

Geotechnical Engineering Services

Wind Rose Neighborhood Development
King County, Washington

for

King County Housing Authority

April 28, 2016



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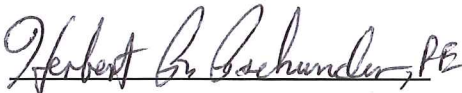
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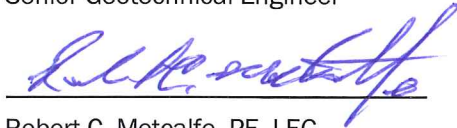
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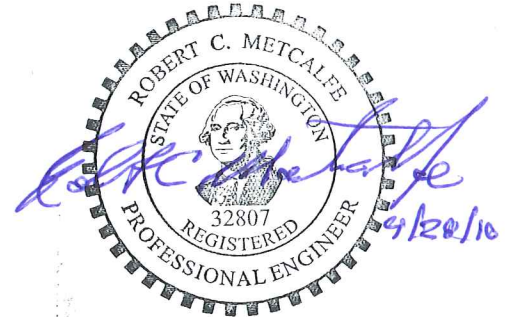


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Table of Contents

INTRODUCTION	1
PROJECT UNDERSTANDING	1
SCOPE OF SERVICES	2
FIELD EXPLORATIONS AND LABORATORY TESTING.....	2
Field Explorations.....	2
Laboratory Testing.....	3
SITE CONDITIONS	3
Geology.....	3
Surface Conditions.....	3
Subsurface Soil Conditions	4
Groundwater Conditions.....	4
CONCLUSIONS AND RECOMMENDATIONS	5
Earthquake Engineering.....	5
2012 IBC Seismic Design Information	5
Seismic Hazards.....	5
Liquefaction Potential.....	5
Slope Assessment.....	6
Steep Slope and Erosion Hazards.....	6
Slope Stability Assessment – MSE Wall	7
Site Preparation and Earthwork.....	8
Stripping, Clearing and Grubbing.....	8
Abandoning Utilities	8
Removal of Existing Fill Soils.....	9
Subgrade Preparation.....	9
Frost Depth.....	10
Erosion and Sedimentation Control	10
Benching.....	11
Excavations and Fill Slopes	11
Structural Fill	13
Weather Considerations	15
Utility Trenches and Backfill	16
Construction Considerations – Existing Slopes.....	17
Foundation Support.....	17
Foundation Design.....	17
Lateral Load Resistance	17
Foundation Settlement.....	17
Footing Drains.....	18
Underslab Drainage	18
Permanent Subsurface Walls.....	18
Cast-in-Place Walls.....	18
Drainage	19
Construction Considerations	19

Table of Contents (continued)

Slab-on-Grade Floors	20
Capillary Break	20
Vapor Retarder	20
Pavement Recommendations.....	20
Subgrade Preparation.....	20
New Residential Pavements.....	20
Asphalt-Treated Base.....	21
Storm Water Detention Pond	21
Soil Conditions	21
Permanent Slopes.....	21
Gravity Block Walls.....	21
Erosion Control.....	22
Outfall Pipe Scour Protection	22
Seepage Protection	23
MSE Wall With Vegetated Face.....	23
Rockerries.....	23
Southeast Soil Cap	24
Permanent Drainage Considerations	25
Recommended Additional Geotechnical Services	25
LIMITATIONS	26
REFERENCES	26

LIST OF FIGURES

Figure 1. Vicinity Map
Figure 2. Site Plan
Figure 3. Steep Slope Critical Areas
Figure 4. Wall Drainage and Backfill
Figure 5. Compaction Criteria for Trench Backfill
Figure 6. Recommended Surcharge Pressure
Figure 7. Typical Gravity Block Retaining Walls Section and Notes
Figure 8. Rockery at Cut Slope
Figure 9. Rockery at Fill Slope

APPENDICES

Appendix A. Field Explorations
Figure A-1 – Key to Exploration Logs
Figures A-2 through A-16 – Log of Test Pits
Appendix B. Laboratory Testing
Figures B-1 through B-3 – Sieve Analysis Results
Appendix C. Previous Explorations
Appendix D. June 16, 2015 Letter on Retaining Wall Design and Calculations for Gravity Block Wall With 2H:1V Backslope
Appendix E. Report Limitations and Guidelines for Use

INTRODUCTION

This report presents the results of our additional geotechnical engineering services related to the King County Housing Authority (KCHA) Wind Rose Neighborhood Development project. The project site is located southeast of the intersection of 4th Avenue SW and SW Roxbury Street in the White Center area of unincorporated King County, Washington.

The site comprises approximately 6 acres and adjoins the south, southeast and east sides of a property known as the “Notch Property”, which will be a future phase of the Wind Rose Neighborhood Development. SW 97th Place extends through the development site. The site is shown relative to surrounding physical features on the Vicinity Map, Figure 1 and the Site Plan, Figure 2.

We previously provided geotechnical consulting services for this project, which included preliminary sections for rockeries and vegetated face geosynthetic and gravity block walls within the site, preliminary details for capping of a dump area in the southeast part of the site, and review of preliminary drawings prepared by the project civil engineer, KPFF Consulting Engineers (KPFF). We summarized those services in a letter report dated June 16, 2015 and in several email messages in early 2015.

The King County Department of Permitting and Environmental Review (DPER) provided engineering review comments for the Wind Rose project in a letter dated November 17, 2015. The comments include several items that pertain to the geotechnical aspects of the project. In the comments, DPER required that a geotechnical engineering report be prepared for the Wind Rose project and to address these items, which include steep slopes, capping of the dump area, a proposed storm drain outfall, and retaining walls.

PROJECT UNDERSTANDING

We understand that the Wind Rose Neighborhood Development will include single family residences on 32 lots flanking both sides of SW 97th Place. Two existing residential structures along the south side of SW 97th Place will be demolished. A storm water detention pond will be constructed near the center of the site and northwest of SW 97th Place with an outfall planned in the southeast area adjacent to Lots 17 and 18. Grading of the lots with fills up to about 15 feet in height will be necessary, and about 16 feet of excavation will be required for the storm water pond.

A retaining wall along the southeast side of Lots 15 through 24 will be located along a slope that extends down to a ravine. We understand that this wall will be a mechanically stabilized earth (MSE) wall with a vegetated facing. The wall will need to be supported on adequate soils and be embedded a sufficient distance below the adjacent slope to provide adequate protection from undermining. The finished down slope surface will be inclined no steeper than 3H:1V (horizontal to vertical).

Gravity block walls with heights up to 15 feet are planned along the southeast, west and north sides of the planned storm water pond. These walls will have a 3H:1V fore slope down to the bottom of the pond. The back slopes above the wall segments will vary from level to 2H:1V.

Rockeries are planned for the perimeter of some of the lots in the southwest part of the site. We anticipate that regrading will consist of relatively thin cuts and fills, except for the storm water pond, MSE wall and

rockery areas. New underground utilities and improvements to SW 97th Place are also planned as part of the project.

GeoEngineers previously completed geologic and geotechnical engineering reports for the overall Greenbridge Redevelopment project and the Wind Rose Neighborhood Development “Notch Property” site. These reports include:

- “Geotechnical Engineering Services, Wind Rose Development, King County, Washington,” dated February 29, 2012.
- “Wind Rose NEPA EA, Summary of Subsurface Conditions and Preliminary Geotechnical Considerations, KCHA Notch Properties near the Greenbridge Redevelopment Project, Southeast Corner 4th Avenue SE and SW Roxbury Street, Unincorporated King County, Washington,” dated August 18, 2010.
- “Update Report, Geotechnical Engineering Services, Greenbridge Hope VI Redevelopment Project, King County, Washington,” dated January 12, 2007.

GeoEngineers identified a localized area of dumped fill and debris in the southeast part of the site during previous studies. This fill area is located across a ravine from, and southeast of, Lots 15 through 19. The dumped fill includes demolition debris, soil waste and organic debris and was noted as a Recognized Environmental Condition (REC) in a 2003 Phase I Environmental Site Assessment (ESA) we completed.

Test pits were completed by GeoEngineers in 2013 to evaluate the extent of debris and to obtain soil samples for chemical testing of potential contaminants of concern. The test results presented in the January 2015 Draft Phase II ESA report indicated that fill soil from three out of five test pits contained cPAHs or arsenic at concentrations exceeding MTCA cleanup levels. We understand KCHA plans to cap the dumped fill area during construction so that it is isolated from direct contact with people and wildlife.

SCOPE OF SERVICES

The purpose of our services is to complete additional subsurface explorations at the site as a basis for developing design recommendations for the geotechnical elements of the Wind Rose Neighborhood Development and to address the DPER review comments. In addition, our scope includes design recommendations for capping the southeast dump area.

Our specific scope of services is outlined in our January 15, 2016 letter that accompanies KCHA Additional Authorization #8 (AA#8) dated January 20, 2016.

FIELD EXPLORATIONS AND LABORATORY TESTING

Field Explorations

Subsurface conditions were evaluated at the site by completing 15 test pit excavations (TP-1 through TP-15) ranging in depth from 4 to 17 feet below the existing ground surface. The test pits were excavated using a track-mounted excavator. The locations of the explorations are shown on the Site Plan, Figure 2. A description of the subsurface exploration program and logs of the test pits are presented in Appendix A. Logs of previous explorations completed at this site are included in Appendix C.

Laboratory Testing

Soil samples obtained from the explorations were transported to GeoEngineers' Redmond laboratory for further evaluation and testing. Selected samples were tested for moisture content and grain size distribution (sieve analyses). A description of the laboratory test procedures and test results are presented in Appendix B or on the explorations logs in Appendix A, as appropriate.

SITE CONDITIONS

Geology

We reviewed the "Geologic Map of Seattle – A Progress Report" developed by Kathy Troost, et al. of the United States Geologic Survey (USGS) dated 2005. The soils mapped at the project site consist of glacial deposits identified as Recessional Outwash deposits (Qvr), Vashon till (Qvt), and Advance Outwash deposits (Qva).

Recessional outwash deposits generally consist of moderately sorted and stratified sand and gravel with minor silt and clay content. These materials were deposited in streams emanating from the retreating glacier and are generally loose to medium dense.

Vashon till deposits consist of dense to very dense silty sand with gravel, cobbles and occasional boulders deposited directly below the glacier.

Advance outwash deposits consist of sand and gravel deposited by streams issuing from an advancing ice sheet. Silt lenses can be locally present in the upper part and are common in the lower part. The advance outwash may be overlain by Vashon till in some areas and is generally dense to very dense.

Although not observed in our recent or previous explorations, the recessional and advance outwash deposits and glacial till commonly contain cobbles and boulders.

Surface Conditions

The site is about 6 acres in size and is roughly L shaped. SW 97th Place extends through the site from southwest to northeast. Two residential structures currently occupy lots along the south side of SW 97th Place; these residences will be demolished prior to site development.

The existing ground surface within the west, northwest and northeast parts of the site generally slopes down to the east and southeast, from approximate Elevation 396 feet (at 4th Avenue SW) to approximate Elevation 374 feet (east of the existing buildings). An area of steep slopes flanking ravines is located within the southeast part of the site. The existing ground surface in this area rapidly slopes down toward the south from approximately Elevation 379 to Elevation 342 feet. A locally higher area exists just north of the southeast corner of the site; this area coincides with the previously described dumped fill area. The steep slopes and ravines continue to the south and east, south of the southeast part of the site.

Subsurface Soil Conditions

Subsurface soil conditions at the site were evaluated based on the test pit explorations (TP-1 through TP-15) and on our previous borings (B-1 and B-4). A description of subsurface conditions within various areas of the site follows:

- **Building Lots (Lots 1 through 31):** Subsurface conditions within the proposed building areas were evaluated based on test pits TP-1 through TP-7 and previous boring B-1, and generally consist of fill overlying outwash deposits and glacial till. Approximately 6 inches of topsoil was encountered in test pit TP-1. The fill generally consists of loose to medium dense silty sand with occasional roots and variable gravel content, and extends to depths of ½ to 6 feet below the ground surface in these test pits. Outwash deposits consisting of medium dense to very dense sand with variable silt and gravel content were encountered in TP-1 and TP-3 through TP-7. Glacial till soils were encountered beneath the fill in test pit TP- 2, and beneath the outwash deposits in test pit TP-4. The glacial till soils consist of medium dense to dense silty sand with gravel. Very dense sand was encountered in boring B-1 at a depth of about 32½ feet.
- **Detention Pond:** Subsurface conditions within the proposed detention pond were evaluated based on test pit TP-8 and previous boring B-4. Subsurface conditions generally consist of fill overlying outwash deposits. The fill consists of loose to medium dense silty sand with occasional roots, gravel and brick fragments. The fill extends to a depth of about 3 feet. Outwash deposits consisting of dense to very dense sand with silt were observed in test pit TP-8. These deposits extend to a depth of approximately 17 feet. Interbedded layers of silt and sand with silt were observed beneath the fill in boring B-1.
- **Rockeries:** Subsurface conditions within the proposed rockery area along 4th Avenue SW were evaluated based on test pit TP-1. Surface conditions generally consist of fill overlying outwash deposits. The fill soils consist of medium dense sand with occasional roots. The fill extends to approximately 3 feet below the existing ground surface. Outwash deposits were encountered below the fill. These deposits consist of dense sand with silt and gravel.
- **MSE Wall Alignment:** Subsurface conditions along the proposed alignment of the MSE wall extending along the southeast side of Lots 15 through 24 were evaluated based on test pits TP-9 through TP-13. Approximately 12 inches of topsoil was observed in test pit TP-9. Fill was encountered at the surface or beneath the topsoil in each of these test pits. The fill consists of loose to medium dense silty sand with roots and occasional gravel. The fill extends to a depth of approximately 3 feet below the ground surface. Outwash deposits were observed beneath the fill, and consist of medium dense to dense silty sand and sand with variable silt content. Outwash deposits extend to the depths explored in the test pits.
- **Southeast Dump Area (Proposed Soil Cap):** Subsurface conditions along the margin of the proposed southeast soil cap area were evaluated based on test pits TP-14 and TP-15. Subsurface conditions observed outside of the dump fill generally consist of topsoil overlying outwash deposits. The outwash deposits consist of medium dense to dense sand with variable silt content.

Groundwater Conditions

Observations of groundwater conditions were made during excavation of the test pits. Groundwater seepage was observed at depths ranging from 3½ to 13 feet in test pits TP-1, TP-2, TP-8, and TP-10 during the time they were being excavated. Groundwater was observed in boring B-4 at approximately 13 feet below the ground surface at the time of drilling. Some of the seepage observed may represent perched

groundwater that exists above lower permeability zones within the outwash deposits and also above the glacial till.

Groundwater observations in the test pits represent a short-term condition that may not be representative of the long-term groundwater conditions at the site. Groundwater levels are anticipated to vary as a function of precipitation, season and other factors.

CONCLUSIONS AND RECOMMENDATIONS

Earthquake Engineering

2012 IBC Seismic Design Information

We recommend the 2012 International Building Code (IBC) parameters for Site Class, short period spectral response acceleration (S_s), 1-second period spectral response acceleration (S_1), and Seismic Coefficients F_A and F_v presented in the following Table 1:

TABLE 1. 2012 IBC PARAMETERS

2012 IBC Parameter	Recommended Value
Average Field Standard Penetration Resistance	15 < N < 50*
Site Class	D
Short Period Spectral Response Acceleration, S_s (percent g)	154
1-Second Period Spectral Response Acceleration, S_1 (percent g)	58.5
Seismic Coefficient, F_A	1.0
Seismic Coefficient, F_v	1.5

Notes:

The above spectral response accelerations are based on data from the USGS National Seismic Hazard Mapping Project

*Based on previous boring B-1.

Seismic Hazards

The site is mapped near the Seattle Fault Zone, which is thought to have a recurrence interval in the order of 1,000 years. The fault zone generally extends in a west-to-east direction and is just north of the site. Based on the long recurrence interval of the fault and the extensive thickness of glacial deposits below the site, the potential for surface fault rupture at the site is low.

Liquefaction Potential

Liquefaction refers to the condition where vibration or shaking of the ground, usually from earthquake forces, results in the development of excess pore pressures in saturated soils with subsequent loss of strength in the deposit of soil so affected. Ground settlement, lateral spreading and/or sand boils may result from soil liquefaction. In general, soils that are susceptible to liquefaction include very loose to medium dense, clean to silty sands that are below the water table.

Conditions favorable to liquefaction occur in loose to medium dense, clean to moderately silty sand that is below the groundwater level. Based on our evaluation of the subsurface conditions observed in the test

pits and previous borings completed at the site, it is our opinion that there is a low potential for liquefaction of the soils below the site.

Slope Assessment

Steep Slope and Erosion Hazards

Steep slopes hazard areas are defined as those areas 40 percent or steeper with a vertical height difference of at least 10 feet. Steep slope hazard areas within and near the southeast part of the Wind Rose site are shown on Figure 3, Steep Slope Critical Areas, and are delineated based on the base survey map produced by Goldsmith and Associates dated December 12, 2014. None of these steep slope hazard areas are identified in the King County Sensitive Areas Map Folio (1990).

Geotechnical recommendations for design and construction of the Greenbridge project including the E02 Storm Water Detention Pond and adjacent steep slopes located south of the Wind Rose site were provided in our report "Update Report, Geotechnical Engineering Services, Greenbridge Hope VI Redevelopment Project, King County, Washington", dated January 12, 2007.

We completed a reconnaissance of the steep slope areas within and adjacent to the southeast part of the site. The reconnaissance was also focused along the planned storm water pond outlet flow path, as shown on Figure 3. Existing slope conditions and potential impacts from the storm water outlet were assessed between the proposed outlet location and an existing manhole structure located approximately 400 feet to the southeast at the bottom of the east-west trending ravine. An existing Seattle City Light (SCL) access road bisects the drainage path between the outlet and manhole structure.

Within the Wind Rose property, and between the planned outlet pipe and SCL access road the existing slopes are typically less than 40 percent, except where previous grading occurred for the former residential housing structures and along the slope where the dumped fill area exists. Vegetation generally consists of large conifer and deciduous trees with a moderate understory, and thin ground cover. The slopes along the southeast dumped fill area are densely covered with blackberry bushes.

The channel bottom along the swale north of the access road generally consists of silty sand and no active scouring or ponded water was observed. There is sparse ground cover along the swale, and the soils appear to be moderately to highly erodible, although they also appear to infiltrate reasonably well. The drainage swale abruptly terminates at the north side of the SCL access road. The bottom of the swale is about 6 feet below the surface of the access road, and the north slope of the access road is very steep (about 1H:1V or steeper), and appears to be unstable and susceptible to erosion.

South of the SCL access road is a natural east-west trending ravine with steep slopes, which the SCL access road traverses along the top. There are numerous shallow slope failures and erosional features along the north and south slopes of the ravine, especially in the areas shown on Figure 3. One failure occurred along the base of the south slope of the ravine located northeast of the E02 pond. The failure apparently occurred around 2009 from discharge from an abandoned storm drain pipe outfall, and is located just up the ravine and across from the SCL access road crossing.

The failed area is roughly 60 feet wide and extends about 30 feet up the slope. The slide mass is about 4 to 6 feet deep with a deeper scoured area on the west side ranging up to 8 feet deep. The exposed native soils consist of dense glacially consolidated advance outwash. The advance outwash is composed of

stratified silty sand and sand with silt with variable gravel. No groundwater seepage was observed in the failed area or along other areas of the south or north slopes of the ravine.

Several other shallow slumps and erosion features were observed on the slopes north and south of the ravine as it widens to the east. The slope immediately below the SCL access road is near vertical in some areas and heavily eroded. However, potential scouring along the base of the ravine appears to be limited to within about 25 feet down slope of the SCL access road crossing, although the flow is confined in a relatively narrow channel created from adjacent surficial slumps. The ravine bottom west of the manhole structure exhibits little to no scour or erosion, and it appears that surface water flow may infiltrate into the sandy soils along the base of the ravine between the manhole structure and the SCL access road crossing.

Based on our observations, we anticipate that the shallow slope failures resulted from shallow surficial soils becoming saturated due to extended rainfall, or uncontrolled pipe discharge, and eventually the shear strength of the soil was exceeded and the slopes failed. We expect that similar shallow slope failures will continue in the future, as these are on-going natural erosional processes along these slopes. However, grading for the proposed neighborhood development will increase the stability of the existing slopes by grading them to flatter inclinations and by controlling surface water runoff.

We did not observe a storm drain pipe under the SCL access road at the intersection with the drainage swale. We observed very steep slopes and active scouring on both slopes below the SCL access road, and scouring along the ravine bottom immediately downslope of the south slope. We recommend consideration be given to installing a storm drain pipe under the SCL access road to accommodate storm water flow from the planned detention pond outlet, and that the slopes upstream and downstream of this crossing be enhanced to prevent future scouring or slope failures. Erosion protection along the upper portion of the east-west ravine bottom downslope of the access road should also be considered.

Slope Stability Assessment – MSE Wall

We evaluated the impact of loading from the MSE wall on the slope below Lots 15 through 24. External global stability of the MSE wall under static and seismic conditions was evaluated using the commercial computer program Slope/W (Version 8.0, 2012) developed by GeoSlope International. The results of our analyses indicate that the slope has a static factor of safety of about 1.6 and a seismic factor of safety of about 1.1. The seismic factor of safety analysis used a horizontal seismic coefficient of 0.21g (or half of the peak horizontal ground acceleration of about 0.41g), which is based on the 2008 USGS probabilistic seismic hazard maps. Results of our slope stability analyses related to the MSE wall are provided in our June 16, 2015 letter, which is included as Appendix D of this report.

Standard slope management practices should be implemented at this site. These practices will reduce potential impacts to the steep slopes, but will not eliminate the natural processes of shallow slope creep and failures. These include:

- no grading such as cutting at the toe or filling at the top of the slope without engineering design;
- no dumping of trash, yard waste and debris on the slope;
- collecting and tightlining roof drain discharge and surface water runoff away from the top of the slopes; and

- no removal of vegetative cover on the slope below the walls without temporary erosion control measures and replacement with permanent vegetative cover. Removal of blackberry bushes are permissible along the base of the wall.

Site Preparation and Earthwork

Stripping, Clearing and Grubbing

All construction debris, not including asphalt and concrete rubble, associated with demolition of the existing buildings or previous buildings, utilities and pavement should be removed from the site. Existing voids or new depressions created during demolition and site preparation should be cleaned of loose soil and debris and backfilled with compacted structural fill. Trash and debris on the slopes within planned grading should be removed during clearing activities.

We recommend grinding existing asphalt roadway pavement that needs to be removed, and that the asphalt and any underlying base course materials be reused on site as structural fill under future pavement and hardscape areas. In addition, we recommend that concrete rubble be crushed and reused as structural fill under pavements and in utility trenches.

Areas to be graded for new development should be cleared of surface and subsurface deleterious matter including any debris, trees and associated stumps and roots. Graded areas should be stripped of organic laden soils. Based on our explorations and site observations, we estimate that, on average, stripping depths on the order of 6 to 12 inches will be necessary to remove the root zone and surficial soils containing organics. Deeper excavation may be needed to remove root balls associated with large trees. Soft soils may exist around the site in localized depressions. If encountered, soft soils should be removed from new building and retaining wall areas.

The organic soil strippings can be stockpiled and used later for landscaping purposes or may be spread over disturbed areas following completion of excavation and grading. If spread out, the organic strippings should be in a layer less than 1 foot thick, should not be placed on slopes steeper than 3H:1V and should be track-rolled to a uniformly compacted condition. Materials that cannot be used for landscaping or protection of disturbed areas should be removed from the project site.

Along the surface drainage zone for the detention pond outfall, we recommend removing selected vegetation to install features that will protect the existing soils from scour and erosion. These measures include may include (1) riprap underlain by a geotextile separator, or (2) a turf reinforcement mat (TRM), such as Propex Landlok 450, which is good for water velocities up to 18 feet per second (ft/sec). Where the TRM is placed, we recommend that the ground surface be hydroseeded. Once vegetation is established, the TRM lined drainage swale will blend in with the surrounding environment.

Abandoning Utilities

The following recommendations apply to abandoning utility pipes at the site prior to vertical construction:

- All utility pipes greater than or equal to 12 inches diameter and located below potential future building areas may be left in place provided that they are fully grouted.
- All utilities less than 12 inches in diameter and located beneath potential future building areas may be left in place provided that they are capped and/or plugged with grout.

- Utility structures should be removed and associated pipes capped/plugged to prevent the movement of groundwater.
- Utility pipes encountered outside of building areas during redevelopment activities should be plugged, capped, or removed to prevent movement of groundwater.

The following recommendations apply to utility pipe issues arising during vertical construction:

- Abandoned utility lines under proposed planned buildings should be identified during construction and the existing trench backfill should be removed and replaced as follows. Utility pipes and/or unsuitable trench backfill encountered during excavation and subgrade preparation for foundations or slabs should be removed and recompacted to a depth of 3 feet below the bottom of the footing or slab, and to a distance of at least 3 feet beyond the edges of the foundation. The excavation should be backfilled with structural fill compacted to at least 95 percent of the maximum dry density (MDD) obtained using ASTM D 1557.
- We recommend that storm water pipes not discharge onto slopes. We recommend that outlet pipes be designed to prevent scour and erosion of existing soils downslope of the outlet pipes.

Removal of Existing Fill Soils

Under planned structures, we recommend that existing fill be removed to expose medium dense to dense native soils and that these areas be replaced with properly compacted structural fill. However, where existing fill soils extend 2 feet or more below future building foundations, we recommend that the foundations be supported on at least 2 feet of properly compacted structural fill that replaces the existing fill. Remaining existing fill below the structural fill pad may be left in place depending on building loads.

The 2-foot-thick structural fill pad should be preceded by overexcavation of the upper 2 feet below foundations and compaction of the exposed subgrade at the base of the excavation. The zone of structural fill should extend horizontally beyond the edges of the foundations a distance equal to the depth of overexcavation.

Subgrade Preparation

Prior to placing new fills, pavement base course materials or gravel below slabs-on-grade, all subgrade areas should be proofrolled or probed by hand to locate zones of soft or pumping soils. Proofrolling can be completed using a piece of heavy tire-mounted equipment or a loaded dump truck. If zones of soft or pumping soils are identified, they should be overexcavated and replaced with compacted structural fill.

If deep pockets of soft or pumping soils are encountered, it may be possible to limit the depth of overexcavation by placing a non-woven geotextile fabric such as Mirafi 500X (or similar geotextile) on the overexcavated subgrade and covering the geotextile with structural fill. The geotextile will provide additional support by bridging over the soft material.

After completing the proofrolling, the subgrade areas should be recompacted to a firm and unyielding condition, if possible. The degree of compaction that can be achieved will depend on when construction is performed. If the work is performed during dry weather conditions, we recommend that all subgrade areas be compacted to at least 95 percent of the MDD obtained using the ASTM D 1557 test procedure. If the work is performed during wet weather conditions or if the exposed subgrade is wet, it may not be possible

to recompact the subgrade to 95 percent of the MDD. In this case, we recommend that the subgrade be compacted to the extent possible without causing undue weaving or pumping of the subgrade soils.

Subgrade disturbance or deterioration could occur if the subgrade is wet and cannot be dried. If the subgrade deteriorates during compaction or while being subjected to construction traffic, it may become necessary to modify the compaction criteria or methods.

A representative of GeoEngineers should observe subgrade preparation to help evaluate the depth of removal of existing fill, soft or pumping soils required, and to evaluate if subgrade disturbance or progressive deterioration is occurring. Subgrade disturbance or deterioration could occur if the subgrade becomes wet. If the subgrade deteriorates due to saturation and disturbance from wheeled equipment, the soil will need to be moisture conditioned and recompact or replaced with imported structural fill prior to placement of base course materials for pavement areas or concrete for the slabs and footings.

Frost Depth

The design frost depth for the Puget Sound area is 12 inches. We recommend that exterior foundations supporting buildings and other sensitive structures extend at least 18 inches below the lowest adjacent finished ground surface. Interior foundations should extend at least 12 inches below adjacent concrete slabs or lowest adjacent grade.

Erosion and Sedimentation Control

In our opinion, the erosion potential of the on-site soils is low to moderate, except in the southeast part of the site along the potential drainage outfall path, where the erosion potential is moderate to high. The areas of high erosion hazard generally correspond to the steep slope hazard areas shown in Figure 3.

Construction activities including stripping and grading will expose soils to the erosional effects of wind and water. The amount and potential impacts of erosion are partly related to the time of year that construction actually occurs. Wet weather construction will increase the amount and extent of erosion and potential sedimentation.

Potential sources or causes of erosion and sedimentation depend upon construction methods, slope length and gradient, amount of soil exposed and/or disturbed, soil type, construction sequencing and weather. Implementing an erosion and sedimentation control plan will reduce the project impact on erosion-prone areas. The erosion and sedimentation control measures should be designed, installed and maintained in accordance with the requirements of King County. The plan should incorporate basic planning principles including:

- Scheduling grading and construction to reduce soil exposure.
- Retaining existing asphalt whenever feasible.
- Revegetating or mulching denuded areas.
- Directing runoff away from denuded areas.
- Reducing the length and steepness of slopes with exposed soils.
- Decreasing runoff velocities.

- Preparing drainage ways and outlets to handle concentrated or increased runoff.
- Confining sediment to the project site.
- Inspecting and maintaining control measures frequently.

In addition, we recommend that all disturbed areas be finish graded and seeded as soon as practicable to reduce the risk of erosion. Some sloughing and raveling of slopes with exposed or disturbed soil should be expected. Temporary erosion protection should be used and maintained in areas with exposed or disturbed soils to help reduce erosion and reduce transport of sediment to adjacent areas and receiving waters. Erosion and sedimentation control measures may be implemented by using a combination of interceptor swales, straw bale barriers, silt fences and straw mulch for temporary erosion protection of exposed soils.

Permanent erosion protection should be provided by paving or landscape planting. Until the permanent erosion protection is established and the site is stabilized, site monitoring should be performed by qualified personnel to evaluate the effectiveness of the erosion control measures and to repair and/or modify them as appropriate. Provisions for modifications to the erosion control system based on monitoring observations should be included in the erosion and sedimentation control plan.

Benching

Where new fill is placed on existing slopes, the new fill should be keyed into the existing slopes as described in Section 2-03.3(14) of the 2016 Washington State Department of Transportation (WSDOT) Standard Specifications for embankment construction, except as noted herein. The benches should be keyed into the slopes and into medium dense to dense native soils, except along the dump fill area where the new cap fill will be benched into the existing dump fill. We recommend that the benches be at least 5 feet wide into the slope, with the vertical height between benches limited to no more than 3 feet. The horizontal portion of each bench should be sloped such that surface water runoff is directed downslope.

All existing unsuitable fill and loose soils should be removed from areas to receive fill. If existing fill soils are not removed from the slopes or entirely from beneath new fills, the performance of the slope and overlying improvements may be jeopardized.

Excavations and Fill Slopes

Excavation Considerations

Fill, outwash deposits and glacial till were observed in the explorations. We anticipate that these soils may be excavated with conventional excavation equipment, such as large excavators and/or dozers. The dense glacial till may be very difficult to excavate, depending upon the depth of cuts planned, and large excavators and/or dozers equipped with rippers may be needed. Although cobbles and boulders were not encountered in our explorations, it is our experience that cobbles and boulders are commonly encountered in these soil deposits and the contractor should be prepared to address them. We recommend that procedures be identified in the project specifications for measurement and payment of work associated with removal of cobbles and boulders.

We anticipate shallow groundwater seepage may enter excavations (such as for the storm water detention pond and utility trenches) depending on the time of year construction takes place, especially during the winter months. However, we expect that this seepage water can be handled by digging

interceptor trenches in the excavations and pumping from sumps. Seepage water not intercepted and removed from the excavations will make it difficult to place and compact structural fill and may destabilize cut slopes.

The excavation for the storm water pond and associated retaining walls and access road may require cuts up to 16 feet high. Smaller cuts will be needed for the MSE wall and the rockeries in the west part of the site. These cuts may be made as temporary open cut slopes depending on site constraints. Excavations are also required for underground utilities. The stability of open cut slopes is a function of soil type, groundwater seepage, slope inclination, slope height and nearby surface loads. The use of inadequately designed open cuts could impact the stability of adjacent work areas and existing utilities, and endanger personnel.

The contractor performing the work has the primary responsibility for protection of workers and adjacent improvements. In our opinion, the contractor will be in the best position to observe subsurface conditions continuously throughout the construction process and to respond to variable soil and groundwater conditions. Therefore, the contractor should have the primary responsibility for deciding whether or not to use open cut slopes for much of the excavations rather than some form of temporary excavation support, and for establishing the safe inclination of cut slopes.

Acceptable slope inclinations for utilities and ancillary excavations should be determined during construction. Because of the diversity of construction techniques and available shoring systems, the design of temporary shoring is most appropriately left up to the contractor proposing to complete the installation. Temporary cut slopes and shoring must comply with the provisions of Title 296 WAC, Part N, "Excavation, Trenching and Shoring."

Temporary Slopes

We recommend using temporary cut slopes no steeper than 1½H:1V in the existing fill, medium dense outwash deposits and surficial soils. Flatter slopes may be necessary if localized sloughing occurs. Temporary cut slopes should be no steeper than 1H:1V in the dense to very dense outwash deposits and glacial till. Localized areas of seepage may exist along less permeable lenses or layers within the outwash deposits and glacial till. We also anticipate shallow perched groundwater conditions will exist during the winter and spring months. Cut slope inclinations may need to be modified by the contractor if localized sloughing occurs

For open cuts at the site we recommend that:

- no traffic, construction equipment, stockpiles or building supplies be allowed at the top of the slopes within a distance of at least 5 feet or ½ the height of the cut (whichever is greater), from the top of the cut;
- exposed soil along the slope should be protected from surface erosion using waterproof tarps or plastic sheeting;
- construction activities be scheduled so that the length of time the temporary cut is left open is minimized;
- erosion control measures be implemented as appropriate such that runoff from the site is reduced to the extent practical;

- surface water be diverted away from the excavation; and
- the general condition of the slopes be observed periodically by a geotechnical engineer to identify potential problems.

Since the contractor has control of the construction operations, the contractor should be made responsible for the stability of cut slopes, as well as the safety of the excavations. Shoring and temporary slopes must conform to applicable local, state and federal safety regulations.

Permanent Cut and Fill Slopes

Permanent cut and fill slopes should be inclined no steeper than 2H:1V. We recommend that all fill placed to construct permanent slopes be placed and compacted as structural fill. The fill should be compacted at the slope face, or the fill embankment should be overbuilt and cut back. If the slope face is not uniformly compacted, slumps may occur during wet weather conditions.

Permanent slopes should be planted or hydroseeded as soon as practicable after grading. Temporary erosion control measures may be necessary until permanent vegetation is established.

Structural Fill

Materials

Structural fill materials used for support of footings, placed below floor slabs, sidewalks, parking areas and pavements, utility trench backfill, embankments, MSE wall construction, and for conventional retaining wall backfill are classified as structural fill for the purpose of this report. Structural fill material quality varies depending upon its use as described below:

1. Structural fill placed to construct embankment and pavement areas, to backfill utility trenches and below-grade walls, and to support foundations may consist of on-site outwash deposits, glacial till or suitable fill soils provided that the soils are properly conditioned for the required compaction. If needed during dry weather, imported soil should meet the criteria for select borrow as described in Section 9-03.14(2) of the 2016 WSDOT Standard Specifications. On-site soils and imported select borrow will be suitable for use as structural fill during dry weather conditions only. If structural fill is placed during wet weather, or between October 1 and May 30, the structural fill should consist of imported gravel borrow as described in Section 9-03.14(1) of the 2016 WSDOT Standard Specifications, with the additional restriction that the fines content be limited to no more than 5 percent. It may be possible to use on-site soils during wet weather for areas requiring 90 percent compaction provided the earthwork contractor implements good wet weather techniques and the soil is properly moisture conditioned. However, for planning purposes we recommend that gravel borrow be used throughout the project during wet weather conditions.
2. Structural fill placed immediately outside below-grade walls (drainage zone) should consist of washed $\frac{3}{8}$ inch to No. 8 pea gravel or conform to Section 9-03.12(4) of the 2016 WSDOT Standard Specifications, as shown in Figure 4.
3. Structural fill for the reinforced fill zone and the retained soil for the MSE wall and for backfilling of the gravity block walls should consist of imported gravel borrow as described in Section 9-03.14(1) of the 2016 WSDOT Standard Specifications, with the additional restriction that the fines content be limited to no more than 5 percent.

4. Structural fill placed as crushed surfacing base course below pavements should conform to Section 9-03.9(3) of the 2016 WSDOT Standard Specifications.
5. Structural fill placed for the capillary break layer below building slabs should consist of 1½-inch minus clean crushed gravel with negligible sand or silt in conformance with Section 9-03.1(4)C, Grading No. 67 of the 2016 WSDOT Standard Specifications.

Reuse of On-Site Native Soils

The on-site native soils are expected to be suitable for structural fill in areas requiring compaction to at least 95 percent of MDD (per ASTM D 1557), provided the work is accomplished during the normally dry season (July through September) and that the soil can be properly moisture conditioned. It may be necessary to import gravel borrow to achieve adequate compaction for support of foundations, pavement areas, floor slabs and structures during wet weather construction. For planning purposes the project should include importing all structural fill for wet weather construction where compaction to at least 90 percent of the MDD is required.

The use of existing on-site native soils as structural fill during wet weather should be planned only for areas requiring compaction to 90 percent of the MDD, as long as the soils are properly protected and not placed during periods of precipitation. The contractor should plan to cover all fill stockpiles with plastic sheeting if it will be used as structural fill. The reuse of on-site soils is highly dependent on the skill of the contractor, schedule, and the weather, and we will work with the design team to maximize the reuse of on-site soils during the wet and dry seasons.

Reuse of On-Site Fill Soils

Fill soil exists across the site, particularly where it has been placed for roadways, building foundations or in other areas where grading activities have occurred. Existing fill can be reused on site where 90 percent compaction is required if careful construction practices are employed. As with the native soils, fill soils should be only considered for dry weather construction. Fill with significant organic materials, rubble or debris should be exported from the site or used in non-structural areas. The fill soils are generally over their optimum moisture content and drying of the soils may be needed in order to reuse them as structural fill.

Existing fill with appreciable amounts of debris should not be considered for reuse as structural fill unless the debris is removed from the fill. Existing fill associated with the southeast dump fill area, should remain on-site and remain beneath the clean cap fill after construction in this area is complete.

Reuse of Existing Asphalt, Base and Concrete Rubble

Existing asphalt pavement and portland cement concrete (PCC) rubble may be reused as structural fill if properly crushed during demolition. Recycled PCC rubble and base course materials may be reused as structural fill throughout the project. Recycled asphalt may be used under new pavement and hardscape areas and in utility trenches

For use as general structural fill across the site, the asphalt and concrete rubble should be crushed or otherwise ground up and should meet the gradation requirements for gravel borrow as described in Section 9-03.14(1) of the 2016 WSDOT Standard Specifications. If recycled asphalt and/or concrete will be used under pavement areas, we recommend that it meet the gradation requirements for crushed surfacing base course as described in Section 9-03.9(3) of the 2016 WSDOT Standard Specifications. Recycled concrete and asphalt should not be used under planned landscape areas.

Fill Placement and Compaction Criteria

Structural fill should be mechanically compacted to a firm, non-yielding condition. Structural fill should be placed in loose lifts not exceeding 12 inches in thickness when using heavy compaction equipment, and not more than 6 inches when using hand-operated compaction equipment. The actual thickness will be dependent on the structural fill material used and the type and size of compaction equipment. Each lift should be conditioned to within 2 percent of optimum moisture content and compacted to the specified density before placing subsequent lifts. Structural fill should be compacted in accordance with ASTM D 1557 to the following criteria:

1. Structural fill placed behind below grade walls and within 5 feet of the wall should be compacted to between 90 to 92 percent of the MDD. Care should be taken when compacting fill near the face of below grade walls to avoid over-compaction and hence overstressing the walls. Structural fill placed beyond the zone immediately behind the walls should be compacted to at least 95 percent of the MDD.
2. Structural fill in new pavement areas, including utility trench backfill, should be compacted to 90 percent of the MDD, except that the upper 2 feet of fill below final subgrade should be compacted to 95 percent of the MDD (see Figure 5). Local utility agencies may require stricter compaction criteria depending on the utility and its location and these requirements shall supersede our recommendations described above.
3. Structural fill placed below floor slabs and approved foundations should be compacted to 95 percent of the MDD.
4. Structural fill for the reinforced fill zone and retained fill for MSE walls should be compacted to 95 percent of the MDD.
5. Structural fill placed on slopes steeper than 5H:1V should be compacted to at least 90 percent of the MDD. In areas intended for future development, a higher degree of compaction should be considered to reduce the settlement potential of the fill soils.
6. Structural fill placed as crushed rock base course below pavements should be compacted to 95 percent of the MDD.
7. Non-structural fill, such as fill placed in landscape areas, should be compacted to at least 90 percent of the MDD, unless otherwise required by the landscape architect.

We recommend that a representative of GeoEngineers be present during proofrolling and to evaluate the exposed subgrade soils in building, retaining wall and pavement areas, and placement of structural fill. We will evaluate the adequacy of the subgrade soils and identify areas needing further work, perform in-place moisture density tests in the fill to verify compliance with the compaction specifications, and advise on any modifications to the procedures which may be appropriate for the prevailing conditions.

Weather Considerations

Disturbance of near surface soils should be expected if earthwork is completed during periods of wet weather. During dry weather the soils will: (1) be less susceptible to disturbance, (2) provide better support for construction equipment, and (3) be more likely to meet the required compaction criteria.

The wet weather season generally begins in October and continues through May in western Washington; however, periods of wet weather may occur during any month of the year. For earthwork activities during wet weather, we recommend that the following steps be taken:

- The ground surface in and around the work area should be sloped so that surface water is directed away from the work area. The ground surface should be graded so that areas of ponded water do not develop. Measures should be taken by the contractor to prevent surface water from collecting in excavations and trenches. Measures should be implemented to remove surface water from the work area.
- Earthwork activities should not take place during periods of heavy precipitation.
- Slopes with exposed soils should be covered with plastic sheeting.
- The contractor should take necessary measures to prevent on-site soils and soils to be used as fill from becoming wet or unstable. These measures may include the use of plastic sheeting, sumps with pumps, and grading. The site soils should not be left uncompacted and exposed to moisture. Sealing the surficial soils by rolling with a smooth-drum roller prior to periods of precipitation will help reduce the extent that these soils become wet or unstable.
- The contractor should cover all soil stockpiles that will be used as structural fill with plastic sheeting.
- Construction traffic should be restricted to specific areas of the site, preferably areas that are surfaced with the existing asphalt or granular materials not susceptible to wet weather disturbance.
- Construction activities should be scheduled so that the length of time that soils are left exposed to moisture is reduced to the extent practical.

Routing of equipment on the fill and native subgrade soils during the wet weather months will be difficult and the subgrade will likely become highly disturbed and rutted. In addition, a significant amount of mud can be produced by routing equipment directly on these soils in wet weather. Therefore, to protect the subgrade soils and to provide an adequate wet weather working surface for the contractor's equipment and labor, we recommend that the contractor protect exposed subgrade soils with sand and gravel, crushed gravel, or asphalt treated base (ATB). The contractor should also plan to limit the size of working areas and to protect other areas from access where possible to protect exposed subgrades.

Utility Trenches and Backfill

Trench excavation, pipe bedding, and trench backfilling should be completed using the general procedures described in the 2016 WSDOT Standard Specifications or other suitable procedures specified by the project civil engineer. The fill soils and native glacial deposits encountered at the site are generally of low corrosivity based on our experience in the Puget Sound area.

Utility trench backfill should consist of structural fill and should be placed in lifts of 12 inches or less (loose thickness) when using heavy compaction equipment such as hoe-packs, and not more than 6 inches when using hand-operated compaction equipment, such that adequate compaction can be achieved throughout the lift. Each lift must be compacted prior to placing the subsequent lift. Prior to compaction, the backfill should be moisture conditioned to within 2 percent of the optimum moisture content, if necessary.

The backfill should be compacted in accordance with the criteria discussed above. Figure 5 illustrates recommended trench compaction criteria under pavement and non-structural areas. We recommend that the lift thickness as well as the compaction criteria be adhered to in order to reduce potential settlement of trench backfill.

Construction Considerations – Existing Slopes

As previously described, up to 3 feet of fill exists along the alignment of the MSE wall. We recommend that this fill be removed from below the planned wall and the wall excavation extended down to medium dense to dense native soils. It may be necessary to place properly compacted structural fill for support of the wall if the excavation extends a significant distance below planned base of wall grades.

Foundation Support

We recommend that the proposed residential buildings be supported on shallow foundations constructed on undisturbed medium dense to very dense native outwash deposits or glacial till or on properly compacted structural fill overlying undisturbed medium dense to dense native soils.

Foundation Design

Perimeter footings should be at least 16 inches wide and interior column footings should be at least 24 inches wide. The design frost depth for the Puget Sound area is 12 inches, therefore, we recommend that exterior footings for structures be founded at least 18 inches below lowest adjacent grade. Interior footings should be founded at least 12 inches below top of slab or adjacent finished grade. Design of the building foundations should comply with the 2012 IBC.

We recommend using an allowable soil bearing capacity of 3,000 psf for shallow foundations bearing on medium dense to very dense native soils or on structural fill extending down to these soils. The allowable soil bearing pressure applies to the total of dead and long-term live loads and may be increased by up to one-third for wind or seismic loads.

The depth to suitable bearing soil will depend on the depth of the existing topsoil and fill soils, and the density of the underlying native soils.

Lateral Load Resistance

Lateral loads can be resisted by a combination of friction between the footing and the supporting soil, and by the passive lateral resistance of the soil surrounding the embedded portions of the footings. A coefficient of friction between concrete and soil of 0.4 and a passive lateral resistance corresponding to an equivalent fluid density of 350 pounds per cubic foot (pcf) may be used for design. The friction coefficient and passive lateral resistance are allowable values and include a factor of safety of about 1.5.

If soils adjacent to footings are disturbed during construction, the disturbed soils must be recompacted; otherwise the lateral passive resistance value must be reduced.

Foundation Settlement

We estimate that the post-construction settlement of footings founded on medium dense to very dense undisturbed native soils or structural fill extending down to medium dense to very dense undisturbed native soils, as recommended above, will be less than ½ inch. Differential settlement between comparably loaded

column footings or along a 25-foot section of continuous wall footing should be smaller than $\frac{1}{2}$ inch. We expect most of the footing settlements will occur as loads are applied.

Immediately prior to placing concrete, loose or disturbed soils that accumulate in the footing excavations during forming and steel placement must be removed. Debris or loose soils not removed from the footing excavations prior to placing concrete will result in additional settlement.

Footing Drains

We recommend that perimeter footing drains be installed around each building. The perimeter drains should be installed at the base of the exterior footings. The perimeter drains should be provided with cleanouts and should consist of at least 4-inch-diameter perforated pipe placed on a 3-inch bed of, and surrounded by, 6 inches of drainage material enclosed in a non-woven geotextile fabric such as Mirafi 140N (or approved equivalent) to prevent fine soil from migrating into the drain material. We recommend that the drainpipe consist of either heavy-wall solid pipe (SDR-35 PVC, or equal) or rigid corrugated smooth interior polyethylene pipe (ADS N-12, or equal). We recommend against using flexible tubing for footing drainpipes.

The drainage material should consist of pea gravel or “Gravel Backfill for Drains” per WSDOT 2016 Standard Specification Section 9-03.12(4), as shown in Figure 4.

The perimeter drains should be sloped to drain by gravity, if practicable, to a suitable discharge point, preferably a storm drain. We recommend that the cleanouts be covered, and be placed in flush mounted utility boxes. Water collected in roof downspout lines must not be routed to the footing drain lines.

Underslab Drainage

Groundwater may accumulate under buildings designed with below grade walls. To mitigate this condition, we recommend that the slabs for the buildings benched into slopes be provided with underdrainage to collect and discharge groundwater from below the slabs. This can be accomplished by installing a 4-inch-diameter, heavy-wall perforated collector pipe in a shallow trench placed below the capillary break layer. The trench should measure about 1-foot-wide by 1-foot-deep and should be backfilled with capillary break fill material or similar clean crushed rock with negligible fines content.

We recommend installing a single underdrain collector pipe below the long axis of the buildings that are benched into slopes and having below-grade walls. The collector pipe should be sloped to drain and discharge into the storm water collection system to convey the water off site. If connected to the footing drain pipe, the invert of the underslab drain pipe must be at a higher elevation to prevent water from flowing under the buildings from the perimeter system. The pipe should also incorporate cleanouts, if possible. The cleanouts could be extended through the foundation walls to be accessible from the outside, or could be placed in flush mounted access boxes cast into floor slabs.

Permanent Subsurface Walls

Cast-in-Place Walls

Conventional cast-in-place walls may be necessary for retaining structures located on-site, such as below-grade walls for residential buildings or cast-in-place walls for grade changes. The lateral soil pressures acting on conventional cast-in-place subsurface walls will depend on the nature, density and

configuration of the soil behind the wall and the amount of lateral wall movement which can occur as backfill is placed.

For walls that are free to yield at the top at least one-thousandth of the height of the wall, soil pressures will be less than if movement is limited by such factors as wall stiffness or bracing. Assuming that the walls are backfilled and drainage is provided as outlined in the following paragraphs, we recommend that yielding walls supporting horizontal backfill be designed using an equivalent fluid density of 35 pcf (triangular distribution), while non-yielding walls supporting horizontal backfill be designed using an equivalent fluid density of 55 pcf (triangular distribution). For unrestrained walls with backfill sloping up at 2H:1V, the design lateral earth pressure should be increased to 55 pcf, while restrained walls with a 2H:1V sloping backfill should be designed using an equivalent fluid density of 75 pcf.

These lateral soil pressures do not include the effects of surcharges such as floor loads, traffic loads or other surface loading. For seismic loading conditions, a rectangular earth pressure equal to $8H$ psf, where H is the height of the wall, should be added to the active/at-rest pressures presented above. Traffic surcharges can be approximated by increasing the wall height by 2 feet. Other surcharge loading should be applied as appropriate, as shown in Figure 6.

Lateral resistance for conventional cast-in-place walls can be provided by frictional resistance along the base of the wall and passive resistance in front of the wall. The allowable frictional resistance may be computed using a coefficient of friction of 0.4 applied to vertical dead-load forces. The allowable passive resistance may be computed using an equivalent fluid density of 350 pcf (triangular distribution). The above coefficient of friction and passive equivalent fluid density values incorporate a factor of safety of about 1.5.

The above soil pressures assume that wall drains will be installed to prevent the buildup of hydrostatic pressure behind the walls, as discussed below.

Drainage

Positive drainage should be provided behind cast-in-place retaining walls by placing a minimum 2-foot-wide zone of wall drainage material, as shown in Figure 4. The drainage zone should extend from the base of the wall to within 2 feet of the finished ground surface. The top 2 feet of fill should consist of relatively impermeable soil, such as on-site glacial till, to prevent infiltration of surface water into the wall drainage zone.

A 4-inch minimum diameter pipe should be located at the base of the wall in the drainage zone to remove water that collects in this zone. The drainpipe should be placed with 0.5 percent minimum slopes and discharge to an appropriate location. Alternatively, drainage can be provided with weep holes designed in accordance with WSDOT Standard Plans if the water exiting the weep holes is acceptable.

Construction Considerations

Backfill placed within 5 feet of below grade walls should be compacted to densities ranging from 90 to 92 percent of the MDD obtained in accordance with the ASTM D 1557 procedure to reduce the potential for development of excess pressure on the walls. If sidewalks or pavement will be placed adjacent to the wall, we recommend that the upper 2 feet of fill be compacted to 95 percent of the MDD. Measures

should be taken to prevent the buildup of excess lateral soil pressures due to over-compaction of the backfill behind the wall; for example, by using hand-operated compaction equipment.

Slab-on-Grade Floors

Conventional slabs may be supported on-grade provided the subgrade soils and structural fill are prepared as recommended under the “Earthwork” section of this report. We recommend that slabs be founded on either undisturbed medium dense to very dense native soils or on structural fill placed over native soils. For slabs designed as a beam on an elastic foundation, a modulus of subgrade reaction of 100 pounds per cubic inch (pci) may be used for subgrade soils prepared as recommended.

Capillary Break

We recommend that the slab-on-grade floors for buildings be underlain by a capillary break gravel layer consisting of 4 inches of material meeting the requirements of 2016 WSDOT Standard Specification 9-03.1(4)C, Grading No. 67 with the exception that this material should have negligible sand or fines (see Figure 4).

Vapor Retarder

If water vapor migration through the slabs is objectionable, an appropriate vapor retarder such as 10-mil plastic sheeting should be placed between the floor slab and the capillary break layer to reduce the upward migration of moisture through the slab. This will be desirable where the slabs will be surfaced with tile or will be carpeted. It may also be prudent to apply a sealer to the slab to further retard the migration of moisture through the floor.

Pavement Recommendations

Subgrade Preparation

We recommend that the subgrade soils in new residential pavement areas be prepared and evaluated as described in the “Earthwork” section of this report. In cut areas exposing medium dense to dense native soils, we recommend that the subgrade surface be compacted to at least 95 percent of the MDD obtained using ASTM D 1557 prior to placing pavement section materials.

Where new or existing fill is present, we recommend that the upper 2 feet of subgrade soil below all pavement areas be compacted or recompact to at least 95 percent of the MDD. If the subgrade soils are loose or soft, it may be necessary to excavate the soils and replace them with structural fill, gravel borrow, or gravel base material. Based on our test pits TP-1 and TP-7, the majority of the pavement subgrade soils are expected to consist of medium dense to dense outwash deposits.

Pavement subgrade conditions should be observed and proofrolled or probed during construction to evaluate the presence of unsuitable subgrade soils and the possible need for over-excavation and placement of a geotextile fabric.

New Residential Pavements

Based on our experience in the site vicinity, we recommend that the hot-mix asphalt (HMA) pavement sections presented in Table 2 below be used for the project. We can evaluate the pavement sections based

on site-specific traffic data, if needed. Parking areas and alleys at the site should be designed in accordance with the Neighborhood Subaccess section recommendations shown in Table 2.

TABLE 2. RECOMMENDED NEW PAVEMENT SECTIONS

Material	Neighborhood Subcollector Section Thickness (inches)	Neighborhood Subaccess Section Thickness (inches)	WSDOT¹ Standard Specification
½ inch HMA; PG 58-22	2.5	2	5-04 and 9-03
Asphalt Treated Base	-	-	4-06
Crushed Surfacing Base Course	6	4	9-03.9(3)

Notes:

¹ WSDOT = Washington State Department of Transportation, 2016, Standard Specifications for Road, Bridge and Municipal Construction.

Asphalt-Treated Base

Because pavements may be constructed during the wet season, consideration may be given to covering the areas to be paved with ATB for protection. Subaccess pavement areas and neighborhood subcollector pavement areas should be surfaced with at least 4 inches of ATB. Prior to placement of the final pavement sections, we recommend that areas of ATB pavement failure be removed and the subgrade repaired. If ATB is used and is serviceable when final pavements are constructed, the crushed surfacing base course can be eliminated, and the design HMA pavement thickness can be placed directly over the ATB.

Storm Water Detention Pond

We understand that a permanent storm water pond is planned near the center of the site and northwest of SW 97th Place with an outfall planned in the southeast area between lots 17 and 18. We understand that the planned bottom of the pond will be at about Elevation 362 feet.

Soil Conditions

We anticipate that the storm water detention pond will be excavated in dense outwash deposits at the site. The dense outwash deposits were observed from about 1½ to 17 feet below the existing ground surface in test pit TP-8. Groundwater seepage was encountered at about 13 feet below the existing ground surface in test pit TP-8.

Permanent Slopes

We recommend that permanent cut and fill slopes within the detention pond be no steeper than 3H:1V. To achieve uniform compaction on interior fill slopes, we recommend that fill slopes be overbuilt slightly and subsequently cut back to expose well compacted fill. This should be discussed with the contractor and incorporated into the project plans.

Gravity Block Walls

We understand that gravity block walls with heights up to 15 feet are planned along the southeast, west and north sides of the storm water detention pond. These walls will have a 3H:1V fore slope down to the bottom of the pond. The back slope above the wall segments will vary from level to 2H:1V. We anticipate that the wall backfill will consist of imported gravel borrow conforming to Section 9-03.14(1) of the

2016 WSDOT Standard Specifications. Native soils excavated from the detention pond should not be used as backfill for the gravity block walls.

Our preliminary wall design recommendations and calculations for the gravity block walls are presented in our June 16, 2015 letter report, which is included as Appendix D in this report. Figure 7 in this report is an updated version and replaces Figure 2 in our June 16, 2015 report. Figure 7 presents typical cross sections and notes for gravity block walls with both a level back slope and a backslope inclined at 2H:1V. Calculations for the gravity block wall with a level backslope are included with the June 16, 2015 report (Appendix D of this report), Calculations related to the wall with 2H:1V backslope are also included in Appendix D.

Erosion Control

To reduce potential erosion and to help establish permanent vegetation on the pond side slopes, we recommend that erosion protection of the slopes include hydroseeding in conjunction with installation of an erosion control blanket. We recommend that the erosion control blanket be staked to disturbed slopes to help reduce the risk of erosion during wet work periods and after the work is completed. We recommend that the erosion control blanket consist of a product such as Curlex II, manufactured by American Excelsior Co., or SC150BN, manufactured by North American Green. We recommend that the erosion control blanket be installed in accordance with the manufacturer's recommendations and that the installation and stapling methods be observed during construction.

Hydroseeding and installation of the erosion control blanket should occur as soon as possible and prior to the wet winter months. Hydroseeding should occur to allow proper germination before the winter. We also recommend that the hydroseed mix include a tackifier to increase adhesion between the hydroseed mixture and the granular native soils.

Outfall Pipe Scour Protection

We recommend that storm water conveyances from the storm water pond outlet extend to properly protected energy dissipation structures, and that potential downstream erosion and scour be prevented, especially in the southeast area where steep slope hazard areas exist. Energy dissipaters should be designed and constructed at the outfall to prevent erosion or scour along the planned clean soil cap for the dump fill area.

We recommend that the area around the outlet pipe be protected from possible scour due to flow through the pipe. This can be achieved by providing a dissipation pad or other erosion control structure around and downstream of the outlet pipe. The erosion control structure should extend far enough and wide enough to prevent future erosion/scour of the slope. If riprap is used, we recommend that a geotextile separator, such as TC Mirafi 180N, be placed under the riprap to help prevent scouring of the underlying soil.

We recommend that the planned storm water drainage path down slope of the outlet pipe be lined to prevent scouring. As discussed previously, the shallow drainage swale (about 1 foot deep) may be lined with a permanent TRM such as Propex Landlok 450. The TRM should be placed over a shallow channel shaped into the existing ground surface. The shaped channel should have side slopes inclined no steeper than 3H:1V, be at least 1 foot deep and at least 1 foot wide at the bottom. The TRM should be placed over the prepared surface and anchored in accordance with the manufacturer's recommendations. The geotechnical engineer should observe the prepared ground surface before placing the TRM to confirm there

is good contact between the TRM and the underlying soil. After installing the TRM, hydroseed consisting of a native ground cover seed mix that grows in a dense nature and in a relatively shady environment should be applied to the TRM and channel.

We understand the design team is also considering lining the channel with riprap. If riprap lining is used, we recommend that it be underlain with a nonwoven geotextile separator consisting of TenCate Mirafi 160N or equal.

Seepage Protection

The design of the detention pond outfall should incorporate features that will protect the slopes from potentially damaging seepage along the outlet pipe. We recommend that potential seepage along the outlet pipe be prevented by constructing seepage cut-off collars around the outlet pipe. A cut-off collar should be constructed about 5 to 10 feet beyond the interior pond slope. We recommend that the cutoff collar extend a minimum of 1 foot into the trench sidewalls perpendicular to the pipe and be at least 12 inches wide along the pipe. The cut-off collar should be constructed of concrete, CDF or a cement-bentonite mixture.

MSE Wall with Vegetated Face

We understand that the MSE wall planned along the southeast side of Lots 15 through 24 will be up to 18 feet high and will have a vegetated face. The wall will be embedded a sufficient distance below the finished downslope surface, which will be inclined no steeper than 3H:1V. The wall will have a level back slope.

We anticipate the wall backfill will consist of (1) imported Gravel Borrow conforming to Section 9-03.14(1) of the 2016 WSDOT Standard Specifications, or (2) suitable granular on-site soil originating from the excavation for the proposed detention pond and other areas of the site. Native soils excavated from the detention pond may be reused in the retained soil zone for the vegetated face retaining wall.

Our preliminary wall design recommendations and calculations for the MSE wall are presented in our June 16, 2015 letter report, which is included as Appendix D in this report. Figure 1 included with the June 16, 2015 report presents a typical cross section and notes for the MSE wall.

The geogrid reinforcement for the MSE wall will extend behind the face of the wall by a horizontal distance equal to about 0.95 percent of the wall height. We understand the buildings will be set back at least 10 feet from the face of the MSE wall. The upper layer of geogrid will be at least 18 inches below the ground surface, and it is permissible to construct the building foundations and floor slabs over the ends of the geogrid, if needed, without compromising the stability or integrity of the wall system.

MSE walls with geogrid reinforcement are commonly designed to include surcharge loading and seismic forces, and our wall design includes a surcharge of 250 psf. With the 10-foot setback distance, this surcharge pressure adequately accounts for the impact of house foundation loading on the MSE wall system.

Rockerries

We understand that rockeries may be used for grade transitions along 4th Avenue SW. The height of the rockeries will range from about 1 to 5½ feet. It is important to realize that rockeries provide only limited

soil retention and are not intended as structural retaining walls. The primary purpose of a rockery is to protect the slope face from erosion and raveling, while providing limited soil retention.

Rockerries may be used in both cut and fill areas. Rockerries with a horizontal backslope should be limited to 8 feet in height in cut areas. If rockeries are to face fill areas greater than 4 feet in height or if they have inclined backslopes, then the fill should be designed and reinforced with geosynthetic materials. The height is measured as the vertical distance from the ground surface in front of the toe of the rockery to the ground surface behind the top of the rockery.

The base of rockeries should be embedded at least one-half the thickness of the lowest course of rocks or 18 inches below the adjacent ground surface, whichever is greater. The rockery should be supported on firm, undisturbed native soils or compacted structural fill. We recommend the condition of the rockery subgrades be observed by GeoEngineers to evaluate whether additional overexcavation is needed. Overexcavated areas should be backfilled with structural fill, with the width of excavation extending beyond the edge of the rockery a distance equal to the depth of overexcavation.

We recommend that rockeries be constructed using rock weights and sizes as specified in Sections 8-24 and 9-13.7(1) of the 2016 WSDOT Standard Specifications. Other requirements by King County may also apply. The rockery face should be constructed with a batter of between 1H:5V and 1H:6V. Rock courses should be gradational in size from top to bottom with the largest rocks of uniform size being placed for the lowest course. A geotextile separator should be placed across the cut slope prior to placing rock backfill behind the rockery.

Rockerries should be installed by a qualified contractor experienced in rockery construction. Typical rockery cross sections and construction guidelines for rockeries in cut and fill conditions are shown on Figures 8 and 9, respectively.

Permanent drainage systems should intercept surface water runoff at the top of rockeries to prevent it from flowing in an uncontrolled manner across the rockeries.

Southeast Soil Cap

As described in previous environmental and geotechnical documents for the Greenbridge and Wind Rose sites, a debris-laden dumped fill exists in the southeast area of the Wind Rose project site. The approximate location of the dumped fill is shown on Figure 2. The fill includes various debris, including plastic, metal, wire, tires, and assorted other materials mixed in with soil fill.

As part of site development, we recommend that the dumped fill area be capped with a layer of clean compacted fill soil. The purpose of the soil cap is to reduce infiltration, provide better surface water control, and prevent people from contacting the existing fill materials in this area. The soil cap is a conservatively protective “preventative” measure for an area that is not planned for future use except as Open Space.

Our recommendations for the cap include the following:

- Regrade existing dumped fill slopes to no steeper than 2H:1V.
- Bench steps into dumped fill slope to receive cap fill and geosynthetic materials.
- Cut key for toe of cap fill slope.

- Cover entire fill area with a nonwoven separation geotextile (TenCate Mirafi 180N). Install geotextile sheets such that seams are perpendicular to slope contours.
- Place a high visibility geosynthetic material (e.g. orange safety fencing) on top of nonwoven geotextile.
- Place 3 feet of clean compacted cap fill over geosynthetic materials. Place fill in 12-inch maximum loose lifts and compact each lift to at least 90 percent MDD obtained using ASTM D 1557. Use on-site silty sand for clean cap fill.
- Extend cap fill at least 3 feet beyond limits of debris fill. Determine lateral extent of existing dumped fill during construction. Do not extend cap beyond east property line.
- Grade cap fill surface to promote positive surface water drainage.
- Install an erosion control blanket on cap fill slopes. Erosion blanket may consist of North American Green S150BN or American Excelsior Curlex II.
- Protect toe of cap from potential scour/erosion from planned detention pond outfall pipe.
- Hydroseed cap to establish a dense stand of permanent vegetation for permanent erosion protection.

Permanent Drainage Considerations

We recommend that all surfaces be sloped to drain away from the proposed building areas. Pavement surfaces and open space areas should be sloped such that the surface water is collected and routed to suitable discharge points.

Roof drains should be connected to tightlines that discharge appropriately. Water collected in roof downspout lines should be routed to appropriate discharge points in separate pipe systems. Roof downspout lines must not be connected to footing drain systems.

Recommended Additional Geotechnical Services

Throughout this report, recommendations are provided where we consider additional geotechnical services to be appropriate. These additional services are summarized below:

- GeoEngineers should be retained to provide additional geotechnical recommendations during final design of the project.
- GeoEngineers should be retained to review the project plans and specifications when complete to confirm that our design recommendations have been implemented as intended.
- During construction, GeoEngineers should evaluate the suitability of the foundation, retaining wall, pavement and slab subgrades, observe installation of rockeries and subsurface drainage measures, observe the construction of the gravity block and MSE walls, observe and test placement and compaction of structural backfill, observe placement of erosion control products including the TRM, and provide a summary letter for our construction observation services. The purposes of GeoEngineers construction phase services are to confirm that the subsurface conditions and construction methods are consistent with those observed in the explorations and other reasons described in Appendix E, Report Limitations and Guidelines for Use.

LIMITATIONS

We have prepared this report for use by King County Housing Authority and other members of the design teams for the southeast part of the Wind Rose Neighborhood Development project.

Our services were provided to assist in the design of structures on sloping ground. Our recommendations are intended to improve the overall stability of the site and to reduce the potential for future property damage related to earth movements, drainage or erosion. Qualified engineering and construction practices can help mitigate the risks inherent in construction on slopes, although those risks cannot be eliminated completely. Favorable performance of structures in the near term is useful information for anticipating future performance, but it cannot predict or imply a certainty of long-term performance, especially under conditions of adverse weather or seismic activity.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering in this area at the time this report was prepared. No warranty or other conditions, express or implied, should be understood.

Any electronic form, facsimile or hard copy of the original document (email, text, table, and/or figure), if provided, and any attachments are only a copy of the original document. The original document is stored by GeoEngineers, Inc. and will serve as the official document of record.

Please refer to Appendix E titled “Report Limitations and Guidelines for Use” for additional information pertaining to use of this report.

REFERENCES

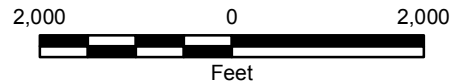
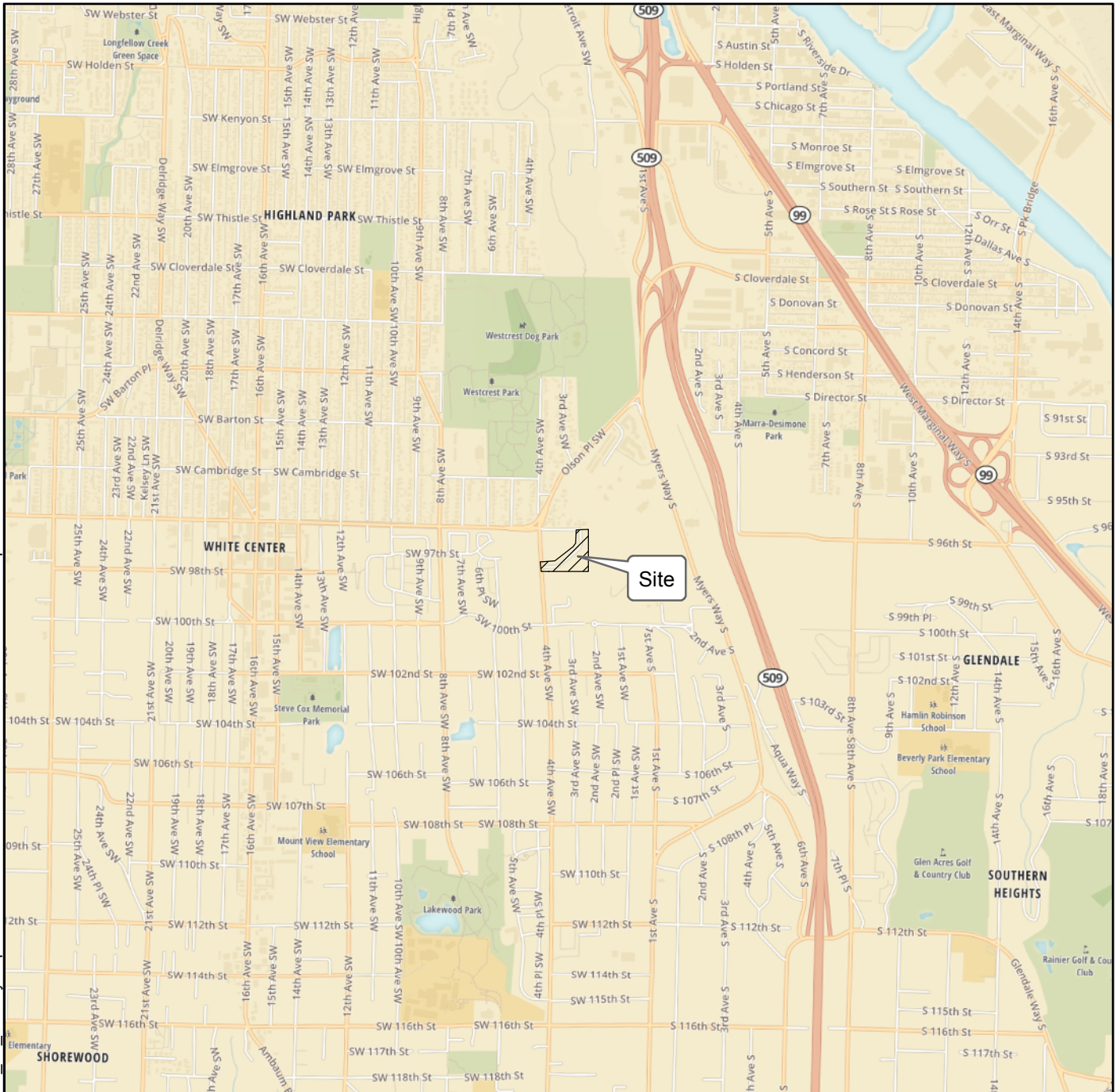
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U.S. Geological Survey, “Earthquake Hazards Program, Interpolated Probabilistic Ground Motion for the Conterminous 48 States by Latitude Longitude, 2002 Data,” accessed using Earthquake Ground Motion Parameters Version 5.0.8 on February 20, 2015.

Washington State Department of Transportation , 2016, “Standard Specifications for Road, Bridge and Municipal Construction.”

Troost, Kathy Goetz, Booth, Derek B., Wisher, Aaron P., and Shimel, Scott A., 2005. The Geologic Map of Seattle – a Progress Report: U.S. Geological Survey Open-File Report 2005-1252.



Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.
3. It is unlawful to copy or reproduce all or any part thereof, whether for personal use or resale, without permission.

Data Sources: Open Street Map 2015

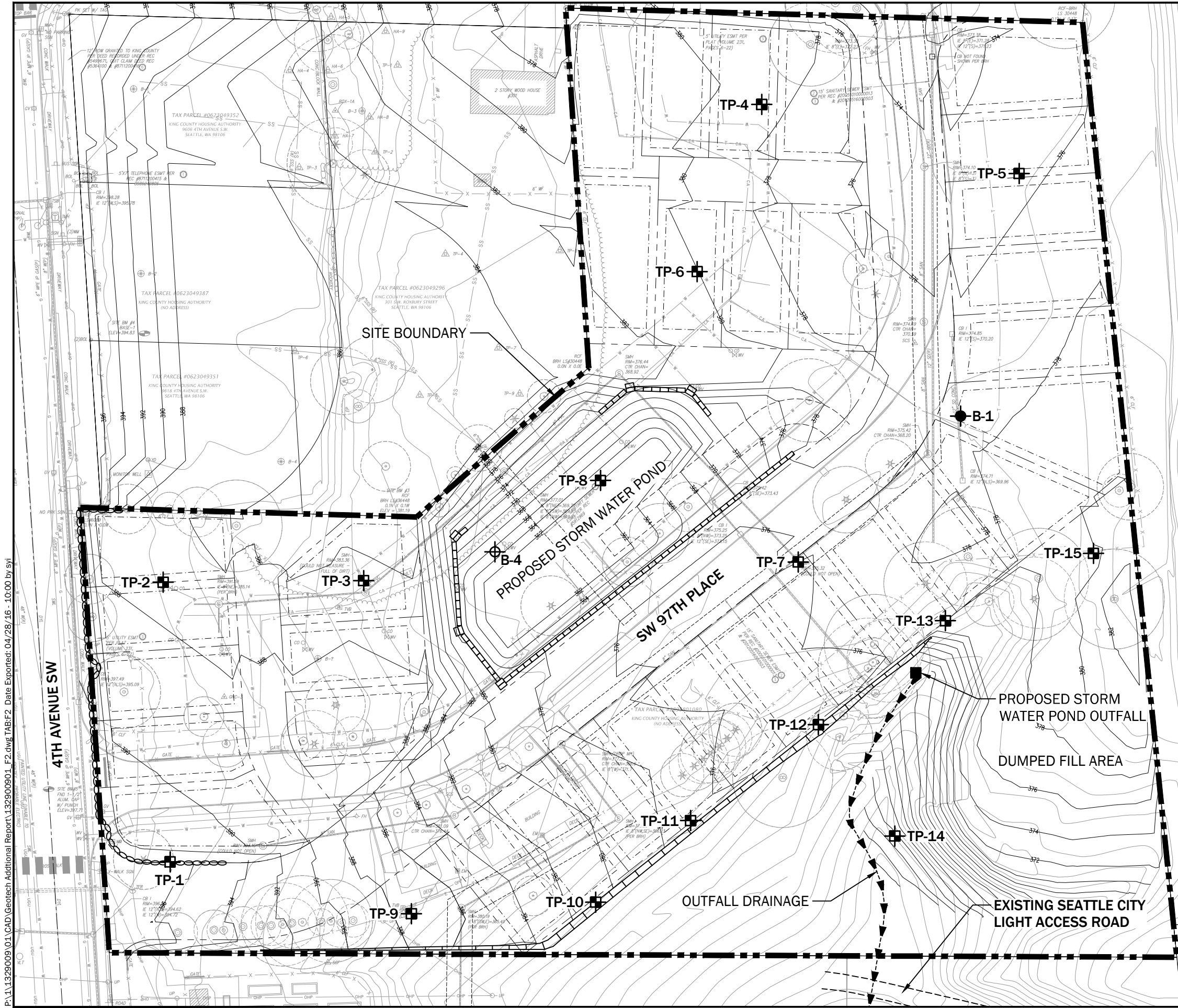
Transverse Mercator, Zone 10 N North, North American Datum 1983
North arrow oriented to grid north

Vicinity Map

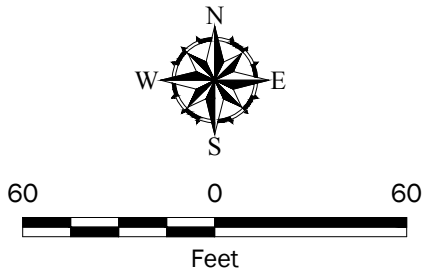
Wind Rose Development
King County, Washington



Figure 1



- Legend**
- TP-1 Test pit completed by GeoEngineers Inc., 2016
 - B-4 Boring completed by GeoEngineers Inc., 2007
 - B-1 Boring completed by GeoEngineers Inc., 2002

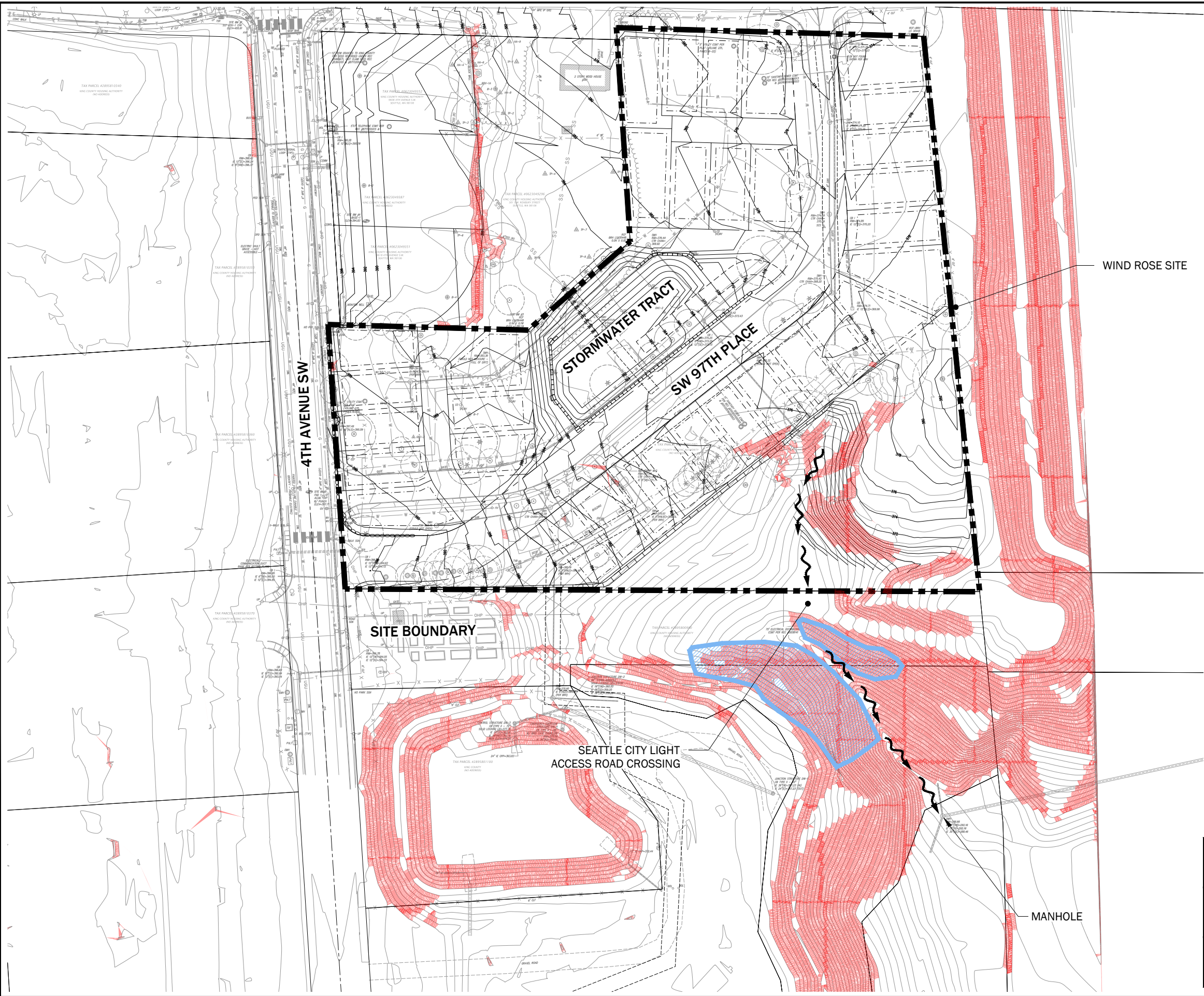


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Data Source: Base drawing provided by KPFF.

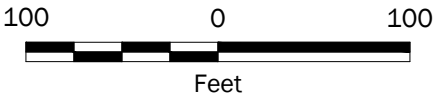
Projection: NAD83 Washington State Planes, North Zone, US Foot.

Site Plan	
Wind Rose Development King County, Washington	
GEOENGINEERS	Figure 2



Legend

- Steep slopes greater than 40% grade
- Slope Erosion and Shallow Slumps
- Stormwater Flow Path

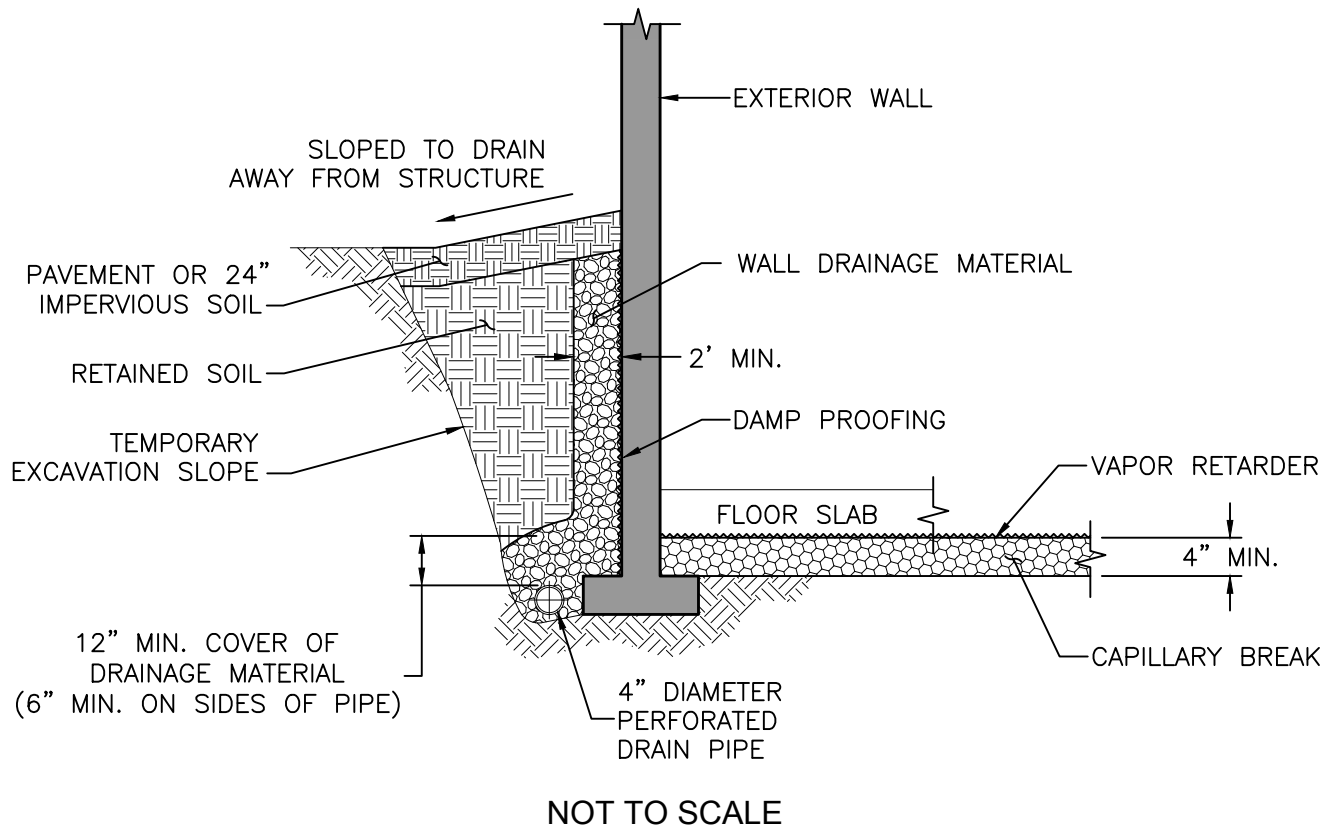


- Notes:
- The locations of all features shown are approximate.
 - This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Base drawing provided by KPFF.

Projection: NAD83 Washington State Planes, North Zone, US Foot.

Steep Slope Critical Areas	
Wind Rose Development King County, Washington	
GEOENGINEERS	Figure 3



Materials:

A. **ALL DRAINAGE MATERIAL:** Must consist of washed #57 to No. 10 pea gravel. Alternatively, the wall drainage material may consist of "Gravel Backfill for Drains" per SDOT Standard Specification 9-12.1, surrounded by a non-woven geotextile such as Mirafi 100N (or approved equivalent).

B. **RETAINED SOIL:** Should consist of structural fill, either on-site soil or imported. The backfill should be compacted in loose lifts not exceeding 6 inches. All backfill supporting building floor slabs should consist of imported sand and gravel per SDOT Standard Specification 9-1.1 or City of Seattle Type 17 compacted to at least 95 percent ASTM D1557. Backfill not supporting building floor slabs, sidewalks, or pavement should be compacted to 90 to 92 percent of the maximum dry density, per ASTM D1557. Backfill supporting sidewalks or pavement areas should be compacted to at least 95 percent in the upper two feet. Only hand-operated equipment should be used for compaction within 5 feet of the walls and no heavy equipment should be allowed within 5 feet of the wall.

C. **CAPILLARY BREAK:** Should consist of at least 4 inches of clean crushed gravel with a maximum size of 1-1/2 inches and negligible sand or fines.

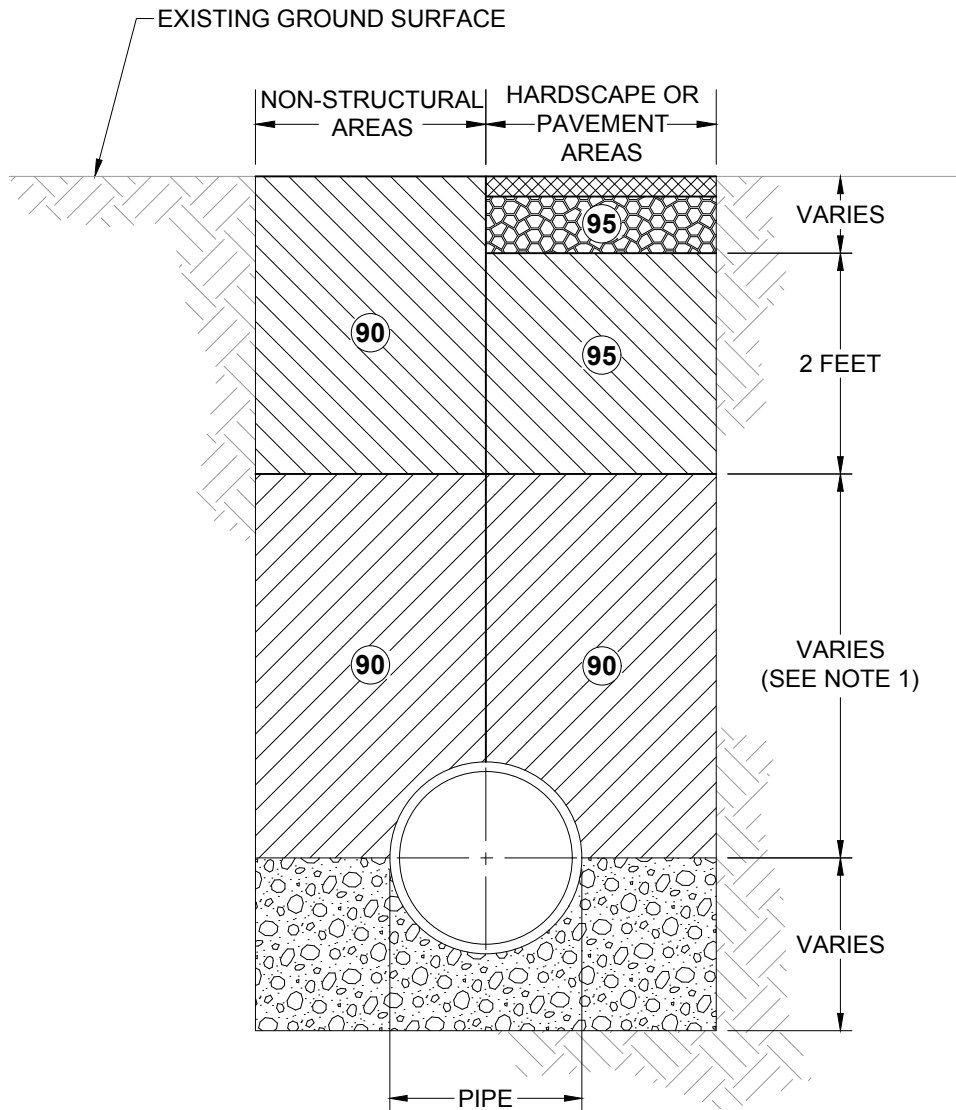
D. **PERFORATED DRAIN PIPE:** Should consist of a 4-inch diameter perforated heavy-wall solid pipe (SDR-35 PVC) or rigid corrugated polyethylene pipe (ADS N-12) or equivalent. Drain pipes should be placed with 0.5 percent minimum slopes and discharge to the storm water collection system.

Wall Drainage and Backfill

Wind Rose Development
King County, Washington



Figure 4



NOT TO SCALE

LEGEND:

95	RECOMMENDED COMPACTION AS A PERCENTAGE OF MAXIMUM DRY DENSITY, BY TEST METHOD ASTM D1557 (MODIFIED PROCTOR)
	CONCRETE OR ASPHALT PAVEMENT
	BASE COURSE
	TRENCH BACKFILL
	PIPE BEDDING

NOTES:

1. All backfill under building areas should be compacted to at least 95 percent per ASTM D1557.

Compaction Criteria for Trench Backfill

Wind Rose Development
King County, Washington



Figure 5

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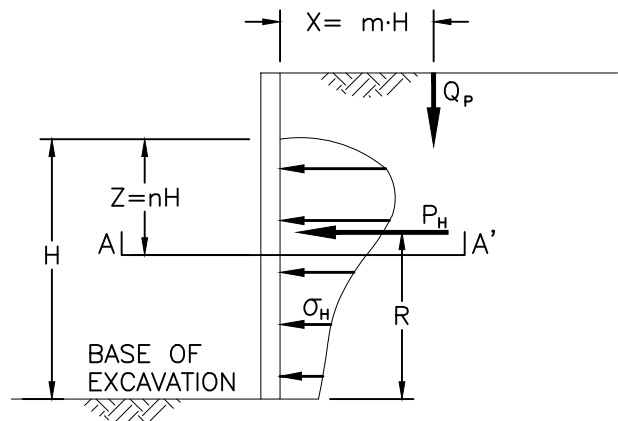
DEFINITIONS:

- Q_p = Point load in pounds
 Q_L = Line load in pounds/foot
 H = Excavation height below footing, feet
 σ_H = Lateral earth pressure from surcharge, psf
 q = Surcharge pressure in psf
 θ = Radians
 σ_H' = Distribution of σ_H in plan view
 P_H = Resultant lateral force acting on wall, pounds
 R = Distance from base of excavation to resultant lateral force, feet

NOTES:

1. Procedures for estimating surcharge pressures shown above are based on Manual 7.2 Naval Facilities Engineering Command, September 1966 (NAVFAC DM 7.2).
2. Lateral earth pressures from surcharge should be added to earth pressures presented on Figures 8 and 9 as appropriate.
3. See report text for where surcharge pressures are appropriate.

LATERAL EARTH PRESSURE FROM
POINT LOAD, Q_p (SPREAD FOOTING)



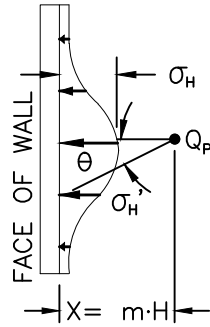
FOR $m \leq 0.4$

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

FOR $m > 0.4$

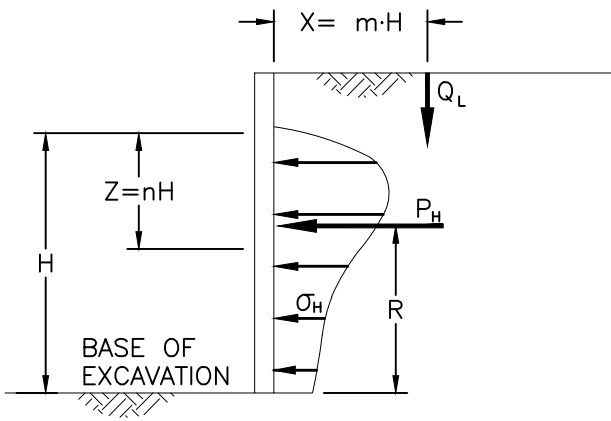
$$\sigma_H = \frac{1.77Q_p m^2 n^2}{H^2(m^2+n^2)^3}$$
$$\sigma_H' = \sigma_H \cos^2(1.1\theta)$$

M	$P_H \left(\frac{H}{Q_p} \right)$	R
0.2	0.78	0.59H
0.4	0.78	0.59H
0.6	0.45	0.48H



SECTION A-A'
Pressures from Point Load Q_p

LATERAL EARTH PRESSURE FROM
LINE LOAD, Q_L (CONTINUOUS WALL FOOTING)



FOR $m \leq 0.4$

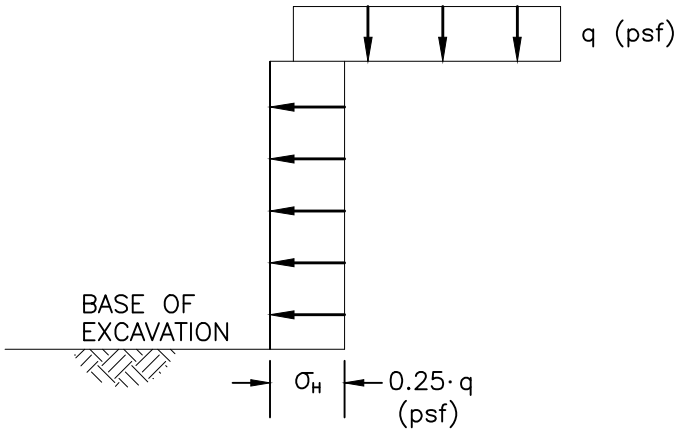
$$\sigma_H = \frac{0.2n \cdot Q_L}{H(0.16+n^2)^2}$$
$$P_H = 0.55Q_L$$

FOR $m > 0.4$

$$\sigma_H = \frac{1.28m^2 n \cdot Q_L}{H(m^2+n^2)^2}$$
$$P_H = \frac{0.64Q_L}{(m^2+1)}$$

M	R
0.1	0.60H
0.3	0.60H
0.5	0.56H
0.7	0.48H

UNIFORM SURCHARGES, q
(FLOOR LOADS, LARGE FOUNDATION
ELEMENTS OR OTHER SURFACE LOADS)



σ_H = LATERAL SURCHARGE PRESSURE
FROM UNIFORM SURCHARGE

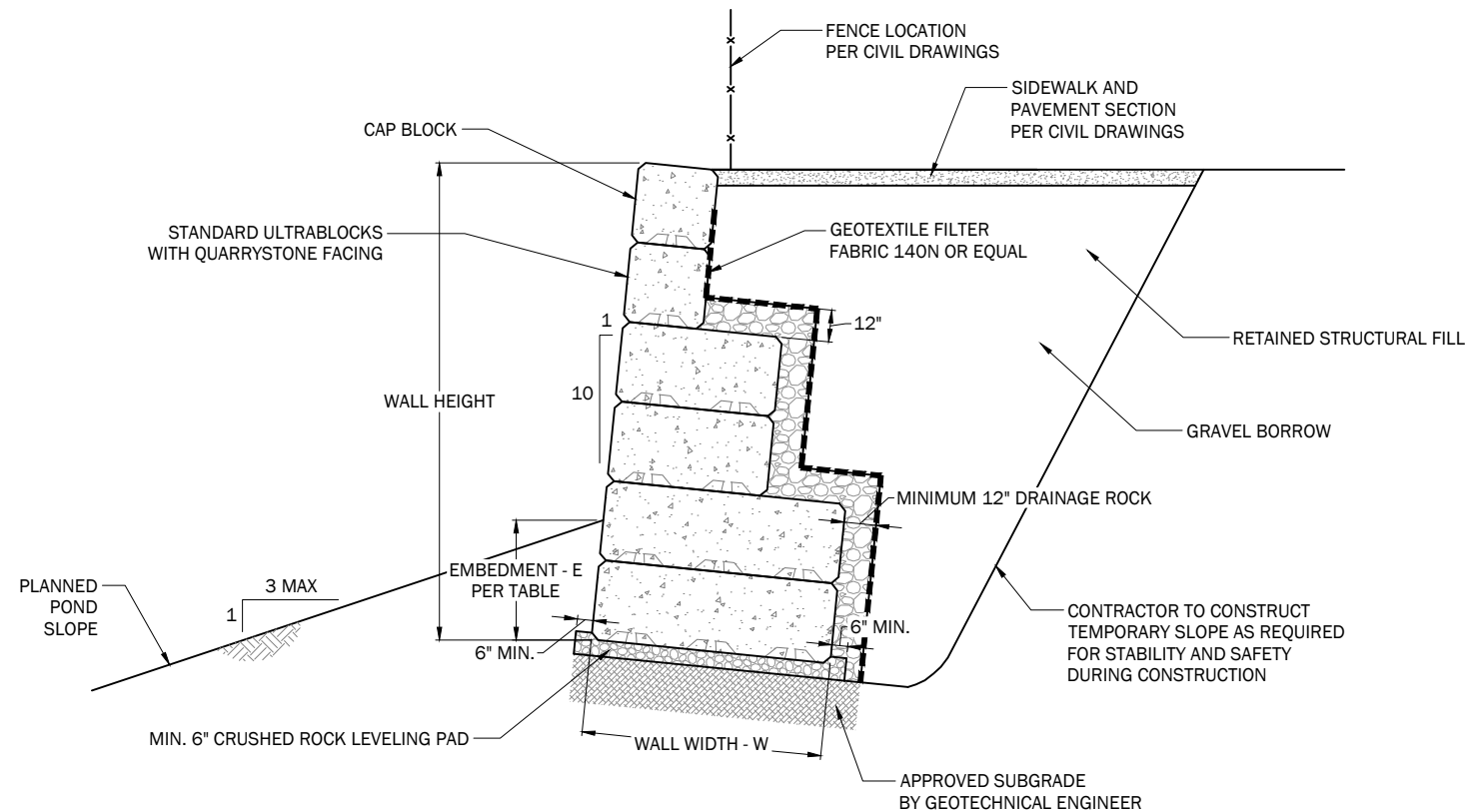
Recommended Surcharge Pressure

Wind Rose Development
King County, Washington



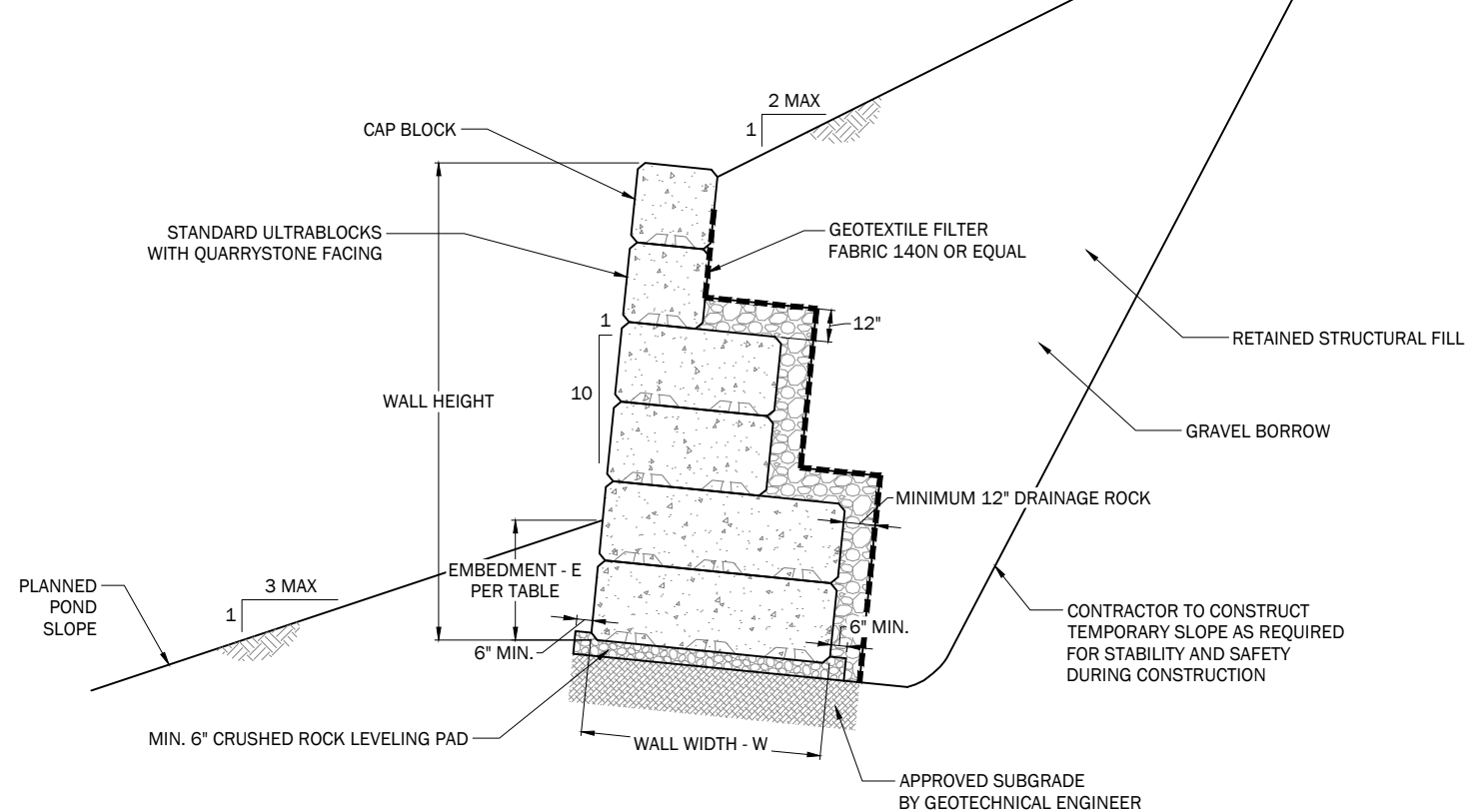
Figure 6

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WALL CROSS SECTION FOR DETENTION POND (LEVEL BACKSLOPE)
NOT TO SCALE

Wall Height (feet)	Number of Turned Base Blocks	Wall Width, W (feet)	Minimum Embedment Depth, E (feet)
15	4	7.5	3
12.5	3	5	2
10	2	5	2
7.5	1	5	1.5
5	0	2.5	1



WALL CROSS SECTION FOR DETENTION POND (2H:1V MAX BACKSLOPE)
NOT TO SCALE

Wall Height (feet)	Number of Turned Base Blocks	Wall Width, W (feet)	Minimum Embedment Depth, E (feet)
15	4	7.5	3
12.5	3	7.5	2
10	2	5	2
7.5	1	5	1.5
5	0	2.5	1

Construction Notes

Materials:

A. Segmental Concrete Units

- Wall units shall be 22.46' x 2.46' x 4.92' blocks produced to ULTRABLOCK specifications.

B. Drainage Rock

- Drainage Rock shall meet WSDOT standard specification 9-03.12(4).

C. Crushed Rock Leveling Pad

- Crushed rock used as a leveling pad at the base of the wall shall meet WSDOT Standard Specification 9-03.9(3) "Crushed Surfacing".

D. Backfill Materials

- Backfill material shall meet WSDOT 9-03.14(1) Gravel Borrow or other material as approved by Geotechnical Engineer.

Execution:

A. Subgrade Preparation

- Subgrade bearing surfaces shall be cleared and grubbed, removing organic or deleterious materials. Soft areas shall be overexcavated and replaced with structural fill prior to placing blocks, as directed by Geotechnical Engineer.
- Subgrade shall be excavated to a depth such that the base of the first course of block is at the minimum embedment depth shown in the table on this sheet, unless otherwise specified in the design or as directed by Geotechnical Engineer.
- Subgrade shall be approved by Geotechnical Engineer. Soil shall be compacted before construction proceeds. Subgrade materials not meeting Geotechnical Engineer's approval shall be removed and replaced with structural fill material at direction of Geotechnical Engineer.
- Overexcavated areas shall be replaced with structural backfill material.

B. Leveling Pad

- Leveling pad shall be placed as shown on the construction plans.
- Leveling pad shall be placed on undisturbed native soils or on properly compacted structural fill.
- Leveling pad shall be compacted to 95 percent of modified proctor dry density to ensure a hard, level surface on which to place the first course of blocks. Pad shall be constructed at the proper elevation and slope to ensure the final elevations and batter as shown on the plans. The manufacturer recommends using 2x6 wood boards and steel stakes to make a form for achieving design batter.
- Leveling pad shall have a minimum thickness of 6 inches. Leveling pad shall extend beyond the blocks in all directions at least 6 inches.

C. Unit Installation

- The first course of wall units shall be placed on the prepared base with the front edges tight together. The units shall be checked for level and alignment as they are placed.
- Ensure that units are in full contact with base. Proper care shall be taken to develop straight lines and smooth curves on base course as per wall layout.
- Fill a minimum of 12 inches behind the base course with Drainage Rock. Use approved soils to backfill behind the wall rock and in front of the base course to firmly lock in place. Check again for level and alignment. Use a plate compactor to consolidate the area behind the first course. All excess material shall be swept from top of units.

D. Backfill Placement

- Backfill material shall be placed in lifts and compacted to a minimum 95 percent of Maximum Dry Density (MDD) as determined by ASTM D-1557.
- Only hand-operated compaction equipment shall be allowed within 5 feet behind the wall. Backfill in this zone shall be compacted to 90 percent of (MDD) as determined by ASTM D-1557.
- The excavated trench area in front of the toe of the wall shall be filled and compacted as the wall is being constructed.

DESIGN PARAMETERS

ALLOWABLE SOIL BEARING PRESSURE = 3,000 PSF
DESIGN OF THE RETAINING STRUCTURE IS BASED ON THE FOLLOWING PARAMETERS:

	FRICTION ANGLE, PHI (degrees)	COHESION, C (psf)	MOIST UNIT WEIGHT (pcf)	BUOYANT UNIT WEIGHT (pcf)
RETAINED SOIL OR BACKFILL	34	0	125	62.6
FOUNDATION SOIL	36	0	125	62.6

EXTERNAL STABILITY OF WALLS:
SURCHARGE LOADING = 250 PSF
SEISMIC ACCELERATION = 0.5xPGA = 0.21g
MINIMUM FACTOR OF SAFETY, BASE SLIDING = 1.5 (1.1 Seismic)
MINIMUM FACTOR OF SAFETY, OVERTURNING = 2.0 (1.1 Seismic)

Typical Gravity Block Retaining Walls
Section and Notes

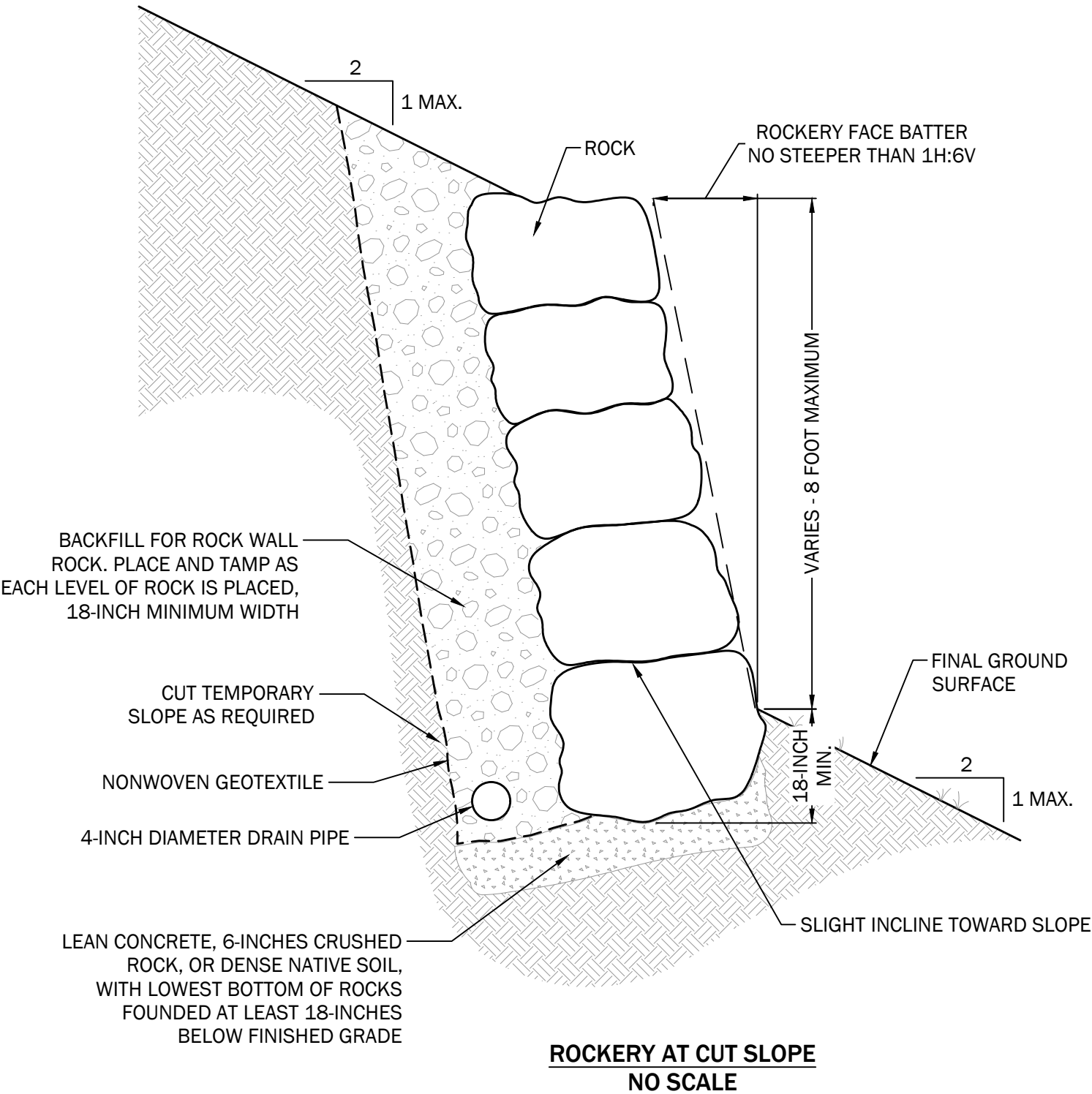
Wind Rose Development
King County, Washington



Figure 7

ROCKERY CONSTRUCTION SPECIFICATIONS

- 1. Rockeries shall be constructed in accordance with WSDOT Standard Specifications. Rock sizes and durability shall comply with WSDOT Standard Specifications, Sections 8-24 and 9.13.7(2). The individual constructing the rockery should be an experienced and skillful craftsman in rockery construction.
- 2. Rock shall be sound unweathered ledge rock from an established source that has demonstrated that it produces suitable rock. The rock shall be free of fractures, clay seams and evidence of weathering.
- 3. The contractor shall use sufficient space so that he can select among a number of rocks for each space in the rockery to be filled. Rocks which have shapes which do not match the spaces offered by the previous course of rock shall be rejected. Rock shall be angular, tabular, or semi-rectangular shaped; any rocks of basically rounded form shall not be used.
- 4. Rock sizes must be arranged with the larger boulders used in the lower courses of the rockery as specified in the WSDOT Standard Specifications, Section 8-24.
- 5. The first course of rocks shall be placed on firm unyielding soil. There shall be full contact between the rock and soil which may require shaping of the ground surface or slamming or dropping the rocks into place so that the soil foundation conforms to the rock face bearing on it. As an alternative, it is satisfactory to use lean concrete in which to seat the first course of rocks or to use 3/4-inch minus crushed rock into which the foundation rocks are seated. The bottom of the first course of rock shall be a minimum of 18 inches below the lowest adjacent grade.
- 6. The rocks shall be placed so that there are no continuous joint planes in the vertical or lateral direction. Each rock shall bear solidly on two or more rocks below it and so there is no sign of instability such as "rocking" or "tipping" of individual boulders. The rocks shall fit so no open spaces or voids larger than 6 inches exist. Rocks shall be placed so that there is some bearing between flat rock faces, rather than on points. Horizontal or nearly horizontal joints shall slope downward into the material protected (away from the rockery face).
- 7. Spalls shall be used behind the rockery rocks to block spaces and, where necessary, to wedge between rocks and to lock them together. This should also serve to prevent washing of backfill material through the rockery.
- 8. Backfill between the rockery and the adjacent soil face shall meet the requirements of backfill for rock wall, WSDOT Standard Specifications, Section 9.13.7(2). The backfill zone shall be at least 18 inches wide and be filled and thoroughly tamped as each course of boulders is placed.
- 9. A nonwoven geotextile separator, such as Mirafi 140N or approved equivalent shall be placed between crushed rock fill and the native soil. This is intended to prevent piping of native sediment through the rockery.
- 10. A 4-inch diameter solid wall PVC perforated drainpipe shall be installed as shown and connected to a suitable storm water collection system. The drainpipe shall have a 0.25 percent minimum slope.
- 11. Rockery construction is an art and depends largely on the skill of the builder. Although rockeries offer some lateral restraint, it is largely indeterminate and they are not normally intended to provide significant lateral support. Even when the foundation and retained material are satisfactory and the rockery materials and construction are satisfactory, there is some risk of movement or failure.



Rockery at Cut Slope

Wind Rose Development
King County, Washington



Figure 8

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REINFORCEMENT SPACING TABLE

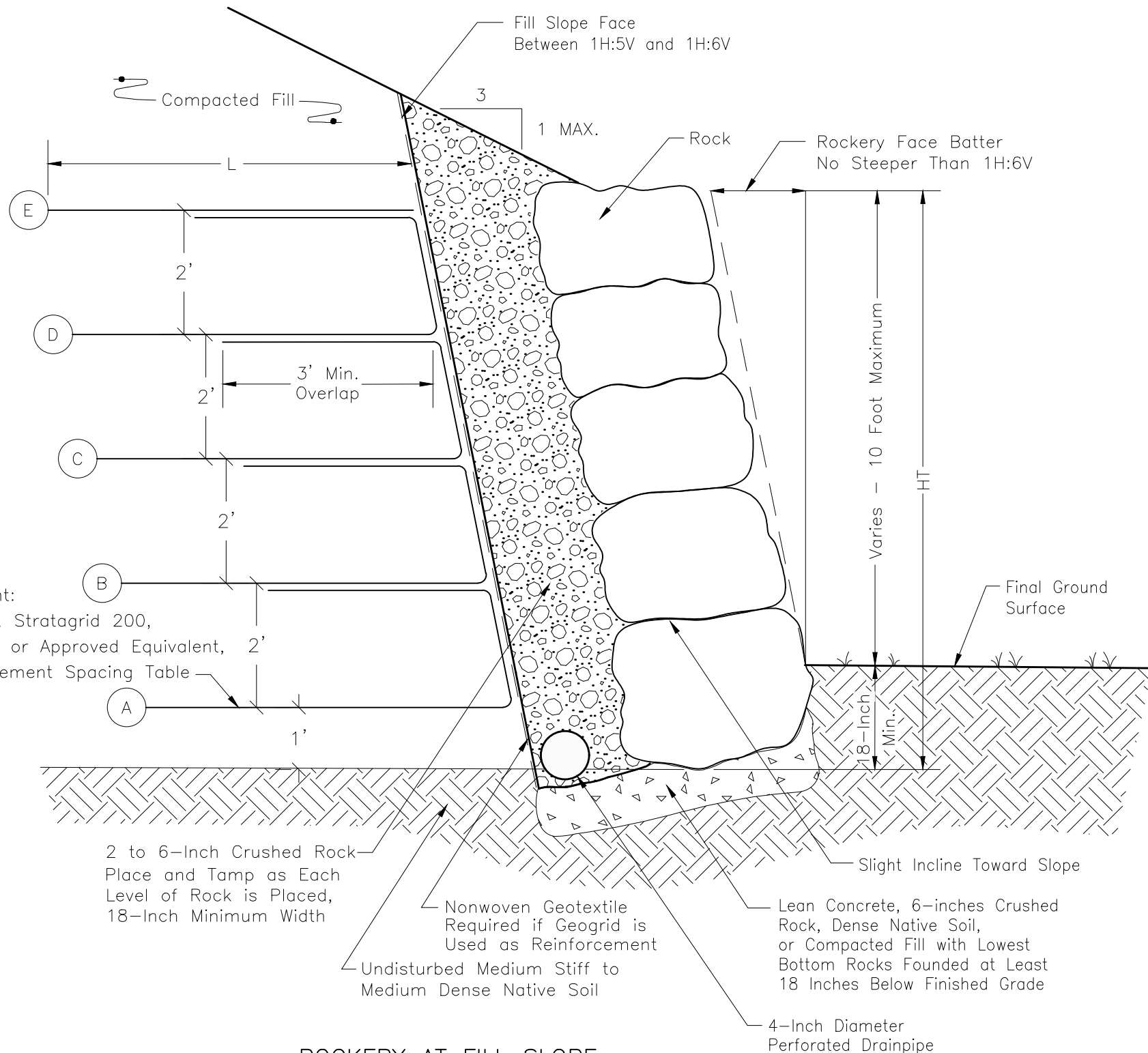
HT (FT)	L (FT)	Reinforcement Spacing
<4	—	None
4–5	5	(A), (B)
6–7	7	(A), (B), (C)
8–9	8	(A), (B), (C), (D)
>9	10	(A), (B), (C), (D), (E)

Required reinforcement length may be reduced for wall batters less than 1H:6V with the approval of the geotechnical engineer.

ROCKERY CONSTRUCTION SPECIFICATIONS

- See the Rockery Construction Specifications on Figure 1 for additional requirements for Rockeries.
- Fills faced with rockeries that are in excess of 4 feet in height are to be constructed with reinforcement. Reinforcement shall consist of Miragrid 3XT or Stratigard 200 geogrid reinforcement, Amoco 2044 geotextile reinforcement, or approved equivalent. Length and spacing of reinforcement to be installed according with the Reinforcement Spacing Table.
- Reinforcement shall be oriented with highest strength axis perpendicular to rockery alignment. Reinforcement shall be pulled taut and anchored at back before backfill placement on reinforcement. Reinforcement shall be continuous throughout their embedment and overlap length(s). Reinforcement panels shall be placed immediately adjacent to each other along the wall length.
- Backfill in reinforced zone shall consist of granular material (i.e. sand and gravel) containing less than 5 percent passing the U.S. No. 200 sieve, or as otherwise approved. The backfill shall be placed in loose lifts less than 12 inches thickness and be compacted to at least 92 percent of maximum dry density as determined by ASTM D1557. Backfill shall be placed and compacted so development of slack in reinforcement is minimized. Backfill outside the reinforced zone shall be compacted to at least 90 percent.
- On-site soils may be used as backfill in both the reinforced zone and the unreinforced zone provided it is properly moisture conditioned to achieve compaction, meets the gradation criteria described above, and is approved by the Geotechnical Engineer.

Reinforcement:
Miragrid 3XT, Stratagrid 200,
Amoco 2044 or Approved Equivalent, 2'
See Reinforcement Spacing Table



ROCKERY AT FILL SLOPE
NO SCALE

Rockery at Fill Slope

Wind Rose Development
King County, Washington



Figure 9

APPENDIX A

Field Explorations

APPENDIX A

FIELD EXPLORATIONS

Subsurface soil and groundwater conditions were evaluated at the site by excavating 15 test pits (TP-1 through TP-15). The test pits were completed on February 10, 2016 to depths ranging from 4 to 17 feet below the existing ground surface.

Test pit locations were estimated in the field by pacing from existing site features. The approximate locations of the test pits are shown on the Site Plan, Figure 2. Ground surface elevations shown on the test pit logs were estimated based on interpolation from contours on the base survey map shown on Figure 2. Test pit locations and elevations should be considered accurate only to the degree implied by the methods used.

The test pits were excavated using a track-mounted excavator provided by Kelly's Excavating, Inc. under subcontract to GeoEngineers. The test pits were continuously observed by a representative from our firm who located the test pits, classified the soils encountered, obtained representative soil samples, observed groundwater conditions, and maintained a detailed log of each test pit. At completion of test pit excavation, the test pits were backfilled in lifts approximately two feet thick and tamped in place with the excavator bucket, and the backfill should not be considered as structural fill for future structures.

The soils encountered in the test pits were visually classified in the field using the soil classification system described in Figure A-1. Figures A-2 through A-16 present the logs of the test pits. The logs reflect our interpretation of the field conditions and the results of laboratory evaluation and testing of samples. They also indicate the depths at which the soil types or their characteristics change, although the change may actually be gradual. If the change occurred between samples, it was interpreted.

Representative soil samples were obtained from the test pits using a shovel and directly from the backhoe bucket. Relative density or consistency of the soils encountered was estimated using a ½-inch-diameter steel hand probe rod and by observing digging action of the track-mounted excavator.

The soil samples we obtained were logged, sealed in plastic bags, and transported to our Redmond laboratory. The field classifications were further evaluated in our laboratory.

Observations of groundwater seepage conditions were made while completing the test pits. The groundwater conditions encountered during excavation are presented on the test pit logs. Groundwater conditions observed while completing the test pits represent a short term condition and may not be representative of the long term groundwater conditions at the site.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS	
			GRAPH	LETTER		
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES	
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES	
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES	
			GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES		
	MORE THAN 50% RETAINED ON NO. 200 SIEVE	SAND AND SANDY SOILS	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS
					SP	POORLY-GRADED SANDS, GRAVELLY SAND
MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES	
			SC	CLAYEY SANDS, SAND - CLAY MIXTURES		
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS, ROCK FLOUR, CLAYEY SILTS WITH SLIGHT PLASTICITY	
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS	
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY	
	MORE THAN 50% PASSING NO. 200 SIEVE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS SILTY SOILS
					CH	INORGANIC CLAYS OF HIGH PLASTICITY
					OH	ORGANIC CLAYS AND SILTS OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS	

NOTE: Multiple symbols are used to indicate borderline or dual soil classifications

Sampler Symbol Descriptions

	2.4-inch I.D. split barrel
	Standard Penetration Test (SPT)
	Shelby tube
	Piston
	Direct-Push
	Bulk or grab
	Continuous Coring

Blowcount is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted). See exploration log for hammer weight and drop.

A "P" indicates sampler pushed using the weight of the drill rig.

A "WOH" indicates sampler pushed using the weight of the hammer.

NOTE: The reader must refer to the discussion in the report text and the logs of explorations for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the specific exploration locations and at the time the explorations were made; they are not warranted to be representative of subsurface conditions at other locations or times.

ADDITIONAL MATERIAL SYMBOLS

SYMBOLS		TYPICAL DESCRIPTIONS
GRAPH	LETTER	
	AC	Asphalt Concrete
	CC	Cement Concrete
	CR	Crushed Rock/Quarry Spalls
	TS	Topsoil/Forest Duff/Sod

Groundwater Contact



Measured groundwater level in exploration, well, or piezometer



Measured free product in well or piezometer

Graphic Log Contact



Distinct contact between soil strata



Approximate contact between soil strata

Material Description Contact



Contact between geologic units



Contact between soil of the same geologic unit

Laboratory / Field Tests

%F	Percent fines
%G	Percent gravel
AL	Atterberg limits
CA	Chemical analysis
CP	Laboratory compaction test
CS	Consolidation test
DS	Direct shear
HA	Hydrometer analysis
MC	Moisture content
MD	Moisture content and dry density
OC	Organic content
PM	Permeability or hydraulic conductivity
PI	Plasticity index
PP	Pocket penetrometer
PPM	Parts per million
SA	Sieve analysis
TX	Triaxial compression
UC	Unconfined compression
VS	Vane shear

Sheen Classification

NS	No Visible Sheen
SS	Slight Sheen
MS	Moderate Sheen
HS	Heavy Sheen
NT	Not Tested

KEY TO EXPLORATION LOGS



FIGURE A-1

Date Excavated: 2/10/2016
 Equipment: Komatsu PC 130 Excavator

Logged By: DTM
 Total Depth (ft) 6

Elevation (feet)	Depth (feet)	SAMPLE		Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
		Testing Sample	Sample Name Testing						
395	1				TS		Approximately 6 inches topsoil (very loose, moist)		Probe depth = 12 to 14 inches
394	2	X	1 MC		SM		Brown silty fine to medium sand with occasional gravel, roots (medium dense, moist) (fill)	15	
393	3								
392	4	X	2 MC		SP-SM		Brown-gray fine to medium sand with silt and gravel (dense, moist) (outwash deposits)	27	Probe depth = 4 to 6 inches
391	5								
390	6	X	3				Minor groundwater seepage observed at 5½ feet		Probe depth < 2 inches
							Approximate ground surface elevation: 396 feet No caving observed		

Notes: See Figure A-1 for explanation of symbols.
 The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to 0.5 foot.

Log of Test Pit TP-1



Project: Wind Rose Development
 Project Location: King County, Washington
 Project Number: 1329-009-01

Figure A-2
 Sheet 1 of 1

Date Excavated: 2/10/2016
 Equipment: Kobelco 120C

Logged By: DTM
 Total Depth (ft) 9

Elevation (feet)	Depth (feet)	SAMPLE		Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
		Testing Sample	Sample Name Testing					
393	1			SM		Gray silty fine to medium sand, grass and roots (medium dense, moist) (topsoil)		Probe depth = 2 to 4 inches
				SM		Dark brown silty fine to medium sand, roots (loose, moist) (fill)		
392	2	X	1 MC				11	Probe depth = 6 to 12 inches
391	3			SM		Brown silty fine to medium sand with occasional gravel (loose, moist)		
390	4	X	2 SA				20	Probe depth = 4 to 8 inches %F = 29
389	5							
388	6			SM		Gray silty fine to medium sand with gravel (medium dense to dense, moist to wet) (glacial till)		
387	7							
386	8					Slow groundwater seepage observed 8 feet		
385	9	X	3 MC				12	

Approximate ground surface elevation: 394 feet
 Minor caving observed at 8 feet

Notes: See Figure A-1 for explanation of symbols.
 The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to 0.5 foot.

Log of Test Pit TP-2



Project: Wind Rose Development
 Project Location: King County, Washington
 Project Number: 1329-009-01

Figure A-3
 Sheet 1 of 1

Date Excavated: 2/10/2016
 Equipment: Komatsu PC 120 Excavator

Logged By: DTM
 Total Depth (ft) 5.5

Elevation (feet)	Depth (feet)	SAMPLE		Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
		Testing Sample	Sample Name Testing						
383	1				SM		Dark brown silty fine to medium sand, roots (loose, moist) (fill)		Probe depth = 8 to 10 inches
382	2	X	1 SA		SM		Brown silty fine to medium sand, occasional roots (loose, moist)	21	Probe depth = 6 to 12 inches %F = 29
381	3								
380	4	X	2 MC		SP-SM		Gray fine to medium sand with silt and gravel (dense, moist) (outwash deposits)	11	Probe depth = 1/2 to 2 inches
379	5								

Approximate ground surface elevation: 384 feet
 No groundwater seepage observed
 No caving observed

Notes: See Figure A-1 for explanation of symbols.
 The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to 0.5 foot.

Log of Test Pit TP-3



Project: Wind Rose Development
 Project Location: King County, Washington
 Project Number: 1329-009-01

Figure A-4
 Sheet 1 of 1

Date Excavated: 2/10/2016
 Equipment: Komatsu PC 120 Excavator

Logged By: DTM
 Total Depth (ft) 6

Elevation (feet)	Depth (feet)	SAMPLE		Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
		Testing Sample	Sample Name Testing						
375	1				SM		Dark brown silty fine to medium sand, roots (loose to medium dense, moist) (fill)		Probe depth < 5 inches
374	2	X	1 MC		SM		Gray/brown/orange silty fine to medium sand with gravel, roots (dense, moist)	19	Probe depth < 3 inches
373	3				GP-GM		Gray fine to coarse gravel with sand and silt (dense, moist) (outwash deposits)		
372	4	X	2 MC						
371	5								
370	6	X	3		SM		Brown silty fine to medium sand with gravel (dense, moist) (glacial till?)	10	Probe depth < 1 inch

Approximate ground surface elevation: 376 feet
 No groundwater seepage observed
 No caving observed

Notes: See Figure A-1 for explanation of symbols.
 The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to 0.5 foot.

Log of Test Pit TP-4



Project: Wind Rose Development
 Project Location: King County, Washington
 Project Number: 1329-009-01

Figure A-5
 Sheet 1 of 1

Date Excavated: 2/10/2016
 Equipment: Komatsu PC 120 Excavator

Logged By: DTM
 Total Depth (ft) 7.5

Elevation (feet)	Depth (feet)	SAMPLE		Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
		Testing Sample	Sample Name Testing					
375	1			SM		Gray silty fine to medium sand with gravel, roots (medium dense, moist) (fill)		Probe depth = 1 to 3 inches
374	2	X	1 SA	SM		Brown silty fine to medium sand with occasional gravel (loose to medium dense, moist)	20	Probe depth = 4 to 6 inches %F = 34
373	3			SM		Grayish, brownish orange silty fine to medium sand (medium dense, moist) (outwash deposits)		
372	4	X	2 MC			Transitions to gray (medium dense to dense)	19	Probe depth = 2 to 4 inches
371	5							
370	6							
369	7	X	3					

Approximate ground surface elevation: 376 feet
 No groundwater seepage observed
 No caving observed

Notes: See Figure A-1 for explanation of symbols.
 The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to 0.5 foot.

Log of Test Pit TP-5



Project: Wind Rose Development
 Project Location: King County, Washington
 Project Number: 1329-009-01

Figure A-6
 Sheet 1 of 1

Date Excavated: 2/10/2016
 Equipment: Komatsu PC 120 Excavator

Logged By: DTM
 Total Depth (ft) 4

Elevation (feet)	Depth (feet)	SAMPLE		Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
		Testing Sample	Sample Name Testing						
376	1				SM		Dark brown silty fine to medium sand, roots (loose, moist) (fill)		Probe depth = 4 to 6 inches
375	2	X	1 MC		SM		Brown-gray/orange silty fine to medium sand with occasional gravel (medium dense, moist) (outwash deposits)	13	Probe depth = 1 to 3 inches
374	3				SP-SM		Brown-gray silty fine to medium sand with occasional gravel (dense to very dense, moist)		
373	4	X	2 MC					16	Probe depth = ½ to 1 inch

Approximate ground surface elevation: 377 feet
 No groundwater seepage observed
 No caving observed

Notes: See Figure A-1 for explanation of symbols.
 The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to 0.5 foot.

Log of Test Pit TP-6



Project: Wind Rose Development
 Project Location: King County, Washington
 Project Number: 1329-009-01

Figure A-7
 Sheet 1 of 1

Date Excavated: 2/10/2016
 Equipment: Komatsu PC 120 Excavator

Logged By: DTM
 Total Depth (ft) 6.5

Elevation (feet)	Depth (feet)	SAMPLE		Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
		Testing Sample	Sample Name Testing						
375	1				SM		Dark brown silty fine to medium sand, roots (loose, moist) (fill)		Probe depth = 6 to 8 inches
374	2	X	1 MC		SM		Light brown silty fine to medium sand with occasional gravel, roots (loose, moist)	15	Probe depth = 8 to 10 inches
373	3								
372	4	X	2 MC		SM		Gray silty fine to medium sand with occasional gravel (medium dense, moist)	20	Probe depth = 4 to 6 inches
371	5								
370	6	X	3 MC		SP-SM		Gray silty fine to medium sand with silt (medium dense to dense, moist) (outwash deposits)	11	

Approximate ground surface elevation: 376 feet
 No groundwater seepage observed
 Minor caving observed at 3½ feet

Notes: See Figure A-1 for explanation of symbols.
 The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to 0.5 foot.

Log of Test Pit TP-7



Project: Wind Rose Development
 Project Location: King County, Washington
 Project Number: 1329-009-01

Figure A-8
 Sheet 1 of 1

Date Excavated: 2/10/2016
 Equipment: Komatsu PC 120 Excavator

Logged By: DTM
 Total Depth (ft) 17

Elevation (feet)	Depth (feet)	SAMPLE		Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
		Testing Sample	Sample Name Testing						
376	1				SM		Gray silty fine to medium sand, roots (loose to medium dense, moist) (fill)		Probe depth = 3 to 5 inches
					SM		Gray-brown/orange silty fine to medium sand (loose to medium dense, moist)		
375	2	1 MC			SM		Brown silty fine to medium sand (dense, moist) (outwash deposits)	15	Probe depth = 1 to 2 inches
374	3								
373	4	2 SA			SM		Gray silty fine sand (dense, moist)	16	Probe depth = 1/2 to 1 inch %F = 47
372	5								
371	6								
370	7								
369	8								
368	9								
367	10								
366	11								
365	12								
364	13	3 SA			SP-SM		Gray fine sand with silt (dense, wet) Moderate groundwater seepage observed from 13 to 17 feet	19	%F = 10
363	14								
362	15						Becomes wet		
361	16								
360	17	4 MC						30	
Approximate ground surface elevation: 377 feet Minor caving observed from 12 to 17 feet									

Notes: See Figure A-1 for explanation of symbols.
 The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to 0.5 foot.

Log of Test Pit TP-8



Project: Wind Rose Development
 Project Location: King County, Washington
 Project Number: 1329-009-01

Figure A-9
 Sheet 1 of 1

Date Excavated: 2/10/2016
 Equipment: Komatsu PC 130 Excavator

Logged By: DTM
 Total Depth (ft) 4

Elevation (feet)	Depth (feet)	SAMPLE		Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
		Testing Sample	Sample Name Testing						
381	1				TS		Topsoil (very loose, moist)		Probe depth = 10 to 14 inches
380	2	X	1 MC		SM		Dark brown silty fine to medium sand, roots (loose, moist) (fill)	17	Probe depth = 6 to 12 inches
379	3				SM		Gray silty fine to medium sand (dense, moist) (outwash deposits)		
378	4	X	2 SA					14	Probe depth < 2 inches %F = 40

Approximate ground surface elevation: 382 feet
 No groundwater seepage observed
 No caving observed

Notes: See Figure A-1 for explanation of symbols.
 The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to 0.5 foot.

Log of Test Pit TP-9



Project: Wind Rose Development
 Project Location: King County, Washington
 Project Number: 1329-009-01

Figure A-10
 Sheet 1 of 1

Date Excavated: 2/10/2016

Logged By: DTM

Equipment: Komatsu PC 130 Excavator

Total Depth (ft) 4

Elevation (feet)	Depth (feet)	SAMPLE		Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
		Testing Sample	Sample Name Testing						
367	1				SM		Dark brown silty fine to medium sand, roots (loose, moist) (fill)		Probe depth = 8 to 10 inches
366	2	X	1 MC		SM		Gray silty fine to medium sand (medium dense, moist) (outwash deposits)	13	Probe depth = 2 to 4 inches
365	3				SP-SM		Gray fine to medium sand with silt (dense, moist)		
364	4	X	2 MC				Minor groundwater seepage observed at 3½ feet	10	Probe depth < 2 inches

Approximate ground surface elevation: 368 feet
No caving observed

Notes: See Figure A-1 for explanation of symbols.

The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to 0.5 foot.

Log of Test Pit TP-10



Project: Wind Rose Development
Project Location: King County, Washington
Project Number: 1329-009-01

Figure A-11
Sheet 1 of 1

Date Excavated: 2/10/2016

Logged By: DTM

Equipment: Komatsu PC 130 Excavator

Total Depth (ft) 6

Elevation (feet)	Depth (feet)	SAMPLE		Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
		Testing Sample	Sample Name Testing						
366	1				SM		Dark brown silty fine to medium sand (loose, moist) (fill)		Probe depth = 4 to 8 inches
					SM		Grayish brown silty fine to medium (medium dense, moist)		
365	2	X	1 MC		SM		Gray silty fine to medium sand (medium dense, moist) (outwash deposits)	15	Probe depth = 2 to 5 inches
364	3								
363	4	X	2 SA		SP-SM		Gray fine to medium sand with silt (dense, moist)	13	Probe depth < 3 inches %F = 9
362	5								
361	6								

Approximate ground surface elevation: 367 feet
No groundwater seepage observed
No caving observed

Notes: See Figure A-1 for explanation of symbols.

The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to 0.5 foot.

Log of Test Pit TP-11



Project: Wind Rose Development
Project Location: King County, Washington
Project Number: 1329-009-01

Figure A-12
Sheet 1 of 1

Date Excavated: 2/10/2016
 Equipment: Komatsu PC 120 Excavator

Logged By: DTM
 Total Depth (ft) 10

Elevation (feet)	Depth (feet)	SAMPLE		Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
		Testing Sample	Sample Name Testing						
361	1				SM		Dark brown silty fine to medium sand (loose, moist) (fill)		Probe depth = 12 to 16 inches
360	2	X	1 MC		SM		Grayish orange/brown silty fine to medium sand with occasional gravel (medium dense, moist)	16	Probe depth = 4 to 8 inches
359	3				SM		Gray silty fine to medium sand (medium dense to dense, moist) (outwash deposits)		
358	4	X	2 SA					14	Probe depth = 1 to 3 inches %F = 30
357	5								
356	6								
355	7								
354	8								
353	9								
352	10	X	3						
Approximate ground surface elevation: 362 feet No groundwater seepage observed No caving observed									
Notes: See Figure A-1 for explanation of symbols. The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to 0.5 foot.									

Log of Test Pit TP-12



Project: Wind Rose Development
 Project Location: King County, Washington
 Project Number: 1329-009-01

Figure A-13
 Sheet 1 of 1

Date Excavated: 2/10/2016
 Equipment: Komatsu PC 120 Excavator

Logged By: DTM
 Total Depth (ft) 11

Elevation (feet)	Depth (feet)	SAMPLE		Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
		Testing Sample	Sample Name Testing						
367	1				SM		Dark brown silty fine to medium sand, roots (loose, moist) (fill)		Probe depth = 10 to 16 inches
366	2	X	1 SA		SM		Orangish gray silty fine to medium sand, roots (loose, moist)	14	Probe depth = 8 to 12 inches %F = 26
365	3				SP-SM		Gray fine to medium sand with silt (medium dense, moist) (outwash deposits)		
364	4	X	2 MC					9	Probe depth = 3 to 6 inches
363	5								
362	6								
361	7								
360	8						Becomes dense		
359	9								
358	10								
357	11	X	3						

Approximate ground surface elevation: 368 feet
 No groundwater seepage observed
 No caving observed

Notes: See Figure A-1 for explanation of symbols.
 The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to 0.5 foot.

Log of Test Pit TP-13



Project: Wind Rose Development
 Project Location: King County, Washington
 Project Number: 1329-009-01

Figure A-14
 Sheet 1 of 1

Date Excavated: 2/10/2016

Logged By: DTM

Equipment: Komatsu PC 120 Excavator

Total Depth (ft) 4

Elevation (feet)	Depth (feet)	SAMPLE		Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
		Testing Sample	Sample Name Testing						
349	1				TS		Dark brown topsoil with roots (loose, moist)		
348	2	X	1 MC		SP-SM		Orange fine to medium sand with silt, roots (medium dense, moist) (outwash deposits)	8	
347	3				SP-SM		Gray silty fine to medium sand with silt (dense, moist)		
346	4	X	2 MC					11	
Approximate ground surface elevation: 350 feet No groundwater seepage observed No caving observed									

Notes: See Figure A-1 for explanation of symbols.

The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to 0.5 foot.

Log of Test Pit TP-14



Project: Wind Rose Development
 Project Location: King County, Washington
 Project Number: 1329-009-01

Figure A-15
 Sheet 1 of 1

Date Excavated: 2/10/2016

Logged By: DTM

Equipment: Komatsu PC 120 Excavator

Total Depth (ft) 4

Elevation (feet)	Depth (feet)	SAMPLE		Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
		Testing Sample	Sample Name Testing						
377	1				SM		Dark brown silty fine to medium sand (loose, moist) (topsoil)		
376	2				SM		Grayish orange silty fine to medium sand (medium dense, moist) (outwash deposits)		
375	3				SM		Gray silty fine to medium sand (dense, moist)		
374	4						Approximate ground surface elevation: 378 feet No groundwater seepage observed No caving observed		

Notes: See Figure A-1 for explanation of symbols.

The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to 0.5 foot.

Log of Test Pit TP-15



Project: Wind Rose Development
 Project Location: King County, Washington
 Project Number: 1329-009-01

Figure A-16
 Sheet 1 of 1

APPENDIX B

Laboratory Testing

APPENDIX B

LABORATORY TESTING

Soil samples obtained from the test pits were transported to our Redmond geotechnical laboratory and evaluated to confirm or modify field classifications, as well as to evaluate engineering properties of the soils. Representative samples were selected for laboratory testing that included moisture content tests and sieve analyses. The tests were conducted using test methods of the American Society for Testing and Materials (ASTM) or other applicable procedures.

Soil Classifications

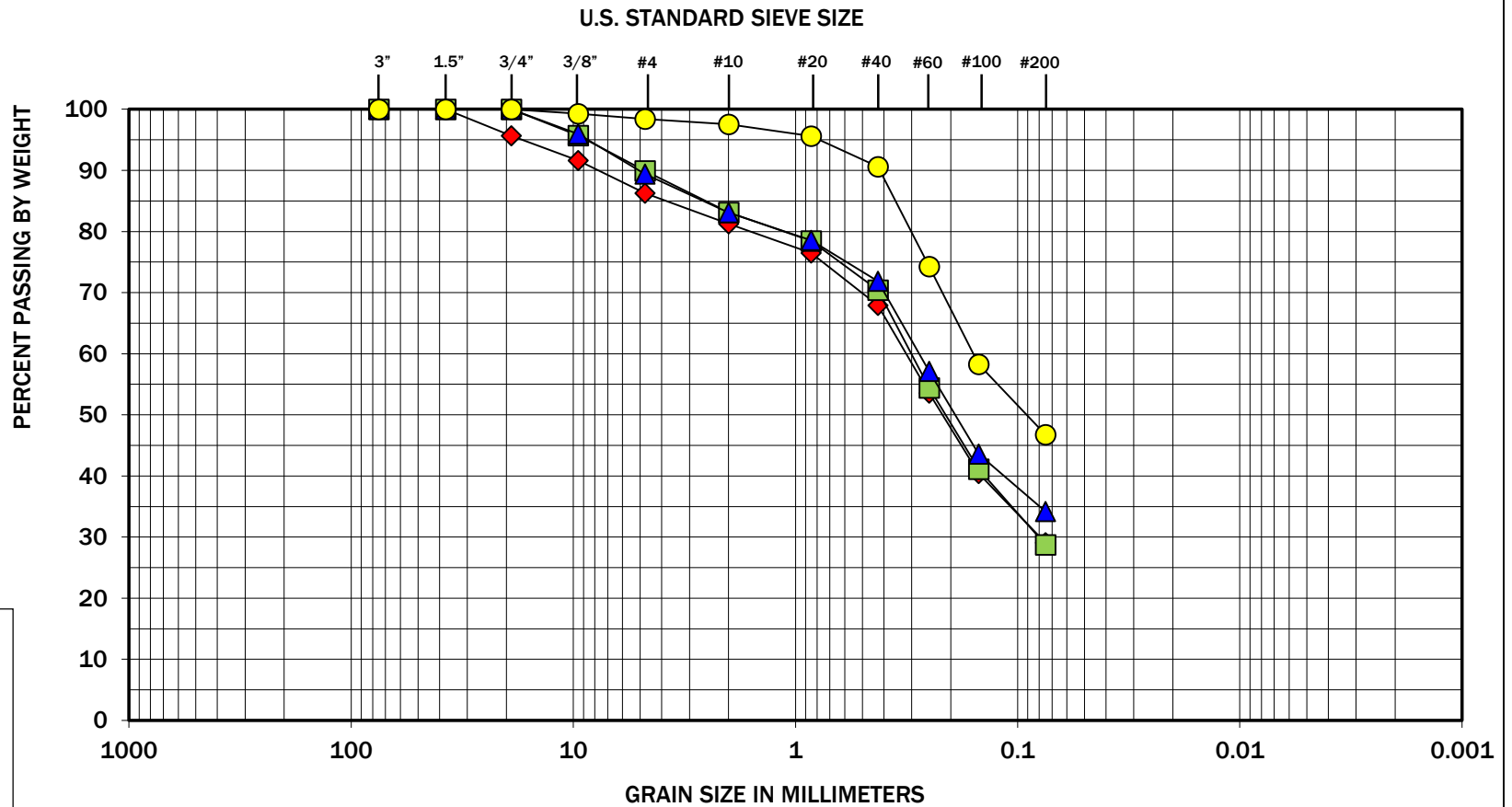
All soil samples obtained from the test pits were visually classified in the field and/or in our laboratory using a system based on the Unified Soil Classification System (USCS) and ASTM classification methods. ASTM test method D 2488 was used to visually classify the soil samples, while ASTM D 2487 was used to classify the soils based on laboratory test results. These classification procedures are incorporated in the test pit logs presented as Figures A-2 through A-16 in Appendix A.

Moisture Content Tests

Moisture content tests were completed using the ASTM D 2216 test method for representative samples obtained from the test pits. The results of these tests are presented on the test pit logs (Appendix A) at the respective sample depths.

Sieve Analyses

Sieve analyses were conducted on nine samples obtained from the test pits. The analyses were conducted using the ASTM D 6913 test method. The wet sieve method was used to estimate the percentage of soil particles retained on the U.S. No. 200 mesh sieve. The results of the sieve analyses were plotted, classified using the USCS, and presented on Figures B-1 through B-3.

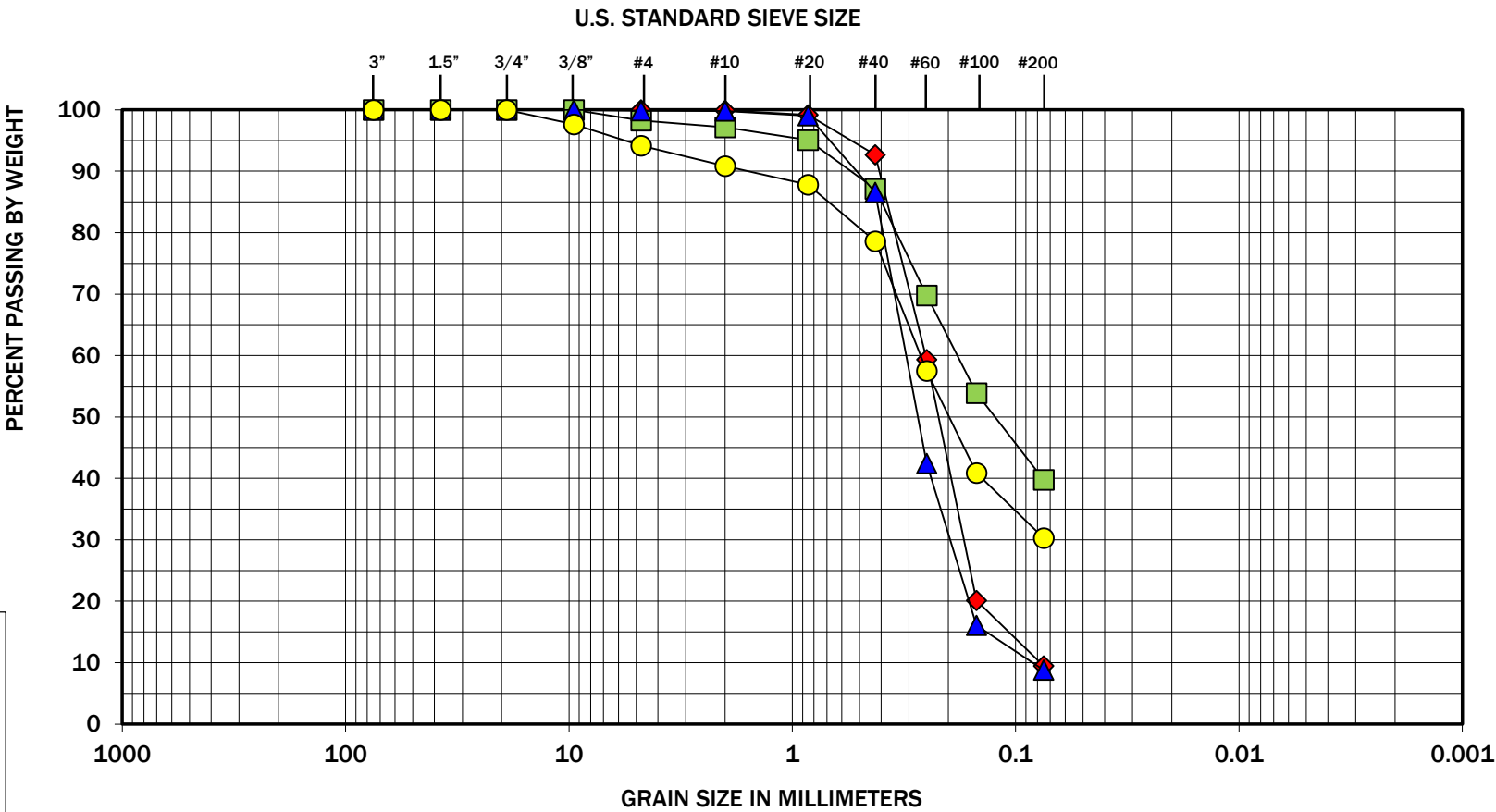


COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Symbol	Test Pit Number	Depth (feet)	Moisture (%)	Soil Description
◆	TP-2	4	20	Silty sand (SM)
■	TP-3	2	21	Silty sand (SM)
▲	TP-5	2	20	Silty sand (SM)
●	TP-8	4	16	Silty sand (SM)

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The grain size analysis results were obtained in general accordance with ASTM D 6913.



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Symbol	Test Pit Number	Depth (feet)	Moisture (%)	Soil Description
◆	TP-8	13	19	Poorly graded sand with silt (SP-SM)
■	TP-9	3.5	14	Silty sand (SM)
▲	TP-11	4	13	Poorly graded sand with silt (SP-SM)
●	TP-12	4	14	Silty sand (SM)

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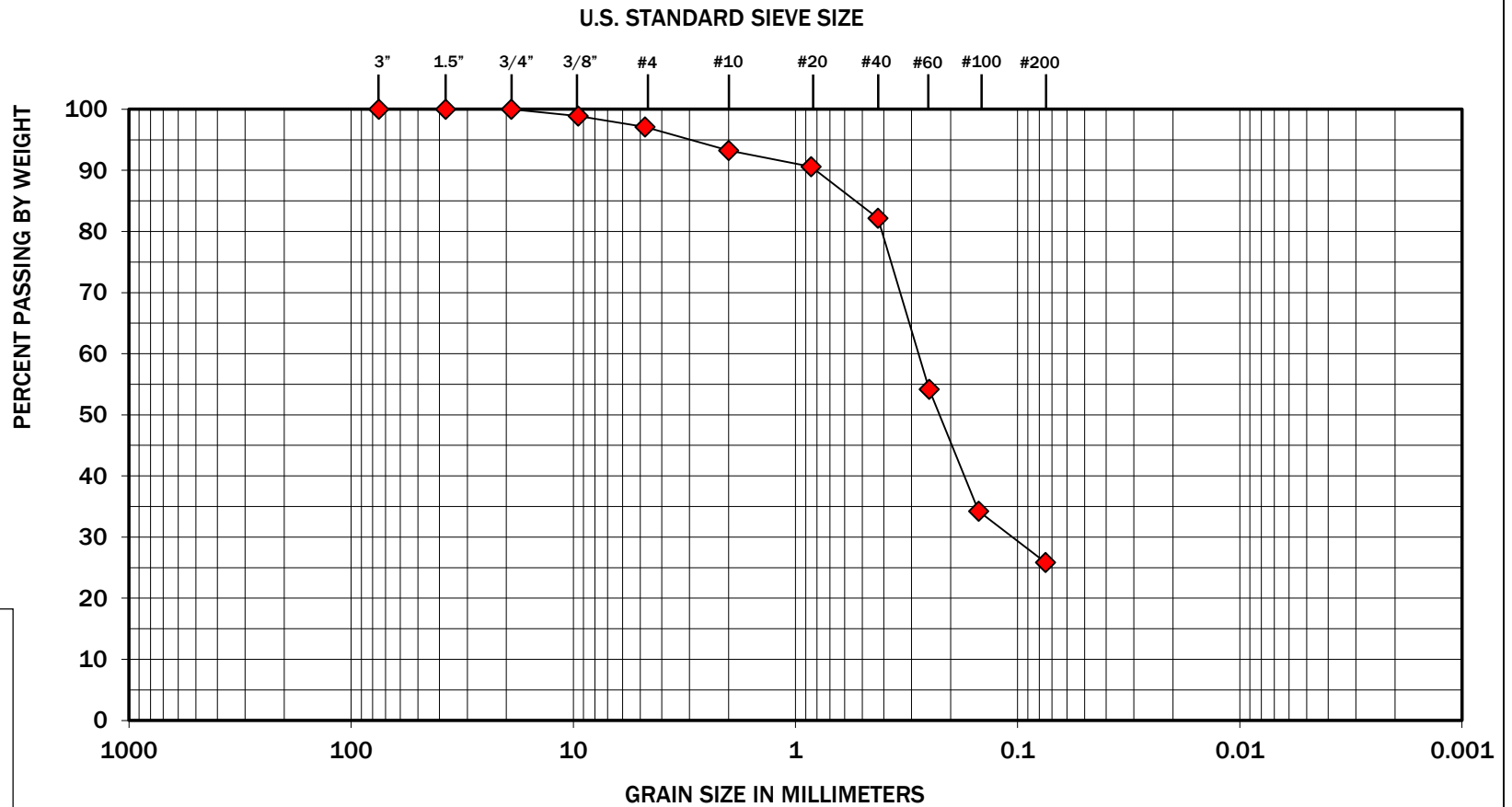
The grain size analysis results were obtained in general accordance with ASTM D 6913.



Wind Rose Development
King County, Washington

Sieve Analysis Results

Figure B-2



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Symbol	Test Pit Number	Depth (feet)	Moisture (%)	Soil Description
◆	TP-13	2	14	Silty sand (SM)

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The grain size analysis results were obtained in general accordance with ASTM D 6913.



APPENDIX C

Previous Explorations

APPENDIX C

PREVIOUS EXPLORATIONS

Appendix C presents the logs of boring B-1 drilled in 2002 and boring B-4 completed (using the direct push method) in 2007 for our previous studies in the site vicinity. Boring B-1 was drilled in the eastern part of the Wind Rose site, while boring B-4 was drilled in the storm water pond area.

The approximate locations of the previous explorations are shown on the Site Plan, Figure 2.

Date(s) Drilled	12/04/02	Logged By	JG2	Checked By	CFE
Drilling Contractor	Gregory Drilling	Drilling Method	Hollow Stem Auger	Sampling Methods	Dames & Moore
Auger Data	5.25-inch HSA	Hammer Data	300 (lb) hammer/ 30 (in) drop	Drilling Equipment	CME 85 Truck-mounted Rig
Total Depth (ft)	76.5	Surface Elevation (ft)	375.9	Groundwater Level (ft. bgs)	None observed
Datum/ System					

Elevation feet	Depth feet	SAMPLES				Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, lbs/ft ³	OTHER TESTS AND NOTES
		Interval	Number	Recovered (in)	Blows/foot							
375	0							SM	Brown silty fine sand (loose, moist) (fill?)			
370	5	1	12	39				SP-SM	Brown fine sand with silt (dense, moist) Becomes dense	7	106	SA
365	10	2	16	51					Becomes very dense	9	107	
360	15	3	14	43					Becomes fine to medium sand with occasional coarse sand (dense)			
355	20	4	16	37								
350	25	5	16	64					Becomes very dense	15	106	
345	30	6	16	36					Becomes fine grained, dense			
35	35							SP	Gray brown fine sand (very dense, moist)			

Note: See Figure A-2 for explanation of symbols

LOG OF BORING B-1



Project: Park Lake Homes Redevelopment
Project Location: Seattle, Washington
Project Number: 1329-003-00

Figure: A-3
Sheet 1 of 2

1329-003-00 GEI GTBORING 2.1.0 P:\111329003\00\FINALS\1329003.GPJ GEIV2 2.GDT 8/27/03

Elevation feet	Depth feet	SAMPLES				Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, lbs/ft ³	OTHER TESTS AND NOTES
		Interval	Number	Recovered (in)	Blows/foot							
340	35	7	16	79					Becomes very dense			
335	40	8	14	50/5"						6	106	
330	45	9	18	50/5.5"								
325	50	10	10	50/5"								
320	55	11	12	50/3"								
								SW	Brown fine to coarse sand and fine gravel (very dense, moist)			
								SP	Brown fine sand (very dense, moist)			
315	60	12	14	50/4"								
310	65	13	11	50/5"					Occasional fine to coarse gravel	8	104	
305	70	14	15	50/5"								
300	75	15	12	50/3"						4	106	

LOG OF BORING B-1 (continued)



Project: Park Lake Homes Redevelopment
 Project Location: Seattle, Washington
 Project Number: 1329-003-00

Figure: A-3
 Sheet 2 of 2

Date(s) Drilled	05/07/07	Logged By	MSL	Checked By	JAR
Drilling Contractor	ESN	Drilling Method	Direct Push	Sampling Methods	Sleeved Sample
Auger Data		Hammer Data		Drilling Equipment	Geoprobe/Stratoprobe
Total Depth (ft)	16	Surface Elevation (ft)		Groundwater Level (ft. bgs)	13
Vertical Datum		Datum/ System		Easting(x): Northing(y):	

Depth feet	SAMPLES					Group Symbol	MATERIAL DESCRIPTION	Sheen	Headspace Vapor PID (ppm)	NOTES
	Interval	Recovered (in)	Blows/foot	Sub-Sample Number	Testing					
0						SM	Brown to dark brown silty fine sand with occasional gravel and bricks (moist) (fill)			
					CA			SS	2.2	
						ML	Brown silt with fine sand (moist)	SS	1.8	
5							becomes mottled brown to gray becomes wet	NS	1.9	
						SW	Brown well graded sand with fine gravel and trace silt (moist)	SS	2.9	
						SP-SM	Brown fine to medium sand with silt (moist)			
10						ML	Brown sandy silt (moist)	NS	2.4	
						SM	Brown silty fine to medium sand (moist)	NS	1.7	
					CA			NS	1.6	
								NS	1.6	
15								NS	1.4	
20										

Note: See Figure A-1 for explanation of symbols.

LOG OF BORING B-4



Project: 301 SW Roxbury
Project Location: Seattle, Washington
Project Number: 1329-003-09

Figure A-5
Sheet 1 of 1

APPENDIX D

**June 16, 2015 Letter on Retaining Wall Design and
Gravity Block Wall Calculations for 2H:1V Backslope**



8410 154th Avenue NE
Redmond, Washington 98052
425.861.6000

June 16, 2015

King County Housing Authority
600 Andover Park West
Seattle, Washington 98188

Attention: John Eliason

Subject: Additional Geotechnical Consulting Services
Retaining Wall Design
Wind Rose Development
King County, Washington
File No. 1329-009-01, Task 100

This letter (Addendum No. 1) summarizes our preliminary geotechnical design recommendations and calculations for retaining walls in the southeast part of the King County Housing Authority (KCHA) Wind Rose Development project site. The project site is located southeast of the intersection of 4th Avenue SW and SW Roxbury Street in the White Center area of unincorporated King County, Washington.

GeoEngineers previously completed geologic and geotechnical engineering services for the overall Greenbridge Redevelopment project and the Wind Rose Development “Notch” site. Those services are summarized in the following reports:

- “Geotechnical Engineering Services, Wind Rose Development, King County, Washington,” dated February 29, 2012.
- “Update Report, Geotechnical Engineering Services, Greenbridge Hope IV Redevelopment Project, King County, Washington,” dated January 12, 2007.

The project site comprises approximately 6 acres and adjoins the south, southeast and east sides of a property known as the “Notch Property,” which will also be part of the Wind Rose Development. SW 97th Place extends through the subject site.

We understand that the southeast part of the Wind Rose Development will include single family residences on 32 lots flanking both sides of SW 97 Place. A retaining wall will be constructed along the southeast side of Lots 15 through 24, which are located along the top of a slope that extends down to a ravine. We understand this wall will be up to 18 feet high and will be a vegetated face, geosynthetic reinforced (mechanically stabilized earth, MSE) wall. The wall will be embedded a sufficient distance below the finished downslope surface, which will be inclined no steeper than 3H:1V (horizontal to vertical). The wall will have a level back slope.



A storm water detention pond will be constructed in the center of the site and along the northwest side of SW 97th Place. We understand that gravity block retaining walls with heights up to 15 feet are planned along the southeast, west and north sides of the pond. These walls will have a 3H:1V fore slope down to the bottom of the pond. The back slope above the wall segments will vary from level to 2H:1V.

We anticipate the wall backfill will consist of (1) imported Gravel Borrow conforming to Section 9-03.14(1) of the 2014 Washington State Department of Transportation (WSDOT) Standard Specifications, or (2) suitable granular on-site soil originating from the excavation for the proposed detention pond and other areas of the site. Native soils excavated from the detention pond may be reused in the retained soil zone for the vegetated retaining wall southeast of Lots 15 through 24, but should not be used as wall backfill for the concrete gravity walls around the pond.

SUBSURFACE CONDITIONS

Based on previous explorations we completed at the Wind Rose site and vicinity, the subsurface conditions generally consist of granular fill soils with varying thicknesses overlying ice contact and recessional outwash deposits.

The fill varies in thickness from about 3 to 8 feet in the site vicinity and consists of loose to medium dense sand with varying amounts of silt and gravel. The fill contains trace organic matter and asphalt debris.

The ice contact and recessional outwash soils were observed below the fill and consist of medium dense to very dense sand with variable amounts of silt and gravel, and with occasional interbeds of very stiff to hard silt and clay. These deposits extend to the maximum depth explored at the site.

Perched groundwater was encountered within the fill and ice contact deposits during our previous explorations. These shallow zones of groundwater were observed at depths ranging from about 10 to 30 feet beneath the existing ground surface, typically in lenses of sand or silty sand that are underlain by low permeability soils. Lateral movement of groundwater within these shallow perched zones may occur in topographically downslope directions, but is generally expected to be limited in volume because of the isolated occurrence of these zones. We anticipate that perched groundwater may exist at various depths in response to seasonal changes in precipitation.

PRELIMINARY WALL DESIGN RECOMMENDATIONS AND CALCULATIONS

An overview of wall analyses methods, preliminary design recommendations, construction considerations, wall performance, and other factors for the two walls is provided in the following sections. Preliminary typical sections for these walls are presented in Figures 1 and 2.

Analysis Methods

Engineering analysis to develop the preliminary typical section for the MSE wall (southeast of Lots 15-24) was completed using the commercial computer program MSEW (Version 3.3, 2014) developed by ADAMA Engineering, Inc. The program includes calculations of internal stability. The typical section for the gravity block wall (between the detention pond and the road) was completed using the commercial

computer program UltraWall (Version 3.3, 2014) by Ultrablock, Inc. External global stability of both the MSE wall and the gravity block wall under static and seismic conditions was evaluated using the commercial computer program Slope/W (Version 8.0, 2012) developed by GeoSlope International.

Subgrade Preparation

Prior to final design, we recommend that a series of subsurface explorations be completed along both wall areas to confirm that the subgrade soils will be suitable for supporting the walls with adequate factors of safety and to confirm the input parameters used in our wall stability calculations.

Prior to placing the fill for the MSE wall or the blocks for the gravity wall, the wall subgrade areas should be prepared as described in the “Subgrade Preparation” section included in our February 29, 2012 report. This is particularly important for the MSE wall along Lots 15-24, where thick deposits of organic materials and loose soils are anticipated on the slope along the wall alignment. If soft, loose, organic or otherwise unsuitable soils are encountered at the wall subgrade level, we recommend these soils be evaluated by GeoEngineers and be removed and replaced with suitable structural fill down to the depth identified by our representative. The structural fill zone should extend horizontally beyond the edge of the wall facing units or gravity blocks (front and back) by the depth of the excavation. The structural fill should be compacted to at least 95 percent of the maximum dry density (MDD) obtained using the ASTM D 1557 test method.

Soil Properties

MSE Wall (Lots 15 to 24)

We recommend the parameters summarized in Table 1 be used for design of the proposed MSE wall along Lots 15 to 24. The soil parameters recommended in Table 1 are based on previous explorations completed at the Wind Rose site and our experience in the area.

The design values shown in Table 1 assume the backfill soils in the reinforced zone are compacted to at least 95 percent of the MDD obtained using ASTM D 1557. Wall backfill within the reinforced zone should consist of imported Gravel Borrow.

TABLE 1. RECOMMENDED MSE WALL DESIGN PARAMETERS

Soil Properties	Backfill Soil (Reinforced Zone)	Retained Soil (Proposed Fill)	Foundation Bearing Soil
Unit Weight (pcf)	125	125	125
Friction Angle (deg)	34	34	36
Cohesion (psf)	0	0	0
Active Earth Pressure Coefficient, K_a	-	0.28	-

Notes:

pcf – pounds per cubic foot

psf – pounds per square foot

For purposes of preliminary internal wall design, we recommend that the groundwater level be assumed below the base of the wall, provided that the wall backfill consists of gravel borrow or other free-draining soil.

Gravity Block Wall along Southeast Side of Detention Pond

We recommend the design parameters summarized in Table 2 be used for design of the proposed gravity block wall along the southeast side of the detention pond and adjacent to SW 97th Place. The soil strength parameters reflect the fact that the base of the wall will be within dense native granular soils.

The design values shown in Table 2 assume the backfill soils in the reinforced zone are compacted to at least 95 percent of the MDD obtained using ASTM D 1557. Wall backfill within the reinforced zone should consist of imported Gravel Borrow.

TABLE 2. RECOMMENDED GRAVITY BLOCK WALL DESIGN PARAMETERS

Soil Properties	Backfill Soil (Reinforced Zone)	Retained Soil (Structural Fill)	Foundation Bearing Soil
Unit Weight (pcf)	125	125	125
Friction Angle (deg)	36	36	38
Cohesion (psf)	0	0	0
Active Earth Pressure Coefficient, K_a	-	0.26	-

For purposes of preliminary internal wall design, we recommend that the groundwater level be assumed at the design high water surface within the pond (Elevation 372 feet).

Earthquake Loads

Based on the 2008 United States Geologic Survey (USGS) probabilistic seismic hazard maps, the peak ground acceleration (PGA) expected at the site from an earthquake with a 2 percent probability of exceedance in 50 years is approximately 0.41g. We recommend the internal stability of the wall be analyzed using a horizontal seismic coefficient of 0.21g. External global stability may utilize half of the expected PGA for pseudo-static seismic analysis.

A summary of the derivation of the PGA from the USGS hazard maps, is attached.

Performance Limit Values

We recommend the performance limit values presented in Table 3 be used as minimum safety factors to design the MSE and gravity block walls.

TABLE 3. RECOMMENDED PERFORMANCE LIMIT VALUES

Criteria	Minimum Static Safety Factor	Minimum Seismic Safety Factor
Geogrid Strength, Geogrid Connection, Pullout, and Sliding	1.5	1.1
Overturning Stability	2	1.5
Bearing Capacity	2	1.5
Global Stability	1.5	1.1



Surcharge Loading

We have assumed that no significant surcharge loads will be applied behind the walls, other than traffic loading. GeoEngineers should be consulted to evaluate surcharge loading conditions so that modifications can be made to our recommendations, as appropriate.

Preliminary Design

GeoEngineers completed preliminary design of the MSE and gravity block walls to develop our recommended typical wall sections (Figures 1 and 2).

Calculations supporting the preliminary design of the MSE and gravity block walls are attached. Results of static and seismic external global stability analyses are attached.

LIMITATIONS

We have prepared this letter for use by KCHA and members of the design team for use in preliminary design of retaining walls for the southeast part of the proposed Wind Rose Development in the White Center area of King County, Washington. This letter is submitted as an addendum to our report dated February 29, 2012.

Our services were provided to assist in the design of two retaining walls for the planned development: (1) a gravity block wall along the southeast side of the detention pond that fronts the road, and (2) an MSE wall along the southeast side of Lots 15-24. Our recommendations are intended to improve the overall stability of the site and to reduce the potential for future property damage related to earth movements, drainage or erosion. Qualified engineering and construction practices can help mitigate the risks inherent in construction on slopes, although those risks cannot be eliminated completely. Favorable performance of structures in the near term is useful information for anticipating future performance, but it cannot predict or imply a certainty of long-term performance, especially under conditions of adverse weather or seismic activity.

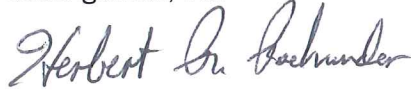
Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering in this area at the time this report was prepared. No warranty or other conditions, express or implied, should be understood.

Any electronic form, facsimile or hard copy of the original document (email, text, table, and/or figure), if provided, and any attachments are only a copy of the original document. The original document is stored by GeoEngineers, Inc. and will serve as the official document of record.



Please refer to Appendix E in our February 29, 2012 geotechnical report for additional information pertaining to use of this report.

Sincerely,
GeoEngineers, Inc.



Herbert R. Pschunder, PE
Senior Geotechnical Engineer



Robert C. Metcalfe, PE, LEG
Principal

HRP:RCM:nld



Attachments:

Figure 1. MSE Wall Section

Figure 2. Gravity Block Retaining Wall Section and Notes

Seismic Design Input

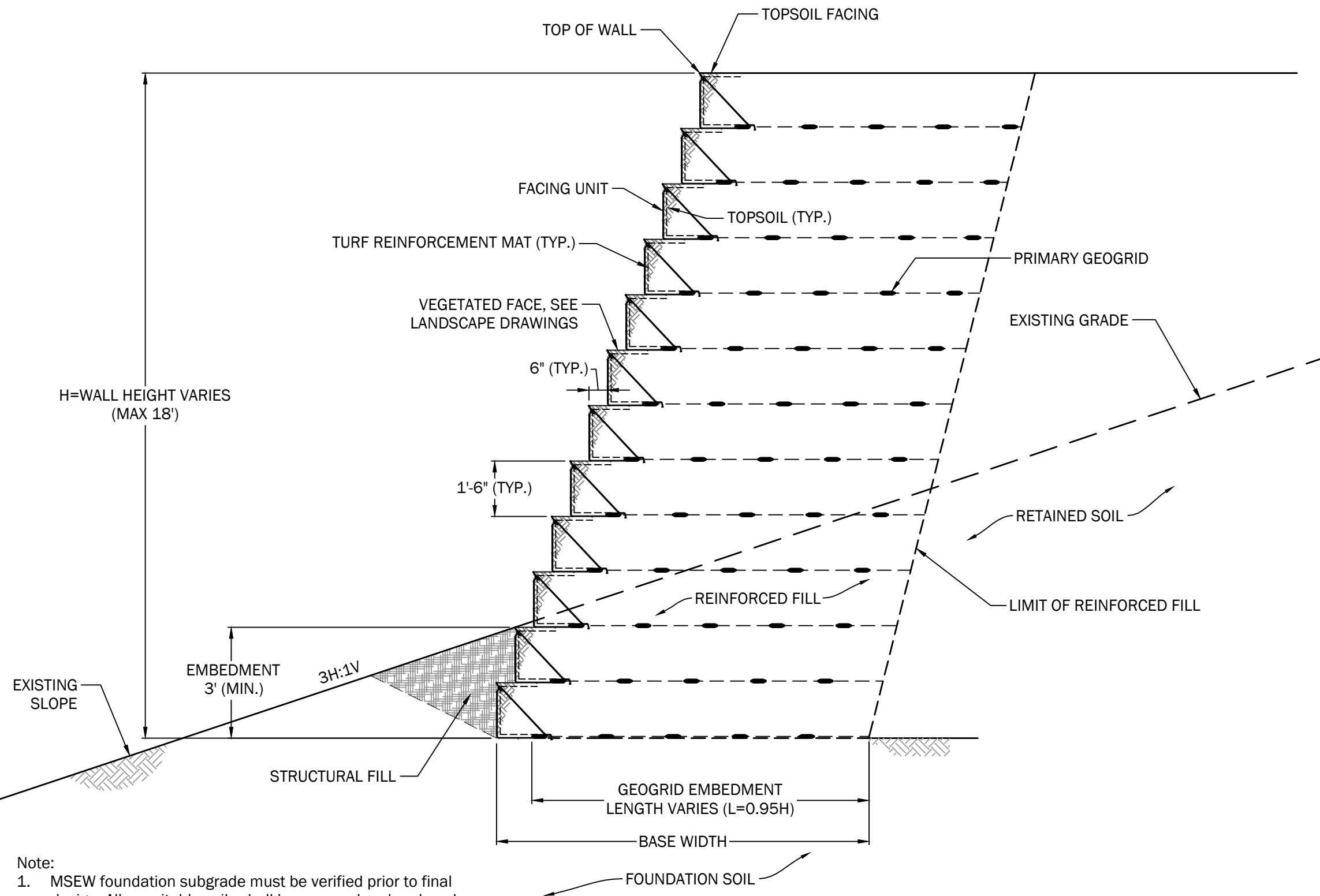
MSE and Gravity Block Wall Calculations

Slope Stability Analyses Results

cc: Kevin Preston, Susan Milan, King County Housing Authority (one copy by email)
Alberto Cisneros, KPFF (one copy by email)
Jill Routt, Brian Fields, Goldsmith Engineering (one copy by email)

Disclaimer: Any electronic form, facsimile or hard copy of the original document (email, text, table, and/or figure), if provided, and any attachments are only a copy of the original document. The original document is stored by GeoEngineers, Inc. and will serve as the official document of record.

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- Note:
1. MSEW foundation subgrade must be verified prior to final design. All unsuitable soils shall be removed and replaced with structural fill.

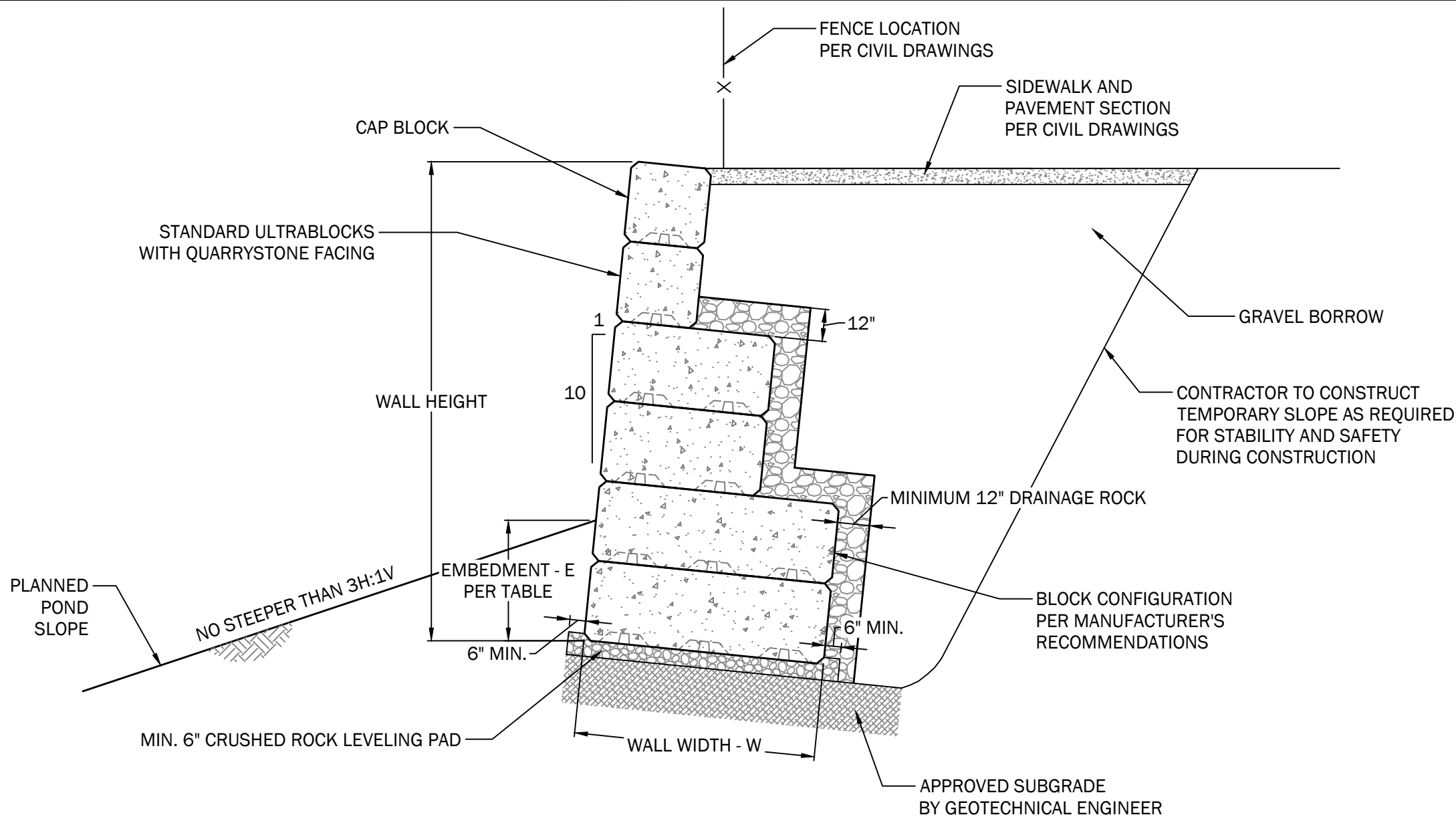
- Notes:
1. The locations of all features shown are approximate.
 2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

TYPICAL MSE WALL SECTION, LOTS 15-24

NOT TO SCALE

MSE Wall Section	
Wind Rose Development	
King County, Washington	
	Figure 1

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TYPICAL GRAVITY BLOCK WALL CROSS SECTION DETENTION POND
NOT TO SCALE

Wall Height (feet)	Number of Turned Base Blocks	Wall Width, W (feet)	Minimum Embedment Depth, E (feet)
15	4	7.5	3
12.5	3	5	2
10	2	5	2
7.5	1	5	1.5
5	0	2.5	1

Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Construction Notes

- Materials:
- A. Segmental Concrete Units
1. Wall units shall be 22.46' x 2.46' x 4.92' blocks produced to ULTRABLOCK specifications.
- B. Drainage Rock
1. Drainage Rock shall meet WSDOT standard specification 9-03.12(4).
- C. Crushed Rock Leveling Pad
1. Crushed rock used as a leveling pad at the base of the wall shall meet WSDOT Standard Specification 9-03.9(3) "Crushed Surfacing".
- D. Backfill Materials
1. Backfill material shall meet WSDOT 9-03.14(1) Gravel Borrow or other material as approved by Geotechnical Engineer.
- Execution:
- A. Subgrade Preparation
1. Subgrade bearing surfaces shall be cleared and grubbed, removing organic or deleterious materials. Soft areas shall be overexcavated and replaced with structural fill prior to placing blocks, as directed by Geotechnical Engineer.
2. Subgrade shall be excavated to a depth such that the base of the first course of block is at the minimum embedment depth shown in the table on this sheet, unless otherwise specified in the design or as directed by Geotechnical Engineer.
3. Subgrade shall be approved by Geotechnical Engineer. Soil shall be compacted before construction proceeds. Subgrade materials not meeting Geotechnical Engineer's approval shall be removed and replaced with structural fill material at direction of Geotechnical Engineer.
4. Overexcavated areas shall be replaced with structural backfill material.
- B. Leveling Pad
1. Leveling pad shall be placed as shown on the construction plans.
2. Leveling pad shall be placed on undisturbed native soils or on properly compacted structural fill.
3. Leveling pad shall be compacted to 95 percent of modified proctor dry density to ensure a hard, level surface on which to place the first course of blocks. Pad shall be constructed at the proper elevation and slope to ensure the final elevations and batter as shown on the plans. The manufacturer recommends using 2x6 wood boards and steel stakes to make a form for achieving design batter.
4. Leveling pad shall have a minimum thickness of 6 inches. Leveling pad shall extend beyond the blocks in all directions at least 6 inches.
- C. Unit Installation
1. The first course of wall units shall be placed on the prepared base with the front edges tight together. The units shall be checked for level and alignment as they are placed.
2. Ensure that units are in full contact with base. Proper care shall be taken to develop straight lines and smooth curves on base course as per wall layout.
3. Fill a minimum of 12 inches behind the base course with Drainage Rock. Use approved soils to backfill behind the wall rock and in front of the base course to firmly lock in place. Check again for level and alignment. Use a plate compactor to consolidate the area behind the first course. All excess material shall be swept from top of units.
- D. Backfill Placement
1. Backfill material shall be placed in lifts and compacted to a minimum 95 percent of Maximum Dry Density (MDD) as determined by ASTM D-1557.
2. Only hand-operated compaction equipment shall be allowed within 5 feet behind the wall. Backfill in this zone shall be compacted to 90 percent of (MDD) as determined by ASTM D-1557.
3. The excavated trench area in front of the toe of the wall shall be filled and compacted as the wall is being constructed.

DESIGN PARAMETERS

ALLOWABLE SOIL BEARING PRESSURE = 3,000 PSF
DESIGN OF THE RETAINING STRUCTURE IS BASED ON THE FOLLOWING PARAMETERS:

	FRICTION ANGLE, PHI (degrees)	COHESION, C (psf)	MOIST UNIT WEIGHT (pcf)	BUOYANT UNIT WEIGHT (pcf)
RETAINED SOIL OR BACKFILL	34	0	125	62.6
FOUNDATION SOIL	36	0	125	62.6

EXTERNAL STABILITY OF WALLS:
SURCHARGE LOADING = 250 PSF
SEISMIC ACCELERATION = 0.5xPGA = 0.21g
MINIMUM FACTOR OF SAFETY, BASE SLIDING - 1.5 (1.1 Seismic)
MINIMUM FACTOR OF SAFETY, OVERTURNING - 2.0 (1.1 Seismic)

Gravity Block Retaining Wall Section and Notes

Wind Rose Development

King County, Washington



Figure 2

 Design Maps Detailed Report

ASCE 7-10 Standard (47.51623°N, 122.34136°W)

Site Class D – “Stiff Soil”, Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_s) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From [Figure 22-1](#) ^[1] $S_s = 1.543 \text{ g}$ From [Figure 22-2](#) ^[2] $S_1 = 0.585 \text{ g}$

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
Any profile with more than 10 ft of soil having the characteristics: • Plasticity index $PI > 20$, • Moisture content $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u < 500 \text{ psf}$			
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4-1: Site Coefficient F_a

Site Class	Mapped MCE _R Spectral Response Acceleration Parameter at Short Period				
	$S_s \leq 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_s

For Site Class = D and $S_s = 1.543$ g, $F_a = 1.000$

Table 11.4-2: Site Coefficient F_v

Site Class	Mapped MCE _R Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_1

For Site Class = D and $S_1 = 0.585$ g, $F_v = 1.500$

Equation (11.4-1):

$$S_{MS} = F_a S_s = 1.000 \times 1.543 = 1.543 \text{ g}$$

Equation (11.4-2):

$$S_{M1} = F_v S_1 = 1.500 \times 0.585 = 0.878 \text{ g}$$

Section 11.4.4 — Design Spectral Acceleration Parameters

Equation (11.4-3):

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 1.543 = 1.029 \text{ g}$$

Equation (11.4-4):

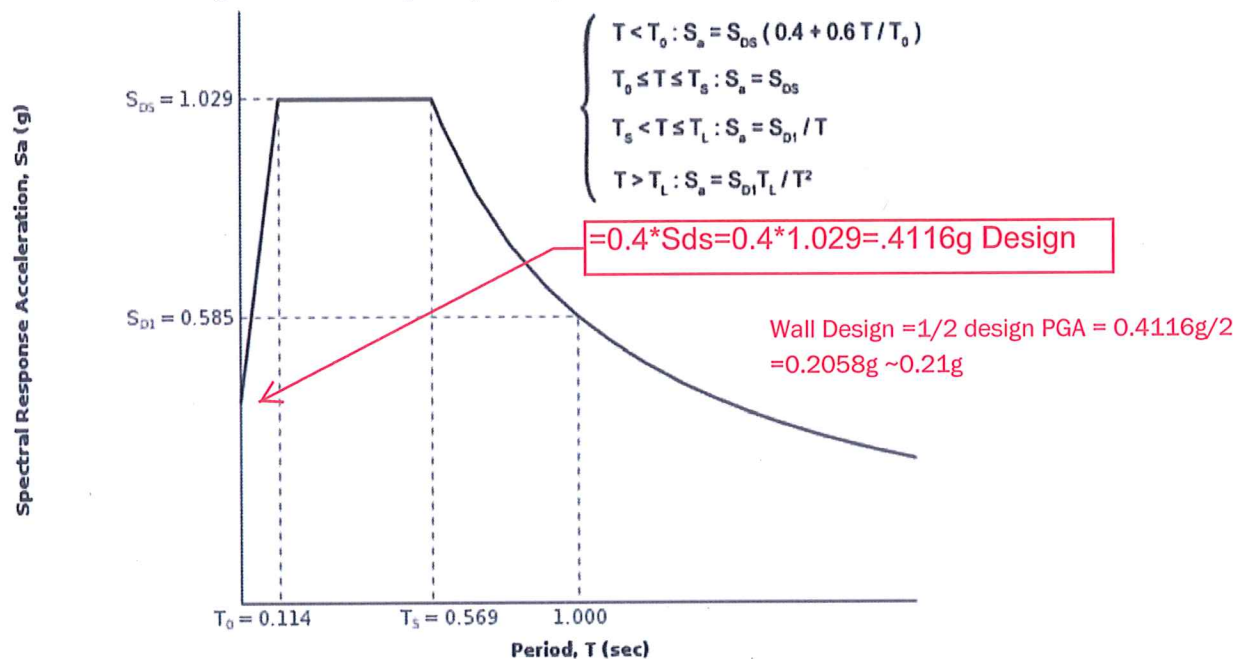
$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 0.878 = 0.585 \text{ g}$$

Section 11.4.5 — Design Response Spectrum

From **Figure 22-12** ^[3]

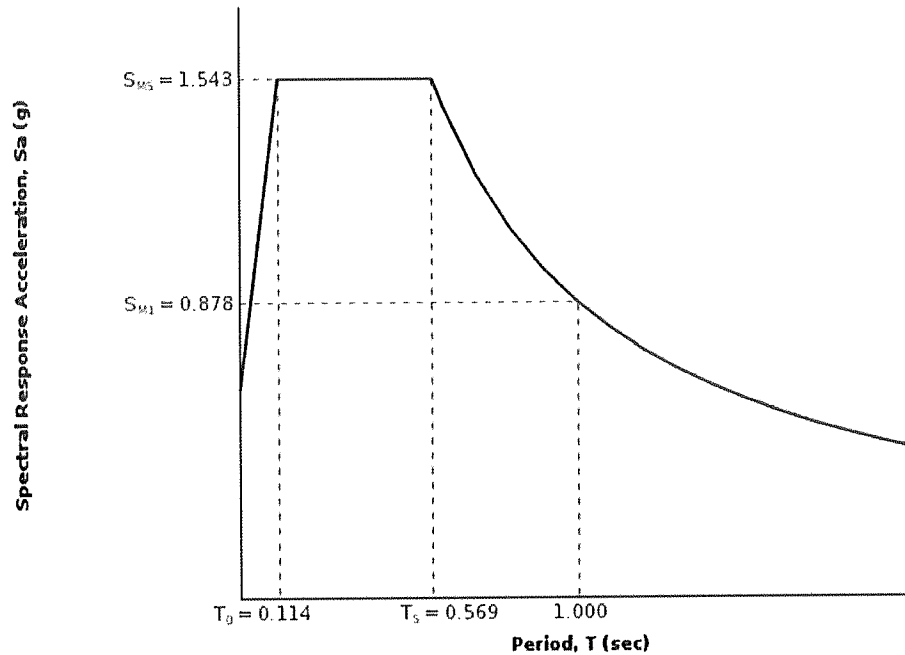
$$T_L = 6 \text{ seconds}$$

Figure 11.4-1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From **Figure 22-7** ^[4]

$$PGA = 0.647$$

Equation (11.8-1):

$$PGA_M = F_{PGA} PGA = 1.000 \times 0.647 = 0.647 \text{ g}$$

Table 11.8-1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.647 g, $F_{PGA} = 1.000$

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From **Figure 22-17** ^[5]

$$C_{RS} = 0.934$$

From **Figure 22-18** ^[6]

$$C_{R1} = 0.916$$

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \leq S_{DS} < 0.33g$	B	B	C
$0.33g \leq S_{DS} < 0.50g$	C	C	D
$0.50g \leq S_{DS}$	D	D	D

For Risk Category = I and $S_{DS} = 1.029g$, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \leq S_{D1} < 0.133g$	B	B	C
$0.133g \leq S_{D1} < 0.20g$	C	C	D
$0.20g \leq S_{D1}$	D	D	D

For Risk Category = I and $S_{D1} = 0.585g$, Seismic Design Category = D

Note: When S_1 is greater than or equal to $0.75g$, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category $\equiv$ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = D

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. Figure 22-1: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. Figure 22-2: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. Figure 22-12: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. Figure 22-7: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. Figure 22-17: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. Figure 22-18: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

MSEW(3.0): Update # 14.94

Title: Wind Rose Development
Project Number: 1329-009-01
Client: King County Housing Authority
Designer: AJH
Station Number:

ANALYSIS of a SIMPLE STRUCTURE using GEOGRID as reinforcing material.

SOIL DATA

REINFORCED SOIL

Unit weight, γ	125.0 lb/ft ³
Design value of internal angle of friction, ϕ	34.0 °

RETAINED SOIL

Unit weight, γ	125.0 lb/ft ³
Design value of internal angle of friction, ϕ	34.0 °

FOUNDATION SOIL (Considered as an equivalent uniform soil)

Equivalent unit weight, $\gamma_{\text{equiv.}}$		125.0 lb/ft ³
Equivalent internal angle of friction, $\phi_{\text{equiv.}}$		36.0°
Equivalent cohesion, $c_{\text{equiv.}}$		0.0 lb/ft ²

Water table does not affect bearing capacity

LATERAL EARTH PRESSURE COEFFICIENTS

K_a (internal stability) = 0.2827 (if batter is less than 10° , K_a is calculated from eq. 15. Otherwise, eq. 38 is utilized)

Inclination of internal slip plane, $\psi = 62.00^\circ$ (see Fig. 28 in DEMO 82).

Ka (external stability) = 0.2827 (if batter is less than 10°, Ka is calculated from eq. 16. Otherwise, eq. 17 is utilized)

BEARING CAPACITY

Bearing capacity coefficients (calculated by MSEW): $N_c = 0.00$ $N_\gamma = 26.71$

SEISMICITY

Maximum ground acceleration coefficient, $A = 0.210$

Design acceleration coefficient in Internal Stability: $K_h = A_m = 0.260$

Design acceleration coefficient in External Stability: $K_{h_d} = 0.260 \Rightarrow K_h = A_m = 0.260$

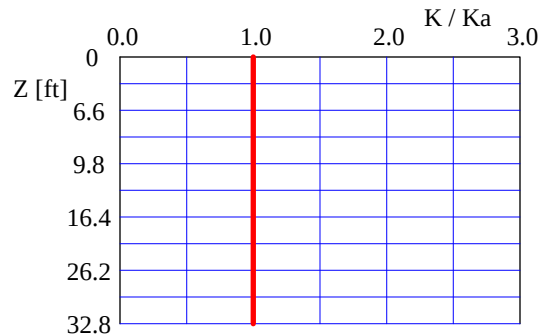
$$K_{ae} (K_h > 0) = 0.4208 \qquad K_{ae} (K_h = 0) = 0.2463 \qquad \Delta K_{ae} = 0.1745$$

Seismic soil-geogrid friction coefficient, F^* is 80.0% of its specified static value.

D A T A	Geogrid type #1	Geogrid type #2	Geogrid type #3	Geogrid type #4	Geogrid type #5
Tult [lb/ft]	5750.0				
Durability reduction factor, RFd	1.10				
Installation-damage reduction factor, RFid	1.10				
Creep reduction factor, RFC	2.00	N/A	N/A	N/A	N/A
Fs-overall for strength	N/A				
Coverage ratio, Rc	1.000				
Friction angle along geogrid-soil interface, ρ	24.22				
Pullout resistance factor, F*	$0.67 \cdot \tan \phi$	N/A	N/A	N/A	N/A
Scale-effect correction factor, α	0.8				

Variation of Lateral Earth Pressure Coefficient With Depth

Z	K / Ka
0 ft	1.00
3.3 ft	1.00
6.6 ft	1.00
9.8 ft	1.00
13.1 ft	1.00
16.4 ft	1.00
19.7 ft	1.00



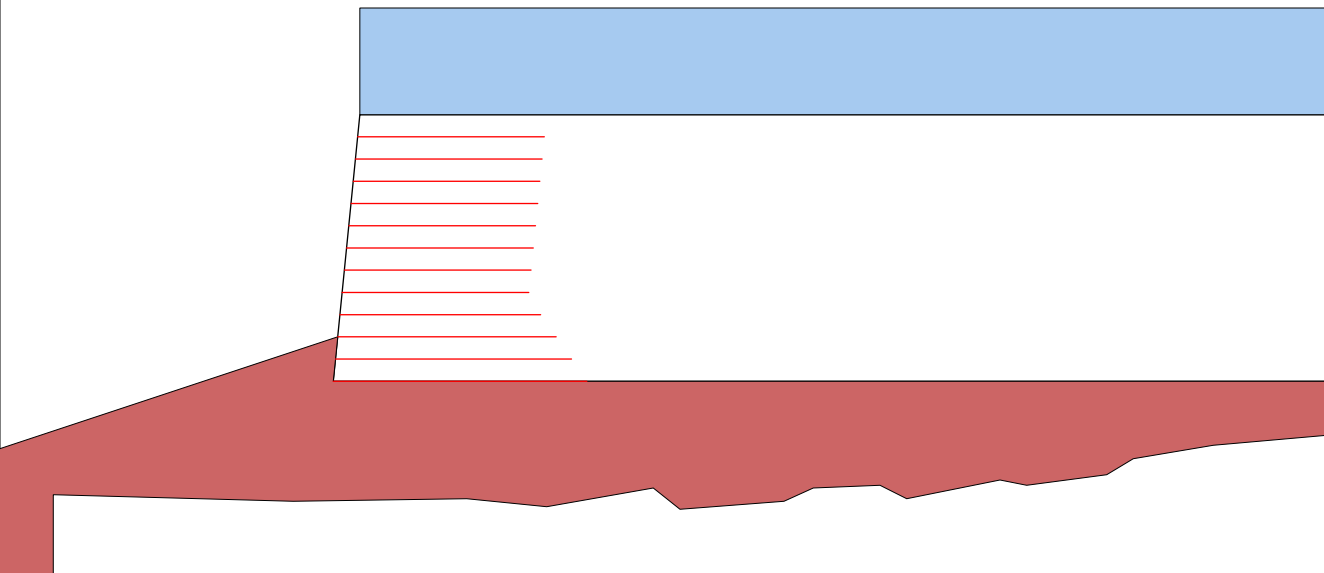
INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)

Design height, Hd	18.00 [ft]	{ Embedded depth is E = 3.00 ft, and height above top of finished bottom grade is H = 15.00 ft }
Batter, ω	5.7 [deg]	
Backslope, β	0.0 [deg]	
Backslope rise	0.0 [ft]	
		Broken back equivalent angle, I = 0.00° (see Fig. 25 in DEMO 82)

UNIFORM SURCHARGE

Uniformly distributed dead load is 0.0 [lb/ft ²], and live load is 250.0 [lb/ft ²]

ANALYZED REINFORCEMENT LAYOUT:



SCALE:

0 2 4 6 8 10 [ft]



ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, $F_s = 10.35$, Meyerhof stress = 2550 lb/ft².

Foundation Interface: Direct sliding, $F_s = 3.518$, Eccentricity, $e/L = 0.0475$, F_s -overturning = 7.17

G E O G R I D				C O N N E C T I O N							
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [pullout resistance]	Fs-overall [connection break]	Fs-overall [geogrid strength]	Geogrid strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.00	17.12	1	N/A	N/A	N/A	4.568	50.501	2.347	0.0475	---
2	1.50	15.93	1	N/A	N/A	N/A	2.423	22.673	2.345	0.0473	---
3	3.00	14.74	1	N/A	N/A	N/A	2.637	20.115	2.344	0.0470	---
4	4.50	13.55	1	N/A	N/A	N/A	2.892	17.156	2.342	0.0467	---
5	6.00	12.60	1	N/A	N/A	N/A	3.202	14.559	2.387	0.0439	---
6	7.50	12.60	1	N/A	N/A	N/A	3.586	13.346	2.651	0.0332	---
7	9.00	12.60	1	N/A	N/A	N/A	4.075	12.093	2.975	0.0239	---
8	10.50	12.60	1	N/A	N/A	N/A	4.718	10.802	3.384	0.0163	---
9	12.00	12.60	1	N/A	N/A	N/A	5.603	9.438	3.915	0.0101	---
10	13.50	12.60	1	N/A	N/A	N/A	6.896	7.952	4.634	0.0054	---
11	15.00	12.60	1	N/A	N/A	N/A	8.965	6.233	5.661	0.0022	---
12	16.50	12.60	1	N/A	N/A	N/A	9.562	2.973	7.248	0.0004	---

ANALYSIS: CALCULATED FACTORS (Seismic conditions)

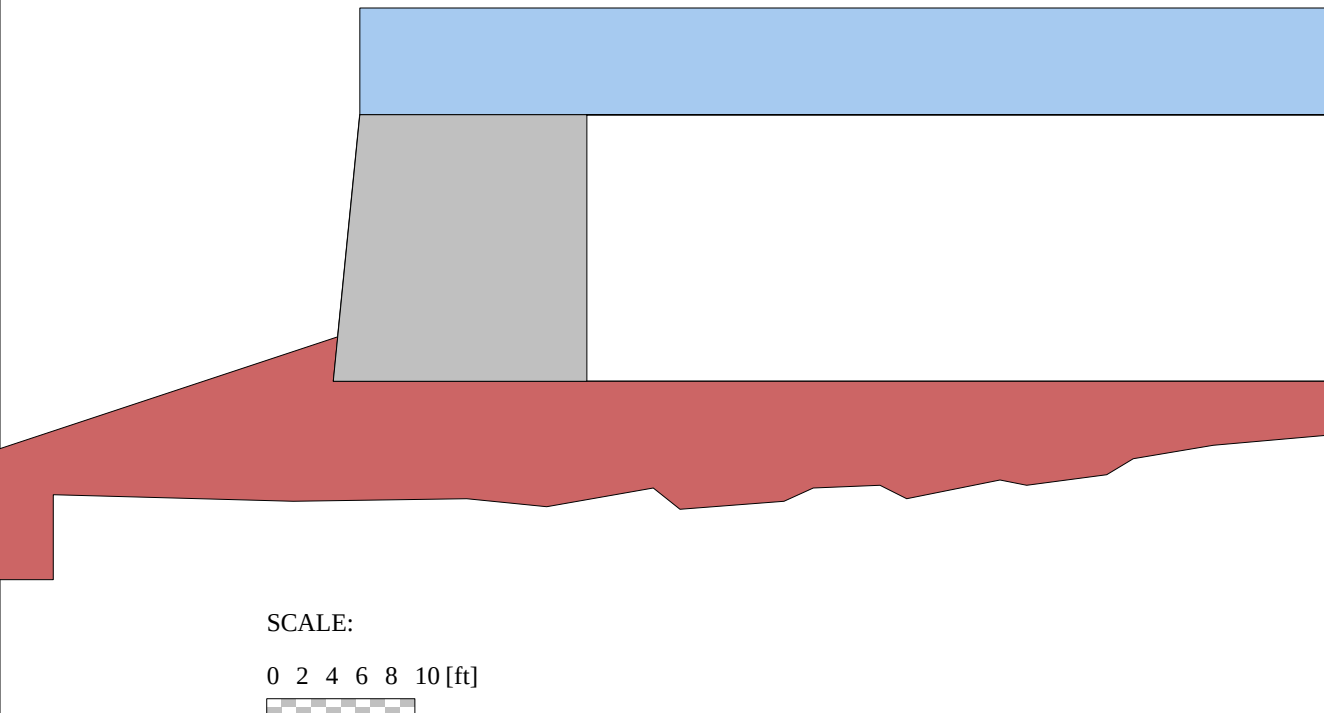
Bearing capacity, $F_s = 6.81$, Meyerhof stress = 3143 lb/ft².

Foundation Interface: Direct sliding, $F_s = 1.822$, Eccentricity, $e/L = 0.1439$, F_s -overturning = 3.10

G E O G R I D				C O N N E C T I O N							
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [pullout resistance]	Fs-overall [connection break]	Fs-overall [geogrid strength]	Geogrid strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.00	17.12	1	N/A	N/A	N/A	3.477	24.818	1.215	0.1439	---
2	1.50	15.93	1	N/A	N/A	N/A	2.109	13.982	1.224	0.1408	---
3	3.00	14.74	1	N/A	N/A	N/A	2.308	12.527	1.235	0.1372	---
4	4.50	13.55	1	N/A	N/A	N/A	2.548	10.811	1.247	0.1330	---
5	6.00	12.60	1	N/A	N/A	N/A	2.837	9.264	1.288	0.1226	---
6	7.50	12.60	1	N/A	N/A	N/A	3.160	8.410	1.453	0.0930	---
7	9.00	12.60	1	N/A	N/A	N/A	3.567	7.528	1.664	0.0677	---
8	10.50	12.60	1	N/A	N/A	N/A	4.093	6.619	1.942	0.0464	---
9	12.00	12.60	1	N/A	N/A	N/A	4.802	5.662	2.327	0.0293	---
10	13.50	12.60	1	N/A	N/A	N/A	5.808	4.627	2.892	0.0161	---
11	15.00	12.60	1	N/A	N/A	N/A	7.346	3.461	3.805	0.0069	---
12	16.50	12.60	1	N/A	N/A	N/A	7.902	1.675	5.531	0.0016	---

BEARING CAPACITY for GIVEN LAYOUT

	STATIC	SEISMIC	UNITS
(Water table does not affect bearing capacity)			
Ultimate bearing capacity, q-ult	26400	21415	[lb/ft ²]
Meyerhof stress, σ_v	2549.7	3143	[lb/ft ²]
Eccentricity, e	0.65	2.14	[ft]
Eccentricity, e/L	0.038	0.125	
Fs calculated	10.35	6.81	
Base length	17.12	17.12	[ft]



At interface with foundation: e/L static = 0.0475, e/L seismic = 0.1439; Overturning: F_s -static = 7.17, F_s -seismic = 3.10

#	Geogrid Elevation [ft]	Geogrid Length [ft]	Fs Static	Fs Seismic	Geogrid Type #	Product name
1	0.00	17.12	2.347	1.215	1	---
2	1.50	15.93	2.345	1.224	1	---
3	3.00	14.74	2.344	1.235	1	---
4	4.50	13.55	2.342	1.247	1	---
5	6.00	12.60	2.387	1.288	1	---
6	7.50	12.60	2.651	1.453	1	---
7	9.00	12.60	2.975	1.664	1	---
8	10.50	12.60	3.384	1.942	1	---
9	12.00	12.60	3.915	2.327	1	---
10	13.50	12.60	4.634	2.892	1	---
11	15.00	12.60	5.661	3.805	1	---
12	16.50	12.60	7.248	5.531	1	---

#	Geogrid Elevation [ft]	Geogrid Length [ft]	e / L Static	e / L Seismic	Geogrid Type #	Product name
1	0.00	17.12	0.0475	0.1439	1	---
2	1.50	15.93	0.0473	0.1408	1	---
3	3.00	14.74	0.0470	0.1372	1	---
4	4.50	13.55	0.0467	0.1330	1	---
5	6.00	12.60	0.0439	0.1226	1	---
6	7.50	12.60	0.0332	0.0930	1	---
7	9.00	12.60	0.0239	0.0677	1	---
8	10.50	12.60	0.0163	0.0464	1	---
9	12.00	12.60	0.0101	0.0293	1	---
10	13.50	12.60	0.0054	0.0161	1	---
11	15.00	12.60	0.0022	0.0069	1	---
12	16.50	12.60	0.0004	0.0016	1	---

RESULTS for STRENGTH

Live Load included in calculating Tmax

#	Geogrid Elevation [ft]	Tavailable [lb/ft]	Tmax [lb/ft]	Tmd [lb/ft]	Specified minimum Fs-overall static	Actual calculated Fs-overall static	Specified minimum Fs-overall seismic	Actual calculated Fs-overall seismic	Product name
1	0.00	2376	520.15	326.59	N/A	4.568	N/A	3.477	---
2	1.50	2376	980.67	291.53	N/A	2.423	N/A	2.109	---
3	3.00	2376	901.15	256.47	N/A	2.637	N/A	2.308	---
4	4.50	2376	821.64	221.41	N/A	2.892	N/A	2.548	---
5	6.00	2376	742.13	190.93	N/A	3.202	N/A	2.837	---
6	7.50	2376	662.61	178.57	N/A	3.586	N/A	3.160	---
7	9.00	2376	583.10	166.21	N/A	4.075	N/A	3.567	---
8	10.50	2376	503.59	153.86	N/A	4.718	N/A	4.093	---
9	12.00	2376	424.07	141.50	N/A	5.603	N/A	4.802	---
10	13.50	2376	344.56	129.14	N/A	6.896	N/A	5.808	---
11	15.00	2376	265.05	116.78	N/A	8.965	N/A	7.346	---
12	16.50	2376	248.48	104.42	N/A	9.562	N/A	7.902	---

RESULTS for PULLOUT

Live Load included in calculating Tmax

NOTE: Live load is not included in calculating the overburden pressure used to assess pullout resistance.

#	Geogrid Elevation [ft]	Coverage Ratio	Tmax [lb/ft]	Tmd [lb/ft]	Le [ft] (see NOTE)	La [ft]	Avail.Static Pullout, Pr [lb/ft]	Specified Static Fs	Actual Static Fs	Avail.Seism. Pullout, Pr [lb/ft]	Specified Seismic Fs	Actual Seismic Fs
1	0.00	1.000	520.2	326.6	17.12	0.00	26268.0	N/A	50.501	21014.4	N/A	24.818
2	1.50	1.000	980.7	291.5	15.28	0.65	22234.9	N/A	22.673	17788.0	N/A	13.982
3	3.00	1.000	901.2	256.5	13.44	1.30	18126.8	N/A	20.115	14501.4	N/A	12.527
4	4.50	1.000	821.6	221.4	11.61	1.94	14096.1	N/A	17.156	11276.9	N/A	10.811
5	6.00	1.000	742.1	190.9	10.01	2.59	10805.0	N/A	14.559	8644.0	N/A	9.264
6	7.50	1.000	662.6	178.6	9.36	3.24	8843.2	N/A	13.346	7074.5	N/A	8.410
7	9.00	1.000	583.1	166.2	8.71	3.89	7051.5	N/A	12.093	5641.2	N/A	7.528
8	10.50	1.000	503.6	153.9	8.07	4.53	5439.7	N/A	10.802	4351.7	N/A	6.619
9	12.00	1.000	424.1	141.5	7.42	5.18	4002.5	N/A	9.438	3202.0	N/A	5.662
10	13.50	1.000	344.6	129.1	6.77	5.83	2740.0	N/A	7.952	2192.0	N/A	4.627
11	15.00	1.000	265.0	116.8	6.12	6.48	1652.0	N/A	6.233	1321.6	N/A	3.461
12	16.50	1.000	248.5	104.4	5.47	7.13	738.7	N/A	2.973	591.0	N/A	1.675



UltraWall

UltraWall v3.3 Build 14226

Project: Wind Rose Pond
 Location: Seattle, WA
 Designer: AJH
 Date: 04/09/2015
 Section: 15' wall
 Design Method: NCMA_09_3rd_Ed
 Design Unit: Ultrablock

Seismic Acc: 0.210

SOIL PARAMETERS	ϕ	coh	γ
Retained Soil:	36 deg	0 psf	125 pcf
Foundation Soil:	40 deg	0 psf	63 pcf
Leveling Pad:	Crushed Stone		

GEOMETRY

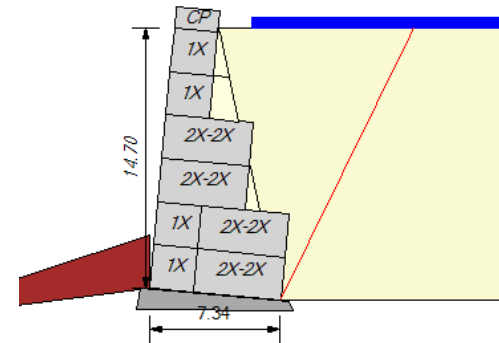
Design Height:	15.00 ft	Live Load:	250 psf
Wall Batter/Tilt:	0.00/ 5.40 deg	Live Load Offset:	2.00 ft
Embedment:	3.00 ft	Live Load Width:	50 ft
Leveling Pad Depth:	0.50 ft	Dead Load:	0 psf
Slope Angle:	0.0 deg	Dead Load Offset:	0.0 ft
Slope Length:	0.0 ft	Dead Load Width:	0 ft
Slope Toe Offset:	0.0 ft	Leveling Pad Width:	8.38 ft
		Toe Slope Angle:	18.20
		Toe Slope Length:	15.00
		Toe Slope Bench:	0.00

FACTORS OF SAFETY (Static / Seismic)

Sliding:	1.50 / 1.125	Overturning:	1.50 / 1.125
Bearing:	2.00 / 1.5		

RESULTS (Static / Seismic)

FoS Sliding:	2.44 (lvlpd) / 2.02	FoS Overturning:	2.31 / 1.77
Bearing:	3036.24 / 3798.84	FoS Bearing:	10.37 / 8.29



Name	Elev.	ka	kae	Pa	Pae	Pir	- PaC	FSsl	FoS OT	siesFSsl	FoS SeisOT
CP	14.68	0.594	0.244	2	2	34	0	100.00	100.00	100.00	99.87
1X	12.24	0.200	0.244	91	111	101	0	100.00	21.81	87.03	6.21
1X	9.79	0.193	0.244	319	403	169	0	45.28	3.90	32.71	2.41
2X-2X	7.34	0.376	0.451	1439	1723	304	0	13.54	3.56	11.52	2.68
2X-2X	4.89	0.319	0.380	2106	2502	440	0	8.81	2.57	7.36	1.94
1X-2X-2X	2.45	0.382	0.450	4008	4712	642	0	10.27	2.78	8.81	2.17
1X-2X-2X	0.00	0.340	0.401	5040	5942	845	0	2.44[3.11]	2.31	2.02[2.68]	1.77



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 (1.49999999999999 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.



DESIGN DATA

TARGET DESIGN VALUES (Factors of Safety - Static / Seismic)

Minimum Factor of Safety for the sliding along the base	FSsl =1.50 /1.125
Minimum Factor of Safety for overturning about the toe	FSot =1.50 /1.125
Minimum Factor of Safety for bearing (foundation shear failure)	FSbr =2.00 /3.000
Seismic factors of safety are 75% of the static values.	

MINIMUM DESIGN REQUIREMENTS

Minimum embedment depth	Min_emb =3.00 ft
-------------------------	------------------

INPUT DATA

Geometry

Wall Geometry

Design Height, top of leveling pad to finished grade at top of wall	H =14.70 ft
Embedment, measured from top of leveling pad to finished grade	emb =3.00 ft
Leveling Pad Depth	LP Thickness =0.50 ft
Face Batter, measured from vertical	i =0.00 deg

Slope Geometry

Slope Angle, measured from horizontal	β =0.00 deg
Slope toe offset, measured from back of the face unit	STL_offset =0.00 ft
Slope Length, measured from back of wall facing	SL_Length =0.00 ft

NOTE: If the slope toe is offset or the slope breaks within three times the wall height, a Coulomb Trial Wedge method of analysis is used.

Surcharge Loading

Live Load, assumed transient loading (e.g. traffic)	LL = 250.00 psf
Live Load Offset, measured from back face of wall	LL_offset =2.00 ft
Live Load Width, assumed strip loading	LL_width = 50.00 ft
Dead Load, assumed permanent loading (e.g. buildings)	DL = 0.00 psf
Dead Load Offset, measured from back face of wall	DL_offset =0.00 ft
Dead Load Width, assumed strip loading	DL_width = 0.00 ft

Soil Parameters

Retained Zone

Angle of Internal Friction	ϕ = 36.00 deg
Cohesion	coh =0.00 psf
Moist Unit Weight	gamma =125.00 pcf

Foundation

Angle of Internal Friction	ϕ = 40.00 deg
Cohesion	coh =0.00 psf
Moist Unit Weight	gamma =62.60 pcf



RETAINING WALL UNITS

STRUCTURAL PROPERTIES:

N is the normal force [or factored normal load] on the base unit

The default leveling pad to base unit shear is $0.8 \tan(\phi)$ [AASHTO 10.6.3.4] or

may be the manufacturer supplied data. ϕ is assumed to be 40 degrees for a stone leveling pad.

The shear equations are setup as $N(\tan \phi) + \text{Intercept}$

Unit Designation: Cap

Unit Dimensions:

Height = 1.23 ft

Width = 4.92 ft

Weight = 423 lbs

Depth = 2.46 ft

Density = 140.00 pcf

Unit to Unit Shear

$$\tau = N \tan(0.00) + 17796.00 \text{ ppf}$$

Unit to Leveling Pad Shear

$$\tau = N \tan(34.00) + 0.00 \text{ ppf}$$

Unit Designation: Full

Unit Dimensions:

Height = 2.46 ft

Width = 4.92 ft

Weight = 846 lbs

Depth = 2.46 ft

Density = 140.00 pcf

Unit to Unit Shear

$$\tau = N \tan(0.00) + 17796.00 \text{ ppf}$$

Unit to Leveling Pad Shear

$$\tau = N \tan(33.80) + 0.00 \text{ ppf}$$



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	2.62	1107.20	0.80	3.86	3.08
2	846.08	2.39	2018.67	113.83	3.84	437.45
3	846.08	2.15	1822.93	311.91	4.05	1264.12
4	1692.15	3.15	5334.31	0.20	5.62	1.12
5	1692.15	2.92	4942.83	106.43	5.59	594.86
6	2538.23	3.92	9946.94	0.00	7.61	0.00
7	2538.23	3.69	9359.72			

Block Weight (Force v) = 10576 ppf

X-Arm = 3.27 ft

Soils Block Weight (Force v) = 533 ppf

X-Arm = 4.47 ft

Active Earth Pressure $P_a = 5040$ ppf

P_{a_h} (Force H) = $P_a \cos(\text{batter} + \delta) = 5040 \times \cos(12.7 + 27.0) = 3875$ ppf

Y-Arm = 5.13 ft

P_{a_v} (Force V) = $P_a \sin(\text{batter} + \delta) = 5040 \times \sin(12.7 + 27.0) = 3223$ ppf

X-Arm = 6.18 ft

Live Load $P_q = 1069$ ppf

P_{q_h} (Force H) = $P_q \cos(\text{batter} + \delta) = 1069 \times \cos(12.7 + 27.0) = 822$ ppf

Y-Arm = 7.70 ft

P_{q_v} (Force V) = $P_q \sin(\text{batter} + \delta) = 1069 \times \sin(12.7 + 27.0) = 683$ ppf

X-Arm = 5.59 ft



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)
Coefficient of active earth pressure

$\delta = 27.0$ deg
 $k_a = 0.340$

External failure plane

Effective Angle from horizontal

Coefficient of passive earth pressure: $k_p = (1 + \sin(\phi)) / (1 - \sin(\phi))$

$\rho = 63$ deg
Eff. Angle = 77.25 deg
 $k_p = 4.60$

$$K_a := \frac{\cos(\phi_1 + i)^2}{\cos(i)^2 \cdot \cos(\delta_1 - i) \left(1 + \frac{\sin(\phi_1 + \delta_1) \cdot \sin(\phi_1 - \beta)}{\cos(\delta_1 - i) \cdot \cos(i + \beta)} \right)^2}$$

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 Surcharge Paqd

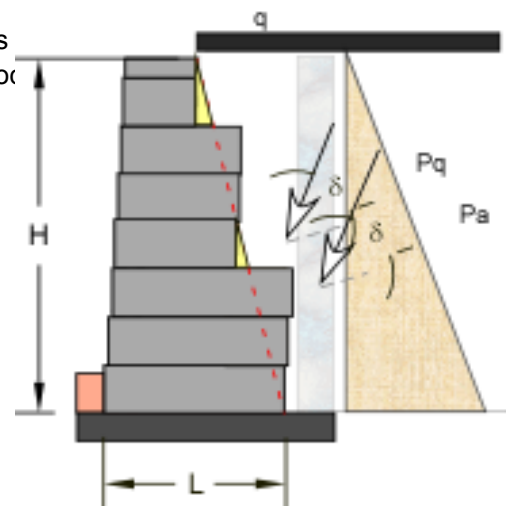
FORCES AND MOMENTS

The program resolves all the geometry into simple geometric shapes 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	10576	--	3.27	--	--	34533
Soil Wedge(W2)	1.00	533	--	4.47	--	--	2382
LvlPad(W18)	1.00	891	--	--	--	--	--
Pa_h	1.00	--	3875	--	5.13	19888	--
Pa_v	1.00	3223	--	6.18	--	--	19907
Pq_h	1.00	--	822	--	7.70	6325	--
Pq_v	1.00	683	--	5.59	--	--	3823
Sum V / H	1.00	15015	4697		Sum Mom	26212	60644

Note: live load forces and moments are not included in SumV or Mr as live loads are not included as resisting forces.





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.

$$\text{Forces Resisting sliding} = W1 + W2 + P_{av} + P_{qv}$$

$$10576 + 533 + 3223 + 683$$

$$N = 15015 \text{ ppf}$$

$$\text{Resisting force at pad} = (N \tan(\text{slope}) + \text{intercept} \times L)$$

$$15015 \times \tan(33.8) + 0.0 \times 7.4$$

where L is the base block width

$$R_{f1} = 11391$$

Friction angle is the lesser of the leveling pad and F_{nd}

N_1 includes N (the leveling pad) + leveling pad (LP)

$$15015 + 891$$

$$\phi = 40.00 \text{ deg}$$

$$N_1 = 15906 \text{ ppf}$$

Passive resistance is calculated using $k_p = (1 + \sin(40)) / (1 - \sin(40))$

Force at top of resisting trapezoid, $d_1 = 3.00$

Force at base of resisting trapezoid, $d_2 = 4.20$

Depth of trapezoid

$$P_p = [(F_{p1} + F_{p2}) / 2] \times \text{depth}$$

$$k_p = 4.60$$

$$F_{p1} = 863.68 \text{ ppf}$$

$$F_{p2} = 1208.32 \text{ ppf}$$

$$\text{depth} = 1.20 \text{ ft}$$

$$P_p = 1240.24$$

$$\text{Resisting force at } f_{nd} = (N_1 \tan(\phi) + c L) + P_p$$

$$15906 \times \tan(40) + 0 \times 7.6 + 1240$$

$$R_{f2} = 14587$$

where LP = $|v| \text{ pad thickness} \times 130 \text{ pcf} \times (L + |v| \text{ pad thickness} / 2)$

Driving force is the horizontal component of

$$P_{ah} + P_{qh}$$

$$3875 + 822$$

$$D_f = 4697$$

$$FS_{sl} = R_{f1} / D_f / R_{f2} / D_f$$

$$FS_{sl} = 2.44 / 3.11$$



OVERTURNING ABOUT THE TOE

Overturning 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). For concrete leveling pads eccentricity is checked at the base of the pad.

Moments resisting eccentricity = $M1 + MPav + MPqv$

$34533 + 2382 + 19907 + 3823$

$Mr = 60644 \text{ ft-lbs}$

Moments causing eccentricity = $MPah + MPqh$

$19888 + 6325$

$Mo = 26212 \text{ ft-lbs}$

$e = L/2 - (Mr - Mo) / N1$

$e = 7.38/2 - (60644 - 26212) / 15906$

$e = 1.39$

$e/L = 0.19$

$FSot = Mr / Mo$

$FSot = 60644 / 26212$

$FSot = 2.31$



Eccentricity is the calculation of the distance of the resultant away from the centroid of mass. In wall design the eccentricity is used to calculate an effective footing width.

Calculation of Eccentricity

$$\text{SumV} = (W1 + W2 + LL + Pa_v + Pq_v)$$

$$e = L/2 - (\text{SumMr} + M_{LL} - \text{SumMo})/(\text{SumV} + LL)$$

$$e = 7.38/2 - (34431 / 15015.13)$$

$$e = 1.394 \text{ ft}$$

Calculation of Bearing Pressures

$$\text{Qult} = c * Nc + q * Nq + 0.5 * \gamma * (B') * Ng$$

where:

$$Nc = 75.31$$

$$Nq = 64.20$$

$$Ng = 109.41$$

$$c = 0.00 \text{ psf}$$

$$q = 219.10 \text{ psf}$$

$$B' = B - 2e + \text{lpad} = 5.09 \text{ ft}$$

$$\text{Gamma} = 63 \text{ pcf}$$

Calculate Ultimate Bearing, Qult

$$\text{Bearing Pressure} = (\text{SumVert} / B') + (\text{LP width} * \text{gamma})$$

Calculated Factors of Safety for Bearing

$$\text{Qult} = 31483 \text{ psf}$$

$$\text{sigma} = 3036.24 \text{ psf}$$

$$\text{Qult/sigma} = 10.37$$



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	A = 0.210
Horizontal Acceleration [kh = A/2]	kh = 0.080
Vertical Acceleration	kv = 0.000

INERTIA FORCES OF THE STRUCTURE

Face (Pif) = (W1)*kh = 10575.95 * 0.080	
	Pif = 845.37 ppf

SEISMIC THRUST

Kae	Kae = 0.401
D_Kae = Kae - Ka = (0.401 - 0.340)	D_Kae = 0.061
Pae = 0.5*gamma*(H)^2*D_Kae	Pae = 902.01 ppf
Pae_h = Pae*cos(δ)	Pae_h = 693.52 ppf
Pae_v = Pae*sin(δ)	Pae_v = 576.75 ppf

TABLE OF RESULTS FOR SEISMIC REACTIONS

Name	Force (V)	Force (H)	X-len	Y-len	Mo	Mr
Face Blocks(W1)	10575.955	--	3.265	--	--	34532.6
Face Soil(W2)	533.173	--	4.468	--	--	2382.01
Pa_h	--	3875.171	--	5.132	19887.54	--
Pa_v	3222.716	--	6.177	--	--	19906.69
Pif	--	845.367	--	9.238	7809.22	--
Pae_h	--	693.522	--	9.238	6406.53	--
Pae_v	576.755	--	6.177	--	--	3562.61



SEISMIC SLIDING

The target factor of safety for seismic is 75% of the static value. Live loads are ignored in the analyses based on the basic premise that the probability of the maximum acceleration occurring at the exact same instant as the maximum live load is small.

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 is $W1 + W2 + Pav + Paev$	SumVs = 14909
Resisting force = $SumVs * \tan(\phi) + \text{intercept} \times L$	
$14908.60 + \tan(33.8) + 0.00 \times 7.38$	FRe = 10871 lbf
Driving force = $Pa_h + Pae_h + Pif$	
$3875 + 694 + 845$	FDr = 5414 lbf
FOS = FRe / FDr [leveling pad / foundation]	FoS = 2.02 / 2.68

SEISMIC OVERTURNING

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

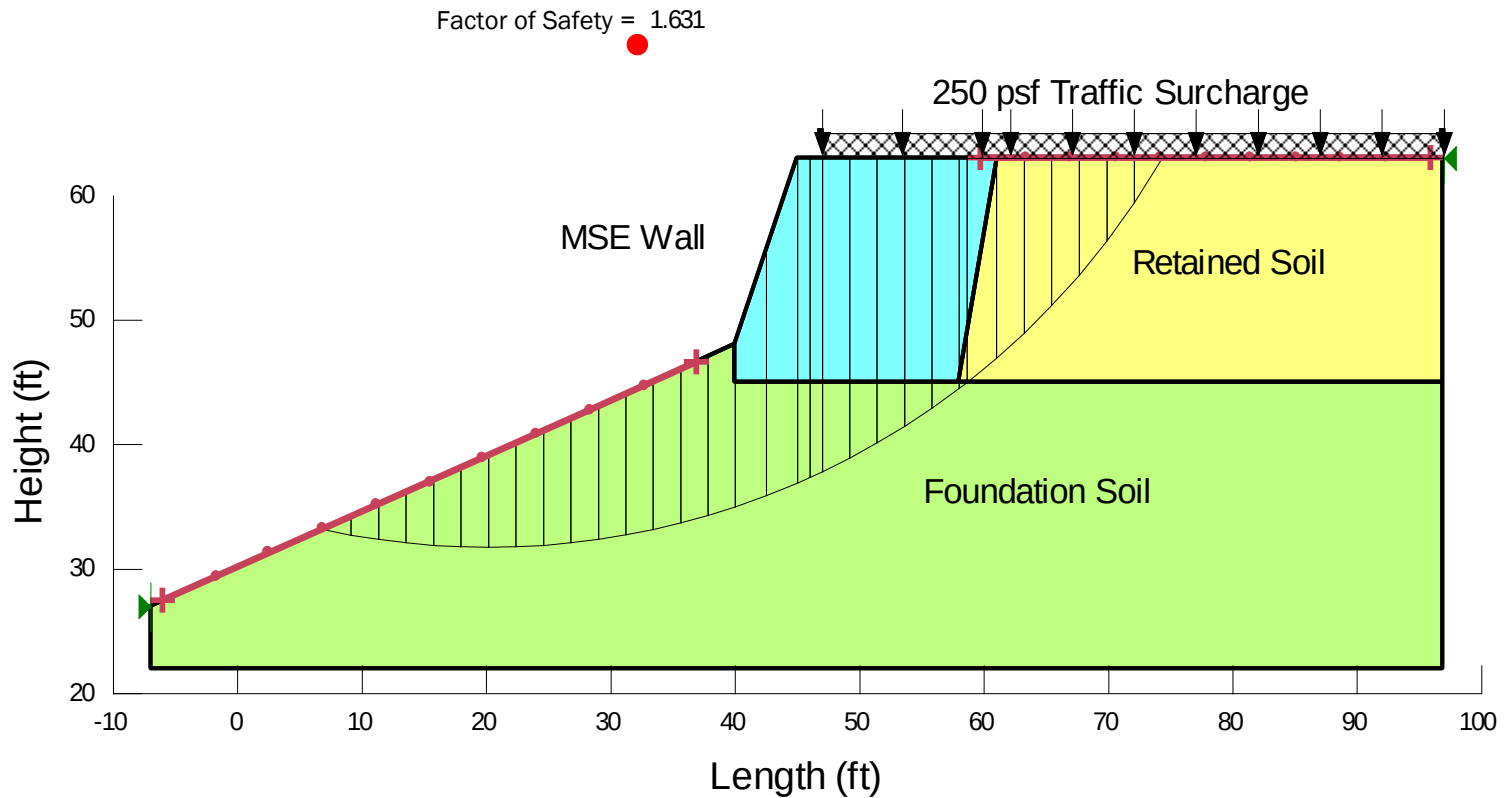
Resisting Moment = $M1 + M2 + MPav + MPaev$	SumMrS = 60384 ft lbf
Driving Moment = $MPah + MPaeh + MPif$	SumMoS = 34103.29 ft lbf
Factor of Safety = $SumMrS / SumMoS$	FoS = 1.77

SEISMIC BEARING

Bearing is the ability of the foundation to support the mass of the structure.

$Qult = c \cdot Nc + q \cdot Nq + 0.5 \cdot \gamma \cdot (B') \cdot Ng$
 where:
 $Nc = 75.31$
 $Nq = 64.20$
 $Ng = 109.41$
 $c = 0.00 \text{ psf}$
 $q = 219.10 \text{ psf}$

Calculate Ultimate Bearing, Qult (seismic)	Qult = 31483.20 psf
eccentricity (e)	e = 1.925
Equivalent Footing Width, $B' = L - 2e + e $ pad	$B' = 4 \text{ ft}$
Bearing Pressure = $sumVs / B'$	sigma = 3799 psf
Factor of Safety for Bearing = $Qult / \text{Bearing}$	FoS = 8.29

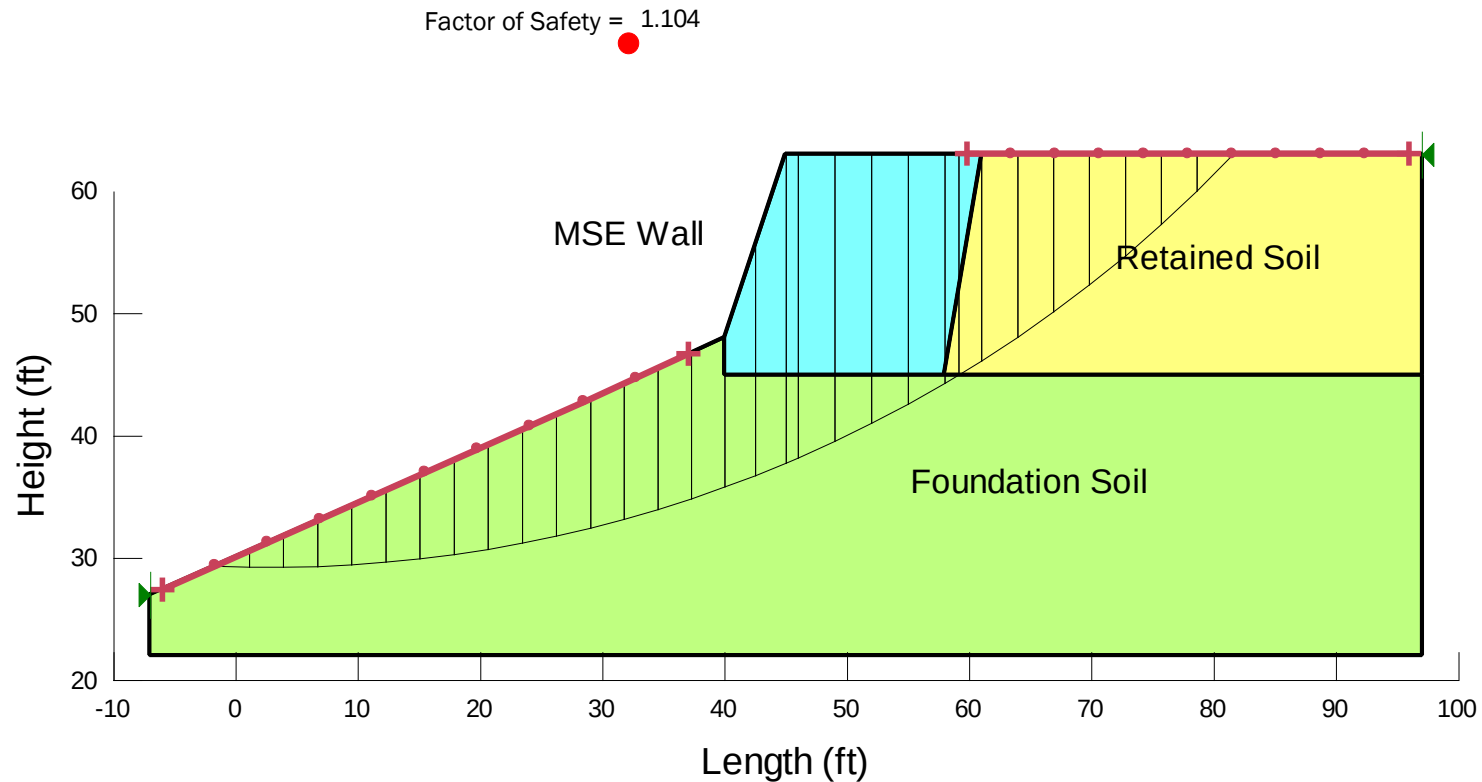


Name: Retained Soil	Unit Weight: 125 pcf	Cohesion': 0 psf	Phi': 34 °
Name: Foundation Soil	Unit Weight: 125 pcf	Cohesion': 0 psf	Phi': 36 °
Name: Reinforced Soil	Unit Weight: 125 pcf	Cohesion': 2,000 psf	Phi': 34 °

18-Foot Tall MSE Wall Section Static Slope Stability

King County Housing Authority
Wind Rose Development
King County, Washington

GEOENGINEERS 

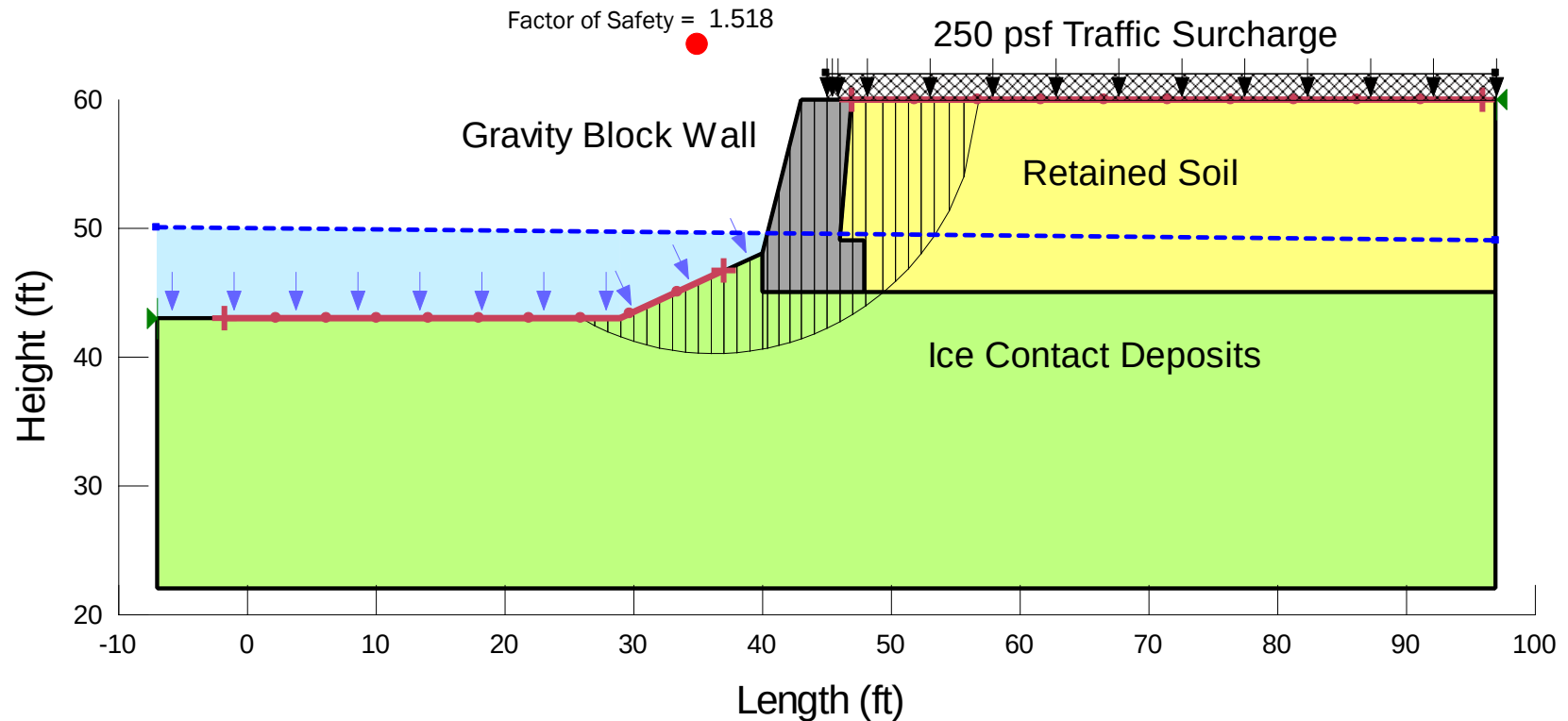


Name: Retained Soil	Unit Weight: 125 pcf	Cohesion': 0 psf	Phi': 34 °
Name: Foundation Soil	Unit Weight: 125 pcf	Cohesion': 0 psf	Phi': 36 °
Name: Reinforced Soil	Unit Weight: 125 pcf	Cohesion': 2,000 psf	Phi': 34 °

18-Foot Tall MSE Wall Section Seismic Slope Stability

King County Housing Authority
Wind Rose Development
King County, Washington

GEOENGINEERS 

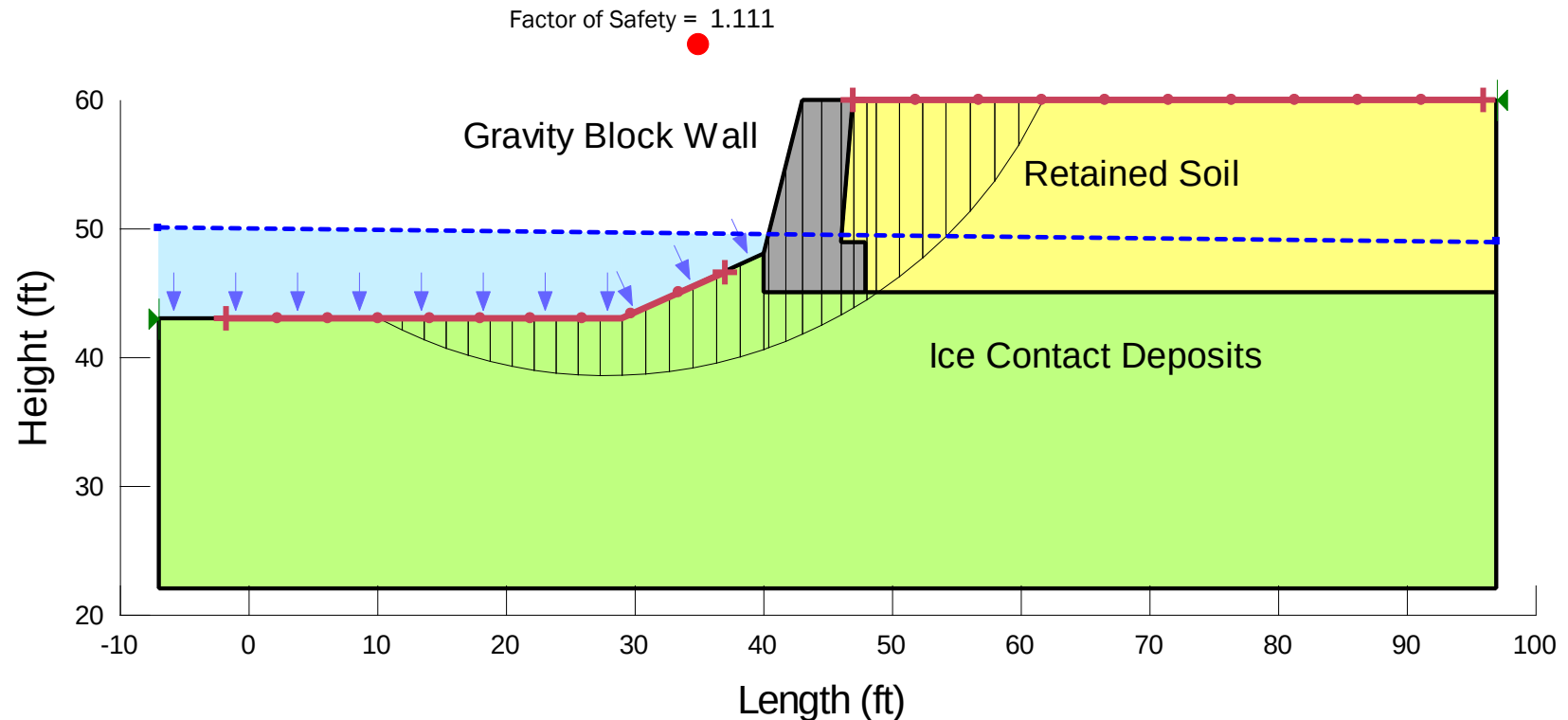


Name: Retained Soil Unit Weight: 125 pcf Cohesion': 0 psf Phi': 36 °
 Name: Ice Contact Deposits Unit Weight: 130 pcf Cohesion': 0 psf Phi': 38 °
 Name: Block Wall Unit Weight: 150 pcf

15-Foot Tall Gravity Block Wall Section Static Slope Stability

King County Housing Authority
Wind Rose Development
King County, Washington

GEOENGINEERS 



Name: Retained Soil Unit Weight: 125 pcf Cohesion': 0 psf Phi': 36 °
 Name: Ice Contact Deposits Unit Weight: 130 pcf Cohesion': 0 psf Phi': 38 °
 Name: Block Wall Unit Weight: 150 pcf

15-Foot Tall Gravity Block Wall Section Seismic Slope Stability

King County Housing Authority
 Wind Rose Development
 King County, Washington

GEOENGINEERS

Note: A horizontal pseudo-static seismic coefficient of 0.21g was included to model the design-level earthquake event.

Reference: GEO-SLOPE International, Ltd., GeoStudio 2012 Version 8.0

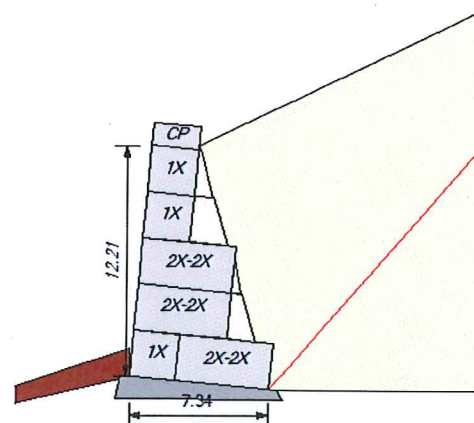


UltraWall

UltraWall v3.3 Build 14226

Project: Wind Rose Pond
 Location: Seattle, WA
 Designer: AJH
 Date: 04/09/2015
 Section: 12.5 wall 2:1 Slope
 Design Method: NCMA_09_3rd_Ed
 Design Unit: Ultrablock

Seismic Acc: 0.210



SOIL PARAMETERS ϕ coh γ
 Retained Soil: 34 deg 0 psf 125 pcf
 Foundation Soil: 36 deg 0 psf 63 pcf
 Leveling Pad: Crushed Stone

GEOMETRY

Design Height:	12.50 ft	Live Load:	0 psf
Wall Batter/Tilt:	0.00/ 5.40 deg	Live Load Offset:	0.00 ft
Embedment:	1.50 ft	Live Load Width:	0 ft
Leveling Pad Depth:	0.50 ft	Dead Load:	0 psf
Slope Angle:	26.0 deg	Dead Load Offset:	0.0 ft
Slope Length:	20.0 ft	Dead Load Width:	0 ft
Slope Toe Offset:	0.0 ft	Leveling Pad Width:	8.38 ft
		Toe Slope Angle:	18.20
		Toe Slope Length:	15.00
		Toe Slope Bench:	0.00

FACTORS OF SAFETY (Static / Seismic)

Sliding:	1.50 / 1.125	Overtaking:	1.50 / 1.125
Bearing:	2.00 / 1.5		

RESULTS (Static / Seismic)

FoS Sliding:	2.00 (lvlpd) / 1.49	FoS Overtaking:	2.28 / 1.65
Bearing:	2568.94 / 3890.50	FoS Bearing:	5.53 / 3.65

Name	Elev.	ka	kae	Pa	Pae	Pir	- PaC	FSsl	FoS OT	siesFSsl	FoS SeisOT
CP	12.24	0.334	0.474	1	1	34	0	100.00	100.00	100.00	100.00
1X	9.79	0.334	0.474	147	209	101	0	100.00	13.58	59.78	4.55
1X	7.34	0.334	0.474	543	772	169	0	35.30	3.45	19.91	1.56
2X-2X	4.89	0.655	0.899	2480	3404	304	0	9.81	3.24	6.20	1.92
2X-2X	2.45	0.539	0.742	3522	4850	440	0	6.43	2.28	4.04	1.34
1X-2X-2X	0.00	0.661	0.838	6882	8725	642	0	2.00[2.13]	2.48	1.49[1.66]	1.65



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 1.499999999999999, 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 (1.4999999999 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.



DESIGN DATA

TARGET DESIGN VALUES (Factors of Safety - Static / Seismic)

Minimum Factor of Safety for the sliding along the base	FSsl = 1.50 / 1.125
Minimum Factor of Safety for overturning about the toe	FSot = 1.50 / 1.125
Minimum Factor of Safety for bearing (foundation shear failure)	FSbr = 2.00 / 3.000
Seismic factors of safety are 75% of the static values.	

MINIMUM DESIGN REQUIREMENTS

Minimum embedment depth	Min_emb = 1.50 ft
-------------------------	-------------------

INPUT DATA

Geometry

Wall Geometry

Design Height, top of leveling pad to finished grade at top of wall	H = 12.21 ft
Embedment, measured from top of leveling pad to finished grade	emb = 1.50 ft
Leveling Pad Depth	LP Thickness = 0.50 ft
Face Batter, measured from vertical	i = 0.00 deg

Slope Geometry

Slope Angle, measured from horizontal	β = 26.00 deg
Slope toe offset, measured from back of the face unit	STL_offset = 0.00 ft
Slope Length, measured from back of wall facing	SL_Length = 20.00 ft

NOTE: If the slope toe is offset or the slope breaks within three times the wall height, a Coulomb Trial Wedge method of analysis is used.

Surcharge Loading

Live Load, assumed transient loading (e.g. traffic)	LL = 0.00 psf
Live Load Offset, measured from back face of wall	LL_offset = 0.00 ft
Live Load Width, assumed strip loading	LL_width = 0.00 ft
Dead Load, assumed permanent loading (e.g. buildings)	DL = 0.00 psf
Dead Load Offset, measured from back face of wall	DL_offset = 0.00 ft
Dead Load Width, assumed strip loading	DL_width = 0.00 ft

Soil Parameters

Retained Zone

Angle of Internal Friction	ϕ = 34.00 deg
Cohesion	coh = 0.00 psf
Moist Unit Weight	gamma = 125.00 pcf

Foundation

Angle of Internal Friction	ϕ = 36.00 deg
Cohesion	coh = 0.00 psf
Moist Unit Weight	gamma = 62.60 pcf



RETAINING WALL UNITS

STRUCTURAL PROPERTIES:

N is the normal force [or factored normal load] on the base unit

The default leveling pad to base unit shear is $0.8 \tan(\phi)$ [AASHTO 10.6.3.4] or

may be the manufacturer supplied data. ϕ is assumed to be 40 degrees for a stone leveling pad.

The shear equations are setup as $N(\tan \phi) + \text{Intercept}$

Unit Designation: Cap

Unit Dimensions:

Height = 1.23 ft

Width = 4.92 ft

Weight = 423 lbs

Unit to Unit Shear

$$\tau = N \tan(0.00) + 17796.00 \text{ ppf}$$

Depth = 2.46 ft

Density = 140.00 pcf

Unit to Leveling Pad Shear

$$\tau = N \tan(34.00) + 0.00 \text{ ppf}$$

Unit Designation: Full

Unit Dimensions:

Height = 2.46 ft

Width = 4.92 ft

Weight = 846 lbs

Unit to Unit Shear

$$\tau = N \tan(0.00) + 17796.00 \text{ ppf}$$

Depth = 2.46 ft

Density = 140.00 pcf

Unit to Leveling Pad Shear

$$\tau = N \tan(33.80) + 0.00 \text{ ppf}$$



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	2.39	1009.33	0.63	3.62	2.27
2	846.08	2.15	1822.93	133.63	3.63	484.70
3	846.08	1.92	1627.19	371.34	3.93	1460.89
4	1692.15	2.92	4942.83	45.77	5.46	249.78
5	1692.15	2.69	4551.35	245.10	5.66	1386.16
6	2538.23	3.69	9359.72			

Block Weight (Force v) = 8038 ppf

X-Arm = 2.90 ft

Soils Block Weight (Force v) = 796 ppf

X-Arm = 4.66 ft

Active Earth Pressure $P_a = 6882$ ppf

P_{a_h} (Force H) = $P_a \cos(\text{batter} + \delta) = 6882 \times \cos(16.1 + 25.5) = 5149$ ppf

Y-Arm = 4.30 ft

P_{a_v} (Force V) = $P_a \sin(\text{batter} + \delta) = 6882 \times \sin(16.1 + 25.5) = 4567$ ppf

X-Arm = 6.10 ft



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)
Coefficient of active earth pressure

$\delta = 25.5$ deg
 $k_a = 0.661$

External failure plane

Effective Angle from horizontal

Coefficient of passive earth pressure: $k_p = (1 + \sin(\phi)) / (1 - \sin(\phi))$

$\rho = 49$ deg
Eff. Angle = 73.93 deg
 $k_p = 3.85$

$$K_a := \frac{\cos(\phi_1 + i)^2}{\cos(i)^2 \cdot \cos(\delta_1 - i) \left(1 + \frac{\sin(\phi_1 + \delta_1) \cdot \sin(\phi_1 - \beta)}{\cos(\delta_1 - i) \cdot \cos(i + \beta)} \right)^2}$$

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 Surcharge Paqd

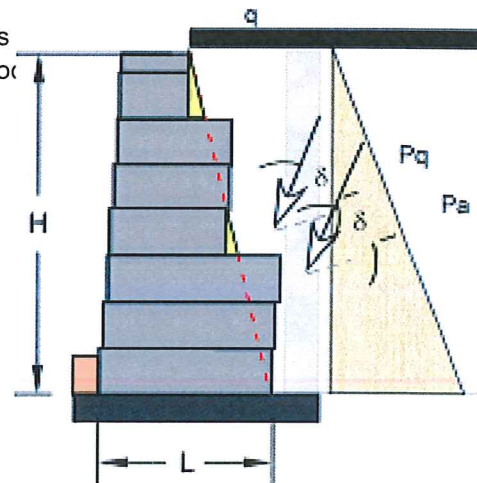
FORCES AND MOMENTS

The program resolves all the geometry into simple geometric shapes 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	8038	--	2.90	--	--	23313
Soil Wedge(W2)	1.00	796	--	4.66	--	--	3712
LvlPad(W18)	1.00	891	--	--	--	--	--
Pa_h	1.00	--	5149	--	4.30	22152	--
Pa_v	1.00	4567	--	6.10	--	--	27856
Sum V / H	1.00	13401	5149		Sum Mom	22152	54881

Note: live load forces and moments are not included in SumV or Mr as live loads are not included as resisting forces.





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.

$$\text{Forces Resisting sliding} = W1 + W2 + P_{av}$$

$$8038 + 796 + 4567$$

$$N = 13401 \text{ ppf}$$

$$\text{Resisting force at pad} = (N \tan(\text{slope}) + \text{intercept} \times L)$$

$$13401 \times \tan(33.8) + 0.0 \times 7.4$$

where L is the base block width

$$R_{f1} = 10256$$

Friction angle is the lesser of the leveling pad and F_{nd}

$N1$ includes N (the leveling pad) + leveling pad (LP)

$$13401 + 891$$

$$\phi = 36.00 \text{ deg}$$

$$N1 = 14292 \text{ ppf}$$

Passive resistance is calculated using $k_p = (1 + \sin(36)) / (1 - \sin(36))$

$$k_p = 3.85$$

Force at top of resisting trapezoid, $d1 = 1.50$

$$F_{p1} = 361.69 \text{ ppf}$$

Force at base of resisting trapezoid, $d2 = 2.70$

$$F_{p2} = 650.35 \text{ ppf}$$

Depth of trapezoid

$$\text{depth} = 1.20 \text{ ft}$$

$$P_p = [(F_{p1} + F_{p2}) / 2] \times \text{depth}$$

$$P_p = 605.78$$

Resisting force at $f_{nd} = (N1 \tan(\phi) + c L) + P_p$

$$14292 \times \tan(36) + 0 \times 7.6 + 606$$

$$R_{f2} = 10990$$

where LP = l_v pad thickness * 130pcf * ($L + l_v$ pad thickness/2)

Driving force is the horizontal component of

P_{ah}

$$5149$$

$$D_f = 5149$$

$$F_{ssl} = R_{f1} / D_f + R_{f2} / D_f$$

$$F_{ssl} = 2.00 / 2.13$$



OVERTURNING ABOUT THE TOE

Overturning 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). For concrete leveling pads eccentricity is checked at the base of the pad.

Moments resisting eccentricity = $M1 + MPav$
 $23313 + 3712 + 27856$

$Mr = 54881 \text{ ft-lbs}$

Moments causing eccentricity = $MPah$
 22152

$Mo = 22152 \text{ ft-lbs}$

$e = L/2 - (Mr - Mo) / N1$
 $e = 7.38/2 - (54881 - 22152) / 14292$

$e = 1.25$
 $e/L = 0.17$

$FSot = Mr / Mo$
 $FSot = 54881 / 22152$

$FSot = 2.48$



ECCENTRICITY AND BEARING

Eccentricity is the calculation of the distance of the resultant away from the centroid of mass. In wall design the eccentricity is used to calculate an effective footing width.

Calculation of Eccentricity

$$\text{SumV} = (W1 + W2 + Pa_v)$$

$$e = L/2 - (\text{SumMr} - \text{SumMo})/(\text{SumV})$$

$$e = 7.38/2 - (32729 / 13400.82)$$

$$e = 1.245 \text{ ft}$$

Calculation of Bearing Pressures

$$Q_{ult} = c * N_c + q * N_q + 0.5 * \gamma * (B') * N_g$$

where:

$$N_c = 50.59$$

$$N_q = 37.75$$

$$N_g = 56.31$$

$$c = 0.00 \text{ psf}$$

$$q = 125.20 \text{ psf}$$

$$B' = B - 2e + \text{lpad} = 5.38 \text{ ft}$$

$$\text{Gamma} = 63 \text{ pcf}$$

Calculate Ultimate Bearing, Q_{ult}

Bearing Pressure = $(\text{SumVert} / B') + (\text{LP width} * \text{gamma})$

Calculated Factors of Safety for Bearing

$$Q_{ult} = 14217 \text{ psf}$$

$$\text{sigma} = 2568.94 \text{ psf}$$

$$Q_{ult}/\text{sigma} = 5.53$$



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
Horizontal Acceleration [$k_h = A/2$]
Vertical Acceleration

$A = 0.210$
 $k_h = 0.080$
 $k_v = 0.000$

INERTIA FORCES OF THE STRUCTURE

Face (P_{if}) = $(W_1) \cdot k_h = 8037.73 \cdot 0.080$

$P_{if} = 642.48$ ppf

SEISMIC THRUST

K_{ae}
 $D_{Kae} = K_{ae} - K_a = (0.838 - 0.661)$
 $P_{ae} = 0.5 \cdot \gamma \cdot (H)^2 \cdot D_{Kae}$
 $P_{ae_h} = P_{ae} \cdot \cos(\delta)$
 $P_{ae_v} = P_{ae} \cdot \sin(\delta)$

$K_{ae} = 0.838$
 $D_{Kae} = 0.177$
 $P_{ae} = 1843.19$ ppf
 $P_{ae_h} = 1378.94$ ppf
 $P_{ae_v} = 1223.05$ ppf

TABLE OF RESULTS FOR SEISMIC REACTIONS

Name	Force (V)	Force (H)	X-len	Y-len	Mo	Mr
Face Blocks(W_1)	8037.726	--	2.9	--	--	23313.35
Face Soil(W_2)	796.47	--	4.66	--	--	3711.65
P_{a_h}	--	5148.694	--	4.302	22151.78	--
P_{a_v}	4566.625	--	6.1	--	--	27855.83
P_{if}	--	642.479	--	7.744	4975.57	--
P_{ae_h}	--	1378.944	--	7.744	10679.0	--
P_{ae_v}	1223.052	--	6.1	--	--	7460.46



SEISMIC SLIDING

The target factor of safety for seismic is 75% of the static value. Live loads are ignored in the analyses based on the basic premise that the probability of the maximum acceleration occurring at the exact same instant as the maximum live load is small.

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 is $W1 + W2 + Pav + Paev$

$$\text{SumVs} = 14624$$

Resisting force = $\text{SumVs} * \tan(\phi) + \text{intercept} * L$

$$14623.87 + \tan(33.8) + 0.00 * 7.38$$

$$\text{FRe} = 10664 \text{ lbf}$$

Driving force = $Pa_h + Pae_h + Pif$

$$5149 + 1379 + 642$$

$$\text{FDr} = 7170 \text{ lbf}$$

FOS = FRe / FDr [leveling pad / foundation]

$$\text{FoS} = 1.49 / 1.66$$

SEISMIC OVERTURNING

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

Resisting Moment = $M1 + M2 + MPav + MPaev$

$$\text{SumMrS} = 62341 \text{ ft lbf}$$

Driving Moment = $MPah + MPaeh + MPif$

$$\text{SumMoS} = 37806.35 \text{ ft lbf}$$

Factor of Safety = $\text{SumMrS} / \text{SumMoS}$

$$\text{FoS} = 1.65$$

SEISMIC BEARING

Bearing is the ability of the foundation to support the mass of the structure.

$$\text{Qult} = c * Nc + q * Nq + 0.5 * \gamma * (B') * Ng$$

where:

$$Nc = 50.59$$

$$Nq = 37.75$$

$$Ng = 56.31$$

$$c = 0.00 \text{ psf}$$

$$q = 125.20 \text{ psf}$$

Calculate Ultimate Bearing, Qult (seismic)
eccentricity (e)

$$\text{Qult} = 14217.16 \text{ psf}$$

$$e = 2.010$$

Equivalent Footing Width, $B' = L - 2e + |v|$ pad

$$B' = 4 \text{ ft}$$

Bearing Pressure = sumVs / B'

$$\sigma = 3891 \text{ psf}$$

Factor of Safety for Bearing = $\text{Qult} / \text{Bearing}$

$$\text{FoS} = 3.65$$

APPENDIX E

Report Limitations and Guidelines for Use

APPENDIX E

REPORT LIMITATIONS AND GUIDELINES FOR USE¹

This appendix provides information to help you manage your risks with respect to the use of this report.

Geotechnical Services Are Performed For Specific Purposes, Persons and Projects

This report has been prepared for the exclusive use of King County Housing Authority (KCHA) and other project team members for the planned Wind Rose Neighborhood Development project. This report is not intended for use by others, and the information contained herein is not applicable to other sites.

GeoEngineers structures our services to meet the specific needs of our clients. For example, a geotechnical or geologic study conducted for a civil engineer or architect may not fulfill the needs of a construction contractor or even another civil engineer or architect that are involved in the same project. Because each geotechnical or geologic study is unique, each geotechnical engineering or geologic report is unique, prepared solely for the specific client and project site. Our report is prepared for the exclusive use of our Client. No other party may rely on the product of our services unless we agree in advance to such reliance in writing. This is to provide our firm with reasonable protection against open-ended liability claims by third parties with whom there would otherwise be no contractual limits to their actions. Within the limitations of scope, schedule and budget, our services have been executed in accordance with Additional Authorization #8 initiated by KCHA and dated January 20, 2016, and generally accepted geotechnical practices in this area at the time this report was prepared. This report should not be applied for any purpose or project except the one originally contemplated.

A Geotechnical Engineering or Geologic Report Is Based On a Unique Set of Project-Specific Factors

This report has been prepared for the planned Wind Rose Neighborhood Development project in King County, Washington. GeoEngineers considered a number of unique, project-specific factors when establishing the scope of services for this project and report. Unless GeoEngineers specifically indicates otherwise, do not rely on this report if it was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

¹ Developed based on material provided by GBA, GeoProfessional Business Association; www.geoprofessional.org.

For example, changes that can affect the applicability of this report include those that affect:

- the function of the proposed structure;
- elevation, configuration, location, orientation or weight of the proposed structure;
- composition of the design team; or
- project ownership.

If important changes are made after the date of this report, GeoEngineers should be given the opportunity to review our interpretations and recommendations and provide written modifications or confirmation, as appropriate.

Subsurface Conditions Can Change

This geotechnical or geologic report is based on conditions that existed at the time the study was performed. The findings and conclusions of this report may be affected by the passage of time, by manmade events such as construction on or adjacent to the site, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations. Always contact GeoEngineers before applying a report to determine if it remains applicable.

Most Geotechnical and Geologic Findings are Professional Opinions

Our interpretations of subsurface conditions are based on field observations from widely spaced sampling locations at the site. Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoEngineers reviewed field and laboratory data and then applied our professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ, sometimes significantly, from those indicated in this report. Our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions.

Geotechnical Engineering Report Recommendations are Not Final

Do not over-rely on the preliminary construction recommendations included in this report. These recommendations are not final, because they were developed principally from GeoEngineers' professional judgment and opinion. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for this report's recommendations if we do not perform construction observation.

Sufficient monitoring, testing and consultation by GeoEngineers should be provided during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether or not earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective method of managing the risks associated with unanticipated conditions.

A Geotechnical Engineering or Geologic Report Could Be Subject to Misinterpretation

Misinterpretation of this report by other design team members can result in costly problems. You could lower that risk by having GeoEngineers confer with appropriate members of the design team after submitting the report. Also retain GeoEngineers to review pertinent elements of the design team's plans

and specifications. Contractors can also misinterpret a geotechnical engineering or geologic report. Reduce that risk by having GeoEngineers participate in pre-bid and preconstruction conferences, and by providing construction observation.

Do not Redraw the Exploration Logs

Geotechnical engineers and geologists prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, but recognize that separating logs from the report can elevate risk.

Give Contractors a Complete Report and Guidance

Some owners and design professionals believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering or geologic report, but preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with GeoEngineers and/or to conduct additional study to obtain the specific types of information they need or prefer. A pre-bid conference can also be valuable. Be sure contractors have sufficient time to perform additional study. Only then might an owner be in a position to give contractors the best information available, while requiring them to at least share the financial responsibilities stemming from unanticipated conditions. Further, a contingency for unanticipated conditions should be included in your project budget and schedule.

Contractors Are Responsible For Site Safety on Their Own Construction Projects

Our geotechnical recommendations are not intended to direct the contractor's procedures, methods, schedule or management of the work site. The contractor is solely responsible for job site safety and for managing construction operations to minimize risks to on-site personnel and to adjacent properties.

Read These Provisions Closely

Some clients, design professionals and contractors may not recognize that the geoscience practices (geotechnical engineering or geology) are far less exact than other engineering and natural science disciplines. This lack of understanding can create unrealistic expectations that could lead to disappointments, claims and disputes. GeoEngineers includes these explanatory "limitations" provisions in our reports to help reduce such risks. Please confer with GeoEngineers if you are unclear how these "Report Limitations and Guidelines for Use" apply to your project or site.

Geotechnical, Geologic and Environmental Reports Should Not Be Interchanged

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

Biological Pollutants

GeoEngineers' Scope of Work specifically excludes the investigation, detection, prevention or assessment of the presence of Biological Pollutants. Accordingly, this report does not include any interpretations, recommendations, findings, or conclusions regarding the detecting, assessing, preventing or abating of Biological Pollutants and no conclusions or inferences should be drawn regarding Biological Pollutants, as they may relate to this project. The term "Biological Pollutants" includes, but is not limited to, molds, fungi, spores, bacteria, and viruses, and/or any of their byproducts.

If Client desires these specialized services, they should be obtained from a consultant who offers services in this specialized field.

Have we delivered World Class Client Service?

Please let us know by visiting [**www.geoengineers.com/feedback**](http://www.geoengineers.com/feedback).

