GEOTECHNICAL AND BRIDGE FOUNDATION REPORT

Walker/Murray Improvements
SW Walker Road and SW Murray Boulevard Intersection
Washington County, Oregon

For
Washington County
Department of Land Use and Transportation
c/o WH Pacific, Inc.
June 23, 2020

GeoDesign Project: WashCo-56-03

June 23, 2020

Washington County
Department of Land Use and Transportation
c/o WH Pacific, Inc.
9755 Barnes Road, Suite 300
Portland, OR 97225

Attention: Wayne Bower, P.E.

Geotechnical and Bridge Foundation Report
Walker/Murray Improvements
SW Walker Road and SW Murray Boulevard Intersection
Washington County, Oregon
GeoDesign Project: WashCo-56-03

GeoDesign, Inc. is pleased to submit our geotechnical and bridge foundation report for the proposed the Walker/Murray Improvements in Washington County, Oregon. This report presents a description of our services and site conditions; a summary of our field explorations and laboratory testing; and our recommendations for design and construction of bridge foundations, embankment fills, and retaining structures. Our recommendations were developed in general accordance with OSSC and standards presented in AASHTO LRFD (2017).

We appreciate the opportunity to be of continued service to you. Please call if you have questions concerning this report or if we can provide additional services.

Sincerely,

GeoDesign, Inc.

Nick Paveglio, P.E.
Senior Associate Engineer

Jeffery D. Tucker, P.E., G.E.
Principal Engineer

cc: Andy Bowen – WH Pacific (via email only)

NNP:JDT:kt:sn
Attachments
One copy submitted (via email only)
Document ID: WashCo-56-03-062320-geor-rev.docx

EXECUTIVE SUMMARY

Based on information provided by the design team and the results of our explorations, laboratory testing, and analyses, it is our opinion that the project is feasible from a geotechnical perspective. The primary geotechnical considerations for the project are summarized as follows:

- The bridges should be supported on driven piles that extend into the very stiff silt and hard clay beneath the site. Driven piles will generate vibrations during installation. Based on experience and published information, we do not anticipate that structures within 50 feet of the piles will be damaged by vibrations.
- The subgrade soil in the areas of the proposed culvert replacements is suitable to support the precast box culvert sections, provided weak and unsuitable subgrade materials are removed and replaced with structural fill and a 2-foot-thick layer of granular material is provided below the culverts.
- The near-surface soil at the site is fine-grained and easily disturbed during wet weather. The contractor should protect the subgrade areas from disturbance during construction.

TABLE OF CONTENTS **PAGE NO.**

TABLE OF CONTENTS	PAGE NO.

ACRONYMS AND ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
AC	asphalt concrete
ASTM	American Society for Testing and Materials
BGS	below ground surface
CIP	cast-in-place
CSZE	Cascadia subduction zone earthquake
FHWA	Federal Highway Administration
g	gravitational acceleration (32.2 feet/second^2)
GDM	Geotechnical Design Manual
H:V	horizontal to vertical
LRFD	Load and Resistance Factor Design
MSE	mechanically stabilized earth
NA	not applicable
ODOT	Oregon Department of Transportation
OSHA	Occupational Safety and Health Administration
OSSC	Oregon Standard Specifications for Construction (2018)
pcf	pounds per cubic foot
pci	pounds per cubic inch
PDA	Pile Driving Analyzer®
PGA	peak ground acceleration
psf	pounds per square foot
psi	pounds per square inch
SPT	standard penetration test
USGS	U.S. Geological Survey
WEAP	wave equation analysis program

1.0 INTRODUCTION

This report presents our geotechnical evaluation for the proposed Walker/Murray Improvements project in Beaverton, Oregon. The project consists of improvements to the intersection of SW Walker Road and SW Murray Boulevard, including replacement of the Walker Road and Murray Boulevard bridges over Cedar Mill Creek and the replacement of the culverts on North Johnson Creek under SW Far Vista Drive and SW Walker Road. The site location with respect to the surrounding physical features is shown on Figure 1. The existing topographic setting of the site is provided on Figure 2. Acronyms and abbreviations used herein are defined above, immediately following the Table of Contents.

The existing Murray Boulevard bridge over Cedar Mill Creek is located on SW Murray Boulevard approximately 275 feet south of the intersection of SW Murray Boulevard and SW Walker Road. The bridge is approximately 66 feet long, 93 feet wide, and supported by driven piles. The existing Walker Road bridge over Cedar Mill Creek is located on SW Walker Road approximately 200 feet east of the intersection of SW Murray Boulevard and SW Walker Road. The bridge is approximately 66 feet long, 88 feet wide, and supported by driven piles.

The existing North Johnson Creek culvert on SW Walker Road is 125 feet long and consists of a 14-foot-wide by 9-foot-tall closed box structure. The existing culvert has a 53-degree skew from the SW Walker Road centerline and is located approximately 190 feet east of the intersection of SW Walker Road and SW Far Vista Drive. The existing North Johnson Creek culvert on SW Far Vista Drive consists of a 14-foot-wide by 9-foot-tall closed box structure. The existing culvert is located approximately 52 feet south of the centerline of SW Walker Road and is approximately 150 feet long. The existing culverts will be replaced with 20-foot-wide by 10-foot-tall precast box culverts. The SW Walker Road culvert replacement will be 143.8 feet in length and the SW Far Vista Drive culvert replacement will be 70 feet in length.

2.0 PURPOSE AND SCOPE

The scope of our geotechnical services was to explore the subsurface conditions along the bridge alignments to provide geotechnical engineering recommendations for design and construction of the replacement bridges and culvert replacement. The specific scope of services included the following:

- Reviewed readily available geologic maps and water well logs that cover the site vicinity.
- Reviewed our in-house files for geotechnical information from nearby projects.
- Coordinated and managed the field investigation, including scheduling drilling contractors, utility locaters, and GeoDesign staff.
- Drilled four borings to depths between 111.5 and 121.5 feet BGS near the proposed abutment locations of the proposed bridge replacements along SW Murray Boulevard and SW Walker Road.
- Drilled four borings to depths between 26.5 and 31.5 feet BGS near the existing North Johnson Creek culverts on SW Walker Road and SW Far Vista Drive.
- Performed infiltration testing in three hand auger borings at depths between 4.5 and 5 feet BGS.

- Maintained a detailed log of each exploration, visually classified the soil encountered, collected soil samples as appropriate for the soil conditions encountered, and observed groundwater conditions in each exploration.
- Conducted the following laboratory tests on soil samples collected from the explorations:
  - Seventy-six moisture content determinations in general conformance with ASTM D2216
  - Four particle-size analyses in general conformance with ASTM D1140
  - Eight Atterberg limits tests in general conformance with ASTM D4318
  - Three sieve with hydrometer tests in general conformance with ASTM C117, ASTM C136, and ASTM D422
- Provided geotechnical engineering construction recommendations for site preparation, structural fill compaction criteria, wet/dry weather earthwork procedures, and retaining walls.
- Provided deep foundation recommendations for the proposed bridges, including bearing capacity, seismic design parameters, and foundation settlement.
- Provided recommendations for retaining wall construction, including preliminary global stability and soil parameters.
- Provided recommendations for design and construction of precast box culverts, including factored bearing resistance and estimated settlements.
- Prepared this report summarizing the results of our subsurface exploration and laboratory testing programs and presenting our geotechnical recommendations and conclusions.

3.0 SITE CONDITIONS

3.1 GEOLOGIC SETTING

The site is located in the Tualatin Basin of the Puget Sound-Willamette Valley physiographic province, a tectonically active lowland located along the convergent Cascadia margin (Orr and Orr, 1999). The Tualatin Basin is formed by a gentle syncline between the uplifted Coast Ranges to the west, the Chehalem Mountains to the south, and the Tualatin Mountains to the north and east. The Tualatin Mountains have been uplifted along northwesterly oriented faults, including the steeply dipping Portland Hills fault located along the eastern flank of the mountains.

The near-surface geologic unit mapped at the site is the fine-grained facies of the Quaternary flood deposits (Madin, 1990). The unit consists of unconsolidated silt and sand deposited by catastrophic floods associated with the sudden release of waters from glacial Lake Missoula during the late Pleistocene Age. Many dozens of these Missoula Floods occurred between approximately 15,500 and 12,500 years ago (and perhaps during earlier glaciations). Flood waters several hundred feet deep swept out of the Columbia Gorge and over the lowlands of the Portland area. The thickness of the flood deposits in the site vicinity is approximately 40 to 50 feet.

Underlying the Quaternary flood deposits are a thick accumulation of moderately to poorly lithified mudstone and sandstone mapped as the Sandy River Mudstone equivalent (Madin, 1990). These were deposited in a large delta where the ancient Columbia and Willamette rivers converged, probably during the middle Miocene to Pleistocene (approximately 6 million to 1 million years ago). The thickness of the unit in the site vicinity is approximately 250 feet.

Basement rocks underlying the Sandy River Mudstone equivalent in the site vicinity consist of the Miocene Columbia River Basalts, emplaced approximately 17 million to 6 million years ago in the Portland area (Madin, 1990). The Columbia River Basalts are exposed in the Tualatin Basin in the highlands surrounding the valley and in a group of mountains south of the site, which include Cooper Mountain and Bull Mountain. The Columbia River Basalts consist of thick flows of basalt erupted from fissures in eastern Oregon, Washington, and western Idaho that traveled down the ancient Columbia River Gorge to fill the lowland areas around Portland.

3.2 SURFACE CONDITIONS

The site is located near the intersection of SW Murray Boulevard and SW Walker Road. The project limits extend from the intersection to approximately 1,900 feet east and 720 feet west along SW Walker Road and from the intersection to approximately 800 feet north and 1,250 feet south along SW Murray Boulevard. SW Murray Boulevard will be widened to the east approximately 15 to 20 feet and will require up to 5 to 6 feet of fill, and SW Walker Road will be widened to the north and south approximately 18 feet and will require up to 6 feet of fill. The Nike Campus berm is located south of SW Walker Road east of the intersection and west of SW Murray Boulevard south of the intersection. A retail facility is located at the northwest corner of the intersection and residential apartments are located at the northeast corner. Residential housing is located both north and south of SW Walker Road east of the intersection and east of SW Murray Boulevard south of the intersection. The ground surface in the area of the proposed fill locations is relatively flat beyond the toe of the existing roadway slopes. Figure 2 shows the topography and features at the site.

3.3 SUBSURFACE CONDITIONS

3.3.1 Explorations

The subsurface conditions were explored at each bridge location by drilling a boring near each bridge abutment. Borings B-1 and B-2 were drilled to depths between 111.5 and 121.5 feet BGS for the Murray Boulevard bridge. Borings B-3 and B-4 were drilled to a depth of 121.5 feet BGS for the Walker Road bridge. Borings B-5, B-6, B-7, and B-8 were drilled to depths between 26.5 and 31.5 feet BGS for the culvert extension under SW Walker Road and the culvert replacement under SW Far Vista Drive.

The approximate locations of the explorations are shown on Figure 2. A detailed description of the subsurface exploration program and the exploration logs are presented in Appendix A. Laboratory testing to determine physical and engineering properties of the soil at the site was completed on select samples from the explorations. Descriptions of the laboratory procedures and test results are presented in Appendix A. The general soil conditions encountered in our explorations are described below.

3.3.2 Subsurface Conditions

3.3.2.1 Murray Boulevard and Walker Road Bridges

The pavement sections near the abutments for the Murray Boulevard and Walker Road bridges consist of approximately 7 to 21 inches of AC underlain by approximately 3 to 12 inches of aggregate base. Beneath the pavement section is approximately 3.5 to 7 feet of gravel, silt, or clay fill with variable sand content and trace organics. Underlying the fill is soft to very stiff alluvial clay and silt. The alluvial silt and clay are generally brown to gray with trace organics and variable sand content. Stratified beds of wet, silty sand less than 1 foot in thickness were present within the silt and clay. The silt and clay are moist to wet and have low plasticity. The recent alluvium extends to depths of 55 to 60 feet BGS in the borings. Laboratory testing of the silt and clay indicates that their moisture content ranged from 26 to 45 percent at the time of explorations. The recent alluvial clay and silt generally have low strength and high compressibility.

Beneath the alluvial silt and clay is Sandy River Mudstone consisting of stiff to very stiff silt and clay. The mudstone is gray to brown in color, generally moist, and ranges from low to high plasticity. The mudstone extends to the maximum depth explored in the borings completed near Murray Boulevard bridge. Medium dense to dense, silty sand between depths of approximately 100 and 115 feet BGS were present at the Walker Road bridge. Laboratory testing indicates that the moisture content of the Sandy River Mudstone ranged from 24 to 45 percent at the time of our explorations.

3.3.2.2 North Johnson Creek Culverts

The pavement sections near the existing North Johnson Creek culvert on SW Far Vista Drive consist of approximately 4.8 to 8.8 inches of AC underlain by approximately 10 to 15 inches of aggregate base. Beneath the pavement section in boring B-5 is approximately 3.5 feet of silt fill. Beneath the fill or pavement section in borings B-5 and B-6 are alluvial silt and clay consistent with the alluvial soil described for the Murray Boulevard and Walker Road bridges. The alluvial silt and clay extend to the maximum depths explored of 31.5 and 26.5 feet BGS in borings B-5 and B-6, respectively.

The pavement sections near the existing North Johnson Creek culvert on SW Walker Road consist of approximately 10 inches of AC underlain by approximately 11 to 14 inches of aggregate base. Beneath the pavement section in boring B-7 is approximately 13.5 feet of gravel fill. The gravel is likely associated with backfilling of the existing culvert. Beneath the fill or pavement section in borings B-7 and B-8 are alluvial silt and clay consistent with the alluvial soil described for the Murray Boulevard and Walker Road bridges. The alluvial silt and clay extend to the maximum depths explored of 26.5 and 31.5 feet BGS in borings B-7 and B-8, respectively.

3.3.3 Groundwater

Groundwater could not be directly measured due to the presence of drilling fluid, but increased soil moisture content was observed within 6 to 12 feet of the existing ground surface in most borings. The depth to groundwater will fluctuate in response to seasonal weather and changes in surface topography, and sites within close proximity to the creeks will likely experience groundwater levels consistent with nearby Cedar Mill Creek and North Johnson Creek water levels.

3.4 INFILTRATION TESTING

Infiltration testing was performed in borings HA-1, HA-2, and HA-3. The bases of the borings were saturated prior to testing the infiltration rate using falling-head procedures and a 3-inch-diameter standpipe. Low-head conditions were tested by maintaining the height of the water column at less than 2 feet. Infiltration tests were performed at depths of 5, 4.6, and 4.5 feet in HA-1, HA-2, and HA-3, respectively. Negligible infiltration was observed at all test locations and infiltration systems are not feasible within the project limits.

4.0 LABORATORY TESTING

In addition to the moisture content testing, four particle-size analyses, eight Atterberg limits tests, and three sieve with hydrometer tests were completed on samples collected from the explorations. Descriptions of the laboratory procedures and test results are presented in Appendix A.

5.0 CONCLUSIONS

Based on the results of subsurface explorations, laboratory testing, and engineering analyses, the project is feasible from a geotechnical perspective. The following factors will have an impact on design and construction of the proposed bridge and culvert improvements:

- The proposed bridge structures should be supported on driven piles advanced into the stiff to very stiff silt and clay. Pile resistance should be verified with dynamic testing.
- The subgrade soil in the areas of the proposed culvert replacements is suitable to support the precast box culvert sections, provided weak and unsuitable subgrade materials are removed and replaced with structural fill and a 2-foot-thick layer of granular material is provided below the culverts.
- The near-surface soil at the site consists primarily of fill and native silt and clay. The silty soil can become disturbed during the wet season with repeated heavy traffic. The contractor should protect the subgrade areas from disturbance during construction.

6.0 DESIGN

6.1 PERMANENT SLOPES

Permanent fill slopes on the site should not exceed a gradient of 2H:1V, unless specifically evaluated for stability. Slopes should be planted with appropriate vegetation or armored to provide protection against erosion as soon as possible after grading. Surface water runoff should be collected and directed away from slopes to prevent water from running down the face of the slope.

6.2 FILLS

New approach fills may be required to widen the bridge structures. Based on review of the plans, approximately 5 to 6 feet of fill will be placed on the east side of SW Murray Boulevard to widen the bridge approach and approximately 6 feet of fill will be placed on the south side of SW Walker Road for the bridge approach widening. The fills are confined to the eastern 25 feet of the approach on the Murray Boulevard bridge and the southern 15 feet of the approach on the Walker Road bridge and will not significantly affect the entire approaches. We recommend that hardscapes such as pavement and sidewalks be delayed until after settlement is complete. We estimate that settlement should essentially be complete within three months after fill placement.

6.3 PRECAST BOX CULVERT RECOMMENDATIONS

Based on the project information, the SW Far Vista Drive replacement culvert invert elevation will be 170.88 feet, with the channel bottom approximately 4 feet above the invert elevation. The SW Far Vista Drive culvert will be 70 feet in length. We understand that approximately 2 feet of cover will be placed above the top of the SW Far Vista Drive culvert. The invert of the Walker Road culvert replacement will be located northwest of the existing culvert and the outlet will be in the same location as the existing structure. The proposed invert elevation is 171.32 feet, with 3.8 feet of fill above the invert elevation.

Based on our subsurface explorations, the culverts will be founded in medium stiff to stiff silt. We recommend that a minimum 2-foot-thick layer of crushed rock be placed below the box culverts. To prevent the migration of fines, we recommend that a geotextile separation fabric be placed between the native fine-grained soil and the crushed rock granular fill. The box culverts should be designed using a factored bearing resistance of 3,000 psf with two feet of granular material provided below the culvert. Lateral earth pressures of the box culvert should be calculated using an at-rest earth pressure of 55 pcf for granular fill material placed above the groundwater elevation and 20 pcf for granular fill below the groundwater level. Traffic surcharge loads equivalent to 2 feet of soil or 250 pcf should be applied to the culvert. Surcharge pressures for the fill material above the top of the culvert should be calculated using a unit weight of 125 pcf.

The estimated total primary settlement of up to 2 inches is anticipated for the proposed precast box culvert, retaining walls, and approach embankment fill. Differential settlement is expected to be approximately half of the total. The settlement of the box culvert should essentially be complete within 60 days after fill placement.

We understand that a concrete saddle will be constructed above a 15-inch-diameter waterline that is present under the proposed Walker Road Culvert location. We recommend that the footings for the saddle structure be designed using a factored bearing resistance of 2,000 psf. We recommend that the fill material placed between the bottom of the culvert and the top of the concrete saddle structure consist of controlled density fill materials.

6.4 SEISMIC CONSIDERATIONS

6.4.1 Geologic Hazards

6.4.1.1 General

In addition to ground shaking, site-specific geologic conditions can influence the potential for earthquake damage. Deep deposits of loose or soft alluvium can amplify ground motions, resulting in increased seismic loads on structures. Other geologic hazards are related to soil failure and permanent ground deformation. Permanent ground deformation could result from liquefaction, lateral spreading, landsliding, and fault surface rupture. The following sections provide discussion regarding potential seismic hazards.

6.4.1.2 Fault Surface Rupture

Fault surface rupture is a seismic hazard that includes abrupt, large, differential ground movements that can cause damage to structures located on a known fault. Based on USGS

(2017) mapping, the closest mapped fault is the Oatfield fault, which is located approximately 3 miles northeast of the site. Due to the distance of the closest mapped fault, fault surface rupture is not a design consideration for the project.

6.4.1.3 Liquefaction

Liquefaction can be defined as the sudden loss of shear strength in a soil due to an excessive buildup of pore water pressure. Liquefied soil layers generally follow a path of least resistance to dissipate pore pressures, often resulting in sudden surface settlement, sand boils or ejections, and/or lateral spreading in extreme cases. Clean, loose, uniform or silty, fine-grained, saturated sand is particularly susceptible to liquefaction.

We performed a liquefaction analysis using the Simplified Procedure per recommendations in Section 6.5.2.2 of the ODOT GDM (2018) for sites having potentially liquefiable soil. Based on subsurface conditions, laboratory testing, and analysis, it is our opinion that liquefaction of sand and strain-softening of fine-grained soil will be negligible under the design levels of shaking.

6.4.1.4 Lateral Spreading

Lateral spreading is a liquefaction-related seismic hazard. Areas subject to lateral spreading are typically gently sloping or flat sites underlain by liquefiable sediment adjacent to an open face (such as riverbanks or sloughs). Liquefied soil adjacent to open faces may "flow" in that direction, resulting in surface cracking and lateral displacement toward the open face. The magnitude of lateral spreading decreases with distance from the open face. Because liquefaction is negligible, lateral spreading is not a design consideration.

6.4.2 General

According to the ODOT GDM (2018), new bridges west of U.S. Highway 97 are to be designed for a two-level seismic design criterion. The criteria are presented below:

"Life-Safety" Design Criteria:

Under this level of shaking, the bridge and approach structures, foundation and approach fills must be able to withstand the design forces and displacements without collapse of any portion of the structure and also be consistent with the Life Safety seismic design criteria described below and in the current ODOT BDDM. In general, bridges that are properly designed and detailed for seismic loads can accommodate relatively large deflections without the danger of collapse.

Ground motions for this criterial have an average return period of 1000 years (7% probability of exceedance in 75 years). This level of services is typically what has been provided for bridge design prior to the 2018 of the ODOT GDM.

"Operational" Design Criteria:

This criterion is associated with bridges remaining in service following a level of ground shaking associated with a full-rupture Cascadia Subduction Zone Earthquake (CSZE). For the Operational performance level, bridges and approach fills are designed to provide access for emergency vehicles associated with a full rupture Cascadia Subduction Zone Earthquake (CSZE). Some *structural damage is anticipated but the damage should be repairable and the bridge should be able to carry emergency vehicles immediately following the earthquake. This holds true for the approach fills leading up to the bridge.*

Approach fill settlement and lateral displacements should be minimal to provide for immediate emergency vehicle access for at least one travel lane. As a general rule of thumb, an estimated lateral embankment displacement of up to 1 foot is considered acceptable in many cases as long as the "operational" performance criteria described above can be met and the structure foundations are adequately designed to withstand the soil loads resulting from the lateral displacements. Vertical settlements on the order of 6" to 12" may be acceptable depending on the roadway geometry, anticipated performance of the bridge end panels and the ability of bridge foundation elements to withstand any imposed downdrag loads.

We note that the Oatfield fault is located approximately 3 miles from the proposed bridge replacements. As described in AASHTO LRFD (2017), a site-specific seismic analysis is required for bridges within 6 miles of an active fault. Based on Sommerville et al. (1997), the spectral acceleration factor associated with the worst-case earthquake scenario is 1.0 between periods of 0.0 seconds and 0.6 second and 1.5 at a period of 5.0 seconds. Based on our experience, the proposed single-span bridges have low seismic requirements and a fundamental period below 0.6 second; therefore, a site-specific seismic analysis is not necessary for the proposed bridge replacements.

6.4.3 Seismic Design Parameters

Due to the presence of liquefiable soil at the site, a site-specific ground motion response may be appropriate. Based on our experience as well as the discussion in Section 6.5.1.4 of the ODOT GDM (2018), the liquefied response spectra are typically lower than non-liquefied response spectra for periods less than approximately 1.0 second. Based on our experience, the proposed single-span bridges will have a fundamental period of less than 1.0 second. Therefore, we did not complete a site-specific ground motion response and conservatively used the code-based spectra parameters.

Earthquake ground motions at the site were evaluated using seismic data from the 2016 ODOT Seismic Design maps. Peak bedrock accelerations for return periods of 500 years and 1,000 years for the "Life Safety" criteria are reported in Table 1. Based on geologic mapping and our explorations, the recommended seismic design Site Class is D. The "Life Safety" seismic design parameters for Site Class D soil are provided in Table 2.

Table 1. ODOT Seismic Design Parameters for Bedrock "Life Safety" Criteria

Parameter	Value	
Site Class (Bedrock)	B	
	Return Period	
	500 years	1,000 years
PGA	0.178 g	0.278 g
S_s	0.387 g	0.605 g
S_1	0.136 g	0.227 g

Table 2. ODOT Seismic Design Parameters for Soil "Life Safety" Criteria

Parameter	Value	
Site Class (Soil)	D	
	Return Period	
	500 years	1,000 years
F_{PGA}	1.444	1.322
F_a	1.490	0.316
F_v	2.238	2.416
S_{DS}	0.384 g	0.127 g
S_{D1}	0.211 g	0.325 g
A_s	0.257 g	0.368 g

Peak bedrock accelerations for the "Operational" criteria are reported in Table 3. Based on geologic mapping and our explorations, the recommended seismic design Site Class is D. The "Operational" seismic design parameters for Site Class "D" soil are provided in Table 4.

Table 3. ODOT Seismic Design Parameters for Bedrock "Operational" Criteria

Parameter	Value
Site Class (Bedrock)	B
	Return Period
	CSZE
PGA	0.155 g
S_s	0.292 g
S_1	0.162 g

Table 4. ODOT Seismic Design Parameters for Soil "Operational" Criteria

Parameter	Value
Site Class (Soil)	D
	Return Period
	CSZE
F_{PGA}	1.490
F_a	1.566
F_v	2.276
S_{DS}	0.305 g
S_{D1}	0.246 g
A_s	0.231 g

6.5 FOUNDATION ANALYSIS AND DESIGN RECOMMENDATIONS

6.5.1 General

Based on correspondence with WHPacific, Inc., 16-inch-diameter by 0.5-inch-wall thickness, closed-ended pipe piles will be used to support the bridge abutments. The Walker Road bridge will have 15 piles, and the Murray Boulevard bridge will have 18 piles. The Strength 1 and Service 1 AASHTO LRFD (2017) loads for Bents 1 and 2 are shown in Table 5.

Table 5. AASHTO LRFD (2017) Service 1 and Strength 1 Loads

Bridge	Maximum Service 1 Limit State Load per Pile (kips)	Maximum Strength 1 Limit State Load per Pile (kips)
Murray Boulevard	369	527
Walker Road	317	455

1. Loads are based on information from WHPacific.

Based on information provided by the design team, the scour depth at both bridge crossings is estimated to be negligible. The following sections provide recommendations for driven piles in accordance with AASHTO LRFD (2017).

6.5.2 Downdrag

New approach fills may be required to raise the new bridges above design flood elevations. The majority of the fills will be negligible at the bridge abutments, with the exception of the east side of the Murray Boulevard bridge and the south side of the Walker Road bridge, where a maximum of 6 feet of fill will be placed for the widened portion of the bridge approaches. Based on the limited area that will receive new fills, we estimate that settlement will essentially be negligible at the pile foundation locations. Therefore, downdrag loads were not applied in our pile analyses.

6.5.3 Driven Piles

6.5.3.1 Driven Pile Axial Capacities

Sixteen-inch-diameter, closed-ended pipe piles will be used to support the bridges. Analysis was completed to determine factored axial resistances and total nominal resistance during driving for individual piles using methods described in AASHTO LRFD (2017). The analysis is based on the following design and construction assumptions:

- The factored pile resistances are based on a resistance factor of 0.65 associated with the use of dynamic pile testing and a calibrated wave equation analysis during construction. This corresponds to an approximate factor of safety of 1.54 for soil resistance on the pile. Without dynamic pile testing, AASHTO LRFD requires that a significantly lower resistance factor of 0.4 be used (an equivalent factor of safety of approximately 2.5), which will result in longer piles.
- The axial resistance factors of the piles should meet the requirements of Section 6.5.4.2 of AASHTO LRFD.
- There are no downdrag effects on the pile.
- There are no scour effects on the pile.
- The minimum pile spacing, clearance, and embedment into the cap as described in this section are adhered to.

Tables 6 and 7 provide estimated minimum tip elevations, estimated minimum tip lengths, and the required axial resistances per pile assuming PDA testing will be conducted on one pile per abutment for the Murray Boulevard and Walker Road bridges. Pile resistance capacity plots for Bents 1 and 2 for the Murray Boulevard bridge are provided on Figures B-1 and B-2, respectively, in Appendix B. Pile resistance capacity plots for Bents 1 and 2 of the Walker Road bridge are provided on Figures B-3 and B-4, respectively, in Appendix B. The plots present the nominal pile resistances, factored axial resistances for the strength limit state, and the factored uplift resistance for single piles. Analysis verified that the service limit loading will induce settlements less than described in the "Deep Foundation Settlement" section.

Table 6. Pile Capacities and Lengths for 16-Inch-Diameter, Closed-Ended Pipe Piles Murray Boulevard Bridge

Bent	LRFD-Factored Load (kips)	Nominal Geotechnical Resistance[1] (kips)	Pile Cut-Off Elevation (feet)	Estimated Tip Elevation (feet)	Estimated Pile Length Below Bent Cap (feet)	Factored Uplift Resistance (kips)
Bent 1	527	811	170.7	64	107	462
Bent 2	527	811	170.7	58	113	406

1. Based on a resistance factor of 0.65.

Table 7. Pile Capacities and Lengths for 16-Inch-Diameter, Closed-Ended Pipe Piles Walker Road Bridge

Bent	LRFD-Factored Load (kips)	Nominal Geotechnical Resistance[1] (kips)	Pile Cut-Off Elevation (feet)	Estimated Tip Elevation (feet)	Estimated Pile Length Below Bent Cap (feet)	Factored Uplift Resistance (kips)
Bent 1	455	700	172.7	90	83	279
Bent 2	455	700	172.7	79	94	308

6.5.3.2 Wave Equation Analysis

Based on AASHTO LRFD (2017), wave equation analysis should be completed to determine the driving criteria for piles where dynamic testing is not completed. The wave equation analysis should be performed using the computer program WEAP 1987 (or subsequently newer version) based on information provided by PDA testing. PDA testing will be completed by the agency's representative (GeoDesign) on a minimum of one pile per pier.

6.5.3.3 Wave Equation Method

A suitable hammer shall be selected for driving piles and for performance of wave equation analyses. The 1987 or newer version of WEAP should be used. The wave equation analyses should be conducted by personnel qualified by training and experience to perform this type of work. The analysis should be completed using a triangular distribution.

The WEAP analysis should be concurrent and submitted with the Pile and Driving Equipment Data Form. The Agency will approve or reject the pile driving equipment submittal after a review of the wave equation analysis conducted by the contractor. The pile driving hammers should meet the following requirements based on wave equation analysis:

1. The energy of the submitted hammer shall produce a wave equation-predicted blow count between 3 and 15 blows per 25 millimeters (1 inch) for the ultimate capacities, pile lengths, and other conditions specified.
2. The pile stresses indicated by the wave equation analysis at the ultimate pile bearing capacity shall not be greater than the stress at the point of impending damage to the pile. Tensile and compressive stresses in the pile shall be limited to 90 percent of the pile material's yield strength for the grade of steel specified at any time during pile installation.

Hammers not meeting these requirements should be rejected. The rejected hammers should be replaced with suitable hammers. Use input values for the wave equation analyses according to Tables 8 and 9.

Table 8. Wave Equation Analyses Parameters – Murray Boulevard Bridge

Bent	LRFD-Factored Load (kips)	Pile Type	Pile Length[1] (feet)	Quake (inches)		Damping (seconds/foot)		% Skin (ITYS)	R_{ndr}[2] (kips)
				Skin	Toe	Skin	Toe		
Bent 1	527	PP 16X0.5	107	0.10	0.13	0.2	0.15	87	811
Bent 2	527	PP 16X0.5	113	0.10	0.13	0.2	0.15	76	811

1. The pile length shown does not include the stick-up length above bottom of pile cap.
2. The nominal driving resistance, R_{ndr}.

Table 9. Wave Equation Analyses Parameters – Walker Road Bridge

Bent	LRFD-Factored Load (kips)	Pile Type	Pile Length[1] (feet)	Quake (inches)		Damping (seconds/foot)		% Skin (ITYS)	R_{ndr}[2] (kips)
				Skin	Toe	Skin	Toe		
Bent 1	455	PP 16X0.5	83	0.10	0.13	0.2	0.15	61	700
Bent 2	455	PP 16X0.5	94	0.10	0.13	0.2	0.15	68	700

1. The pile length shown does not include the stick-up length above bottom of pile cap.
2. The nominal driving resistance, R_{ndr}.

6.5.3.4 Minimum Pile Spacing, Clearance, and Embedment into Cap

The efficiency of pile groups should be evaluated when piles are closely spaced and overlapping stresses from adjacent piles are present or when the pile cap is not in contact with the ground. Driven pile spacing should not be less than 2.5B (diameter) or 30 inches, center to center, to negate pile group effects to the factored resistances.

The distance from the side of any pile to the nearest edge of the pile cap shall not be less than 9 inches. The top of piles should extend at least 12 inches into the pile cap after all damaged material has been removed. If the pile is attached to the cap by embedded bars or strands, the pile shall extend no less than 6 inches into the cap.

Where a reinforced concrete beam is CIP and used as a bent cap supported by piles, the concrete cover on the side of the pile shall not be less than 6 inches, plus an allowance for permissible pile misalignment. Where pile reinforcement is anchored in the cap satisfying Section 5.12.4.1 of AASHTO LRFD (2017), the concrete cover may be less than 6 inches.

6.5.4 Deep Foundation Settlement

Settlement of driven steel pipe piles or drilled piles embedded into dense gravel or sand will be negligible beyond the elastic compression of the pile. For design purposes, a maximum settlement of 1 inch should be estimated for deep foundations designed and constructed in accordance with the recommendations in this report.

6.5.5 Lateral Analysis Parameters

Lateral analysis parameters for analyzing the deep foundations with the LPILE computer software program are presented in Tables C-1 and C-2 for the Murray Boulevard bridge and Tables C-3 and C-4 for the Walker Road bridge (see Appendix C).

In addition to the lateral resistance provided by the foundation elements, lateral forces can be resisted by passive pressure of the pile caps. Provided that the pile caps are at least 2 feet below the ground surface, an available passive earth pressure of 350 psf can be used for loading away from Cedar Mill Creek and where the passive pressures do not exert forces on any planned retaining wall face. We recommend that the available passive earth pressure be reduced to 200 psf where the forces will act on the retaining walls. This additional pressure will need to be accounted for in retaining wall design if they are within the influence zone of retaining walls.

6.5.6 Impact Panels

Impact panels should be constructed on embankment structural fill compacted to at least 95 percent of the maximum dry density, as determined by AASHTO T 99. The total post-construction settlement is expected to be less than 1 inch, and differential settlement between parallel edges of the impact panels should be less than 1 inch given the anticipated loads. This assumes that the full duration of the embankment settlement will occur prior to construction of the impact panels. It is anticipated that the permanent contact pressures imposed on the impact panels will be low enough such that resulting additional settlement will not be significant.

6.6 CIP RETAINING WALLS

6.6.1 General

Based on correspondence with Kittelson & Associates and WHPacific, CIP retaining walls will be required at the bridge abutments. The retaining wall design recommendations presented below are based on the following assumptions: (1) the walls are conventional MSE-type construction, (2) the walls will be less than 15 feet in height, (3) the backfill is granular, (4) the retaining walls are unrestrained and free to rotate, and (5) the appropriate wall surcharges are included in the design as described in this section.

6.6.2 CIP Wall Design Parameters

For unrestrained retaining walls, we recommend an active equivalent fluid pressure of 35 pcf be used for design. If retaining walls are restrained from rotation prior to being backfilled, an equivalent fluid pressure of 55 pcf should be used for design. For embedded building walls, a superimposed seismic lateral force should be calculated based on a dynamic force of $6H^2$ pounds per lineal foot of wall (where H is the height of the wall in feet). The load should be applied as a distributed load with the centroid located at a distance of 0.6H from the base of the wall. Traffic and retained slope surcharges should be included where appropriate as described in the "Wall Surcharges" section. In addition, the foundation soil for CIP and gravity walls should be firm, granular native or fill soil prepared in conformance with the "Construction" section. Backfill behind the walls should consist of granular wall backfill described in this section.

6.6.3 Wall Surcharges

Where traffic loads are located within a horizontal distance from the top of the wall equal to one-half the wall height, the lateral earth pressure shall be increased by a surcharge load using the following equation and Tables 10 and 11:

$\Delta p = k\gamma_s h_{eq}$ where,

Δp = constant horizontal earth pressure due to live load surcharge (psf)
k = coefficient of lateral earth pressure (should be taken as 0.28)
γ_s = total unit weight of soil (pcf), should be assumed as 125 pcf
h_{eq} = equivalent height of soil for vehicular load (per Tables 10 and 11

Table 10. Equivalent Height of Soil for Vehicular Loading on Abutments Perpendicular to Traffic

Bent Height (feet)	h_{eq} (feet)
5	4
10	3
> 20	2

Table 11. Equivalent Height of Soil for Vehicular Loading on Abutments Parallel to Traffic

Bent Height (feet)	h_{eq} (feet) Distance from Wall Backface to Edge of Traffic	
	0 feet	1 Foot or Farther
5	5	2
10	3.5	2
> 20	2	2

The design equivalent fluid pressure should also be increased for walls that retain sloping soil. We recommend the following lateral earth pressures be increased using the following factors (Table 12) when designing walls that retain sloping soil.

Table 12. Lateral Earth Pressure Increase Factors for Sloping Soil Backfill

Slope of Retained Soil (degrees)	Lateral Earth Pressure Increase Factor
0	1.00
5	1.06
10	1.12
20	1.33
25	1.52
30	2.27

Seismic forces should be modeled based on the pseudo-static approach developed by the Mononobe-Okobe method. As described in the AASHTO LRFD (2017), a seismic coefficient of one-half of the PGA should be used when analyzing internal stability.

6.6.4 Temporary Cuts

Temporary cuts for retaining walls should be completed as described in the "Excavation" section.

6.6.5 Wall Foundations

Retaining wall foundations may be supported on firm native soil or organics-free fill soil provided the foundation subgrade soil is evaluated as described in the "Site Preparation" section. Footings supporting the proposed retaining walls should be designed based on a net allowable bearing pressure of 2,000 psf. The weight of the footing and overlying backfill can be ignored in calculating footing sizes. The recommended allowable bearing pressure applies to the total of dead plus long-term live loads and may be increased by one-third for short-term loads, such as those resulting from wind or seismic forces. Continuous footings constructed on or above slopes should be at least 18 inches wide and at least 24 inches below the lowest adjacent final grade. A minimum horizontal distance of at least 5 feet from the top of the footing shall be provided between the near face of the footing and the nearest face of any finished slope. Lateral loads on footings can be resisted by passive earth pressure on the sides of the structures and by friction on the base of the footings. Our analysis indicates that the available passive earth pressure for footings is 350 pcf. Adjacent footing elements, pavements, or the upper 12-inch depth of adjacent, unpaved areas should not be considered when calculating passive resistance. A coefficient of friction equal to 0.40 may be used when calculating resistance to sliding for footings placed on the native or fill gravel.

6.6.6 Wall Backfill and Drains

Granular wall backfill materials placed behind CIP concrete walls should extend at least 1 foot behind the heel of the wall. The granular wall backfill materials should extend to at least 1 foot below the top of the wall where the backfill is level and at least to the top of the wall where the backfill is sloped. Sloping backfill above the retaining wall may consist of general borrow structural fill. Specifications for the fill materials are discussed in detail in the “Materials” section.

A minimum 12-inch-wide by 12-inch-tall zone of drain rock should be placed at the heel of all CIP concrete retaining walls. Perforated collector pipes should be embedded at the base of the drain rock. The drain rock should meet the requirements provided in the “Materials” section. The perforated collector pipes should discharge at an appropriate location away from the base of the wall.

The wall backfill should be compacted as recommended in the “Materials” section. Backfill placed within 3 feet of the wall should be compacted in lifts less than 6 inches thick using hand-operated tamping equipment (such as a jumping jack or vibratory plate compactor).

6.6.7 Construction Considerations

All footing subgrades should be evaluated by the project geotechnical engineer or their representative to confirm suitable bearing conditions. Observations should also confirm that all loose or soft material, organics, unsuitable fill, prior topsoil zones, and softened subgrades (if present) have been removed. Localized deepening of footing excavations may be required to penetrate any deleterious material.

If construction is undertaken during periods of rain, we recommend placing at least 3 to 4 inches of imported granular material over the prepared footing subgrades to help protect the subgrade from disturbance due to the elements and foot traffic. The imported material should meet the specifications for imported granular material as discussed in the “Materials” section.

6.7 PREFABRICATED MODULAR BLOCK WALLS

6.7.1 Wall Geometry

A prefabricated modular block wall is proposed along the south side of the channel realignment of North Johnson Creek. An existing residence is located approximately 22 feet south of the proposed wall location. The wall alignment and section information were provided by Kittelson & Associates. The maximum exposed heights for the wall are expected to range from approximately 4.5 to 6.5 feet with riprap placed at the toe of the wall at a slope of 2H:1V. We understand the riprap is required for scour protection and the riprap layer will have a thickness of 3 feet. The prefabricated modular block wall will require a minimum toe embedment of 2 feet and a wall batter of 1H:8V. The cut for the wall will not be supporting roadways or sidewalks, so a traffic surcharge load should not be applied in design. The contractor should consult the wall manufacturer during construction to assist with block arrangement. Blocks should be arranged so that the entire wall interlocks and acts as a whole and not in independent sections.

6.7.2 Wall Design Parameters

The soil parameters presented in Table 13 should be used for analysis of the prefabricated modular block wall. The material used to backfill behind the walls is discussed in the “Wall Backfill and Drains” section as well as the “Materials” section.

Table 13. Wall Design Parameters

Soil Type	Unit Weight (pcf)	Friction Angle, φ (degrees)	Cohesion (psf)
Retained Soil – Granular Wall Backfill[1]	130	35	0
Retained Soil – Native Silt[2]	120	28	50
Foundation Soil – Native Silt[3]	120	28	50

1. Wall backfill for prefabricated modular block, conventional segmental block, and CIP walls should meet the specifications provided in OSSC 00510.12 – Granular Wall Backfill.
2. Retained soil should meet the specifications provided in OSSC 00330.12 – Borrow Material.
3. Foundation soil should be native or fill soil prepared in conformance with the "Site Preparation" section.

6.7.3 Loading Conditions

There are no buildings or roadways located within a horizontal distance from the back of the wall equal to the height of the wall. As a result, other surcharges were not modeled in the design of the prefabricated modular block wall.

6.7.4 Prefabricated Modular Block Wall Design

Preliminary design analyses were completed for the proposed prefabricated modular block wall. The contractor should consult the wall manufacturer to perform final design and block arrangement. The preliminary design of the prefabricated modular block retaining wall for this project was performed using the computer program UltraWall that was developed by Race Engineering & Assoc, LLC. This program was specifically designed to analyze the internal stability and global stability of UltaBlock® walls. The wall was analyzed using the AASHTO LRFD 2012 load factors and method of analysis. The soil parameters from Table 13 were input into the program to generate a block configuration for the wall.

Seismic forces were modeled based on the pseudo-static approach developed by the Mononobe-Okobe method. A seismic coefficient of one-half of the PGA was used to analyze both the internal and external stability. The 1,000-year event PGA value of 0.368 g (shown in Table 2) was used in wall analyses.

The calculation packages presented in Appendix D include the UltraWall® design outputs for the critical wall section for static and seismic conditions with the groundwater level at the ordinary high water elevation of 179.85 feet on North Johnson Creek in the area of the proposed wall location, which was provided by Kittelson & Associates. The UltraWall® output for the global stability under both static and seismic conditions are also presented Appendix D. Our preliminary analysis indicates that a four-block-high configuration will satisfy the design requirements as shown in Appendix D. The global stability analysis of the wall indicates that the wall will have a factor of safety greater than 1.3 for static conditions and 1.1 for seismic conditions.

6.7.5 Temporary Cuts

Temporary cuts will be required to construct the proposed retaining wall. Excavations into the slope need to be carefully planned so as not to destabilize the slope. A residential structure is located approximately 22 feet south of the proposed wall location. Cuts less than 4 feet should stand vertical. Deeper excavations should be cut back at an inclination of 1.5H:1V or flatter or be shored. Because of the adverse effects that water can have on slope stability, we anticipate that this process will not be feasible for winter-time construction, and wall construction should occur in dry summer months or use temporary shoring. If shoring is used, we recommend that the type and design of the shoring system be the responsibility of the contractor, who is in the best position to choose a system that fits the overall plan of operation.

6.7.6 Prefabricated Modular Block Wall Foundations

Conventional prefabricated modular block wall foundations may be supported on firm native soil or organic-free fill soil, provided the foundation subgrade soils is evaluated as described in the "Site Preparation" section. Walls should be constructed on a leveling course placed over the subgrade excavation. The leveling course should consist of crushed rock placed over the subgrade soil. The leveling course of crushed rock should have a minimum layer thickness of 12 inches and meet the requirement of the retaining wall leveling pad in the "Materials" section.

6.7.7 Wall Backfill and Drains

Granular wall backfill materials placed behind conventional prefabricated modular block walls should extend at least 1 foot behind the heel of the walls. The granular wall backfill should extend a minimum horizontal distance equal to ½H, where H is the height of the retaining wall, from the back of the wall. The granular wall backfill materials should extend to at least 1 foot below the top of the wall where the backfill is level and at least to the top of the wall where the backfill is sloped. A drainage geotextile fabric should be placed between the granular wall backfill and the retained soil, as described in the "Granular Wall Backfill" section. Sloping backfill above the retaining wall may consist of general borrow structural fill. The specifications for the fill materials, including compaction requirements, are discussed in detail in the "Materials" section.

A minimum 12-inch-wide by 12-inch-tall zone of drain rock should be placed at the heel of the prefabricated modular block retaining wall. A minimum 4-inch-diameter, perforated collector pipe should be placed at the base of the wall in the drain rock. The drain rock should meet the requirements provided in the "Materials" section. The perforated collector pipe should discharge at an appropriate location away from the base of the wall. The discharge pipe(s) should not be tied directly into stormwater drain systems unless measures are taken to prevent backflow into the drainage system of the wall.

6.7.8 Wall Settlement

Settlement of up to 1 percent of the wall height commonly occurs immediately adjacent to retaining walls as the wall rotates and develops active lateral earth pressures. Consequently, we recommend that construction of flatwork adjacent to retaining walls be postponed at least four weeks after backfilling of the wall, unless survey data indicates that settlement is complete prior to that time.

6.7.9 Construction Considerations

All footing subgrades should be evaluated by the project geotechnical engineer or their representative to confirm suitable bearing conditions. Observations should also confirm that all loose or soft material, organics, unsuitable fill, prior topsoil zones, and softened subgrades (if present) have been removed. Localized deepening of footing excavations may be required to penetrate any deleterious material.

If construction is undertaken during periods of rain, we recommend placing at least 3 to 4 inches of imported granular material over the prepared footing subgrades to help protect the subgrade from disturbance due to the elements and foot traffic. The imported material should meet the specifications for imported granular material as discussed in the "Materials" section.

Technical staff from the wall manufacturer should be on site at the start of wall construction to assist with block arrangement at the bottom of the walls.

6.8 GROUNDWATER CONSIDERATIONS

Groundwater levels should be carefully considered by the foundation contractor to establish the appropriate construction sequence and construction method. In general, the depth to groundwater will likely correspond to the elevation of Cedar Mill and North Johnson Creeks at the time of construction. The observed groundwater levels may rise significantly during or after extended periods of wet weather.

6.9 SITE DRAINAGE

We recommend that surface drains and other subsurface drains be connected to a tightline leading to a suitable discharge. The bridges and roadways should be sloped such that surface water runoff is collected and routed to suitable discharge points.

7.0 CONSTRUCTION

7.1 SITE PREPARATION

7.1.1 General

Earthwork operations should be planned and executed to minimize subgrade disturbance. Managing construction traffic and protection of exposed soil subgrades is the responsibility of the contractor. We can assist the contractor and project team regarding wet weather construction guidelines, if necessary.

7.1.2 Demolition

Demolition includes complete removal of the existing bridges, AC pavement, guardrails, and piling in areas proposed for construction. Demolished material should be transported off site for disposal, except as noted in the "Recycled On-Site Materials" section. Within all structural areas (bridges, pavement, and fill), excavations left from removing foundations, utilities, and other subsurface elements should be backfilled with structural fill. The bottoms of the excavations should be excavated to expose firm subgrade before filling. The sides of the excavations should be cut into firm material and sloped a minimum of 1H:1V. Utility lines abandoned under new structural components should be completely removed or grouted full if left in place. Soft or loose soil encountered in utility line excavations should be removed and replaced with structural fill.

7.1.3 Recycled On-Site Materials

On-site conventional concrete and oversized rock may be used as fill if they are processed to meet the requirements for their intended use. Processing includes crushing and screening, grinding in place, or other methods to meet the requirements for structural fill. The processed material should be fairly well graded and contain no metal, organic, or other deleterious material. The processed material may be mixed with on-site soil or imported fill to assist in achieving the gradation requirements. We recommend that processed recycled fill have the maximum particle sizes as presented in Table 14.

Table 14. Processed Fill Maximum Particle Size

Depth of Placement[1]	Maximum Particle Size
0 to 2 feet	Not recommended
2 to 6 feet	4 inches
6 to 10 feet	8 inches
deeper than 10 feet	12 inches

1. Below subgrade of structural element.

Recycled on-site fill material should not be used within a depth of 2 feet from foundations and pavement. Excavation through recycled material that is placed as structural fill may be difficult if a significant fraction of oversized particles is present. In addition, these excavations may also be prone to significant raveling and caving.

7.1.4 Subgrade Preparation

Stripping and grubbing should be performed in accordance with OSSC 00320.41 – Grubbing Operations. Trees should be removed from all proposed bridge, fill, and roadway areas. In addition, root balls should be grubbed out to the depth of the roots, which could exceed a depth of 3 feet BGS. Depending on the methods used to remove the root balls, considerable disturbance and loosening of the subgrade could occur during site grubbing. We recommend that this disturbed soil be removed to expose firm, undisturbed embankment subgrade per OSSC 00330.41(b) – Foundation Excavation. Organic material should be disposed of off site or in a suitable location determined by Washington County. Subgrade preparation should be performed in accordance with OSSC 00330.41 – Excavations and OSSC 00330.42 – Embankment, Fills, and Backfills.

7.1.5 Wet Weather Considerations

The soil at the site can be easily disturbed during the wet season and when it is moist. If not carefully executed, site preparation, utility trench work, and roadway embankment construction can create extensive soft areas and significant subgrade repair costs can result. To avoid elevated construction costs associated with wet weather construction, we strongly recommend that earthwork be scheduled for the summer and early fall when extended periods of dry weather are expected. If construction is planned when the surficial soil is wet (or may become wet), the construction methods and schedule should be carefully considered with respect to protecting the subgrade to reduce the need to over-excavate disturbed or softened soil. Construction traffic will significantly disturb these areas when wet. The project budget should reflect the recommendations below if construction is planned during wet weather or when the surficial soil is wet.

If construction occurs when wet soil is present, site preparation activities may need to be accomplished using track-mounted excavating equipment that loads removed material into trucks supported on granular haul roads. The thickness of the granular material for haul roads and staging areas will depend on the amount and type of construction traffic. Generally, an 18- to 24-inch-thick mat of granular material is sufficient. The actual thickness of haul roads and staging areas should be based on the contractor's approach to site development and the amount and type of construction traffic. In areas requiring subgrade protection, we recommend using Tensar BX 1200 biaxial geogrid (or similar) overlain by quarry-run rock, crushed rock, or crushed gravel and sand meeting the requirements in OSSC 00330.14 – Selected Granular Backfill and OSSC 00330.15 – Selected Stone Backfill, with a maximum particle size of 6 inches and less than 5 percent by dry weight passing the U.S. Standard No. 4 sieve.

7.2 EROSION CONTROL

Erosion control plans are required on construction projects located within the applicable Washington County jurisdiction. The on-site soil is susceptible to erosion, and erosion control measures should be carefully planned and in place before construction begins. Consequently, we recommend that slopes be covered with an appropriate erosion control product if construction occurs during periods of wet weather. We recommend that all slope surfaces be planted as soon as practical to minimize erosion. Surface water runoff should be collected and directed away from slopes to prevent water from running down the slope face. Erosion control measures such as straw bales, sediment fences, and temporary detention and settling basins should be used in accordance with local and state ordinances.

7.3 EXCAVATION

7.3.1 General

The contractor should be aware of, and become familiar with, applicable local, state, and federal safety regulations, including current OSHA excavation and trench safety standards. Excavations shall be completed in conformance with the relevant sections of the OSSC, including, but not limited to, OSSC 00330 – Earthwork, OSSC 00400 – Drainage and Sewers, and OSSC 00500 – Bridges. The information provided below is general in nature and should not be relied upon by the contractor during construction without their own evaluation of excavation stability.

7.3.2 Trench Cuts and Temporary Shoring

Temporary cuts will be required for the project. Excavations into the slopes need to be carefully planned so as not to destabilize the slope. Cuts less than 4 feet should stand vertical provided groundwater is not present. Deeper excavations should initially be cut back at an inclination 1.5H:1V. Depending on the depth of cuts and the soil/rock encountered, excavation inclinations may be able to be steepened to 1.25H:1V, 1H:1V, or greater. Washington County and GeoDesign should be contacted at the time of excavation to observe the soil and rock conditions to determine if the steeper slope excavations are feasible.

If shoring is used, we recommend that the type and design of the shoring system be the responsibility of the contractor, who is in the best position to choose a system that fits the construction plan.

7.3.3 Dewatering

If groundwater is present in the base of any excavations, we recommend over-excavation by 18 to 24 inches and placing stabilization material in the base. Stabilization material should meet the requirements in the "Materials" section. The material should be free of organic matter and other deleterious material. Stabilization material should be placed in one lift and compacted until well keyed. Once the stabilization rock is in place, a submersible pump can be used to expel water from the base of the excavation.

7.4 MATERIALS

7.4.1 General

A variety of material may be used as structural fill at the site. Fill should only be placed over subgrade that has been prepared in conformance with the "Site Preparation" section. Structural fill should meet the specifications provided in the OSSC 00330 – Earthwork, OSSC 00400 – Drainage and Sewers, and OSSC 02600 – Aggregates, depending on the application. A brief characterization of some of the acceptable materials and our recommendations for their use as structural fill is provided below.

7.4.2 On-Site Soil

The material at the site should be suitable for use as general embankment fill, provided it is properly moisture conditioned, free of organic material and particles over 6 inches in diameter, and meets the specifications provided in OSSC 00330.12 – Borrow Material. Laboratory testing indicates the moisture content of the on-site soil is higher than the optimum moisture required for compaction as structural fill. Therefore, moderate to extensive moisture conditioning (i.e., drying) of the on-site soil will be required in order to achieve proper compaction; therefore, the use of on-site soil for structural fill will be restricted to periods of prolonged dry weather.

When used as structural fill, native soil should be placed in lifts with a maximum uncompacted thickness of 6 to 8 inches and compacted to not less than 95 percent of the maximum dry density, as determined by AASHTO T 99.

7.4.3 Imported Granular Material

Imported granular material used as structural fill should be pit- or quarry-run rock, crushed rock, or crushed gravel and sand and should meet the specifications provided in OSSC 00330.14 – Selected Granular Backfill or OSSC 00330.15 – Selected Stone Backfill. The imported granular material should also be angular, should be fairly well graded between coarse and fine material, should have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve, and should have at least two mechanically fractured faces.

Imported granular material should be placed in lifts with a maximum uncompacted thickness of 8 inches and compacted to not less than 95 percent of the maximum dry density, as determined by AASHTO T 99. During the wet season or when wet subgrade conditions exist, the initial lift should be approximately 12 to 18 inches in uncompacted thickness and should be compacted by rolling with a smooth-drum roller without using vibratory action.

Where the imported granular material is placed atop a fine-grained subgrade, a geotextile should be placed as a barrier between the native soil subgrade and the imported granular material. The placement of the imported granular fill should be done in conformance with the specifications provided in OSSC 00331 – Subgrade Stabilization. The geotextile should meet the specifications provided in OSSC 02320.20 – Geotextile Property Values for soil separation. The geotextile should be installed in conformance with OSSC 00350 – Geosynthetic Installation.

7.4.4 Stabilization Material

Stabilization material should consist of pit- or quarry-run rock, crushed rock, or crushed gravel and should meet the specifications provided in OSSC 00330.16 – Stone Embankment Material. In addition, the material should have a maximum particle size of 6 inches, should have less than 5 percent by dry weight passing the U.S. Standard No. 4 sieve, and should have at least two mechanically fractured faces. The material should be free of organic matter and other deleterious material. Stabilization material should be placed in lifts between 12 and 18 inches thick and compacted to a firm condition.

Where the stabilization material is used to stabilize soft subgrade beneath pavement or construction haul roads, a geotextile should be placed as a barrier between the soil subgrade and the imported granular material. The placement of the imported granular fill should be done in conformance with the specifications provided in OSSC 00331 – Subgrade Stabilization. The geotextile should meet the specifications provided in OSSC 02320.20 – Geotextile Property Values for soil separation. The geotextile should be installed in conformance with OSSC 00350 – Geosynthetic Installation. Geotextile is not required where stabilization material is used at the base of utility trenches.

7.4.5 Granular Wall Backfill

Granular wall backfill placed behind the conventional segmental retaining walls should consist of select granular material that meets the specifications provided in OSSC 02630.10 – Dense-Graded Aggregate ¾-inch to 1½-inch minus material. The material should be placed and compacted to a minimum specified compaction level of 95 percent of its maximum dry density, as determined by AASHTO T 99.

We recommend the granular wall backfill be separated from general fill, native soil, and/or topsoil using a Type 1 drainage geotextile that meets the specifications provided in OSSC Table 02320-1 – Geotextile Property Values. The geotextile should be installed in conformance with OSSC 00350 – Geosynthetic Installation.

7.4.6 Drain Rock Material

Drain rock should consist of angular, granular material that meets the specifications provided in OSSC 00430.11 – Granular Drain Backfill Material. The drain rock should be wrapped in a Type 1 drainage geotextile that meets the specifications provided in OSSC Table 02320-1 – Geotextile Property Values. The geotextile should have at least two mechanically fractured faces and be installed in conformance with OSSC 00350 – Geosynthetic Installation.

7.4.7 Retaining Wall Leveling Pad

Imported granular material placed at the base of retaining wall footings should consist of select granular material that meets the specifications provided in OSSC 00510.13 – Granular Structure Backfill. The granular material should meet either the 1"-0 or ¾"-0 aggregate size listed in OSSC Table 02630-1 – Grading Requirements for Dense-Graded Aggregate and have at least two mechanically fractured faces. The leveling pad material should be placed in a 6- to 12-inch lift and compacted to not less than 95 percent of the maximum dry density, as determined by AASHTO T 99.

7.4.8 Trench Backfill

Trench backfill placed beneath, adjacent to, and for at least 12 inches above utility lines (i.e., the pipe zone) should consist of well-graded granular material with a maximum particle size of 1 inch and less than 10 percent by dry weight passing the U.S. Standard No. 200 sieve and meet the specifications provided in OSSC 00405.13 – Pipe Zone Material. The pipe zone backfill should be compacted to at least 90 percent of the maximum dry density, as determined by AASHTO T 99, or as required by the pipe manufacturer or local agency.

Within roadway alignments, the remainder of the trench backfill up to the subgrade elevation should consist of well-graded granular material with a maximum particle size of 2½ inches and less than 7 percent by dry weight passing the U.S. Standard No. 200 sieve and should meet OSSC 00405.14 – Trench Backfill, Class B, C, or D. This material should be compacted to at least 90 percent of the maximum dry density, as determined by AASHTO T 99, or as required by the pipe manufacturer or local agency. The upper 3 feet of the trench backfill should be compacted to at least 95 percent of the maximum dry density, as determined by AASHTO T 99.

7.4.9 Existing AC Pavement, Concrete, and Aggregate Base

AC pavement, concrete, and aggregate base from the existing roadways can be used in general structural fills provided particles greater than 6 inches are not present and it is thoroughly mixed with soil so that there are no voids between the fragments. This material should only be used at depths greater than 3 feet below the finished subgrade in general fill areas and at least 3 feet above the pipe zone in trenches. The recycled materials should meet the specifications provided in OSSC 00330.12 – Borrow Material and other appropriate specifications.

7.4.10 Soil Amendment with Cement

As an alternative to the use of imported granular material or as an alternative to scarification and compaction during wet periods, an experienced contractor may be able to amend the on-site silt soil with portland cement to obtain suitable support properties. It is generally less costly to amend on-site soil than to remove and replace soft soil with granular material. Based on the moisture contents, soil types, and processing speed, cement amendment would be more suitable at this site than lime amendment.

The permeability of cement-amended soil is extremely low and should not be completed in landscape areas or the cement-amended material should be removed from landscape areas prior to planting. We should be contacted if this approach is considered. Cement amendment should not be used if runoff during construction cannot be directed away from adjacent wetlands, if present.

We recommend a target strength for cement-amended soil of 100 psi. The amount of cement used to achieve this target generally varies with moisture content and soil type. It is difficult to predict field performance of soil to cement amendment due to variability in soil response, and we recommend laboratory testing to confirm expectations. However, for preliminary design purposes, 5 percent cement by weight of dry soil can generally be used when the soil moisture content does not exceed approximately 20 percent. If the soil moisture content is in the range of 25 to 35 percent, 5 to 7 percent by weight of dry soil is recommended. The amount of cement added to the soil may need to be adjusted based on field observations and performance. Moreover, depending on the time of year and moisture content levels during amendment, water may need to be applied during tilling to appropriately condition the soil moisture content. The amount of cement used during treatment should be based on an assumed soil dry unit weight of 100 pcf. We recommend a baseline cement content of 7 percent for bidding.

In order to use wet on-site soil that would not otherwise be suitable for structural fill, it may be amended and placed as fill over a subgrade prepared in conformance with the “Site Preparation” section. Typically, a minimum curing of four days is required between treatment and construction traffic access. Consecutive lifts of fill may be treated immediately after the previous lift has been amended and compacted (e.g., the four-day wait period does not apply). However, where the final lift of fill is a building or roadway subgrade, the four-day wait curing period is in effect.

The amended surface should be protected from abrasion by placing a minimum 4-inch thickness of crushed rock. When used for staging pads and haul roads, the crushed rock may become contaminated with soil during construction. Contaminated base rock should be removed and replaced with clean rock in pavement areas such that the minimum thickness of free-draining base at the surface is 3 inches.

It is not possible to amend soil during heavy or continuous rainfall. Work should be completed during suitable conditions. To prevent strength loss during curing, cement-amended soil should be allowed to cure for a minimum of four days prior to access by construction traffic.

8.0 OBSERVATION OF CONSTRUCTION

Satisfactory earthwork and foundation performance depends to a large degree on the quality of construction. Subsurface conditions observed during construction should be compared with those encountered during the subsurface explorations. Recognition of changed conditions often requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated. In addition, sufficient monitoring of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. Construction observation and testing should be provided in accordance with the 2018 version of OSSC.

We recommend that GeoDesign be part of the team participating in observing earthwork and foundation installation activities. This observational work will include stripping, proof rolling of the subgrade and repair of soft areas, footing subgrade preparation, drilled piles installation, performing laboratory compaction and field moisture-density tests, observation of wall subgrade preparation and geogrid placement, and other appropriate activities.

9.0 LIMITATIONS

We have prepared this report for use by Washington County, WHPacific, and the members of their design team for the proposed project. The data and report can be used for bidding or estimating purposes, but our report, conclusions, and interpretations should not be construed as warranty of the subsurface conditions and are not applicable to other sites.

Exploration observations indicate soil conditions only at specific locations and only to the depths penetrated. They do not necessarily reflect soil strata or water level variations that may exist between exploration locations. If subsurface conditions differing from those described are noted during the course of excavation and construction, re-evaluation will be necessary. If different conditions are encountered during construction, some slight modifications of our recommendations may be required. Alternatively, GeoDesign can perform additional explorations and analysis prior to construction in order to refine our model of soil conditions.

The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design. Within the limitations of scope, schedule, and budget, our services have been executed in accordance with generally accepted practices in this area at the time the report was prepared. No warranty, express or implied, should be understood.

♦ ♦ ♦

We appreciate the opportunity to be of service to you. Please call if you have questions concerning this report or if we can provide additional services.

Sincerely,

GeoDesign, Inc.

Nick Paveglio, P.E.
Senior Associate Engineer

Jeffery D. Tucker, P.E., G.E.
Principal Engineer

REFERENCES

AASHTO, 2017. *AASHTO LRFD Bridge Design Specifications,* Eighth Edition with 2018 interim revisions, American Association of State Highway and Transportation Officials.

Madin, I. P., 1990. *Earthquake-Hazard Geology Maps of the Portland Metropolitan Area, Oregon: Text and Map Explanation.* Oregon Department of Geology and Mineral Industries, Open-File Report O-90-2, 21 pp., 8 maps, scale 1:24,000.

ODOT, 2016. *2016 ODOT Seismic Design Maps, 500 Year Peak Ground Acceleration.*

ODOT, 2016. *2016 ODOT Seismic Design Maps, 500 Year 0.2 SEC. Spectral Acceleration.*

ODOT, 2016. *2016 ODOT Seismic Design Maps, 500 Year 1.0 SEC. Spectral Acceleration.*

ODOT, 2016. *2016 ODOT Seismic Design Maps, 1,000 Year Peak Ground Acceleration.*

ODOT, 2016. *2016 ODOT Seismic Design Maps, ,1000 Year 0.2 SEC. Spectral Acceleration.*

ODOT, 2016. *2016 ODOT Seismic Design Maps, 1,000 Year 1.0 SEC. Spectral Acceleration.*

ODOT, 2016. *2016 ODOT Seismic Design Maps, CSZE Peak Ground Acceleration.*

ODOT, 2016. *2016 ODOT Seismic Design Maps, CSZE 0.2 SEC. Spectral Acceleration.*

ODOT, 2016. *2016 ODOT Seismic Design Maps, CSZE 1.0 SEC. Spectral Acceleration.*

ODOT, 2018. *Geotechnical Design Manual.* June 2018 Edition.

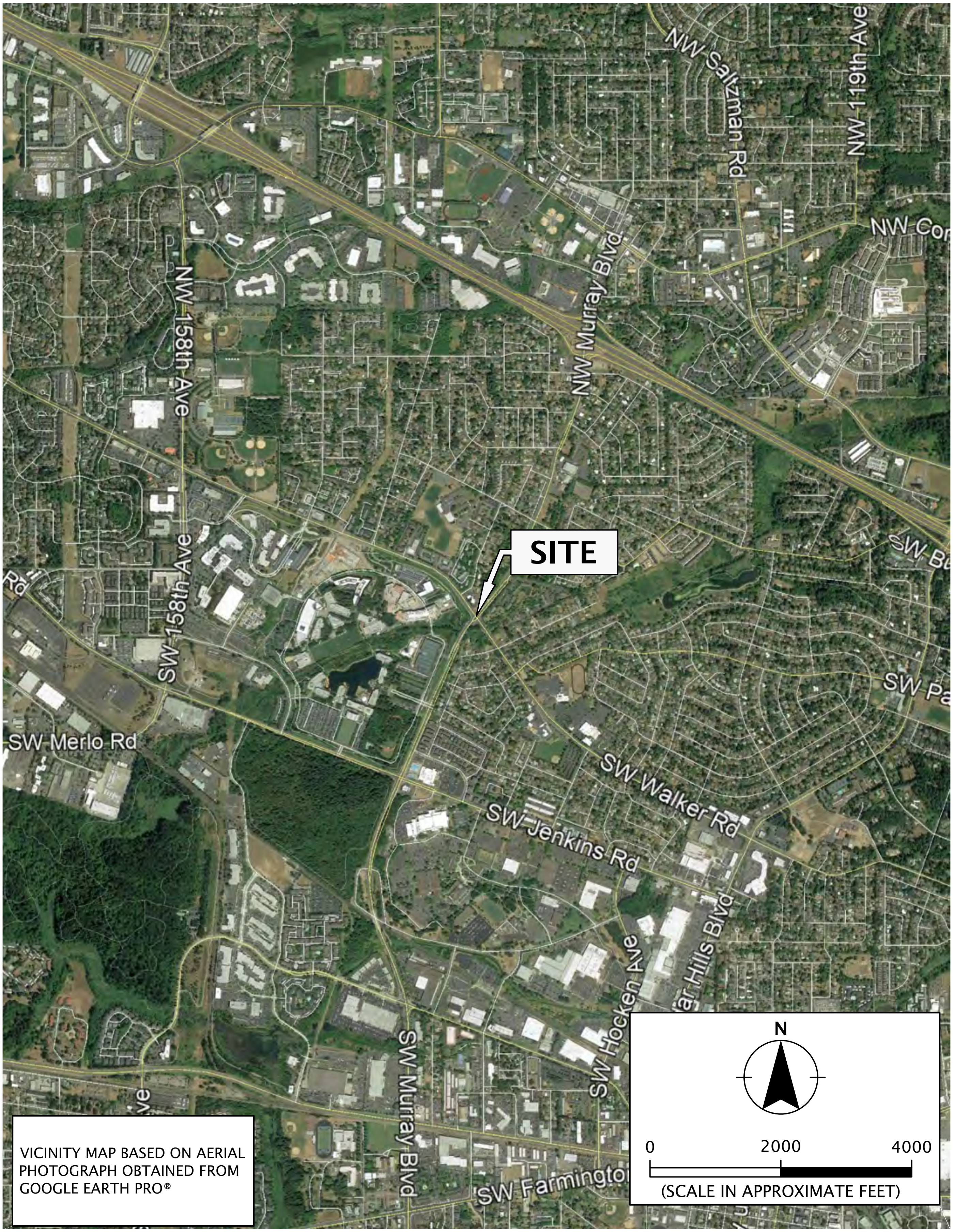
Orr, E. L. and Orr, W. N., 1999. *Geology of Oregon.* Iowa: Kendall/Hunt Publishing, 254 pp.

Somerville, P. G., et al., 1997. Modification of Empirical Strong Ground Motion Attenuation Relations to Include the Amplitude and Duration Effects of Rupture Directivity, *Seismological Research Letters*, Volume 68, Number 1, p. 199.

USGS, 2017. National Seismic Hazard Maps. Retrieved from http://geohazards.usgs.gov

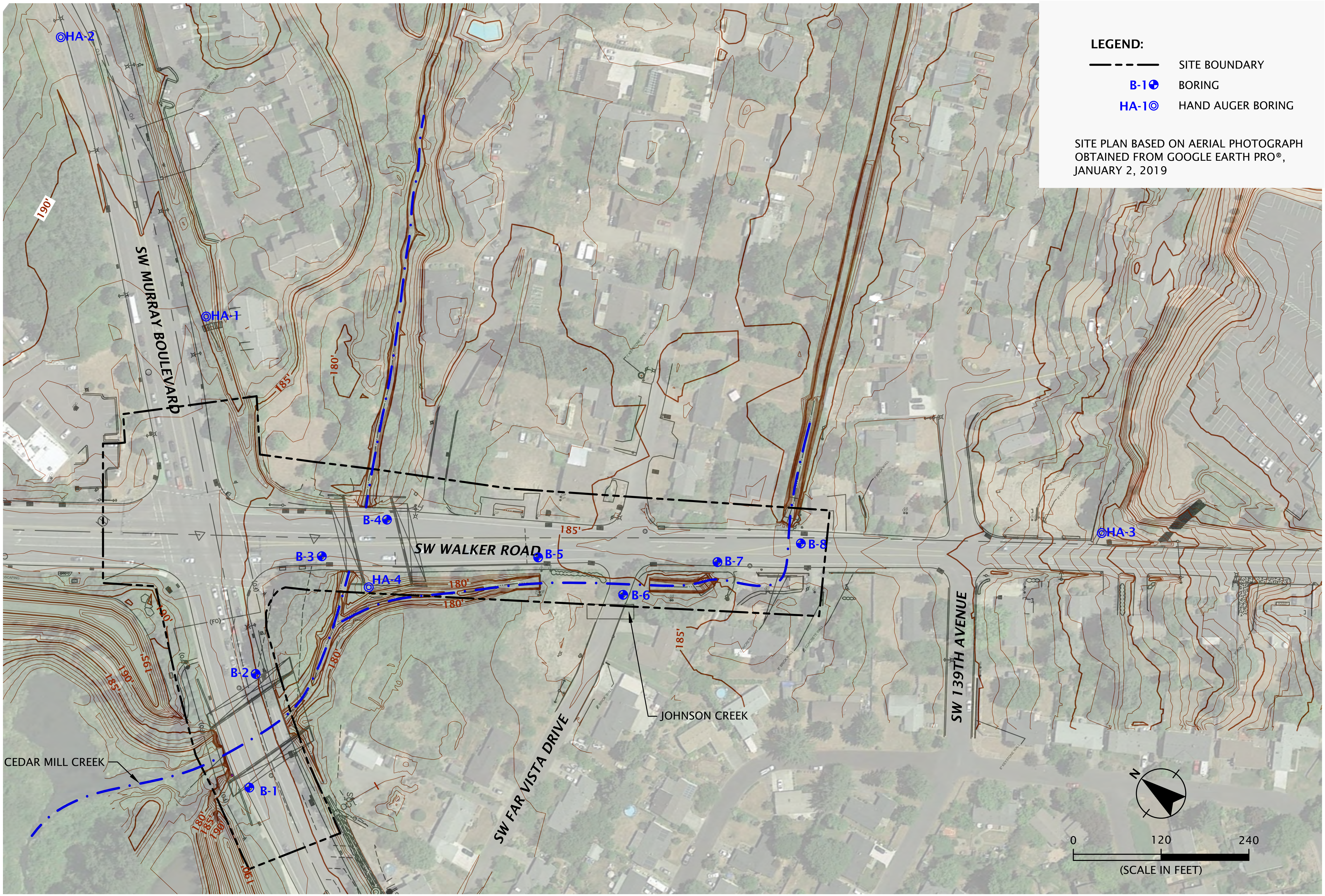
Weaver, C. S. and Shedlock, K. M., 1991. Program for Earthquake Hazards Assessment in the Pacific Northwest: U.S. Geological Survey Circular 1067, 29 pp.

FIGURES



VICINITY MAP BASED ON AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH PRO®

GEODESIGN INC AN NV5 COMPANY	WASHCO-56-03	VICINITY MAP	
	JUNE 2020	WALKER/MURRAY IMPROVEMENTS WASHINGTON COUNTY, OR	FIGURE 1



SITE PLAN

FIGURE 2

WALKER/MURRAY IMPROVEMENTS
WASHINGTON COUNTY, OR

WASHCO-56-03

JUNE 2020

GEODESIGN
AN NV5 COMPANY

Printed By: mmiller | Print Date: 6/17/2020 11:22:45 AM
File Name: J:\S-Z\WashCo\WashCo-56\WashCo-56-03\Figures\CAD\WashCo-56-03-SP03.dwg | Layout: FIGURE 2

APPENDIX A

APPENDIX A

FIELD EXPLORATIONS

GENERAL

Subsurface conditions at the site were explored by drilling eight borings (B-1 through B-8) at the proposed bridge abutments and culverts. The borings were drilled to depths between 26.5 and 121.5 feet BGS. Four hand auger borings were also advanced to depths between 4 and 6 feet BGS to perform infiltration tests and collect samples for particle-size analysis testing. The exploration logs are presented in this appendix.

Borings B-1 through B-88 were drilled using mud rotary methods by Western States Soil Conservation, Inc. of Hubbard, Oregon, in March 2014. The borings were drilled with a conventional rubber tire, truck-mounted drill rig under the supervision of GeoDesign personnel. We chose the locations of the explorations based on information provided by the design team.

SOIL SAMPLING

We collected representative samples of the various soils encountered during drilling for geotechnical laboratory testing. Samples were collected from the borings using 1½-inch-inner diameter, split-spoon SPT sampler in general accordance with ASTM D1586. The samplers were driven into the soil with a 140-pound automatic trip hammer free-falling 30 inches. The sampler was driven a total distance of 18 inches. The number of blows required to drive the sampler the final 12 inches is recorded on the exploration logs, unless otherwise noted. Higher quality, relatively undisturbed samples were collected using a standard Shelby tube in general accordance with ASTM D1587.

Disturbed soil samples were collected from the tip of the hand auger bit. Sample intervals are shown on the exploration logs.

Sampling methods and intervals are shown on the exploration logs.

SOIL CLASSIFICATION

The soil samples were classified in the field in accordance with the "Exploration Key" (Table A-1) and "Soil Classification System" (Table A-2), which are presented in this appendix. The exploration logs indicate the depths at which the soil characteristics change, although the change actually could be gradual. If the change occurred between sample locations, the depth was interpreted. Classifications are shown on the exploration logs.

LABORATORY TESTING

Laboratory tests were conducted on selected soil samples to confirm field classifications and determine the index engineering properties and strength characteristics. Locations of the tested samples are shown on the exploration logs. Descriptions of the tests and results of the testing completed are presented below.

ATTERBERG LIMITS TESTING

The plastic limit and liquid limit (Atterberg limits) of select soil samples were determined in accordance with ASTM D4318. The Atterberg limits and the plasticity index were completed to aid in the classification of the soil and evaluation of liquefaction susceptibility. The plastic limit is defined as the moisture content (in percent) where the soil becomes brittle. The liquid limit is defined as the moisture content where the soil begins to act similar to a liquid. The plasticity index is the difference between the liquid and plastic limits. The test results are presented in this appendix.

GRAIN-SIZE TESTING

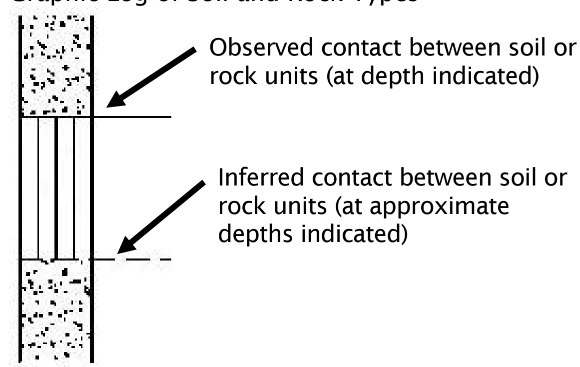
We completed grain-size testing on select soil samples in order to determine the distribution of soil particle sizes. The testing included sieve analyses in general accordance with ASTM C136 and percent fines determinations (percent passing the U.S. Standard No. 200 Sieve) in general accordance with ASTM C117. Hydrometer testing was also conducted in general accordance with ASTM D422. The test results are presented in this appendix.

MOISTURE CONTENT

We tested the natural moisture content of select soil samples in general accordance with ASTM D2216. The test results are presented in this appendix.

SYMBOL	SAMPLING DESCRIPTION
	Location of sample collected in general accordance with ASTM D1586 using Standard Penetration Test with recovery
	Location of sample collected using thin-wall Shelby tube or Geoprobe® sampler in general accordance with ASTM D1587 with recovery
	Location of sample collected using Dames & Moore sampler and 300-pound hammer or pushed with recovery
	Location of sample collected using Dames & Moore sampler and 140-pound hammer or pushed with recovery
	Location of sample collected using 3-inch-O.D. California split-spoon sampler and 140-pound hammer with recovery
	Location of grab sample
	Rock coring interval
	Water level during drilling
	Water level taken on date shown

Graphic Log of Soil and Rock Types



GEOTECHNICAL TESTING EXPLANATIONS

ATT	Atterberg Limits	P	Pushed Sample
CBR	California Bearing Ratio	PP	Pocket Penetrometer
CON	Consolidation	P200	Percent Passing U.S. Standard No. 200 Sieve
DD	Dry Density		
DS	Direct Shear	RES	Resilient Modulus
HYD	Hydrometer Gradation	SIEV	Sieve Gradation
MC	Moisture Content	TOR	Torvane
MD	Moisture-Density Relationship	UC	Unconfined Compressive Strength
NP	Non-Plastic	VS	Vane Shear
OC	Organic Content	kPa	Kilopascal

ENVIRONMENTAL TESTING EXPLANATIONS

CA	Sample Submitted for Chemical Analysis	ND	Not Detected
P	Pushed Sample	NS	No Visible Sheen
PID	Photoionization Detector Headspace Analysis	SS	Slight Sheen
		MS	Moderate Sheen
ppm	Parts per Million	HS	Heavy Sheen

GEODESIGN INC AN NV5 COMPANY	EXPLORATION KEY	TABLE A-1

RELATIVE DENSITY - COARSE-GRAINED SOIL

Relative Density	Standard Penetration Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)
Very Loose	0 – 4	0 – 11	0 – 4
Loose	4 – 10	11 – 26	4 – 10
Medium Dense	10 – 30	26 – 74	10 – 30
Dense	30 – 50	74 – 120	30 – 47
Very Dense	More than 50	More than 120	More than 47

CONSISTENCY - FINE-GRAINED SOIL

Consistency	Standard Penetration Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)	Unconfined Compressive Strength (tsf)
Very Soft	Less than 2	Less than 3	Less than 2	Less than 0.25
Soft	2 – 4	3 – 6	2 – 5	0.25 – 0.50
Medium Stiff	4 – 8	6 – 12	5 – 9	0.50 – 1.0
Stiff	8 – 15	12 – 25	9 – 19	1.0 – 2.0
Very Stiff	15 – 30	25 – 65	19 – 31	2.0 – 4.0
Hard	More than 30	More than 65	More than 31	More than 4.0

PRIMARY SOIL DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE-GRAINED SOIL (more than 50% retained on No. 200 sieve)	GRAVEL (more than 50% of coarse fraction retained on No. 4 sieve)	CLEAN GRAVEL (< 5% fines)	GW or GP	GRAVEL
		GRAVEL WITH FINES (≥ 5% and ≤ 12% fines)	GW-GM or GP-GM	GRAVEL with silt
			GW-GC or GP-GC	GRAVEL with clay
		GRAVEL WITH FINES (> 12% fines)	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND (50% or more of coarse fraction passing No. 4 sieve)	CLEAN SAND (<5% fines)	SW or SP	SAND
		SAND WITH FINES (≥ 5% and ≤ 12% fines)	SW-SM or SP-SM	SAND with silt
			SW-SC or SP-SC	SAND with clay
		SAND WITH FINES (> 12% fines)	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOIL (50% or more passing No. 200 sieve)	SILT AND CLAY	Liquid limit less than 50	ML	SILT
			CL	CLAY
			CL-ML	silty CLAY
			OL	ORGANIC SILT or ORGANIC CLAY
		Liquid limit 50 or greater	MH	SILT
			CH	CLAY
			OH	ORGANIC SILT or ORGANIC CLAY
HIGHLY ORGANIC SOIL			PT	PEAT

MOISTURE CLASSIFICATION

Term	Field Test
dry	very low moisture, dry to touch
moist	damp, without visible moisture
wet	visible free water, usually saturated

ADDITIONAL CONSTITUENTS

Secondary granular components or other materials such as organics, man-made debris, etc.

Percent	Silt and Clay In: Fine-Grained Soil	Silt and Clay In: Coarse-Grained Soil	Percent	Sand and Gravel In: Fine-Grained Soil	Sand and Gravel In: Coarse-Grained Soil
< 5	trace	trace	< 5	trace	trace
5 – 12	minor	with	5 – 15	minor	minor
> 12	some	silty/clayey	15 – 30	with	with
			> 30	sandy/gravelly	Indicate %

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0	ASPHALT CONCRETE (7.0 inches).	187.5 / 186.9 / 0.6			
	AGGREGATE BASE (7.0 inches).	186.3 / 1.2			
	Very stiff, light brown-brown SILT (ML), some clay, minor sand; moist, sand is fine - **FILL**.			24	
5	stiff to very stiff, brown-orange, trace organics (woody debris); organics are mm scale at 5.0 feet			15	
	Stiff, gray CLAY with sand (CL), trace organics (woody debris); moist, sand is fine, organics are up to 1/4-inch diameter (alluvium).	179.5 / 8.0	ATT	11	LL = 41% PL = 23%
10	Soft to medium stiff, gray CLAY (CL), some silt, minor sand, trace organics (woody debris); moist, sand is fine, organics are mm scale.	177.5 / 10.0		4	
				P	
15	Soft, brown with orange-gray mottled SILT (ML), minor clay, trace sand; moist, sand is fine.	173.0 / 14.5	P200	3	P200 = 89%
20	soft to medium stiff, light brown, without clay and organics; wet at 20.0 feet			4	
25	Stiff, gray to dark gray SILT with sand (ML); moist, stratified beds of wet, silty sand (up to 6 inches thick) and moist silt and trace fine sand.	162.5 / 25.0		9	
				P	
30	grades to medium stiff at 30.0 feet		P200	7	P200 = 97%
35	soft, trace sand and organics (woody debris); wet, sand is fine, organics are <1/8-inch diameter at 35.0 feet			2	
				P	
40					

▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (scale 0 – 50 – 100)

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: JGH COMPLETED: 03/11/14

BORING METHOD: mud rotary (see report text) BORING BIT DIAMETER: 3 7/8-inch/4 7/8-inch

BORING LOG - GDI-NV5 - 1 PER PAGE WASHCO-56-03-B1_8-HA1_4.GPJ GDI_NV5.GDT PRINT DATE: 6/17/20:KM:KT

GEODESIGN INC AN NV5 COMPANY	WASHCO-56-03	**BORING B-1**	
	JUNE 2020	WALKER/MURRAY IMPROVEMENTS WASHINGTON COUNTY, OR	**FIGURE A-1**

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
40	(continued from previous page)			21	
45	Stiff, gray SILT (ML), minor sand, trace organics (woody debris); moist to wet, sand is fine, organics are <1/8-inch diameter, laminated to stratified beds of silty sand.	142.5 / 45.0		12	
50	medium stiff to stiff, trace sand; moist at 50.0 feet			8	
55	very stiff, with sand, without organics; moist to wet, stratified beds of wet, silty sand at 55.0 feet			28	
60	Very stiff, gray CLAY (CL), some silt, minor sand; moist, sand is fine to coarse (Sandy River Mudstone equivalent).	127.5 / 60.0	ATT	19	LL = 29% PL = 18%
65	Stiff, light brown-light gray CLAY (CH), trace silt; moist.	122.5 / 65.0		13	
70	very stiff, gray-light gray at 70.0 feet			16	
75	stiff, light brown-gray at 75.0 feet			13	
80					

▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (scale 0 – 50 – 100)

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: JGH COMPLETED: 03/11/14

BORING METHOD: mud rotary (see report text) BORING BIT DIAMETER: 3 7/8-inch/4 7/8-inch

GEODESIGN INC — AN NV5 COMPANY

WASHCO-56-03

JUNE 2020

BORING B-1
(continued)

WALKER/MURRAY IMPROVEMENTS
WASHINGTON COUNTY, OR

FIGURE A-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	BLOW COUNT / MOISTURE CONTENT %	INSTALLATION AND COMMENTS
80	Very stiff, brown to light gray SILT (ML), some clay, trace sand; moist, sand is fine.	107.5 / 80.0			17	
	Very stiff, light gray CLAY (CH); moist.	106.5 / 81.0				
90	stiff at 90.0 feet				9	
100	grades to very stiff, gray, some silt at 100.0 feet				21	
110	Very stiff, brown-light gray CLAY (CL), some silt; moist.	77.5 / 110.0			25	
	Exploration completed at a depth of 111.5 feet.	76.0 / 111.5				

Legend: ▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC%

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: JGH COMPLETED: 03/11/14

BORING METHOD: mud rotary (see report text) BORING BIT DIAMETER: 3 7/8-inch/4 7/8-inch

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0	ASPHALT CONCRETE (11.8 inches).	185.0			
	AGGREGATE BASE (12.3 inches).	184.0 / 1.0			
	Very dense, gray GRAVEL with silt and sand (GW-GM); moist - **FILL**.	183.0 / 2.0		54	
	Medium dense gray-brown, silty GRAVEL (GM), trace organics (woody debris) and sand; moist - **FILL**.	181.0 / 4.0			
5				4	
	Soft to medium stiff, gray CLAY (CL), trace organics (roots); moist, organics are <1/4-inch diameter (alluvium).	179.5 / 5.5			
10	brown-light gray, minor sand, without clay; moist to wet, sand is fine at 10.0 feet		ATT	5	LL = 40% PL = 23%
15	with sand, without organics; wet at 15.0 feet			5	
20	stiff; stratified beds of silty sand at 20.0 feet			13	
25	gray-dark gray, minor sand; stratified beds of silty sand at 25.0 feet			12	
30	grades to medium stiff, with sand; stratified bed of silty sand (up to 7 inches thick) at 30.0 feet			6	
35	stiff to very stiff, minor sand at 35.0 feet			15	
40					

▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (scale 0 – 50 – 100)

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: JGH COMPLETED: 03/13/14

BORING METHOD: mud rotary (see report text) BORING BIT DIAMETER: 4 7/8-inch

GEODESIGN INC — AN NV5 COMPANY

WASHCO-56-03	**BORING B-2**	
JUNE 2020	WALKER/MURRAY IMPROVEMENTS WASHINGTON COUNTY, OR	**FIGURE A-2**

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
40	stiff; moist to wet at 40.0 feet			12	
45	medium stiff to stiff, trace sand; homogeneous at 45.0 feet			8	
50	stiff; wet at 50.0 feet			10	
55	Very stiff, gray-dark gray SILT with sand (ML); wet.	130.0 / 55.0	P200	25	P200 = 83%
60	Very stiff, gray CLAY (CH); moist (Sandy River Mudstone equivalent).	125.0 / 60.0		20	
65	light gray-brown, some silt at 65.0 feet			26	
70				16	
75					
80					

▲ BLOW COUNT
● MOISTURE CONTENT %
RQD% CORE REC%
0 50 100

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: JGH COMPLETED: 03/13/14

BORING METHOD: mud rotary (see report text) BORING BIT DIAMETER: 4 7/8-inch

GEODESIGN INC
AN NV5 COMPANY

WASHCO-56-03

JUNE 2020

BORING B-2
(continued)

WALKER/MURRAY IMPROVEMENTS
WASHINGTON COUNTY, OR

FIGURE A-2

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 – 50 – 100)	INSTALLATION AND COMMENTS
80		stiff, light gray to light brown at 80.0 feet				▲ 12 ●	
85							Driller Comment: a little softer at approx. 87.0 feet.
90		very stiff, gray, without silt at 90.0 feet				▲ 19	
		Very stiff, gray CLAY (CL); moist.	94.0 / 91.0				
95							
100		Very stiff, gray CLAY (CH); moist.	85.0 / 100.0			▲ 28 ●	
105							
110		light gray-orange at 110.0 feet				▲ 24	
115							
120							

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: JGH COMPLETED: 03/13/14

BORING METHOD: mud rotary (see report text) BORING BIT DIAMETER: 4 7/8-inch

GEODESIGN INC
AN NV5 COMPANY

WASHCO-56-03

JUNE 2020

BORING B-2
(continued)

WALKER/MURRAY IMPROVEMENTS
WASHINGTON COUNTY, OR

FIGURE A-2

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 50 100)	INSTALLATION AND COMMENTS
120		(continued from previous page)				59	
		Exploration completed at a depth of 121.5 feet.	63.5 / 121.5				
125							
130							
135							
140							
145							
150							
155							
160							

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: JGH COMPLETED: 03/13/14

BORING METHOD: mud rotary (see report text) BORING BIT DIAMETER: 4 7/8-inch

GEODESIGN INC
AN NV5 COMPANY

WASHCO-56-03

JUNE 2020

BORING B-2
(continued)

WALKER/MURRAY IMPROVEMENTS
WASHINGTON COUNTY, OR

FIGURE A-2

BORING LOG - GDI-NV5 - 1 PER PAGE WASHCO-56-03-B1_8-HA1_4.GPJ GDI_NV5.GDT PRINT DATE: 6/17/20:KM:KT

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0	ASPHALT CONCRETE (14.0 inches).	184.0			
	AGGREGATE BASE (10.0 inches).	182.8 / 1.2			
	Medium dense, gray-brown GRAVEL with silt and sand (GW-GM); moist - **FILL**.	182.0 / 2.0		13	
5	loose at 5.0 feet			7	Driller Comment: still gravelly, caving at 5.0 feet. ▽ 6.0 feet, during drilling
	Medium stiff to stiff, brown-gray SILT (ML), some clay, trace to minor organics (woody debris); moist to wet.	177.0 / 7.0		8	Fairly thick (3-inch-long and 3/8-inch-diameter woody debris in cuttings.
10	stiff at 10.0 feet			11	Smooth to 10.0 feet with woody debris.
				10	
15	medium stiff, light brown, minor sand; sand is fine at 15.0 feet			6	
20	very stiff, brown at 20.0 feet gray at 21.0 feet		ATT	27	LL = 34% PL = 26%
25	Soft to medium stiff, gray-dark gray SILT (ML), minor sand; wet, stratified beds of silty sand.	159.0 / 25.0	SIEV	4	Switch to 3 7/8-inch drag bit at 25.0 feet. Possible flowing sand at 25.0 feet.
30	Stiff, dark gray SILT with sand to (ML); wet, sand is fine, stratified beds of silty sand (up to 4 inches thick).	154.0 / 30.0		9	
35	very stiff, minor sand at 35.0 feet			16	
40					

▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 – 50 – 100)

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: JGH COMPLETED: 03/17/14

BORING METHOD: mud rotary (see report text) BORING BIT DIAMETER: 4 7/8-inch/3 7/8-inch

GEODESIGN INC AN NV5 COMPANY

WASHCO-56-03

JUNE 2020

BORING B-3

WALKER/MURRAY IMPROVEMENTS
WASHINGTON COUNTY, OR

FIGURE A-3

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	BLOW COUNT
40	stiff, with sand; stratified beds of silty sand (2 to 4 inches thick) at 40.0 feet		14
45	very stiff; moist to wet, stratified beds of silty sand (1 to 7 inches thick) at 45.0 feet		20
50	medium stiff; wet, homogeneous at 50.0 feet		6
55	Very stiff, gray CLAY (CL), some silt, minor sand; moist, sand is fine (Sandy River Mudstone equivalent).	128.5 / 55.5	29
60	Very stiff, light gray CLAY (CH); moist.	124.0 / 60.0	26
	light brown, some silt at 61.0 feet		
65	stiff, gray to light brown at 65.0 feet		13
70	Very stiff, light brown CLAY (CL), some silt, trace sand; moist, sand is fine.	114.0 / 70.0	28
75	Very stiff, light gray CLAY (CH); moist.	109.0 / 75.0	21
	Very stiff, light brown SILT (ML), some clay; moist.	108.5 / 75.5	
80			

▲ BLOW COUNT
● MOISTURE CONTENT %
RQD% CORE REC%

TESTING | SAMPLE | INSTALLATION AND COMMENTS

DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: JGH
COMPLETED: 03/17/14

BORING METHOD: mud rotary (see report text)
BORING BIT DIAMETER: 4 7/8-inch/3 7/8-inch

GEODESIGN INC
AN NV5 COMPANY

WASHCO-56-03

JUNE 2020

BORING B-3
(continued)

WALKER/MURRAY IMPROVEMENTS
WASHINGTON COUNTY, OR

FIGURE A-3

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	BLOW COUNT / MOISTURE CONTENT % (RQD% / CORE REC%) 0–50–100	INSTALLATION AND COMMENTS
80	Very stiff, light gray-light brown CLAY (CL/CH), some silt, minor sand; moist, stratified beds of sand (up to 2 to 3 inches thick).	104.0 / 80.0			▲ 28	
85	Stiff, gray CLAY (CH); moist.	99.0 / 85.0			▲ 11 ●	
	Stiff, gray SILT (ML), some clay; moist.	98.5 / 85.5				Driller Comment: harder clay at 87.0 feet.
95	Very stiff, light gray-gray CLAY (CH); moist, homogeneous.	89.0 / 95.0			▲ 28 ●	
105	Medium dense, brown-orange, silty SAND (SM), minor clay; moist to wet, fine, laminated to stratified beds of silt and clay.	79.0 / 105.0			▲ 28	
115	Hard, brown-orange SILT with sand (ML), some clay; moist to wet, sand is fine, stratified beds of silty sand, clay, and silt.	69.0 / 115.0			▲ 39 ●	
120						

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: JGH COMPLETED: 03/17/14

BORING METHOD: mud rotary (see report text) BORING BIT DIAMETER: 4 7/8-inch/3 7/8-inch

GEODESIGN INC AN NV5 COMPANY

WASHCO-56-03

JUNE 2020

BORING B-3
(continued)

WALKER/MURRAY IMPROVEMENTS
WASHINGTON COUNTY, OR

FIGURE A-3

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 50 100)	INSTALLATION AND COMMENTS
120		stiff to very stiff at 120.0 feet	63.5 / 120.5			19	
		Very stiff, light gray CLAY (CH); moist.	62.5 / 121.5				
		Exploration completed at a depth of 121.5 feet.					
125							
130							
135							
140							
145							
150							
155							
160							

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: JGH COMPLETED: 03/17/14

BORING METHOD: mud rotary (see report text) BORING BIT DIAMETER: 4 7/8-inch/3 7/8-inch

GEODESIGN INC AN NV5 COMPANY

WASHCO-56-03

JUNE 2020

BORING B-3
(continued)

WALKER/MURRAY IMPROVEMENTS
WASHINGTON COUNTY, OR

FIGURE A-3

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	BLOW COUNT / MOISTURE CONTENT %	INSTALLATION AND COMMENTS
0	ASPHALT CONCRETE (possible asphalt concrete over portland cement concrete; 21.0 inches).	184.0			
	AGGREGATE BASE (3.0 inches).	182.2 / 1.8			
	Medium stiff, gray CLAY (CL/CH), some silt, minor gravel, trace organics (wood fragments); moist - **FILL**.	182.0 / 2.0		5	Drill rig chatter to 2.0 feet, then smooth.
5	soft at 5.0 feet			0-2-50/3"	
	Very dense, gray, silty GRAVEL with sand and portland cement concrete debris (GM); moist, angular - **FILL**.	178.0 / 6.0			Drill rig chatter at 6.0 feet. Large PPC debris at 6.0 feet, hole caving in, moved 2 feet wet and re-drilled hole. Switch to 4 7/8-inch bit at 7.0 feet. Smooth drilling at 7.0 feet. LL = 59% PL = 18%
	Soft, brown-gray CLAY (CH), trace sand and organics; moist.	177.0 / 7.0	ATT	3	
10				P	
	Soft to medium stiff, brown with orange mottled CLAY (CL), some silt, trace organics; moist.	172.5 / 11.5		4	
	Medium stiff, brown SILT (ML), trace sand; moist, sand is fine.	170.0 / 14.0			
15	gray, minor sand; wet at 15.5 feet with sand at 16.2 feet			7	15.5 feet, during drilling
20	very stiff, minor sand, trace clay and organics; moist at 20.0 feet interbeds of sandy silt at 21.0 feet			16	
25	medium stiff; moist to wet at 25.0 feet interbeds of silty sand (2 inches thick) at 26.0 feet			7	
30	medium stiff to stiff at 30.0 feet interbeds of silty sand at 31.0 feet			8	
35	very stiff, some clay, trace sand; moist at 35.0 feet			16	
40					

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: NAK/JGH COMPLETED: 03/19/14

BORING METHOD: mud rotary (see report text) BORING BIT DIAMETER: 6-inch/4 7/8-inch/3 7/8-inch

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
40	(continued from previous page)			16	
45	Stiff, gray CLAY (CL), some silt; moist.	139.0 / 45.0	ATT	10	LL = 32% PL = 21%
50	minor clay at 51.0 feet			13	
55	minor sand at 55.0 feet			23	
	Very stiff, gray-green CLAY (CH); moist (Sandy River Mudstone equivalent).	128.5 / 55.5			
60	stiff to very stiff, brown or gray with black streaks at 60.0 feet			15	
65	stiff, light gray-green at 65.0 feet			10	
	Very stiff, light brown SILT (ML), trace clay and sand; moist, sand is fine.	116.0 / 68.0			
70				29	
	Stiff, gray CLAY (CH), trace sand; moist, sand is fine.	111.0 / 73.0			
75				11	
80					

▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 – 50 – 100)

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: NAK/JGH COMPLETED: 03/19/14

BORING METHOD: mud rotary (see report text) BORING BIT DIAMETER: 6-inch/4 7/8-inch/3 7/8-inch

GEODESIGN INC AN NV5 COMPANY

WASHCO-56-03

JUNE 2020

BORING B-4
(continued)

WALKER/MURRAY IMPROVEMENTS
WASHINGTON COUNTY, OR

FIGURE A-4

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	BLOW COUNT
80	(continued from previous page) very stiff; interbeds of clayey sand (4 inches thick) at 80.5 feet		18
85	Hard, gray to light gray SILT (ML), minor sand; moist to wet, sand is fine (ash).	99.0 / 85.0	47
90	Hard, light gray-gray CLAY (CH); moist.	94.0 / 90.0	40
95	very stiff at 95.0 feet some silt, minor sand, trace organics (plant debris); sand is fine at 96.0 feet		21
100	Dense, light brown-orange, silty SAND (SM); wet, fine.	84.0 / 100.0	42
105	minor clay; moist to wet, stratified beds of moist silt, some clay (up to 4 inches thick) at 105.0 feet		48
110			44
115	Very stiff, light gray-gray CLAY (CL/CH); moist.	69.0 / 115.0	28
120			

▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (scale 0 – 50 – 100)

INSTALLATION AND COMMENTS

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: NAK/JGH COMPLETED: 03/19/14

BORING METHOD: mud rotary (see report text) BORING BIT DIAMETER: 6-inch/4 7/8-inch/3 7/8-inch

GEODESIGN INC, AN NV5 COMPANY	WASHCO-56-03	BORING B-4 (continued)	
	JUNE 2020	WALKER/MURRAY IMPROVEMENTS WASHINGTON COUNTY, OR	FIGURE A-4

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 50 100)	INSTALLATION AND COMMENTS
120		Hard, light gray CLAY with sand (CL), some silt; moist to wet, sand is fine.	64.0 / 120.0			32	
		Exploration completed at a depth of 121.5 feet.	62.5 / 121.5				
125							
130							
135							
140							
145							
150							
155							
160							

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: NAK/JGH COMPLETED: 03/19/14

BORING METHOD: mud rotary (see report text) BORING BIT DIAMETER: 6-inch/4 7/8-inch/3 7/8-inch

GEODESIGN INC AN NV5 COMPANY	WASHCO-56-03	**BORING B-4** (continued)	
	JUNE 2020	WALKER/MURRAY IMPROVEMENTS WASHINGTON COUNTY, OR	**FIGURE A-4**

DEPTH FEET	ELEVATION / DEPTH	MATERIAL DESCRIPTION	TESTING	SAMPLE	BLOW COUNT	INSTALLATION AND COMMENTS
0	185.5					
	184.8 / 0.7	ASPHALT CONCRETE (8.8 inches).				
	183.5 / 2.0	AGGREGATE BASE (15.3 inches).				
		Soft, brown SILT with gravel (ML), minor sand; moist - **FILL**.			2	
5	180.0 / 5.5				2	
		Soft, gray-dark gray SILT (ML), minor sand, trace organics (roots); moist (alluvium). medium stiff to stiff, light brown-light gray, some clay, trace sand, without organics at 7.5 feet			8	
10		medium stiff, gray-dark gray, minor sand, without clay; wet; sand is fine at 10.0 feet	ATT		7	LL = 34% PL = 27% 10.0 feet, during drilling
				P		
15		grades to soft, dark brown-gray, trace sand and organics (roots/woody debris); moist to wet at 15.0 feet			3	
20		very stiff, dark gray, with sand; wet, blocky mottled texture at 20.0 feet some clay at 21.0 feet				
25		stiff, gray-dark gray, trace sand and organics (woody debris); organics are <1/8-inch diameter at 25.0 feet			12	
30		grades to medium stiff at 30.0 feet			6	
	154.0 / 31.5	Exploration completed at a depth of 31.5 feet.				
35						
40						

▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC%

0 50 100

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: JGH COMPLETED: 03/20/14

BORING METHOD: mud rotary (see report text) BORING BIT DIAMETER: 4 7/8-inch

GEODESIGN INC — AN NV5 COMPANY

WASHCO-56-03

JUNE 2020

BORING B-5

WALKER/MURRAY IMPROVEMENTS
WASHINGTON COUNTY, OR

FIGURE A-5

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0	ASPHALT CONCRETE (4.8 inches).	185.0 / 184.6 / 0.4			
	AGGREGATE BASE (10.3 inches).	183.7 / 1.3			Driller Comment: smooth at 1.3 feet.
	Medium stiff, light brown with orange-gray mottled SILT (ML), minor sand, trace organics (carbonized wood); moist, sand is fine.			7	
5	grades to soft to medium stiff, without organics; moist to wet at 5.0 feet		P200	4	P200 = 89%
	Medium stiff to stiff, brown CLAY (CH); moist.	177.5 / 7.5		8	
10	Medium stiff, brown-light brown SILT (ML), some clay, trace sand and organics (roots); moist, sand is fine.	175.0 / 10.0		6	
15	gray-dark gray, with sand, without clay and organics; wet at 15.0 feet			6	
				P	
20	stiff at 20.0 feet			9	
25	minor sand at 25.0 feet			13	
	Exploration completed at a depth of 26.5 feet.	158.5 / 26.5			
30					
35					
40					

Legend: ▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (scale 0 – 50 – 100)

DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: JGH

COMPLETED: 03/20/14

BORING METHOD: mud rotary (see report text)

BORING BIT DIAMETER: 4 7/8-inch

GEODESIGN INC — AN NV5 COMPANY

WASHCO-56-03

JUNE 2020

BORING B-6

WALKER/MURRAY IMPROVEMENTS
WASHINGTON COUNTY, OR

FIGURE A-6

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0	ASPHALT CONCRETE (10.5 inches).	186.5			
	AGGREGATE BASE (11.5 inches).	185.6 / 0.9			
	Medium dense, brown-orange, silty GRAVEL with sand (GM), minor clay; moist - **FILL**.	184.7 / 1.8	SIEV	11	Driller Comment: gravelly to 2.5 feet.
5	grades to with debris (isolated metal fragment) at 5.0 feet			15	Driller Comment: caving at approx. 5.0 feet.
	loose at 7.5 feet			7	Crunchy to 7.5 feet.
10	medium dense at 10.0 feet			16	Woody debris, drilling to 10.0 feet, but gravelly.
					Driller Comment: crunchy at 12.5 feet.
	Medium stiff, light brown-orange SILT with sand (ML); wet, sand is fine (alluvium).	172.5 / 14.0			Driller Comment: smooth at 14.0 feet.
15				5	
	P				
20	gray to dark gray, minor sand; moist to wet, stratified beds of silty sand (up to 4 inches thick) at 20.0 feet			7	
25	grades to very stiff, with sand; wet at 25.0 feet			18	
	Exploration completed at a depth of 26.5 feet.	160.0 / 26.5			
30					
35					
40					

▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC%

0 50 100

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: JGH COMPLETED: 03/20/14

BORING METHOD: mud rotary (see report text) BORING BIT DIAMETER: 4 7/8-inch

GEODESIGN INC AN NV5 COMPANY

WASHCO-56-03

JUNE 2020

BORING B-7

WALKER/MURRAY IMPROVEMENTS
WASHINGTON COUNTY, OR

FIGURE A-7

BORING LOG - GDI-NV5 - 1 PER PAGE WASHCO-56-03-B1_8-HA1_4.GPJ GDI_NV5.GDT PRINT DATE: 6/17/20:KM:KT

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0	ASPHALT CONCRETE (10.3 inches).	187.0			
	AGGREGATE BASE (13.8 inches).	186.1 / 0.9			
	Soft to medium stiff, brown-orange SILT (ML), some clay, trace organics (woody debris); moist to wet, organics are <1/8-inch diameter, blocky mottled texture.	185.0 / 2.0		4	Gravelly cuttings to 2.0 feet Consistent shallow clay, perched "zone."
5	Soft to medium stiff, gray-light gray CLAY (CH), trace organics (rootlets); moist.	183.5 / 3.5		P	
	Medium stiff, light brown-light gray SILT (ML), trace sand; moist, sand is fine.	180.0 / 7.0	ATT	7	LL = 39% PL = 27%
10	minor sand; wet, stratified beds of silty sand (up to 3 inches thick) at 12.0 feet			P; 5	
15	grades to stiff, with sand at 16.0 feet			10	
20	Medium stiff, gray-dark gray SILT with sand to SILT (ML), minor sand, trace organics (woody debris); wet, sand is fine, organics are <1/8-inch diameter, stratified beds of silty sand.	167.0 / 20.0		7	
25	Soft to medium stiff, gray to dark gray SILT (ML), minor sand, trace organics (woody debris); wet, sand is fine, organics are <1/8-inch diameter.	162.0 / 25.0		4	
30	medium stiff at 30.0 feet			7	
	Exploration completed at a depth of 31.5 feet.	155.5 / 31.5			
35					
40					

▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 – 50 – 100)

DRILLED BY: Western States Soil Conservation, Inc.   LOGGED BY: JGH   COMPLETED: 03/21/14

BORING METHOD: mud rotary (see report text)   BORING BIT DIAMETER: 4 7/8-inch

GEODESIGN INC — AN NV5 COMPANY

WASHCO-56-03	**BORING B-8**	
JUNE 2020	WALKER/MURRAY IMPROVEMENTS WASHINGTON COUNTY, OR	**FIGURE A-8**

HA-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	COMMENTS
0.0	Soft to medium stiff, brown SILT (ML), trace organics; moist (topsoil to approximately 2 inches).			
	without organics at 2.0 feet			
2.5	dry to moist at 2.5 feet			
	medium stiff to stiff, gray-brown, trace sand; moist at 3.0 feet		PP	PP = 0.7 tsf
	soft to medium stiff, brown, without sand at 4.0 feet		PP	PP = 0.5 tsf
5.0	medium stiff to stiff at 5.0 feet		PP	PP = 1.25 tsf Infiltration test at 5.0 feet.
	Exploration completed at a depth of 6.0 feet.	6.0		Surface elevation was not measured at the time of exploration.
7.5				

HA-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	COMMENTS
0.0	Soft to medium stiff, brown SILT (ML); moist (topsoil to approximately 4 inches).			
	medium stiff to stiff at 2.0 feet			
2.5			PP	PP = 1.0 tsf
	medium stiff 4.5 feet		PP	Infiltration test at 4.6 feet. PP = 0.9 tsf
5.0				
	Exploration completed at a depth of 5.5 feet.	5.5		Surface elevation was not measured at the time of exploration.
7.5				

▲ BLOW COUNT ● MOISTURE CONTENT % (0 50 100)

DRILLED BY: GeoDesign, Inc. staff LOGGED BY: JLM/LAG COMPLETED: 11/02/16

BORING METHOD: hand auger (see document text) BORING BIT DIAMETER: 3 inches

GEODESIGN INC — AN NV5 COMPANY

WASHCO-56-03 | JUNE 2020

BORING

WALKER/MURRAY IMPROVEMENTS
WASHINGTON COUNTY, OR

FIGURE A-9

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT %	COMMENTS

HA-3

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	SAMPLE	COMMENTS
0.0	Soft to medium stiff, brown SILT (ML); moist (topsoil to approximately 2 inches). without gravel at 0.5 foot			
2.5			☒	
	brown-gray at 3.0 feet			
				Infiltration test at 4.5 feet.
5.0			☒ ☒	Surface elevation was not measured at the time of exploration.
	Exploration completed at a depth of 5.8 feet.	5.8		
7.5				

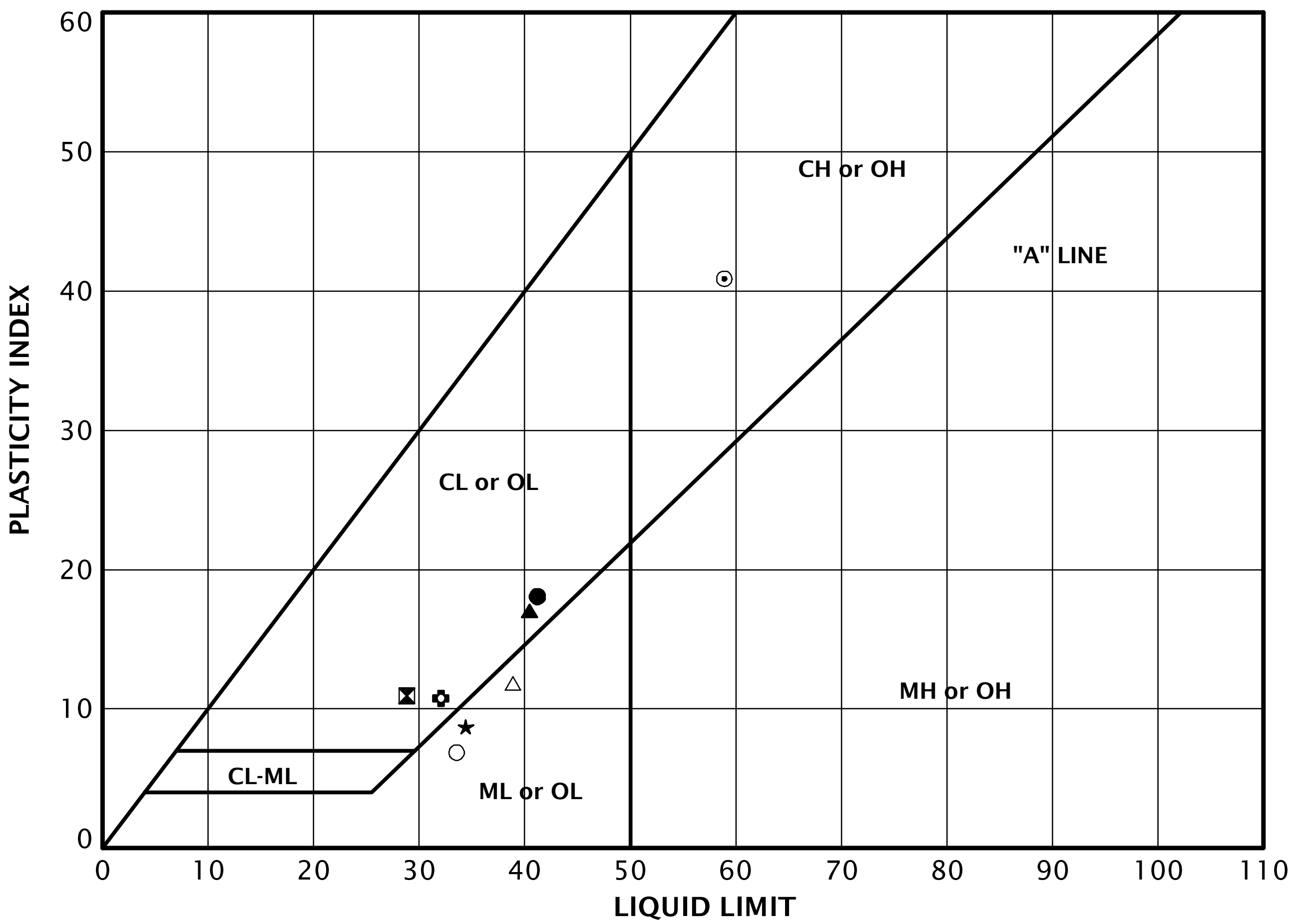
HA-4

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	SAMPLE	COMMENTS
0.0	Medium stiff, brown SILT with organics (ML); moist (topsoil to 2 inches).			
2.5	without organics at 2.0 feet			
			☒	
	Exploration completed at a depth of 4.0 feet.	4.0		Surface elevation was not measured at the time of exploration.
5.0				
7.5				

DRILLED BY: GeoDesign, Inc. staff | LOGGED BY: LAG | COMPLETED: 11/02/16

BORING METHOD: hand auger (see document text) | BORING BIT DIAMETER: 3 inches

GEODESIGN INC AN NV5 COMPANY	WASHCO-56-03	BORING (continued)	
	JUNE 2020	WALKER/MURRAY IMPROVEMENTS WASHINGTON COUNTY, OR	FIGURE A-10



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
●	B-1	7.5	27	41	23	18
⧗	B-1	60.0	25	29	18	11
▲	B-2	10.0	37	40	23	17
★	B-3	20.0	33	34	26	8
⊙	B-4	7.5	37	59	18	41
✿	B-4	45.0	33	32	21	11
○	B-5	10.0	32	34	27	7
△	B-8	7.5	37	39	27	12

ATTERBERG_LIMITS 14 WASHCO-56-03-B1_8-HA1_4.GPJ GEODESIGN.GDT PRINT DATE: 6/17/20:SN

GEODESIGN INC
AN NV5 COMPANY

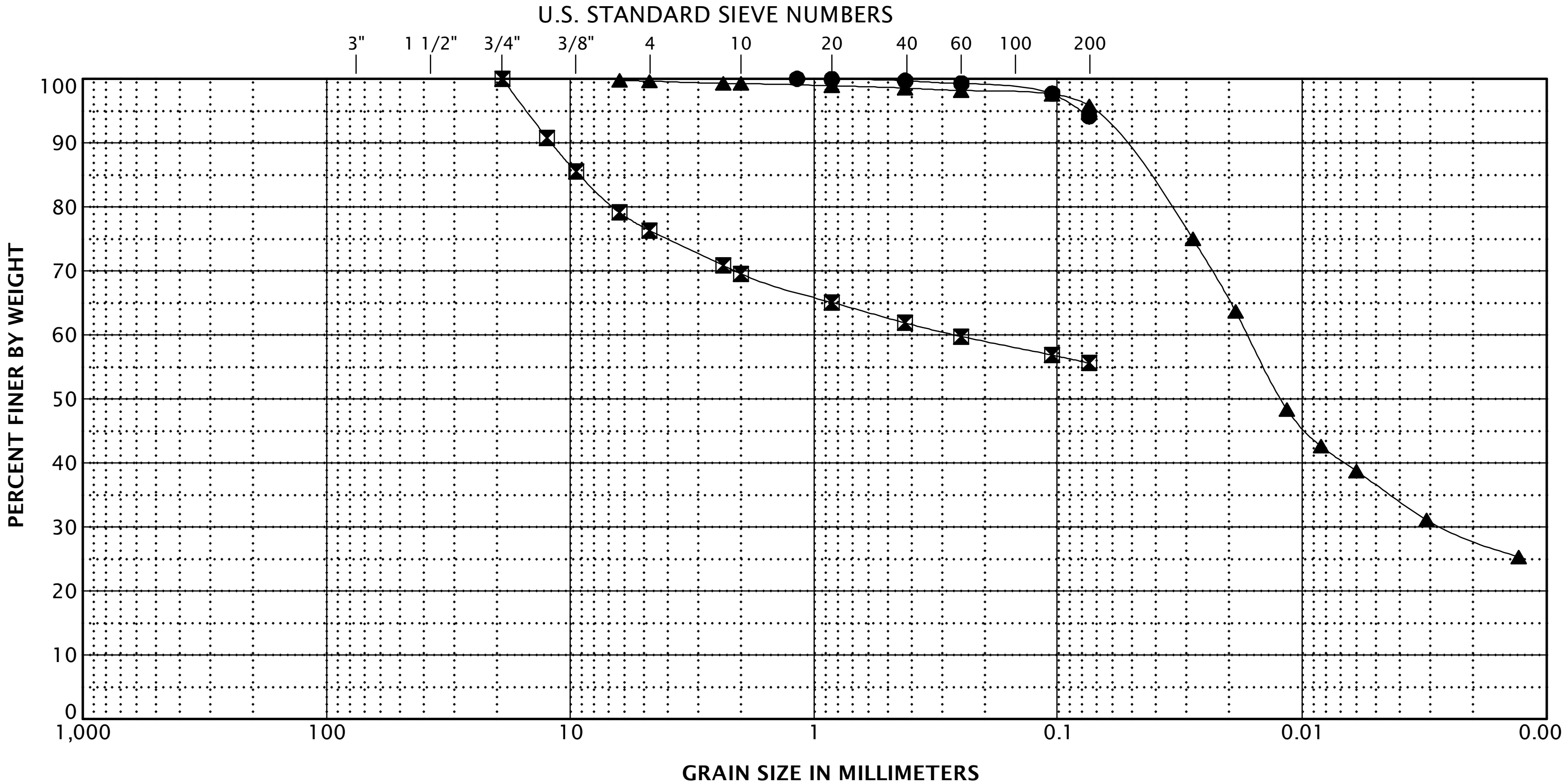
WASHCO-56-03

JUNE 2020

ATTERBERG LIMITS TEST RESULTS

WALKER/MURRAY IMPROVEMENTS
WASHINGTON COUNTY, OR

FIGURE A-11



BOULDERS	COBBLES	GRAVEL		SAND			FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY

KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	D60	D50	D30	D10	D5	GRAVEL (PERCENT)	SAND (PERCENT)	SILT (PERCENT)	CLAY (PERCENT)
●	B-3	25.0	36						0	6	94	
⧗	B-7	2.5	17	0.27					24	21	56	
▲	HA-4	4.0	28	0.02	0.01	0.00			0	4	59	37

GEODESIGN INC
AN NV5 COMPANY

WASHCO-56-03

JUNE 2020

GRAIN-SIZE TEST RESULTS

WALKER/MURRAY IMPROVEMENTS
WASHINGTON COUNTY, OR

FIGURE A-12

SAMPLE INFORMATION			MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
B-1	2.5	185.0	23							
B-1	5.0	182.5	23							
B-1	7.5	180.0	27					41	23	18
B-1	10.0	177.5	28							
B-1	15.0	172.5	39				89			
B-1	20.0	167.5	40							
B-1	25.0	162.5	37							
B-1	30.0	157.5	40				97			
B-1	35.0	152.5	45							
B-1	40.0	147.5	29							
B-1	50.0	137.5	32							
B-1	60.0	127.5	25					29	18	11
B-1	70.0	117.5	37							
B-1	80.0	107.5	38							
B-1	100.0	87.5	30							
B-2	5.0	180.0	29							
B-2	10.0	175.0	37					40	23	17
B-2	15.0	170.0	42							
B-2	20.0	165.0	33							
B-2	25.0	160.0	36							
B-2	30.0	155.0	39							
B-2	35.0	150.0	35							
B-2	45.0	140.0	35							
B-2	50.0	135.0	32							
B-2	55.0	130.0	26				83			
B-2	65.0	120.0	28							
B-2	80.0	105.0	45							

SAMPLE INFORMATION			MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
B-2	100.0	85.0	27							
B-2	120.0	65.0	31							
B-3	15.0	169.0	34							
B-3	20.0	164.0	33					34	26	8
B-3	25.0	159.0	36		0	6	94			
B-3	30.0	154.0	37							
B-3	40.0	144.0	37							
B-3	50.0	134.0	41							
B-3	55.0	129.0	40							
B-3	65.0	119.0	43							
B-3	75.0	109.0	43							
B-3	85.0	99.0	33							
B-3	95.0	89.0	27							
B-3	115.0	69.0	36							
B-4	7.5	176.5	37					59	18	41
B-4	12.0	172.0	37							
B-4	15.0	169.0	35							
B-4	20.0	164.0	31							
B-4	30.0	154.0	33							
B-4	40.0	144.0	30							
B-4	45.0	139.0	33					32	21	11
B-4	50.0	134.0	38							
B-4	60.0	124.0	29							
B-4	70.0	114.0	33							
B-4	80.0	104.0	34							
B-4	90.0	94.0	24							
B-4	100.0	84.0	34							

SAMPLE INFORMATION			MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
B-4	110.0	74.0	33							
B-4	120.0	64.0	39							
B-5	7.5	178.0	29							
B-5	10.0	175.5	32					34	27	7
B-5	13.0	172.5	34							
B-5	20.0	165.5	51							
B-5	30.0	155.5	42							
B-6	2.5	182.5	29							
B-6	5.0	180.0	29				89			
B-6	7.5	177.5	28							
B-6	10.0	175.0	37							
B-6	15.0	170.0	37							
B-6	20.0	165.0	37							
B-7	2.5	184.0	17		24	21	56			
B-7	15.0	171.5	39							
B-7	20.0	166.5	38							
B-7	25.0	161.5	35							
B-8	2.5	184.5	27							
B-8	7.5	179.5	37					39	27	12
B-8	12.0	175.0	42							
B-8	20.0	167.0	37							
B-8	30.0	157.0	34							
HA-4	4.0				0	4	96			

APPENDIX B

APPENDIX B

DRIVEN PILE DESIGN INFORMATION

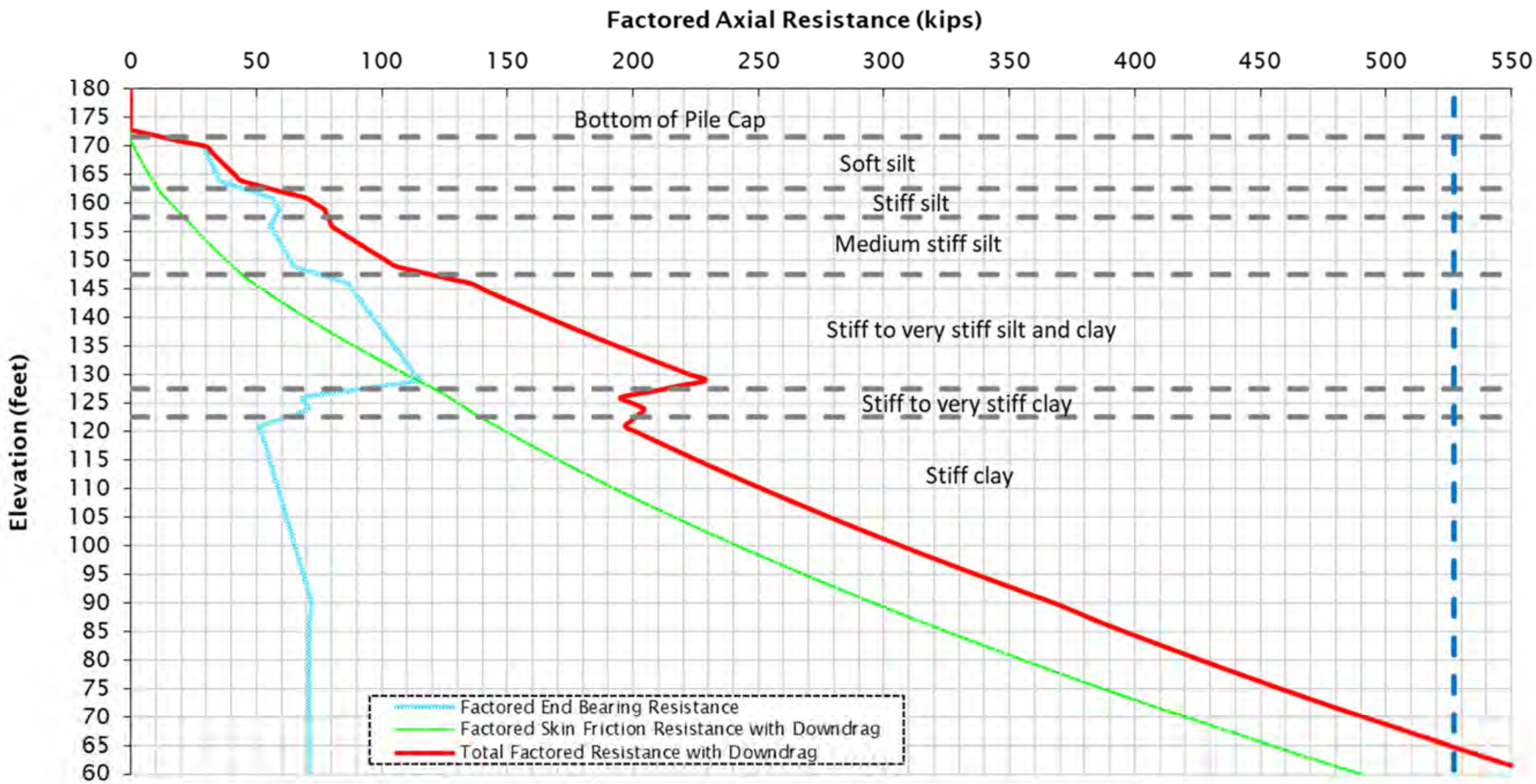
GENERAL

Design information for pile design is presented in this appendix. Our engineering design calculations were completed in general accordance with the FHWA manual *Design and Construction of Driven Pile Foundations, 2016; AASHTO LRFD Bridge Design Specifications, 2017* and generally accepted practices for geotechnical engineering design. Our design calculations are on file in our office and can be provided upon request. Summaries of design information are presented in this appendix.

PILE AXIAL CAPACITY DESIGN

Plots of geotechnical resistances versus depth for PP16X0.5, closed-ended pipe piles at the proposed bridge abutments for the Murray Boulevard and Walker Road bridges are presented in this appendix. A resistance factor of 0.65 was used in calculating the factored axial and uplift capacities as shown on Figures B-1 through B-4, which is the maximum acceptable resistance factor to be computed for piles used with dynamic pile testing.

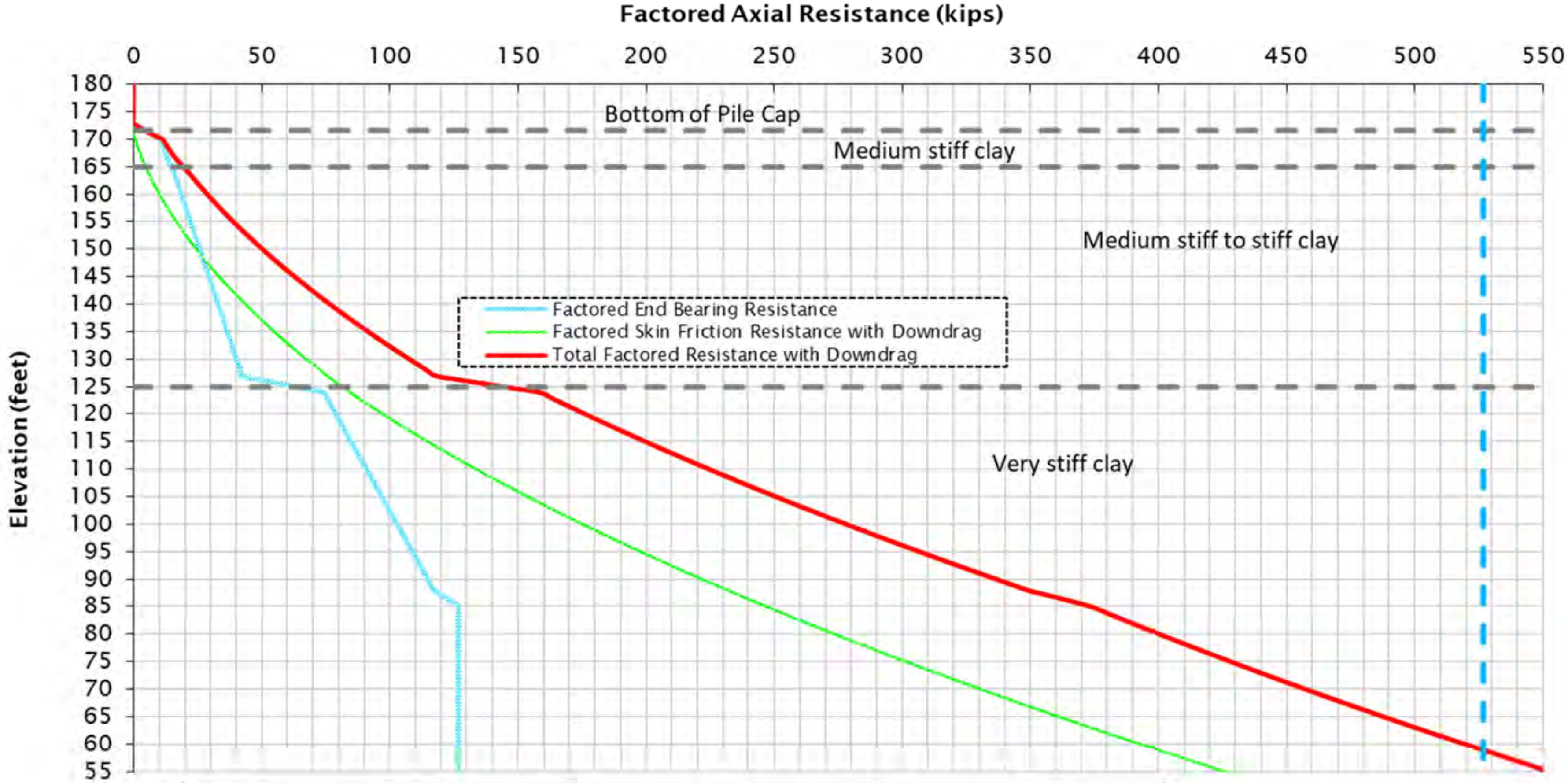
Single PP 16 x 0.500 Driven Pile



NOTES:
1. Capacities include a resistance factor of 0.65, which can only be used if PDA testing is completed.
2. Depth of geologic units may vary and should be verified during construction.
3. Figure should be used in conjunction with report text.

GEODESIGN INC — AN NV5 COMPANY	WASHCO-56-03	FACTORED AXIAL RESISTANCE – MURRAY BOULEVARD BRIDGE BENT 1	
	JUNE 2020	WALKER/MURRAY IMPROVEMENTS WASHINGTON COUNTY, OR	FIGURE B-1

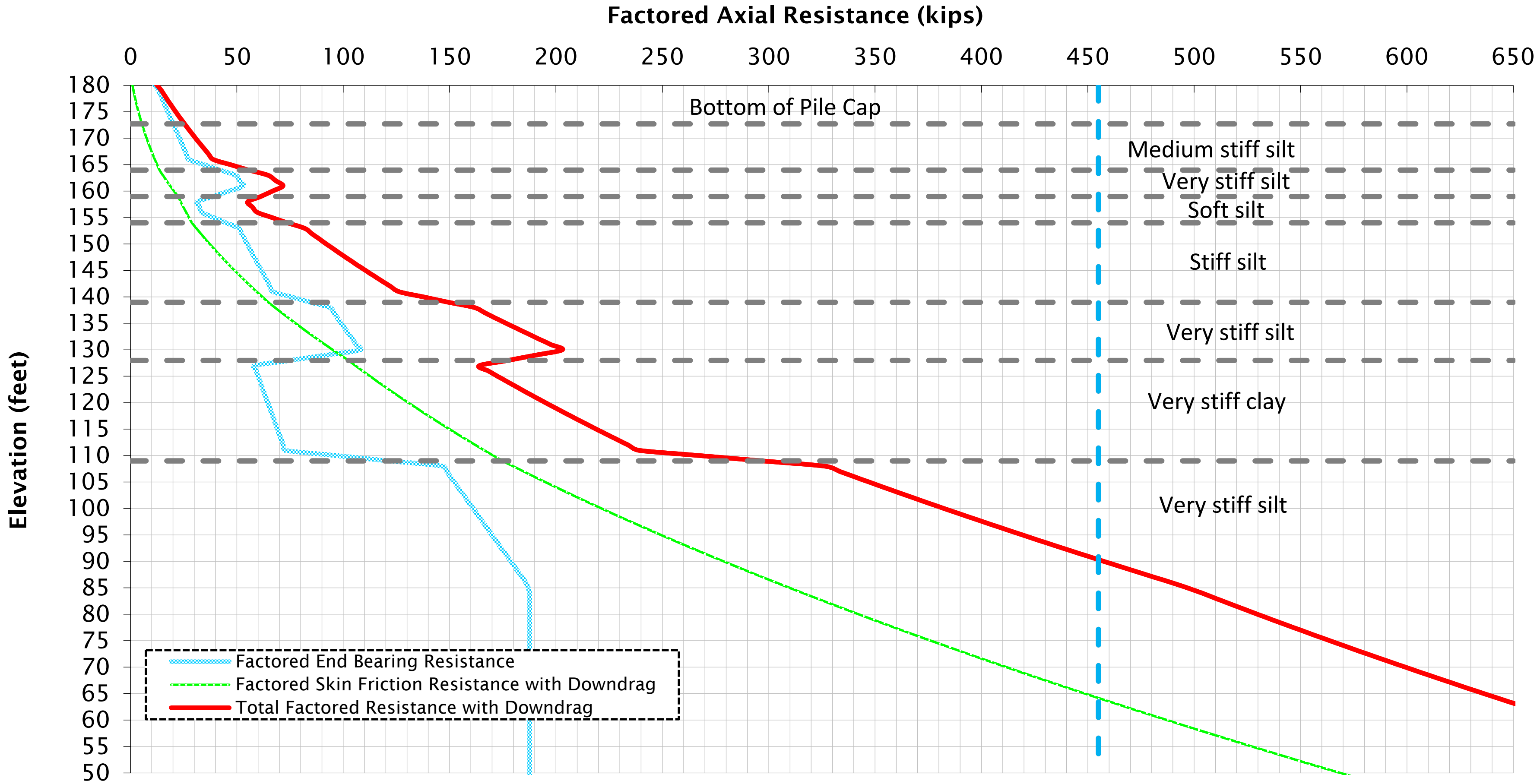
Single PP 16 x 0.500 Driven Pile



NOTES:
1. Capacities include a resistance factor of 0.65, which can only be used if PDA testing is completed.
2. Depth of geologic units may vary and should be verified during construction.
3. Figure should be used in conjunction with report text.

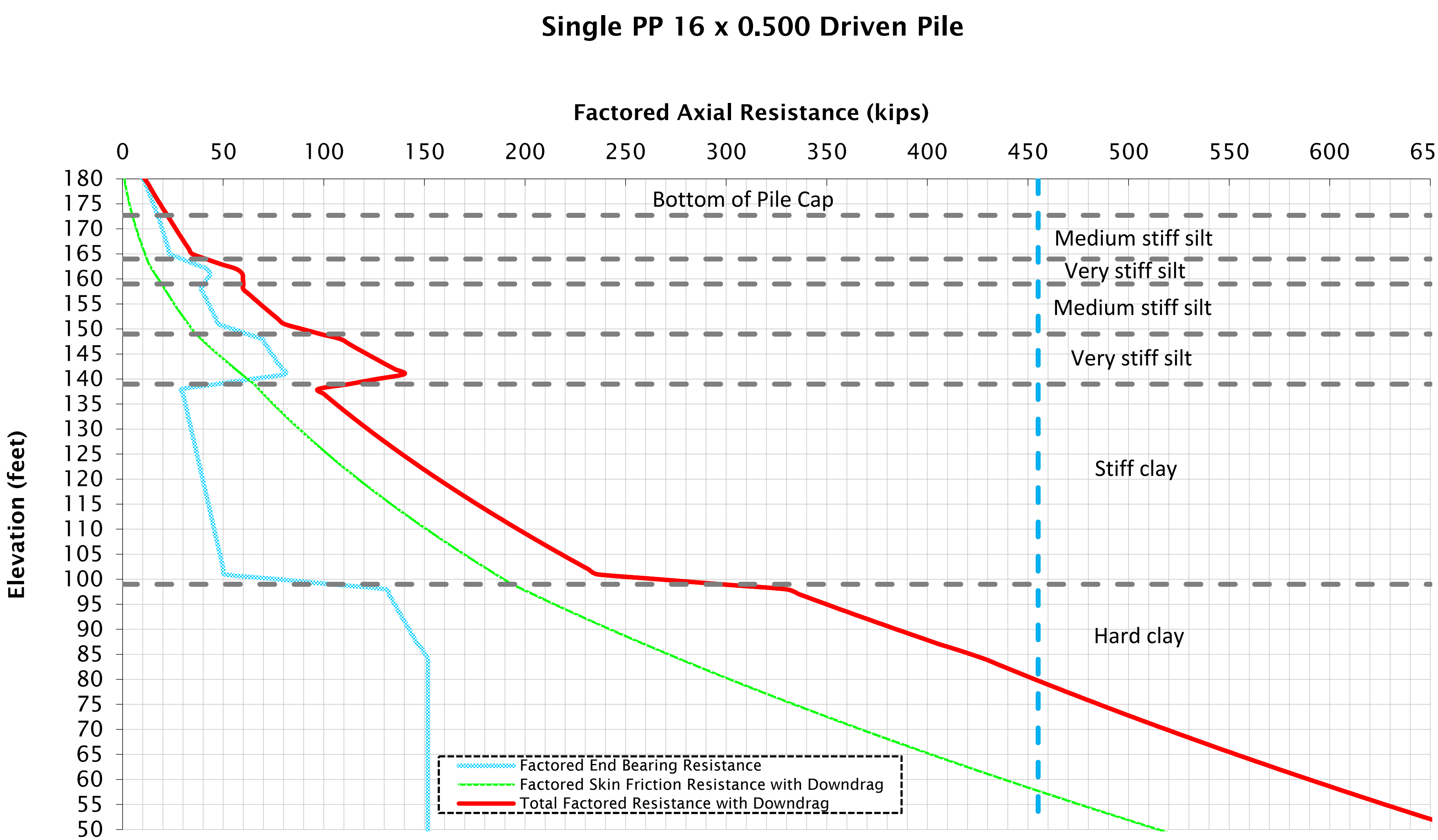
GEODESIGN INC — AN NV5 COMPANY	WASHCO-56-03	**FACTORED AXIAL RESISTANCE – MURRAY BOULEVARD BRIDGE BENT 2**
	JUNE 2020	WALKER/MURRAY IMPROVEMENTS WASHINGTON COUNTY, OR — **FIGURE B-2**

Single PP 16 x 0.500 Driven Pile



NOTES:
1. Capacities include a resistance factor of 0.65, which can only be used if PDA testing is completed.
2. Depth of geologic units may vary and should be verified during construction.
3. Figure should be used in conjunction with report text.

GEODESIGN INC — AN NV5 COMPANY	WASHCO-56-03	FACTORED AXIAL RESISTANCE – WALKER ROAD BRIDGE BENT 1	
	JUNE 2020	WALKER/MURRAY IMPROVEMENTS WASHINGTON COUNTY, OR	FIGURE B-3



NOTES:
1. Capacities include a resistance factor of 0.65, which can only be used if PDA testing is completed.
2. Depth of geologic units may vary and should be verified during construction.
3. Figure should be used in conjunction with report text.

GEODESIGN INC — AN NV5 COMPANY	WASHCO-56-03	FACTORED AXIAL RESISTANCE – WALKER ROAD BRIDGE BENT 2	
	JUNE 2020	WALKER/MURRAY IMPROVEMENTS WASHINGTON COUNTY, OR	FIGURE B-4

APPENDIX C

APPENDIX C

LPILE PARAMETERS

Parameters for lateral analysis and design of the bridge structures for the computer software program LPILE 6.0 are presented in this appendix. Parameters are provided for Bents 1 and 2 of the Murray Boulevard bridge in Tables C-1 and C-2, respectively, and for Bents 1 and 2 of the Walker Road bridge in Tables C-3 and C-4, respectively, in this appendix.

TABLE C-1
LPILE Soil Parameters
Murray Boulevard Bridge - Bent 1
Walker/Murray Improvements
Washington County, Oregon

Soil Conditions	Soil Model (LPILE 6.0)	Elev. @ Top, ft [1]	Layer Thick., ft	Elev @ Bot., ft	Depth @ Top, ft	Depth @ Bot, ft	g', pcf [2]	c', psf	e_{50}	Friction Angle, ϕ' (degrees)	k, pci
Stiff Clay Fill	Stiff Clay without free water	187	15.5	171.5	0.0	15.5	115	500	0.01	NA	100
Soft Silt	Soft Clay (Matlock)	171.5	9.0	162.5	15.5	24.5	115 (52)	250	0.02	NA	30
Medium Stiff to Stiff Silt	Stiff Clay with free water	162.5	15.0	147.5	24.5	39.5	115 (52)	500	0.01	NA	100
Very Stiff Silt and Clay	Stiff Clay with free water	147.5	25.0	122.5	39.5	64.5	115 (52)	1,000	0.05	NA	500
Stiff Clay	Stiff Clay with free water	122.5	72.5	50.0	64.5	137.0	115 (52)	500	0.010	NA	100

Notes:
1. Elevation at top corresponds to the existing ground surface.
2. Saturated unit weight in (). Assume saturated soil at elevation 180.0 feet.

Shaft Center-to-Center Spacing (in the direction of loading)	P-Multipliers,		
	Row 1	Row 2	Row 3 and Higher
3B	0.8	0.4	0.3
5B	1	0.85	0.7
8B	1	1.0	1.0

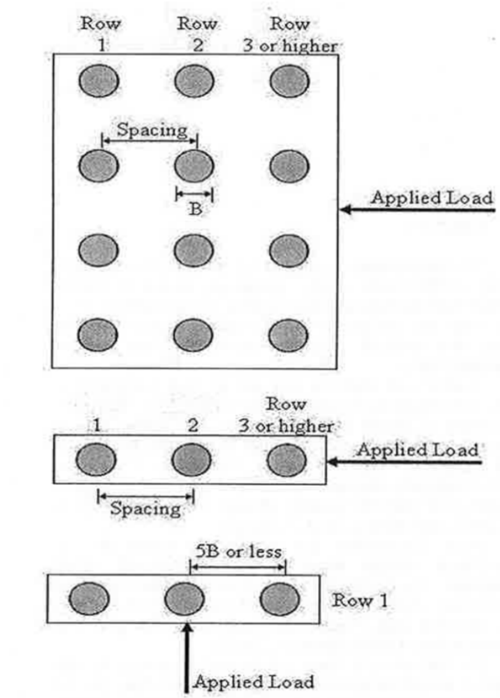


TABLE C-2
LPILE Soil Parameters
Murray Boulevard Bridge - Bent 2
Walker/Murray Improvements
Washington County, Oregon

Soil Conditions	Soil Model (LPILE 6.0)	Elev. @ Top, ft [1]	Layer Thick., ft	Elev @ Bot., ft	Depth @ Top, ft	Depth @ Bot, ft	g', pcf [2]	c', psf	e_{50}	Friction Angle, ϕ' (degrees)	k, pci
Soft to Medium Stiff Clay	Soft Clay (Matlock)	185	13.5	171.5	0.0	13.5	115	250	0.02	NA	30
Medium Stiff to Stiff Silt and Clay	Stiff Clay with free water	171.5	46.5	125.0	13.5	60.0	115 (52)	500	0.01	NA	100
Very Stiff Clay	Stiff Clay with free water	125	50.0	75.0	60.0	110.0	115 (52)	1,000	0.05	NA	500

Notes:
1. Elevation at top corresponds to the existing ground surface
2. Saturated unit weight in (). Assume saturated soil at elevation 180.0 feet.

Shaft Center-to-Center Spacing (in the direction of loading)	P-Multipliers,		
	Row 1	Row 2	Row 3 and Higher
3B	0.8	0.4	0.3
5B	1	0.85	0.7
8B	1	1.0	1.0

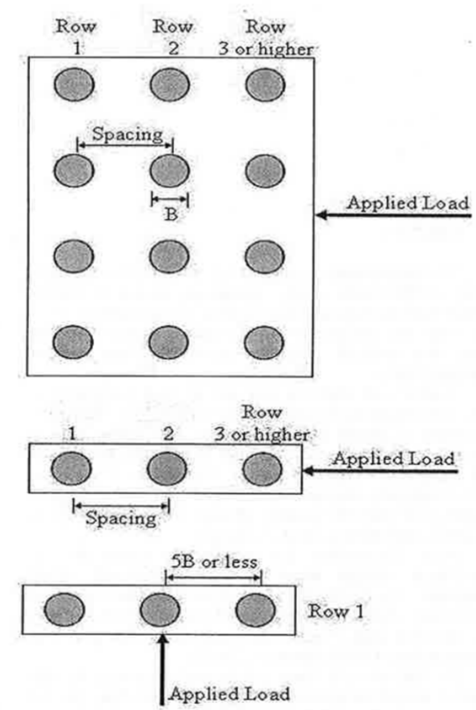


TABLE C-3
LPILE Soil Parameters
Walker Road Bridge - Bent 1
Walker/Murray Improvements
Washington County, Oregon

Soil Conditions	Soil Model (LPILE 6.0)	Elev. @ Top, ft [1]	Layer Thick., ft	Elev @ Bot., ft	Depth @ Top, ft	Depth @ Bot, ft	g', pcf [2]	c', psf	e_{50}	Friction Angle, ϕ' (degrees)	k, pci
Medium Dense Gravel Fill	Sand (Reese)	184	12.5	171.5	0.0	12.5	120 (58)	NA	NA	35	125
Medium Stiff Silt	Stiff Clay with free water	171.5	7.5	164.0	12.5	20.0	115 (52)	500	0.01	NA	100
Very Stiff Silt	Stiff Clay with free water	164	5.0	159.0	20.0	25.0	115 (52)	1,000	0.05	NA	500
Soft Silt	Soft Clay (Matlock)	159	5.0	154.0	25.0	30.0	115 (52)	250	0.02	NA	30
Stiff Silt	Stiff Clay with free water	154	15.0	139.0	30.0	45.0	115 (52)	500	0.01	NA	100
Very Stiff Silt and Clay	Stiff Clay with free water	139	65.0	74.0	45.0	110.0	115 (52)	1,000	0.05	NA	500

Notes:
1. Elevation at top corresponds to the existing ground surface
2. Saturated unit weight in (). Assume saturated soil at elevation 180.0 feet.

Shaft Center-to-Center Spacing (in the direction of loading)	P-Multipliers,		
	Row 1	Row 2	Row 3 and Higher
3B	0.8	0.4	0.3
5B	1	0.85	0.7
8B	1	1.0	1.0

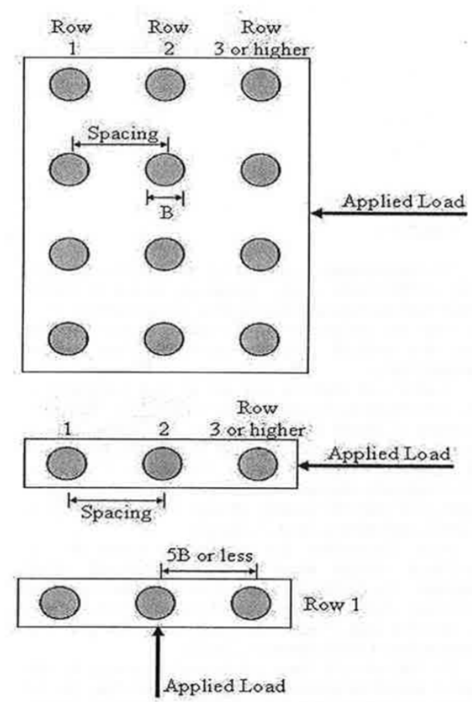
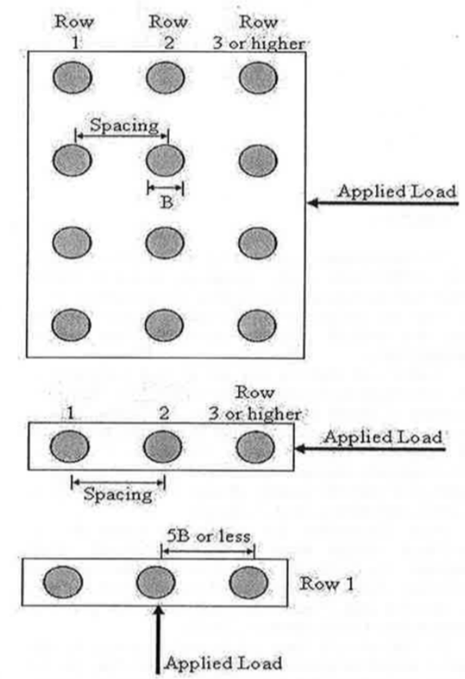


TABLE C-4
LPILE Soil Parameters
Walker Road Bridge - Bent 2
Walker/Murray Improvements
Washington County, Oregon

Soil Conditions	Soil Model (LPILE 6.0)	Elev. @ Top, ft [1]	Layer Thick., ft	Elev @ Bot., ft	Depth @ Top, ft	Depth @ Bot, ft	g', pcf [2]	c', psf	e_{50}	Friction Angle, ϕ' (degrees)	k, pci
Medium Dense Gravel Fill	Sand (Reese)	184	12.5	171.5	0.0	12.5	120 (58)	NA	NA	35	125
Medium Stiff Silt	Stiff Clay with free water	171.5	7.5	164.0	12.5	20.0	115 (52)	500	0.010	NA	100
Very Stiff Silt	Stiff Clay with free water	164	5.0	159.0	20.0	25.0	115 (52)	1,000	0.050	NA	500
Medium Stiff Silt	Stiff Clay with free water	159	10.0	149.0	25.0	35.0	115 (52)	500	0.010	NA	100
Very Stiff Silt	Stiff Clay with free water	149	10.0	139.0	35.0	45.0	115 (52)	1,000	0.050	NA	500
Stiff Clay	Stiff Clay with free water	139	40.0	99.0	45.0	85.0	115 (52)	500	0.010	NA	100
Hard Clay	Stiff Clay with free water	99	25.0	74.0	85.0	110.0	115 (52)	2,000	0.005	NA	1,000

Notes:
1. Elevation at top corresponds to the existing ground surface
2. Saturated unit weight in (). Assume saturated soil at elevation 180.0 feet.

Shaft Center-to-Center Spacing (in the direction of loading)	P-Multipliers,		
	Row 1	Row 2	Row 3 and Higher
3B	0.8	0.4	0.3
5B	1	0.85	0.7
8B	1	1.0	1.0



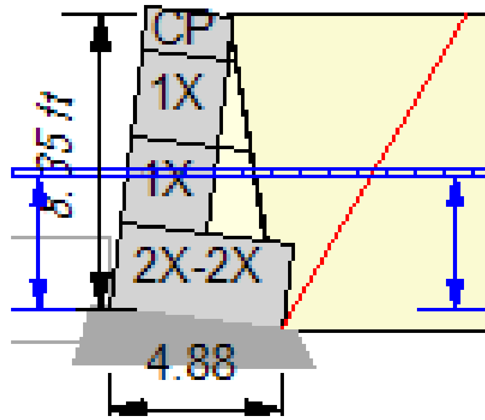
APPENDIX D

APPENDIX D

MODULAR BLOCK RETAINING WALL DESIGN OUTPUT

The output files for the modular block retaining wall design using the computer software UltraWall® are presented in this appendix.

UltraWall



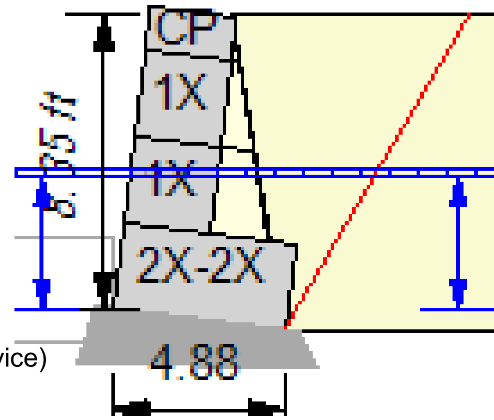
Project:	Murray/Walker Improvements
Location:	Johnson Creek Retaining Wall
Designer:	JDT
Date:	2/22/2019
Section:	Section 1
Design Method:	AASHTO_LRFD_2012
Design Unit:	UltraBlock

SOIL PARAMETERS	φ	coh	γ	e	γsat
Retained Soil:	28 deg	50 psf	115 pcf	0.40	138 pcf
Foundation Soil:	28 deg	50 psf	115 pcf	0.40	138 pcf
Leveling Pad:	35 deg	0 psf	130 pcf	0.40	138 pcf
	Crushed Stone				

GEOMETRY

Design Height:	8.35 ft	Live Load:	0 psf
Wall Batter/Tilt:	0.0/ 7.13 deg	Live Load Offset:	0.00 ft
Embedment:	2.00 ft	Live Load Width:	0 ft
Leveling Pad Depth:	1.00 ft	Dead Load:	0 psf
Slope Angle:	0.0 deg	Dead Load Offset:	0.0 ft
Slope Length:	0.0 ft	Dead Load Width:	0 ft
Slope Toe Offset:	0.0 ft	Leveling Pad Width:	5.92 ft
Vertical δ on Single Depth		Toe Slope Angle:	26.67
		Toe Slope Length:	0.00
		Toe Slope Bench:	0.00

Water Level:	3.70 ft
Water Back of Face:	3.70 ft
Drain Depth:	1.00 ft



RESULTS

CDR Sliding:	1.38 (lvlpd)	CDR Bearing:	2.27
Eccentricity (e/L):	0.14 (e/L <= 0.33)	Bearing:	1470 / 1025 (Service)
Ecc Internal(e/L):	0.10 (e/L <= 0.45)		

Name	Elev.[dpth]	ka	Paw1	Paw2	Paw3	Pwd	(Pwr)	Paq	Paqd	(PaC)	PaT	CDRsl(Lvl Pad)	e/L	e/L (Srvs)	%D/H
CP	7.32[1.03]	0.279	29	0	0	0	0	0	0	35	0	100.00	--	--	200%
1X	4.88[3.47]	0.279	229	0	0	0	0	0	0	100	129	86.12	--	--	71%
1X	2.44[5.91]	0.279	347	234	26	50	-50	0	0	164	447	25.72	0.10	--	42%
2X-2X	0.00[8.35]	0.401	498	923	282	427	-427	0	0	284	1442	1.38 (1.82)	0.14	0.10	59%

Column Descriptions:

- ka: active earth pressure coefficient
- Paw1: active earth pressure of soils above the water line
- Paw2: active earth pressures (1) of submerged soils below the water line
- Paw3: active earth pressures (2) of submerged soils below the water line
- Pwd: driving pressures of water from behind the face
- Pwr: resisting pressures of water in front of wall
- Paq: live surcharge earth pressure
- Paqd: dead surcharge earth pressure
- (PaC): reduction in load due to cohesion
- PaT: sum of all earth pressures
- CDRsl (lvlPad): 'Capacity/Demand Ratio' for sliding at each layer. (CDR sliding below the leveling pad)
- e/L: eccentricity/base width ratio
- e/L (Srvs): service state condition eccentricity/base width ratio
- %D/H: ratio of based depth to height (warning for narrow walls, < 35%)

DESIGN DATA

Load Factors for Design

AASHTO Table 3.4.1-1 & 3.4.1-2

Load Case	Str_Max	Str_Min	Extreme Max	Extreme Min	Service
Str I Dead Load (DC)	1.25	0.90	1.00	1.00	1.00
Soil Load Driving (EH)	1.50	0.90	1.00	1.00	1.00
Str I Vert Earth Load (EV)	1.35	1.00	1.00	1.00	1.00
Dead Load Surcharge (ES)	1.50	0.75	1.00	1.00	1.00
Live Load (LL, PL, LS)	1.75	0.00	1.00	1.00	1.00

AASHTO Table 11.5.7-1 & Article 11.5.8

MSE Resistance Case	Strength	Extreme	Service
Bearing Resistance (RFbr)	0.65	0.90	1.00
Sliding Soil to Soil (RFsl)	1.00	1.00	1.00
Sliding Soil to Reinforcement (RFslrf)	1.00	1.00	1.00
Tensile Resistance (RFten)	0.90	1.20	1.00
Pullout Resistance (RFpo)	0.90	1.20	1.00
Overturning Resistance (RFot)	0.60	1.00	1.00

AASHTO Table 10.5.5.2.2-1

Gravity Resistance Case	Resistance Factor	Extreme	Service
Bearing Resistance (RFbr)	0.45	0.80	1.00
Cast-In-Place Concrete to Stone (RFsl_cip)	0.80	1.00	1.00
Precast Concrete to Stone (RFsl_c)	0.90	1.00	1.00
Stone to Soil (RFsl_s)	0.90	1.00	1.00
Passive EP (RFep)	0.50	1.00	1.00

Application of Load Factors

Group	DC	EV	LS	EH	Probable Use
Strength I-a	0.90	1.00	1.75	1.50	BC/EC/SL
Strength I-b	1.25	1.35	1.75	1.50	BC (max. value)
Service I	1.00	1.00	1.00	1.00	Settlement

Notes: BC - Bearing Capacity; EC - Eccentricity; SL - Sliding

By Inspection:

• Strength I-a (minimum vertical loads and maximum horizontal loads) will govern for the case of sliding and eccentricity (overturning); and

• For the case of bearing capacity, maximum vertical loads will govern, and the factored loads must be compared for Strength I-b and Strength IV.

NOTES ON DESIGN UNITS

The wall section is designed on a 'per unit width bases' (lb/ft/ft of wall or kN/m/meter of wall). In the calculations the software shows lb/ft or kN/m, neglecting the unit width factor for simplicity.

The weights for the wall unit are shown as lbs / ft3 (kN / m3). For SRW design a 1 sf unit is typically 1 ft deep, 1.5 ft wide and 8 inches tall (or 1 ft3). therefore a typical value of 120 pcf is shown. With larger units the unit weight will vary with the size of the unit. Say we have 4 ft wide unit, 1.5 ft tall and 24 inches deep with a tapered shape (sides narrow), built with 150 pcf concrete. We add up the concrete, the gravel fill and divide by the volume and the results may come out to 140 pcf, as shown in the table. The units with more gravel may have lower effective unit weights based on the calculations.

Hollow Units
Hollow units with gravel fill are treated differently in AASHTO. If the fill can fall out as the unit is lifted, then AASHTO only allows 80% of the weight of the fill to be used for eccentricity (overturning calculations). In the properties page for the units the weight of the concrete may be as low as 75 pcf. This is the effective unit weight of the concrete only (e.g. the weight of the concrete divided by the volume of the unit). The density of the concrete maybe 150 pcf, but not the effective weight including the volume of the void spaces used for gravel fill.

Rounding Errors
When doing hand calculations the values may vary from the values shown in the software. The program is designed using double precision values (64 bit precision: 14 decimal places). Over several calculations the results may differ from the single calculation the user is making, probably inputting one or two already rounded values.

Result Rounding
As noted above the software is based on double precision values. For example, using an NCMA design method an allowable factor of safety of 1.5 the software may calculate a value of 1.49999999999999, since this is less than 1.5, it would be false (NG), even though the results shown is 1.50 (results are rounded to 2 places on the screen). In the design check we round to 2 decimal places to check against the suggested value (1.49999999999 rounds to 1.50). Given the precision of the calculation, this will provide a safe design even though the 'absolute' value is less than the minimum suggested.

RETAINING WALL UNITS

STRUCTURAL PROPERTIES:

N is the normal force [or factored normal load] on unit to unit interface
The unit to unit shear is N x Tan(0.0) + 17796.0

N is the normal force [or factored normal load] on the base unit
The default leveling pad to base unit shear is 0.8 * ATan(35), or 29 deg. [AASHTO LRFD 10.6.3.4-2] or may be the manufacturer supplied data.
[Note: concrete to concrete has a coefficient of φ0.6 N. 0 [AASHTO LRFD 5.8.4]

Table of Values:

Unit	Ht (in)	Width (in)	Depth (in)	Equiv_Density (pcf)	Equiv_CG (in)
Cap	14.75	59.00	29.50	140.00	14.75
Full	29.50	59.00	29.50	140.00	14.75
Double	29.50	59.00	59.00	140.00	29.50
Triple	29.50	59.00	88.50	140.00	44.25
15 in Tall Unit	14.75	59.00	29.50	140.00	14.75

FORCE DETAILS

The details below shown how the forces and moments are calculated for each force component. The values shown are not factored. All loads are based on a unit width (ppf / kNpm).

With hydrostatic forces, the 'w' in the index indicates the unit is under water.

Layer	Block Wt	X-Arm	Moment	Soil Wt	X-Arm	Moment
1	423.04	2.14	907.23	11.76	3.44	40.42
2	846.08	1.84	1556.30	164.75	3.44	566.68
3 w	652.69	1.53	1001.42	295.65	3.50	1036.13
4 w	937.94	2.46	2305.76			

Block Weight (Force v) = block: 3,807 X-Arm = 2.15 ft
Soils Block Weight (Force v) = 472 ppf X-Arm = 3.62 ft

Active Earth Pressure Pa = (Paw1 + Paw2 + Paw3) = 1,703 ppf
Active Earth Pressure Paw1 = 498 ppf
Active Earth Pressure Paw2 = 923 ppf
Active Earth Pressure Paw3 = 282 ppf
Pa_h (Force H) = (Paw1 + Paw2 + Paw3) cos(δ batter) = 1,703 x cos(21.010.0 +) = 1,217 ppf
Y-Arm = 2.74 ft
Pa_v (Force V) = Pa sin(batter + δ) = 1,703 x sin(21.010.0) = 731 ppf
X-Arm = 4.37 ft

Passive Earth Pressures
Passive earth pressures are used for resistance of the Leveling Pad, but may be extended upward to assist with the resistance of the wall facing for walls that have deep embedments.

Passive Earth Pressure:
kp = 2.77
Pp = 951.60 ppf

CALCULATION RESULTS

OVERVIEW

UltraWall calculates stability assuming the wall is a rigid body. Forces and moments are calculated about the base and the front toe of the wall. The base block width is used in the calculations. The concrete units and granular fill over the blocks are used as resisting forces.

EARTH PRESSURES

The method of analysis uses the Coulomb Earth Pressure equation (below) to calculate active earth pressures. Wall friction is assumed to act at the back of the wall face. The component of earth pressure is assumed to act perpendicular to the boundary surface. The effective δ angle is δ minus the wall batter at the back face. If the slope breaks within the failure zone, a trial wedge method of analysis is used.

EXTERNAL EARTH PRESSURES

Effective δ angle (3/4 retained phi)	δ = 21.0 deg
Coefficient of active earth pressure	ka = 0.401
External failure plane	ρ = 58 deg
Back Face Angle from horizontal	Face Angle =80.01 deg
Coefficient of passive earth pressure	kp =2.77

$$Ka := \frac{\cos(\phi_i + i)^2}{\cos(i)^2 \cdot \cos(\delta_i - i)\left(1 + \sqrt{\frac{\sin(\phi_i + \delta_i)\cdot \sin(\phi_i - \beta)}{\cos(\delta_i - i)\cdot \cos(i + \beta)}}\right)^2}$$

FORCES AND MOMENTS

The program resolves all the geometry into simple geometric shapes to make checking easier. All x and y coordinates are referenced to a zero point at the front toe of the base block.

UNFACTORED LOADS

Name	Factor γ	Force (V)	Force (H)	X-len	Y-len	Mo	Mr
Face Blocks(W1)	1.00	3807	--	2.15	--	--	8173
Soil Wedge(W2)	1.00	472	--	3.62	--	--	1711
LvlPad(W18)	1.00	410	--	--	--	--	--
Pa_h	1.00	--	1217	--	2.74	3337	--
Pa_v	1.00	731	--	4.37	--	--	3194
Water_Fd (W22)	1.00	--	427	--	1.23	527	--
Water_Fr (W23)	1.00	--	-427	--	1.23	-527	--
Sum V / H	1.00	5010	1217		Sum Mom	3337	13078

W0: stone within units

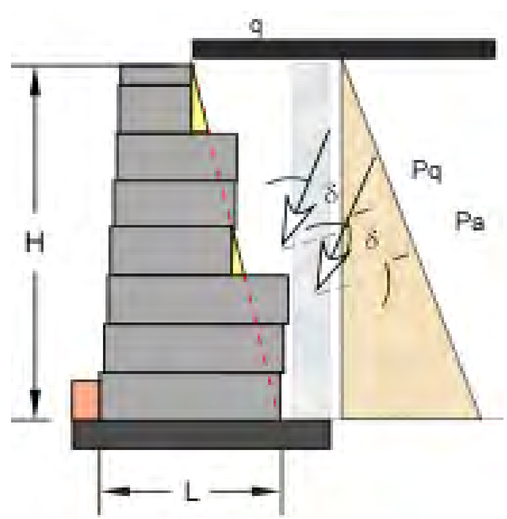
W1: facing units

W2: stone over the tails

W9: Driving force Pa

W10: Driving Surcharge load Paq

W11: Driving Dead Load Surchage Paqd



Note: Calculations and quantities are for PRELIMINARY ANALYTICAL USE ONLY and MUST NOT be used for final design or construction without the independent review, verification, and approval by a qualified professional engineer.

FORCES AND MOMENTS FACTORED FOR Str I-a

UltraWall increases all driving forces and reduces the resisting forces by the factors shown for Str I.

FACTORED LOADS: Str Ia

Name	Factor γ	Force (V)	Force (H)	X-len	Y-len	Mo	Mr
Face Blocks(W1)	0.90	3427	--	2.15	--	--	7355
Soil Wedge(W2)	1.00	472	--	3.62	--	--	1711
Pa_h	1.50	--	1826	--	2.74	5006	--
Pa_v	1.50	1096	--	4.37	--	--	4792
Sum V / H		4995	1826		Sum Mom	5006	13858

FORCES AND MOMENTS FACTORED FOR Str I-b

UltraWall increase resisting loads and increases driving loads by the factors shown for Str I-b.

FACTORED LOADS: Str Ib

Name	Factor γ	Force (V)	Force (H)	X-len	Y-len	Mo	Mr
Face Blocks(W1)	1.25	4759	--	2.15	--	--	10216
Soil Wedge(W2)	1.35	637	--	3.62	--	--	2310
Pa_h	1.50	--	1826	--	2.74	5006	--
Pa_v	1.50	1096	--	4.37	--	--	4792
Pq_h	1.75	--	0	--	4.48	0	--
Pq_v	1.75	0	--	4.12	--	--	0
Sum V / H		6493	1826		Sum Mom	5006	17317

BASE SLIDING

Sliding at the base is checked at the block to leveling pad interface between the base block and the leveling pad. Sliding is also checked between the leveling pad and the foundation soils.

Forces Resisting sliding = W1(DCr) + W2(EVr) + Pav(EHd)
(3807 x 0.90) + (472 x 1.00) + (731 x 1.50) N =4995 ppf

Resisting force at pad = (N tan(slope) + intercept) x RFsl
(4995 x tan(31.0) + 0.0) x 0.90 Rf1 =2518
where L is the base block width

N1 includes N + leveling pad
where LvlPd = lvl pad thickness * 76pcf * L + lvl pad thickness/2 LvlPd = 410
Forces Resisting sliding = W1(DCr) + W2(EVr) + LvlPd(EVr) + Pav(EHd)
(3807 x 0.90) + (472 x 1.00) + (410 x 1.00) + (731 x 1.50)
N1 =5405 ppf

Friction angle is the lesser of the leveling pad and Fnd φ =28.00 deg

Passive resistance is calculated using kp = (1 + sin(28))/(1 - sin(28)) kp = 2.77
Force at top of resisting trapezoid, d1 = 2.00 Fp1 = 419.75
Force at base of resisting trapezoid, d2 = 3.62 Fp2 = 758.70
Depth of trapezoid depth = 1.62
Pp = (Fp1 + Fp2) / 2 * depth 951.60

Resisting force at fnd = (N1 tan(phi) + c L) * RFsl + Pp x RFps
(5405 x tan(28) + 50 x 5.92) * 0.90 + (952 x 0.50)
Rf2 = 3327 [the value printed is the minimum
foundation soil.

Driving force is the horizontal component of
Pah(EHd) + Pwd(1.0)
(1217 x 1.50) + (427 x 1.00) Df =1826

CDR = (Rf1 / Df) / (Rf2 / Df2) CDR =1.38 / 1.82

ECCENTRICITY ABOUT THE TOE

Eccentricity at the base is checked by assuming rotation about the front toe by the block mass and the soil retained on the blocks. Allowable overturning can be defined by eccentricity (e/L).

Moments resisting eccentricity =
M1(DCr) + M2(EVr) + MPav(EHd)
((8173 x 0.90) + (1711 x 1.00) + (3194 x 1.50)

Mr =13858 ft-lbs

Moments causing eccentricity = MPah(EHd)
(3337 x 1.50) Mo =5006 ft-lbs

e = L/2 - (Mr - Mo)/ N
e =4.92/2 - (13858 - 5006) /4995 e =0.69
e/L = 0.14

GRAVITY RESULTS

The table below shows the forces and moments for each layer of face units. For concrete leveling pads, sliding and eccentricity is checked at the base blocks and at the base of the concrete leveling pad.

TABLE OF RESULTS

Layer	Ht	ka	Batter	Delta	W0	Len0X	W1	Len1X	W2	Len2X	Pah	Pav	PaLenY	PaLenX	PqLenY	PqLenX
0	7.32	0.279	7.1	14.00	0	0.00	423	1.30	0	0.00	0	0	0.45	2.48		
1	4.88	0.279	7.1	14.00	0	0.00	1269	1.45	0	2.62	128	0	1.26	2.54		
2	2.44	0.279	7.1	14.00	0	0.00	2115	1.62	0	2.78	440	0	1.87	2.64		
3	0.00	0.401	-10.0	21.00	0	0.00	3807	2.15	472	3.62	1217	731	2.74	4.37		

Layer	M0[0]	M1[1]	Mr[2]	Mlp	MPah	MPav	Forces Re	Forces DR	SumV	Mr	Mo	CDRsl	ecc/L
0	0	548	0	0	0	0	16208	1	381	493	0	100.00	-0.03
1	0	1838	1	0	162	0	16592	193	1143	1655	243	86.12	0.00
2	0	3435	1	0	822	0	16975	660	1904	3093	1233	25.72	0.10
3	0	8173	1711	0	3337	3194	2518	1826	4995	13858	5006	1.38	0.14

ECCENTRICITY AND BEARING

Eccentricity is the calculation of the distance of the resultant force away from the centroid of the mass. This measure is an indication of the overturning of the mass. UltraWall uses an allowable eccentricity of 9/20 L for concrete to concrete bearing surfaces and a concrete leveling pad (thickness > 1.0 ft), or L/3 for bearing on soil per the AASHTO LRFD guidelines. Eccentricity is still used as a guide to design in some design methods.

UltraWall calculates three eccentricities:

1) Maximum eccentricity (overturning) where it uses the maximum driving forces combined with the minimum resisting forces (see overturning) [Str I-a]. 2) Maximum bearing where it uses the maximum driving forces combined with the maximum resisting forces [Str I-b]. 3) Service: Maximum bearing where it uses the actual driving forces combined with the actual resisting forces in Service loading.

Calculation of Eccentricity for maximum bearing

Moments resisting = M1(DCd) + M2(EVd) + MPa(EHd)

(8173 x 1.25) + (1711 x 1.35) + (3194 x 1.50)

Mr =17317 ft-lbs

Moments driving = MPah(EHd)

(3337 x 1.50) Mo =5006 ft-lbs

Nb = W1(DCd) + W2(EVd) + WPa(EHd)

(3807 x 1.25) + (472 x 1.35) + (731 x 1.50)

Nb =6493 ppf

N bearing = W1(DCd) + W2(EVd) + WPlvlpad(EVd) + WPa(EHd)

(3807 x 1.25) + (472 x 1.35) + (410 x 1.35) + (731 x 1.50)

Nbrg =7047 ppf

Calculate Eccentricity

e = L/2 - (Mr - Mo)/ Nb

e =4.92/2 - (17317 - 5006) /6493 e =0.562

B' = B - 2e + lvl pad thickness =

4.92 - 2 x 0.56 + 1.00 B' = 4.79 ft

Calculation of Bearing Pressures

Qult = (c * Nc + q * Nq * RW1 + 0.5 * γ * B' * Ng * RW2) * RFbrg

[(50.00 x 25.80) + (345 x 14.72 x 0.50) + (0.5 x 115 x 4.79 x 16.72 x 0.50)] x 0.45

Qult =2760 psf

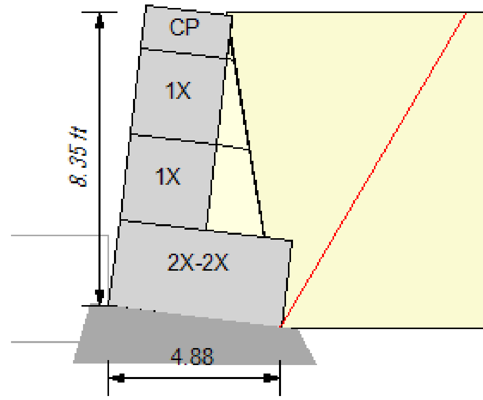
Applied Bearing Pressures (σ) Nbrg/B' =1470 psf

Calculated CDR for bearing Qult/σ =1.88

Design	Sum Vert	Mo	Mr	e	Qult	Sigma	CDR
Strength I-a	5405	5006	13858	0.686	2706	1189	2.27
Strength I-b	7047	5006	17317	0.562	2760	1470	1.88
Service	5010	3337	13078	0.514	2780	1025	2.71

UltraWall

Project:	Murray/Walker Improvements
Location:	Johnson Creek Retaining Wall
Designer:	JDT
Date:	2/22/2019
Section:	Section 1
Design Method:	AASHTO_LRFD_2012
Design Unit:	UltraBlock



Seismic Acc: 0.368

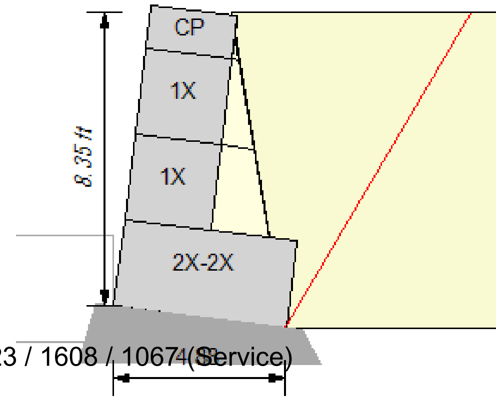
SOIL PARAMETERS	φ	coh	γ
Retained Soil:	28 deg	50 psf	115 pcf
Foundation Soil:	28 deg	50 psf	115 pcf
Leveling Pad:	35 deg	0 psf	130 pcf
	Crushed Stone		

GEOMETRY

Design Height:	8.35 ft	Live Load:	0 psf
Wall Batter/Tilt:	0.0/ 7.13 deg	Live Load Offset:	0.00 ft
Embedment:	2.00 ft	Live Load Width:	0 ft
Leveling Pad Depth:	1.00 ft	Dead Load:	0 psf
Slope Angle:	0.0 deg	Dead Load Offset:	0.0 ft
Slope Length:	0.0 ft	Dead Load Width:	0 ft
Slope Toe Offset:	0.0 ft	Leveling Pad Width:	5.92 ft
Vertical δ on Single Depth		Toe Slope Angle:	26.67
		Toe Slope Length:	0.00
		Toe Slope Bench:	0.00

Note: Calculations and quantities are for PRELIMINARY ANALYTICAL USE ONLY and MUST NOT be used for final design or construction without the independent review, verification, and approval by a qualified professional engineer.

UltraWall 5.0.18138



RESULTS (Static / Seismic)

CDR Sliding: 1.30 (lvlpd) / 1.02 CDR Bearing: 3.58 / 5.70

Eccentricity (e/L): 0.15 (e/L <= 0.33) / [0.23 (e/L <= 0.40)] Bearing: 1623 / 1608 / 1067 (Service)

Ecc Internal(e/L): 0.15 (e/L <= 0.45)

Name	Elev.	ka	kae	Pa	Pae	Pir	- PaC	CDRsl	e/L	siesCDRsl [Pae/Pir]	Seis e/L
CP	7.32	0.279	0.418	29	43	78	35	100.00	-0.03	374.12 / 239.69	0.01 / 0.03
1X	4.88	0.279	0.418	229	343	234	100	86.12	0.00	53.93 / 46.60	0.11 / 0.15
1X	2.44	0.279	0.418	621	928	389	164	24.96	0.15	20.46 / 18.60	0.35 / 0.37
2X-2X	0.00	0.401	0.568	1849	2621	701	284	1.30[1.89]	0.15	1.35 / 1.02	0.26 / 0.23

Column Descriptions:

- ka: active earth pressure coefficient
- Pa: active earth pressure
- Paq: live surcharge earth pressure
- Paqd: dead surcharge earth pressure
- (PaC): reduction in load due to cohesion
- PaT: sum of all earth pressures
- CDRsl (lvlPad): 'Capacity/Demand Ratio' for sliding at each layer. (CDR sliding below the leveling pad)
- e/L: eccentricity/base width ratio
- e/L (Srvs): service state condition eccentricity/base width ratio
- %D/H: ratio of based depth to height (warning for narrow walls, < 35%)

DESIGN DATA

Load Factors for Design

AASHTO Table 3.4.1-1 & 3.4.1-2

Load Case	Str_Max	Str_Min	Extreme Max	Extreme Min	Service
Str I Dead Load (DC)	1.25	0.90	1.00	1.00	1.00
Soil Load Driving (EH)	1.50	0.90	1.00	1.00	1.00
Str I Vert Earth Load (EV)	1.35	1.00	1.00	1.00	1.00
Dead Load Surcharge (ES)	1.50	0.75	1.00	1.00	1.00
Live Load (LL, PL, LS)	1.75	0.00	1.00	1.00	1.00

AASHTO Table 11.5.7-1 & Article 11.5.8

MSE Resistance Case	Strength	Extreme	Service
Bearing Resistance (RFbr)	0.65	0.90	1.00
Sliding Soil to Soil (RFsl)	1.00	1.00	1.00
Sliding Soil to Reinforcement (RFslrf)	1.00	1.00	1.00
Tensile Resistance (RFten)	0.90	1.20	1.00
Pullout Resistance (RFpo)	0.90	1.20	1.00
Overturning Resistance (RFot)	0.60	1.00	1.00

AASHTO Table 10.5.5.2.2-1

Gravity Resistance Case	Resistance Factor	Extreme	Service
Bearing Resistance (RFbr)	0.45	0.80	1.00
Cast-In-Place Concrete to Stone (RFsl_cip)	0.80	1.00	1.00
Precast Concrete to Stone (RFsl_c)	0.90	1.00	1.00
Stone to Soil (RFsl_s)	0.90	1.00	1.00
Passive EP (RFep)	0.50	1.00	1.00

Application of Load Factors

Group	DC	EV	LS	EH	Probable Use
Strength I-a	0.90	1.00	1.75	1.50	BC/EC/SL
Strength I-b	1.25	1.35	1.75	1.50	BC (max. value)
Service I	1.00	1.00	1.00	1.00	Settlement

Notes: BC - Bearing Capacity; EC - Eccentricity; SL - Sliding

By Inspection:

• Strength I-a (minimum vertical loads and maximum horizontal loads) will govern for the case of sliding and eccentricity (overturning); and

• For the case of bearing capacity, maximum vertical loads will govern, and the factored loads must be compared for Strength I-b and Strength IV.

NOTES ON DESIGN UNITS

The wall section is designed on a 'per unit width bases' (lb/ft/ft of wall or kN/m/meter of wall). In the calculations the software shows lb/ft or kN/m, neglecting the unit width factor for simplicity.

The weights for the wall unit are shown as lbs / ft3 (kN / m3). For SRW design a 1 sf unit is typically 1 ft deep, 1.5 ft wide and 8 inches tall (or 1 ft3). therefore a typical value of 120 pcf is shown. With larger units the unit weight will vary with the size of the unit. Say we have 4 ft wide unit, 1.5 ft tall and 24 inches deep with a tapered shape (sides narrow), built with 150 pcf concrete. We add up the concrete, the gravel fill and divide by the volume and the results may come out to 140 pcf, as shown in the table. The units with more gravel may have lower effective unit weights based on the calculations.

Hollow Units

Hollow units with gravel fill are treated differently in AASHTO. If the fill can fall out as the unit is lifted, then AASHTO only allows 80% of the weight of the fill to be used for eccentricity (overturning calculations). In the properties page for the units the weight of the concrete may be as low as 75 pcf. This is the effective unit weight of the concrete only (e.g. the weight of the concrete divided by the volume of the unit). The density of the concrete maybe 150 pcf, but not the effective weight including the volume of the void spaces used for gravel fill.

Rounding Errors

When doing hand calculations the values may vary from the values shown in the software. The program is designed using double precision values (64 bit precision: 14 decimal places). Over several calculations the results may differ from the single calculation the user is making, probably inputting one or two already rounded values.

Result Rounding

As noted above the software is based on double precision values. For example, using an NCMA design method an allowable factor of safety of 1.5 the software may calculate a value of 1.49999999999999, since this is less than 1.5, it would be false (NG), even though the results shown is 1.50 (results are rounded to 2 places on the screen). In the design check we round to 2 decimal places to check against the suggested value (1.49999999999 rounds to 1.50). Given the precision of the calculation, this will provide a safe design even though the 'absolute' value is less than the minimum suggested.

RETAINING WALL UNITS

STRUCTURAL PROPERTIES:

N is the normal force [or factored normal load] on unit to unit interface
The unit to unit shear is N x Tan(0.0) + 17796.0

N is the normal force [or factored normal load] on the base unit
The default leveling pad to base unit shear is 0.8 * ATan(35), or 29 deg. [AASHTO LRFD 10.6.3.4-2] or may be the manufacturer supplied data.
[Note: concrete to concrete has a coefficient of φ0.6 N. 0 [AASHTO LRFD 5.8.4]

Table of Values:

Unit	Ht (in)	Width (in)	Depth (in)	Equiv_Density (pcf)	Equiv_CG (in)
Cap	14.75	59.00	29.50	140.00	14.75
Full	29.50	59.00	29.50	140.00	14.75
Double	29.50	59.00	59.00	140.00	29.50
Triple	29.50	59.00	88.50	140.00	44.25
15 in Tall Unit	14.75	59.00	29.50	140.00	14.75

FORCE DETAILS

The details below shown how the forces and moments are calculated for each force component. The values shown are not factored. All loads are based on a unit width (ppf / kNpm).

Layer	Block Wt	X-Arm	Moment	Soil Wt	X-Arm	Moment
1	423.04	2.14	907.23	11.76	3.44	40.42
2	846.08	1.84	1556.30	164.75	3.44	566.68
3	846.08	1.53	1298.13	386.34	3.50	1353.99
4	1692.15	2.46	4159.88			

Block Weight (Force v) = block: 3,807 X-Arm = 2.21 ft
Soils Block Weight (Force v) = 563 ppf X-Arm = 3.62 ft

Active Earth Pressure Pa = 1,849 ppf
Pa_h (Force H) = Pa cos(δ batter) = 1,849 x cos(21.010.0) = 1,342 ppf
Y-Arm = 2.99 ft
Pa_v (Force V) = Pa sin(δ batter) = 1,849 x sin(21.010.0) = 806 ppf
X-Arm = 4.37 ft

Passive Earth Pressures
Passive earth pressures are used for resistance of the Leveling Pad, but may be extended upward to assist with the resistance of the wall facing for walls that have deep embedments.

Passive Earth Pressure:
kp = 2.77
Pp = 1,444.27 ppf

CALCULATION RESULTS

OVERVIEW

UltraWall calculates stability assuming the wall is a rigid body. Forces and moments are calculated about the base and the front toe of the wall. The base block width is used in the calculations. The concrete units and granular fill over the blocks are used as resisting forces.

EARTH PRESSURES

The method of analysis uses the Coulomb Earth Pressure equation (below) to calculate active earth pressures. Wall friction is assumed to act at the back of the wall face. The component of earth pressure is assumed to act perpendicular to the boundary surface. The effective δ angle is δ minus the wall batter at the back face. If the slope breaks within the failure zone, a trial wedge method of analysis is used.

EXTERNAL EARTH PRESSURES

Effective δ angle (3/4 retained phi)	δ = 21.0 deg
Coefficient of active earth pressure	ka = 0.401
External failure plane	ρ = 58 deg
Back Face Angle from horizontal	Face Angle =80.01 deg
Coefficient of passive earth pressure	kp =2.77

$$Ka := \frac{\cos(\phi_i + i)^2}{\cos(i)^2 \cdot \cos(\delta_i - i)\left(1 + \sqrt{\frac{\sin(\phi_i + \delta_i)\cdot \sin(\phi_i - \beta)}{\cos(\delta_i - i)\cdot \cos(i + \beta)}}\right)^2}$$

FORCES AND MOMENTS

The program resolves all the geometry into simple geometric shapes to make checking easier. All x and y coordinates are referenced to a zero point at the front toe of the base block.

UNFACTORED LOADS

Name	Factor γ	Force (V)	Force (H)	X-len	Y-len	Mo	Mr
Face Blocks(W1)	1.00	3807	--	2.21	--	--	8418
Soil Wedge(W2)	1.00	563	--	3.62	--	--	2040
LvlPad(W18)	1.00	704	--	--	--	--	--
Pa_h	1.00	--	1342	--	2.99	4009	--
Pa_v	1.00	806	--	4.37	--	--	3523
Sum V / H	1.00	5176	1342		Sum Mom	4009	13980

W0: stone within units

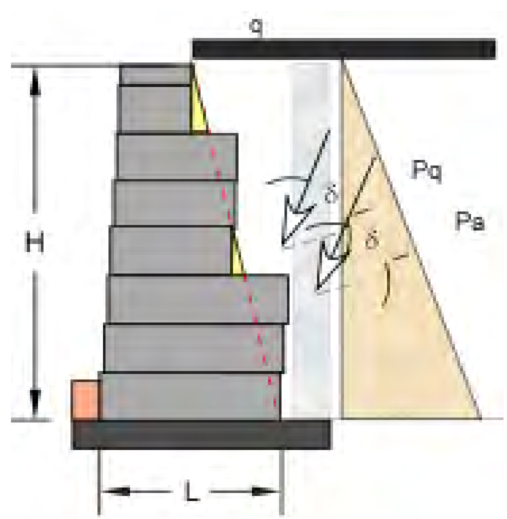
W1: facing units

W2: stone over the tails

W9: Driving force Pa

W10: Driving Surcharge load Paq

W11: Driving Dead Load Surchage Paqd



FORCES AND MOMENTS FACTORED FOR Str I-a

UltraWall increases all driving forces and reduces the resisting forces by the factors shown for Str I.

FACTORED LOADS: Str Ia

Name	Factor γ	Force (V)	Force (H)	X-len	Y-len	Mo	Mr
Face Blocks(W1)	0.90	3427	--	2.21	--	--	7576
Soil Wedge(W2)	1.00	563	--	3.62	--	--	2040
Pa_h	1.50	--	2013	--	2.99	6013	--
Pa_v	1.50	1209	--	4.37	--	--	5284
Sum V / H		5198	2013		Sum Mom	6013	14900

FORCES AND MOMENTS FACTORED FOR Str I-b

UltraWall increase resisting loads and increases driving loads by the factors shown for Str I-b.

FACTORED LOADS: Str Ib

Name	Factor γ	Force (V)	Force (H)	X-len	Y-len	Mo	Mr
Face Blocks(W1)	1.25	4759	--	2.21	--	--	10522
Soil Wedge(W2)	1.35	760	--	3.62	--	--	2753
Pa_h	1.50	--	2013	--	2.99	6013	--
Pa_v	1.50	1209	--	4.37	--	--	5284
Pq_h	1.75	--	0	--	4.48	0	--
Pq_v	1.75	0	--	4.12	--	--	0
Sum V / H		6728	2013		Sum Mom	6013	18560

BASE SLIDING

Sliding at the base is checked at the block to leveling pad interface between the base block and the leveling pad. Sliding is also checked between the leveling pad and the foundation soils.

Forces Resisting sliding = W1(DCr) + W2(EVr) + Pav(EHd)	
(3807 x 0.90) + (563 x 1.00) + (806 x 1.50)	N =5198 ppf
Resisting force at pad = (N tan(slope) + intercept) x RFsl	
(5198 x tan(31.0) + 0.0) x 0.90	Rf1 =2621
where L is the base block width	
N1 includes N + leveling pad	
where LvlPd = lvl pad thickness * 130pcf * L + lvl pad thickness/2	LvlPd = 704
Forces Resisting sliding = W1(DCr) + W2(EVr) + LvlPd(EVr) + Pav(EHd)	
(0 x 0.90) + (0 x 1.00) + (704 x 1.00)	
	N1 =5903 ppf
Friction angle is the lesser of the leveling pad and Fnd	φ =28.00 deg
Passive resistance is calculated using kp = (1 + sin(28))/(1 - sin(28))	kp = 2.77
Force at top of resisting trapezoid, d1 = 2.00	Fp1 = 637.06
Force at base of resisting trapezoid, d2 = 3.62	Fp2 = 1151.49
Depth of trapezoid	depth = 1.62
Pp = (Fp1 + Fp2) / 2 * depth	1444.27
Resisting force at fnd = (N1 tan(phi) + c L) * RFsl + Pp x RFps	
(5903 x tan(28) + 50 x 5.92) * 0.90 + (1444 x 0.50)	
Rf2 = 3811	[the value printed is the minimum

foundation soil.

Driving force is the horizontal component of	
Pah(EHd)	
(1342 x 1.50)	Df =2013
CDR = (Rf1 / Df) / (Rf2 / Df2)	CDR =1.30 / 1.89

ECCENTRICITY ABOUT THE TOE

Eccentricity at the base is checked by assuming rotation about the front toe by the block mass and the soil retained on the blocks. Allowable overturning can be defined by eccentricity (e/L).

Moments resisting eccentricity =
M1(DCr) + M2(EVr) + MPav(EHd)
((8418 x 0.90) + (2040 x 1.00) + (3523 x 1.50)

Mr =14900 ft-lbs

Moments causing eccentricity = MPah(EHd)
(4009 x 1.50) Mo =6013 ft-lbs

e = L/2 - (Mr - Mo)/ N
e =4.92/2 - (14900 - 6013) /5198 e =0.75
e/L = 0.15

Note: Calculations and quantities are for PRELIMINARY ANALYTICAL USE ONLY and MUST NOT be used for final design or construction without the independent review, verification, and approval by a qualified professional engineer.

GRAVITY RESULTS

The table below shows the forces and moments for each layer of face units. For concrete leveling pads, sliding and eccentricity is checked at the base blocks and at the base of the concrete leveling pad.

TABLE OF RESULTS

Layer	Ht	ka	Batter	Delta	W0	Len0X	W1	Len1X	W2	Len2X	Pah	Pav	PaLenY	PaLenX	PqLenY	PqLenX
0	7.32	0.279	7.1	14.00	0	0.00	423	1.30	0	0.00	0	0	0.45	2.48		
1	4.88	0.279	7.1	14.00	0	0.00	1269	1.45	0	2.62	128	0	1.26	2.54		
2	2.44	0.279	7.1	14.00	0	0.00	2115	1.60	1	2.78	453	0	2.07	2.64		
3	0.00	0.401	-10.0	21.00	0	0.00	3807	2.21	563	3.62	1342	806	2.99	4.37		

Layer	M0[0]	M1[1]	Mr[2]	Mlp	MPah	MPav	Forces Re	Forces DR	SumV	Mr	Mo	CDRsl	ecc/L
0	0	548	0	0	0	0	16208	1	381	493	0	100.00	-0.03
1	0	1838	1	0	162	0	16592	193	1143	1655	243	86.12	0.00
2	0	3387	1	0	939	0	16975	680	1904	3049	1409	24.96	0.15
3	0	8418	2040	0	4009	3523	2621	2013	5198	14900	6013	1.30	0.15

ECCENTRICITY AND BEARING

Eccentricity is the calculation of the distance of the resultant force away from the centroid of the mass. This measure is an indication of the overturning of the mass. UltraWall uses an allowable eccentricity of 9/20 L for concrete to concrete bearing surfaces and a concrete leveling pad (thickness > 1.0 ft), or L/3 for bearing on soil per the AASHTO LRFD guidelines. Eccentricity is still used as a guide to design in some design methods.

UltraWall calculates three eccentricities:

1) Maximum eccentricity (overturning) where it uses the maximum driving forces combined with the minimum resisting forces (see overturning) [Str I-a]. 2) Maximum bearing where it uses the maximum driving forces combined with the maximum resisting forces [Str I-b]. 3) Service: Maximum bearing where it uses the actual driving forces combined with the actual resisting forces in Service loading.

Calculation of Eccentricity for maximum bearing
Moments resisting = M1(DCd) + M2(EVd) + MPa(EHd)
(8418 x 1.25) + (2040 x 1.35) + (3523 x 1.50)
Mr =18560 ft-lbs

Moments driving = MPah(EHd)
(4009 x 1.50) Mo =6013 ft-lbs

Nb = W1(DCd) + W2(EVd) + WPa(EHd)
(3807 x 1.25) + (563 x 1.35) + (806 x 1.50)
Nb =6728 ppf

N bearing = W1(DCd) + W2(EVd) + WPlvlpad(EVd) + WPa(EHd)
(3807 x 1.25) + (563 x 1.35) + (704 x 1.35) + (806 x 1.50)
Nbrg =7679 ppf

Calculate Eccentricity
e = L/2 - (Mr - Mo)/ Nb
e =4.92/2 - (18560 - 6013) /6728 e =0.593

B' = B - 2e + lvl pad thickness =
4.92 - 2 x 0.59 + 1.00 B' = 4.73 ft

Calculation of Bearing Pressures
Qult = (c * Nc + q * Nq + 0.5 * γ * B' * Ng) * RFbrg
[(50.00 x 25.80) + (345 x 14.72) + (0.5 x 115 x 4.73 x 16.72)] x 0.45 Qult =4912 psf
Ultimate Bearing, Qult Qult =4912 psf
Applied Bearing Pressures (σ) Nbrg/B' =1623 psf
Calculated CDR for bearing Qult/σ =3.03

Design	Sum Vert	Mo	Mr	e	Qult	Sigma	CDR
Strength I-a	5903	6013	14900	0.749	4777	1336	3.58
Strength I-b	7679	6013	18560	0.593	4912	1623	3.03
Service	5176	4009	13980	0.532	4965	1067	4.65

Note: Calculations and quantities are for PRELIMINARY ANALYTICAL USE ONLY and MUST NOT be used for final design or construction without the independent review, verification, and approval by a qualified professional engineer.

SEISMIC CALCULATIONS

The loads considered under seismic loading are primarily inertial loadings. The wave passes the structure putting the mass into motion and then the mass will try to continue in the direction of the initial wave. In the calculations you see the one dynamic earth pressure from the wedge of the soil behind the reinforced mass, and then all the other forces come from inertia calculations of the face put into motion and then trying to be held in place.

Design Ground Acceleration As = Fpae * PGA = 0.368

Assumed deformation = Def = 3.00 in

(with deformation, the kh is reduced by 50%)

Horizontal Acceleration kh =0.184

Vertical Acceleration kv =0.000

INERTIA FORCES OF THE STRUCTURE

Pif = (W1 * kh)

(3807.34 * 0.184) Pif = 613.60

SEISMIC THRUST

Coefficient of active seismic earth pressure Kae Kae =0.568

D_Kae = Kae - Ka = (0.568 -0.401) D_Kae =0.167

Pae = 0.5*gamma*(H)^2 * DKae Pae =771.46 ppf

Pae_h = Pae*cos(delta) Pae_h =661.37 ppf

Pae_v = Pae*sin(delta) Pae_v =397.17 ppf

In AASHTO LRFD, two cases are looked at: 1) 100% Pae and 50% of Pir, and 2) 50% Pae and 100% Pir.

TABLE OF RESULTS FOR SEISMIC REACTIONS

Name	Force (V)	Force (H)	X-len	Y-len	Mo	Mr
Face Blocks(W1)	3807	--	2.21	--	--	8418
Soil Block(W2)	563	--	3.62	--	--	2040
Pa_h	--	1342	--	2.99	4009	--
Pa_v	806	--	4.37	--	--	3523
Pif	--	614	--	5.38	3299	--
Pae_h	--	661	--	5.38	3556	--
Pae_v	397	--	4.37	--	--	1736

SEISMIC SLIDING

Details are only shown for sliding at the base of blocks, a check is made at the foundation level with the answer only shown.

The vertical resisting forces are
Units(1.0) + SoilWedge(1.0) + Pa_v(1.0) + Pae_v(1.0)

(3807 * 1.0) + (563 * 1.0) + (806 * 1.0) + (397 * 1.0)
Note Pae_v changes between case 1 and case 2, to Pae_v/2)

Case 1, 100% Pae & 50% Pir
SumVs_Pae = 5573

Case 2, 50% Pae & 100% Pir
SumVs_Pir = 4773

Resisting force 1 = (SumVs_Pae * tan(slope) + intercept * L) * RFsl FReS1 =3122 ppf
Resisting force 2 = (SumVs_Pir * tan(slope) + intercept * L) * RFsl FReS2 =2674 ppf

The inertial force Pif = Face*kh(1.0) + Soil Wedge*kh*(1.0)
Driving force = + Pa_h(1.0) + Pif_h/2(1.0) + Pae_h(1.0)
[case 2 is 50% Pae, 100% Pir]

(1342 * 1.0) + (614/2 * 1.0) + (661 * 1.0)
FDrS1 =2310 ppf
FDrS2 =2627 ppf

CDR = (FReS1/FDrS1) / (FReS2/FDrS2) CDR =1.35 / 1.02

SEISMIC ECCENTRICITY

Eccentricity is rotation about the front toe of the wall and is a check on overturning.

Resisting Moment = Units(1.0) + SoilWedge(1.0) + Pa_v(1.0) + Pae_v(1.0)
(case 2 is 50% Pae)
(8418 * 1.0) + (2040 * 1.0) + (3523 * 1.0) + (1736 * 1.0)
MomReS [Pae] =0 ft ppf
MomReS [Pir] =0 ft ppf

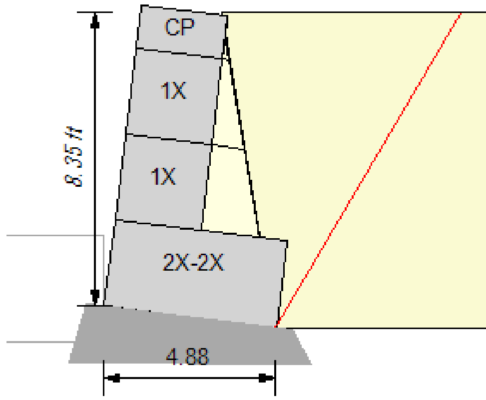
Driving Moment =
Pa_h(1.0) + Pif_h/2(1.0) + Pae_h(1.0)
(4009 * 1.0) + (3299/2 * 1.0) + (3556 * 1.0)
MomDrS [Pae] =9214 ft ppf
MomDrS [Pir] =9312 ft ppf

e = L/2 - (Mr - Mo)/ N
e [Pae] =4.92/2 - (15716 - 9214) /5573 e =0.00
e/L = 0.00 e [Pir] =4.92/2 - (15742 - 9312) /4773
e/L = 0.00

SEISMIC BEARING

UltraWall

Project:	Murray/Walker Improvements
Location:	Johnson Creek Retaining Wall
Designer:	JDT
Date:	2/22/2019
Section:	Section 1
Design Method:	AASHTO_LRFD_2012
Design Unit:	UltraBlock



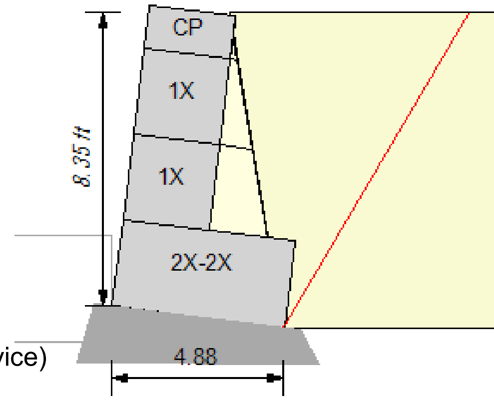
SOIL PARAMETERS	φ	coh	γ	e	γsat
Retained Soil:	28 deg	50 psf	115 pcf	0.40	138 pcf
Foundation Soil:	28 deg	50 psf	115 pcf	0.40	138 pcf
Leveling Pad:	35 deg	0 psf	130 pcf	0.40	138 pcf
	Crushed Stone				

GEOMETRY

Design Height:	8.35 ft	Live Load:	0 psf
Wall Batter/Tilt:	0.0/ 7.13 deg	Live Load Offset:	0.00 ft
Embedment:	2.00 ft	Live Load Width:	0 ft
Leveling Pad Depth:	1.00 ft	Dead Load:	0 psf
Slope Angle:	0.0 deg	Dead Load Offset:	0.0 ft
Slope Length:	0.0 ft	Dead Load Width:	0 ft
Slope Toe Offset:	0.0 ft	Leveling Pad Width:	5.92 ft
Vertical δ on Single Depth		Toe Slope Angle:	26.67
		Toe Slope Length:	0.00
		Toe Slope Bench:	0.00
Water Level:	3.70 ft		
Water Back of Face:	3.70 ft		
Drain Depth:	1.00 ft		

Note: Calculations and quantities are for PRELIMINARY ANALYTICAL USE ONLY and MUST NOT be used for final design or construction without the independent review, verification, and approval by a qualified professional engineer.

UltraWall 5.0.18138



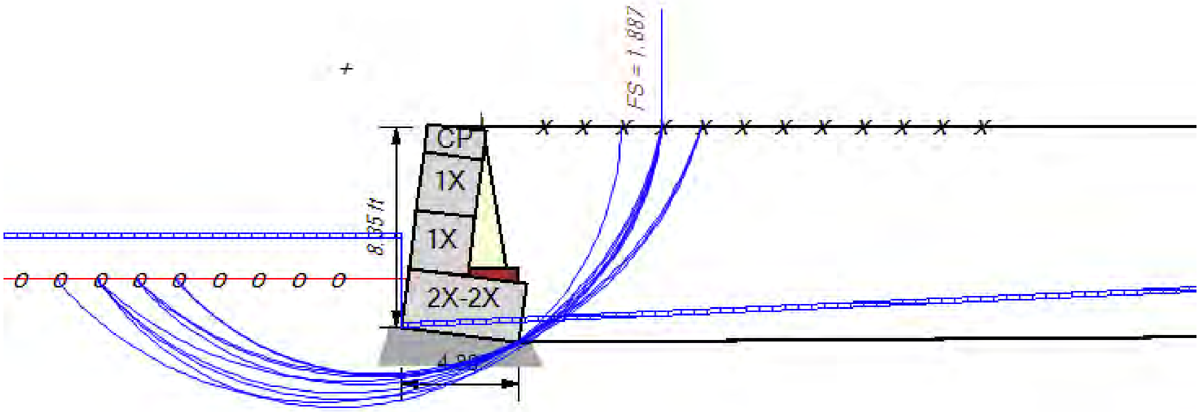
RESULTS

CDR Sliding:	1.30 (lvlpd)	CDR Bearing:	3.58
Eccentricity (e/L):	0.15 (e/L <= 0.33)	Bearing:	1623 / 1067 (Service)
Ecc Internal(e/L):	0.15 (e/L <= 0.45)		

Name	Elev.[dpth]	ka	Pa	Paq	Paqd	(PaC)	PaT	CDRsl(Lvl Pad)	e/L	e/L (Srvs)	%D/H
CP	7.32[1.03]	0.279	29	0	0	35	0	100.00	--	--	200%
1X	4.88[3.47]	0.279	229	0	0	100	129	86.12	--	--	71%
1X	2.44[5.91]	0.279	621	0	0	164	457	24.96	0.15	0.03	42%
2X-2X	0.00[8.35]	0.401	1849	0	0	284	1566	1.30 (1.89)	0.15	0.11	59%

Column Descriptions:

- ka: active earth pressure coefficient
- Paw1: active earth pressure of soils above the water line
- Paw2: active earth pressures (1) of submerged soils below the water line
- Paw3: active earth pressures (2) of submerged soils below the water line
- Pwd: driving pressures of water from behind the face
- Pwr: resisting pressures of water in front of wall
- Paq: live surcharge earth pressure
- Paqd: dead surcharge earth pressure
- (PaC): reduction in load due to cohesion
- PaT: sum of all earth pressures
- CDRsl (lvlPad): 'Capacity/Demand Ratio' for sliding at each layer. (CDR sliding below the leveling pad)
- e/L: eccentricity/base width ratio
- e/L (Srvs): service state condition eccentricity/base width ratio
- %D/H: ratio of based depth to height (warning for narrow walls, < 35%)



FS = 1.887
CP
1X
1X
2X-2X
8.35 ft

GLOBAL RESULTS

Global stability is a global analysis (Bishop) with the failure planes originating at the top of the slope / wall and exiting out below the wall in the area infront of the structure. For MSE walls, the resistance of the geogrid reinforcement is included in the analysis. The curve may go through the base of the wall and the wall shear would be included. In most cases the failure plane will pass below the structure.

ID	Enter Point X	Enter Point Y	Exit Point X	Exit Point Y	Center X	Center Y	Radius	FoS
1	10.89	8.35	-12.77	2.00	-2.45	10.80	13.56	1.887
1	10.89	8.35	-11.10	2.00	-1.56	10.20	12.58	1.904
1	10.89	8.35	-9.43	2.00	-0.65	9.60	11.61	1.917
2	10.89	8.35	-9.43	2.00	-0.64	9.56	11.59	1.920
2	10.89	8.35	-12.77	2.00	-2.33	10.37	13.38	1.929
1	10.89	8.35	-14.44	2.00	-3.33	11.39	14.55	1.929
1	12.56	8.35	-11.10	2.00	-1.20	12.37	14.34	1.945
1	12.56	8.35	-12.77	2.00	-2.10	13.15	15.43	1.946
2	12.56	8.35	-11.10	2.00	-1.14	12.14	14.22	1.953
1	9.22	8.35	-12.77	2.00	-2.77	8.63	11.99	1.954

DESIGN DATA

Load Factors for Design

AASHTO Table 3.4.1-1 & 3.4.1-2

Load Case	Str_Max	Str_Min	Extreme Max	Extreme Min	Service
Str I Dead Load (DC)	1.25	0.90	1.00	1.00	1.00
Soil Load Driving (EH)	1.50	0.90	1.00	1.00	1.00
Str I Vert Earth Load (EV)	1.35	1.00	1.00	1.00	1.00
Dead Load Surcharge (ES)	1.50	0.75	1.00	1.00	1.00
Live Load (LL, PL, LS)	1.75	0.00	1.00	1.00	1.00

AASHTO Table 11.5.7-1 & Article 11.5.8

MSE Resistance Case	Strength	Extreme	Service
Bearing Resistance (RFbr)	0.65	0.90	1.00
Sliding Soil to Soil (RFsl)	1.00	1.00	1.00
Sliding Soil to Reinforcement (RFslrf)	1.00	1.00	1.00
Tensile Resistance (RFten)	0.90	1.20	1.00
Pullout Resistance (RFpo)	0.90	1.20	1.00
Overturning Resistance (RFot)	0.60	1.00	1.00

AASHTO Table 10.5.5.2.2-1

Gravity Resistance Case	Resistance Factor	Extreme	Service
Bearing Resistance (RFbr)	0.45	0.80	1.00
Cast-In-Place Concrete to Stone (RFsl_cip)	0.80	1.00	1.00
Precast Concrete to Stone (RFsl_c)	0.90	1.00	1.00
Stone to Soil (RFsl_s)	0.90	1.00	1.00
Passive EP (RFep)	0.50	1.00	1.00

Application of Load Factors

Group	DC	EV	LS	EH	Probable Use
Strength I-a	0.90	1.00	1.75	1.50	BC/EC/SL
Strength I-b	1.25	1.35	1.75	1.50	BC (max. value)
Service I	1.00	1.00	1.00	1.00	Settlement

Notes: BC - Bearing Capacity; EC - Eccentricity; SL - Sliding

By Inspection:

• Strength I-a (minimum vertical loads and maximum horizontal loads) will govern for the case of sliding and eccentricity (overturning); and

• For the case of bearing capacity, maximum vertical loads will govern, and the factored loads must be compared for Strength I-b and Strength IV.

NOTES ON DESIGN UNITS

The wall section is designed on a 'per unit width bases' (lb/ft/ft of wall or kN/m/meter of wall). In the calculations the software shows lb/ft or kN/m, neglecting the unit width factor for simplicity.

The weights for the wall unit are shown as lbs / ft3 (kN / m3). For SRW design a 1 sf unit is typically 1 ft deep, 1.5 ft wide and 8 inches tall (or 1 ft3). therefore a typical value of 120 pcf is shown. With larger units the unit weight will vary with the size of the unit. Say we have 4 ft wide unit, 1.5 ft tall and 24 inches deep with a tapered shape (sides narrow), built with 150 pcf concrete. We add up the concrete, the gravel fill and divide by the volume and the results may come out to 140 pcf, as shown in the table. The units with more gravel may have lower effective unit weights based on the calculations.

Hollow Units
Hollow units with gravel fill are treated differently in AASHTO. If the fill can fall out as the unit is lifted, then AASHTO only allows 80% of the weight of the fill to be used for eccentricity (overturning calculations). In the properties page for the units the weight of the concrete may be as low as 75 pcf. This is the effective unit weight of the concrete only (e.g. the weight of the concrete divided by the volume of the unit). The density of the concrete maybe 150 pcf, but not the effective weight including the volume of the void spaces used for gravel fill.

Rounding Errors
When doing hand calculations the values may vary from the values shown in the software. The program is designed using double precision values (64 bit precision: 14 decimal places). Over several calculations the results may differ from the single calculation the user is making, probably inputting one or two already rounded values.

Result Rounding
As noted above the software is based on double precision values. For example, using an NCMA design method an allowable factor of safety of 1.5 the software may calculate a value of 1.49999999999999, since this is less than 1.5, it would be false (NG), even though the results shown is 1.50 (results are rounded to 2 places on the screen). In the design check we round to 2 decimal places to check against the suggested value (1.49999999999 rounds to 1.50). Given the precision of the calculation, this will provide a safe design even though the 'absolute' value is less than the minimum suggested.

RETAINING WALL UNITS

STRUCTURAL PROPERTIES:

N is the normal force [or factored normal load] on unit to unit interface
The unit to unit shear is N x Tan(0.0) + 17796.0

N is the normal force [or factored normal load] on the base unit
The default leveling pad to base unit shear is 0.8 * ATan(35), or 29 deg. [AASHTO LRFD 10.6.3.4-2] or may be the manufacturer supplied data.
[Note: concrete to concrete has a coefficient of φ0.6 N. 0 [AASHTO LRFD 5.8.4]

Table of Values:

Unit	Ht (in)	Width (in)	Depth (in)	Equiv_Density (pcf)	Equiv_CG (in)
Cap	14.75	59.00	29.50	140.00	14.75
Full	29.50	59.00	29.50	140.00	14.75
Double	29.50	59.00	59.00	140.00	29.50
Triple	29.50	59.00	88.50	140.00	44.25
15 in Tall Unit	14.75	59.00	29.50	140.00	14.75

FORCE DETAILS

The details below shown how the forces and moments are calculated for each force component. The values shown are not factored. All loads are based on a unit width (ppf / kNpm).

With hydrostatic forces, the 'w' in the index indicates the unit is under water.

Layer	Block Wt	X-Arm	Moment	Soil Wt	X-Arm	Moment
1	423.04	2.14	907.23	11.76	3.44	40.42
2	846.08	1.84	1556.30	164.75	3.44	566.68
3	846.08	1.53	1298.13	386.34	3.50	1353.99
4	1692.15	2.46	4159.88			

Block Weight (Force v) = block: 3,807 X-Arm = 2.21 ft
Soils Block Weight (Force v) = 563 ppf X-Arm = 3.62 ft

Active Earth Pressure Pa = (Paw1 + Paw2 + Paw3) = 0 ppf
Active Earth Pressure Paw1 = 0 ppf
Active Earth Pressure Paw2 = 0 ppf
Active Earth Pressure Paw3 = 0 ppf
Pa_h (Force H) = (Paw1 + Paw2 + Paw3) cos(δ batter) = 0 x cos(21.010.0 +) = 1,342 ppf
Y-Arm = 2.99 ft
Pa_v (Force V) = Pa sin(batter + δ) = 0 x sin(21.010.0) = 806 ppf
X-Arm = 4.37 ft

Passive Earth Pressures
Passive earth pressures are used for resistance of the Leveling Pad, but may be extended upward to assist with the resistance of the wall facing for walls that have deep embedments.

Passive Earth Pressure:
kp = 2.77
Pp = 1,444.27 ppf

CALCULATION RESULTS

OVERVIEW

UltraWall calculates stability assuming the wall is a rigid body. Forces and moments are calculated about the base and the front toe of the wall. The base block width is used in the calculations. The concrete units and granular fill over the blocks are used as resisting forces.

EARTH PRESSURES

The method of analysis uses the Coulomb Earth Pressure equation (below) to calculate active earth pressures. Wall friction is assumed to act at the back of the wall face. The component of earth pressure is assumed to act perpendicular to the boundary surface. The effective δ angle is δ minus the wall batter at the back face. If the slope breaks within the failure zone, a trial wedge method of analysis is used.

EXTERNAL EARTH PRESSURES

Effective δ angle (3/4 retained phi)	δ = 21.0 deg
Coefficient of active earth pressure	ka = 0.401
External failure plane	ρ = 58 deg
Back Face Angle from horizontal	Face Angle =80.01 deg
Coefficient of passive earth pressure	kp =2.77

$$Ka := \frac{\cos(\phi_i + i)^2}{\cos(i)^2 \cdot \cos(\delta_i - i)\left(1 + \sqrt{\dfrac{\sin(\phi_i + \delta_i)\cdot \sin(\phi_i - \beta)}{\cos(\delta_i - i)\cdot \cos(i + \beta)}}\right)^2}$$

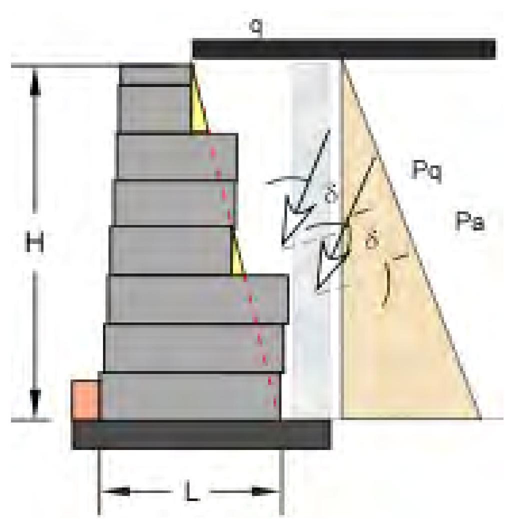
FORCES AND MOMENTS

The program resolves all the geometry into simple geometric shapes to make checking easier. All x and y coordinates are referenced to a zero point at the front toe of the base block.

UNFACTORED LOADS

Name	Factor γ	Force (V)	Force (H)	X-len	Y-len	Mo	Mr
Face Blocks(W1)	1.00	3807	--	2.21	--	--	8418
Soil Wedge(W2)	1.00	563	--	3.62	--	--	2040
LvlPad(W18)	1.00	704	--	--	--	--	--
Pa_h	1.00	--	1342	--	2.99	4009	--
Pa_v	1.00	806	--	4.37	--	--	3523
Sum V / H	1.00	5176	1342		Sum Mom	4009	13980

W0: stone within units
W1: facing units
W2: stone over the tails
W9: Driving force Pa
W10: Driving Surcharge load Paq
W11: Driving Dead Load Surchage Paqd



FORCES AND MOMENTS FACTORED FOR Str I-a

UltraWall increases all driving forces and reduces the resisting forces by the factors shown for Str I.

FACTORED LOADS: Str Ia

Name	Factor γ	Force (V)	Force (H)	X-len	Y-len	Mo	Mr
Face Blocks(W1)	0.90	3427	--	2.21	--	--	7576
Soil Wedge(W2)	1.00	563	--	3.62	--	--	2040
Pa_h	1.50	--	2013	--	2.99	6013	--
Pa_v	1.50	1209	--	4.37	--	--	5284
Sum V / H		5198	2013		Sum Mom	6013	14900

FORCES AND MOMENTS FACTORED FOR Str I-b

UltraWall increase resisting loads and increases driving loads by the factors shown for Str I-b.

FACTORED LOADS: Str Ib

Name	Factor γ	Force (V)	Force (H)	X-len	Y-len	Mo	Mr
Face Blocks(W1)	1.25	4759	--	2.21	--	--	10522
Soil Wedge(W2)	1.35	760	--	3.62	--	--	2753
Pa_h	1.50	--	2013	--	2.99	6013	--
Pa_v	1.50	1209	--	4.37	--	--	5284
Pq_h	1.75	--	0	--	4.48	0	--
Pq_v	1.75	0	--	4.12	--	--	0
Sum V / H		6728	2013		Sum Mom	6013	18560

BASE SLIDING

Sliding at the base is checked at the block to leveling pad interface between the base block and the leveling pad. Sliding is also checked between the leveling pad and the foundation soils.

Forces Resisting sliding = W1(DCr) + W2(EVr) + Pav(EHd)	
(3807 x 0.90) + (563 x 1.00) + (806 x 1.50)	N =5198 ppf
Resisting force at pad = (N tan(slope) + intercept) x RFsl	
(5198 x tan(31.0) + 0.0) x 0.90	Rf1 =2621
where L is the base block width	
N1 includes N + leveling pad	
where LvlPd = lvl pad thickness * 76pcf * L + lvl pad thickness/2	LvlPd = 704
Forces Resisting sliding = W1(DCr) + W2(EVr) + LvlPd(EVr) + Pav(EHd)	
(0 x 0.90) + (0 x 1.00) + (704 x 1.00)	
	N1 =5903 ppf
Friction angle is the lesser of the leveling pad and Fnd	φ =28.00 deg
Passive resistance is calculated using kp = (1 + sin(28))/(1 - sin(28))	kp = 2.77
Force at top of resisting trapezoid, d1 = 2.00	Fp1 = 637.06
Force at base of resisting trapezoid, d2 = 3.62	Fp2 = 1151.49
Depth of trapezoid	depth = 1.62
Pp = (Fp1 + Fp2) / 2 * depth	1444.27
Resisting force at fnd = (N1 tan(phi) + c L) * RFsl + Pp x RFps	
(5903 x tan(28) + 50 x 5.92) * 0.90 + (1444 x 0.50)	
Rf2 = 3811	[the value printed is the minimum

foundation soil.

Driving force is the horizontal component of	
Pah(EHd)	
(1342 x 1.50)	Df =2013
CDR = (Rf1 / Df) / (Rf2 / Df2)	CDR =1.30 / 1.89

Note: Calculations and quantities are for PRELIMINARY ANALYTICAL USE ONLY and MUST NOT be used for final design or construction without the independent review, verification, and approval by a qualified professional engineer.

ECCENTRICITY ABOUT THE TOE

Eccentricity at the base is checked by assuming rotation about the front toe by the block mass and the soil retained on the blocks. Allowable overturning can be defined by eccentricity (e/L).

Moments resisting eccentricity =
M1(DCr) + M2(EVr) + MPav(EHd)
((8418 x 0.90) + (2040 x 1.00) + (3523 x 1.50)

Mr =14900 ft-lbs

Moments causing eccentricity = MPah(EHd)
(4009 x 1.50) Mo =6013 ft-lbs

e = L/2 - (Mr - Mo)/ N
e =4.92/2 - (14900 - 6013) /5198 e =0.75
e/L = 0.15

GRAVITY RESULTS

The table below shows the forces and moments for each layer of face units. For concrete leveling pads, sliding and eccentricity is checked at the base blocks and at the base of the concrete leveling pad.

TABLE OF RESULTS

Layer	Ht	ka	Batter	Delta	W0	Len0X	W1	Len1X	W2	Len2X	Pah	Pav	PaLenY	PaLenX	PqLenY	PqLenX
0	7.32	0.279	7.1	14.00	0	0.00	423	1.30	0	0.00	0	0	0.45	2.48		
1	4.88	0.279	7.1	14.00	0	0.00	1269	1.45	0	2.62	128	0	1.26	2.54		
2	2.44	0.279	7.1	14.00	0	0.00	2115	1.60	1	2.78	453	0	2.07	2.64		
3	0.00	0.401	-10.0	21.00	0	0.00	3807	2.21	563	3.62	1342	806	2.99	4.37		

Layer	M0[0]	M1[1]	Mr[2]	Mlp	MPah	MPav	Forces Re	Forces DR	SumV	Mr	Mo	CDRsl	ecc/L
0	0	548	0	0	0	0	16208	1	381	493	0	100.00	-0.03
1	0	1838	1	0	162	0	16592	193	1143	1655	243	86.12	0.00
2	0	3387	1	0	939	0	16975	680	1904	3049	1409	24.96	0.15
3	0	8418	2040	0	4009	3523	2621	2013	5198	14900	6013	1.30	0.15

ECCENTRICITY AND BEARING

Eccentricity is the calculation of the distance of the resultant force away from the centroid of the mass. This measure is an indication of the overturning of the mass. UltraWall uses an allowable eccentricity of 9/20 L for concrete to concrete bearing surfaces and a concrete leveling pad (thickness > 1.0 ft), or L/3 for bearing on soil per the AASHTO LRFD guidelines. Eccentricity is still used as a guide to design in some design methods.

UltraWall calculates three eccentricities:

1) Maximum eccentricity (overturning) where it uses the maximum driving forces combined with the minimum resisting forces (see overturning) [Str I-a]. 2) Maximum bearing where it uses the maximum driving forces combined with the maximum resisting forces [Str I-b]. 3) Service: Maximum bearing where it uses the actual driving forces combined with the actual resisting forces in Service loading.

Calculation of Eccentricity for maximum bearing

Moments resisting = M1(DCd) + M2(EVd) + MPa(EHd)

(8418 x 1.25) + (2040 x 1.35) + (3523 x 1.50)

Mr =18560 ft-lbs

Moments driving = MPah(EHd)

(4009 x 1.50) Mo =6013 ft-lbs

Nb = W1(DCd) + W2(EVd) + WPa(EHd)

(3807 x 1.25) + (563 x 1.35) + (806 x 1.50)

Nb =6728 ppf

N bearing = W1(DCd) + W2(EVd) + WPlvlpad(EVd) + WPa(EHd)

(3807 x 1.25) + (563 x 1.35) + (704 x 1.35) + (806 x 1.50)

Nbrg =7679 ppf

Calculate Eccentricity

e = L/2 - (Mr - Mo)/ Nb

e =4.92/2 - (18560 - 6013) /6728 e =0.593

B' = B - 2e + lvl pad thickness =

4.92 - 2 x 0.59 + 1.00 B' = 4.73 ft

Calculation of Bearing Pressures

Qult = (c * Nc + q * Nq * RW1 + 0.5 * γ * B' * Ng * RW2) * RFbrg

[(50.00 x 25.80) + (345 x 14.72 x 1.00) + (0.5 x 115 x 4.73 x 16.72 x 1.00)] x 0.45

Qult =4912 psf

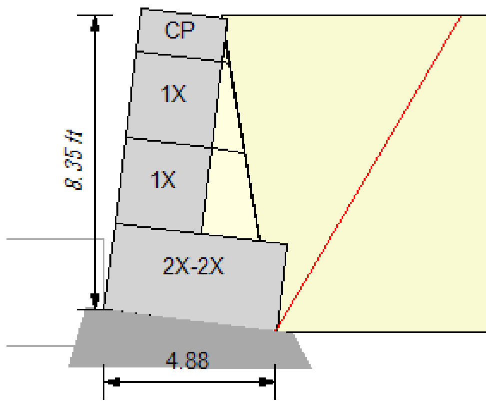
Applied Bearing Pressures (σ) Nbrg/B' =1623 psf

Calculated CDR for bearing Qult/σ =3.03

Design	Sum Vert	Mo	Mr	e	Qult	Sigma	CDR
Strength I-a	5903	6013	14900	0.749	4777	1336	3.58
Strength I-b	7679	6013	18560	0.593	4912	1623	3.03
Service	5176	4009	13980	0.532	4965	1067	4.65

UltraWall

Project:	Murray/Walker Improvements
Location:	Johnson Creek Retaining Wall
Designer:	JDT
Date:	2/22/2019
Section:	Section 1
Design Method:	AASHTO_LRFD_2012
Design Unit:	UltraBlock



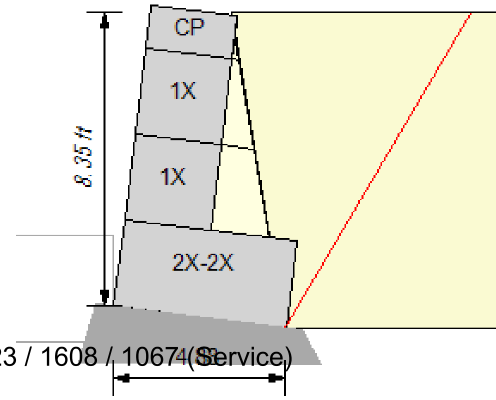
Seismic Acc: 0.368

SOIL PARAMETERS	φ	coh	γ
Retained Soil:	28 deg	50 psf	115 pcf
Foundation Soil:	28 deg	50 psf	115 pcf
Leveling Pad:	35 deg	0 psf	130 pcf
	Crushed Stone		

GEOMETRY

Design Height:	8.35 ft	Live Load:	0 psf
Wall Batter/Tilt:	0.0/ 7.13 deg	Live Load Offset:	0.00 ft
Embedment:	2.00 ft	Live Load Width:	0 ft
Leveling Pad Depth:	1.00 ft	Dead Load:	0 psf
Slope Angle:	0.0 deg	Dead Load Offset:	0.0 ft
Slope Length:	0.0 ft	Dead Load Width:	0 ft
Slope Toe Offset:	0.0 ft	Leveling Pad Width:	5.92 ft
Vertical δ on Single Depth		Toe Slope Angle:	26.67
		Toe Slope Length:	0.00
		Toe Slope Bench:	0.00

Note: Calculations and quantities are for PRELIMINARY ANALYTICAL USE ONLY and MUST NOT be used for final design or construction without the independent review, verification, and approval by a qualified professional engineer.



RESULTS (Static / Seismic)

CDR Sliding: 1.30 (lvlpd) / 1.02 CDR Bearing: 3.58 / 5.70

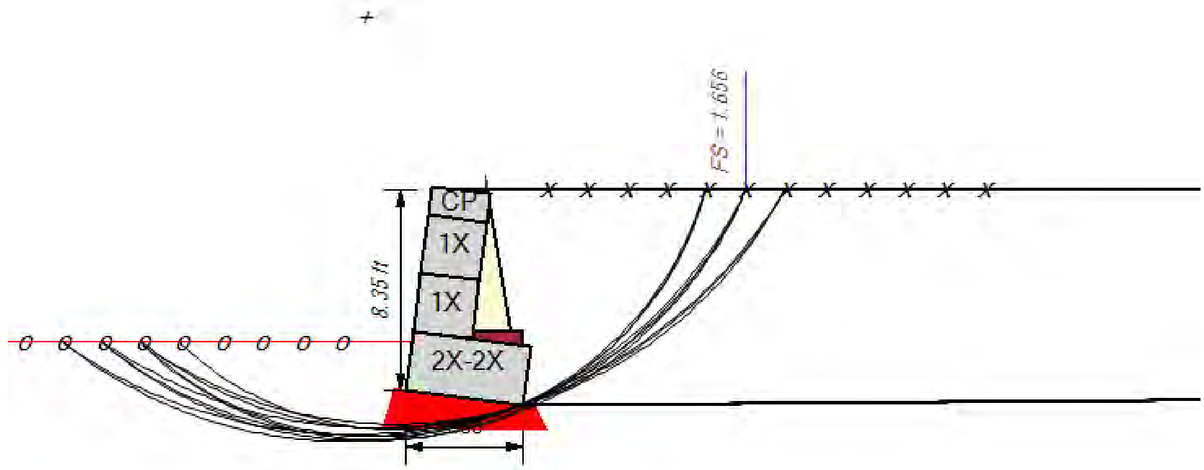
Eccentricity (e/L): 0.15 (e/L <= 0.33) / [0.23 (e/L <= 0.40)] Bearing: 1623 / 1608 / 1067 (Service)

Ecc Internal(e/L): 0.15 (e/L <= 0.45)

Name	Elev.	ka	kae	Pa	Pae	Pir	- PaC	CDRsl	e/L	siesCDRsl [Pae/Pir]	Seis e/L
CP	7.32	0.279	0.418	29	43	78	35	100.00	-0.03	374.12 / 239.69	0.01 / 0.03
1X	4.88	0.279	0.418	229	343	234	100	86.12	0.00	53.93 / 46.60	0.11 / 0.15
1X	2.44	0.279	0.418	621	928	389	164	24.96	0.15	20.46 / 18.60	0.35 / 0.37
2X-2X	0.00	0.401	0.568	1849	2621	701	284	1.30[1.89]	0.15	1.35 / 1.02	0.26 / 0.23

Column Descriptions:

- ka: active earth pressure coefficient
- Pa: active earth pressure
- Paq: live surcharge earth pressure
- Paqd: dead surcharge earth pressure
- (PaC): reduction in load due to cohesion
- PaT: sum of all earth pressures
- CDRsl (lvlPad): 'Capacity/Demand Ratio' for sliding at each layer. (CDR sliding below the leveling pad)
- e/L: eccentricity/base width ratio
- e/L (Srvs): service state condition eccentricity/base width ratio
- %D/H: ratio of based depth to height (warning for narrow walls, < 35%)



FS = 1.656
CP
1X
1X
2X-2X
8.35 ft

GLOBAL RESULTS

Global stability is a global analysis (Bishop) with the failure planes originating at the top of the slope / wall and exiting out below the wall in the area infront of the structure. For MSE walls, the resistance of the geogrid reinforcement is included in the analysis. The curve may go through the base of the wall and the wall shear would be included. In most cases the failure plane will pass below the structure.

ID	Enter Point X	Enter Point Y	Exit Point X	Exit Point Y	Center X	Center Y	Radius	FoS
1	14.23	8.35	-12.77	2.00	-1.73	15.65	17.55	1.656
1	14.23	8.35	-11.10	2.00	-0.82	14.70	16.34	1.664
1	12.56	8.35	-11.10	2.00	-1.20	12.37	14.34	1.666
1	15.90	8.35	-14.44	2.00	-2.25	19.42	21.26	1.674
2	14.23	8.35	-12.77	2.00	-1.61	15.14	17.24	1.676
1	15.90	8.35	-12.77	2.00	-1.34	18.30	19.90	1.679
1	12.56	8.35	-9.43	2.00	-0.29	11.59	13.25	1.679
1	14.23	8.35	-14.44	2.00	-2.63	16.59	18.77	1.682
2	12.56	8.35	-11.10	2.00	-1.14	12.14	14.22	1.684
1	15.90	8.35	-11.10	2.00	-0.42	17.16	18.55	1.684

DESIGN DATA

Load Factors for Design

AASHTO Table 3.4.1-1 & 3.4.1-2

Load Case	Str_Max	Str_Min	Extreme Max	Extreme Min	Service
Str I Dead Load (DC)	1.25	0.90	1.00	1.00	1.00
Soil Load Driving (EH)	1.50	0.90	1.00	1.00	1.00
Str I Vert Earth Load (EV)	1.35	1.00	1.00	1.00	1.00
Dead Load Surcharge (ES)	1.50	0.75	1.00	1.00	1.00
Live Load (LL, PL, LS)	1.75	0.00	1.00	1.00	1.00

AASHTO Table 11.5.7-1 & Article 11.5.8

MSE Resistance Case	Strength	Extreme	Service
Bearing Resistance (RFbr)	0.65	0.90	1.00
Sliding Soil to Soil (RFsl)	1.00	1.00	1.00
Sliding Soil to Reinforcement (RFslrf)	1.00	1.00	1.00
Tensile Resistance (RFten)	0.90	1.20	1.00
Pullout Resistance (RFpo)	0.90	1.20	1.00
Overturning Resistance (RFot)	0.60	1.00	1.00

AASHTO Table 10.5.5.2.2-1

Gravity Resistance Case	Resistance Factor	Extreme	Service
Bearing Resistance (RFbr)	0.45	0.80	1.00
Cast-In-Place Concrete to Stone (RFsl_cip)	0.80	1.00	1.00
Precast Concrete to Stone (RFsl_c)	0.90	1.00	1.00
Stone to Soil (RFsl_s)	0.90	1.00	1.00
Passive EP (RFep)	0.50	1.00	1.00

Application of Load Factors

Group	DC	EV	LS	EH	Probable Use
Strength I-a	0.90	1.00	1.75	1.50	BC/EC/SL
Strength I-b	1.25	1.35	1.75	1.50	BC (max. value)
Service I	1.00	1.00	1.00	1.00	Settlement

Notes: BC - Bearing Capacity; EC - Eccentricity; SL - Sliding

By Inspection:

• Strength I-a (minimum vertical loads and maximum horizontal loads) will govern for the case of sliding and eccentricity (overturning); and

• For the case of bearing capacity, maximum vertical loads will govern, and the factored loads must be compared for Strength I-b and Strength IV.

NOTES ON DESIGN UNITS

The wall section is designed on a 'per unit width bases' (lb/ft/ft of wall or kN/m/meter of wall). In the calculations the software shows lb/ft or kN/m, neglecting the unit width factor for simplicity.

The weights for the wall unit are shown as lbs / ft3 (kN / m3). For SRW design a 1 sf unit is typically 1 ft deep, 1.5 ft wide and 8 inches tall (or 1 ft3). therefore a typical value of 120 pcf is shown. With larger units the unit weight will vary with the size of the unit. Say we have 4 ft wide unit, 1.5 ft tall and 24 inches deep with a tapered shape (sides narrow), built with 150 pcf concrete. We add up the concrete, the gravel fill and divide by the volume and the results may come out to 140 pcf, as shown in the table. The units with more gravel may have lower effective unit weights based on the calculations.

Hollow Units

Hollow units with gravel fill are treated differently in AASHTO. If the fill can fall out as the unit is lifted, then AASHTO only allows 80% of the weight of the fill to be used for eccentricity (overturning calculations). In the properties page for the units the weight of the concrete may be as low as 75 pcf. This is the effective unit weight of the concrete only (e.g. the weight of the concrete divided by the volume of the unit). The density of the concrete maybe 150 pcf, but not the effective weight including the volume of the void spaces used for gravel fill.

Rounding Errors

When doing hand calculations the values may vary from the values shown in the software. The program is designed using double precision values (64 bit precision: 14 decimal places). Over several calculations the results may differ from the single calculation the user is making, probably inputting one or two already rounded values.

Result Rounding

As noted above the software is based on double precision values. For example, using an NCMA design method an allowable factor of safety of 1.5 the software may calculate a value of 1.49999999999999, since this is less than 1.5, it would be false (NG), even though the results shown is 1.50 (results are rounded to 2 places on the screen). In the design check we round to 2 decimal places to check against the suggested value (1.49999999999 rounds to 1.50). Given the precision of the calculation, this will provide a safe design even though the 'absolute' value is less than the minimum suggested.

RETAINING WALL UNITS

STRUCTURAL PROPERTIES:

N is the normal force [or factored normal load] on unit to unit interface
The unit to unit shear is N x Tan(0.0) + 17796.0

N is the normal force [or factored normal load] on the base unit
The default leveling pad to base unit shear is 0.8 * ATan(35), or 29 deg. [AASHTO LRFD 10.6.3.4-2] or may be the manufacturer supplied data.
[Note: concrete to concrete has a coefficient of φ0.6 N. 0 [AASHTO LRFD 5.8.4]

Table of Values:

Unit	Ht (in)	Width (in)	Depth (in)	Equiv_Density (pcf)	Equiv_CG (in)
Cap	14.75	59.00	29.50	140.00	14.75
Full	29.50	59.00	29.50	140.00	14.75
Double	29.50	59.00	59.00	140.00	29.50
Triple	29.50	59.00	88.50	140.00	44.25
15 in Tall Unit	14.75	59.00	29.50	140.00	14.75

FORCE DETAILS

The details below shown how the forces and moments are calculated for each force component. The values shown are not factored. All loads are based on a unit width (ppf / kNpm).

Layer	Block Wt	X-Arm	Moment	Soil Wt	X-Arm	Moment
1	423.04	2.14	907.23	11.76	3.44	40.42
2	846.08	1.84	1556.30	164.75	3.44	566.68
3	846.08	1.53	1298.13	386.34	3.50	1353.99
4	1692.15	2.46	4159.88			

Block Weight (Force v) = block: 3,807 X-Arm = 2.21 ft
Soils Block Weight (Force v) = 563 ppf X-Arm = 3.62 ft

Active Earth Pressure Pa = 1,849 ppf
Pa_h (Force H) = Pa cos(δ batter) = 1,849 x cos(21.010.0) = 1,342 ppf
Y-Arm = 2.99 ft
Pa_v (Force V) = Pa sin(δ batter) = 1,849 x sin(21.010.0) = 806 ppf
X-Arm = 4.37 ft

Passive Earth Pressures
Passive earth pressures are used for resistance of the Leveling Pad, but may be extended upward to assist with the resistance of the wall facing for walls that have deep embedments.

Passive Earth Pressure:
kp = 2.77
Pp = 1,444.27 ppf

CALCULATION RESULTS

OVERVIEW

UltraWall calculates stability assuming the wall is a rigid body. Forces and moments are calculated about the base and the front toe of the wall. The base block width is used in the calculations. The concrete units and granular fill over the blocks are used as resisting forces.

EARTH PRESSURES

The method of analysis uses the Coulomb Earth Pressure equation (below) to calculate active earth pressures. Wall friction is assumed to act at the back of the wall face. The component of earth pressure is assumed to act perpendicular to the boundary surface. The effective δ angle is δ minus the wall batter at the back face. If the slope breaks within the failure zone, a trial wedge method of analysis is used.

EXTERNAL EARTH PRESSURES

Effective δ angle (3/4 retained phi)	δ = 21.0 deg
Coefficient of active earth pressure	ka = 0.401
External failure plane	ρ = 58 deg
Back Face Angle from horizontal	Face Angle =80.01 deg
Coefficient of passive earth pressure	kp =2.77

$$Ka := \frac{\cos(\phi_i + i)^2}{\cos(i)^2 \cdot \cos(\delta_i - i)\left(1 + \sqrt{\frac{\sin(\phi_i + \delta_i)\cdot\sin(\phi_i - \beta)}{\cos(\delta_i - i)\cdot\cos(i + \beta)}}\right)^2}$$

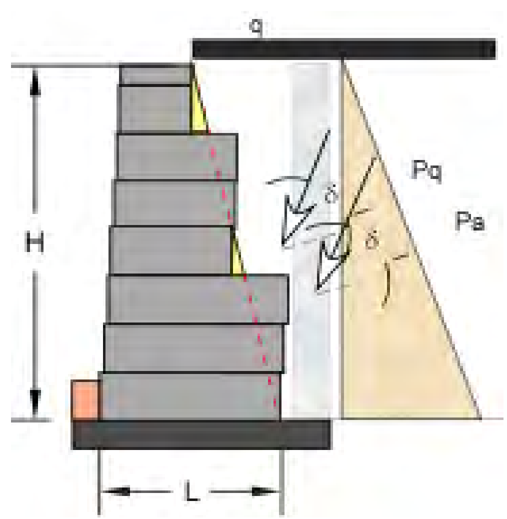
FORCES AND MOMENTS

The program resolves all the geometry into simple geometric shapes to make checking easier. All x and y coordinates are referenced to a zero point at the front toe of the base block.

UNFACTORED LOADS

Name	Factor γ	Force (V)	Force (H)	X-len	Y-len	Mo	Mr
Face Blocks(W1)	1.00	3807	--	2.21	--	--	8418
Soil Wedge(W2)	1.00	563	--	3.62	--	--	2040
LvlPad(W18)	1.00	704	--	--	--	--	--
Pa_h	1.00	--	1342	--	2.99	4009	--
Pa_v	1.00	806	--	4.37	--	--	3523
Sum V / H	1.00	5176	1342		Sum Mom	4009	13980

W0: stone within units
W1: facing units
W2: stone over the tails
W9: Driving force Pa
W10: Driving Surcharge load Paq
W11: Driving Dead Load Surchage Paqd



Note: Calculations and quantities are for PRELIMINARY ANALYTICAL USE ONLY and MUST NOT be used for final design or construction without the independent review, verification, and approval by a qualified professional engineer.

FORCES AND MOMENTS FACTORED FOR Str I-a

UltraWall increases all driving forces and reduces the resisting forces by the factors shown for Str I.

FACTORED LOADS: Str Ia

Name	Factor γ	Force (V)	Force (H)	X-len	Y-len	Mo	Mr
Face Blocks(W1)	0.90	3427	--	2.21	--	--	7576
Soil Wedge(W2)	1.00	563	--	3.62	--	--	2040
Pa_h	1.50	--	2013	--	2.99	6013	--
Pa_v	1.50	1209	--	4.37	--	--	5284
Sum V / H		5198	2013		Sum Mom	6013	14900

FORCES AND MOMENTS FACTORED FOR Str I-b

UltraWall increase resisting loads and increases driving loads by the factors shown for Str I-b.

FACTORED LOADS: Str Ib

Name	Factor γ	Force (V)	Force (H)	X-len	Y-len	Mo	Mr
Face Blocks(W1)	1.25	4759	--	2.21	--	--	10522
Soil Wedge(W2)	1.35	760	--	3.62	--	--	2753
Pa_h	1.50	--	2013	--	2.99	6013	--
Pa_v	1.50	1209	--	4.37	--	--	5284
Pq_h	1.75	--	0	--	4.48	0	--
Pq_v	1.75	0	--	4.12	--	--	0
Sum V / H		6728	2013		Sum Mom	6013	18560

BASE SLIDING

Sliding at the base is checked at the block to leveling pad interface between the base block and the leveling pad. Sliding is also checked between the leveling pad and the foundation soils.

Forces Resisting sliding = W1(DCr) + W2(EVr) + Pav(EHd)
(3807 x 0.90) + (563 x 1.00) + (806 x 1.50) N =5198 ppf

Resisting force at pad = (N tan(slope) + intercept) x RFsl
(5198 x tan(31.0) + 0.0) x 0.90 Rf1 =2621
where L is the base block width

N1 includes N + leveling pad
where LvlPd = lvl pad thickness * 130pcf * L + lvl pad thickness/2 LvlPd = 704
Forces Resisting sliding = W1(DCr) + W2(EVr) + LvlPd(EVr) + Pav(EHd)
(0 x 0.90) + (0 x 1.00) + (704 x 1.00)
N1 =5903 ppf

Friction angle is the lesser of the leveling pad and Fnd φ =28.00 deg

Passive resistance is calculated using kp = (1 + sin(28))/(1 - sin(28)) kp = 2.77
Force at top of resisting trapezoid, d1 = 2.00 Fp1 = 637.06
Force at base of resisting trapezoid, d2 = 3.62 Fp2 = 1151.49
Depth of trapezoid depth = 1.62
Pp = (Fp1 + Fp2) / 2 * depth 1444.27

Resisting force at fnd = (N1 tan(phi) + c L) * RFsl + Pp x RFps
(5903 x tan(28) + 50 x 5.92) * 0.90 + (1444 x 0.50)
Rf2 = 3811 [the value printed is the minimum
foundation soil.

Driving force is the horizontal component of
Pah(EHd)
(1342 x 1.50) Df =2013

CDR = (Rf1 / Df) / (Rf2 / Df2) CDR =1.30 / 1.89

ECCENTRICITY ABOUT THE TOE

Eccentricity at the base is checked by assuming rotation about the front toe by the block mass and the soil retained on the blocks. Allowable overturning can be defined by eccentricity (e/L).

Moments resisting eccentricity =
M1(DCr) + M2(EVr) + MPav(EHd)
((8418 x 0.90) + (2040 x 1.00) + (3523 x 1.50)

Mr =14900 ft-lbs

Moments causing eccentricity = MPah(EHd)
(4009 x 1.50)

Mo =6013 ft-lbs

e = L/2 - (Mr - Mo)/ N
e =4.92/2 - (14900 - 6013) /5198

e =0.75
e/L = 0.15

GRAVITY RESULTS

The table below shows the forces and moments for each layer of face units. For concrete leveling pads, sliding and eccentricity is checked at the base blocks and at the base of the concrete leveling pad.

TABLE OF RESULTS

Layer	Ht	ka	Batter	Delta	W0	Len0X	W1	Len1X	W2	Len2X	Pah	Pav	PaLenY	PaLenX	PqLenY	PqLenX
0	7.32	0.279	7.1	14.00	0	0.00	423	1.30	0	0.00	0	0	0.45	2.48		
1	4.88	0.279	7.1	14.00	0	0.00	1269	1.45	0	2.62	128	0	1.26	2.54		
2	2.44	0.279	7.1	14.00	0	0.00	2115	1.60	1	2.78	453	0	2.07	2.64		
3	0.00	0.401	-10.0	21.00	0	0.00	3807	2.21	563	3.62	1342	806	2.99	4.37		

Layer	M0[0]	M1[1]	Mr[2]	Mlp	MPah	MPav	Forces Re	Forces DR	SumV	Mr	Mo	CDRsl	ecc/L
0	0	548	0	0	0	0	16208	1	381	493	0	100.00	-0.03
1	0	1838	1	0	162	0	16592	193	1143	1655	243	86.12	0.00
2	0	3387	1	0	939	0	16975	680	1904	3049	1409	24.96	0.15
3	0	8418	2040	0	4009	3523	2621	2013	5198	14900	6013	1.30	0.15

ECCENTRICITY AND BEARING

Eccentricity is the calculation of the distance of the resultant force away from the centroid of the mass. This measure is an indication of the overturning of the mass. UltraWall uses an allowable eccentricity of 9/20 L for concrete to concrete bearing surfaces and a concrete leveling pad (thickness > 1.0 ft), or L/3 for bearing on soil per the AASHTO LRFD guidelines. Eccentricity is still used as a guide to design in some design methods.

UltraWall calculates three eccentricities:

1) Maximum eccentricity (overturning) where it uses the maximum driving forces combined with the minimum resisting forces (see overturning) [Str I-a]. 2) Maximum bearing where it uses the maximum driving forces combined with the maximum resisting forces [Str I-b]. 3) Service: Maximum bearing where it uses the actual driving forces combined with the actual resisting forces in Service loading.

Calculation of Eccentricity for maximum bearing
Moments resisting = M1(DCd) + M2(EVd) + MPa(EHd)
(8418 x 1.25) + (2040 x 1.35) + (3523 x 1.50)
Mr =18560 ft-lbs

Moments driving = MPah(EHd)
(4009 x 1.50) Mo =6013 ft-lbs

Nb = W1(DCd) + W2(EVd) + WPa(EHd)
(3807 x 1.25) + (563 x 1.35) + (806 x 1.50)
Nb =6728 ppf

N bearing = W1(DCd) + W2(EVd) + WPlvlpad(EVd) + WPa(EHd)
(3807 x 1.25) + (563 x 1.35) + (704 x 1.35) + (806 x 1.50)
Nbrg =7679 ppf

Calculate Eccentricity
e = L/2 - (Mr - Mo)/ Nb
e =4.92/2 - (18560 - 6013) /6728 e =0.593

B' = B - 2e + lvl pad thickness =
4.92 - 2 x 0.59 + 1.00 B' = 4.73 ft

Calculation of Bearing Pressures
Qult = (c * Nc + q * Nq + 0.5 * γ * B' * Ng) * RFbrg
[(50.00 x 25.80) + (345 x 14.72) + (0.5 x 115 x 4.73 x 16.72)] x 0.45 Qult =4912 psf
Ultimate Bearing, Qult Qult =4912 psf
Applied Bearing Pressures (σ) Nbrg/B' =1623 psf
Calculated CDR for bearing Qult/σ =3.03

Design	Sum Vert	Mo	Mr	e	Qult	Sigma	CDR
Strength I-a	5903	6013	14900	0.749	4777	1336	3.58
Strength I-b	7679	6013	18560	0.593	4912	1623	3.03
Service	5176	4009	13980	0.532	4965	1067	4.65

Note: Calculations and quantities are for PRELIMINARY ANALYTICAL USE ONLY and MUST NOT be used for final design or construction without the independent review, verification, and approval by a qualified professional engineer.

SEISMIC CALCULATIONS

The loads considered under seismic loading are primarily inertial loadings. The wave passes the structure putting the mass into motion and then the mass will try to continue in the direction of the initial wave. In the calculations you see the one dynamic earth pressure from the wedge of the soil behind the reinforced mass, and then all the other forces come from inertia calculations of the face put into motion and then trying to be held in place.

Design Ground Acceleration — As = Fpae * PGA = 0.368

Assumed deformation = — Def = 3.00 in

(with deformation, the kh is reduced by 50%)

Horizontal Acceleration — kh =0.184

Vertical Acceleration — kv =0.000

INERTIA FORCES OF THE STRUCTURE

Pif = (W1 * kh)

(3807.34 * 0.184) — Pif = 613.60

SEISMIC THRUST

Coefficient of active seismic earth pressure Kae — Kae =0.568

D_Kae = Kae - Ka = (0.568 -0.401) — D_Kae =0.167

Pae = 0.5*gamma*(H)^2 * DKae — Pae =771.46 ppf

Pae_h = Pae*cos(delta) — Pae_h =661.37 ppf

Pae_v = Pae*sin(delta) — Pae_v =397.17 ppf

In AASHTO LRFD, two cases are looked at: 1) 100% Pae and 50% of Pir, and 2) 50% Pae and 100% Pir.

TABLE OF RESULTS FOR SEISMIC REACTIONS

Name	Force (V)	Force (H)	X-len	Y-len	Mo	Mr
Face Blocks(W1)	3807	--	2.21	--	--	8418
Soil Block(W2)	563	--	3.62	--	--	2040
Pa_h	--	1342	--	2.99	4009	--
Pa_v	806	--	4.37	--	--	3523
Pif	--	614	--	5.38	3299	--
Pae_h	--	661	--	5.38	3556	--
Pae_v	397	--	4.37	--	--	1736

SEISMIC SLIDING

Details are only shown for sliding at the base of blocks, a check is made at the foundation level with the answer only shown.

The vertical resisting forces are
Units(1.0) + SoilWedge(1.0) + Pa_v(1.0) + Pae_v(1.0)

(3807 * 1.0) + (563 * 1.0) + (806 * 1.0) + (397 * 1.0)
Note Pae_v changes between case 1 and case 2, to Pae_v/2)

Case 1, 100% Pae & 50% Pir
SumVs_Pae = 5573

Case 2, 50% Pae & 100% Pir
SumVs_Pir = 4773

Resisting force 1 = (SumVs_Pae * tan(slope) + intercept * L) * RFsl FReS1 =3122 ppf
Resisting force 2 = (SumVs_Pir * tan(slope) + intercept * L) * RFsl FReS2 =2674 ppf

The inertial force Pif = Face*kh(1.0) + Soil Wedge*kh*(1.0)
Driving force = + Pa_h(1.0) + Pif_h/2(1.0) + Pae_h(1.0)
[case 2 is 50% Pae, 100% Pir]

(1342 * 1.0) + (614/2 * 1.0) + (661 * 1.0)

FDrS1 =2310 ppf
FDrS2 =2627 ppf

CDR = (FReS1/FDrS1) / (FReS2/FDrS2) CDR =1.35 / 1.02

SEISMIC ECCENTRICITY

Eccentricity is rotation about the front toe of the wall and is a check on overturning.

Resisting Moment = Units(1.0) + SoilWedge(1.0) + Pa_v(1.0) + Pae_v(1.0)
(case 2 is 50% Pae)
(8418 * 1.0) + (2040 * 1.0) + (3523 * 1.0) + (1736 * 1.0)

MomReS [Pae] =0 ft ppf
MomReS [Pir] =0 ft ppf

Driving Moment =
Pa_h(1.0) + Pif_h/2(1.0) + Pae_h(1.0)
(4009 * 1.0) + (3299/2 * 1.0) + (3556 * 1.0)

MomDrS [Pae] =9214 ft ppf
MomDrS [Pir] =9312 ft ppf

e = L/2 - (Mr - Mo)/ N
e [Pae] =4.92/2 - (15716 - 9214) /5573 e =0.00
e/L = 0.00 e [Pir] =4.92/2 - (15742 - 9312) /4773
e/L = 0.00

SEISMIC BEARING

www.geodesigninc.com