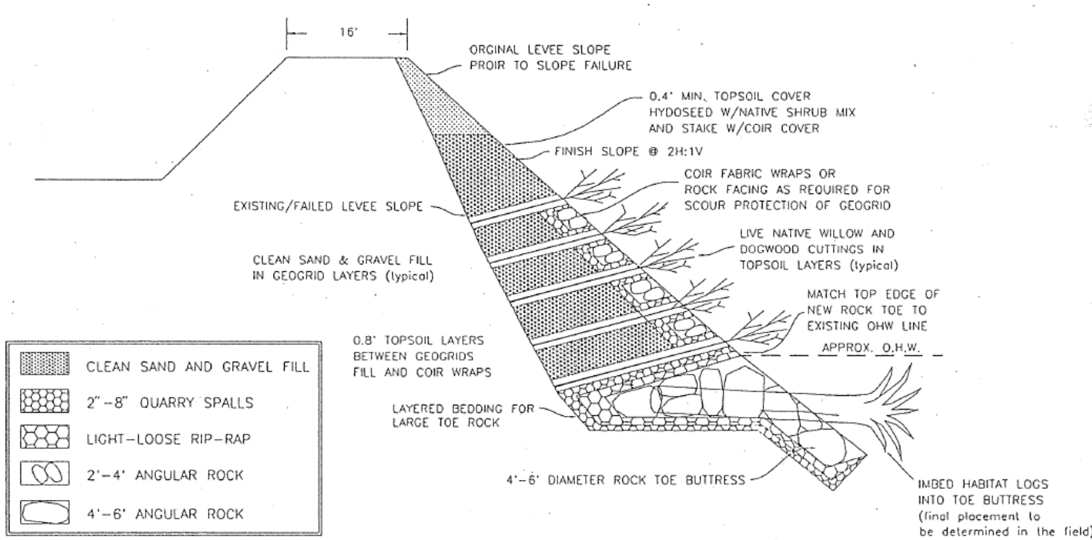

Figure 4. Revetment Typical Repair Section (1996)

More recently, revetment inspection records including land and float inspections conducted by County staff between 2011 and 2020, included in Appendix B, generally indicate evidence of on-going and intermittent scour-related damage to the revetment including oversteepened conditions, displaced toe and armor rocks, large voids between armor rocks, and undermined trees. Cracks in the asphalt surface of the trail were also noted in 2018, but it was unclear if these cracks were the result of revetment instability or root growth from the large, mature cottonwood trees that are present across the top of the revetment. Anecdotally, we understand that the revetment experienced minor damage but no failures during the last two major flood events in January 2009 and February 2020.

Utility locating information from the recent soil borings indicate that there are no known buried utilities within the revetment. We understand that a buried fiber optics cable utility is present beneath the trail further downstream, but not at the site.

2.1.2 Preliminary Geotechnical Data Report

The County Road Services Division completed a geotechnical investigation and preliminary geotechnical data report (King County, 2020a) for the project that included three soil borings advanced to depths of between 50.5 to 70 feet below the trail surface along the revetment, as well as several test pits and borings along the right bank of the river in the vicinity of the site (See Figure 2 and Appendix A). Our interpretation and discussion of the subsurface conditions at the revetment is included in Section 2.4 of this report.

2.2 Site Reconnaissance

Aspect staff completed a reconnaissance of the revetment and surrounding areas on July 30, 2019, and August 15, 2019. Our reconnaissance work was completed by walking the trail, traversing the revetment where accessible, and observing the revetment from the upstream area of the left bank and the adjacent right bank. During our reconnaissance, we observed and noted visible features, such as revetment inclination and geometry, trail conditions, armor rock configuration and displacement, soil exposures, and vegetation patterns. Select photographs from our site reconnaissance are included in Appendix C.

The waterward side of the revetment extends between 25 and 30 feet above the river channel and the inclination varies but is typically steeper over the lower half of the revetment with inclinations ranging from 37 degrees (1.33H:1V) to 45 degrees (1H:1V) with localized areas that are near vertical. The upper half of the revetment is typically inclined at 20 degrees (2.75H:1V) to 30 degrees (1.73H:1V). The crest width of the revetment is approximately 16 feet and includes approximately 12 feet of asphalt-paved trail. The landward side of the revetment slopes down towards the west and SR 169 at an inclination of about 25 degrees (2.15H:1V), forming a gentle swale between the trail and SR 169. The base of the swale is approximately 8 feet below the trail surface and located well-above river water levels. Our observations of the revetment geometry generally confirm the topographic and bathymetric ground survey data used to formulate our slope stability models for this study and are described in Section 3.3.2.2 of this report.

At the site, the river flows almost directly at (perpendicular to) the revetment in a westerly direction before being redirected by the revetment towards the northwest and parallel to the trail and revetment (Figure 2). We did not observe any obvious signs of

recent or incipient revetment failure at the site. However, we did observe indicators of distress that included the following:

- A relatively deep scour hole is present at the upstream end of the revetment. Based on the available bathymetric data, the scour hole extends approximately 6 feet below the adjacent channel bed and extends to the toe of the revetment. The deepest part of the scour hole is located about 50 feet from the revetment.
- Displaced armor rocks lie within the river channel at the upstream end of the revetment.
- Sporadic voids between the revetment armor rocks, up to a couple feet wide.
- An area at the top of the revetment on the waterward side of the trail where the ground surface has settled creating a concave shape and a vertical offset on the order of 1 to 3 feet. Timber planks had been placed along the edge of the trail for interim stabilization and appeared to be at least 2 to 3 years old.
- Cracks within the trail asphalt surface. The cracks emanate outward from the waterward side of the trail in perpendicular to sub-perpendicular patterns from the river channel and appear more consistent with distress caused by tree root growth below the trail surface rather than cracks related to revetment instability that we would expect to be arcuate-shaped and oriented parallel to the river channel.
- Mature vegetation on the revetment consists of deciduous cottonwood trees. While deciduous trees are not reliable indicators of stability (or instability) due to their tendency to grow towards favorable light conditions, there is evidence of undermining of some trees on the revetment and associated trunk lean. Trees can improve stability through effective cohesion from the root systems, but as the trees get larger, they can also reduce stability when those roots are leveraged against the slope by windthrowing which is the destabilizing force of an overturning moment exerted on the slope as a result of wind forces blowing against the tree.

2.3 Geologic Setting

The geology of the site and vicinity is characterized by Cedar River alluvium deposited on and within a sequence of Fraser-age and pre-Fraser glacial deposits overlying Tertiary bedrock of the Renton Formation at depth (Booth, 1995). Quaternary glacial deposits in the site vicinity show a record of multiple ice advances through the region. Drainage of a regional glacial lake caused rapid erosion, creating the Cedar River valley within the thick, complex sequence of glacial and nonglacial deposits. The surficial geology of the present-day Cedar River valley is characterized by relatively coarse-grained recent alluvium within the floodplain consisting of boulders, cobbles, gravel, sand, and silty sand with steep side slopes, often mantled by colluvium from episodic landslides and soil creep (Booth, 1995).

2.4 Subsurface Conditions and Soil Engineering Units

Our understanding and characterization of the subsurface conditions at and near the revetment was primarily based on the preliminary geotechnical data report completed by the County Road Services Division (King County, 2020a; Appendix A).

The general stratigraphy of the revetment includes fill overlying alluvium overlying Pre-Fraser Glacial deposits. Detailed description of the geologic units is included below.

2.4.1 Revetment Fill

Soil interpreted to be fill was encountered under the trail in borings B-1 through B-3 to depths of approximately 20 feet below the ground surface (bgs). The fill soils typically consist of fine to medium sand and silty sand (SP, SM)[1], fine and coarse gravel (GP), and cobbles. The fill was likely placed during the construction of the former railroad grade and/or revetment that underlies the trail alignment; however, the fill typically has a jumbled and unstructured appearance which is similar to the appearance of landslide deposits. Landslide or mass wasting deposits are not mapped at the revetment location, but are mapped along the lower portion of the slope immediately west of SR 169, and it is possible that a portion of the fill noted in borings B-1 through B-3 consists of or was sourced from landslide deposits.

The standard penetration test blow count values (“N-values”) ranged from 5 to 18 blows per foot (bpf), with an average N-value of 9 bpf, indicating the fill was placed in a loose to medium dense configuration. No groundwater was encountered within the fill.

Based on soil type and density, the fill can be assumed to possess moderate shear strength, low to moderate compressibility, low to moderate moisture sensitivity due to its variable fines content, no susceptibility to liquefaction (no groundwater encountered in this unit), and high permeability.

2.4.2 Alluvium Deposits

Soil interpreted to be high-energy alluvial deposits was encountered underlying the fill in borings B-1 through B-3. The alluvium was encountered to approximately 30 feet bgs in B-1 through B-3. The alluvium typically consists of an upper layer of silty sand or fine to medium sand (SM, SP-SM) which we interpret to be overbank deposits. This upper layer of alluvium is underlain by more granular deposits consisting of fine to medium sand (SP, SW) and fine and coarse gravel (GP, GW), and cobbles which we interpret to be coarser sediment deposited in the higher energy environment of the river channel (or former river channels) .

Based on data across the floodplain and at the revetment, the overbank deposits are between a couple inches and 2.5 feet thick.

The N-values within this unit ranged from 8 to greater than 50 bpf. Accounting for the potential that some blow counts are overstated due to the presence of gravel and cobbles, we consider the alluvium to be in a generally medium dense to dense configuration.

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[1]Soils classified in accordance with the Unified Soil Classification System (USCS), ASTM D2488.

Based on soil type and density, the alluvial deposits can be assumed to possess moderate to high shear strength, low compressibility, low to moderate moisture sensitivity due to its variable fines content, low to moderate susceptibility to liquefaction, and high permeability.

2.4.3 Pre-Fraser Glacial Deposits

Sediment interpreted to consist of pre-Fraser glacial deposits was encountered in borings B-1 through B-3 from approximately 30 feet bgs to the bottom of explorations (50 to 70 feet bgs). At boring B-5 within the floodplain, similar soils were encountered in the sample collected at 20 feet bgs. The pre-Fraser glacial deposits typically consist of fine to medium sand (SP, SW-SM, SW) that is weakly to moderately oxidized.

The N-values within this unit were typically greater than 50 bpf, because the sediments were glacially overridden to a very dense configuration after deposition. Groundwater was present throughout this unit.

Based on soil type and density, the pre-Fraser glacial deposits can be assumed to possess high shear strength, low compressibility, low moisture sensitivity due to its low fines content, low susceptibility to liquefaction due to its dense configuration, and high permeability.

2.5 Groundwater

Groundwater at the revetment was encountered in all three borings between 21 and 22 feet below the surface of the trail. This is consistent with the river water levels and groundwater levels encountered in the other explorations across the floodplain, such as boring B5. Hydraulic continuity at the site is relatively high, meaning groundwater levels are expected to fluctuate with changes in river levels.

3 Hydraulic and Geomorphic Conditions

The following is a qualitative summary of the hydraulic and geomorphic conditions at the site under existing conditions and the project alternatives as provided by Tetra Tech and Watershed Geodynamics.

3.1 Existing Conditions

Under the existing conditions, river flows are directed at the base of the revetment by the existing levee and westerly flow of the river. During flood events, the existing levee can be overtopped, which reduces the flow rate and velocity at the revetment; however, during large flood events, these reductions are offset by additional flow contributions coming from the upstream Rutledge-Johnson area. Flow velocities for the 100-year flood are highest at the revetment based on the hydraulic modeling results. The scour hole present at the upstream end of the revetment has created a pool which provides a small amount of cushioning of the approaching river flows; however, this effect is limited to the pool itself and does not extend downstream along the rest of the revetment.

3.2 Potential Future Conditions Under Project Alternatives

When considering the project alternatives, which are similar with respect to their hydraulic and geomorphic impacts, the following potential conditions are anticipated:

- Flow diversion into proposed side channels will reduce the water levels in the greater project area except at the outlet of the side channels. At these locations near RM 12.96 and 13.06, return flow from the side channels will create a localized backwater condition.
- For larger flood events, removal of the existing levee will allow unconstrained flow into the floodplain along the right bank, reducing water levels upstream of RM 13.12. Downstream of this point, the large volume of floodwater reentering the main river channel from the floodplain will cause an increase in the water levels.
- Flow velocity will be reduced in the river near the revetment. Velocity reduction upstream of RM 13.12 will be due to less flow in the main river channel. Downstream of this location, floodwaters returning to the main river channel from the floodplain will cause backwater conditions, increasing the water level and flow area resulting in lower velocities.
- The hydraulic analysis showed that shear stresses on the revetment will be reduced under all modeled flows. As a result, revetment bank erosion potential is expected to be lower with construction of the alternatives compared to existing conditions.
- Removal of the existing levee and construction of new side channels in the floodplain will allow the river to migrate into the right floodplain. Based on other levee removal projects in the vicinity, it is anticipated that channel migration will

proceed via channel widening at the apex of the meander bend near RM 13.3 to 13.4 (at the upstream end of the proposed primary side channel).

- The rate of widening/migration is anticipated to be on the order of up to 20 feet over the short term (0-5 years) and up to 70 feet over the medium term (5 – 20 years) assuming moderate flows occur during this period.
- Over the long term, it is anticipated that several scenarios could occur ranging between those shown on Figure 5: 1) the river could migrate into the right floodplain, bump up against the new setback levee, and direct the main river channel back toward the revetment; 2) the river could avulse into the primary side channel; or 3) the river could continue to migrate with a more gentle curve.
- Over the long term, it is anticipated the bend at RM 13.35 will continue migrating laterally toward the new setback levee, likely resulting in one of two planform configurations, as it moves through the Project site:
  - Into the primary side channel which could convert the primary side channel into the main channel and the current main channel into a side channel. This would result in less flow against the CRT7 revetment at the current location but could shift erosive forces to a location closer to RM 13.00.
  - If the channel migrates into the curve of the setback levee, the levee could direct the flow around the bend and in a southerly direction back towards the CRT7 revetment at a point upstream of the current impingement point. This scenario is less likely but possible over the long term.

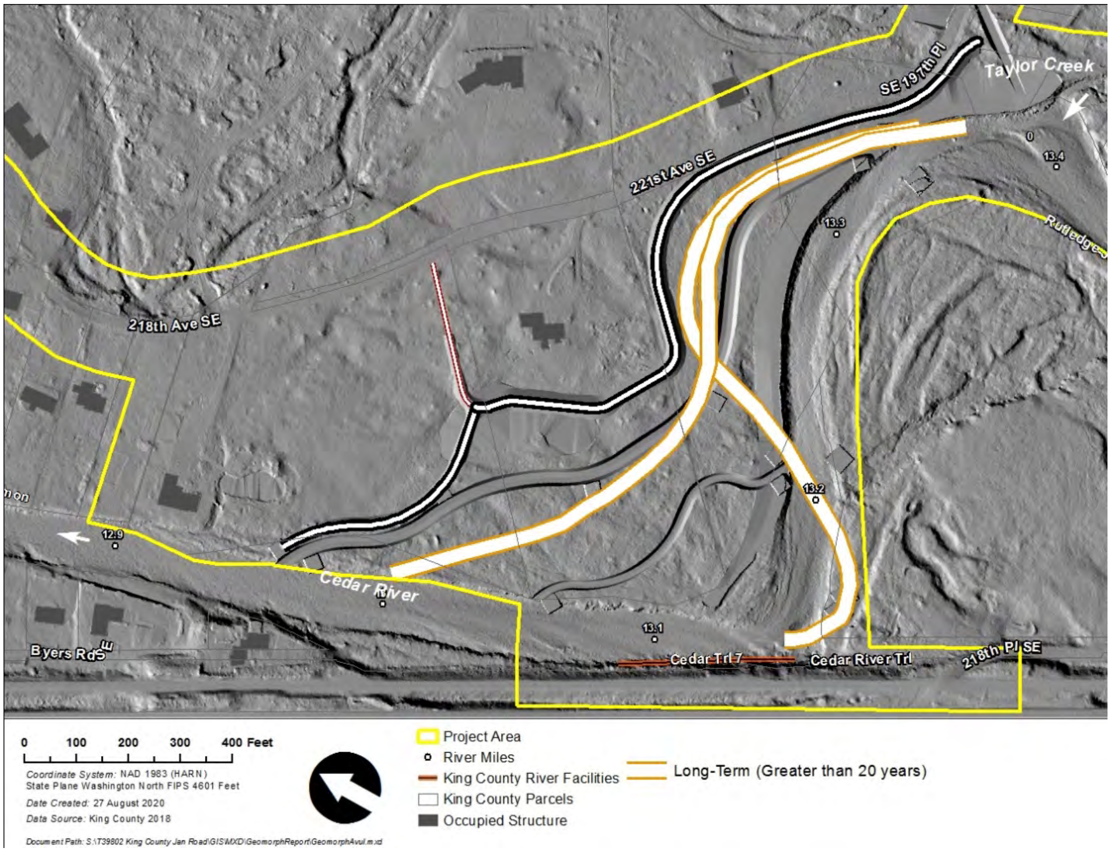


Figure 5. Potential Long-Term Channel Migration Pathways (King County, 2020c)

3.3 Rutledge-Johnson Project Effects

The Rutledge-Johnson project site located along the left bank of the river, upstream of the Jan Road Neighborhood project and revetment will have potential effects on the revetment depending on how that project is implemented. It is anticipated some portion of the Rutledge-Johnson levee may be removed and new side channels created through the left bank floodplain.

In general, river flow diverted through the Rutledge-Johnson project site due to levee removal will decrease the water level between RM 13.2 and 13.5 by about 0.4 feet for the 100-year flood. Water level reduction in this reach will reduce the volume of overflow to SE 197th Pl along the right bank, but potentially increase the volume of flow in the river at RM 13.2 and the revetment. For the 100-year flood, increased flow in the river will also increase the water level by about 0.10 feet immediately downstream of the Rutledge-Johnson project site and by about 0.05 feet at RM 12.9. The magnitude in the change in water levels and flow is about the same with and without the Jan Road project alternatives.

4 Geotechnical Engineering Analyses

The scope of this evaluation requires detailed slope stability analyses to assess the vulnerability of the revetment under existing conditions and proposed project conditions, based on the project design alternatives. The following sections describe inputs developed for our slope stability analyses, the methodologies used, and results.

4.1 Soil Engineering Properties

The soil engineering properties used in our slope stability analyses were developed based on our review of the preliminary geotechnical data report, empirical correlations with standard penetrometer test (SPT) blow counts, literature review, and our knowledge of the local geology. To validate our assumed soil engineering properties, we completed model testing within our stability models to confirm the engineering properties chosen resulted in stable conditions at the revetment during known events and scenarios. These events included the magnitude 6.8 2001 Nisqually Earthquake and the 2008 flood on the Cedar River. The estimated peak ground acceleration (PGA) at the site during the 2001 Nisqually Earthquake was approximately 0.12g based on the records from nearby ground motion monitoring stations (USGS, 2019). Significant damage/failure of the revetment was not observed during these events and our stability models showed similar results indicating the assumed soil engineering properties were appropriate.

To account for the natural range in variability of the engineering properties of each material, we also varied the engineering properties used in our analyses to investigate if changes in the properties across their reasonable range (typically +/- 10 percent) significantly affected the stability results. Through this exercise, we confirmed the stability results were similar for a reasonable range of unit weights, friction angles, effective cohesions, and hydraulic conductivities.

We utilized the Unified Soil Classification System (USCS) soil classifications and empirical correlations based on grain size analyses (Massman, 2003) to develop reasonable assumptions of hydraulic conductivity for each soil unit.

Specific references utilized in the development of the soil engineering properties include the Washington State Department of Transportation (WSDOT) Geotechnical Design Manual (GDM; WSDOT, 2019), the Navy Facilities Engineering Command (NAVFAC) Design Manual 7.1 (NAVFAC, 1986), and Holtz and Kovacs (1981).

Based on the estimated hydraulic conductivities, all of the soil units at and near the revetment were assumed to have high permeability characteristics; therefore, effective (drained) shear strength parameters were used throughout our analyses. The soil units and engineering properties used in our stability analyses are shown on Table 1.

Table 1. Soil Units and Engineering Properties

Soil Unit	Unit Weight (moist)	Unit Weight (saturated)	Effective (drained) Friction Angle	Effective (drained) Cohesion	Hydraulic Conductivity
	pcf	pcf	degrees	psf	cm/s
CRT7 Fill	115	120	32	50	$3x10^{-2}$
Overbank Alluvial Deposits	110	115	32	50	$3x10^{-2}$
High-Energy Alluvial Deposits	125	130	35	0	$4x10^{-1}$
Pre-Fraser Glacial Deposits	130	135	40	0	$2x10^{-2}$
CRT7 Revetment	135	140	45	0	>10

Notes:

1) pcf = pounds per cubic foot.
2) psf = pounds per square foot.
3) cm/s = centimeters per second.

4.2 Earthquake Engineering

The site is located within the Puget Lowland physiographic province, an area of active seismicity that is subject to earthquakes on shallow crustal faults and deeper subduction zone earthquakes. The site lies about 3.6 miles south of the Seattle fault zone, which consists of shallow crustal structures that are considered active (evidence for movement within the Holocene [since about 15,000 years ago]) and is capable of producing earthquakes of magnitude 7.3 or greater. The recurrence interval of earthquakes on this fault zone is on the order of 1,000 years or more. The most recent large earthquake on the Seattle fault occurred about 1,100 years ago (Pratt et al., 2015). There are also several other shallow crustal faults in the region capable of producing earthquakes and strong ground shaking.

The site also lies within the zone of strong ground shaking from earthquakes associated with the Cascadia Subduction Zone (CSZ). Subduction zone earthquakes occur due to rupture between the subducting oceanic plate and the overlying continental plate. The CSZ can produce earthquakes up to magnitude 9.3 and the recurrence interval is thought to be on the order of about 500 years. A recent study provides strong evidence that the most recent CSZ earthquake occurred 320 years ago in January 1700 (Atwater et al., 2015).

Deep intraslab earthquakes, which occur from tensional rupture of the sinking oceanic plate, are also associated with the CSZ. An example of this type of seismicity is the 2001 Nisqually earthquake. Deep intraslab earthquakes typically are magnitude 7.5 or less and occur approximately every 10 to 30 years.

The following sections present descriptions of seismic design considerations for this evaluation.

4.2.1 Ground Motions

The stability and vulnerability of the revetment under seismic conditions is an important component of this evaluation and risk assessment. Ground motions for seismic analyses were selected in accordance with guidance from USACE ER 1110-2-1806 (USACE, 2016) which recommends consideration of ground motions reflecting the Operating-Basis Earthquake (OBE) which has a 50 percent probability of exceedance in 100 years or a return period of 144 years. Using guidance from Chapter 6 of the WSDOT GDM, existing embankments that do not support or impact structures such as bridges, tunnels, and walls, do not require mitigation for seismic slope stability; therefore, we did not evaluate the revetment or SR 169 for the WSDOT/AASHTO[2] design level earthquake for this risk assessment.

Based on the available subsurface exploration data, we concluded the appropriate seismic site class for the site and revetment is Site Class D and we utilized the USGS National Seismic Hazard Map data (USGS, 2014) to obtain the respective ground motion parameters and earthquake characteristics for the OBE.

Table 2. Operating-Basis Earthquake Parameters

Earthquake	Return Period (years)	Peak Ground Acceleration (g)	Earthquake Magnitude[(1)]	Modal Source-to-Site Distance (km)[(1)]
OBE	144	0.204	7.10	68.13

Notes:
1) Based on USGS Probabilistic Seismic Hazard Deaggregation.

4.2.2 Surficial Ground Rupture

An inferred surficial trace of the southernmost splay of the Seattle Fault Zone, an active, east-west trending, south-dipping thrust fault system, is located approximately 3.6 miles north-northeast of the site (DNR, 2019). This fault zone consists of multiple strands of southerly dipping shallow thrust faults and northerly dipping backthrust planes that extended to the ground surface in the late Holocene (Pratt et al., 2015). However, due to the suspected long recurrence interval and the distance of the site to the mapped fault zone, the potential for surficial ground rupture at the site is considered low.

4.2.3 Liquefaction

Liquefaction occurs when loose, saturated, and relatively cohesionless soil deposits temporarily lose strength from seismic shaking. Potential effects of soil liquefaction include temporary loss of bearing capacity and lateral soil resistance, liquefaction-induced settlement, riverbank slope failure, and lateral spreading, any of which could result in damage to the revetment, trail, or SR 169. The primary factors controlling the onset of liquefaction include intensity and duration of strong ground motion, characteristics of subsurface soil, *in situ* stress conditions, and the groundwater conditions.

[2] American Association of State Highway and Transportation Officials

The Washington Department of Natural Resources (DNR) maps the site as having moderate to high liquefaction susceptibility (DNR, 2004).

We conducted a liquefaction analysis at the revetment with the aid of WSliq, a liquefaction analysis software program that was created as part of an extended research project supported by the WSDOT and authored by Steve Kramer (Kramer, 2008). Our liquefaction analysis utilized the data from boring B-2 which we determined represents the most conservative (i.e. liquefaction-prone) conditions at the revetment. Groundwater and river levels for the liquefaction analyses were assumed at mean annual levels approximately 21 feet below the surface of the trail. Ground motions reflecting the OBE were used in the liquefaction evaluations.

Given the relative density (medium dense to dense), grain size distribution (gravelly alluvium with cobbles), and geologic origin of the saturated soils (Pre-Fraser deposits at depth) at the site, the results of our liquefaction analyses indicate that wide-spread liquefaction is not predicted to occur at the revetment for the OBE. The factors of safety against liquefaction (FS_L) for the saturated soil units as a function of depth below the trail surface are shown on Table 3.

Table 3. Liquefaction Analysis Results

Soil Unit	Depth Below Trail (feet)	FS_L OBE
Alluvium	22	1.10
Alluvium	29	1.50
Alluvium	42.5	1.54
Alluvium	52.5	29.00
Pre-Fraser Deposits	62.5	31.17

Notes:
1) FS_L = factor of safety against liquefaction.

4.3 Slope Stability Analyses

The revetment is a unique structure as its classification, both in form and function, is in-between a traditional levee and highway embankment. In order to develop an appropriate rationale for assessing the stability and vulnerability of the revetment, we combined guidance from the following standards:

- USACE EM 1110-2-1902 (Slope Stability Manual)
- USACE EM 1110-2-1913 (Levee Design Manual)
- WSDOT Geotechnical Design Manual (GDM)

The combined guidance from the above standards indicate the revetment should be evaluated for stability under three primary analysis cases:

- Static (steady state)

- Rapid Drawdown
- Seismic

Additional details of the analysis cases are included in Section 3.3.1. The recommended minimum factors of safety for each analysis case based on the referenced standards and the minimum factors of safety we used for this evaluation are shown in Table 4. This methodology and the minimum factors of safety used for this evaluation are consistent with other similar studies along the Cedar River Trail corridor (Shannon & Wilson, Inc, 2020)

Table 4. Summary of Minimum Factors of Safety (FS) for Revetment Stability

Analysis Case	USACE EM 1110-2-1902	USACE EM 1110-2-1913	WSDOT GDM M46-03.11	Minimum FS Criteria for this Evaluation
Static	1.3	1.4	1.25 to 1.5	1.3
Seismic	N/A	N/A	1.05 to 1.1	1.1
Rapid Drawdown	1.1 to 1.3	1.0 to 1.2	N/A	1.1

Notes:
1) N/A = not specified.

4.3.1 Stability Analysis Cases

In accordance with guidance presented in the referenced standards, we evaluated three primary analysis cases for revetment stability. Each analysis case was evaluated under river hydraulic conditions representative of existing and proposed project conditions, under both unscoured and scoured riverbed conditions. The scoured riverbed condition is based on the 100-year flood (1% ACF). Due to the geometry of the revetment that includes only a shallow and gentle to moderate landward side slope west of the trail, we did not evaluate the landward side stability of the revetment. Similarly, the groundwater levels at the revetment present strong hydraulic continuity with the river resulting in a flat groundwater surface, so steady-state seepage calculations were not conducted. Detailed description of the analysis cases, sub-cases, and key rationale are included below.

4.3.1.1 Static

This case occurs when the river levels remain at or near an identified stage long enough so that the revetment becomes saturated and a steady-state condition occurs. Due to the high permeability of the soil units at the site, steady-state conditions occur quickly, on the order of hours or less.

We evaluated the static case for both the 100-year flood (peak flood level) condition and the base flow condition. Effective shear strengths were used for all of the soil types since the static case is approximating steady-state conditions.

4.3.1.2 Seismic

This case evaluates the stability of the revetment during strong shaking from an earthquake and for instability after an earthquake if liquefaction occurs. We evaluated the seismic case considering the OBE (144-year return period) seismic event. Using guidance

from Chapter 6 of the WSDOT GDM, existing embankments that do not support or impact structures such as bridges, tunnels, and walls, do not require mitigation for seismic slope stability; therefore, we did not evaluate the revetment or SR 169 for the WSDOT/AASHTO design level earthquake for this risk assessment.

Our seismic analyses were accomplished using an inertial load representative of the design earthquake and analyzing the stability of the revetment through a pseudostatic seismic slope stability analysis. We applied a horizontal acceleration coefficient of one-half of the PGA from the OBE seismic event (Table 2) for these pseudostatic analyses.

In cases where liquefaction is likely, the shear strength of the liquefied soils will be reduced resulting in a residual strength condition and the potential for a flow failure or large deformations after an earthquake are possible. As described in Section 3.2.3, liquefaction is not anticipated for the OBE.

The base flow conditions were used to inform the river and groundwater levels for the seismic analyses. Drained or effective shear strengths were used for all of the soil types due to their free-draining nature.

4.3.1.3 Rapid Drawdown

This case represents the condition where a prolonged flood stage saturates the revetment and underlying soils. When the flood recedes, it also quickly removes the buttressing pressure of the flood waters against the waterward side of the revetment adding to the potential for unstable conditions. Rapid drawdown can also cause elevated pore-water pressures in slow-draining soils, reducing their shear strength; however, the free-draining nature of the site soils indicate that this condition is unlikely to occur.

In collaboration with Tetra Tech and the County, based on typical flood hydrographs for the river at and near the site and the free-draining characteristics of the site soils, we determined that the rapid drawdown analysis case should consider drawdown from the 100-year flood level to a water surface representing two days post-peak flood conditions.

Drained or effective shear strengths were used for all of the soil types due to their free-draining nature.

4.3.2 Analysis Methodology

Our slope stability analyses were conducted using the computer model Slide (Rocscience, 2017), which uses 2-D limit equilibrium methods to analyze slope stability. The Slide program performs slope stability computations based on the modeled cross-section conditions and calculates a factor of safety against slope failure, which is defined as the ratio of the resisting forces to the driving forces acting on a soil mass. A factor of safety of one indicates a “just-stable” condition, and a factor of safety less than one would indicate unstable conditions. Spencer’s analysis method was used as the primary analysis in Slide as it satisfies both moment and force equilibrium criteria for the potential sliding soil mass.

Within the revetment cross section stability models, failure surfaces were generated and analyzed using a dense grid-search method, automatic search function, and specific failure surface locations of interest within Slide which computes the factors of safety

against sliding corresponding to the critical failure surface and other specified surfaces for the given cross section and analysis condition. The failure surfaces generated include both circular failure surfaces and composite surfaces (combination circular and block). Only significant failure surfaces, at least 4 feet thick, were considered. Shallow failure surfaces that are less than 4 feet thick are considered maintenance issues and not a true slope failure.

For the rapid drawdown analysis case, we utilized the three-stage method as described in USACE EM 1110-2-1913 (USACE, 2000) which is available as an advanced function in the Slide software.

For all analysis cases, we considered three potential failure surface locations:

- Failure along the back of the revetment armor rock.
- Failure intersecting the riverward edge of the trail.
- Failure intersecting the riverward edge of SR 169.

Inputs required for the slope stability models include topography, bathymetry, soil units and engineering properties, river levels, groundwater conditions, and long-term scour. The selection of the analysis section locations and the model inputs are described further below. The soil units and engineering properties are discussed in Section 3.1 and shown on Table 1.

4.3.2.1 Analysis Section Locations and Rationale

We evaluated three representative cross sections of the revetment: B-B', C-C', and E-E' (Figure 2). These sections were chosen in collaboration with Tetra Tech and the County based on the topography and bathymetry at each location, to evaluate representative sections along the revetment, and in consideration of key geomorphic features such as the existing scour hole present in the river channel near the toe of the revetment at RM 13.15 and coinciding with Section B-B'.

Based on our observations and analyses, it is our opinion that Section B-B' located at approximately RM 13.15 represents the critical section of the revetment with respect to stability due to the overall steepness and height of the revetment at this location, the presence of a prominent scour hole in the river channel, and the orientation of the river flows that are directed almost perpendicularly at the revetment.

4.3.2.2 Topography and Bathymetry

The river channel shape and ground surface at each analysis section was developed using data from three sources as compiled and provided by the County. The three sources are stitched together, shown as contour lines on Figure 2, and include:

- A bathymetric survey conducted by Pacific Geomatic Services, Inc. (PGS, 2019)
- Cross section topographic surveys conducted by PGS
- 2019 LIDAR data provided by the County

4.3.2.3 Water and Groundwater Levels

Water levels at each critical section were developed for the 100-year flood, base flow conditions, and a water surface representing two days post-peak flood conditions for the rapid drawdown analysis case. We understand the 100-year flood water levels are based on a river discharge of 9,440 cubic feet per second (cfs) and the base flow water levels are based on a mean annual river discharge of 590 cfs.

The water levels are summarized in Table 5 and were provided from the project hydraulic model by Tetra Tech for the existing conditions and considering the hydraulic changes from the project alternatives. The project hydraulic model was updated and calibrated to the February flood event after our CRT7 stability analyses, resulting in slightly different reported water levels (King County, 2020b), but the values used in our analyses and shown in Table 5 are typically within 0.1 to 0.2 feet of the updated water levels and no greater than 0.4 feet different at the critical section B-B'. The water levels are presented as elevations referenced to the NAVD88 datum. The project alternatives are not projected to change the base flow water levels at the revetment. Reductions in the flood water levels for the 100-year flood are on the order of 1.5 to 2.0 feet when compared to the existing conditions anticipated for the project alternatives. Groundwater conditions extending into the revetment were assumed in hydraulic continuity with the river levels in all analyses as indicated by the findings from the Preliminary Geotechnical Data Report (County, 2020).

Table 5. Water Levels

Section B-B'			
Water Level	**Existing Conditions**	**Alternative 1**	**Alternative 2**
Base Flow	254.70	254.70	254.70
100 Year Flood	264.51	262.65	262.80
2-Days Post-Flood	262.68	260.98	261.02
Section C-C'			
Base Flow	254.70	254.70	254.70
100 Year Flood	264.39	262.58	262.73
2-Days Post-Flood	262.56	260.87	260.90
Section E-E'			
Base Flow	254.70	254.70	254.70
100 Year Flood	262.71	261.45	261.62
2-Days Post-Flood	261.18	259.75	259.78

4.3.2.4 Scour Conditions

Scour was accounted for in our slope stability evaluations to understand the vulnerability of the revetment with projected riverbed scour conditions reflecting the 100-year flood scenarios. The risk of scour and scour-related instability is highest at river bends such as the location of the revetment where scour forces are concentrated at the toe of the revetment during flood events.

The shape and configuration of the modeled scour hole was developed through evaluation with soil mechanics that accounts for the angle of repose of the riverbed materials and the dynamic and short duration nature of scour processes and bolstered by anecdotal observations of scour conditions along similar rivers in King County and through collaboration with Tetra Tech and the County. The modeled scour hole shape is an inverted triangular envelope that starts at the toe of the revetment and extends below the channel at a 45-degree angle to the recommended scour elevations.

The scour elevations were provided by Tetra Tech for the existing conditions and considering the hydraulic and geomorphic changes from the project alternatives 1 and 2. For the existing conditions, the scour elevation we used was 242.6 (NAVD 88[3]). For Alternatives 1 and 2, the scour elevations used were 244.3 and 244.2 (NAVD 88), respectively. In general, the project alternatives reduce estimated scour depth at the revetment by 1.6 to 1.7 feet.

The longitudinal extent of the scour for the existing condition was estimated by Tetra Tech to be about 310 feet and extend from about RM 13.17 to RM 13.11. For the alternatives, the longitudinal extent of the scour was estimated to be about 270 feet and extend from about RM 13.17 to RM 13.12. However, there is a high level of uncertainty in both the location and length of the scour pool so our stability analysis conservatively assumed the scoured conditions would extend uniformly across the critical sections B-B' through E-E'.

4.4 Slope Stability Analysis Results

The results of our revetment stability analyses are summarized on Tables 7 through 9 (after the text). The results include calculations for the three primary stability cases (static, seismic, and rapid drawdown) along with water and scour levels reflecting existing conditions and the project alternatives. The anticipated hydraulic and geomorphic effects of the adjacent Rutledge-Johnson project are within the bounds of those applied for the existing conditions and project alternative conditions at the revetment indicating the stability results are relatively representative of the potential Rutledge-Johnson project effects.

The result tables are color-coded with green cells showing the analysis cases, locations, and conditions that meet our recommended stability criteria (Table 4), orange cells showing the analysis cases, locations, and conditions that do not meet our recommended stability criteria, and red cells showing the analysis cases, locations, and conditions that are projecting failure (factor of safety of less than 1.0).

[3] North American Vertical Datum of 1988

We've identified Section B-B' as the critical section of the revetment and detailed graphical outputs of our slope stability model and results for all analysis cases and conditions are included in Appendix D.

5 Conclusions

5.1 Revetment Stability

Overall, the revetment is tall, steep, and composed primarily of loose fill soils creating a baseline condition of marginal stability. The stability of the revetment varies depending on the three potential failure surfaces considered as summarized in Table 6 and presented in detail in Tables 7 through 9 (after the text). The results across the three cross sections analyzed (Figure 2) are similar; therefore, the conclusions described below are applicable to the entirety of the revetment:

Table 6. Revetment Stability Summary

Potential Failure Surface Location	Summary of Results
Along back of revetment armor rock	Unstable for all cases and conditions, except rapid drawdown cases under non-scoured conditions.
Intersecting riverward edge of the trail	Generally stable under non-scoured conditions, but the number of unstable cases increase under potential scour conditions.
Intersecting the riverward edge of SR 169	Stable for all cases and conditions.

A workshop was held on June 4, 2020 to discuss these conclusions and risk implications to the revetment, a copy of the workshop minutes is attached in Appendix E.

5.2 Effects of Project Alternatives

The project alternatives result in minor reductions in the water levels and predicted scour depths at the revetment, on the order of 2 feet. The potential effects of the adjacent Rutledge-Johnson project are not fully known but anticipated to be similar to the project alternatives. These changes do not reduce the stability of the revetment when compared to existing conditions nor do they significantly improve the stability of the revetment. The revetment, when considering the effects of both project alternatives, has minor changes in the calculated factors of safety but no change with respect to meeting the recommended stability criteria (Table 4).

5.3 Risk

A summary of our characterization of the infrastructure risk, due to revetment instability is as follows:

- The likelihood of instability or failure of the armor rock and waterward side revetment is high. The consequence of this type of failure is the exposure of the underlying revetment soils to additional bank scour during subsequent flood events and a progressive failure that could compromise the trail. A progressive

failure is unlikely to manifest rapidly and unlikely to present a hazard to the life-safety of trail users. The trail is not a vital transportation corridor, but it is a secondary regional multi-modal path. The revetment and trail do not contain other critical infrastructure like significant utilities. Therefore, the risk of progressive instability or failure of the trail is moderate. The risk of a progressive failure of SR 169 is low.

- The likelihood of instability or failure of the trail outside of the progressive failure scenario described above is moderate. A sudden failure of the trail could pose a hazard to the life-safety of trail users. The trail is not a vital transportation corridor, but it is a secondary regional multi-modal path and the revetment and trail do not contain other critical infrastructure like significant utilities. Therefore, the risk of instability or failure of the trail is moderate.
- The likelihood of instability or failure of SR 169 is very low. SR 169 is a vital transportation corridor and sudden failure of SR 169 would pose a significant hazard to the life-safety of the highway users. However, due to the very low likelihood of this scenario occurring, the risk of instability or failure of SR 169 is very low.
- The risks described above are not significantly changed by the project alternatives or what is currently known about the adjacent Rutledge-Johnson project.
- Over the long term (20+ years), channel migration is likely to result in one of two planform configurations, as discussed in Section 3.2. Anticipated changes to the vulnerability of the revetment and associated infrastructure risk are different for the two estimated planform conditions:
    - The river could migrate into the right floodplain, encroach against the new setback levee and direct the main river channel back toward the revetment. This will likely increase flows and shear stresses on the revetment, thereby potentially increasing the vulnerability of the revetment and the related risk elements described above.
    - The river could avulse into the new primary side channel on the right floodplain and likely further reduce flows and shear stresses on the revetment; thereby potentially reducing the vulnerability of the revetment and the related risk elements described above.

5.4 Risk Reduction Concepts

Risk related to revetment instability could be reduced through various methods. Additional technical investigations and analyses would be required to further define the feasibility and benefits of the methods described below.

The stability of the revetment could be improved through reactive or proactive approaches. A reactive approach would be to continue standard inspections/monitoring of the revetment, and plan to repair the revetment upon failure. Proactive approaches to stabilize the revetment could include:

- Install buttressing along the toe of the revetment.
- Regrade the bank slope to a flatter angle (2H:1V).
- Place large wood within the main channel to redirect river flows away from and limit channel migration towards critical infrastructure and reduce impacts to existing habitat.

6 References

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

Atwater, B.F., S. Musumi-Rokkaku, D. Satake, Y. Tsuji, K. Ueda, and D.K. Yamaguci (Atwater et al.), 2015, The orphan tsunami of 1700—Japanese clues to a parent earthquake in North America, U.S. Geological Survey, Professional Paper 1707.

Booth, D.B., 1995, Surficial Geologic Map of the Maple Valley Quadrangle, King County, Washington, U.S. Geological Survey, Miscellaneous Field Studies Map MF-2297, Scale 1:24,000.

Holtz and Kovacs, 1981, An Introduction to Geotechnical Engineering, Prentice-Hall, Inc.

King County, 2020a, Preliminary Geotechnical Data Report – Revision No. 1 Jan Road Neighborhood Improvements Project, April 21, 2020.

King County, 2020b, Hydraulic Analysis of Alternatives, Jan Road Neighborhood Improvements Project, prepared by Tetra Tech, Inc. for King County Department of Natural Resources and Parks Water and Land Resources Division, Seattle, WA.

King County, 2020c, Existing Conditions Geomorphic Assessment Report, Jan Road Neighborhood Improvements Project, prepared by Watershed GeoDynamics for King County Department of Natural Resources and Parks Water and Land Resources Division, Seattle, WA.

Kramer, S., 2008, Evaluation of Liquefaction Hazards in Washington State, prepared for the Washington State Transportation Commission.

Massman, J., 2003, A Design Manual for Sizing Infiltration Ponds, October 2003.

Navy Facilities Engineering Command (NAVFAC), 1986, Soil Mechanics, Design Manual 7.01. September 1, 1986.

Pacific Geomatic Services, Inc., 2019, Topographic and Bathymetric Survey, Jan Road Geotech Profiles, September 25, 2019 [AutoCAD survey file].

Pratt, T.L., K.G. Troost, J.K. Odum, and W.J. Stephenson (Pratt et al.), 2015, Kinematics of shallow backthrusts in the Seattle fault zone, Washington State, Geosphere, v. 11, no. 6, p. 1–27, doi:10.1130/GES01179.1.

Rocscience, 2017, Slide 7.024 Analysis Program. May 2017.

Shannon & Wilson, Inc., 2020, Geotechnical Engineering Report, Cedar River Trail Site 2 Revetment Geotechnical Design Support, King County, Washington, dated July 30, 2020.

United States Army Corps of Engineers (USACE), 2000, Design and Construction of Levees, USACE Engineering Manual EM 1110-2-1913, dated April 30, 2000.

United States Army Corps of Engineers (USACE), 2003, Design Guidance for Slope Stability, USACE Engineering Manual ETL 1110-2-1902, dated October 31, 2003.

United States Army Corps of Engineers (USACE), 2016, Earthquake Design and Evaluation for Civil Works Projects, USACE Engineering Regulation ER 1110-2-1806, dated May 31, 2016.

United States Geological Survey (USGS), 2014, Unified Hazard Tool, https://earthquake.usgs.gov/hazards/interactive/

United States Geological Survey (USGS), 2019, Nisqually EQ Shakemap, https://earthquake.usgs.gov/earthquakes/eventpage/uw10530748/map?shakemap-code=atlas20010228185432&shakemap-source=atlas&shakemap-pga=true&shakemap-stations=true&shakemap-mmi-contours=false, accessed November, 2019.

Washington State Department of Natural Resources (DNR), 2004, Liquefaction Susceptibility and Site Class Maps of Washington State, By County, Washington Division of Geology and Earth Resources Open File Report 2004-20, by Palmer, S.P., S.L. Magsino, E.L. Bilderback, J.L. Poelstra, D.S. Folger, and R.A. Niggemann, 2004, September 2004.

Washington State Department of Natural Resources (DNR), 2019, Washington Geologic Information Portal, Accessed September 12, 2019 from https://geologyportal.dnr.wa.gov/.

Washington State Department of Transportation (WSDOT), 2019, Geotechnical Design Manual M 46-03.

7 Limitations

Work for this project was performed for Tetra Tech, Inc. and King County Department of Natural Resources and Parks, Water, and Land Resources Division, River and Floodplain Management Section (Client), and 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 F 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 Henry H. Haselton, PE, PMP, Principal Geotechnical Engineer, (206) 838-5852.

TABLES

Legend	
(red)	FS < 1
(orange)	FS < Recommended Minimum
(green)	FS >= Recommended Minimum

Table 7.1: Cross Section B-B', Existing Conditions

				Factor of Safety, Non-Scoured Conditions			Factor of Safety with Future Scour**		
Analysis Case	Water Level Condition	Water Level Elevation	Recommended minimum Fs	Along back of revetment	Edge of trail	Edge of SR 169	Along back of revetment	Edge of trail	Edge of SR 169
Static Conditions	Long-Term Base Flow	254.70	1.3	1.18	1.32	3.50	0.85	1.19	3.48
Seismic Conditions*	Long-Term Base Flow	254.70	1.1	0.99	1.08	2.34	0.68	0.97	2.33
Peak Flood Level	100-year flood	264.51	1.3	1.23	1.32	3.35	0.94	1.19	3.33
Rapid Drawdown	100-year flood to 2-day post-flood	264.51 -> 262.68	1.1	1.20	1.31	3.38	0.92	1.19	3.37

*The OBE seismic event.

**Scour elevation = 242.6

Table 7.2: Cross Section B-B', Alternative 1

				Factor of Safety, Non-Scoured Conditions			Factor of Safety with Future Scour**		
Analysis Case	Water Level Condition	Water Level Elevation	Recommended minimum Fs	Along back of revetment	Edge of trail	Edge of SR 169	Along back of revetment	Edge of trail	Edge of SR 169
Static Conditions	Long-Term Base Flow	254.70	1.3	1.18	1.32	3.49	0.92	1.16	3.48
Seismic Conditions*	Long-Term Base Flow	254.70	1.1	0.99	1.08	2.34	0.76	0.94	2.33
Peak Flood Level	100-year flood	262.65	1.3	1.21	1.30	3.32	1.00	1.15	3.30
Rapid Drawdown	100-year flood to 2-day post-flood	262.65 -> 260.98	1.1	1.18	1.31	3.38	0.98	1.16	3.37

*The OBE seismic event.

**Scour elevation = 244.3

Table 7.3: Cross Section B-B', Alternative 2

				Factor of Safety, Non-Scoured Conditions			Factor of Safety with Future Scour**		
Analysis Case	Water Level Condition	Water Level Elevation	Recommended minimum Fs	Along back of revetment	Edge of trail	Edge of SR 169	Along back of revetment	Edge of trail	Edge of SR 169
Static Conditions	Long-Term Base Flow	254.70	1.3	1.18	1.33	3.49	0.91	1.19	3.48
Seismic Conditions*	Long-Term Base Flow	254.70	1.1	0.99	1.08	2.34	0.74	0.97	2.33
Peak Flood Level	100-year flood	262.80	1.3	1.21	1.32	3.33	0.97	1.18	3.31
Rapid Drawdown	100-year flood to 2-day post-flood	262.8 -> 261.02	1.1	1.18	1.32	3.38	0.94	1.18	3.37

*The OBE seismic event.

**Scour elevation = 244.2

Legend	
(red)	FS < 1
(orange)	FS < Recommended Minimum
(green)	FS >= Recommended Minimum

Table 8.1: Cross Section C-C', Existing Conditions

				Factor of Safety, Non-Scoured Conditions			Factor of Safety with Future Scour**		
Analysis Case	Water Level Condition	Water Level Elevation	Recommended minimum Fs	Along back of revetment	Edge of trail	Edge of SR 169	Along back of revetment	Edge of trail	Edge of SR 169
Static Conditions	Long-Term Base Flow	254.70	1.3	1.15	1.22	3.71	0.96	1.17	3.78
Seismic Conditions*	Long-Term Base Flow	254.70	1.1	0.97	1.00	2.39	0.79	0.97	2.45
Peak Flood Level	100-year flood	264.39	1.3	1.19	1.22	3.71	1.07	1.17	3.82
Rapid Drawdown	100-year flood to 2-day post-flood	264.39 -> 262.56	1.1	1.16	1.21	3.71	1.05	1.17	3.81

*The OBE seismic event.

**Scour elevation = 242.6

Table 8.2: Cross Section C-C', Alternative 1

				Factor of Safety, Non-Scoured Conditions			Factor of Safety with Future Scour**		
Analysis Case	Water Level Condition	Water Level Elevation	Recommended minimum Fs	Along back of revetment	Edge of trail	Edge of SR 169	Along back of revetment	Edge of trail	Edge of SR 169
Static Conditions	Long-Term Base Flow	254.70	1.3	1.15	1.21	3.86	1.01	1.15	3.78
Seismic Conditions*	Long-Term Base Flow	254.70	1.1	0.97	1.00	2.49	0.80	0.94	2.45
Peak Flood Level	100-year flood	262.58	1.3	1.16	1.19	3.85	1.12	1.15	3.75
Rapid Drawdown	100-year flood to 2-day post-flood	262.58 -> 260.87	1.1	1.14	1.20	3.88	1.08	1.15	3.37

*The OBE seismic event.

**Scour elevation = 244.3

Table 8.3: Cross Section C-C', Alternative 2

				Factor of Safety, Non-Scoured Conditions			Factor of Safety with Future Scour**		
Analysis Case	Water Level Condition	Water Level Elevation	Recommended minimum Fs	Along back of revetment	Edge of trail	Edge of SR 169	Along back of revetment	Edge of trail	Edge of SR 169
Static Conditions	Long-Term Base Flow	254.70	1.3	1.15	1.23	3.86	0.99	1.07	3.77
Seismic Conditions*	Long-Term Base Flow	254.70	1.1	0.97	1.01	2.49	0.81	0.87	2.45
Peak Flood Level	100-year flood	262.73	1.3	1.16	1.21	3.85	1.11	1.08	3.76
Rapid Drawdown	100-year flood to 2-day post-flood	262.73 -> 260.9	1.1	1.14	1.22	3.88	1.07	1.07	3.78

*The OBE seismic event.

**Scour elevation = 244.2

Legend	
(red)	FS < 1
(orange)	FS < Recommended Minimum
(green)	FS >= Recommended Minimum

Table 9.1: Cross Section E-E', Existing Conditions

				Factor of Safety, Non-Scoured Conditions			Factor of Safety with Future Scour**		
Analysis Case	Water Level Condition	Water Level Elevation	Recommended minimum Fs	Along back of revetment	Edge of trail	Edge of SR 169	Along back of revetment	Edge of trail	Edge of SR 169
Static Conditions	Long-Term Base Flow	254.70	1.3	1.24	1.32	5.04	1.00	1.24	4.85
Seismic Conditions*	Long-Term Base Flow	254.70	1.1	1.04	1.09	2.97	0.80	1.01	2.92
Peak Flood Level	100-year flood	262.71	1.3	0.96	1.03	2.77	1.08	1.23	5.09
Rapid Drawdown	100-year flood to 2-day post-flood	262.71 -> 261.18	1.1	1.18	1.30	5.22	1.06	1.23	5.02

*The OBE seismic event.

**Scour elevation = 242.6

Table 9.2: Cross Section E-E', Alternative 1

				Factor of Safety, Non-Scoured Conditions			Factor of Safety with Future Scour**		
Analysis Case	Water Level Condition	Water Level Elevation	Recommended minimum Fs	Along back of revetment	Edge of trail	Edge of SR 169	Along back of revetment	Edge of trail	Edge of SR 169
Static Conditions	Long-Term Base Flow	254.70	1.3	1.21	1.32	5.05	1.00	1.20	4.85
Seismic Conditions*	Long-Term Base Flow	254.70	1.1	1.02	1.09	2.99	0.83	0.97	2.92
Peak Flood Level	100-year flood	261.45	1.3	1.17	1.29	5.21	1.06	1.19	4.99
Rapid Drawdown	100-year flood to 2-day post-flood	261.45 -> 259.75	1.1	1.18	1.29	5.17	1.04	1.18	4.94

*The OBE seismic event.

**Scour elevation = 244.3

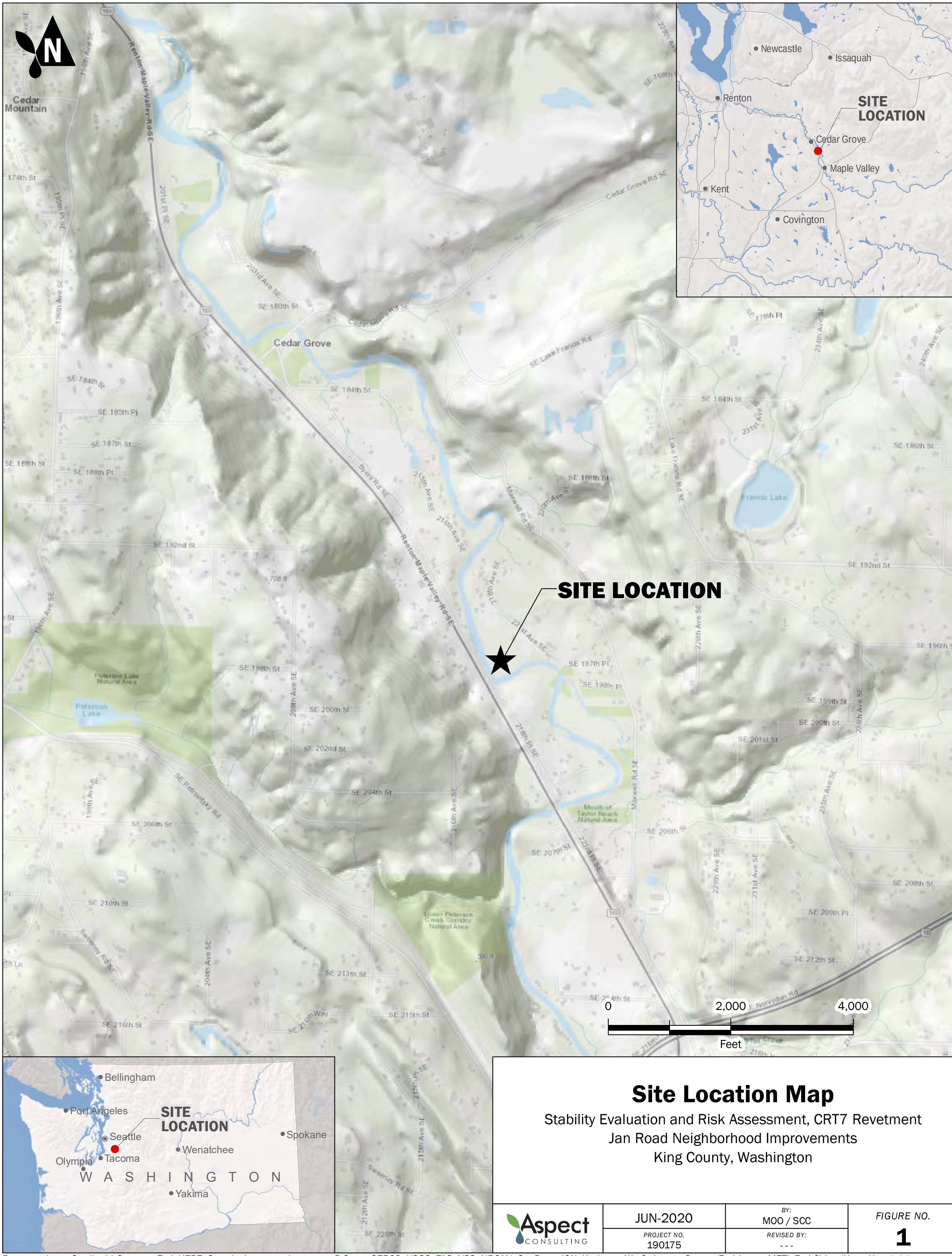
Table 9.3: Cross Section E-E', Alternative 2

				Factor of Safety, Non-Scoured Conditions			Factor of Safety with Future Scour**		
Analysis Case	Water Level Condition	Water Level Elevation	Recommended minimum Fs	Along back of revetment	Edge of trail	Edge of SR 169	Along back of revetment	Edge of trail	Edge of SR 169
Static Conditions	Long-Term Base Flow	254.70	1.3	1.21	1.39	5.05	1.00	1.23	4.85
Seismic Conditions*	Long-Term Base Flow	254.70	1.1	1.02	1.15	2.99	0.81	1.00	2.92
Peak Flood Level	100-year flood	261.62	1.3	1.17	1.32	5.22	1.04	1.21	5.00
Rapid Drawdown	100-year flood to 2-day post-flood	261.62 -> 259.78	1.1	1.18	1.33	5.17	1.02	1.20	4.95

*The OBE seismic event.

**Scour elevation = 244.2

FIGURES



Site Location Map

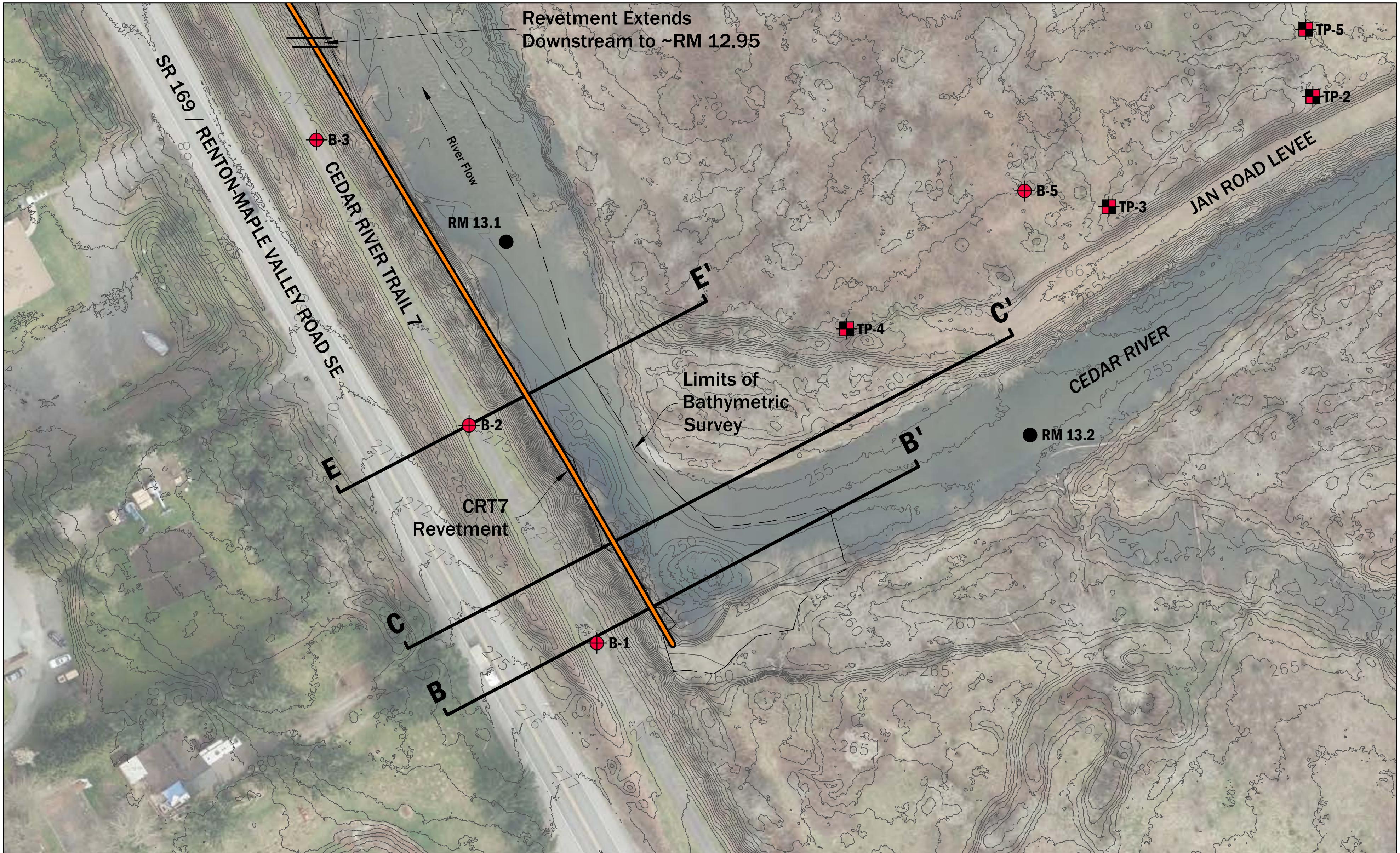
Stability Evaluation and Risk Assessment, CRT7 Revetment
Jan Road Neighborhood Improvements
King County, Washington

Aspect CONSULTING

JUN-2020	BY: MOO / SCC	FIGURE NO.
PROJECT NO. 190175	REVISED BY: - - -	1

GIS Path: T:\projects_8\KingCounty_JanRoadNeighborhoodLevee_190175\Delivered\Stability Evaluation and Risk Assessment, CRT7 Revetment\01 Site Location Map.mxd || Coordinate System: NAD 1983 StatePlane Washington North FIPS 4601 Feet || Date Saved: 6/18/2020 || User: bgrimm || Print Date: 6/18/2020

Basemap Layer Credits || 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

Boring Location (King County, 2019)

Test Pit Location (King County, 2019)

RM 13.1 River Mile Marker

B B' Cross Section Locations for Stability Analyses

Source: Pacific Geomatic Services, Inc., *2019, Topographic and Bathymetric Survey, Jan Road Geotech Profiles*, September 25, 2019. Aerial Imagery provided by King County, 2020.

0 100 200 Feet

Site and Exploration Map

Stability Evaluation and Risk Assessment, CRT7 Revetment
Jan Road Neighborhood Improvements
King County, Washington

Aspect CONSULTING

Jul-2020	BY: MOO/SCC	FIGURE NO.
PROJECT NO. 190175	REVISED BY: SCC	2

CAD Path: Q:\King County\190175 Jan Road Neighborhood Levee\King County CAD Sheets\190175-02 (Jan Road - Geotech Profiles).dwg 02 Site and Exploration Map Letter || Date Saved: Jul 28, 2020 4:07pm || User: scudd

APPENDIX A

Preliminary Geotechnical Data Report, King County, 2020

King County

Department of Local Services
Road Services Division
Materials Laboratory

April 21, 2020

TO: Dan Heckendorf, P.E., Engineer III, River and Floodplain Management Section, Water and Land Resources Division, KCDNRP

VIA: Douglas T. Walters, P.E., Interim Materials Engineer, Drainage, Survey, Utility and Materials Group, Engineering Services Section, KCDLS

FM: Timothy R. Hyden, Engineer III, Drainage, Survey, Utility and Materials Group, Engineering Services Section, KCDLS

RE: Preliminary Geotechnical Data Report – Revision No. 1
Jan Road Neighborhood Improvements Project

1.0 INTRODUCTION

This report presents results from our geotechnical investigation of the referenced site and provides a preliminary characterization of site soil and groundwater conditions for the Jan Road Neighborhood Improvements Project (Project). Work performed includes literature review, subsurface exploration by drilling and heavy equipment excavation, installation of a groundwater monitoring well, laboratory testing of select soil samples and a description of subsurface soil and groundwater conditions.

We previously submitted a Preliminary Geotechnical Data Report dated September 24, 2019. This revised report (Revision No. 1) was prepared to incorporate information from additional subsurface exploration completed along the existing levee, proposed levee extension and the existing floodplain. The additional exploration included test pits (TP-8 through TP-12) and hand auger excavations (AHA-1 and AHA-2), located upstream and adjacent to the area previously explored.

2.0 PROJECT DESCRIPTION/SUMMARY

The project site is located adjacent to State Route 169 (SR 169), and includes a section of the King County Parks Cedar River Trail (Trail) and multiple parcels owned by either King County or private citizens. The total site area is roughly 20 acres. A Site Vicinity Map is attached as Figure 1.

The floodplain area of the site is generally flat and slopes gently to the northwest along the Cedar River valley. The Cedar River Trail Site 7 (CRT 7) Revetment (revetment), located on the southwest side (left bank) of the Cedar River and parallel

to SR 169, provides flood protection to the trail and SR 169 (See Figure 2). Along the revetment, the trail is roughly 25 to 30 feet above the thalweg elevation of the Cedar River. Jan Road Levee provides flood protection and is located on the north side (right bank) of the Cedar River. The top of the levee is estimated to be 13 feet above the Cedar River thalweg elevation. Side slopes of the revetment and levee are estimated to be about 1.5 Horizontal:1.0 Vertical.

Proposed improvements within the exploration area may include:

- Relocating the existing levee by setting it back away from the river channel;
- Excavation of side channels through the right bank floodplain to increase flood conveyance;
- Installation of engineered log jams or similar structures for wildlife habitat and flow control;
- Extending the levee a few hundred feet upstream to improve flood protection;
- Roadway and drainage upgrades along portions of 197th Avenue SE, 218th Avenue SE, and 221st Avenue SE.

Although bank stabilization efforts along the CRT 7 Revetment are not currently planned, a risk assessment to evaluate its current vulnerability is scheduled as part of the project.

3.0 <u>GEOLOGIC HAZARDS</u>

An internet review of King County iMAP and the Washington State DNR Geologic information portal was performed to determine if geologic hazards are present on or adjacent to the site.

No fault trenches or seismogenic folds or faults are mapped by DNR in the site vicinity. However, the entire project site area is designated by iMAP and/or DNR as a Seismic Hazard with a moderate to high susceptibility of liquefaction.

Within the project confines, revetment slopes from the Cedar River Trail down to the Cedar River and sections of the levee are designated by iMAP as Potential Landslide Hazard areas and/or Potential Steep Slope Hazard areas. In addition, upland slopes outside the project area on the north and south sides of the Cedar River are Landslide Hazard areas, Steep Slope Hazard areas and Erosion Hazards. iMAP identifies the Cedar River channel and portions of the floodplain area behind the levee as moderate to severe risk Channel Migration Hazard Areas.

Literature review was performed for preliminary identification of geologic hazards on and adjacent to the site. The presence of Geologic hazards may trigger specific requirements from regulatory authorities, potentially having an impact on design, permitting and/or construction. Identification of specific impacts was beyond the

scope of our investigation and should be further evaluated as project design progresses.

4.0 GEOLOGIC MAPPING

We reviewed the 1995 USGS Geologic Map of the Maple Valley Quadrangle to identify surficial geologic units near the site. The portion of the site associated with our initial report was located within the geologic unit Younger Alluvium (Qyal). The expanded site area now includes the adjacent geologic unit Wetland Deposits (Qw), further upstream and to the south. Geologic units on the site are described as follows:

> Younger Alluvium – Holocene (*Qyal*): Moderately sorted deposits of cobble gravel, pebbly sand, and sandy silt along major rivers and stream channels.
>
> Wetland Deposits – Holocene (*Qw*): Peat and alluvium that are poorly drained and intermittently wet annually. Grades into unit Qyal.

Two other geologic units are located adjacent to the site, but outside the areas of our investigation. Alluvial Fan Deposits (Qf) and Older Alluvium (Qoal) are located along the southwest and northeast sides of the site, respectively, and described as follows:

> Alluvial Fan Deposits – Holocene (*Qf*): Boulders, cobbles and sand deposited in lobate form where streams emerge from confining valleys and reduced gradients cause sediment loads to be deposited.
>
> Older Alluvium – Holocene and Pleistocene (*Qoal*): Texturally equivalent to unit Qyal but deposits lie at higher elevations and typically have greater relief than younger alluvium.

5.0 SUBSURFACE EXPLORATION

Contractors completed a series of six borings and sixteen backhoe/trackhoe test pits under the direction of representatives from our office. In addition, two excavations were hand-augered by Aspect Consulting, LLC (Aspect), geotechnical consultant to the King County River and Floodplain Management Section (RFMS) for this project. Boring, test pit and hand-augered excavation locations were selected by RFMS and/or Aspect, and are shown on the attached Figure 2 – Subsurface Exploration Location Plan.

Contractors licensed by the State of Washington and under contract to King County completed the borings (Holocene Drilling) and test pits (Jensen Excavating and Rainier Wood Recyclers). Prior to beginning work, excavation areas were determined to be clear of underground utilities by the Washington Utility Notification Center and a private utility locating company under contract to King County, Applied Professional Services, Inc. In addition, Holocene Drilling notified the Washington

State Department of Ecology (DOE) of their planned work activities prior to drilling and obtained the required start-cards.

Subsurface exploration began on May 10, 2019 and continued periodically through May 28, 2019 for all test borings and test pits TP-1 through TP-7. On March 6, 2020, test pits TP-8 through TP-12 and hand auger excavations AHA-1 and AHA-2 were completed. A cultural resources consultant, contracted through RFMS, was on site during subsurface exploration.

Representatives from our office coordinated and observed drilling and test pit excavation activities. Logs from our borings and text pits are attached as Appendix A. Our Logs describe exploration methods, soil types, groundwater conditions, sample methods, sample locations, sample intervals, and other pertinent information. Soils descriptions generally follow the Unified Soil Classification System (ASTM D-2488 - Standard Practice for Description and Identification of Soils).

An Engineer with Aspect performed the exploration for hand auger excavations AHA-1 and AHA-2. A memorandum from Aspect dated April 3, 2020 is attached as Appendix B. Their memorandum describes the hand auger excavations and subsurface soil profile, and includes Logs from their exploration.

5.1 Test Pit Excavations

Test pit excavations were completed at the top of the levee embankment, at the levee embankment toe and within the right bank floodplain (See Figure 2). Test pits TP-1 through TP-7 were excavated using a John Deere 410D rubber-tired backhoe and test pits TP-8 through TP-12 were excavated using a Takeuchi TB235 mini-excavator. Both excavating buckets had smooth faces and were 2 to 3 feet wide. The test pit excavations were approximately 6 to 8 feet in length and ranged in depth from 4 to 8 feet below the ground surface (bgs).

Bulk or grab samples were obtained at select intervals, typically when there was a change in materials, to aid in characterization of the soil profile. Samples were sealed in plastic bags and returned to our laboratory for further evaluation and/or testing.

5.2 Geotechnical Drilling and Sampling

Borings B-1 through B-3 were completed on the Trail adjacent to SR 169. Due to difficulties with heave inside the auger and auger refusal, three borings were attempted at the B-1 location (B-1, B-1a and B-1b). The B-1 boring log provided in Appendix A includes a combination of field and lab soil classifications and Standard Penetration Testing (SPT) blow counts selected from these three boring attempts. The B-1 boring log in Appendix A incorporates information from the following:

- Boring B-1 from the ground surface to a depth of 20 feet;
- Boring B-1a from 20 feet to 51.5 feet using hollow-stem and mud rotary; and
- Boring B-1b sonic samples between 6.5' to 10' and 31' to 35'.

Borings B-4 through B-6 were located in the floodplain on the north side of the Cedar River (See Figure 2). In addition, Boring B-5 was completed as a nominal 2-inch diameter monitoring well (Well Tag I.D. BLT729) from the ground surface to a depth of 15 feet bgs.

Diedrich D50 track-mounted drill rigs were used to complete borings B-1, B-4, B-5 and B-6 in general conformance with ASTM D-6151 (Standard Practice for Using Hollow-Stem Augers for Geotechnical Exploration and Soil Sampling). Borings B-1 and B-5 were advanced using nominal 4.25-inch diameter hollow-stem auger, except mud rotary drilling was used to complete boring B-1a from 35 feet bgs to the termination depth at 51.5 feet bgs. Borings B-4 and B-6 were advanced using nominal 3.25-inch I.D. hollow-stem auger.

Borings B-2 and B-3 were completed using a Geoprobe 8140 LC Sonic track-mounted drill rig based on procedures outlined by ASTM D-6914 (Standard Practice for Sonic Drilling for Site Characterization and the Installation of Subsurface Monitoring Devices). The borings were advanced using a series of casing ranging from approximately 4 inches to 10 inches in diameter.

SPT tests based on ASTM D-1586 (Standard Test Method for Standard Penetration Test and Split-Barrel Sampling of Soils) were performed at 2.5' or 5' intervals throughout the borings. SPT blow counts and the amount of sample retrieved is noted on the Boring Logs. Soil samples were visually classified, placed in sealed plastic bags and stored for further evaluation and laboratory testing.

Reports of efficiency rating for the hammer used to drive SPT samples were provided by Holocene Drilling. Hammer efficiency testing was conducted by Robert Miner Dynamic Testing, Inc. based on ASTM D-4633 (Standard Test Method for Energy Measurement for Dynamic Penetrometers). Hammer efficiency test results are summarized below in Table 1:

TABLE 1

Boring No.	Drill Rig Type	Drill Rig No.	Efficiency Rating
B-1	Diedrich D-50	No. 95	93%
B-2 and B-3	Geoprobe 8140 LC Sonic	No. 80	96%
B-4, B-5 and B-6	Diedrich D-50	No. 26	89%

6.0 GROUNDWATER

Groundwater was encountered in all borings, the two test pits and one hand auger excavation for extension of the levee, and all but one of the floodplain test pits. Where the levee slope toe intercepts the floodplain, groundwater was encountered in two of the four test pit locations. Groundwater was not encountered in the levee embankment test pits and one of the hand auger excavations. Table 2 summarizes the depths below the ground surface to groundwater:

TABLE 2		
Exploration No.	**Location Description**	**Groundwater Depth (bgs)**
B-1	Trail	22.0’
B-2 and B-3	Trail	21.0’
B-4, B-5 and B-6	Floodplain	7.5’
TP-1, TP-2, TP-3 and TP-4	Levee Embankment	None
TP-1 and TP-4	Levee Toe	None
TP-2	Levee Toe	4.0’
TP-3	Levee Toe	6.5’
TP-5	Floodplain	7.0’
TP-6 and TP-7	Floodplain	4.8’
TP-8	Floodplain	None
TP-9	Floodplain	7.8’
TP-10	Levee Embankment	None
TP-11	Levee Extension	5.2’
TP-12	Levee Extension	5.0’
AHA-1	Levee Extension	0.5’ (Seep)
AHA-2	Levee Extension	None

7.0 LABORATORY TESTING

Laboratory testing was performed on 27 samples obtained during exploratory drilling and test pit excavations. Samples for testing were selected by Aspect and/or Tetra Tech, through direction from RFMS staff regarding conceptual design feature alignment and profile. Laboratory test reports are provided in Appendix C.

Testing was performed in general accordance with AASHTO T27 – Procedure A (Standard Method of Test for Sieve Analysis of Fine and Coarse Aggregates) and AASHTO T11 (Standard Method of Test for Materials Finer than 75-um (No. 200) Sieve in Mineral Aggregate by Washing). However, the mass of some test samples was less than the minimum specified by the AASHTO test methods due to the limited amount of available material. In some of the samples, the amount of fine soil coating gravel particles (plus ¼ inch or No. 4 screen size) was negligible after initial sieving. Therefore, the gravel fraction of the sample was not washed over the No. 200 sieve.

8.0 SUBSURFACE SOIL PROFILE

8.1 CRT 7 Revetment

The trail is surfaced with approximately 2 inches of asphalt over 4 inches of crushed surfacing along the Revetment. Below the pavement section, soils consist of fill to a depth of approximately 20 feet bgs. The fill soils are in a loose to medium dense state and were likely placed with minimal controls for material type and compaction. The fill soils predominantly consist of mixed sands and gravels with varying amounts of cobbles and a relatively low percentage of material finer than a No. 200 mesh sieve. However, occasional layers of silty sand were encountered.

Groundwater was encountered at depths of 21 to 22 feet bgs where soils interpreted as naturally deposited alluvium are present. Wood debris was encountered in one of the borings (B-3) at the transition to naturally deposited soils. Between the depths of about 20 and approximately 30 to 35 feet bgs, a mixture of medium dense to very dense sands and gravels with cobbles are present. From about 35 feet bgs to the boring termination depths (between 51.5 and 70 feet bgs), the soils were typically very dense poorly or well-graded sands and sands with silt. The soils transitioned from being a mixture of tan, reddish brown and brown in color to gray between the depths of about 45, 23 and 30 feet in borings B-1, B-2 and B-3, respectively.

8.2 Jan Road Levee

8.2.1 Jan Road Levee Embankment

Four test pits (TP-1 Levee, TP-2 Levee, TP-3 Levee and TP-4 Levee) were excavated by scraping material along the landward slope from the top of the levee embankment down to the toe. One test pit (TP-10) was excavated parallel with the levee alignment, at the top of the embankment and adjacent to the intercept with the outside slope. The change in elevation between the top of levee and slope toe ranged from about 4 to 8 feet. All of the soils within the levee embankment were classified as fill, appear to have been loosely deposited and may have been generated by dredging material from the river channel.

The levee surface was covered with a mantling of silty sand with gravel to well graded sand with silt that ranged in thickness from about 0.25 to 1.0 feet. Fine roots were present to a depth of about 0.5 to 1.5 feet bgs. The material was slightly moist and the excavations readily caved below the mantling of surface soils containing more than a trace of silt. Throughout the depth of exploration, the soils consisted of poorly and well-graded gravels with varying quantities of sand and cobbles. No groundwater was encountered in the test pit excavations for the Levee Embankment.

8.2.2 Jan Road Levee Toe

Four test pits were excavated at the slope toe along the landward side of the levee (test pits TP-1 Toe, TP-2 Toe, TP-3 Toe and TP-4 Toe). The test pit excavations extended to depths between 5 and 8 feet bgs.

The surficial soil layer consisted of silty sand or sand with silt and varying amounts of gravel and cobbles to depths of 1.5 to 2.5 feet bgs. Debris was encountered in TP-1 Toe and TP-3 Toe at depths of 1.5 and 2.5 feet bgs, respectively. Fine roots from low lying vegetation and tree roots up to about 3-inches in diameter were present throughout most of the surficial soil layers.

Below the surficial layer, a layer of poorly graded sand with varying amounts of gravel, cobbles and occasional boulders was encountered between the depths of 2.5 to 5.0 feet bgs in TP-3 Toe and 1.5 to 3.0 feet bgs in TP-4 Toe.

Well-graded gravel with varying amounts of sand, cobbles and occasional boulders was present below the silty sand in TP-1 Toe and TP-2 Toe, and below the poorly graded sand in TP-3 Toe and TP-4 Toe. The well-graded gravel continued to the bottom of the test pits.

Soils throughout the test pits were relatively easy to excavate and considered to be in a loose to medium dense state. Soils became more difficult to excavate with depth due to the type of bucket, caving and, in general, an increasing cobble content. The soils were in a moist condition and became saturated below the groundwater at depths of 4 and 6.5 feet bgs, respectively, in TP-2 Toe and TP-3 Toe. Test pit sidewalls typically began to cave below the root zones at 1.5 to 2.5 feet bgs.

8.3 <u>Floodplain</u>

Exploration in the floodplain included five test pits (TP-5, TP-6, TP-7, TP-8 and TP-9) and three borings (B-4, B-5 and B-6).

8.3.1 Floodplain Test Pits (TP-5, TP-6, TP-7, TP-8 and TP-9)

The five test pits excavated in the existing floodplain extended from the ground surface to depths between 5.0 and 8.0 feet bgs. At the TP-5, TP-7 and TP-9 locations, silty sand, sometimes containing gravel and cobbles, was present from the ground surface to depths of 0.25 to 2.5 feet bgs. Poorly graded sands underlie the silty sand in TP-5 and TP-7 to depths of 2.0 to 4.75 feet bgs. Silty gravel soils were present at the ground surface and extended to depths of up to 2.5 feet bgs in TP-6 and TP-8, except a layer of sand with silt was present in TP-8 between 1.0 to 2.25 feet bgs. Poorly and well-graded gravels were present in all five of the floodplain test pits. The gravels began at depths between 2.0 and 4.25 feet bgs and extended to the test pit termination depths between 5 and 8 feet bgs.

Excavation was relatively easy and the soils were in a loose to medium dense state. The flat tip/face of the backhoe/trackhoe bucket made excavation more difficult when cobbles were more prevalent or small boulders present. Soils were in a moist condition and became saturated once groundwater was encountered. Caving occurred in all five floodplain test pits, typically beginning just below surficial soils with increased silt content and/or the root zone at depths of 0.5 to 2.5 feet bgs.

8.3.2 Floodplain Borings (B-4, B-5 and B-6)

Loose to medium dense poorly-graded sand, poorly-graded sand with silt and silty sand were present from the ground surface to depths of 2.5 to 5.0 feet bgs. The upper sand layer was underlain by medium dense to very dense well-graded gravel or well-graded gravel with silt to a depth of 7.5, 17.5 and 16 feet bgs in borings B-4, B-5 and B-6, respectively. Dense to very dense poorly-graded sand, well-graded sand and poorly-graded sand with silt was encountered below the gravels to the 21.5 foot boring termination depth.

Soils above the groundwater elevation were typically in a moist condition. Groundwater was encountered at 7.5 feet bgs in all three borings and soils were typically saturated, except the silty gravel encountered at 16 feet in boring B-6 was considered to be in a moist condition.

8.4 Levee Extension Test Pits (TP-11 and TP-12) and Hand Auger Excavations (AHA-1 and AHA-2)

Two test pits were excavated upstream of the existing levee to provide soils information for evaluation of a levee extension. In addition, Aspect completed two hand auger excavations in the same general vicinity.

Soils encountered in TP-11 and TP-12 were similar to those encountered for the floodplain test pits. Silty sand or sandy silt soils were present from the ground surface to a depth of 1.7 feet, followed by well-graded gravels continuing to the test pit termination depths of 5.0 and 5.25 feet bgs. Test pit excavations were terminated due to caving when groundwater was encountered.

Test pit excavation was relatively easy with soils considered to be in a loose to medium dense state. Soils were moist throughout the test pits and became saturated below the groundwater surface. Caving in the test pits began about 1.7' bgs at the transition between silty sands and the underlying gravels.

A detailed description of soil conditions encountered by Aspect during their hand auger exploration is provided with their Memorandum (Appendix B). Soils in AHA-1 are summarily described as silt to a depth of 3.0 feet bgs, followed by poorly graded sand to a depth of 5.5 feet bgs and then sand with gravel to the exploration termination depth at 6.0 feet bgs. AHA-2 soils consisted of topsoil to a depth of 0.5

feet bgs, followed by silty sand to a depth of about 3 feet bgs and then sand with gravel to the exploration termination depth at about 3.1 feet bgs.

9.0 REFERENCES

AASHTO Standard Specification for Transportation Materials and Methods of Sampling and Testing, *AASHTO T27 Standard Method of Test for Sieve Analysis of Fine and Coarse Aggregates*, 2019.

AASHTO Standard Specification for Transportation Materials and Methods of Sampling and Testing, *AASHTO T11 Standard Method of Test for Materials Finer Than 75-um (No. 200) Sieve in Mineral Aggregates by Washing*, 2019.

ASTM Standards, Volume 04.08, *ASTM D1586 Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils*, 2018.

ASTM Standards, Volume 04.08, *ASTM D4633 Standard Test Method for Energy Measurement for Dynamic Penetrometers*, 2018.

ASTM Standards, Volume 04.09, *ASTM D6151 Standard Practice for Hollow-Stem Augers for Geotechnical Exploration and Soil Sampling,* 2018.

ASTM Standards, Volume 04.09, *ASTM D6914 Standard Practice for Sonic Drilling for Site Characterization and the Installation of Subsurface Monitoring Devices,* 2018.

Booth, Derek B., 1995, *United States Geologic Survey Map of the Maple Valley Quadrangle*, King County, Washington, Scale 1:24,000.

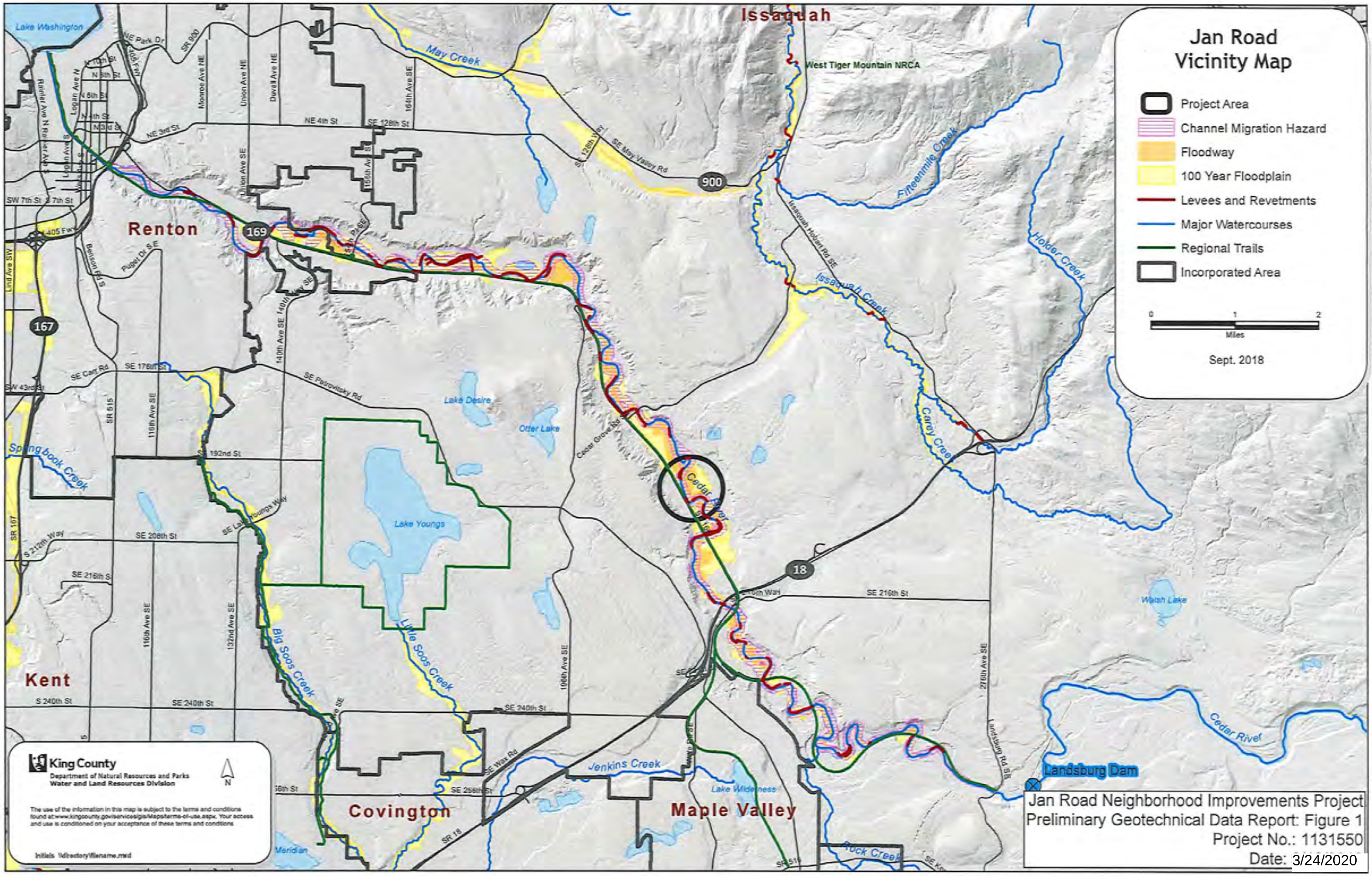
King County iMAP, https://gismaps.kingcounty.gov/iMap/

Washington State Department of Natural Resources, *Geologic Information Portal, https://www.dnr.wa.gov/geologyportal*

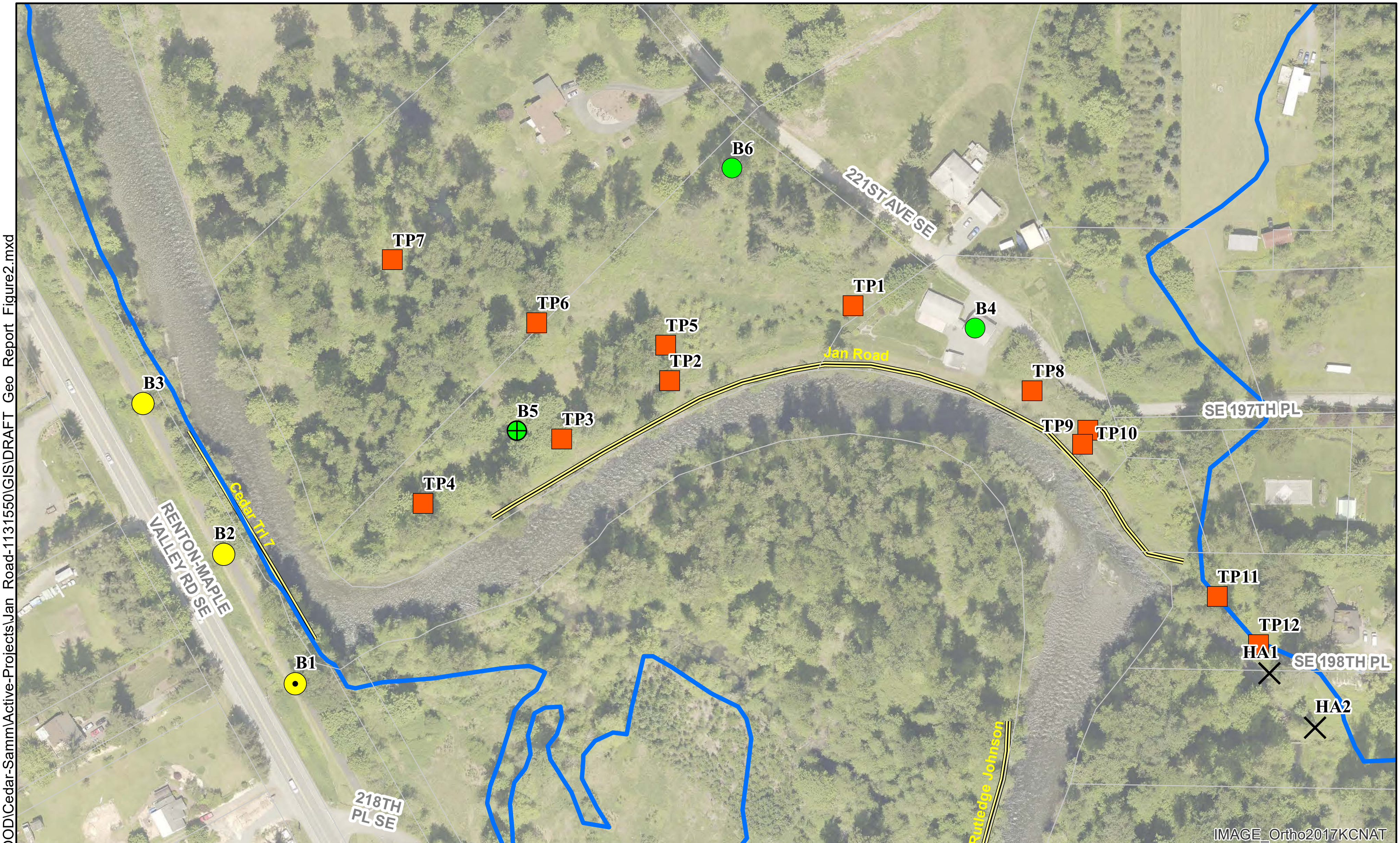
10.0 CLOSURE

We appreciate the opportunity to have provided services for this project and trust this revised report meets your current needs and adequately addresses the agreed upon scope of work. Should you have questions, comments or desire clarification, please contact Tim Hyden (206-477-0983) or Doug Walters (206-477-2112) at your convenience.

Attachments
Figure 1 – Site Vicinity Map
Figure 2 – Subsurface Exploration Location Plan
Appendix A – Boring and Test Pit Logs (KC Roads Services)
Appendix B – Aspect Memorandum and Hand Auger Exploration Logs
Appendix C – Laboratory Test Reports



Jan Road Neighborhood Improvements Project
Preliminary Geotechnical Data Report: Figure 1
Project No.: 1131550
Date: 3/24/2020



Geotechnical Explorations, By Method

- Boring (Hollow-Stem Auger)
- Boring (Hollow-Stem Auger, Shallow Groundwater Well)
- Boring (Sonic Core)
- Boring (Hollow-Stem Auger, Mud Rotary, and Sonic Core)
- Test Pit
- Hand Auger

- Parcels
- 100 Year Floodplain
- King County River facilities

0 100 200 400 Feet

N

Jan Road Neighborhood Improvements Project
Project Number: 1131550
Date: 3/24/2020

Figure 2 - Subsurface Exploration Location Plan

Document Path: G:\FLOOD\Cedar-Samm\Active-Projects\Jan Road-1131550\GIS\DRAFT Geo Report Figure2.mxd

King County

Department of Local Services
Road Services Division

APPENDIX A – REVISION NO. 1
BORING LOGS AND TEST PIT LOGS

PRELIMINARY GEOTECHNICAL SITE CHARACTERIZATION REPORT
JAN ROAD NEIGHBORHOOD IMPROVEMENTS

Mailstop: KSC-TR-0313 | 201 South Jackson St., Seattle, WA 98104-3856
206-477-8100 | maint.roads@kingcounty.gov | www.kingcounty.gov/roads

KEY TO SYMBOLS

Symbol	Description
	Strata symbols
	Paving
	Gravel
	Well graded gravel
	Silty sand
	Well graded sand with silt
	Poorly graded sand
	Well graded sand
	Poorly graded gravel with silt
	Poorly graded gravel
	Poorly graded sand with silt
	Silty gravel
	Well graded gravel with silt
	Silt
	Misc. Symbols
	Water table during drilling
	Boring continues
	Soil Samplers
	Standard penetration test
	Bulk/Grab sample
	Monitor Well Details
	covered riser
	concrete seal

Notes:

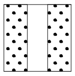
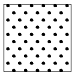
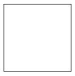
1. Borings were completed on 5-20, 5-28, 5-29, 5-30 and 5-31, 2019.
2. Borings 1, 4, 5 and 6 were completed by a track-mounted Diedrich D-50 drill rig.
3. 3.5 inch I.D. hollow-stem auger was used for Borings 1, 4 and 6 and 4.25 inch I.D. hollow-stem auger was used for Boring 5.
4. Borings 2 and 3 were completed by a track-mounted Geoprobe 8140 LC Sonic drill rig.
5. Test pits 1 - 7 were excavated on 5-10 and 5-13, 2019 using a John Deere 410D rubber tired backhoe with a two foot wide smooth tipped bucket.
6. Test pits 8 - 12 were excavated on 3-6-2020 using a Takeuchi TB Mini-Excavator with a three foot wide smooth tipped bucket.
7. Depth to groundwater and/or caving, if encountered, is reported on the Boring Logs and Test Pit Logs.
8. These logs are subject to the limitations and conclusions in this report.

FIGURE A-1 (FOR BORING LOGS AND TEST PIT LOGS)
PRELIMINARY GEOTECHNICAL DATA REPORT - JAN ROAD NEIGHBORHOOD IMPROVEMENTS

KEY TO SYMBOLS

Symbol Description

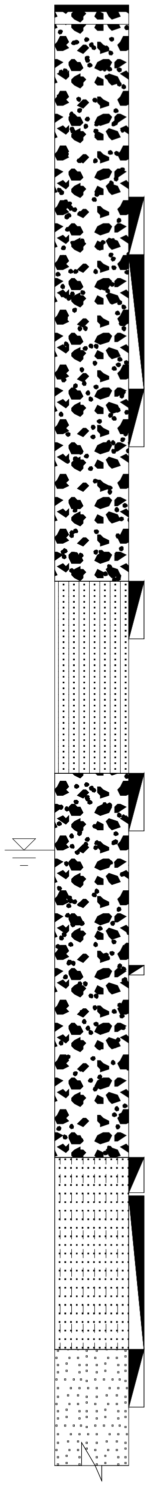
<u>Monitor Well Details</u>

Symbol	Description
	**silica sand, blank PVC**
	**silica sand, no pipe (end plug)**
	**end of well installation**

LOG OF BORING
BORING B-1

PROJECT: **Jan Road Neighborhood Improvements**
BORING LOCATION: **See Attached Site Plan**
DRILLING EQUIPMENT: **Diedrich D50; 3.5" Hollow-Stem**
DRILLING CONTRACTOR: **Holocene Drilling**
DEPTH TO - Water: **22'** Caving: **Yes**

DATE: **5-20-2019**
START: **10:30 AM (5-20)**
FINISH: **12:00 (5-21)**
LOGGER: **Tim Hyden**
DATE CHECKED: **5-21-2019**



ELEVATION/ DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Description	Moist (%)	-200 (%)	Remarks
0		GW	2" Asphalt Surfacing. 4" Crushed Surfacing. Reddish Brown, Loose, Well Graded Gravel with Sand and Cobbles, Moist. (Fill)			
5	4,4,4				3.7	
10	4,4,4					
15	3,2,3	SM	Reddish Brown, Very Loose, Fine to Medium Grain Silty Sand, Non-Plastic, Moist. (Fill)		16.8	
20	5,26,34	GW	Reddish Brown, Very Dense, Sandy Well Graded Gravel with Cobbles, Moist and Becomes Saturated at 22' BGS.			6" Recovery
25	50/3"					2" Recovery.
30	8,50/6"	SW-SM	Reddish Brown, Very Dense, Well Graded San with Silt and Gravel, Wet. Reddish Brown, Very Dense, Well Graded Sand with Silt and Gravel, Wet.		6.7	12" Recovery.
35	23,43,40	SP	Reddish Brown, Very Dense, Fine to Medium Grain Poorly Graded Sand, Wet.			14" Recovery. Switch to Mud Rotary.

FIGURE NO. **A-2**

LOG OF BORING

BORING B-1

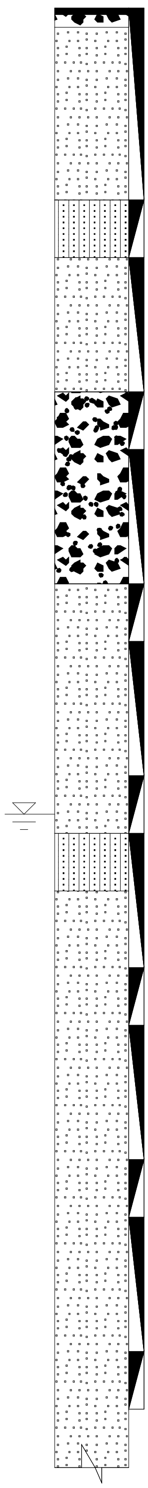
(continued)

ELEVATION/ DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Description	Moist (%)	-200 (%)	Remarks
40	16,50/6"	SP	Brown, Very Dense, Gravelly Fine to Medium Grain Poorly Graded Sand, Wet.			6" Recovery.
45	20,40,50/6		Brown, Very Dense, Poorly Graded Sand with Silt, Wet.		11.4	16" Recovery.
50	26,45,49		Gray, Very Dense, Fine Grain Poorly Graded Sand with Gravel, Wet.			16" Recovery.
			Boring Terminated at 51.5 BGS Due to Stuck Auger. Auger Abandoned In-Place on 5-22-2019.			
55						
60						
65						
70						
75						
80						

LOG OF BORING
BORING B-2

PROJECT: **Jan Road Neighborhood Improvements**
BORING LOCATION: **See Attached Site Plan**
DRILLING EQUIPMENT: **Geoprobe 8140LC Sonic**
DRILLING CONTRACTOR: **Holocene Drilling**
DEPTH TO - Water: **21'** Caving: **No - Cased**

DATE: **5-30-2019**
START: **8:00 AM**
FINISH: **2:30 PM**
LOGGER: **Tim Hyden**
DATE CHECKED: **5-30-2019**



ELEVATION/ DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Description	Moist (%)	-200 (%)	Remarks
0		SP	2" Asphalt Surfacing 4" Crushed Surfacing Light Brown, Loose, Gravelly Fine to Medium Grain Poorly Graded Sand, Moist. (Fill)			
5	6,3,3	SM	Tan, Loose, Silty Sand with Gravel and Cobbles, Moist. (Fill)		17.0	5" Recovery.
		SP	Reddish Brown, Loose, Gravelly Fine to Medium Grain Poorly Graded, Moist. (Fill)			
10	8,5,5	GW	Reddish Brown, Well Graded Gravel with Sand and Cobbles, Includes Few 6" Layers of Silty Sand, Moist. (Fill)		4.9	6" Recovery.
15	5,3,3,	SP	Tan, Loose to Dense, Fine to Medium Grain Poorly Graded Sand with Gravel and Cobbles, Moist becoming Wet at 21' BGS. (Fill)			14" Recovery.
20	24,22,9					10" Recovery.
		SM	Gray, Medium Dense, Silty Sand with Gravel, Wet. (Sample Mixed with Wood Debris)			
		SP	Reddish Brown Transition to Gray, Medium Dense, Fine to Medium Grain Poorly Graded Sand with Gravel, Wet.			
25	10,11,9					8" Recovery. Adding Mud Due to Heave.
30	8,10,15	SP	Tan to Gray, Medium Dense to Very Dense, Gravelly Fine to Medium Grain Poorly Graded Sand, Wet.			6" Recovery.
35	50/5"					5" Recovery.

LOG OF BORING

BORING B-2

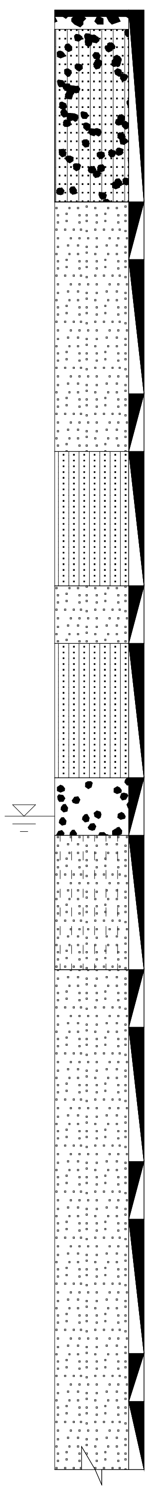
(continued)

ELEVATION/ DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Description	Moist (%)	-200 (%)	Remarks
		SP	Tan to Gray, Medium Dense to Very Dense, Gravelly Fine to Medium Grain Poorly Graded Sand with Cobbles, Wet.			
40	35,50/2"					No Recovery.
45	28,22,38					18" Recovery.
50	17,32,40	SW	Gray, Very Dense, Well Graded Sand with Gravel and Cobbles, Wet.			8" Recovery Small Piece Wood Debris.
55	20,25,50/5	SP	Gray, Very Dense, Gravelly Fine to Medium Grain Poorly Graded Sand, Wet.			16" Recovery.
60	23,50/5"					8" Recovery.
		SW	Gray, Dense to Very Dense, Gravelly Well Graded Sand, Wet.			
65	21,50/6"	SP	Gray, Very Dense, Fine to Medium Grain Poorly Graded Sand with Gravel, Wet.			12" Recovery.
70			Boring Terminated at 70' BGS Due to Heave.			
75						
80						

LOG OF BORING
BORING B-3

PROJECT: **Jan Road Neighborhood Improvements**
BORING LOCATION: **See Attached Site Plan**
DRILLING EQUIPMENT: **Geoprobe 8140LC Sonic**
DRILLING CONTRACTOR: **Holocene Drilling**
DEPTH TO - Water: **21'** Caving: **No - Cased**

DATE: **5-31-2019**
START: **7:30 AM**
FINISH: **2:30 PM**
LOGGER: **Tim Hyden**
DATE CHECKED: **5-31-2019**



ELEVATION/ DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Description	Moist (%)	-200 (%)	Remarks
0			2" Asphalt Surfacing			
		GP-GM	4" Crushed Surfacing Brown, Loose, Sandy Poorly Graded Gravel with Silt, Moist. (Fill)			
5	36,13,5	SP	Light Brown, Loose to Medium Dense, Fine to Medium Grain Poorly Graded Sand with Gravel and Cobbles, Moist. (Fill)			3" Recovery.
10	16,3,3					7" Recovery.
		SM	Brown, Medium Dense, Silty Sand, Moist. (Fill)		26.8	
15	9,9,9	SP	Light Brown, Medium Dense, Fine to Medium Grain Poorly Graded Sand with Gravel and Cobbles, Moist. (Fill)			8" Recovery. Scattered Charcoal.
		SM	Light Brown, Medium Dense, Silty Sand with Gravel and Cobbles, Moist. (Fill)			
20	9,14,14	GP	Brown, Medium Dense, Sandy Poorly Graded Gravel with Cobbles, Wet.			8" Recovery.
		SP-SM	Brown, Medium Dense, Poorly Graded Sand with Silt, Gravel and Cobbles, Wet.		5.5	
25	14,16,17	SP	Brown, Dense to Very Dense, Gravelly Fine to Medium Grain Poorly Graded Sand, Wet.			6" Recovery.
30	34,24,28		(Transitions to Gray)			15" Recovery.
35	35,43,50/3	SP	Gray, Very Dense, Fine to Medium Grain Poorly Graded Sand with Gravel, Wet.			15" Recovery.

FIGURE NO. **A-4**

LOG OF BORING
BORING B-3
(continued)

ELEVATION/ DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Description	Moist (%)	-200 (%)	Remarks
40	15,20,25	SP	Gray, Very Dense, Fine to Medium Grain Poorly Graded Sand with Gravel, Wet.			4" Recovery. Scattered Wood Debris.
45	25,50/6"					3" Recovery.
50	16,25,50/5					7" Recovery.
55	22,28,32					8" Recovery
60	50/4"					4" Recovery.
			Boring Terminated at 60' 4" BGS due to Heave.			
65						
70						
75						
80						

LOG OF BORING
BORING B-4

PROJECT: **Jan Road Neighborhood Improvements**
BORING LOCATION: **See Attached Site Plan**
DRILLING EQUIPMENT: **Diedrich D50; 3.5" Hollow-Stem**
DRILLING CONTRACTOR: **Holocene Drilling**
DEPTH TO - Water: **7.5'** Caving: **Yes**

DATE: **5-28-2019**
START: **8:30 AM**
FINISH: **9:40 AM**
LOGGER: **Tim Hyden**
DATE CHECKED: **5-28-2019**

ELEVATION/ DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Description	Moist (%)	-200 (%)	Remarks
0	4,6,9	SM SP	Brown, Mediumm Dense, Silty Fine Sand with Gravel, Moist.			6" Recovery
	8,16,14	GW	Tan, Medium Dense, Gravelly Poorly Graded Fine to Medium Grain Sand, Moist.			6" Recovery.
5	25,36,42		Tan, Medium Dense to Very Dense, Sandy Well Graded Gravel with Cobbles, Moist Becoming Wet at 7.5' BGS.			9" Recovery.
	24,12,10	SP-SM	Tan, Medium Dense, Poorly Graded Sand with Silt and Gravel, Wet.			4" Recovery.
10	14,10,14				5.6	10" Recovery.
	50/0"					No Recovery
15	50/1"					1" Recovery.
20	10,12,31	SP	Gray, Dense, Fine to Medium Grain Poorly Graded Sand with Gravel, Wet.			16" Recovery
			Boring Terminated at 21.5 Feet BGS.			
25						
30						
35						

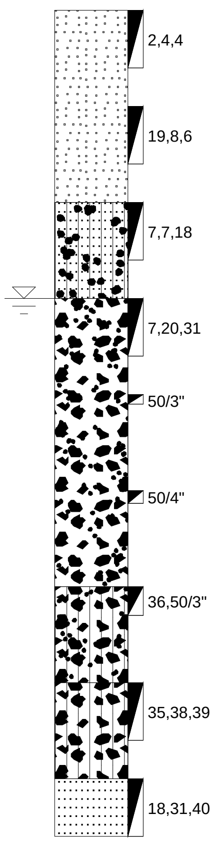
FIGURE NO. **A-5**

LOG OF MONITOR WELL INSTALLATION
WELL NO. B-5

PROJECT: **Jan Road Neighborhood Improvements**
BORING LOCATION: **See Attached Site Plan**
DRILL METHOD: **Diedrich D50; 3.5" Hollow-Stem**
DRILLER: **Holocene Drilling**
DEPTH TO - Water: **7.5'**

DATE: **5-28-2019**
START: **9:50 AM**
FINISH: **12:00 PM**
LOGGER: **Tim Hyden**
DATE CHECKED: **5-28-2019**

Washington State
Well ID: BLT 729



ELEVATION/ DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Description	Moist (%)	-200 (%)	Remarks
0	2,4,4	SP	Brown/Tan, Loose to Medium Dense, Gravelly Fine to Medium Grain Poorly Graded Sand with Occasional Cobbles, Moist.			3" Recovery. Gravel in Sampler Tip.
	19,8,6					8" Recovery
5	7,7,18	GP-GM	Brown, Medium Dense, Poorly Graded Gravel with Silt, Sand and Cobbles, Moist.		5.0	3" Recovery. Gravel in Sampler Tip.
	7,20,31	GW	Tan, Very Dense, Sandy Well Graded Gravel with Occasional Cobbles, Wet.			
10	50/3"				6.4	No Recovery.
	50/4"					4" Recovery.
15	36,50/3"	GW-GM	Gray, Very Dense, Well Graded Gravel with Silt, Sand and Cobbles, Wet.			9" Recovery.
	35,38,39	GM	Reddish Brown, Very Dense, Silty Poorly Graded Gravel with Sand, Non-Plastic, Wet.			16" Recovery.
20	18,31,40	SW	Ten/Reddish Brown, Very Dense Well Graded Sand with Gravel, Wet.			18" Recovery.
25			Boring Terminated at 21.5' BGS.			
30						

Monitor Well Construction Schematic

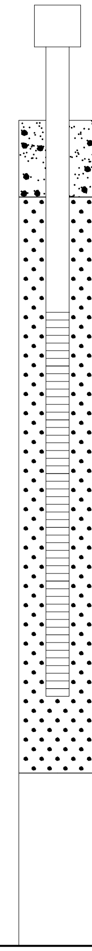


FIGURE **A-6**

LOG OF BORING
BORING B-6

PROJECT: **Jan Road Neighborhood Improvements**
BORING LOCATION: **See Attached Site Plan**
DRILLING EQUIPMENT: **Diedrich D50; 3.5" Hollow-Stem**
DRILLING CONTRACTOR: **Holocene Drilling**
DEPTH TO - Water: **7.5'** Caving: **Yes**

DATE: **5-28-2019**
START: **1:00 PM**
FINISH: **2:00 PM**
LOGGER: **Tim Hyden**
DATE CHECKED: **5-28-2019**

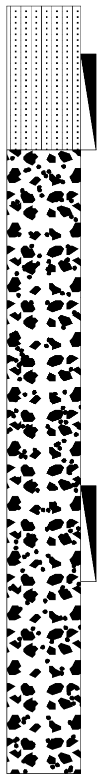
ELEVATION/ DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Description	Moist (%)	-200 (%)	Remarks
0	4,6,7	SM	Dark Brown, Medium Dense, Fine to Medium Grain Silty Sand with Gravel, Moist.			
	4,9,12	SP-SM	Tan, Medium Dense, Fine to Medium Grain Poorly Graded Sand with Silt and Gravel, Moist.			8" Recovery.
5	5,12,18	GW	Tan, Medium Dense, Sandy Well Graded Gravel, Moist.			9" Recovery. Gravel in Sampler Tip.
	38,50/6"	GW-GM	Tan/Brown, Dense to Very Dense, Well Graded Gravel with Silt, Sand and Cobbles, Wet.			6" Recovery. Gravel in Sampler Tip.
10	18,20,21					10" Recovery.
	7,18,23	SP	Tan, Dense, Gravelly Fine to Medium Grain Poorly Graded Sand with Occasional Cobbles, Moist.			13" Recovery.
15	6,30,48					17" Recovery.
		GM	Reddish Brown, Very Dense, Silty Gravel with Sand, Moist.			
20	12,22,23	SP-SM	Reddish Brown, Dense, Fine to Medium Grain Poorly Graded Sand with Silt and Gravel, Wet.			18" Recovery.
			Boring Terminated at 21.5 Feet BGS.			
25						
30						
35						

FIGURE NO. **A-7**

TEST PIT LOG
NUMBER: TP1 TOE

PROJECT: **Jan Road Neighborhood Improvements**
TEST PIT LOCATION: **See Attached Site Plan**
EXCAVATION EQUIPMENT: **John Deere 410D Backhoe**
CONTRACTOR: **Jensen Excavating**
DEPTH TO - WATER: **N/A** CAVING: **~1.5'**

DATE: **5-10-2019**
START: **8:15 AM**
FINISH: **9:30 PM**
LOGGER: **Tim Hyden**
DATE CHECKED: **5-10-2019**



ELEVATION/ DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Description	Moist (%)	-200 (%)	Remarks
0		SM	Dark Brown, Fine to Medium Grain Silty Sand with Gravel and Occasional Cobbles, Moist, Fine Roots to About 18". (Fill?)		21.8	Debris Noted in Upper 1.5 Feet.
2		GW	Light Brown/Tan, Well Graded Sandy Gravel with Cobbles and Occasional Small Boulders, Moist (Moisture Content Increasing with Depth).			Increased Cobble Content at 4' to 8'.
4						
6						
8			Test Pit Terminated at 8' BGS.			
10						
12						
14						

FIGURE NO. **A-8**

TEST PIT LOG
NUMBER: TP1 LEVEE

PROJECT: **Jan Road Neighborhood Improvements**
TEST PIT LOCATION: **See Attached Site Plan**
EXCAVATION EQUIPMENT: **John Deere 410D Backhoe**
CONTRACTOR: **Jensen Excavating**
DEPTH TO - WATER: **N/A** CAVING: **0.5' - 1.0'**

DATE: **5-10-19**
START: **9:30 AM**
FINISH: **10:00 AM**
LOGGER: **Tim Hyden**
DATE CHECKED: **5-10-2019**

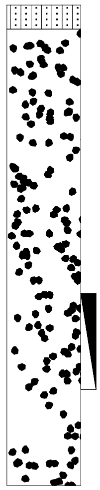
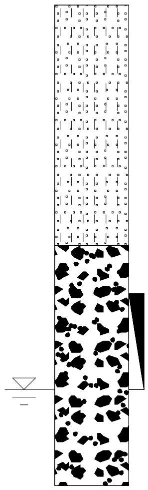
ELEVATION/ DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Description	Moist (%)	-200 (%)	Remarks
0		SM	Brown, Silty Sand with Gravel, Non-Plastic, Moist, Fine Roots to Approximately 18". (Topsoil)			
		GP	Light Brown/Tan, Sandy Poorly Graded Gravel with Occasional Cobbles, Moist. (Fill)		2.5	
2						
4						
			Test Pit Terminated at 5' BGS.			
6						
8						
10						
12						
14						

FIGURE NO. **A-9**

TEST PIT LOG
NUMBER: TP2 TOE

PROJECT: **Jan Road Neighborhood Improvements**
TEST PIT LOCATION: **See Attached Site Plan**
EXCAVATION EQUIPMENT: **John Deere 410D Backhoe**
CONTRACTOR: **Jensen Excavating**
DEPTH TO - WATER: **4'** CAVING: **~2.0'**

DATE: **5-10-2019**
START: **10:15 AM**
FINISH: **11:00 AM**
LOGGER: **Tim Hyden**
DATE CHECKED: **5-10-2019**



ELEVATION/ DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Description	Moist (%)	-200 (%)	Remarks
0		SP-SM	Brown, Fine to Medium Grain Poorly Graded Sand with Gravel and Silt, Abundant Cobbles, Moist, Fine Roots to Approximately 2'.			
2		GW	Light/Reddish Brown, Well Graded Gravel with Sand and Cobbles, Moist becoming Saturated at 4'BGS.			
4						
			Test Pit Terminated at 5' BGS.			
6						
8						
10						
12						
14						

FIGURE NO. **A-10**

TEST PIT LOG
NUMBER: TP2 LEVEE

PROJECT: **Jan Road Neighborhood Improvements**
TEST PIT LOCATION: **See Attached Site Plan**
EXCAVATION EQUIPMENT: **John Deere 410D Backhoe**
CONTRACTOR: **Jensen Excavating**
DEPTH TO - WATER: **N/A** CAVING: **0.5' - 1.0'**

DATE: **5-10-2019**
START: **10:20 AM**
FINISH: **11:00 AM**
LOGGER: **Tim Hyden**
DATE CHECKED: **5/10/2019**

ELEVATION/ DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Description	Moist (%)	-200 (%)	Remarks
0		SM	Brown, Silty Sand with Gravel, Non-Plastic, Moist. (Topsoil with Fine Roots)			
		GW	Tan/Reddish Brown, Sandy Well Graded Gravel with Cobbles, Moist, Fine Roots to Approximately 18".			
8			Test Pit Terminated at 8' BGS.			

Depth scale: 0, 2, 4, 6, 8, 10, 12, 14

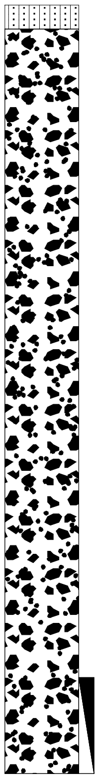
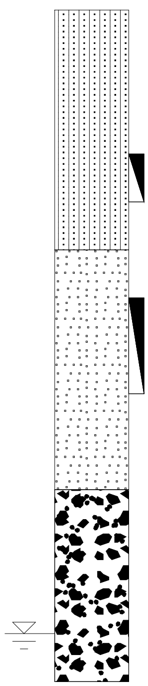


FIGURE NO. **A-11**

TEST PIT LOG
NUMBER: TP3 TOE

PROJECT: **Jan Road Neighborhood Improvements**
TEST PIT LOCATION: **See Attached Site Plan**
EXCAVATION EQUIPMENT: **John Deere 410D Backhoe**
CONTRACTOR: **Jensen Excavating**
DEPTH TO - WATER: **6.5'** CAVING: **~2.5'**

DATE: **5-10-2019**
START: **1:00 PM**
FINISH: **1:30**
LOGGER: **Tim Hyden**
DATE CHECKED: **5/10/2019**



ELEVATION/ DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Description	Moist (%)	-200 (%)	Remarks
0		SM	Brown, Silty Fine to Medium Grained Sand, Trace of Gravel, Non-Plastic, Moist. (Abundant Roots 1/2" to 3" Diameter to a Depth of 2.5")			
2		SP	Tan, Fine to Medium Grain Poorly Graded Sand with Gravel and Cobbles, Moist.			Fill - Brick Encountered about 2.5' BGS.
4						
6		GW	Tan/Reddish Brown, Well Graded Sandy Gravel with Cobbles, Moist Becoming Wet at 6.5' BGS.			
8			Test Pit Terminated at 7' BGS.			
10						
12						
14						

FIGURE NO. **A-12**

TEST PIT LOG
NUMBER: TP3 LEVEE

PROJECT: **Jan Road Neighborhood Improvements**
TEST PIT LOCATION: **See Attached Site Plan**
EXCAVATION EQUIPMENT: **John Deere 410D Backhoe**
CONTRACTOR: **Jensen Excavating**
DEPTH TO - WATER: **N/A** CAVING: **0.5' - 1.0'**

DATE: **5/10/2019**
START: **1:30 PM**
FINISH: **1:50 PM**
LOGGER: **Tim Hyden**
DATE CHECKED: **5/10/2019**

ELEVATION/ DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Description	Moist (%)	-200 (%)	Remarks
0		SM	Brown, Silty Sand with Gravel, Non-plastic, Moist. (Topsoil with Fine Roots)			
		GP	Tan, Poorly Graded Gravel with Sand and Cobbles, Moist. (Fill)			
2					2.3	
4						
6						
7			Test Pit Terminated at 7' BGS.			
8						
10						
12						
14						

FIGURE NO. **A-13**

TEST PIT LOG
NUMBER: TP4 TOE

PROJECT: **Jan Road Neighborhood Improvements**
TEST PIT LOCATION: **See Attached Site Plan**
EXCAVATION EQUIPMENT: **John Deere 410D Backhoe**
CONTRACTOR: **Jensen Excavating**
DEPTH TO - WATER: **N/A** CAVING: **~1.5'**

DATE: **5-13-19**
START: **8:40 AM**
FINISH: **9:10 AM**
LOGGER: **Tim Hyden**
DATE CHECKED: **5/13/2019**

ELEVATION/ DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Description	Moist (%)	-200 (%)	Remarks
0		SM	Brown, Silty Fine Grain Sand with Gravel, Non-Plastic, Moist, Fine to 1-Inch Diameter Roots to 18".			
2		SP	Tan, Gravelly, Fine to Medium Grain Poorly Graded Sand with Cobbles and Occasional Small Boulders, Moist.			
4		GW	Tan/Reddish Brown, Well Graded Gravel with Sand, Cobbles and Occasional Boulders, Moist.		1.5	
6			Test Pit Terminated at 6' BGS.			
8						
10						
12						
14						

FIGURE NO. **A-14**

TEST PIT LOG
NUMBER: TP4 LEVEE

PROJECT: **Jan Road Neighborhood Improvements**
TEST PIT LOCATION: **See Attached Site Plan**
EXCAVATION EQUIPMENT: **John Deere 410D Backhoe**
CONTRACTOR: **Jensen Excavating**
DEPTH TO - WATER: **N/A** CAVING: **~1.0'**

DATE: **5-13-19**
START: **9:00 AM**
FINISH: **9:30 AM**
LOGGER: **Tim Hyden**
DATE CHECKED: **5-13-19**

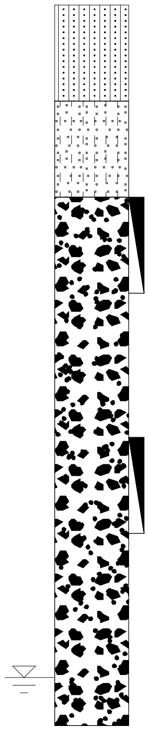
ELEVATION/ DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Description	Moist (%)	-200 (%)	Remarks
0		GW	Light Brown/Tan, Sandy, Well Graded Gravel with Cobbles and Occasional Small Boulders, Moist. Fine Roots to Approximately 12" to 18". (Fill)			
2						
4			Test Pit Terminated at 4' BGS.			
6						
8						
10						
12						
14						

FIGURE NO. **A-15**

TEST PIT LOG
NUMBER: TP5

PROJECT: **Jan Road Neighborhood Improvements**
TEST PIT LOCATION: **See Attached Site Plan**
EXCAVATION EQUIPMENT: **John Deere 410D Backhoe**
CONTRACTOR: **Jensen Excavating**
DEPTH TO - WATER: **7'** CAVING: **~1.0'**

DATE: **5-10-19**
START: **10:20 AM**
FINISH: **11:00 AM**
LOGGER: **Tim Hyden**
DATE CHECKED: **5-20-2019**

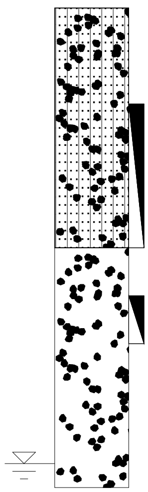


ELEVATION/ DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Description	Moist (%)	-200 (%)	Remarks
0		SM	Dark Brown, Fine to Medium Grain Silty Sand with Gravel, Occasional Cobbles, Non-Plastic, Moist, Fine Roots to 18".			
		SP-SM	Light Brown/Tan, Gravelly Poorly Graded Sand with Silt and Cobbles, Moist.			
2						
4		GW	Tan/Reddish Brown, Well Graded Gravel with Sand, Cobbles and Occasional Small Boulders, Moist Becoming Wet at 7' BGS.		1.4	Increasing Cobble Content and Occasional Small Boulders at 3'.
6						
8			Test Pit Terminated at 7.5' BGS.			
10						
12						
14						

TEST PIT LOG
NUMBER: TP6

PROJECT: **Jan Road Neighborhood Improvements**
TEST PIT LOCATION: **See Attached Site Plan**
EXCAVATION EQUIPMENT: **John Deere 410D Backhoe**
CONTRACTOR: **Jensen Excavating**
DEPTH TO - WATER: **4.75'** CAVING: **~2.5'**

DATE: **5-13-19**
START: **9:30 am**
FINISH: **10:15 am**
LOGGER: **Tim Hyden**
DATE CHECKED: **5/13/2019**



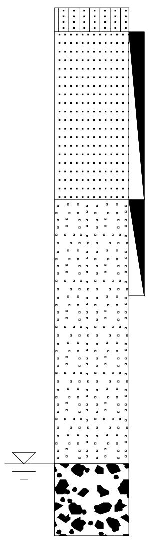
ELEVATION/ DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Description	Moist (%)	-200 (%)	Remarks
0		GP-GM	Brown, Poorly Graded Gravel with Silt, Sand and Cobbles, Moist. Approximately 0.5" to 2" Diameter Roots to 2.5'.		8.1	
2		GP	Light Brown, Sandy Poorly Graded Gravel with Cobbles, Moist Becoming Wet at 4.75'.			
4						
6			Test Pit Terminated at 5' BGS.			
8						
10						
12						
14						

FIGURE NO. **A-17**

TEST PIT LOG
NUMBER: TP7

PROJECT: **Jan Road Neighborhood Improvements**
TEST PIT LOCATION: **See Attached Site Plan**
EXCAVATION EQUIPMENT: **John Deere 410D Backhoe**
CONTRACTOR: **Jensen Excavating**
DEPTH TO - WATER: **4.75'** CAVING: **~0.5'**

DATE: **5-13-19**
START: **10:25 am**
FINISH: **11:00 am**
LOGGER: **Tim Hyden**
DATE CHECKED: **5/13/2019**

ELEVATION/ DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Description	Moist (%)	-200 (%)	Remarks
0		SM	Dark Brown, Silty Fine to Medium Grain Sand with Gravel, Moist, Approximately 1" Diameter Roots to 12".			
		SW	Gray/Tan, Well Graded Gravelly Sand with Some Cobbles, Moist.			
2		SP	Gray/Tan, Poorly Graded Sand, Moist.		0.2	
		GW	Tan, Sandy Well Graded Gravel with Some Cobbles, Wet.			
			Test Pit Terminated at 5.5' BGS.			

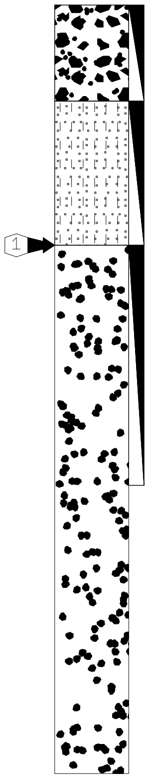
Depth scale: 0, 2, 4, 6, 8, 10, 12, 14

FIGURE NO. **A-18**

TEST PIT LOG
NUMBER: TP-8

PROJECT: **Jan Road Neighborhood Improvements**
TEST PIT LOCATION: **See Attached Site Plan**
EXCAVATION EQUIPMENT: **Takeuchi TB 235 Mini-Excavator**
CONTRACTOR: **Rivers Edge Env. Services, Inc.**
DEPTH TO - WATER: **N/A** CAVING: **2.5'**

DATE: **3-6-2020**
START: **11:08 AM**
FINISH: **11:46 PM**
LOGGER: **Tim Hyden**
DATE CHECKED: **3-6-2020**

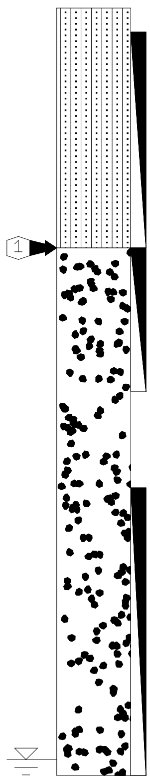


ELEVATION/ DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Description	Moist (%)	-200 (%)	Remarks
0		GW	Brown, Well Graded Gravel with Sand, 5% Cobbles, Non-Plastic, Moist, Fine Roots.	7.1	4.3	Possible Subgrade From Access Road.
		SP-SM	Tan, Fine Sand with Silt, Non-Plastic, Moist.	22.8		
2	1	GP	Gray/Light Brown, Poorly Graded Gravel with Sand, 20% to 30% Cobbles, Moist.	2.9	0.7	
4						
6						
8			Test Pit Terminated at 8 Feet Due to Caving and Equipment Capability.			
10						
12						
14						

TEST PIT LOG
NUMBER: TP-9

PROJECT: **Jan Road Neighborhood Improvements**
TEST PIT LOCATION: **See Attached Site Plan**
EXCAVATION EQUIPMENT: **Takeuchi TB 235 Mini-Excavator**
CONTRACTOR: **Rivers Edge Env. Services, Inc.**
DEPTH TO - WATER: **7.833** CAVING: **2.5'**

DATE: **3-6-2020**
START: **10:43 AM**
FINISH: **12:55 PM**
LOGGER: **Tim Hyden**
DATE CHECKED: **3-6-2020**



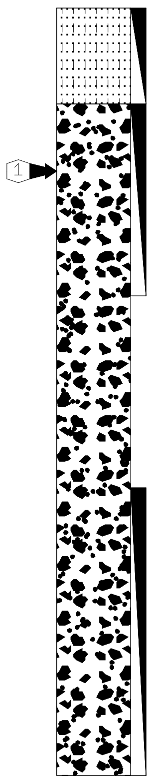
ELEVATION/ DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Description	Moist (%)	-200 (%)	Remarks
0		SM	Dark Brown, Fine Grain Silty Sand, Non-Plastic, Moist to Near Saturated, Fine Roots near Surface and 1/4" to 1" Diameter Roots to 2.5' Depth.	26.7	43.3	
2						
		GP	Gray, Poorly Graded Gravel with Sand, 20% Cobbles, Moist becoming Saturated at Bottom of Test Pit.	3.4		
4						
				7.5	2.8	
6						
8			Test Pit Terminated at 8 Feet Due to Water and Caving.			
10						
12						
14						

FIGURE NO. **A-20**

TEST PIT LOG
NUMBER: TP-10

PROJECT: **Jan Road Neighborhood Improvements**
TEST PIT LOCATION: **See Attached Site Plan**
EXCAVATION EQUIPMENT: **Takeuchi TB 235 Mini-Excavator**
CONTRACTOR: **Rivers Edge Env. Services, Inc.**
DEPTH TO - WATER: **N/A** CAVING: **1.7'**

DATE: **3-6-2020**
START: **10:19 AM**
FINISH: **12:23 PM**
LOGGER: **Tim Hyden**
DATE CHECKED: **3-6-2020**

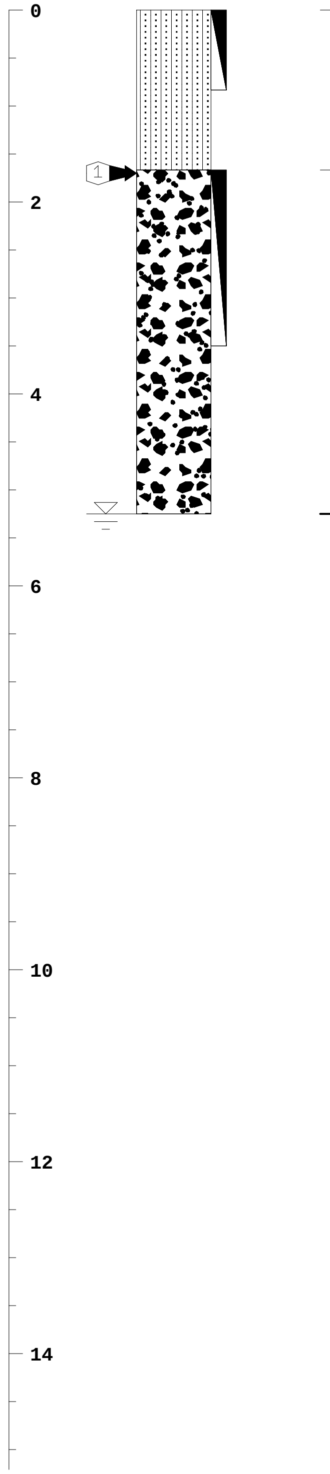


ELEVATION/ DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Description	Moist (%)	-200 (%)	Remarks
0		SW-SM	Light Brown, Gravelly Well Graded Sand with Silt, Moist.(Fine Roots to Depth of Approx. 6")	7.2		
	1	GW	Gray, Well Graded Gravel with Sand, 20 to 30 Percent Cobbles, Moist.	3.3	0.8	
2						
4						
		GW	Gray, Well Graded Gravel with Sand, 20 to 30 Percent Cobbles, Moist.	3.4	1.0	
6						
8			Test Pit Terminated at 8 Feet Due to Caving and Equipment Capability.			
10						
12						
14						

TEST PIT LOG
NUMBER: TP-11

PROJECT: **Jan Road Neighborhood Improvements**
TEST PIT LOCATION: **See Attached Site Plan**
EXCAVATION EQUIPMENT: **Takeuchi TB 235 Mini-Excavator**
CONTRACTOR: **Rivers Edge Env. Services, Inc.**
DEPTH TO - WATER: **5.25'** CAVING: **1.7'**

DATE: **3-6-2020**
START: **9:53 AM**
FINISH: **1:31 PM**
LOGGER: **Tim Hyden**
DATE CHECKED: **3-6-2020**



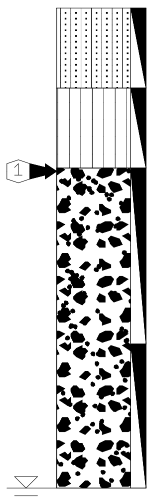
ELEVATION/ DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Description	Moist (%)	-200 (%)	Remarks
0		SM	Dark Brown, Fine Silty Sand, Trace of Gravel, Non-Plastic, Moist. Non-Plastic, Moist, Minor Fine Roots. (Resident Burn Area)	32.3		
		GW	Tan, Well Graded Gravel with Sand, Approx. 20% 3" to 6" Cobbles, Moist.	4.7	1.5	
			Test Pit Terminated at 5.25' Due to Water.			

FIGURE NO. **A-22**

TEST PIT LOG
NUMBER: TP-12

PROJECT: **Jan Road Neighborhood Improvements**
TEST PIT LOCATION: **See Attached Site Plan**
EXCAVATION EQUIPMENT: **Takeuchi TB 235 Mini-Excavator**
CONTRACTOR: **Rivers Edge Env. Services, Inc.**
DEPTH TO - WATER: **5'** CAVING: **1.7'**

DATE: **3-6-2020**
START: **9:24 AM**
FINISH: **2:23 PM**
LOGGER: **Tim Hyden**
DATE CHECKED: **3-6-2020**



ELEVATION/ DEPTH	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Description	Moist (%)	-200 (%)	Remarks
0		SM	Dark Brown Fine Silty Sand with Gravel, Non-Plastic, Moist, Minor Fine Roots. (Resident Burn Area)	22.2		
		ML	Tan, Fine Sandy Silt, Non-Plastic to Low Plasticity, Moist, 1" to 1.5" Diameter Roots.	28.5	57.9	
2		GW	Tan, Sandy, Well Graded Gravel, Moist.		4.0	
				7.9	1.2	
4						
			Test Pit Terminated at 5' Due to Water.			
6						
8						
10						
12						
14						

FIGURE NO. **A-23**

King County

Department of Local Services
Road Services Division

APPENDIX B – REVISION NO. 1
ASPECT CONSULTING APRIL 3, 2020 MEMORANDUM

JAN ROAD NEIGHBORHOOD IMPROVEMENTS
PRELIMINARY HAND AUGER INVESTIGATIONS

MEMORANDUM

Project No. 190175-600.3

April 3, 2020

To: Jerry Scheller, PE, Senior Project Manager
Tetra Tech, Inc.

cc: Dan Heckendorf, PE, CFM, Senior Engineer
King County Water and Land Resources Division

From:

Andrew J. Holmson, PE
Associate Geotechnical Engineer
aholmson@aspectconsulting.com

Henry H. Haselton, PE, PMP
Principal Geotechnical Engineer
hhaselton@aspectconsulting.com

Mari Otto, EIT
Project Engineer
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Re: **Jan Road Neighborhood Improvements - Preliminary Hand Auger Investigations**

Introduction

This memo presents preliminary shallow hand auger exploration data collected by Aspect Consulting, LLC (Aspect) for the King County Water and Land Resources Division's (County) Jan Road Neighborhood Improvements Design (Project). The Project area is located along the Cedar River between Renton and Maple Valley, Washington, approximately 1.3 miles northwest of the Highway 18 and State Route 169 (SR 169) interchange (Site).

On March 6, 2020, an Aspect engineer was on-Site to:

- Partially observe test pit explorations being performed for the County under the direction of Tim Hyden with the King County Road Services Division (Roads Services)
- Perform preliminary hand auger explorations near the confluence of Taylor Creek and the Cedar River to the west of the terminus of SE 198th Place

We understand that Roads Services is preparing a data report to describe the test pit explorations and characterize the soil conditions at those exploration locations. This memo will be included as an attachment to their data report.

Hand Auger Explorations

An Aspect engineer performed two hand auger explorations, designated AHA-01 and AHA-02, to depths of practical refusal at 6 feet and 3 feet below ground surface (bgs), respectively.

The soil conditions encountered in AHA-01 consisted of up to about 5.5 feet of very loose to medium dense, wet, brown to gray silt (ML) and sand (SP) that we interpreted to be overbank alluvium deposits. Underlying the overbank alluvium deposits, we encountered medium dense, wet, brown sand with gravel (SW) that we interpreted to be high energy alluvium deposits. We encountered hand auger refusal in AHA-01 at a depth of 6 feet bgs in the high energy alluvium deposits.

The soil conditions in AHA-02 consisted of up to 0.5 feet of very loose, grassy topsoil overlying 2.5 feet of very loose, very moist, brown silty sand (SM) overbank alluvium deposits, and encountered hand auger refusal conditions in medium dense, very moist, light brown sand with gravel (SW) high energy alluvium deposits at 3 feet bgs.

Groundwater was encountered at 0.5 feet in AHA-01. No groundwater was encountered in AHA-02.

The hand auger exploration logs are presented in Appendix A. Approximate locations for the hand auger explorations are presented in Table 1 below.

Table 1. Hand Auger Locations

Exploration ID	Latitude	Longitude
AHA-01	47.424065°	-122.045330°
AHA-02	47.423868°	-122.044998°

Note: Hand auger exploration locations are approximate, and accurate to ± 20 feet.

Limitations

Work for this project was performed for Tetra Tech (Client), and 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.

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 B 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 Henry H. Haselton, PE, PMP at 206.225.7379.

Attachments: Appendix A – Preliminary Subsurface Investigation: Hand Auger Exploration Logs
Appendix B – Report Limitations and Guidelines for Use

V:\190175 Jan Road Neighborhood Improvements Design\Deliverables\Hand Auger Investigations\Final_20200403\190175-600.3_Jan Road Hand Auger Investigations_20200403.docx

APPENDIX A

Preliminary Subsurface Investigation: Hand Auger 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
- GS = Grain 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	Jan Rd Neighborhood Improvements - 190175			Geotechnical Exploration Log	
	Project Address & Site Specific Location Levee Extension - Cedar River near Taylor Creek, Right bank near RM 13.35			*Coordinates (Lat,Lon WGS84)* 47.4241, -122.0453 (est)	*Exploration Number* **AHA-01**
Contractor Aspect Consulting, LLC	*Equipment* Hand	*Sampling Method* Grab		*Ground Surface Elev. (NAVD88)* 261' (est)	
Operator MO	*Exploration Method(s)* Hand tools	*Work Start/Completion Dates* 3/6/2020		*Top of Casing Elev. (NAVD88)* NA	*Depth to Water (Below GS)* 0.5' (Seep)

Depth (feet)	Elev. (feet)	Exploration Completion and Notes	Sample Type/ID	Blows/foot ▲ Water Content (%) ● 0 10 20 30 40 50	Blows/6"	Tests	Material Type	Description	Depth (ft)
0	261	3/6/2020 (water level)					ML	**OVERBANK ALLUVIUM DEPOSITS** SILT (ML); very loose to loose, very moist, brown; non-plastic, few fine sand. Becomes wet at 0.5 ft bgs.	
1	260	Backfilled with excavated soil.							1
2	259							Encountered a subangular cobble approximately 12 inches long and 5 inches wide.	2
3	258						SP	SAND (SP); loose to medium dense, wet, gray; predominately fine sand.	3
4	257								4
5	256								5
5.5							SW	**HIGH ENERGY ALLUVIUM DEPOSITS** SAND WITH GRAVEL (SW); medium dense, wet, brown; fine to coarse sand, fine subrounded to subangular gravel.	
6	255							Bottom of exploration at 6 ft. bgs. Note: Hand auger terminated due to refusal on sand with gravel.	6
7	254								7
8	253								8
9	252								9

Legend

Sample Type

Plastic Limit ⊢—⊣ Liquid Limit

Water Level: Water Level (Seepage)

See Exploration Log Key for explanation of symbols

Logged by: MO
Approved by: AJH

Exploration Log AHA-01

Sheet 1 of 1

Aspect CONSULTING	Jan Rd Neighborhood Improvements - 190175			Geotechnical Exploration Log	
	Project Address & Site Specific Location Levee Extension - Cedar River near Taylor Creek, Right bank near RM 13.4			*Coordinates (Lat,Lon WGS84)* 47.4239, -122.0450 (est)	*Exploration Number* **AHA-02**
Contractor Aspect Consulting, LLC	*Equipment* Hand	*Sampling Method* Grab		*Ground Surface Elev. (NAVD88)* 267' (est)	
Operator MO	*Exploration Method(s)* Hand tools	*Work Start/Completion Dates* 3/6/2020		*Top of Casing Elev. (NAVD88)* NA	*Depth to Water (Below GS)* No Water Encountered

Depth (feet)	Elev. (feet)	Exploration Completion and Notes	Sample Type/ID	Blows/foot ▲ Water Content (%)● 0 10 20 30 40 50	Blows/6"	Tests	Material Type	Description	Depth (ft)
								TOPSOIL TOPSOIL; very loose, very moist, brown; organics are grass, rootlets, and organic fines.	
								OVERBANK ALLUVIUM DEPOSITS SILTY SAND (SM); very loose, very moist, brown; predominately fine sand.	
1	266	Backfilled with excavated soil.							1
2	265								2
3	264							**HIGH ENERGY ALLUVIUM DEPOSITS** SAND WITH GRAVEL (SW); medium dense, very moist, light brown; fine to coarse sand, fine subrounded gravel. Bottom of exploration at 3 ft. bgs. Note: Hand auger terminated due to refusal on sand with gravel.	3
4	263								4
5	262								5
6	261								6
7	260								7
8	259								8
9	258								9

Legend

Sample Type

Plastic Limit ├──┤ Liquid Limit

Water Level: No Water Encountered

See Exploration Log Key for explanation of symbols

Logged by: MO
Approved by: AJH

Exploration Log AHA-02

Sheet 1 of 1

APPENDIX B

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.

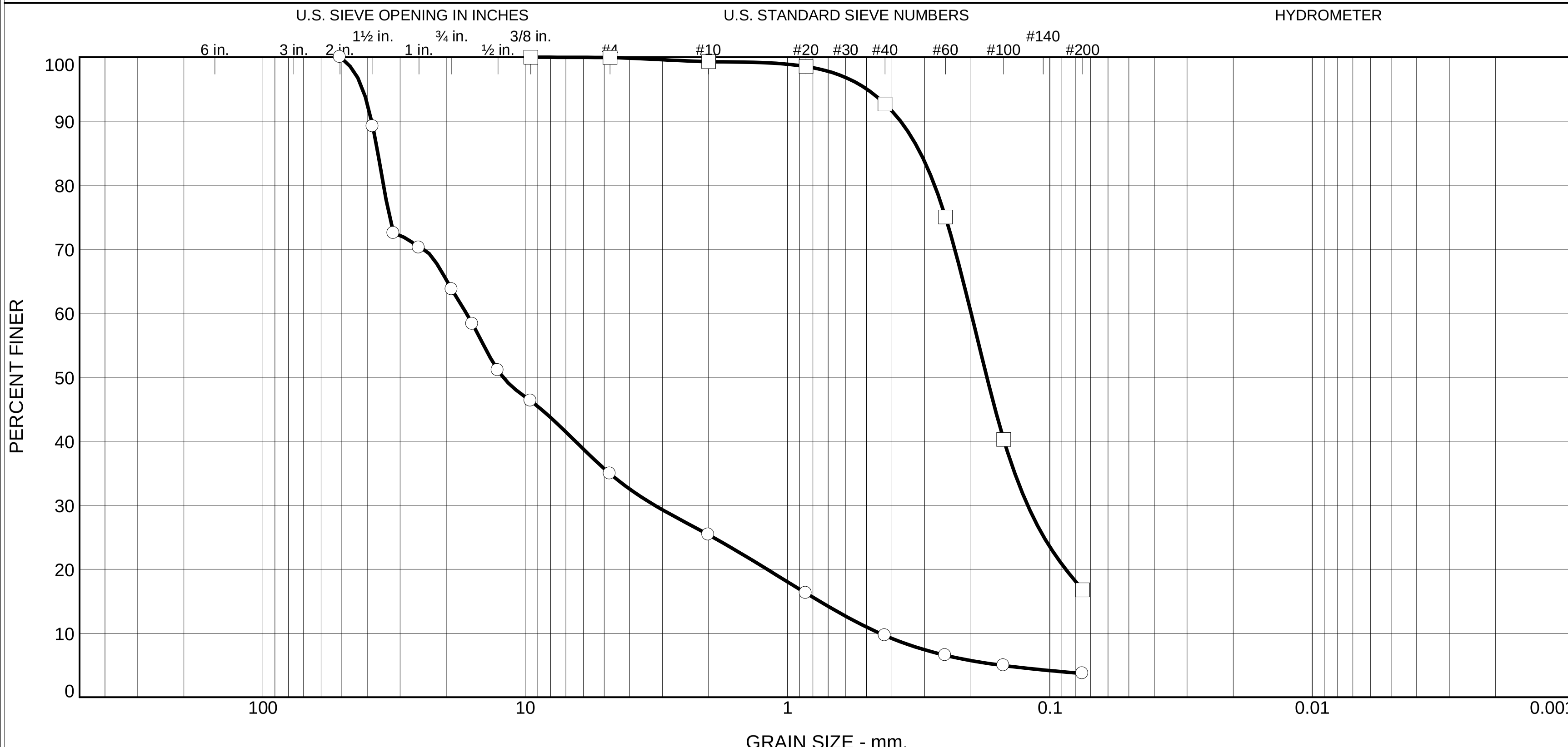
King County

Department of Local Services
Road Services Division

APPENDIX C – REVISION NO. 1
LABORATORY TEST REPORTS

PRELIMINARY GEOTECHNICAL DATA REPORT
JAN ROAD NEIGHBORHOOD IMPROVEMENTS

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	36.3	28.8	9.5	15.8	5.9	3.7	
□	0.0	0.0	0.0	0.7	6.6	75.9	16.8	

	Source	Sample #	Depth/Elev.	Date Sampled	USCS	Material Description	NM %	LL	PL
○	B-1	KC-296	6.5'-10'	5-20-19	GW	Well-graded Gravel with Sand			
□	B-1	KC-293	15'-16.5'	5-20-19	SM	Silty Sand	18.9		

Client King County DNRP

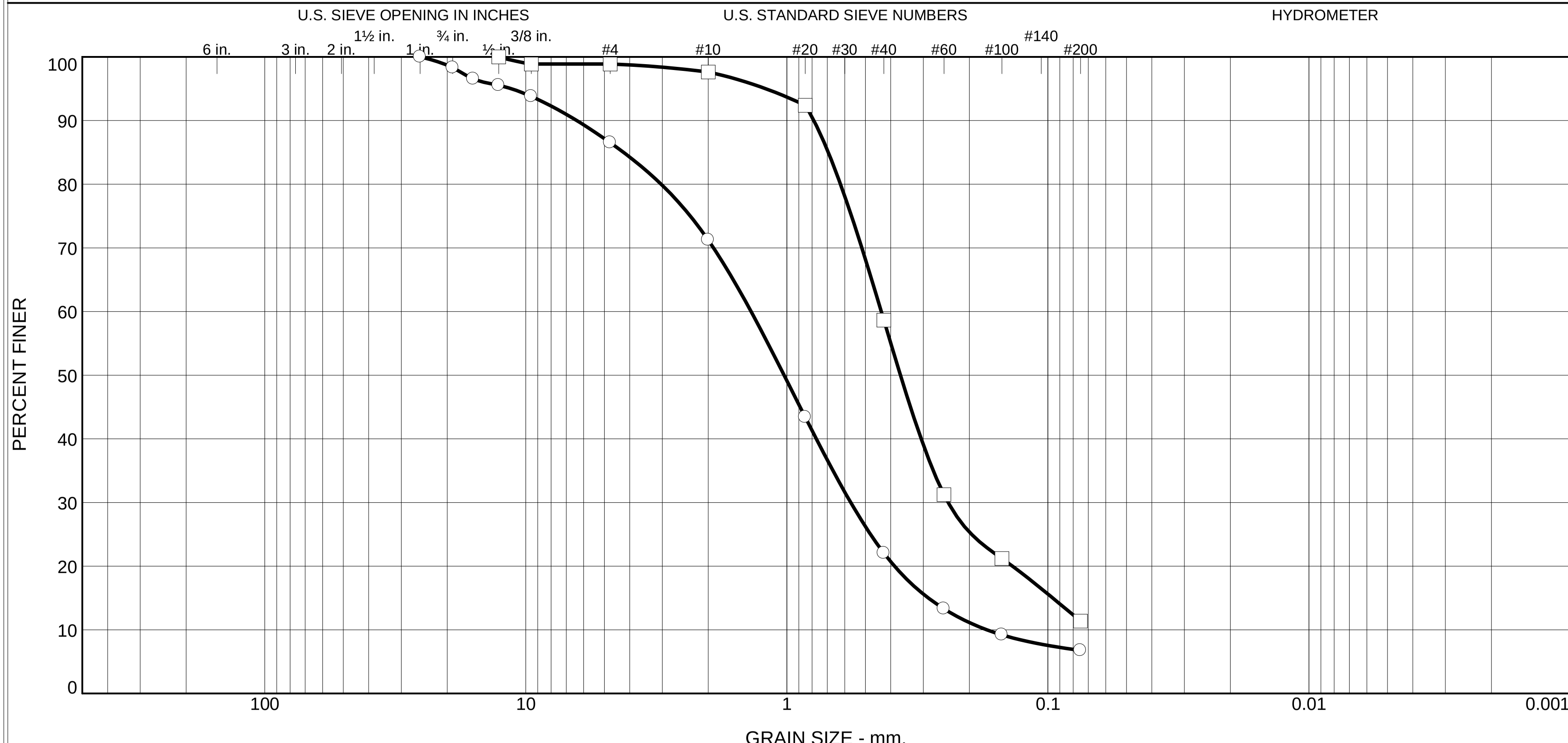
Project Jan Road Neighborhood Improvements

Project No. 1131550 Figure C-1

KING COUNTY

MATERIALS LABORATORY

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	1.7	11.8	15.3	49.1	15.4	6.7	
□	0.0	0.0	1.1	1.3	38.9	47.3	11.4	

	Source	Sample #	Depth/Elev.	Date Sampled	USCS	Material Description	NM %	LL	PL
○	B-1	KC-294	31.5'-35'	5-20-19	SW-SM	Well-graded Sand with Silt			
□	B-1	KC-295	45'-46.5'	5-20-19	SP-SM	Poorly Graded Sand with Silt			

Client King County DNRP

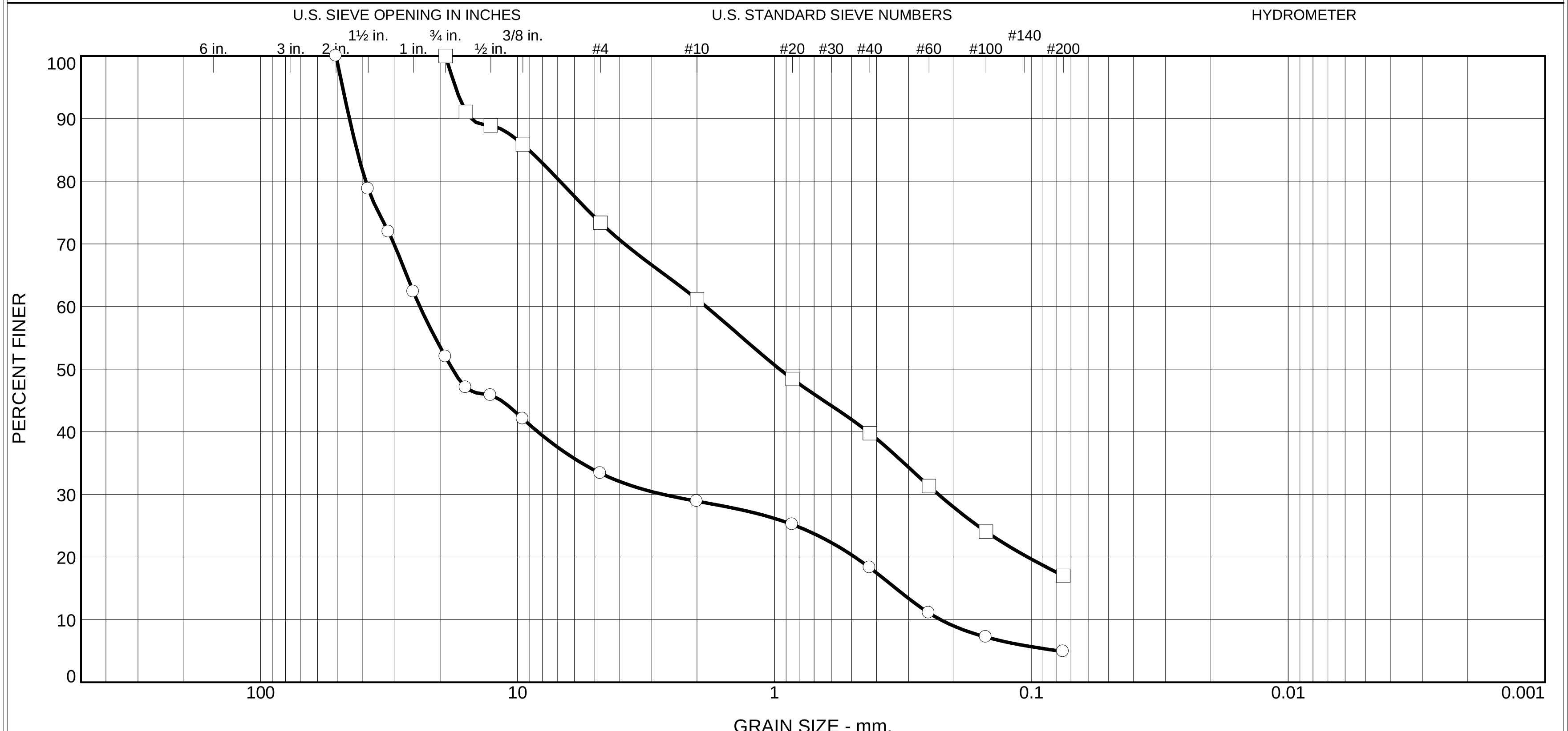
Project Jan Road Neighborhood Improvements

Project No. 1131550 Figure C-2

KING COUNTY

MATERIALS LABORATORY

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	48.0	18.6	4.5	10.6	13.4	4.9	
□	0.0	0.0	26.6	12.2	21.4	22.8	17.0	

	Source	Sample #	Depth/Elev.	Date Sampled	USCS	Material Description	NM %	LL	PL
○	B-2	KC-297	10.5'-15'	5-30-19	GW	Well-graded Gravel with Sand	3.8		
□	B-2	KC-298	5'-6.5'	5-30-19	SM	Silty Sand with Gravel			

Client King County DNRP

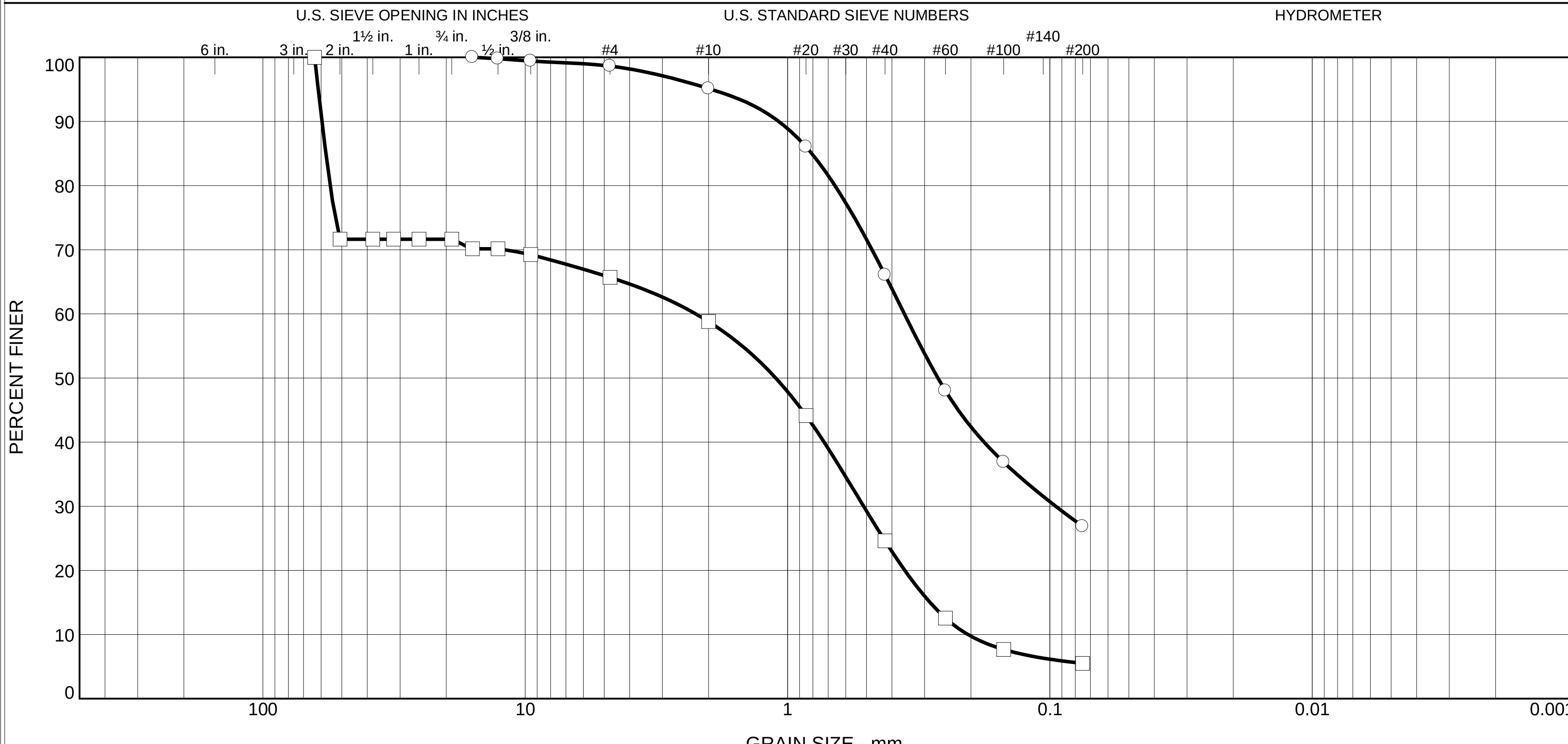
Project Jan Road Neighborhood Improvements

Project No. 1131550 Figure C-3

KING COUNTY

MATERIALS LABORATORY

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.0	1.4	3.5	29.0	39.3	26.8	
□	0.0	28.4	5.9	6.9	34.2	19.1	5.5	

	Source	Sample #	Depth/Elev.	Date Sampled	USCS	Material Description	NM %	LL	PL
○	B-3	KC-299	11.5'-15'	5-31-19	SM	Silty Sand	5.7		
□	B-3	KC-300	21.5'-25'	5-31-19	SP-SM	Poorly Graded Sand with Silt and Gravel			

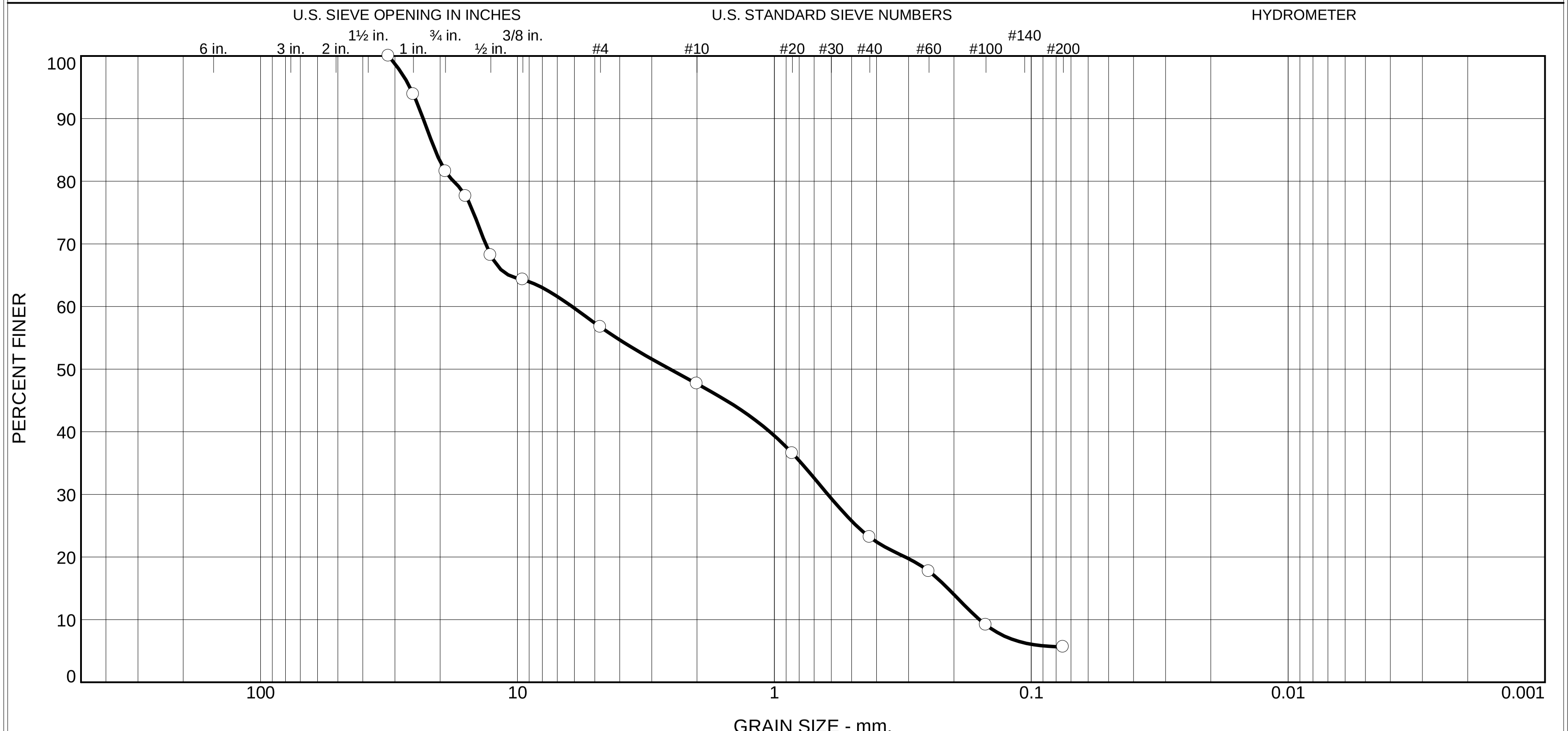
Client King County DNRP
Project Jan Road Neighborhood Improvements

Project No. 1131550 Figure C-4

KING COUNTY

MATERIALS LABORATORY

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	18.5	24.8	9.0	24.5	17.6	5.6	

Source	Sample #	Depth/Elev.	Date Sampled	USCS	Material Description	NM %	LL	PL
B-4	KC-301	10'-11.5'	5-28-19	SP-SM	Poorly Graded Sand with Silt and Gravel	10.7		

Client King County DNRP

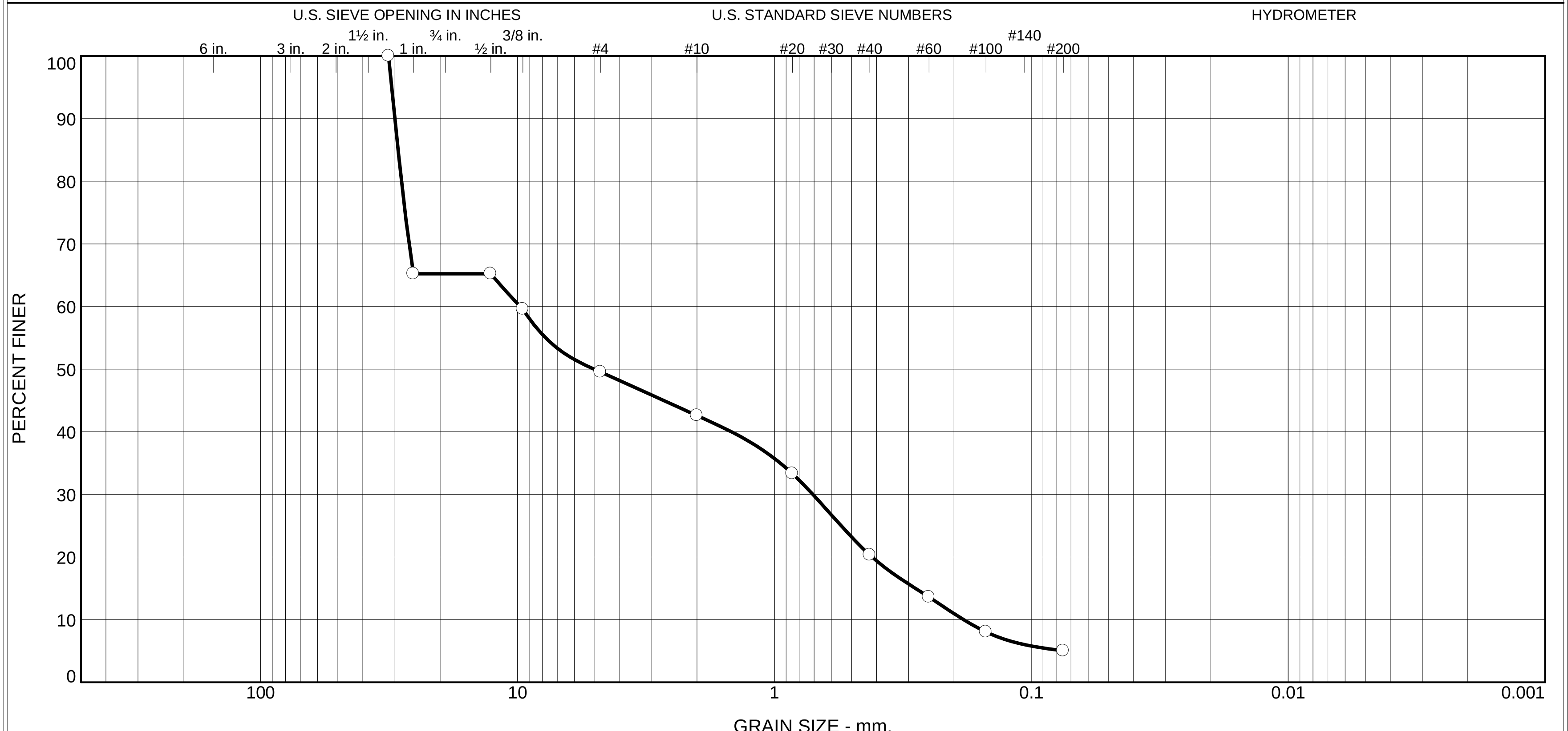
Project Jan Road Neighborhood Improvements

Project No. 1131550 Figure C-5

KING COUNTY

MATERIALS LABORATORY

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	34.8	15.7	6.9	22.3	15.3	5.0	

Source	Sample #	Depth/Elev.	Date Sampled	USCS	Material Description	NM %	LL	PL
B-5	KC-302	5'-6.5'	5-28-19	GP-GM	Poorly Graded Gravel with Silt and Sand			

Client King County DNRP

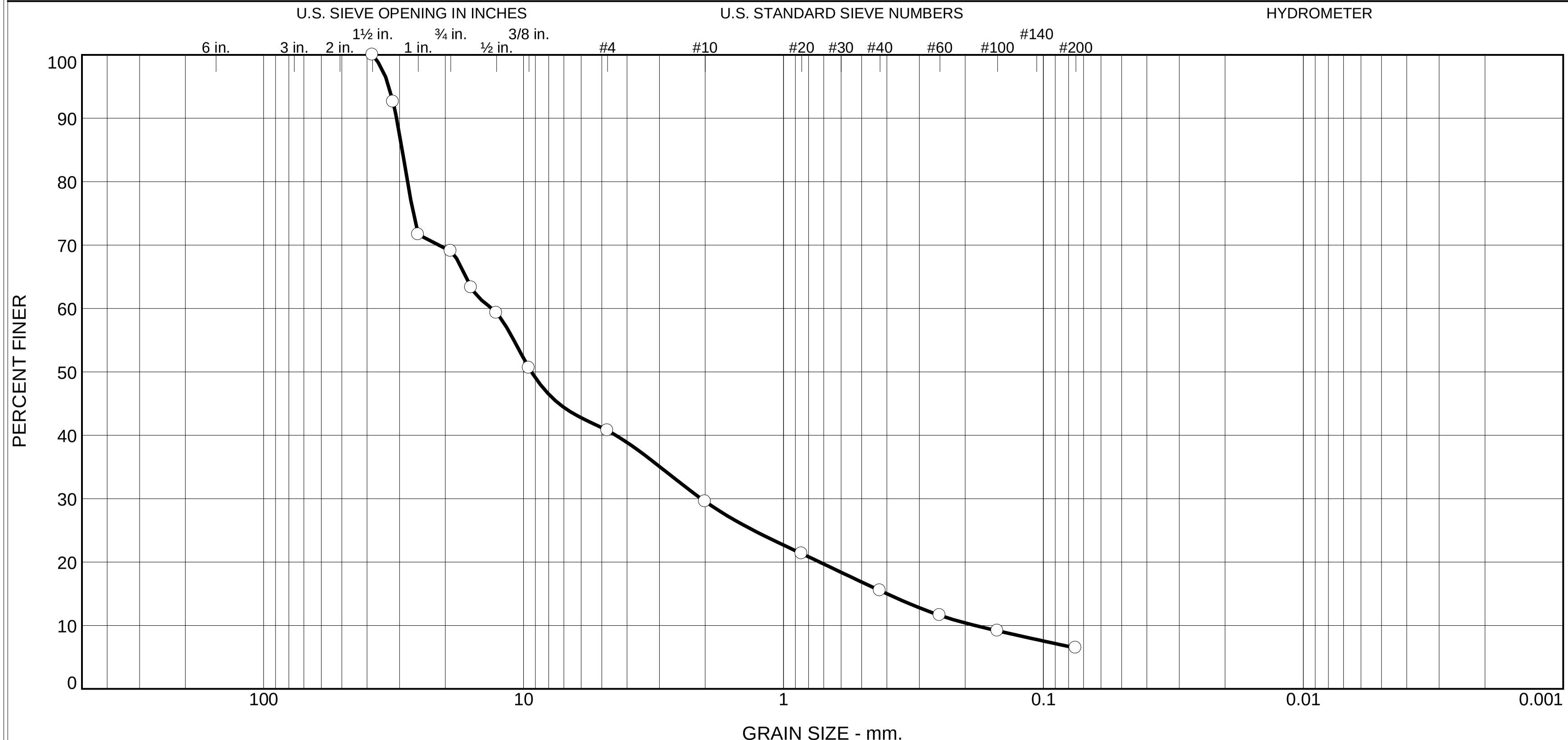
Project Jan Road Neighborhood Improvements

Project No. 1131550 Figure C-6

KING COUNTY

MATERIALS LABORATORY

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	31.0	28.3	11.1	14.1	9.1	6.4	

Source	Sample #	Depth/Elev.	Date Sampled	USCS	Material Description	NM %	LL	PL
B-6	KC-303	10'-11.5'	5-28-19	GW-GM	Well-graded Gravel with Silt and Sand	8.5		

Client King County DNRP

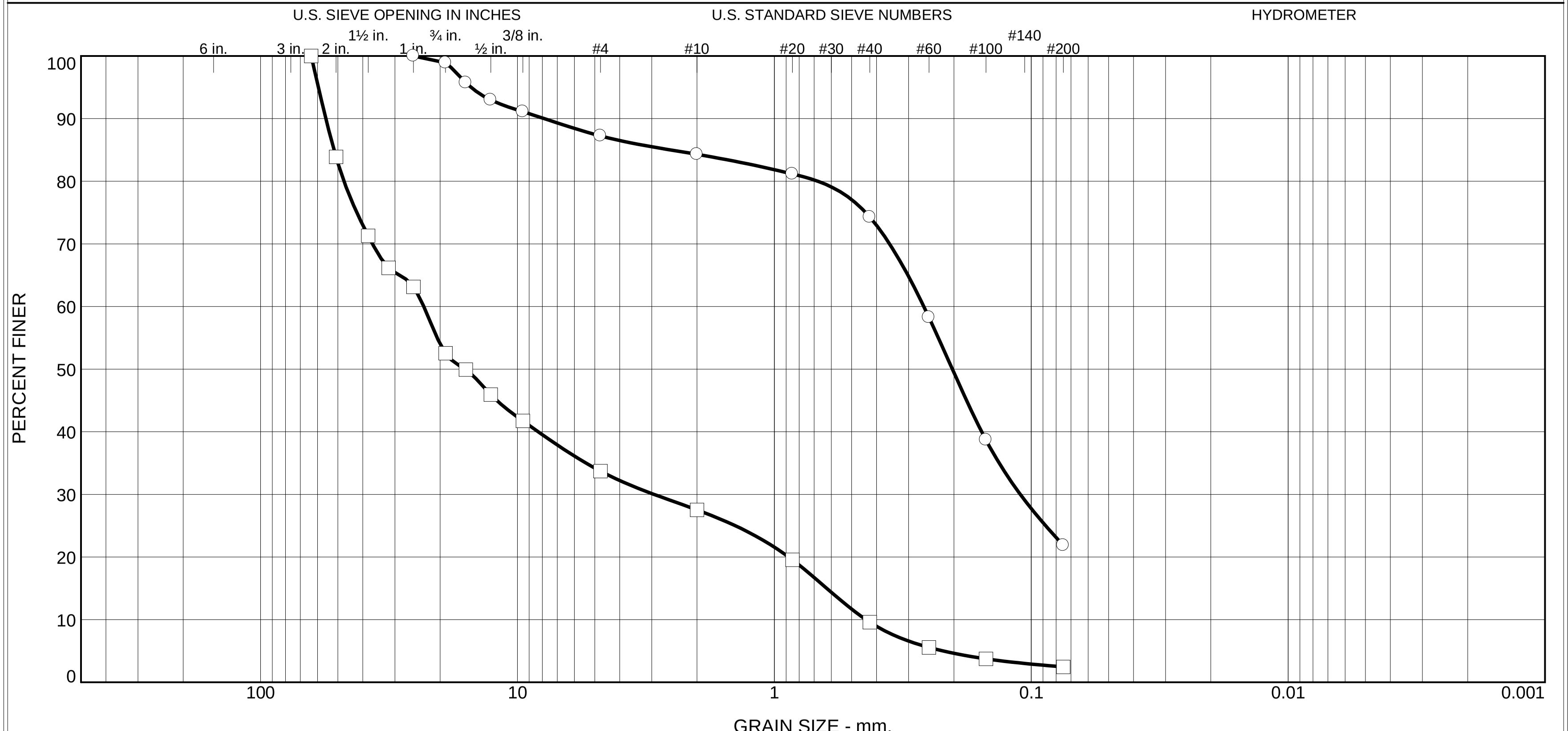
Project Jan Road Neighborhood Improvements

Project No. 1131550 Figure C-7

KING COUNTY

MATERIALS LABORATORY

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	1.1	11.7	2.9	10.0	52.5	21.8	
□	0.0	47.5	18.8	6.2	17.9	7.1	2.5	

	Source	Sample #	Depth/Elev.	Date Sampled	USCS	Material Description	NM %	LL	PL
○	TP1 Levee Toe	KC-306	0.5'-1.5'	5-10-19	SM	Silty Sand	19.6		
□	TP1 Levee Embank.	KC-305	3'-4'	5-10-19	GP	Poorly Graded Gravel with Sand			

Client King County DNRP

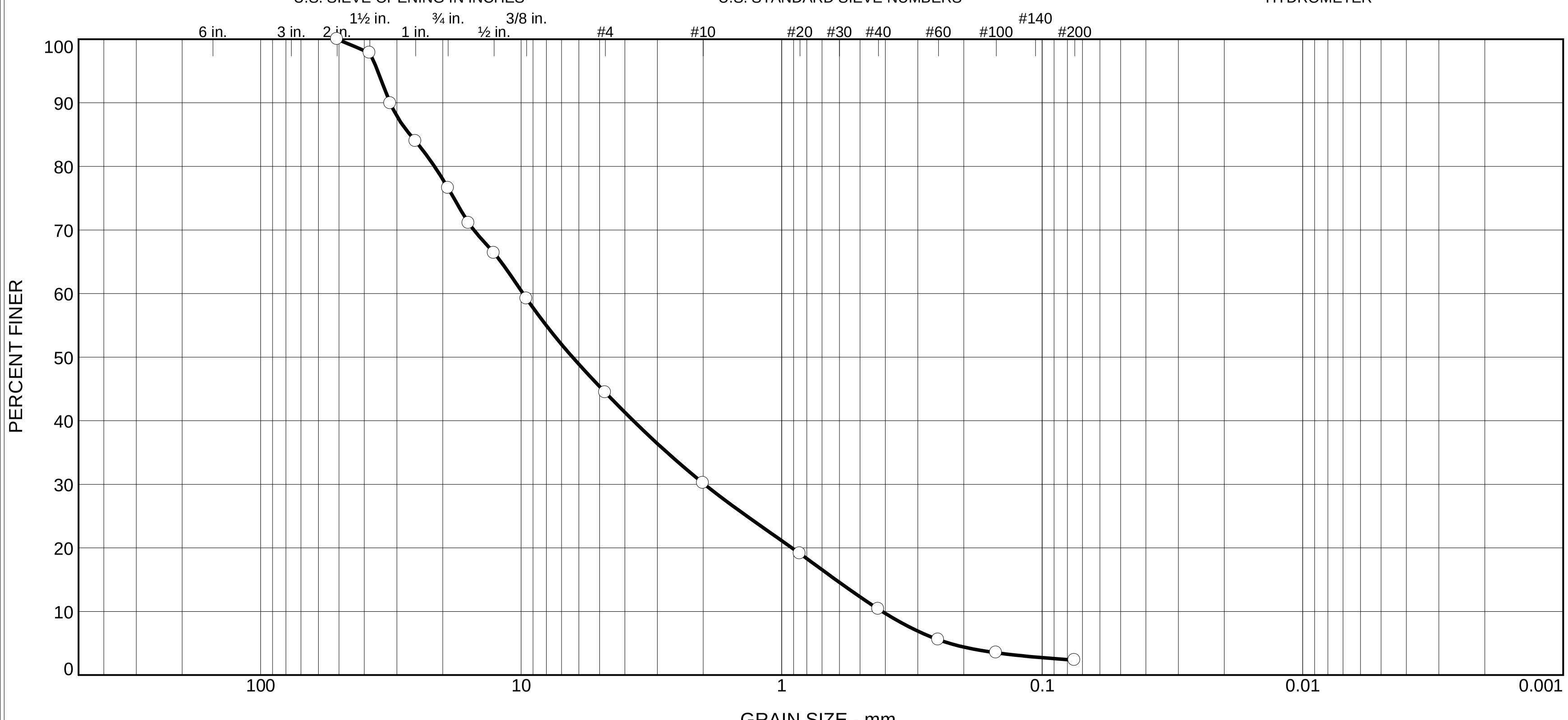
Project Jan Road Neighborhood Improvements

Project No. 1131550 Figure C-8

KING COUNTY

MATERIALS LABORATORY

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	23.4	32.2	14.2	19.8	8.1	2.3	

Source	Sample #	Depth/Elev.	Date Sampled	USCS	Material Description	NM %	LL	PL
TP3 Levee Embank.	KC-307	2.5'-3.5'	5-10-19	GP	Poorly Graded Gravel with Sand	3.6		

Client King County DNRP

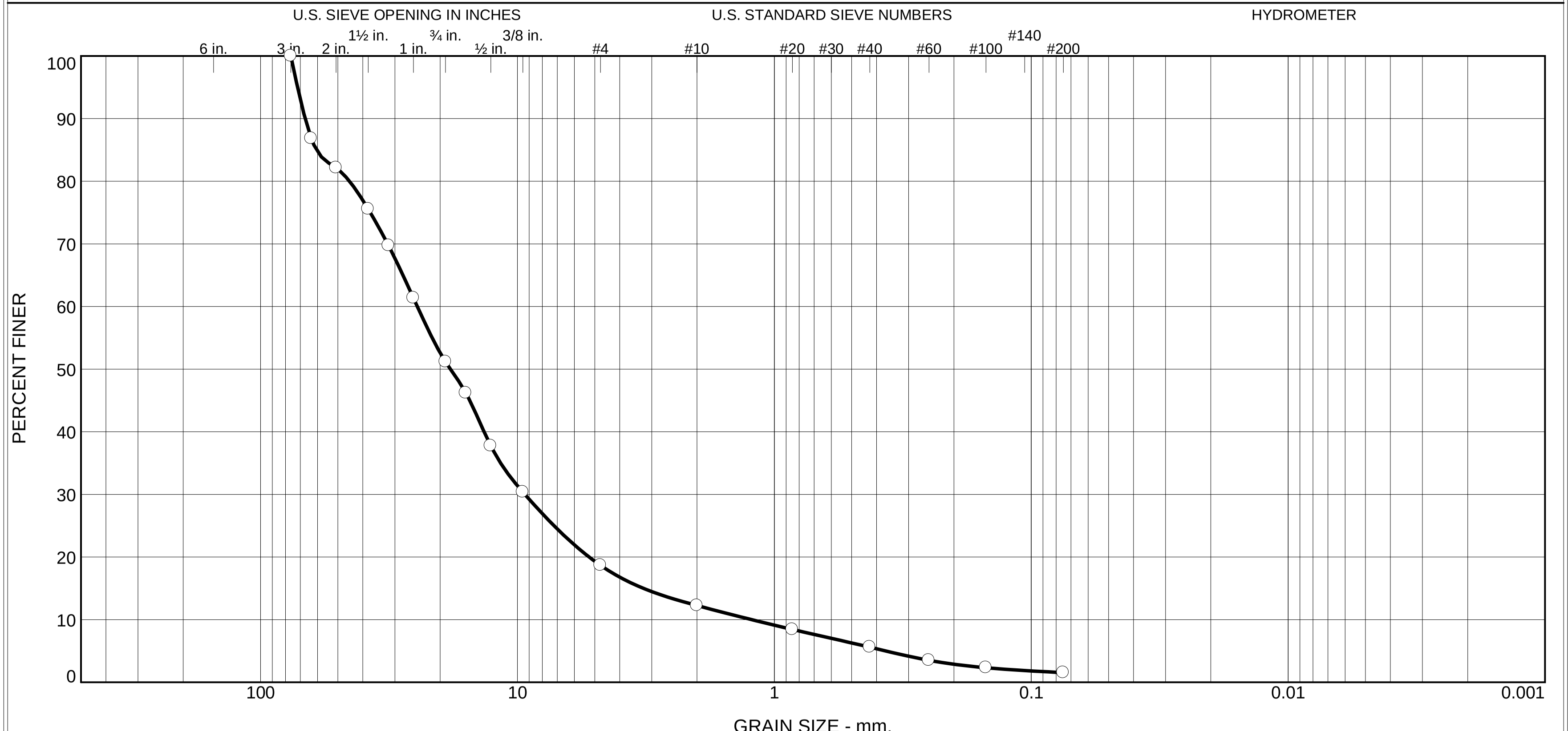
Project Jan Road Neighborhood Improvements

Project No. 1131550 Figure C-9

KING COUNTY

MATERIALS LABORATORY

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	48.8	32.5	6.4	6.7	4.1	1.5	

Source	Sample #	Depth/Elev.	Date Sampled	USCS	Material Description	NM %	LL	PL
TP4 Levee Toe	KC-308	4'-5'	5-13-19	GW	Well-graded Gravel with Sand			

Client King County DNRP

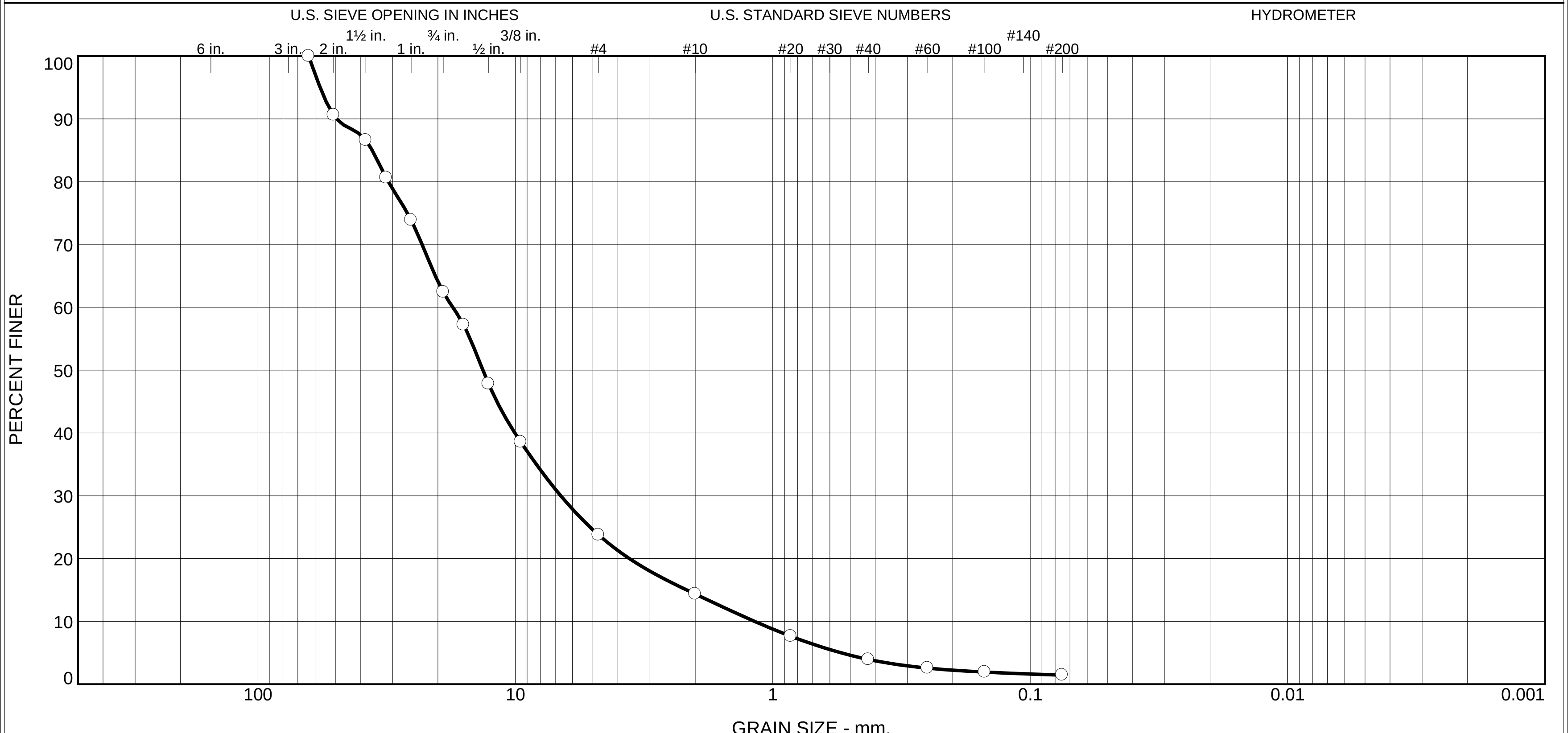
Project Jan Road Neighborhood Improvements

Project No. 1131550 Figure C-10

KING COUNTY

MATERIALS LABORATORY

Particle Size Distribution Report



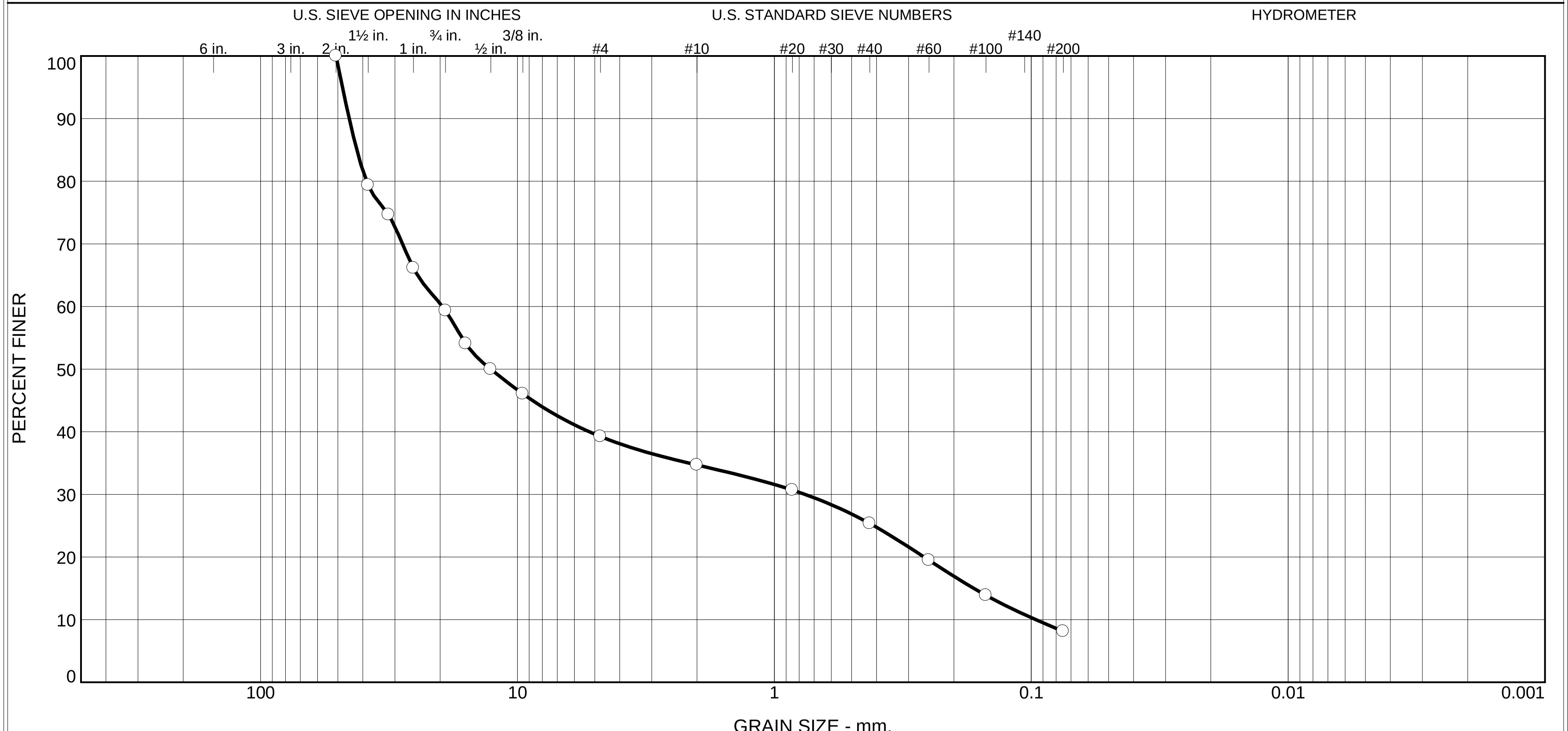
% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	37.6	38.6	9.5	10.4	2.5	1.4	

Source	Sample #	Depth/Elev.	Date Sampled	USCS	Material Description	NM %	LL	PL
TP5	KC-309	4.5'-5.5'	5-10-19	GW	Well-graded Gravel with Sand			

Client King County DNRP
Project Jan Road Neighborhood Improvements
Project No. 1131550 Figure C-11

KING COUNTY
MATERIALS LABORATORY

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	40.7	20.1	4.5	9.3	17.3	8.1	

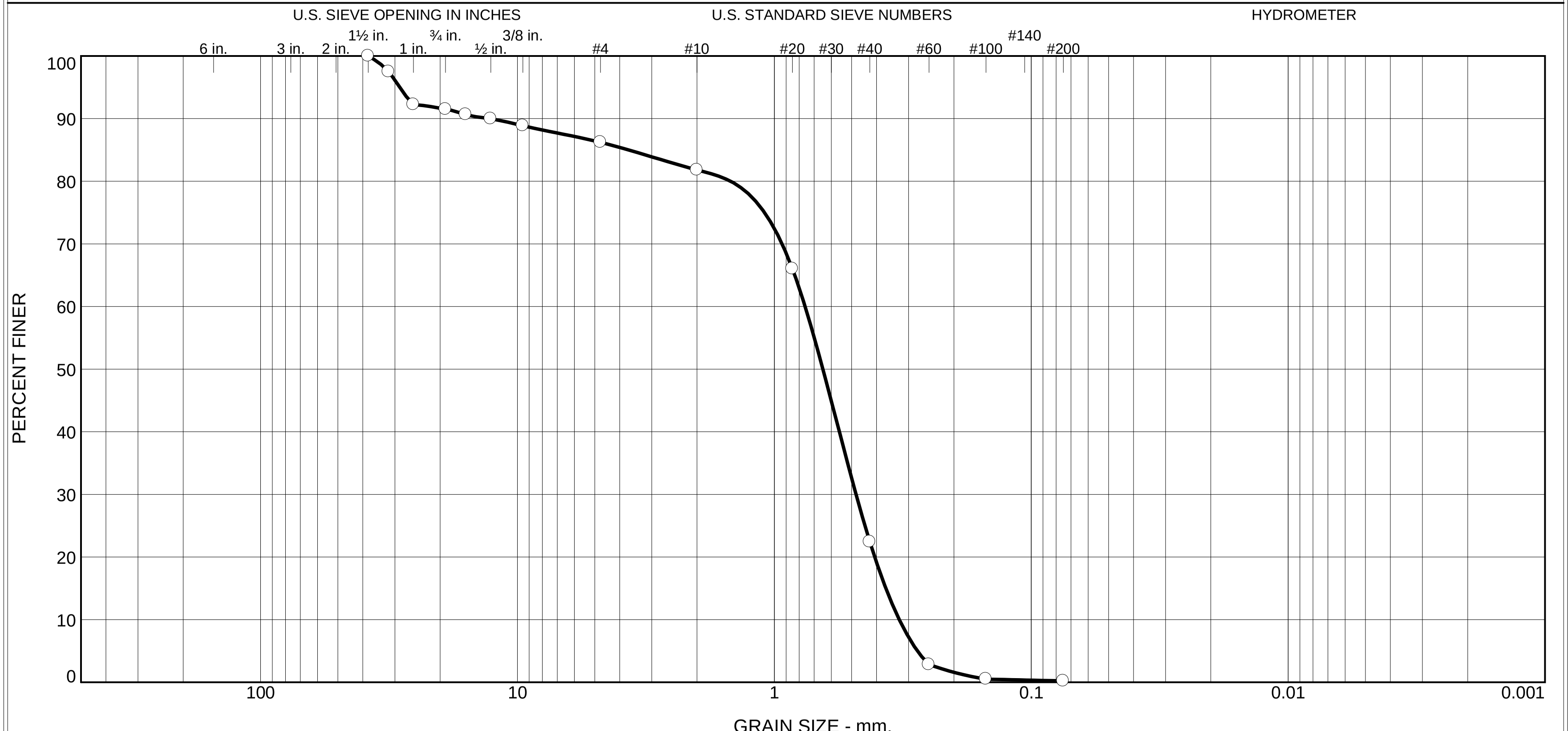
Source	Sample #	Depth/Elev.	Date Sampled	USCS	Material Description	NM %	LL	PL
TP6	KC-310	1'-2.5'	5-13-19	GP-GM	Poorly Graded Gravel with Silt and Sand			

Client King County DNRP
Project Jan Road Neighborhood Improvements
Project No. 1131550 Figure C-12

KING COUNTY

MATERIALS LABORATORY

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	8.5	5.3	4.4	59.4	22.2	0.2	

Source	Sample #	Depth/Elev.	Date Sampled	USCS	Material Description	NM %	LL	PL
TP7	KC-311	2'-3'	5-13-19	SP	Poorly Graded Sand			

Client King County DNRP

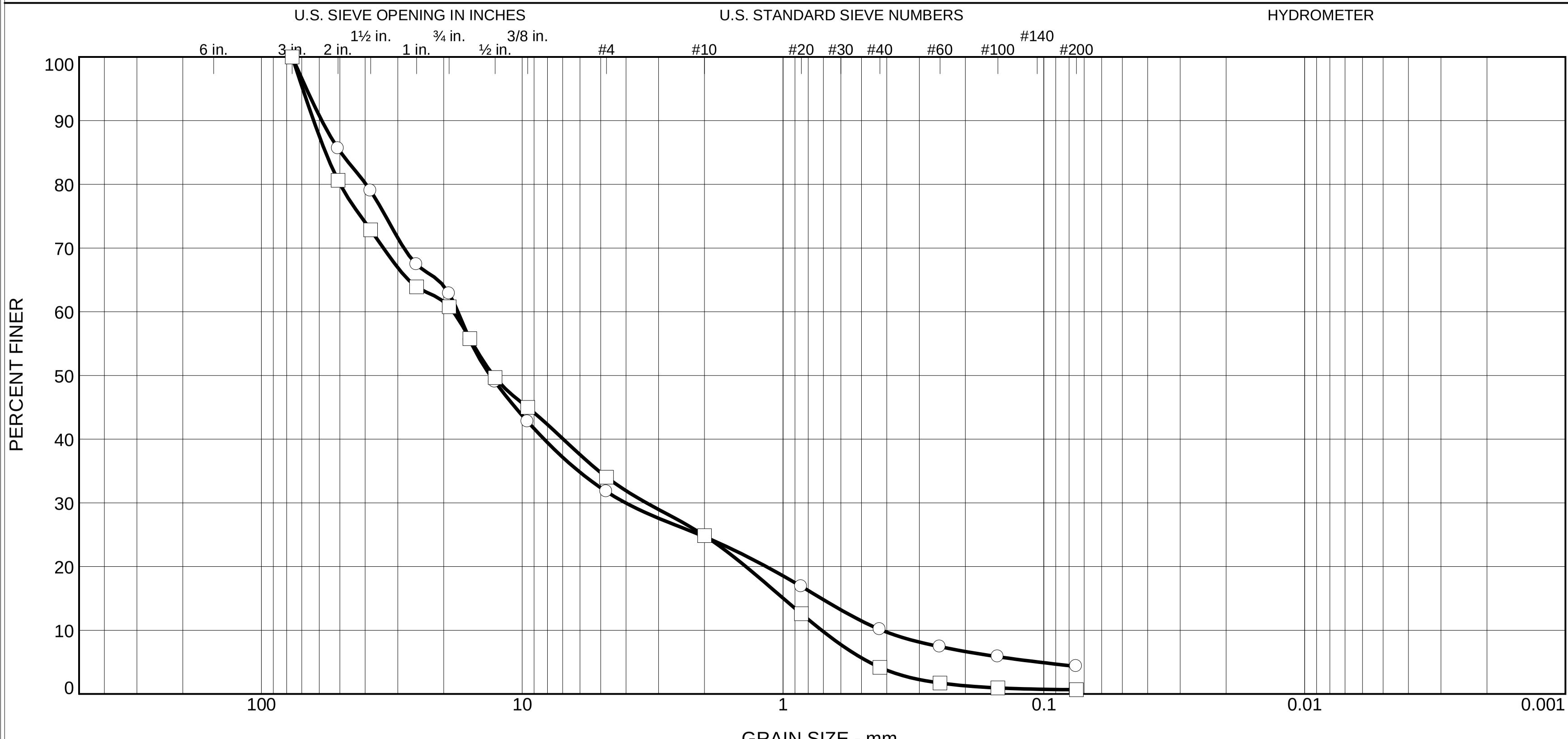
Project Jan Road Neighborhood Improvements

Project No. 1131550 Figure C-13

KING COUNTY

MATERIALS LABORATORY

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	37.2	31.0	7.1	14.6	5.8	4.3	
□	0.0	39.2	26.8	9.1	20.7	3.5	0.7	

	Source	Sample #	Depth/Elev.	Date Sampled	USCS	Material Description	NM %	LL	PL
○	TP-8	KCN-20-47	0' - 1'	3-6-2020	GW	Well-Graded Gravel with Sand	7.1		
□	TP-8	KCN-20-59	2.5' - 8.0'	3-6-2020	GP	Poorly Graded Gravel with Sand	2.9		

Client DNRP

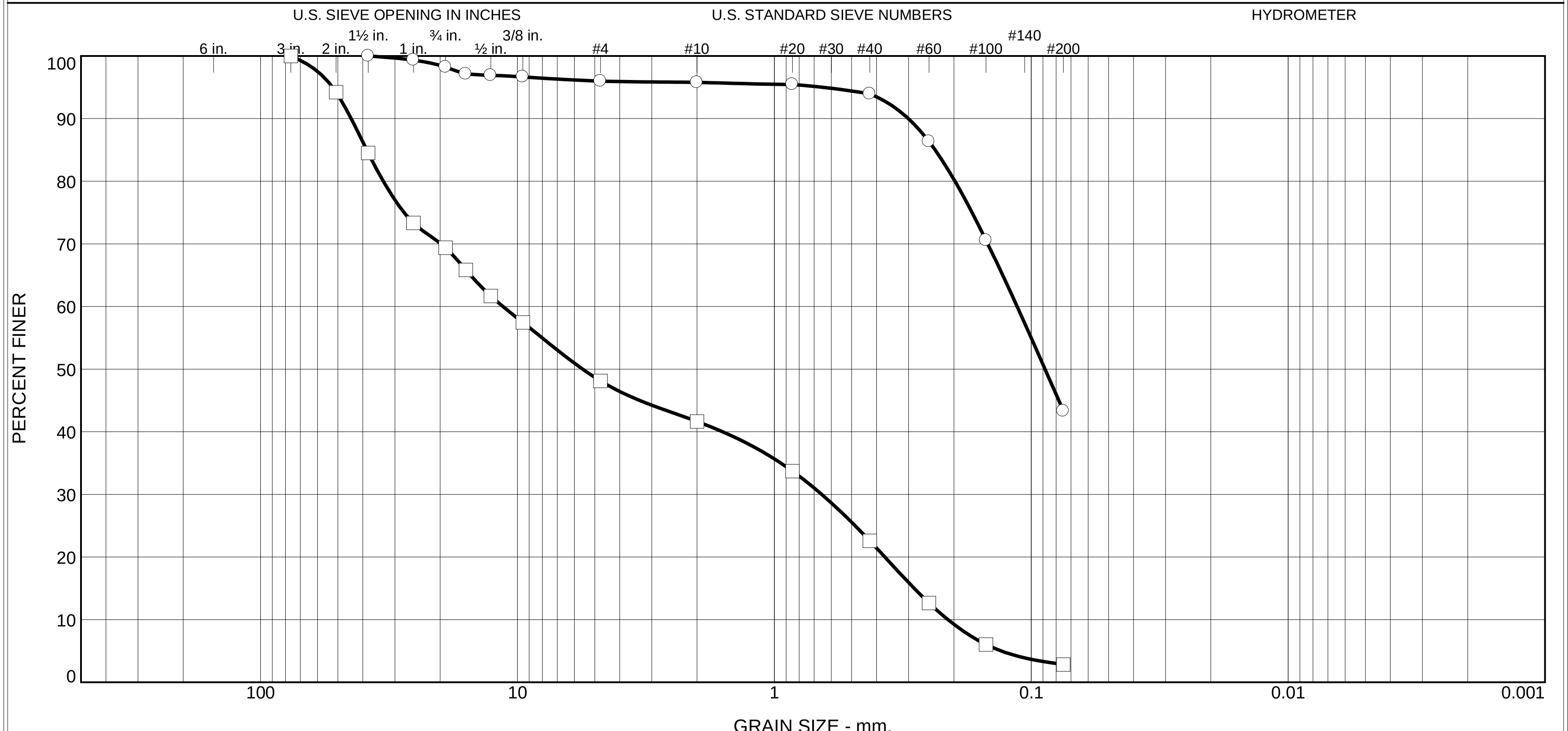
Project Jan Road Neighborhood Improvements

Project No. 1131550 Figure C-14

KING COUNTY

MATERIALS LABORATORY

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	1.8	2.2	0.2	1.8	50.7	43.3	
□	0.0	30.6	21.3	6.5	19.0	19.8	2.8	

	Source	Sample #	Depth/Elev.	Date Sampled	USCS	Material Description	NM %	LL	PL
○	TP-9	KCN-20-50	0.25' - 2.5'	3-6-2020	SM	Silty Sand	26.7		
□	TP-9	KCN-20-61	5' - 8'	3-6-2020	GP	Poorly Graded Gravel with Sand	7.5		

Client DNRP

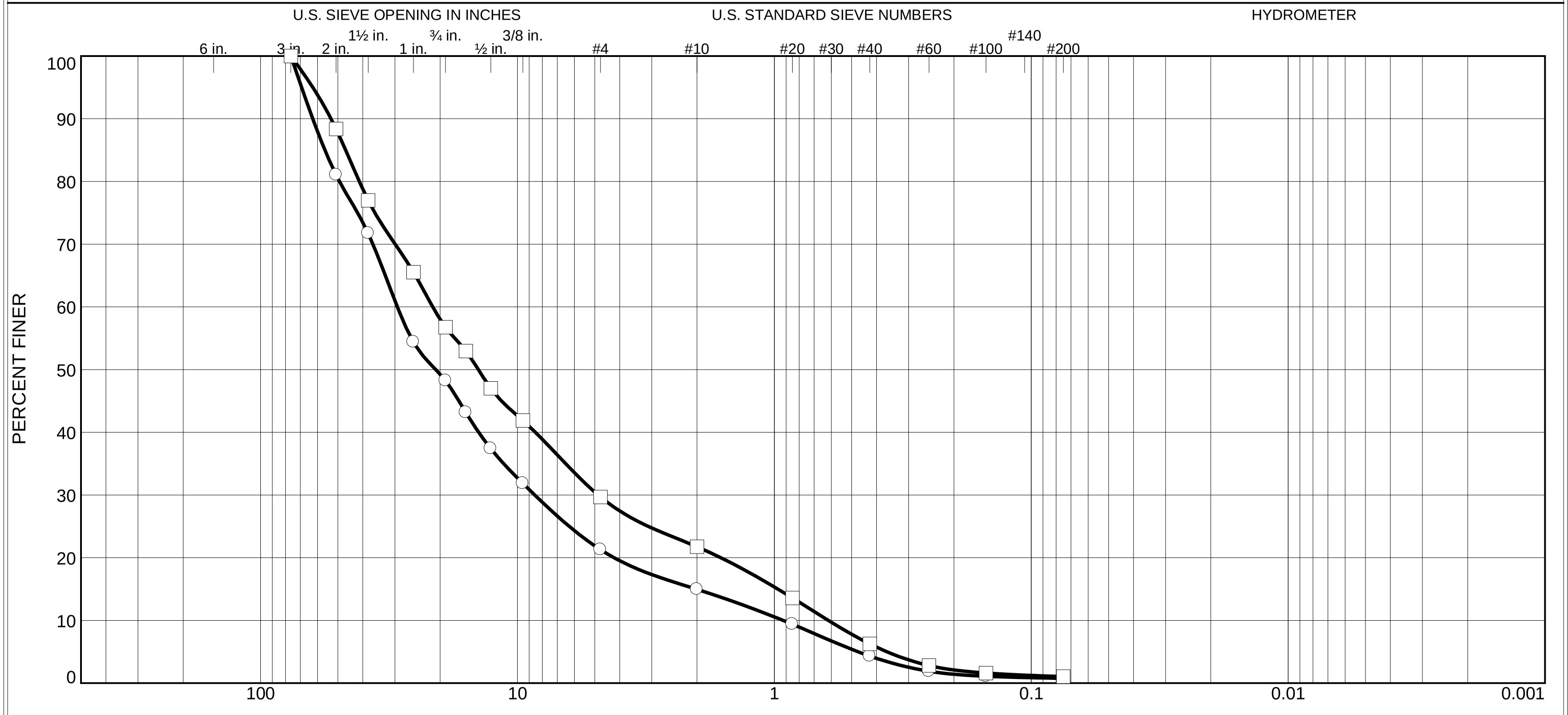
Project Jan Road Neighborhood Improvements

Project No. 1131550 Figure C-15

KING COUNTY

MATERIALS LABORATORY

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	51.8	26.9	6.3	10.7	3.5	0.8	
□	0.0	43.2	27.1	8.0	15.4	5.3	1.0	

	Source	Sample #	Depth/Elev.	Date Sampled	USCS	Material Description	NM %	LL	PL
○	TP-10	KCN-20-53	1' - 3'	3-6-2020	GW	Well-Graded Gravel with Sand	3.3		
□	TP-10	KCN-20-60	5' - 8'	3-6-2020	GW	Well-Graded Gravel with Sand	3.4		

Client DNRP

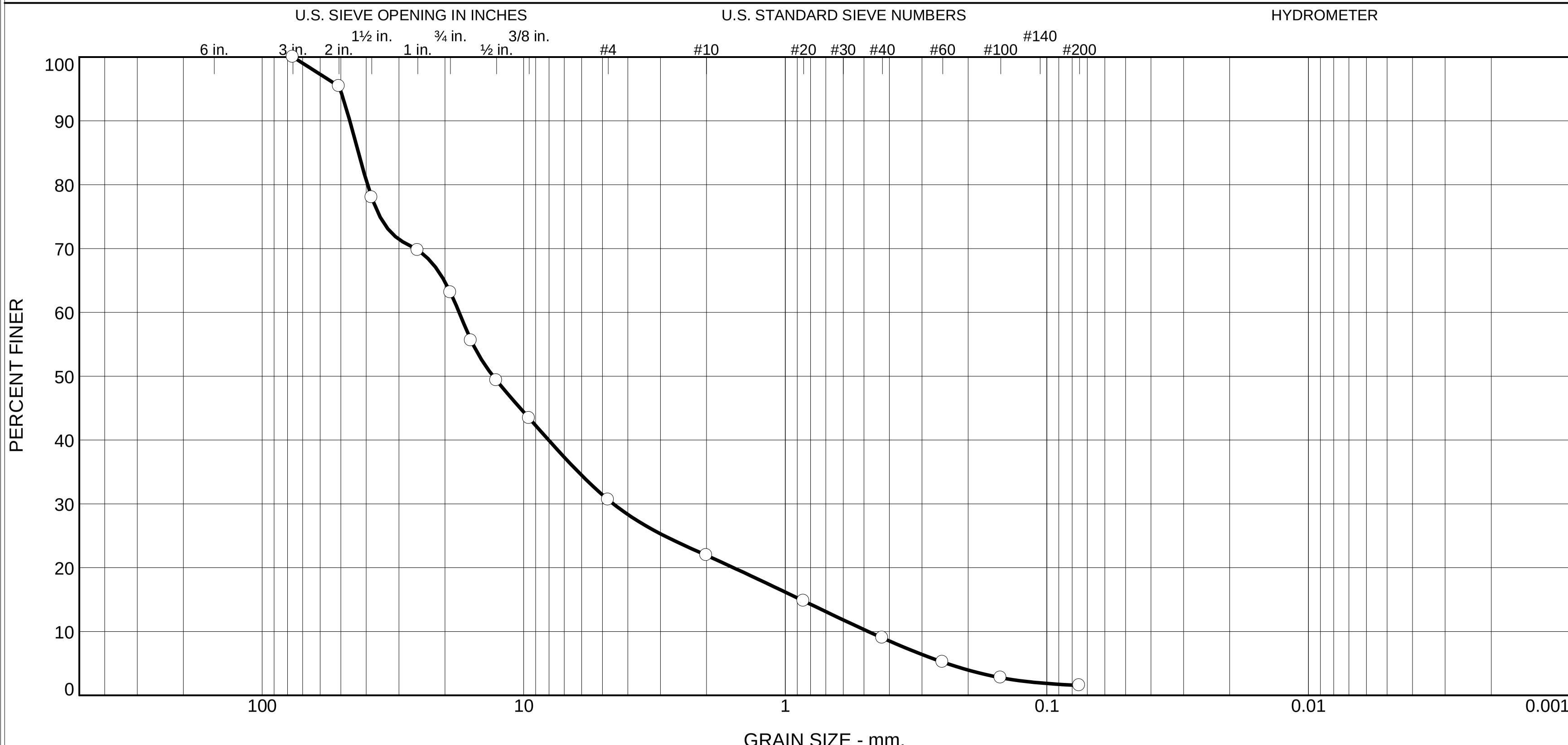
Project Jan Road Neighborhood Improvements

Project No. 1131550 Figure C-16

KING COUNTY

MATERIALS LABORATORY

Particle Size Distribution Report



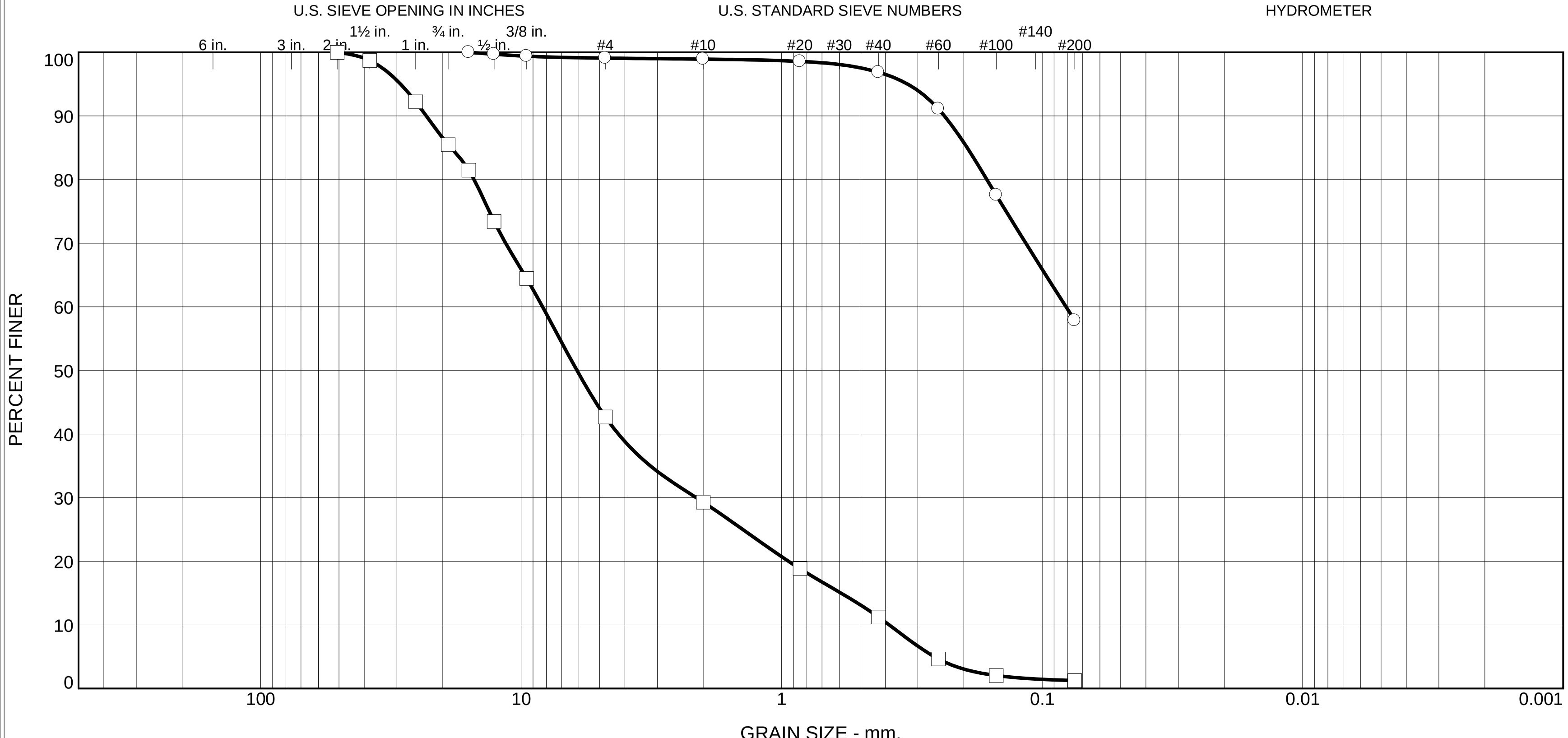
% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	36.9	32.5	8.7	12.9	7.5	1.5	

Source	Sample #	Depth/Elev.	Date Sampled	USCS	Material Description	NM %	LL	PL
TP-11	KCN-20-55	1.7' - 3.5'	3-6-2020	GW	Well-Graded Gravel with Sand	4.7		

Client DNRP
Project Jan Road Neighborhood Improvements
Project No. 1131550 Figure C-17

KING COUNTY
MATERIALS LABORATORY

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.0	0.9	0.2	2.0	39.0	57.9	
□	0.0	14.5	42.8	13.4	18.1	10.0	1.2	

	Source	Sample #	Depth/Elev.	Date Sampled	USCS	Material Description	NM %	LL	PL
○	TP-12	KCN-20-57	0.8' - 1.7	3-6-2020	ML	Sandy Silt	28.5		
□	TP-12	KCN-20-62	3.5' - 5.0'	3-6-2020	GW	Well-Graded Gravel with Sand	7.9		

Client DNRP

Project Jan Road Neighborhood Improvements

Project No. 1131550 Figure C-18

KING COUNTY

MATERIALS LABORATORY

APPENDIX B

King County Revetment Inspection Records (2011 to 2020)

Rivers Facility Inventory Inspection Tickets: 12/10/2018

If you have any questions about this data, please contact **Kyle Comanor** at **206-477-4076** or email **kyle.comanor@kingcounty.gov** .

Facility Name	River Name	Inspection Date	Inspection Type	Is Partial Inspection	Inspection RM	Inspector 1	Flow Condition	Inspection Mode	Is QA/QC	Modified By	Inspection Notes
Cedar Trl 7	Cedar River	5/30/2018	Routine Inspection	No		Hardy, Jessy	Low - toe visible	From River	No	Hardy, Jessy	Float Inspection (2018-05-30) Land Inspection (2018-06-05) Upstream of facility: Oversteepened, toe rock displaced, scour at the toe is evident. Upstream eddy is increasing in size. Along facility: Vegetation is intact. Detritus creates a matting that makes the face of the facility appear smooth and full. However, there are large voids between the large face rocks. Large trees are holding in place. Trail: cracks in the asphalt are evident, which may have been caused by tree root growth.
Cedar Trl 7	Cedar River	9/22/2017	Routine Inspection	No		Heckendorf, Dan	Low - toe visible	Same Bank	No	Heckendorf, Dan	Entire facility significantly oversteepened; Approx 1:1 Toe rock ~2-3' round Approximately 30ft change in elevation from river water surface to trail Concave section (about 20') in about the middle of facility; consider repair. Large cottonwood being undermined at u/s end of facility (height about 80') Scour along section of bank directly upstream of facility; pool reforming migrating towards upstream end of facility, creating potential flanking condition.
Cedar Trl 7	Cedar River	10/9/2014	Routine Inspection	No		Garric, Craig	Low - toe visible	Same Bank	No	Walker, Eric	U/S end BB from bend in river to 1996 repair. Upper bank slope mow? River inspection required. Repair appears intact. No sign of damage visible thru reach but entire bank oversteepened. Unable to see toe from TOB due to veg and steepness and height. Moving d/s thalweg moves to RB
Cedar Trl 7	Cedar River	10/18/2012	Routine Inspection	No		Faegenburg, Nancy	Low - toe visible	Same Bank	No	Walker, Eric	No sign of any damage. Recommend water side inspection for toe- difficult to inspect from land.
Cedar Trl 7	Cedar River	10/6/2011	Routine Inspection	No		Barton, Chase	Low - toe visible	Same Bank	No	Walker, Eric	Very dense vegetation/willows. Hard to see down to toe but no apparent damage.
Cedar Trl 7	Cedar River	2/16/2011	Routine Inspection	No		Butchart, Carolyn	OHW	Same Bank	No	Walker, Eric	Potential fresh scour @ u/s end where river meets trail. Big eddy- doesnt' appear to be immediate danger

You're in: **Facility Inventory Home** > **Inspection Home** > Inspection Ticket

Faciltiy Inspection Ticket

(•) Add Inspection Ticket
( ) Find Inspection Ticket

[Add New Inspection Ticket] [Clone Inspection Ticket]

Update Inspection Ticket

* River:	Cedar River ▼
* Facility:	Cedar Trl 7 ▼
* Inspection Date:	3/9/2020
* Inspection Type:	Post-flood Inspection ▼
Is Partial Inspection?	[ ] *If unchecked then entire facility is inspected*
* RM Downstream:	0
* RM Upstream:	0
* Inspector 1:	Heckendorf, Dan ▼
Inspector 2:	Brummer, Chris ▼
Inspector 3:	Select ... ▼
* Flow Type:	Low - toe visible ▼
* Inspection Mode:	From River ▼
* Equipment:	No Equipment used ▼
* Used Hard Copy?	(•) Yes ( ) No
Is QA/QC?	[x]
Is Valid?	[x] *If unchecked then it is a false entry*
Inspection Notes:	Sections of minor damage observed along approximately 300 feet (total) of facility, primarily focused near upstream end. Damage along this segment consists primarily of: (1) displacement of toe and face rock (observed in channel), (2) generally over-steepened and undercut banks. Scour has undermined numerous large trees, likely to fall into the channel potentially resulting in further damage of the bank (see inspection photos and maps). Flow during inspection approximately 700 cfs (USGS 12119000).

[Submit] [Cancel] [Update Detail Info]

APPENDIX C

Site Reconnaissance Photos

PHOTO 1: Cedar River Trail 7, Near B-2, looking north, pavement cracks in white

PHOTO 2: Cedar River Trail 7, Between B-1 and B-2, looking northwest, area of Trail settlement and vertical offset

PHOTO 3: Cedar River Trail 7, Near B-2, looking south

PHOTO 4: Cedar River Trail 7, Near B-3, looking south

Site Photographs

(Photographs 1 through 4)

Stability Evaluation and Risk Assessment, CRT7 Revetment
Jan Road Neighborhood Improvements
King County, Washington

Aspect CONSULTING	JUNE-2020	BY: MO	FIGURE NO.
	PROJECT NO. 190175	REVIEWED BY: AJH	C-1

PHOTO 5: River Left Bank, approximately River Mile13.17, looking west at toe of Revetment

PHOTO 6: River Left Bank, approximately River Mile 13.17, looking west at toe of Revetment (2)

PHOTO 7: River Left Bank, approximately River Mile 13.17, looking northwest

PHOTO 8: River Left Bank, approximately River Mile 13.17, looking north

Site Photographs
(Photographs 5 through 8)

Stability Evaluation and Risk Assessment, CRT7 Revetment
Jan Road Neighborhood Improvements
King County, Washington

Aspect CONSULTING	JUNE-2020	BY: MO	FIGURE NO.
	PROJECT NO. 190175	REVIEWED BY: AJH	C-2

PHOTO 9: River Right Bank, approximately River Mile 13.15, looking southwest

PHOTO 10: River Right Bank, approximately River Mile 13.15, looking west

PHOTO 11: River Right Bank, approximately River Mile 13.15, looking northwest

PHOTO 12: River Right Bank, approximately River Mile 13.15, looking northwest (close-up)

Site Photographs
(Photographs 9 through 12)

Stability Evaluation and Risk Assessment, CRT7 Revetment
Jan Road Neighborhood Improvements
King County, Washington

Aspect CONSULTING	JUNE-2020	BY: MO	FIGURE NO.
	PROJECT NO. 190175	REVIEWED BY: AJH	C-3

PHOTO 13: River Right Bank, approximately River Mile 13.1, looking southwest

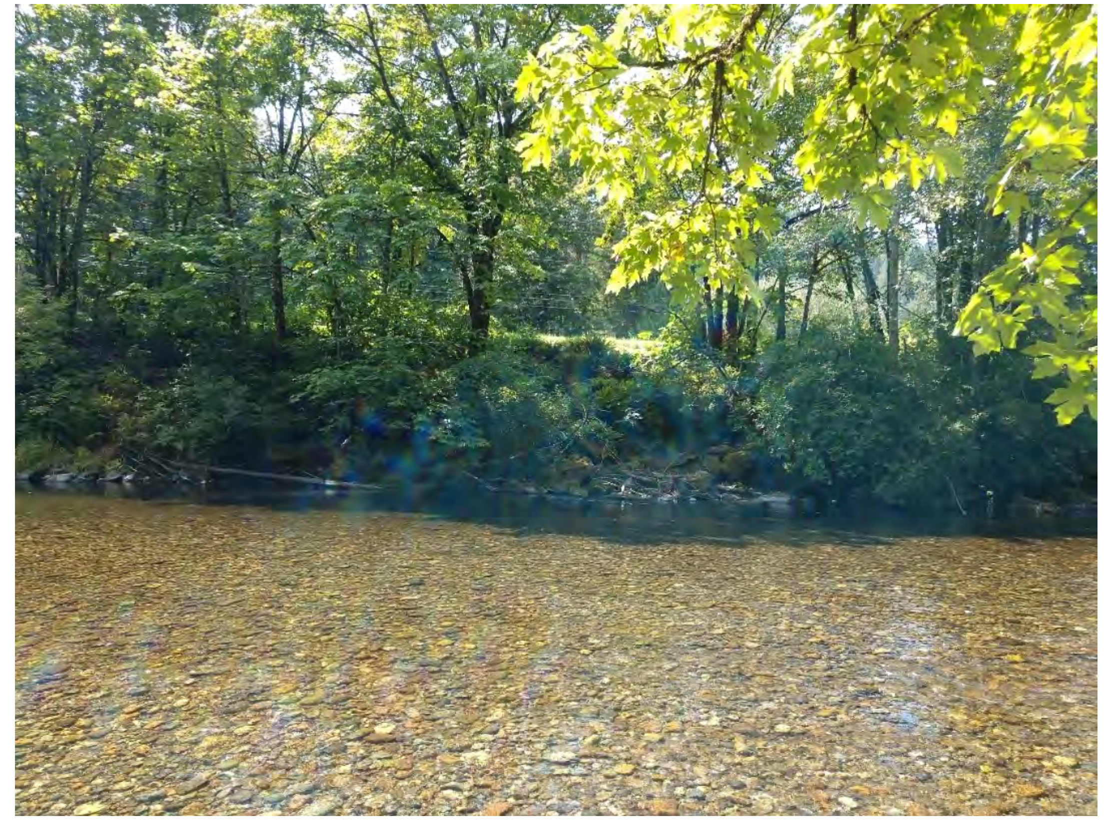


PHOTO 14: River Right Bank, approximately River Mile 13.1, looking west

PHOTO 15: Revetment Rock (typical)

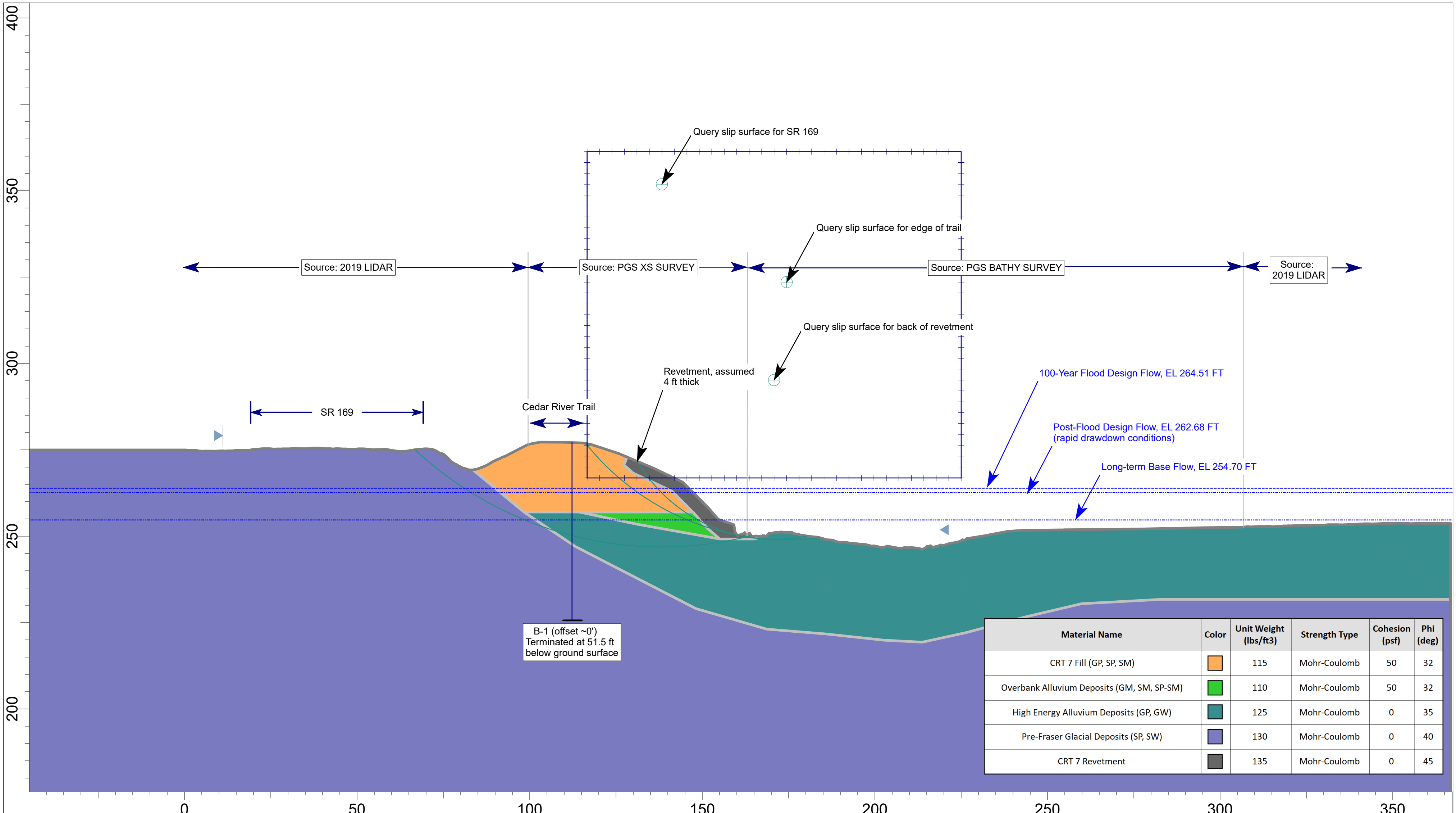
Site Photographs
(Photographs 13 through 15

Stability Evaluation and Risk Assessment, CRT7 Revetment
Jan Road Neighborhood Improvements
King County, Washington

Aspect CONSULTING	JUNE-2020	BY: MO	FIGURE NO.
	PROJECT NO. 190175	REVIEWED BY: AJH	C-4

APPENDIX D

Slope Stability Analysis Outputs
Section B-B’



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
CRT 7 Fill (GP, SP, SM)		115	Mohr-Coulomb	50	32
Overbank Alluvium Deposits (GM, SM, SP-SM)		110	Mohr-Coulomb	50	32
High Energy Alluvium Deposits (GP, GW)		125	Mohr-Coulomb	0	35
Pre-Fraser Glacial Deposits (SP, SW)		130	Mohr-Coulomb	0	40
CRT 7 Revetment		135	Mohr-Coulomb	0	45

Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

Cross Section B-B', Existing Conditions, Non-Scoured Conditions Master Scenario

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment, CRT7 Revetment
Jan Road Neighborhood Improvements
King County, Washington

SLIDE 8.020

SCALE: 1:480

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Aspect CONSULTING

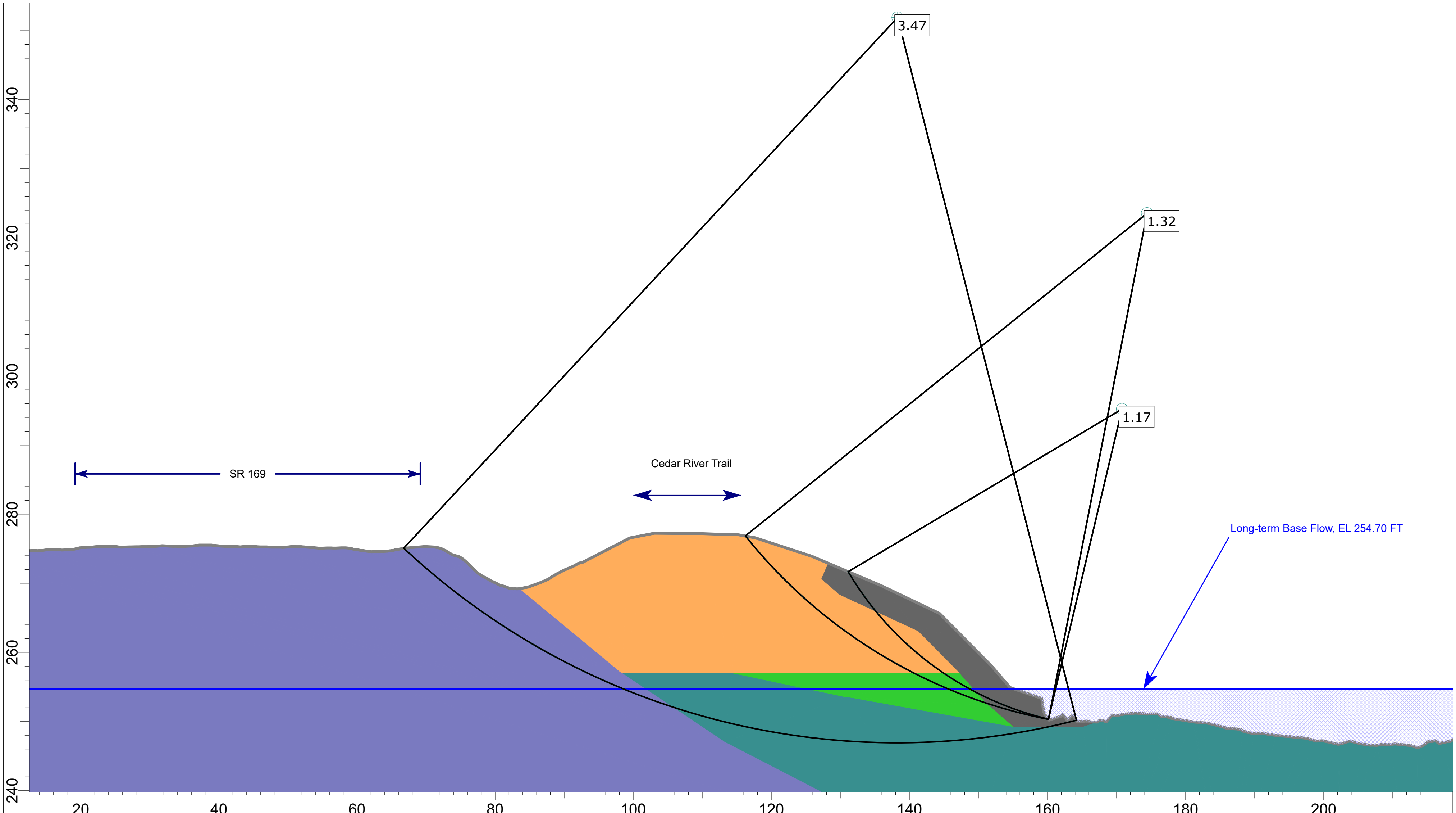
6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-1**



Legend

- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Existing Conditions, Non-Scoured Conditions Static Conditions

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment, CRT7 Revetment
Jan Road Neighborhood Improvements
King County, Washington

SLIDEINTERPRET 8.020

SCALE: 1:240

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Aspect CONSULTING

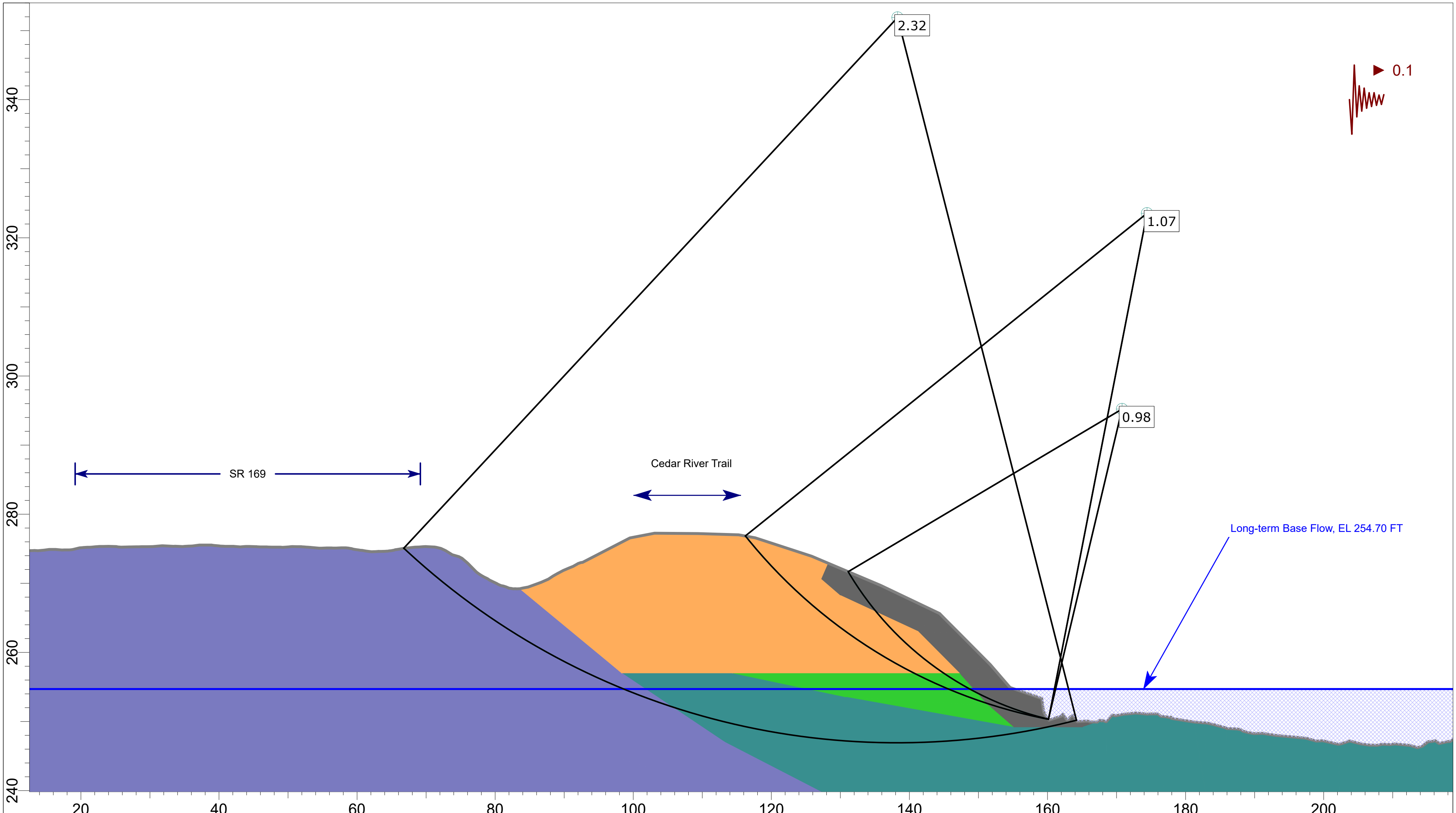
6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-2**



Legend

- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Existing Conditions, Non-Scoured Conditions Seismic Conditions

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment, CRT7 Revetment
Jan Road Neighborhood Improvements
King County, Washington

SLIDEINTERPRET 8.020

SCALE: 1:240

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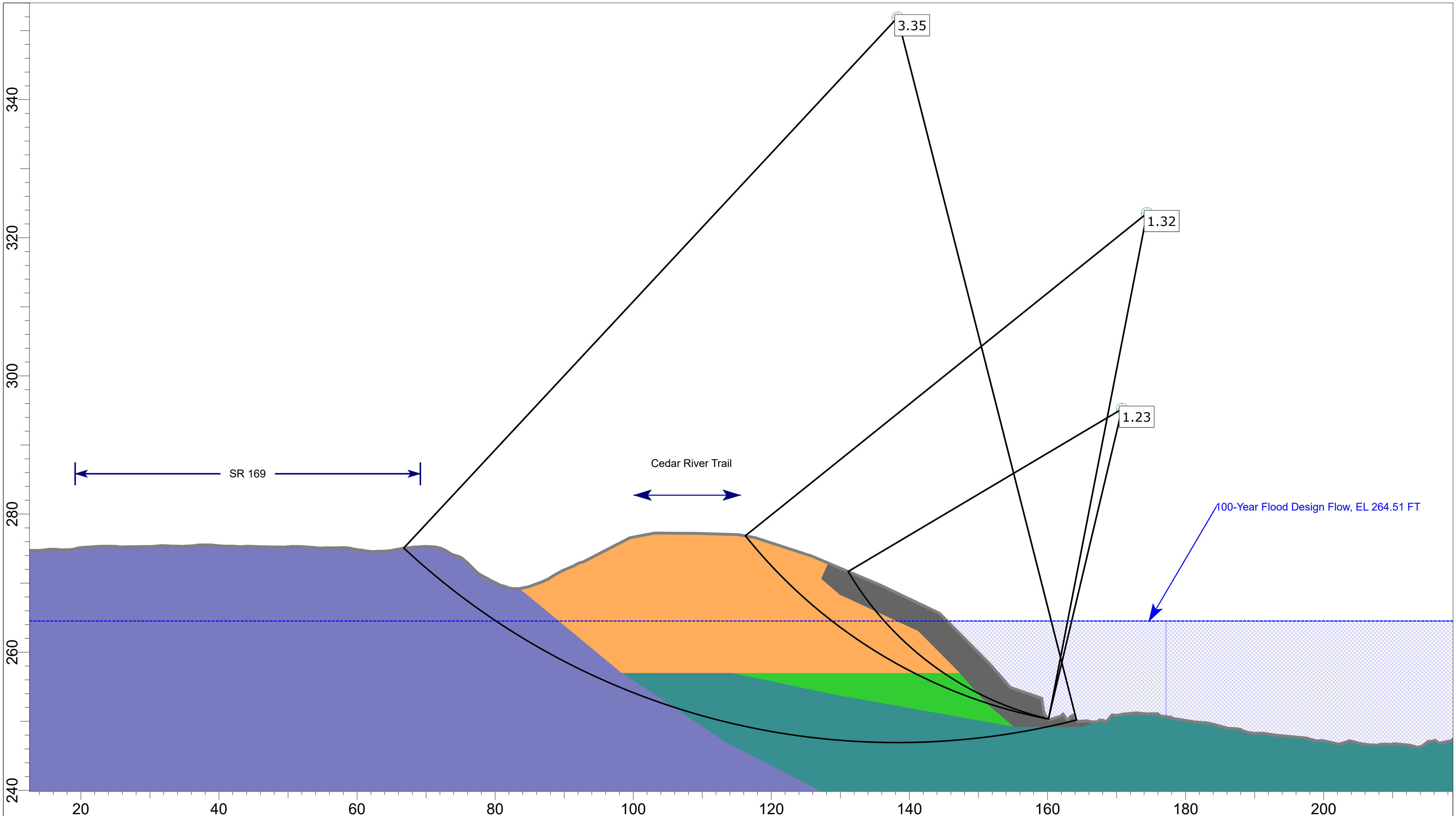
Aspect CONSULTING

6/23/2020	BY: MO	APPENDIX:
PROJECT NO. 190175	REVIEWED BY: AJH	D-3



Legend

- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Existing Conditions, Non-Scoured Conditions Peak Flood Level

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment, CRT7 Revetment
Jan Road Neighborhood Improvements
King County, Washington

SLIDEINTERPRET 8.020

SCALE: 1:240

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Aspect CONSULTING

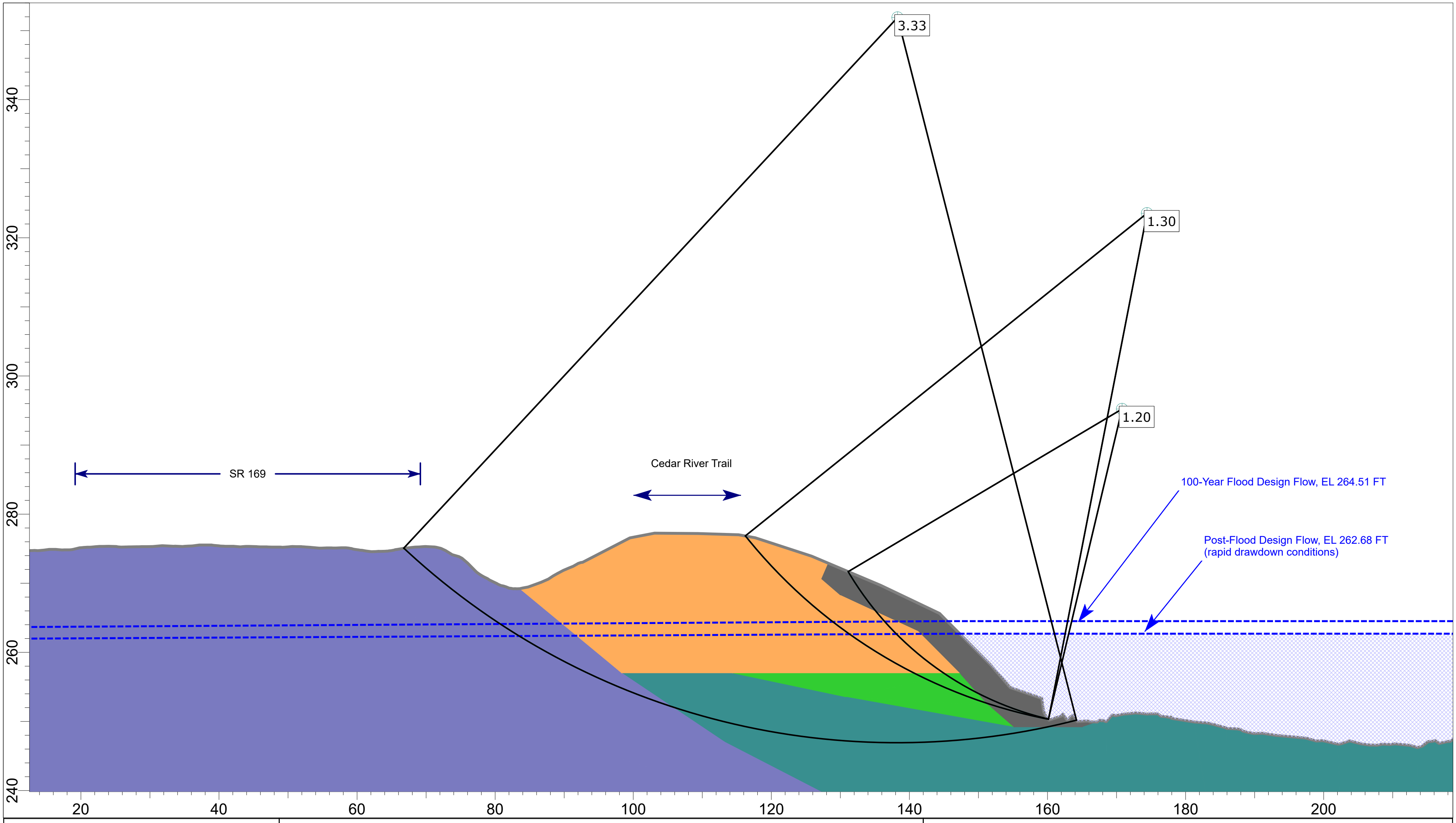
6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-4**



Legend

- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Existing Conditions, Non-Scoured Conditions Rapid Drawdown

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment, CRT7 Revetment
Jan Road Neighborhood Improvements
King County, Washington

SLIDEINTERPRET 8.020

SCALE: 1:240

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Aspect CONSULTING

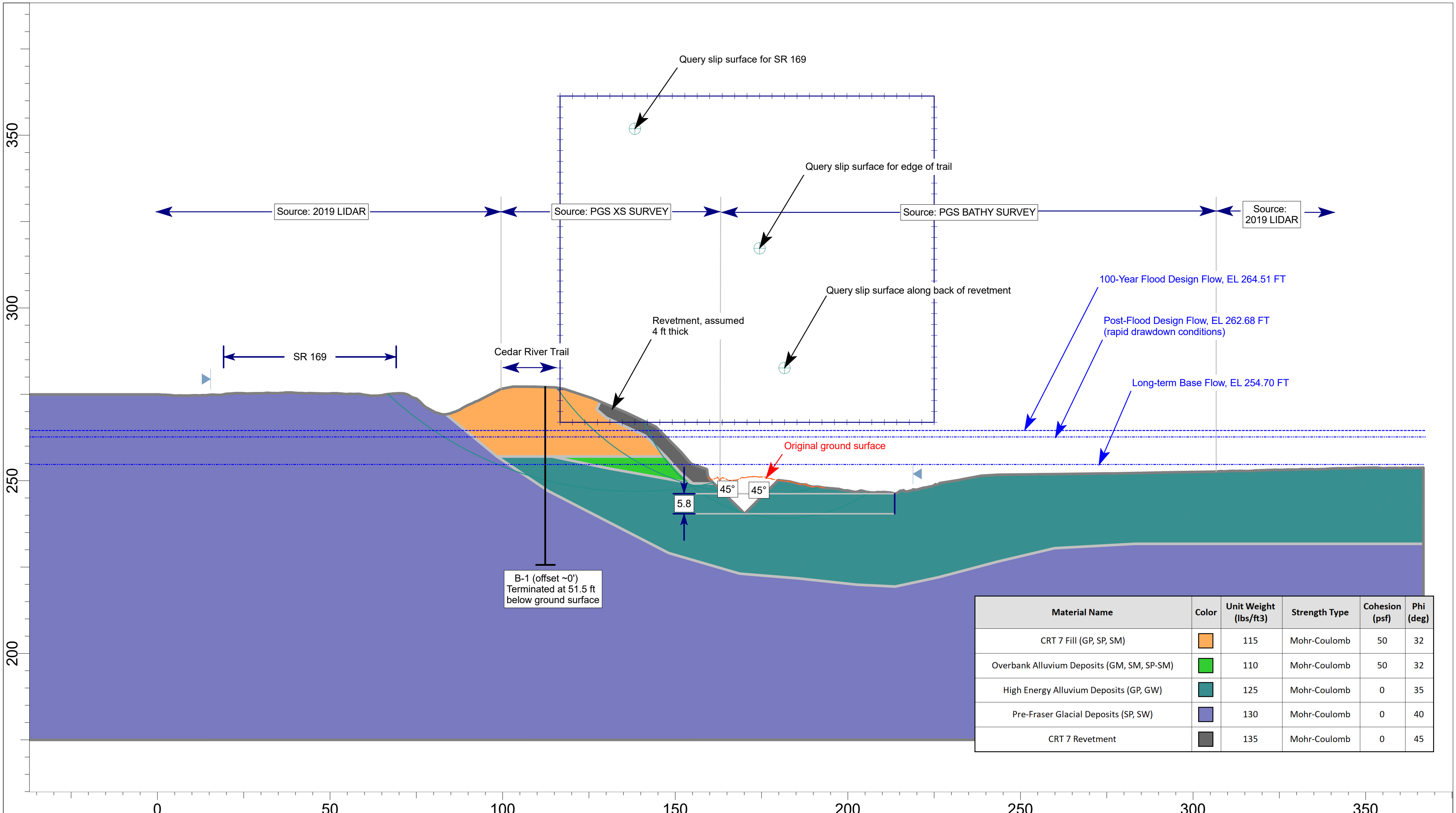
6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-5**



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
CRT 7 Fill (GP, SP, SM)		115	Mohr-Coulomb	50	32
Overbank Alluvium Deposits (GM, SM, SP-SM)		110	Mohr-Coulomb	50	32
High Energy Alluvium Deposits (GP, GW)		125	Mohr-Coulomb	0	35
Pre-Fraser Glacial Deposits (SP, SW)		130	Mohr-Coulomb	0	40
CRT 7 Revetment		135	Mohr-Coulomb	0	45

Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

Cross Section B-B', Existing Conditions With Potential Scour Master Scenario

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDE 8.020

SCALE: 1:480

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Aspect CONSULTING

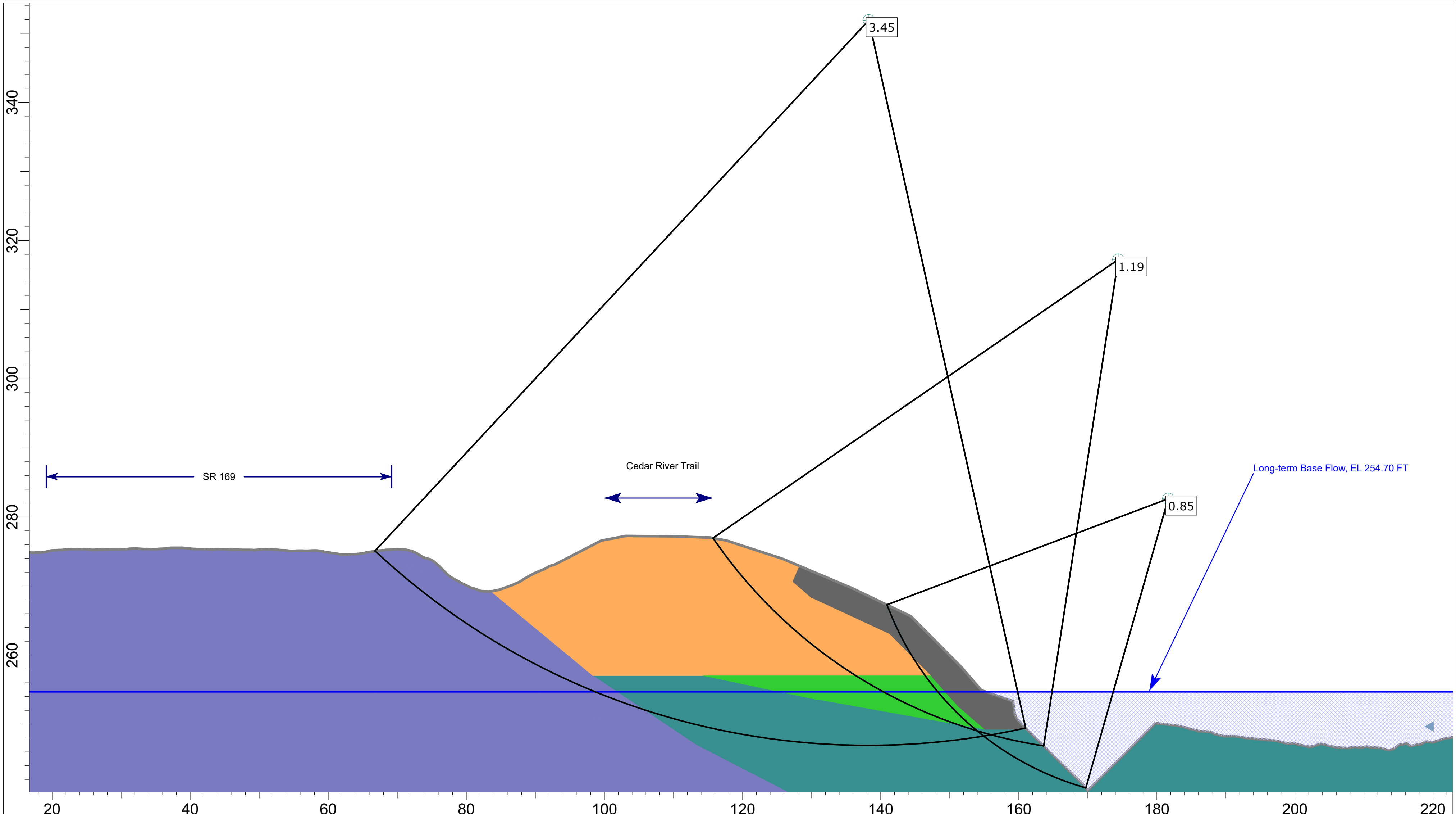
6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-6**



Legend

- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Existing Conditions With Potential Scour Static Conditions

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020

SCALE: 1:240

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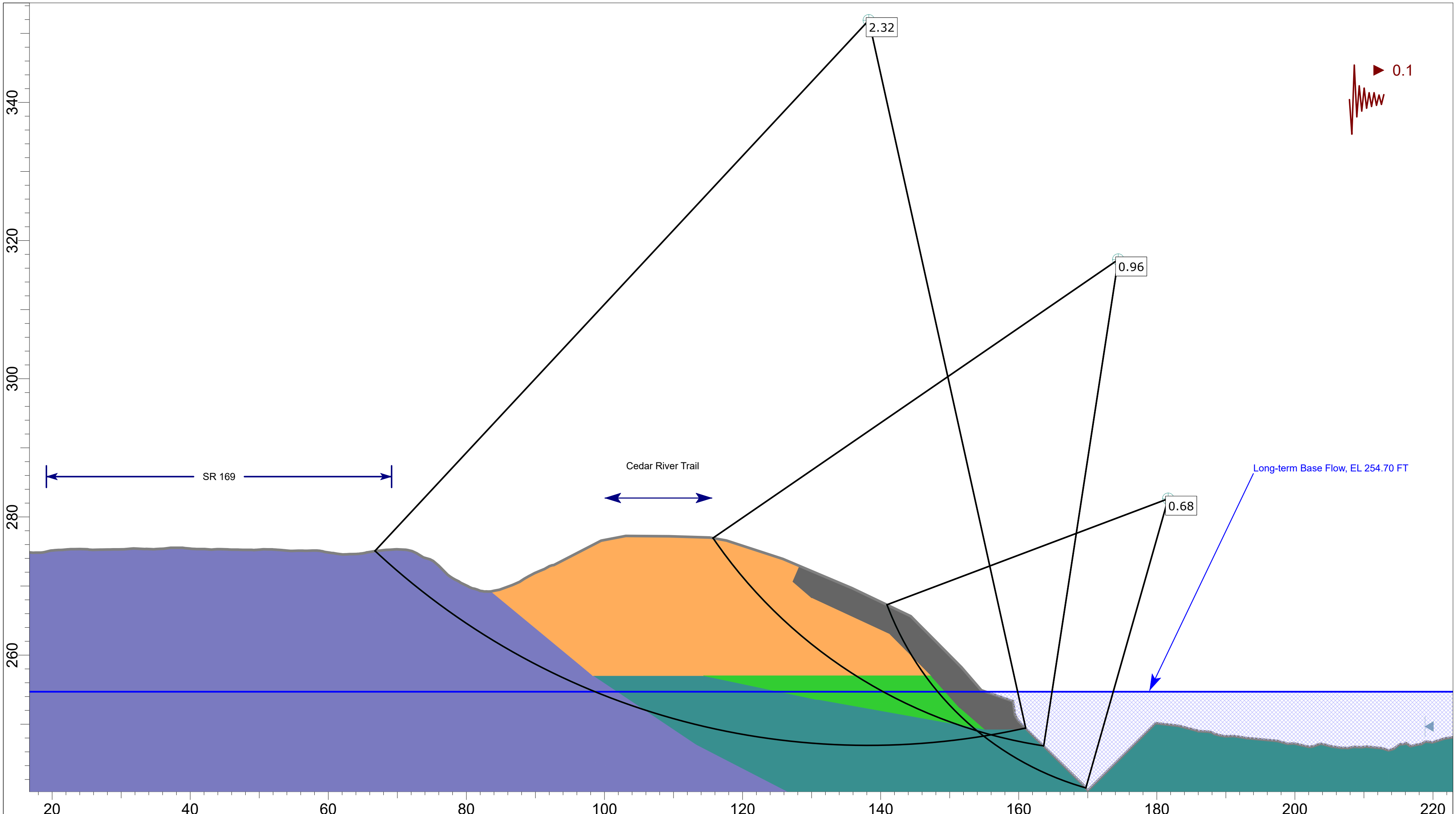
6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-7**



Legend

- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Existing Conditions With Potential Scour Seismic Conditions

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020

SCALE: 1:240

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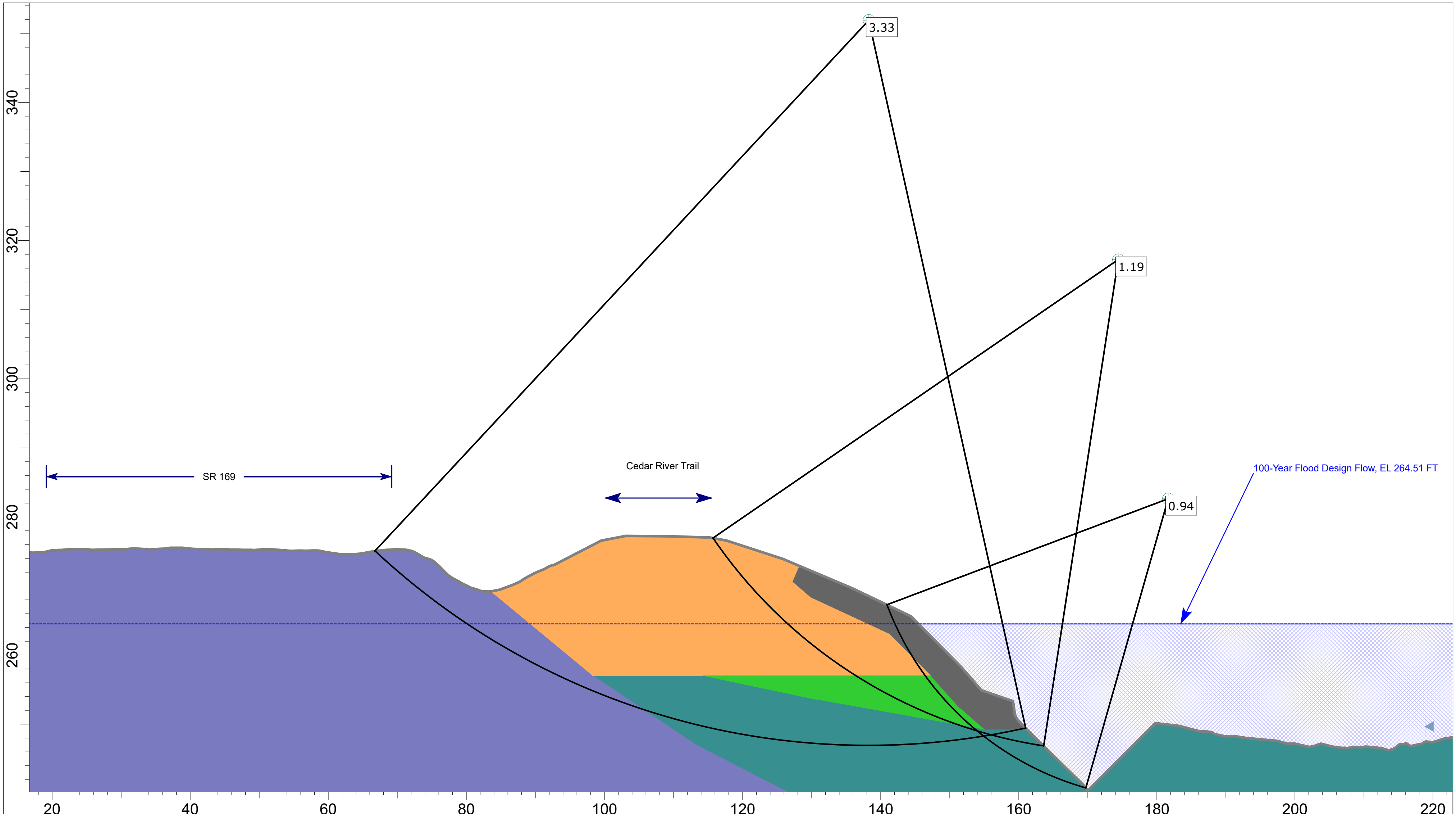
6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-8**



Legend

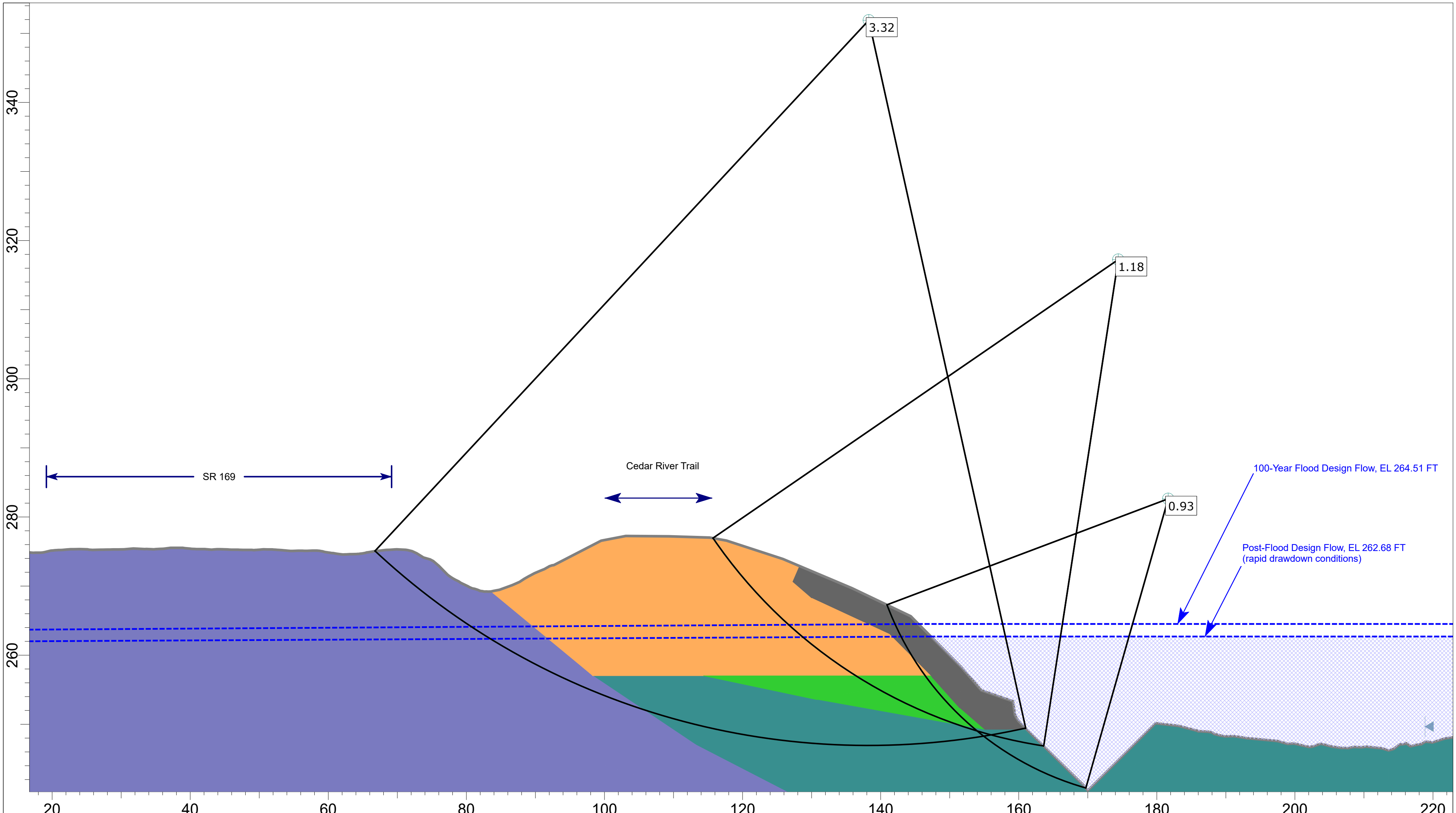
- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Existing Conditions With Potential Scour Peak Flood Level

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020	SCALE: 1:240 S:\King County\Jan Road Neighborhood Levee\Data\Analyses\04_SSA CRT7\crt7_2020.05.07_existing conditions w scour analysis.slmd	Aspect CONSULTING	6/23/2020 PROJECT NO. 190175	BY: MO REVIEWED BY: AJH	APPENDIX: **D-9**



Legend

- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Existing Conditions With Potential Scour Rapid Drawdown

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020

SCALE: 1:240

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Aspect CONSULTING

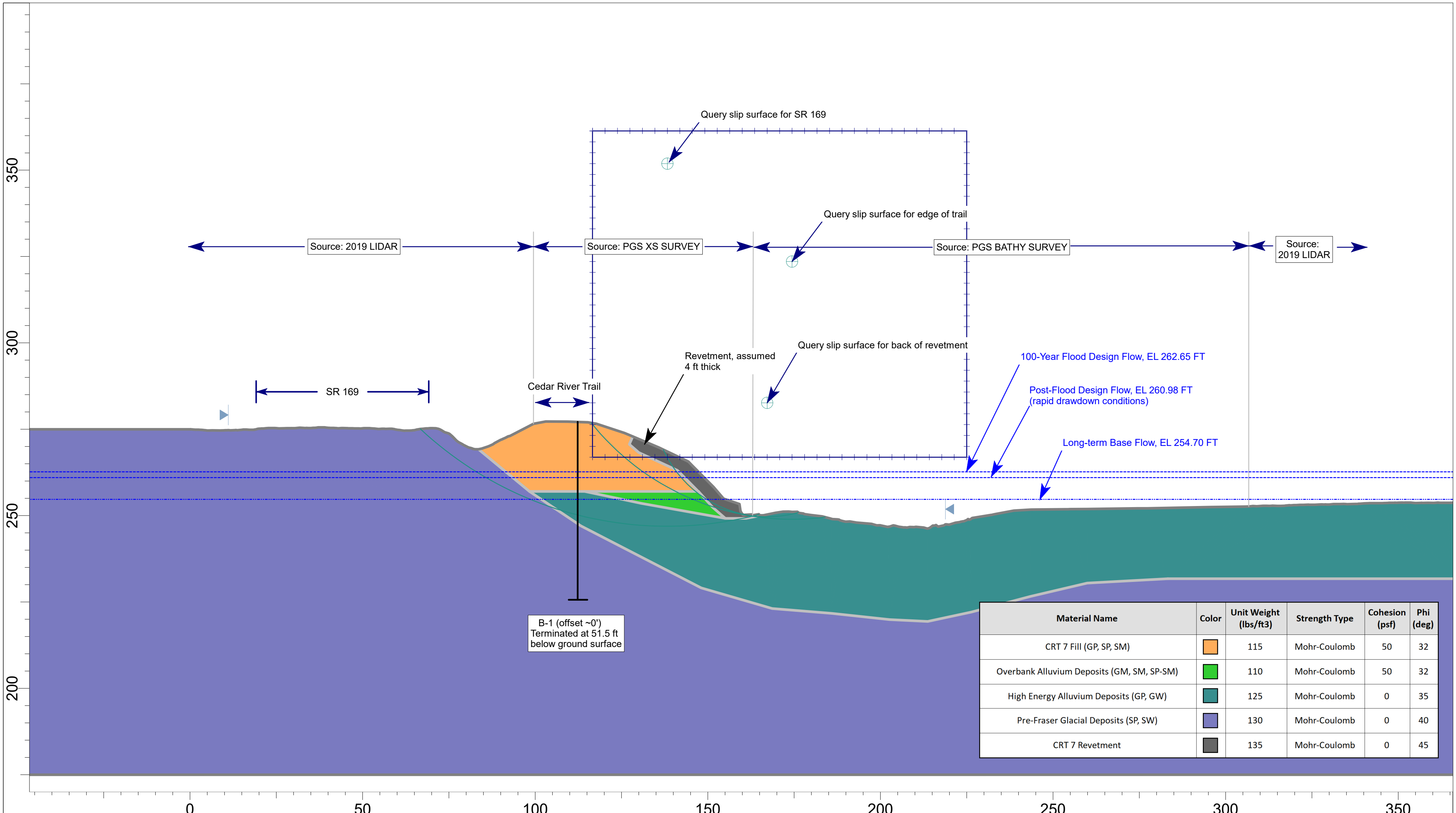
6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-10**



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
CRT 7 Fill (GP, SP, SM)		115	Mohr-Coulomb	50	32
Overbank Alluvium Deposits (GM, SM, SP-SM)		110	Mohr-Coulomb	50	32
High Energy Alluvium Deposits (GP, GW)		125	Mohr-Coulomb	0	35
Pre-Fraser Glacial Deposits (SP, SW)		130	Mohr-Coulomb	0	40
CRT 7 Revetment		135	Mohr-Coulomb	0	45

Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

Cross Section B-B', Alternative 1, Non-Scoured Conditions Master Scenario

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDE 8.020

SCALE: 1:480
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Aspect CONSULTING

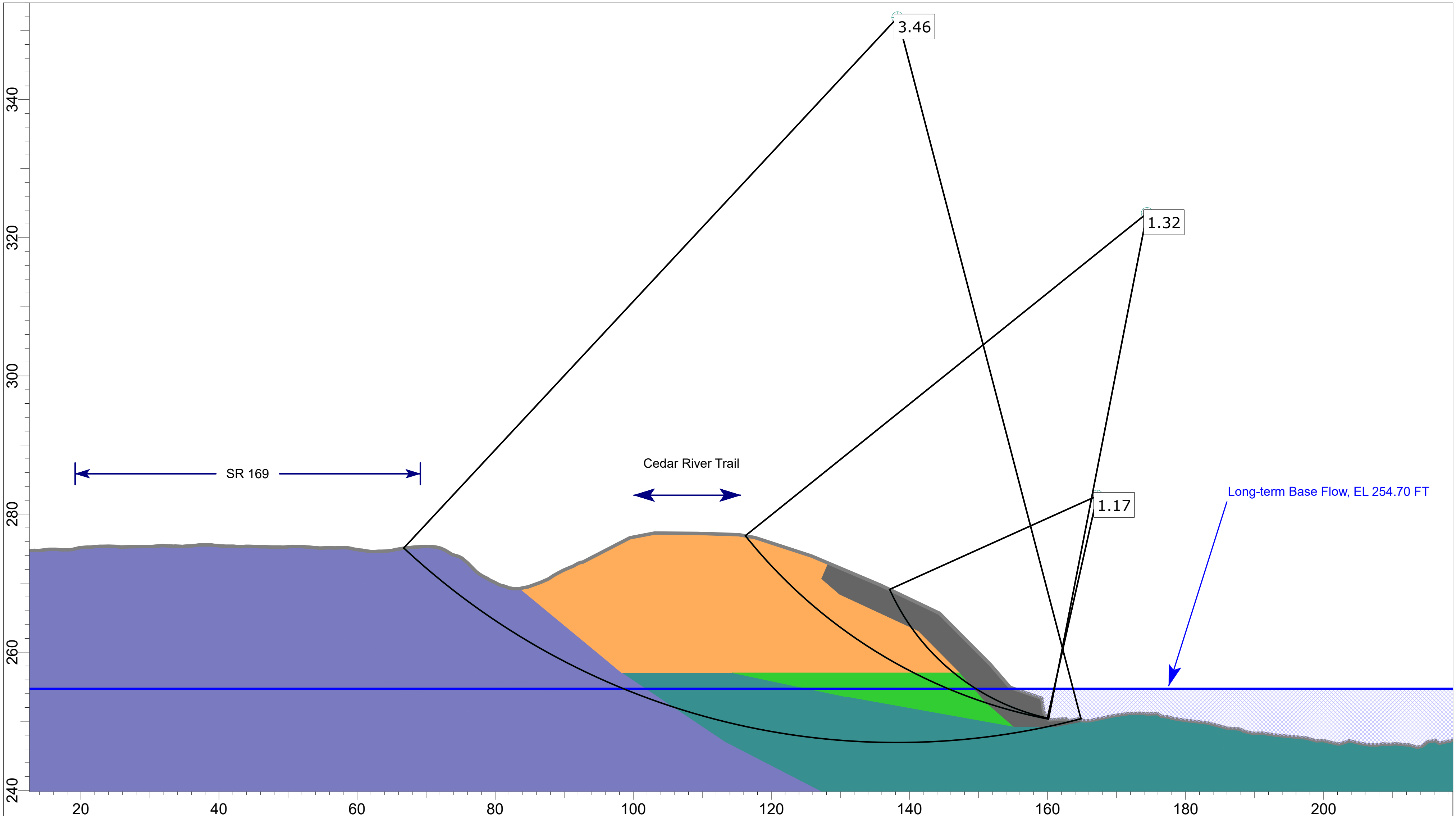
6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-11**



Cross Section B-B', Alternative 1, Non-Scoured Conditions
Static Conditions

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020

SCALE: 1:240

S:\King County\Jan Road Neighborhood Levee\Data\Analyses\04_SSA CRT7\crt7_2020.04.22_alternative 1 existing conditions.slmd

Aspect CONSULTING

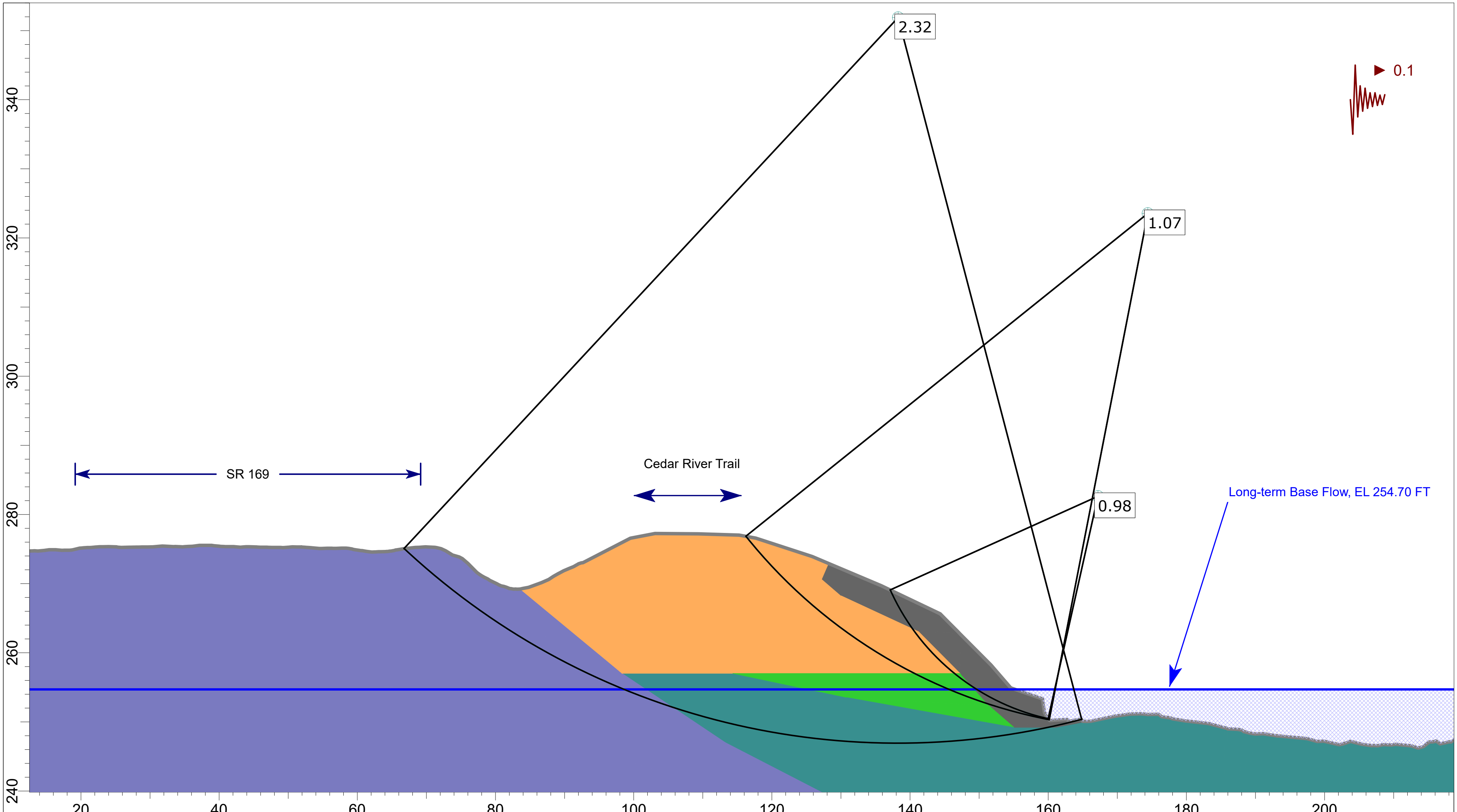
6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-12**



Legend

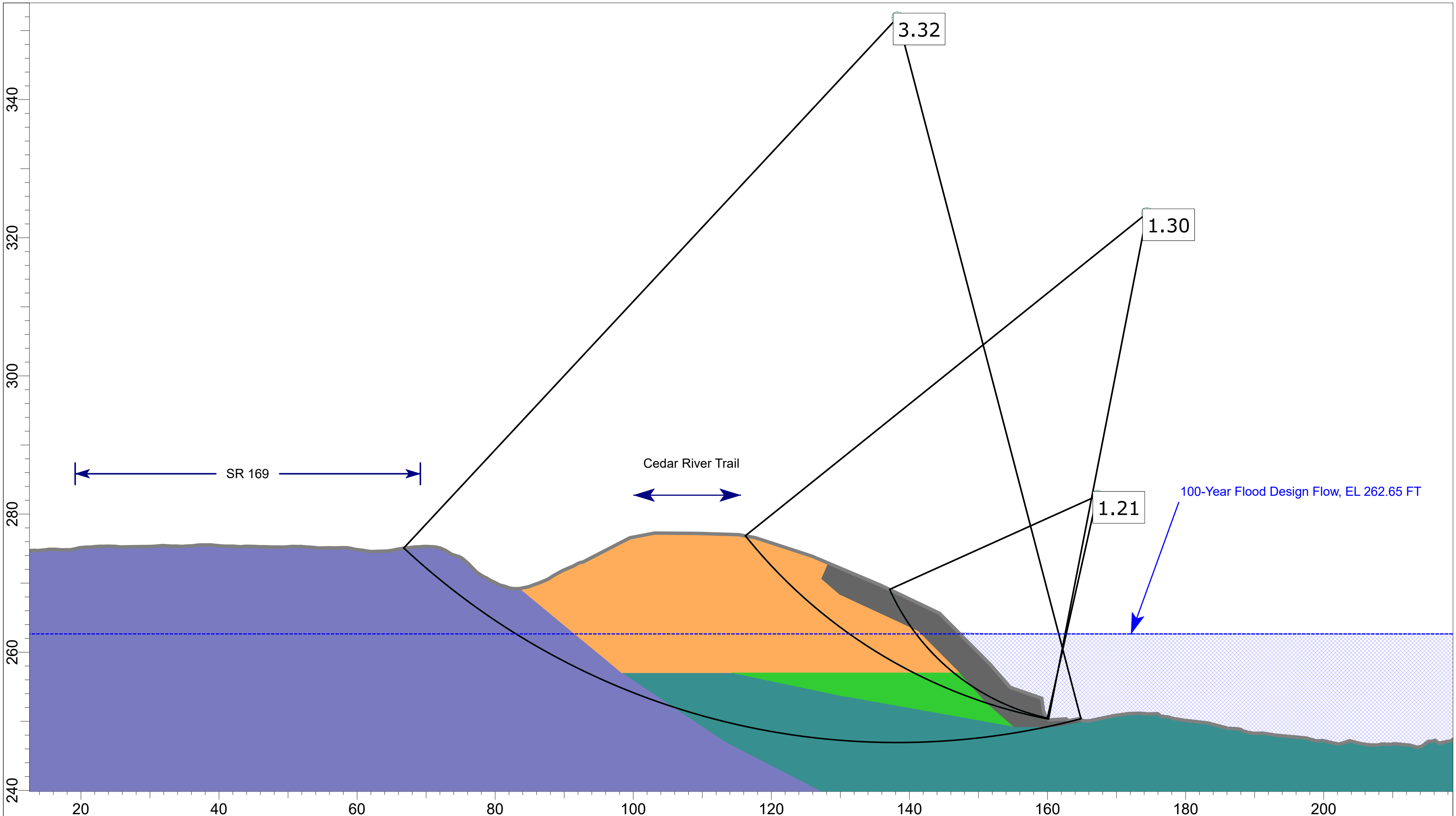
- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Alternative 1, Non-Scoured Conditions
Seismic Conditions

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020	SCALE: 1:240 S:\King County\Jan Road Neighborhood Levee\Data\Analyses\04_SSA CRT7\crt7_2020.04.22_alternative 1 existing conditions.slmd	Aspect CONSULTING	6/23/2020 PROJECT NO. 190175	BY: MO REVIEWED BY: AJH	APPENDIX: **D-13**



Legend

- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Alternative 1, Non-Scoured Conditions
Peak Flood Level

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020

SCALE: 1:240

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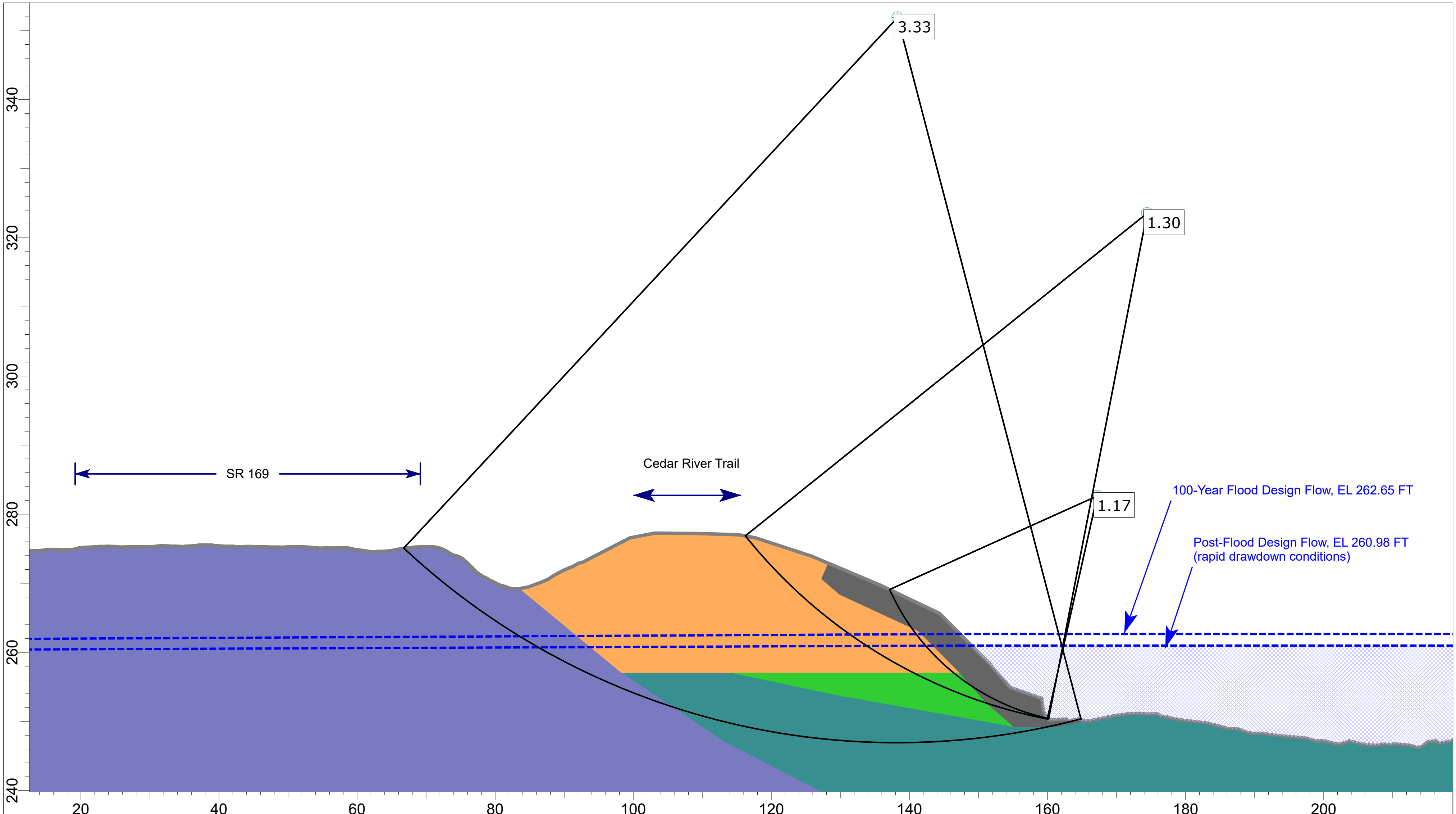
Aspect CONSULTING

6/23/2020	BY: MO	APPENDIX: D-14
PROJECT NO. 190175	REVIEWED BY: AJH	



Legend

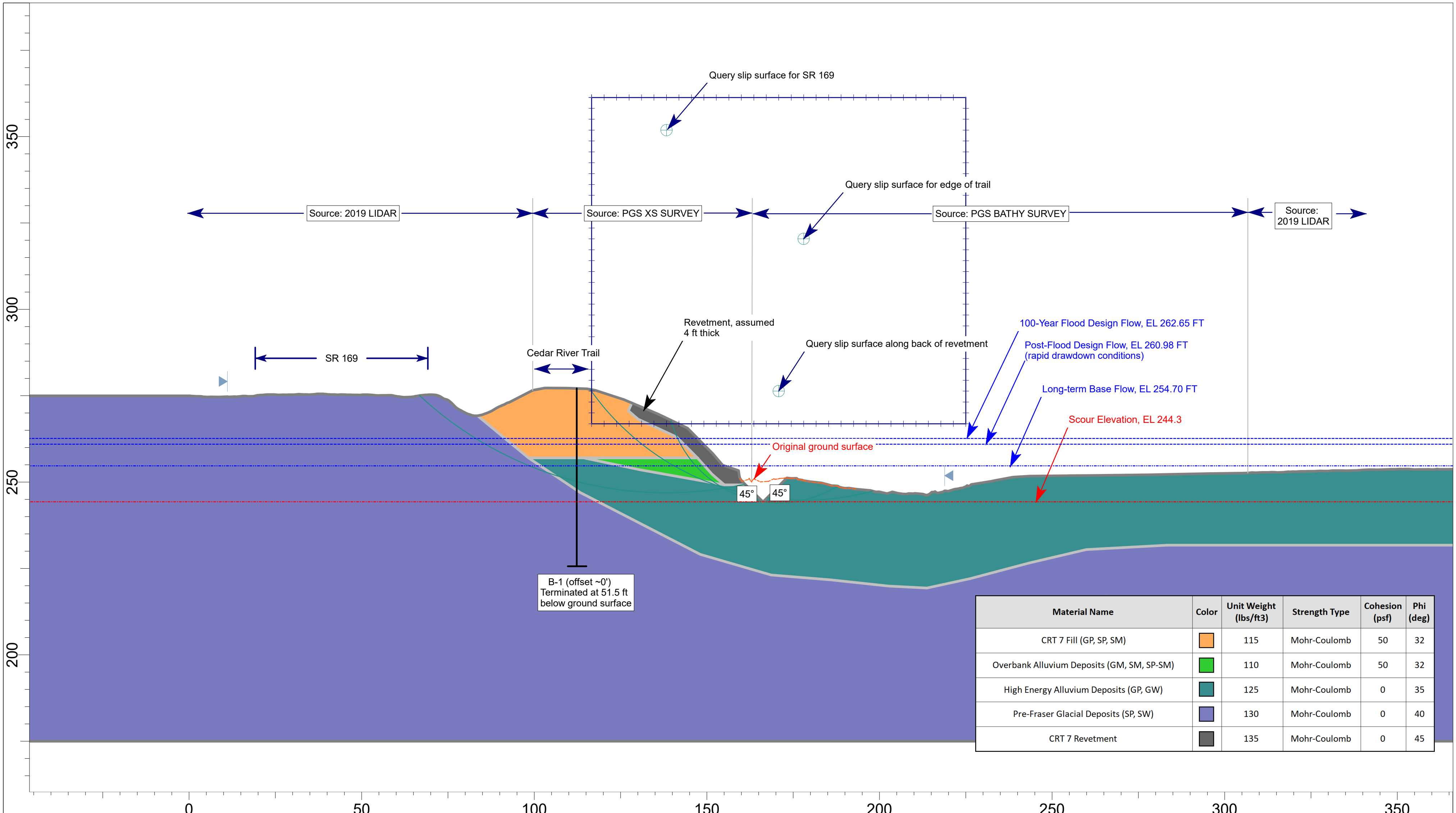
Query Slip Surface
Search Limits
Modeled Groundwater Level

Cross Section B-B', Alternative 1, Non-Scoured Conditions
Rapid Drawdown

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020	SCALE: 1:240 S:\King County\Jan Road Neighborhood Levee\Data\Analyses\04_SSA CRT7\crt7_2020.04.22_alternative 1 existing conditions.slmd	Aspect CONSULTING	6/23/2020 PROJECT NO. 190175	BY: MO REVIEWED BY: AJH	APPENDIX: **D-15**



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
CRT 7 Fill (GP, SP, SM)		115	Mohr-Coulomb	50	32
Overbank Alluvium Deposits (GM, SM, SP-SM)		110	Mohr-Coulomb	50	32
High Energy Alluvium Deposits (GP, GW)		125	Mohr-Coulomb	0	35
Pre-Fraser Glacial Deposits (SP, SW)		130	Mohr-Coulomb	0	40
CRT 7 Revetment		135	Mohr-Coulomb	0	45

Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

Cross Section B-B', Alternative 1 With Potential Scour Master Scenario

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDE 8.020

SCALE: 1:480

S:\King County\Jan Road Neighborhood Levee\Data\Analyses\04_SSA CRT7\crt7_2020.04.22_alternative 1 with scour.slmd

Aspect CONSULTING

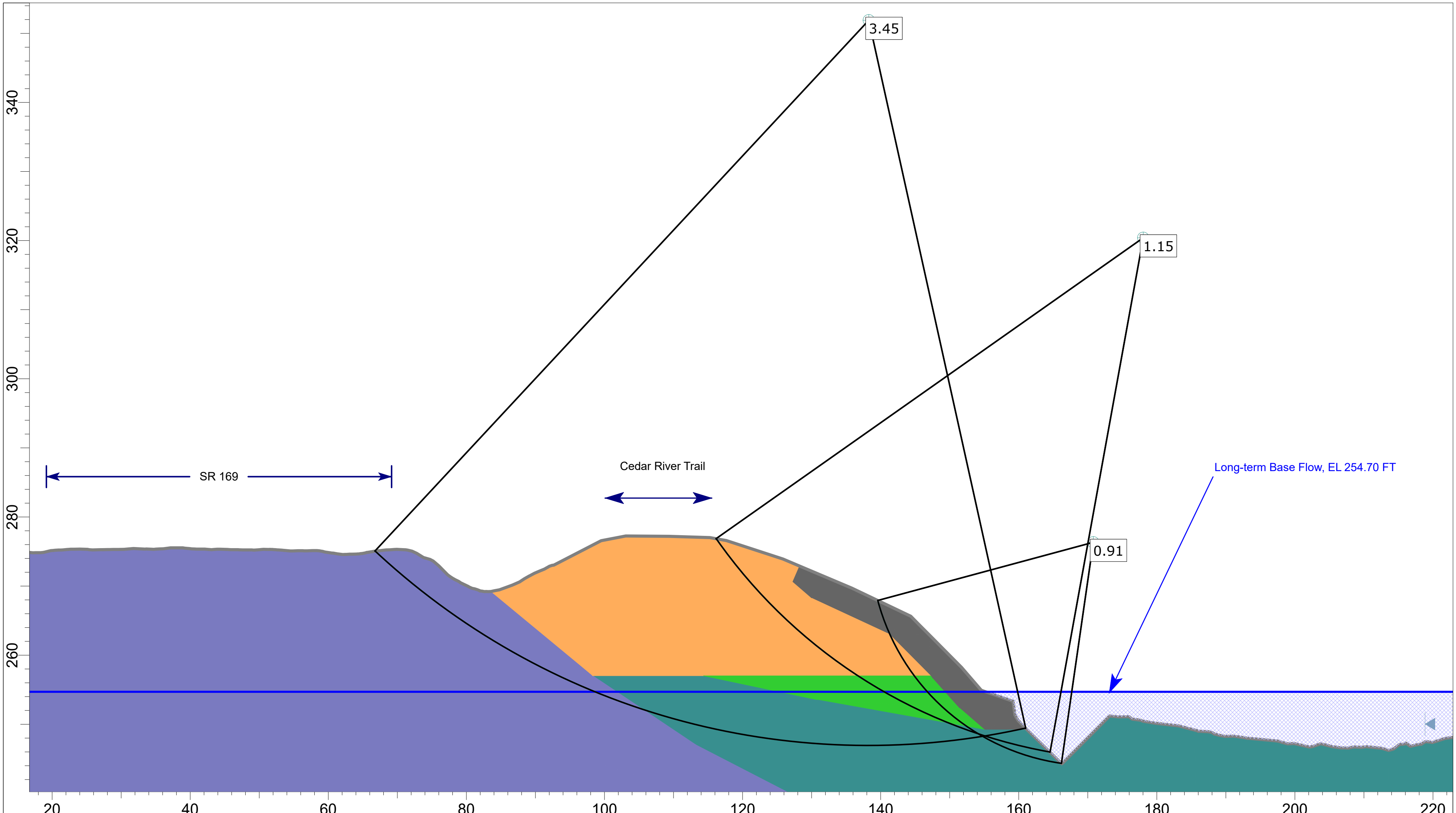
6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-16**



Legend

- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Alternative 1 With Potential Scour
Static Conditions

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020

SCALE: 1:240

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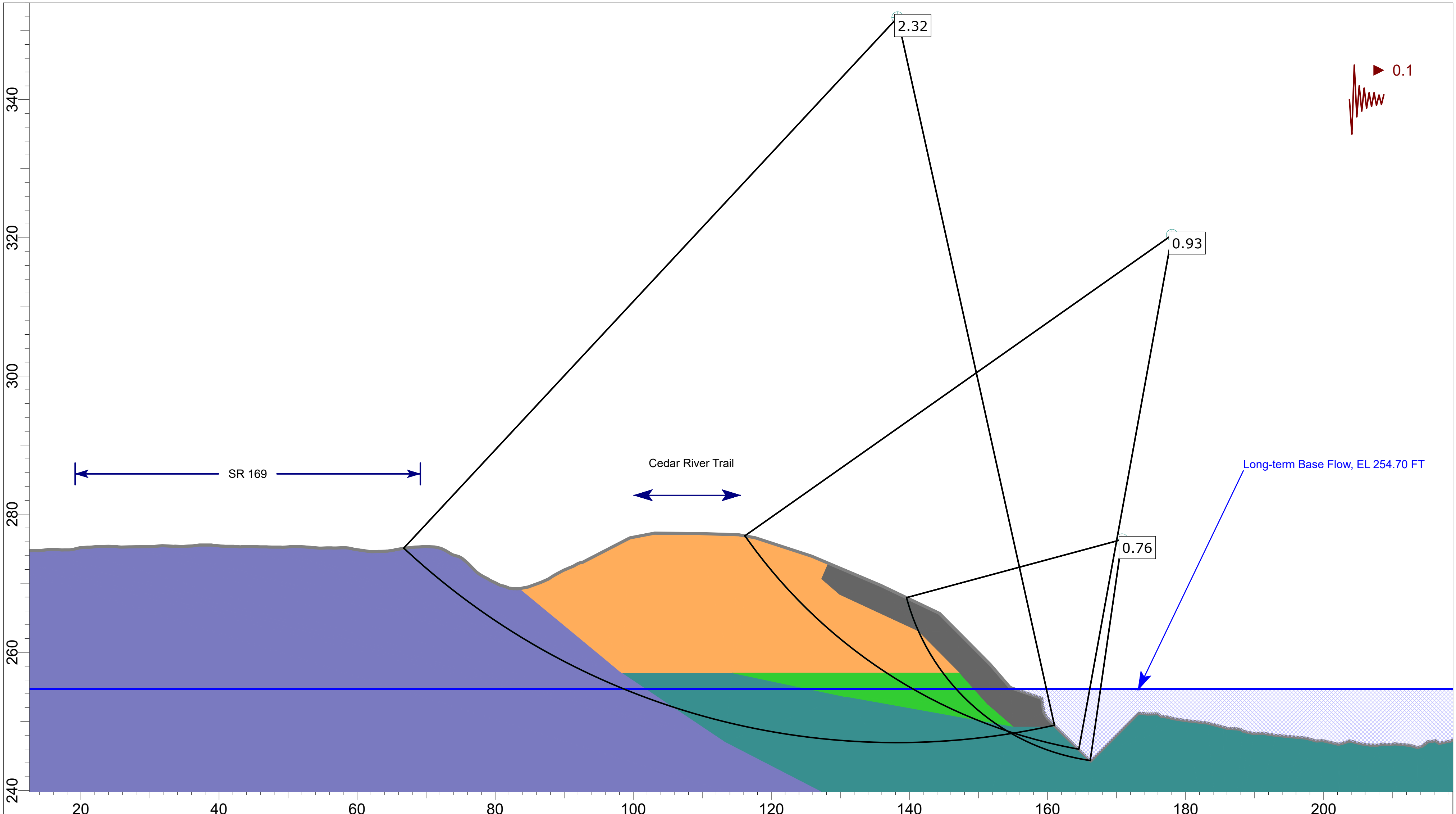
PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-17**



Legend

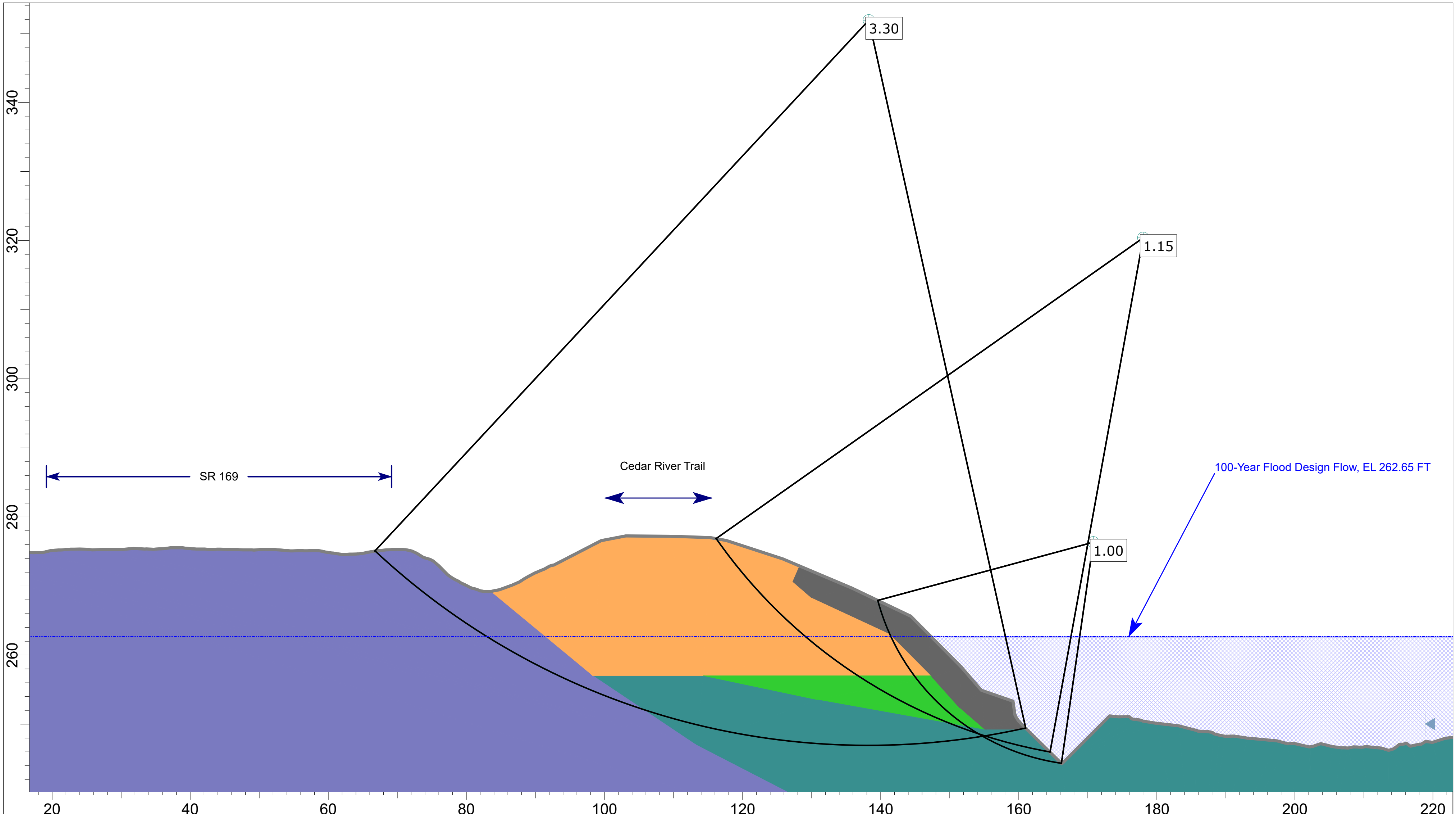
- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Alternative 1 With Potential Scour
Seismic Conditions

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020	SCALE: 1:240 S:\King County\Jan Road Neighborhood Levee\Data\Analyses\04_SSA CRT7\crt7_2020.04.22_alternative 1 with scour.slmd	Aspect CONSULTING	6/23/2020 PROJECT NO. 190175	BY: MO REVIEWED BY: AJH	APPENDIX: **D-18**



Legend

- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Alternative 1 With Potential Scour
Peak Flood Level

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020

SCALE: 1:240

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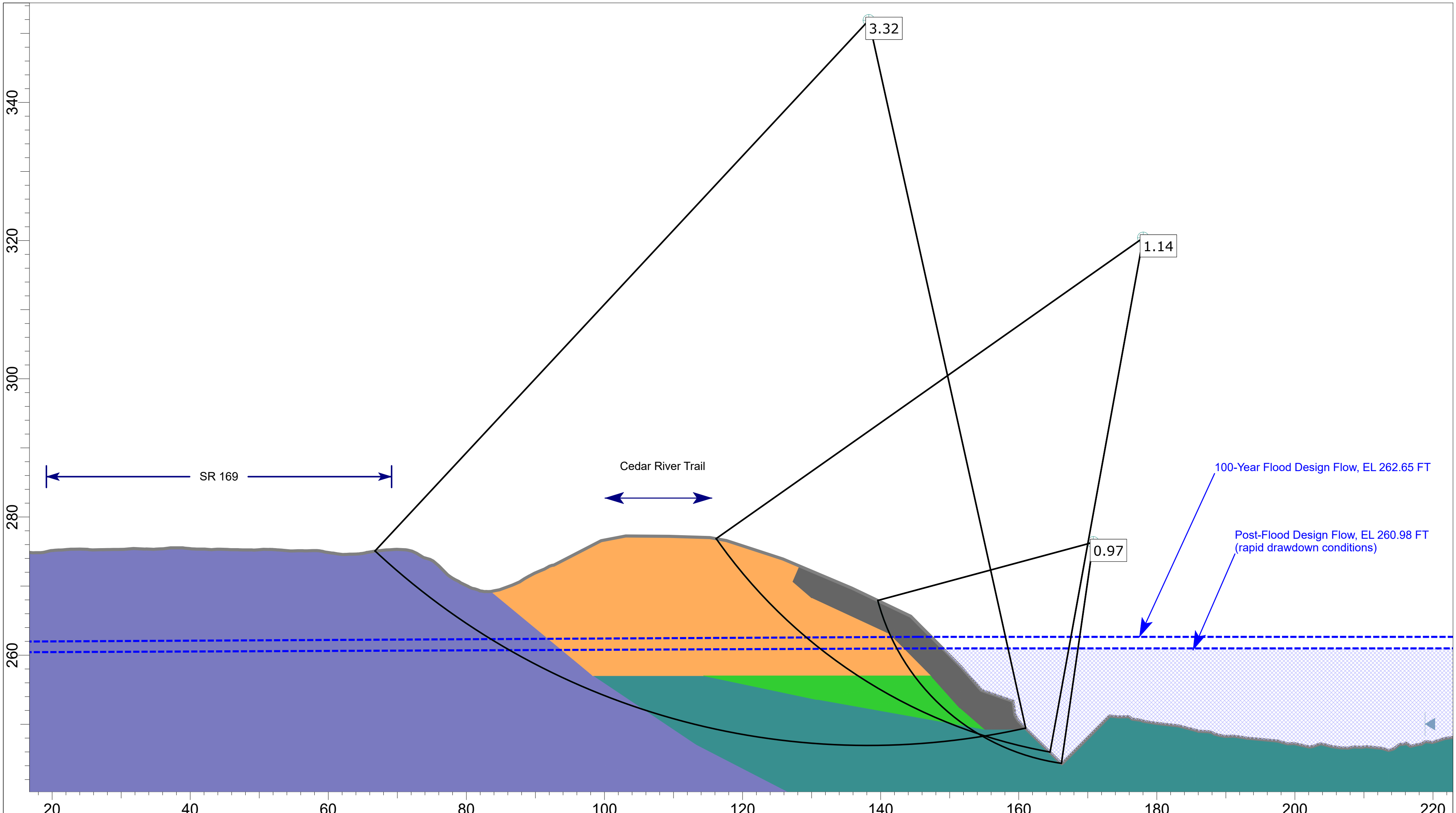
6/23/2020	BY: MO	APPENDIX:	
PROJECT NO. 190175	REVIEWED BY: AJH	D-19	



Legend

- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Alternative 1 With Potential Scour
Rapid Drawdown

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020

SCALE: 1:240
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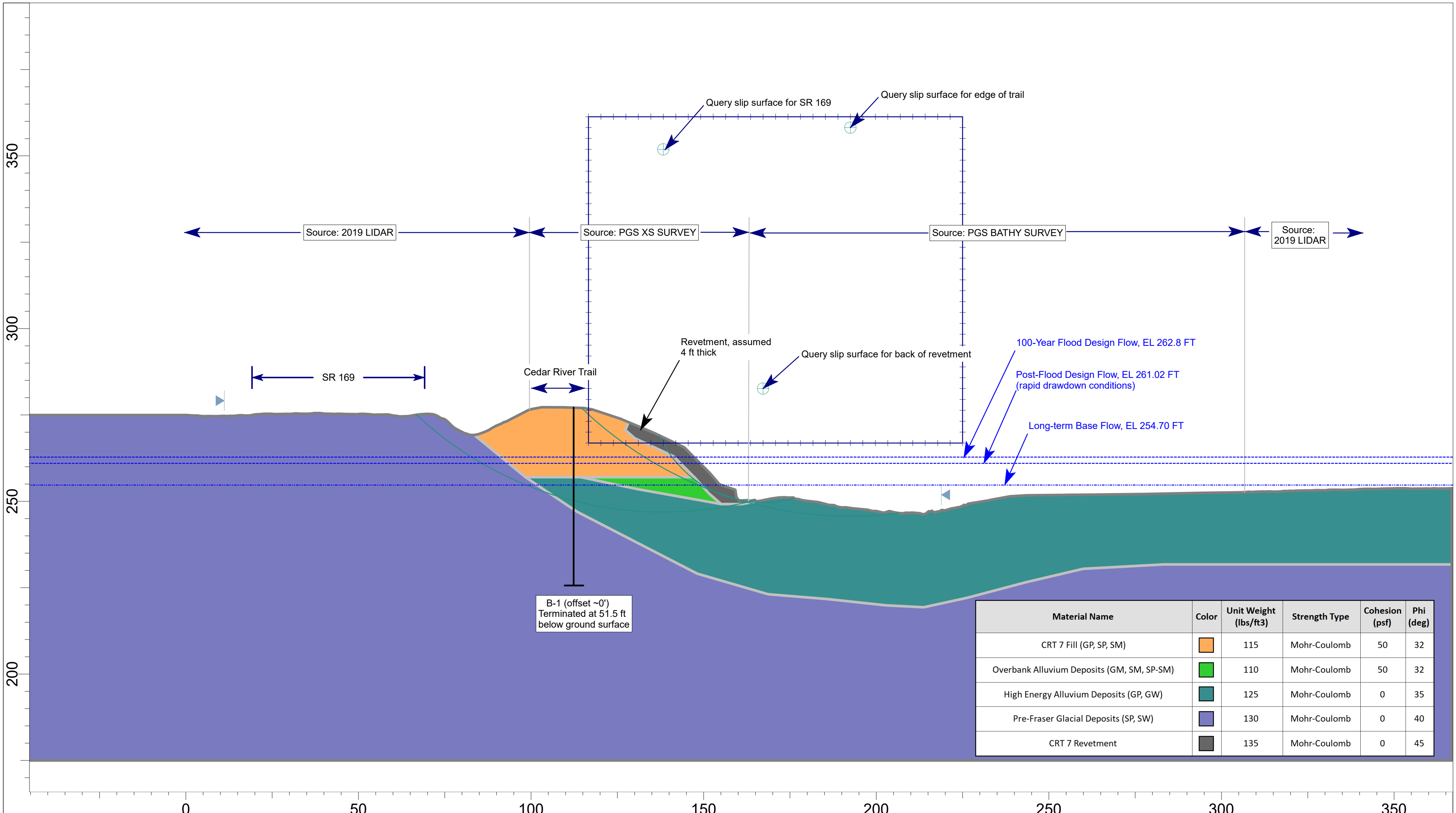
PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-20**



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
CRT 7 Fill (GP, SP, SM)		115	Mohr-Coulomb	50	32
Overbank Alluvium Deposits (GM, SM, SP-SM)		110	Mohr-Coulomb	50	32
High Energy Alluvium Deposits (GP, GW)		125	Mohr-Coulomb	0	35
Pre-Fraser Glacial Deposits (SP, SW)		130	Mohr-Coulomb	0	40
CRT 7 Revetment		135	Mohr-Coulomb	0	45

Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

Cross Section B-B', Alternative 2, Non-Scoured Conditions
Master Scenario

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDE 8.020

SCALE: 1:480

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Aspect CONSULTING

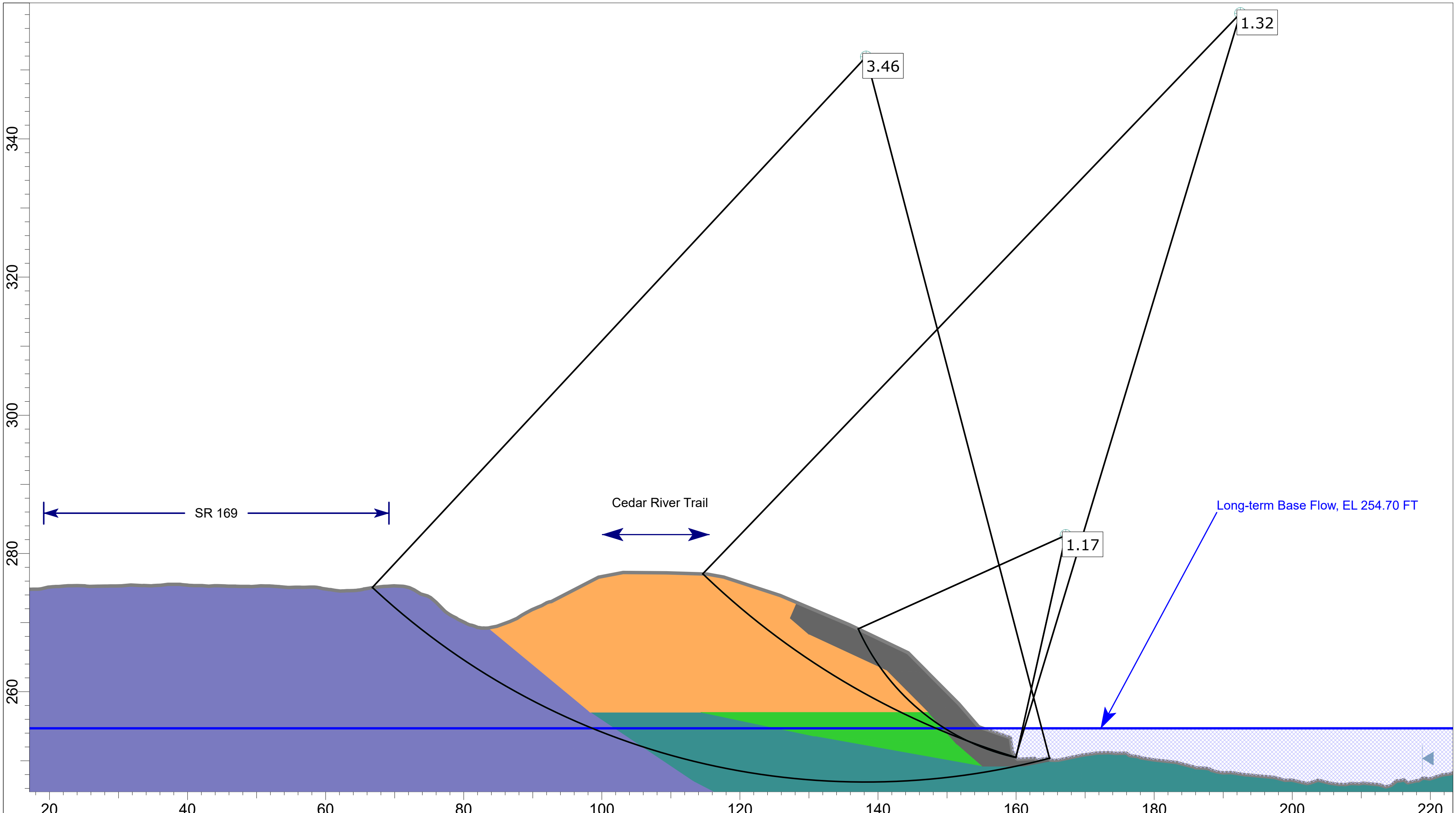
6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-21**



Legend

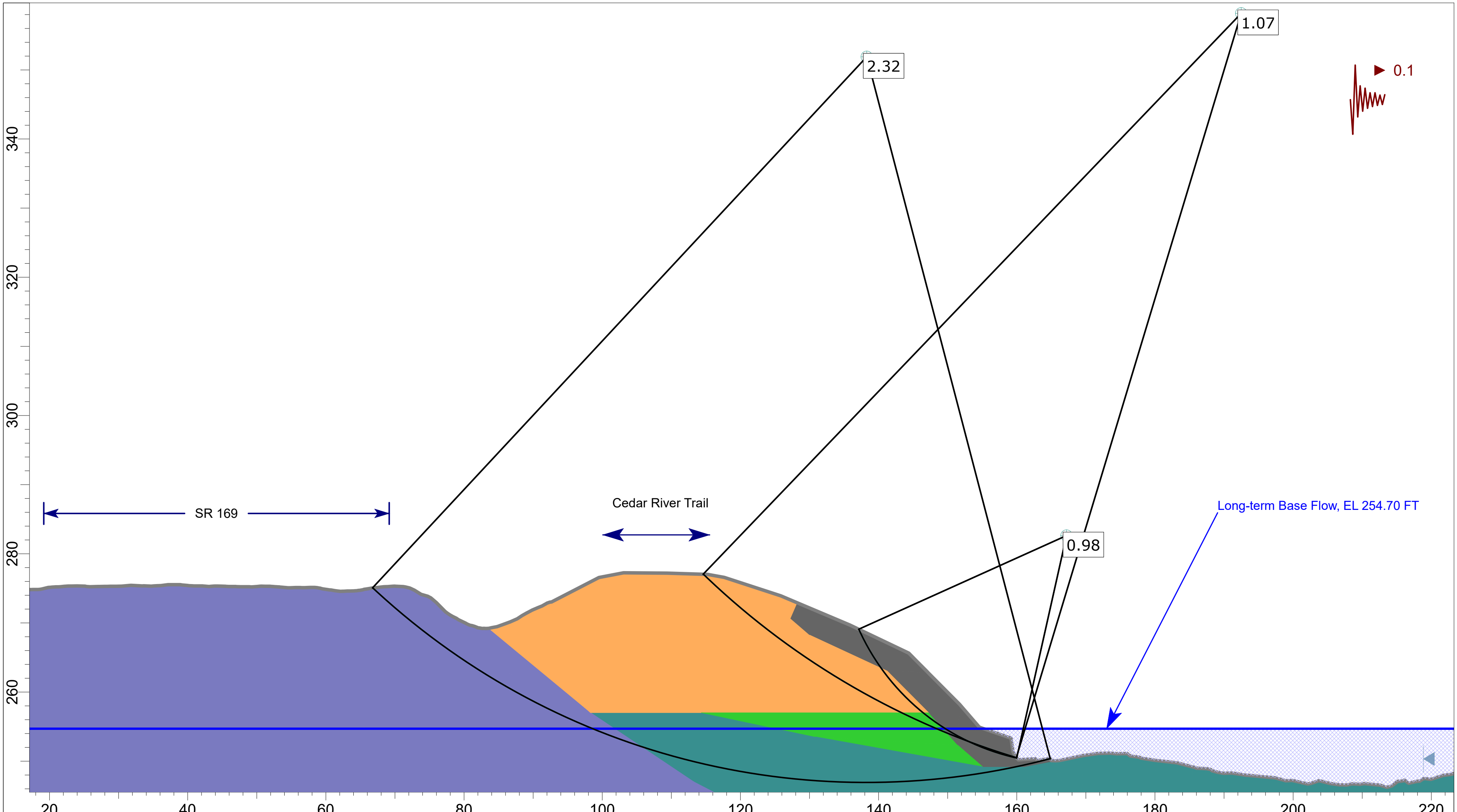
- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Alternative 2, Non-Scoured Conditions
Static Conditions

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020	SCALE: 1:240 S:\King County\Jan Road Neighborhood Levee\Data\Analyses\04_SSA CRT7\crt7_2020.04.22_alternative 2 existing conditions.slmd	Aspect CONSULTING	6/23/2020 PROJECT NO. 190175	BY: MO REVIEWED BY: AJH	APPENDIX: **D-22**



Legend

- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Alternative 2, Non-Scoured Conditions
Seismic Conditions

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020

SCALE: 1:240

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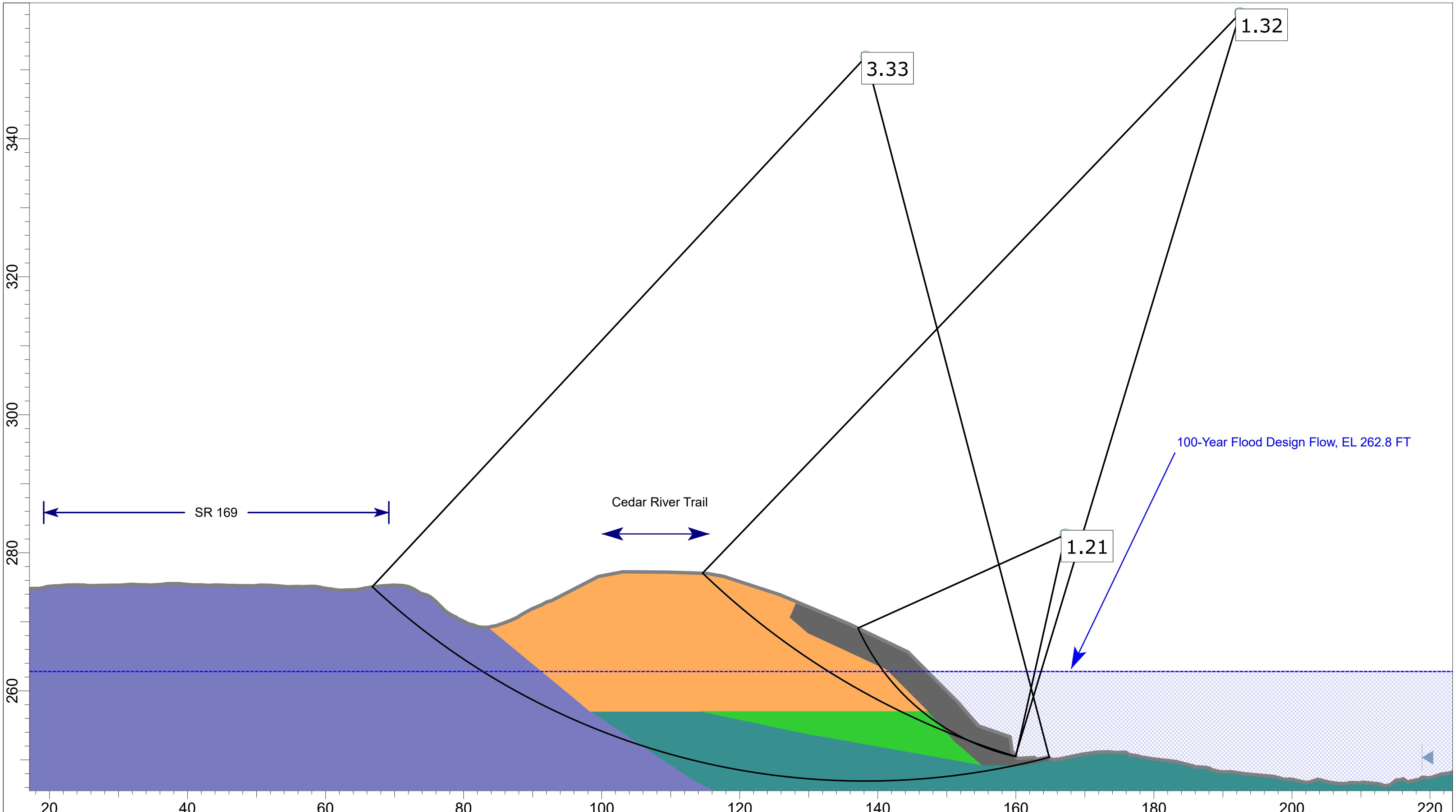
6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-23**



Legend

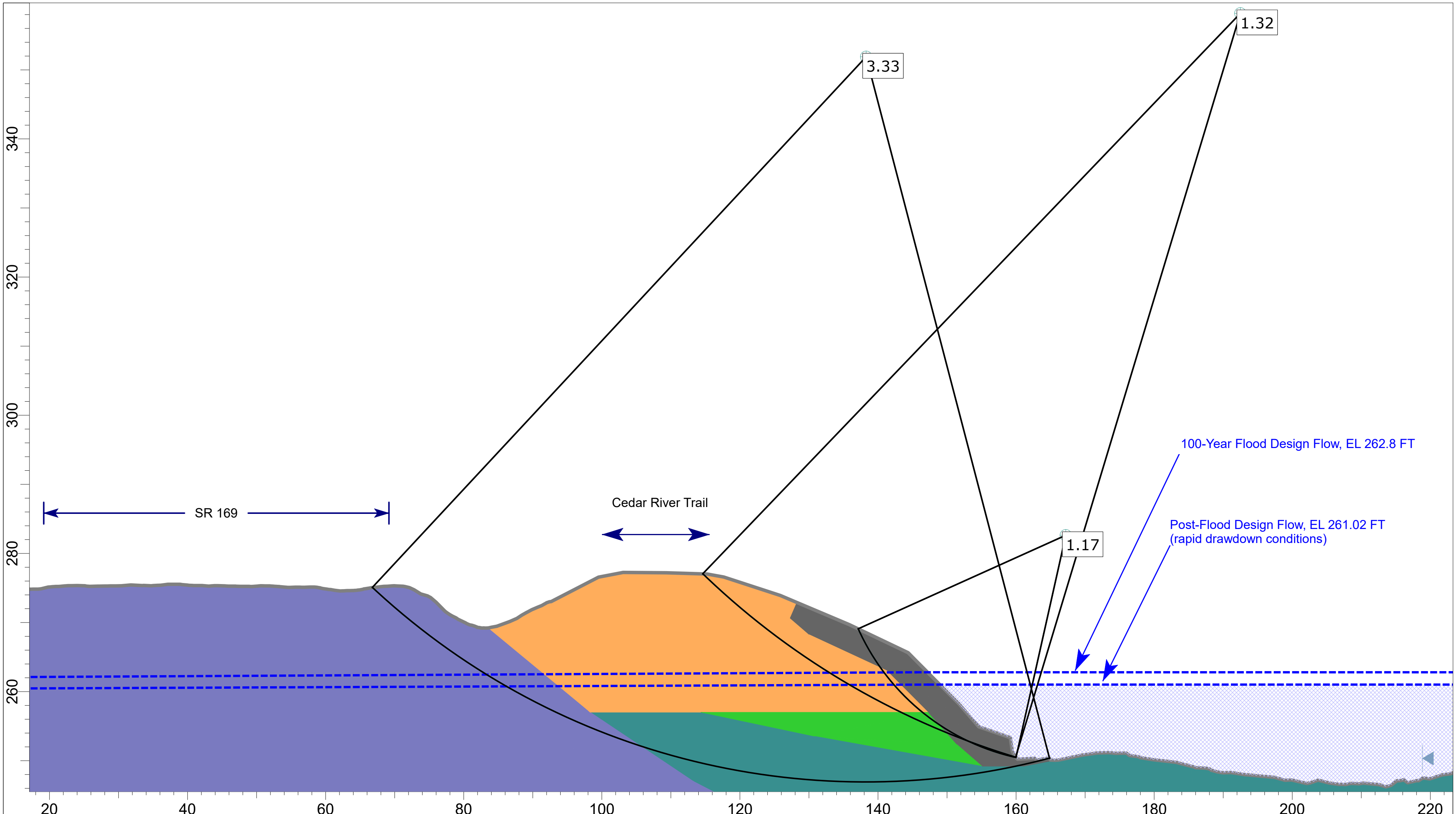
- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Alternative 2, Non-Scoured Conditions
Peak Flood Level

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020	SCALE: 1:240 S:\King County\Jan Road Neighborhood Levee\Data\Analyses\04_SSA CRT7\crt7_2020.04.22_alternative 2 existing conditions.slmd	Aspect CONSULTING	6/23/2020 PROJECT NO. 190175	BY: MO REVIEWED BY: AJH	APPENDIX: **D-24**



Legend

- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Alternative 2, Non-Scoured Conditions Rapid Drawdown

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020

SCALE: 1:240

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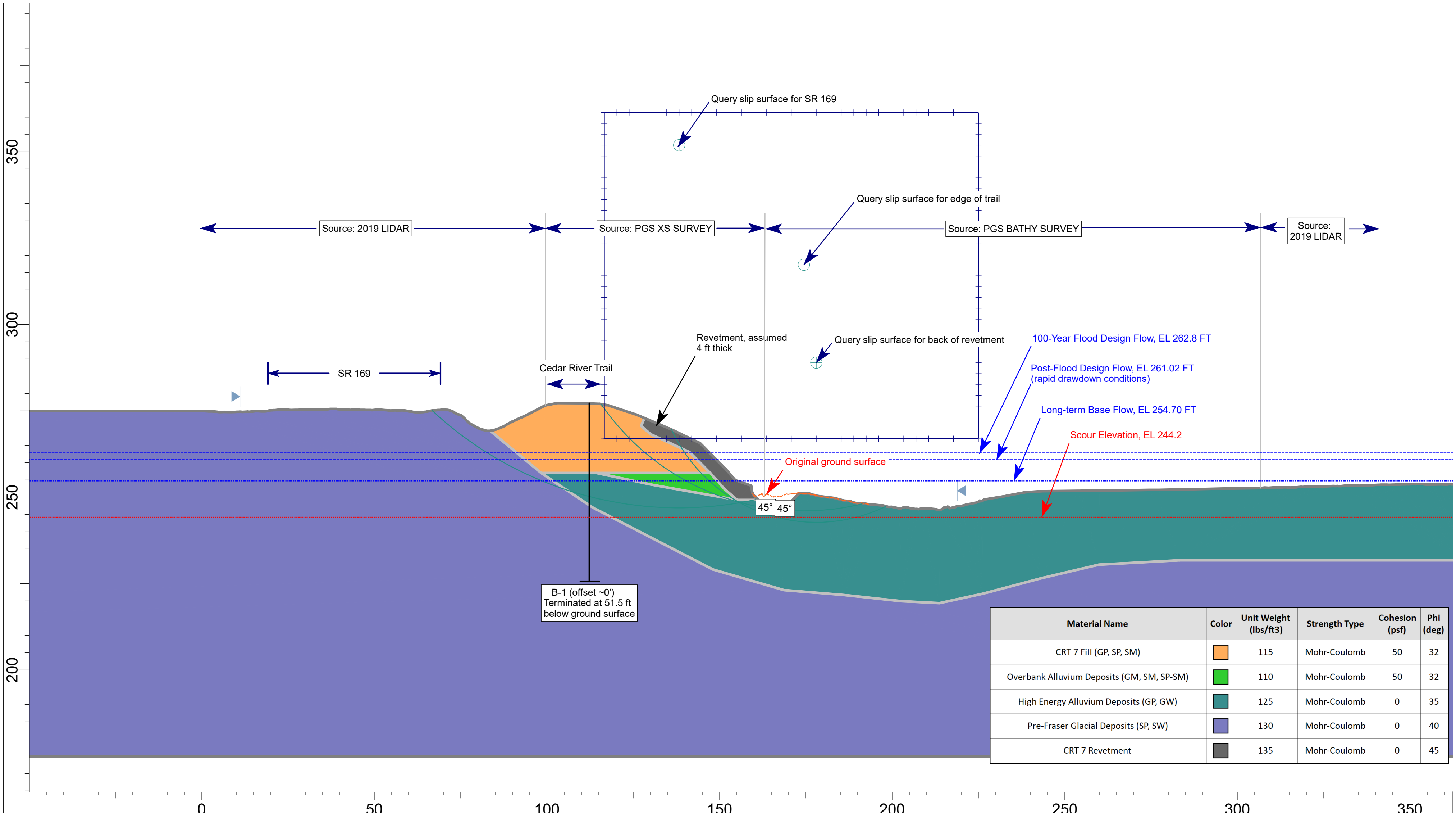
6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-25**



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
CRT 7 Fill (GP, SP, SM)		115	Mohr-Coulomb	50	32
Overbank Alluvium Deposits (GM, SM, SP-SM)		110	Mohr-Coulomb	50	32
High Energy Alluvium Deposits (GP, GW)		125	Mohr-Coulomb	0	35
Pre-Fraser Glacial Deposits (SP, SW)		130	Mohr-Coulomb	0	40
CRT 7 Revetment		135	Mohr-Coulomb	0	45

Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

Cross Section B-B', Alternative 2 With Potential Scour
Master Scenario

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDE 8.020

SCALE: 1:480

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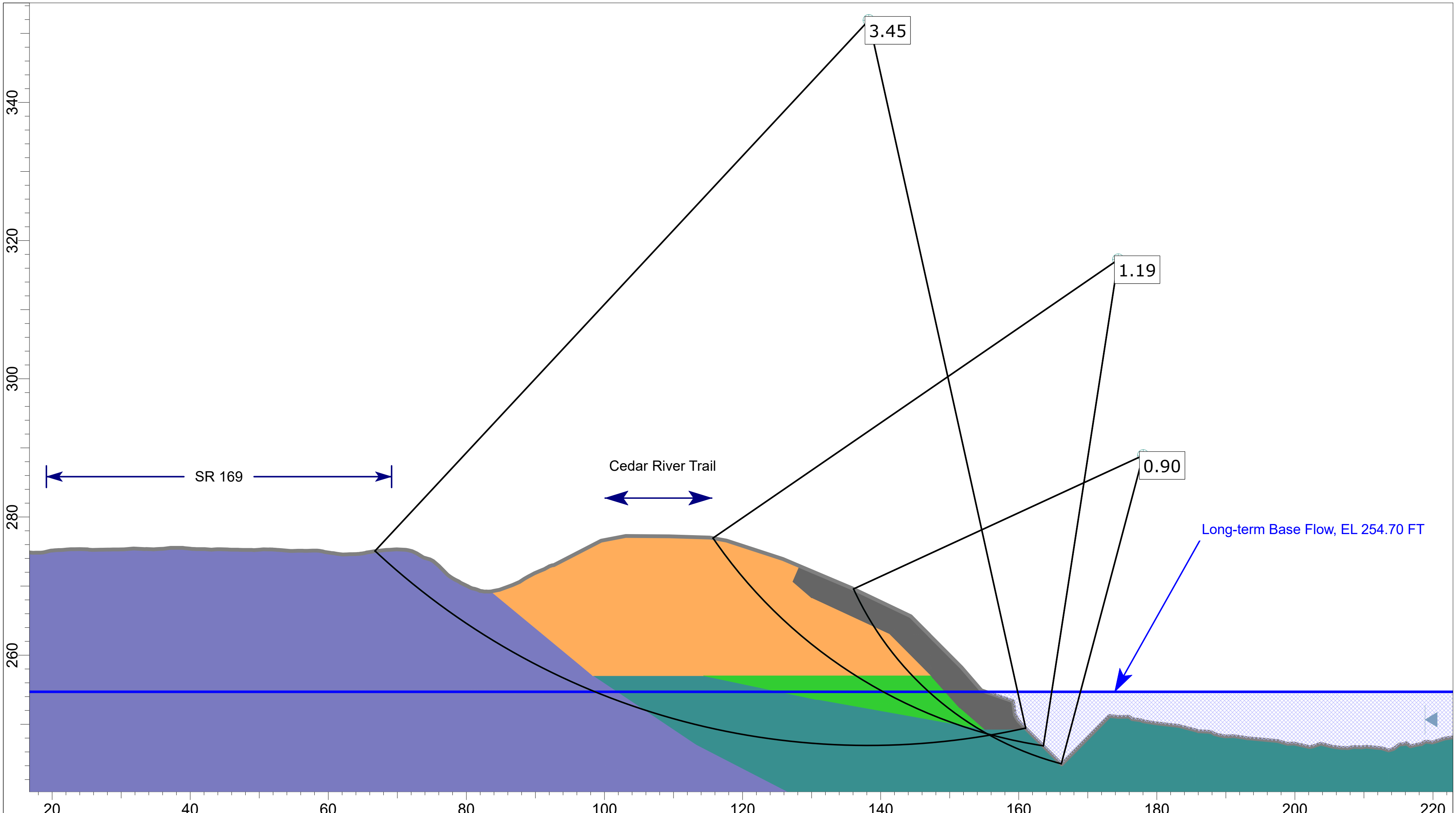
6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-26**



Legend

- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Alternative 2 With Potential Scour
Static Conditions

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020

SCALE: 1:240

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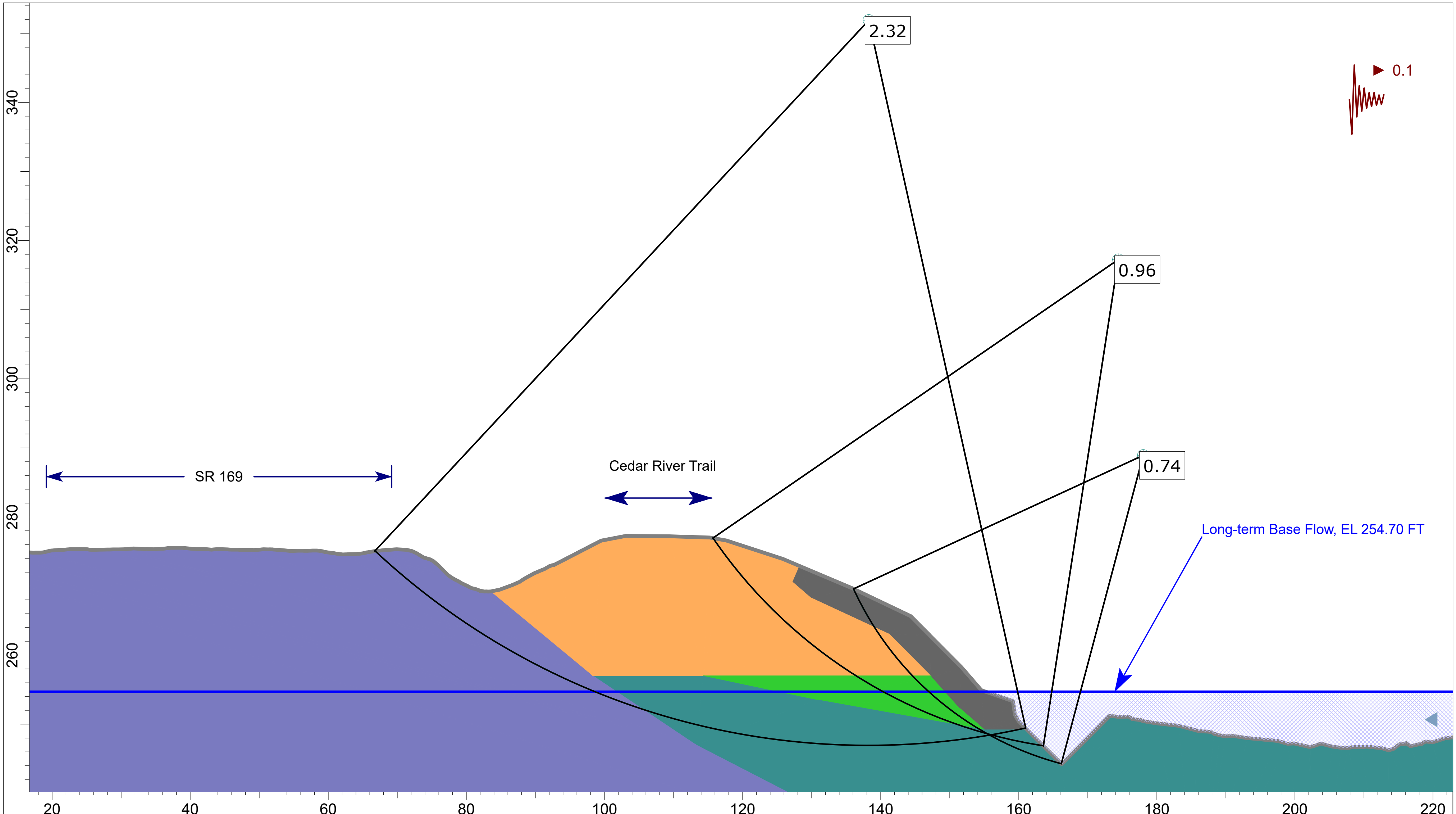
6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-27**



Legend

- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Alternative 2 With Potential Scour
Seismic Conditions

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020

SCALE: 1:240

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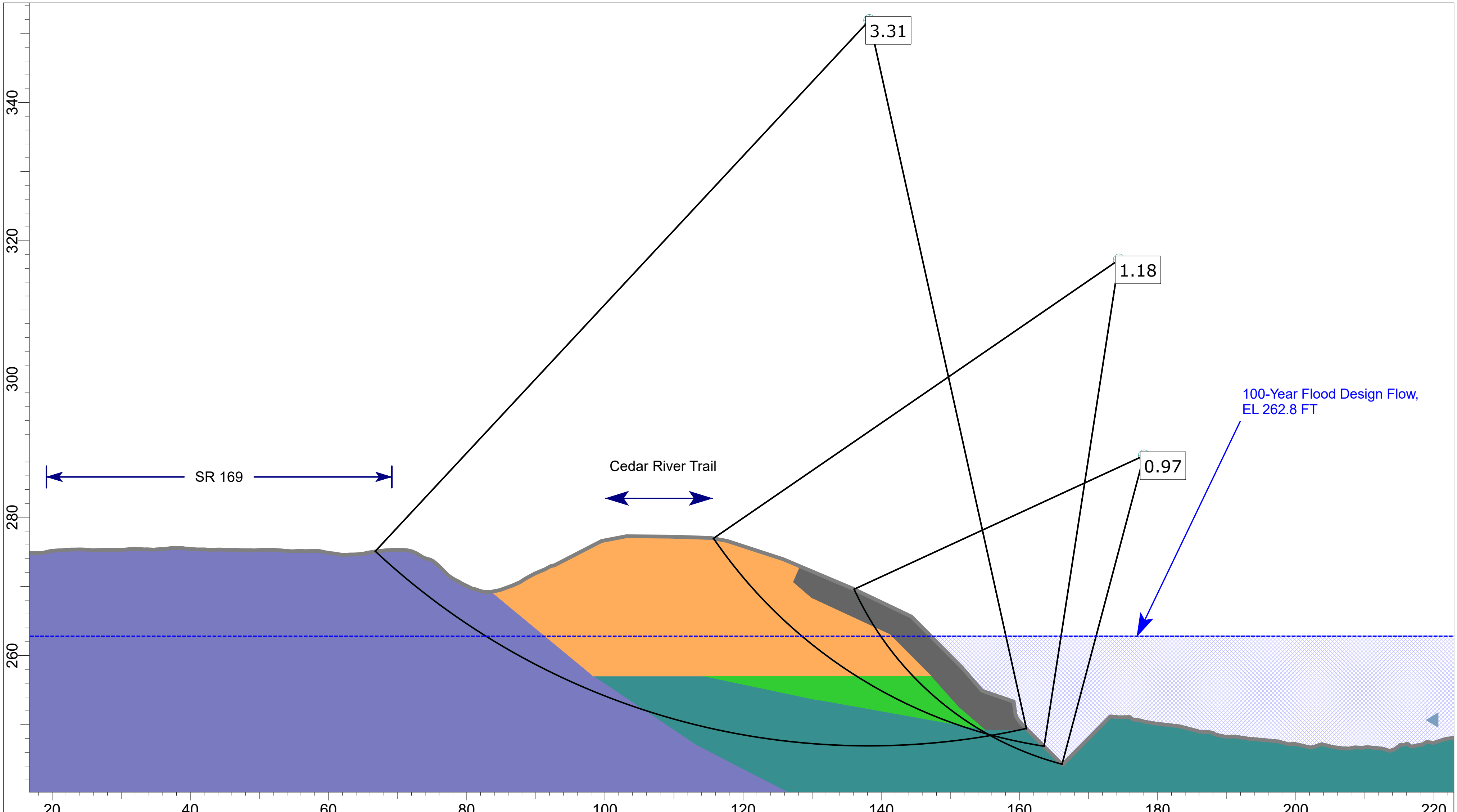
6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-28**



Legend

- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Alternative 2 With Potential Scour
Peak Flood Level

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020

SCALE: 1:240

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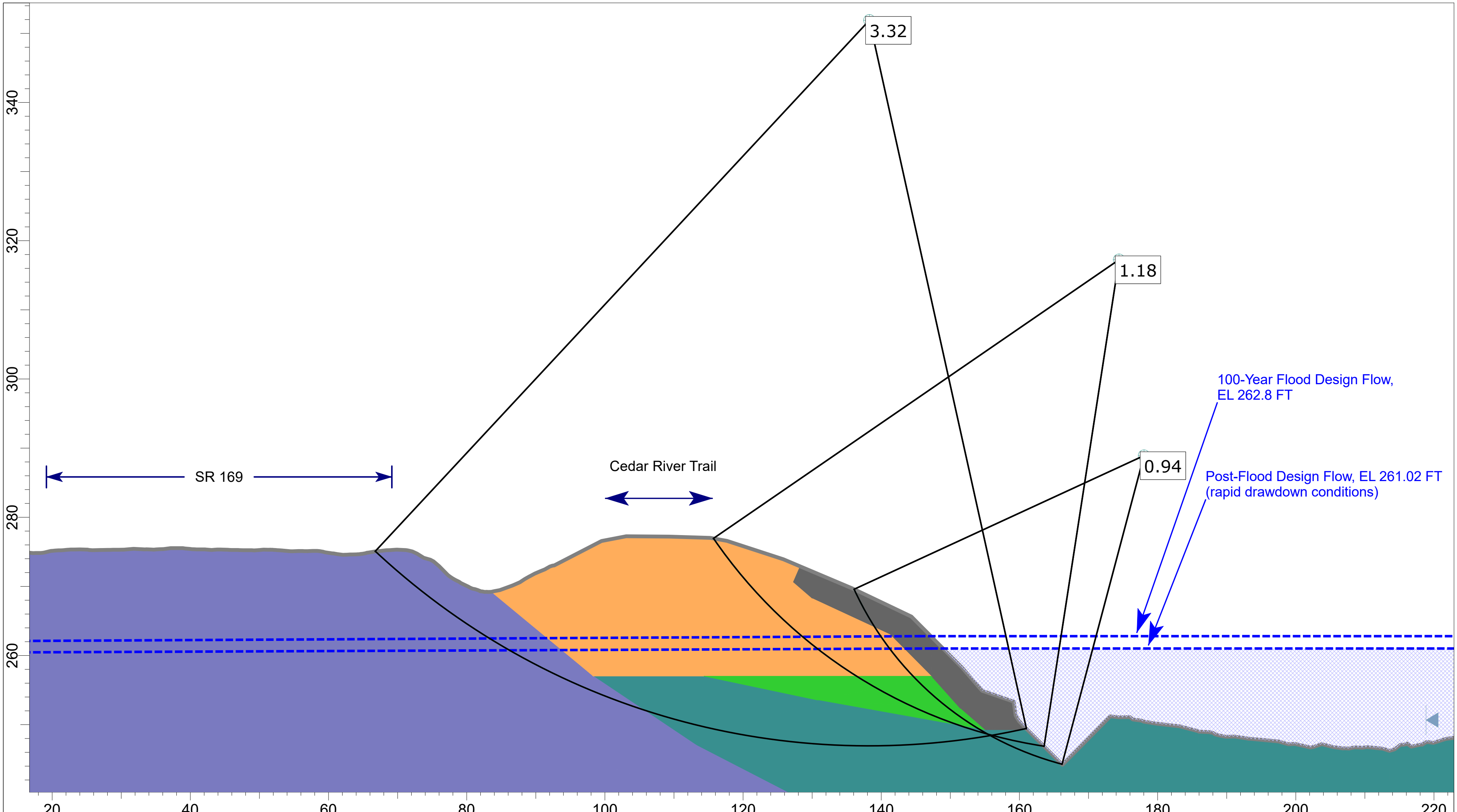
6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-29**



Legend

Query Slip Surface

Search Limits

Modeled Groundwater Level

Cross Section B-B', Alternative 2 With Potential Scour Rapid Drawdown

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020

SCALE: 1:240

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6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-30**

APPENDIX E

CRT7 Revetment Risk Assessment Workshop: Meeting Minutes

Meeting Minutes

Project No.: 190175-600.2

June 4, 2020

Attendees:	Tetra Tech	Jerry Scheller
	Aspect Consulting	Andy Holmson Henry Haselton Mari Otto Jordan Sanford
	King County Department of Natural Resources	Dan Heckendorf Chris Brummer Judi Radloff
	Watershed GeoDynamics	Kathy Dube
Date/Time:	June 4, 2020, 2:30 – 4:30 PM	
Location:	Teams Meeting (video conference)	
Called By:	Jerry Scheller	
Re:	**CRT7 Revetment Risk Assessment Workshop: Meeting Minutes**	

The purpose of this meeting was to review the findings from the Cedar River Trail Site 7 (CRT7) Revetment (Revetment) stability analysis to inform on the general existing vulnerability of the Revetment. Additionally, potential benefits and impacts, resulting from implementation of Jan Road project alternatives and adjacent Rutledge – Johnson project (upstream) concept design. This meeting was recorded by Jerry Scheller.

Revetment Existing Conditions

Andy presented an understanding of the existing conditions of the Revetment, including an overview of cross-sections, geology, boring locations, boring logs, armoring, adjacent 1998 damage and repair documentation, and riprap movement. Kathy provided input describing the historical active channel margins and movement trends noting indicators of migration occurring at the upstream end of the Revetment and identifying the high value of the existing deep pool at the base of the Revetment as aquatic habitat.

Jerry provided an overview of the hydraulic modeling and related notes/trends at and near the Revetment that included modeled flood results with depth, velocity, and critical particle diameter maps.

Summary of Stability Analyses for Existing Conditions

Andy presented a review of relevant stability criteria (USACE Levee Design Manual; WSDOT Geotechnical Design Manual), and presented the Revetment slope stability results based on existing conditions with future scour amongst four scenarios focusing on two key slip surfaces: 1) back of revetment and 2) edge of the Cedar River Trail (trail).

Graphical analysis results were reviewed, showing cross sections with failure surfaces and results relative to factor-of-safety and comparison to the relevant stability criteria (USACE, WSDOT). A description of incorporation of scour into the modeling and soil mechanics analyses was presented.

A summary of the results showed that the Revetment is marginally stable (factors of safety greater than 1.0) under existing conditions and does meet WSDOT stability criteria under certain scenarios; however, the Revetment is generally unstable under scoured conditions (factor of safety less than 1.0) and does not meet relevant stability criteria. Factors of safety along the back of the armor rock are less than factors of safety for failure surface passing through the edge of the trail.

Review of Project Alternatives

A summary of the project alternatives was provided by Jerry. The alternatives include:

- **Alternative 1:** Elevating the majority of the private roadway (SE 197th PL/221st Ave SE/218th Ave SE) and installation of cross culverts, setting back the Jan Road Levee, and floodplain regrading with side channels.

- **Alternative 2:** Setting back the Jan Road Levee and extending it to the east (upstream), floodplain regrading with side channels, and upgrades to the existing culvert under SE 197th.

Given the similarity in the alternatives, and resulting hydraulic model output, it was determined that discussion of the results for one of the alternatives would be sufficient. Jerry and Kathy provided a review of modeling results for instream velocity, water levels, sediment transport, and scour based on Alternative 2, relative to existing conditions. They also discussed potential channel migration pathways resulting from Alternative 2, and modeled shear stress as it relates to observed deposition and scour patterns. Changes in regard to water levels and scour elevations at the Revetment in the context of the stability analyses were summarized and presented by Andy.

Review of Stability Analyses for Alternatives

While there are minor changes in the calculated Revetment factor of safety values, when compared to the USACE and WSDOT stability criteria, there is not significant change due to the project alternatives. The Revetment is tall, steep, and comprised of loose fill soils, laying the framework for marginal stability. The Revetment is not prone to instability under rapid drawdown conditions due to relatively free-draining soils and typical Cedar River flood hydrograph shape (not flashy). Revetment stability is not sensitive to minor changes in water levels and scour. Based on observations made by Dan and Chris during recent flood conditions, modeled conditions appear consistent with observed response of revetment and flood hydrograph record.

Review of Adjacent Project (Rutledge-Johnson)

Kathy and Jerry presented modeling results of potential impacts from the adjacent and upstream Rutledge-Johnson levee removal project, based on a single preliminary design concept that would remove a downstream portion of the levee and activate an existing backwater channel. The results showed little difference at the Revetment relative to changes in river conditions considering the Jan Road project alternatives only.

Risk Discussion

The Revetment is not stable when compared to commonly accepted design criteria (USACE and WSDOT), but it is calculated to have a factor-of-safety greater than 1.0 based on the engineering properties assumed for the analysis. While mathematically computed as stable, performance of the Revetment under the conditions analyzed is less certain than it would be if the commonly accepted design criteria were satisfied, due to the inherent uncertainty of the engineering properties of geologic materials. Failure of the armor rock is considered the greatest risk when scour occurs; however, failures extending back into the trail are considered to carry a lower risk.

The analyses described above do not account for probable channel movement, as a result of project implementation. A comprehensive evaluation of risk would consider likely channel migration over time, and the implications of those temporal considerations on the overall vulnerability of the Revetment. Modeled conditions for Alternative 2 suggest less scour and erosive forces (velocity) along the CRT7 revetment relative to existing conditions. Scour depths along the facility are likely to lessen further over the medium-term (10 years), further reducing potential revetment instabilities related to scour. Further reduction in velocities are also anticipated to occur over this time frame, resulting in a greater likelihood for depositional zones near the facility, likely resulting in further limitation of scour at this location. Assuming Alternative 2 conditions, substantive channel migration is not expected to occur in the short-term. Long-term (50 years), the river may try to migrate and erode upstream of the Revetment pool; therefore, the project may consider installing flow deflection elements, upstream of this location, to direct flows away from the upstream end of the facility. Given the long-term nature of this projection, monitoring of this potential channel evolution scenario was recommended as a minimum action.

Conceptual Stabilization Alternatives

Andy provided a discussion of potential conceptual approaches to improving the stability of the Revetment that included reactive or proactive approaches. A reactive approach would be to repair the Revetment if it fails, in a manner similar to the adjacent 1998 repair. Proactive approaches to stabilize the Revetment to an acceptable level of stability when compared to the USACE and WSDOT design criteria could include:

- Proactive repair of the Revetment in a manner similar to the adjacent 1998 repair.
  - Could offer improved in-water habitat (wood debris incorporated into repair).
  - Clearing of existing vegetation necessary for the repair would eliminate shade and cooling from the established large trees growing on the Revetment.
- Buttressing along the toe of the Revetment and scour hole.
  - Logistics of in-water construction and impacts of buttress on stream channel identified as potential drawbacks.

- Regrade the Revetment to a flatter angle (2H:1V)
  - More favorable for long-term stability.
  - Would require moving trail and possibly impacting highway right-of-way.
  - Clearing of vegetation would be a temporary impact to shade and cooling.

In considering these conceptual stabilization approaches, the County suggested that the co-managers would likely prioritize protection of habitat and resist impacts to habitat, suggesting that the reactive approach may be favored. The County will discuss this internally.

Mitigation of Revetment vulnerabilities through project design by controlling flows, velocities, and channel migration area away from the Revetment was discussed. The team agreed that the project could include design elements and provisions to influence the geomorphology and hydraulics in a way that would focus less severe forces against the Revetment.

Next Steps

- County to follow-up internally following this presentation to discuss the presented content, any additional questions/comments, and expectations for the risk assessment reporting. Dan to provide feedback to the consultant team.

- A design study for the CRT2 revetment (downstream) was completed in 2019 and will be provided by the County. Aspect will review the study for the purpose of applying consistent methodologies for similar structures/evaluations within the Cedar River.

- Project team will revise presentation materials and redistribute to County and provide these meeting notes.

- Aspect to proceed with preparing draft CRT7 Revetment Risk Assessment Memorandum under Task 600.2.

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APPENDIX F

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.

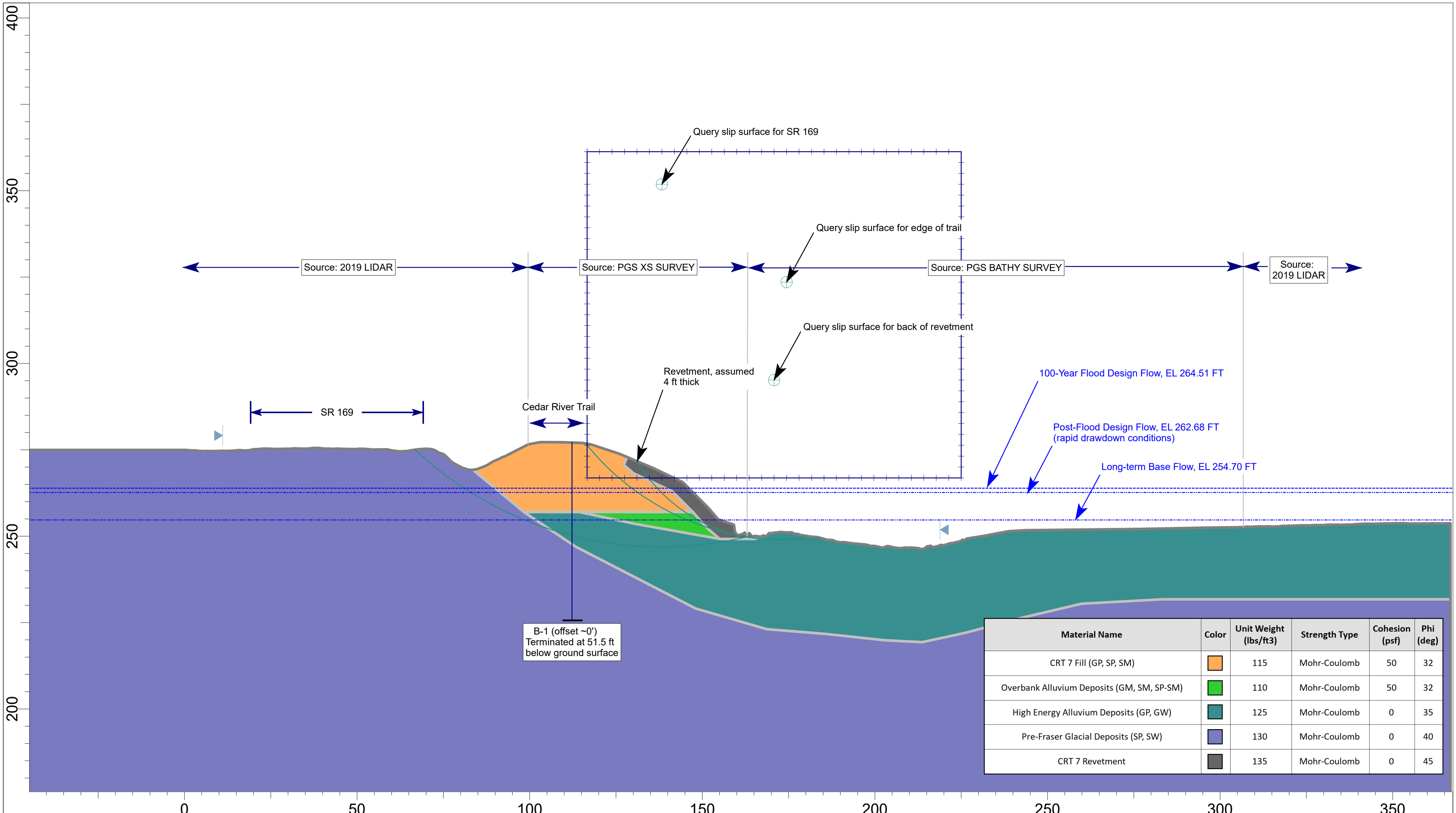
Geotechnical, Geologic, and Environmental Reports Are Not Interchangeable

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.

APPENDIX D

Slope Stability Analysis Outputs Section B-B’



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
CRT 7 Fill (GP, SP, SM)		115	Mohr-Coulomb	50	32
Overbank Alluvium Deposits (GM, SM, SP-SM)		110	Mohr-Coulomb	50	32
High Energy Alluvium Deposits (GP, GW)		125	Mohr-Coulomb	0	35
Pre-Fraser Glacial Deposits (SP, SW)		130	Mohr-Coulomb	0	40
CRT 7 Revetment		135	Mohr-Coulomb	0	45

Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

Cross Section B-B', Existing Conditions, Non-Scoured Conditions Master Scenario

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment, CRT7 Revetment
Jan Road Neighborhood Improvements
King County, Washington

SLIDE 8.020

SCALE: 1:480

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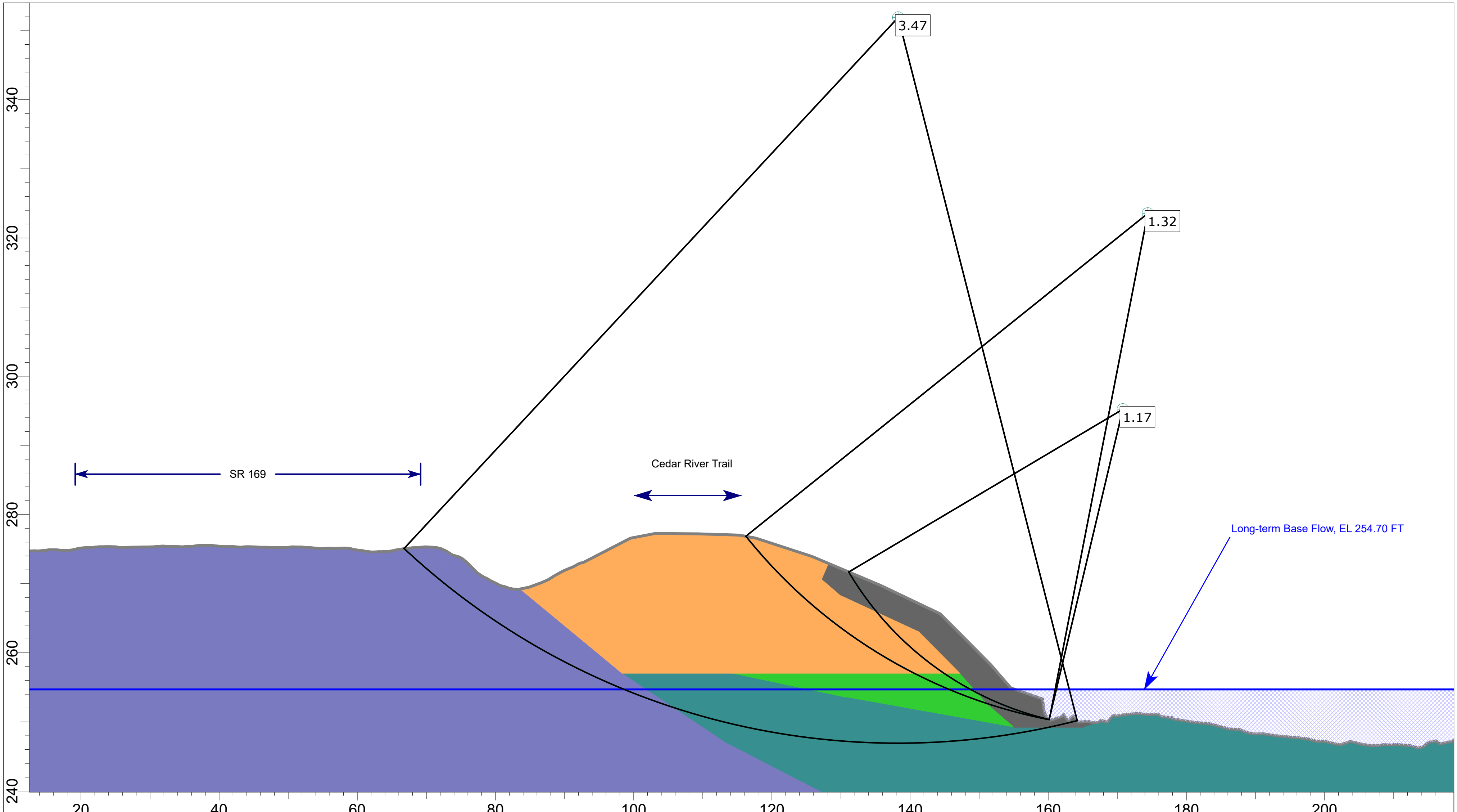
6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: D-1



Legend

- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Existing Conditions, Non-Scoured Conditions Static Conditions

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment, CRT7 Revetment
Jan Road Neighborhood Improvements
King County, Washington

SLIDEINTERPRET 8.020

SCALE: 1:240

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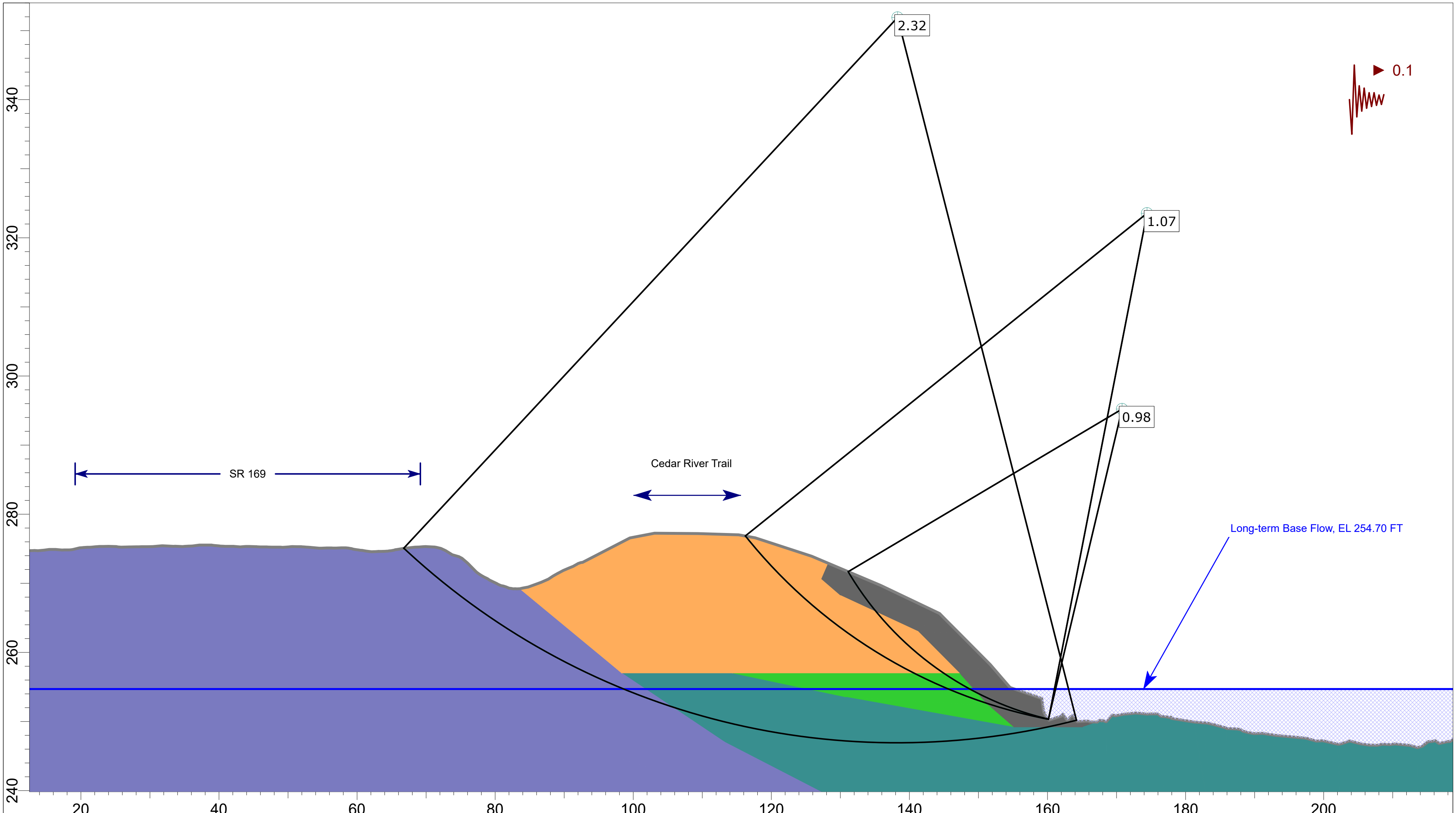
6/23/2020	BY: MO	APPENDIX:	D-2
PROJECT NO. 190175	REVIEWED BY: AJH		



Legend

- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Existing Conditions, Non-Scoured Conditions Seismic Conditions

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment, CRT7 Revetment
Jan Road Neighborhood Improvements
King County, Washington

SLIDEINTERPRET 8.020

SCALE: 1:240

S:\King County\Jan Road Neighborhood Levee\Data\Analyses\04_SSA CRT7\crt7_2020.05.07_Existing Conditions.slmd

Aspect CONSULTING

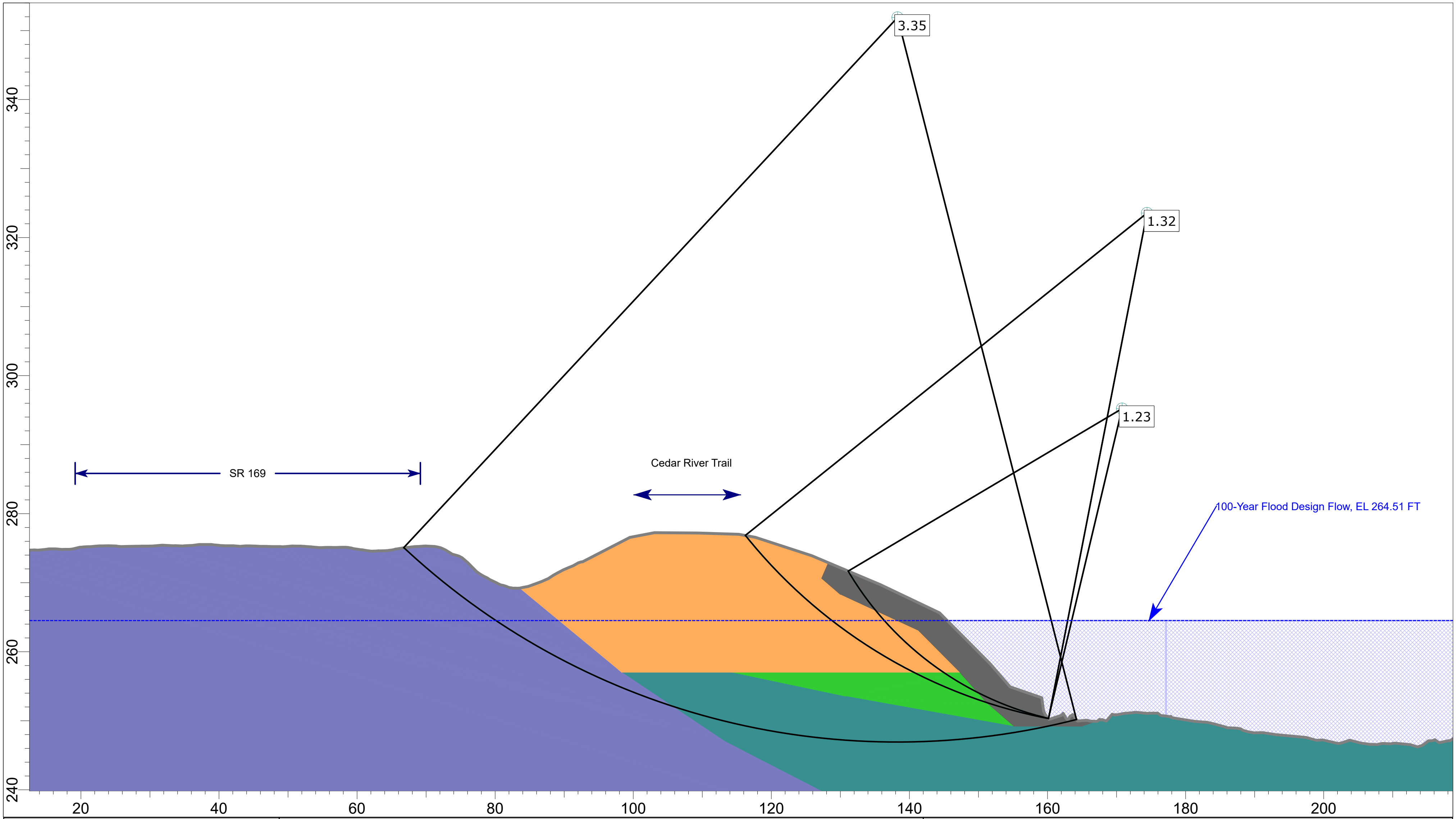
6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-3**



Legend

Query Slip Surface
Search Limits
Modeled Groundwater Level

Cross Section B-B', Existing Conditions, Non-Scoured Conditions Peak Flood Level

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment, CRT7 Revetment
Jan Road Neighborhood Improvements
King County, Washington

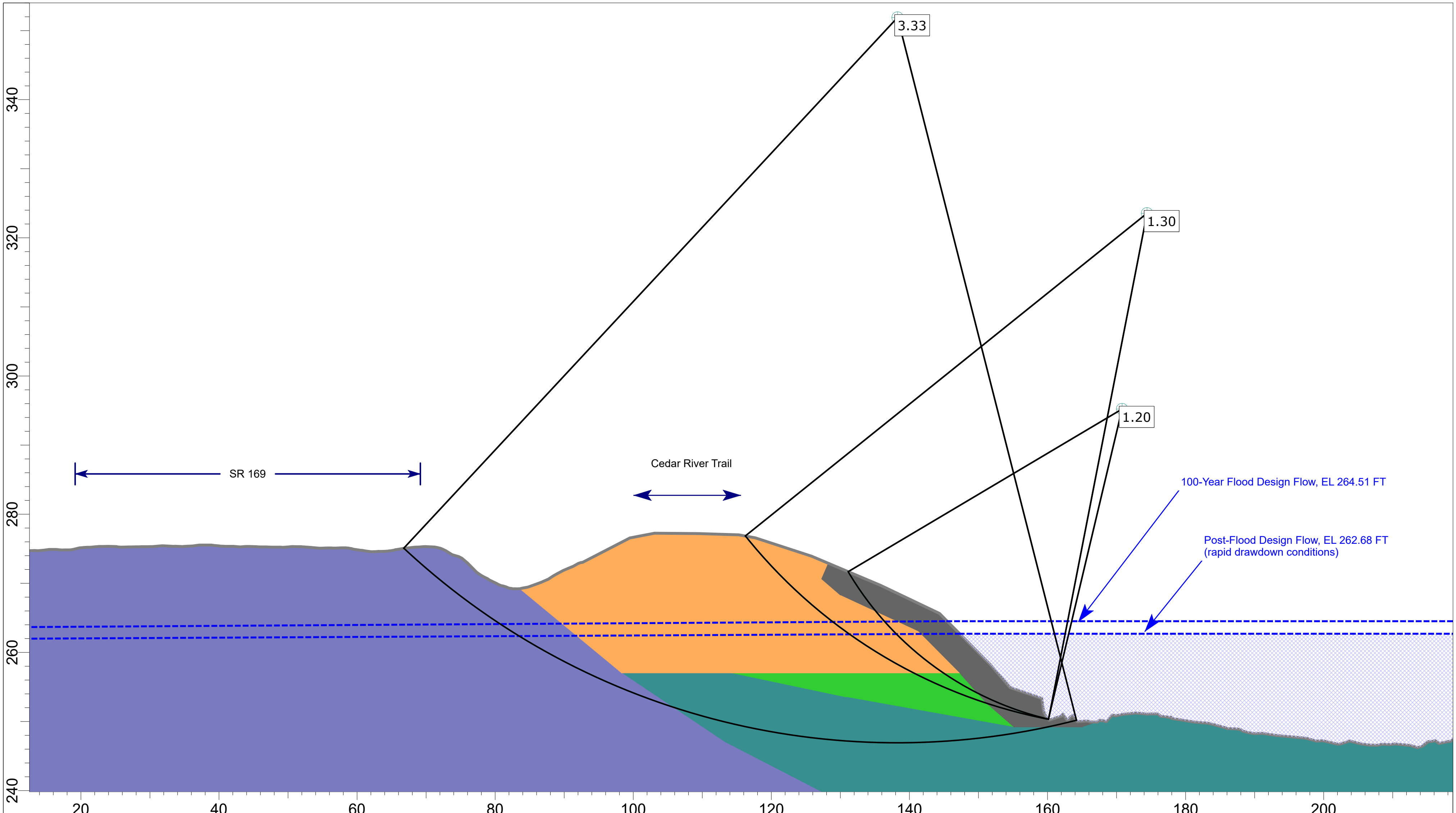
SLIDEINTERPRET 8.020

SCALE: 1:240

S:\King County\Jan Road Neighborhood Levee\Data\Analyses\04_SSA CRT7\crt7_2020.05.07_Existing Conditions.slmd

Aspect CONSULTING

6/23/2020	BY: MO	APPENDIX:
PROJECT NO. 190175	REVIEWED BY: AJH	**D-4**



Legend

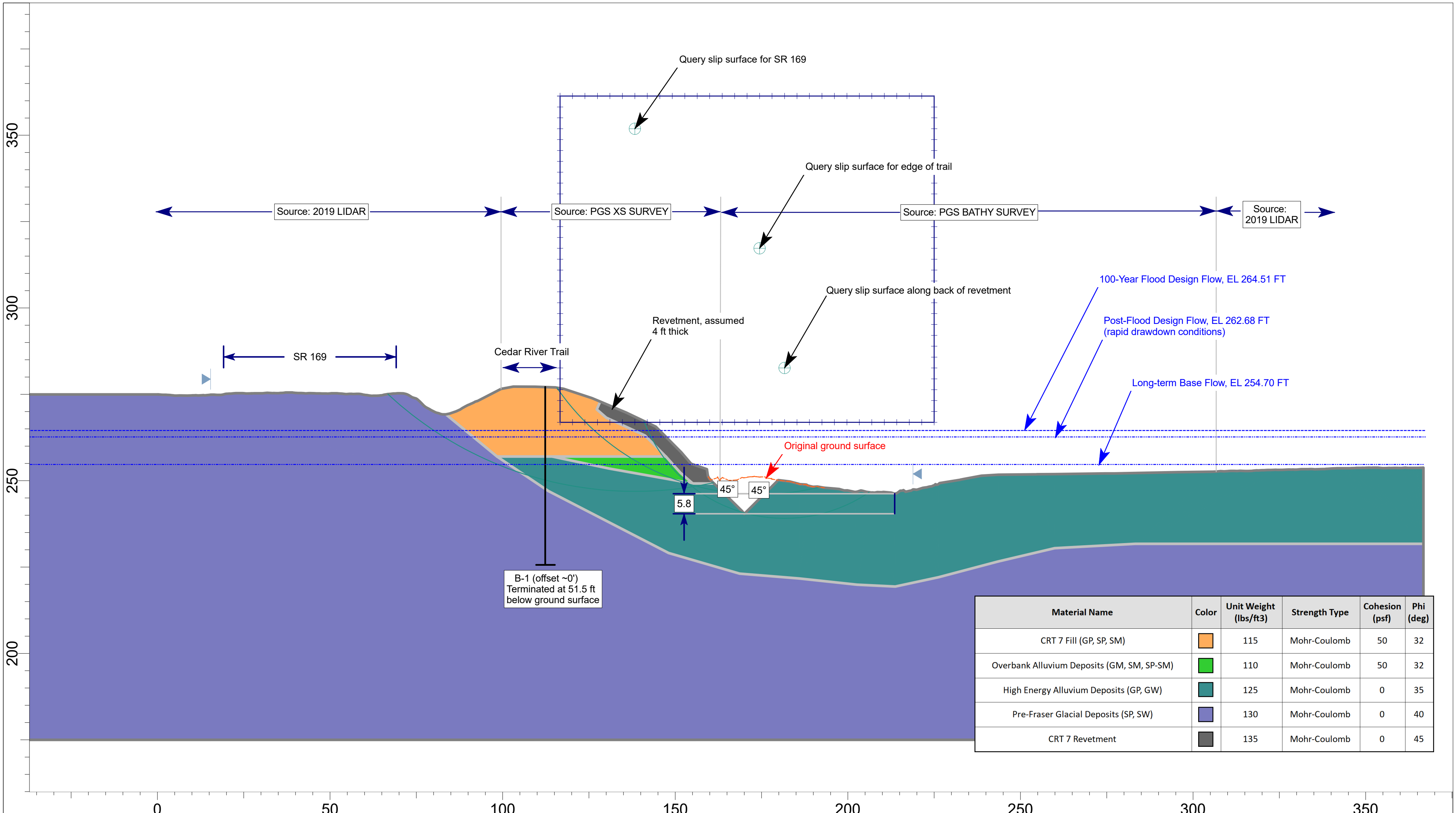
Query Slip Surface
Search Limits
Modeled Groundwater Level

Cross Section B-B', Existing Conditions, Non-Scoured Conditions Rapid Drawdown

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment, CRT7 Revetment
Jan Road Neighborhood Improvements
King County, Washington

SLIDEINTERPRET 8.020	SCALE: 1:240 S:\King County\Jan Road Neighborhood Levee\Data\Analyses\04_SSA CRT7\crt7_2020.05.07_Existing Conditions.slmd	Aspect CONSULTING	6/23/2020 PROJECT NO. 190175	BY: MO REVIEWED BY: AJH	APPENDIX: **D-5**



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
CRT 7 Fill (GP, SP, SM)		115	Mohr-Coulomb	50	32
Overbank Alluvium Deposits (GM, SM, SP-SM)		110	Mohr-Coulomb	50	32
High Energy Alluvium Deposits (GP, GW)		125	Mohr-Coulomb	0	35
Pre-Fraser Glacial Deposits (SP, SW)		130	Mohr-Coulomb	0	40
CRT 7 Revetment		135	Mohr-Coulomb	0	45

Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

Cross Section B-B', Existing Conditions With Potential Scour Master Scenario

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDE 8.020

SCALE: 1:480

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Aspect CONSULTING

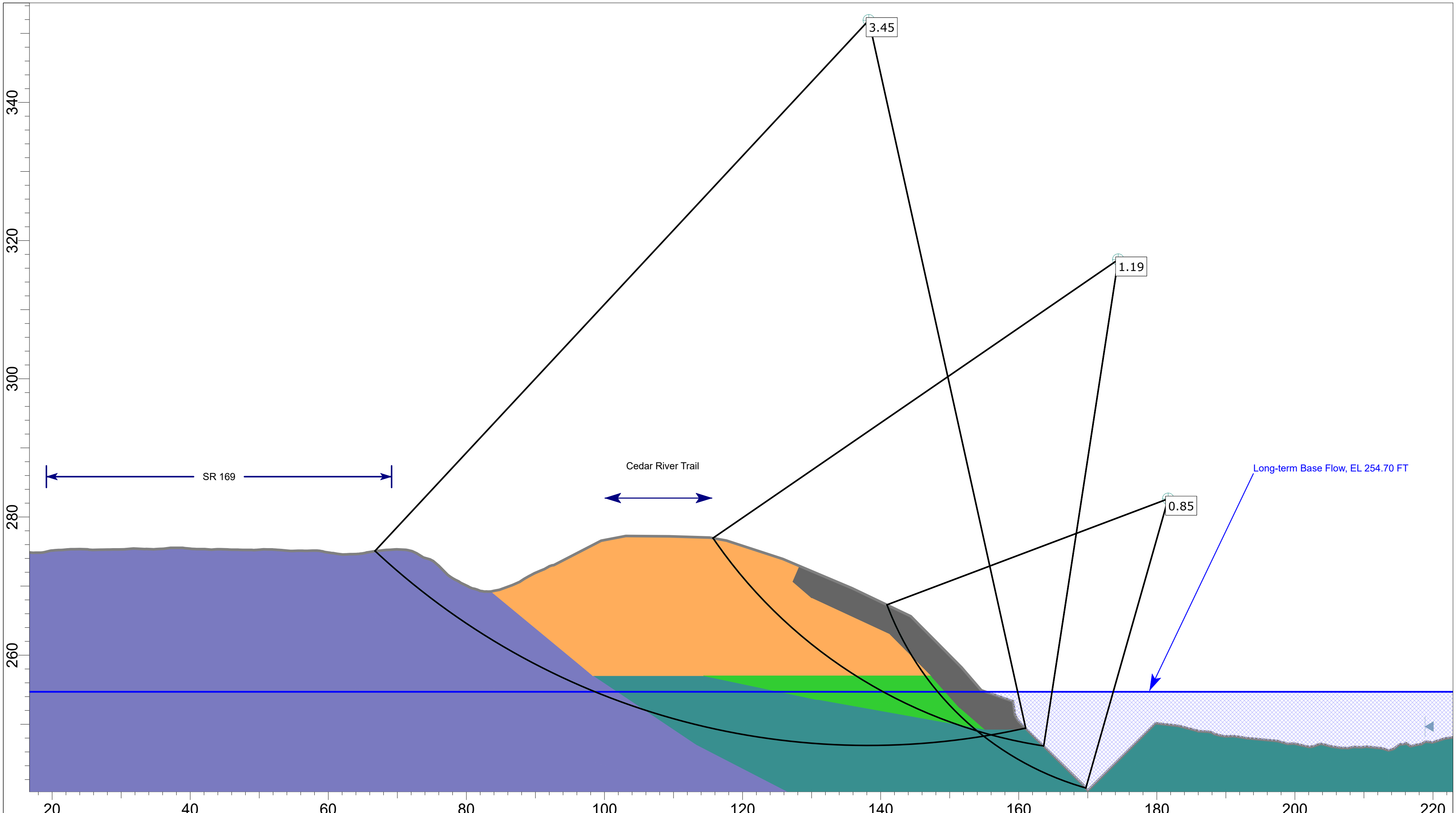
6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: D-6



Legend

- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Existing Conditions With Potential Scour Static Conditions

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020

SCALE: 1:240

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Aspect CONSULTING

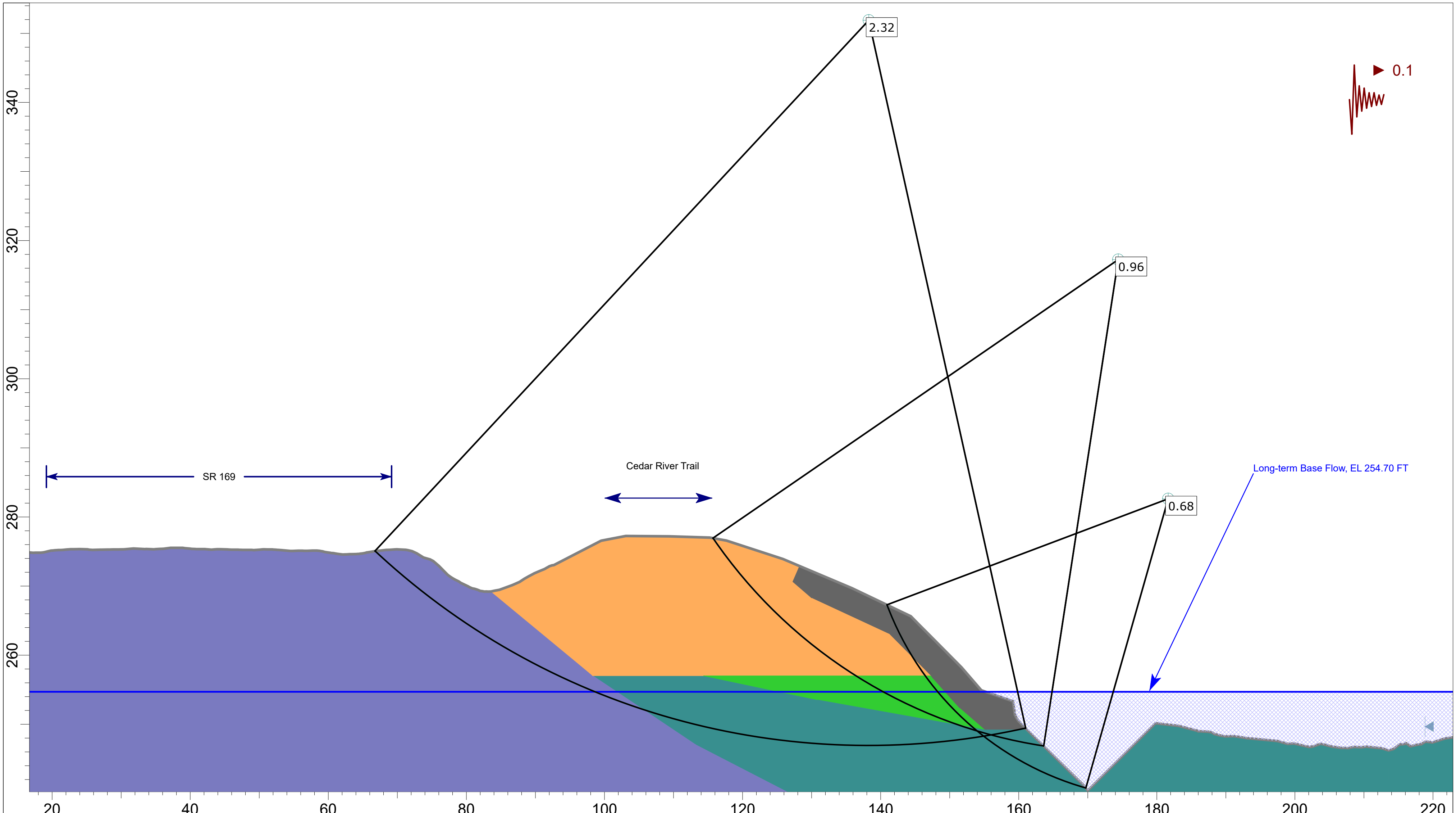
6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-7**



Legend

- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Existing Conditions With Potential Scour Seismic Conditions

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020

SCALE: 1:240

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Aspect CONSULTING

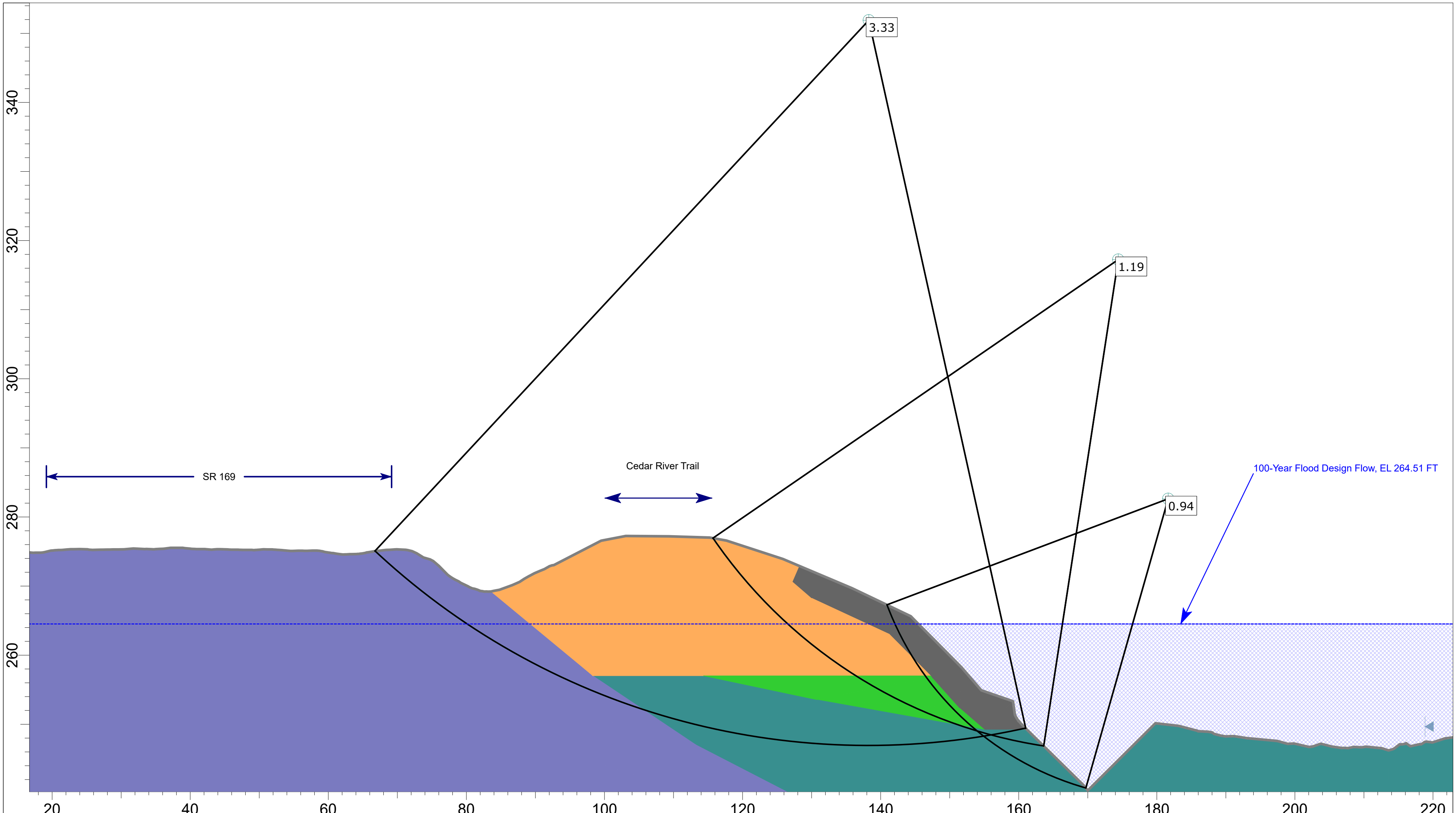
6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-8**



Legend

- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Existing Conditions With Potential Scour Peak Flood Level

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

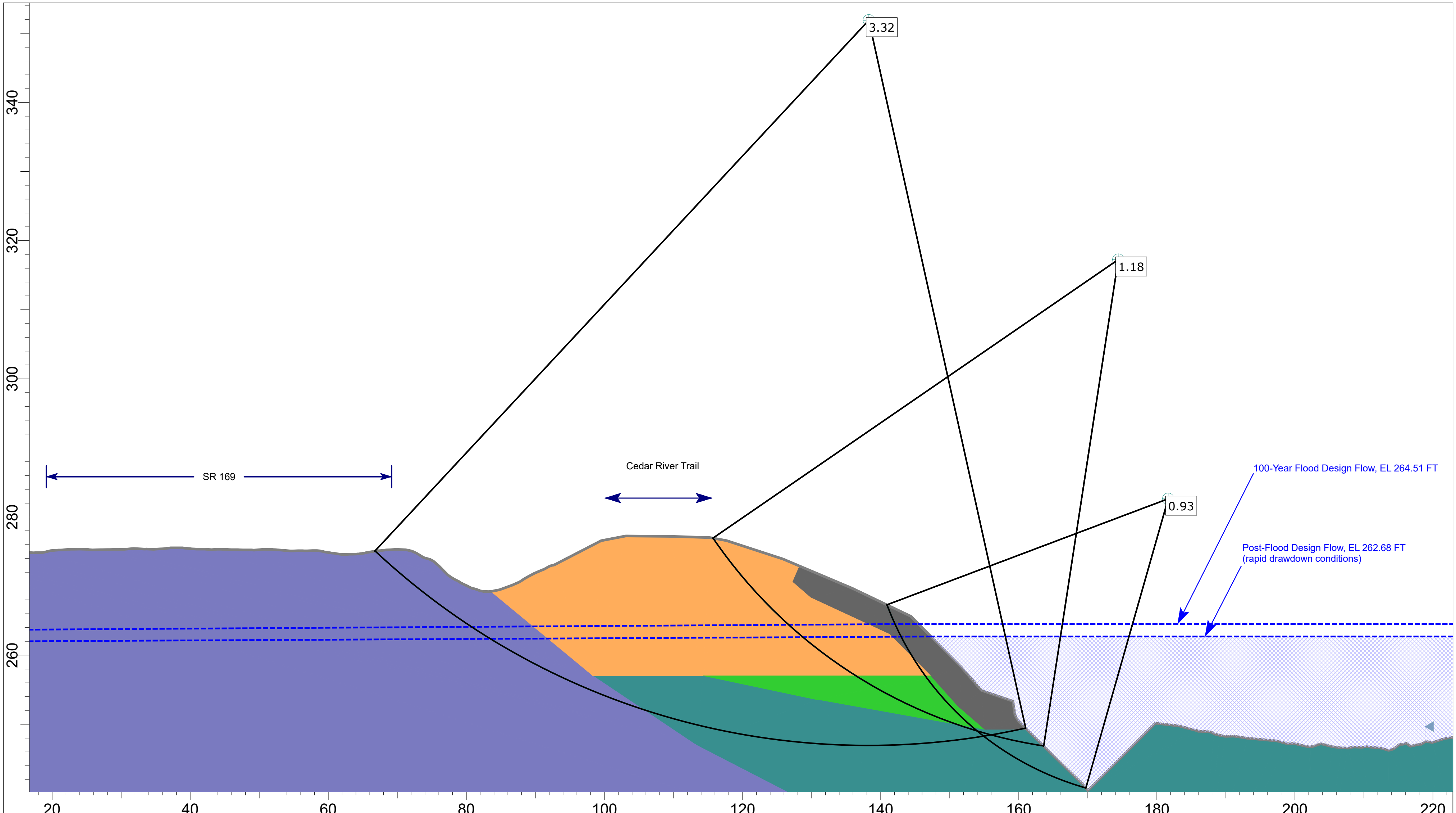
SLIDEINTERPRET 8.020

SCALE: 1:240

S:\King County\Jan Road Neighborhood Levee\Data\Analyses\04_SSA CRT7\crt7_2020.05.07_existing conditions w scour analysis.slmd

Aspect CONSULTING

6/23/2020	BY: MO	APPENDIX:
PROJECT NO. 190175	REVIEWED BY: AJH	**D-9**



Legend

Query Slip Surface
Search Limits
Modeled Groundwater Level

Cross Section B-B', Existing Conditions With Potential Scour Rapid Drawdown

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

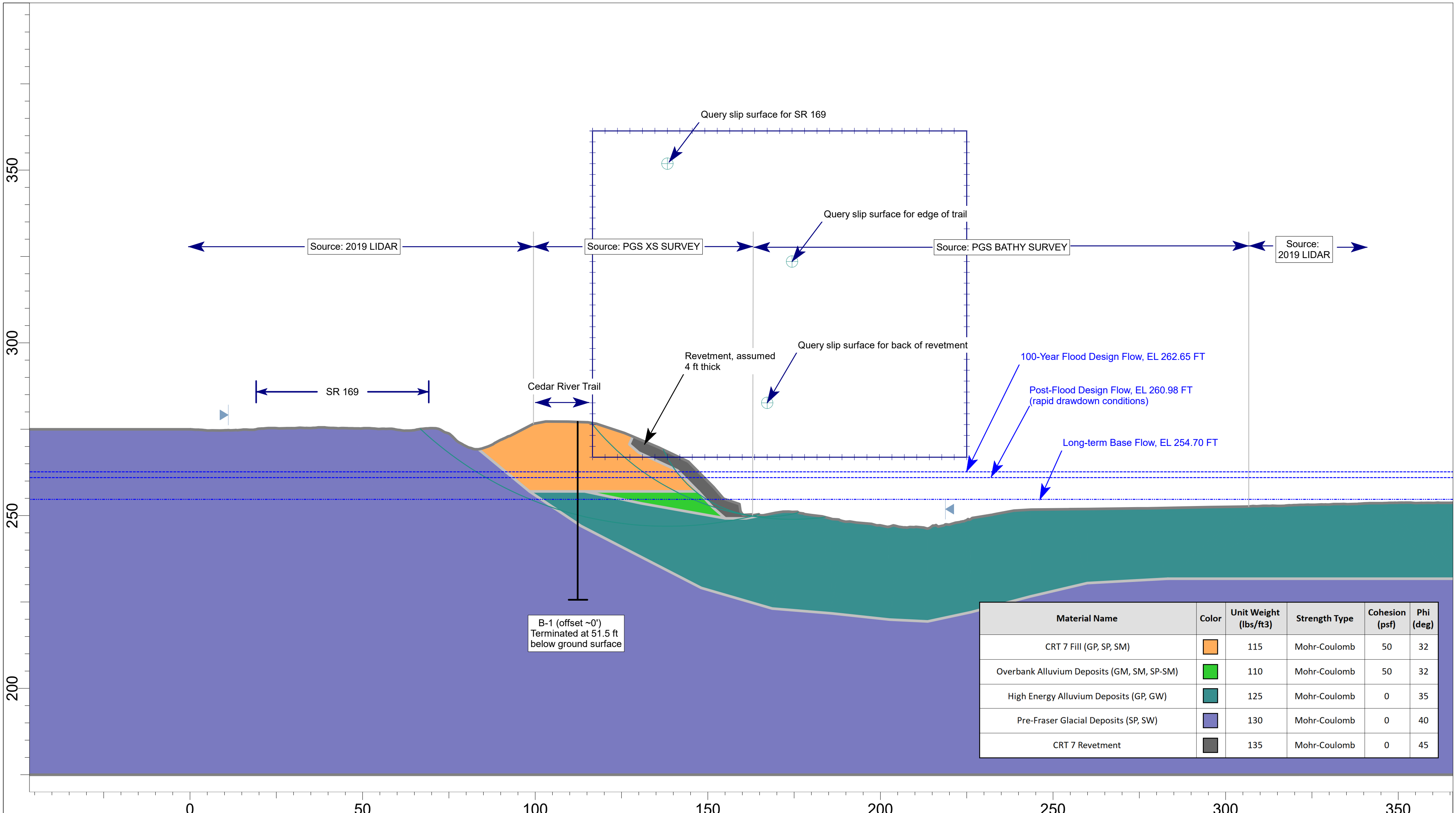
SLIDEINTERPRET 8.020

SCALE: 1:240

S:\King County\Jan Road Neighborhood Levee\Data\Analyses\04_SSA CRT7\crt7_2020.05.07_existing conditions w scour analysis.slmd

Aspect CONSULTING

6/23/2020	BY: MO	APPENDIX:
PROJECT NO. 190175	REVIEWED BY: AJH	D-10



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
CRT 7 Fill (GP, SP, SM)		115	Mohr-Coulomb	50	32
Overbank Alluvium Deposits (GM, SM, SP-SM)		110	Mohr-Coulomb	50	32
High Energy Alluvium Deposits (GP, GW)		125	Mohr-Coulomb	0	35
Pre-Fraser Glacial Deposits (SP, SW)		130	Mohr-Coulomb	0	40
CRT 7 Revetment		135	Mohr-Coulomb	0	45

Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

Cross Section B-B', Alternative 1, Non-Scoured Conditions Master Scenario

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDE 8.020

SCALE: 1:480

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Aspect CONSULTING

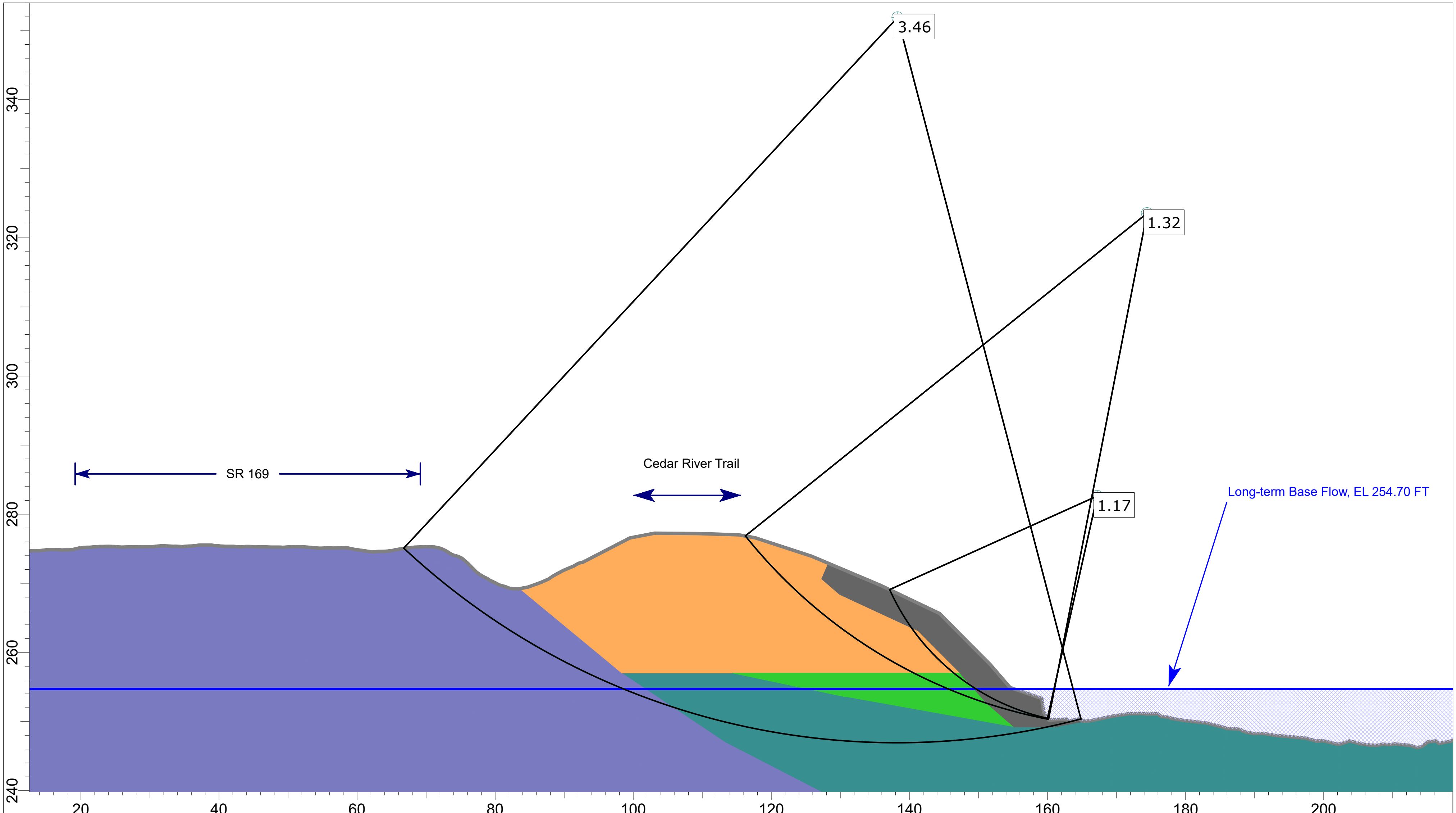
6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-11**



Legend

- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Alternative 1, Non-Scoured Conditions
Static Conditions

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020

SCALE: 1:240
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Aspect CONSULTING

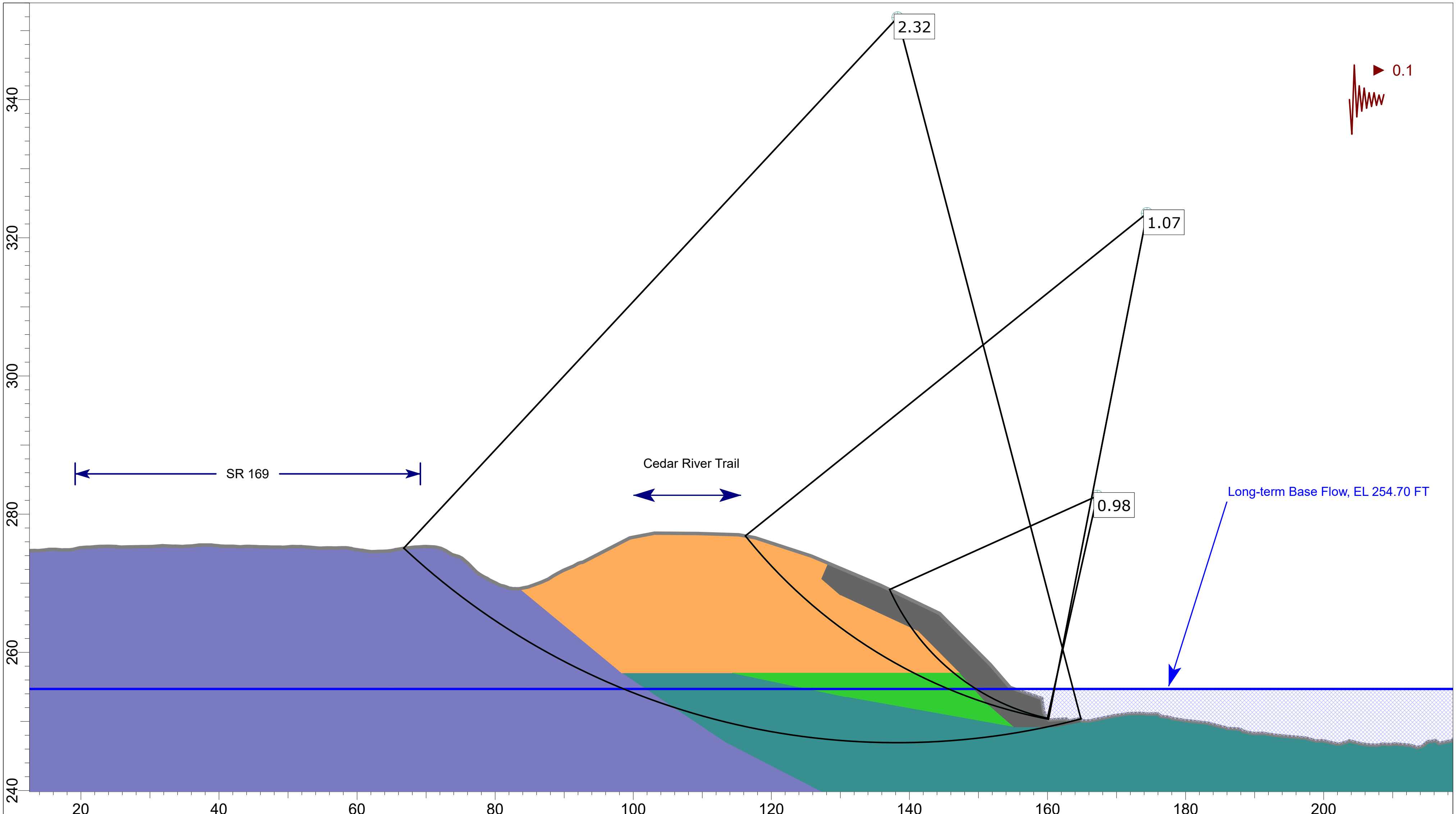
6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-12**



Legend

- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Alternative 1, Non-Scoured Conditions
Seismic Conditions

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020

SCALE: 1:240

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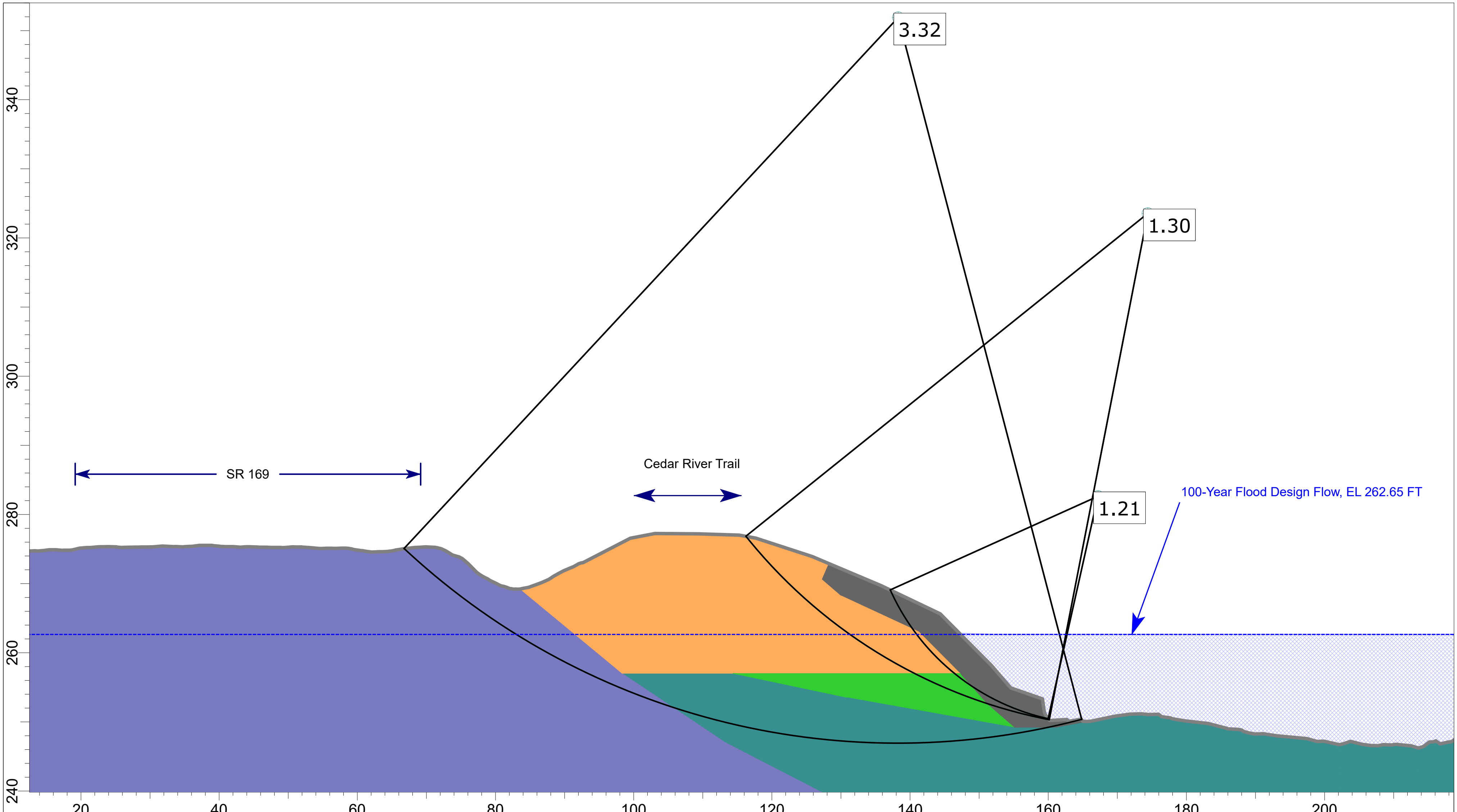
Aspect CONSULTING

6/23/2020	BY: MO	APPENDIX:
PROJECT NO. 190175	REVIEWED BY: AJH	**D-13**



Legend

- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Alternative 1, Non-Scoured Conditions
Peak Flood Level

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

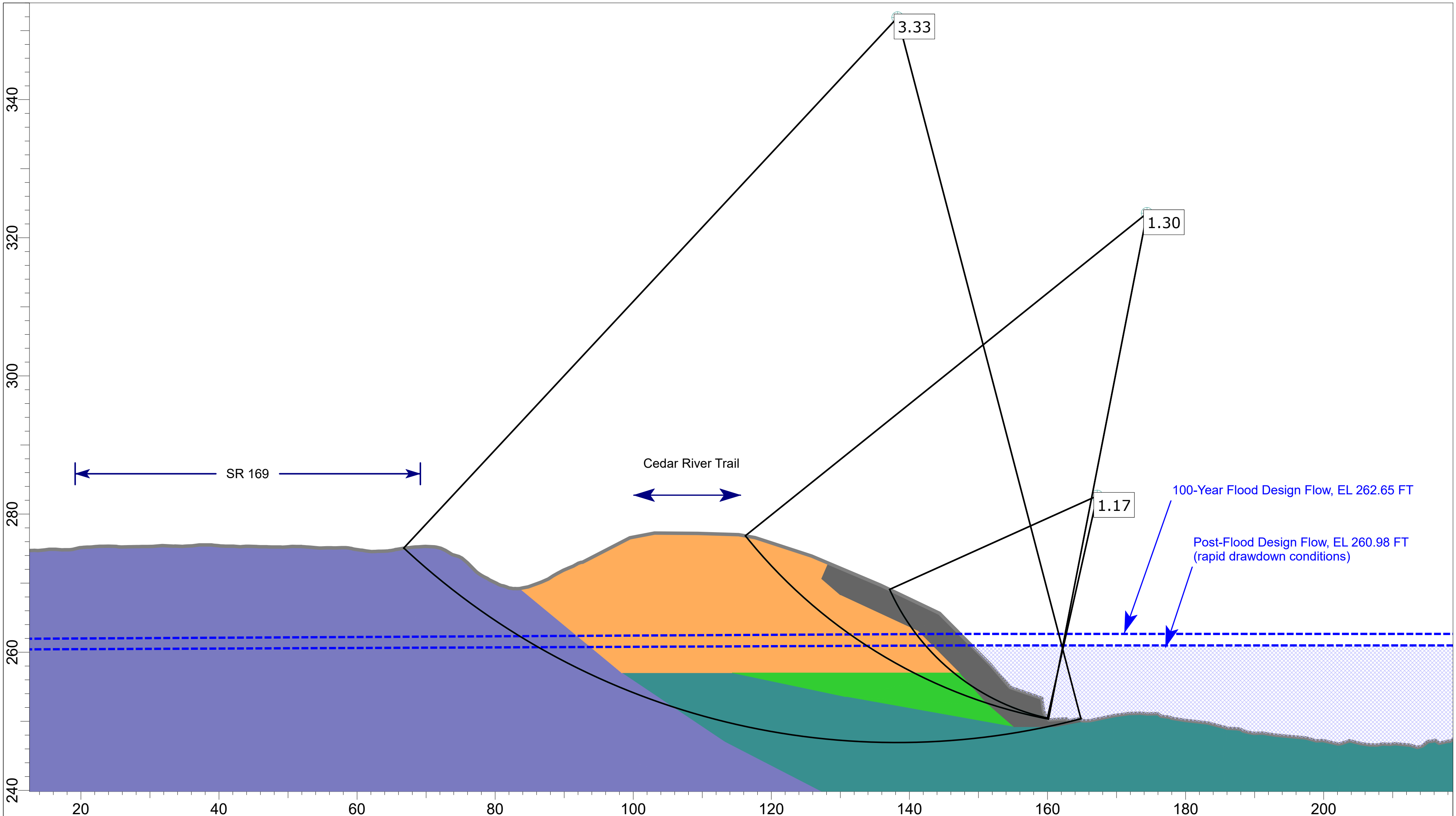
SLIDEINTERPRET 8.020

SCALE: 1:240

S:\King County\Jan Road Neighborhood Levee\Data\Analyses\04_SSA CRT7\crt7_2020.04.22_alternative 1 existing conditions.slmd

Aspect CONSULTING

6/23/2020	BY: MO	APPENDIX:
PROJECT NO. 190175	REVIEWED BY: AJH	D-14



Legend

- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Alternative 1, Non-Scoured Conditions Rapid Drawdown

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020

SCALE: 1:240
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Aspect CONSULTING

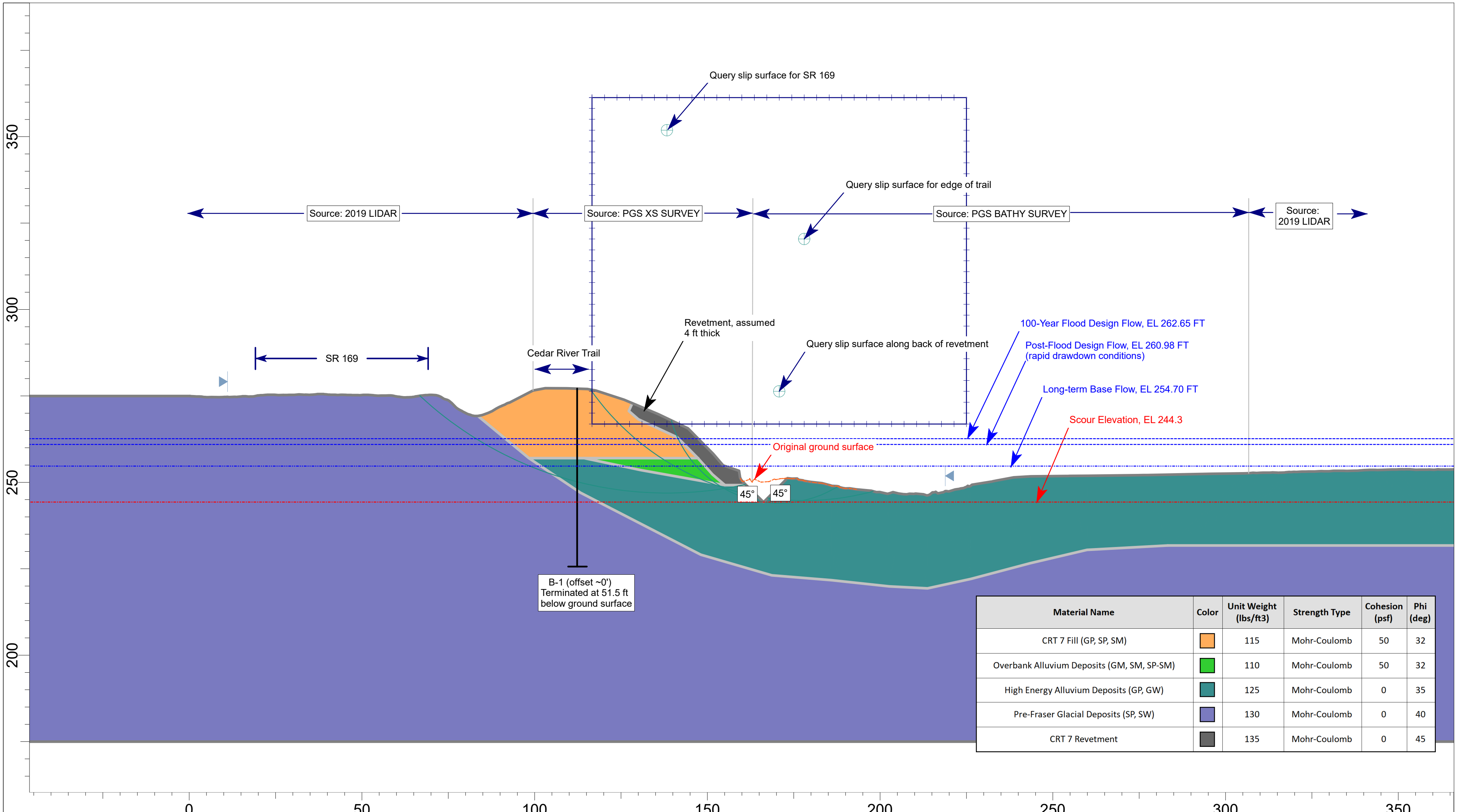
6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-15**



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
CRT 7 Fill (GP, SP, SM)		115	Mohr-Coulomb	50	32
Overbank Alluvium Deposits (GM, SM, SP-SM)		110	Mohr-Coulomb	50	32
High Energy Alluvium Deposits (GP, GW)		125	Mohr-Coulomb	0	35
Pre-Fraser Glacial Deposits (SP, SW)		130	Mohr-Coulomb	0	40
CRT 7 Revetment		135	Mohr-Coulomb	0	45

Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

Cross Section B-B', Alternative 1 With Potential Scour
Master Scenario

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDE 8.020

SCALE: 1:480

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Aspect CONSULTING

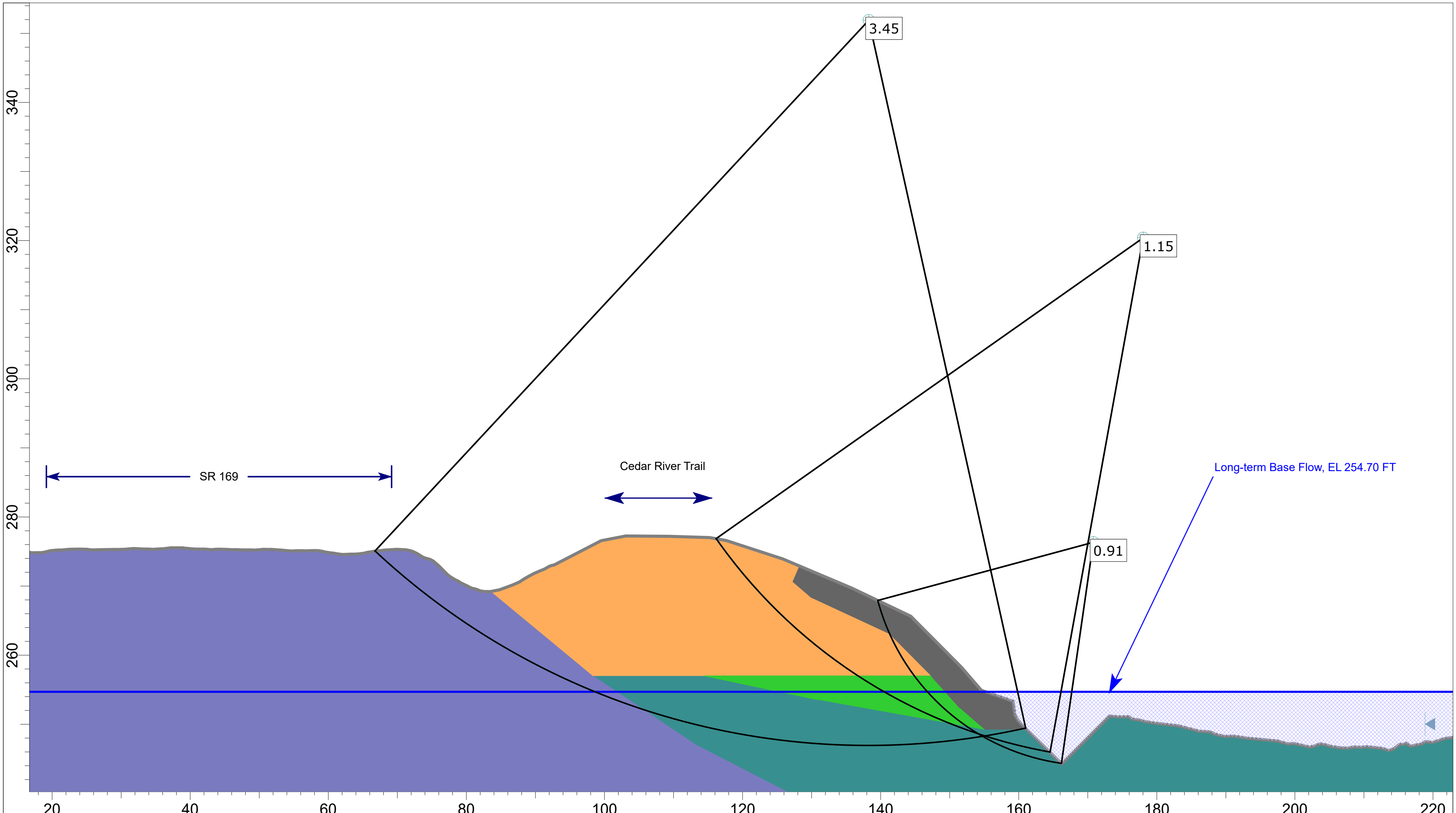
6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-16**



Legend

- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Alternative 1 With Potential Scour
Static Conditions

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020

SCALE: 1:240

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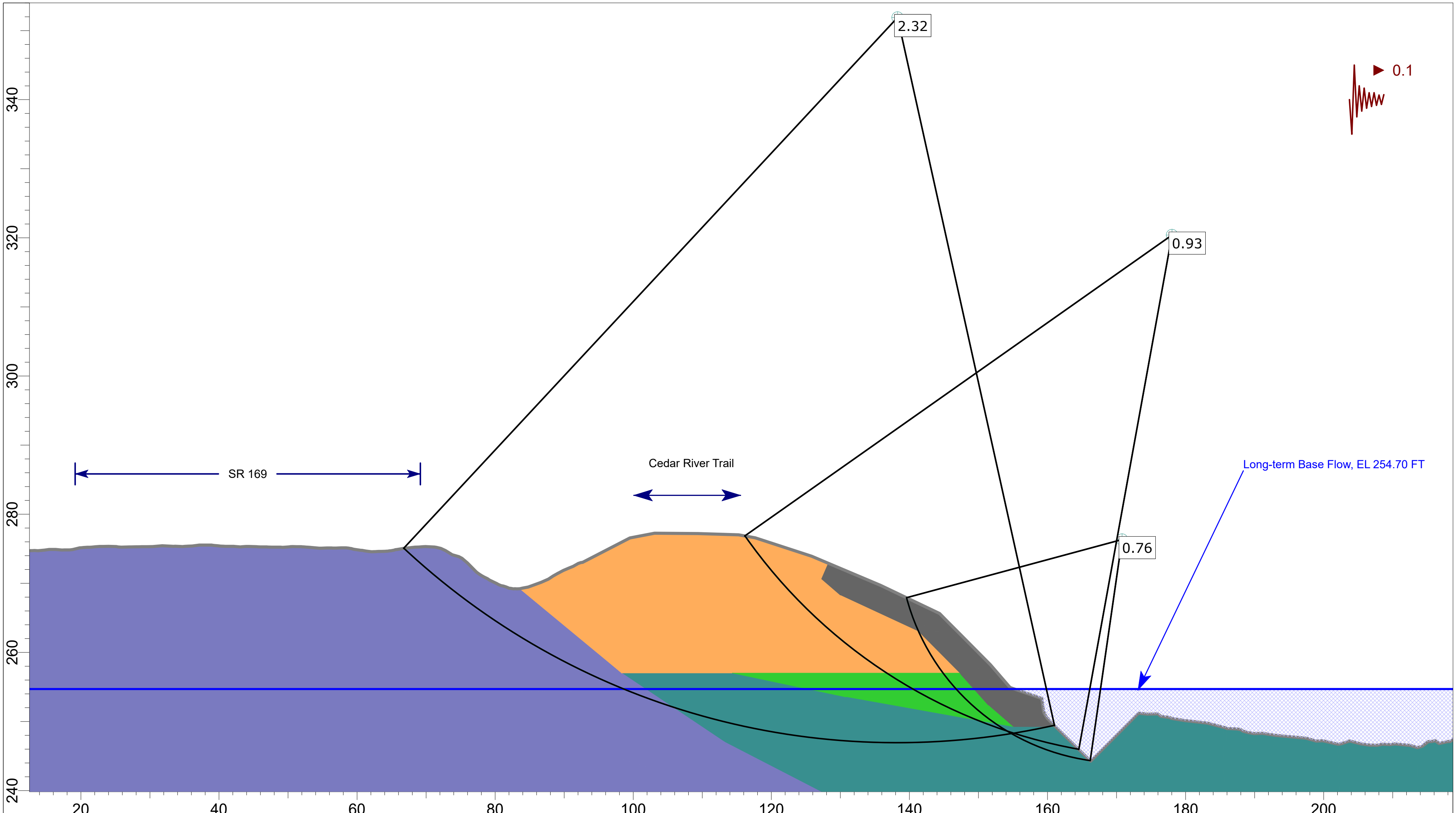
Aspect CONSULTING

6/23/2020	BY: MO	APPENDIX:
PROJECT NO. 190175	REVIEWED BY: AJH	D-17



Legend

- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Alternative 1 With Potential Scour
Seismic Conditions

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020

SCALE: 1:240

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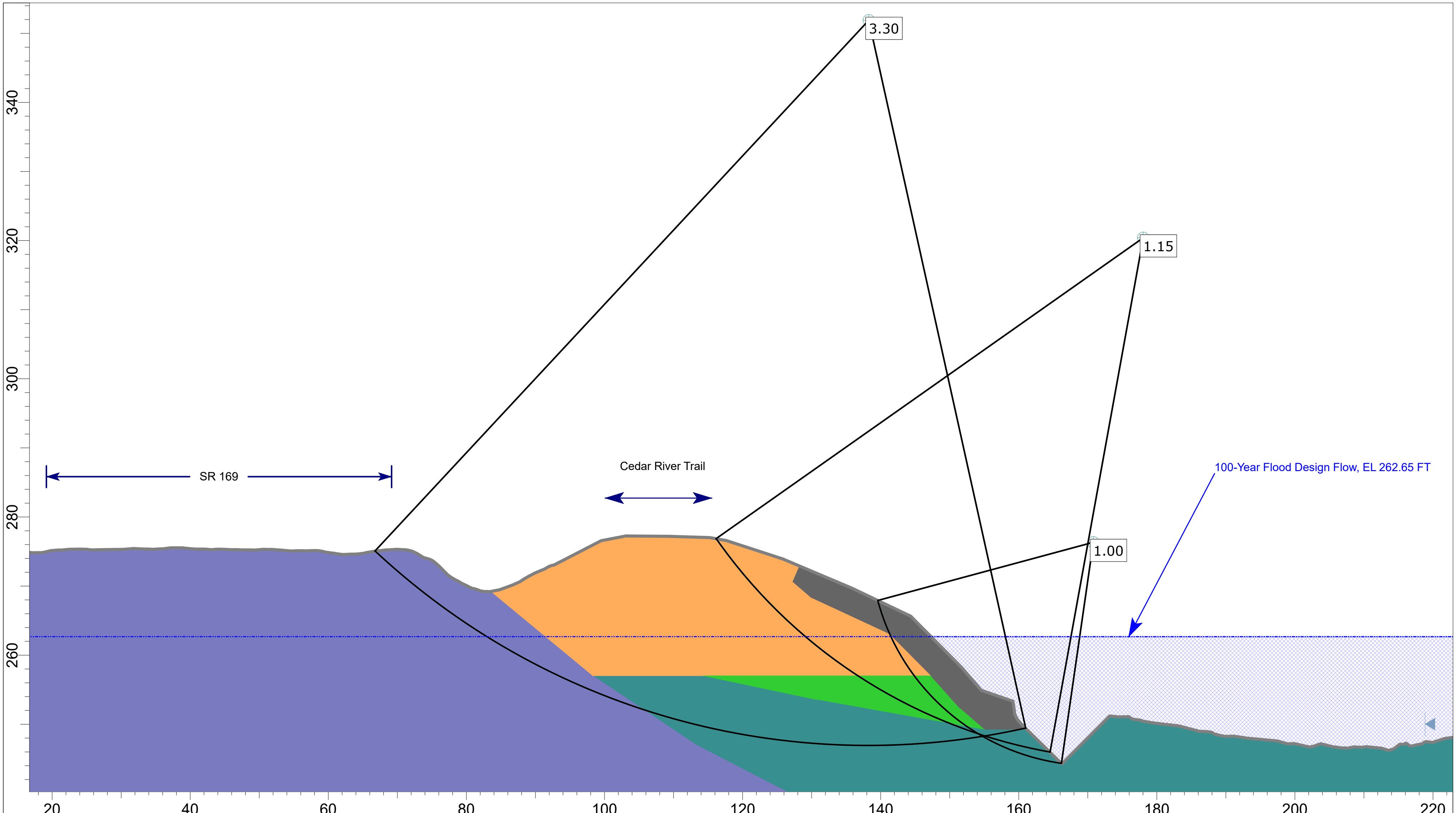
Aspect CONSULTING

6/23/2020	BY: MO	APPENDIX:
PROJECT NO. 190175	REVIEWED BY: AJH	**D-18**



Legend

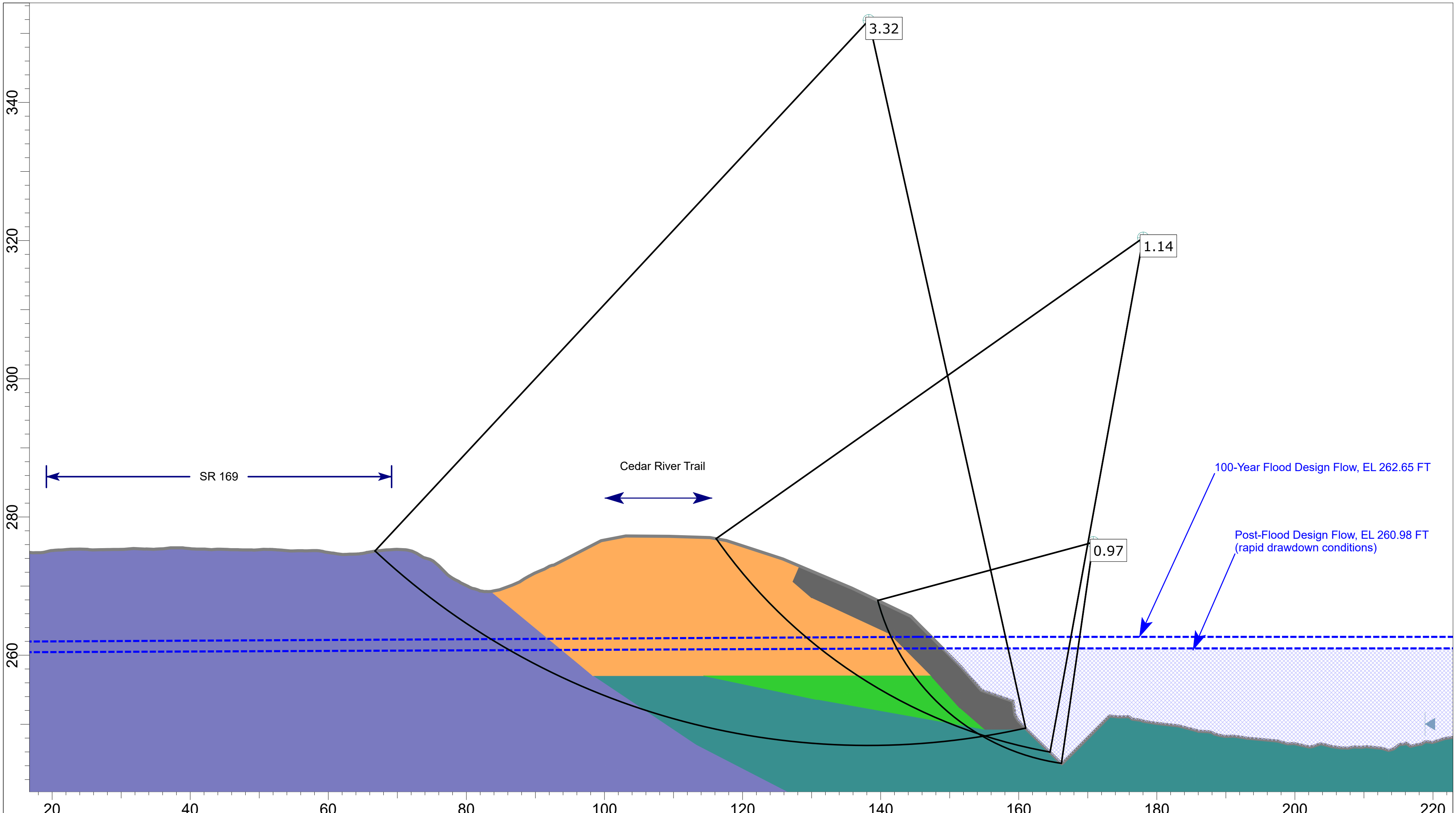
- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Alternative 1 With Potential Scour
Peak Flood Level

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020	SCALE: 1:240 S:\King County\Jan Road Neighborhood Levee\Data\Analyses\04_SSA CRT7\crt7_2020.04.22_alternative 1 with scour.slmd	Aspect CONSULTING	6/23/2020 PROJECT NO. 190175	BY: MO REVIEWED BY: AJH	APPENDIX: D-19



Legend

- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Alternative 1 With Potential Scour
Rapid Drawdown

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020

SCALE: 1:240

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Aspect CONSULTING

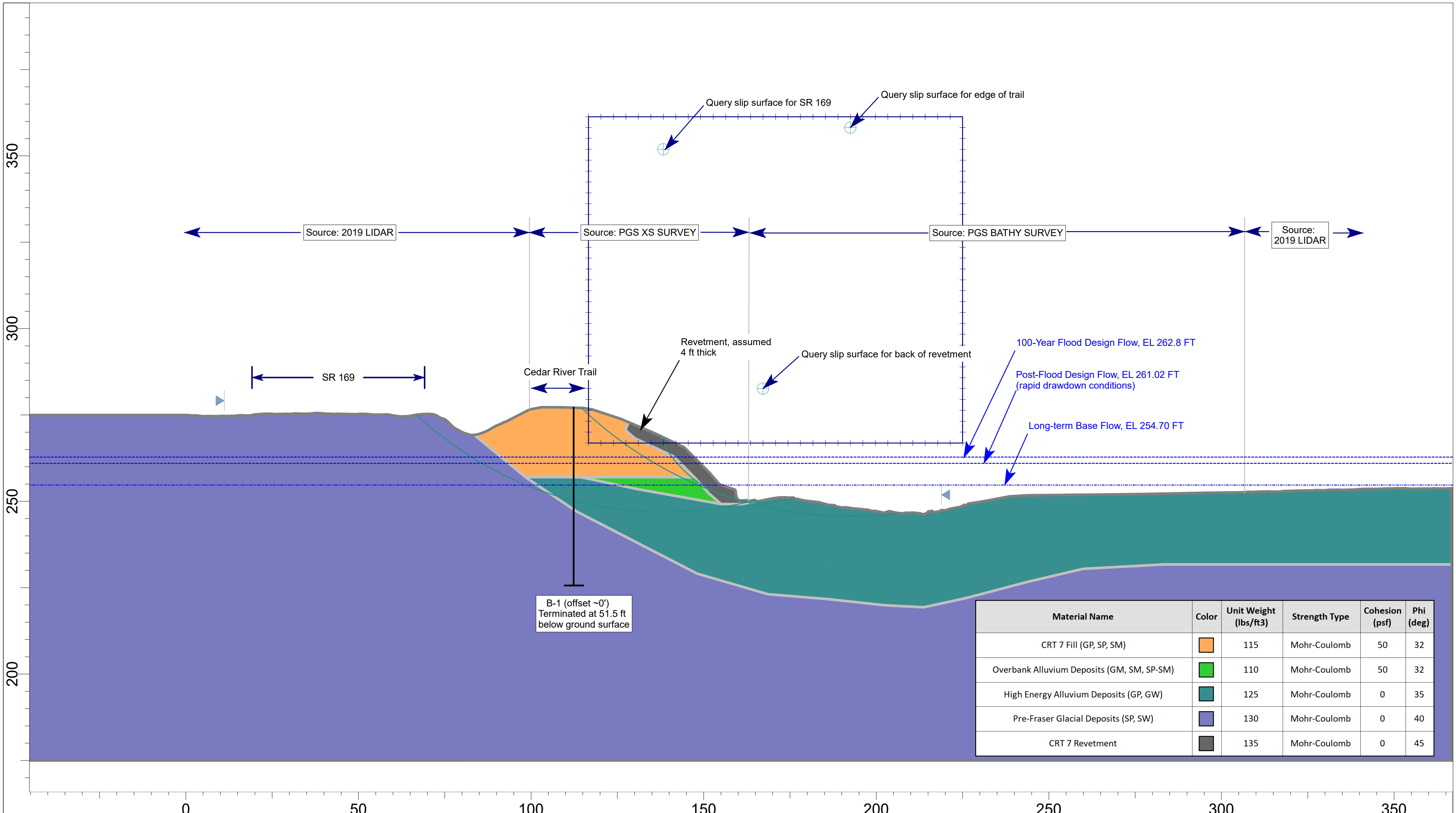
6/23/2020

BY: MO

PROJECT NO. 190175

REVIEWED BY: AJH

APPENDIX: **D-20**



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
CRT 7 Fill (GP, SP, SM)		115	Mohr-Coulomb	50	32
Overbank Alluvium Deposits (GM, SM, SP-SM)		110	Mohr-Coulomb	50	32
High Energy Alluvium Deposits (GP, GW)		125	Mohr-Coulomb	0	35
Pre-Fraser Glacial Deposits (SP, SW)		130	Mohr-Coulomb	0	40
CRT 7 Revetment		135	Mohr-Coulomb	0	45

Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

Cross Section B-B', Alternative 2, Non-Scoured Conditions Master Scenario

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDE 8.020

SCALE: 1:480

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Aspect CONSULTING

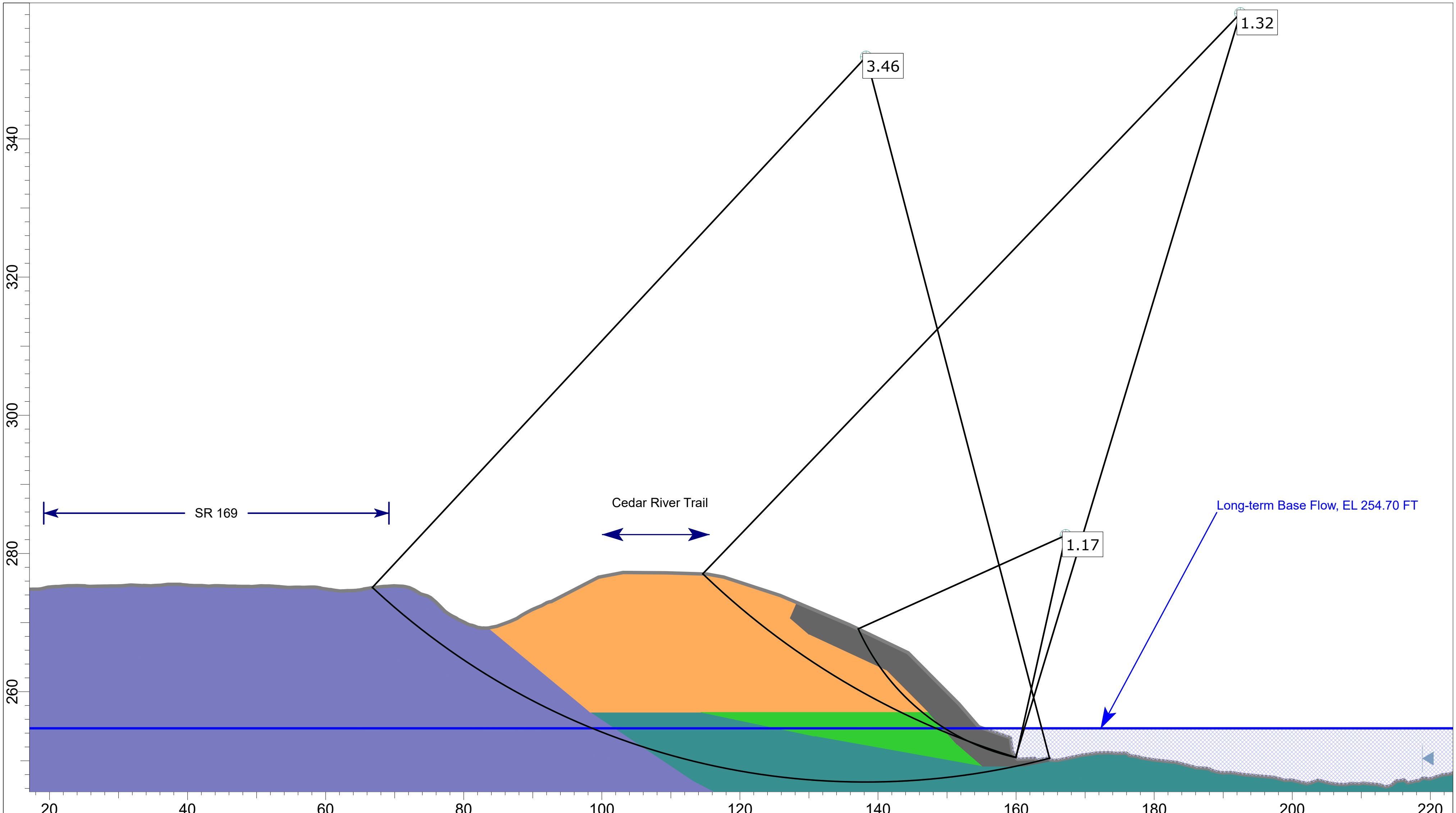
6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-21**



Legend

Query Slip Surface
Search Limits
Modeled Groundwater Level

Cross Section B-B', Alternative 2, Non-Scoured Conditions
Static Conditions

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020

SCALE: 1:240

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Aspect CONSULTING

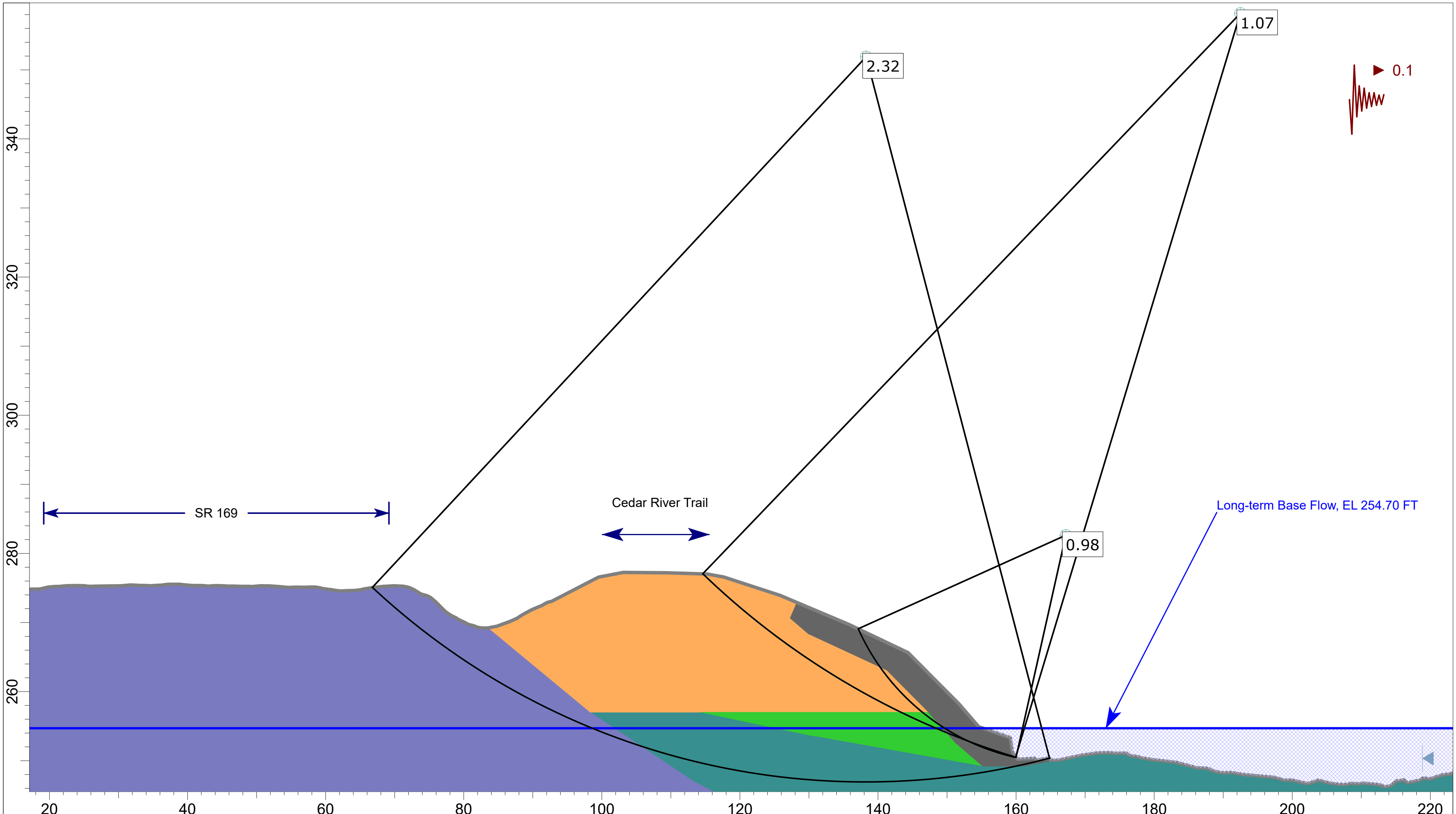
6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: D-22



Legend

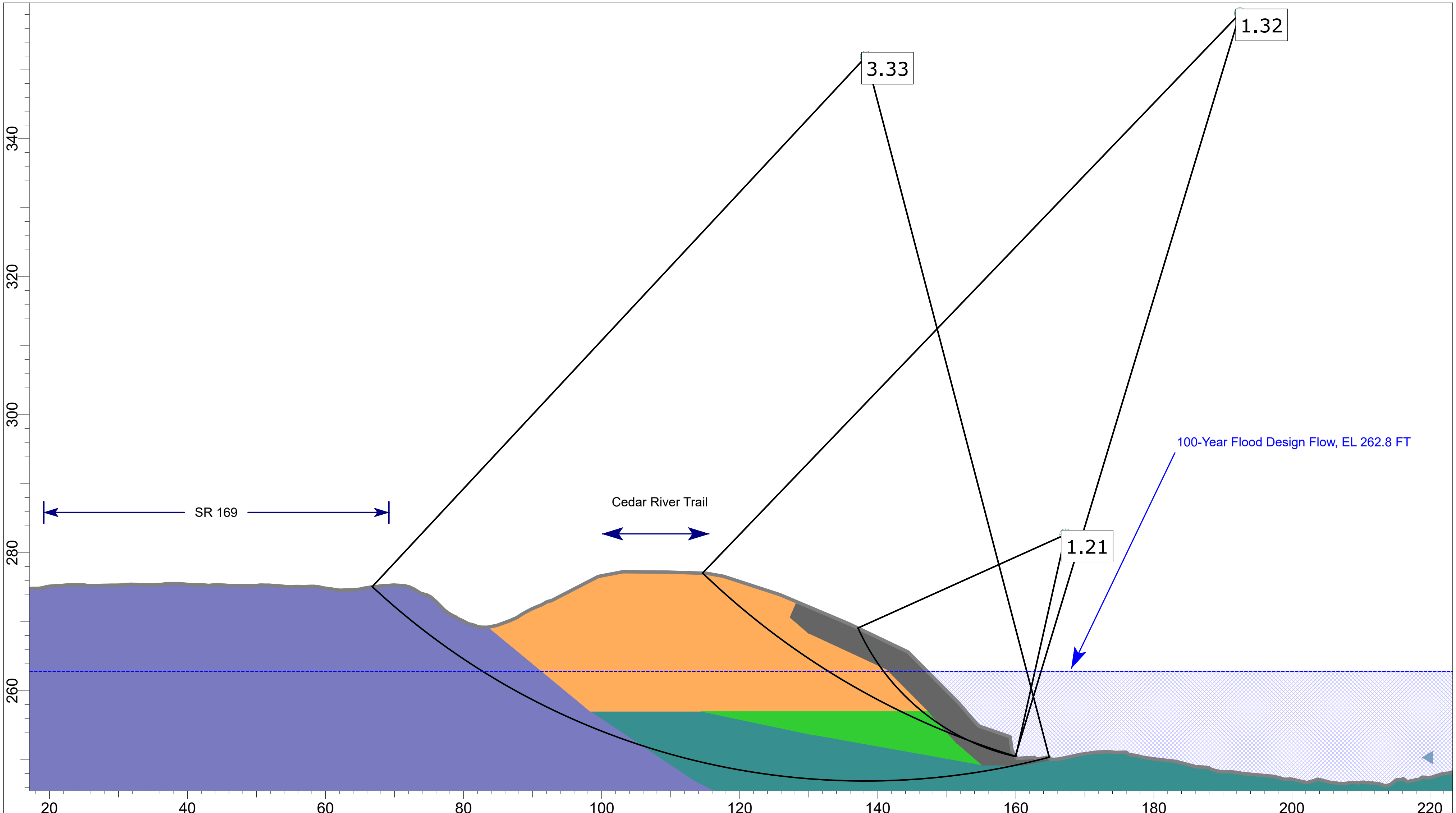
- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Alternative 2, Non-Scoured Conditions
Seismic Conditions

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020	SCALE: 1:240 S:\King County\Jan Road Neighborhood Levee\Data\Analyses\04_SSA CRT7\crt7_2020.04.22_alternative 2 existing conditions.slmd	Aspect CONSULTING	6/23/2020 PROJECT NO. 190175	BY: MO REVIEWED BY: AJH	APPENDIX: **D-23**



Legend

- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Alternative 2, Non-Scoured Conditions Peak Flood Level

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020

SCALE: 1:240

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Aspect CONSULTING

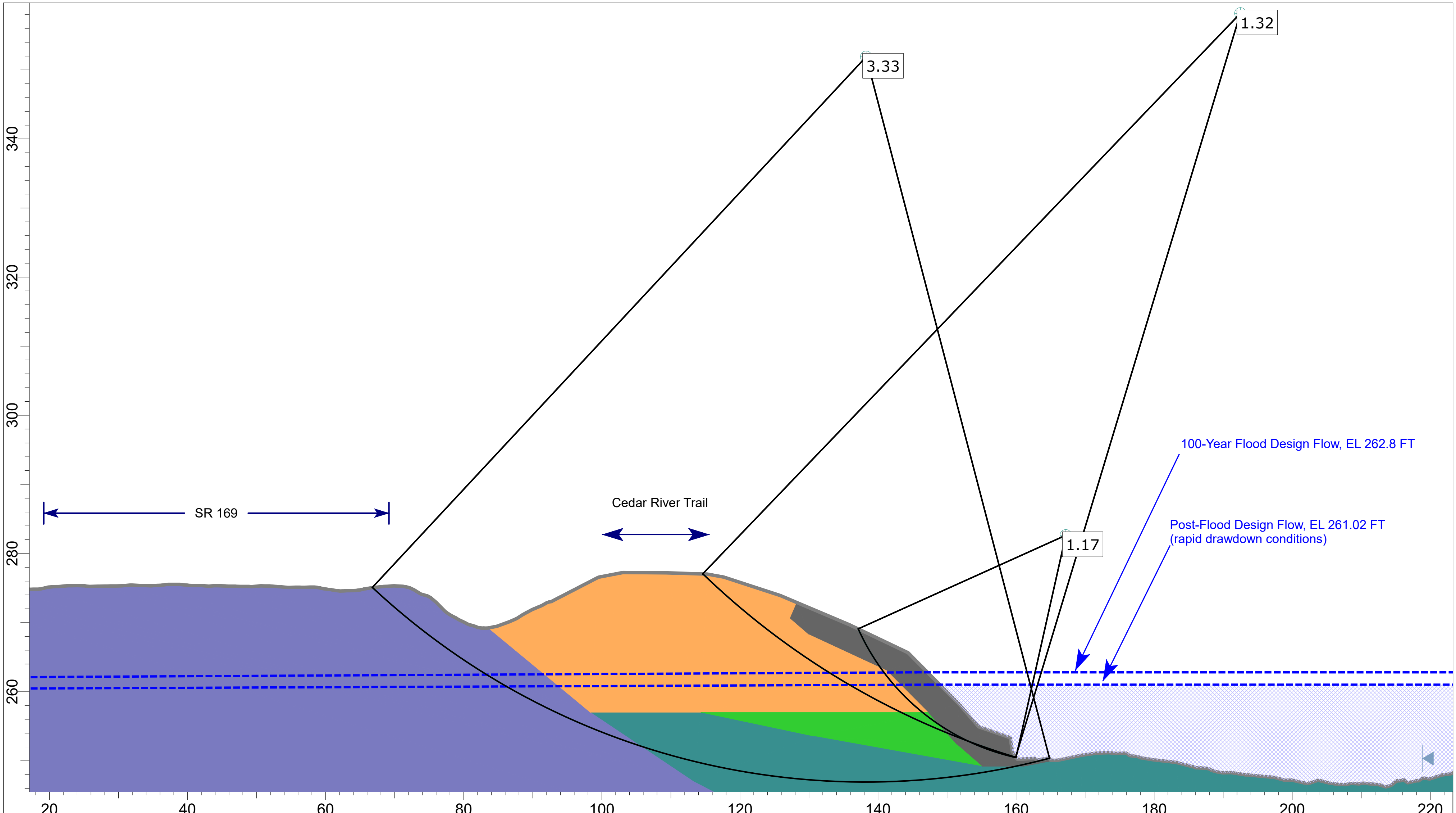
6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-24**



Legend

- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Alternative 2, Non-Scoured Conditions Rapid Drawdown

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020

SCALE: 1:240

S:\King County\Jan Road Neighborhood Levee\Data\Analyses\04_SSA CRT7\crt7_2020.04.22_alternative 2 existing conditions.slmd

Aspect CONSULTING

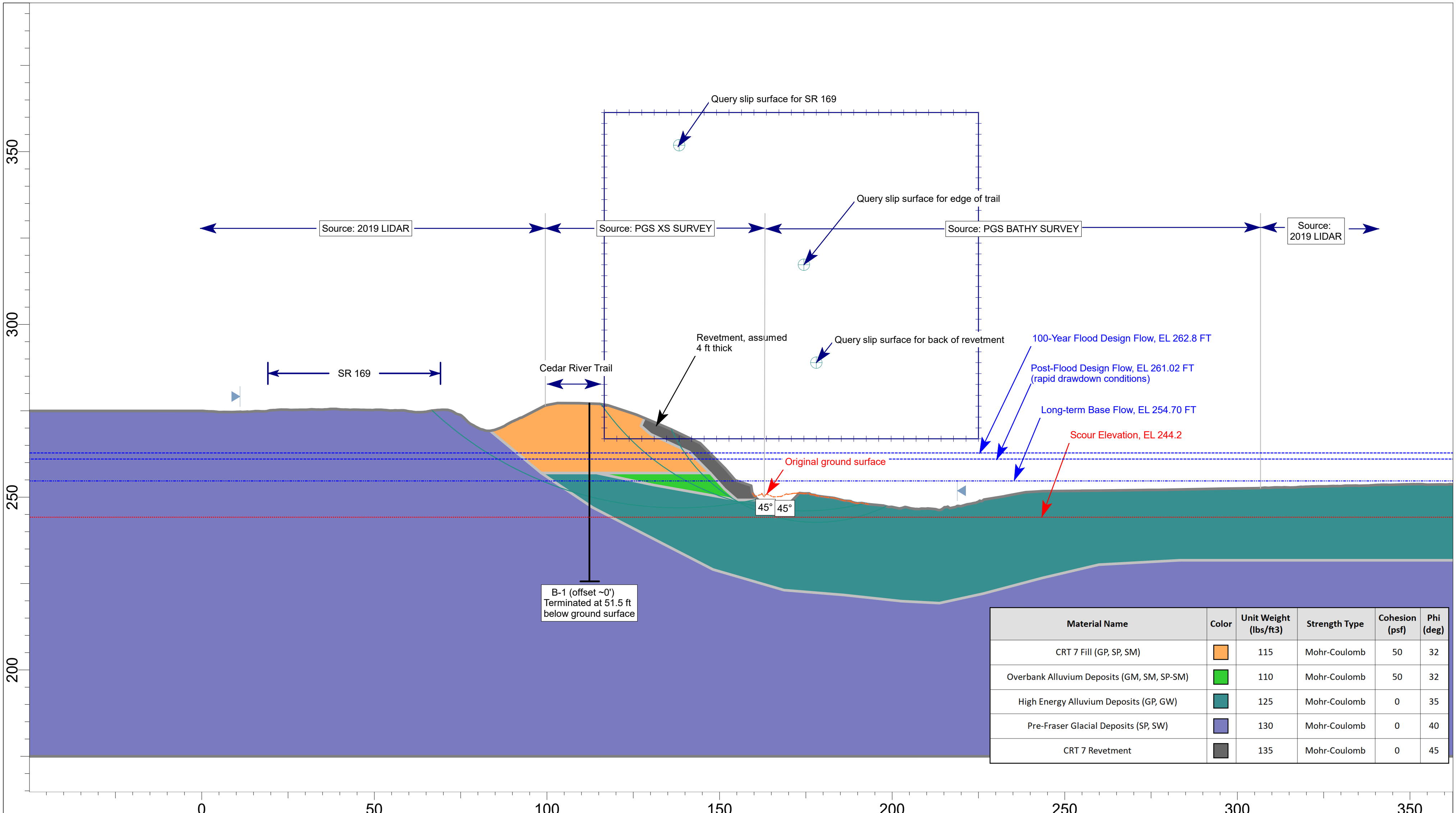
6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-25**



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
CRT 7 Fill (GP, SP, SM)		115	Mohr-Coulomb	50	32
Overbank Alluvium Deposits (GM, SM, SP-SM)		110	Mohr-Coulomb	50	32
High Energy Alluvium Deposits (GP, GW)		125	Mohr-Coulomb	0	35
Pre-Fraser Glacial Deposits (SP, SW)		130	Mohr-Coulomb	0	40
CRT 7 Revetment		135	Mohr-Coulomb	0	45

Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

Cross Section B-B', Alternative 2 With Potential Scour Master Scenario

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDE 8.020

SCALE: 1:480

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Aspect CONSULTING

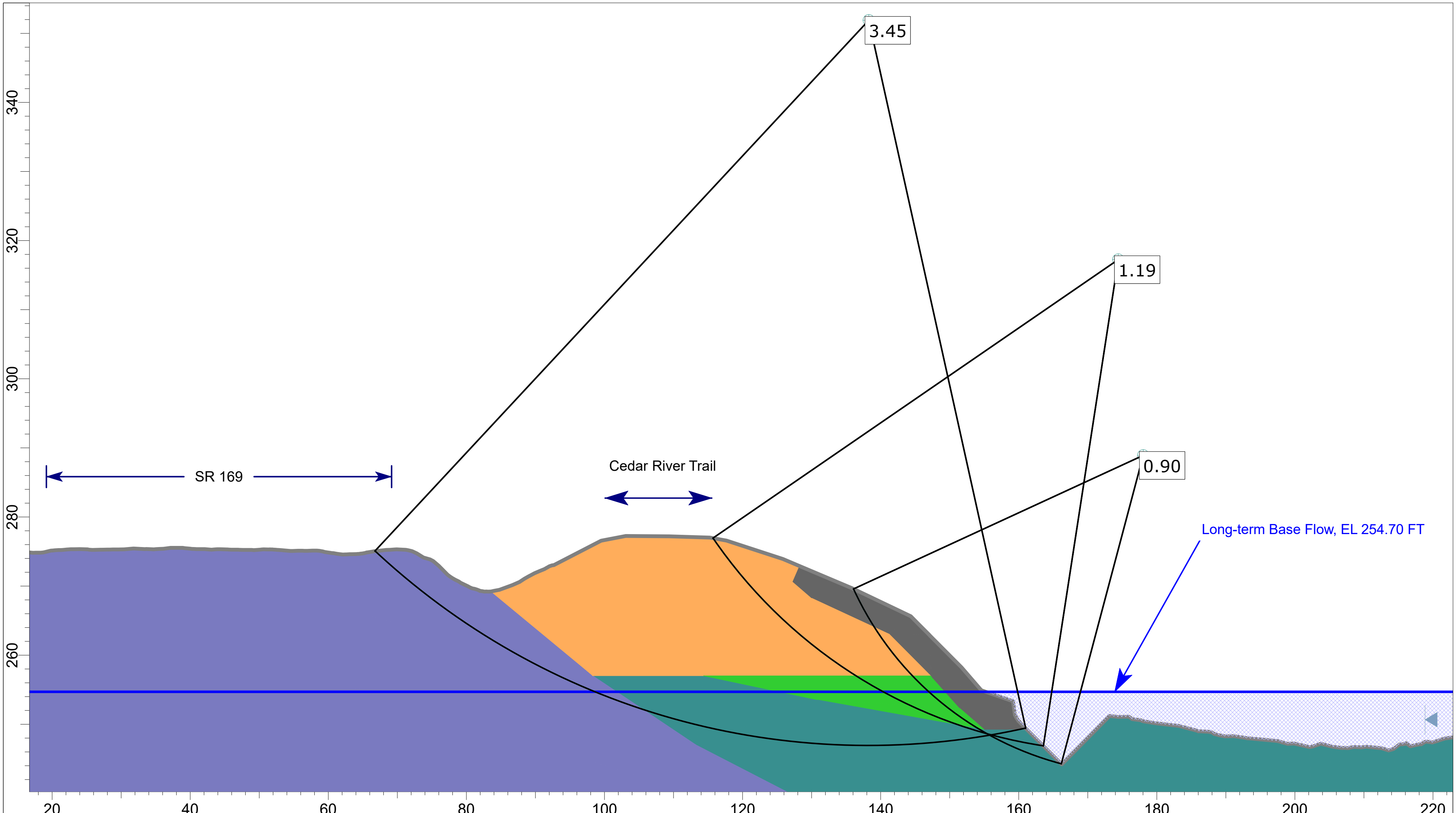
6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-26**



Legend

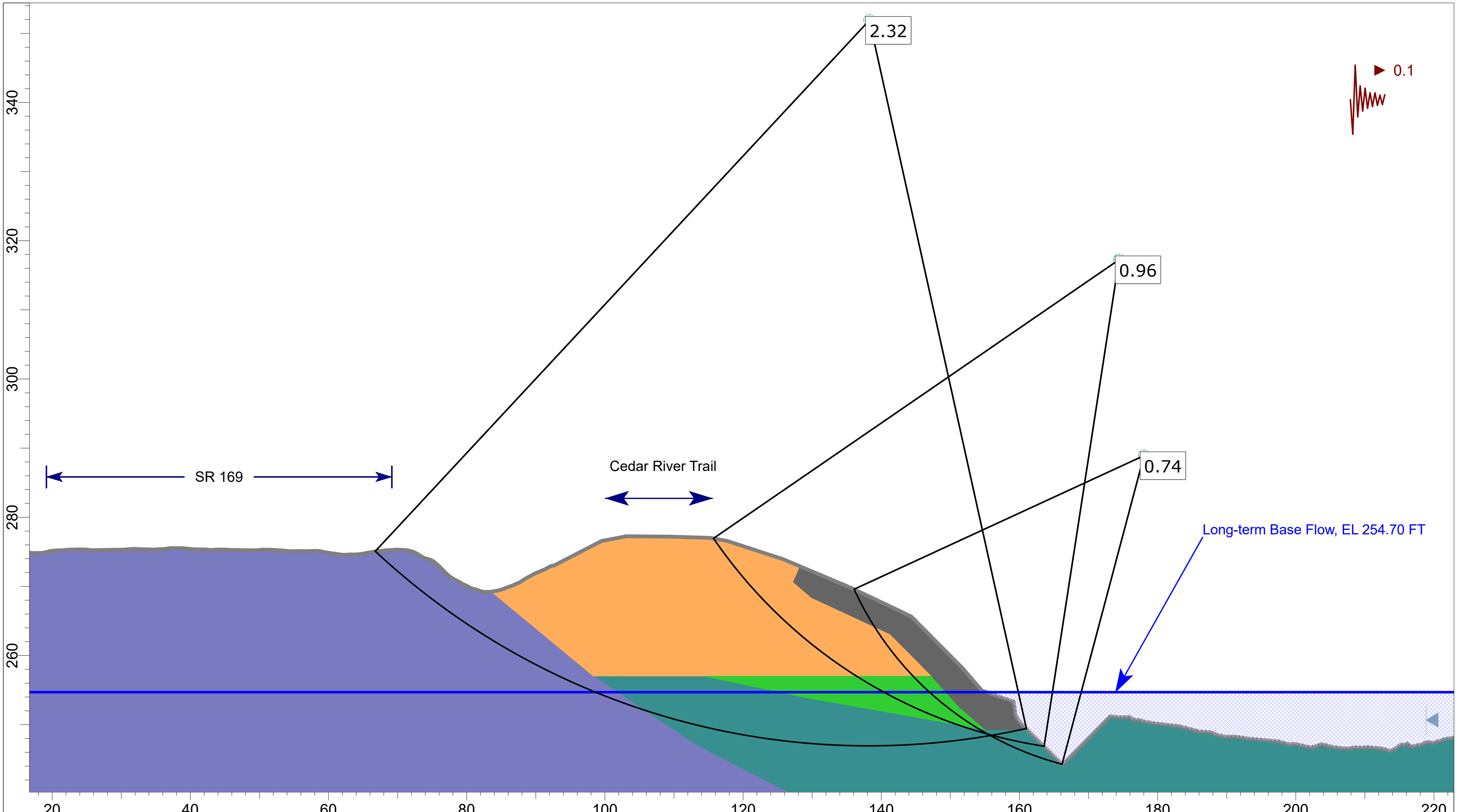
- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Alternative 2 With Potential Scour
Static Conditions

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020	SCALE: 1:240 S:\King County\Jan Road Neighborhood Levee\Data\Analyses\04_SSA CRT7\crt7_2020.04.22_alternative 2 with scour.slmd	Aspect CONSULTING	6/23/2020 PROJECT NO. 190175	BY: MO REVIEWED BY: AJH	APPENDIX: **D-27**



Legend

- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Alternative 2 With Potential Scour
Seismic Conditions

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020

SCALE: 1:240

S:\King County\Jan Road Neighborhood Levee\Data\Analyses\04_SSA CRT7\crt7_2020.04.22_alternative 2 with scour.slmd

Aspect CONSULTING

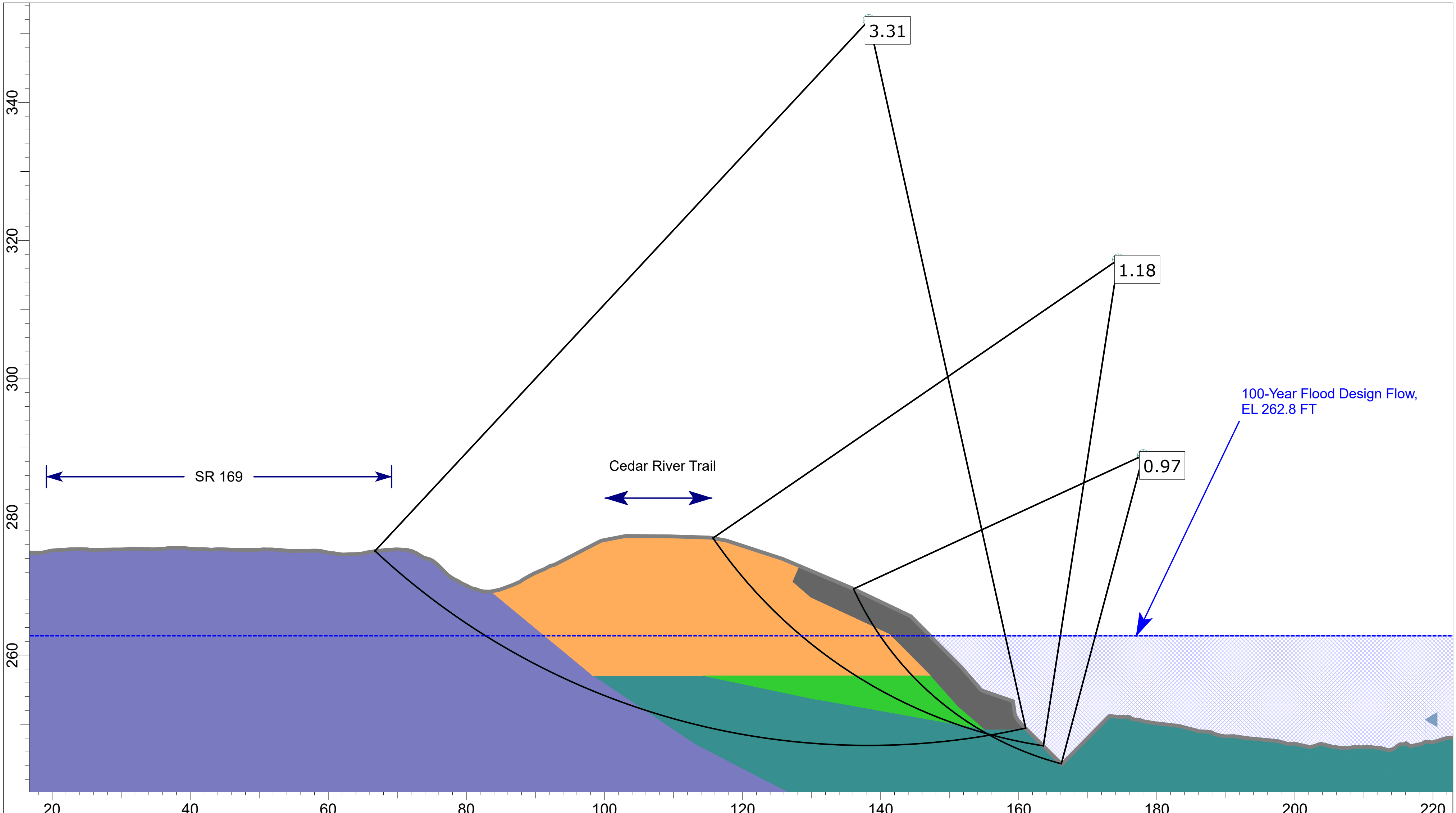
6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-28**



Legend

- Query Slip Surface
- Search Limits
- Modeled Groundwater Level

Cross Section B-B', Alternative 2 With Potential Scour
Peak Flood Level

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

SLIDEINTERPRET 8.020

SCALE: 1:240

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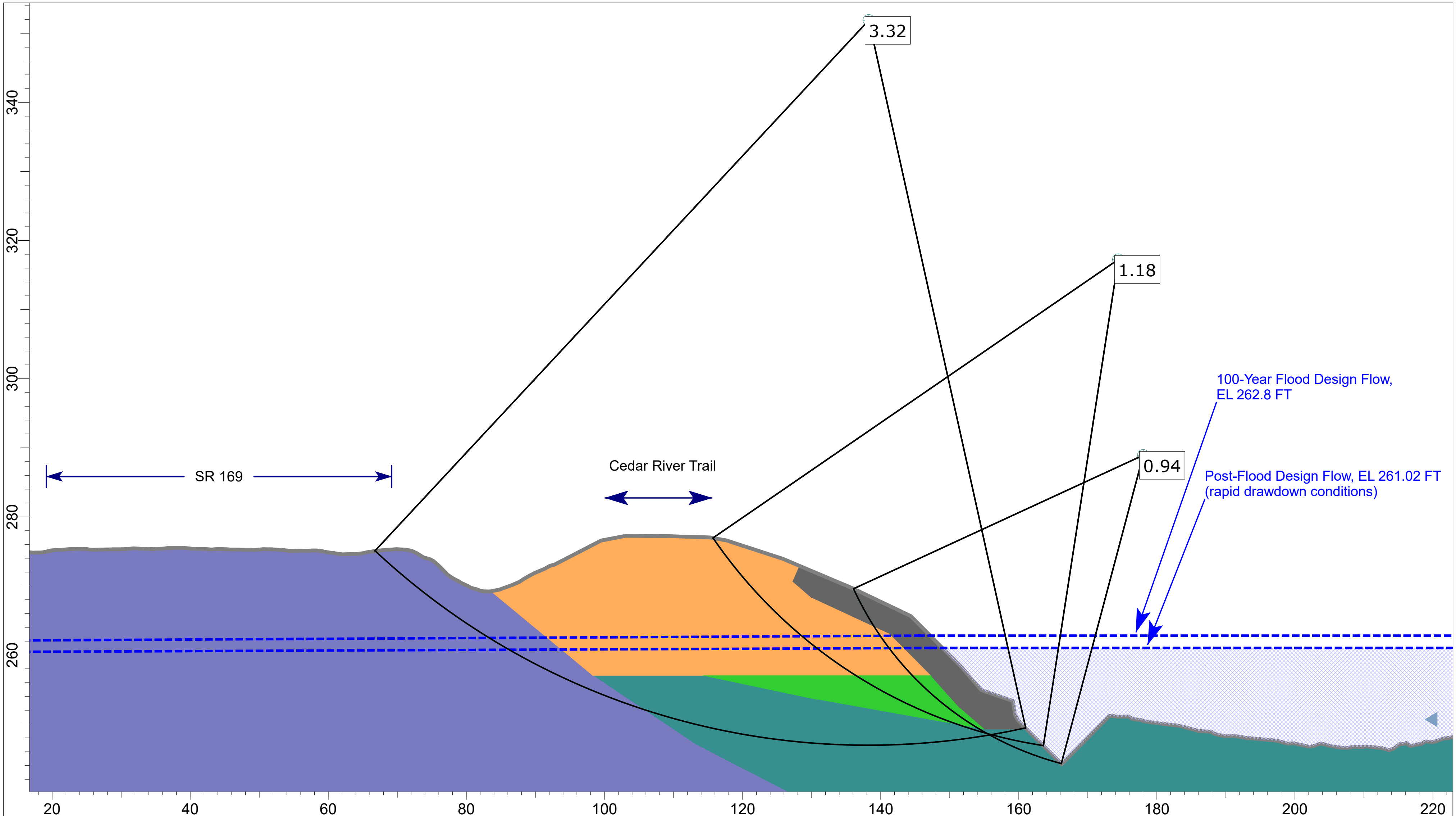
PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-29**



Legend

Query Slip Surface
Search Limits
Modeled Groundwater Level

Cross Section B-B', Alternative 2 With Potential Scour
Rapid Drawdown

CRT7 Slope Stability Analysis

Stability Evaluation and Risk Assessment
Jan Road Neighborhood Improvements
King County

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Aspect CONSULTING

6/23/2020

PROJECT NO. 190175

BY: MO

REVIEWED BY: AJH

APPENDIX: **D-30**

APPENDIX E

CRT7 Revetment Risk Assessment Workshop: Meeting Minutes

Aspect CONSULTING

Meeting Minutes

Project No.: 190175-600.2

June 4, 2020

Attendees:	Tetra Tech	Jerry Scheller
	Aspect Consulting	Andy Holmson Henry Haselton Mari Otto Jordan Sanford
	King County Department of Natural Resources	Dan Heckendorf Chris Brummer Judi Radloff
	Watershed GeoDynamics	Kathy Dube
Date/Time:	June 4, 2020, 2:30 – 4:30 PM	
Location:	Teams Meeting (video conference)	
Called By:	Jerry Scheller	
Re:	**CRT7 Revetment Risk Assessment Workshop: Meeting Minutes**	

The purpose of this meeting was to review the findings from the Cedar River Trail Site 7 (CRT7) Revetment (Revetment) stability analysis to inform on the general existing vulnerability of the Revetment. Additionally, potential benefits and impacts, resulting from implementation of Jan Road project alternatives and adjacent Rutledge – Johnson project (upstream) concept design. This meeting was recorded by Jerry Scheller.

Revetment Existing Conditions

Andy presented an understanding of the existing conditions of the Revetment, including an overview of cross-sections, geology, boring locations, boring logs, armoring, adjacent 1998 damage and repair documentation, and riprap movement. Kathy provided input describing the historical active channel margins and movement trends noting indicators of migration occurring at the upstream end of the Revetment and identifying the high value of the existing deep pool at the base of the Revetment as aquatic habitat.

Jerry provided an overview of the hydraulic modeling and related notes/trends at and near the Revetment that included modeled flood results with depth, velocity, and critical particle diameter maps.

Summary of Stability Analyses for Existing Conditions

Andy presented a review of relevant stability criteria (USACE Levee Design Manual; WSDOT Geotechnical Design Manual), and presented the Revetment slope stability results based on existing conditions with future scour amongst four scenarios focusing on two key slip surfaces: 1) back of revetment and 2) edge of the Cedar River Trail (trail).

Graphical analysis results were reviewed, showing cross sections with failure surfaces and results relative to factor-of-safety and comparison to the relevant stability criteria (USACE, WSDOT). A description of incorporation of scour into the modeling and soil mechanics analyses was presented.

A summary of the results showed that the Revetment is marginally stable (factors of safety greater than 1.0) under existing conditions and does meet WSDOT stability criteria under certain scenarios; however, the Revetment is generally unstable under scoured conditions (factor of safety less than 1.0) and does not meet relevant stability criteria. Factors of safety along the back of the armor rock are less than factors of safety for failure surface passing through the edge of the trail.

Review of Project Alternatives

A summary of the project alternatives was provided by Jerry. The alternatives include:

- **Alternative 1:** Elevating the majority of the private roadway (SE 197th PL/221st Ave SE/218th Ave SE) and installation of cross culverts, setting back the Jan Road Levee, and floodplain regrading with side channels.

- **Alternative 2:** Setting back the Jan Road Levee and extending it to the east (upstream), floodplain regrading with side channels, and upgrades to the existing culvert under SE 197th.

Given the similarity in the alternatives, and resulting hydraulic model output, it was determined that discussion of the results for one of the alternatives would be sufficient. Jerry and Kathy provided a review of modeling results for instream velocity, water levels, sediment transport, and scour based on Alternative 2, relative to existing conditions. They also discussed potential channel migration pathways resulting from Alternative 2, and modeled shear stress as it relates to observed deposition and scour patterns. Changes in regard to water levels and scour elevations at the Revetment in the context of the stability analyses were summarized and presented by Andy.

Review of Stability Analyses for Alternatives

While there are minor changes in the calculated Revetment factor of safety values, when compared to the USACE and WSDOT stability criteria, there is not significant change due to the project alternatives. The Revetment is tall, steep, and comprised of loose fill soils, laying the framework for marginal stability. The Revetment is not prone to instability under rapid drawdown conditions due to relatively free-draining soils and typical Cedar River flood hydrograph shape (not flashy). Revetment stability is not sensitive to minor changes in water levels and scour. Based on observations made by Dan and Chris during recent flood conditions, modeled conditions appear consistent with observed response of revetment and flood hydrograph record.

Review of Adjacent Project (Rutledge-Johnson)

Kathy and Jerry presented modeling results of potential impacts from the adjacent and upstream Rutledge-Johnson levee removal project, based on a single preliminary design concept that would remove a downstream portion of the levee and activate an existing backwater channel. The results showed little difference at the Revetment relative to changes in river conditions considering the Jan Road project alternatives only.

Risk Discussion

The Revetment is not stable when compared to commonly accepted design criteria (USACE and WSDOT), but it is calculated to have a factor-of-safety greater than 1.0 based on the engineering properties assumed for the analysis. While mathematically computed as stable, performance of the Revetment under the conditions analyzed is less certain than it would be if the commonly accepted design criteria were satisfied, due to the inherent uncertainty of the engineering properties of geologic materials. Failure of the armor rock is considered the greatest risk when scour occurs; however, failures extending back into the trail are considered to carry a lower risk.

The analyses described above do not account for probable channel movement, as a result of project implementation. A comprehensive evaluation of risk would consider likely channel migration over time, and the implications of those temporal considerations on the overall vulnerability of the Revetment. Modeled conditions for Alternative 2 suggest less scour and erosive forces (velocity) along the CRT7 revetment relative to existing conditions. Scour depths along the facility are likely to lessen further over the medium-term (10 years), further reducing potential revetment instabilities related to scour. Further reduction in velocities are also anticipated to occur over this time frame, resulting in a greater likelihood for depositional zones near the facility, likely resulting in further limitation of scour at this location. Assuming Alternative 2 conditions, substantive channel migration is not expected to occur in the short-term. Long-term (50 years), the river may try to migrate and erode upstream of the Revetment pool; therefore, the project may consider installing flow deflection elements, upstream of this location, to direct flows away from the upstream end of the facility. Given the long-term nature of this projection, monitoring of this potential channel evolution scenario was recommended as a minimum action.

Conceptual Stabilization Alternatives

Andy provided a discussion of potential conceptual approaches to improving the stability of the Revetment that included reactive or proactive approaches. A reactive approach would be to repair the Revetment if it fails, in a manner similar to the adjacent 1998 repair. Proactive approaches to stabilize the Revetment to an acceptable level of stability when compared to the USACE and WSDOT design criteria could include:

- Proactive repair of the Revetment in a manner similar to the adjacent 1998 repair.
    - Could offer improved in-water habitat (wood debris incorporated into repair).
    - Clearing of existing vegetation necessary for the repair would eliminate shade and cooling from the established large trees growing on the Revetment.
- Buttressing along the toe of the Revetment and scour hole.
    - Logistics of in-water construction and impacts of buttress on stream channel identified as potential drawbacks.

- Regrade the Revetment to a flatter angle (2H:1V)
  - More favorable for long-term stability.
  - Would require moving trail and possibly impacting highway right-of-way.
  - Clearing of vegetation would be a temporary impact to shade and cooling.

In considering these conceptual stabilization approaches, the County suggested that the co-managers would likely prioritize protection of habitat and resist impacts to habitat, suggesting that the reactive approach may be favored. The County will discuss this internally.

Mitigation of Revetment vulnerabilities through project design by controlling flows, velocities, and channel migration area away from the Revetment was discussed. The team agreed that the project could include design elements and provisions to influence the geomorphology and hydraulics in a way that would focus less severe forces against the Revetment.

Next Steps

- County to follow-up internally following this presentation to discuss the presented content, any additional questions/comments, and expectations for the risk assessment reporting. Dan to provide feedback to the consultant team.

- A design study for the CRT2 revetment (downstream) was completed in 2019 and will be provided by the County. Aspect will review the study for the purpose of applying consistent methodologies for similar structures/evaluations within the Cedar River.

- Project team will revise presentation materials and redistribute to County and provide these meeting notes.

- Aspect to proceed with preparing draft CRT7 Revetment Risk Assessment Memorandum under Task 600.2.

V:\190175 Jan Road Neighborhood Improvements Design\Deliverables\CRT7 Revetment Risk Assess\Final\CRT7 Meeting Minutes_06042020_final.docx

APPENDIX F

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.

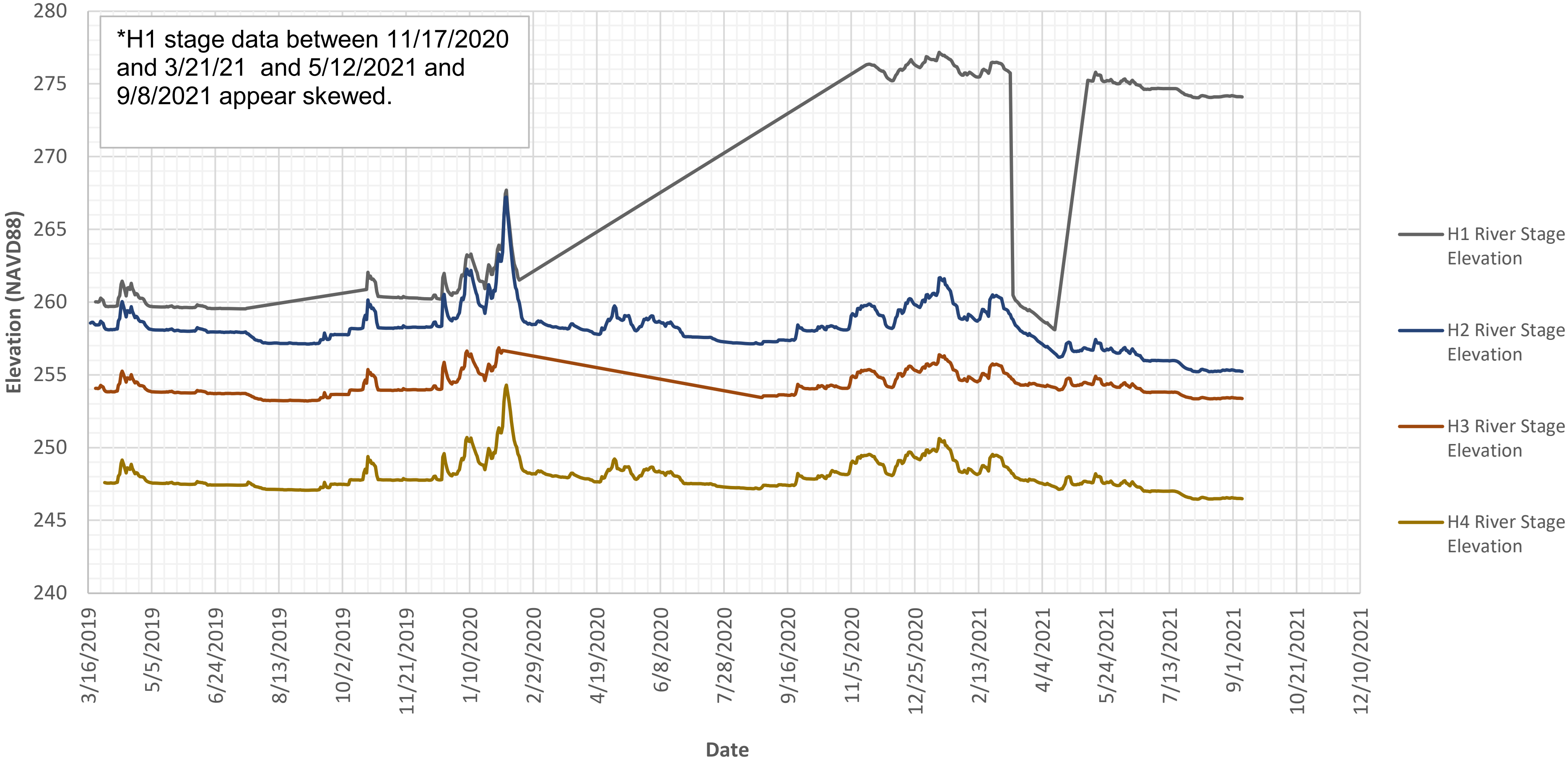
Geotechnical, Geologic, and Environmental Reports Are Not Interchangeable

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.

APPENDIX D

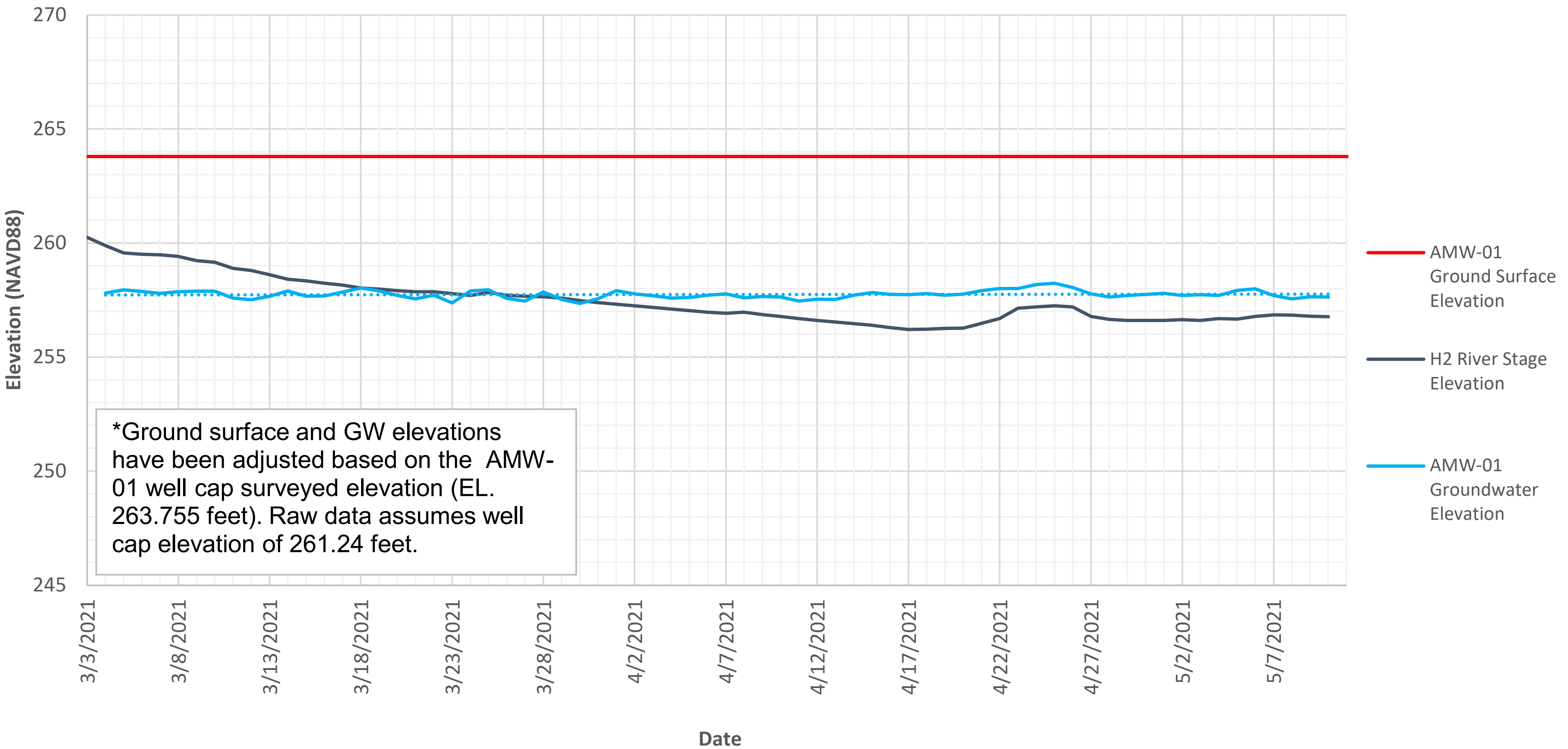
Groundwater Monitoring



Notes:

1. Groundwater and river stage data instrumented and monitored by King County. Data downloaded from King County Hydrologic Information Center: https://green2.kingcounty.gov/hydrology/GaugeMap.aspx
2. Elevation of existing ground surface and correlation to water elevation approximated from contours shown on Figure 2 Site and Exploration Map

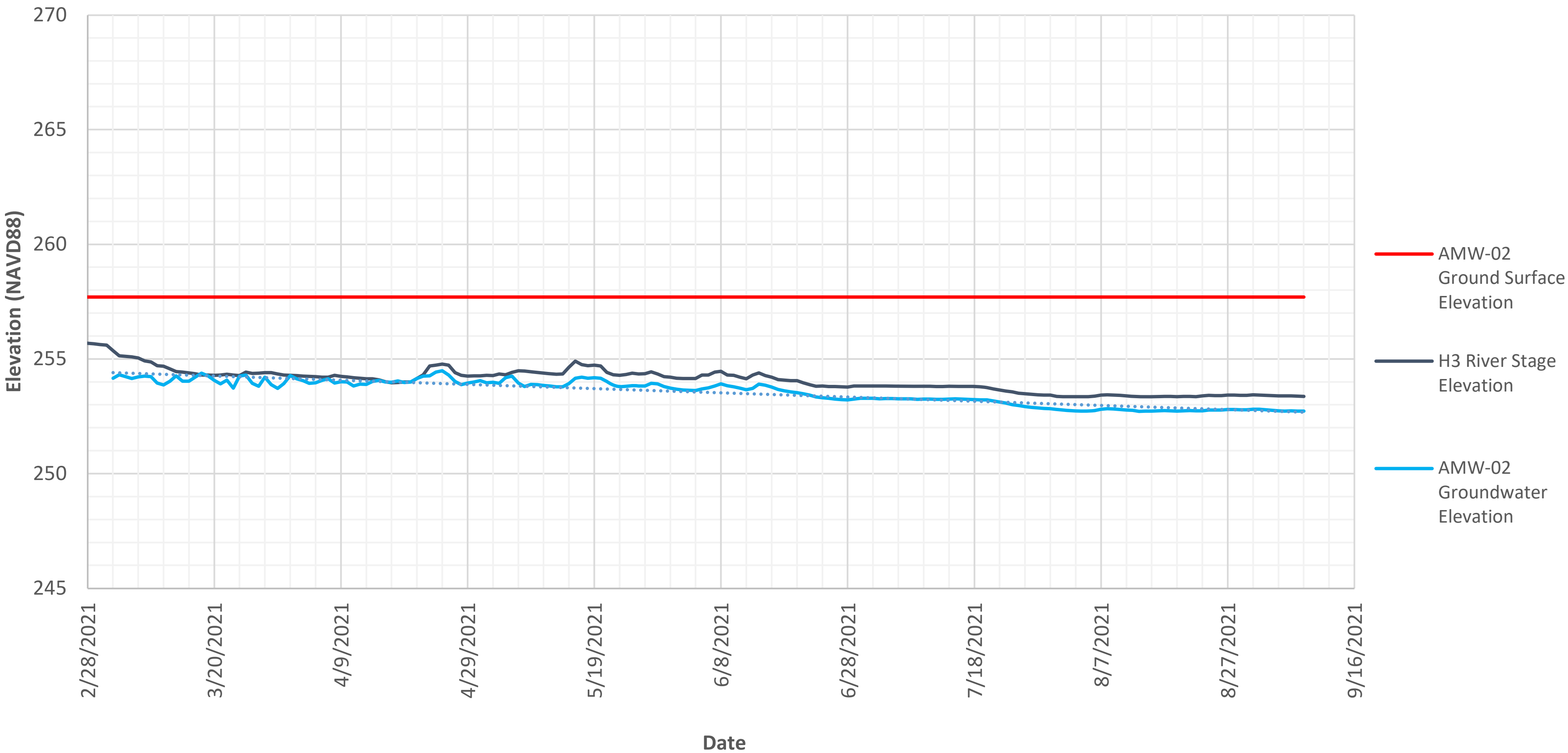
River Stage Monitoring Data Jan Road Neighborhood Improvements King County, Washington		
Project No.: 190175	**Analysis By / Reviewed By:** MO / AJH	**Updated:** 2/3/2022
Aspect CONSULTING		**Figure** **D-1**



Notes:

1. Groundwater and river stage data instrumented and monitored by King County. Data downloaded from King County Hydrologic Information Center: https://green2.kingcounty.gov/hydrology/GaugeMap.aspx
2. Elevation of existing ground surface and correlation to water elevation approximated from contours shown on Figure 2 Site and Exploration Map

AMW-01 Groundwater/River Stage Monitoring Jan Road Neighborhood Improvements King County, Washington		
Project No.: 190175	**Analysis By / Reviewed By:** MO / AJH	**Updated:** 2/3/2022
Aspect CONSULTING		**Figure D-2**



Notes:

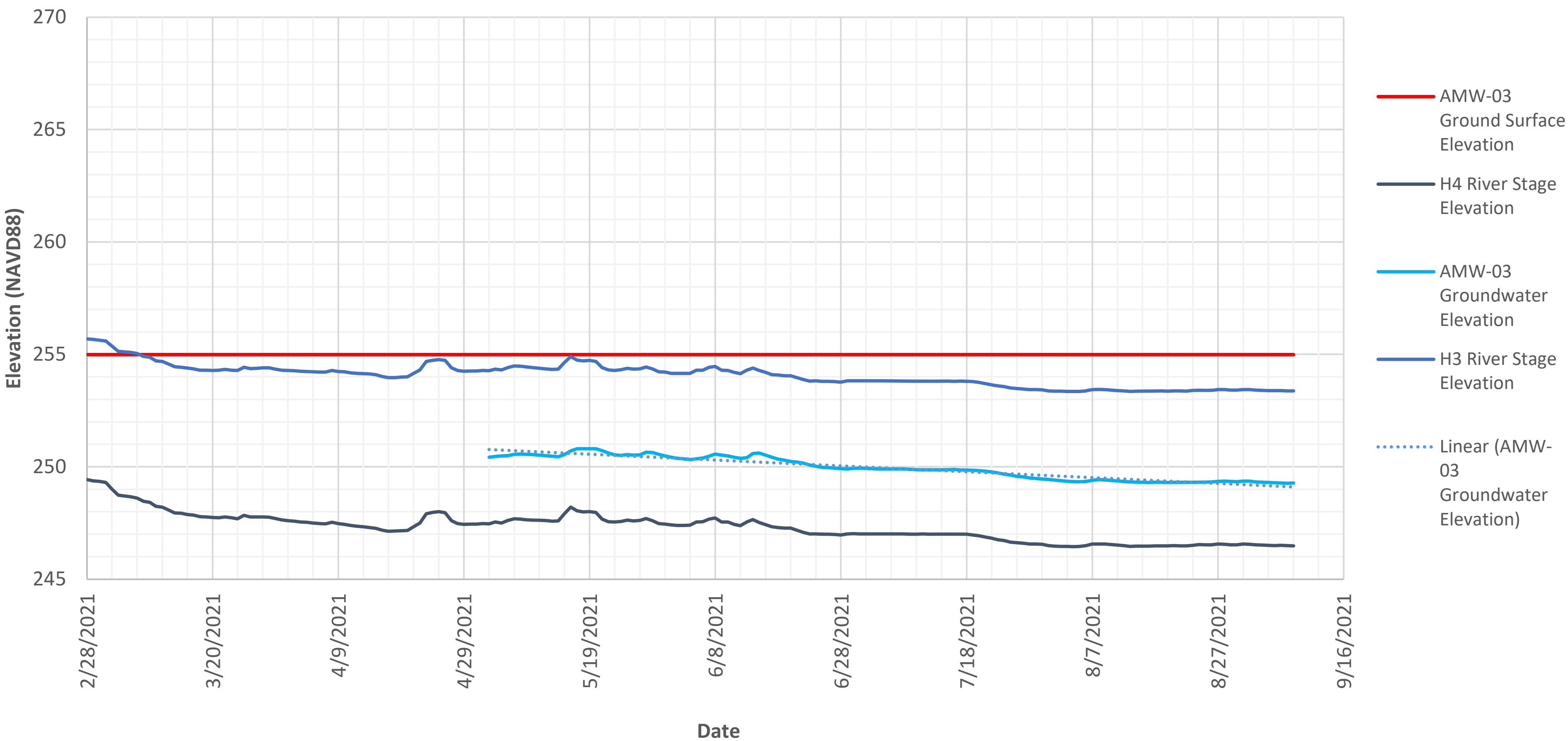
1. Groundwater and river stage data instrumented and monitored by King County. Data downloaded from King County Hydrologic Information Center: https://green2.kingcounty.gov/hydrology/GaugeMap.aspx
2. Elevation of existing ground surface and correlation to water elevation approximated from contours shown on Figure 2 Site and Exploration Map

AMW-02 Groundwater/River Stage Monitoring
Jan Road Neighborhood Improvements
King County, Washington

Project No.:	Analysis By / Reviewed By:	Updated:
190175	MO / AJH	2/3/2022

Aspect CONSULTING

Figure D-3



Notes:

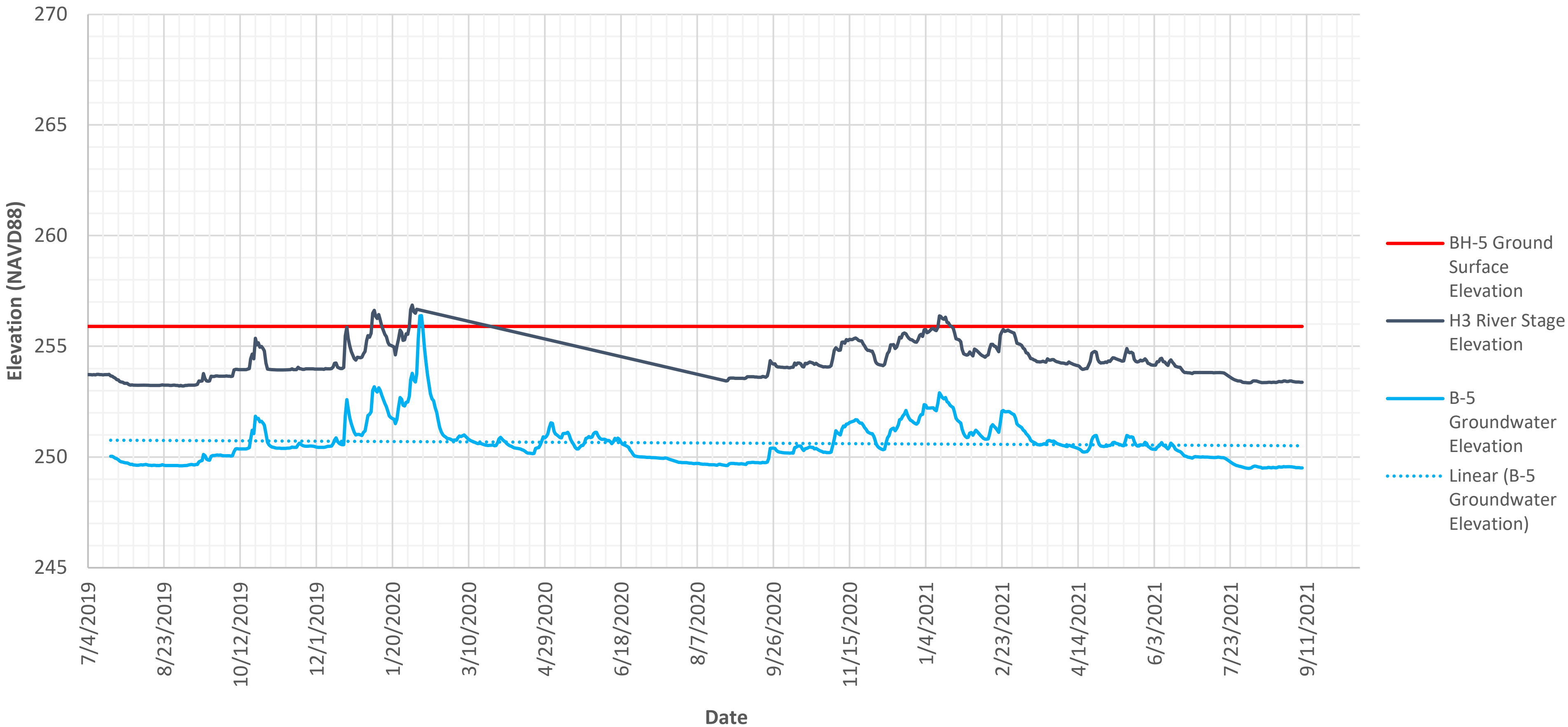
1. Groundwater and river stage data instrumented and monitored by King County. Data downloaded from King County Hydrologic Information Center: https://green2.kingcounty.gov/hydrology/GaugeMap.aspx
2. Elevation of existing ground surface and correlation to water elevation approximated from contours shown on Figure 2 Site and Exploration Map

AMW-03 Groundwater/River Stage Monitoring
Jan Road Neighborhood Improvements
King County, Washington

Project No.:	Analysis By / Reviewed By:	Updated:
190175	MO / AJH	2/3/2022

Aspect CONSULTING

Figure D-4



Notes:

1. Groundwater and river stage data instrumented and monitored by King County. Data downloaded from King County Hydrologic Information Center: https://green2.kingcounty.gov/hydrology/GaugeMap.aspx
2. Elevation of existing ground surface and correlation to water elevation approximated from contours shown on Figure 2 Site and Exploration Map

BH-5 Groundwater/River Stage Monitoring
Jan Road Neighborhood Improvements
King County, Washington

Project No.:	Analysis By / Reviewed Bv:	Updated:
190175	MO / AJH	2/3/2022

Aspect CONSULTING

Figure
D-5

APPENDIX E

Settlement Analyses

E. Settlement Analyses

E.1. Methodology

We evaluated settlement using the software program Settle3D (Rocscience, 2019), a 3-dimensional finite element analysis program that calculates settlement, stress, and pore pressures through a 3-dimensional soil volume.

Key inputs into Settle3D are soil engineering parameters such as elastic modulus, time-dependent consolidation parameters and groundwater conditions, and loading conditions. Through finite element analysis within the subsurface, Settle3D computes three-dimensional stress changes and one-dimensional (vertical) settlement and pore-pressures.

The key outputs from the settlement analyses are elastic settlement, primary consolidation, and secondary compression at user-defined time intervals.

E.2. Design Assumptions and Criteria

There are no design criteria specific to allowable settlement thresholds, as defined by the Project. We conducted settlement analyses to determine the magnitude and rate of anticipated settlement along the setback levee alignment, and inform the need for settlement mitigation measures.

E.3. Design Conditions for Analysis

We performed settlement analyses at five critical sections along the setback levee alignment. We considered conservative groundwater conditions based on the seasonal high groundwater levels informed by continuous groundwater monitoring at the Site. We created the subsurface profiles used in the models based on reference explorations along the setback levee alignment, identified in Table E-1 below.

Typical levee section geometry inputs into the Settle3D models are presented in Section 4.2 of the report text. We conservatively modeled the embankment using an assumed unit weight of 140 pounds per cubic foot (pcf) to represent a combination of the proposed levee fill and rip rap. The embankment load was applied in up to six stages of equal thickness (±0.3 feet) to represent construction over a period of one to three months. We modelled the elastic soil modulus of the subsurface soil units based on correlations with standard penetration test (SPT) N-values collected during the subsurface explorations (Bowles, 1996). Since fine-grained soils were not encountered during the subsurface exploration program, consolidation parameters were not assigned to the on-Site soils. The soil parameters for settlement analyses are presented in Table 1.

The critical section locations, reference explorations, and water conditions used in our analyses are presented in Table E-1 below.

Table E-1. Seepage Analysis Section Locations and Details

Section	Stationing(1)	Reference Exploration	Planned Levee Height (ft)(2)	Depth to Groundwater, (ft)(3)
A-A'	3+50	ATP-06 AMW-03	5.8	1.0
B-B'	12+37.30	ATP-04 AB-03	3.9	3.0
C-C'	16+25	ATP-02 AB-01	6.3	4.5
D-D'	19+00	B-6	5.7	4.0
E-E'	24+00	B-4 AMW-01	4.3	5.0

Notes:
1) Stationing per draft 100 percent Project drawings and as shown on Figure 2.
2) Levee height is measured at the top of the levee (at centerline) to the base of levee core fill.
3) Depth to groundwater based on seasonal high groundwater surface elevation. Based on continuous groundwater monitoring at the Site.

E.4. Soil Parameters for Settlement Analyses

The soil stratigraphy used for each design section was based on our interpretation of the subsurface conditions as shown on the geologic profile (Figures 3-4) with corresponding engineering properties shown in Table 1. We determined soil settlement parameters such as the elastic modulus for each geologic deposit encountered at the Site based on observed soil conditions, correlations from *in situ* standard penetration tests, correlations with particle size test results, and our experience with local geology. No cohesionless soils were observed in our subsurface investigation program, therefore consolidation and secondary compression parameters were not assigned to any of the soil units. Our selection of design settlement values was supported by sensitivity analyses, described in Section E-6.

E.5. Settlement Analysis Results

We anticipate all settlement will occur elastically as the setback levee is constructed. We estimate elastic settlement on the order of 0.5 inches or less.

Since no cohesive soils were observed at the site, we do not anticipate long-term settlement and do not recommend any settlement mitigation (overbuild) measures.

E.6. Settlement Sensitivity Analysis

We conducted sensitivity analyses of the settlement models to account for the inherent variability of natural soil deposits. We performed our sensitivity analyses at the critical section for settlement: Section C-C', where planned levee heights are relatively tall and all major geologic soil units are anticipated in the subsurface.

APPENDIX F

Seepage Analyses

F. Seepage Analyses

F.1 Methodology

To evaluate seepage through the proposed setback levee embankment, we performed steady-state seepage analyses using the finite element analysis groundwater module within the computer program Slide (Rocscience, 2017). The Slide program groundwater module is a two-dimensional, finite-element program that simulates fluid flow and estimates pore-water pressure distribution in saturated and unsaturated porous material.

Key inputs into the Slide groundwater module are levee embankment geometry, soil hydraulic properties (horizontal hydraulic conductivity, k_h, and ratio of vertical to horizontal hydraulic conductivity (k_v:k_h) and known boundary conditions (in this case, steady-state flood total head conditions). Through iterative calculations of successive finite element runs, the groundwater analysis module computes the pressure head throughout the model and determines the piezometric surface, flow directions, vertical hydraulic gradients, and seepage discharge potential.

Key outputs from the Slide groundwater module are the vertical hydraulic gradients at/near the toe of the levee and discharge through the levee toe per 100 linear feet of embankment.

F.2 Design Assumptions and Criteria

We performed seepage analyses in accordance with the United States Army Corps of Engineers (USACE) Engineering Technical Letter (ETL) 1110-2-569 *Design Guidance for Levee Underseepage* (USACE, 2005). These guidelines recommend a maximum vertical hydraulic gradient at/near the landside toe of the levee embankment of 0.5 (dimensionless). In the case of nearby drainage features such as a landside drainage ditch, the vertical hydraulic gradients should not exceed 0.8 (USACE, 2005). In addition, we've assumed that excessive levee discharge for the Project can be defined as more than 200 gallons per minute (gpm) over 100 linear feet of embankment.

F.3 Design Conditions for Analysis

We performed steady-state seepage analyses to evaluate groundwater flow through and beneath the proposed levee at five critical sections along the proposed levee alignment. We considered the water level representing the 100-year flood water surface elevation (WSE) provided by TetraTech (TetraTech, 2021). We assessed seepage conditions in long-term (scoured) conditions to represent conservative seepage conditions. We created the subsurface profiles used in the models based on reference explorations performed during our subsurface explorations, identified in Table F-1 below.

The critical section locations, reference explorations, and water and scour conditions used in our analyses are presented in Table F-1 below. Typical levee section geometry in long-term scoured conditions is presented in Sections 4.2 and 5.1.1 of the report text.

Table F-1. Seepage Analysis Section Locations and Details

Section	Stationing(1)	Riverward 100-yr(2) WSE(3)	Landward 100-yr(6) WSE	Scour Elevation(7)
A-A’	3+50	257.2(4)	255.5	244.2
B-B’	12+37.30	259.4	-	245.3
C-C’	16+25	263.4	-	246.3
D-D’	19+00	264.0	-	240.9
E-E’	24+00	267.6	-	245.4

Notes:
1) Stationing per draft 100 percent Project drawings and as shown on Figure 2.
2) 1% annual probability of exceedance flood.
3) WSE = water surface elevation, yr = year.
4) WSE information provided by Tetra Tech (Tetra Tech, 2021).
5) Minor flooding of landward side of levee predicted by hydraulic modeling.
6) Long-term elevations at riverside levee toe due to scour or projected future channel migration. Scour elevations provided by Tetra Tech (Tetra Tech, 2021).

F.4 Soil Parameters for Seepage Analyses

The soil stratigraphy used for each design section was based on our interpretation of the subsurface conditions as shown on the geologic profile (Figures 3-4) with corresponding engineering properties shown in Table 1. Discussion of the hydraulic property selection process for native soils and fill is included below. Our selection of design hydraulic conductivity values was supported by sensitivity analyses, described in Section F.6.

Native soils

We determined hydraulic properties for the native Alluvial Deposits – Sand Facies, Alluvial Deposits – Gravel Facies, and Pre-Fraser Deposits encountered at the Site based on particle size correlations (Massman, 2003) and slug testing described in Section 3.6. We assumed that these naturally deposited soils have a ratio of vertical to horizontal hydraulic conductivity (kv:kh) between 0.05 and 0.2. We used a design kv:kh value of 0.1 for these geologic deposits.

Fill

The levee prism will consist of Select Fill Type 1 which we anticipate will be at least partially sourced from re-used on-Site Sand and Gravel Facies, imported, relatively impermeable core fill (Levee Core Fill), and launchable scour protection (Riprap).

Additionally, between approximately Stations 12+00 and 13+00 (represented by Section B-B'), the levee will be built over an existing pond that will be filled with Ballast Fill, sourced from on-Site Gravel Facies or similar imported soils.

- We based hydraulic conductivity of the Select Fill Type 1 and Ballast Fill material on particle size correlations from on-Site Alluvial Deposits – Sand Facies, and Alluvial Deposits – Gravel Facies, respectively.
- The allowable range of hydraulic conductivity values for the Levee Core Fill was informed by our sensitivity analysis described in Section F.6. The design hydraulic conductivity of the Levee Core Fill was taken as the average of the allowable range.
- We applied relatively high-permeability hydraulic conductivity values to the Riprap modeled in the typical levee sections.
- We assume that all fills will be placed as relatively homogenous units. Therefore, we applied a design k_v:k_h value of 1 to these soils.

We performed sensitivity analyses to support the selection of the hydraulic conductivity properties, as described in Section F.6.

F.5 Seepage Analyses Results

The results of our seepage analyses are presented in Table F-2 below and shown graphically in the attached model setup and output figures (Figures F-1 through F-10). Our steady-state seepage analyses indicate that seepage through the levee embankment is unlikely to manifest in excessive vertical hydraulic gradients or excessive discharge, indicating the proposed levee sections and design are sufficient from a seepage perspective and meet the Project design criteria.

Table F-2. Seepage Analysis Results, 100-Year Flood Scenario

Section	Stationing(1)	Vertical Hydraulic Gradient (unitless)(2)	Discharge near the levee toe per 100 linear feet of embankment (gpm)(3)
A-A'	3+50	< 0.1	9
B-B'	12+37.30	0.3	48
C-C'	16+25	0.3	10
D-D'	19+00	0.1	< 5
E-E'	24+00	0.4	9

Notes:

1) Stationing per draft 100 percent Project drawings and as shown on Figure 2.
2) Maximum vertical hydraulic gradient anticipated during the 100-year flood WSE. Modeled with long-term (scoured) levee geometry. Measured at/near the landward toe of the levee embankment.
3) Total discharge anticipated during the 100-year flood WSE. Modeled with long-term (scoured) levee geometry. Measured at/near the landward toe of the levee embankment.

The geometry of the landward toe of Section B-B' leads to unique seepage conditions: the swale at the landward levee toe is expected to accumulate water as seepage occurs in the 100-year flood scenario. We assessed the vertical hydraulic gradient at the landward toe and rate of discharge into the swale as it fills up. Our results are summarized in Table F-3 below. Based on our analyses, we anticipate that the swale will accumulate water to a level approaching but not exceeding the 100-year flood elevation. As the swale fills, the vertical hydraulic gradient and rate of discharge will diminish and approach zero. Based on our assessment, we consider the risk of water levels in the swale exceeding the 100-year flood or otherwise uncontrolled flooding from seepage extending beyond the swale extents to be low. Section B-B' is representative of the setback levee between Stations 12+00 and 13+00.

Table F-3. Swale Fill Sensitivity, Section B-B', 100-Year Flood Scenario

Percentage of Swale Fill(1)	Vertical Hydraulic Gradient (unitless)(2)	Discharge through the levee toe per 100 linear feet of embankment (gpm)(3)
0%	0.3	48
25%	0.2	47
50%	0.1	27
75%	< 0.1	22
100%	< 0.1	6

Notes:

1) Where maximum swale fill (i.e. 100 percent full) is equal to the existing 100-year flood elevation modeled at this location. The existing 100-year flood elevation at this location is 258.3 (Tetra Tech, 2022).
2) Maximum vertical hydraulic gradient anticipated during the 100-year flood WSE, in the modeled swale fill scenario. Modeled with long-term (scoured) levee geometry. Measured at/near the landward toe of the levee embankment.
3) Total discharge anticipated during the 100-year flood WSE in the modeled swale fill scenario. Modeled with long-term (scoured) levee geometry. Measured at/near the landward toe of the levee embankment.

F.6. Seepage Sensitivity Analysis

We conducted sensitivity analyses of the seepage models to account for the inherent variability of natural soil deposits and better inform the levee select fill and core geometry requirements. We performed our sensitivity analyses at the critical sections for seepage: B-B' (where discharge at the landward embankment toe was greatest) and E-E' (where vertical hydraulic gradients at the levee toe were greatest). Our sensitivity analyses considered the 100-year flood scenario.

We isolated the horizontal hydraulic conductivity (k_h) of the native soil deposits, select fill, levee core fill, and ballast fill and varied them by an order of magnitude. For example, we varied the hydraulic conductivity of Select Fill Type 1 and the native sand and alluvial gravel facies between $1x10^{-2}$ feet per second (ft/s) and $1x10^{-4}$ ft/s. Likewise, we isolated the ratio of vertical to horizontal hydraulic conductivities (k_v:k_h) for the fill soils and varied it between 0.5 and 2; we varied k_v:k_h for the native deposits between 0.05 and 0.2. We assessed the sensitivity of the seepage model by observing how the vertical

hydraulic gradient and discharge at and near the levee toe varied throughout the analyses. Our results are summarized in Table F-4 below.

Table F-4. Seepage Sensitivity Results

Variable	Variable Range	Section B-B'		Section E-E'	
		Range of vertical hydraulic gradient (unitless)	Range of discharge through the levee toe per 100 linear feet of embankment (gpm)	Range of vertical hydraulic gradient (unitless)	Range of discharge through the levee toe per 100 linear feet of embankment (gpm)
Alluvial Deposits – Sand Facies, k_h	$1.1x10^{-3} \pm 10^{-1}$ ft/s	0.3±0.3	48±26	N/A(1)	N/A(1)
Alluvial Deposits – Sand Facies, k_v:k_h	0.1 ± 0.05	0.3±0.0	48±5	N/A(1)	N/A(1)
Alluvial Deposits – Gravel Facies, k_h	$1.1x10^{-3} \pm 10^{-1}$ ft/s	0.3±0.1	48±9	0.4±0.1	9±6
Alluvial Deposits – Gravel Facies, k_v:k_h	0.1 ± 0.05	0.3±0.0	48±1	0.4±0.1	9±9
Levee Select Fill Type 1, k_h	$1.1x10^{-3} \pm 10^{-1}$ ft/s	0.3±0.4(2)	48±51	0.4±0.1	9±3
Levee Select Fill Type 1, k_v:k_h	1 ± 0. 5	0.3±0.1	48±3	0.4±0.1	9±3
Levee Core Fill, k_h	$2.9x10^{-6} \pm 10^{-2}$ ft/s	0.3±0.0	48±2	0.4±0.1	9±5
Levee Core Fill, kv:k_h	1 ± 0. 5	0.3±0.0	48±3	0.4±0.0	9±0
Ballast Fill, k_h	$1.1x10^{-3} \pm 10^{-1}$ ft/s	0.3±0.1	48±49	N/A(3)	N/A(3)
Ballast Fill, k_v:k_h	1 ± 0. 5	0.3±0.0	48±2	N/A(3)	N/A(3)

Notes:

1) Based on nearby subsurface exploration data, Alluvial Deposits – Sand Facies, are not present near Section E-E'.

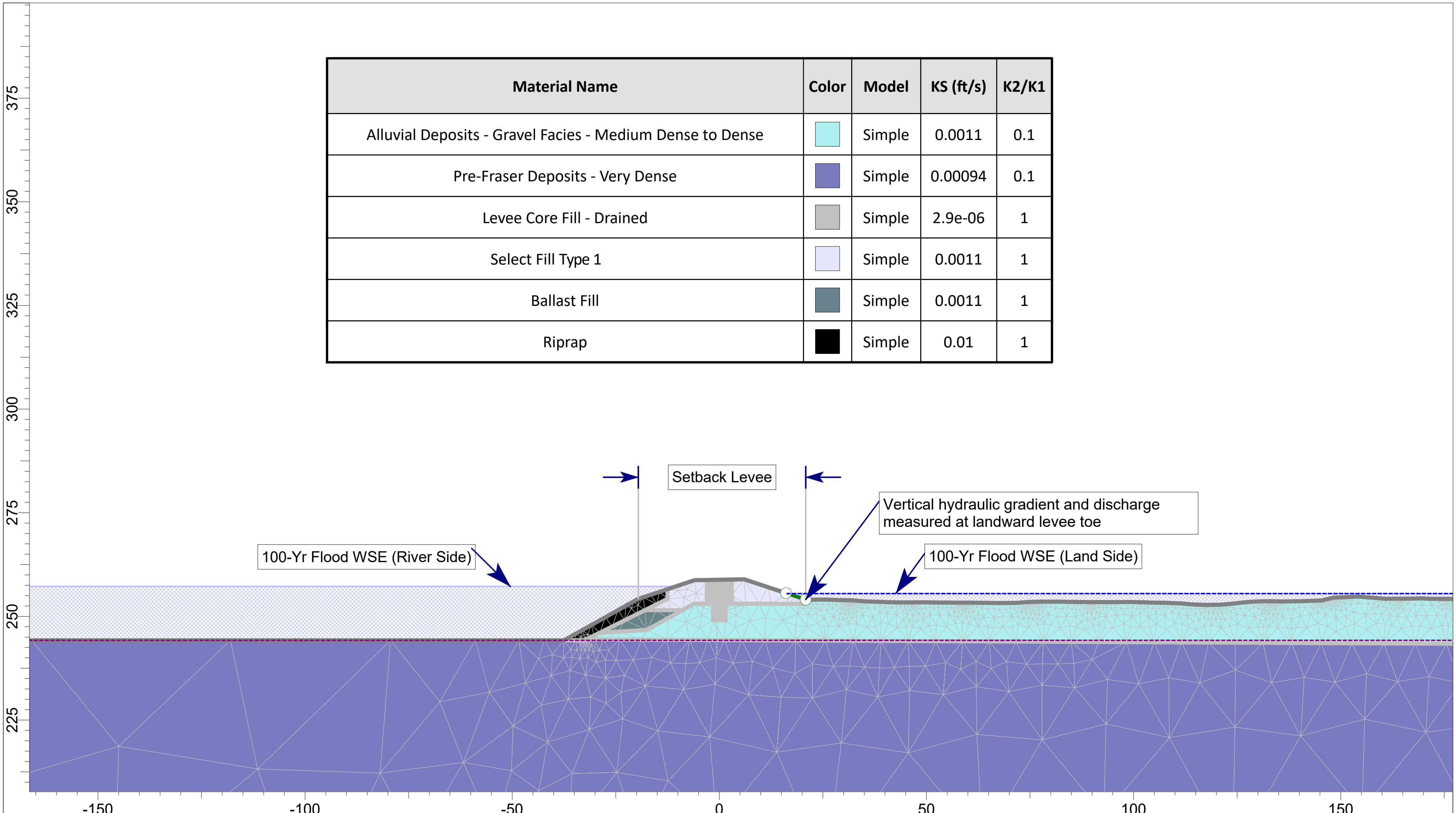
2) Our sensitivity analyses indicate that vertical hydraulic gradients greater than the threshold value of 0.5 are anticipated at Section B-B' when the horizontal hydraulic conductivity of Levee Select Fill Type 1 is less than 5E-04 feet per second.
3) Only minor amounts of Ballast Fill are planned for Section E-E'; we did not do a sensitivity analysis for Ballast Fill at Section E-E'.

We also performed sensitivity analyses on the levee core geometry at Section B-B'. The levee at this section will be built atop a filled in pond, and therefore no inspection trench will be necessary. A rectangular levee core cross-section is appropriate and allows for more straightforward construction at this location compared to the T-shaped levee core cross sections typical of the rest of the setback levee, where an inspection trench is required. We varied the width of the levee core between 6 and 12 feet. Our results indicated vertical hydraulic gradient and rate of discharge through the levee toe exhibited low sensitivity to the width of the levee core. We consider a levee core of 6 feet to be economical and provide sufficient seepage control at Section B-B'. Section B-B' is representative of the setback levee between Stations 12+00 and 13+00.

Our seepage sensitivity analyses indicate the Levee Core Fill may have a horizontal hydraulic conductivity ranging between $2.9x10^{-4}$ ft/s and $2.9x10^{-8}$ ft/s. Our seepage sensitivity analyses also indicate that the Select Fill Type 1 used for the levee embankment beyond the levee core fill should be carefully controlled to limit its horizontal hydraulic conductivity to a range between approximately $1x10^{-2}$ ft/s and $4x10^{-4}$ ft/s. The gradations of the Levee Core Fill and Select Fill Type 1 described in Sections 6.5.3 and 6.5.4, respectively, are anticipated to result in a horizontal hydraulic conductivity within this range.

APPENDIX F
FIGURES

Material Name	Color	Model	KS (ft/s)	K2/K1
Alluvial Deposits - Gravel Facies - Medium Dense to Dense		Simple	0.0011	0.1
Pre-Fraser Deposits - Very Dense		Simple	0.00094	0.1
Levee Core Fill - Drained		Simple	2.9e-06	1
Select Fill Type 1		Simple	0.0011	1
Ballast Fill		Simple	0.0011	1
Riprap		Simple	0.01	1



Legend

Finite Element Mesh

Discharge Measurement

A-A' STA 3+50, Long-Term Conditions Waterward, Steady-State Seepage, 100-Yr Flood

Seepage Analysis

Geotechnical Design Report
Jan Road Neighborhood Improvements
King County, Washington

SLIDE 8.032

SCALE: 1:400

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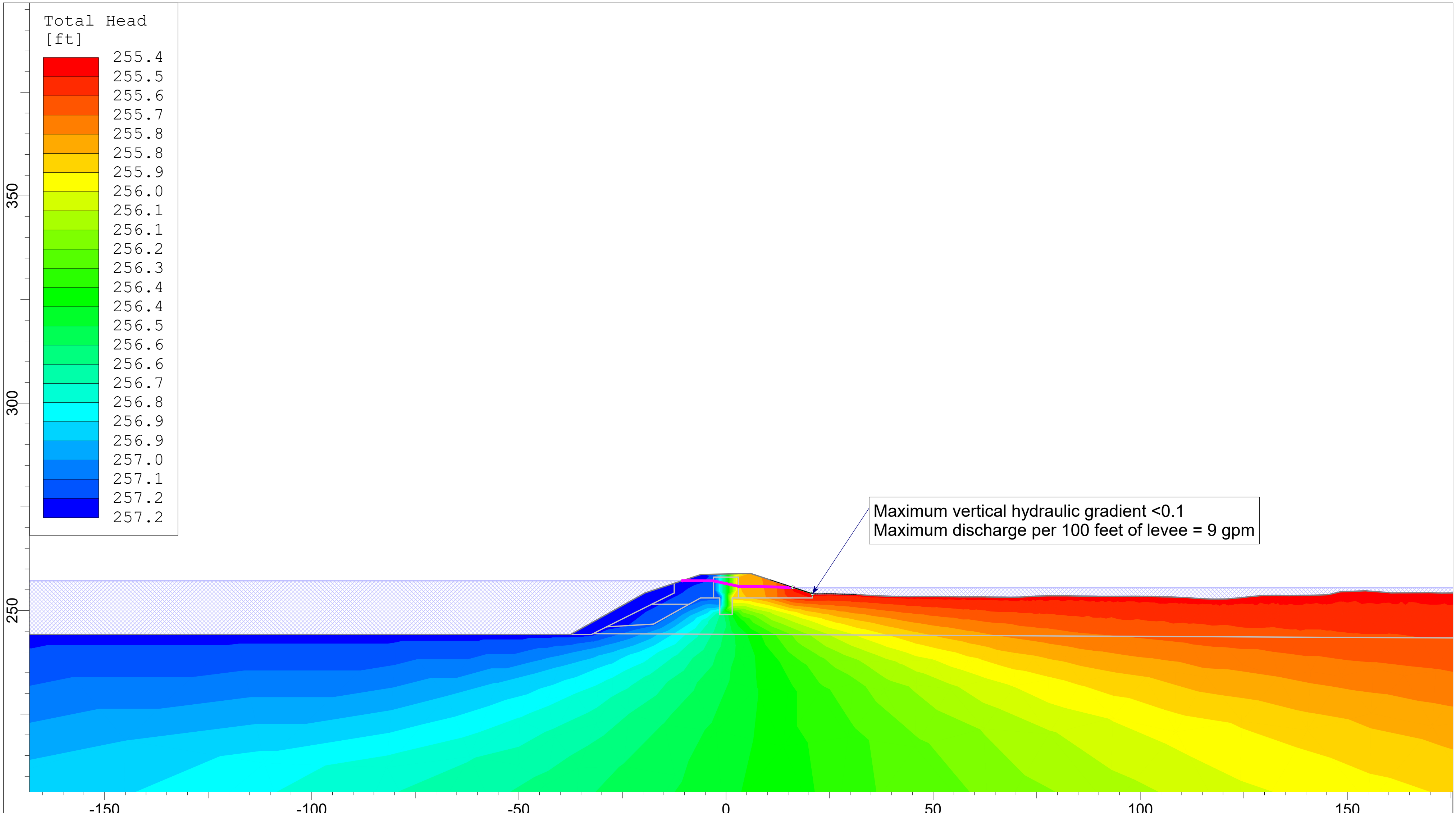
REVIEWED BY: A. Holmson

APPENDIX: **F-1**



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Steady-State Seepage Groundwater Table

A-A' STA 3+50, Long-Term Conditions Waterward, Steady-State Seepage, 100-Yr Flood

Seepage Analysis

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King County, Washington

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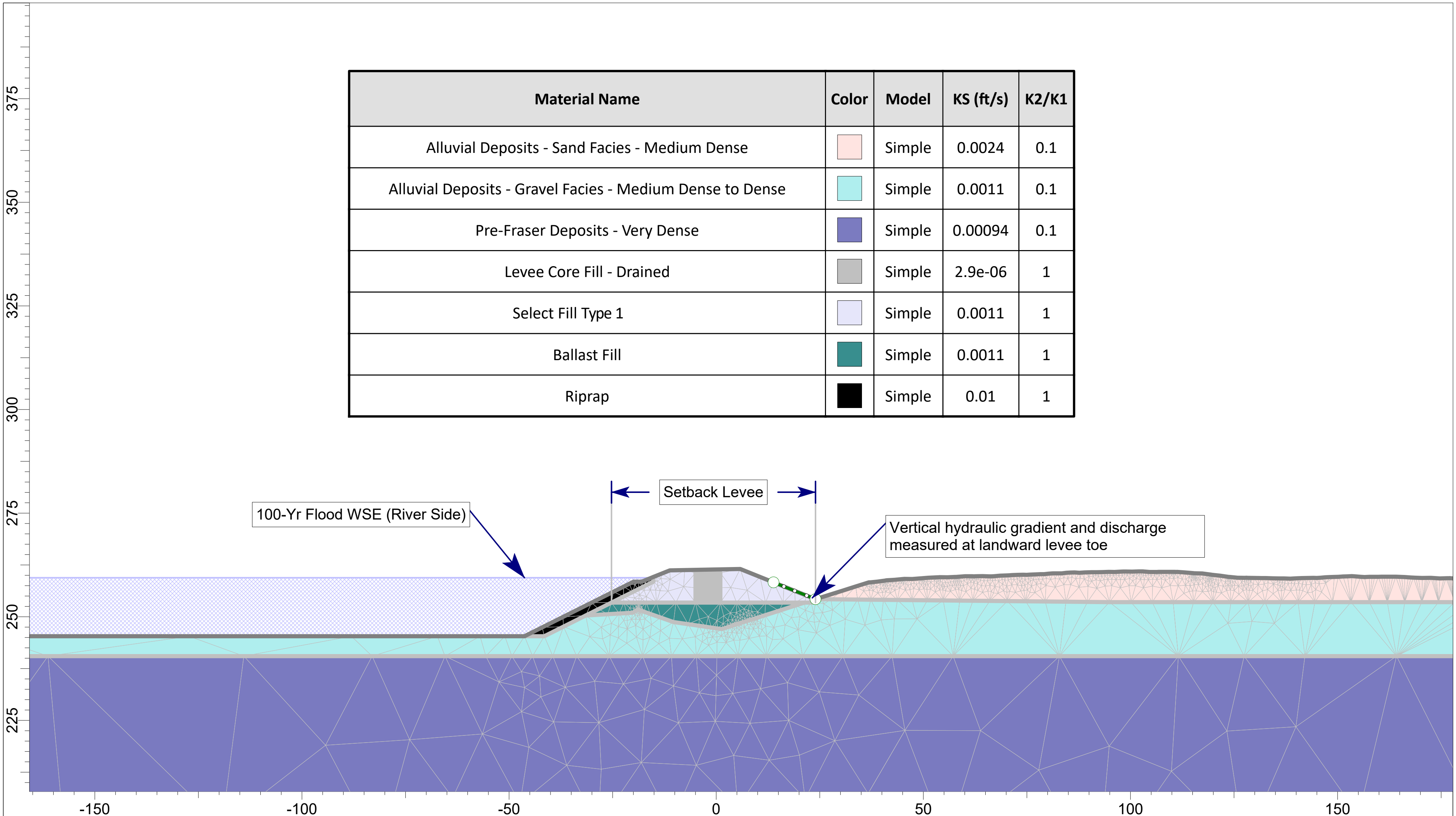
REVIEWED BY: A. Holmson

APPENDIX: F-2

Material Name	Color	Model	KS (ft/s)	K2/K1
Alluvial Deposits - Sand Facies - Medium Dense		Simple	0.0024	0.1
Alluvial Deposits - Gravel Facies - Medium Dense to Dense		Simple	0.0011	0.1
Pre-Fraser Deposits - Very Dense		Simple	0.00094	0.1
Levee Core Fill - Drained		Simple	2.9e-06	1
Select Fill Type 1		Simple	0.0011	1
Ballast Fill		Simple	0.0011	1
Riprap		Simple	0.01	1



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Finite Element Mesh

Discharge Measurement

B-B' STA 12+37.30, Long Term Conditions Waterward, Steady-State Seepage, 100-Yr Flood

Seepage Analysis

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King County, Washington

SLIDE 8.032

SCALE: 1:400

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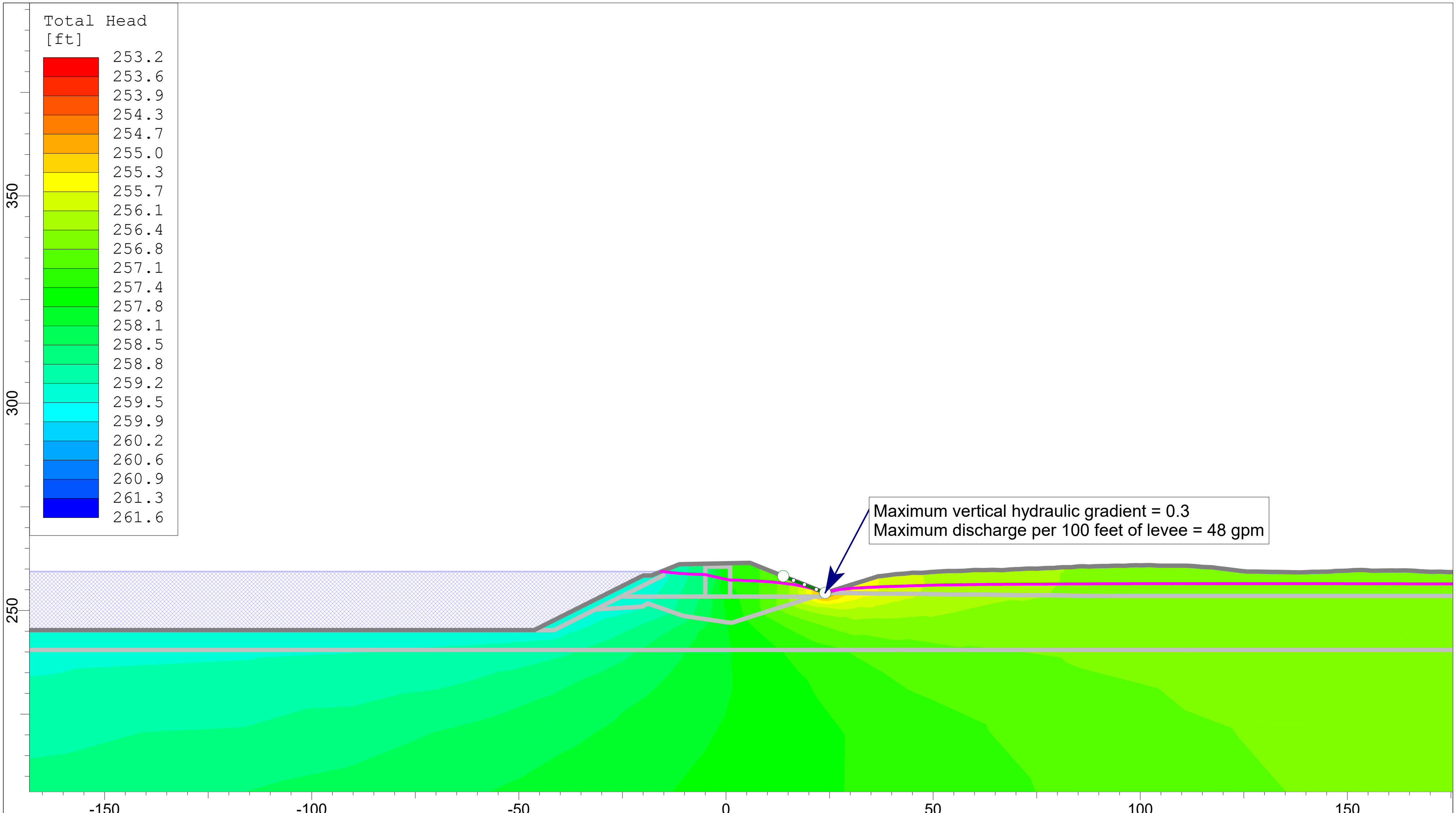
APPENDIX: F-3



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Steady-State Seepage Groundwater Table

B-B' STA 12+37.30, Long Term Conditions Waterward, Steady-State Seepage, 100-Yr Flood

Seepage Analysis

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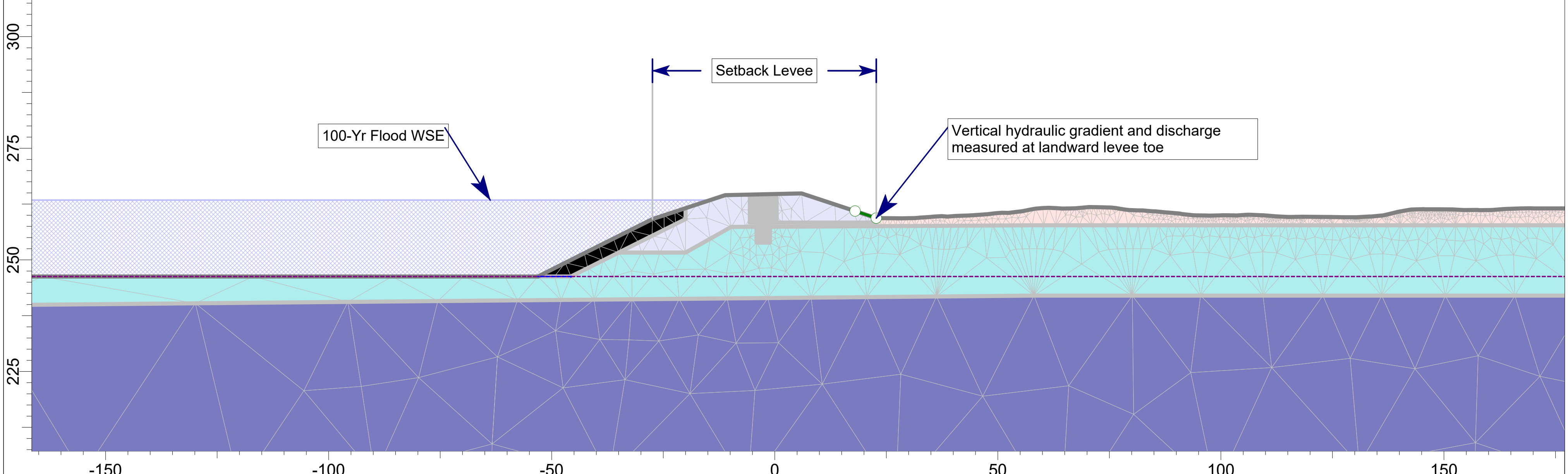
APPENDIX: F-4

Material Name	Color	Model	KS (ft/s)	K2/K1
Alluvial Deposits - Sand Facies - Medium Dense		Simple	0.0024	0.1
Alluvial Deposits - Gravel Facies - Medium Dense to Dense		Simple	0.0011	0.1
Pre-Fraser Deposits - Very Dense		Simple	0.00094	0.1
Levee Core Fill - Drained		Simple	2.9e-06	1
Select Fill Type 1		Simple	0.0011	1
Riprap		Simple	0.01	1



Legend

Finite Element Mesh

Discharge Measurement

C-C' STA 16+25, Long-Term Conditions Waterward, Steady-State Seepage, 100-Yr Flood

Seepage Analysis

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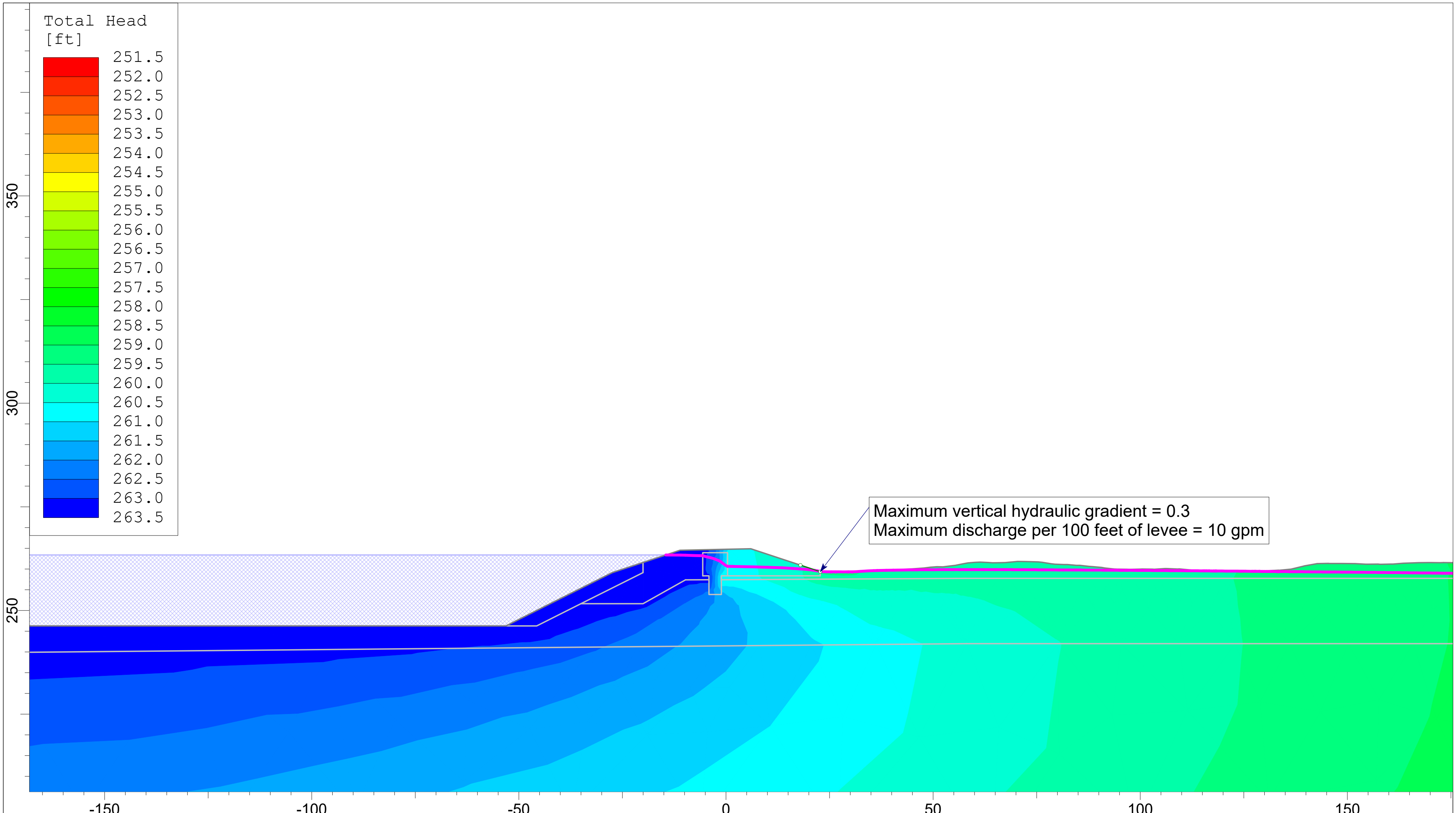
APPENDIX: **F-5**



Legend

Steady-State Seepage Groundwater Table

C-C' STA 16+25, Long-Term Conditions Waterward, Steady-State Seepage, 100-Yr Flood

Seepage Analysis

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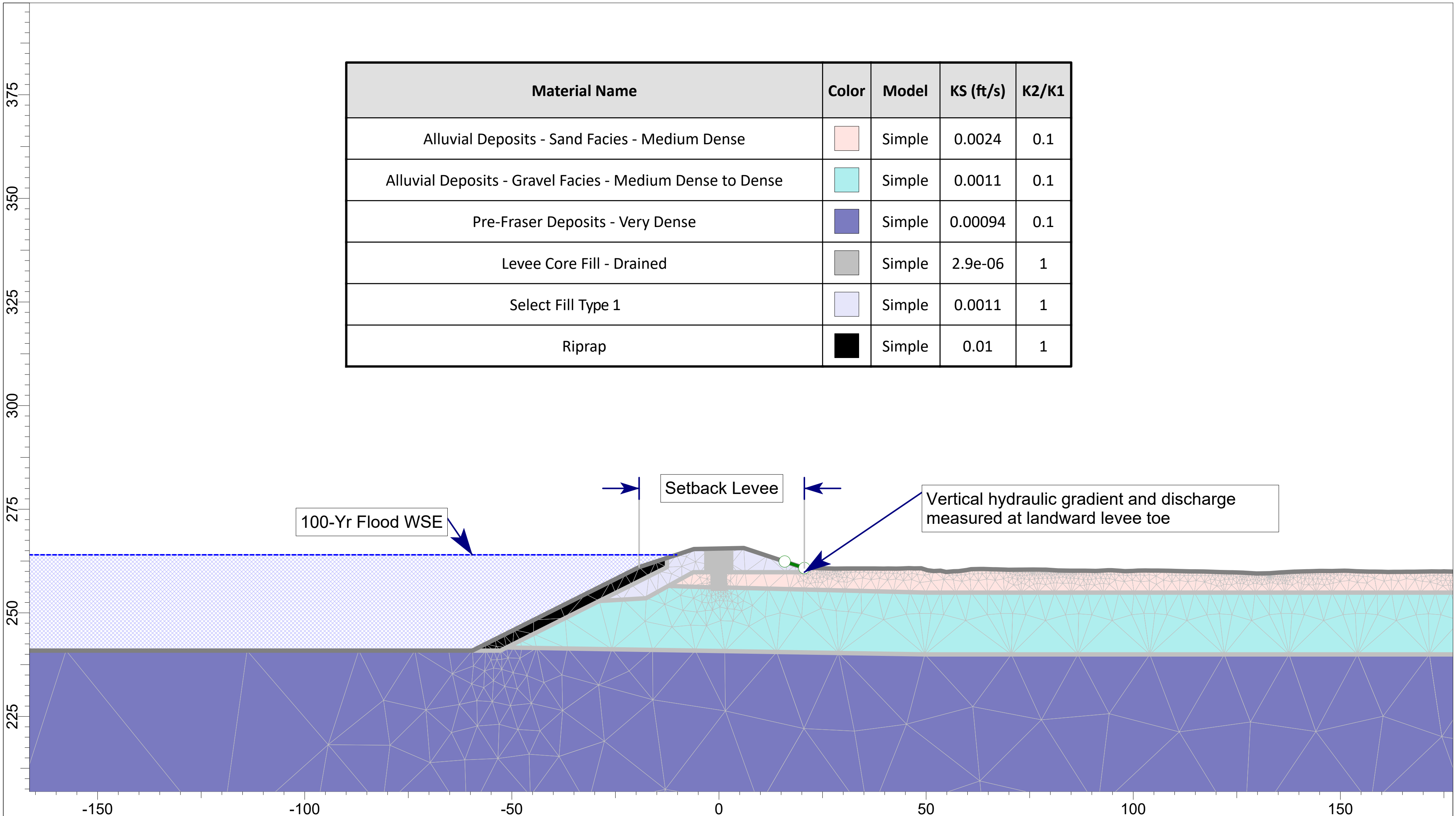
REVIEWED BY: A. Holmson

APPENDIX: **F-6**

Material Name	Color	Model	KS (ft/s)	K2/K1
Alluvial Deposits - Sand Facies - Medium Dense		Simple	0.0024	0.1
Alluvial Deposits - Gravel Facies - Medium Dense to Dense		Simple	0.0011	0.1
Pre-Fraser Deposits - Very Dense		Simple	0.00094	0.1
Levee Core Fill - Drained		Simple	2.9e-06	1
Select Fill Type 1		Simple	0.0011	1
Riprap		Simple	0.01	1



Legend

Finite Element Mesh

Discharge Measurement

D-D' STA 19+00, Long-Term Conditions Waterward, Steady-State Seepage, 100-Yr Flood

Seepage Analysis

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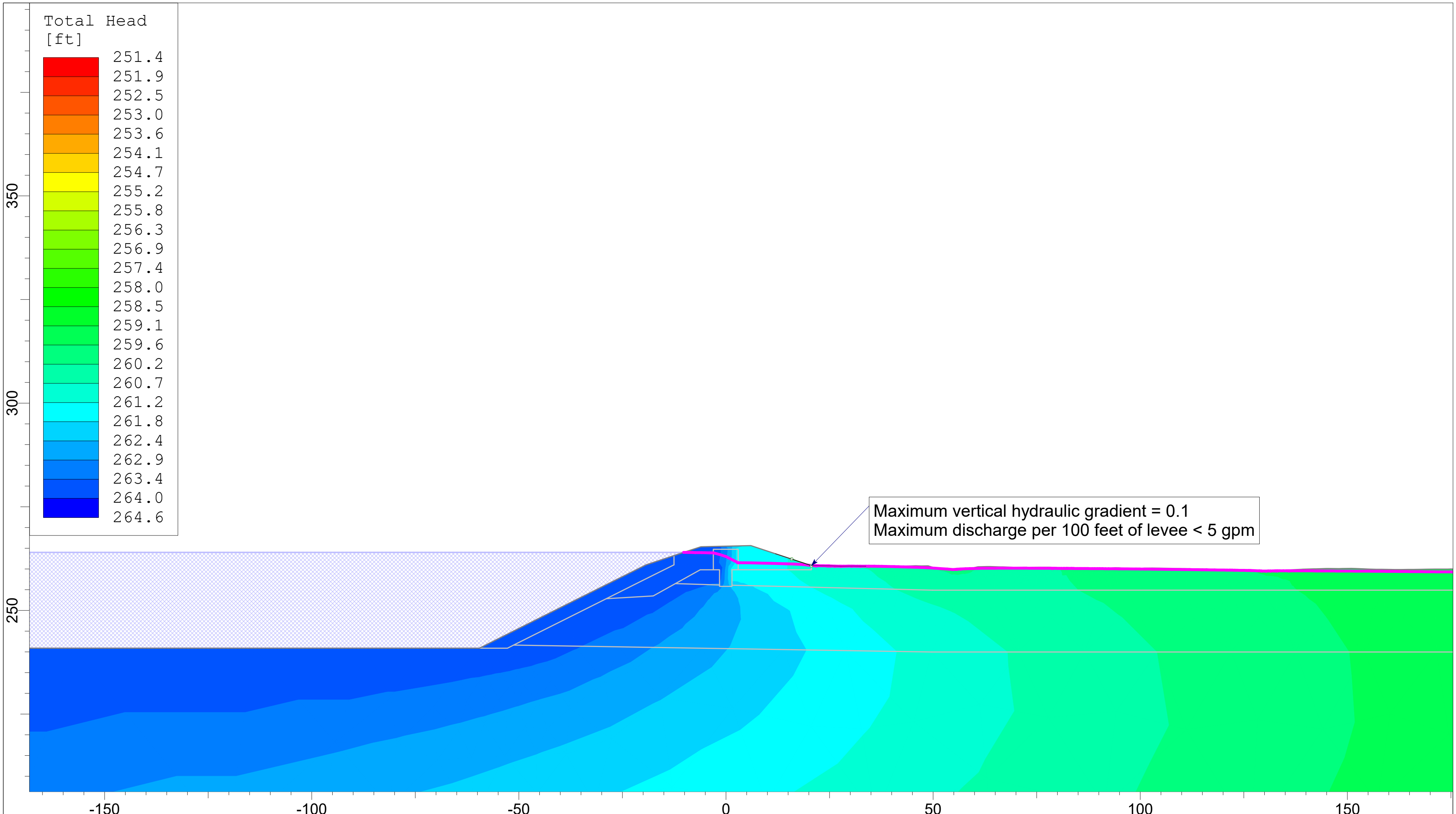
APPENDIX: **F-7**



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Steady-State Seepage Groundwater Table

D-D' STA 19+00, Long-Term Conditions Waterward, Steady-State Seepage, 100-Yr Flood

Seepage Analysis

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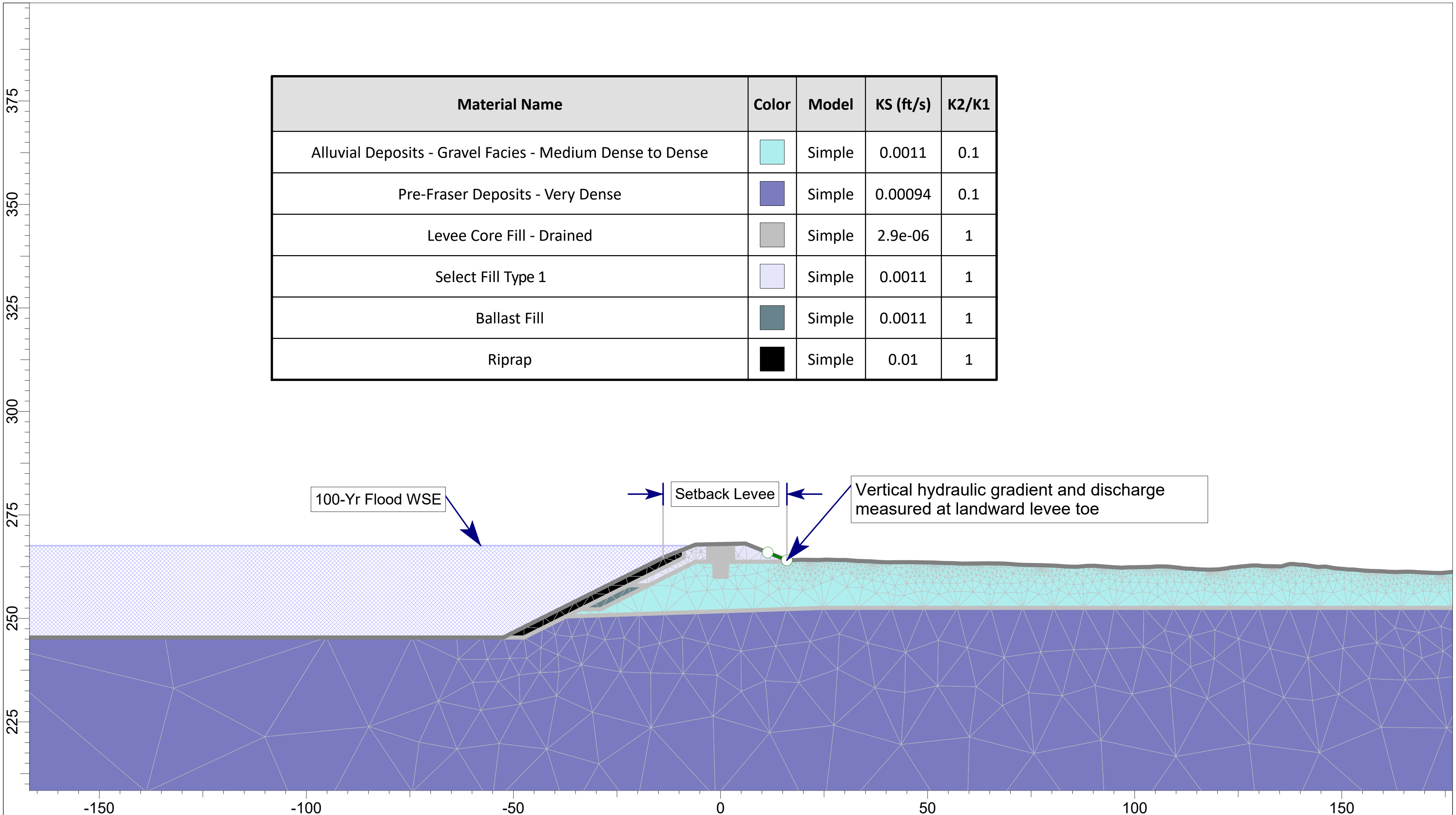
Material Name	Color	Model	KS (ft/s)	K2/K1
Alluvial Deposits - Gravel Facies - Medium Dense to Dense		Simple	0.0011	0.1
Pre-Fraser Deposits - Very Dense		Simple	0.00094	0.1
Levee Core Fill - Drained		Simple	2.9e-06	1
Select Fill Type 1		Simple	0.0011	1
Ballast Fill		Simple	0.0011	1
Riprap		Simple	0.01	1



Legend

Finite Element Mesh

Discharge Measurement

E-E' STA 24+00, Long-Term Conditions Waterward, Steady-State Seepage, 100-Yr Flood

Seepage Analysis

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M. Otto

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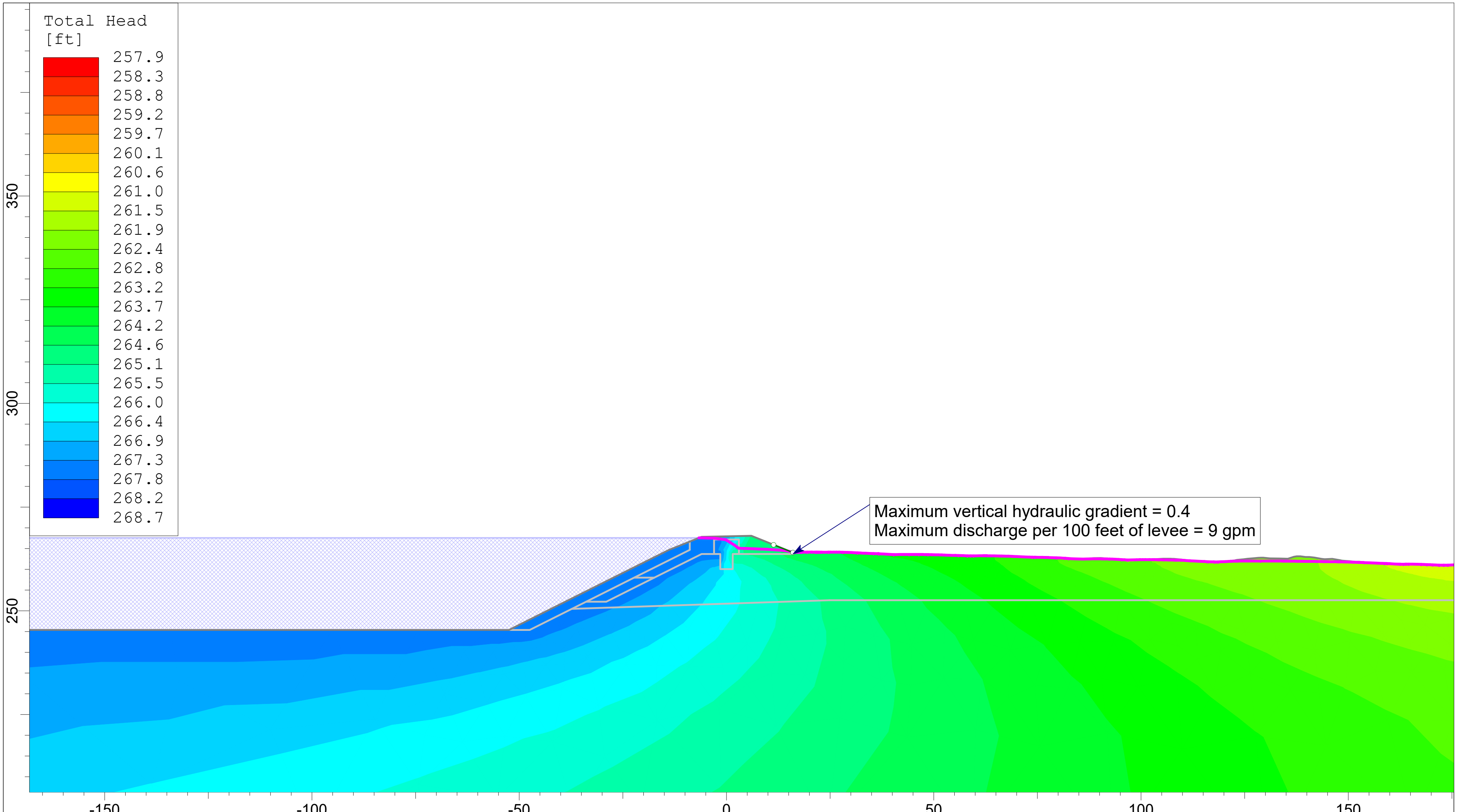
A. Holmson

APPENDIX:
F-9



Legend

Steady-State Seepage Groundwater Table

E-E' STA 24+00, Long-Term Conditions Waterward, Steady-State Seepage, 100-Yr Flood

Seepage Analysis

Geotechnical Design Report
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APPENDIX G

Slope Stability Analyses

G. Slope Stability Analyses

G.1. Methodology

To evaluate slope stability of the proposed setback levee embankment, we performed slope stability analyses (SSA) using the slope stability module within the computer program Slide (Rocscience, 2017). The Slide slope stability module is a two-dimensional, finite-element program that performs slope stability computations based on the modeled slope conditions and computes a factor of safety against slope failure, FS_{SSA}, which is defined as:

$$FS_{SSA} = s/\tau$$

where “s” is the available shear strength of the soil and “τ” is the shear stress required for “just-stable” equilibrium. A just-stable equilibrium condition would result in a factor of safety of 1.0 while an unstable condition would result in a factor of safety less than 1.0.

Key inputs into the Slide slope stability module are levee embankment geometry, soil parameters such as unit weight, soil shear strength parameters (friction angle and cohesion), and groundwater conditions. We used Spencer’s method in our Slide analyses. Through iterative calculations of successive finite element runs, the slope stability module computes forces and performs limit equilibrium calculations on thousands of potential slip surfaces.

Key outputs from the Slide slope stability module are the factors of safety of each slip surface throughout both the riverside and landside of the levee embankment. Analysis outputs for this Project control slip surface searches to include only failures that intersect the levee crest or beyond—surficial failures (less than 3 to 5 feet thick) are considered maintenance issues and are not included, since they do not affect the global stability and/or core of the levee.

G.2. Design Assumptions and Criteria

We considered stability criteria associated with multiple analysis cases in accordance with the United States Army Corps of Engineers (USACE) Engineering Manual (EM) 1110-2-1913 *Design and Construction of Levees* (USACE, 2000). These guidelines recommend that three analysis cases be considered in design: end-of-construction, steady state seepage, and sudden drawdown from full flood stage. The minimum factor of safety requirement for each design case is presented in Table G-1 below. Each design case is further described below. Additionally, we evaluated seismic conditions, considering the Operating Basis Earthquake (OBE) and Maximum Design Earthquake (MDE), which are further described in Section 4.4.

Table G-1. Minimum Factors of Safety for Levee Stability

Analysis Case	Minimum Factor of Safety (FS)[1]
End-of-Construction	1.3
Steady-State Seepage	1.4
Sudden Drawdown	1.1[2]
Seismic, OBE	1.1
Seismic, MDE	1.0

Notes:
1) Applies to both the riverside and landside levee slopes.
2) USACE guidance suggests a FS of 1.0 for short duration flood conditions and a FS of 1.2 for long duration flood conditions. We assumed medium flood duration and, accordingly, a minimum FS of 1.1.

G.2.1. End of Construction

This case represents the conditions immediately following the construction of the new setback levee. The new levee embankment will impose a load on the foundation soils and underlying geology that will elevate the pore-water pressures within the soil matrix and create undrained conditions within the Levee Core Fill immediately following construction. Drained conditions within the underlying granular, cohesionless soils will exist since excess pore-water pressures dissipate quickly in granular, cohesionless soils. The mean water surface elevation (WSE) was used for the end-of-construction water surface and no future scoured conditions were assumed since a flood event that will induce scour is unlikely to occur during the first few weeks/months following construction. End-of-construction conditions assume that immediate settlement has occurred (as the levee was constructed). No long-term consolidation settlement or secondary compression is anticipated along the proposed setback levee alignment.

G.2.2. Steady-State Seepage

The steady-state seepage condition occurs when the river levels remain at or near an identified stage long enough that the levee embankment becomes saturated and a condition of steady-state seepage through and below the embankment occurs. We evaluated the steady-state seepage condition for the100-yr WSE. We modeled the steady-state piezometric surfaces using the seepage module of the Slide program. Our seepage analyses are described in Appendix F.

We modeled the levee with future projected scour conditions (“Long-Term Conditions”), which assume that processes such as channel migration and flood events have scoured away the toe of the waterward side of the levee embankment to a predicted scour elevation (Tetra Tech, 2022). The Long-Term Conditions also assume that launchable scour protection elements are in the launched configuration. We applied drained or effective strengths for all soil types since the steady-state seepage case is a long-term condition anticipated to occur after the pore-water pressures have dissipated from cohesive soils within the system, namely the Levee Core Fill.

G.2.3. Sudden Drawdown

This case represents the condition where a prolonged flood stage saturates the levee embankment and underlying soils (i.e., steady-state seepage conditions). Then, as the flood recedes, the flood waters recede faster than the soils can drain, causing elevated pore-water pressures and reducing the shear strength of slower draining soil units such as the Levee Core Fill. When the flood recedes, it also quickly removes the buttressing pressure of the flood waters against the waterside of the levee, adding to the potential for unstable soil conditions.

We modeled sudden drawdown conditions using an initial water level equivalent to the 100-year WSE, assuming the piezometric surface identified in our Seepage analyses (described in Appendix F). We assumed the drawdown water level to be equivalent to the mean annual WSE. We assumed excess pore-water pressures would develop in the cohesionless Levee Core Fill material. We assumed excess pore-water pressures would drain relatively quickly from the other, more coarse-grained soils in the system. Accordingly, we used the lesser of drained versus total stress undrained shear strength parameters for the Levee Core Fill during sudden drawdown analyses. We used drained strength parameters for the other soils.

G.3. Design Conditions for Analysis

We performed slope stability calculations at five critical sections along the proposed levee alignment. We considered two water levels in our stability analyses representing the mean annual WSE and the 100-year WSE. We also considered two seismic design scenarios: the Operating Basis Earthquake (OBE) and the Maximum Design Earthquake (MDE). Seismic design parameters are presented in Section 4.4. Seismic conditions in Slide were modeled using a horizontal pseudostatic coefficient, calculated as 0.5*PGA, where PGA is defined as the peak ground accelerations anticipated to develop at the Site during the OBE and MDE.

We created the subsurface profiles used in the models based on reference explorations performed during our subsurface explorations. The critical section locations, reference explorations, and water and scour conditions used in our analyses are presented in Table G-2 below. Typical levee section geometry and launchable scour toe geometry modeled in Slide are presented in Sections 4.2 and 5.1.1 of the report text.

Table G-2. Levee Analysis Section Locations and Details

Section	Stationing(1)	Reference Exploration(s)	Planned Levee Height(2)	Riverward 100-yr(3) WSE(4)(5)	Landward 100-yr(6) WSE	Mean Annual WSE	Scour Elevation(7)
A-A’	3+50	ATP-06 AMW-03	5.8	257.2	255.5	251.5	244.2
B-B’	12+37.30	ATP-04 AB-03	3.9	259.4	-	253.4	245.3
C-C’	16+25	ATP-02 AB-01	6.3	263.4	-	251.5	246.3

Section	Stationing(1)	Reference Exploration(s)	Planned Levee Height(2)	Riverward 100-yr(3) WSE(4)(5)	Landward 100-yr(6) WSE	Mean Annual WSE	Scour Elevation(7)
D-D’	19+00	B-6	5.7	264.0	-	251.5	240.9
E-E’	24+00	B-4 AMW-01	4.3	267.6	-	258.0	245.4

Notes:

1) Stationing per draft 100 percent Project drawings and as shown on Figure 2.
2) Levee height is measured at the top of levee (at centerline) to base of levee core fill.
3) 1% annual probability of exceedance flood.
4) WSE = water surface elevation, yr = year.
5) WSE information provided by Tetra Tech (Tetra Tech, 2022).
6) Minor flooding of landward side of levee predicted by hydraulic modeling.
7) Long-term elevations at riverside levee toe due to scour or projected future channel migration. Scour elevations provided by Tetra Tech (Tetra Tech, 2022).

G.4. Soil Parameters for Slope Stability Analyses

The soil stratigraphy used for each design section was based on our interpretation of the subsurface conditions as shown on the geologic profile (Figures 3-4) with corresponding engineering properties shown in Table 1. The setback levee embankment geometry was determined through an iterative design process with Tetra Tech. Discussion of the soil shear strength selection process for native soils and fill is included below.

Native Soils

We determined soil shear strength properties for each geologic deposit encountered at the Site based on observed soil conditions, correlations from *in situ* standard penetration tests, correlations with particle size test results, and our experience with local geology.

Fill

We anticipate the levee prism will consist of primarily re-used Sand and Gravel Facies sourced from on-Site (Select Fill Type 1), imported, relatively impermeable core fill (Levee Core Fill), and launchable scour protection (Riprap). Additionally, between approximately Stations 12+00 and 13+00 (represented by Section B-B’), the levee will be built over an existing pond that will be filled with Ballast Fill, anticipated to be sourced primarily from on-site Gravel Facies or similar imported soils.

We assigned shear strength properties of the re-used soil based on drained shear strength properties for the geologic unit from which they will be derived. We assigned drained and undrained shear strength properties of the imported Levee Core Fill based on our experience with cohesionless soils commonly used as levee core fill material. Drained shear strength parameters were applied in scenarios with long durations, which would allow pore pressures to dissipate over time (Steady-State Seepage). Undrained shear strength parameters were applied in short-duration scenarios (End of Construction, Sudden Drawdown, Seismic). We assigned a typical friction angle of the Riprap based on experience with commonly used materials for scour protection.

We supported our shear strength property assignments with sensitivity analyses to identify allowable ranges of shear strength properties for these soils. Our sensitivity analyses are described further in Section G-6.

G.5. Slope Stability Analyses Results

The results of our slope stability analyses are presented in Table G-3 below and shown graphically in the attached analysis outputs (Figures G-1 through G-60).

Our results indicate that the levee will be sufficiently stable on both the waterward and landward sides in most analysis cases. In the MDE seismic scenario, our results indicated the waterward side of the levee will be marginally stable (FS just under or equal to 1.0). These results indicate that some deformation (on the order of inches to a couple feet) could occur and damage the levee embankment surface and should be anticipated in the MDE scenario, for which we anticipate maintenance-level repairs will be necessary. We do not anticipate a global instability or catastrophic failure of the levee embankment resulting from the MDE seismic scenario.

Table G-3. Slope Stability Analysis Results

Case	Water Level(1)(2)	Minimum Required FS(1)	Waterward / Landward	FS Resulting at the Critical Section(4) (Reference Figure)(5)				
				A-A'	B-B'	C-C'	D-D'	E-E'
End of Construction	Mean Annual WSE	1.3	Waterward Landward	2.6 (G-3) 2.4 (G-4)	2.9 (G-15) 1.9 (G-16)	2.5 (G-27) 2.3 (G-28)	2.6 (G-39) 2.4 (G-40)	2.5 (G-51) 2.1 (G-52)
Steady-State Seepage	100-Yr Flood(3)	1.4	Waterward Landward	1.8 (G-5) 2.1 (G-6)	1.8 (G-17) 1.8 (G-18)	1.8 (G-29) 2.1 (G-30)	1.7 (G-41) 2.1 (G-42)	1.7 (G-53) 2.0 (G-54)
Sudden Drawdown	100-Yr Flood(3) -> Mean Annual WSE	1.1	Waterward Landward	1.7 (G-7) 2.4 (G-8)	1.7 (G-19) 1.9 (G-20)	1.7 (G-31) 2.3 (G-32)	1.6 (G-43) 2.4 (G-44)	1.5 (G-55) 2.1 (G-56)
Seismic, OBE	Mean Annual WSE	1.1	Waterward Landward	1.2 (G-9) 1.7 (G-10)	1.2 (G-21) 1.4 (G-22)	1.3 (G-33) 1.7 (G-34)	1.2 (G-45) 1.7 (G-46)	1.1 (G-57) 1.6 (G-58)
Seismic, MD	Mean Annual WSE	1.0	Waterward Landward	1.0 (G-11) 1.3 (G-12)	0.9 (G-23) 1.1 (G-24)	1.0 (G-35) 1.3 (G-36)	0.9 (G-47) 1.3 (G-48)	0.9 (G-59) 1.2 (G-60)

Notes:

1) WSE = water surface elevation, yr = year; FS = factor of safety.
2) WSE information provided by Tetra Tech (Tetra Tech, 2022).
3) 1% annual probability of exceedance flood.
4) Minimum factor of safety affecting the levee crest or beyond. Red values indicate a factor of safety less than the minimum required FS.
5) Model setups are shown on Figures G-1 and G-2 (Section A-A'), Figures G-13 and G-14 (Section B-B'), Figures G-25 and G-26 (Section C-C'), Figures G-37 and G-38 (Section D-D'), and Figures G-49 and G-50 (Section E-E').

G.6. Slope Stability Sensitivity Analyses

We conducted sensitivity analyses of the slope stability models to account for the inherent variability of natural soil deposits and better inform the requirements of the fill soils. We performed our sensitivity analyses at the critical section for slope stability in the most critical scenario where relatively low FS results were observed and long-term

anticipated scoured conditions are greatest: Section E-E' in the seismic MDE scenario, on the waterward side of the levee embankment. Since critical slope stability conditions are anticipated in long-term conditions versus end-of-construction conditions, we performed sensitivity analyses assuming long-term conditions only.

We isolated the soil and shear strength parameters (unit weight, friction angle, and cohesion) of the materials in the slope stability model and varied them over a reasonable range of values. We assessed the sensitivity of the slope stability model by using the Statistics – Sensitivity module within Slide. The Statistics – Sensitivity module calculates the range of the minimum factor of safety as a single parameter is varied and other parameters are held at a constant value.

The results of our sensitivity analyses are summarized in Table G-4 below. Our assessment of the results is that the slope stability results at the levee crest or beyond are not sensitive to reasonable ranges in soil unit weight and shear strength parameters within the model.

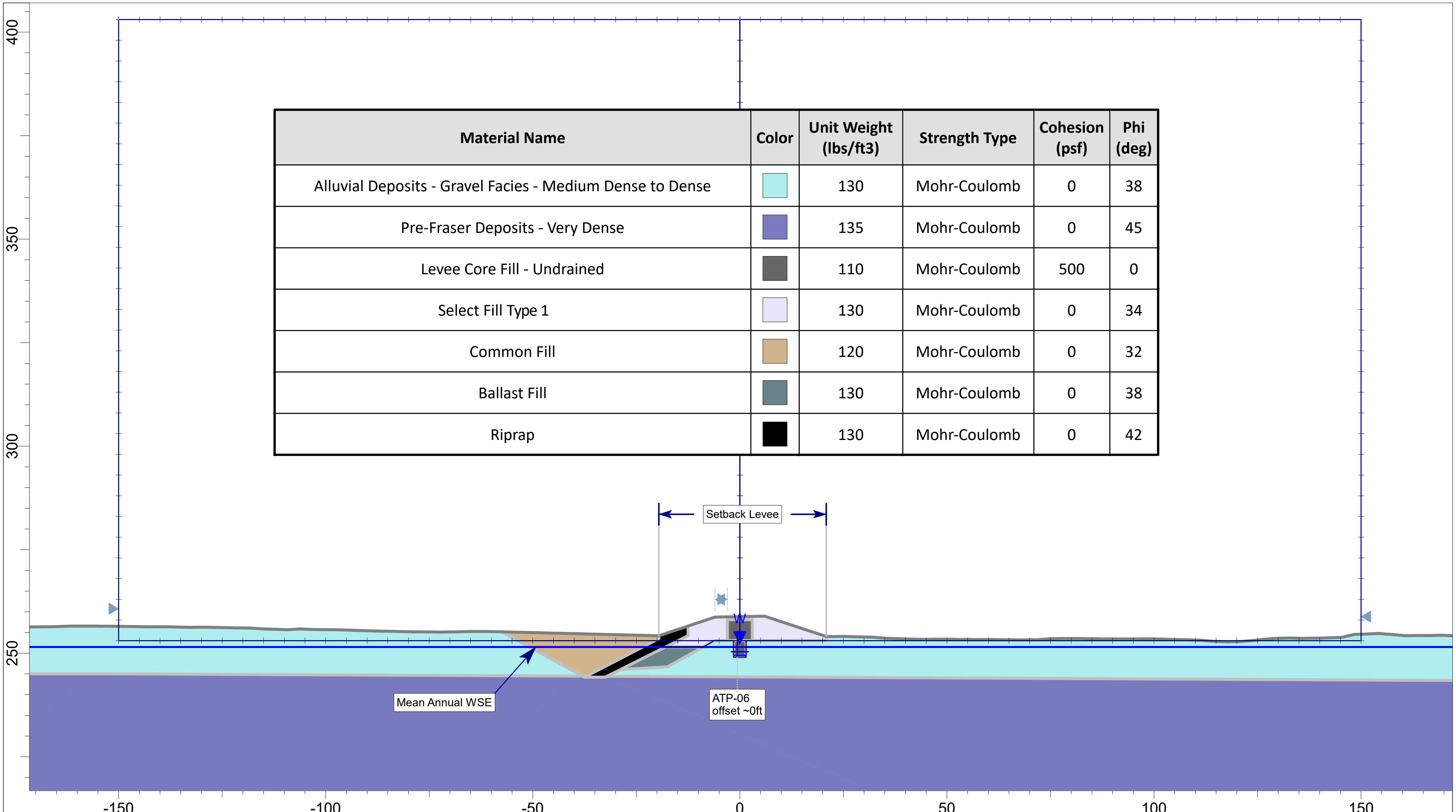
Table G-4. Slope Stability Sensitivity Results

Variable[(1)]	Variable Range[(1)]	Range of minimum factor of safety
Alluvial Deposits – Gravel Facies, φ'	38 ± 4 deg	1.4 ± 0 (not sensitive)
Alluvial Deposits – Gravel Facies, γ	130 ± 15 pcf	
Levee Core Fill (Undrained), Su	500 ± 500 psf	
Levee Core Fill (Undrained), γ	110 ± 15 pcf	
Levee Select Fill Type 1, φ'	38 ± 4 deg	
Levee Select Fill Type 1, γ	130 ± 15 pcf	
Ballast Fill, φ'	38 ± 4 deg	
Ballast Fill, γ	130 ± 15 pcf	
Riprap, φ'	42 ± 4 deg	
Riprap , γ	130 ± 15 pcf	

Notes:

1) φ' = effective friction angle; γ = unit weight; Su = undrained shear strength; deg = degree; pcf = pounds per cubic foot; psf = pounds per square foot.

APPENDIX G
FIGURES



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Alluvial Deposits - Gravel Facies - Medium Dense to Dense		130	Mohr-Coulomb	0	38
Pre-Fraser Deposits - Very Dense		135	Mohr-Coulomb	0	45
Levee Core Fill - Undrained		110	Mohr-Coulomb	500	0
Select Fill Type 1		130	Mohr-Coulomb	0	34
Common Fill		120	Mohr-Coulomb	0	32
Ballast Fill		130	Mohr-Coulomb	0	38
Riprap		130	Mohr-Coulomb	0	42

Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

A-A' STA 3+50, End of Construction Master Scenario

Slope Stability Analysis

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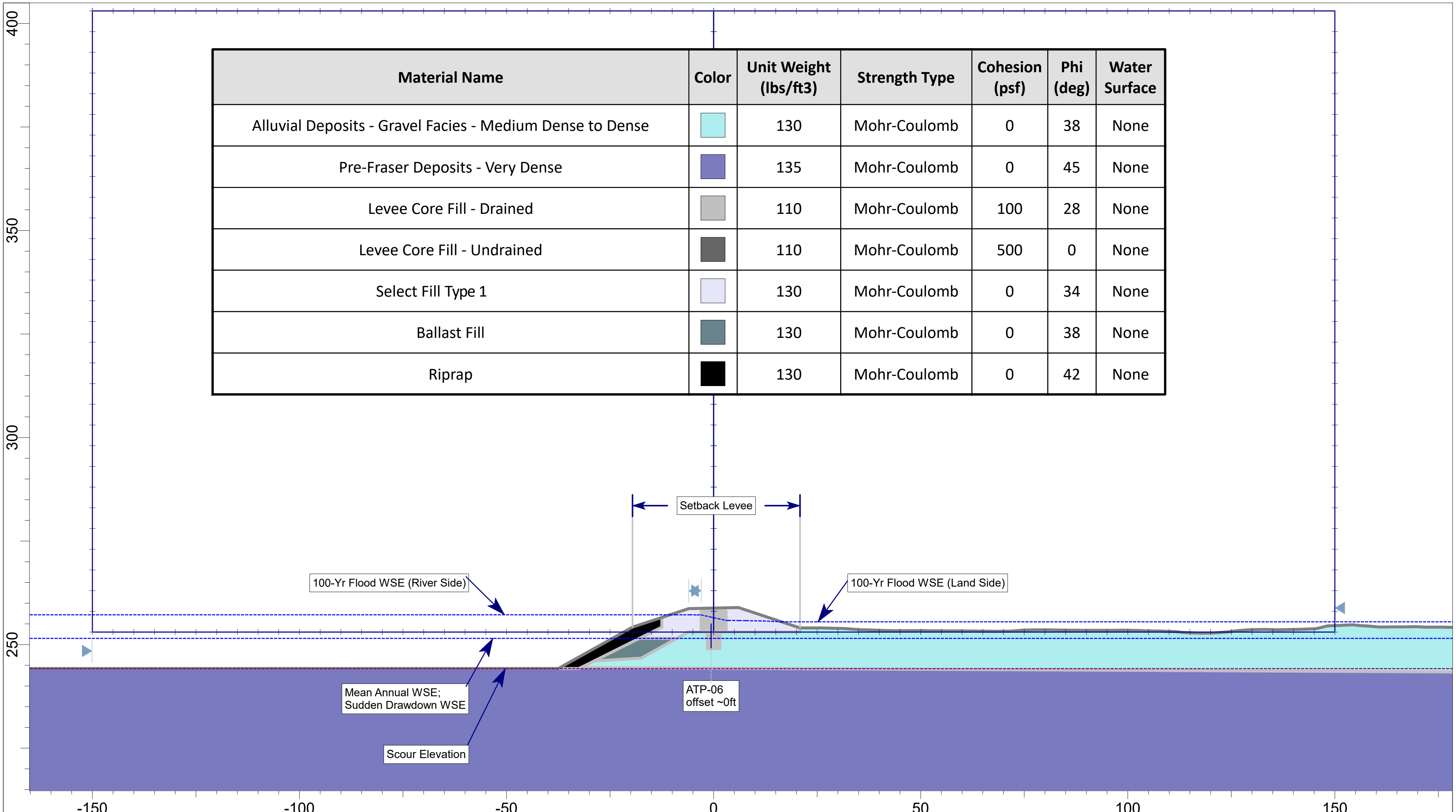
APPENDIX: G-1

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Alluvial Deposits - Gravel Facies - Medium Dense to Dense		130	Mohr-Coulomb	0	38	None
Pre-Fraser Deposits - Very Dense		135	Mohr-Coulomb	0	45	None
Levee Core Fill - Drained		110	Mohr-Coulomb	100	28	None
Levee Core Fill - Undrained		110	Mohr-Coulomb	500	0	None
Select Fill Type 1		130	Mohr-Coulomb	0	34	None
Ballast Fill		130	Mohr-Coulomb	0	38	None
Riprap		130	Mohr-Coulomb	0	42	None



Legend

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- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

A-A' STA 3+50, Long-Term Conditions Master Scenario

Slope Stability Analysis

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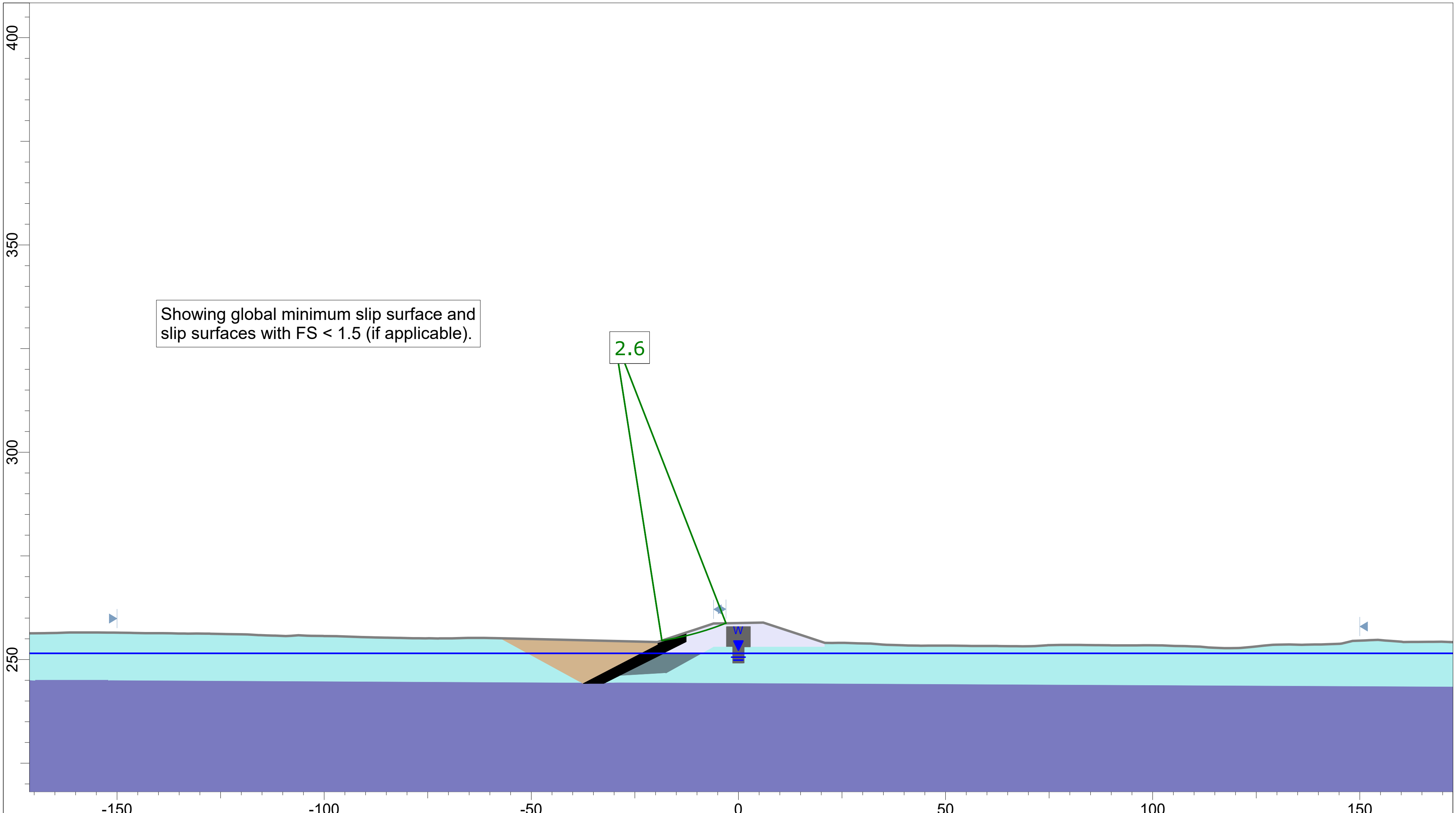
APPENDIX: **G-2**



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- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

A-A' STA 3+50, End of Construction Waterward, Mean Annual WSE

Slope Stability Analysis

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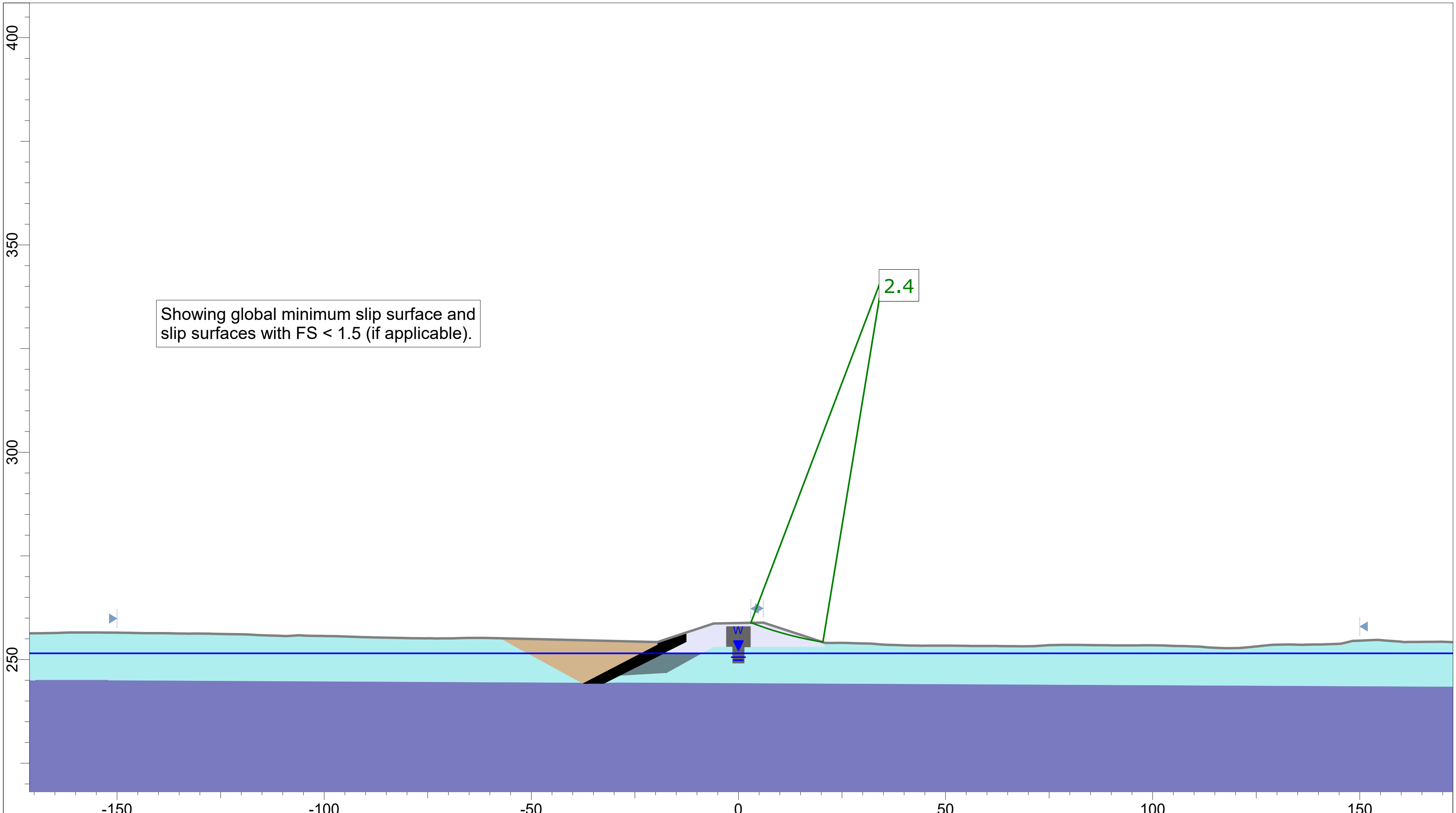

Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

A-A' STA 3+50, End of Construction Landward, Mean Annual WSE

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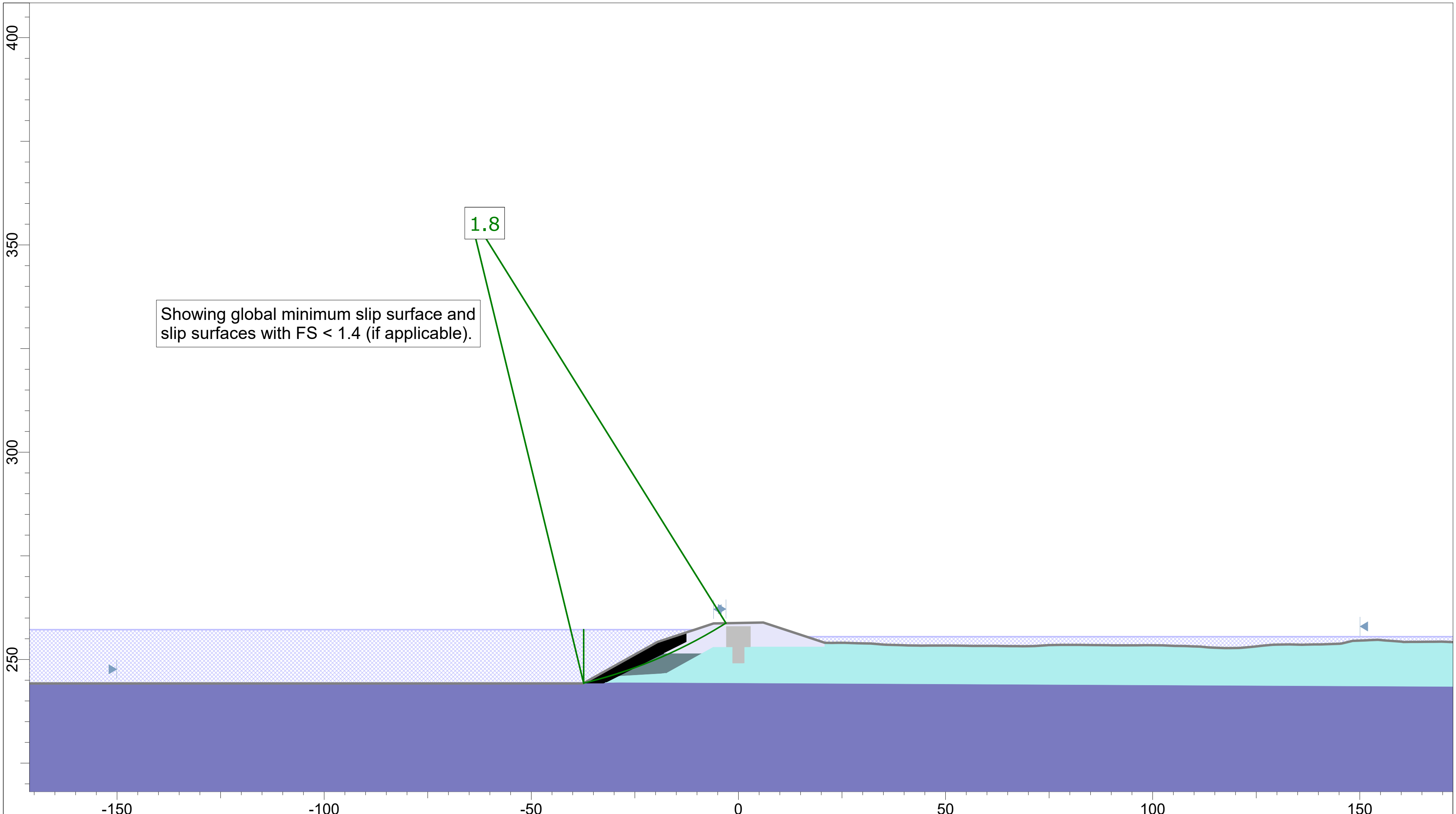
APPENDIX: G-4



Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

A-A' STA 3+50, Long-Term Conditions Waterward, Steady-State Seepage, 100-Yr Flood

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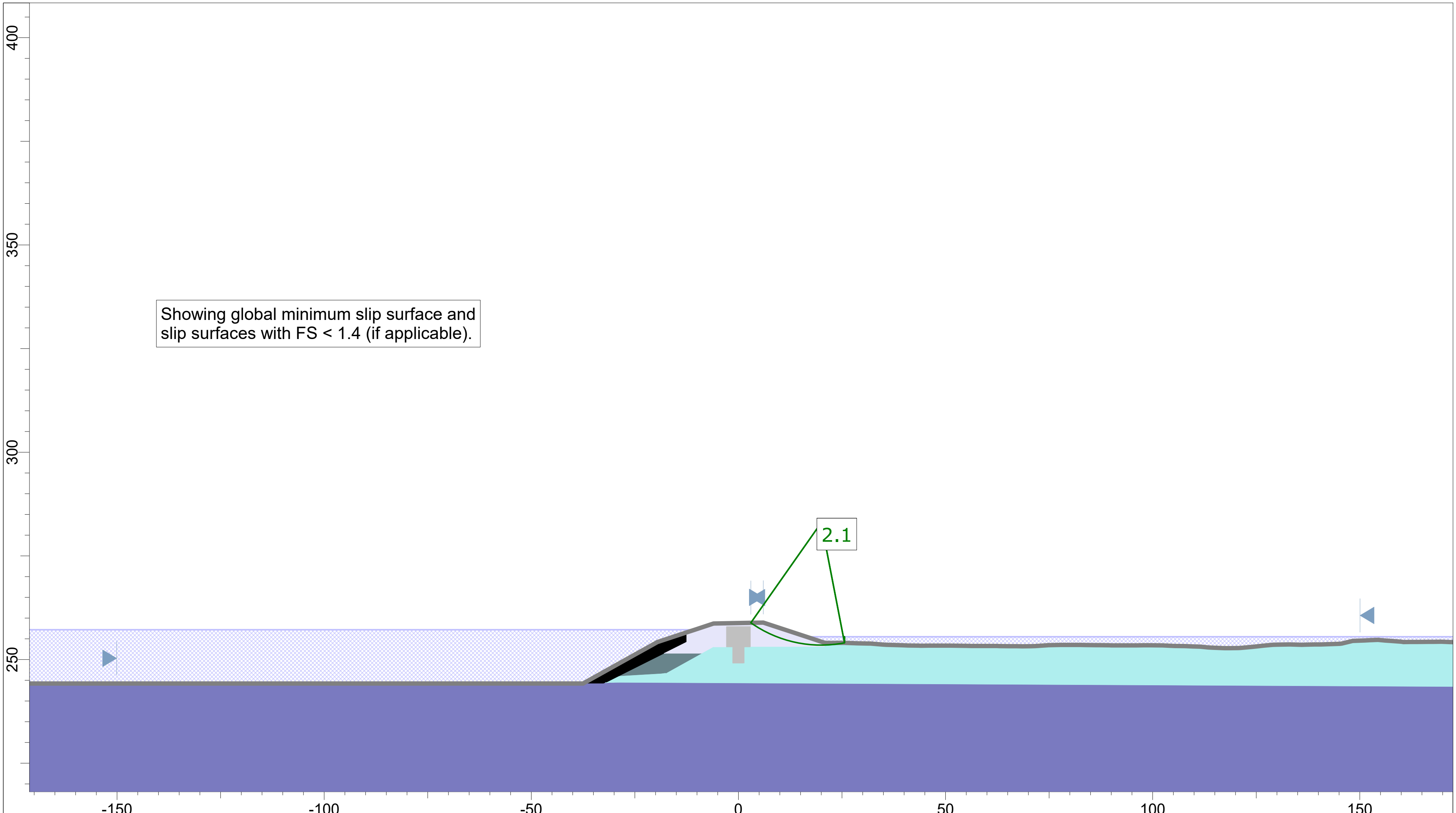

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- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

A-A' STA 3+50, Long-Term Conditions Landward, Steady-State Seepage, 100-Yr Flood

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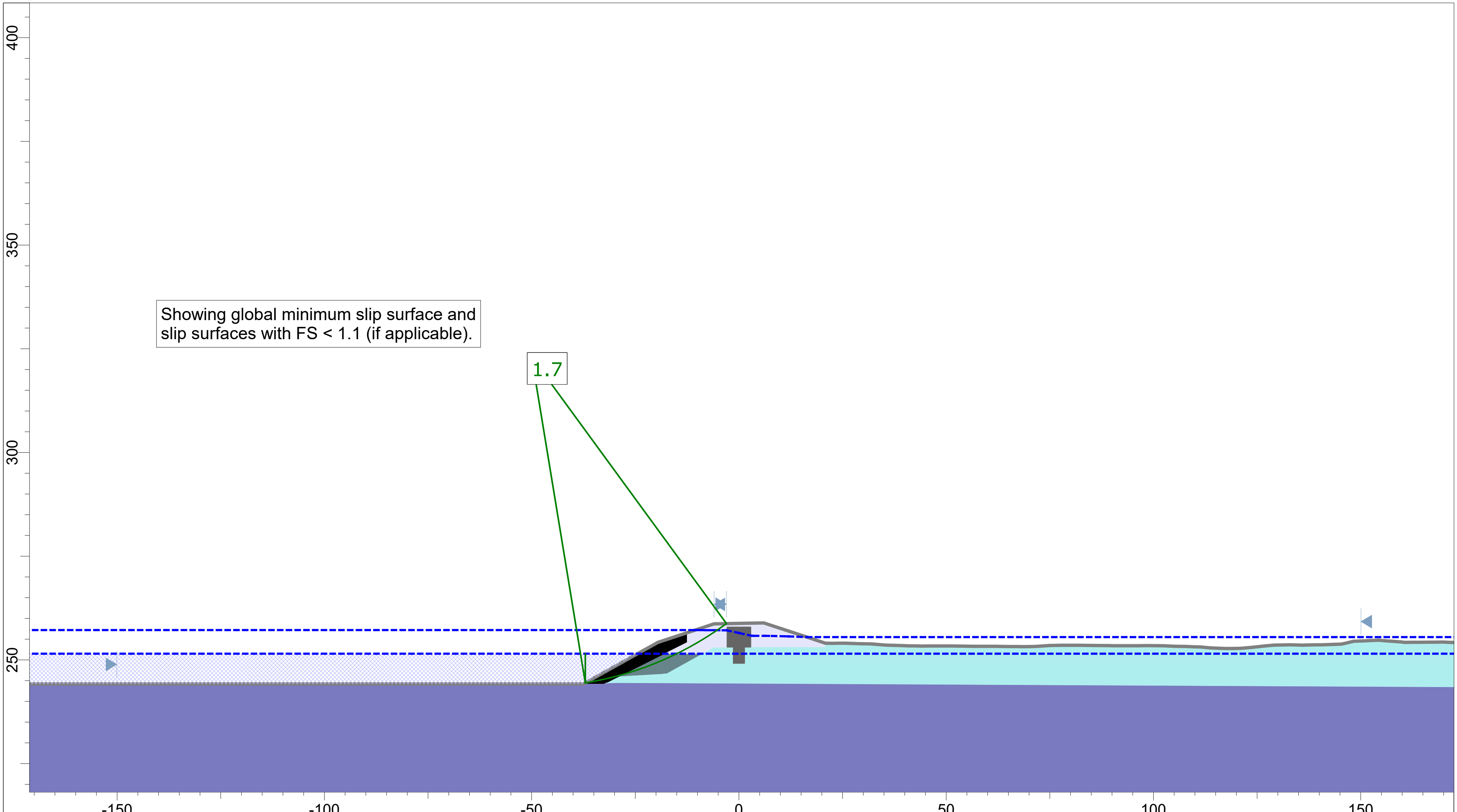

Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

A-A' STA 3+50, Long-Term Conditions Waterward, Sudden Drawdown, 100-Yr Flood

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APPENDIX:
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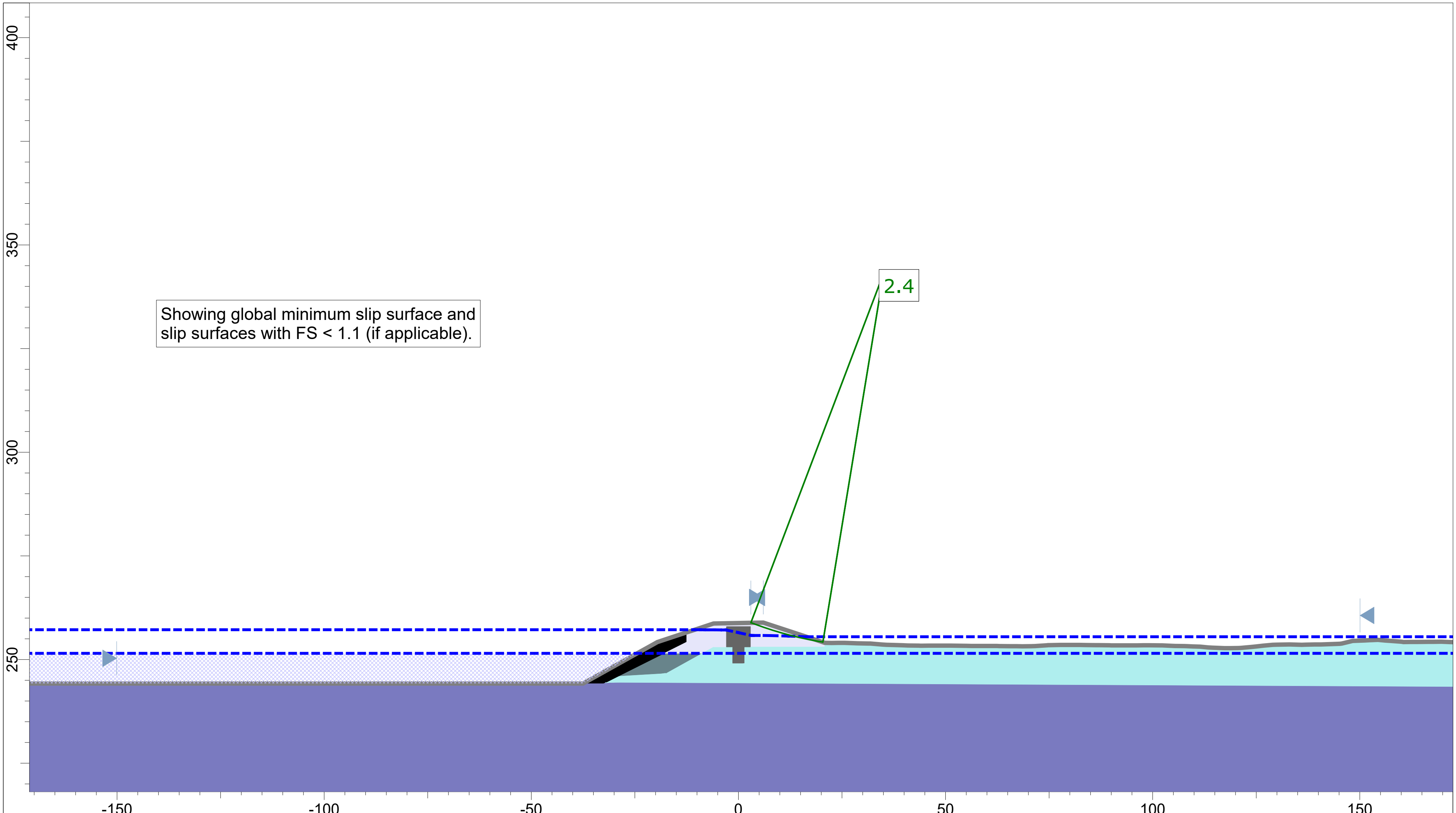

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- Search Grid
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- Modeled Groundwater Level
- Boring Location and Depth

A-A' STA 3+50, Long-Term Conditions Landward, Sudden Drawdown, 100-Yr Flood

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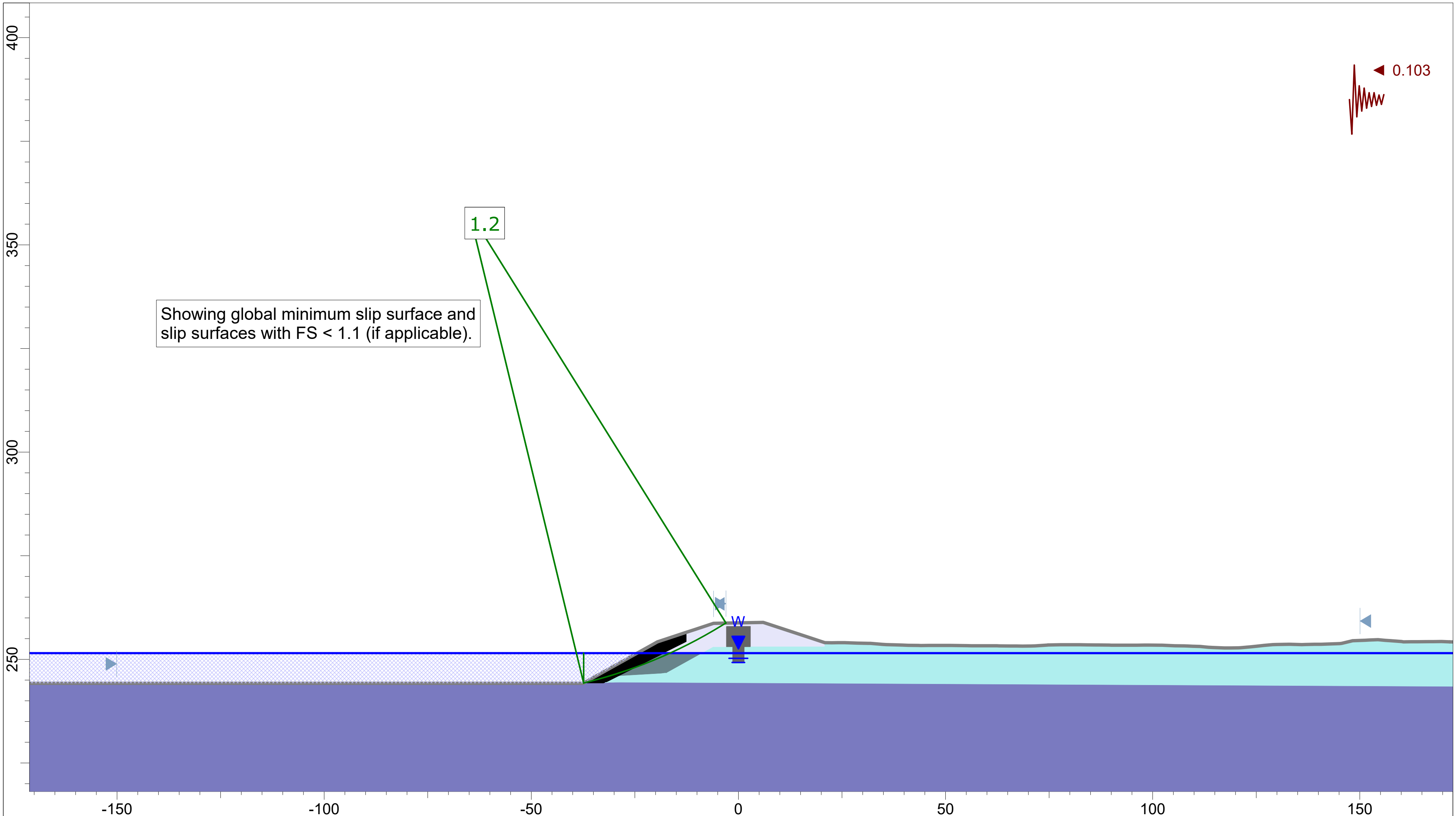

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- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

A-A' STA 3+50, Long-Term Conditions Waterward, Seismic, OBE

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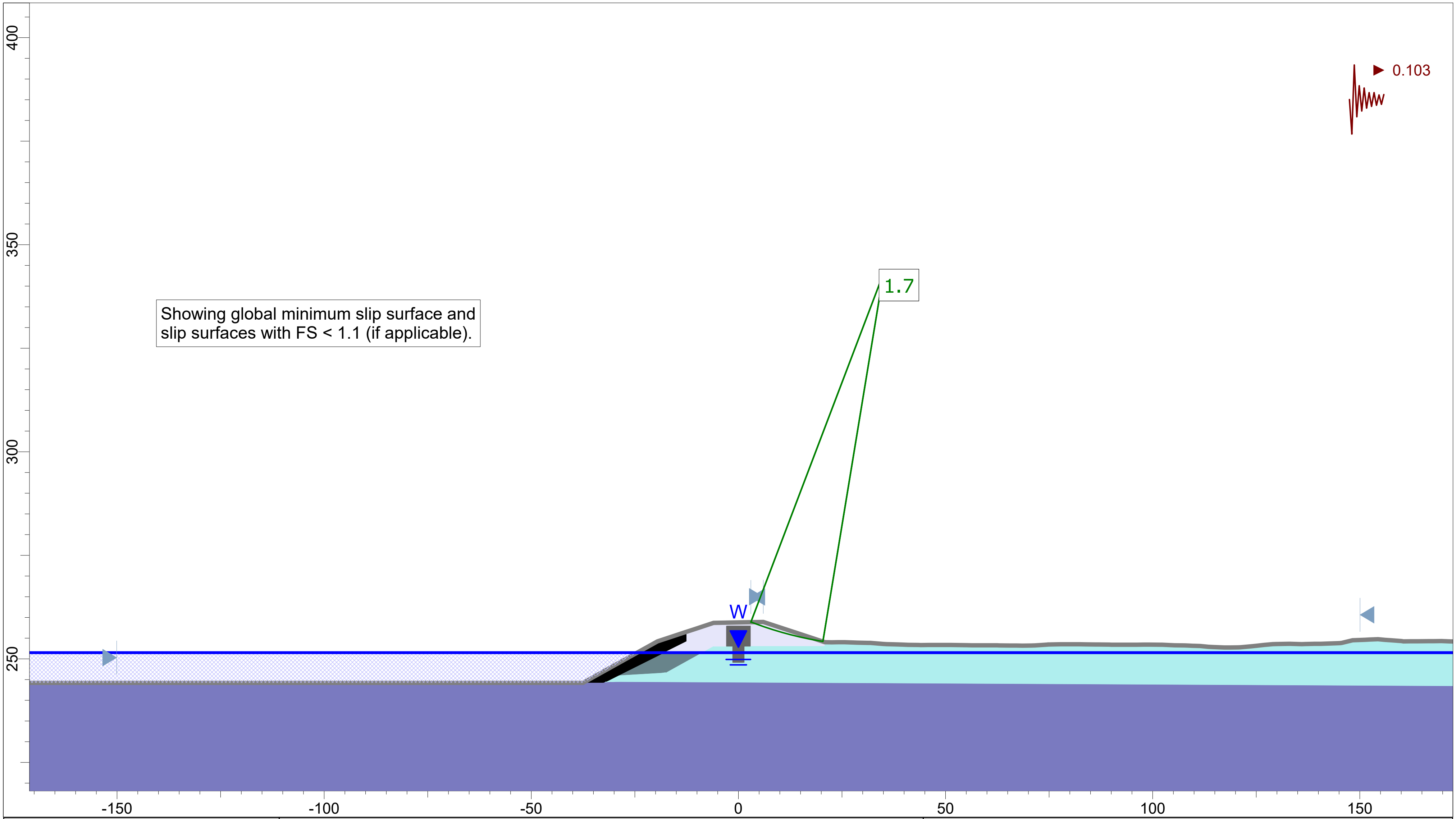

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- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

A-A' STA 3+50, Long-Term Conditions Landward, Seismic, OBE

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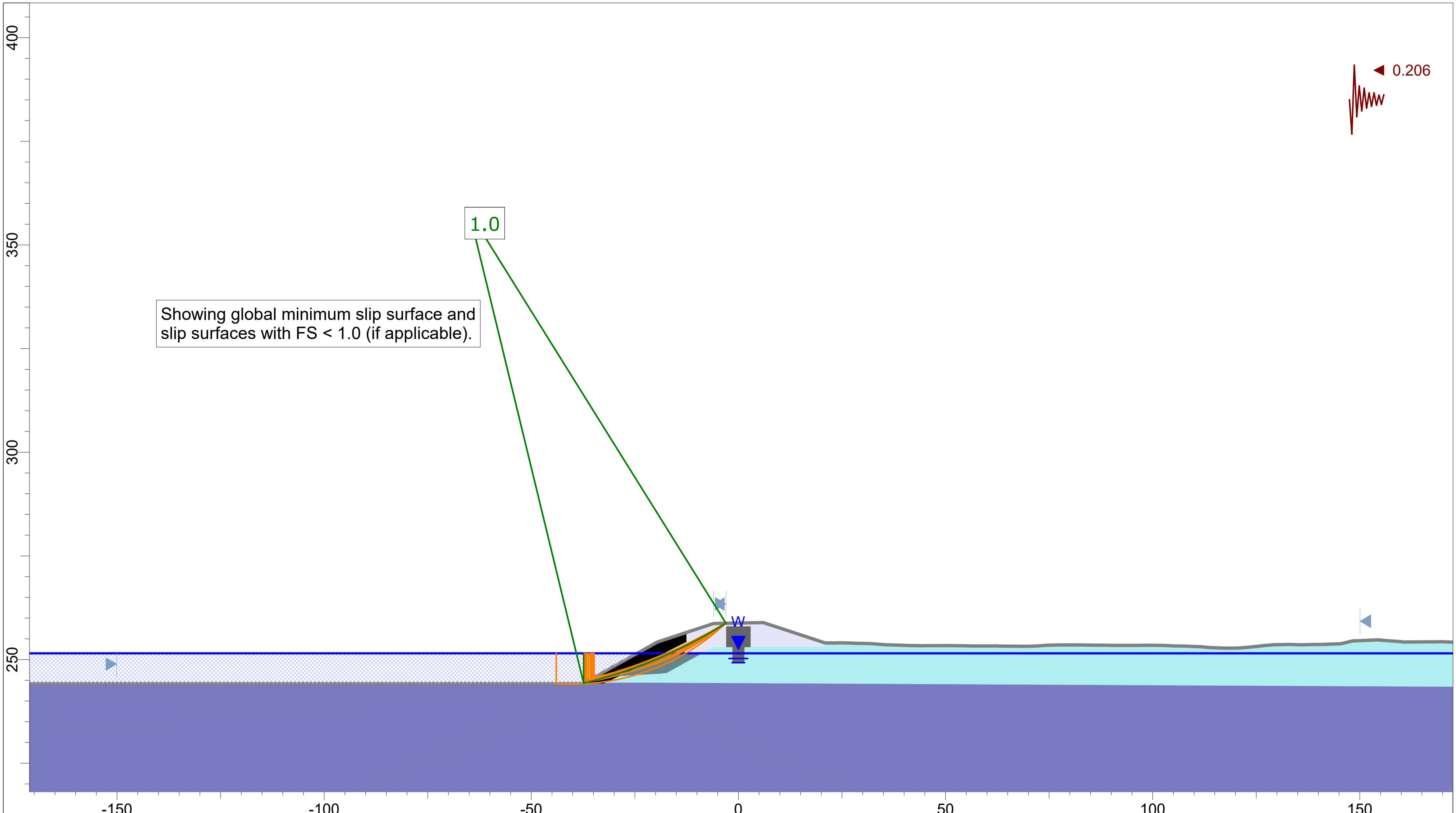

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- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

A-A' STA 3+50, Long-Term Conditions Waterward, Seismic, MDE

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APPENDIX:
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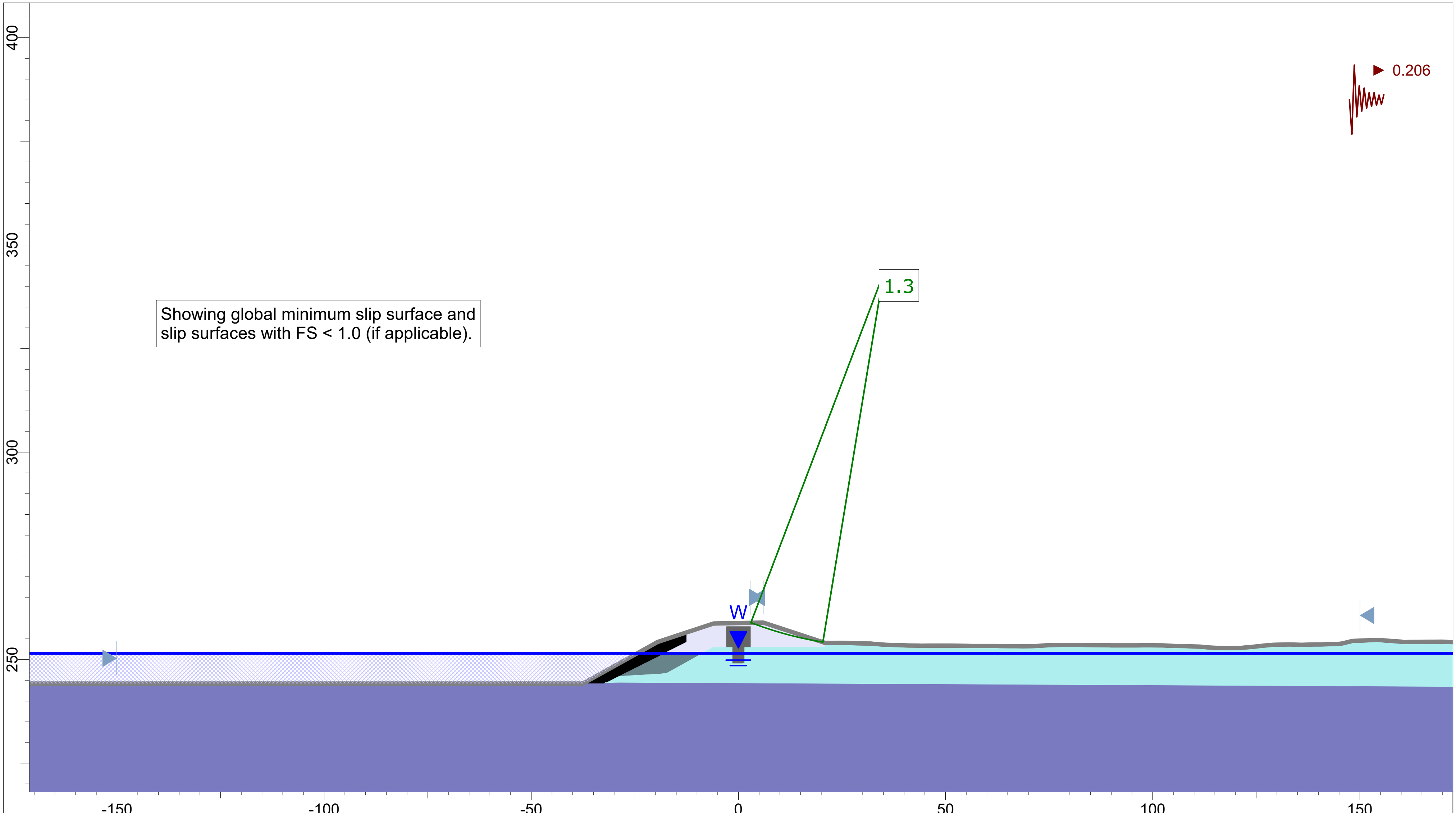

Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

A-A' STA 3+50, Long-Term Conditions Landward, Seismic, MDE

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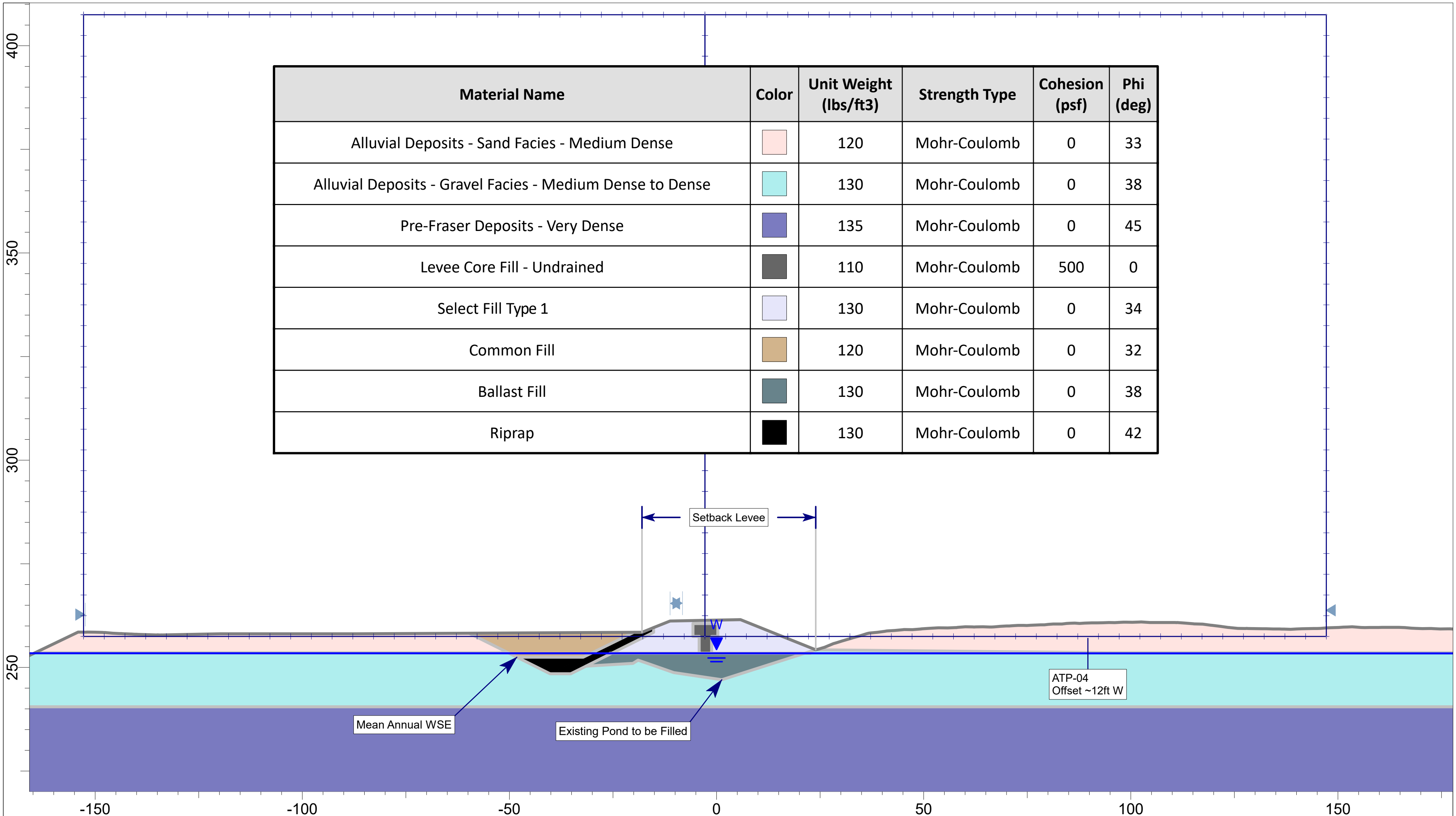
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Alluvial Deposits - Sand Facies - Medium Dense		120	Mohr-Coulomb	0	33
Alluvial Deposits - Gravel Facies - Medium Dense to Dense		130	Mohr-Coulomb	0	38
Pre-Fraser Deposits - Very Dense		135	Mohr-Coulomb	0	45
Levee Core Fill - Undrained		110	Mohr-Coulomb	500	0
Select Fill Type 1		130	Mohr-Coulomb	0	34
Common Fill		120	Mohr-Coulomb	0	32
Ballast Fill		130	Mohr-Coulomb	0	38
Riprap		130	Mohr-Coulomb	0	42



Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

B-B' STA 12+37.30, End of Construction Master Scenario

Slope Stability Analysis

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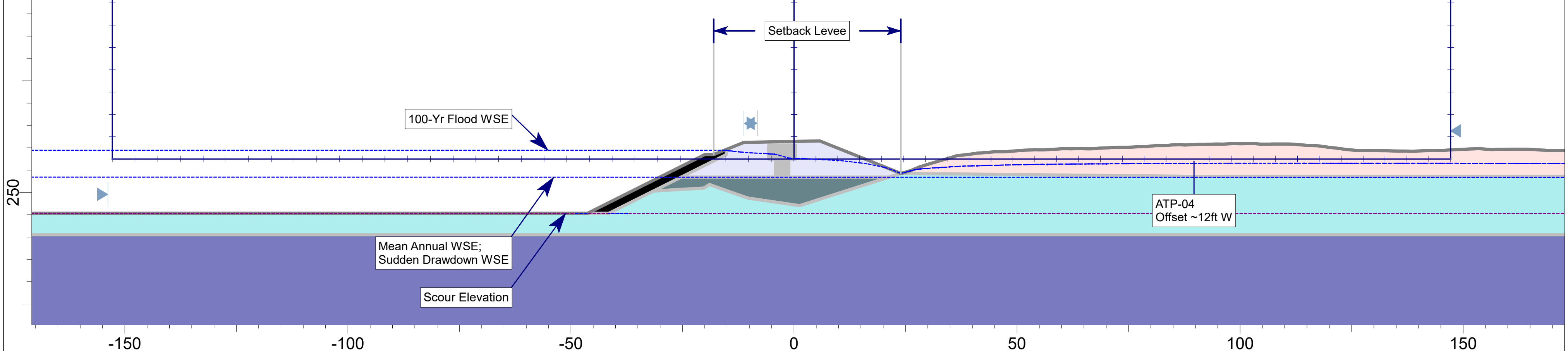
REVIEWED BY: A. Holmson

APPENDIX: G-13

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Alluvial Deposits - Sand Facies - Medium Dense		120	Mohr-Coulomb	0	33
Alluvial Deposits - Gravel Facies - Medium Dense to Dense		130	Mohr-Coulomb	0	38
Pre-Fraser Deposits - Very Dense		135	Mohr-Coulomb	0	45
Levee Core Fill - Drained		110	Mohr-Coulomb	100	28
Levee Core Fill - Undrained		110	Mohr-Coulomb	500	0
Select Fill Type 1		130	Mohr-Coulomb	0	34
Ballast Fill		130	Mohr-Coulomb	0	38
Riprap		130	Mohr-Coulomb	0	42



Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

B-B' STA 12+37.30, Long-Term Conditions Master Scenario

Slope Stability Analysis

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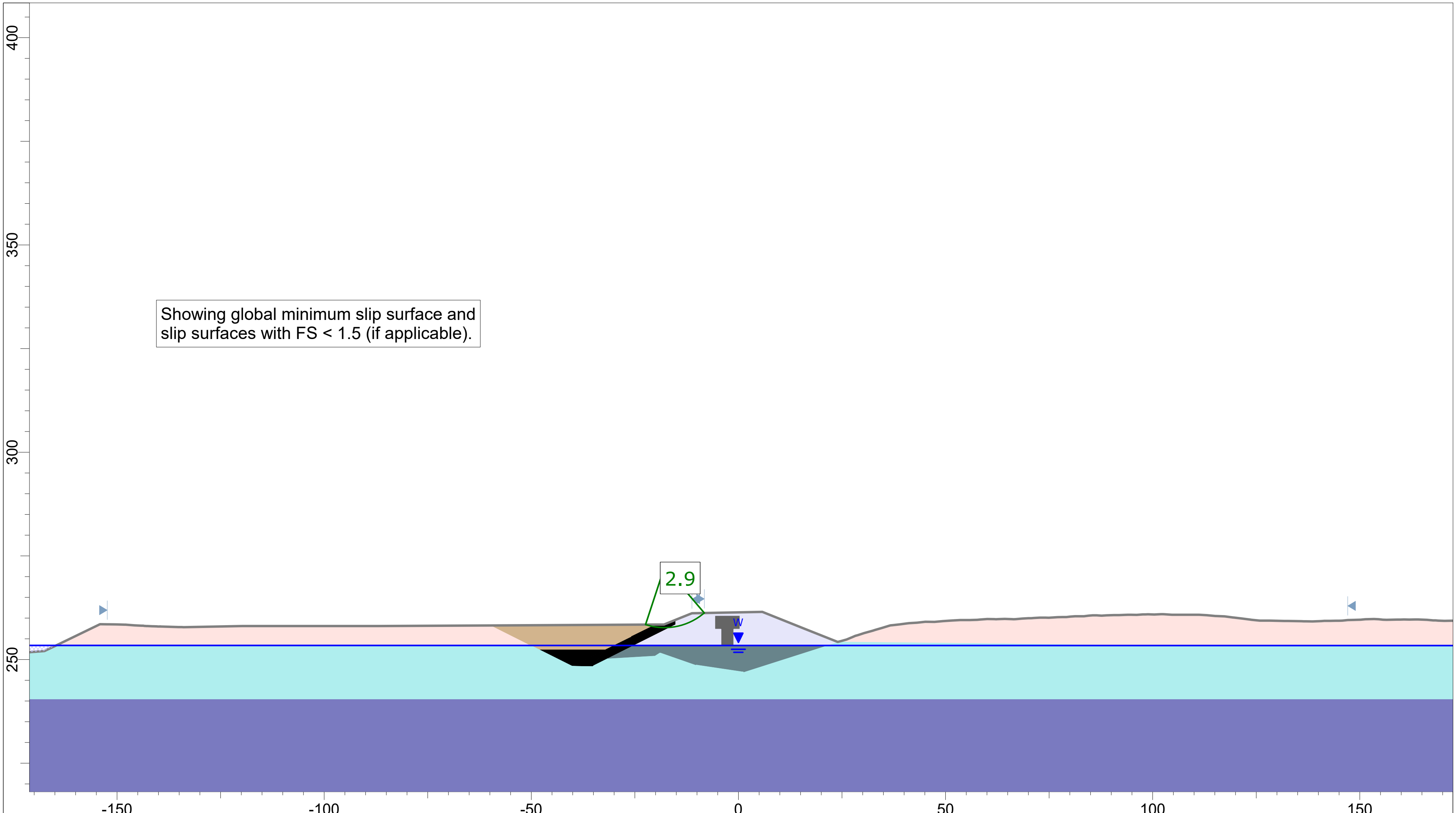

Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

B-B' STA 12+37.30, End of Construction Waterward, Mean Annual WSE

Slope Stability Analysis

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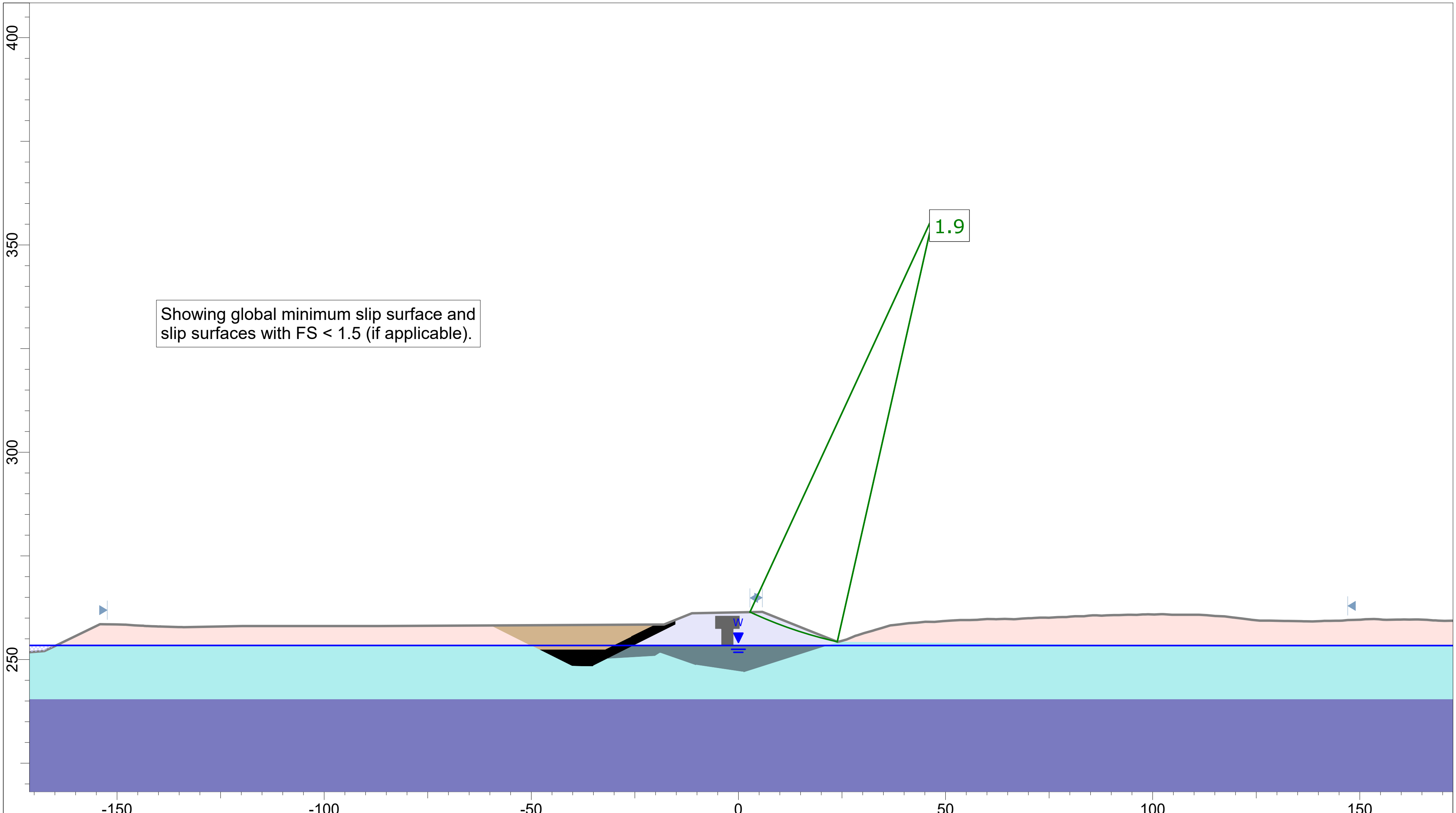

Legend

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- Modeled Groundwater Level
- Boring Location and Depth

B-B' STA 12+37.30, End of Construction Landward, Mean Annual WSE

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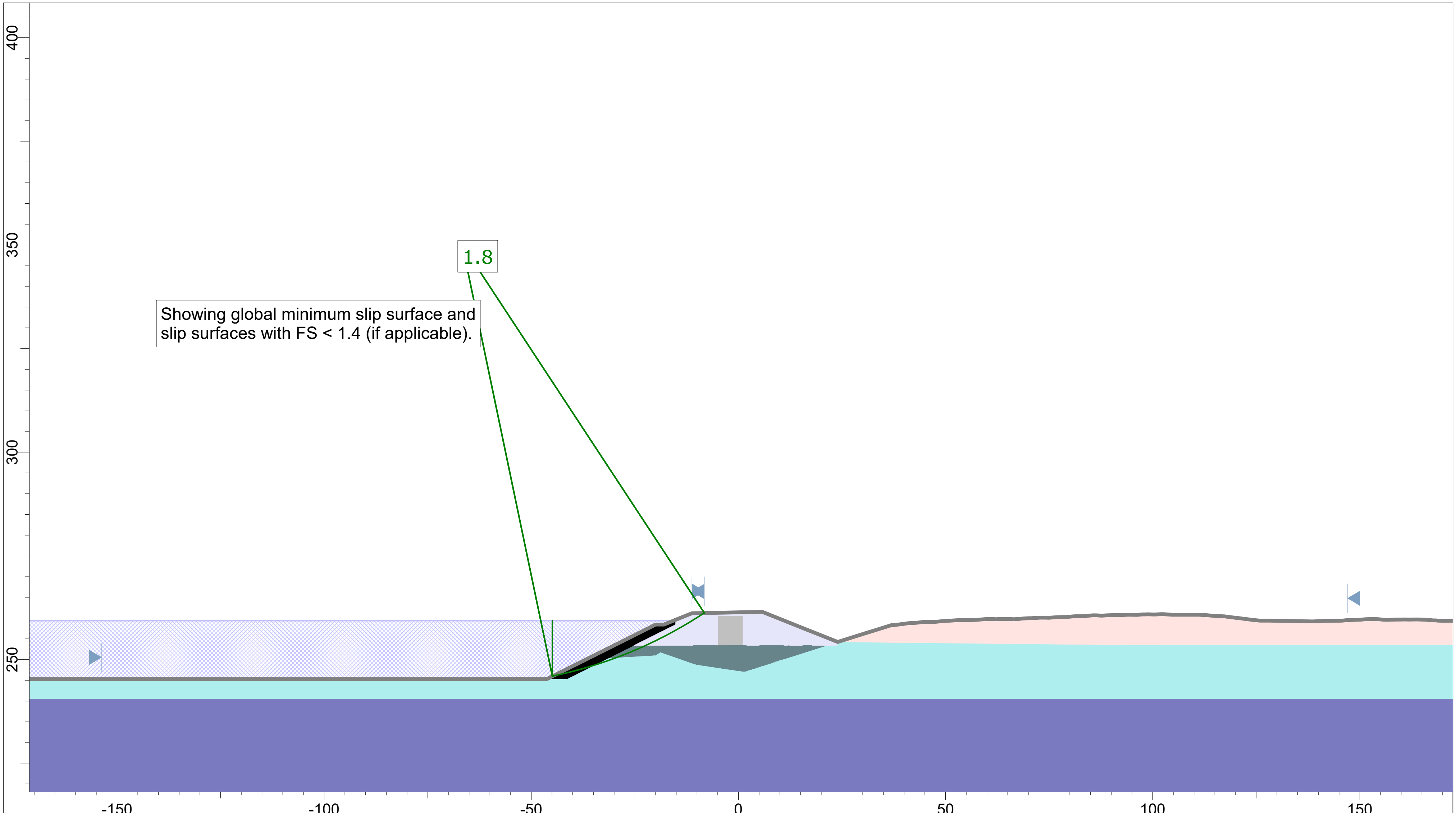

Legend

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- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

B-B' STA 12+37.30, Long-Term Conditions Waterward, Steady-State Seepage, 100-Yr Flood

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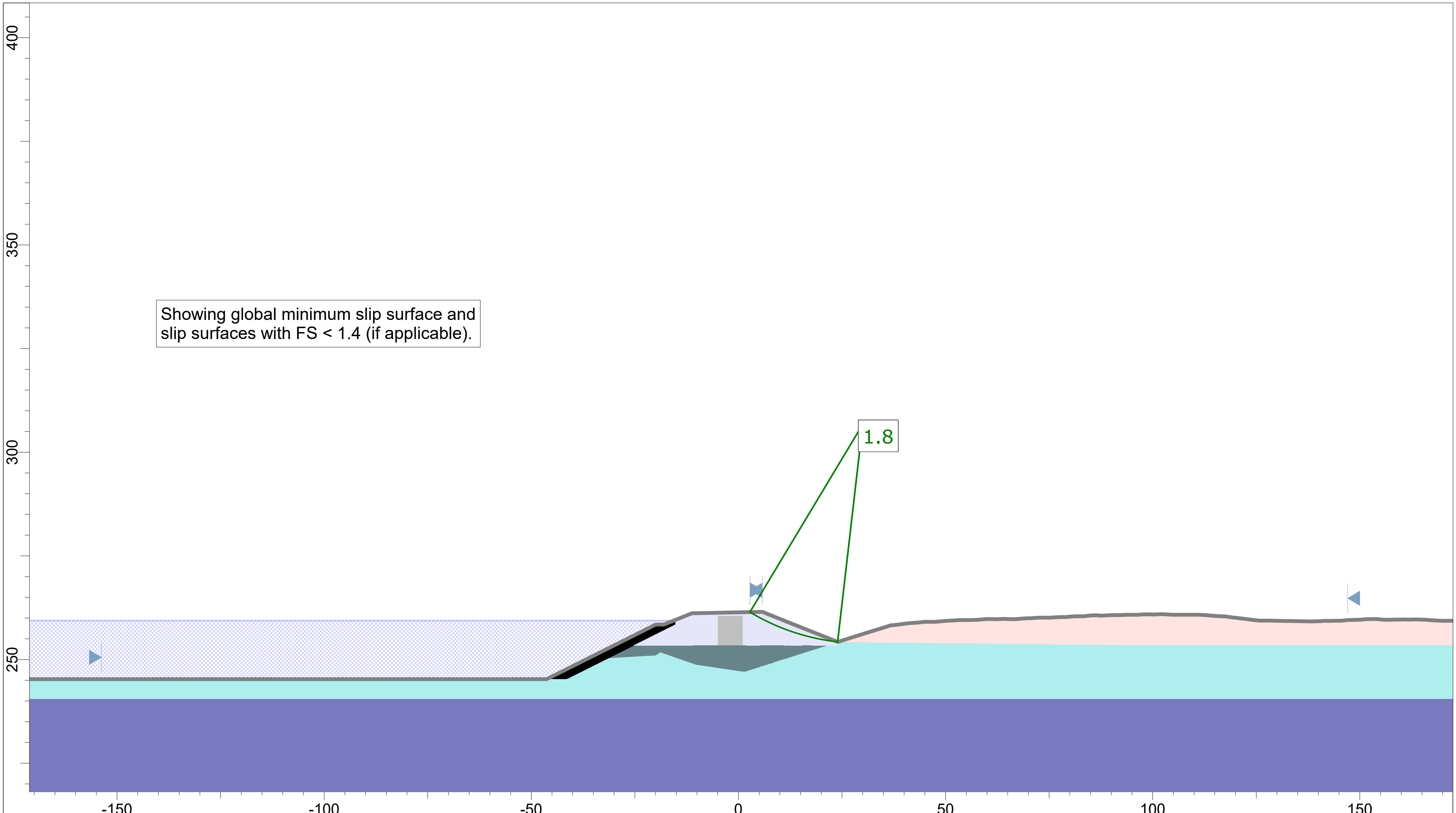

Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

B-B' STA 12+37.30, Long-Term Conditions Landward, Steady-State Seepage, 100-Yr Flood

Slope Stability Analysis

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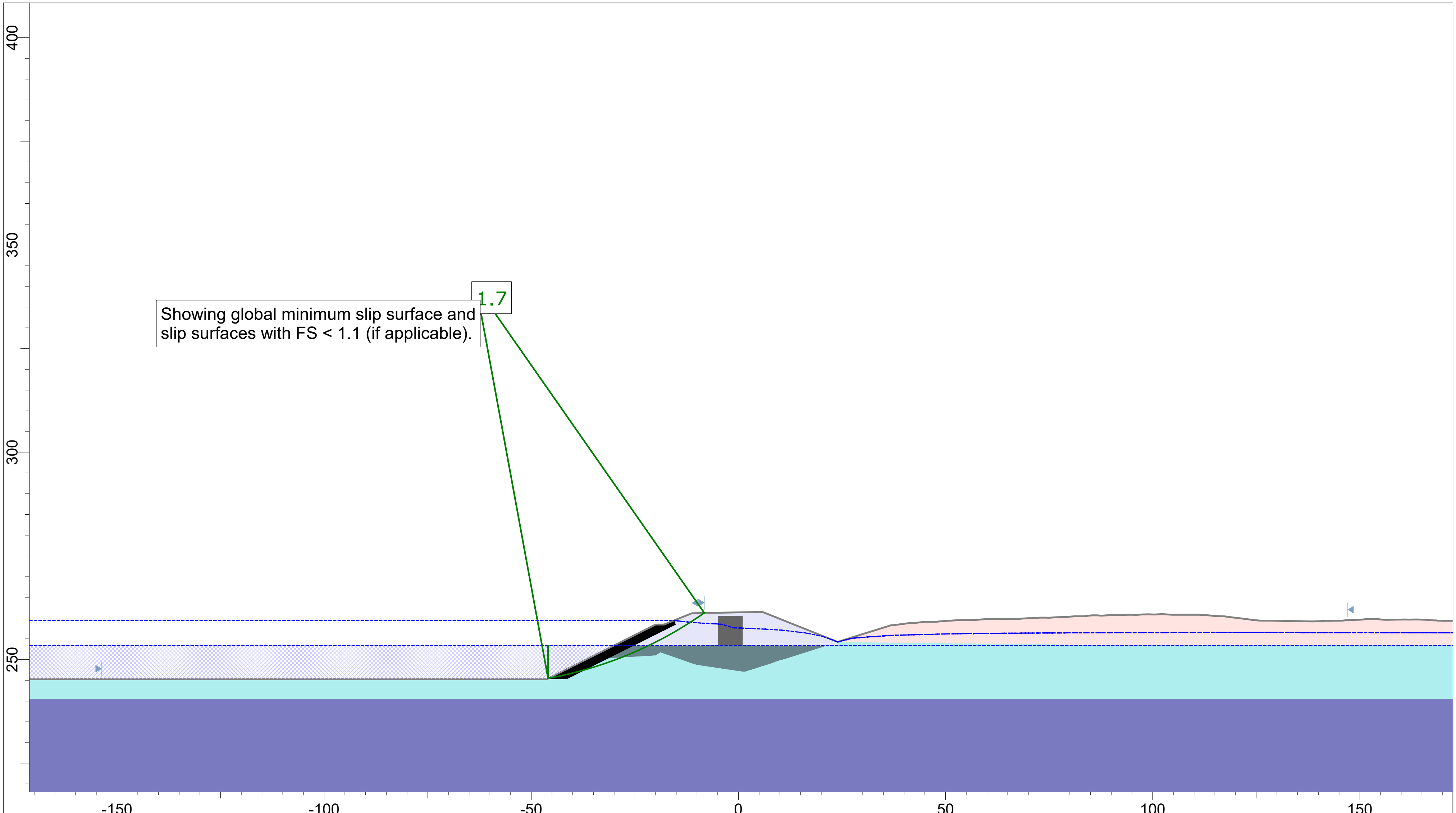
REVIEWED BY: A. Holmson

APPENDIX: **G-18**



Legend

- Search Grid
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- Modeled Groundwater Level
- Boring Location and Depth

B-B' STA 12+37.30, Long-Term Conditions Waterward, Sudden Drawdown, 100-Yr Flood

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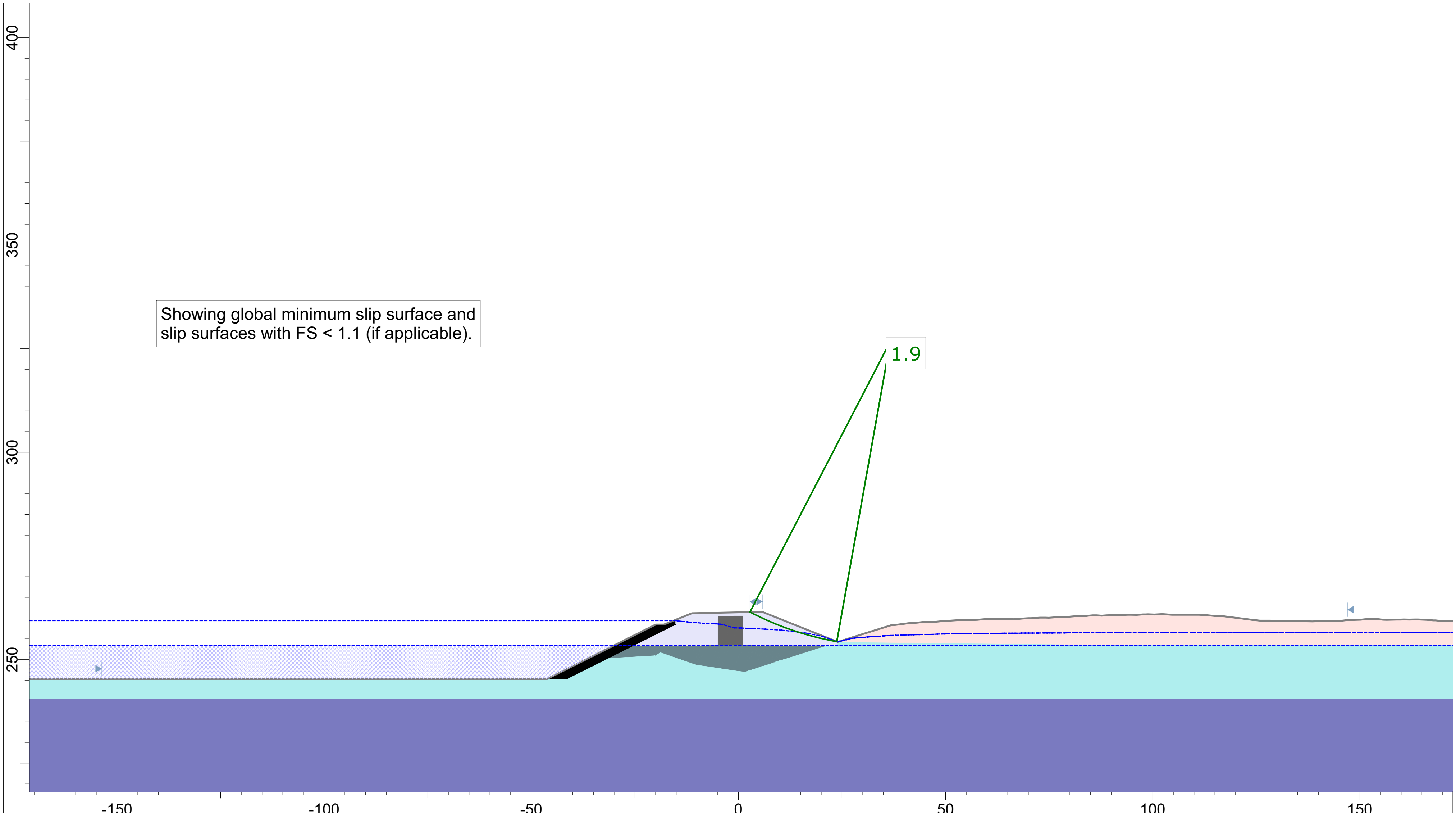

Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

B-B' STA 12+37.30, Long-Term Conditions Landward, Sudden Drawdown, 100-Yr Flood

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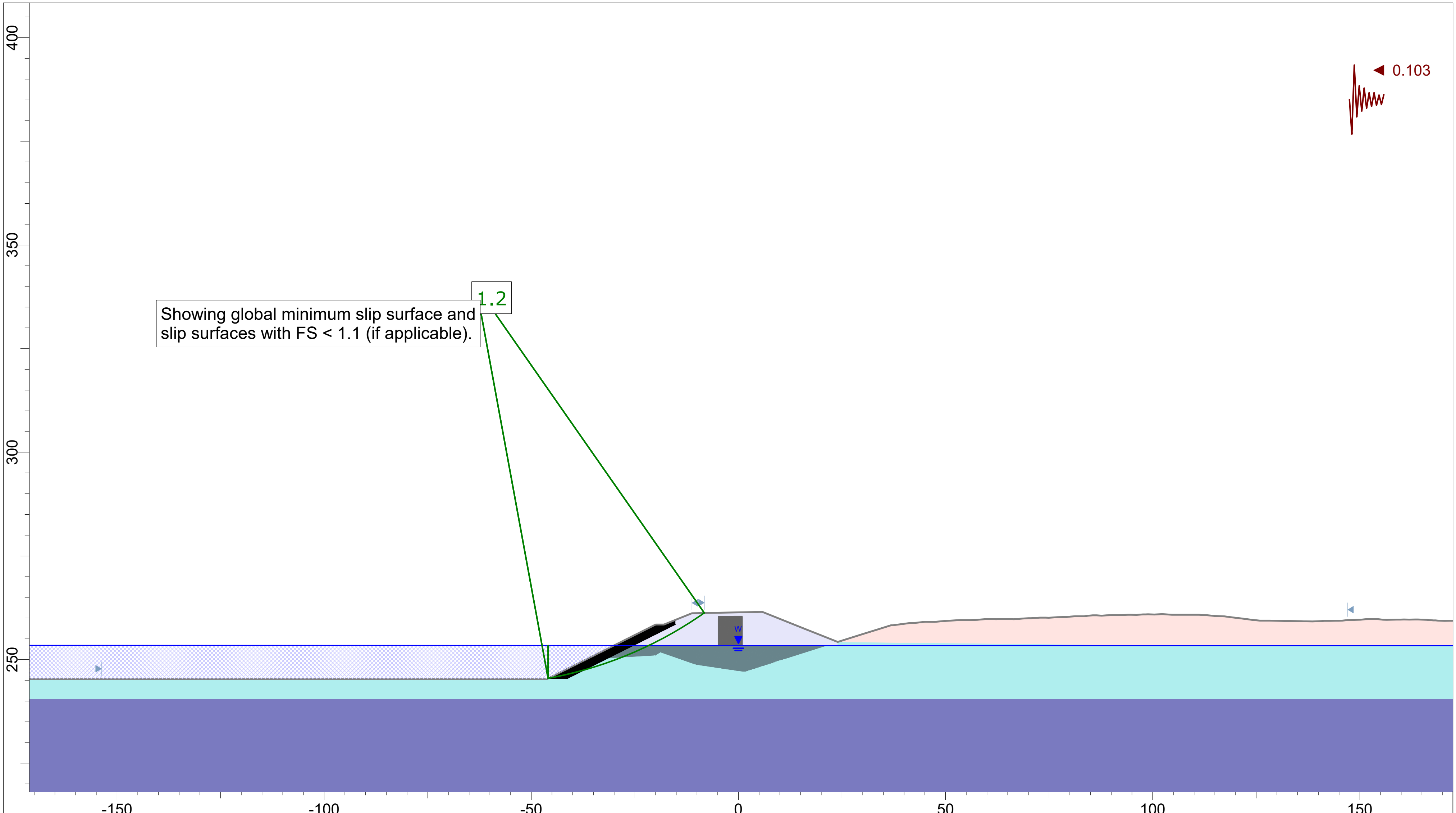
1/28/2022	BY: M. Otto	APPENDIX:
PROJECT NO. 190175-A	REVIEWED BY: A. Holmson	G-20



Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

B-B' STA 12+37.30, Long-Term Conditions Waterward, Seismic, OBE

Slope Stability Analysis

Geotechnical Design Report
Jan Road Neighborhood Improvements
King County, Washington

SLIDEINTERPRET 8.032

SCALE: 1:400

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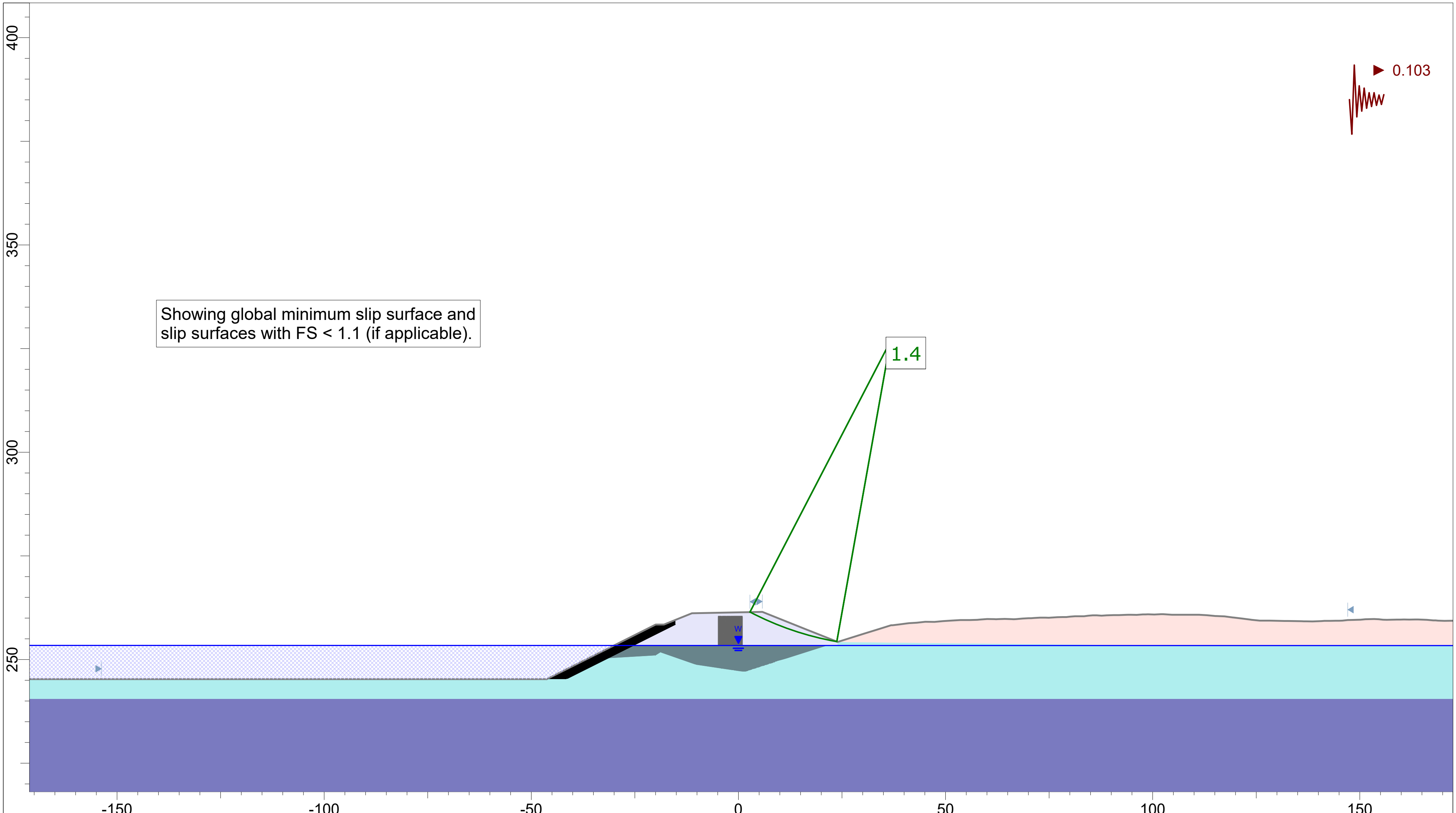
1/28/2022	BY: M. Otto	APPENDIX:
PROJECT NO. 190175-A	REVIEWED BY: A. Holmson	G-21



Legend

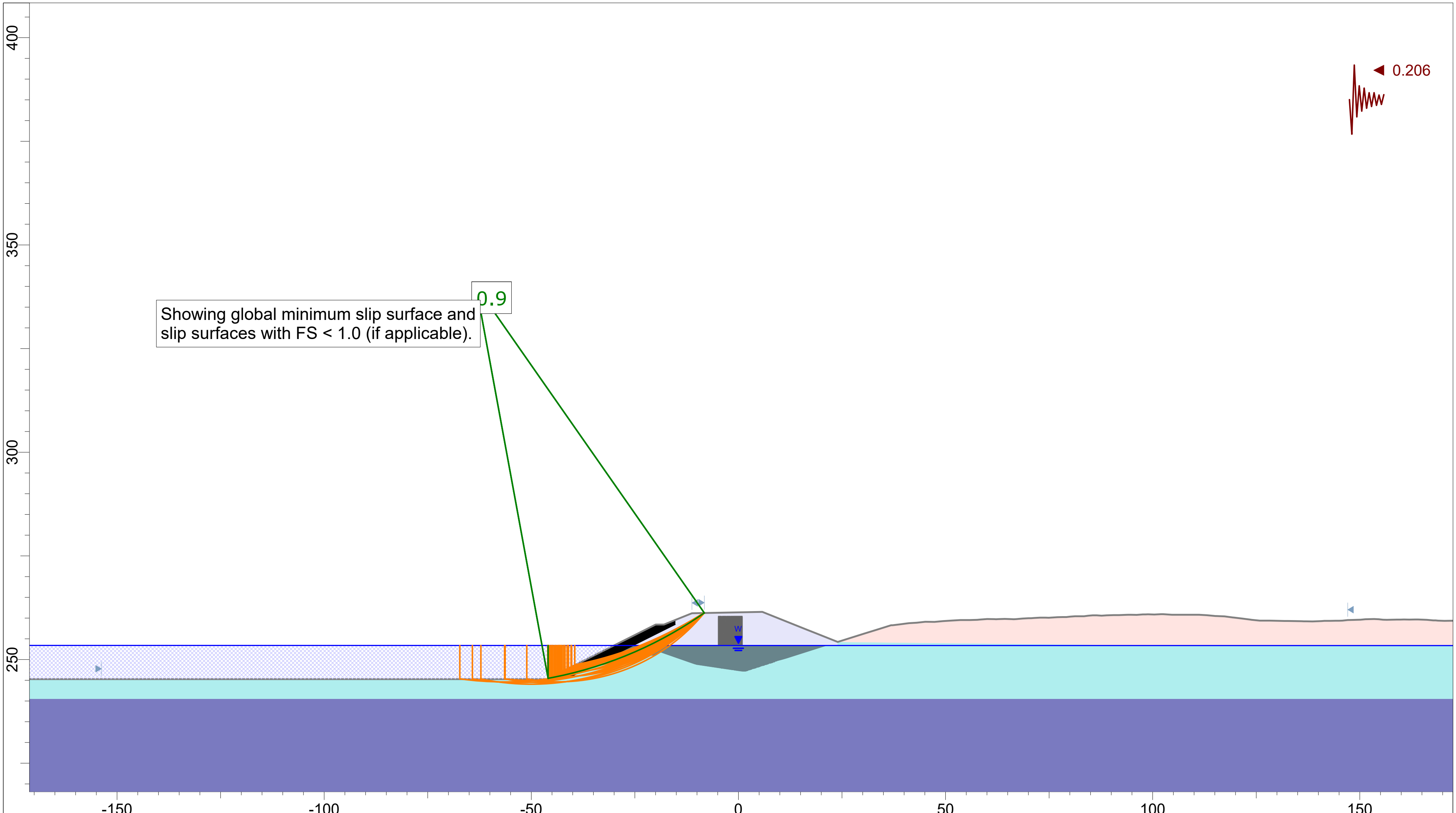
- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

B-B' STA 12+37.30, Long-Term Conditions Landward, Seismic, OBE

Slope Stability Analysis

Geotechnical Design Report
Jan Road Neighborhood Improvements
King County, Washington

SLIDEINTERPRET 8.032	SCALE: 1:400 C:\Users\motto\OneDrive - Aspect Consulting LLC\Slide\2022.01.28_B-B' STA 12+37, Long-Term Conditions.slmd	Aspect CONSULTING	1/28/2022 PROJECT NO. 190175-A	BY: M. Otto REVIEWED BY: A. Holmson	APPENDIX: **G-22**



Legend

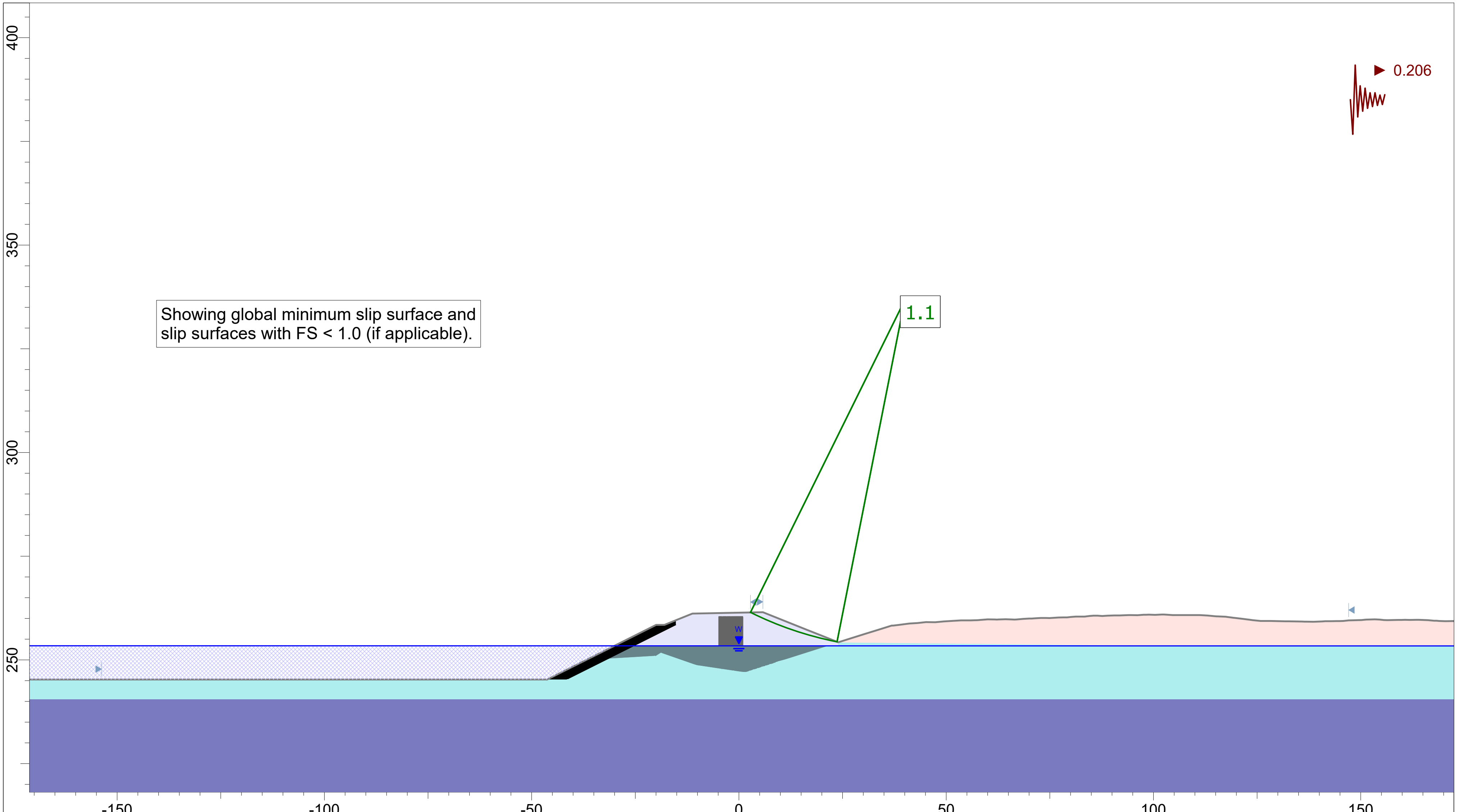
- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

B-B' STA 12+37.30, Long-Term Conditions Waterward, Seismic, MDE

Slope Stability Analysis

Geotechnical Design Report
Jan Road Neighborhood Improvements
King County, Washington

SLIDEINTERPRET 8.032	SCALE: 1:400 C:\Users\motto\OneDrive - Aspect Consulting LLC\Slide\2022.01.28_B-B' STA 12+37, Long-Term Conditions.slmd	Aspect CONSULTING	1/28/2022 PROJECT NO. 190175-A	BY: M. Otto REVIEWED BY: A. Holmson	APPENDIX: **G-23**



Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

B-B' STA 12+37.30, Long-Term Conditions Landward, Seismic, MDE

Slope Stability Analysis

Geotechnical Design Report
Jan Road Neighborhood Improvements
King County, Washington

SLIDEINTERPRET 8.032

SCALE: 1:400

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Aspect CONSULTING

1/28/2022

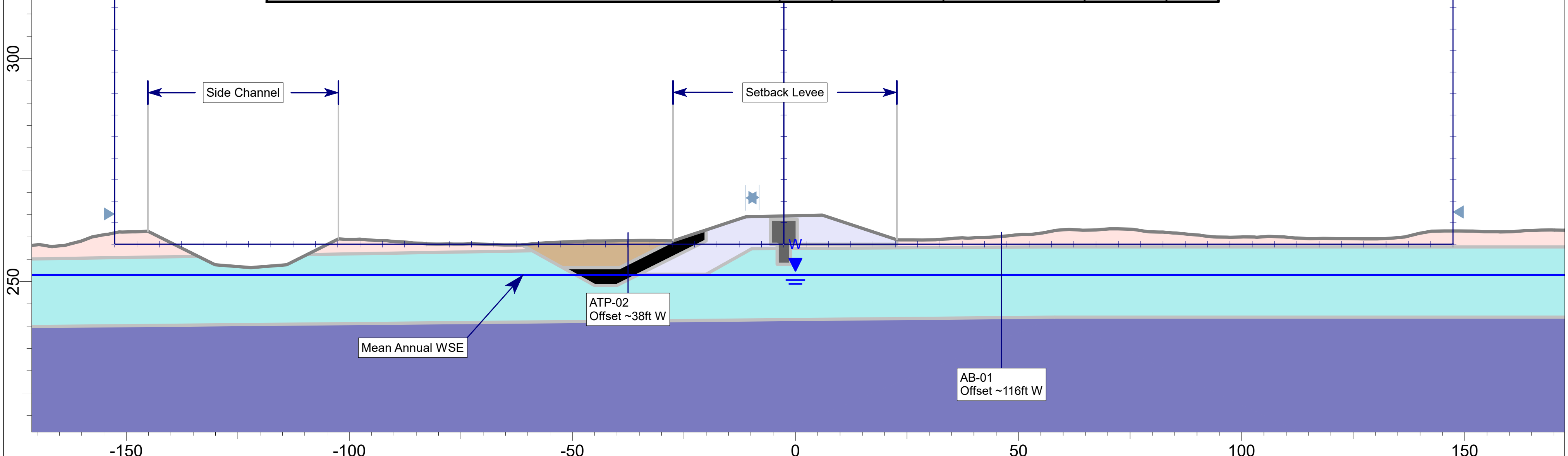
PROJECT NO. 190175-A

BY: M. Otto

REVIEWED BY: A. Holmson

APPENDIX: **G-24**

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Alluvial Deposits - Sand Facies - Medium Dense		120	Mohr-Coulomb	0	33
Alluvial Deposits - Gravel Facies - Medium Dense to Dense		130	Mohr-Coulomb	0	38
Pre-Fraser Deposits - Very Dense		135	Mohr-Coulomb	0	45
Levee Core Fill - Undrained		110	Mohr-Coulomb	500	0
Select Fill Type 1		130	Mohr-Coulomb	0	34
Common Fill		120	Mohr-Coulomb	0	32
Riprap		130	Mohr-Coulomb	0	42



Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

C-C' STA 16+25, End of Construction
Master Scenario

Slope Stability Analysis

Geotechnical Design Report
Jan Road Neighborhood Improvements
King County, Washington

SLIDE 8.032

SCALE: 1:400

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Aspect CONSULTING

1/28/2022

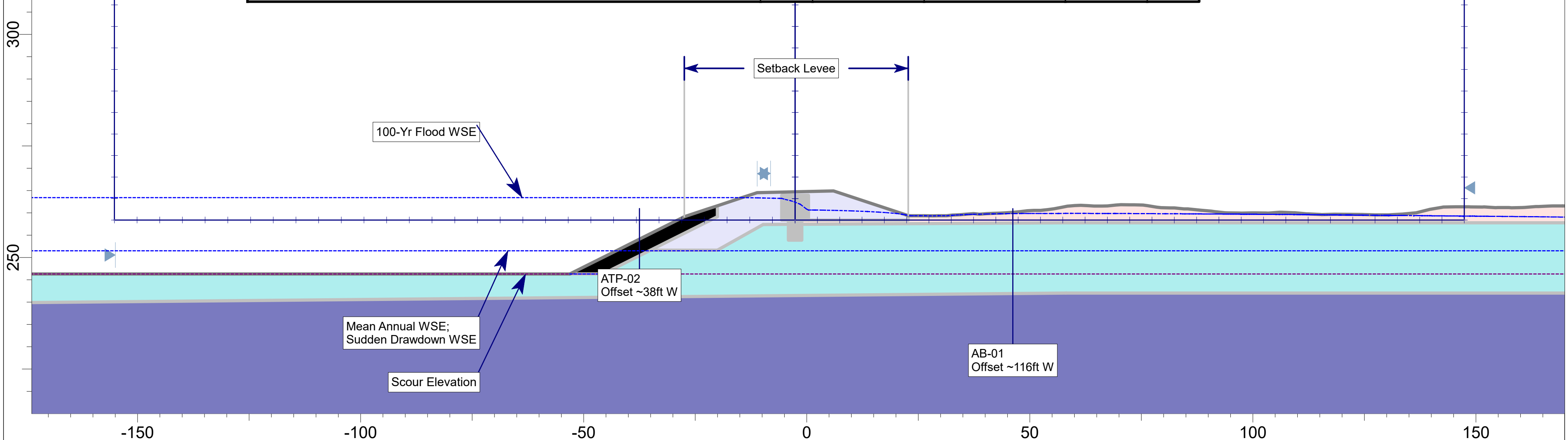
PROJECT NO. 190175-A

BY: M. Otto

REVIEWED BY: A. Holmson

APPENDIX: G-25

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Alluvial Deposits - Sand Facies - Medium Dense		120	Mohr-Coulomb	0	33
Alluvial Deposits - Gravel Facies - Medium Dense to Dense		130	Mohr-Coulomb	0	38
Pre-Fraser Deposits - Very Dense		135	Mohr-Coulomb	0	45
Levee Core Fill - Drained		110	Mohr-Coulomb	100	28
Levee Core Fill - Undrained		110	Mohr-Coulomb	500	0
Select Fill Type 1		130	Mohr-Coulomb	0	34
Riprap		130	Mohr-Coulomb	0	42



Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

C-C' STA 16+25, Long-Term Conditions
Master Scenario

Slope Stability Analysis

Geotechnical Design Report
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King County, Washington

SLIDE 8.032

SCALE: 1:400

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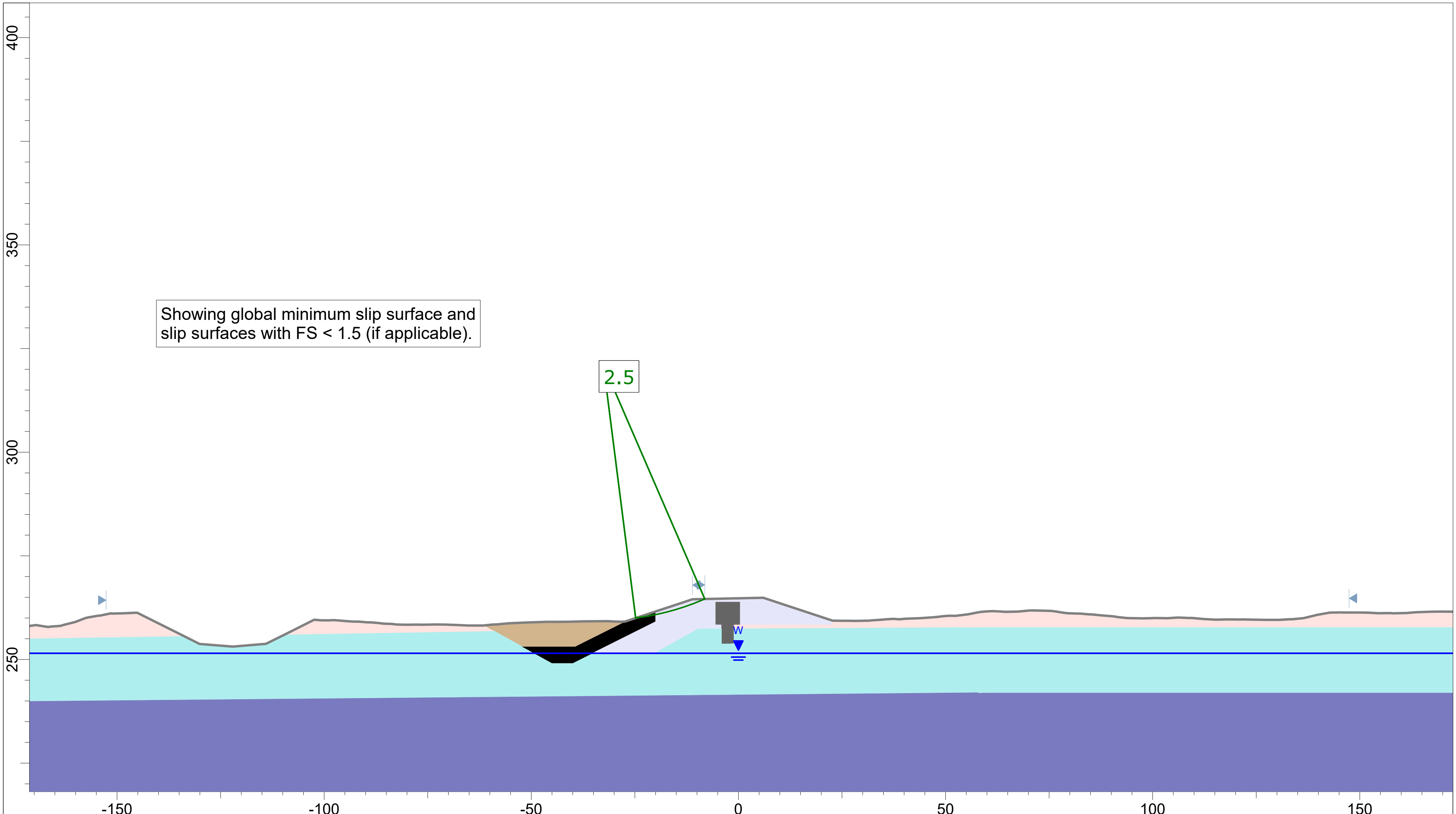
1/28/2022

PROJECT NO.
190175-A

BY:
M. Otto

REVIEWED BY:
A. Holmson

APPENDIX:
G-26



Legend

Search Grid

Search Limits

Modeled Groundwater Level

Boring Location and Depth

C-C' STA 16+25, End of Construction Waterward, Mean Annual WSE

Slope Stability Analysis

Geotechnical Design Report
Jan Road Neighborhood Improvements
King County, Washington

SLIDEINTERPRET 8.032

SCALE: 1:400

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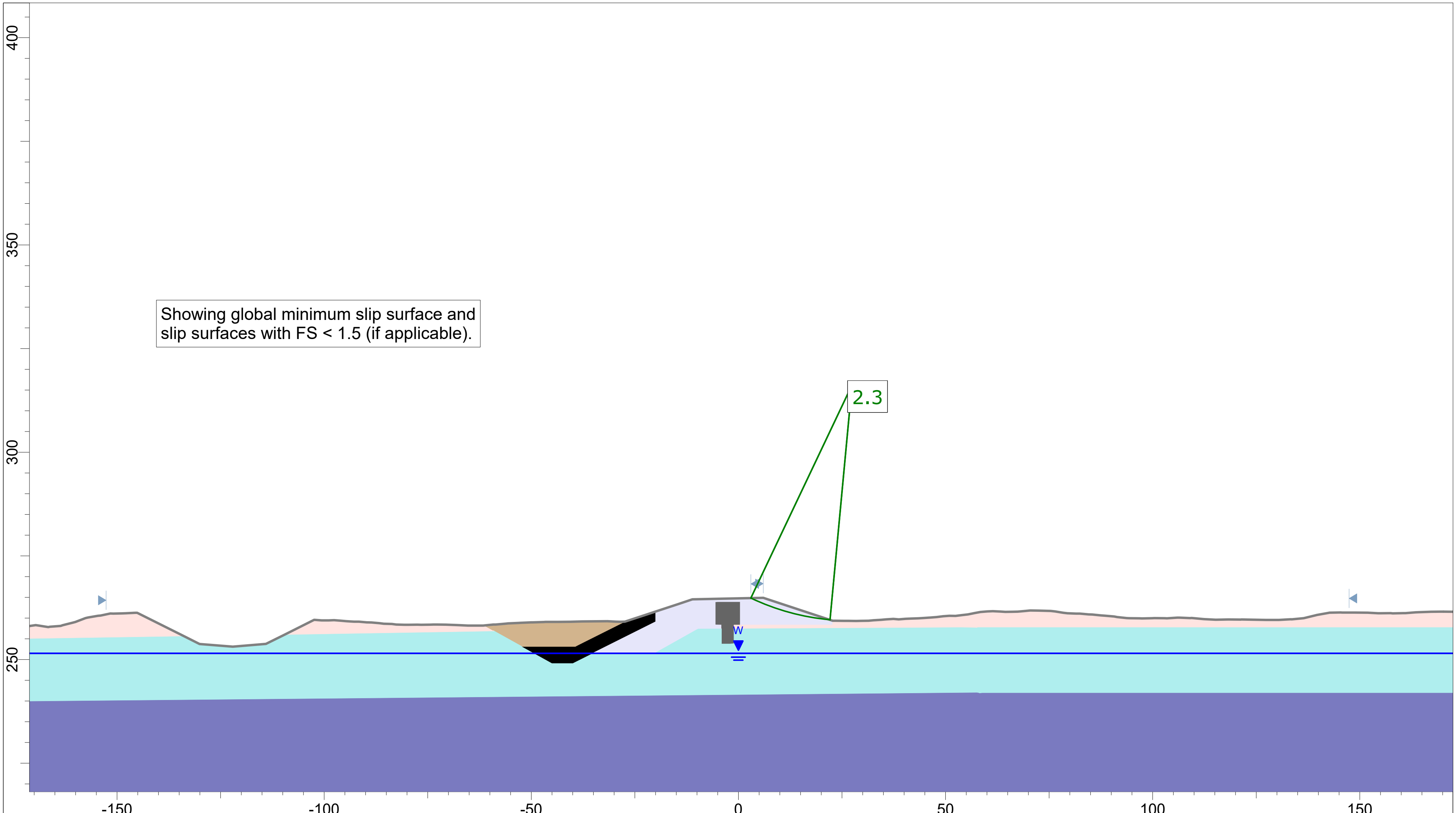
1/28/2022

PROJECT NO.
190175

BY:
M. Otto

REVIEWED BY:
A. Holmson

APPENDIX:
G-27



Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

C-C' STA 16+25, End of Construction Landward, Mean Annual WSE

Slope Stability Analysis

Geotechnical Design Report
Jan Road Neighborhood Improvements
King County, Washington

SLIDEINTERPRET 8.032

SCALE: 1:400

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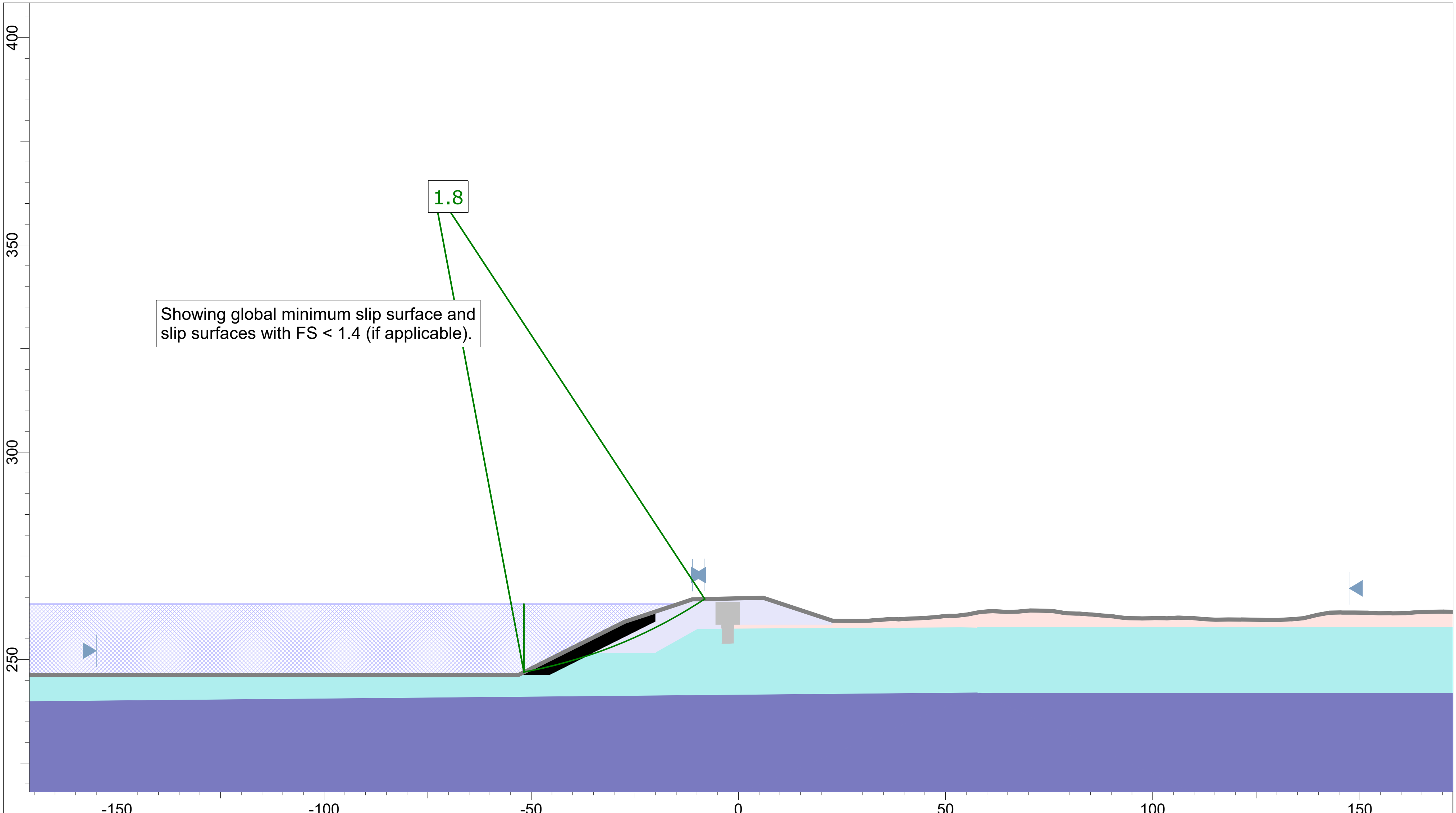
Aspect CONSULTING

1/28/2022	BY: M. Otto	APPENDIX:
PROJECT NO. 190175	REVIEWED BY: A. Holmson	G-28



Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

C-C' STA 16+25, Long-Term Conditions Waterward, Steady-State Seepage, 100-Yr Flood

Slope Stability Analysis

Geotechnical Design Report
Jan Road Neighborhood Improvements
King County, Washington

SLIDEINTERPRET 8.032

SCALE: 1:400

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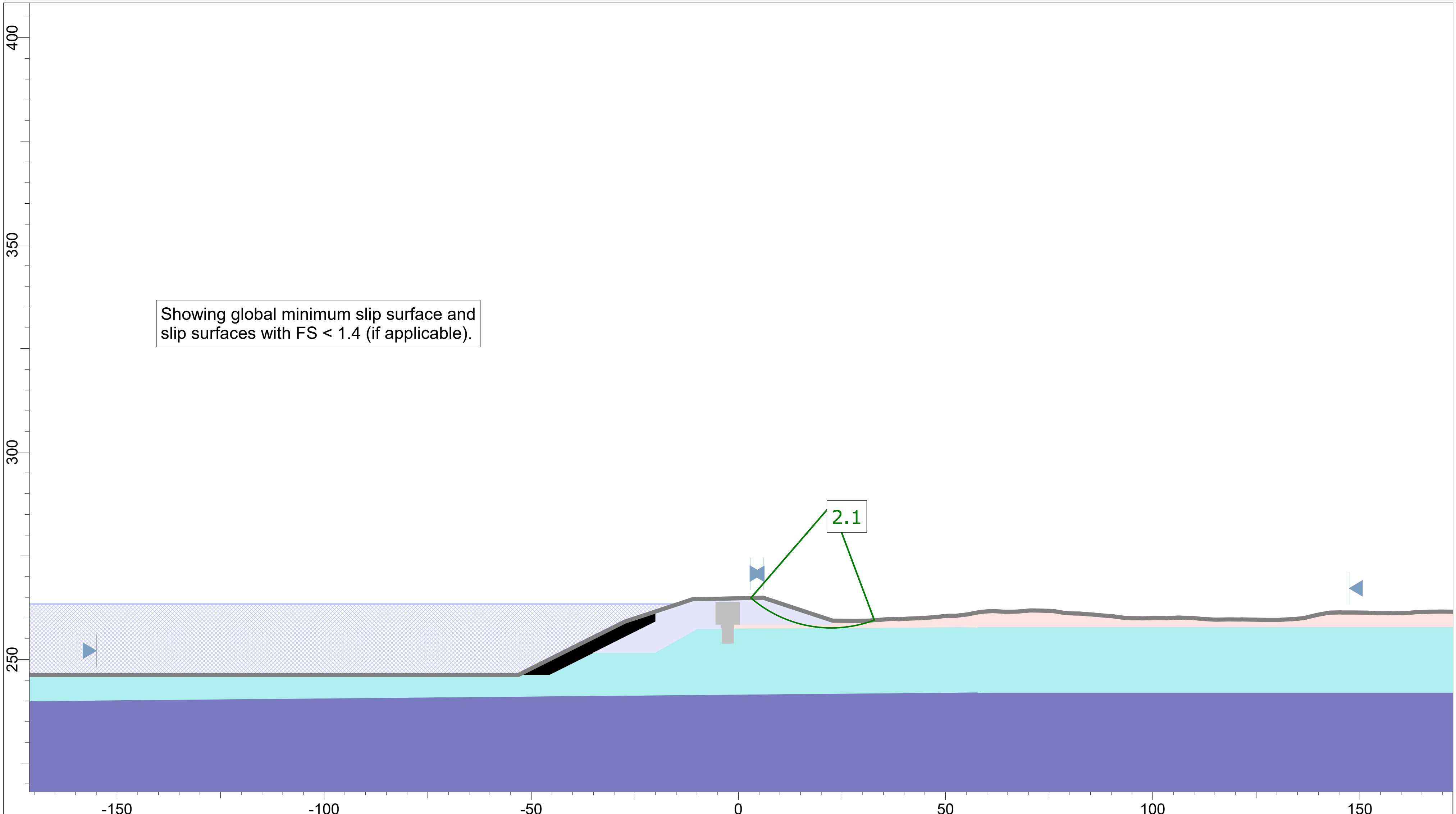
1/28/2022

PROJECT NO. 190175-A

BY: M. Otto

REVIEWED BY: A. Holmson

APPENDIX: **G-29**



Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

C-C' STA 16+25, Long-Term Conditions Landward, Steady-State Seepage, 100-Yr Flood

Slope Stability Analysis

Geotechnical Design Report
Jan Road Neighborhood Improvements
King County, Washington

SLIDEINTERPRET 8.032

SCALE: 1:400

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Aspect CONSULTING

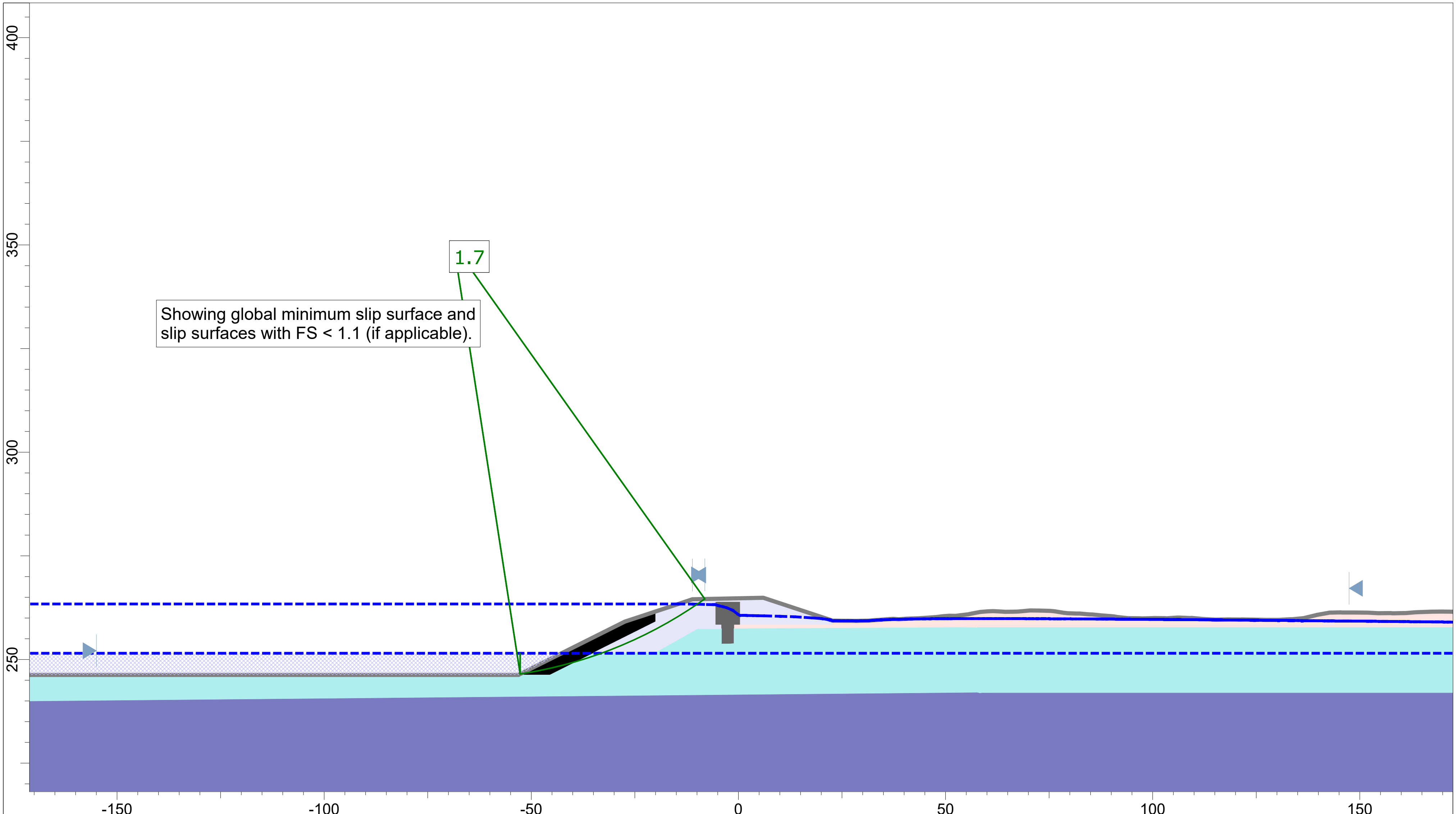
1/28/2022

PROJECT NO. 190175-A

BY: M. Otto

REVIEWED BY: A. Holmson

APPENDIX: **G-30**



Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

C-C' STA 16+25, Long-Term Conditions Waterward, Sudden Drawdown, 100-Yr Flood

Slope Stability Analysis

Geotechnical Design Report
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King County, Washington

SLIDEINTERPRET 8.032

SCALE: 1:400

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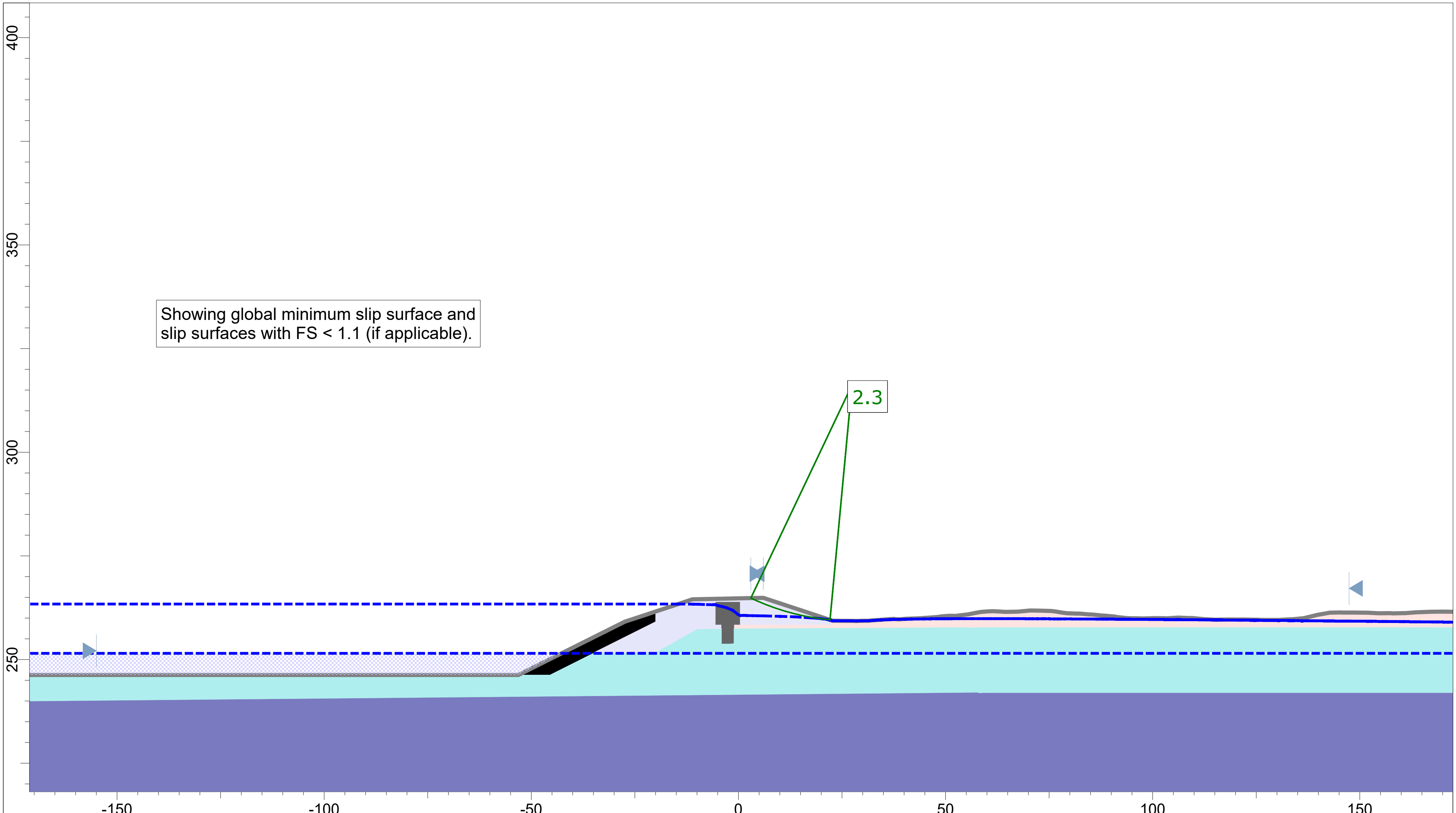
1/28/2022	BY: M. Otto	APPENDIX:
PROJECT NO. 190175-A	REVIEWED BY: A. Holmson	G-31



Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

C-C' STA 16+25, Long-Term Conditions Landward, Sudden Drawdown, 100-Yr Flood

Slope Stability Analysis

Geotechnical Design Report
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King County, Washington

SLIDEINTERPRET 8.032

SCALE: 1:400

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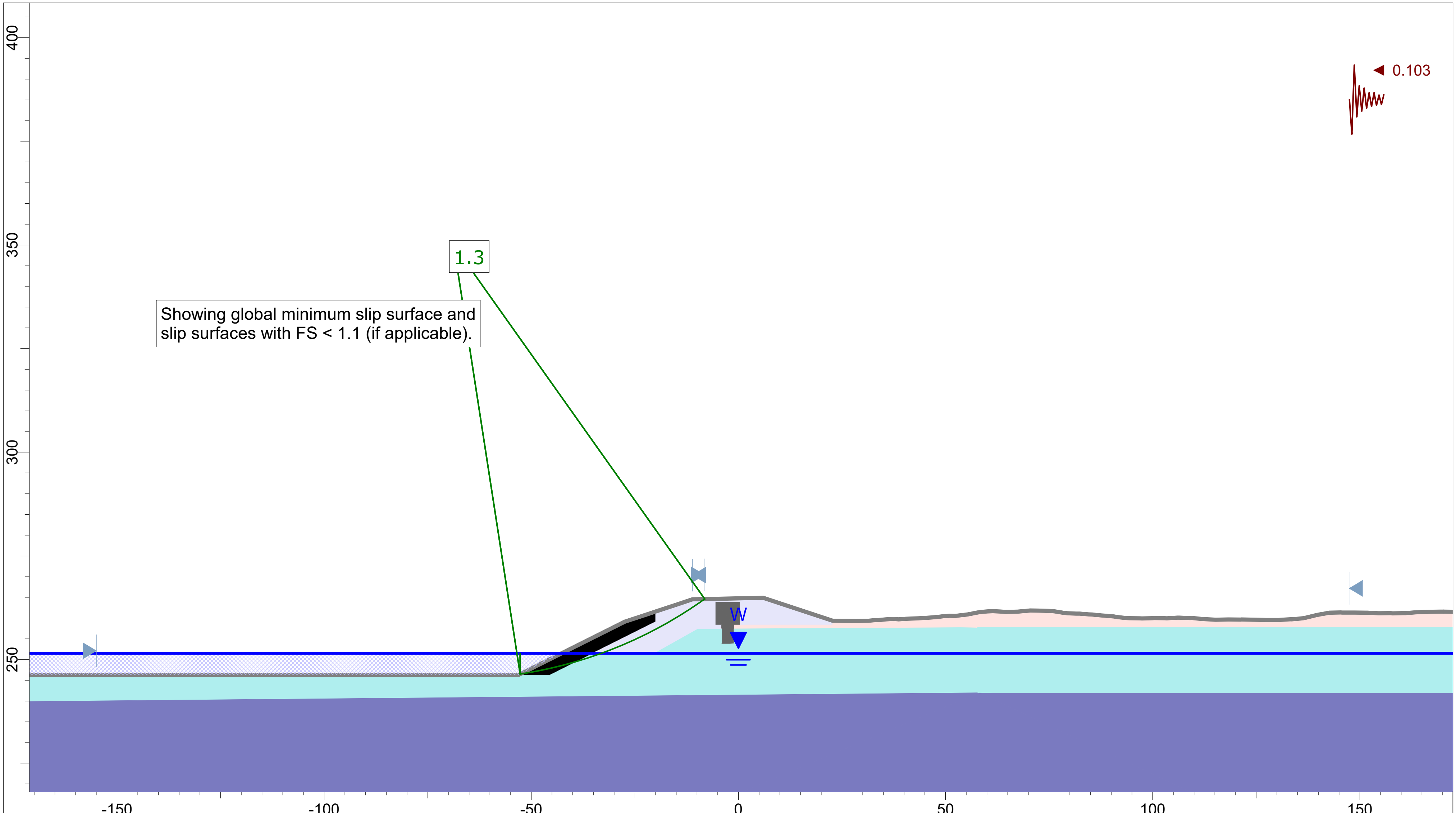
Aspect CONSULTING

1/28/2022	BY: M. Otto	APPENDIX:
PROJECT NO. 190175-A	REVIEWED BY: A. Holmson	**G-32**



Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

C-C' STA 16+25, Long-Term Conditions Waterward, Seismic, OBE

Slope Stability Analysis

Geotechnical Design Report
Jan Road Neighborhood Improvements
King County, Washington

SLIDEINTERPRET 8.032

SCALE: 1:400

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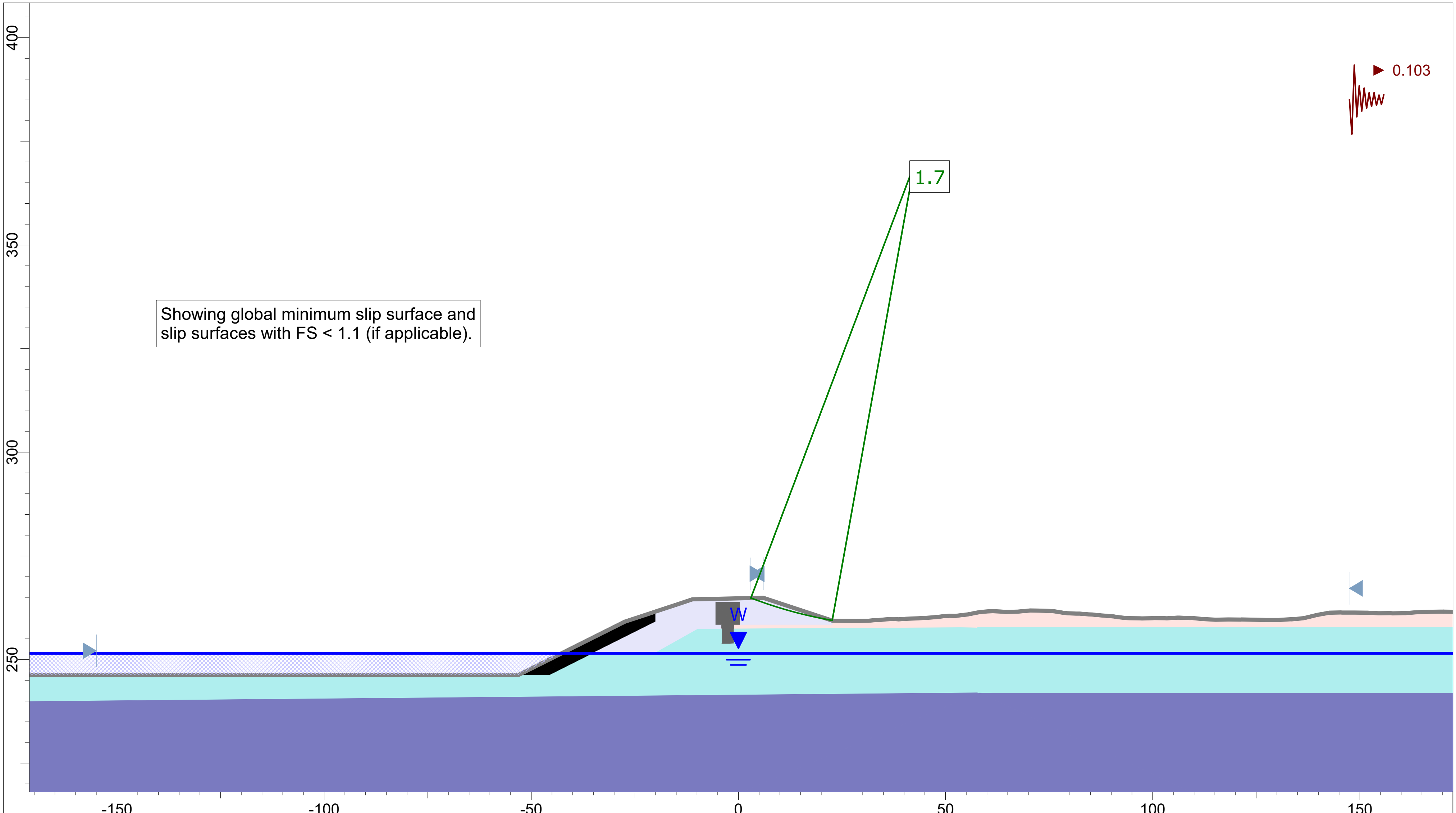
1/28/2022	BY: M. Otto	APPENDIX:
PROJECT NO. 190175-A	REVIEWED BY: A. Holmson	G-33



Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

C-C' STA 16+25, Long-Term Conditions Landward, Seismic, OBE

Slope Stability Analysis

Geotechnical Design Report
Jan Road Neighborhood Improvements
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SLIDEINTERPRET 8.032

SCALE: 1:400

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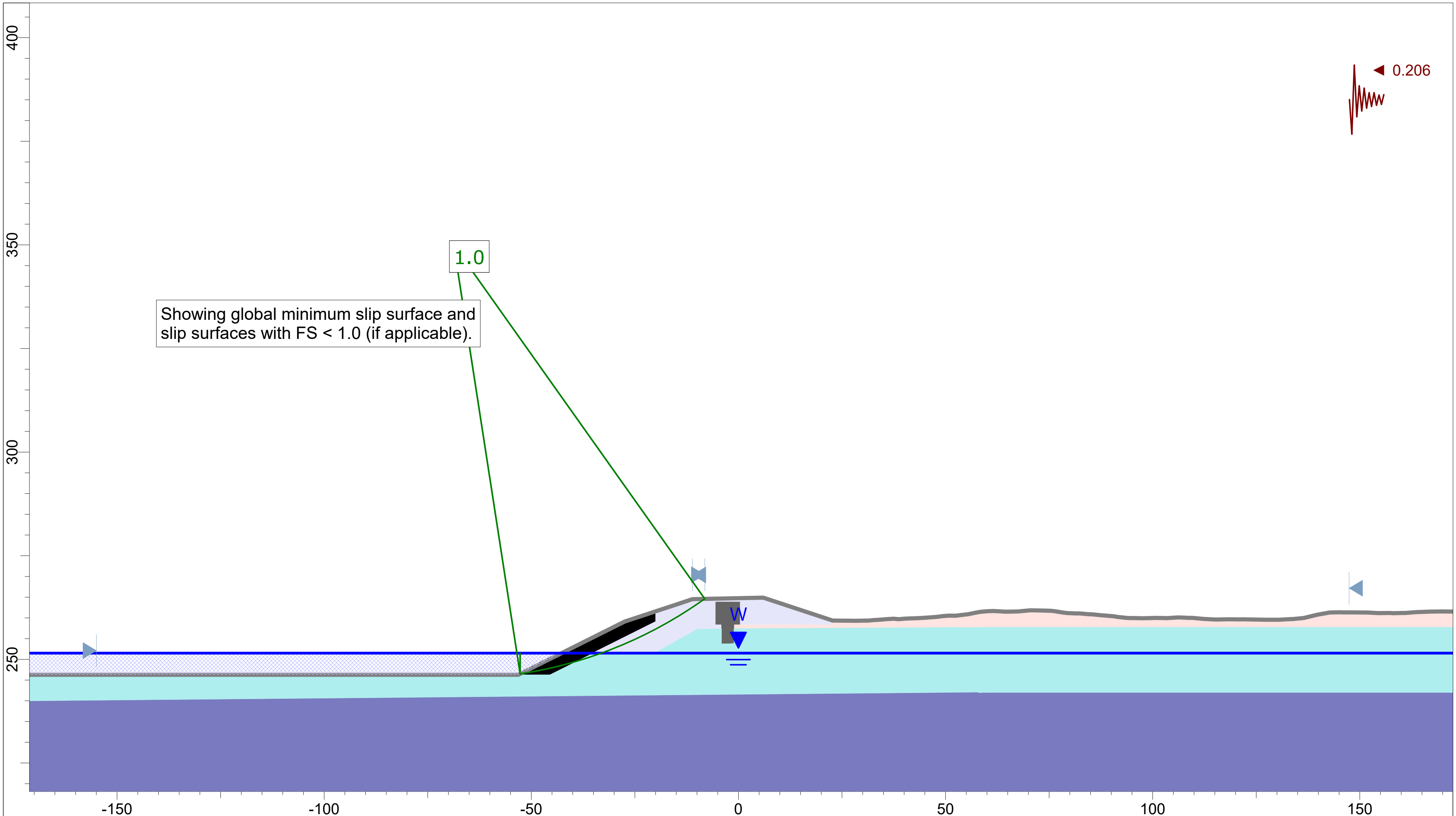
Aspect CONSULTING

1/28/2022	BY: M. Otto	APPENDIX:
PROJECT NO. 190175-A	REVIEWED BY: A. Holmson	G-34



Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

C-C' STA 16+25, Long-Term Conditions Waterward, Seismic, MDE

Slope Stability Analysis

Geotechnical Design Report
Jan Road Neighborhood Improvements
King County, Washington

SLIDEINTERPRET 8.032

SCALE: 1:400

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Aspect CONSULTING

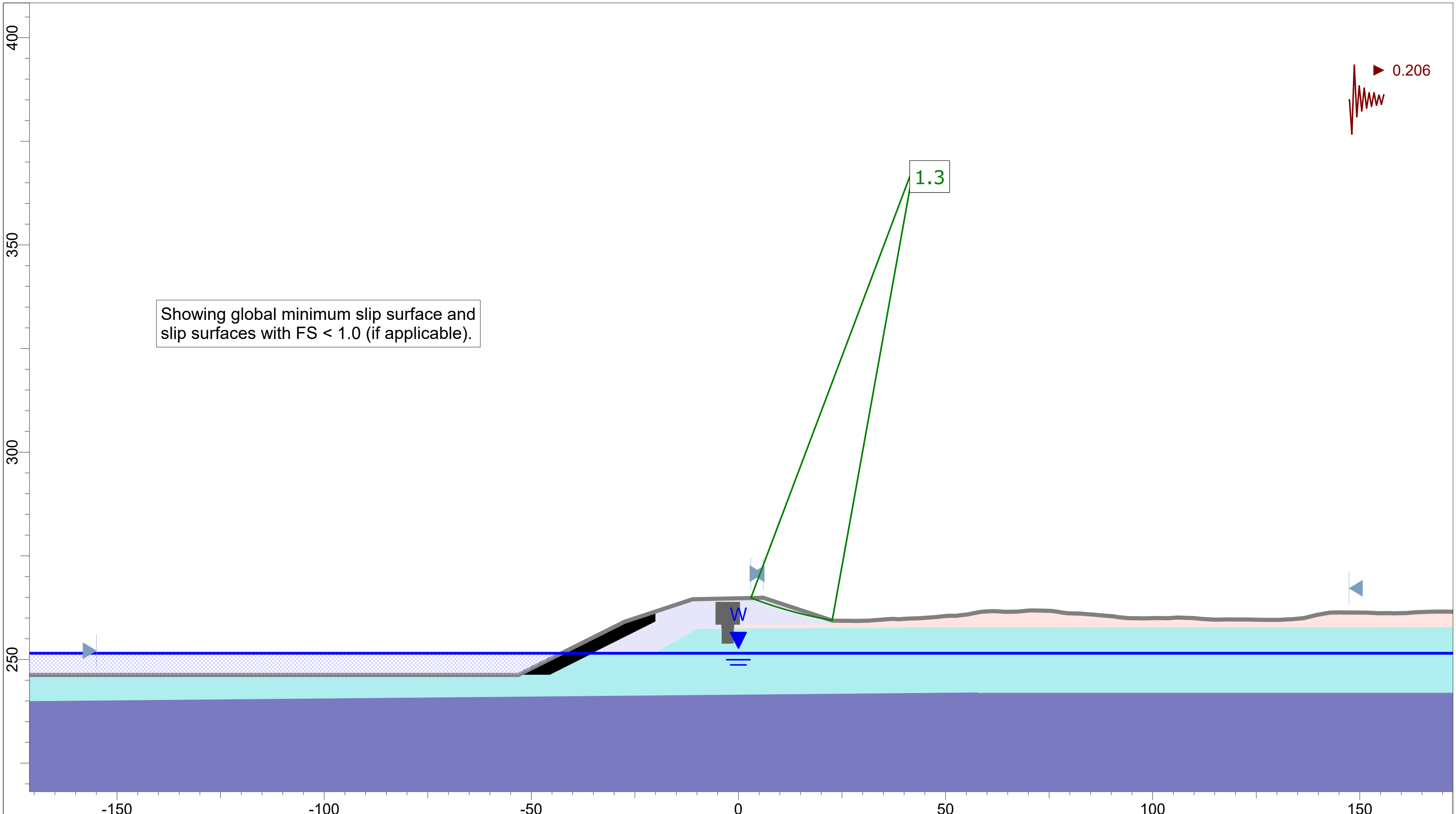
1/28/2022

PROJECT NO. 190175-A

BY: M. Otto

REVIEWED BY: A. Holmson

APPENDIX: G-35



Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

C-C' STA 16+25, Long-Term Conditions Landward, Seismic, MDE

Slope Stability Analysis

Geotechnical Design Report
Jan Road Neighborhood Improvements
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SLIDEINTERPRET 8.032

SCALE: 1:400

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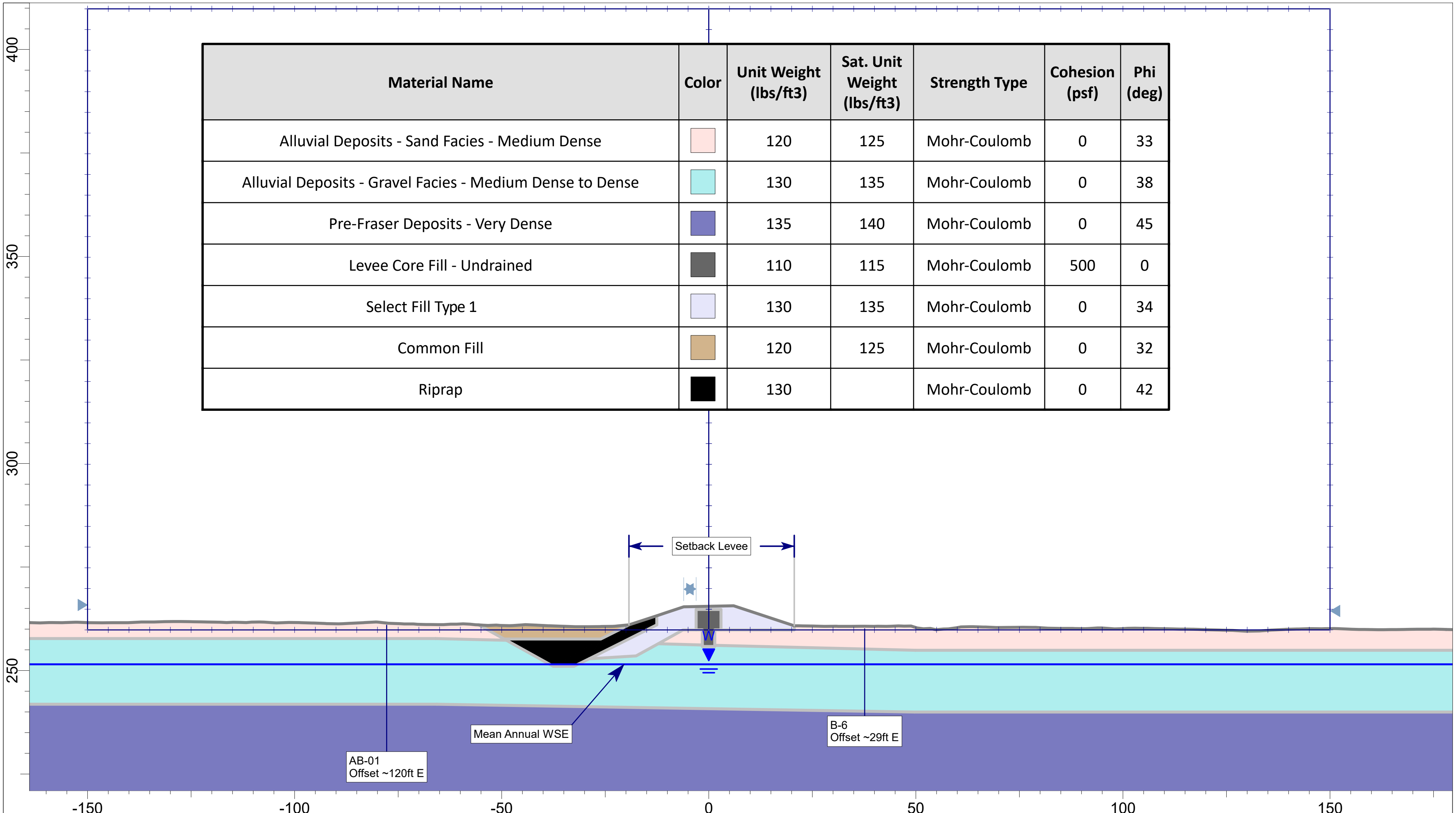
1/28/2022	BY: M. Otto	APPENDIX:
PROJECT NO. 190175-A	REVIEWED BY: A. Holmson	**G-36**

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Alluvial Deposits - Sand Facies - Medium Dense		120	125	Mohr-Coulomb	0	33
Alluvial Deposits - Gravel Facies - Medium Dense to Dense		130	135	Mohr-Coulomb	0	38
Pre-Fraser Deposits - Very Dense		135	140	Mohr-Coulomb	0	45
Levee Core Fill - Undrained		110	115	Mohr-Coulomb	500	0
Select Fill Type 1		130	135	Mohr-Coulomb	0	34
Common Fill		120	125	Mohr-Coulomb	0	32
Riprap		130		Mohr-Coulomb	0	42



Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

D-D' STA 19+00, End of Construction Master Scenario

Slope Stability Analysis

Geotechnical Design Report
Jan Road Neighborhood Improvements
King County, Washington

SLIDE 8.032

SCALE: 1:400

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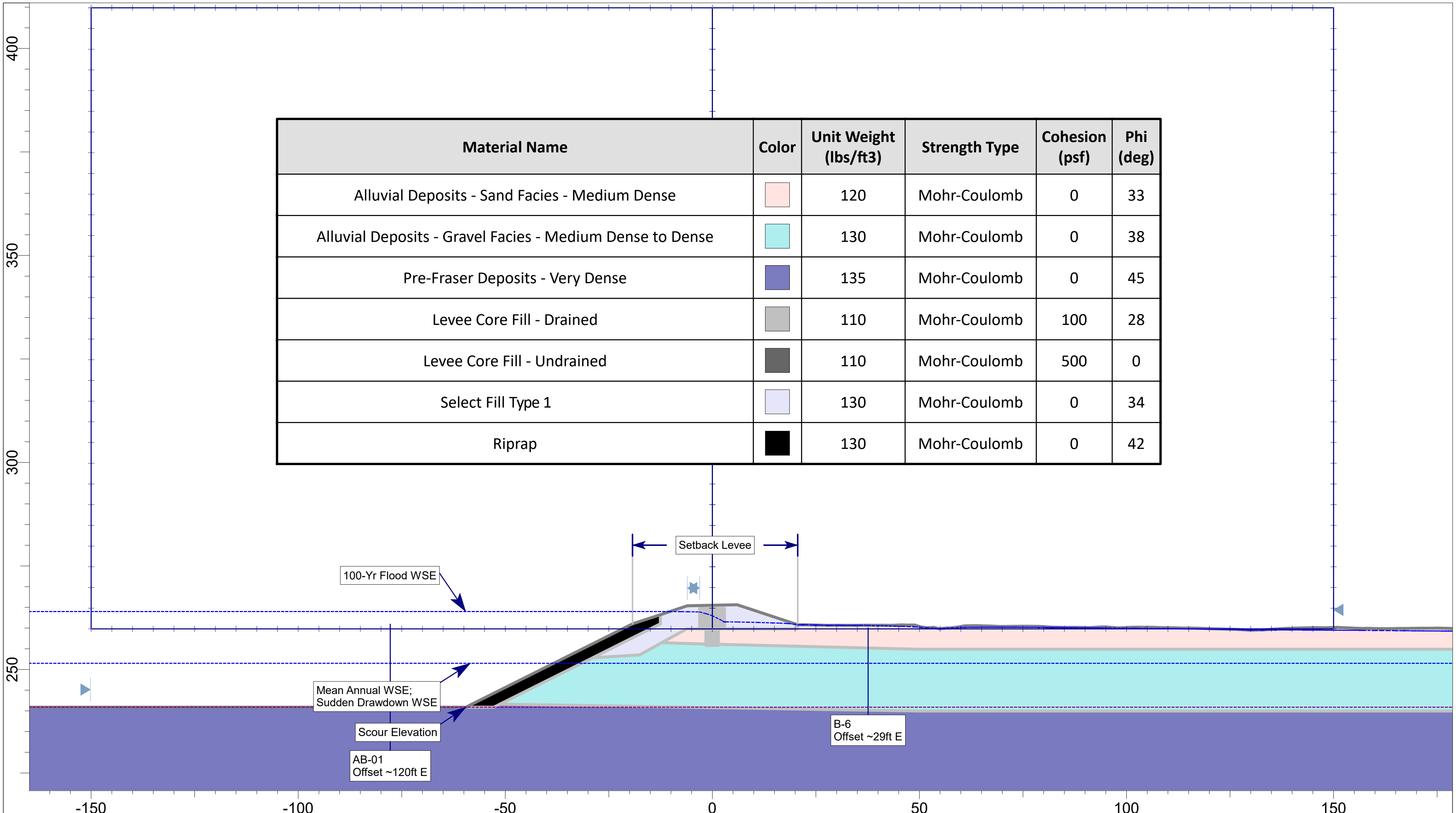
1/28/2022

PROJECT NO. 190175-A

BY: M. Otto

REVIEWED BY: A. Holmson

APPENDIX: G-37



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Alluvial Deposits - Sand Facies - Medium Dense		120	Mohr-Coulomb	0	33
Alluvial Deposits - Gravel Facies - Medium Dense to Dense		130	Mohr-Coulomb	0	38
Pre-Fraser Deposits - Very Dense		135	Mohr-Coulomb	0	45
Levee Core Fill - Drained		110	Mohr-Coulomb	100	28
Levee Core Fill - Undrained		110	Mohr-Coulomb	500	0
Select Fill Type 1		130	Mohr-Coulomb	0	34
Riprap		130	Mohr-Coulomb	0	42

Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

D-D' STA 19+00, Long-Term Conditions Master Scenario

Slope Stability Analysis

Geotechnical Design Report
Jan Road Neighborhood Improvements
King County, Washington

SLIDE 8.032

SCALE: 1:400

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Aspect CONSULTING

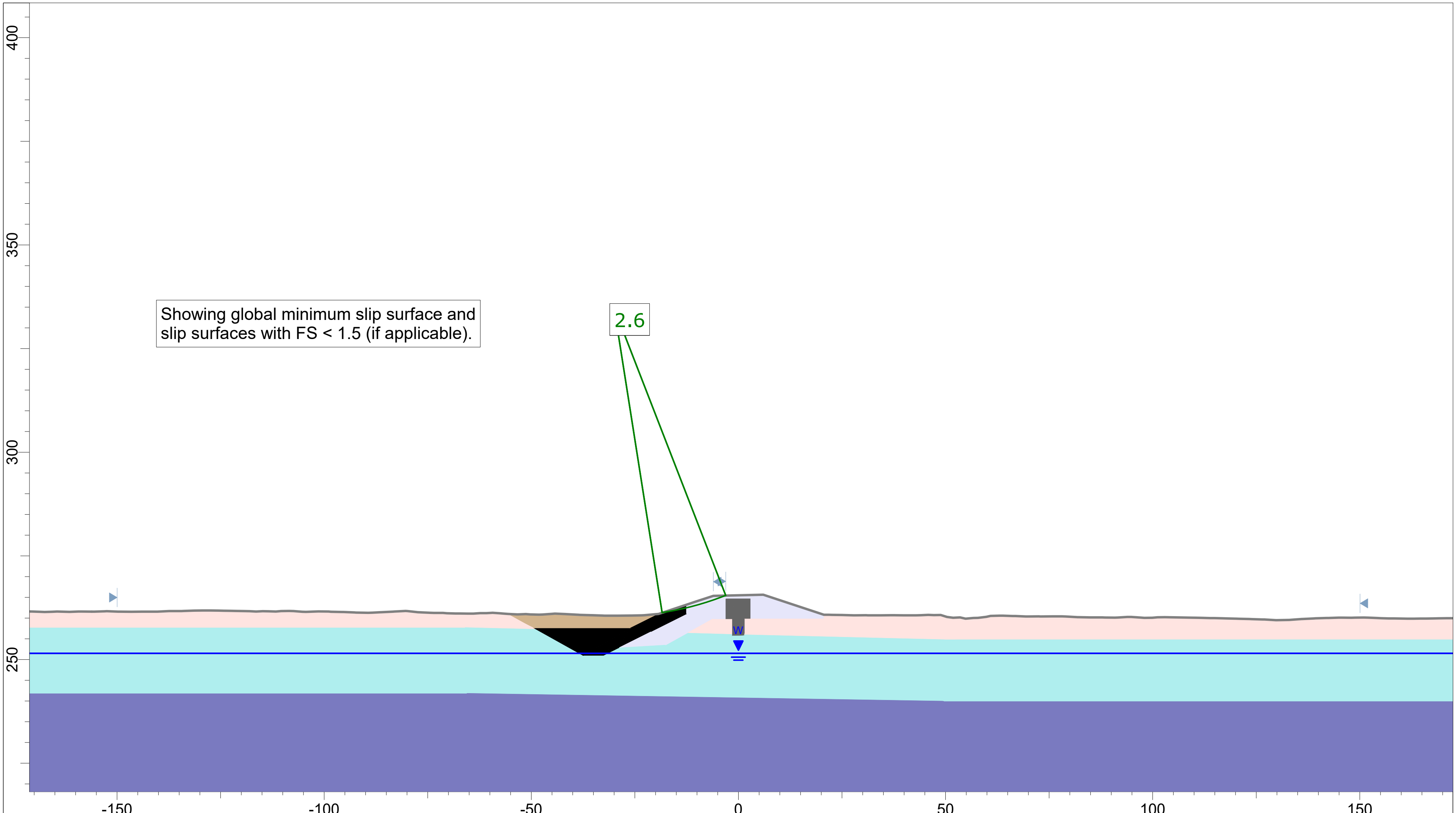
1/28/2022

PROJECT NO. 190175-A

BY: M. Otto

REVIEWED BY: A. Holmson

APPENDIX: G-38



Legend

Search Grid

Search Limits

Modeled Groundwater Level

Boring Location and Depth

D-D' STA 19+00, End of Construction Waterward, Mean Annual WSE

Slope Stability Analysis

Geotechnical Design Report
Jan Road Neighborhood Improvements
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SCALE: 1:400

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Aspect CONSULTING

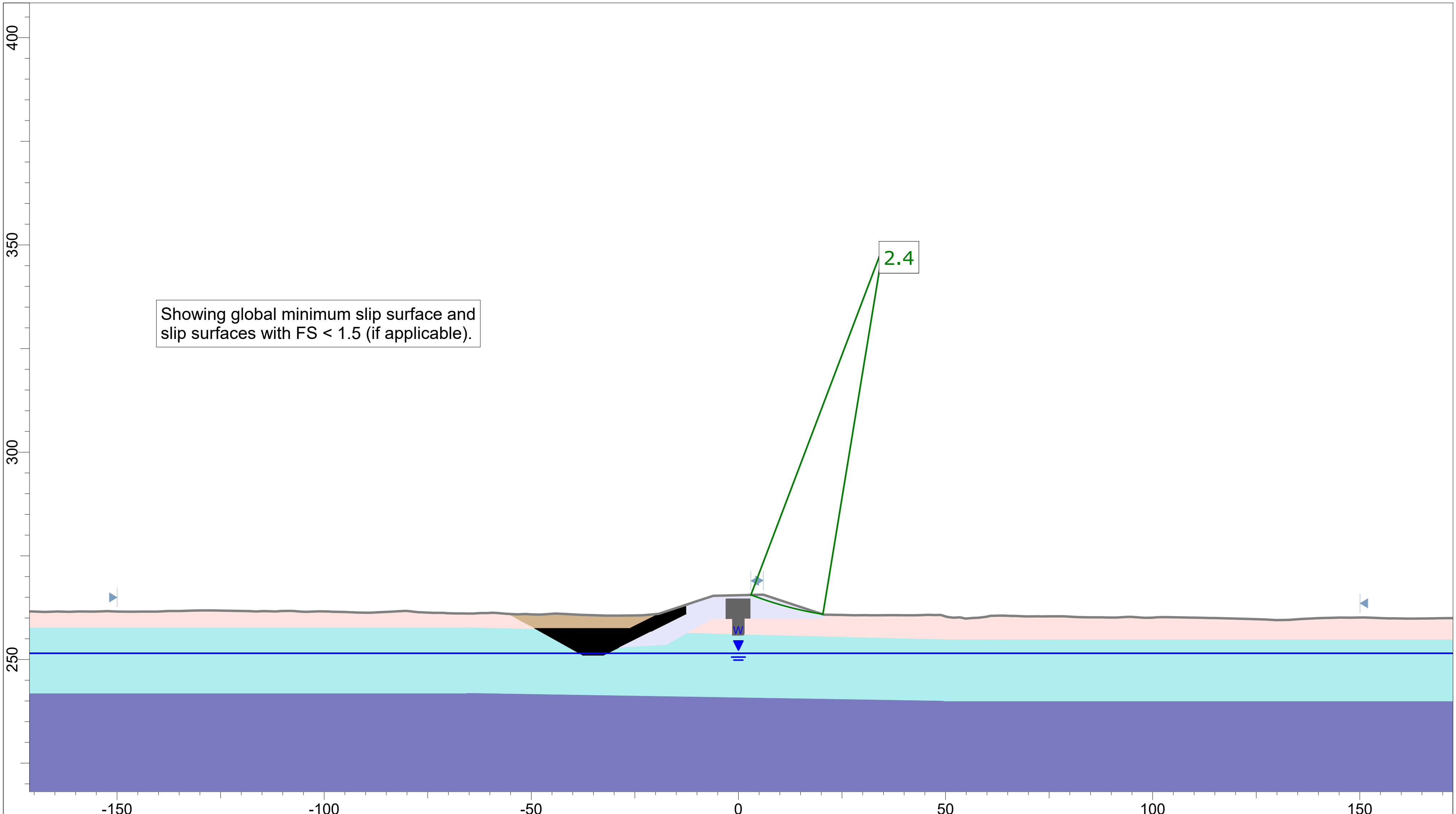
1/28/2022

PROJECT NO. 190175

BY: M. Otto

REVIEWED BY: A. Holmson

APPENDIX: G-39



Legend

Search Grid

Search Limits

Modeled Groundwater Level

Boring Location and Depth

D-D' STA 19+00, End of Construction Landward, Mean Annual WSE

Slope Stability Analysis

Geotechnical Design Report
Jan Road Neighborhood Improvements
King County, Washington

SLIDEINTERPRET 8.032

SCALE: 1:400

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Aspect CONSULTING

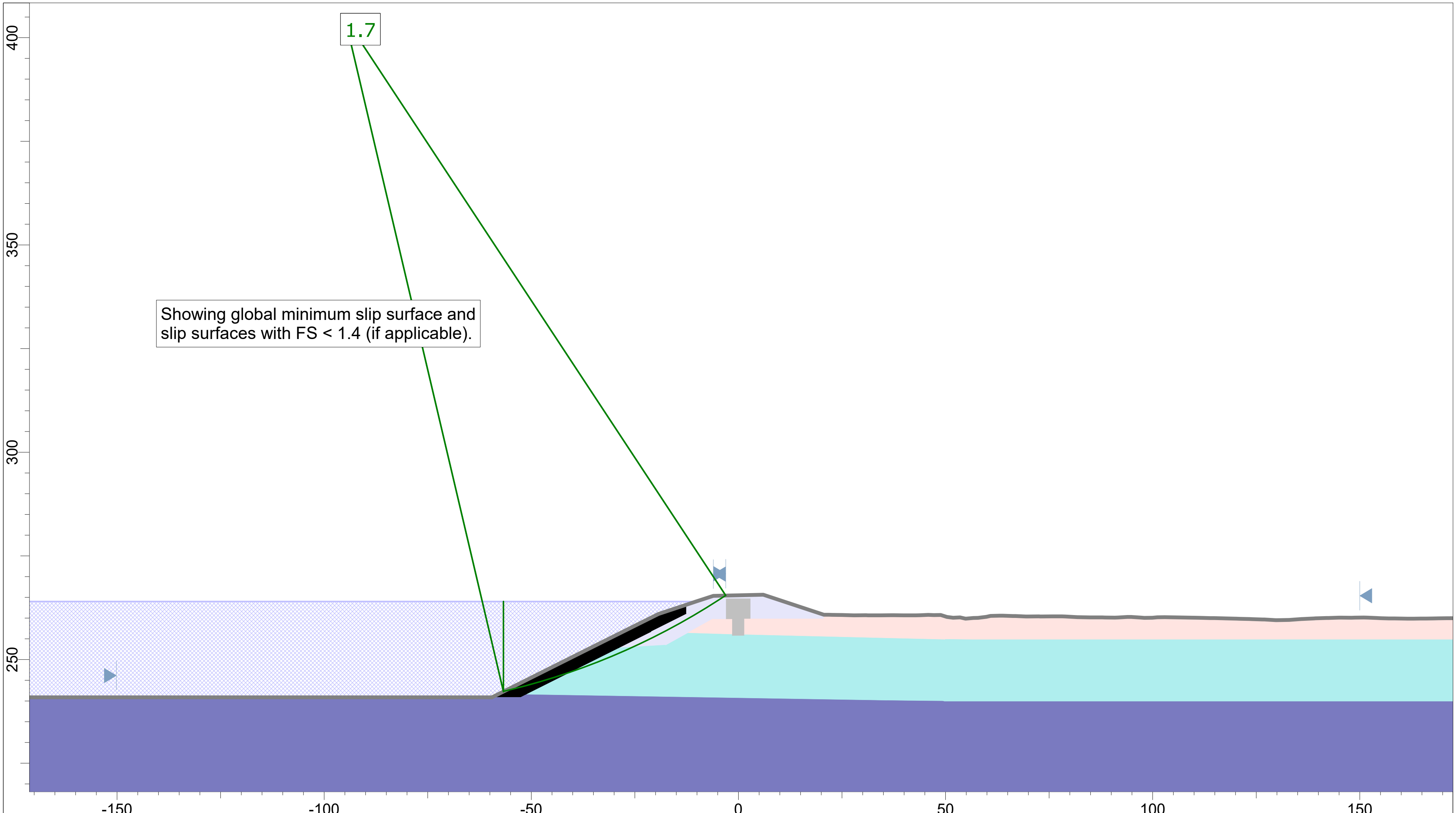
1/28/2022

PROJECT NO. 190175

BY: M. Otto

REVIEWED BY: A. Holmson

APPENDIX: G-40



Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

D-D' STA 19+00, Long-Term Conditions Waterward, Steady-State Seepage, 100-Yr Flood

Slope Stability Analysis

Geotechnical Design Report
Jan Road Neighborhood Improvements
King County, Washington

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SCALE: 1:400

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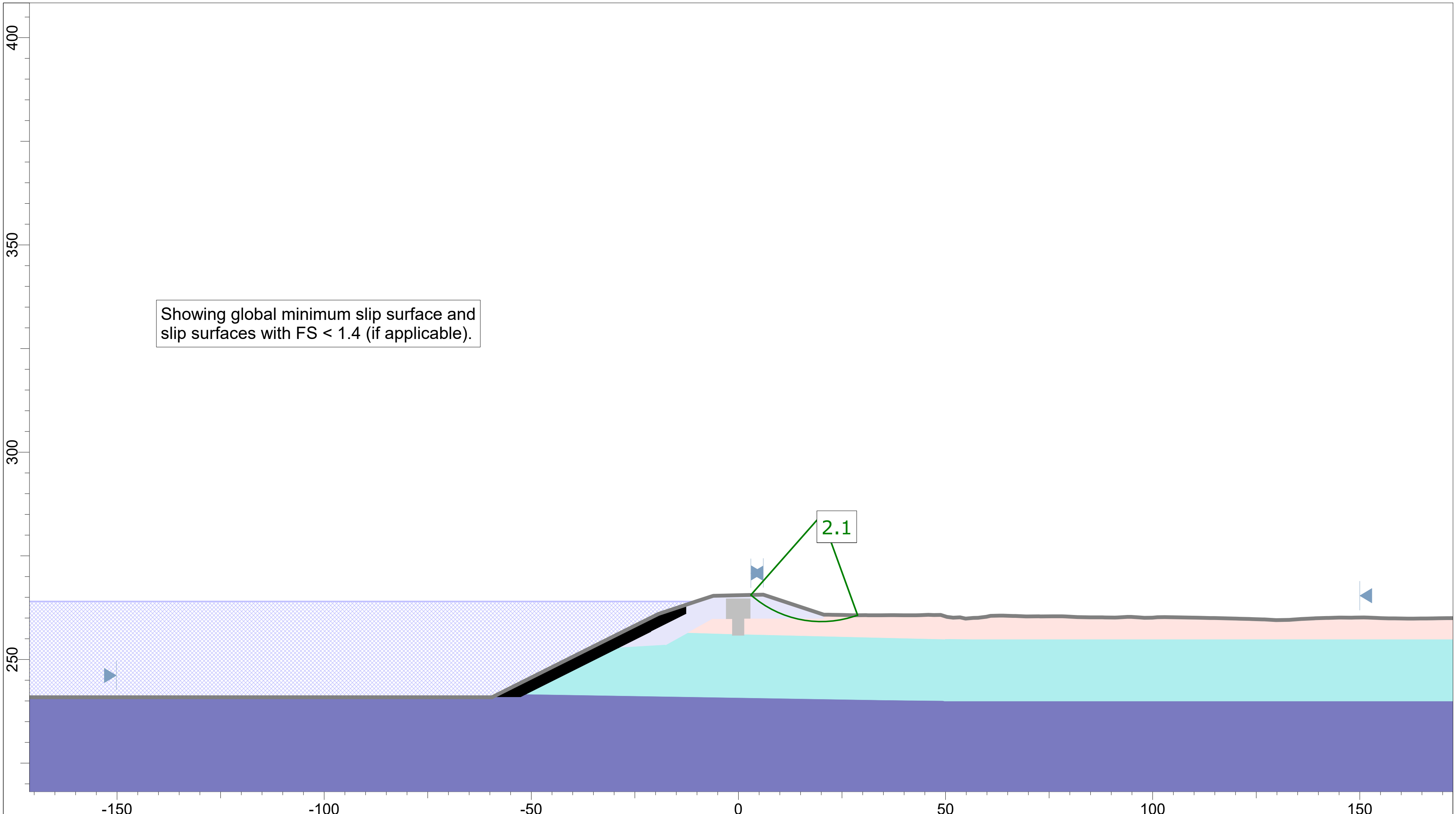
Aspect CONSULTING

1/28/2022	BY: M. Otto	APPENDIX:
PROJECT NO. 190175-A	REVIEWED BY: A. Holmson	**G-41**



Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

D-D' STA 19+00, Long-Term Conditions Landward, Steady-State Seepage, 100-Yr Flood

Slope Stability Analysis

Geotechnical Design Report
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SLIDEINTERPRET 8.032

SCALE: 1:400

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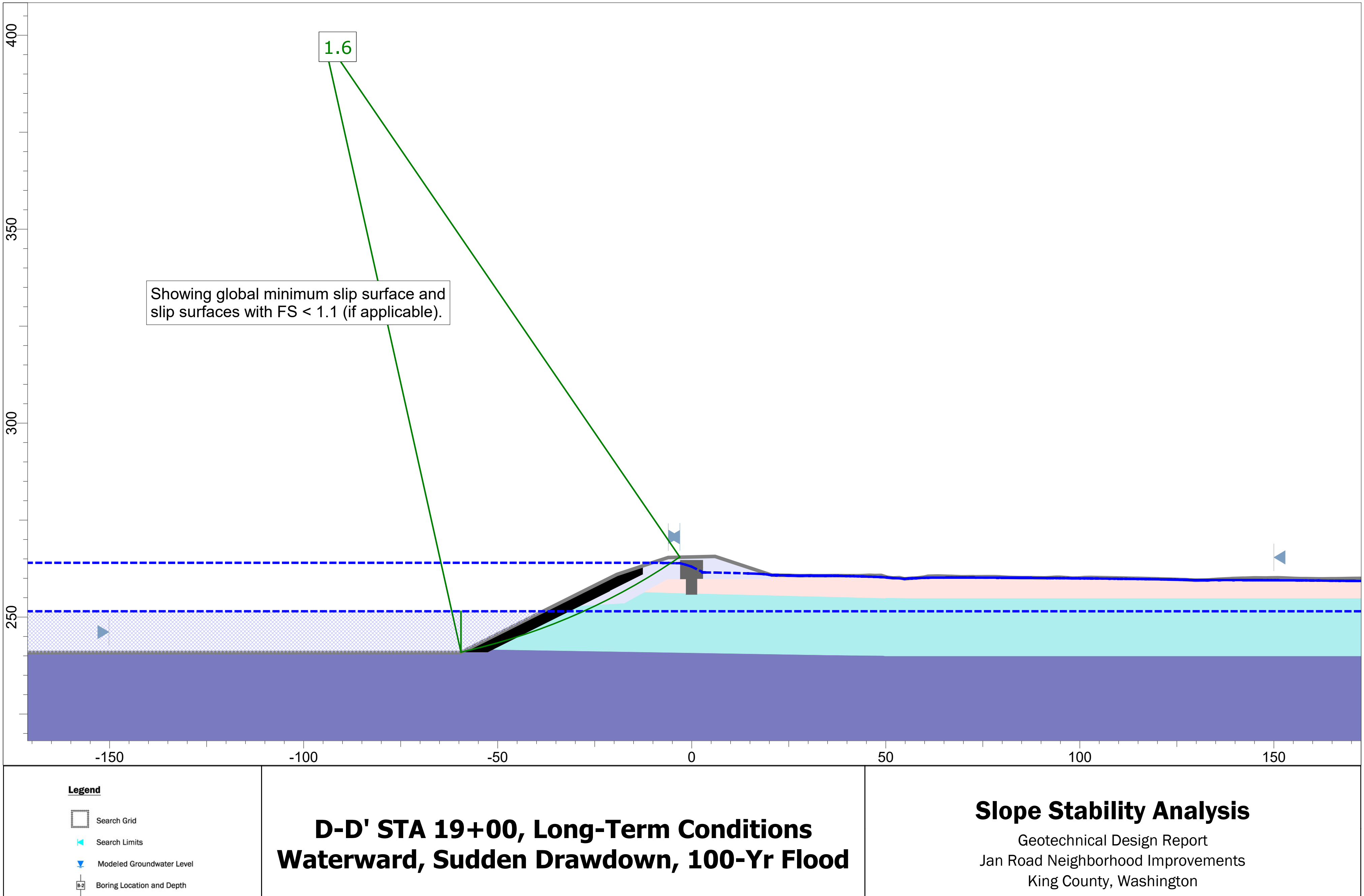
1/28/2022	BY: M. Otto	APPENDIX:
PROJECT NO. 190175-A	REVIEWED BY: A. Holmson	G-42



D-D' STA 19+00, Long-Term Conditions Waterward, Sudden Drawdown, 100-Yr Flood

Slope Stability Analysis

Geotechnical Design Report
Jan Road Neighborhood Improvements
King County, Washington

SLIDEINTERPRET 8.032

SCALE: 1:400

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Aspect CONSULTING

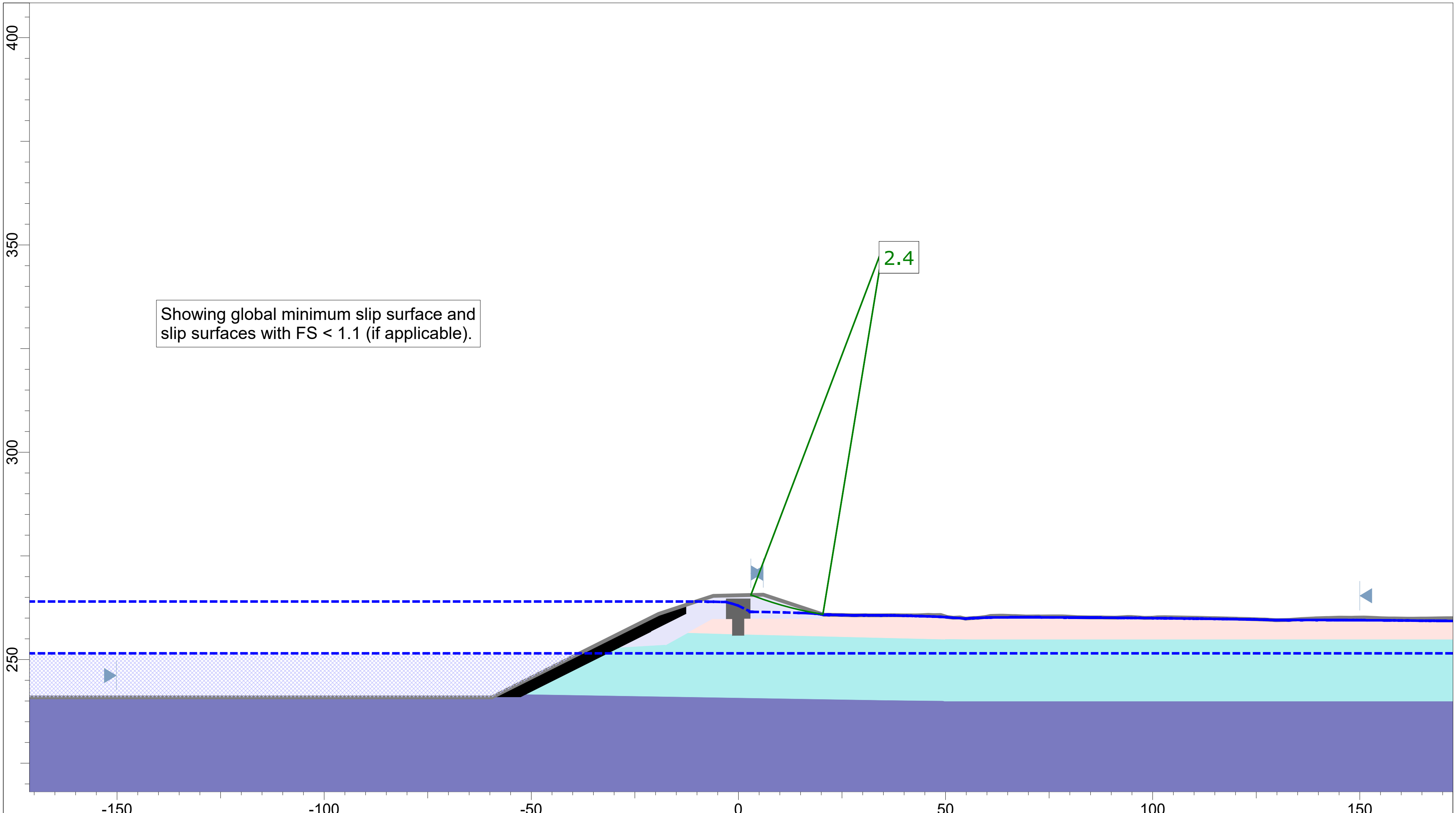
1/28/2022

PROJECT NO. 190175-A

BY: M. Otto

REVIEWED BY: A. Holmson

APPENDIX: G-43



Legend

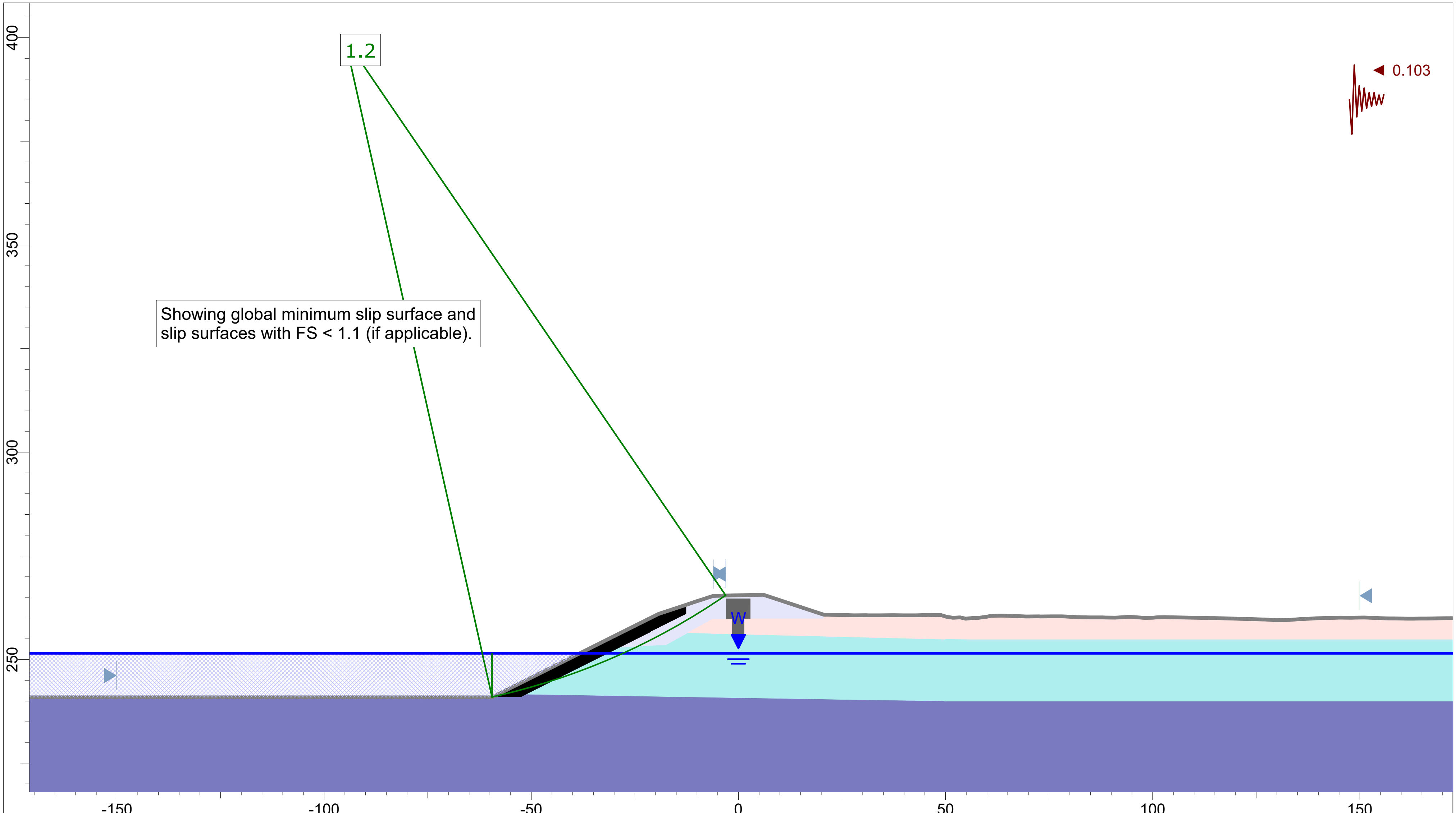
- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

D-D' STA 19+00, Long-Term Conditions Landward, Sudden Drawdown, 100-Yr Flood

Slope Stability Analysis

Geotechnical Design Report
Jan Road Neighborhood Improvements
King County, Washington

SLIDEINTERPRET 8.032	SCALE: 1:400 C:\Users\motto\OneDrive - Aspect Consulting LLC\Slide\2022.01.03_D-D' STA 19+00, Long-Term Conditions.slmd	Aspect CONSULTING	1/28/2022 PROJECT NO. 190175-A	BY: M. Otto REVIEWED BY: A. Holmson	APPENDIX: G-44



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- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

D-D' STA 19+00, Long-Term Conditions Waterward, Seismic OBE

Slope Stability Analysis

Geotechnical Design Report
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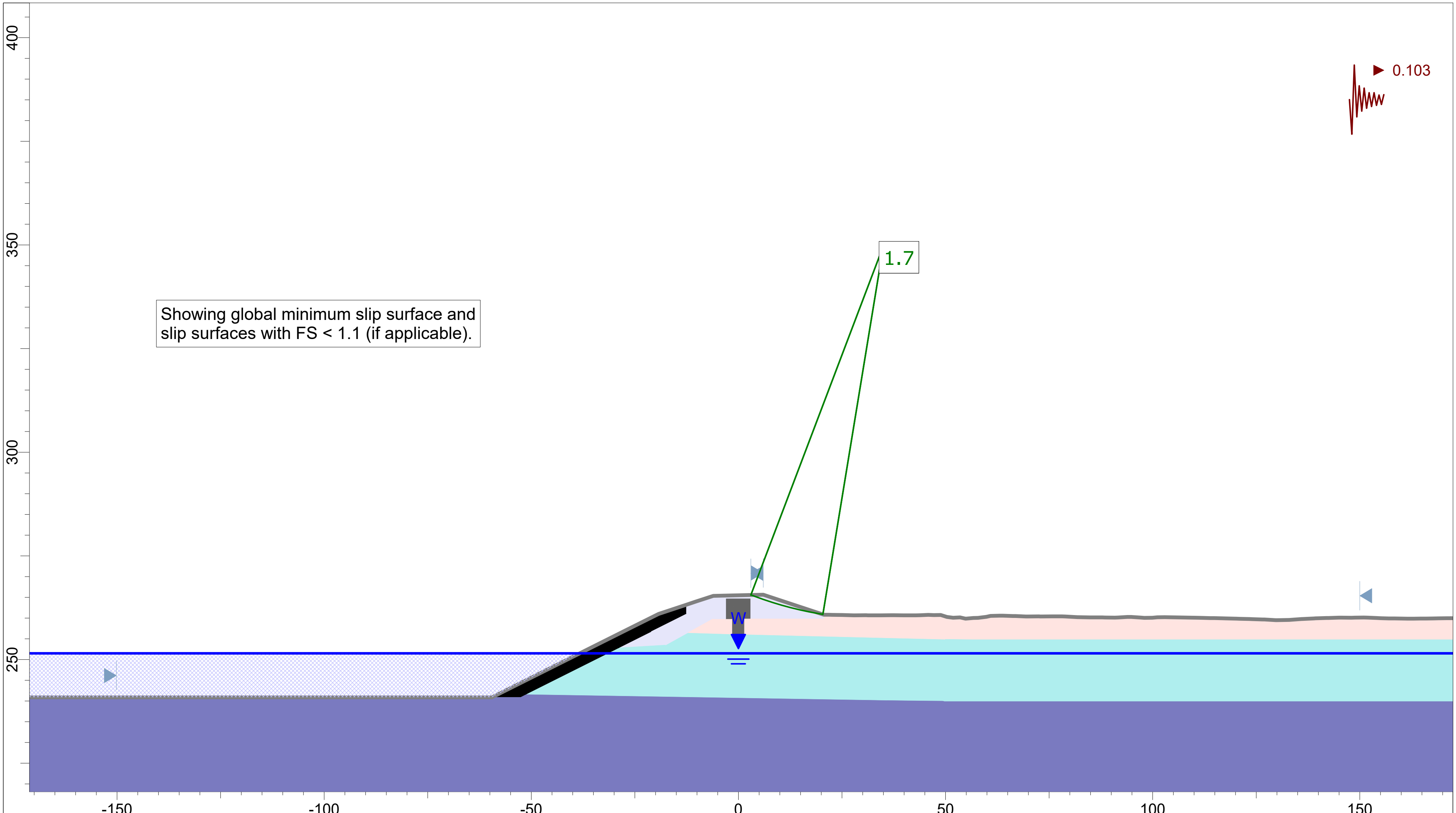
PROJECT NO. 190175-A

BY: M. Otto

REVIEWED BY: A. Holmson

APPENDIX: G-45



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- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

D-D' STA 19+00, Long-Term Conditions Landward, Seismic OBE

Slope Stability Analysis

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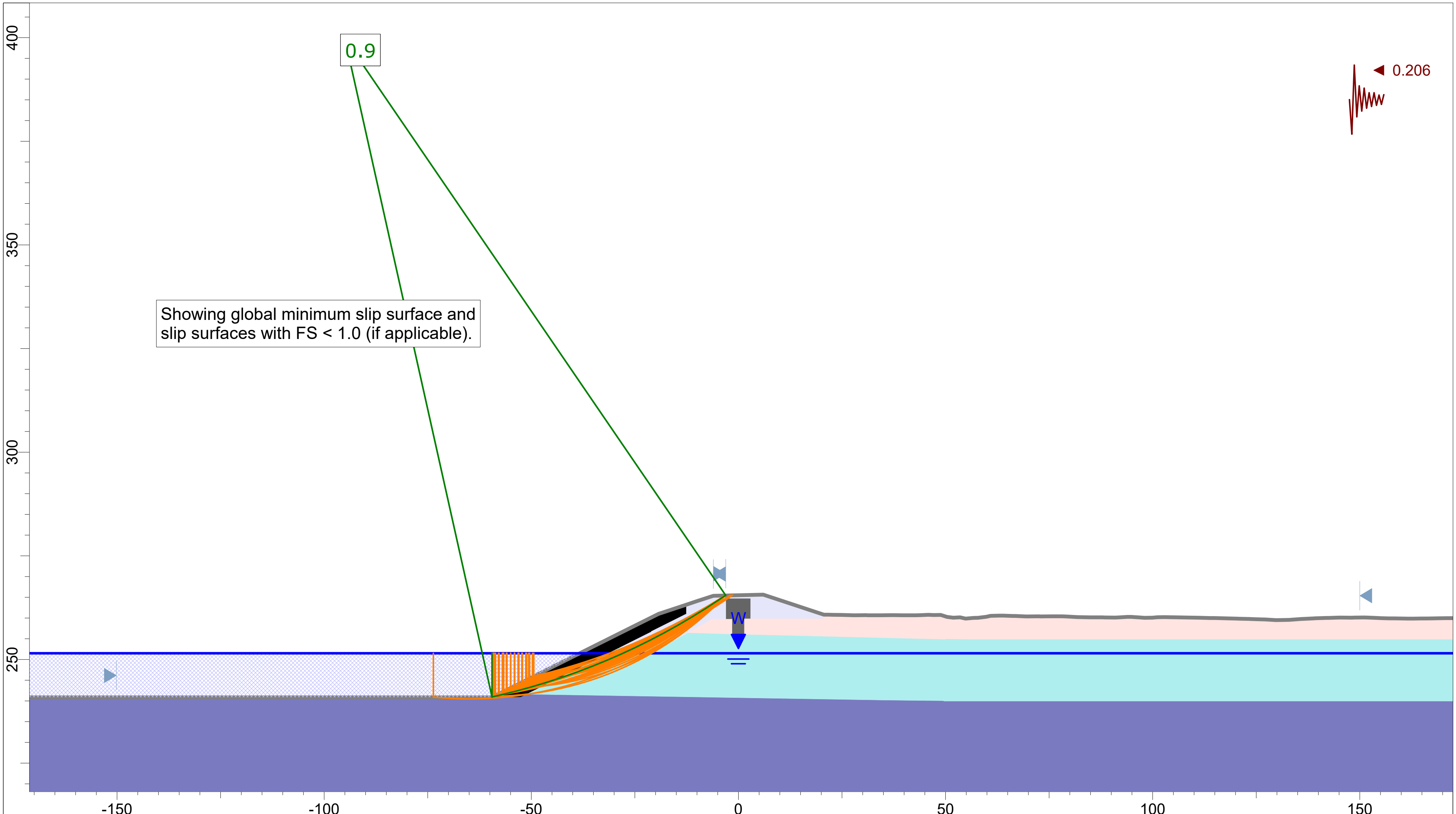
REVIEWED BY: A. Holmson

APPENDIX: G-46



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- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

D-D' STA 19+00, Long-Term Conditions Waterward, Seismic MDE

Slope Stability Analysis

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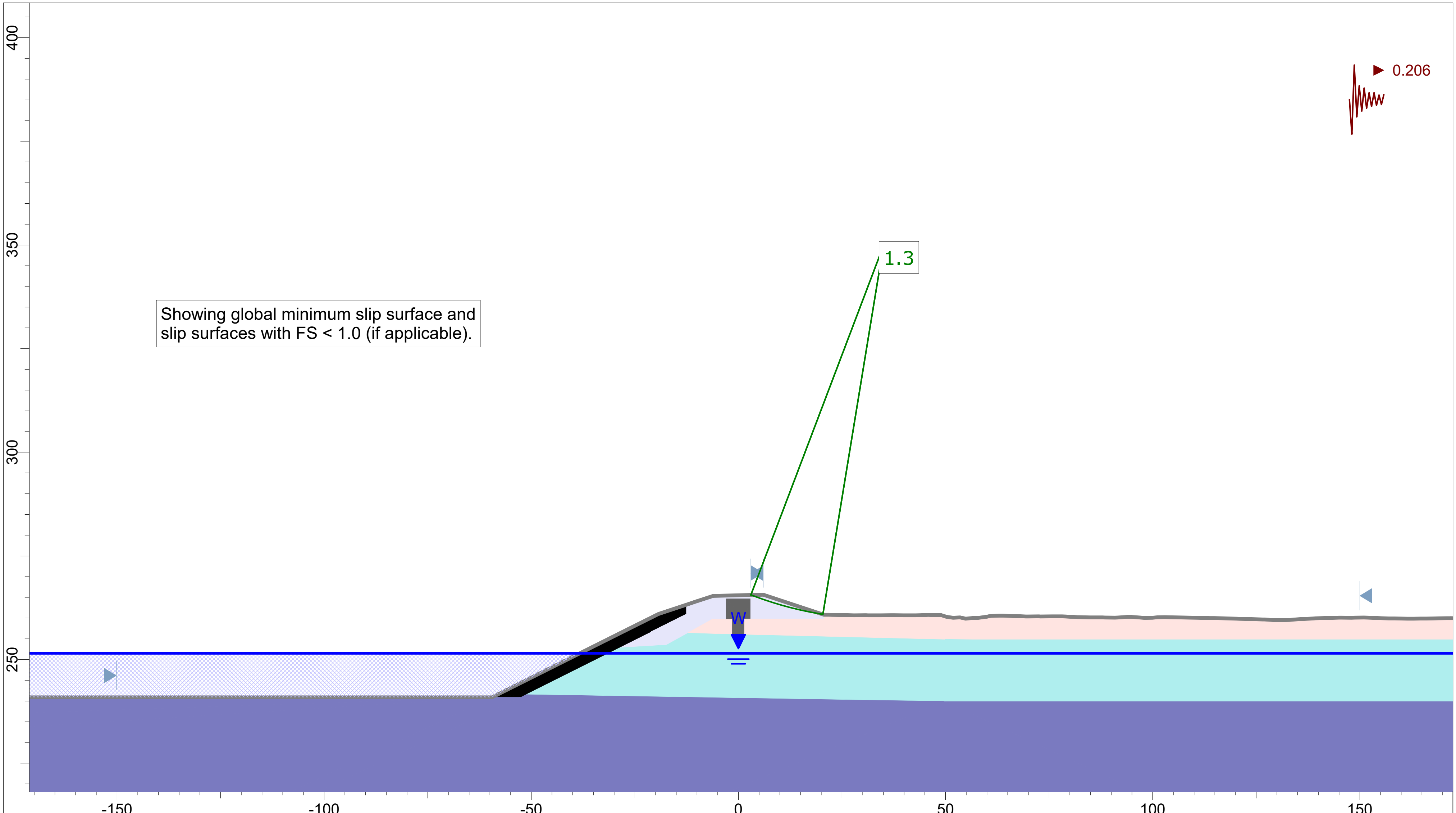
BY: M. Otto

REVIEWED BY: A. Holmson

APPENDIX: **G-47**



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- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

D-D' STA 19+00, Long-Term Conditions Landward, Seismic MDE

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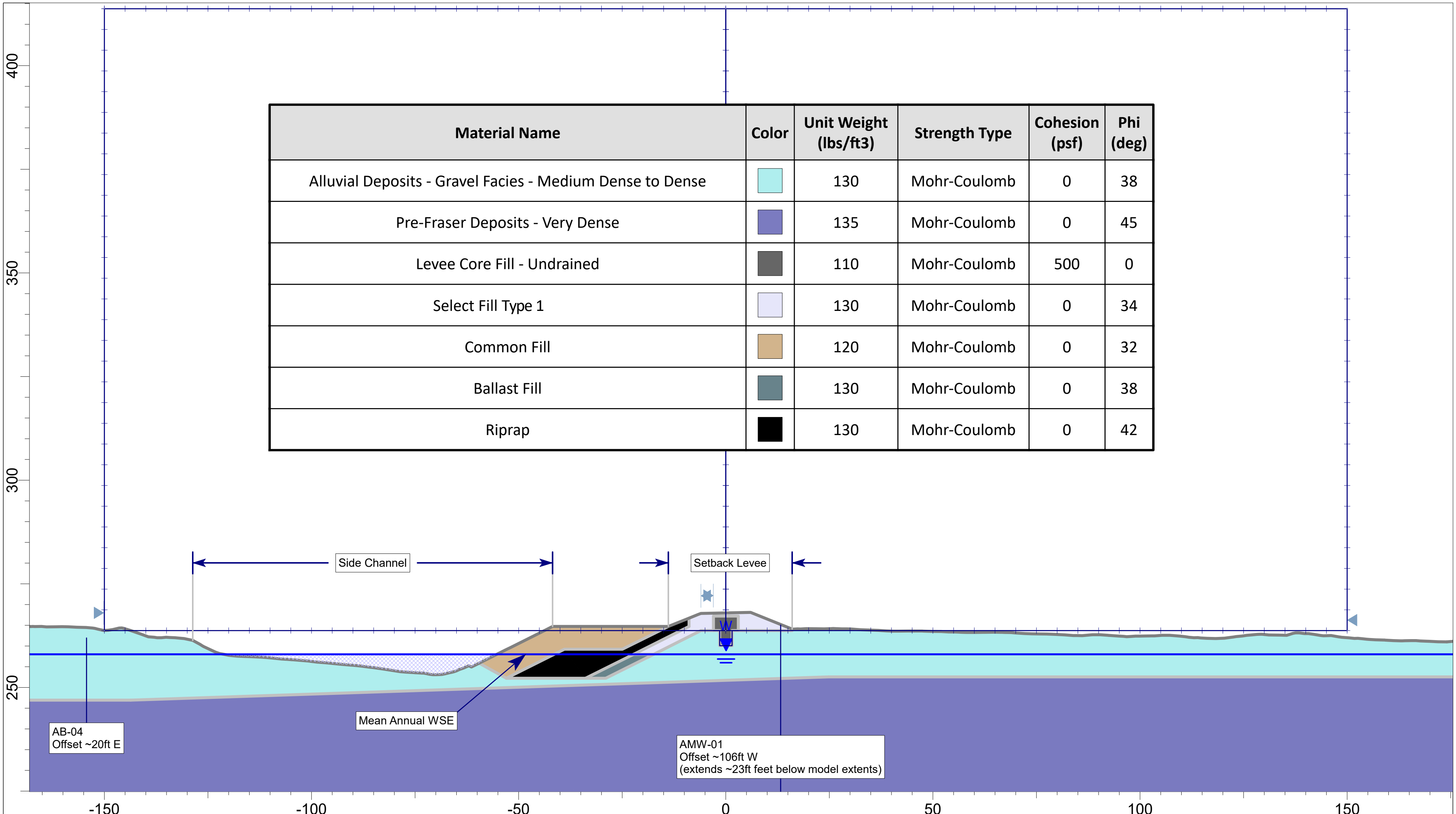
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Alluvial Deposits - Gravel Facies - Medium Dense to Dense		130	Mohr-Coulomb	0	38
Pre-Fraser Deposits - Very Dense		135	Mohr-Coulomb	0	45
Levee Core Fill - Undrained		110	Mohr-Coulomb	500	0
Select Fill Type 1		130	Mohr-Coulomb	0	34
Common Fill		120	Mohr-Coulomb	0	32
Ballast Fill		130	Mohr-Coulomb	0	38
Riprap		130	Mohr-Coulomb	0	42



Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

E-E' STA 24+00, End of Construction Master Scenario

Slope Stability Analysis

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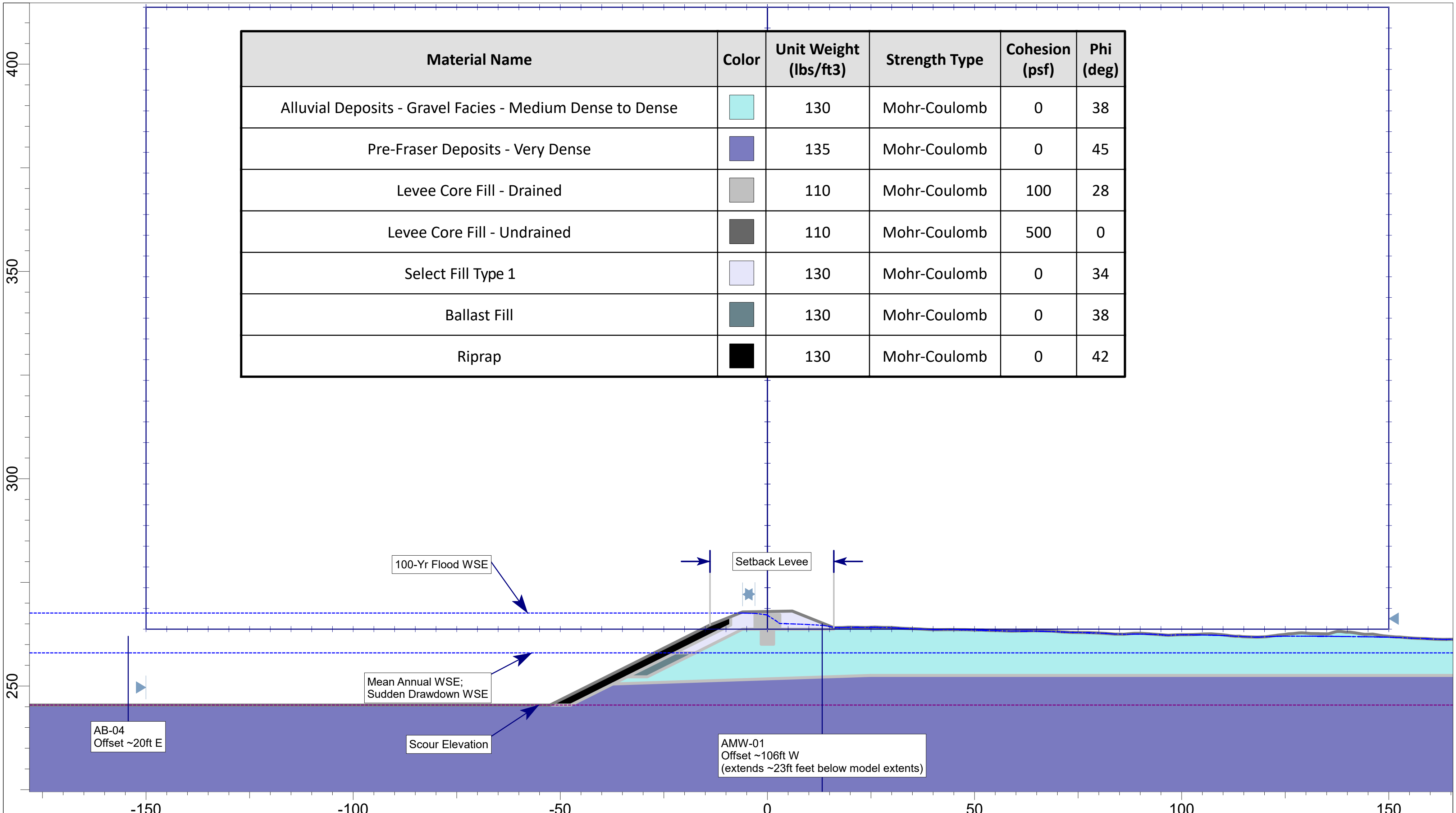
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Alluvial Deposits - Gravel Facies - Medium Dense to Dense		130	Mohr-Coulomb	0	38
Pre-Fraser Deposits - Very Dense		135	Mohr-Coulomb	0	45
Levee Core Fill - Drained		110	Mohr-Coulomb	100	28
Levee Core Fill - Undrained		110	Mohr-Coulomb	500	0
Select Fill Type 1		130	Mohr-Coulomb	0	34
Ballast Fill		130	Mohr-Coulomb	0	38
Riprap		130	Mohr-Coulomb	0	42



Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

E-E' STA 24+00, Long-Term Conditions Master Scenario

Slope Stability Analysis

Geotechnical Design Report
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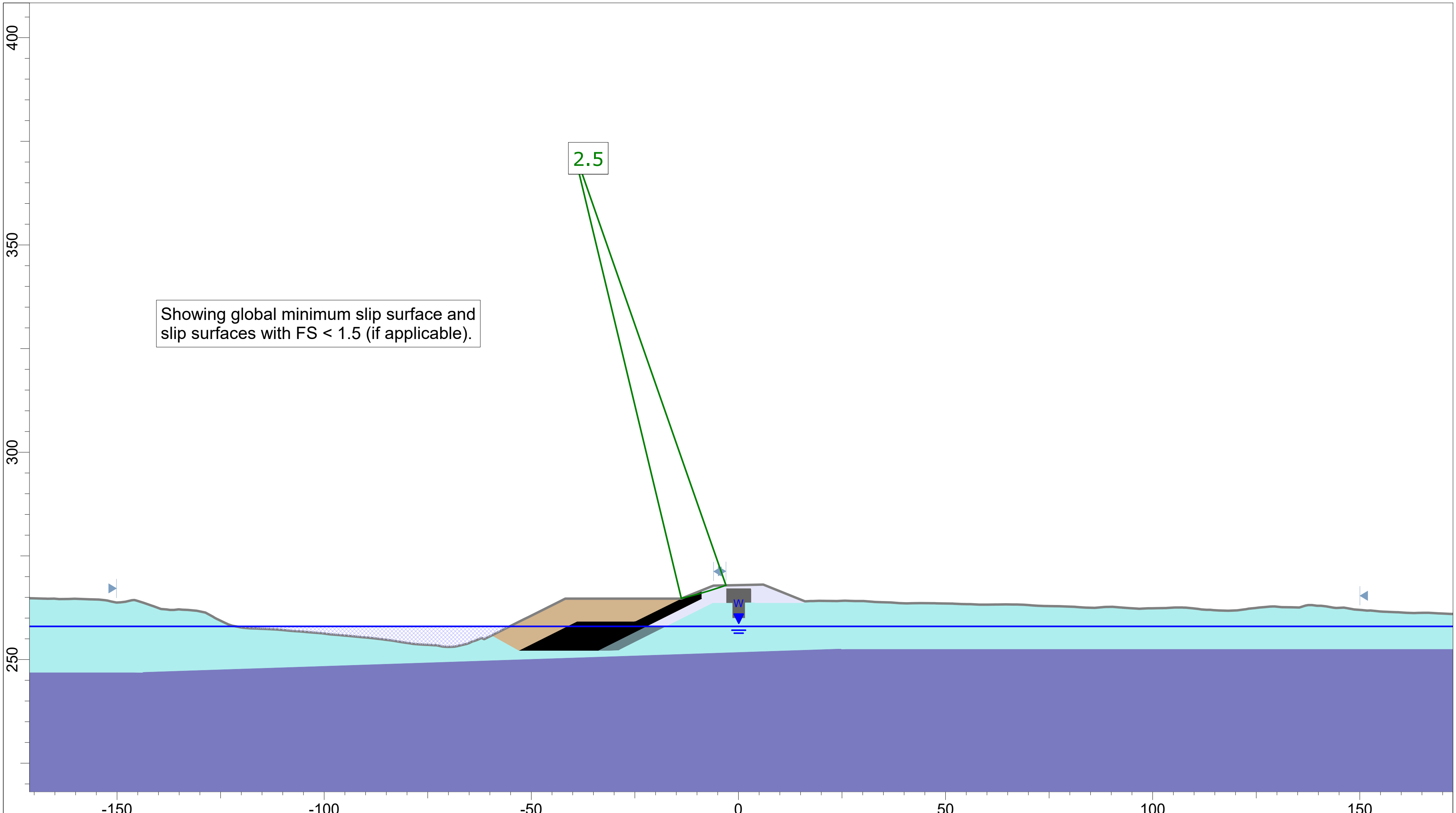
REVIEWED BY: A. Holmson

APPENDIX: **G-50**



Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

E-E' STA 24+00, End of Construction Waterward, Mean Annual WSE

Slope Stability Analysis

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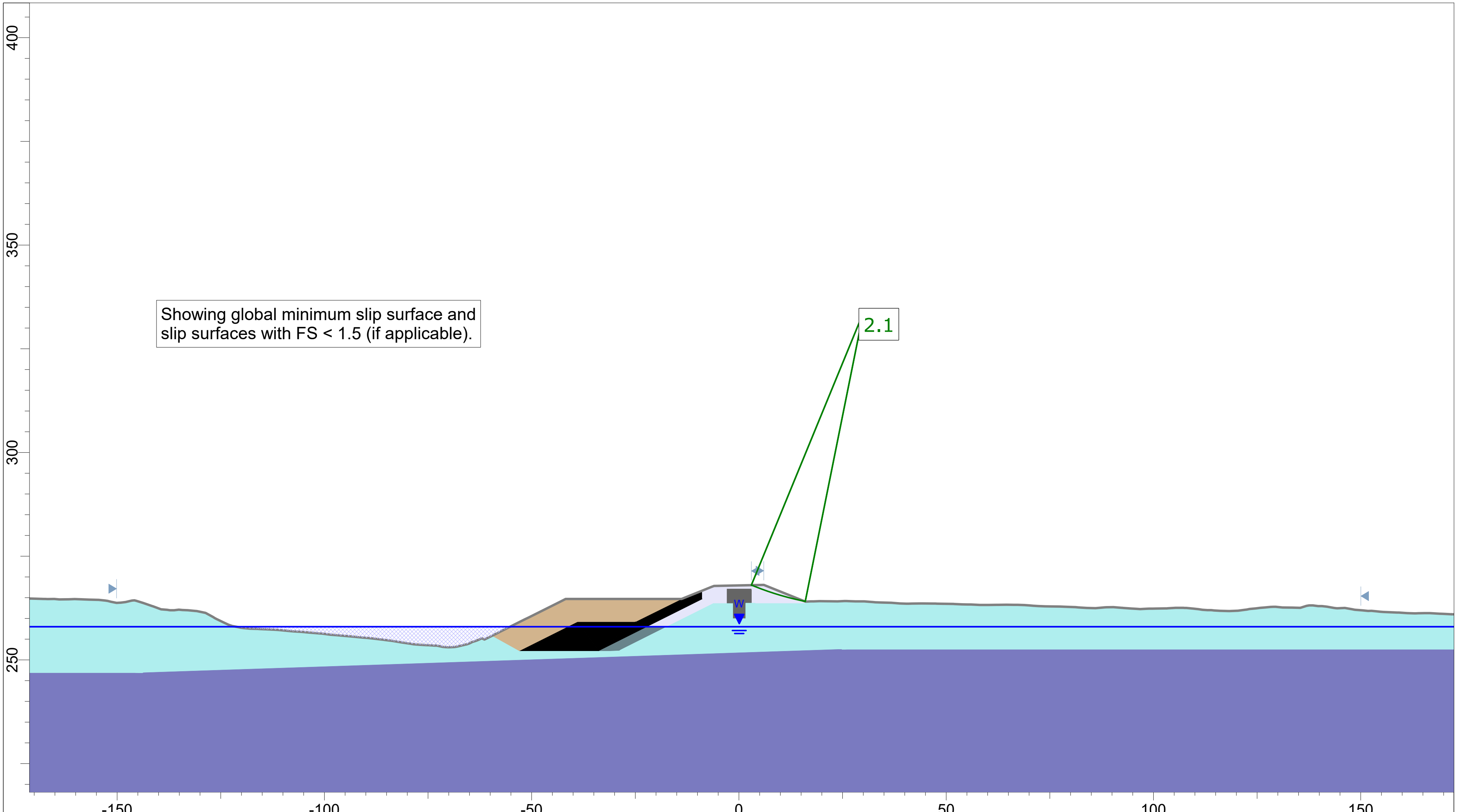

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- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

E-E' STA 24+00, End of Construction Landward, Mean Annual WSE

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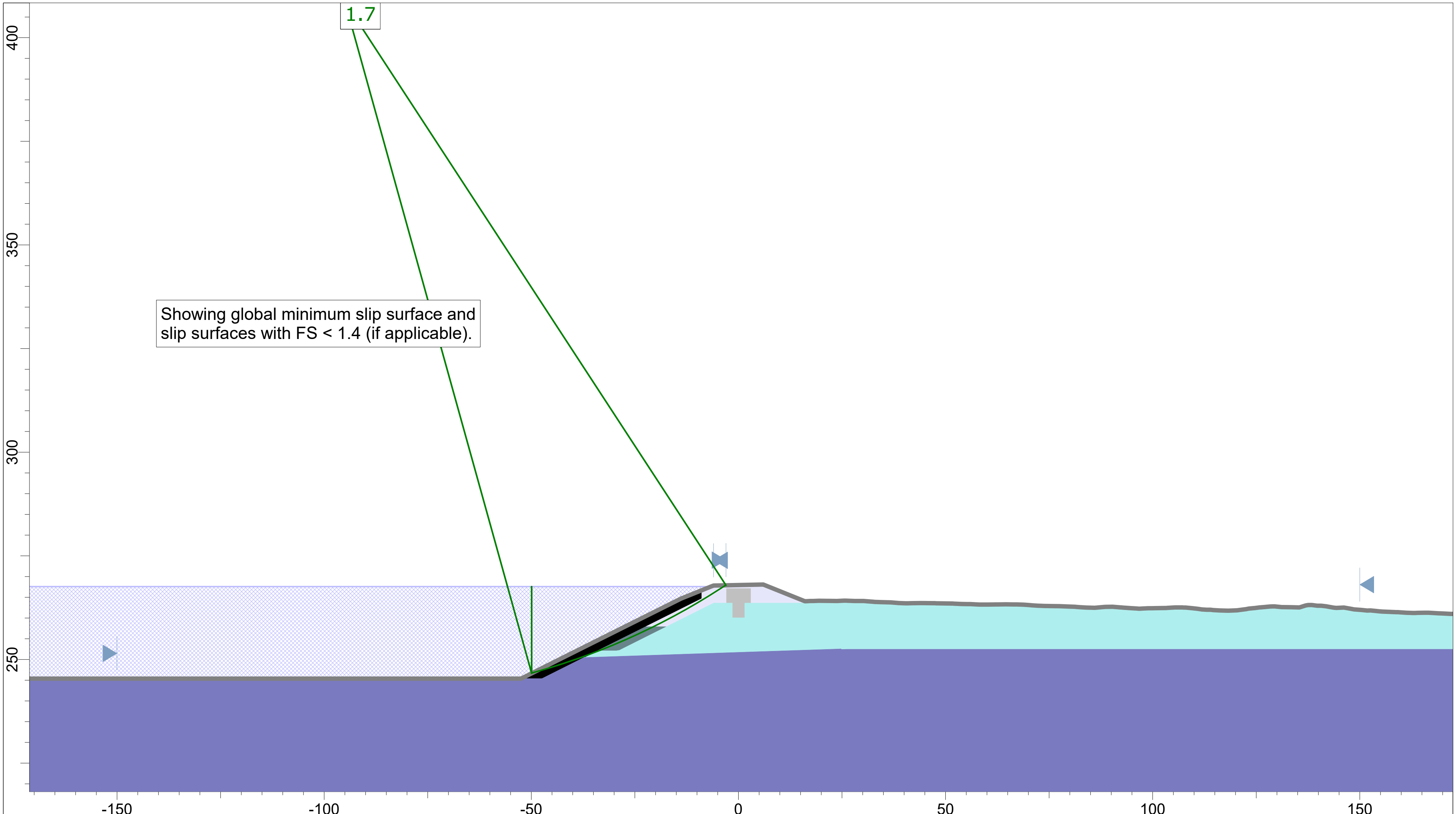
REVIEWED BY: A. Holmson

APPENDIX: **G-52**



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- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

E-E' STA 24+00, Long-Term Conditions Waterward, Steady-State Seepage, 100-Yr Flood

Slope Stability Analysis

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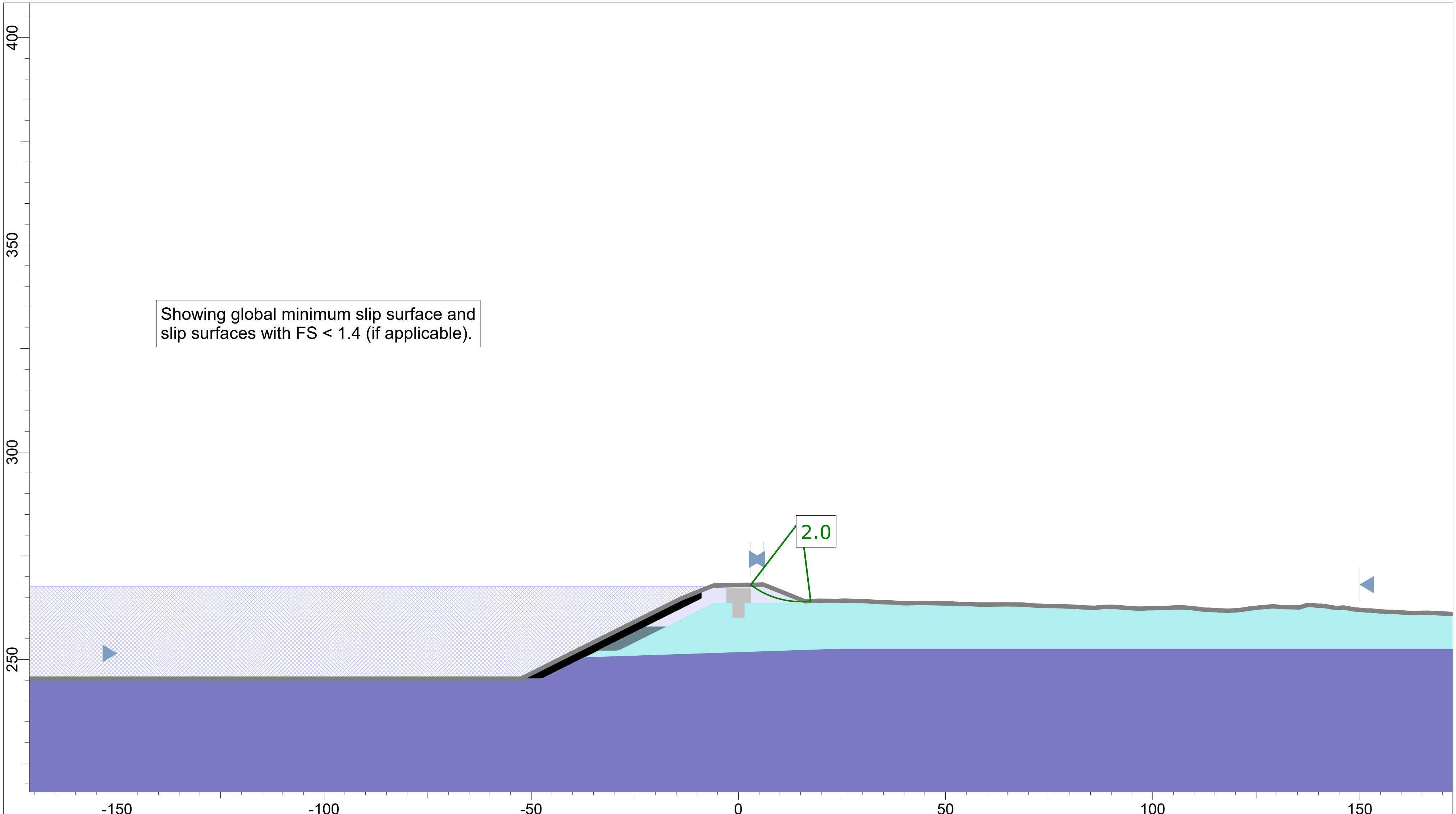

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- Modeled Groundwater Level
- Boring Location and Depth

E-E' STA 24+00, Long-Term Conditions Landward, Steady-State Seepage, 100-Yr Flood

Slope Stability Analysis

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1/28/2022	BY: M. Otto	APPENDIX:
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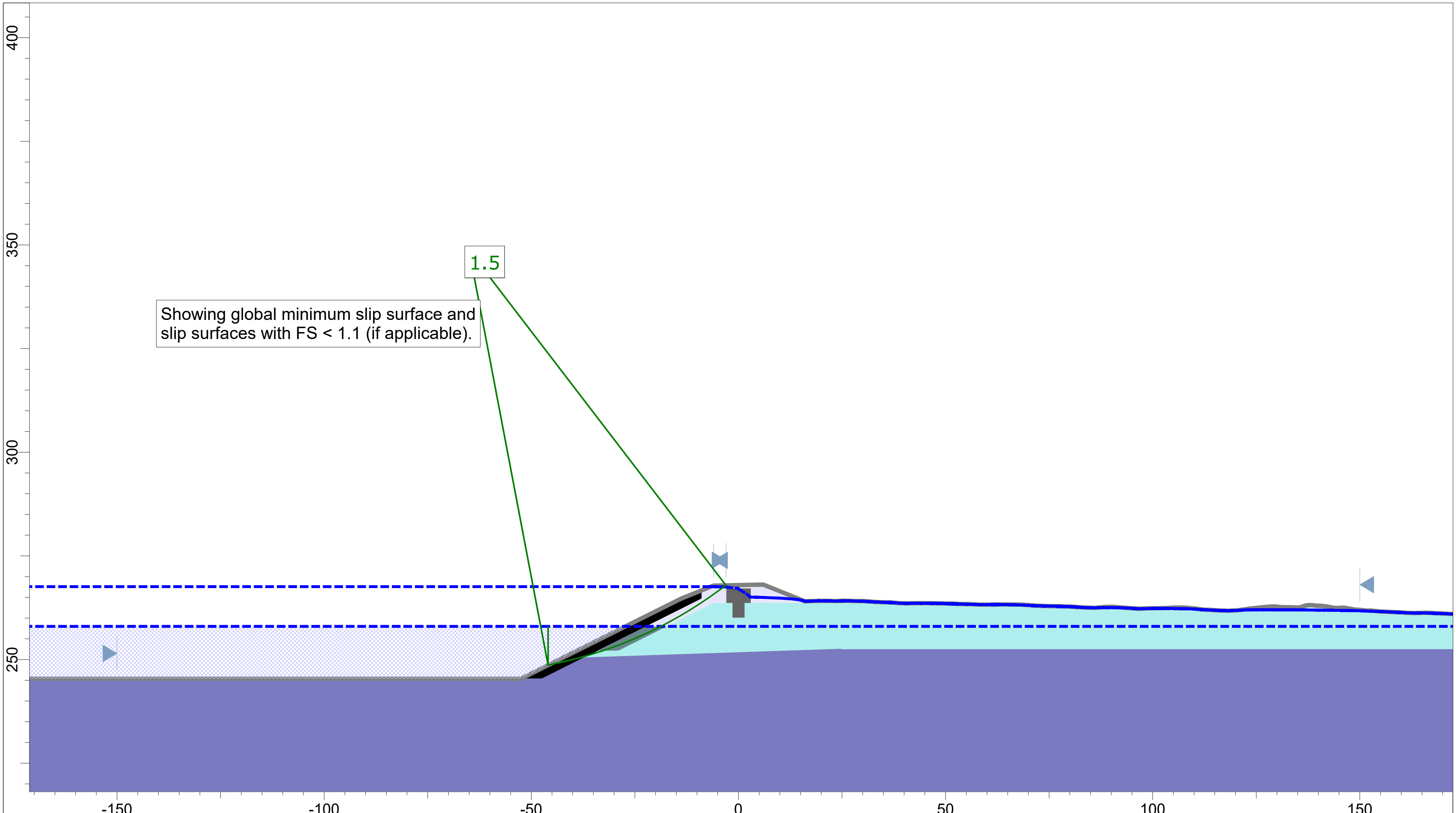

Legend

Search Grid

Search Limits

Modeled Groundwater Level

Boring Location and Depth

E-E' STA 24+00, Long-Term Conditions Waterward, Sudden Drawdown, 100-Yr Flood

Slope Stability Analysis

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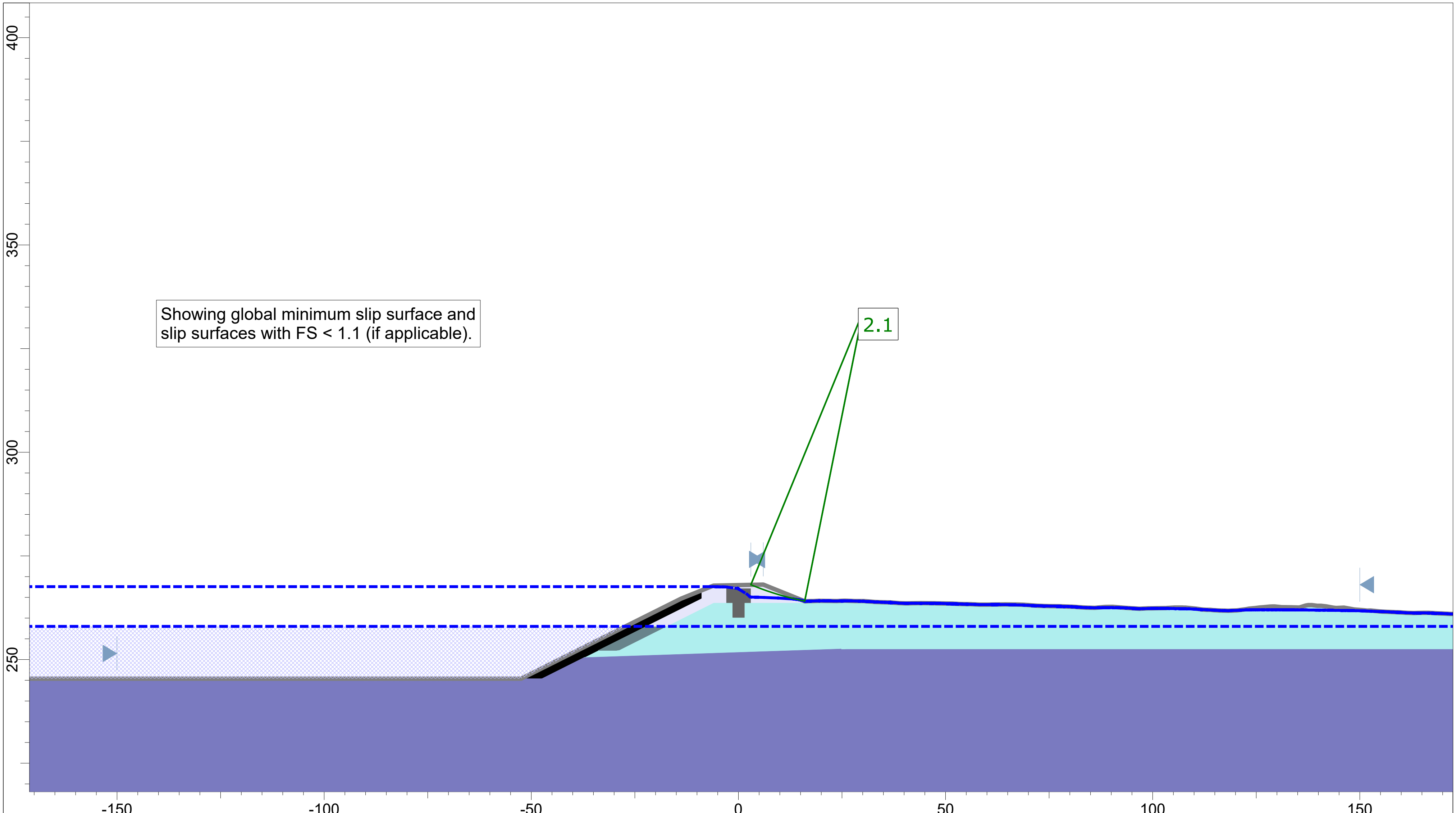
PROJECT NO. 190175-A

BY: M. Otto

REVIEWED BY: A. Holmson

APPENDIX: **G-55**



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- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

E-E' STA 24+00, Long-Term Conditions Landward, Sudden Drawdown, 100-Yr Flood

Slope Stability Analysis

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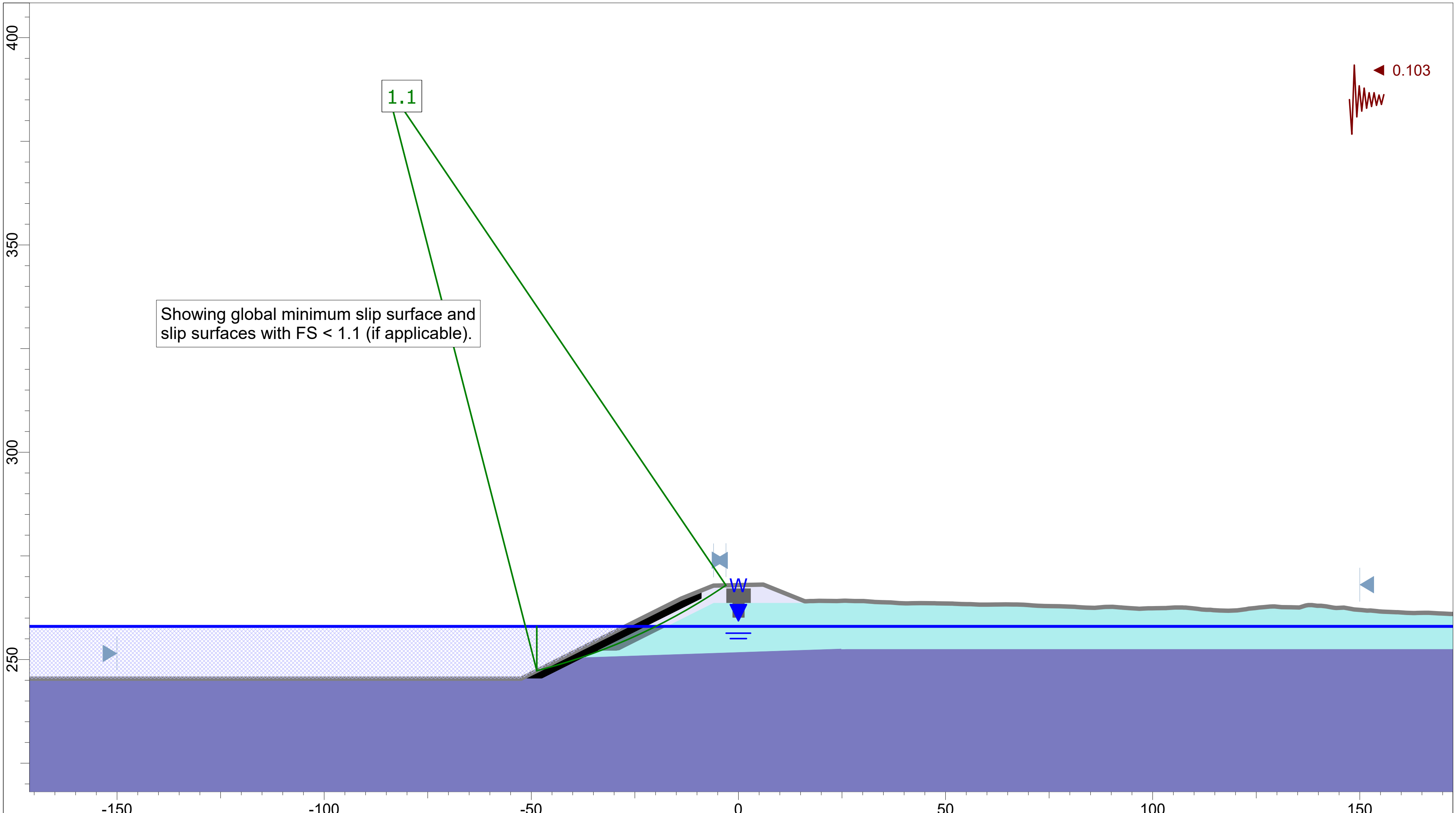

Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

E-E' STA 24+00, Long-Term Conditions Waterward, Seismic, OBE

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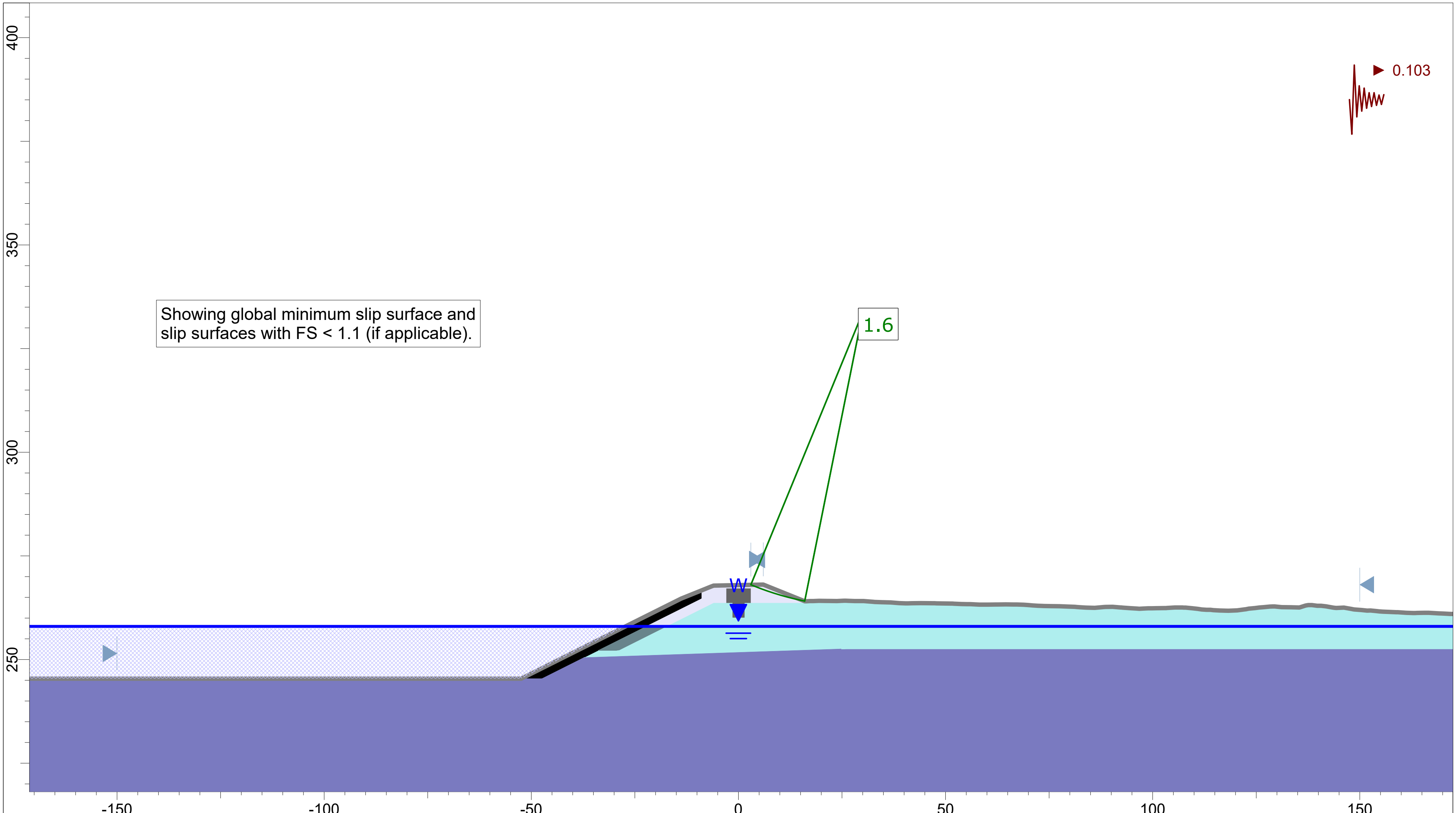
PROJECT NO. 190175-A	REVIEWED BY: A. Holmson	



Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

E-E' STA 24+00, Long-Term Conditions Landward, Seismic, OBE

Slope Stability Analysis

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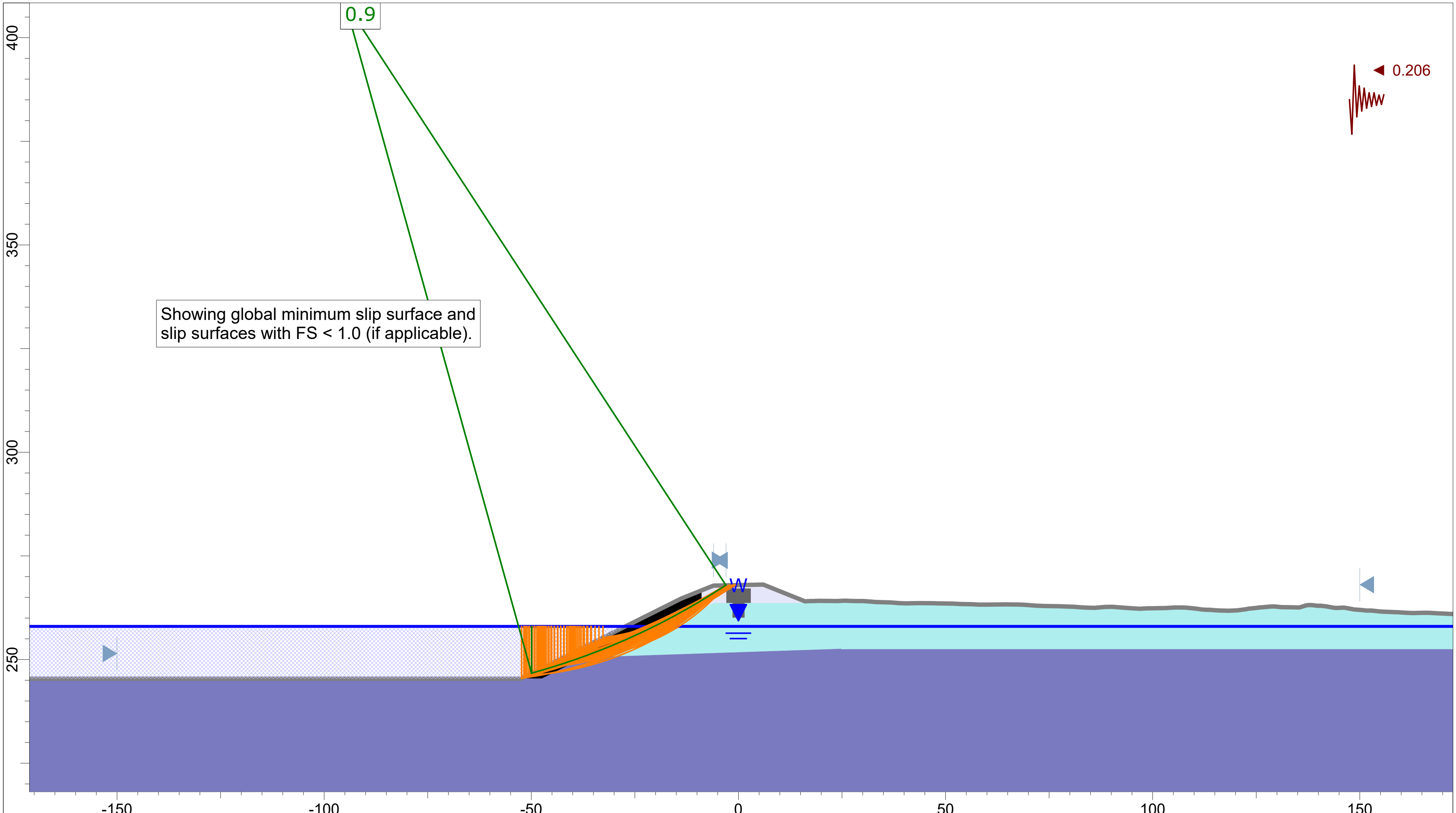

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- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

E-E' STA 24+00, Long-Term Conditions Waterward, Seismic, MDE

Slope Stability Analysis

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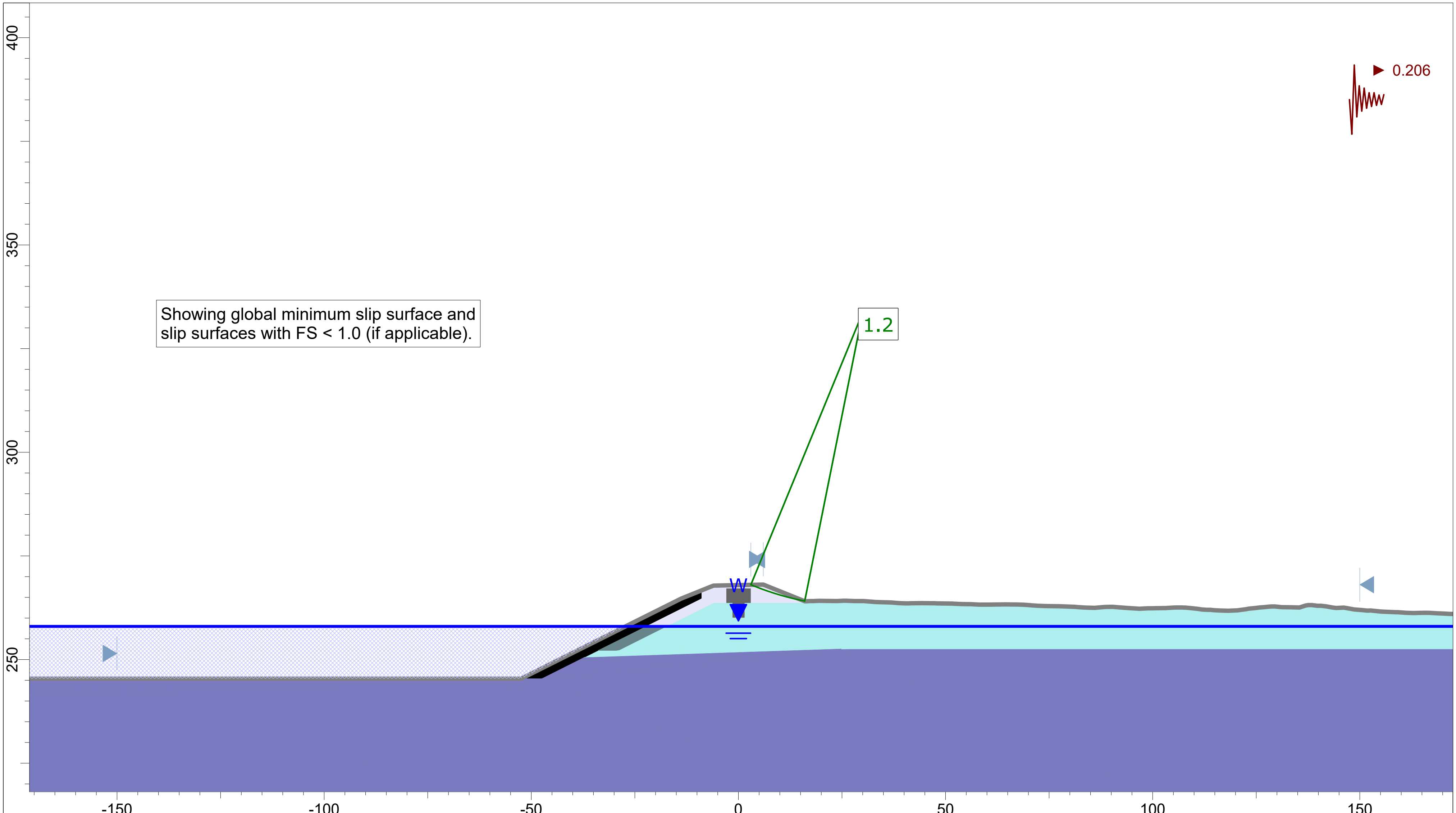
APPENDIX: **G-59**



Legend

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- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

E-E' STA 24+00, Long-Term Conditions Landward, Seismic, MDE

Slope Stability Analysis

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APPENDIX H

Liquefaction Analyses

H. Liquefaction Analyses

H.1. Methodology

Aspect conducted liquefaction evaluations using WSLiq, a liquefaction analysis software that was created as part of an extended research project supported by the Washington State Department of Transportation (WSDOT). Liquefaction occurs when loose, saturated, and relatively cohesionless soil deposits temporarily lose strength as a result of earthquake shaking. Primary factors controlling the development of liquefaction include intensity and duration of strong ground motion, characteristics of subsurface soil, *in situ* stress conditions, and the depth to groundwater. Potential effects of soil liquefaction include temporary loss of shear strength and liquefaction-induced settlement and ground movement, which could result in damage to the setback levee embankment or nearby structures.

Key inputs into WSliq are the subsurface profile (including standard penetration blow counts (“N-values”), fines contents, moisture content and groundwater levels, grain-size d_{50} values, Atterberg limits, surface geometry (including distance to the nearest free-face and height of the free face), and design earthquake conditions (including earthquake magnitude, distance to epicenter, and peak ground acceleration [PGA]).

WSliq computes the factor of safety against liquefaction (FS_{liq}), which is defined as:

$$FS_{liq} = CRR / CSR$$

Where “CRR” is the ratio of cyclic resistance ratio, defined as the capacity of the soil layer to resist liquefaction, and “CSR” is the cyclic stress ratio, defined as the seismic demand applied to the soil layer. For soil layers with FS_{liq} less than 1.0, WSliq can then compute the response spectrum of the soils, potential for lateral spreading and settlement, and residual strength of liquefied soils after the earthquake event.

Key outputs from WSliq include the FS_{liq} of the soil layers to liquefaction, anticipated settlement due to liquefaction, and magnitude of lateral spreading (lateral movement of a point of interest towards a free face – in this case, the Cedar River).

H.2. Design Assumptions and Criteria

We followed the procedures outlined in United States Army Corps of Engineers (USACE) Engineering Manual (EM) 1110-2-6053 *Earthquake Design and Evaluation of Concrete Hydraulic Structures* (USACE, 2007) to determine the seismic parameters for design presented in Table H-1 below. The USACE requires design for an Operating Basis Earthquake (OBE) and Maximum Design Earthquake (MDE), which are further defined in Section 4.4 of the main report and repeated below.

- The OBE is the level of ground motion for which the levee is able to maintain operation with little to no damage. It corresponds to a 50-percent probability of

exceedance over a project service life of 100 years, which corresponds to a 144-year return period event.

- The MDE is the maximum level of ground motion under which the levee should not experience catastrophic failure. It corresponds to a 10-percent probability of exceedance over a project life of 100 years, which corresponds to a 1000-year return period event. ‘=

Each design earthquake scenario has an associated moment magnitude (Mw) and effective peak ground acceleration (PGA).

The National Earthquake Hazard Mapping Program (NEHRP) expresses the effects of site-specific subsurface conditions on the ground motion response in terms of the “site class” (NEHRP, 2003). The site class can be correlated to the average standard penetration resistance (SPT) or average shear wave velocity in the upper 100 feet of the soil profile. Based on our observations from the subsurface exploration programs, we recommend Site Class D as a representative site class description of the upper 100 feet of the soil profile at the Site.

The USACE EM 1110-2-6053 presents guidelines for determining the effective PGA based on the soil Site Class designation. The U.S. Geological Survey (USGS) has completed probabilistic ground motion studies and design maps for the state of Washington (USGS, 2020b).

Table H-1 below outlines the earthquake parameters used in our liquefaction analyses.

Table H-1. Design Ground Motion Parameters

Parameter	OBE Value[1]	MDE Value[1]
Site Soil Class	“D” – Stiff Soil	
Approximate Return Interval	144-year	1000-year
Mean Magnitude Earthquake, M_w[2,3]	7.10	7.11
Mean Distance to Epicenter, km[2,3]	68.04	66.92
Short Period Spectral Acceleration, g (S_s)	0.34	0.91
Site Coefficient (F_a)	1.53	1.14
Design Short Period Spectral Acceleration (S_{DS})	0.52	1.14
Effective Peak Ground Acceleration, g (PGA)	0.21	0.41
Pseudostatic horizontal seismic coefficient, kh	0.106	0.206

Notes: (1) OBE = Operating Basis Earthquake; MDE = Maximum Design Earthquake
(2) Based on the latitude and longitude of the Site: 47.426601°, -122.051578°
(3) Values taken from the USGS Unified Hazard Tool (USGS, 2020a)

H.3. Design Conditions for Analysis

We performed liquefaction analyses at five critical sections along the proposed levee alignment. We used the mean annual water surface elevation (WSE) to represent groundwater conditions at these locations. Based on continued groundwater monitoring data collected at the Site between 2019 and 2021, we estimate mean annual groundwater to be present 5 feet below ground surface. We created the subsurface profiles used in the models based on reference explorations performed during our subsurface explorations, identified in Table H-2 below.

Table H-2. Levee Analysis Section Locations and Details

Section	Stationing(1)	Reference Exploration(s)
A-A’	3+50	ATP-06 AMW-03
B-B’	12+37.30	ATP-04 AB-03
C-C’	16+25	ATP-02 AB-01
D-D’	19+00	B-6
E-E’	24+00	B-4 AMW-01

Notes: (1) Stationing per draft 100 percent Project drawings and as shown on Figure 2.

H.4. Soil Parameters for Liquefaction Analysis

The soil stratigraphy used for each design section was based on our interpretation of the subsurface conditions as shown on the geologic profile (Figures 3-4) with corresponding engineering properties shown in Table 1. Standard penetration N-values input into WSliq are based on the reference explorations presented in Table H-2 (exploration logs are available in Appendix A). Relevant geotechnical laboratory testing is presented in Appendix B.

H.5. Liquefaction Analysis Results

Based on the results of our liquefaction analyses, we do not anticipate liquefaction, or corresponding vertical settlement and lateral spreading towards the Cedar River, to occur along the setback levee alignment during the OBE or MDE. Minimum factors of safety, as well as the approximate depth at which the minimum factor of safety against liquefaction was calculated, are included in Table H-3 below.

Table H-3. Liquefaction Analysis Results

Section	Stationing[1]	Minimum FS_{liq}, OBE[2]	Minimum FS_{liq}, MDE[3]	Approximate Depth of Min FS_{liq}, ft bgs[4]
A-A’	3+50	13.7	6.2	4
B-B’	12+37.30	5.1	2.6	4
C-C’	16+25	3.2	1.6	6
D-D’	19+00	13.7	6.8	20
E-E’	24+00	9	4.6	50

Notes: (1) Stationing per draft 100 percent Project drawings and as shown on Figure 2.
(2) OBE = Operating Basis Earthquake
(3) MDE = Maximum Design Earthquake
(4) ft = feet; bgs = below ground surface

H.6. Liquefaction Sensitivity Analysis

Since input parameters for liquefaction analysis are based on quantitative field and laboratory measurements (blow counts and particle size distribution) and prescriptive seismic peak ground accelerations, we did not perform in-depth sensitivity analyses for liquefaction.

APPENDIX I

Engineered Log Jams and Biorevetment – Timber Pile Capacity and Drivability

I. Engineered Log Jams and Biorevetment – Timber Pile Capacity and Drivability

The Project includes habitat and scour protection features along the left and right banks of the Cedar River. Included in the habitat and scour protection features are several different types of engineered log jams (ELJs) and biorevetments. The ELJs and a biorevetment structure on the waterside of the new setback levee between Stations 23+00 and 25+25 will be supported by driven timber piles. Design of the ELJs is being completed by King County with Aspect providing support for the geotechnical element. Design of the biorevetment is being completed by Tetra Tech, with Aspect providing support for the geotechnical elements of the timber pile design.

We conducted uplift capacity, lateral capacity, and drivability analyses of the timber piles. The following sections describe our analyses, including detailed methodologies, design assumptions and criteria, and results that have been incorporated into the 100-percent Project design.

I.1. General Design Assumptions and Criteria

Based on iterative design in collaboration with King County, we understand that there will be four types of ELJs with timber pile supports (designated ELJ Type 1 through Type 4). The configurations and locations of the ELJ structures are shown on Sheets SP2 and EW1 through EW11 of the 100-percent plans (Tetra Tech, 2021). Timber piles will also be used to support the biorevetment between Stations 23+00 and 25+25, as shown on Sheets BR5 of the 100-percent plans (Tetra Tech, 2021).

The timber piles will be subjected to drag-induced lateral forces from the 100-year flood event as well as uplift forces due to buoyancy and flow-induced lift. The magnitude of the forces on each structure is dependent on the size and configuration of the structure, including the number and size of the timber pile elements. An iterative design process was required to determine the number of timber piles needed to support each structure and design minimum pile size, length, and embedment depth.

The primary geotechnical design considerations for the timber pile supports are uplift capacity, lateral capacity, and drivability for pile installation. Discussion of design methodology, design criteria and assumptions, and results specific to each type of analysis are presented in Sections I-3 through I-5. General assumptions for the timber pile supports are summarized below.

Timber piles will be untreated Pacific Coast Douglas Fir round timber piles and will conform to ASTM International (ASTM) D-25, which is the standard specification for round timber piles. Timber pile material properties will be in general accordance with those shown in Table I-A below. Through iterative design in collaboration with King County, we identified pile lengths and diameters of two typical timber piles to be used for the Project. The timber piles will be:

- 50-foot-long piles with tapered diameters ranging from 18 inches at the butt to 14 inches at the tip

- 35-foot-long piles with tapered diameters ranging from 16 inches at the butt to 12 inches at the tip

Table I-A. Timber Pile Material Properties

Modulus of Elasticity (ksi)(1,2)	Allowable Working Compressive Stress (ksi)(1,3)	Maximum Dynamic Compressive Stress (ksi)(1,3)	Nominal Shear Stress Capacity (psi)(1,2)	Nominal Bending Stress Capacity (psi)(1,2)
1,500	1.25	3.8	160	2,050

Notes:

1. ksi = kips per square inch; psi = pounds per square inch
2. Per the 2012 National Design Specification (NDS) for Wood Construction, Load and Resistance Factor Design (LRFD) standards for round timber piles (AWC, 2012).
3. The timber piles will be subjected to compressive stresses during driving with a vibratory or impact hammer. Per the Timber Pile Design & Construction Manual (AWPI, 2016), the maximum dynamic stress (short term stresses due to pile installation) is three times the allowable working stress of a timber pile.

The predicted scour for individual structures refers to localized scour as opposed to widespread (throughout the floodplain) channel degradation scour. Localized and channel degradation scour are illustrated in Figure I-A below. Further discussion of scour processes and extents is included in the Basis of Design Report (Tetra Tech, 2022).

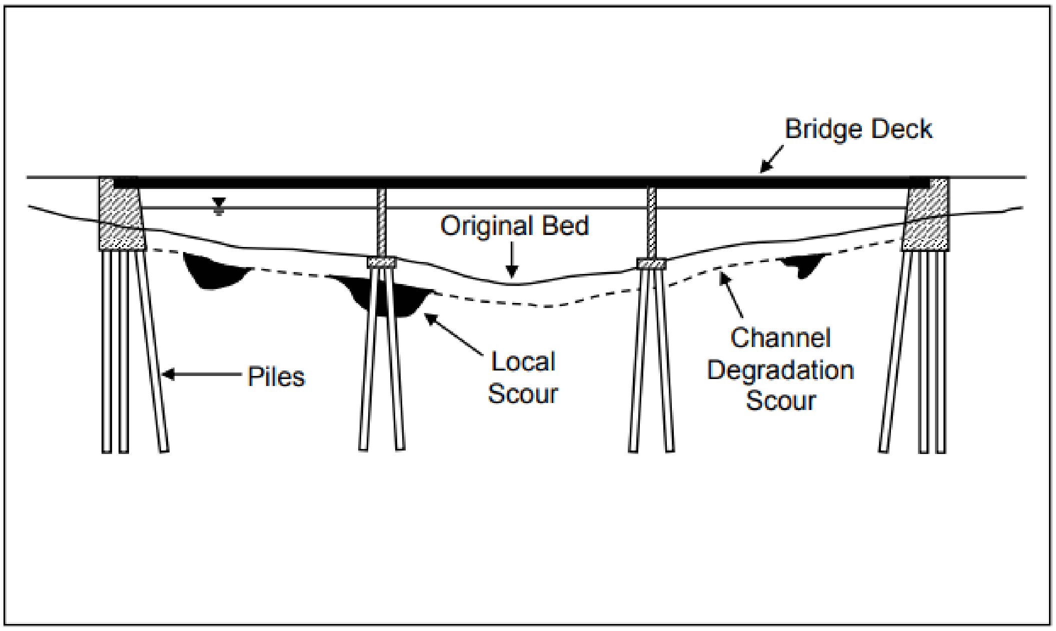

Figure I-A: Illustration of Local versus Channel Degradation Scour. Figure 7-36 from the Federal Highway Administration (FHWA) Design and Construction of Driven Pile Foundations (FHWA, 2016).

I.2. Soil Parameters for Wood Pile Design

The soil conditions at the ELJ and biorevetment locations were derived based on the existing subsurface exploration data for the Project described in Appendix A, B, and C of this report. Timber piles will generally be installed through medium dense to dense Alluvial Deposits – Gravel Facies, and very dense Pre-Fraser Deposits. Scour estimates provided by King County and Tetra Tech (Tetra Tech, 2021) indicate the timber piles will primarily gain long-term support against lateral and uplift forces from very dense Pre-Fraser Deposits, with some contribution from the overlying dense Alluvial Deposits – Gravel Facies. The engineering properties for these soil deposits are shown in Table I-B below.

Table I-B. Soil Parameters for Use in Uplift Capacity, Lateral Capacity, and Drivability of Timber Piles

Soil Layer	Unit Weight / Effective Unit Weight (pcf)[1]	Effective Friction Angle (degrees)	Undrained Cohesion (psf)[1]	LPILE Soil Model	Soil Modulus, k (pci)[1]
Alluvial Deposits – Gravel Facies, medium dense to dense	130 / 67.6	38 - 42	N/A	Sand (Reese)	125
Pre-Fraser Deposits, very dense	135 / 72.6	45	N/A	Sand (Reese)	125

Notes: 1. pcf = pounds per cubic foot; psf = pounds per square foot; pci = pounds per cubic inch

I.3. Pile Uplift Capacity

I.3.1. Methodology

We evaluated the uplift capacity of the timber support piles using the Nordlund pile capacity method (load carrying capacity of a single pile in cohesionless soils), as presented in the Federal Highway Administration (FHWA) design manual for design and construction of driven pile foundations (FHWA, 2016) and adopted by the Timber Pile Design and Construction Manual (AWPI, 2002).

The uplift capacity of a pile is a function of its skin friction capacity and effective weight. Key inputs into the Nordlund uplift capacity calculations are the soil parameters (friction angle of the soil, friction angle of the soil/pile interface, effective unit weight of the soil), the pile unit weight (assuming completely saturated piles), geometry (pile taper angle, length, perimeter), and the volume of soil displaced during pile installation.

The key output of the uplift capacity calculation is the pile’s ultimate capacity to upward (pullout) forces. A factor of safety of 3 is typically applied to the ultimate uplift capacity to determine the allowable uplift capacity per pile.

I.3.2. Design Assumptions and Criteria

We applied the following assumptions to our uplift capacity analyses:

- We assumed the pile dimensions, lengths, embedment below predicted scour elevations, and applied buoyant force shown in Table I-1 (attached), which were provided by King County and Tetra Tech and reported in the Basis of Design Report (Tetra Tech, 2022). These values were determined through an iterative design process.
- We assumed a pile taper angle of zero in determining the coefficient of lateral earth pressures along the pile. Using the Nordlund equation, this assumption produces conservative uplift capacity results.
- We assumed only soils below the predicted scour depth will contribute to the pile uplift capacity.
- We assumed localized scour conditions. Where localized scour conditions are expected, it is typically acceptable to neglect scour when calculating the effective overburden pressures within uplift capacity calculations. Therefore, we calculated effective overburden pressures along the length of the pile using the proposed ground elevation rather than anticipated scour elevation.
- We assumed effective overburden pressures increase with depth until 20*D (where D is the average pile diameter), below which overburden pressures are constant.
- We recommend a factor of safety of 3 be applied to the ultimate uplift capacity results to determine the allowable uplift capacity.

I.3.3. Conditions for Analysis

We created the subsurface profiles for our analyses based on subsurface explorations that we consider to be conservatively representative of subsurface conditions for each ELJ type and the biorevetment support piles, shown on Table I-1 (attached). Logs for these explorations are presented in Appendix A.

The planned pile geometry, pile top elevations, predicted 100-year flood water surface elevation (WSE), predicted scour depth, and predicted total uplift force on the timber piles were provided by King County and Tetra Tech and reported in the Basis of Design Report (Tetra Tech, 2022), and are shown in Table I-1 (attached).

I.3.4. Uplift Capacity Analysis Results

We compared the applied buoyant force on single piles within each structure to the pile's allowable capacity to determine recommended minimum pile lengths and embedment below their respective anticipated scour elevations. These values were also used to determine the minimum pile driving depth below the planned construction platform elevation at each structure.

The results of our uplift capacity analyses are summarized in Table I-1 (attached).

I.3.5. Uplift Capacity Sensitivity Analysis

We conducted sensitivity analyses of the uplift capacity models to account for the inherent variability of natural soils deposits and predicted scour depths and to determine sensitivity to changes in pile length, diameter, and number of support piles per structure. We performed our sensitivity analyses using the model for ELJ Type 2, which is anticipated to be subjected to the highest uplift forces and deepest predicted scour. The variables adjusted in our sensitivity analyses, the range of adjusted values, and resulting range in allowable uplift capacities are shown in Table I-C below.

Table I-C. Uplift Capacity Sensitivity Results

Variable(1)	Variable Range(1)	Range in Allowable Uplift Capacity (kips)
Alluvial Deposits – Gravel Facies, φ’	38 ± 4 deg	15.9 ± 0.0
Alluvial Deposits – Gravel Facies, γ_{sat}	135 ± 15 pcf	15.9 ± 0.8
Pre-Fraser Deposits, φ’	45 ± 5 deg	15.9 ± 1.5
Pre-Fraser Deposits , γ_{sat}	140 ± 15 pcf	15.9 ± 1.2
Pile Length	50 ± 5 ft	15.9 ± 4.5
Average Pile Diameter	16 ± 2 inch	15.9 ± 1.9
Predicted Scour Depth	15.4 ± 5 ft	15.9± 2.7

Notes: **(**1) φ’ = effective friction angle; γ_{sat} = saturated unit weight; deg = degree; pcf = pounds per cubic foot; ft = foot.

Based on the results we conclude the uplift capacity at the timber piles is not sensitive to changes in the effective friction angle of Alluvial Deposits – Gravel Facies, and moderately sensitive to the other variables included in Table I-C.

I.4. Pile Lateral Capacity

I.4.1. Methodology

We performed lateral pile analyses using the computer model LPile (Ensoft, 2019) to determine the lateral capacities of the ELJ support piles. The LPile program is based on rational procedures for analyzing pile deflections and stresses under lateral loading using the lateral load-transfer (p-y) method. Using this program, we can model an individual pile as a beam-column, supported by soil type-specific p-y curves, and assess its reaction as it is subjected to external loads.

Key inputs into LPile are the subsurface profile and the soil modulus (“k”) parameter, which is based on soil type and relative density as determined from nearby reference subsurface explorations, and position in relation to groundwater or design flood elevation. Other key inputs were the pile type and size, anticipated scour elevation, the 100-year flood water surface elevation (WSE), and the corresponding total drag force from the 100-year flood event on the ELJ structure; these values were provided to us by King County and Tetra Tech.

Key outputs from LPile are the anticipated deflection, shear, and moment diagrams of the pile under the given external loads. We compared these outcomes to the maximum factored bending and shear stress of a single pile (a function of pile material and size) to determine the appropriate number of support piles needed per ELJ to support the total drag force on the structure. LPile outputs also depict the depths at which deflection, shear, and moment fixity is achieved; the resulting depth of fixity was used to recommend minimum pile lengths and embedment depths.

I.4.2. Design Assumptions and Criteria

We assumed the following:

- Maximum long-term scoured conditions during the 100-year flood event to represent the worst-case loading condition.
- The hydraulic drag forces would be distributed equally among the support piles in the structure, and the hydraulic drag force on each pile would be applied as a uniform load over the height of exposed pile between the 100-year flood elevation and the anticipated maximum scour elevation.
- Piles in LPILE could be modelled using the 'free-head' condition, meaning the top of pile is not restrained against lateral movement.
- No axial load or moment will be applied at the top of pile.
- Pile spacing in the primary direction of loading will be on the order of 7 feet or greater. For piles up to 20 inches in diameter with this spacing, we did not consider group interaction effects.
- The lateral capacity of the timber piles is limited by a lateral deflection at the pile top of 5 feet, and by the maximum factored shear and bending stresses determined for the design pile sections.

We determined the maximum factored shear and bending stresses of an individual pile using guidance from the 2012 NDS for Wood Construction LRFD standards for timber piles (AWC, 2012). This guidance calculates the maximum factored shear stress of a timber pile as

$$F_v' = (F_v)(C_t)(C_{ct})(2.88)(0.75)(\lambda) \qquad \text{(Eqn I-1)}$$

and maximum factored bending stress of a timber pile as

$$F_b' = (F_b)(C_t)(C_{ct})(C_F)(C_{ls})(2.54)(0.85)(\lambda) \qquad \text{(Eqn I-2)}$$

The variables in the equations above are further explained in Table I-D below:

Table I-D. Timber Pile LRFD Factored Stress Inputs

Input	Description	Design Value
Fv	Nominal shear stress capacity of a timber pile (psi)	160 psi [1]
Fb	Nominal bending stress capacity of a timber pile (psi)	2,060 psi [1]
Ct	Temperature factor (unitless)	1.0 [2]
Cct	Condition treatment factor (unitless)	1.0 [3]
CF	Size factor (unitless)	1.0 [4]
Cls	Load sharing factor (unitless)	1.08 (Type 2 ELJ piles) 1.05 (all other piles) [5]
λ	Time effect factor (unitless)	0.9 [6]
Fv’	Maximum factored shear stress of timber pile	311 psi [7]
Fb’	Maximum factored bending stress of timber pile	4.302 psi (Type 2 ELJ piles) 4,182 psi (all other piles) [8]

Notes:

1. Referenced from Tabel 6A from the National Design Specification Supplement, Design Values for Wood Construction (AWS, 2018). Assumes values for a Pacific Coast Douglas Fir timber pile, treated per ASTM D25.
2. References Section 2.3.3 of the 2012 NDS for Wood Construction (AWC, 2012). Assumes timber piles will generally be exposed to temperatures below 100 degrees Fahrenheit.
3. References Section 6.3.5 of the 2012 NDS for Wood Construction (AWC, 2012). Assumes air-dried conditioning.
4. References Sections 4.3.6.2 and 4.3.6.3 of the 2012 NDS for Wood Construction (AWC, 2012). Calculated based on the average pile diameter.
5. References Table 6.3.11 of the 2012 NDS for Wood Construction (AWC, 2012). Seven support piles were assumed for ELJ Type 2 structures. Two support piles are assumed for ELJ Type 3 and 5 structures.
6. References Appendix N.3.3 of the 2012 NDS for Wood Construction (AWC, 2012). We anticipate the 100-year flood event will subject the ELJ pile supports to lateral loads over the course of a few hours. The NDS guidelines do not present a specific time effect factor for a flood event of such duration, so we have chosen a time effect factor that we consider conservative for the anticipated loading duration.
7. Calculated per Eqn I-1.
8. Calculated per Eqn I-2. Results differ based on proposed pile count (See Note 5).

I.4.3. Conditions for Analysis

We created the subsurface profiles for our analyses based on subsurface explorations that we consider to be conservatively representative of subsurface conditions for each ELJ Type and the biorevetment support piles, shown on Table I-1 (attached). Logs for these explorations are presented in Appendix A.

The planned pile geometry, pile top elevations, predicted 100-year flood water surface elevation (WSE), predicted scour depth, and predicted total uplift force on the timber

piles were provided by King County and Tetra Tech and reported in the draft Basis of Design Report (Tetra Tech, 2022), and are shown in Table I-1 (attached).

I.4.4. Lateral Pile Capacity Analysis Results

The results of our lateral capacity analyses are summarized in Table I-1 (attached).

In general, our results indicate that the minimum pile lengths determined in the uplift capacity analysis satisfy lateral capacity requirements.

I.4.5. ELJ Wood Recruitment Analysis Results

At the request of King County, we performed additional lateral capacity analyses on each ELJ structure that considered the potential for wood recruitment (wood debris collected by the ELJ structure during a flood event) on the ELJ structure. Our analyses accounted for an increase in lateral drag force and decrease in predicted scour that would result from 133 percent additional wood recruited. The drag loads and predicted scour for each ELJ structure in the wood recruitment scenario were provided by King County and are shown on Table I-1 (attached).

The results of the wood recruitment check indicate that the lateral capacity of ELJ Type 2 structures would be exceeded, but not for the other ELJ structures. Based on discussion with King County, we understand that this exceedance is acceptable in the 133 percent wood recruitment scenario. This is further discussed in the Basis of Design Report (Tetra Tech, 2022).

I.4.6. Lateral Capacity Sensitivity Analysis

We conducted sensitivity analyses of the lateral capacity models to account for the inherent variability of natural soils deposits and predicted scour depths and to determine sensitivity to changes in pile length, diameter, and number of support piles per structure. We performed our sensitivity analyses using the model for ELJ Type 2, which is anticipated to be subjected to the largest drag forces and deepest predicted scour. Additionally, we assessed the sensitivity of the pile lateral capacities to variations in distribution of the drag force applied to them, with further discussion below.

Descriptions and results of our sensitivity analyses are described below:

- We isolated the friction angles of the Alluvium – Gravel Facies and Pre-Fraser deposits and varied them between 90 and 100 percent of their design values shown in Table 4, which corresponds to a range of 34 to 38 degrees for the gravel facies and 40 to 45 degrees for the Pre-Fraser deposits. These changes resulted in almost no change to the bending and shear stresses on the pile (very low sensitivity), minimal effect on the predicted pile top deflection (+4 inches; minor sensitivity), and no change to the minimum required pile embedment (no sensitivity).

- We isolated the lateral load distribution on the pile and varied it between a uniform load between the 100-year flood WSE and the predicted scour depth (H) and a point load applied at 0.6*H. Our results show the pile bending and shear stresses are very sensitive to the load distribution pattern, with a point

load distribution being more critical than a uniform load distribution. However, we rationalize using a uniform load distribution for design with the following reasoning:

- Hydraulic drag forces will occur relatively uniformly over the portion of the log where horizontal members will be chain-lashed (over the upper 15 feet or so of the structure).
- In a flood event, scour is predicted to occur unevenly in/around the structure, and there will be debris (largely natural wood debris) that will get caught up in the ELJs, perhaps throughout the water column. The debris could potentially load up the entire pile length.

- We isolated the predicted scour depth and varied it between 75 and 125 percent of the predicted scour, or between 15 and 25 feet below proposed ground surface. These changes resulted in +-1000 pounds per square inch (psi) in bending stress (high sensitivity), +-50psi in shear stress (moderate sensitivity), +-10 inches in pile top deflection (moderate sensitivity), and +- 5 feet in minimum pile embedment (high sensitivity).
- We isolated the pile diameter (keeping the taper from top to bottom consistent) and varied it between a 16-inch top/12-inch bottom and a 20-inch top/16-inch bottom. These changes resulted in +-2000 psi in bending stress (high sensitivity), +-50 psi in shear stress (moderately sensitive), +-13 inches in pile top deflection (moderately sensitive), and +-0 feet in minimum pile embedment (not sensitive).

I.5. Drivability

I.5.1. Methodology

We completed drivability analyses for timber pile supports using the computer program GRLWEAP (Pile Dynamics, 2010). GRLWEAP is a one-dimensional wave equation analysis program that simulates motions and forces in a pile when driven by either an impact or vibratory hammer, and can be used to select appropriate hammers and driving systems with known pile parameters and soil conditions. For this Project, we assessed the feasibility of both vibratory hammers and open-ended diesel impact hammers to install timber piles at the Site.

Key inputs into GRLWEAP are the subsurface profile and soil types and densities (correlated from standard penetration blow counts [“N-values”]), hammer size and driving system, and pile type, size, and material properties. The subsurface profiles were modelled based on representative drilled borings performed during the subsurface exploration program. Hammer sizes and driving systems were chosen from a library within the GRLWEAP program. The pile type and size were determined through iterative design with King County and TetraTech, and are documented in the 100 percent draft Project plans (Tetra Tech, 2022). Pile material properties were derived from the 2012 NDS for Wood Construction (AWC, 2012), as shown in Table I-D.

The key output from GRLWEAP is the penetration rate through the subsurface at prescribed depth increments (in seconds per foot for vibratory hammers and in blows per foot for impact hammers), which can be used to estimate total installation time for a pile and identify driving depth at which driving refusal may occur. Aa key output was an estimate of driving stresses imparted to the pile during driving, which is used to anticipate if the piles will be overstressed during driving.

I.6. Design Assumptions and Criteria

We understand pile spacing will be on the order of 7 feet or greater. For piles with an average diameter of 14 to 16 inches, we do not anticipate soil densification during pile driving will be a design factor.

Timber piles will not be subjected to significant axial loads; therefore, the ultimate toe capacity of the piles were not a major design requirement for the Project.

Design recommendations for pile installation time and maximum compressive stress on the pile are described below:

- We recommend a practical refusal criteria of up to 300 seconds per foot of pile (for vibratory hammers) and about 100 blows per foot of pile (for impact hammers) to maintain economical installation times.
- Per the Timber Pile Design & Construction Manual (AWPI, 2016), we recommend a maximum dynamic driving stress (short term stress due to pile installation with a hammer) of 3.8 kips per square inch (ksi).

Input values into GRLWEAP for drivability analyses with vibratory hammers and impact hammers are summarized in Table I-E below.

Table I-E. Input Parameters for Drivability Analysis Using Vibratory Hammers

Input Name	Input Value for Vibratory Hammers	Input for Impact Hammers
Soil Damping Option[(1)]	Smith-Viscous	Smith
Pile Damping Option[(2)]	5	5
Hammer Damping Option[(3)]	2	2
Residual Stress Analysis[(4)]	0	0
Soil Setup Factors[(5)]	2.5	1
Toe Gain/Loss Factor[(6)]	0.7	0.7
Shaft Quake[(7)]	0.1 inch	0.1
Toe Quake[(8)]	D/120 inch[(12)]	D/120 inch[(12)]
Shaft Damping[(9)]	0.1 second / foot	0.1 second / foot
Toe Damping[(10)]	0.15	0.15
Hammer Efficiency[(11)]	0.5	0.84
Driving system	N/A	Default driving system for timber piles[(13)]

Notes:

1. Soil damping model per GRLWEAP user manual recommendations for vibratory and impact hammers, respectively.
2. Pile damping model per GRLWEAP user manual recommendations for timber piles.
3. Hammer damping model set to default (input value of 2) per GRLWEAP user manual recommendations.
4. Residual stress analysis is not recommended for piles that are not tapered high-yield strength piles.
5. Soil setup factors are recommended for sands/gravels + hammer type.
6. Toe Gain/Loss Factor is recommended for saturated sands/gravels.
7. Shaft quake values are recommended for all soil types.
8. Toe quake values are recommended for dense or hard soils + displacement piles.
9. Shaft damping model is recommended for non-cohesive soils + hammer type.
10. Toe damping model is recommended for all soil types.
11. Vibratory hammer efficiency as recommended by hammer manufacturers. Hammer efficiency for impact hammer was chosen to be consistent with the Washington State Department of Transportation (WSDOT) 2022 *Standard Specifications for Road, Bridge, and Municipal Construction* recommendations for single-acting diesel hammers (WSDOT, 2022).
12. D = pile diameter at the toe of the pile.
13. The default driving system for timber piles in GRLWEAP specifies the following: lead size = 21 inches or less; helmet weight = 3 kips; hammer cushion consisting of 2-inch-thick nylon with a modulus of elasticity of 175 kips per square inch (ksi), an area of 398 square inches, and a coefficient of restitution of 0.92.

I.7. Conditions for Analysis

We performed drivability analyses at two representative ELJ structure locations on the left bank and right bank of the Cedar River. We created the subsurface profiles for our analyses based on reference explorations, namely AB-05 and AB-03. Logs for these explorations are presented in Appendix A. In general, subsurface conditions at these locations consist of up to 15 feet of Alluvial Deposits – Gravel Facies (N-values ranging from 13 to 56 blows per foot [bpf]) atop Pre-Fraser deposits (N-values greater than 50 bpf).

Due to the generally consistent subsurface conditions throughout the Site, we consider the subsurface models used for drivability analyses to be conservatively representative of Site conditions.

I.7.1. Drivability Analysis Results

The results of our drivability analyses indicate that vibratory hammer driving on its own is not feasible at the Site because the dense and coarse nature of the subsurface soils would require oversized hammers (APE 600 or equivalent) that we predict will overstress and damage the piles.

Our results indicate that impact hammer driving is marginally feasible at the Site with an open-ended diesel hammer similar in size to an APE D12-42. Such a hammer could theoretically drive the timber piles to just about 55 feet below ground surface before reaching practical refusal. GRLWEAP outputs for impact hammer driving are presented in Figures I-1 through I-4 (attached).

In our opinion and based on our subsurface exploration data and experience with local geology, cobbles and boulders should be anticipated in the subsurface and could present difficult driving conditions that cannot be fully captured using GRLWEAP. Because of this, we recommend the following:

- We anticipate that pile driving with a vibratory hammer will require oversized hammers that risk overstressing the timber piles. Therefore, we consider pile drivability to be infeasible at the Site with only the use of vibratory hammers.
- Our drivability results show that pile driving is marginally feasible at the Site with an impact hammer.
- Due to the presence of cobbles and boulders in the subsurface, and the associated potential for hard driving conditions, we recommend the pile locations be pre-drilled to within 2 feet of the pile tip elevation prior to driving the piles. The diameter of the pre-drill should be approximately the diameter of the timber pile tip or slightly larger.
- We anticipate the Contractor will elect to use pre-drilling in conjunction with impact hammer driving to install the piles. The impact hammer used should be appropriately sized for the pile and soil conditions. We recommend an impact hammer similar to, or smaller than, an APE D 12-42. Larger impact hammers will likely be oversized for the timber piles, and use of larger impact hammers could lead to pile damage.
- The piles should have steel tips/points for protection during driving. The purpose of the steel tips is to protect the piles from damage should cobbles or other obstructions migrate back into the driving alignment between the pre-drilling and pile driving process.
- If the piles cannot be driven to planned embedment depth following this process, pre-drilling with temporary steel casing may be necessary. The piles could be placed within the temporary steel casing, then the casing could be backfilled with drill spoils. The temporary casing would then be extracted and reused for

subsequent pile installations. We recommend the construction budget for pile installation should have a contingency to account for the potential need for drill casing. The contract plans and/or specifications should also include language stating that piles shall be installed via drill casing if the intended embedment is not reached.

- Obstructions encountered during pile drilling/driving may cause some of the piles to be driven out-of-plumb, or to "drift" off of the design horizontal location. Also, if significant obstructions are encountered at certain locations, it may be necessary to adjust certain pile locations to avoid the obstructions. Because of this potential effect, some flexibility should be allowed in the design to enable adjustment of pile locations. Any such situations that arise during construction should be evaluated on a case-by-case by the Project Representative.

In general, pile driving construction, including load testing requirements, should follow the guidelines set forth in the Project specifications. The Project contract documents should require that the Contractor provide submittals detailing the selected piles along with information regarding the respective pile driving systems and equipment that will be used for installing the piles prior to the start of construction. The Project geotechnical engineer should be engaged to review and comment on the pile driving submittals.

I.7.2. Drivability Sensitivity Analysis

Drivability analyses produce estimates of driving stresses and installation times. The actual driving conditions will vary with Contractor means and methods, and with the use of installation measures such as pre-drilling or installing casing. Accordingly, we did not conduct sensitivity analyses for drivability.

APPENDIX I
TABLES

Table I.1. Engineered Log Jam and Biorevetment - Timber Pile Design Inputs and Results

Project No. 190175, Jan Road Neighborhood Improvements

Timber Pile Structure	Reference Subsurface Exploration	Planned Pile Diameter	Planned Pile Top Elevation (ft, NAVD88)	Predicted 100-Yr Flood Water Surface Elevation (ft, NAVD88)	Planned Ground Surface Elevation (ft, NAVD88)	Planned Pile Length, ft	Planned number of support piles per structure	Scenario	Predicted Scour Depth (ft below planned ground surface)	Predicted Scour Elevation (ft, NAVD88)	Predicted Factored Lateral Drag Force Applied Per ELJ (kip)(1,2)	Predicted Factored Lateral Drag Force Applied Per Pile (kip)(1)	Predicted Buoyant Force Applied per ELJ (kip)(3)	Predicted Buoyant Force Applied per Pile (kip)	Estimated Maximum Factored Lateral Drag Capacity Per Pile (kip)(1)	Estimated Ultimate Uplift Capacity Per Pile (kip)
ELJ Type 1	AB-03	16- to 12-inch (tapered top to bottom)	258.3	263.7	255.3	35	4 (4)	Design	5	250.3	10.6	5.3	29.2	7.3	**15.8**	**31.0**
								Wood Recruitment	4	251.3	21.1	10.6	38.8	9.7	**17.2**	**31.0**
ELJ Type 2	AB-03	18- to 14-inch (tapered top to bottom)	264	262.8	253	50	7	Design	15	238.0	58.7	8.4	98.1	14.0	**9.6**	**47.8**
								Wood Recruitment	12.3	240.7	117.4	16.8	108.2	15.5	**10.7**	**50.8**
ELJ Type 3	AB-01	16- to 12-inch (tapered top to bottom)	258.6	263	254.6	35	2 (4)	Design	5	249.6	7.0	3.5	17.9	9.0	**14.6**	**29.4**
								Wood Recruitment	4	250.6	14.1	7.1	23.8	11.9	**15.8**	**29.4**
ELJ Type 4	AB-03	16- to 12-inch (tapered top to bottom)	258.3	263.7	255.3	35	4 (4)	Design	5	250.3	10.6	5.3	18.9	4.7	**15.8**	**31.0**
								Wood Recruitment	4	251.3	21.1	10.6	25.2	6.3	**17.2**	**31**
Biorevetment	AMW-01	18- to 14-inch (tapered top to bottom)	262	267.6	265.0	38	2	Design	19.1	245.9	28.6	14.3	20.2	10.1	**14.3**	**48.3**

Notes:

1. Factored lateral drag force applied per ELJ includes a load factor of 1.0, per AASHTO LRFD BDS, 2020 design guidelines for water and stream pressures.
2. Predicted factored lateral drag force was provided by King County for ELJ Types 1, 2, 3, and 4 and by Tetra Tech for the biorevetment structure. Lateral drag forces are assumed to act equally on each pile in the structure with uniform distribution along the exposed height of the pile that is subjected to 100-year flood loads (above scour elevation).
3. Predicted buoyant force was provided by King County for ELJ Types 1, 2, 3, and 4 and by Tetra Tech for the biorevetment structure. Buoyant forces are assumed to act equally on each pile in the structure.
4. The number of piles in Type 1, 4, and 5 ELJs were simplified to 2 for lateral pile capacity analysis.

APPENDIX I
FIGURES

Aspect Consulting | Jun 10 2021
Right Bank, | GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 1.000 / 0.700

Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
6.0	44.7	4.2	40.5	5.9	1.055	0.000	5.86	15.0
12.0	56.6	16.1	40.5	7.4	1.118	0.000	6.07	14.6
18.0	91.4	27.9	63.5	12.9	1.240	-0.105	6.61	14.0
24.0	104.7	41.2	63.5	14.9	1.240	-0.133	6.78	13.9
30.0	121.8	58.3	63.5	17.7	1.231	-0.126	6.98	13.8
36.0	191.1	74.3	116.8	31.3	1.448	-0.179	7.80	14.4
42.0	227.8	92.6	135.2	44.6	1.474	-0.164	8.15	14.6
48.0	248.8	113.6	135.2	57.7	1.405	-0.118	8.31	14.5
52.0	281.0	129.1	151.9	92.2	1.445	-0.087	8.54	14.7
55.0	293.4	141.5	151.9	119.7	1.413	-0.049	8.58	14.5

Total Continuous Driving Time 38.00 minutes; Total Number of Blows 1582 (starting at penetration 6.0 ft)

Depth ft	Unit Shaft Resist ksf	Unit Toe Resist ksf	Skin Quake in	Toe Quake in	Skin Damping s/ft	Toe Damping s/ft	Setup Factor	Limit Distance ft	Setup Time hours	Toe Area in^2
0.000	0.000	15.196	0.100	0.117	0.100	0.150	1.000	0.000	0.0	154.00
2.500	0.121	15.196	0.100	0.117	0.100	0.150	1.000	0.000	0.0	154.00
2.500	0.175	18.016	0.100	0.117	0.100	0.150	1.000	0.000	0.0	154.00
5.000	0.312	18.016	0.100	0.117	0.100	0.150	1.000	0.000	0.0	154.00
5.000	0.337	54.111	0.100	0.117	0.100	0.150	1.000	0.000	0.0	154.00
12.500	0.695	54.111	0.100	0.117	0.100	0.150	1.000	0.000	0.0	154.00
12.500	0.497	64.497	0.100	0.117	0.100	0.150	1.000	0.000	0.0	154.00
15.000	0.582	64.497	0.100	0.117	0.100	0.150	1.000	0.000	0.0	154.00
15.000	0.399	84.817	0.100	0.117	0.100	0.150	1.000	0.000	0.0	154.00
30.000	0.798	84.817	0.100	0.117	0.100	0.150	1.000	0.000	0.0	154.00
30.000	0.582	139.669	0.100	0.117	0.100	0.150	1.000	0.000	0.0	154.00
35.000	0.686	139.669	0.100	0.117	0.100	0.150	1.000	0.000	0.0	154.00
35.000	0.653	156.040	0.100	0.117	0.100	0.150	1.000	0.000	0.0	154.00
40.000	0.752	156.040	0.100	0.117	0.100	0.150	1.000	0.000	0.0	154.00
40.000	0.725	180.596	0.100	0.117	0.100	0.150	1.000	0.000	0.0	154.00
50.000	0.916	180.596	0.100	0.117	0.100	0.150	1.000	0.000	0.0	154.00
50.000	0.889	202.911	0.100	0.117	0.100	0.150	1.000	0.000	0.0	154.00
60.000	1.075	202.911	0.100	0.117	0.100	0.150	1.000	0.000	0.0	154.00

Notes
1. See text for analysis assumptions and other inputs.

Hammer Type	**APE D12-42**
Hammer Efficiency	**0.84**
Helmet Weight (kips)	**3**
Cushion Thickness (in)	**2**
Pile Type	**Timber**
Pile Penetration (ft)	**55**

Driveability Analysis Results, AB-03
Jan Road Neighborhood Improvements
King County, Washington

Project No.:	**Analysis By / Reviewed By:**	**Updated:**
190175	MO / AJH	6/10/2021

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Figure
I-1

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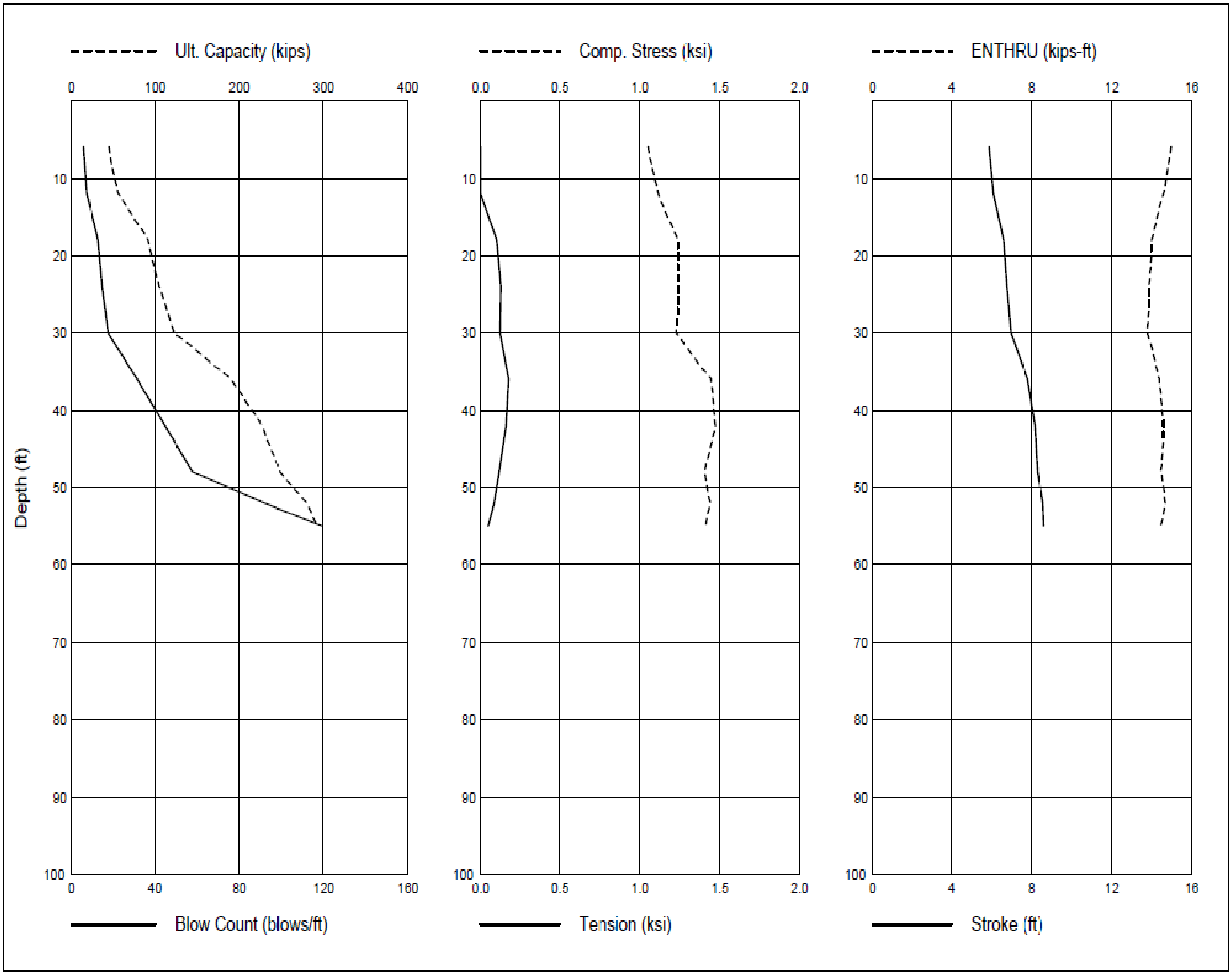
Right Bank,

Jun 10 2021
GRLWEAP Version 2010

Gain/Loss 1 at Shaft and Toe 1.000 / 0.700



Notes

1. See text for analysis assumptions and other inputs.

Hammer Type	**APE D12-42**
Hammer Efficiency	**0.84**
Helmet Weight (kips)	**3**
Cushion Thickness (in)	**2**
Pile Type	**Timber**
Pile Penetration (ft)	**55**

Driveability Analysis Results, AB-03
Jan Road Neighborhood Improvements
King County, Washington

Project No.:	**Analysis By / Reviewed By:**	**Updated:**
190175	MO / AJH	6/10/2021

Aspect CONSULTING

Figure

I-2

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Jan Road

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Gain/Loss 1 at Shaft and Toe 1.000 / 0.700

Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Stroke ft	ENTHRU kips-ft
6.0	51.2	4.0	47.2	6.8	1.090	0.000	5.98	14.7
12.0	49.5	14.9	34.5	6.0	1.074	0.000	5.88	14.9
18.0	82.6	28.1	54.6	10.8	1.194	-0.092	6.42	14.2
24.0	128.4	41.2	87.2	17.8	1.365	-0.154	6.98	13.9
30.0	143.4	56.2	87.2	19.9	1.339	-0.140	7.12	13.9
36.0	184.3	72.3	111.9	26.9	1.518	-0.160	7.65	14.3
42.0	228.1	91.3	136.9	38.6	1.632	-0.166	8.08	14.7
48.0	249.3	112.4	136.9	47.6	1.562	-0.124	8.23	14.6
52.0	280.9	128.1	152.9	68.3	1.597	-0.101	8.50	14.8
55.0	293.5	140.6	152.9	82.3	1.556	-0.073	8.56	14.7

Total Continuous Driving Time 32.00 minutes; Total Number of Blows 1340 (starting at penetration 6.0 ft)

Depth ft	Unit Shaft Resist ksf	Unit Toe Resist ksf	Skin Quake in	Toe Quake in	Skin Damping s/ft	Toe Damping s/ft	Setup Factor	Limit Distance ft	Setup Time hours	Toe Area in^2
0.000	0.000	0.000	0.100	0.117	0.100	0.150	1.500	0.000	0.0	154.00
2.000	0.134	10.712	0.100	0.117	0.100	0.150	1.500	0.000	0.0	154.00
2.000	0.161	14.582	0.100	0.111	0.050	0.150	1.000	0.000	0.0	154.00
5.000	0.241	14.582	0.100	0.111	0.050	0.150	1.000	0.000	0.0	154.00
5.000	0.297	63.084	0.100	0.111	0.050	0.150	1.000	0.000	0.0	154.00
10.000	0.627	63.084	0.100	0.111	0.050	0.150	1.000	0.000	0.0	154.00
10.000	0.420	46.143	0.100	0.111	0.050	0.150	1.000	0.000	0.0	154.00
15.500	0.625	46.143	0.100	0.111	0.050	0.150	1.000	0.000	0.0	154.00
15.500	0.527	72.874	0.100	0.111	0.050	0.150	1.000	0.000	0.0	154.00
20.000	0.682	72.874	0.100	0.111	0.050	0.150	1.000	0.000	0.0	154.00
20.000	0.447	116.471	0.100	0.111	0.050	0.150	1.000	0.000	0.0	154.00
30.000	0.691	116.471	0.100	0.111	0.050	0.150	1.000	0.000	0.0	154.00
30.000	0.586	149.506	0.100	0.100	0.050	0.150	1.000	0.000	0.0	154.00
40.000	0.797	149.506	0.100	0.100	0.050	0.150	1.000	0.000	0.0	154.00
40.000	0.734	182.835	0.100	0.100	0.050	0.150	1.000	0.000	0.0	154.00
50.000	0.928	182.835	0.100	0.100	0.050	0.150	1.000	0.000	0.0	154.00
50.000	0.903	204.177	0.100	0.100	0.050	0.150	1.000	0.000	0.0	154.00
60.000	1.089	204.177	0.100	0.100	0.050	0.150	1.000	0.000	0.0	154.00
60.000	1.129	208.854	0.100	0.100	0.050	0.150	1.000	0.000	0.0	154.00
70.000	1.321	208.854	0.100	0.100	0.050	0.150	1.000	0.000	0.0	154.00

Notes

1. See text for analysis assumptions and other inputs.

Hammer Type	**APE D12-42**
Hammer Efficiency	**0.84**
Helmet Weight (kips)	**3**
Cushion Thickness (in)	**2**
Pile Type	**Timber**
Pile Penetration (ft)	**55**

Driveability Analysis Results, AB-05
Jan Road Neighborhood Improvements
King County, Washington

Project No.:	**Analysis By / Reviewed By:**	**Updated:**
190175	MO / AJH	6/10/2021

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Figure
I-3

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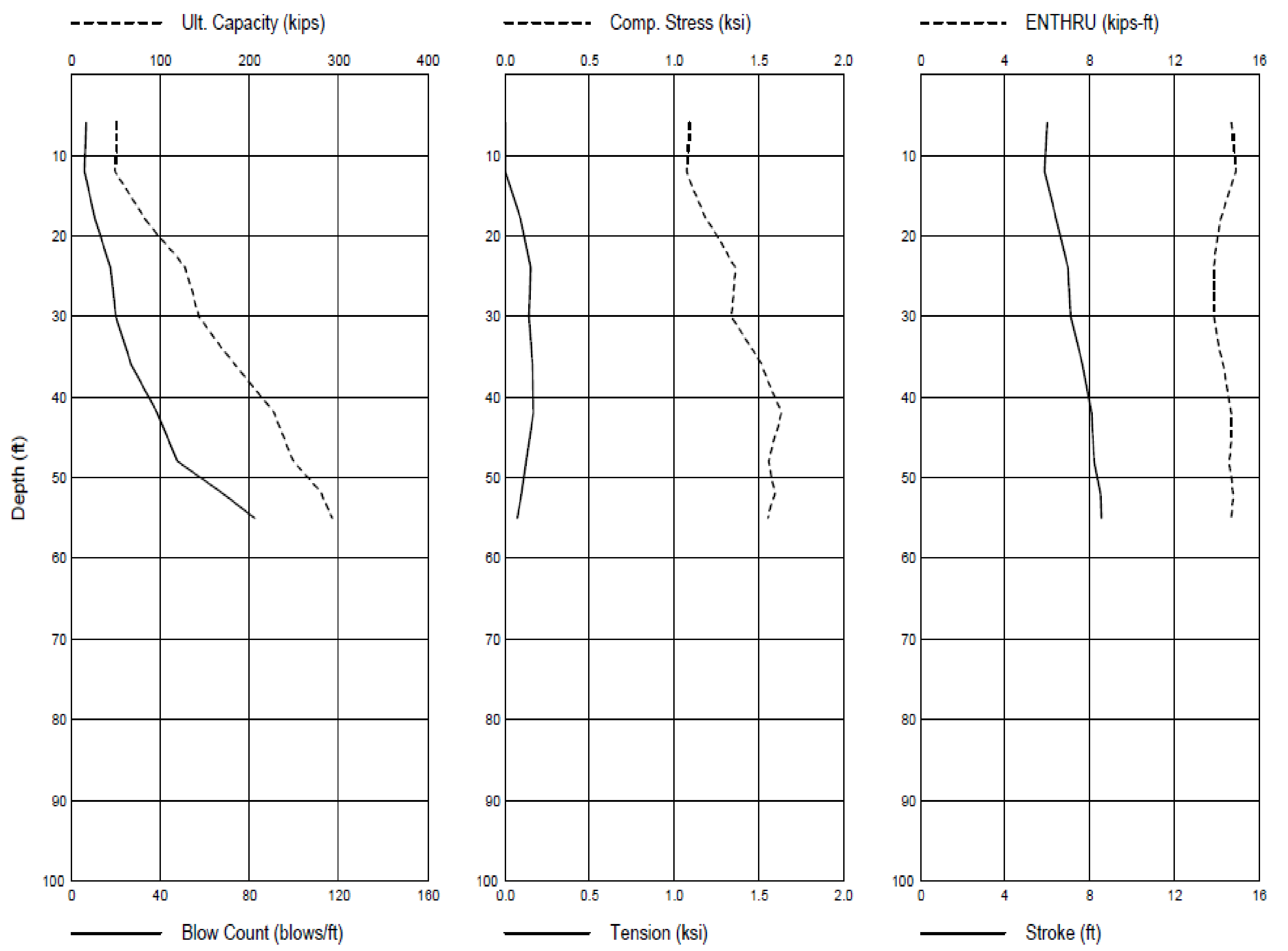
Gain/Loss 1 at Shaft and Toe 1.000 / 0.700



Notes	**Hammer Type**	**APE D12-42**	**Driveability Analysis Results, AB-05** Jan Road Neighborhood Improvements King County, Washington		
1. See text for analysis assumptions and other inputs.	**Hammer Efficiency**	**0.84**			
	Helmet Weight (kips)	**3**	**Project No.:**	**Analysis By / Reviewed By:**	**Updated:**
	Cushion Thickness (in)	**2**	190175	MO / AJH	6/10/2021
	Pile Type	**Timber**	Aspect CONSULTING		**Figure**
	Pile Penetration (ft)	**55**			I-4

APPENDIX J

Geotechnical Analyses for Culvert Design

J. Geotechnical Analyses for Culvert Design

J.1. Methodology

The existing culvert under SE 197th Place will be replaced with a larger, four-sided concrete box culvert structure located just north of the upstream end of the new setback levee. The box culvert will be approximately 16 feet wide and 30 feet long with angled wingwalls to achieve grade control around the culvert. The culvert will aid with conveying overbank flows from Taylor Creek. To inform the design of the culvert, we conducted geotechnical engineering analyses in accordance with the current AASHTO LRFD Bridge Design Specifications (BDS) (AASHTO, 2020) and WSDOT Bridge Design Manual (BDM) (WSDOT, 2020).

J.2. Foundations

We recommend constructing the culvert and wingwall foundations atop a 12-inch-thick rock fill bearing pad (bearing pad) comprised of Permeable Ballast per WSDOT Standard Specification 9-03.9(2) (WSDOT, 2021). The Permeable Ballast should be wrapped in a woven geotextile for soil stabilization meeting the requirements of WSDOT Standard Specification 9-33.2(1) – Table 3 placed over undisturbed and dense alluvial deposits.

The bearing pad will provide relatively uniform subgrade conditions directly beneath the concrete footings and "bridge" potential hard spots below foundations caused by the edges/points of larger cobbles and boulders in the alluvial deposits. The 12-inch-thick bearing pad should extend at least 2 feet in all directions (horizontally) beyond the edges of the culvert footings and at least 1 foot in all directions (horizontally) beyond the edges of the wingwall footings.

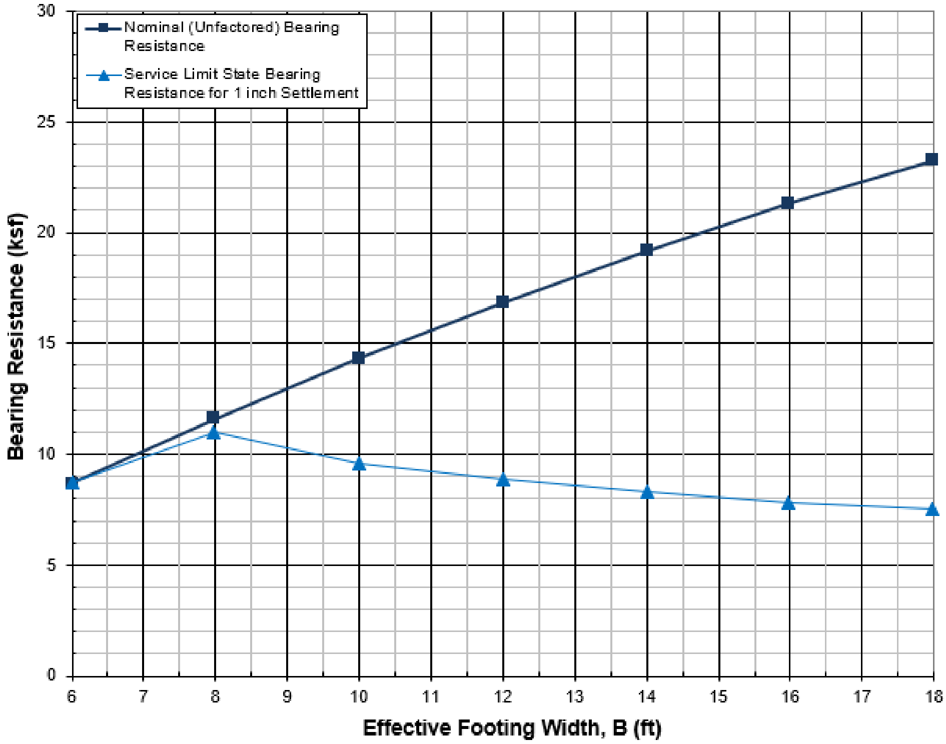
Prior to placing the bearing pad, the subgrade should be prepared to a relatively firm and level condition that is generally free of protruding cobbles and boulders edges/points, which might require some targeted removal of cobbles/boulders. Voids created from cobble/boulder removal should be backfilled with Permeable Ballast. The Project Representative should evaluate the foundation subgrade prior to placement of the fill pad and the foundations to verify conditions.

J.2.1.1. Foundation Bearing Resistance

The bearing resistance values presented below assume the culvert and wingwall footings are placed built atop subgrade materials prepared as described above.

Recommended Nominal (unfactored) and Service Limit State bearing resistances are presented as a function of footing width in Figure J-1 below. The presented Service Limit State bearing resistances are for a total settlement estimate of 1 inch and are controlled by the Nominal Bearing Resistances for footings less than 8 feet wide. The bearing

resistances do not account for any scour and assume the foundation subgrade soils are periodically submerged by high water levels.



Notes:
1. Assumes foundation embedment depth of at least 1.5 feet below streambed.
2. Doesn't account for any scour.
3. Assumes groundwater conditions 2 foot above the streambed.
4. For footing widths less than 8 feet, the service limit state bearing resistance is limited to less than or equal to the nominal bearing resistance.
5. Apply Resistance Factors to Nominal (Unfactored) Bearing Resistance to calculate Strength and Extreme Limit State Bearing Resistances.
   Bearing Resistance Factors: Strength L.S. (AASHTO LRFD BDS Table 10.5.5.2.1-1) = 0.45;

Figure J-1: Culvert and Wingwall Foundation Bearing Resistance

The recommended Load and Resistance Factor Design (LRFD) resistance factors required to calculate Strength and Extreme Limit State Bearing Resistances from the recommended Nominal Bearing Resistance are provided in Table J-1 below.

Differential settlement over the foundation lengths are estimated to be about half the total settlement estimates stated above. Settlement is anticipated to occur as loads are applied during construction.

Table J-1. LRFD Resistance Factors for Shallow Foundations

Limit State	Bearing Resistance, ϕ_b	Shear Resistance to Sliding, ϕ_τ	Passive Pressure Resistance to Sliding, ϕ_{ep}
Service	1.0	-	-
Strength	0.45	0.9$^{(1)}$	0.5
Extreme	0.9	0.9	0.9

Note: (1) for precast concrete

J.2.1.2. Sliding Resistance

Sliding resistance is developed from the friction occurring between the bottom of the concrete footings and the rock fill pad and the passive resistance developed from the soil around the foundation. The frictional and passive resistance values presented assume the precast concrete footings bear on the materials described above, the culvert is backfilled with on-Site material derived from the alluvial deposits or imported material meeting the minimum requirements for Gravel Borrow, WSDOT Standard Specification 9-03.14(1)(WSDOT, 2021) and compacted per our recommendations in Section 6.7 of the main text.

For passive resistance, we recommend the nominal passive values provided in Table J-1 be used for design. For frictional resistance along the bottoms of footings, we recommend an unfactored coefficient of 0.40 assuming precast concrete is used. LRFD Resistance Factors for determining limit state sliding and passive resistance are provided in Table J-1 above.

J.3. Culvert Wall and Wingwall Design

The culvert walls will be integral to the structure and supported by the culvert footings, while the culvert wingwalls will not be structurally connected to the culvert and will be supported on their own shallow spread footings.

The following sections present design recommendations for lateral earth pressures, surcharge considerations, and drainage requirements for the culvert walls and wingwalls.

J.3.1.1. Lateral Earth Pressures

The lateral earth pressures acting on the culvert and wingwalls for active and at-rest conditions are shown below in Table J-2 below. These values assume properly compacted structural fill (as described in Section 6.7) around the culvert and behind the wingwalls and level backslope and foreslope conditions.

To invoke active earth pressure conditions, a wall must be capable of yielding laterally at least 0.001 to 0.002H, where H is the exposed height of the wall; otherwise, at-rest conditions should be assumed.

Table J-2. Lateral Earth Pressure Parameters for Culvert and Wingwalls

Earth Pressure Condition	Earth Pressure Coefficient	Equivalent Fluid Density[3] (pcf)[1]	Uniform Lateral Surcharge Pressure[4] (psf)[1]
Active[2]	0.26	36	0.26*S
At-Rest	0.41	56	0.41*S
Passive[5]	3.82	260	-

Notes:

1. pcf = pounds per cubic foot; psf = pounds per square foot
2. Active earth pressures assume the wall is capable of yielding laterally at least 0.001*H to 0.002*H, where H is the exposed height of wall.
3. The equivalent fluid densities provided above are distributed triangularly along the exposed height of the wall. Equivalent fluid densities assume drained conditions (i.e., no buildup of hydrostatic pressures will be allowed)
4. The uniform lateral surcharge pressures are distributed uniformly (rectangularly) along the exposed height of the wall.
5. Ultimate passive pressures are presented; LRFD Resistance Factors for determining limit state passive resistance are provided in Table J-1 above. Passive resistance within a depth of 2 feet of the ground surface in front of the walls should be ignored.

J.4. Drainage

Surface runoff from the paved roadway surface should be diverted or sloped away from culverts and wingwalls to the extent possible. We recommend culvert backfill and backfill behind the wingwalls consist of Select Fill Type 2, as described in Section 6.5.4, which is expected to be granular and relatively free-draining and will prevent the buildup of unbalanced hydrostatic pressures behind the walls.

APPENDIX K

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.