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November 3, 2017 W1212-T01.1 GEOTECHNICAL RPT

DRAFT

Clark County Public Works Department – Engineering and Construction
PO Box 9810
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Attention: Scot J. Brantley

SUBJECT: Geotechnical Report
Manley Road & Culvert Replacement Project
Clark County, WA

As requested, GRI completed a geotechnical investigation for proposed Manley Road improvements and culvert replacement in Clark County, Washington. The general location of the site is shown on the Vicinity Map, Figure 1. The evaluation included subsurface explorations, field infiltration testing, laboratory testing, engineering analyses, and preparation of this report. The report summarizes our findings and presents our preliminary conclusions and recommendations related to the planned improvements.

Unless otherwise noted, all elevations in this report refer to the National Geodetic Datum of 1929 (NGVD 29).

PROJECT DESCRIPTION

It is our understanding that Clark County Public Works (County) plans to make improvements to NE Manley Road from NE 82nd Avenue to about 800 ft south of NE 249th Street, as shown on the Site Plans, Figures 2 through 6. The project includes new culverts and retaining walls, which will be designed in accordance with the American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design Bridge Design Specifications (LRFD BDS) and applicable Washington State Department of Transportation (WSDOT) guidelines.

A section of NE Manley Road about 1,200 ft north of NE 249th Street is adjacent to Manley Creek (also referred to as Daybreak Creek) and is located on an embankment fill. The creek is eroding the embankment, resulting in overly steep slopes. As a result, the County is considering shifting this portion of the alignment to the west onto a new embankment fill that will be up to 14 ft thick. In order to accommodate the realignment, a 7- to 10-ft cut retaining wall (referred to as Wall 1) will be required with a nearly 1H:1V (Horizontal to Vertical) backslope between about Station 41+00 and 42+00. Alternatively, some consideration is being given to shifting the alignment of Manley Creek to the east away from the embankment and reconfiguring the embankment slopes at 2H:1V.

The project includes the replacement of three existing culverts along Manley Creek with new culverts, designated as the north, center, and south culverts. Each of the culverts will be about 20 ft wide. Several culvert types are being considered for each crossing. For the southern and center culverts, potential culvert types being considered include a precast concrete structure, single-radius steel arches, and multiple-radius steel arches. At the northern culvert, potential culvert replacement types considered include a precast concrete structure or slab bridge. The top of the southern and center culverts will be embedded about 10 ft below roadway grades. The embankment above the culvert headwalls will be inclined at 2H:1V. The top of the northern culvert will be immediately below the pavement section.

We understand the roadway will be realigned at three locations. In order to accommodate the potential realignment between Stations 34+00 and 37+00, two new 7- to 10-ft tall cut walls (referred to as Walls 2 and 3) may be needed. We understand potential wall types being considered for these walls include cast-in-place (CIP) concrete, mechanically stabilized earth (MSE), modular block, and soldier pile walls (with or without tiebacks).

Two new stormwater facilities and new underground stormwater pipelines are also planned as part of the project. One of the ponds is located on the Richards property just northwest of the northern culvert. The second pond is planned for the Lindberg property, which is located on the west side of NE Manley Road between Station 63+00 and Station 70+50. New asphalt concrete (AC) pavement will be constructed in areas where the roadway is realigned.

SITE DESCRIPTION

Topography and Surface Conditions

In the planned project area, NE Manley Road runs in a general northwest to southeast direction between NE 82nd Avenue and NE 244th Street. Northeast Manley Road is paved with asphalt and generally about 22 to 25 ft wide with a gravel-shoulder width varying from 0 to 2 ft. Manley Creek flows from south to north and generally parallels the alignment and crosses under NE Manley Road through three existing corrugated metal pipe (CMP) culverts at about stations 47+00 (north culvert), 41+00 (center culvert), and 24+50 (south culvert).

The project alignment undergoes an elevation change of approximately 150 ft. The road surface slopes gently upwards to the southeast from about elevation 90 ft at the intersection of NE Manley Road and NE 82nd Avenue to elevation 100 ft at Station 47+00. The existing road surface along this section of NE Manley Road is located within several feet of the adjacent ground surface, except between Station 68+00 and 50+00 and on the northeast side of NE Manley Road, where the roadway is elevated 5 to 10 ft above adjacent site grades.

Between Station 47+00 and Station 37+00, NE Manley Road is constructed on a fill embankment; the roadway grades increase to about elevation 180 ft within this stretch of the alignment and Manley Creek generally parallels the east side of the embankment from the center culvert to approximately 400 ft north of the center culvert. The embankment sideslopes in this area range from about 1¼H:1V to 1¾H:1V. Manley Creek is actively eroding the toe of the embankment, resulting in localized areas of embankment slopes inclined at ½H:1V to ¾H:1V. Sloughing embankment side slopes are also present on the west side of the embankment just south of the center culvert, resulting in embankment slopes inclined at about 1¼H:1V in this area.

At the location of the proposed embankment reconfiguration (i.e, on the west side of NE Manley Road between Stations 47+00 and 43+00), the sideslopes of the embankment are inclined at 2H:1V and the ground surface at the base of the embankment ranges from about elevation 105 and 120 ft. Between Stations 43+00 and 40+50, a slope inclined at about 1.2H:1V is located west of NE Manley Road. The top of the slope is at about elevation 215 ft. The slope is covered with large evergreen trees with no obvious indications of global instability such as bowed trees or hummocky or irregular terrain. Wall 1 is planned at this location between about Stations 41+00 and 42+00.

Between Stations 37+00 and Station 33+00, NE Manley Road is situated within an existing cut. Walls 2 and 3 are planned along this stretch of the alignment. At the location of Wall 2, the ground surface behind the proposed wall is inclined at about 1.5H:1V. At the location of Wall 3, the ground surface behind the proposed wall is inclined at about 2H:1V. South of Station 33+00, the grades along NE Manley Road generally increase to about elevation 225 ft at station 24+00.

The north culvert passes under an embankment fill at about station 47+00. The roadway elevation at this location is at about 100 ft and the fill thickness over the culvert varies from about 2 to 6 ft. The embankment at this location is approximately 5 to 7 ft tall. There is a near-vertical rockery wall at the upstream face of the culvert and a 2H:1V embankment slope at the downstream end.

The center culvert is located at about station 41+00. The road surface at the center culvert is at about elevation 143 ft and the culvert is covered by 12 to 15 ft of fill. The streambed on the upstream side of the culvert is at about elevation 130 ft while the streambed on the downstream side of the culvert is at about elevation 123 ft. The embankment fill at this location is approximately 20 ft tall and the side slopes of the embankment are inclined at about 1.5H:1V and 1H:1V on the upstream (south) and downstream (north) sides, respectively.

The south culvert passes under an approximate 15-ft tall embankment fill at about Station 24+50. Northeast Manley Road is at about elevation 225 ft. The downstream (west) side of the embankment slopes to the down to Manley Creek at 1.5H:1V. The upstream (east) side of the embankment is declined at 1H:1V. The ground surface on the upstream side of the embankment is overly steep and has been repaired in the past with a quarry spall infill.

Geology

Geologic information for the project indicates the portion of NE Manley Road between NE 82nd Avenue and the north culvert is mantled with recent alluvium consisting of gravels, cobbles, and boulders (Howard, 2002). South of the north culvert to just north of NE 249th Street, the near-surface geology is mapped as late Pliocene- and Pleistocene-age (about 3.5 to 1 million years old) sedimentary rocks consisting of conglomerate and siltstone referred to as the Troutdale formation. South of NE 249th Street, the near-surface geology is mapped as Pleistocene-age Missoula flood deposits consisting of silt and sand.

The mapped geology is generally consistent with the conditions observed in the borings and during our site reconnaissance; however, the gravel, cobble, and boulder alluvium identified in the northern portion of the site is generally mantled with sand and silt alluvium, and much of the middle portion of the alignment is mantled with embankment fill.

SUBSURFACE CONDITIONS

General

Subsurface materials and conditions along the proposed alignment were evaluated with 13 borings, designated B-1 through B-13, advanced between August 21 and 23, 2017, and four small-scale Pilot

Infiltration Tests (PIT), designated TP-1 through TP-4, completed between October 9 and 11, 2017. The explorations were advanced at the approximate locations shown on the Site Plan, Figures 2 through 6. The borings were advanced to depths of 11.5 to 41.5 ft below the existing ground surface using hollow-stem auger drilling techniques. The test pits were advanced to depths of 3 to 4 ft and the infiltration tests were completed at the base of each test pit. A detailed discussion of the field exploration and laboratory testing program completed for this investigation is provided in Appendix A. Logs of the borings and test pits are provided on Figures 1A through 15A. The terms and symbols used to describe the soils encountered in the borings and test pits are defined in Table 3A and on the attached legend. The results of laboratory testing are summarized in Table 4A and on the boring logs.

For discussion, we have grouped the subsurface conditions along the planned project alignment for six general areas, as shown in Table 1 below.

Table 1: PROJECT AREAS

Location	Explorations
NE 82nd Avenue to North Culvert (Sta. 47+00) (approx.)	B-1, B-2, B-3, B-4, TP-1, TP-2, TP-3, and TP-4
North Culvert (Sta. 47+00)	B-5
Embankment Realignment (Sta. 47+00 to 43+00)	B-6
Wall 1 (Sta 42+00 to 41+00)	B-7
Center Culvert (Sta. 41+00)	B-8 and B-9
Walls 2 and 3 (Sta. 37+00 and 33+00)	B-10 and B-11
South Culvert (Sta. 24+50)	B-12 and B-13

Soil

For the purpose of discussion, the soil conditions in each of the six project areas have been grouped into categories based on their physical characteristics and engineering properties.

NE 82nd Avenue to North Culvert

Borings B-1 through B-4 and TP1 through TP-4 were advanced in the northern portion of the alignment located between NE 82nd Avenue and the north culvert. The following units were encountered in this portion of the alignment:

1. **PAVEMENT**
2. **SILT and SAND (Alluvium)**
3. **Sandy GRAVEL (Alluvium)**
4. **Silty SAND (Alluvium)**

The following paragraphs provide a detailed description of these units.

1. PAVEMENT. In boring B-2, 10 in. of AC pavement was encountered at the ground surface.

2. SILT and SAND (Alluvium). Alluvial silt and sand was encountered at the ground surface in explorations B-1, B-3, B-4, and TP-1 through TP-4 and below the pavement section in boring B-2. The upper 6 to 9 in. of silt and sand encountered in boring B-3 and test pits TP-1 through TP-4 is heavily rooted. These deposits were encountered to depths of 5, 7, 6.8, 2, 3, 2, 2, and 2.5 ft in explorations B-1, B-2, B-3, B-4, TP-1, TP-2, TP-3, and TP-4, respectively.

Silt was encountered to depths of 5, 2.5, 5, 3, and 2 ft in explorations B-1, B-2, B-3, TP-1, and TP-2, respectively. The silt is brown and contains variable fine-grained sand contents, ranging from trace to sandy. Based on SPT N-values of 4 to 10 blows/ft and observations made during digging of the test pits, the relative consistency of the silt is soft to stiff. The silt has a natural moisture content of about 19 to 31%.

Sand was encountered below the silt in borings B-2 and B-3 and at the ground surface in explorations B-4, TP-3, and TP-4. The sand is typically brown, silty, and fine grained. The sand in boring B-4 is fine to medium grained to a depth of 2 ft and grades to gravelly sand containing a trace to some silt, which extends to 5 ft of depth. Based on SPT N-values of 2 to 6 blows/ft, the relative density of the silty sand is very loose to loose. Based on an SPT N-value of 36 blows/ft, the relative density of the gravelly sand in boring B-4 is dense. The silty sand has a natural moisture content of about 16 to 23% and the gravelly sand has a natural moisture content of about 5%.

3. Sandy GRAVEL (Alluvium). Sandy gravel was encountered in explorations B-1 through B-4 and TP-1 through TP-4 below the silt and sand. The sandy gravel extends to a depth of 15 ft in borings B-1, B-3, and B-4. Boring B-2 and test pits TP-1, TP-2, TP-3, and TP-4 were terminated in the sandy gravel at depths of 11.5, 4, 4, 3, and 4, respectively. The sandy gravel typically contains trace to some silt. Cobbles were encountered in the test pits in this layer, and based on drilling action, cobbles and boulders are likely present in the sandy gravel at the boring locations. Based on SPT N-values ranging from 11 blows/ft to refusal conditions, defined as more than 50 blows for 6 in. of sampler penetration, and refusal CMS N*-values, the relative density of the sandy gravel is medium dense to very dense.

4. Silty SAND. Silty sand was encountered from 15 to 16.5 ft (maximum depth explored) in borings B-1, B-3, and B-4. The sand is brown mottled rust to red-brown and typically fine to coarse grained. The silty sand in boring B-1 is fine to medium grained, and the sand in boring B-4 contains a trace of gravel. Based on SPT N-values of 13 to 22 blows/ft and a CMS N*-value of 61 blows/ft, the silty sand is medium dense. The natural moisture content of the silty sand is about 25 to 30%.

North Culvert

Borings B-5 was located in the southbound lane of NE Manley Road about 5 ft north of the north culvert. The following units were encountered at this location:

1. **PAVEMENT**
2. **FILL**
3. **Silty SAND (Alluvium)**
4. **GRAVEL (Alluvium)**
5. **Sandy SILT (Alluvium)**

The following paragraphs provide a detailed description of these units.

1. PAVEMENT. In boring B-5, 10 in. of AC pavement was encountered at the ground surface.

2. FILL. Fill was encountered below the pavement in boring B-5 to a depth of 5 ft. The fill is brown and consists of gravelly sand to sandy gravel. The sand is fine to coarse grained. Based on SPT N-values of 35 and 41 blows/ft, the relative density of the fill is dense. The fill has a natural moisture content of about 8%.

3. Silty SAND (Alluvium). Silty sand was encountered below the fill in boring B-5 to a depth of 10 ft below the ground surface. The sand is brown, is fine to coarse grained, and contains a trace to some gravel. Based on SPT N-values of 5 to 8 blows/ft, the relative density of the silty sand is loose. The silty sand has a natural moisture content of about 23 to 24%.

4. GRAVEL (Alluvium). Gravel was encountered in boring B-5 from 10 to 19 ft below the ground surface. The gravel has a fine- to coarse-grained sand content ranging from some sand to sandy and a silt content varying from some silt to silty, though it is typically sandy and contains a trace to some silt. The gravel is subrounded to subangular and contains cobbles and possible boulders. Based on SPT N-values of 29 to 66 blows/ft, the relative density of the gravel is medium dense to very dense and typically dense to very dense.

4. Sandy SILT (Alluvium). Sandy silt was encountered in boring B-5 between 19 and 21.5 ft below the ground surface, the maximum depth explored. The sandy silt is brown and contains fine- to coarse-grained sand and a trace to some gravel. Based on an SPT N-value of 7 blows/ft, the relative consistency of the sandy silt is medium stiff. The sandy silt has a natural moisture content of about 37%.

Embankment Realignment

Boring B-6 was advanced in the southbound lane of NE Manley Road near the location of the planned embankment realignment. The following units were encountered:

1. **PAVEMENT**
2. **FILL**
3. **Sandy GRAVEL (Alluvium)**
4. **Sandy GRAVEL (Troutdale Formation)**

The following paragraphs provide a detailed description of these units.

1. PAVEMENT. In boring B-6, 10 in. of AC pavement was encountered at the ground surface.

2. FILL. Sandy gravel fill was encountered below the pavement to a depth of 12.5 ft. The sandy gravel contains trace silt, and based on drill action, it includes cobbles and boulders. Based on SPT N-values of 9 to 25 blows/ft, the relative density of the sandy gravel is medium dense.

3. Sandy GRAVEL (Alluvium). Sandy gravel alluvium was encountered below the fill to a depth of 30 ft. The sandy gravel typically contains trace silt, and based on drill action, it includes cobbles and boulders. Below 25 ft, the alluvial gravels contain variable sand and silt content ranging from some sand to sandy and some silt to silty. Based on SPT N-values of 7 blows/ft to refusal blow counts and CMS N*-value refusal blow counts, the relative density of the sandy gravel alluvium is loose to very dense and is typically medium dense to very dense.

4. Sandy GRAVEL (Troutdale Formation). Sandy gravel of the Troutdale formation was encountered below the alluvial sandy gravel to the maximum depth explored, about 31.5 ft. The sandy gravel typically contains trace silt, and based on drill action, it includes cobbles and boulders. Based on an SPT N-value of 53 blows/ft, the relative density of the Troutdale-formation sandy gravel is very dense. The sandy gravel encountered in the borings exhibited minimal to very low cementation.

Wall 1

Boring B-7 was advanced near the location of the Wall 1. The following units were encountered:

1. **PAVEMENT**
2. **FILL**
3. **Sandy GRAVEL (Troutdale Formation)**

The following paragraphs provide a detailed description of these units.

1. PAVEMENT. In boring B-7, 10 in. of AC pavement was encountered at the ground surface.

2. FILL. Sandy gravel and gravelly sand fill was encountered below the pavement to a depth of 21.5 ft. The sandy gravel and gravelly sand fill typically contains some silt as well as cobbles and possible boulders based on observations made during drilling. Based on SPT N-values of 5 to 32 blows/ft, the relative density of the sandy gravel and gravelly sand is loose to dense and is typically loose to medium dense. The natural moisture content of a sample of the gravelly sand fill was 11%.

3. Sandy GRAVEL (Troutdale Formation). Sandy gravel of the Troutdale formation was encountered below the fill to the maximum depth explored, about 31.5 ft. The sandy gravel contains trace silt and based on drill action, includes cobbles and boulders. Based on SPT N-values of 48 to 52 blows/ft, the relative density of the Troutdale-formation sandy gravel is dense to very dense. The sandy gravel encountered in the borings exhibited minimal to very low cementation.

Center Culvert

Borings B-8 and B-9 were located in the northbound and southbound lanes of NE Manley Road, respectively. The following units were encountered in these explorations:

1. **PAVEMENT**
2. **FILL**
3. **Sandy GRAVEL (Alluvium)**
4. **SILT (Troutdale Formation)**

The following paragraphs provide a detailed description of these units.

1. PAVEMENT. In borings B-8 and B-9, 10 in. of AC pavement was encountered at the ground surface.

2. FILL. Sandy gravel and gravelly sand embankment fill was encountered below the pavement to depths of 18 and 20 ft in borings B-8 and B-9, respectively. The sandy gravel and gravelly sand typically contains trace to some silt as well as cobbles and possible boulders based on drill action. Based on SPT N-values of

8 to 66 blows/ft, the relative density of the sandy gravel and gravelly sand is loose to very dense and is typically medium dense. The natural moisture content of the sandy gravel and gravelly sand fill ranges from about 9 to 12%.

3. Sandy GRAVEL (Alluvium). Sandy gravel alluvium was encountered below the fill to depths of 21 and 25 ft in borings B-8 and B-9, respectively. The sandy gravel contains some silt as well as cobbles and possible boulders based on drill action. Based on an SPT N-value of 23 blows/ft, the relative density of the sandy gravel alluvium is medium dense.

4. SILT (Troutdale Formation). Silt of the Troutdale formation was encountered below the alluvial gravel in both boring B-8 and B-9 to the maximum depth explored, 41.5 ft. The silt contains some decomposed rock structure. The silt is gray contains fine-grained sand in varying amounts from a trace to sandy, and becomes clayey below 35 ft. Based on SPT N-values of 7 to 35 blows/ft, the relative consistency of the silt is medium stiff to hard and is typically very stiff. The natural moisture content of the silt ranges from about 29 to 36%.

Walls 2 and 3

Borings B-10 and B-11 were advanced in the northbound travel lane of NE Manley Road, near the proposed locations of Walls 2 and 3. The following units were encountered:

1. **PAVEMENT**
2. **Gravelly SAND to Sandy GRAVEL (Troutdale Formation)**

The following paragraphs provide a detailed description of these units.

1. PAVEMENT. In borings B-10 and B-11, 10 in. of AC pavement was encountered at the ground surface.

2. Gravelly SAND to Sandy GRAVEL (Troutdale Formation). Gravelly sand to sandy gravel of the Troutdale formation was encountered below the pavement section to the maximum depths explored of 15.5 and 16.5 ft in borings B-10 and B-11, respectively. The gravelly sand to sandy gravel typically contains trace to some silt. Based on drilling action, cobbles and boulders are likely present within the unit. Based on SPT N-values of 13 blows/ft to more than 50 blows for less than 6 in. of sampler penetration, the relative density of the gravelly sand to sandy gravel is medium dense to very dense. The natural moisture content of the gravelly sand to sandy gravel ranges from about 13 to 14%. The sandy gravel encountered in the borings exhibited minimal to very low cementation. An outcrop of Troutdale formation on the west side of NE Manley Road was very strongly cemented.

South Culvert

Borings B-12 and B-13 were located in the northbound and southbound lanes of NE Manley Road, respectively, at the location of the south culvert. The following units were encountered:

1. **PAVEMENT**
2. **FILL**
3. **GRAVEL (Troutdale Formation)**

The following paragraphs provide a detailed description of these units.

1. PAVEMENT. In borings B-12 and B-13, 9 in. of AC pavement was encountered at the ground surface.

2. FILL. Fill soil was encountered below the pavement section to a depth of about 17 ft in both boring B-12 and B-13. The fill consists of gravelly sand with trace silt to depths of 2.5 and 3.5 ft in borings B-12 and B-13, respectively. Based on SPT N-values of 16 and 22 blows/ft, the relative density of the gravelly sand is medium dense. In boring B-12, silty sand was encountered below the gravelly sand to a depth of 5 ft. The silty sand is brown and fine grained. Based on an SPT N-value of 4 blows/ft, the relative density of the silty sand is very loose to loose. The natural moisture content of the silty sand is 21%

The fill encountered in borings B-12 and B-13 below depths of 5 and 3.5 ft, respectively, consists of silt with a variable sand content, ranging from a trace of sand to sandy, and up to some clay. Trace gravel and wood debris were encountered in boring B-12 at 12.5 ft, and wood debris was encountered in boring B-13 at 10 ft. Based on SPT N-values of 2 to 4 blows/ft, the relative consistency of the silt is soft to medium stiff. The natural moisture content of the silt ranges from about 22 to 42%.

3. GRAVEL (Troutdale Formation). Gravel of the Troutdale formation was encountered below the fill to the maximum depths explored of 35.8 and 36.4 ft in borings B-12 and B-13, respectively. The sand content of the gravel varies from some to sandy and the silt content varies from a trace of silt to silty. Based on drilling action, the gravel contains cobbles and possible boulders. Based on SPT N-values of 11 blows/ft to refusal, the relative density of the gravel is medium dense to very dense, becoming very dense at a depth of about 25 ft. The sandy gravel encountered in the borings exhibited minimal to very low cementation.

Groundwater

All borings advanced for this investigation were advanced using hollow-stem auger drilling techniques, which allows for the measurement of groundwater in the boring at the time of drilling. In addition, standpipe piezometers were installed in borings B-1, B-3, and B-4 at a depth of 15 ft below ground surface in order to measure the seasonal variation of the groundwater table near the potential stormwater infiltration locations. The depth to groundwater in these piezometers was measured on August 23, 2017, and October 9, 2017. Table 2 summarizes the groundwater data collected for this project.

Table 2: SUMMARY OF GROUNDWATER DATA

Boring Designation	Date	Depth to Groundwater	Remarks
B-1	8/23/2017	12.1	Measured in Piezometer
B-1	10/9/2017	12.2	Measured in Piezometer
B-2	8/21/2017	---	Not Encountered
B-3	8/23/17	12.2	Measured in Piezometer
B-3	10/9/2017	12.3	Measured in Piezometer
B-4	8/23/2017	13.8	Measured in Piezometer
B-4	10/9/2017	14.1	Measured in Piezometer
B-5	8/21/2017	7.5	At Time of Drilling
B-6	8/22/2017	20.0	At Time of Drilling
B-7	8/22/2017	20.0	At Time of Drilling
B-8	8/21/2017	20.0	At Time of Drilling
B-9	8/22/2017	20.0	At Time of Drilling
B-10	8/22/2017	---	Not Encountered
B-11	8/22/2017	---	Not Encountered
B-12	8/23/2017	16	At Time of Drilling
B-13	8/23/2017	16	At Time of Drilling
TP-1	10/11/2017	---	Not Encountered
TP-2	10/10/2017	---	Not Encountered
TP-3	10/9/2017	---	Not Encountered
TP-4	10/9/2017	---	Not Encountered

We anticipate the groundwater level near the culverts will be within several feet of Manley Creek's water level and fluctuate in response to seasonal precipitation. We anticipate perched groundwater could approach the ground surface during periods of sustained wet weather, especially in the northern portion of the site.

Infiltration Testing

Four infiltration tests were conducted in general conformance with the small-scale pilot infiltration test procedure described in Appendix 1-C of the 2015 Clark County Stormwater Manual. Details of the infiltration testing method are provided in Appendix A. The test results are summarized in the Stormwater Infiltration section of this report.

CONCLUSIONS AND RECOMMENDATIONS

General

The investigation disclosed that the portion of NE Manley Road located between NE 82nd Avenue and the north culvert is mantled with alluvial deposits of sand and silt that are in turn underlain by alluvial gravels. Based on standpipe piezometer readings, the groundwater in this portion of the alignment is located 12 to 14 ft below the existing ground surface during the dry summer months.

Extending south from the north culvert to the center culvert, the existing roadway alignment is constructed on an embankment fill consisting of gravelly sand and sandy gravel. The embankment fill is underlain by alluvial gravels and silt and/or gravel of the Troutdale formation. The steep slope located west of NE Manley Road between Station 42+00 and Station 41+00 (boring B-7) consists of Troutdale-formation gravel. Based on the conditions observed in our borings and during our reconnaissance, the Troutdale-formation gravel exhibits minimal to very low cementation; however, we anticipate more-cemented Troutdale-formation gravel may be encountered during soldier pile and tieback installation. The observed groundwater level in this portion of the alignment generally corresponds to within several feet of the water level in Manley Creek.

Troutdale-formation gravel is present at the locations of Walls 2 and 3. Based on the conditions observed in our borings and during our reconnaissance, the Troutdale-formation gravel exhibits minimal to very low cementation; however, we anticipate strongly cemented Troutdale-formation gravel may be encountered during wall construction. Groundwater was not encountered in the borings in these locations. In the location of the south culvert, sand and silt embankment fill was encountered to a depth of about 17 ft. The embankment fill is underlain by Troutdale-formation gravel. The groundwater level at this location generally corresponds to within several feet of the water level in Manley Creek.

Based on the conditions observed in the borings and the results of our engineering analysis, it is our opinion each of the replacement culverts can be supported by shallow foundations. The excavations for the culvert foundations will extend below the groundwater table, requiring extensive dewatering in order to construct the foundations. In our opinion, feasible retaining wall types for Walls 2 and 3 include CIP concrete, MSE, modular block, and soldier pile walls. Because of the steep backslope at the location of Wall 1, we anticipate a cantilevered or anchored soldier pile wall will be required.

Our conclusions and recommendations for design and construction of the improvements are discussed below.

Earthwork

Site Preparation. Clearing and grubbing should be completed in accordance with the requirements of Section 2-01 of the WSDOT Standard Specifications. The ground surface within the limits of potential fills should be stripped of vegetation, surface organics, and loose surface soils. The lateral limits of stripping and grubbing should extend at least 10 ft beyond improvement areas. We anticipate a stripping depth of around 6 to 12 in. will be required to remove vegetation and loose soil present at the base of the existing embankments or in other undeveloped areas. Deeper stripping and grubbing should be anticipated to remove stumps and roots associated with the larger trees present on the site. Strippings will not be suitable for structural fill and should only be used in landscaped areas or removed from the site. Stripped areas to receive structural fill should be evaluated by a qualified member of GRI's geotechnical engineering staff. Stripping and excavation should be accomplished using equipment with smooth cutting surfaces. Soft areas, if encountered, should be overexcavated and backfilled with structural fill prior to placing new fill.

Subgrade Preparation and Grading. Following site preparation activities and any additional excavation needed to reach the planned subgrade, areas to receive fill or other improvements should be evaluated by experienced geotechnical engineering staff from our firm. Loose, soft, or disturbed areas should either be moisture-conditioned and re-compacted as structural fill (dry-weather conditions only) or removed and replaced with imported structural fill. Proof rolling with a loaded dump truck or other heavy, rubber-tired vehicle may be part of the evaluation. During wet weather or wet-ground conditions or due to accessibility constraints, the subgrade can be probed instead of proof rolled to minimize the potential for subgrade disturbance.

Silty sand, sandy silt, and silt were encountered at the ground surface to the north of about Station 47+00 and will likely be present at other locations along the alignment, including near the base of the existing fill embankments and south of Station 33+00. These silty soils are sensitive to moisture content. During wet conditions, silty soils are easily disturbed, rutted, and weakened by construction activities. For this reason, we recommend that all site preparation and earthwork be accomplished during the dry summer months, typically extending from mid-May to mid-October. If the subgrade is disturbed during construction, soft, disturbed soils should be overexcavated to firm soil and backfilled with properly compacted structural fill.

South of about Station 47+00, the near-surface soils consist predominately of sandy gravel or gravelly sand with up to some silt. These deposits can typically be worked throughout the year; however, the contractor should take care to protect sandy and gravelly subgrades during wet weather or wet-ground conditions.

During wet weather or wet-ground conditions, it should be anticipated that haul roads or granular work pads constructed from granular structural fill consisting of crushed rock will be necessary to provide access to the work site and protect silty subgrade from damage due to construction traffic. In our opinion, a 12-in.-thick granular work pad should be sufficient to prevent disturbance of the silt subgrade by lighter construction equipment and limited traffic by dump trucks. Haul roads and other high-density traffic areas will require at least 18 to 24 in. of crushed rock to prevent subgrade deterioration. Haul road requirements will be minimized if work is accomplished during the normally dry months of the year. The performance of haul roads can usually be improved by placing a geotextile over the subgrade prior to placing the rock.

Structural Fill. In our opinion, any materials free of organics or other deleterious materials and approved by the geotechnical engineer are suitable for use as structural fill to establish site grades. However, silty soils are sensitive to moisture content and can be placed and adequately compacted only during the dry summer months. For construction during the wet winter and spring months, fills should be constructed using granular materials with less than 7% fines. The on-site gravels are suitable for use as general embankment fill. However, some of the gravels are silty and more sensitive to moisture during wet conditions.

Structural fill should be placed and compacted in accordance with the requirements for Method C of Section 2-03.3(14)(C) of the WSDOT Standard Specifications. In general, approved, fine-grained soils (i.e., silt and silty sand) used as structural fill should be placed in 8-in.-thick lifts (loose) and compacted using a medium-sized (48-in.-diameter) segmented-pad roller compactor to at least 95% of the maximum dry density determined according by AASHTO T99 or ASTM D698. Organic debris or pieces of rock or concrete larger than about 6 in. should be removed from the fill prior to compaction. The moisture content of fine-grained soils at the time of compaction should be controlled to within 3% of optimum. Some aeration and drying of fine-grained soils may be required to achieve the recommended compaction criteria.

Granular structural fill material used to construct structural fills or work pads during wet weather can consist of relatively clean, granular material, such as sand, sand and gravel, or crushed rock with a maximum size of about 4 in. and not more than about 7% passing the No. 200 sieve (washed analysis). Gravel Borrow meeting the requirements of Section 9-03.14(1) of the WSDOT Standard Specifications meets these requirements and can be used as granular structural fill. The first lift of granular fill material placed over a silt subgrade should be in the range of 12 to 18 in. thick (loose). Subsequent lifts should be 12 in. thick (loose). Granular structural fill material should be compacted to at least 95% of the maximum dry density as determined by AASHTO T99/ASTM D698 (material with less than 30% retained on the No. 4 sieve) or AASTHTO T180/ASTM D698 (material with more than 30% retained on the No. 30 sieve) using a medium-weight (48-in.-diameter drum), smooth, steel-wheeled vibratory roller.

Culvert Excavation, Temporary Shoring, and Dewatering. We understand the roadway will be fully closed while the culverts are replaced and the excavations will generally be made using open-cut methods. Groundwater will generally be near the streambed of Manley Creek, which is at a depth of about 5 ft below NE Manley Road at the north culvert, about 20 ft below NE Manley Road at the center culvert, about 16 ft below NE Manley Road at the south culvert location. We anticipate excavations needed to construct the north, center, and south culvert foundations will extend to depths of about 10, 25, and 20 ft, respectively. Consequently, we anticipate about 5 ft of groundwater drawdown will be needed to construct the culverts.

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

Based on the borings and our site observations, we anticipate the excavations for the north culvert will generally encounter existing embankment fill and alluvial silt and sand; excavations for the center culvert will encounter existing embankment fill and alluvial gravels; and excavations at the south culvert location will encounter existing embankment fill and Troutdale-formation gravel. Excavations in the embankment fill and alluvium can be made with conventional excavation equipment. We anticipate excavations into the Troutdale formation gravel at the south culvert location can be made with a large trackhoe equipped with a bucket fitted with rock teeth. Cobbles and boulders will be encountered in the excavation throughout the fill, alluvial gravels, and Troutdale-formation gravel. Deeper excavations into strongly cemented Troutdale-formation gravel, if encountered, may require rock excavation techniques, such as the use of a large trackhoe equipped with a hydraulic splitter. Although obvious indications of significant cementation were not encountered in the borings, stronger cementation was observed in the cut slope opposite of Wall 2 and at the nearby Tapani quarry.

The soils anticipated at each of the project sites classify as a Type C soils per Chapter 296-155 of the Washington Administrative Code. Temporary excavations greater than 4 ft should either be sloped no steeper than 1.5H:1V or shored. If groundwater seepage or sloughing soils are encountered, flatter excavations slopes may be necessary.

A temporary excavation shoring system may be needed in areas where existing or proposed improvements are located in close proximity to the culverts. Potential methods of excavation support could include slide-rail shoring systems, cantilevered or anchored sheet pile walls, or cantilevered or anchored soldier pile walls. The presence of cobbles and boulders in the embankment fill, alluvial gravels, and Troutdale formation could present hard pile driving installation, particularly for sheet piles. Construction of drilled soldier pile walls is an alternative to driven piles depending on the actual conditions and required pile embedment. Cemented Troutdale-formation gravels, if present, could further limit the penetration of sheet piles. Internal/self-bracing shoring, such as slide-rail shoring or a trench box, may be an effective system to consider for this project. The slide-rail system is installed top-down as the excavation advances and does not require penetration below the base of the excavation. Temporary shoring for excavations should be designed to resist lateral earth pressures, hydrostatic pressures, and surcharge effects from traffic, equipment, materials, and excavation spoils.

The groundwater level should be lowered a minimum of 2 ft below the base of the excavation in order to allow construction of the culverts to occur in the dry soil and maintain stable subgrade conditions. At each culvert location, we anticipate groundwater drawdowns of about 5 ft will be needed in order to maintain dry subgrade conditions. We anticipate the soil to be dewatered at the north and center culverts will consist of alluvial gravels, while the soil to be dewatered at the south culvert will consist of Troutdale-formation gravels. These soil types are anticipated to be highly permeable and will yield large quantities of groundwater into the excavations. We anticipate sump pumps placed below the base of the excavation, deep wells, or a drilled-in well point system could be used to dewater the culvert excavations. Because of the presence of cobbles and boulders, we anticipate installation of a jetted well point system for this project will be infeasible. At the center culvert location, it may be possible to reduce groundwater inflow into the excavation by driving driven sheets into the underlying silt.

Permanent Slopes. The maximum inclination of final graded structural fill and cut slopes should be no steeper than 2H:1V. The area to be filled should be benched to provide a relatively level surface for fill placement and compaction. Typical benching requirements are shown on Figure 7. Structural fill slopes should be placed and compacted a minimum of 2 ft beyond the final slope configuration and then trimmed back to final grade. To reduce erosion and raveling, permanent cut or fill slopes should be hydroseeded as soon as practical or covered with mulch or erosion-control netting/blankets.

Seismic Design Considerations

General. We understand the proposed improvements will be seismically designed in accordance with the 2014 AASHTO LRFD BDS and applicable WSDOT guidelines. Section 11.5.4.2 of the AASHTO LRFD BDS indicates seismic design of retaining walls is not required if the effective peak ground acceleration, A_s, is less than 0.4g. The effective peak ground acceleration at the site is 0.32 g. Accordingly, seismic design for the proposed retaining walls is not required by the code. We understand the project structural engineer may evaluate seismic loads in their design.

Seismic Design Considerations. The earthquake-induced peak bedrock acceleration and spectral response accelerations for the site are based on an approximate 1,000-year return interval seismic hazard (probability of exceedance of 7% in 75 years). The site response can be determined in accordance with AASHTO LRFD BDS using the seismic design parameters provided in Table 3.

Table 3: SEISMIC DESIGN PARAMETERS

Site Latitude and Longitude at Mid-Point of Alignment:	45.8042 / -122.5798	
Site Class Based on Soil Conditions:	Site Class =	D
Peak Horizontal Ground-Acceleration Coefficient on Class B Rock, g:	PGA =	0.25
0.2-Sec Period Spectral Acceleration Coefficient on Class B Rock, g:	S_s =	0.59
1.0-Sec Period Spectral Acceleration Coefficient on Class B Rock, g:	S_1 =	0.21
Site Coefficient for the Peak Ground-Acceleration Coefficient:	F_{pga} =	1.30
Site Coefficient for 0.2-Sec Period Spectral Acceleration:	F_a =	1.33
Site Coefficient for 1.0-Sec Period Spectral Acceleration:	F_v =	1.98
Effective Peak Ground-Acceleration Coefficient, g:	$A_s = F_{pga}(PGA)$ =	0.32
Design Earthquake Response Spectral Acceleration Coefficient at 0.2-Sec Period, g:	$S_{DS} = F_aS_s$ =	0.78
Design Earthquake Response Spectral Acceleration Coefficient at 1.0-Sec Period, g:	$S_{D1} = F_vS_1$ =	0.41

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Liquefaction. Liquefaction is a process by which loose, saturated, granular materials, such as sand and, to a somewhat lesser degree, soft non-plastic and low-plasticity silts, temporarily lose strength during and immediately after a seismic event. Liquefaction occurs as seismic shear stresses propagate through a saturated soil and distort the soil structure, causing loosely packed groups of particles to contract or collapse. If drainage is impeded and cannot occur quickly, the collapsing soil structure increases the pore-water pressure between the soil grains. If the pore-water pressure increases to a level approaching the weight of the overlying soil, the granular layer temporarily behaves as a viscous liquid rather than a solid. As strength is lost, there is an increased risk of settlement, lateral spreading, and/or bearing capacity failures of structures supported by shallow foundations. Liquefaction-induced settlement occurs as the elevated pore-water pressures dissipate and the soil consolidates after the earthquake.

The soils at the center and south culvert locations and Walls 2 and 3 consist of either alluvial gravels or silt and gravel of the Troutdale formation. Due to the soil composition, soil density/consistency, and/or depth to groundwater, the risk of liquefaction at the locations of these project elements is considered to be low. Saturated, loose/soft alluvial silts and sands were encountered in boring B-5 at the north culvert location; these saturated alluvial deposits are considered to be susceptible to liquefaction as a result of seismic shaking.

The potential of liquefaction at the north culvert location was evaluated using the simplified procedure described by Idriss and Boulanger (2008). Based on the deaggregation of the earthquake sources, subduction, subcrustal, and local crustal earthquakes all contribute to the AASHTO LRFD BDS code-based seismic hazard at the site. Therefore, a range of magnitude and peak ground acceleration pairs were evaluated to consider the relative contributions of the varying earthquake sources. Our analysis indicates subduction zone earthquakes tend to control the earthquake hazard at the site. In our liquefaction evaluation, we considered a magnitude (M) 9.0 earthquake with an effective peak ground acceleration of 0.32 g to model a subduction zone earthquake. Groundwater was assumed at elevation +95 ft, which is the approximate elevation of the existing creek at the north culvert location. Our analysis indicates the silty sand present between the creek bed elevation at elevation +90 ft and the sandy silt encountered below elevation +81 ft are susceptible to liquefaction as a result of the code-based design-level earthquake.

We estimated the liquefaction-induced free-field settlement at the north culvert location using an empirical methodology developed by Ishihara and Yoshimine (1992), which is based on case histories of areas that have undergone liquefaction. Using this procedure, we estimate between 2 and 4 in. of free-field liquefaction settlement may occur in the sandy silt encountered below elevation +81 ft as a result of the code-based design-level earthquake. The silty sand present below the streambed and above elevation +90 ft may also settle as a result of liquefaction; however, we recommend these soils be removed from below the base of the culvert foundation (approximate 2 ft deep) to reduce potential seismic settlement. We estimate differential settlement along the length of the culvert may be on the order of one-half of the total settlement described above.

Other Seismic Hazards. The ground surface at the location of the north culvert is relatively flat and the width of the streambed is relatively narrow. Therefore, it is our opinion the risk of lateral spreading at this location is low, even though our analysis indicates the potential for liquefaction at this location exists. Based on the elevation and location of the site, the risk of damage by tsunami and/or seiche is absent. Review of available geologic information disclosed no mapped faults on or near the site, and it is our opinion the potential for fault rupture at the site is low, unless occurring on a previously unmapped or unknown fault.

Culvert Foundation Design

General. The preliminary design drawings included in Otak's September 27, 2017, Alternatives Analysis report indicate the foundation for the south culvert crossing will be situated at about elevation 208 ft, the foundation for the center culvert will range from about elevation 118 ft at the north end to about elevation 126 ft at the south end, and the foundation for the north culvert will be situated at about elevation 92 ft. Based on the conditions observed in the borings advanced for this investigation, we anticipate the soil at the planned subgrade elevation at the south culvert location will consist of medium-dense to very dense gravel (Troutdale formation), the soil at the subgrade elevation at the center culvert location will consist of upwards of 5 ft of medium-dense gravel (alluvium) underlain by medium-stiff to hard silt (Troutdale formation), and the soil at the subgrade elevation at the north culvert location will consist of about 2 ft of loose silty sand underlain by medium-dense to very dense gravel (alluvium). For foundation support, we recommend culvert footings be provided with a minimum of 1.5 ft of embedment below the anticipated scour depth.

In general, the soils encountered at the planned foundation subgrade elevations will provide suitable support for the culverts. However, the silty sand encountered at the planned subgrade elevation for the north culvert location is wet and will be easily softened by construction activities. In addition, this material is considered to be potentially susceptible to liquefaction. In order to provide a firm base for support of the culvert and increase the culvert's seismic performance, we recommend the silty sand material be removed (overexcavated) to expose the underlying gravels (anticipated at about elevation 90 ft) and the overexcavation be backfilled with Gravel Backfill for Foundations meeting the requirements of Section 9-03.12(1) of the WSDOT Standard Specifications.

Excavation for the culverts should be made with excavators equipped with smooth-edge buckets and the subgrade should be observed by a qualified member of GRI's geotechnical engineering staff. If loose soils are encountered at the base of the excavation, it will be necessary to overexcavate and replace the unsuitable materials with Gravel Backfill for Foundations meeting the requirements of Section 9-03.12(1) of the WSDOT Standard Specifications. To provide uniform support, all culvert and wing wall footings should be founded on a minimum 6-in. thickness of compacted crushed rock, such as Crushed Surfacing Base Course meeting the requirements of Section 9-03.9(3) of the WSDOT Standard Specifications. Backfill placed below culverts or wing walls should be backfilled in accordance with Section 2-09.3(1)E of the WSDOT Standard Specifications.

For the Strength I and Extreme Event I limit states, the nominal bearing resistance, qn, for the culvert foundations founded as discussed above may be proportioned using the following equation:

$$qn\ (psf) = 3000 + 1500B'$$

where B′ is the effective footing width in feet.

For evaluation of the footings at the Strength I and Extreme Event I limit states, resistance factors of 0.45 and 1.00, respectively, should be applied to the nominal bearing resistance.

In our opinion, a resistance factor of 1.00 and nominal bearing resistances of 4,000 psf, 2,500 psf, and 4,000 psf are appropriate at the Service I limit state for the south, center, and north culverts, respectively. The nominal bearing resistances were selected to limit total foundation settlement to less than 1 in. for effective culvert footing widths of 10 ft or less. We anticipate most of the settlement described above will occur during construction or as the excavation is backfilled over the culvert. Assuming similarly loaded foundation elements, we estimate differential settlement between two points along the length of the culverts will be 1/2 in. or less. If the foundations are dissimilarly loaded, differential settlement and distortion will be greater than what is estimated above.

Recommendations for lateral earth pressures for culvert walls are provided in the Retaining Walls section of this report.

Retaining Walls

General. Based on the conditions observed in our explorations, the retaining walls will generally be founded on and retain medium-dense to very dense gravel of the Troutdale formation. The Troutdale formation encountered in our borings exhibited minimal cementation or very light cementation. However, GRI did observe strongly cemented Troutdale gravel on a cut slope located opposite of Wall 2 and at the nearby Tapani quarry. The contractor should be prepared to make excavations into the strongly cemented Troutdale gravels.

For Walls 2 and 3, we anticipate feasible retaining wall types could include CIP concrete or modular concrete block retaining walls. MSE walls may be used, provided sufficient space is available to make temporary construction backslopes and there is sufficient frictional resistance to resist the lateral loads. The existing slope uphill of Wall 1 is inclined at about 1.25H:1V, which will result in large lateral earth pressures that likely exceed the capacity of a CIP or gravity-type wall. Accordingly, a cantilevered or anchored soldier pile wall will likely be required at this location. Cantilevered or anchored soldier pile walls are also feasible alternatives for Walls 2 and 3.

Recommendations for the design of CIP concrete, modular concrete block, and MSE walls are provided in this section of the report. Recommendations for soldier pile walls are provided in the Soldier Pile Walls section of this report.

CIP Concrete Walls. Design lateral earth pressures for retaining walls depend on the type of construction, i.e., the ability of the wall to yield; the inclination of the backslope; soil properties; and surcharge loads. The two possible conditions regarding the ability of the wall to yield include the at-rest and the active earth pressure cases. The at-rest earth pressure case is applicable to a wall considered to be relatively rigid and laterally supported at its top and bottom and therefore unable to yield. The active earth pressure case is applicable to a wall capable of yielding slightly away from the backfill by either sliding or rotating about its base. A conventional cantilevered retaining wall is an example of a wall that develops the active earth pressure case by yielding.

Table 4 provides soil properties and active and at-rest earth pressure coefficients for CIP concrete wall design. The earth pressures assume drained conditions behind the wall and are provided for walls with level backslopes and backslopes up to 2H:1V above the horizontal for both the at-rest and active earth pressure conditions. The retaining walls may be subjected to the influence of surcharge loading due to vehicular traffic and construction loads and should be designed for additional horizontal pressure. It is typical design practice to accommodate traffic and typical construction equipment loading with a vertical surcharge pressure of 250 psf. Larger surcharge loads, such as loads induced by cranes, should be addressed individually or by use of a higher surcharge pressure. Lateral pressures due to surcharge loadings in the backfill area can be estimated using the guidelines provided on Figure 8.

Table 4: DESIGN PARAMETERS FOR CIP CONCRETE RETAINING WALLS (1)

	Wall Backfill Inclination	
Parameter	**Level**	**2H:1V**
Soil Unit Weight, γ	135 pcf	
Soil Friction Angle, ϕ	36°	
Active (Static) Earth Pressure Coefficient, K_a	0.26	0.37
Active Seismic Earth Pressure Coefficient, K_{ae}	0.35	0.72
At-Rest (Static) Earth Pressure Coefficient, K_o	0.41	0.59
At-Rest Seismic Earth Pressure Coefficient, K_{oe}	0.48	0.72(2)

Note:

1) To determine the equivalent fluid weight (pcf), multiply the soil unit weight by the applicable earth pressure coefficient.

2) At-rest seismic earth pressure coefficient for the 2H:1V backfill condition assumes the wall will yield during seismic loading such that active conditions develop.

For the Extreme Event I limit state, the horizontal earth pressure should be calculated and distributed as a single triangular pressure as specified in Section 11.6.5 of the 2014 AASHTO LRFD BDS. The resultant of the horizontal earth pressure can be assumed to act at a point $^{1}/_{3}$H above the base of the wall, where H is the height of the wall (including embedment) in feet. The seismic earth pressure coefficients for the Extreme Event I limit state include both static and dynamic lateral pressures and must not be added to the static lateral earth pressure.

Global stability of CIP concrete walls can be evaluated by GRI once the configuration of the retaining walls is determined.

Modular Block and MSE Walls. We recommend the soil parameters provided in Table 5 be used for design of modular block or MSE walls. The recommended parameters assume imported granular material will be used to backfill the wall, as described in the Wall Backfill and Drainage Considerations section of this report.

Table 5: MODULAR BLOCK AND MSE WALL SOIL DESIGN PARAMETERS

Soil Property	Wall Backfill	Retained Soil	Foundation Soil
Unit Weight, pcf	135	135	135
Friction Angle	36°	38°	38°
Cohesion, psf	0	0	0
	Strength I Limit State	**Extreme Event I Limit State**	
Effective Peak Ground Acceleration, A_s	Not Applicable	0.32g	

Design of modular block and MSE walls should include appropriate lateral earth pressures caused by sloping backfill and adjacent surface surcharge loads.

Modular block and MSE retaining walls are typically designed by the contractor. The design should include an evaluation of the internal stability, bearing capacity, foundation settlement, and external stability (i.e., sliding, overturning, and global stability) of the walls and be completed by a licensed civil or structural engineer. The design should be submitted to the County for review prior to construction.

Retaining Wall Foundation Design. CIP concrete, modular block, or MSE walls should be founded in firm, undisturbed, medium-dense to very dense gravel (Troutdale formation) or on structural fill placed above these materials. Excavations for the walls should be made with excavators equipped with a smooth-edge bucket and the subgrade should be observed by a qualified member of GRI's geotechnical engineering staff. If loose soils are encountered at the base of the excavation, it will be necessary to overexcavate and replace the unsuitable materials with Gravel Backfill for Foundations meeting the requirements of Section 9-03.12(1) of the WSDOT Standard Specifications. All prepared foundation bearing surfaces should be free of loose soil and water. To provide uniform support, the CIP retaining wall foundation and the facing units of the modular block or MSE walls should be founded on a minimum 6-in. thickness of compacted crushed rock, such as Crushed Surfacing Base Course meeting the requirements of Section 9-03.9(3) of the WSDOT Standard Specifications. All wall foundation materials should be placed and compacted in accordance with Section 2-09.3(1)E of the WSDOT Standard Specifications.

For foundation support, we recommend CIP concrete retaining walls be provided with a minimum of 1.5 ft of embedment. The toe of each modular block or MSE wall should be embedded at least 1 ft below the adjacent ground surface.

For the Strength I and Extreme Event I limit states, the nominal bearing resistance, qn, for the proposed retaining walls founded as discussed above may be proportioned using the following equation:

$$qn\ (psf) = 6500 + 5500B'$$

where B′ is the effective footing width in feet.

The nominal bearing resistances provided above assume the ground in front of the wall is horizontal and the groundwater table is located at a depth equal to at least 1.5B′. Shallow groundwater or a sloping ground surface would result in significantly lower nominal bearing resistances than what is provided above.

For evaluation of the walls at the Strength I limit state, resistance factors of 0.55, 0.65, and 0.45 should be applied to the nominal bearing resistances for CIP concrete, modular block, and MSE walls, respectively. For Extreme Event I limit states, the resistance factor should be set at 1.00 for all three wall types.

In our opinion, a nominal bearing resistance of 4,000 psf and a resistance factor of 1.00 is appropriate at the Service I limit state for all wall types. Provided the wall has an effective footing width of less than 10 ft and a service load of less than 4,000 psf, we estimate total foundation settlement will be less than 1 in. We anticipate most of the settlement described above will occur as the wall is backfilled (CIP concrete walls) or during construction (modular block and MSE walls). Assuming similarly loaded foundation elements, we estimate differential settlement between two points spaced 100 ft apart along the length of a wall will be 1/2 in. or less. If the foundations are dissimilarly loaded, differential settlement and distortion will be greater than what is estimated above.

Resistance to Lateral Loads. Friction developed between the base of the foundation and the supporting subgrade will resist lateral loads transmitted from the wall to the foundation soil. Passive earth pressures should not be included in lateral load resistance calculations. Table 6 provides nominal sliding coefficients to compute the nominal sliding resistance between the soil and the foundation. The nominal sliding coefficient should be applied to vertical buoyant dead loads only. Resistance factors for each wall type and for the Strength I and Extreme Event I limit states are also provided in Table 6.

Table 6: NOMINAL SLIDING COEFFICIENT AND RESISTANCE FACTORS

Wall/Foundation Type	Nominal Sliding Coefficient	Strength I Limit State Resistance Factor	Extreme Event I Limit State Resistance Factor
CIP Concrete Walls	0.73	1.00	1.00
CIP Concrete Culvert Footing	0.73	0.80	1.00
Precast Concrete Modular Block Walls	0.55	0.90	1.00
MSE with Discontinuous Reinforcement	0.73	1.00	1.00
MSE with Continuous Reinforcement	0.36	1.00	1.00

Wall Backfill and Drainage Considerations. We recommend the wall backfill consist of imported granular material. Wall backfill for CIP concrete and modular block walls should meet the requirements for Gravel Backfill for Walls in Section 9-03.12(2) of the WSDOT Standard Specifications, while backfill located within the reinforced zone of MSE walls should meet the requirements for Gravel Borrow for Structural Earth Walls in Section 9-03.14(4) of the WSDOT Standard Specifications. Wall backfill placement and compaction for CIP concrete walls should be completed as described in Section 6-11.3(5) of the WSDOT Standard Specifications. Wall backfill and placement for MSE walls should be completed as described in Section 6-13.3(7) of the WSDOT Standard Specifications. To limit the buildup of excess lateral pressures on the walls, we recommend all backfill located within 5 ft of the back of the wall be compacted with hand-operated or lightweight compaction equipment.

Drainage of the wall backfill is an essential element of wall design. We recommend a minimum 2-ft-wide by 2-ft-tall continuous drainage zone be placed behind the wall. The drainage zone should be located behind the heel of CIP concrete retaining walls, beyond the lowest block of modular block retaining walls, and beyond the reinforced zoned for MSE walls. The drainage zone should consist of Gravel Backfill for Drains meeting the requirements of Section 9-03.12(4) of the WSDOT Standard Specifications. The Gravel Backfill for Drains should be encapsulated by a Non-Woven Geotextile for Underground Drainage meeting the requirements for Moderate Survivability in Section 9-33.2(1) of the WSDOT Standard Specifications. A minimum 4-in.-diameter perforated drain pipe should be placed 6 in. above the base of the drainage blanket. The drain pipe should discharge into the stormwater drainage system or an approved location.

To reduce the possibility of water ponding and infiltrating into the subsurface behind retaining walls, the adjacent ground surface behind the wall should be sloped to promote runoff away from the top of the wall. Alternatively, a ditch could be constructed along the top of the wall to collect surface water runoff and route it to the storm drain system.

Soldier Pile Walls

General. Based on the conditions observed in our borings and during our geologic reconnaissance, we anticipate medium-dense to very dense gravelly sand and sandy gravel of the Troutdale formation will be exposed at the face of the soldier pile walls. The Troutdale-formation gravel exposed on the face of the existing slopes and in the nearby borings is very lightly cemented. Strongly cemented Troutdale gravel has been observed in the project area, including on the cut slope across from Wall 2 and at the existing Tapani pit located southwest of the site. The soldier pile retaining walls should be constructed in accordance with Section 6-16 of the WSDOT Standard Specifications. Design and installation of permanent ground anchors should be completed in accordance with Section 6-17 of the WSDOT Standard Specifications.

Lateral Earth Pressures. Lateral earth pressure diagrams for design of cantilevered and tieback soldier pile walls at the locations of Wall 1, Wall 2, and Wall 3 are provided on Figures 9 and 10 of this report. The lateral earth pressure diagrams include active, seismic earth, and nominal passive earth pressures. Surcharge pressures can be addressed using the criteria provided on Figure 8. Active, surcharge, and seismic earth pressures should be applied over the soldier pile spacing above the base of the excavation. Active and surcharge pressures should be applied over the width of the soldier pile below the base of the excavation. The nominal passive resistance should be applied over three soldier pile shaft diameters or the pile spacing, whichever is less.

A load factor of 1.0 is included in the active earth and seismic earth pressures presented on Figures 9 and 10. The earth pressures should be factored for the Strength I and Extreme Event I limit states in accordance with the 2014 AASHTO LRFD BDS. For the Strength I limit state, a resistance factor of 0.75 should be applied to the nominal passive resistance. For the Extreme Event I limit state, the resistance factor for the nominal passive pressure should be taken as 1.0.

Tieback Anchors. We recommend the bonded length of the tieback anchors begin a minimum of 5 ft beyond the no-load zone and in the medium-dense to very dense Troutdale-formation sands and gravels. The no-load zone extends upwards from the base of the wall at an angle of 60° above the horizontal. Typical bonded-zone lengths are on the order of 20 to 30 ft. Assuming a bond zone in this range of lengths, we anticipate a tieback anchor installed in a 6-in.-diameter hole drilled into the medium-dense to very dense gravelly sand and sandy gravel can be designed to develop a nominal anchor pullout resistance of 200 to 300 kips.

The upper portion of the Troutdale formation is not cemented, has minimal fines content, and is anticipated to cave during tieback anchor installation. As such, we anticipate the tieback anchors will need to be installed using cased installation methods. Following drilling and placement of the anchor bar or strands, grout is tremied under pressure through a pipe inserted to the bottom of the hole. The temporary casing is withdrawn as the grout is placed. A bond breaker, such as a plastic PVC sleeve attached to the bar or strands, should be placed in the unbonded zone. Provisions should be provided for secondary grouting, and it should be expected that at least one episode of secondary grouting will be needed to develop the nominal anchor pullout resistance described above.

The specific design and installation procedures for tieback anchors are developed by the specialty foundation contractor based on the performance criteria provided by the owner's geotechnical and structural engineers. Section 6.17.3(8)B of the WSDOT Standard Specifications requires that 5% of and a minimum of three production anchors be performance tested. All production anchors should be proof tested, except anchors that are performance tested. The performance and proof test loading schedules are provided in Section 6-17.3(8)B and Section 6-17.3(8)C of the WSDOT Standard Specifications. Acceptance criteria for the proof and performance tests are provided in Section 6-17.3(9) of the WSDOT

Standard Specifications. Guidelines in Section 11.5.7-1 of the 2014 AASHTO LRFD BDS allow use of a resistance factor of 1.0 to compute the factored soil-to-grout bond resistance when proof tests are completed on all production anchors.

Following successful testing, the production anchors should be locked off at 60% of the factored design load. We recommend the installation and testing of the tieback anchors be observed on a full-time basis by qualified geotechnical engineering staff representing the design team and Clark County.

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

Stormwater Infiltration

Four infiltration tests, designated TP-1 through TP-4, were completed to estimate the infiltration rate (in./hr). The location of the infiltration tests are shown on the Site Plans, Figures 2 to 6. Each test was completed in general accordance of the Small-Scale Pilot Infiltration Test procedure as described in Appendix 1-C in Book 1 of the 2015 Clark County Stormwater Manual. Tests TP-1 and TP-2 were completed on the Lindberg Property, while tests TP-3 and TP-4 were completed on the Richards Property. The soil encountered in tests TP-2 through TP-4 consist of sandy gravel with trace silt. The soil encountered at the location of test TP-1 is similar in composition but siltier, resulting a lower measured infiltration rate than the other tests. Cobbles were encountered at each location. The tests results are summarized in Table 7.

Table 7: SUMMARY OF MEASURED INFILTRATION RATE (IN./HR)

Test Pit Designation	Test Location	Depth of Infiltration Test, ft	Soil Type at Test Depth	Measured Infiltration Rate, in./hr
TP-1	Lindberg Property	4	Sandy GRAVEL, some silt, contains cobbles	10.4
TP-2	Lindberg Property	4	Sandy GRAVEL, trace silt, contains cobbles	26.2
TP-3	Richards Property	3	Sandy GRAVEL, trace silt, contains cobbles	25.6
TP-4	Richards Property	4	Sandy GRAVEL, trace silt, contains cobbles	39.9

The field infiltration rate should be reduced using the correction factors provided in Table 4.2 of Book 1 of the 2015 Clark County Stormwater Manual. Provided the lower of the measured infiltration rates at each property is used, the soils correction factor can be taken as 2.0.

We understand porous pavement is being considered to dispose of stormwater. Based on the conditions observed in our explorations, we anticipate the soils located between about Station 47+00 and 33+00 consist of gravelly sand or sandy gravel with variable amounts of silt. In our opinion, infiltration of stormwater into these soils using porous pavement is likely feasible; however, we recommend additional infiltration testing be completed to further evaluate the infiltration rate in this area and the impacts of introducing stormwater to the stability of the embankment slopes. North of Station 47+00, the near-surface soils consist primarily of sandy silt or silty sand, which will likely have a very low infiltration rate. In our opinion, the feasibility of using porous pavement to infiltrate stormwater in the northern reaches of the project alignment is low. Except for at the south culvert location, GRI did not complete any geotechnical investigations in the portion of the alignment located south of about Station 33+00. According to the geologic mapping, the near-surface soils along this stretch are Missoula flood deposits consisting of sand and silt. We anticipate these deposits could have a very low infiltration rate and may not be suitable for porous pavements.

Pavement Design

We understand pavement design for the project will be completed using the Clark County Standard Details, which provide recommended asphalt pavement and crushed-rock thicknesses for varying road classifications and AASHTO soil classifications. North of Station 47+00, the near-surface soils observed in our borings classify as AASHTO Soil Type A-4. Between Stations 47+00 and 33+00, the near-surface soils would be classified as AASHTO Soil Type A-1. GRI did not complete any investigations south of Station 33+00, but we anticipate the near-surface soils in this area would classify as AASHTO Soil Type A-4 based on our review of geologic mapping and our experience. We recommend this be verified as part of final design.

DESIGN REVIEW AND CONSTRUCTION SERVICES

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

LIMITATIONS

This report has been prepared to aid the project team in the design of this project. The scope is limited to the specific project and location described herein, and our description of the project represents our understanding of the significant aspects of the project relevant to site preparation, earthwork, and design and construction of culverts, retaining walls, stormwater facilities, and pavements. In the event any changes in the design and locations of the project elements as outlined in this report are planned, we should be given the opportunity to review the changes and modify or reaffirm the conclusions and recommendations of this report in writing.

The conclusions and recommendations submitted in this report are based on the data obtained from the borings and infiltration test pits made at the locations indicated on Figures 2 through 6 and other sources of information discussed in this report. In the performance of subsurface investigations, specific information is obtained at specific locations at specific times. However, it is acknowledged that variations in soil conditions may exist between exploration locations. This report does not reflect any variations that may occur between these explorations. The nature and extent of variation may not become evident until construction. If, during construction, subsurface conditions differ from those encountered in the explorations, we should be advised at once so we can observe and review these conditions and reconsider our recommendations where necessary.

Please contact the undersigned if you have any questions regarding this report.

Submitted for GRI,

Matthew S. Shanahan, PE Principal	Brian A. Bennetts, PE Senior Engineer	Thomas P. Gayne, PE Staff Engineer

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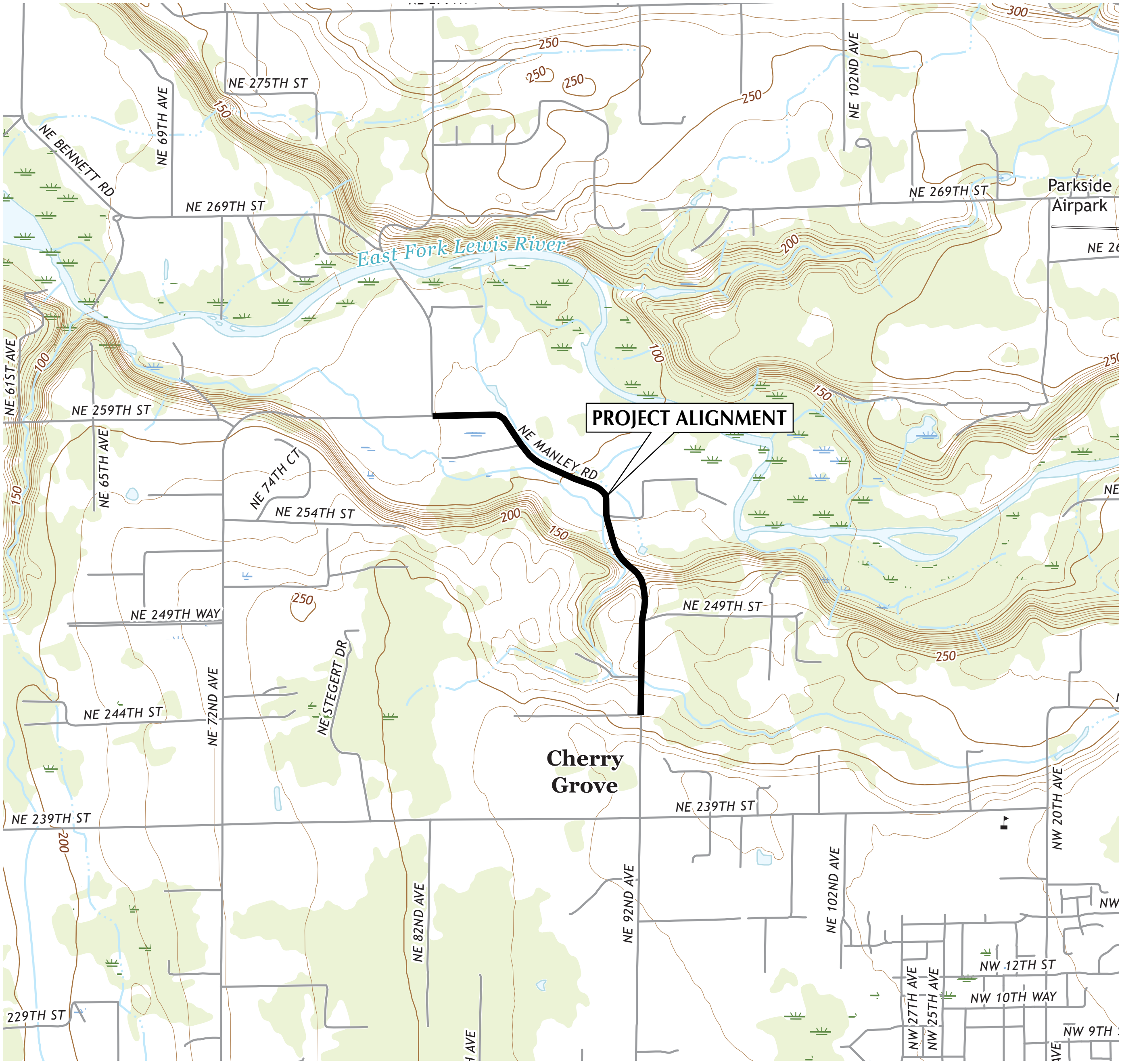
AASHTO, 2014, AASHTO LRFD Bridge Design Specifications, 7th edition.

Howard, K.A., 2002, Geologic map of the Battle Ground 7.5-minute quadrangle, Clark County, Washington: USGS, Miscellaneous Field Studies, MF-2395.

Idriss, I. M., and Boulanger, R. W., 2008, Soil liquefaction during earthquakes: Earthquake Engineering Research Institute, Oakland, California.

Ishihara, K., and Yoshimine, M., 1992, Evaluation of settlements in sand deposits following liquefaction during earthquakes: Soils and Foundations, v. 32, no. 1, pp. 173-188.

This document has been submitted electronically.



USGS TOPOGRAPHIC MAP
BATTLE GROUND, WASH. (2017)

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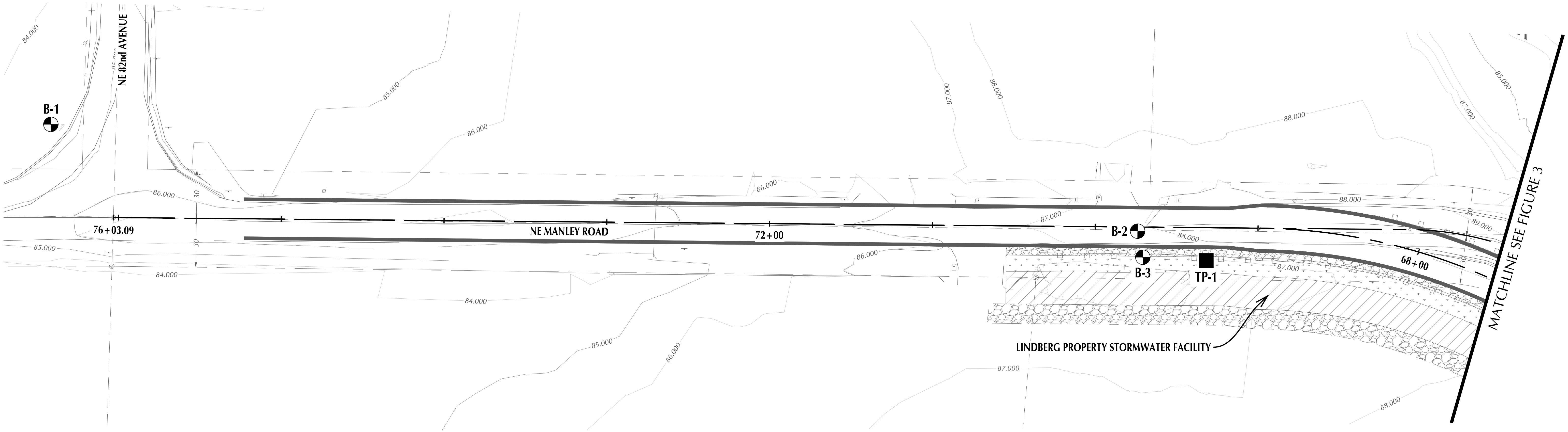
VICINITY MAP

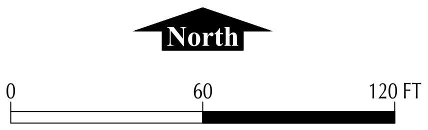
NOV. 2017 JOB NO. W1212-1 FIG. 1



■ TEST PIT COMPLETED BY GRI
(OCTOBER 11, 2017)

◒ BORING COMPLETED BY GRI
(AUGUST 21 - 22, 2017)

SITE PLAN FROM FILE BY CLARK COUNTY PUBLIC WORKS



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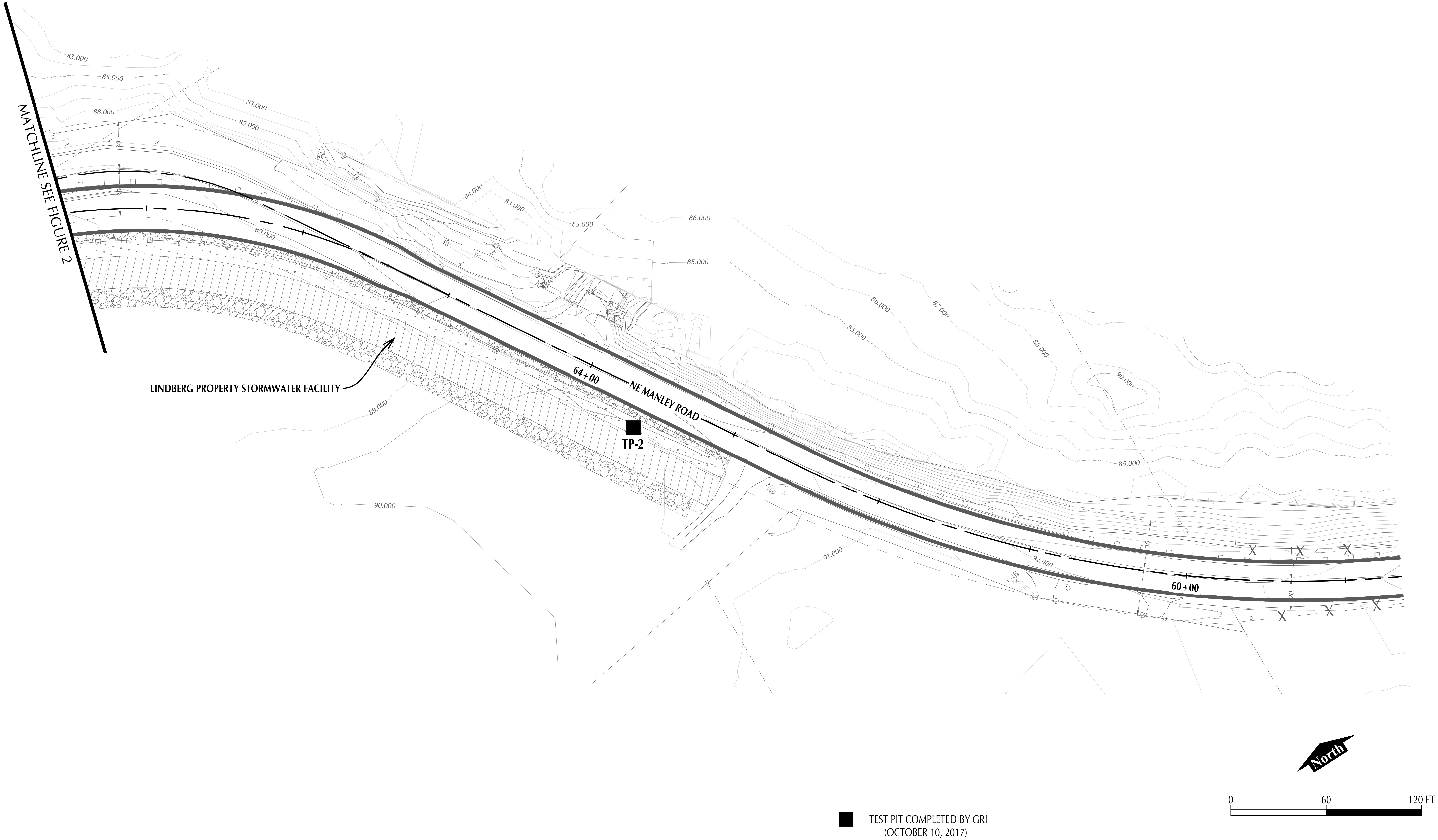
SITE PLAN

NOV. 2017 JOB NO. W1212-1 FIG. 2



■ TEST PIT COMPLETED BY GRI
(OCTOBER 10, 2017)

SITE PLAN FROM FILE BY CLARK COUNTY PUBLIC WORKS

GRI

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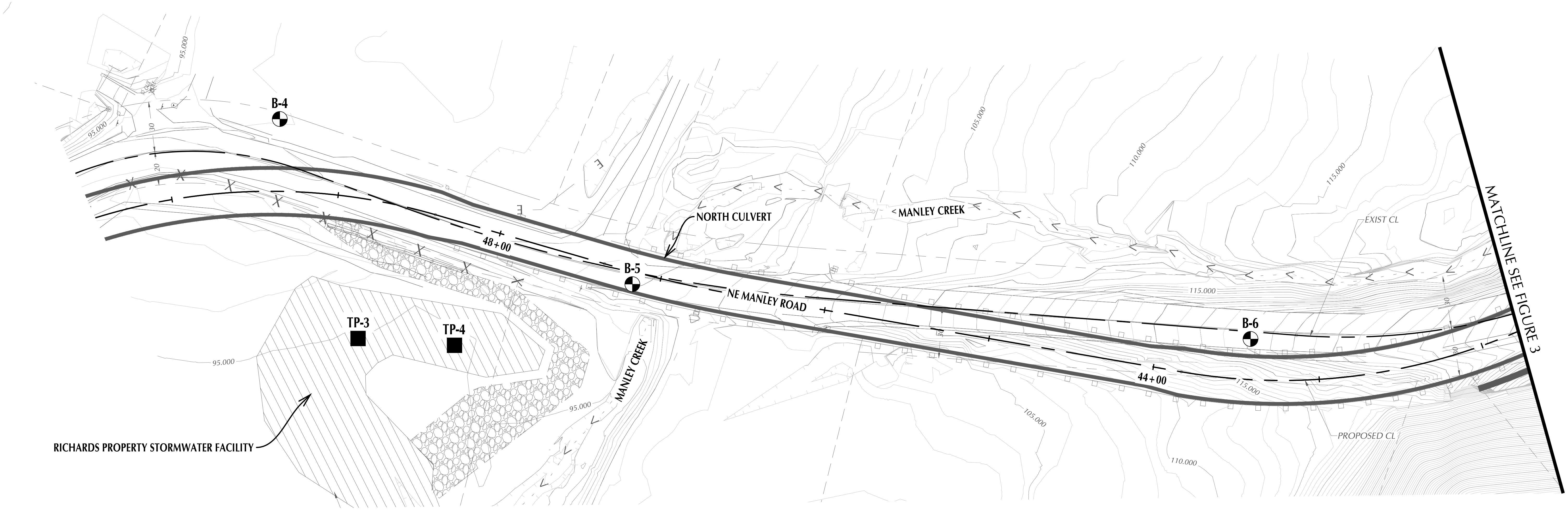
SITE PLAN

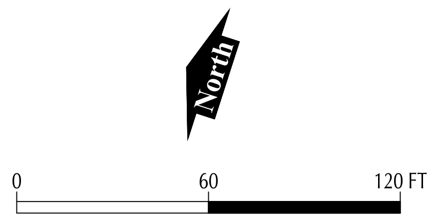
NOV. 2017 JOB NO. W1212-1 FIG. 3



■ TEST PIT COMPLETED BY GRI
(OCTOBER 9, 2017)

◐ BORING COMPLETED BY GRI
(AUGUST 21 - 22, 2017)

SITE PLAN FROM FILE BY CLARK COUNTY PUBLIC WORKS



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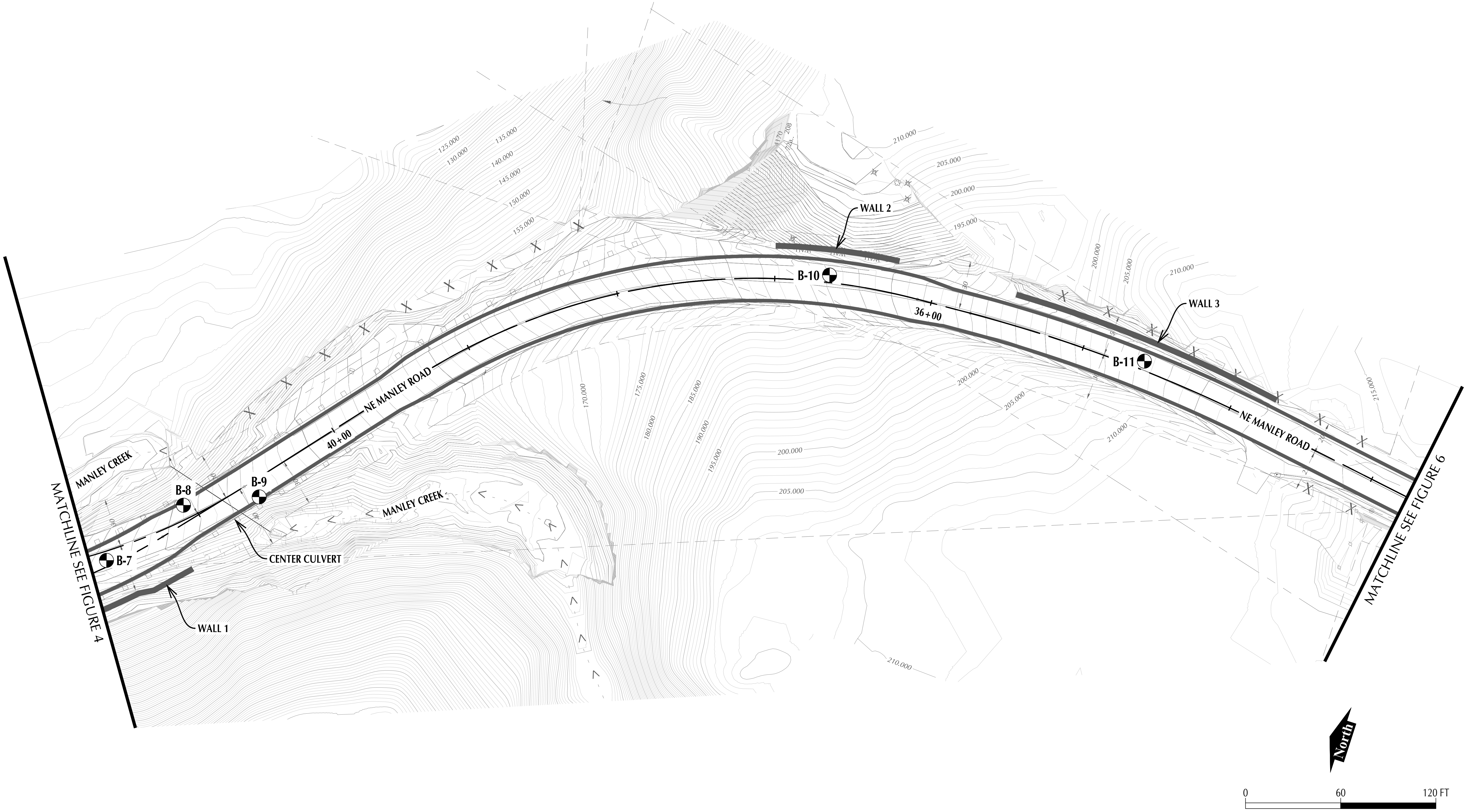
SITE PLAN

NOV. 2017 JOB NO. W1212-1 FIG. 4



⊕ BORING COMPLETED BY GRI
(AUGUST 21 - 22, 2017)

SITE PLAN FROM FILE BY CLARK COUNTY PUBLIC WORKS

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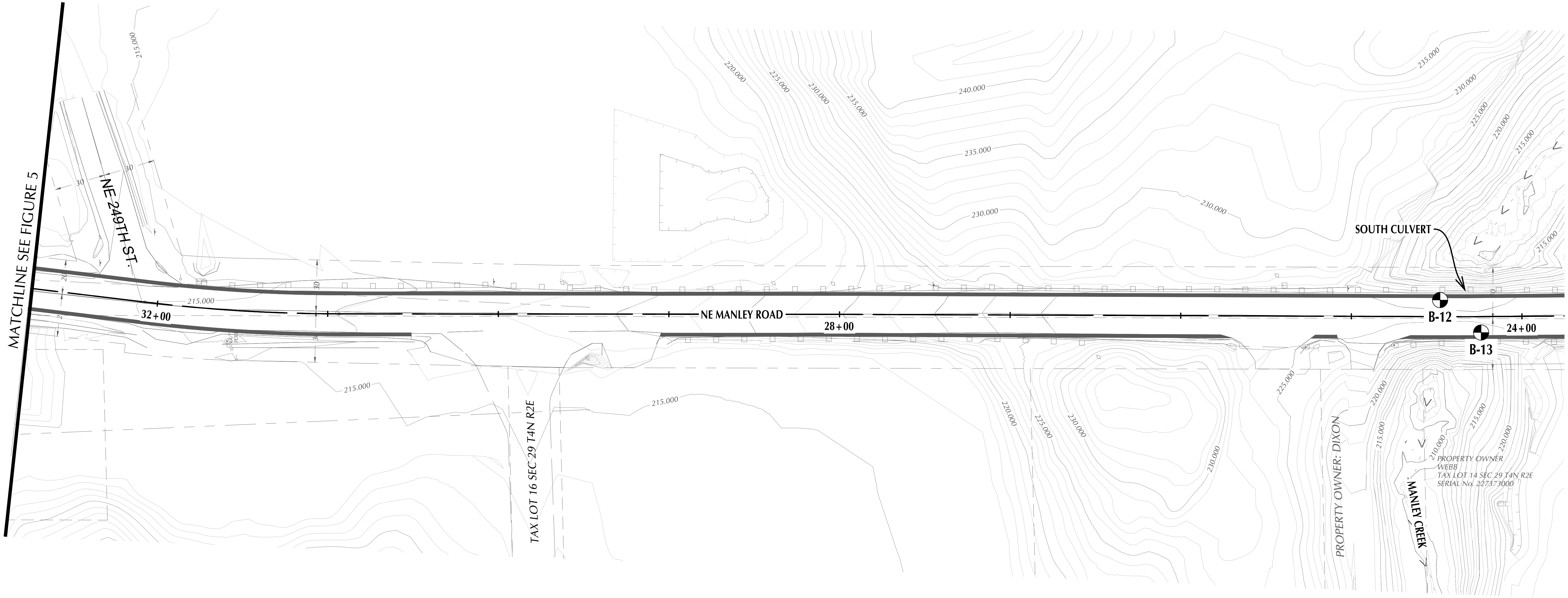
NE MANLEY ROAD IMPROVEMENTS

SITE PLAN

NOV. 2017 JOB NO. W1212-1 FIG. 5



● BORING COMPLETED BY GRI
(AUGUST 23, 2017)

SITE PLAN FROM FILE BY CLARK COUNTY PUBLIC WORKS

North

0 60 120 FT

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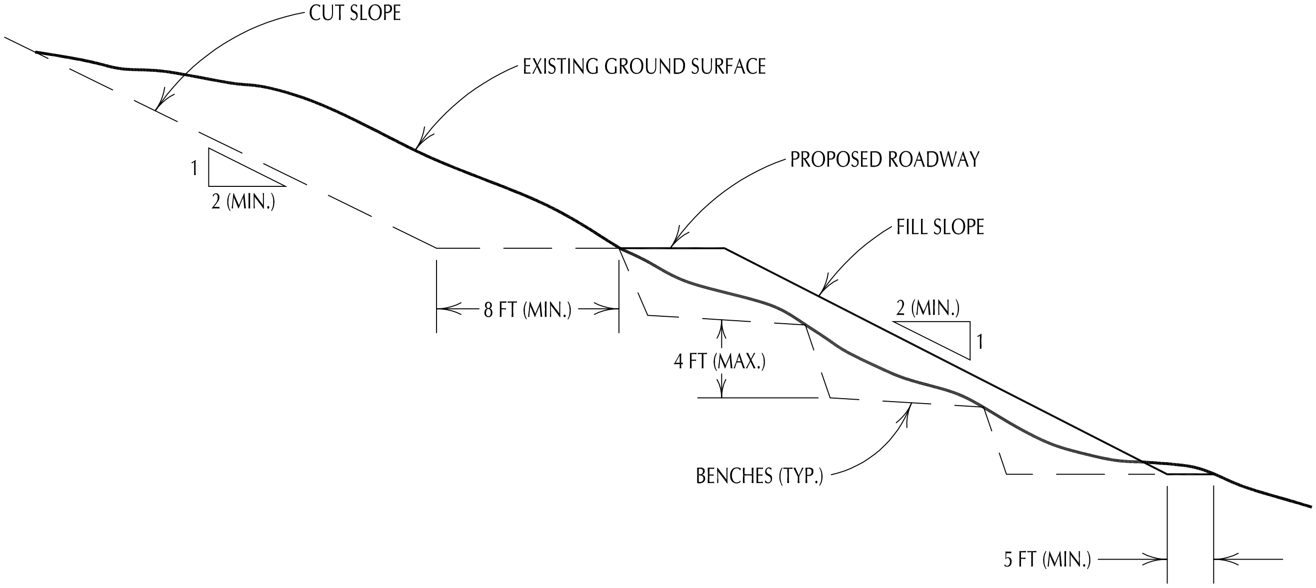
SITE PLAN

NOV. 2017 JOB NO. W1212-1 FIG. 6



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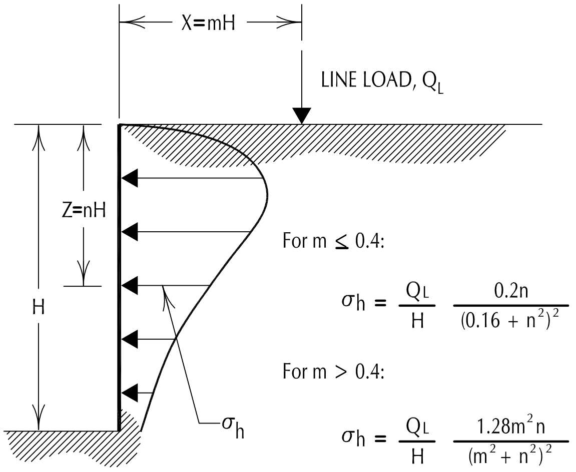
TYPICAL DETAIL FOR FILLING ON SLOPES

NOV. 2017 JOB NO. W1212-1 FIG. 7



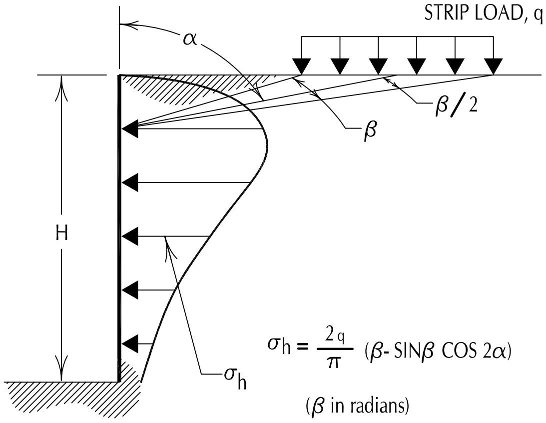
For m ≤ 0.4:

$$\sigma_h = \frac{Q_L}{H} \quad \frac{0.2n}{(0.16 + n^2)^2}$$

For m > 0.4:

$$\sigma_h = \frac{Q_L}{H} \quad \frac{1.28m^2 n}{(m^2 + n^2)^2}$$

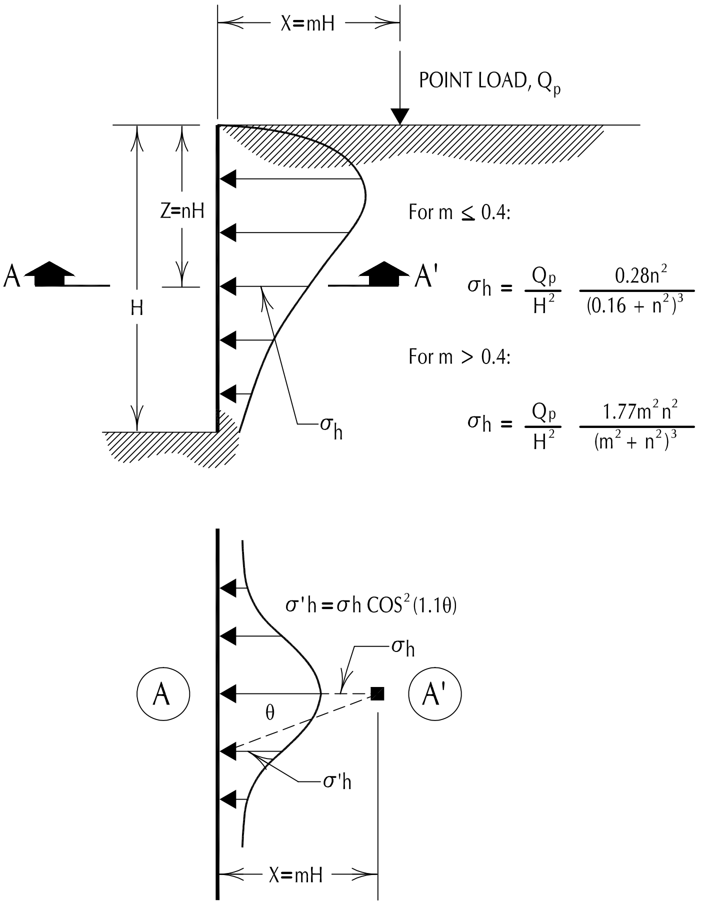
LINE LOAD PARALLEL TO WALL



$$\sigma_h = \frac{2q}{\pi} \; (\beta\text{- SIN}\beta \text{ COS } 2\alpha)$$

(β in radians)

STRIP LOAD PARALLEL TO WALL



For m ≤ 0.4:

$$\sigma_h = \frac{Q_p}{H^2} \quad \frac{0.28n^2}{(0.16 + n^2)^3}$$

For m > 0.4:

$$\sigma_h = \frac{Q_p}{H^2} \quad \frac{1.77m^2 n^2}{(m^2 + n^2)^3}$$

DISTRIBUTION OF HORIZONTAL PRESSURES

VERTICAL POINT LOAD

NOTES:

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

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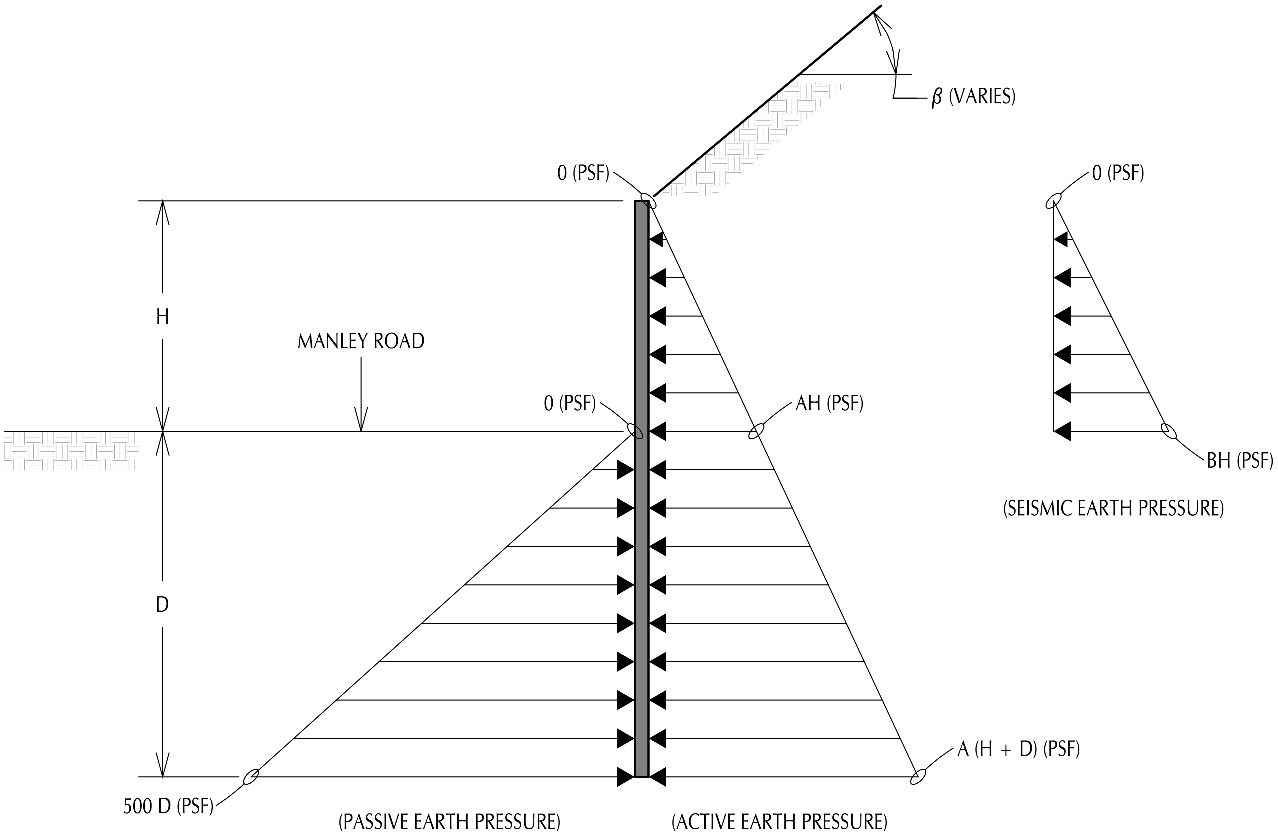
SURCHARGE-INDUCED LATERAL PRESSURE

NOV. 2017 JOB NO. W1212-1 FIG. 8



β	A	B
0° - LEVEL BACKSLOPE	32	12
26.6° (2H: 1V)	45	33
40.0° (1.25H: 1V)	77	280

NOTES:

1) SOLDIER PILE WALL SHOULD BE DESIGNED IN ACCORDANCE WITH THE 2014 AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS AND THE TEXT OF THIS REPORT.
2) ALL PRESSURES PRESENTED ARE IN TERMS OF POUNDS PER SQUARE FOOT (PSF), WITH ALL DIMENSIONS IN FEET.
3) ACTIVE EARTH PRESSURE AND SEISMIC EARTH PRESSURES ASSUMES A CANTILEVERED SOLDIER PILE WALL.
4) THE ACTIVE EARTH PRESSURE AND SEISMIC EARTH PRESSURES ARE DEPENDENT ON THE BACKSLOPE BEHIND THE WALL. SEE TABLE FOR COEFFICIENTS, A AND B, FOR HORIZONTAL BACKSLOPES, AND FOR BACKSLOPES INCLINED AT 2H:1V, AND BACKSLOPES INCLINED AT 1.25H 1V.
5) SURCHARGE INDUCED LATERAL PRESSURES, IF ANY, SHOULD BE INCLUDED IN THE DESIGN USING THE GUIDELINES PROVIDED ON FIGURE 8.
6) ACTIVE EARTH PRESSURE, SURCHARGE-INDUCED LATERAL PRESSURE, AND SEISMIC EARTH PRESSURE CAN BE ASSUMED TO ACT OVER THE ENTIRE EXPOSED WALL AREA AND OVER THE WIDTH OF THE SOLDIER PILE BELOW THE LAGGING.
7) A LOAD FACTOR OF 1.0 IS INCLUDED IN THE ACTIVE EARTH PRESSURE, SURCHARGE-INDUCED LATERAL EARTH PRESSURE AND SEISMIC EARTH PRESSURES.
8) THE PASSIVE EARTH PRESSURE CAN BE ASSUMED TO ACT OVER 2 SOLDIER PILE DIAMETERS (ACTUAL AREA) FOR MINIMUM SPACING OF THREE DIAMETERS.
9) A RESISTANCE FACTOR OF 0.75 AND 1.00 SHOULD BE APPLIED TO THE PASSIVE EARTH PRESSURE FOR THE STRENGTH I AND EXTREME EVENT I LIMIT STATE, RESPECTIVELY.

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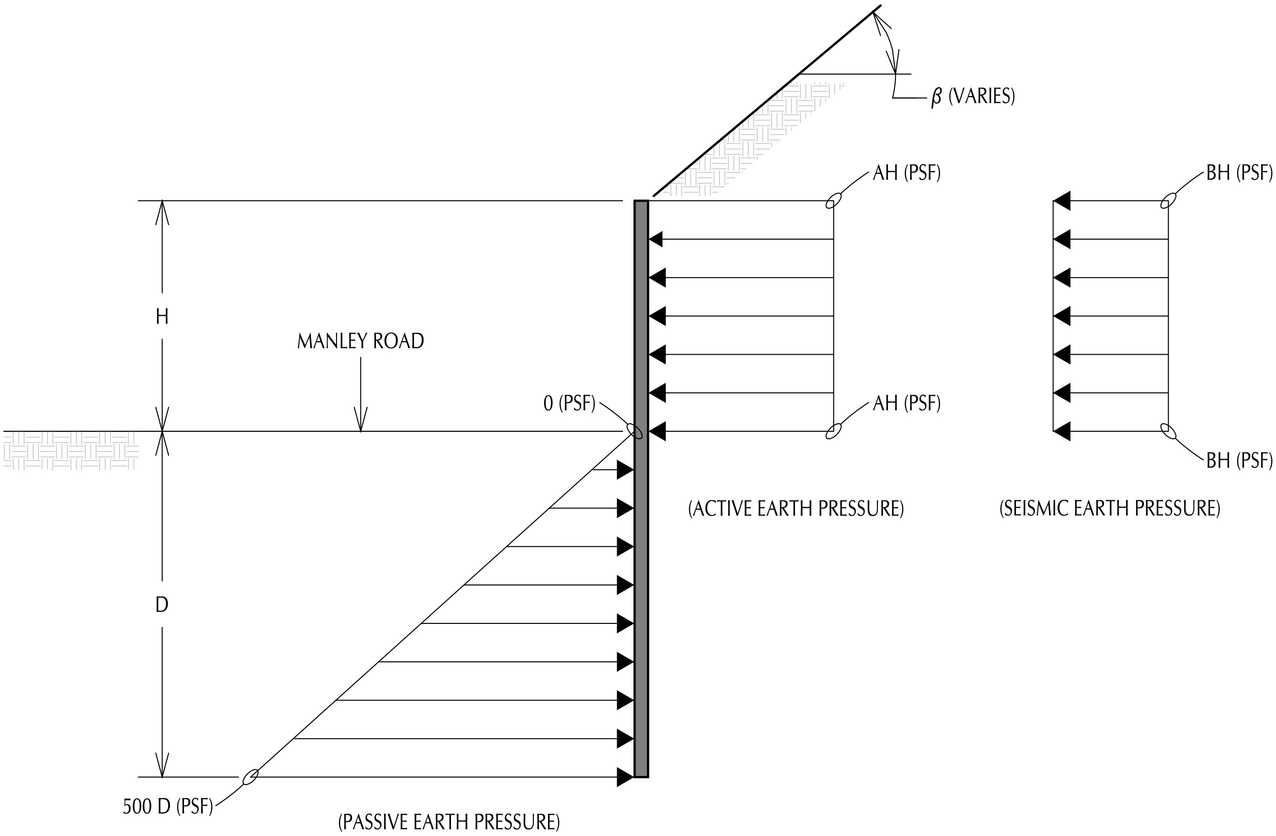
LATERAL EARTH PRESSURES

(CANTILEVERED SOLDIER PILE - WALLS 1, 2, AND 3)

NOV. 2017 JOB NO. W1212-1 FIG. 9



β	A	B
0° - LEVEL BACKSLOPE	21	8
26.6° (2H: 1V)	29	21
40.0° (1.25H: 1V)	50	183

NOTES:

1) SOLDIER PILE WALL SHOULD BE DESIGNED IN ACCORDANCE WITH THE 2014 AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS AND THE TEXT OF THIS REPORT.
2) ALL PRESSURES PRESENTED ARE IN TERMS OF POUNDS PER SQUARE FOOT (PSF), WITH ALL DIMENSIONS IN FEET.
3) ACTIVE EARTH PRESSURE AND SEISMIC EARTH PRESSURE DIAGRAM IS FOR A TIEBACK SOLDIER PILE WALL.
4) THE ACTIVE EARTH PRESSURE AND SEISMIC EARTH PRESSURES ARE DEPENDENT ON THE BACKSLOPE BEHIND THE WALL. SEE TABLE FOR COEFFICIENTS, A AND B, FOR HORIZONTAL BACKSLOPES, BACKSLOPES INCLINED AT 2H:1V, AND BACKSLOPES INCLINED AT 1.25H: 1V.
5) SURCHARGE INDUCED LATERAL PRESSURES, IF ANY, SHOULD BE INCLUDED IN THE DESIGN USING THE GUIDELINES PROVIDED ON FIGURE 8.
6) ACTIVE EARTH PRESSURE, SURCHARGE-INDUCED LATERAL PRESSURE AND SEISMIC EARTH PRESSURE CAN BE ASSUMED TO ACT OVER THE ENTIRE EXPOSED WALL AREA.
7) A LOAD FACTOR OF 1.0 IS INCLUDED IN THE ACTIVE EARTH PRESSURE, SURCHARGE-INDUCED LATERAL PRESSURE, AND SEISMIC EARTH PRESSURES.
8) THE PASSIVE EARTH PRESSURE CAN BE ASSUMED TO ACT OVER 2 SOLDIER PILE DIAMETERS (ACTUAL AREA) FOR MINIMUM SPACING OF THREE DIAMETERS.
9) A RESISTANCE FACTOR OF 0.75 AND 1.00 SHOULD BE APPLIED TO THE PASSIVE EARTH PRESSURE FOR THE STRENGTH I AND EXTREME EVENT I LIMIT STATE, RESPECTIVELY.

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LATERAL EARTH PRESSURES

(TIEBACK SOLDIER PILE - WALLS 1, 2, AND 3)

NOV. 2017 JOB NO. W1212-1 FIG. 10

APPENDIX A

Subsurface Explorations and Laboratory Testing

APPENDIX A

SUBSURFACE EXPLORATIONS AND LABORATORY TESTING

SUBSURFACE EXPLORATIONS

Subsurface materials and conditions at the site were evaluated between August 21 and August 23, 2017, and between October 9 and 11, 2017, with 13 borings, designated B-1 through B-13, and four test pits, designated TP-1 through TP-4. The approximate locations of the explorations are shown on Figures 2 through 6. All field operations were observed by an experienced member of GRI's engineering staff.

Borings

The borings were advanced to depths of 11.5 to 41.5 ft below the ground surface using a truck-mounted drill rig and the hollow-stem auger drilling techniques. The borings were advanced by Holocene Drilling, Inc., of Puyallup, Washington, under subcontract to GRI. Disturbed soil samples were typically obtained from the borings at 2.5- to 5-ft intervals of depth using a standard split-spoon (SPT) sampler or California-modified sampler (CMS). The outside diameters of the SPT sampler and CMS used were 2 and 3 in., respectively. The CMS was used in order to collect more-representative samples of gravelly soils. Penetration tests were conducted by driving the sampler into the soil a distance of 18 in. using a 140-lb hammer dropped 30 in. The number of blows required to drive the SPT sampler the last 12 in. is known as the standard penetration resistance, or SPT N-value. The number of blows required to drive the CMS the last 12 in. is denoted the N*-value. The SPT N- and N*-values provide a measure of the relative density of granular soils, such as sand, and the relative consistency, or stiffness, of cohesive soils, such as silt. The SPT samples were carefully examined in the field and representative portions were saved in airtight jars or bags and returned to our laboratory for further classification and testing.

Relatively undisturbed samples of fine-grained, cohesive soils were obtained by pushing 3-in.-outside-diameter (O.D.) Shelby tubes into the undisturbed soil a maximum distance of 24 in. using the drill rig. The soils exposed in the ends of the Shelby tubes were examined and classified in the field. After classification, the tubes were sealed with rubber caps and tape to preserve the natural moisture content of the soils. All samples were returned to our laboratory for further examination and testing.

Logs of the borings are provided on Figures 1A through 13A. Each log presents a descriptive summary of the various types of materials encountered in the borings and notes the depths where the materials and/or characteristics of the materials change. To the right of the descriptive summary, the numbers and types of samples obtained during the drilling operation are indicated. Farther to the right, SPT N- or N*-values are shown graphically, along with the natural moisture content and percentage passing the No. 200 sieve. The terms and symbols used to describe the soils encountered in the borings are defined in Table 3A and the attached legend.

Small-Scale Pilot Infiltration Tests

Four small-scale pilot infiltration tests were conducted in test pits TP-1, TP-2, TP-3, and TP-4 at depths of 4, 4, 3, and 4 ft, respectively, between October 9 and October 11, 2017, at the locations shown on Figures 2 through 6. The tests were conducted in general conformance with Appendix 1-C of the 2015 Clark County Stormwater Manual. Logs of the test pits are provided on Figures 14A and 15A.

The small-scale, pilot infiltration tests were completed by excavating a test pit to the selected depth using a small, track-mounted excavator with a smooth-edged bucket. The excavator was supplied and operated by Scott Lee Excavating, Inc., of Battle Ground, Washington, under subcontract to GRI. The bottom area of the test pit was carefully measured and water was added to the excavation to maintain an approximate 12-in. head during a minimum 1.5-hr soaking period. During the soaking period, the water flow into the excavation was continuously adjusted in order to maintain the 12-in. head. After the soaking period, the water level was maintained in the pit for an additional 1 hr. After completion of the infiltration testing, the water flow rate into the pit was estimated by measuring the amount of time needed to fill a 5-gal. bucket. After testing, the drop in water level in the test pit was monitored for a period of 55 to 205 min. After completion of testing, the excavations were backfilled with excavated soil and compacted using the excavator bucket. Data from the small-scale infiltration tests are provided in Tables 1A and 2A.

Table 1A: SMALL-SCALE PILOT INFILTRATION TEST DATA

Location	Test Depth, ft	Soil at Depth of Test	Test Pit Dimensions, ft	Flow Rate, gal./min
TP-1	4	Sandy GRAVEL, some silt, contains cobbles	2 by 6	1.3
TP-2	4	Sandy GRAVEL, trace silt, contains cobbles	5.2 by 6.7	9.4
TP-3	3	Sandy GRAVEL, trace silt, contains cobbles	6 by 8	12.8
TP-4	4	Sandy GRAVEL, trace silt, contains cobbles	4.4 by 5.5	9.4

Table 2A: SMALL-SCALE PILOT INFILTRATION TEST FALLING HEAD DATA

TP-1		TP-2		TP-3		TP-4	
Time, min	Head, in.	Time, min	Head, in.	Time, min	Head, in.	Time, min	Head, in.
0	12	0	11	0	14.7	0	12
30	9	3	10	8	12	3	11
50	7.3	22	7	28	8	6	10
63	6.3	35	5.5	42	5.3	12	9
92	5	53	4	56	3	16	8
107	4	102	0	80	0	21	7
205	0	-	-	-	-	28	6
-	-	-	-	-	-	32	5
-	-	-	-	-	-	38	4
-	-	-	-	-	-	44	3
-	-	-	-	-	-	49	2
-	-	-	-	-	-	52	1
-	-	-	-	-	-	55	0

LABORATORY TESTING

General

All samples obtained from the borings were returned to our laboratory, where the physical characteristics of the samples were noted and the field classifications modified where necessary. The laboratory testing program included standard classification tests, natural moisture content determinations, and washed gradations. A summary of the laboratory test results is provided in Table 4A. The following paragraphs describe the laboratory testing program in more detail.

Natural Moisture Content

The natural moisture contents of selected soil samples were determined in conformance with ASTM D2216. The results are shown on Figures 1A through 13A and summarized in Table 4A.

Grain-Size Analysis (Washed Sieve)

Washed-sieve analyses were performed on representative samples of the soils to assist in their classification. The test is performed by taking a sample of known dry weight and washing it over a No. 200 sieve. The material retained on the No. 200 sieve is oven-dried and re-weighed, and the percentage of material (by weight) that passed the No. 200 sieve is calculated. The test results are provided on Figures 1A through 13A and summarized in Table 4A.

Table 3A: GUIDELINES FOR CLASSIFICATION OF SOIL

Description of Relative Density for Granular Soil

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

Description of Consistency for Fine-Grained (Cohesive) Soils

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

Grain-Size Classification

Boulders:
    >12 in.

Cobbles:
    3 - 12 in.

Gravel:
    1/4 - 3/4 in. (fine)
    3/4 - 3 in. (coarse)

Sand:
    No. 200 - No. 40 sieve (fine)
    No. 40 - No. 10 sieve (medium)
    No. 10 - No. 4 sieve (coarse)

Silt/Clay:
    pass No. 200 sieve

Modifier for Subclassification

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

Table 4A

SUMMARY OF LABORATORY RESULTS

Sample Information						Atterberg Limits			
Location	Sample	Depth, ft	Elevation, ft	Moisture Content, %	Dry Unit Weight, pcf	Liquid Limit, %	Plasticity Index, %	Fines Content, %	Soil Type
B-1	S-1	2.5	81.5	29	–	–	–	70	Sandy SILT
	S-2	5.0	79.0	6	–	–	–	–	Sandy GRAVEL
	S-8	15.0	69.0	30	–	–	–	–	Silty SAND
B-2	S-1	0.8	87.2	31	–	–	–	–	SILT
	S-2	2.5	85.5	23	–	–	–	46	Silty SAND
	S-3	5.0	83.0	23	–	–	–	25	Silty SAND
B-3	S-1	2.5	84.5	19	–	–	–	–	SILT
	S-2	5.0	82.0	16	–	–	–	–	Silty SAND
	S-6	15.0	72.0	25	–	–	–	15	Silty SAND
B-4	S-1	2.5	94.5	5	–	–	–	–	Gravelly SAND
	S-7	15.0	82.0	25	–	–	–	–	Silty SAND
B-5	S-1	0.8	99.2	8	–	–	–	–	FILL
	S-2	2.5	97.5	8	–	–	–	–	FILL
	S-3	5.0	95.0	23	–	–	–	–	Silty SAND
	S-4	7.5	92.5	24	–	–	–	–	Silty SAND
	S-8	20.0	80.0	37	–	–	–	–	Sandy SILT
B-7	S-3	5.0	132.5	11	–	–	–	–	FILL
B-8	S-1	0.8	141.2	9	–	–	–	–	FILL
	S-3	5.0	137.0	12	–	–	–	–	FILL
	S-6	12.5	129.5	12	–	–	–	–	FILL
	S-9	25.0	117.0	32	–	–	–	75	SILT
	S-10	30.0	112.0	30	–	–	–	–	SILT
	S-11	35.0	107.0	31	–	–	–	–	Clayey SILT
	S-12	40.0	102.0	33	–	–	–	–	Clayey SILT
B-9	S-3	5.0	141.0	11	–	–	–	–	FILL
	S-10	30.0	116.0	29	–	–	–	66	Sandy SILT
	S-11	35.0	111.0	31	–	–	–	–	Clayey SILT
	S-12	40.0	106.0	36	–	–	–	–	Clayey SILT
B-10	S-3	5.0	181.0	13	–	–	–	–	Gravelly SAND
B-11	S-3	5.0	202.5	14	–	–	–	–	Sandy GRAVEL
B-12	S-2	2.5	223.5	21	–	–	–	–	FILL
	S-3	5.0	221.0	26	–	–	–	–	FILL
	S-4	7.5	218.5	28	–	–	–	–	FILL
	S-5	10.0	216.0	31	–	–	–	–	FILL
	S-6	12.5	213.5	29	–	–	–	–	FILL
B-13	S-3	5.0	221.0	22	–	–	–	–	FILL
	S-6	12.0	214.0	42	–	–	–	–	FILL
	S-7	15.0	211.0	29	–	–	–	–	FILL

BORING AND TEST PIT LOG LEGEND

SOIL SYMBOLS

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

BEDROCK SYMBOLS

Symbol	Typical Description
⊞	BASALT
▤	MUDSTONE
▤	SILTSTONE
▤	SANDSTONE

SURFACE MATERIAL SYMBOLS

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

SAMPLER SYMBOLS

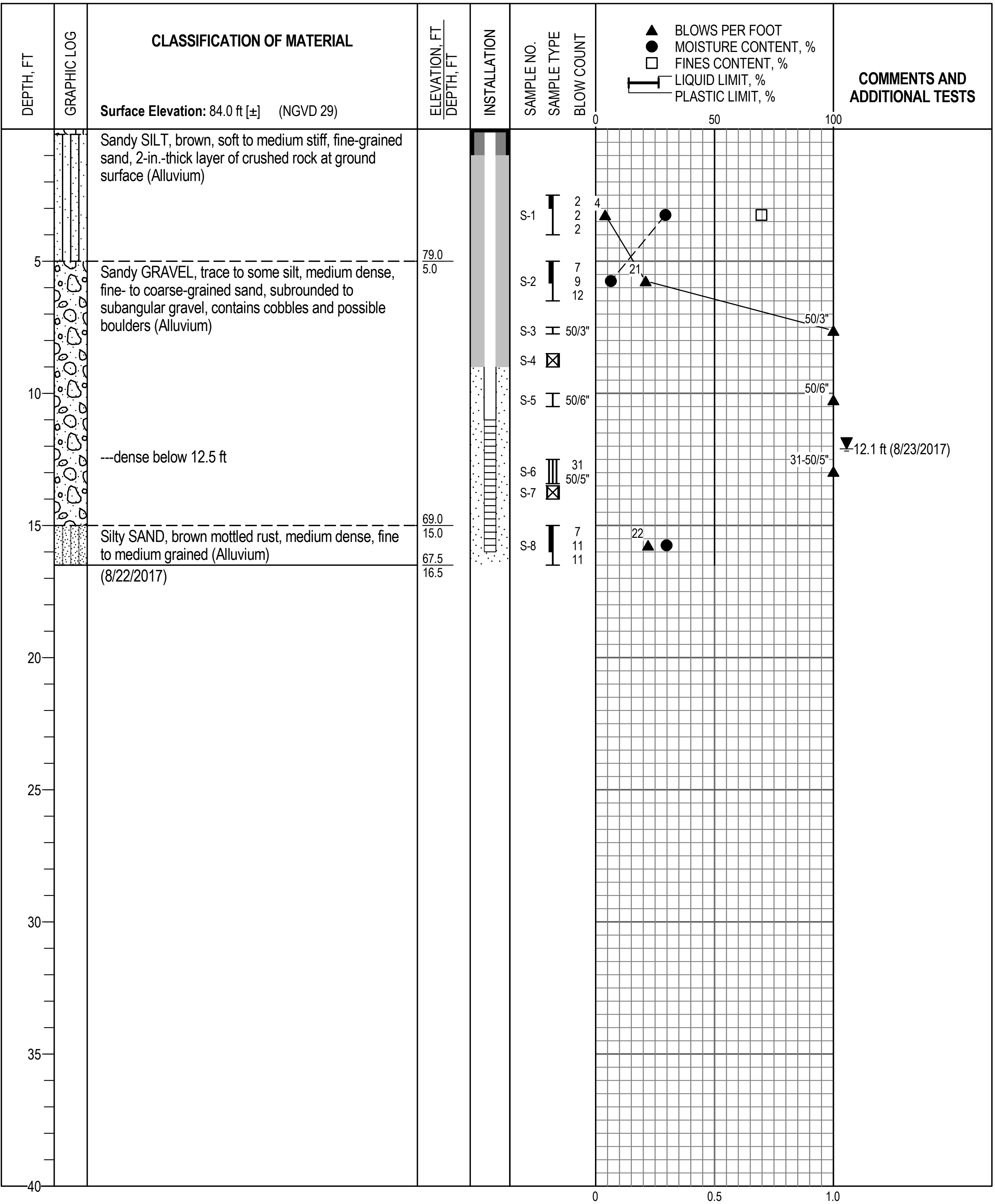
Symbol	Sampler Description
I	2.0-in. O.D. split-spoon sampler and Standard Penetration Test with recovery (ASTM D1586)
▮	Shelby tube sampler with recovery (ASTM D1587)
▮	3.0-in. O.D. split-spoon sampler with recovery (ASTM D3550)
⊠	Grab Sample
▯	Rock core sample interval
▮	Sonic core sample interval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

INSTALLATION SYMBOLS

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

FIELD MEASUREMENTS

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



CLASSIFICATION OF MATERIAL

Surface Elevation: 84.0 ft [±] (NGVD 29)

COMMENTS AND ADDITIONAL TESTS

Sandy SILT, brown, soft to medium stiff, fine-grained sand, 2-in.-thick layer of crushed rock at ground surface (Alluvium)

Sandy GRAVEL, trace to some silt, medium dense, fine- to coarse-grained sand, subrounded to subangular gravel, contains cobbles and possible boulders (Alluvium)

---dense below 12.5 ft

Silty SAND, brown mottled rust, medium dense, fine to medium grained (Alluvium)

(8/22/2017)

GRI BORING LOG (LAT/LONG) GRI DATA TEMPLATE.GDT 10/24/17

Logged By: T. Gayne
Drilled by: Holocene Drilling
Date Started: 8/22/17
Coordinates: Not Available
Drilling Method: Hollow-Stem Auger
Equipment: Foremost Mobile B-58
Hole Diameter: 5 in.
Note: See Legend for Explanation of Symbols
Hammer Type: Auto Hammer
Weight: 140 lb
Drop: 30 in.
Energy Ratio:

GRI

BORING B-1

NOV. 2017 JOB NO. W1212-T01 FIG. 1A

CLASSIFICATION OF MATERIAL

Surface Elevation: 88.0 ft [±] (NGVD 29)

DEPTH, FT: 5, 10, 15, 20, 25, 30, 35, 40

CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT
Asphalt concrte PAVEMENT (10 in.)	87.2 0.8
SILT, some clay, trace to some fine-grained sand, brown, stiff (Alluvium)	85.5 2.5
Silty SAND, brown, very loose, fine grained (Alluvium)	81.0 7.0
Sandy GRAVEL, dense, fine- to coarse-grained sand, subangular to subrounded gravel, contains cobbles and possible boulders (Alluvium)	76.5 11.5

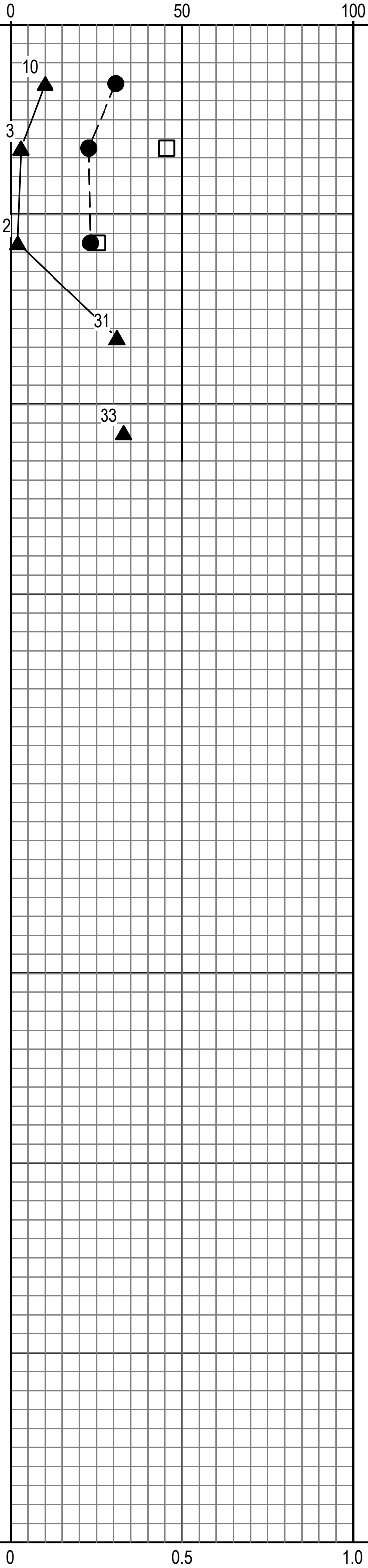
(8/21/2017)

Groundwater not encountered

SAMPLE NO.	SAMPLE TYPE	BLOW COUNT
S-1		19 5 5
S-2		2 1 2
S-3		1 1 1
S-4		9 15 16
S-5	☒	
S-6		4 17 16

INSTALLATION

▲ BLOWS PER FOOT
● MOISTURE CONTENT, %
☐ FINES CONTENT, %
LIQUID LIMIT, %
PLASTIC LIMIT, %



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

COMMENTS AND ADDITIONAL TESTS

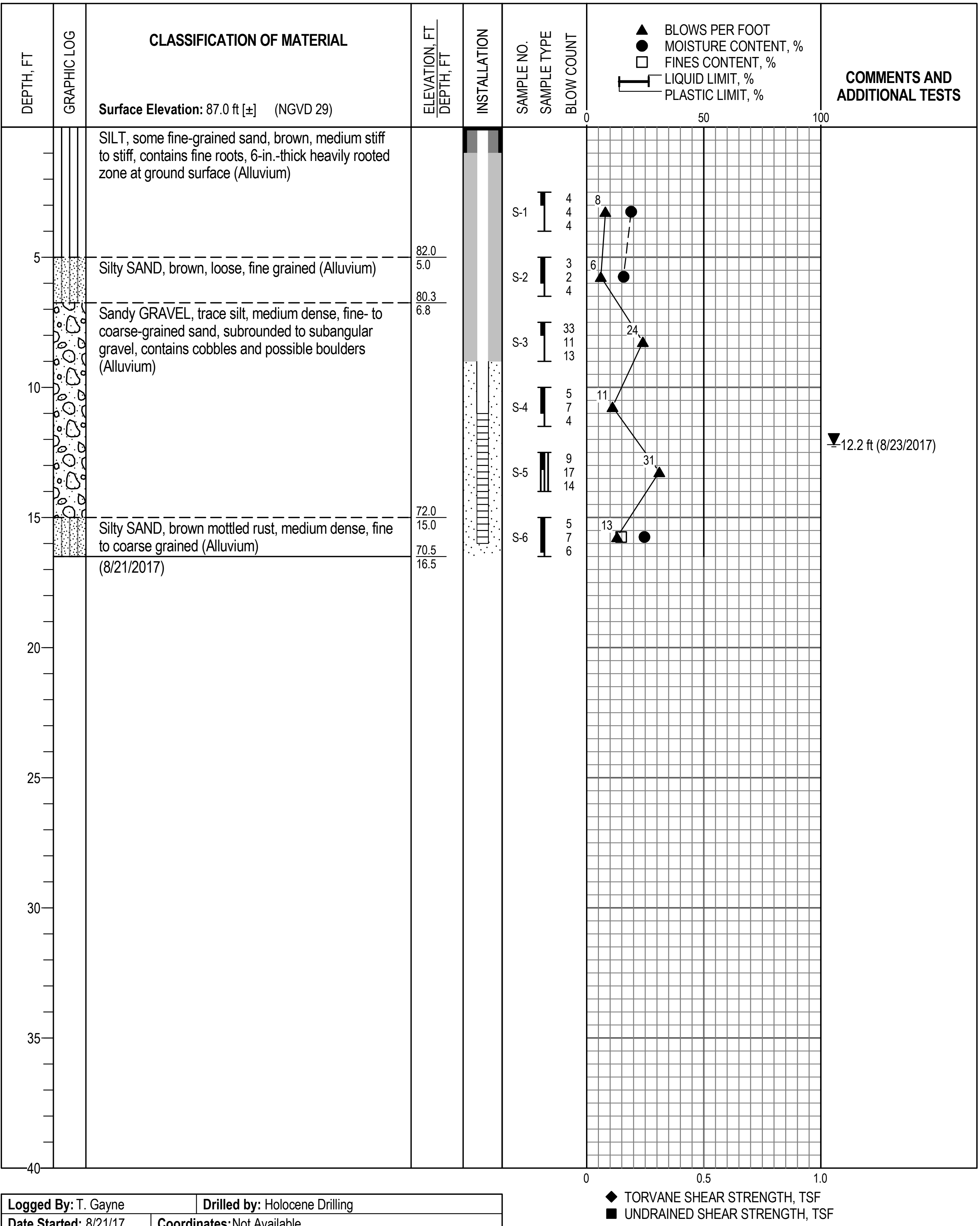
Logged By: T. Gayne
Drilled by: Holocene Drilling
Date Started: 8/21/17
Coordinates: Not Available
Drilling Method: Hollow-Stem Auger
Equipment: Foremost Mobile B-58
Hole Diameter: 5 in.
Note: See Legend for Explanation of Symbols
Hammer Type: Auto Hammer
Weight: 140 lb
Drop: 30 in.
Energy Ratio:

GRI

BORING B-2

NOV. 2017 JOB NO. W1212-T01 FIG. 2A

DEPTH, FT	CLASSIFICATION OF MATERIAL Surface Elevation: 87.0 ft [±] (NGVD 29)	ELEVATION, FT DEPTH, FT	SAMPLE NO.	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
0	SILT, some fine-grained sand, brown, medium stiff to stiff, contains fine roots, 6-in.-thick heavily rooted zone at ground surface (Alluvium)		S-1	4 4 4	
5	Silty SAND, brown, loose, fine grained (Alluvium)	82.0 5.0	S-2	3 2 4	
	Sandy GRAVEL, trace silt, medium dense, fine- to coarse-grained sand, subrounded to subangular gravel, contains cobbles and possible boulders (Alluvium)	80.3 6.8	S-3	33 11 13	
10			S-4	5 7 4	
					▼12.2 ft (8/23/2017)
			S-5	9 17 14	
15	Silty SAND, brown mottled rust, medium dense, fine to coarse grained (Alluvium)	72.0 15.0	S-6	5 7 6	
	(8/21/2017)	70.5 16.5			



Logged By: T. Gayne	**Drilled by:** Holocene Drilling	
Date Started: 8/21/17	**Coordinates:** Not Available	
Drilling Method: Hollow-Stem Auger **Equipment:** Foremost Mobile B-58 **Hole Diameter:** 5 in.		**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:**
Note: See Legend for Explanation of Symbols		

GRI

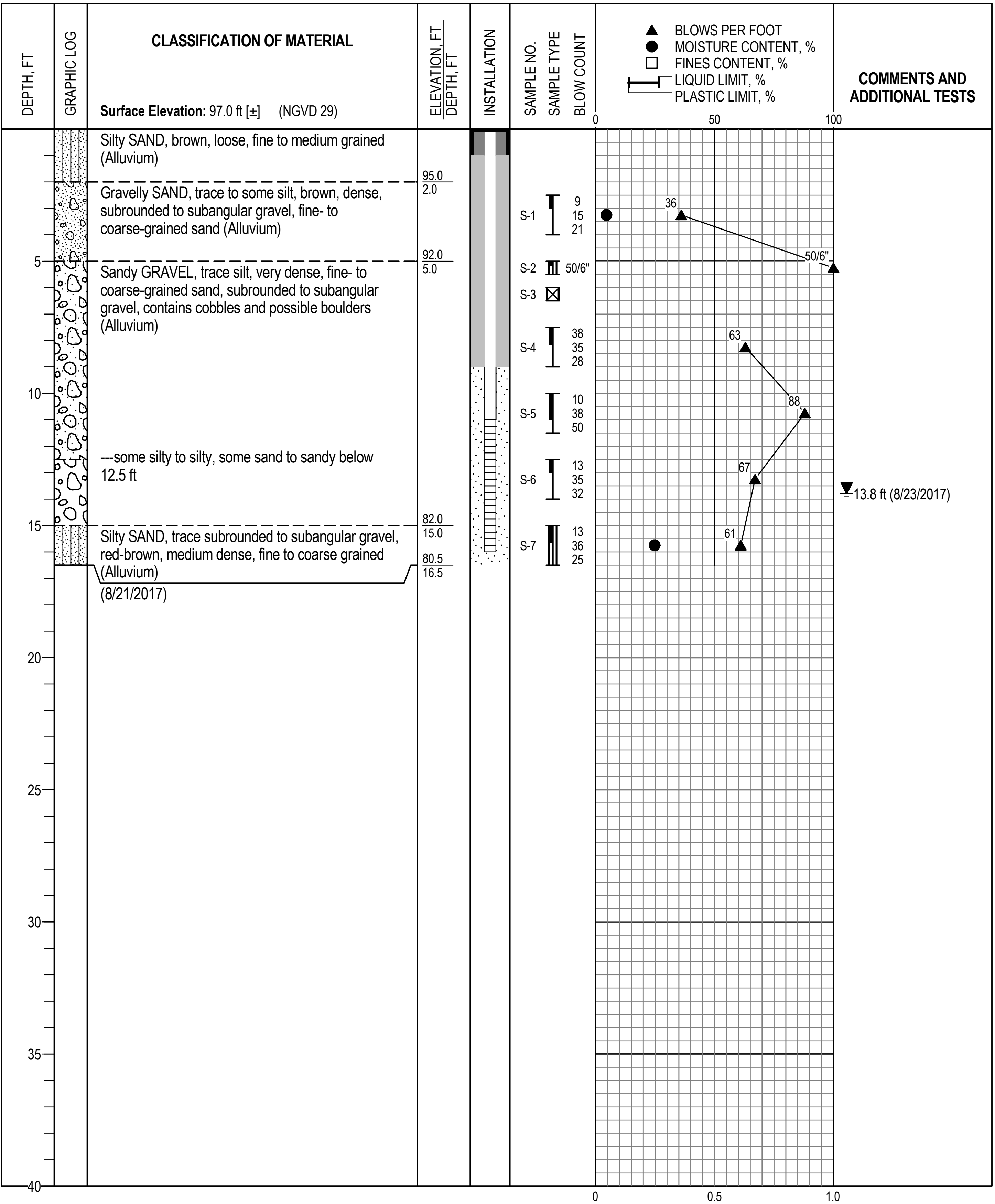
BORING B-3

NOV. 2017 JOB NO. W1212-T01 FIG. 3A

CLASSIFICATION OF MATERIAL

Surface Elevation: 97.0 ft [±] (NGVD 29)

COMMENTS AND ADDITIONAL TESTS



Silty SAND, brown, loose, fine to medium grained (Alluvium)

Gravelly SAND, trace to some silt, brown, dense, subrounded to subangular gravel, fine- to coarse-grained sand (Alluvium)

Sandy GRAVEL, trace silt, very dense, fine- to coarse-grained sand, subrounded to subangular gravel, contains cobbles and possible boulders (Alluvium)

---some silty to silty, some sand to sandy below 12.5 ft

Silty SAND, trace subrounded to subangular gravel, red-brown, medium dense, fine to coarse grained (Alluvium)

(8/21/2017)

GRI BORING LOG (LAT/LONG) GRI DATA TEMPLATE.GDT 10/24/17

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

Logged By: T. Gayne
Drilled by: Holocene Drilling
Date Started: 8/21/17
Coordinates: Not Available
Drilling Method: Hollow-Stem Auger
Equipment: Foremost Mobile B-58
Hole Diameter: 5 in.
Hammer Type: Auto Hammer
Weight: 140 lb
Drop: 30 in.
Energy Ratio:
Note: See Legend for Explanation of Symbols

GRI

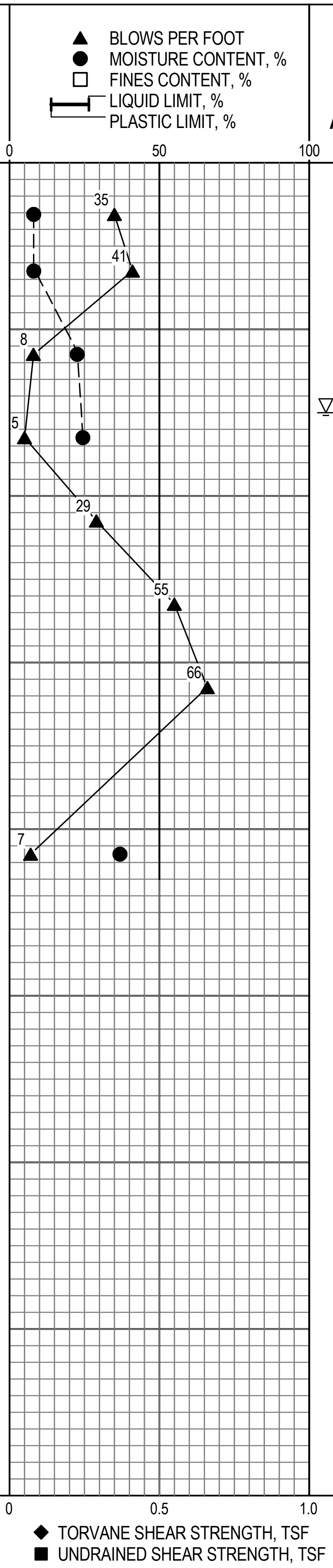
BORING B-4

NOV. 2017 JOB NO. W1212-T01 FIG. 4A

Surface Elevation: 100.0 ft [±] (NGVD 29)

DEPTH, FT	CLASSIFICATION OF MATERIAL	ELEVATION, FT DEPTH, FT
	Asphalt concrete PAVEMENT (10 in.)	99.2 0.8
	Gravelly SAND to sandy GRAVEL, brown, dense, subangular to subrounded gravel, fine- to coarse-grained sand (Fill)	95.0 5.0
5	Silty SAND, trace to some subrounded to subangular gravel, brown, loose, fine to coarse grained (Alluvium)	90.0 10.0
10	GRAVEL, some fine- to coarse-grained sand to sandy, some silt to silty, medium dense, subrounded to subangular, contains cobbles and possible boulders (Alluvium) ---sandy, trace to some silt, very dense below 12.5 ft	81.0 19.0
20	Sandy SILT, trace to some subrounded to subangular gravel, brown, medium stiff, fine- to coarse-grained sand (Alluvium)	78.5 21.5
	(8/21/2017)	

SAMPLE NO.	BLOW COUNT
S-1	18 19 16
S-2	19 16 25
S-3	2 5 3
S-4	2 1 4
S-5	12 14 15
S-6	15 19 36
S-7	15 32 34
S-8	1 1 6



COMMENTS AND ADDITIONAL TESTS

▽7.5 ft (8/21/2017)

Logged By: T. Gayne
Drilled by: Holocene Drilling
Date Started: 8/21/17
Coordinates: Not Available
Drilling Method: Hollow-Stem Auger
Equipment: Foremost Mobile B-58
Hole Diameter: 5 in.
Note: See Legend for Explanation of Symbols
Hammer Type: Auto Hammer
Weight: 140 lb
Drop: 30 in.
Energy Ratio:

GRI

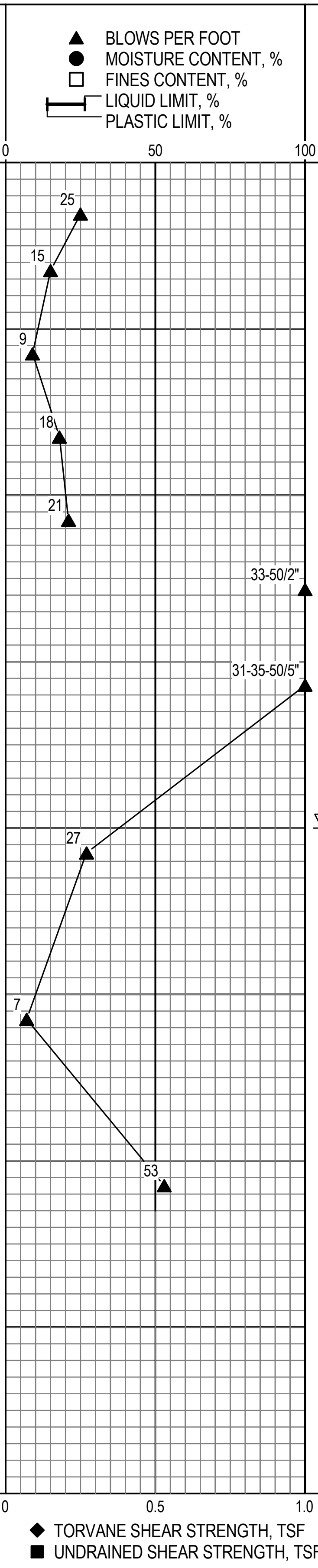
BORING B-5

CLASSIFICATION OF MATERIAL

Surface Elevation: 124.0 ft [±] (NGVD 29)

DEPTH, FT	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT
	Asphalt concrete PAVEMENT (10 in.)	123.2 / 0.8
	Sandy GRAVEL, trace silt, medium dense, fine- to coarse-grained sand, subangular to subrounded gravel (Fill)	
5	---loose, contains organics below 5 ft	
	---medium dense below 7.5 ft	
10		111.5 / 12.5
15	Sandy GRAVEL, trace silt, very dense, fine- to coarse-grained sand, subangular to subrounded gravel, contains cobbles and possible boulders (Alluvium)	
20	---medium dense below 20 ft	
25	---some sand to sandy, some silt to silty, loose below 25 ft	
30	Sandy GRAVEL, trace silt, very dense, fine- to coarse-grained sand, subangular to subrounded gravel, contains cobbles and possible boulders (Troutdale Formation)	94.0 / 30.0
	(8/22/2017)	92.5 / 31.5
35		
40		

SAMPLE NO.	BLOW COUNT
S-1	13 13 12
S-2	6 6 9
S-3	4 5 4
S-4	5 11 7
S-5	5 8 13
S-6	
S-7	33 50/2"
S-8	31 35 50/5"
S-9	8 8 19
S-10	3 3 4
S-11	38 25 28



COMMENTS AND ADDITIONAL TESTS

∇20.0 ft (8/22/2017)

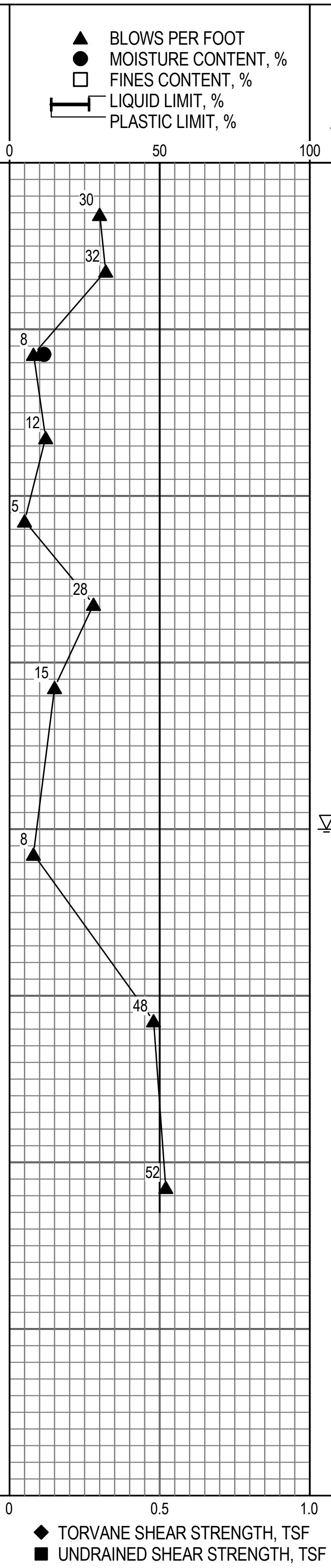
Logged By: T. Gayne | **Drilled by:** Holocene Drilling
Date Started: 8/22/17 | **Coordinates:** Not Available
Drilling Method: Hollow-Stem Auger
Equipment: Foremost Mobile B-58
Hole Diameter: 5 in.
Note: See Legend for Explanation of Symbols
Hammer Type: Auto Hammer
Weight: 140 lb
Drop: 30 in.
Energy Ratio:

GRI BORING LOG (LAT/LONG) GRI DATA TEMPLATE.GDT 10/24/17

GRI

BORING B-6

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	ELEVATION, FT DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
		Surface Elevation: 137.5 ft [±] (NGVD 29)						
0		Asphalt concrete PAVEMENT (10 in.)	136.7 0.8					
		Gravelly SAND to sandy GRAVEL, some silt, brown, medium dense to dense, subangular to subrounded gravel, fine- to coarse-grained sand, contains cobbles and possible boulders (Fill)			S-1		12 17 13	
					S-2		8 22 10	
5		---loose at 5 ft			S-3		4 4 4	
					S-4		4 5 7	
10		---loose at 10 ft			S-5		3 3 2	
					S-6		10 14 14	
15					S-7		9 7 8	
20		---loose at 20 ft			S-8		6 4 4	∇20.0 ft (8/22/2017)
			116.0 21.5					
		Sandy GRAVEL, trace silt, dense to very dense, fine- to coarse-grained sand, subangular to subrounded gravel, contains cobbles and possible boulders (Troutdale Formation)						
25					S-9		13 14 34	
30					S-10		14 26 26	
			106.0 31.5					
		(8/22/2017)						
35								
40								



Logged By: T. Gayne	**Drilled by:** Holocene Drilling
Date Started: 8/22/17	**Coordinates:** Not Available
Drilling Method: Hollow-Stem Auger **Equipment:** Foremost Mobile B-58 **Hole Diameter:** 5 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:**
Note: See Legend for Explanation of Symbols	

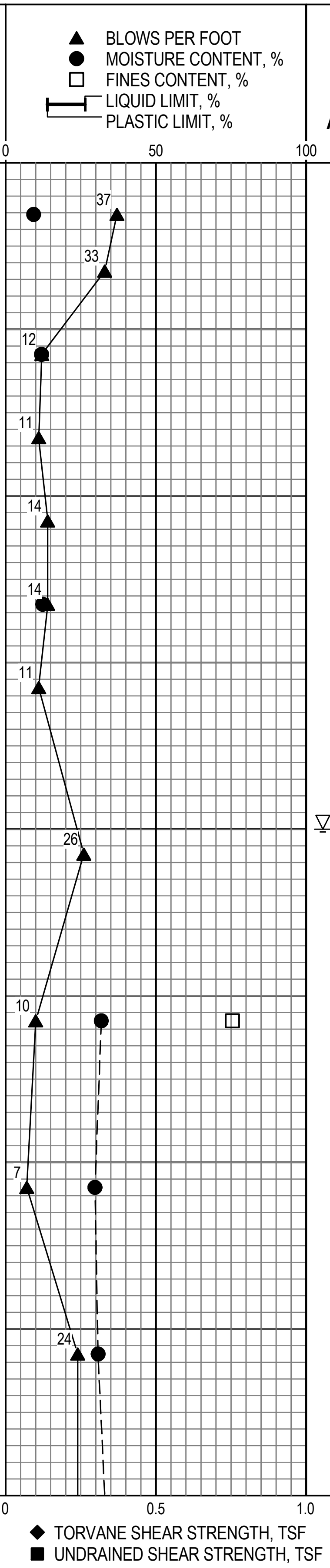
GRI

BORING B-7

NOV. 2017 JOB NO. W1212-T01 FIG. 7A

Surface Elevation: 142.0 ft [±] (NGVD 29)

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	ELEVATION, FT DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
		Asphalt concrete PAVEMENT (10 in.)	141.2 0.8					
		Gravelly SAND to sandy GRAVEL, trace to some silt, brown, dense, subangular to subrounded gravel, fine- to coarse-grained sand, contains cobbles and possible boulders (Fill)			S-1		12 13 24	
					S-2		32 16 17	
5		---medium dense below 5 ft			S-3		4 6 6	
					S-4		4 5 6	
10					S-5		8 9 5	
					S-6		5 5 9	
15					S-7		5 7 4	
		Sandy GRAVEL, some silt, medium dense, fine- to coarse-grained sand, subangular to subrounded gravel, contains cobbles and possible boulders (Alluvium)	124.0 18.0					
20					S-8		9 10 16	∇20.0 ft (8/21/2017)
		SILT, some fine-grained sand, gray, stiff (Troutdale Formation)	121.0 21.0					
25					S-9		3 6 4	
30		---trace sand, medium stiff below 30 ft			S-10		4 3 4	
35		---clayey, very stiff below 35 ft			S-11		5 10 14	
40								



(CONTINUED NEXT PAGE)

Logged By: T. Gayne **Drilled by:** Holocene Drilling
Date Started: 8/21/17 **Coordinates:** Not Available
Drilling Method: Hollow-Stem Auger
Equipment: Foremost Mobile B-58
Hole Diameter: 5 in.
Note: See Legend for Explanation of Symbols
Hammer Type: Auto Hammer
Weight: 140 lb
Drop: 30 in.
Energy Ratio:

GRI

BORING B-8

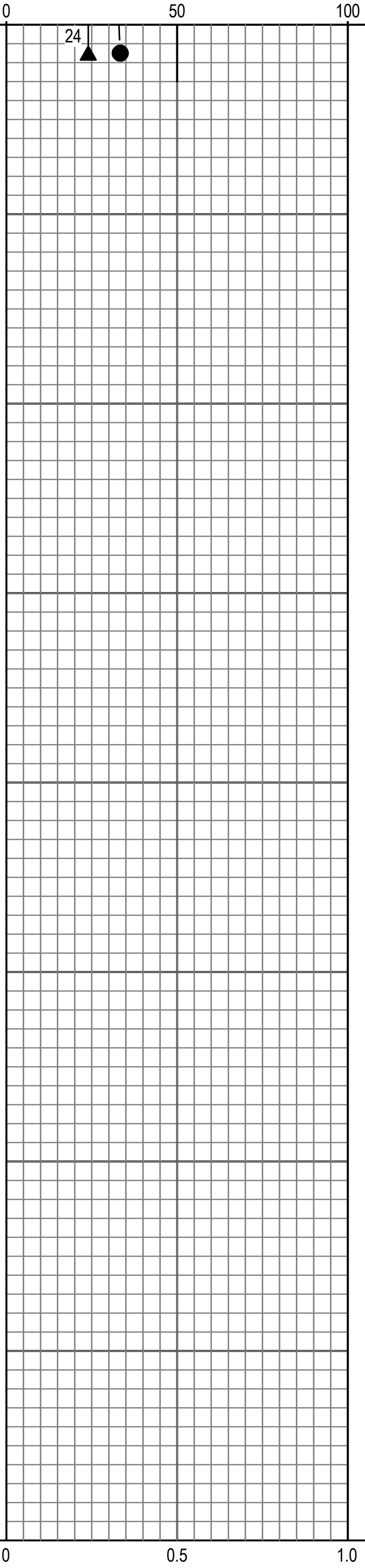
NOV. 2017 JOB NO. W1212-T01 FIG. 8A

CLASSIFICATION OF MATERIAL

Surface Elevation: 142.0 ft [±] (NGVD 29)

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
	▨	Clayey SILT, trace fine-grained sand, gray, very stiff (Troutdale Formation)	100.5 41.5		S-12	▮	6 9 15	
		(8/21/2017)						
45								
50								
55								
60								
65								
70								
75								
80								

▲ BLOWS PER FOOT
● MOISTURE CONTENT, %
☐ FINES CONTENT, %
LIQUID LIMIT, %
PLASTIC LIMIT, %



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

GRI BORING LOG (LAT/LONG) GRI DATA TEMPLATE.GDT 10/24/17

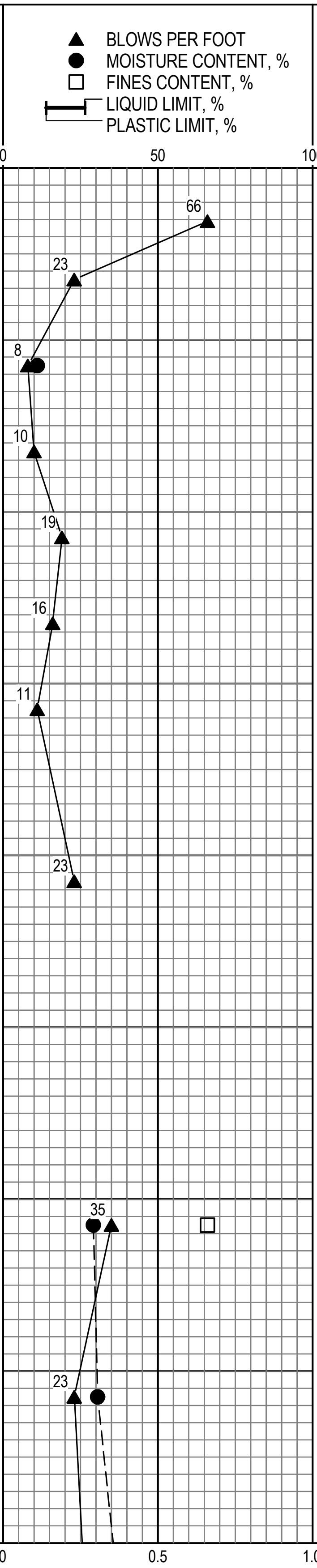
GRI

BORING B-8

NOV. 2017 JOB NO. W1212-T01 FIG. 8A

Surface Elevation: 146.0 ft [±] (NGVD 29)

DEPTH, FT	CLASSIFICATION OF MATERIAL	ELEVATION, FT DEPTH, FT	SAMPLE NO.	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
	Asphalt concrete PAVEMENT (10 in.)	145.2 0.8			
	Gravelly SAND to sandy GRAVEL, trace to some silt, brown, very dense, subangular to subrounded gravel, fine- to coarse-grained sand, contains cobbles and possible boulders (Fill) ---loose to medium dense below 2.5 ft		S-1	22 23 43	
			S-2	12 13 10	
5			S-3	5 4 4	
			S-4	4 5 5	
10			S-5	6 10 9	
			S-6	6 8 8	
15			S-7	5 5 6	
20	Sandy GRAVEL, some silt, medium dense, fine- to coarse-grained sand, subangular to subrounded gravel, contains cobbles and possible boulders (Alluvium)	126.0 20.0	S-8	10 11 12	∇20.0 ft (8/22/2017)
25	Sandy SILT, gray, stiff to very stiff, fine-grained sand (Troutdale Formation)	121.0 25.0	S-9		
30	---hard below 30 ft		S-10	10 12 23	
35	---clayey, trace sand, very stiff below 35 ft		S-11	6 10 13	
40					



(CONTINUED NEXT PAGE)

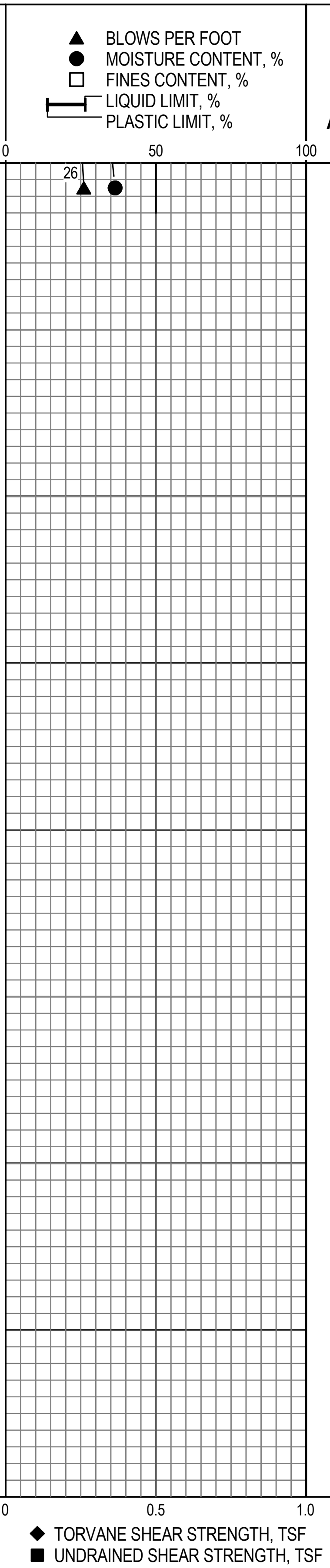
Logged By: T. Gayne	**Drilled by:** Holocene Drilling
Date Started: 8/22/17	**Coordinates:** Not Available
Drilling Method: Hollow-Stem Auger **Equipment:** Foremost Mobile B-58 **Hole Diameter:** 5 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:**
Note: See Legend for Explanation of Symbols	

GRI

BORING B-9

NOV. 2017 JOB NO. W1212-T01 FIG. 9A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL **Surface Elevation:** 146.0 ft [±] (NGVD 29)	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
	▨	Clayey SILT, trace fine-grained sand, gray, very stiff (Troutdale Formation)	104.5 41.5		S-12	▮	8 12 14	
		(8/22/2017)						
45								
50								
55								
60								
65								
70								
75								
80								



GRI BORING LOG (LAT/LONG) GRI DATA TEMPLATE.GDT 10/24/17

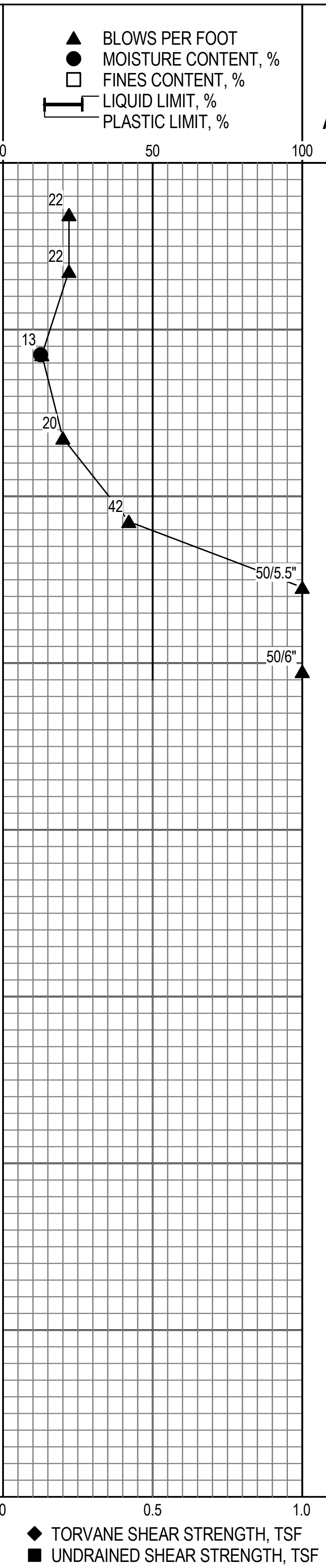
GRI

BORING B-9

NOV. 2017 JOB NO. W1212-T01 FIG. 9A

Surface Elevation: 186.0 ft [±] (NGVD 29)

DEPTH, FT	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
	Asphalt concrete PAVEMENT (10 in.)						
	Gravelly SAND to sandy GRAVEL, trace to some silt, brown, medium dense, subangular to subrounded gravel, fine- to coarse-grained sand, contains cobbles and possible boulders (Troutdale Formation)	185.2 0.8		S-1		12 12 10	
				S-2		15 13 9	
5				S-3		7 6 7	
				S-4		4 10 10	
10	---dense below 10 ft			S-5		6 18 24	
	---very dense below 12.5 ft			S-6		50/5.5"	
15				S-7		50/6"	
	(8/22/2017) Groundwater not encountered	170.5 15.5					
20							
25							
30							
35							
40							



Logged By: T. Gayne	**Drilled by:** Holocene Drilling
Date Started: 8/22/17	**Coordinates:** Not Available
Drilling Method: Hollow-Stem Auger **Equipment:** Foremost Mobile B-58 **Hole Diameter:** 5 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:**
Note: See Legend for Explanation of Symbols	

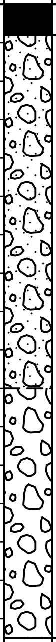
GRI

BORING B-10

NOV. 2017 JOB NO. W1212-T01 FIG. 10A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS

Surface Elevation: 207.5 ft [±] (NGVD 29)



Asphalt concrete PAVEMENT (10 in.)

206.7
0.8

Gravelly SAND to sandy GRAVEL, trace to some silt, brown, medium dense to very dense, subangular to subrounded gravel, fine- to coarse-grained sand, contains cobbles and possible boulders (Troutdale Formation)

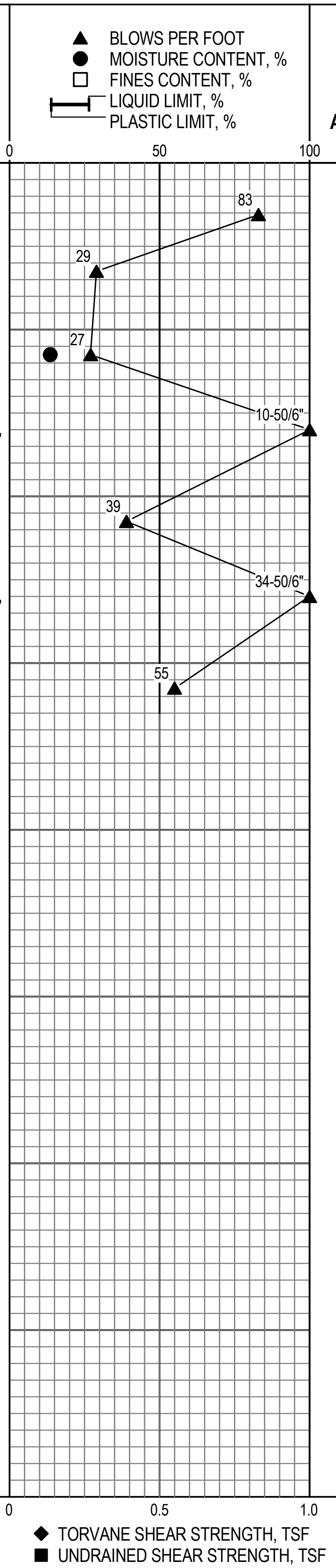
---some sand to sandy, some silt to silty, dense to very dense below 10 ft

191.0
16.5

(8/22/2017)

Groundwater not encountered

S-1	21 40 43	83
S-2	7 14 15	29
S-3	25 20 7	27
S-4	10 50/6"	10-50/6"
S-5	7 17 22	39
S-6	34 50/6"	34-50/6"
S-7	20 28 27	55



Depth, ft: 5, 10, 15, 20, 25, 30, 35, 40

Logged By: T. Gayne	**Drilled by:** Holocene Drilling
Date Started: 8/22/17	**Coordinates:** Not Available
Drilling Method: Hollow-Stem Auger **Equipment:** Foremost Mobile B-58 **Hole Diameter:** 5 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:**
Note: See Legend for Explanation of Symbols	

GRI

BORING B-11

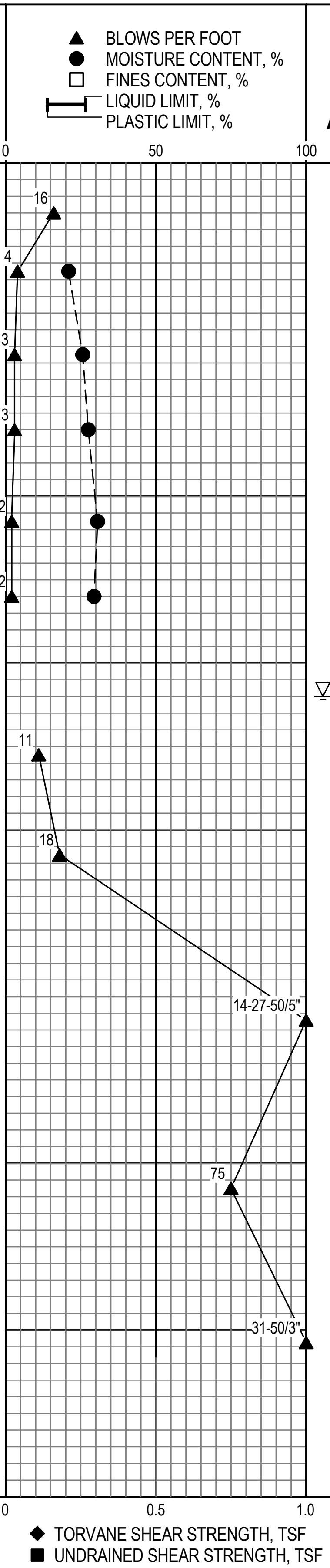
NOV. 2017 JOB NO. W1212-T01 FIG. 11A

CLASSIFICATION OF MATERIAL

Surface Elevation: 226.0 ft [±] (NGVD 29)

DEPTH, FT	CLASSIFICATION OF MATERIAL	ELEVATION, FT DEPTH, FT
	Asphalt concrete PAVEMENT (9 in.)	225.3 0.7
	Gravelly SAND, trace silt, brown, medium dense, subrounded to subangular gravel, fine- to coarse-grained sand (Fill)	223.5 2.5
	Silty SAND, brown, very loose, fine grained, contains fine roots (Fill)	221.0 5.0
5	Sandy SILT, brown, soft, fine-grained sand (Fill)	
	---trace to some clay, trace sand below 7.5 ft	
10	---very soft to soft below 10 ft	
	---trace subrounded to subangular gravel, gray, contains wood debris below 12.5 ft	
15		209.0 17.0
	Sandy GRAVEL, some silt, medium dense, fine- to coarse-grained sand, subangular to subrounded gravel, contains cobbles and possible boulders (Troutdale Formation)	
20		
25	---very dense below 25 ft	
30		
35		190.2 35.8
	(8/23/2017)	
40		

SAMPLE NO.	BLOW COUNT
S-1	7 11 5
S-2	3 2 2
S-3	2 1 2
S-4	2 1 2
S-5	1 1 1
S-6	1 1 1
S-7	
S-8	2 4 7
S-9	6 11 7
S-10	14 27 50/5"
S-11	9 30 45
S-12	31 50/3"



COMMENTS AND ADDITIONAL TESTS

∇16.0 ft (8/23/2017)

Logged By: T. Gayne	**Drilled by:** Holocene Drilling
Date Started: 8/23/17	**Coordinates:** Not Available
Drilling Method: Hollow-Stem Auger **Equipment:** Foremost Mobile B-58 **Hole Diameter:** 5 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:**
Note: See Legend for Explanation of Symbols	

GRI

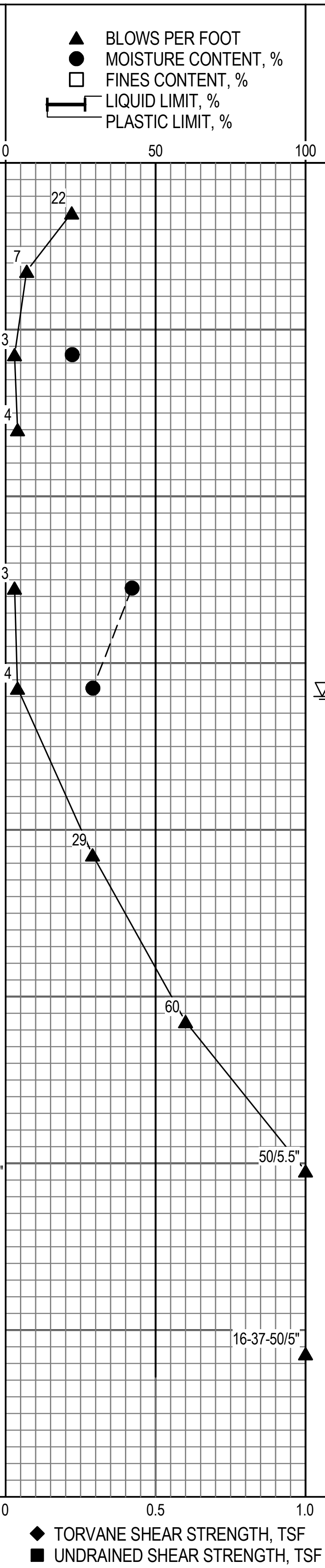
BORING B-12

NOV. 2017 JOB NO. W1212-T01 FIG. 12A

CLASSIFICATION OF MATERIAL

Surface Elevation: 226.0 ft [±] (NGVD 29)

DEPTH, FT	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
	Asphalt concrete PAVEMENT (9 in.)	225.3 / 0.7				
	Gravelly SAND, trace silt, brown, medium dense, subrounded to subangular gravel, fine- to coarse-grained sand (Fill)			S-1	14 14 8	
		222.5 / 3.5		S-2	6 4 3	
	SILT, trace to some fine-grained sand, brown, medium stiff (Fill)					
5	---soft below 5 ft			S-3	1 2 1	
	---sandy, soft to medium stiff below 7.5 ft			S-4	1 2 2	
10	---trace sand, brown and gray, soft, contains wood debris below 10 ft			S-5		
				S-6	1 1 2	
15	---soft to medium stiff, wood debris absent below 15 ft			S-7	1 1 3	∇16.0 ft (8/23/2017)
	GRAVEL, some fine- to coarse-grained sand to sandy, some silt to silty, dense, subangular to subrounded, contains cobbles and possible boulders (Troutdale Formation)	209.0 / 17.0				
20				S-8	14 16 13	
25	---sandy, trace silt, very dense below 25 ft			S-9	14 28 32	
30				S-10	50/5.5"	
35				S-11	16 37 50/5"	
	(8/23/2017)	189.6 / 36.4				
40						



Logged By: T. Gayne	**Drilled by:** Holocene Drilling
Date Started: 8/23/17	**Coordinates:** Not Available
Drilling Method: Hollow-Stem Auger **Equipment:** Foremost Mobile B-58 **Hole Diameter:** 5 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:**
Note: See Legend for Explanation of Symbols	

BORING B-13

NOV. 2017 JOB NO. W1212-T01 FIG. 13A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	SAMPLE NO. SAMPLE TYPE	● MOISTURE CONTENT, % ☐ FINES CONTENT, % ├─┤ LIQUID LIMIT, % PLASTIC LIMIT, % 0 50 100	COMMENTS AND ADDITIONAL TESTS

TP-1 **Surface Elevation:** Not Available

0 50 100

DEPTH, FT	CLASSIFICATION OF MATERIAL	DEPTH, FT	COMMENTS AND ADDITIONAL TESTS
	Sandy SILT, brown, soft to medium stiff, fine-grained sand, 6-in.-thick heavily rooted zone at ground surface		
	Sandy GRAVEL, some silt, medium dense, fine- to coarse-grained sand, subrounded gravel, contains cobbles	3.0	
	(10/11/2017)	4.0	
5	Infiltration testing performed at 4 ft		
	Groundwater not encountered		

0 0.5 1.0

◆ TORVANE SHEAR STRENGTH, TSF

Logged By: S. Reddy **Excavated by:** Scott Lee Excavating, Inc. **Equipment:** Track-Mounted Excavator

Date Started: 10/11/17 **Coordinates:** Not Available **Note:** See Legend for Explanation of Symbols

TP-2 **Surface Elevation:** Not Available

0 50 100

DEPTH, FT	CLASSIFICATION OF MATERIAL	DEPTH, FT	COMMENTS AND ADDITIONAL TESTS
	Sandy SILT, brown, soft to medium stiff, fine-grained sand, 6-in.-thick heavily rooted zone at ground surface		
	Sandy GRAVEL, trace silt, medium dense, fine- to coarse-grained sand, subrounded gravel, contains cobbles	2.0	Decreasing cobble content with depth
	(10/10/2017)	4.0	
5	Infiltration testing performed at 4 ft		
	Groundwater not encountered		

0 0.5 1.0

◆ TORVANE SHEAR STRENGTH, TSF

Logged By: S. Reddy **Excavated by:** Scott Lee Excavating, Inc. **Equipment:** Track-Mounted Excavator

Date Started: 10/10/17 **Coordinates:** Not Available **Note:** See Legend for Explanation of Symbols

GRI

TEST PITS

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	SAMPLE NO. SAMPLE TYPE	● MOISTURE CONTENT, % ☐ FINES CONTENT, % LIQUID LIMIT, % PLASTIC LIMIT, % 0 50 100	COMMENTS AND ADDITIONAL TESTS

TP-3 **Surface Elevation:** Not Available

0 50 100

Silty SAND, brown, loose to medium dense, fine grained, 9-in.-thick heavily rooted zone at ground surface

2.0

Sandy GRAVEL, trace silt, medium dense, fine- to coarse-grained sand, subrounded gravel, contains cobbles

Decreasing cobble content with depth

3.0

(10/9/2017)

Infiltration testing performed at 3 ft

Groundwater not encountered

5

0 0.5 1.0

◆ TORVANE SHEAR STRENGTH, TSF

Logged By: B. Bennetts **Excavated by:** Scott Lee Excavating, Inc. **Equipment:** Track-Mounted Excavator

Date Started: 10/9/17 **Coordinates:** Not Available **Note:** See Legend for Explanation of Symbols

TP-4 **Surface Elevation:** Not Available

0 50 100

Silty SAND, brown, loose to medium dense, fine grained, 6-in.-thick heavily rooted zone at ground surface

2.5

Sandy GRAVEL, trace silt, medium dense, fine- to coarse-grained sand, subrounded gravel, contains cobbles

4.0

(10/9/2017)

Infiltration testing performed at 4 ft

Groundwater not encountered

5

0 0.5 1.0

◆ TORVANE SHEAR STRENGTH, TSF

Logged By: B. Bennetts **Excavated by:** Scott Lee Excavating, Inc. **Equipment:** Track-Mounted Excavator

Date Started: 10/9/17 **Coordinates:** Not Available **Note:** See Legend for Explanation of Symbols

GRI

TEST PITS

NOV. 2017 JOB NO. W1212-T01 FIG. 15A