Wind Rose at Greenbridge (Lot 32, Plat of Windrose) for King County Housing Authority

Technical Information Report

July 2020

GOLDSMITH
LAND DEVELOPMENT SERVICES

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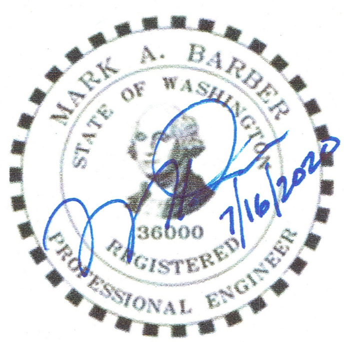
Technical Information Report

Prepared for:
King County Housing Authority

Prepared by:
Goldsmith



July 2020
Job Number: 20067

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FIGURES

APPENDICES

TECHNICAL INFORMATION REPORT (TIR) WORKSHEET

Part 1 PROJECT OWNER AND PROJECT ENGINEER

Project Owner: King County Housing Authority
Phone: (206) 574-1100
Address: 600 Andover Park West
Tukwila, WA 98188
Project Engineer: Mark Barber
Company: Goldsmith Engineering
Phone: 425-462-1080

Part 2 PROJECT LOCATION AND DESCRIPTION

Project Name: Wind Rose at Greenbridge (Lot 32, Plat of Windrose) Site Dev. Permit
DPER Permit #: PRE20-0033
Location Township: Township 23N
Range: 08E. W.M.
Section: NE ¼ SE ¼ Section 61
Site Address: Intersection of 4th Avenue SW and SW Roxbury Street

Part 3 TYPE OF PERMIT APPLICATION

- [x] Landuse (e.g.,Subdivision / Short Subd. / UPD)
- [ ] Building (e.g.,M/F / Commercial / SFR)
- [ ] Clearing and Grading
- [ ] Right-of-Way Use
- [x] Other Site Development Permit

Part 4 OTHER REVIEWS AND PERMITS

- [ ] DFW HPA
- [ ] COE 404
- [ ] DOE Dam Safety
- [ ] FEMA Floodplain
- [ ] COE Wetlands
- [ ] Other ________
- [ ] Shoreline Management
- [ ] Structural Rockery/Vault/_____
- [ ] ESA Section 7

Part 5 PLAN AND REPORT INFORMATION

Technical Information Report

Type of Drainage Review (check one):
- [x] Full
- [ ] Targeted
- [ ] Simplified
- [ ] Large Project
- [ ] Directed

Date (include revision dates): 07/17/2020

Date of Final: ________

Site Improvement Plan (Engr. Plans)

Plan Type (check one):
- [x] Full
- [ ] Modified
- [ ] Simplified

Date (include revision dates): 07/17/2020

Date of Final: ________

Part 6 SWDM ADJUSTMENT APPROVALS

Type (circle one): Standard / Experimental / Blanket

Description: (include conditions in TIR Section 2)

____

____

____

Approved Adjustment No. ______________________ Date of Approval: ______________________

TECHNICAL INFORMATION REPORT (TIR) WORKSHEET

Part 7 MONITORING REQUIREMENTS

Monitoring Required: Yes / No

Start Date: TBD

Completion Date: TBD

Describe: ____

Re: KCSWDM Adjustment No. ____

Part 8 SITE COMMUNITY AND DRAINAGE BASIN

Community Plan : White Center

Special District Overlays: N/A

Drainage Basin: Duwamish River (Hamm Ck)

Stormwater Requirements: Level 3 Flow Control (2016 KCSWDM)

Part 9 ONSITE AND ADJACENT SENSITIVE AREAS

- ☐ River/Stream ____
- ☐ Lake ____
- ☐ Wetlands ____
- ☐ Closed Depression ____
- ☐ Floodplain ____
- ☐ Other ____
- ☐ Steep Slope ____
- ☐ Erosion Hazard ____
- ☐ Landslide Hazard ____
- ☐ Coal Mine Hazard ____
- ☐ Seismic Hazard ____
- ☐ Habitat Protection ____
- ☐ ____

Part 10 SOILS

Soil Type	Slopes	Erosion Potential
Urban-AgD complex	5-12%	low

- ☐ High Groundwater Table (within 5 feet)
- ☐ Other ____
- ☐ Sole Source Aquifer
- ☐ Seeps/Springs
- ☐ Additional Sheets Attached

TECHNICAL INFORMATION REPORT (TIR) WORKSHEET

Part 11 DRAINAGE DESIGN LIMITATIONS

REFERENCE	LIMITATION / SITE CONSTRAINT
☒ Core 2 – Offsite Analysis	Hamm Ck Flooding Requires Level 3 Flow Control
☐ Sensitive/Critical Areas	
☐ SEPA	
☐ LID Infeasibility	
☐ Other	
☐	

☐ Additional Sheets Attached

Part 12 TIR SUMMARY SHEET (provide one TIR Summary Sheet per Threshold Discharge Area)

Threshold Discharge Area:
(name or description)

Core Requirements (all 8 apply):

Discharge at Natural Location	Number of Natural Discharge Locations: 1
Offsite Analysis	Level: 1 / 2 / 3 dated: N/A
Flow Control (include facility summary sheet)	Level: 1 / 2 / 3 or Exemption Number Flow Control BMPs
Conveyance System	Spill containment located at: TBD
Erosion and Sediment Control / Construction Stormwater Pollution Prevention	CSWPP/CESCL/ESC Site Supervisor: TBD Contact Phone: TBD After Hours Phone: TBD
Maintenance and Operation	Responsibility (circle one): Private / Public If Private, Maintenance Log Required: Yes / No
Financial Guarantees and Liability	Provided: Yes / No
Water Quality (include facility summary sheet)	Type (circle one): Basic / Sens. Lake / Enhanced Basic / Bog or Exemption No. Landscape Management Plan: Yes / No

Special Requirements (as applicable): None

Area Specific Drainage Requirements	Type: CDA / SDO / MDP / BP / LMP / Shared Fac. / None Name:
Floodplain/Floodway Delineation	Type (circle one): Major / Minor / Exemption / None 100-year Base Flood Elevation (or range): Datum:
Flood Protection Facilities	Describe: N/A

TECHNICAL INFORMATION REPORT (TIR) WORKSHEET

Part 12 TIR SUMMARY SHEET (provide one TIR Summary Sheet per Threshold Discharge Area)

Source Control (commercial / industrial land use)	Describe land use: Multi-Family / Sr. Assisted Living Housing Describe any structural controls: N/A
Required for all properties with Commercial Activities (A-1), Landscape and Vegetation Management (A-26).	
Oil Control	High-use Site: Yes / No (circled: No) Treatment BMP: ____________ Maintenance Agreement: Yes / No (circled: No) with whom? ____________

Other Drainage Structures

Describe:

Part 13 EROSION AND SEDIMENT CONTROL REQUIREMENTS

MINIMUM ESC REQUIREMENTS DURING CONSTRUCTION	MINIMUM ESC REQUIREMENTS AFTER CONSTRUCTION
■ Clearing Limits	■ Stabilize exposed surfaces
■ Cover Measures	■ Remove and restore Temporary ESC Facilities
■ Perimeter Protection	■ Clean and remove all silt and debris, ensure operation of Permanent Facilities, restore operation of Flow Control BMP Facilities as necessary
■ Traffic Area Stabilization	☐ Flag limits of SAO and open space preservation areas
■ Sediment Retention	☐ Other ____________
■ Surface Water Collection	
☐ Dewatering Control	
■ Dust Control	
■ Flow Control	
■ Protection of Flow Control BMP Facilities (existing and proposed)	
■ Maintain BMPs / Manage Project	

Part 14 STORMWATER FACILITY DESCRIPTIONS (Note: Include Facility Summary and Sketch)

Flow Control	Type/Description	Water Quality	Type/Description
■ Detention	Off-Site Det. Pond	☐ Vegetated Flowpath	
☐ Infiltration		☐ Wetpool	
☐ Regional Facility		☐ Filtration	
☐ Shared Facility		☐ Oil Control	
☐ Flow Control BMPs		☐ Spill Control	
☐ Other		☐ Flow Control BMPs	Off-Site Wetpond
		■ Other	Bioretention Swale

TECHNICAL INFORMATION REPORT (TIR) WORKSHEET

Part 15 EASEMENTS/TRACTS	Part 16 STRUCTURAL ANALYSIS
☐ Drainage Easement	☐ Cast in Place Vault
☐ Covenant	☐ Retaining Wall
☐ Native Growth Protection Covenant	☐ Rockery > 4' High
☐ Tract	☐ Structural on Steep Slope
☐ Other ____________	☐ Other ____________

Part 17 SIGNATURE OF PROFESSIONAL ENGINEER
I, or a civil engineer under my supervision, have visited the site. Actual site conditions as observed were incorporated into this worksheet and the attached Technical Information Report. To the best of my knowledge the information provided here is accurate.
7/16/2020
Signed/Date

1. Project Overview

Introduction

This Technical Information Report (TIR) is prepared on behalf of the King County Housing Authority associated with a submittal of a Site Development Permit application for the *Wind Rose at Greenbridge* (Lot 32, Plat of Windrose) ("Wind Rose at Greenbridge") site. Wind Rose at Greenbridge is the ±1.9 acre Lot 32 of the Plat of Windrose (Recording No. 20190502000861). The site has previously been the subject of the Wind Rose Preliminary Short Subdivision (File #L10S0013) approval and subsequent minor Modifications. The objective of the SDP application is to obtain a new land use approval for the same proposed future development area with virtually the same project conditions, and approval of the same modifications and waivers to code recognizing this as part of the Greenbridge demonstration project, as outlined in the previous Short Plat and Modifications.

The overall stormwater control plan for this project was prepared and approved with the Wind Rose plat per the *Wind Rose (incl. a Division of Greenbridge) Technical Information Report* dated Rev. October 2017 and associated engineering plans (STRV15-0006). The stormwater plan is based on existing information about the site and its downstream drainage systems. This information includes detailed field investigations by Goldsmith, as-built data, drainage reports, drainage complaints and observations by others.

Per discussions with King County Staff, this TIR is submitted to support the review and approval of a new land use application, a Site Development Permit, to be processed as a Type II land use permit. It provides an overview of the existing stormwater control drainage system serving the plat of Windrose, which includes this property - Lot 32.

Under both existing and developed conditions stormwater runoff from the project site is directed to the previously constructed off-site conveyance system and the stormwater flow control and water quality treatment facility in Tract RD-701 in the Windrose plat (STRV15-0006) as shown on the Developed Conditions Basin Plan (Figure 3).

This report provides confirmation that the existing detention / wet-pond stormwater facility, previously permitted and constructed with the plat of Windrose, provides the required level of flow control for future site development assuming 90% impervious surface coverage. This assumption is consistent with the previous design of the off-site stormwater facility and conveyance system that had used a 90% impervious surface coverage for the Wind Rose Parcel at Greenbridge site. More specifically, this TIR also includes additional details of the stormwater plan that are required to verify that 2016 King County Surface Water Design Manual (KCSWDM) stormwater design standards are met for future building permit application, including water quality treatment. It is anticipated that this SDP will carry its own conditions of approvals, TOGETHER with accepted conditions and SPECIFIC requested modification clarifications / exceptions to the previous land use approval contained in the Modification Request submitted with the SDP..

Key features of the Wind Rose at Greenbridge (Lot 32, Plat of Windrose) drainage control plan include:

- The Wind Rose at Greenbridge (Lot 32, Plat of Windrose) development areas are subject to the 2016 KCSWDM stormwater development standards which establishes that the historic forested site condition be used as the existing condition for sizing and assessment of the stormwater facilities

- Stormwater flow control will be provided by the existing off-site Windrose Plat combined detention / wet pond (RD-701). The existing facility has been analyzed to verify that Flood Problem (Level 3) flow control as required by downstream conditions within the Hamm Creek Basin. is provided for the Wind Rose at Greenbridge (Lot 32, Plat of Windrose) site. This analysis is based on the 2016 KCSWDM flow control standards and hydrologic modeling software MGSFlood.
- Under the 2016 KCSWDM the project will provide on-site enhanced treatment per Core Requirement #8.
- On-site Flow control BMPs are required and shall be selected and designed according to the 2016 KCSWDM with future building permit application and engineering plans.

Project Location

The Wind Rose at Greenbridge (Lot 32, Plat of Windrose) project site is Lot 32 of the Plat of Windrose lying in the northeast corner of the overall Greenbridge Development project area within unincorporated King County located between the City of Seattle and the City of Burien. The site is specifically located in Section 6, Township 23 N, Range 4 E, W.M. The site is bounded on the north by SW Roxbury Street coincident with the City of Seattle limits, and to the west by 4th Avenue S.W. The east and south boundaries abut single family portions of the Plat of Windrose. A King County Regional Stormwater Control Facility (White Center Pond) is located on the east boundary of the overall Windrose Plat. This facility is also classified as a Class 2 Wetland and is inventoried by King County as Salmon Creek Wetland 1. Further east of the overall Greenbridge site perimeter is defined by a steep sloped area with an abandoned sand/gravel pit operation. A vicinity map showing the location of the project site is shown on Figure 2.

Project Description

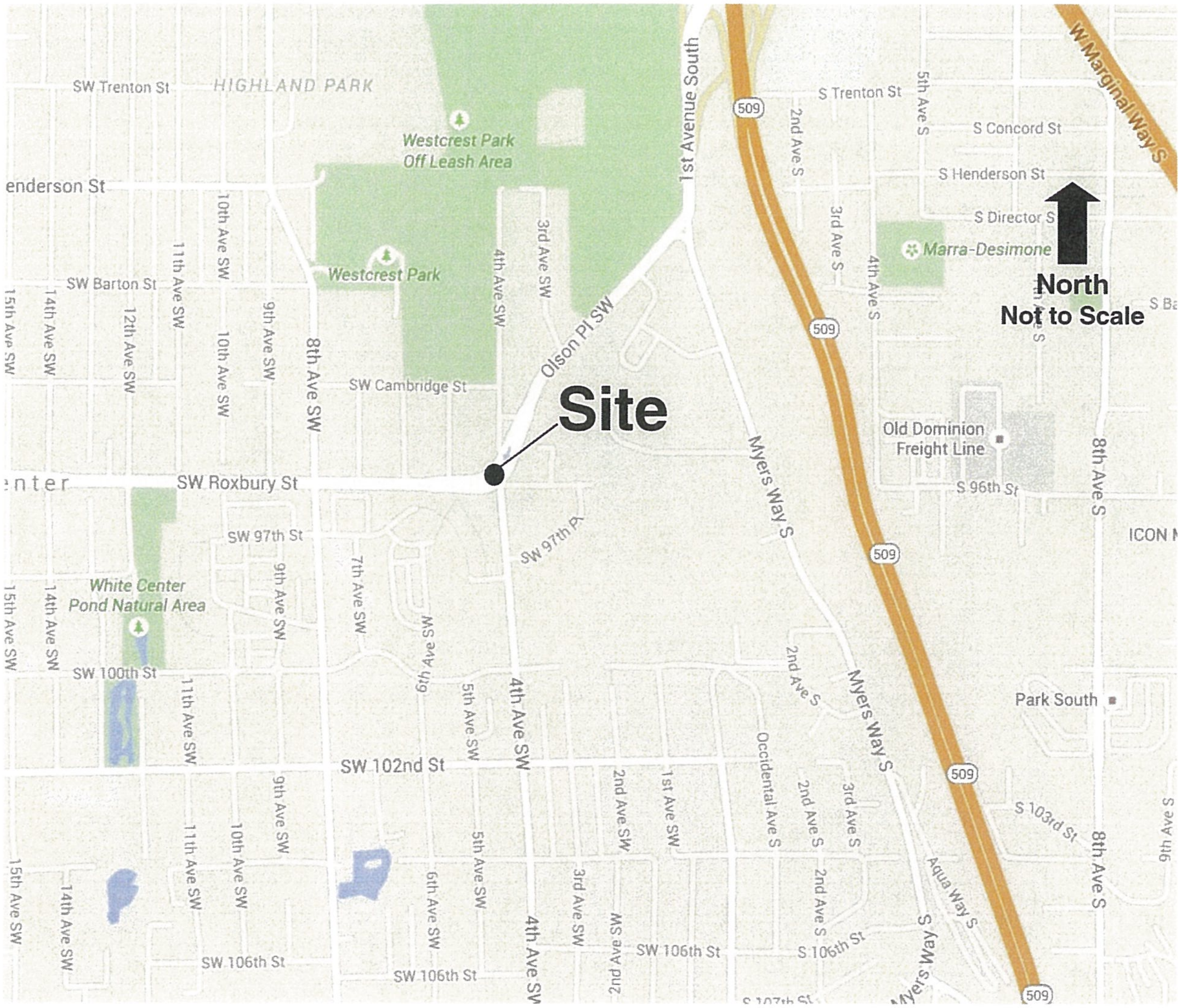
Wind Rose at Greenbridge is Lot 32 of the Plat of Windrose lying in the northeast corner of the overall Greenbridge Development project area. The Wind Rose at Greenbridge (Lot 32, Plat of Windrose) is essentially the same future development project as approved by the previous Wind Rose Preliminary Short Subdivision (File #L10S0013) and subsequent provisions of the Wind Rose Minor Modification #1.

The Site Development application proposed elements include the following features:

- 1.9 ± acre project site (Mixed Use Area) zoned R18,
- Recognized as lying within the Greenbridge Demonstration Project,
- Approval with the Modifications and Waivers as outlined in Attachments A – J of the Wind Rose Preliminary Short Subdivision,
- Density and Dimensions Development Standards as outlined in Attachment K of the Wind Rose Preliminary Short Subdivision as modified by Minor Modification #1 of said Short Plat, confirming the approved density of a Maximum of 24 dwelling units if solely market rate and 80 dwelling units (39 market rate and 41 affordable or low income senior) if meeting income requirements of KCC 21A.55.060.D.3, including the height limitations outlined in Minor Modification #1,
- Acceptance of the SEPA / NEPA documents for Greenbridge as adequate SEPA review for this Site Development Permit consistent with the previous Wind Rose Preliminary Short Subdivision approval,

- Inclusion and clarification of an ADDITIONAL OPTION to modify the building type to allow ALL 80 units as 80% Average Median Income (AMI) affordable housing units,
- Internal circulation provided by a private parking lot and drive isles,
- On-site parking requirements for 40 Parking Stalls is met and 68 parking stalls are proposed. The Applicant requests that an additional option be allowed to provide increased parking, if necessary, under the building. The on-site exterior parking is planned but the additional parking option could be needed.
- Single access point is proposed from SW Roxbury Street between 4th Avenue SW and 2nd Avenue SW – both public rights-of-way modifying Condition 12 of the Wind Rose Preliminary Short Subdivision and elimination of Minor Modification #1 modified condition 1.4,
- Proposed water quality treatment meeting the 2016 KCSWDM standard for enhanced treatment standard will be provided on-site, via bioretention BMPs as permitted under the demonstration ordinance for Greenbridge site development. These BMPs will be permitted with future building permit and engineering plan submittals
- Stormwater control and flow control is proposed to be provided in the existing off site combined detention and wet-pond facility constructed with the Windrose Plat (Tract RD-701) consistent with Minor Modification #1 modified condition 1.3 which provides flow control for Wind Rose at Greenbridge compliant with the 2016 KCSWDM standards for the project site.

No building permit is being sought commensurate with this Site Development Permit. It is anticipated that a future building permit for construction of a multifamily / senior assisted low income housing project would be submitted to King County after approval of the Site Development Permit approval is in place.



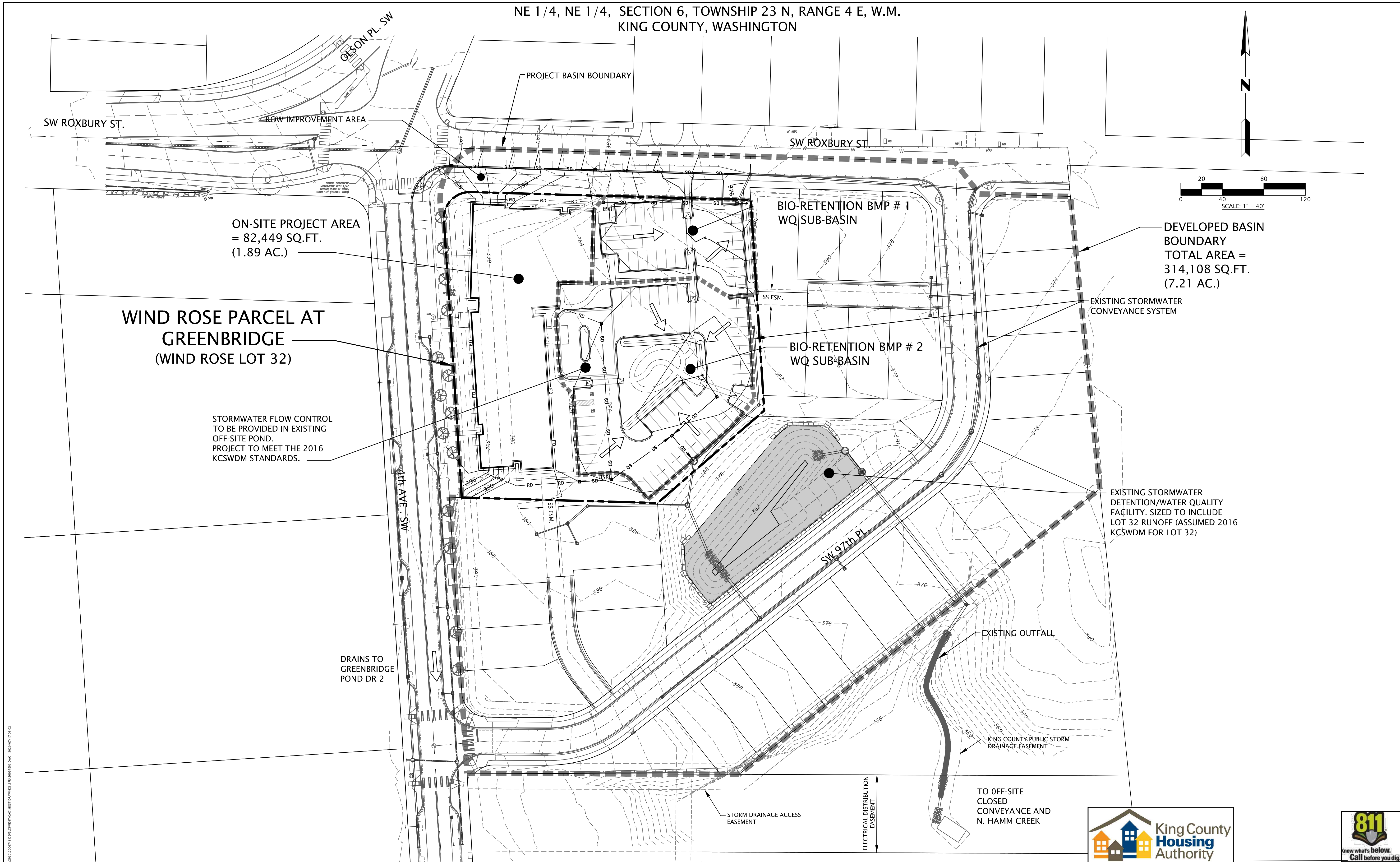
<u>Vicinity Map</u>

4th SW and SW Roxbury, Seattle

FIGURE 2

GOLDSMITH
LAND DEVELOPMENT SERVICES

02075-15 Vicinity Map.docx
June 1, 2015



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PLOTTED: 2020/07/17 08:02 Hwerlinger
DRAWN: HWERLINGER
APPROVED: KGOLDSMITH

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7/17/2020 — KEITH J. GOLDSMITH, STATE OF WASHINGTON, REGISTERED PROFESSIONAL ENGINEER, 24597

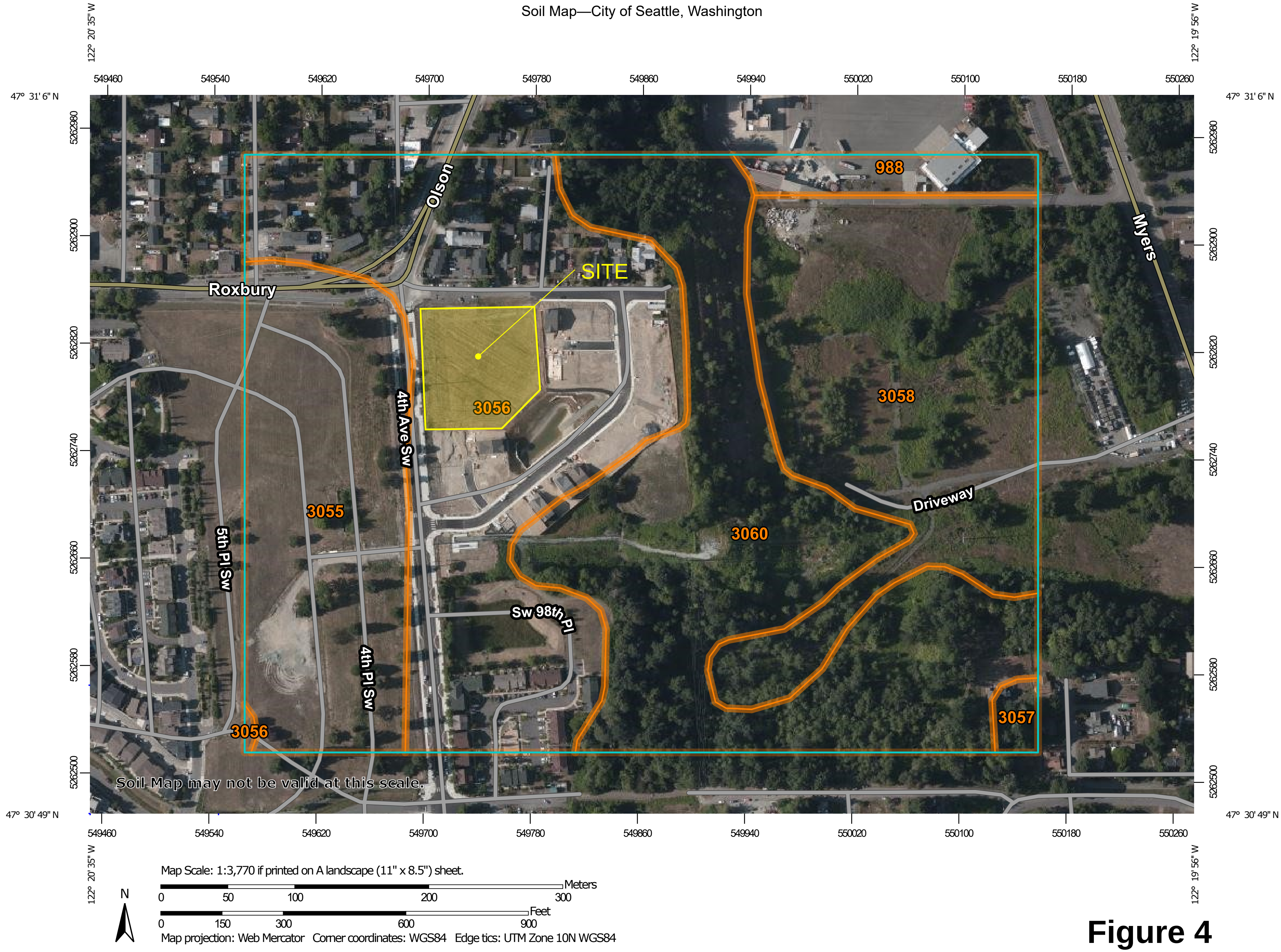
7/17/2020 — MARK A. BARBER, STATE OF WASHINGTON, REGISTERED PROFESSIONAL ENGINEER, 36000

KING COUNTY HOUSING AUTHORITY

DEVELOPED CONDITIONS DRAINAGE PLAN
FOR
WIND ROSE AT GREENBRIDGE
(LOT 32, PLAT OF WINDROSE)

KING COUNTY WASHINGTON

JOB NO. 20067
SHEET
FIG.3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122° 19' 56" W
47° 31' 6" N
47° 30' 49" N
549460
549540
549620
549700
549780
549860
549940
550020
550100
550180
550260
5262980
5262900
5262820
5262740
5262660
5262580
5262500
Olson
Roxbury
SITE
Myers
988
3058
3056
3055
3060
3057
4th Ave Sw
5th Pl Sw
4th Pl Sw
Sw 98th Pl
Driveway
Soil Map may not be valid at this scale.
Map Scale: 1:3,770 if printed on A landscape (11" x 8.5") sheet.
N
Meters
0 50 100 200 300
Feet
0 150 300 600 900
Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 10N WGS84

Figure 4

MAP LEGEND

Area of Interest (AOI)
- Area of Interest (AOI)

Soils
- Soil Map Unit Polygons
- Soil Map Unit Lines
- Soil Map Unit Points

Special Point Features
- Blowout
- Borrow Pit
- Clay Spot
- Closed Depression
- Gravel Pit
- Gravelly Spot
- Landfill
- Lava Flow
- Marsh or swamp
- Mine or Quarry
- Miscellaneous Water
- Perennial Water
- Rock Outcrop
- Saline Spot
- Sandy Spot
- Severely Eroded Spot
- Sinkhole
- Slide or Slip
- Sodic Spot
- Spoil Area
- Stony Spot
- Very Stony Spot
- Wet Spot
- Other
- Special Line Features

Water Features
- Streams and Canals

Transportation
- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

Background
- Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:12,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: City of Seattle, Washington
Survey Area Data: Version 3, Sep 16, 2019

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Jun 29, 2019—Jul 21, 2019

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Figure 4

Map Unit Legend

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
988	Urban land, 0 to 5 percent slopes	1.6	2.5%
3055	Urban land-Alderwood complex, 0 to 5 percent slopes	10.7	16.4%
3056	Urban land-Alderwood complex, 5 to 12 percent slopes	19.5	29.9%
3057	Urban land-Alderwood complex, 12 to 35 percent slopes	0.4	0.7%
3058	Alderwood-Everett-Urban land complex, 0 to 12 percent slopes	14.7	22.5%
3060	Alderwood-Everett-Urban land complex, 35 to 60 percent slopes	18.4	28.1%
Totals for Area of Interest		**65.4**	**100.0%**

Figure 4

2. Conditions and Requirements Summary

This proposal for the Wind Rose at Greenbridge (Lot 32, Plat of Windrose) Site Development Permit (SDP) is a new land use application submitted by the King County Housing Authority for approval. The site has previously been the subject of the Wind Rose Preliminary Short Subdivision (File #L10S0013) approval and subsequent minor Modifications.

The Wind Rose Short Subdivision and Greenbridge Preliminary Plat conditions of approval are cited below. As discussed in Chapter 1, . it is anticipated that this SDP will carry its own conditions of approvals, TOGETHER with accepted conditions and SPECIFIC requested modification clarifications / exceptions to the previous land use approval contained in the Modification Request submitted with the SDP. Therefore, the Wind Rose Short Subdivision conditions are provided for reference. Additionally, the conditions of the Greenbridge Preliminary Plat (L03P0022) have been addressed as they relate to the SDP application

WIND ROSE Short Subdivision (File No. L10S0013)

Wind Rose Short Subdivision L10S0013, received September 30, and the zoning modifications and waivers L10VA001, received September 30, 2010 are GRANTED APPROVAL; subject to the following conditions of approval:

1. KCC Title 19A – Compliance with Final Short Subdivision Requirements

All submittals (building permits, final plats, etc.) shall stand on their own for all requirements:

a. Compliance with all of the Land Segregation provisions of King County Code (KCC) Title 19A.

b. The final short subdivision recording documents must be prepared by a professional land surveyor, licensed in the State of Washington. These documents shall comply with the conditions of approval listed in this letter.

c. The final review process must be completed prior to the recording of the short subdivision or the sale of any lots contained within. The Department of Development and Environmental Services (DDES) strongly recommends that the Final Short Plat review package be submitted to the department at least one year prior to the expiration date of the preliminary approval letter.

d. All persons having an ownership interest in the subject property shall sign on the face of the final short subdivision.

e. All utilities within proposed rights-of-way must be included within a franchise approved by the King County Council prior to final short plat recording.

f. Prior to recording KCC 19A.08.160 requires that the following site work is completed:

1. Drainage best management practices (BMP's) facilities and erosion control measures are consistent with K.C.C. 9.04.090;
2. Water mains and hydrants (if required) are installed and fire flow available;
3. Grading as necessary so that all lots are accessible by passenger vehicle;
4. Specific site improvements are completed that are required and conditioned prior to short plat recording or required to remove any safety hazard.

2. Review Process

A. Preliminary Approval

Per K.C. C. 19A.12.040, preliminary short subdivision approval shall be effective for sixty months. In the alternative, per K.C.C. 19A.12.040, if the application receives preliminary approval prior to December 31, 2011 and the applicant meets the provisions of K.C.C. 19A.12.040B.2.a, b and c, the preliminary subdivision approval shall be effective for eighty-four months.

B. Built Green™**:** A three-star rating under the Built Green™ "Green Communities" Program must be achieved for the combined Wind Rose and Greenbridge project site. This note shall be shown on the face of the final short plat. If a building permit is submitted prior to any final short plat, the building permit application materials shall demonstrate how the project site, as an addition to Greenbridge, achieves a three-star rating under the Built Green™ "Green Communities" Program.

C. The development shall have a minimum of 8 (if low income) or 11 (if market rate) units, a maximum of 80 units, a maximum of 6 lots and an option for a maximum of 10,000 gross square feet of non-residential uses All dimensions of the lots shall be shown on the face of the approved preliminary short plat. Minor revisions to the plat may be approved at the discretion of DDES as described in Condition 6.

3. Site Development Standards

The site development standards specified below apply to all developments within Wind Rose. These standards supersede and modify Title 21A Zoning Code development standards for density and dimensions, design, parking, landscaping and uses. Title 21A development standards which are not specifically modified or waived shall apply. Further modifications to the site development standards may be approved pursuant to Condition 6.

A. Density and Dimensions

The density and dimension standards are provided in Attachment K. A note shall be placed on the final plat requiring conformance with these standards.

B. Design Requirements

The design requirements are provided in Attachment L. A note shall be placed on the final plat requiring conformance with these standards.

C. Landscaping

The landscape requirements are provided in Attachment M. A note shall be placed on the final plat requiring conformance with these standards.

D. Parking and Circulation

The parking and circulation requirements are provided in Attachment N. A note shall be placed on the final plat requiring conformance with these standards.

E. Permitted Uses

The permitted uses are provided in Attachment O. A note shall be placed on the final plat requiring conformance with these standards.

An Authorized Modification Request is submitted with the Site Development Permit application (SDP) requesting confirmation of acceptance and inclusion of the conditions identified above as conditions of approval for the new Site Development Permit land use application for

Wind Rose at Greenbridge (Lot 32, Plat of Windrose). See the Request for Authorization or Administrative Modification under separate cover with the formal SDP Application.

4. Other Development Standards

Except as modified in this approval, all County codes and regulations adopted and in effect on the date of application submittal for the preliminary short plat (September 30, 2010) shall apply to Wind Rose.

An Authorized Modification Request is submitted with the Site Development Permit application (SDP) requesting confirmation of acceptance and inclusion of these conditions, the Wind Rose Administrative Minor Modification #1 conditions and requested modifications contained therein. See the Request for Authorization or Administrative Modification under separate cover with the formal SDP Application.

5. Vesting of Development Standards, Mitigation, and Fees

All development within Wind Rose shall be governed by the development standards in effect on September 30, 2010 and as approved in this permit, notwithstanding any conflicting or different development standards or requirements elsewhere in County code. These standards shall be implemented through plats, binding site plans, building and grading permits and other permits and approvals from the County. During the buildout period, the County shall neither modify or impose new or additional conditions or impact fees beyond those set forth in this permit nor apply subsequently adopted ordinances or other regulations, except as follows:

A. Building permit applications shall be subject to building codes in effect at the time of application for each given building permit

B. Application and review fees for subsequent permits and approvals shall be those fees in effect at the time of future applications.

C. Where King County determines subsequently adopted standards are necessary to address imminent public health and safety hazards or new conditions are imposed to facilitate a major permit modification.

An Authorized Modification Request is submitted with the Site Development Permit application (SDP) requesting confirmation of acceptance and inclusion of these conditions, the Wind Rose Administrative Minor Modification #1 conditions and requested modifications contained therein. See the Request for Authorization or Administrative Modification under separate cover with the formal SDP Application.

6. Flexibility and Modification of Standards

K.C.C. 21A.55.060 (J) acknowledges the possibility that site plan elements or conditions of approval of the preliminary short plat may be requested to be amended or modified. This section provides a framework for distinguishing various types of modifications and the process necessary to review such modifications.

Three categories or levels of preliminary short plat modifications have been identified and are provided below: Authorized Modifications, Minor Modifications and Major Modifications. The Director of DDES, or his/her designee, shall have the authority to review and render decisions on Authorized Modifications and Administrative Minor Modifications. Major modifications shall be treated as new applications.

A. Authorized Modifications:

1. Changes in the location and number of overall dwelling units, provided:
    a. the total number of dwelling units is no greater than 80 (the maximum number approved);
    b. the total number of dwelling units is no less than 8 low income units (the minimum number based on K.C.C. 21A.55.060 density calculations) or no less than 11 market rate units (the minimum numbers based on K.C.C. 21A.12); and
2. Changes in the location and number of lots (i.e. consolidate, alter, reconfigure or relocate lots), provided the total number of lots identified for construction of buildings is not increased above 6.
3. Code modifications submitted in conjunction with the authorized changes listed above.
4. Other amendments or modifications requested by the applicant, which DDES determines to be reasonably consistent with the pre-approved ranges or development standards of the Wind Rose demonstration project.

    The director/designee shall review each requested authorized modification to verify that the modification requested is within the scope of those identified above and to verify that no other regulated feature has been affected by the authorized modification. If these verifications are made, the request will be granted. If these verifications are not made, the request may be considered as a minor or major modification of consideration under those standards provided in the following two sections.

B. Administrative Minor Modifications:

1. Minor changes to the location and design of access. However, changes to the design standards which are not consistent with the provisions of this permit or the King County Road Standards will be subject to approval by the County Road Engineer.
2. Development of the entire site without a final short plat, if no land conveyance is necessary and development is done solely with building permits.
3. Code modifications submitted in conjunction with the authorized changes listed above.
4. The applicant's election to comply with a county standard adopted subsequent to the approval of this project, if the director/designee determines that no interdependency or critical relationship to other development standards exist.
5. Other amendments or modifications to the preliminary plat or preliminary plat conditions which DDES determines to be reasonably consistent with the purpose of the approved uses and development standards for the Wind Rose demonstration project.

    The director/designee may approve, or approve with conditions, the requested minor modification upon determining that the proposed modifications reasonably meet or exceed the protections provided by the original requirement; otherwise, it shall be denied. No separate variance or other revision procedure is required hereunder, except as may be required by the County Road Engineer. The decision shall be provided in writing, and King County shall maintain a cumulative list of all approved administrative minor modifications. The time period for review shall be consistent with the time period established for the underlying permit.

C. Major modifications:

Proposed major modifications shall be reviewed through a new short plat application. For vesting purposes, a major modification is considered to be a new application when:

1. Changes in the number of residential units where the change will result in an increase above the maximum number of units (80).
2. Changes in the number of residential units where the change will result in a decrease below 8 low income housing units or below 11 market rate housing units.
3. Changes in the number of lots where the change will result in an increase above the maximum number of lots (6).
4. Any other change which does not qualify (or was denied) as an administrative minor modification.

D. Proposed major modifications shall be reviewed using the same procedures and requirements as a Type 2 land use decision. For vesting purposes, a major modification is considered to be a new application. However, the change in vesting shall only apply to that aspect of the development approval being proposed for major modification.

An Authorized Modification Request is submitted with the Site Development Permit application (SDP) requesting confirmation of acceptance and inclusion of these conditions, the Wind Rose Administrative Minor Modification #1 conditions and requested modifications contained therein. See the Request for Authorization or Administrative Modification under separate cover with the formal SDP Application.

7. Review Process for Future Code Modifications and Waivers

A. Except as provided in Condition 6 above, the range of proposed future modifications and waivers to develop regulations shall only include the following King County code regulations and related public rules:

1. Drainage review requirements pursuant to K.C.C. Chapter 9.04 and the 2009 Surface Water Design Manual.
2. King County Road Standards pursuant to K.C.C. 14.42 and the 2007 King County Road Design and Construction Standards.
3. Density and Dimension standards established by K.C.C. Chapter 21A.12, except as otherwise specifically provided in this condition.
4. Design Requirements established by K.C.C. Chapter 21A.14, which are not authorized changes.
5. Landscaping and Water Use requirements established by K.C.C. Chapter 21A.16 which are not authorized changes.
6. Parking and Circulation requirements established by K.C.C. Chapter 21A.18 which are not authorized changes.
7. Sign requirements established by K.C.C. chapter 21A.20.
8. Critical Areas requirements established by K.C.C. Chapter 21A.24, provided such modifications and waivers must comply with the requirement of K.C.C. 21A.55.060(D)(8).

9. Uses established by K.C.C. 21A.55.060E, including modifications and waivers of requirements of K.C.C. 21A.08.030, 21A.08.040, 21A.08.050, 21A.08.060, 21A.08.070, 21A.08.080 and 21A.08.100.

B. The procedure for review of future code modifications and waivers shall be as follows:

1. The applicant shall submit a written request for a waiver or modification, together with supporting documentation, which it believes to be a qualified modification or waiver either before or in conjunction with an application for one of the following permits, including implementation approvals (e.g. final short plat approval):
   a. a site development permit,
   b. a binding site plan,
   c. a building permit,
   d. a short subdivision,
   e. a subdivision,
   f. a conditional use permit,
   g. a clearing and grading permit.
2. Except for an applicant's request for a modification or waiver that implements the preliminary plat approval, is in conjunction with a preliminary plat amendment or modification request, or is a new subdivision, modification or waiver, applications shall be Type II decisions. Drainage adjustments and road variances shall be handled per K.C.C. 9.04 and K.C.C. 14.42 respectively. Requests in conjunction with a preliminary short plat amendment or modification shall be reviewed as authorized as minor preliminary short plat amendments or modifications, provided the application shall meet the review standards set forth below.
3. If the reviewing department determines that the request complies with the standards set forth below, the modification or waiver shall be approved.
4. Any appeal regarding a requested modification or waiver shall be consolidated with an appeal of the underlying permit.

8. Review Criteria for Code Modifications and Waivers

A. Proposals to modify or waive development regulations for a development application must be consistent with general health, safety and public welfare standards, and must not violate state or federal law.

. Applications must demonstrate how the proposed project, when considered as a whole with the approved preliminary plat/subdivision modifications and proposed modifications or waivers to the code, will meet all of the criteria listed in this subsection, as compared to development without the modification or waiver, and achieves higher quality urban development; enhances infill, redevelopment and greenfield development; optimizes site utilization; stimulates neighborhood redevelopment; and enhances pedestrian experiences and sense of place and community.

2. Any individual request for a modification or waiver, when considered together with the approved preliminary plat/subdivision modifications, must meet two or more of criteria as follows:

a. Uses the natural site characteristics to protect the natural systems;

b. Addresses stormwater and drainage safety, function, appearance, environmental protection and maintainability based upon sound engineering judgment;

c. Contributes to achievement of a two-star or a three-star rating for the project site under the Built Green™ "Green Communities" program recognized by the Master Builders Association of King and Snohomish Counties; or

d. Where applicable, reduces housing costs for future project residents or tenants without decreasing environmental protection.

3. The criteria of this section supersede other variance, modification or waiver criteria and provisions of K.C.C. Title 9 and Title 21A.

9. Community-Oriented Uses

If non-residential uses are provided, there shall be a maximum of 10,000 square feet of non-residential uses in the project, excluding the parking area for such uses.

10. Geotechnical

a. All grading, building, and development activities shall honor the recommendations presented in the project geotechnical study by GeoEngineers, dated August 18, 2010.

b. Each grading and building permit application shall be accompanied by a review and approval of the development proposal by the project geotechnical engineer. Onsite inspections and approvals of site development during construction activity may be required.

11. Surface Water Management and Control (Title 9 KCC)
2009 King County Surface Water Design Manual (SWDM)

Final short plat approval or building permit approval, as applicable, shall require full compliance with the drainage provisions set forth in King County Code (KCC) 9.04. Compliance may result in reducing the number and/or location of lots as shown on the preliminary approved plat. Preliminary review has identified the following conditions of approval, which represent portions of the drainage requirements. All other applicable requirements in KCC 9.04 and the SWDM must also be satisfied during engineering and final review unless otherwise approved by DDES.

a. Drainage plans and analysis shall comply with the 2009 SWDM. DDES approval of the drainage and roadway plans is required prior to any construction.

b. Offsite swale stabilization and culvert installation are proposed per the Preliminary Drainage Plan received February 11, 2011. These improvements shall be shown on the engineering plans. Appropriate drainage easements shall be submitted for these improvements.

c. Standard plan notes and a construction sequence as specified in the SWDM shall be shown on the engineering plans (Reference Section 7B).

d. As required in Chapter 2 of the drainage manual, a storm water pollution prevention and spill (SWPPS) plan shall be included with the project engineering plans.

e. To implement the required Best Management Practices (BMP) for treatment of storm water, the final engineering plans and technical information report (TIR) shall clearly demonstrate compliance with all applicable design standards. As described in Chapter 5 of the drainage manual, a subdivision project may implement the required BMPs or defer the BMP requirements until future review of building permits. In either case, the final engineering plans shall clearly indicate the applicable BMP standards and requirements for implementation on the recorded plat. Any proposed clearing and grading of the site shall also comply with the soil amendment requirements in KCC 16.82.100.

f. A proposal to implement the required BMPs for development of the subdivision and to receive credit in sizing the flow control facility should be included with the engineering submittal. The engineering plans and technical information report shall provide all required design standards and procedures for implementing the BMPs when applied. During engineering review, the applicant may also choose alternative designs for best management practices as allowed by the SWDM. The final recorded plat shall include covenants, easements, notes, and other details to implement the BMP's for site development when applied

g. Storm water facilities shall be designed using the KCRTS Level 3 Flow Control Standard. Water quality facilities shall also be provided using the Basic protection menu. If runoff control facilities are to be maintained by King County, the runoff control facilities shall be located in a separate tract and/or right-of-way dedicated to King County. If runoff control facilities are to be maintained by a private party or by KCHA, or if portions of the drainage area/facility are used for recreation space in accordance with KCC 21A.14.180 and/or right-of way is privately maintained, a public drainage easement shall be provided. If required, the size of the proposed drainage tracts may have to increase to accommodate the required detention storage volumes and water quality facilities

h. If the storm water BMP's are deferred until building permit review, the following note shall be shown on the final recorded plat:

"Permit applications for buildings or other improvements constructed on lots created by this subdivision must be reviewed by King County for compliance with Best Management Practices (BMP's) and other applicable drainage standards as specified in the SWDM. As determined by King County, the permit applicant for each lot must prepare a drainage site plan with procedures for design and maintenance details, and record a declaration of covenant and grant of easement for implementation of the BMPs."

An Authorized Modification Request is submitted with the Site Development Permit application (SDP) to confirm acceptance requested in the approved Wind Rose Administrative Minor Modification #1, Condition 1.3 to allow the use of a single stormwater control facility for both the Plat of Windrose and Wind Rose at Greenbridge to provide flow control AND modify the Original Wind Rose Short Subdivision Conditions 11.a. and II.g confirming Wind Rose at Greenbridge flow control is compliant with Core Requirement #3 of the 2016 KCSWDM. See the Request for Authorization or Administrative Modification under separate cover with the formal SDP Application.

12. Access Requirements (Title 14 KCC) 2007 King County Road Design and Construction Standards (KCRDCS)

Roadway improvements are required to address access requirements and impacts to existing roads and right-of-way. The extent of improvements (conditioned below) requires submittal of engineering plan and profiles and appropriate review fees. Plans shall be prepared and stamped by a professional engineer licensed in the State of Washington and contain the applicable elements outlined in KCRDCS and/or the 2009

Surface Water Design Manual (see Section 2.2.2). Please note that the applicant should submit the plans a minimum of one year prior to the preliminary approval expiration date.

All construction and upgrading of public and private roads shall be done in accordance with the 2007 KCRDCS established and adopted by Ordinance No. 15753, as amended.

The proposed short subdivision shall comply with the KCRDCS including the following requirements, unless otherwise approved by DDES:

a. The frontage along SW Roxbury Street shall be improved to the urban subcollector street standard (south side) and as necessary to incorporate the proposed channelization and parking plan. These improvements are shown on the Conceptual Traffic Improvement Plan (Signal, Striping, Lighting and Signing) received February 3, 2011.

b. Revise the signalization system at the intersection of 4th Avenue SW and SW Roxbury Street as shown on the Conceptual Traffic Improvement Plan (Signal, Striping, Lighting and Signing) received February 3, 2011. This shall include revisions to the channelization and, potentially, roadway illumination system. Plans for these revisions shall be submitted to the City of Seattle. Copies of the approved plans or other correspondence confirming the proposed revisions shall be provided to KCDOT.

c. Minor reconstruction of 4th Avenue SW urban shoulder improvements (curbs and gutters, sidewalks, and existing driveways) will be required upon the submittal of subsequent permit activity. This may include removal of the existing driveway approaches that do not conform to the new access configuration -- and replacement of any non-conforming driveways with full-width sidewalk and standard vertical curbing with gutters.

d. No more than one direct access to 4th Avenue SW (driveway or private road) will be approved.

Modifications to the above road improvement conditions may be considered by King County pursuant to the variance procedures in KCRDCS 1.12. Any request for a road variance shall be submitted to DDES on the appropriate form and with the minimum fee deposit. Other engineering details that may be shown on the preliminary site plan with the exception of the above may not have been reviewed for compliance with KCRDCS. If differences exist, the final design shall be modified to meet KCRDCS. In addition to the above conditions, right-of-way construction permit is required for any utility work in County right-of-way.

13. The applicant or subsequent owner shall comply with Road Mitigation Payment System (MPS), King County Code 14.75, by paying the required MPS fee and administration fee as determined by King County Department of Transportation. The applicant has an option to either:

a. Pay the MPS fee at final short plat recording, or

b. Pay the MPS fee at the time of building permit issuance.

If the first option is chosen, the fee paid shall be the fee in effect at the time of short plat application and a note shall be placed on the face of the short plat that reads, “All fees required by King County Code 14.75, Mitigation Payment System (MPS) have been paid.”

If the second option is chosen, the fee paid shall be the amount in effect as of the date of the building permit application.

The applicants may request an exemption, through King County Housing and Community Development, for the Roads MPS fee for qualifying low- and moderate-income homes.

14. Site Improvement Inspections, Fees and Financial Guarantees (Title 19A & 27 KCC)

This short plat was conditioned to construct/reconstruct road access/right-of-way improvements and/or drainage facilities. Approved engineering plans, inspection fee and applicable financial guarantees are required prior to either starting construction or recording this short plat. At the time of engineering plan approval, you will be notified of the fee amount that will be required to inspect construction and the amount shall be deposited with DDES and of the financial guarantee amount(s) required prior to scheduling of the pre-construction meeting. Please note that the pre-construction meeting is mandatory prior to the start of any work (including site clearing) or the recording of the short plat.

15. Fire Code (KCC Title 17) - Section 503 of the International Fire Code (IFC)

Preliminary Fire Engineering approval has been granted with the following condition, which shall appear on the final plat:

**** SPRINKLERS REQUIRED ****

Any future residences constructed within this subdivision are required to be sprinkled unless the requirement is removed by the King County Fire Marshal or his/her designee.

16. Zoning Code (KCC 21A)

Density and Dimensions (KCC 21A.12)

All lots and development shall meet the density and dimensions requirements of the R-18 and NB zone classifications, unless otherwise modified or waived in accordance with Attachment K. Minor revisions to the short subdivision, which do not result in substantial changes and/or do not create additional lots, may be approved at the discretion of the Department of Development and Environmental Services.

An Authorized Modification Request is submitted with the Site Development Permit application (SDP) requesting confirmation of acceptance and inclusion of these conditions, the Wind Rose Administrative Minor Modification #1 conditions and requested modifications contained therein. See the Request for Authorization or Administrative Modification under separate cover with the formal SDP Application.

17. Street trees shall be provided as follows (per KCRDCS 5.03 and KCC 21A.16.050). See Attachment H for additional conditions:

A. The applicant shall submit a street tree plan and bond quantity worksheet for review and approval by DDES prior to engineering plan approval (if required), or if engineering plans are not required, at the time of the required pre-construction meeting. As an alternate, the street tree plan and bond quantity worksheet may be submitted for review and approve by DDES prior to building permit approval and issuance.

B. If street trees are proposed along 4th Avenue SW, the street tree plan shall be reviewed by Metro as 4th Avenue SW is on a bus route.

C. The street trees must be installed and inspected, or a performance bond posted (if applicable) prior to recording of the plat or at issuance of the building permit, whichever is first. If a performance bond is posted, the street trees must be installed and inspected within one year of recording of the plat or within three months of final occupancy, whichever is first. At the time of inspection, if the trees are found to be installed per the approved plan, a maintenance bond must be submitted or the performance bond replaced with a maintenance bond, and held for two years. After two years, the maintenance bond may be released after DDES has completed a second inspection and determined that the trees have been kept healthy and thriving.

D. A landscape inspection fee shall also be submitted prior to plat recording or building permit issuance, whichever is first. The inspection fee is subject to change based on the current County fees.

18. Recreation Space

Suitable recreation space shall be provided consistent with the requirements of KCC 21A.14.180 and KCC 21A.14.190 (i.e., sport court[s], children's play equipment, picnic table[s], etc.).

1. A phased detailed recreation space plan (i.e., landscape specs, equipment specs, etc.) consistent with the overall conceptual plan shall be submitted to DDES for review and approval prior to or concurrent with the submittal of the engineering plan or prior to submittal of the first commercial building permit with residential uses, whichever is first.
2. A performance bond, if necessary, for recreation space improvements shall be posted prior to recording of the plat or the approval of a building permit containing recreation space and facilities, whichever is first.

19. In the event that any archaeological objects are uncovered on the site, the applicant shall comply with RCW Chapter 27.53, Archaeological Sites and Resources. Immediate notification and consultation with the State Office of Archaeology and Historical Preservation, King County Office of Cultural Resources and relevant tribes (including the Suquamish, Puyallup and Muckleshoot tribes) is required if discovered materials are prehistoric and/or an archaeological site is present.

20. A homeowners' association or other workable organization shall be established to the satisfaction of DDES which provides for the ownership and continued maintenance of the recreation space.

21. The following conditions have been established under SEPA authority as requirements necessary to mitigate the adverse environmental impacts of this development. These mitigation measures were adopted to eliminate or minimize adverse environmental impacts. The applicant shall demonstrate compliance with these items, where applicable, prior to final plat approval or issuance of a building permit, whichever is first:

a. Mitigate noise levels for units within a 200±-foot arc of the northwest corner of the site by implementing building sound proofing through special construction techniques, or through a modification to site design. (See Appendix F, "Air Quality and Environmental Noise Impact Reviews", prepared by ENVIRON International Corporation) Also, see Section 3.12.6 of the Environmental Assessment document for specific construction recommendations provided by the project Architect, GGLO.

b. Mitigate for identified localized areas of potential or actual soil contamination: "Removal of some localized areas of oil-stained vegetation near the northwestern property line of parcel 0623049296 and supplemental soil sampling on this parcel to confirm that arsenic and/or cadmium concentrations are less than MTCA

Method A cleanup levels". (See Appendix E, "Summary of Existing Conditions Regarding Environmental Assessment Studies", prepared by GeoEngineers, Inc.)

22. Health (KCC 13)

This short plat is exempt from further King County Health Department review. However, if improvements are required by the sewer and/or water purveyor to serve the lots in this short plat, then written documentation shall be provided from said purveyor(s) to verify that the required improvements have been bonded and/or installed, and all necessary easements have been provided, prior to final recording of the short plat.

<u>Other Considerations</u>

A. Preliminary approval of this application does not limit the applicant's responsibility to obtain any required permit or license from the State or other regulatory body. This may include, but not be limited to, obtaining a forest practice permit, an HPA permit, building permits, and other types of entitlements as necessitated by circumstances.

B. Development of the subject property may require registration with the Washington State Department of Licensing, Real Estate Division.

GREENBRIDGE Preliminary Plat (L03P0022)

1. Review Process

A. Preliminary Approval

Per K.C.C. 21A.55.060(J), the preliminary subdivision approval shall be effective for eighty-four months.

Acknowledged; Preliminary Plat approval obtained in 2004. Per Greenbridge Administrative Minor Modification #5 (approved Nov. 13, 2013) the Greenbridge Preliminary Plat approval is valid until July 16, 2016, with a five year extension potential to July 16, 2021.

B. Plat Extensions

Per K.C.C. 21A.55.060 (J), the director may grant a one-time preliminary approval extension for an additional five years, but only if the applicant has shown substantial progress towards development of the demonstration project.

Acknowledged; Preliminary Plat approval obtained in 2004. Per Greenbridge Administrative Minor Modification #5 (approved Nov. 13, 2013) the Greenbridge Preliminary Plat approval is valid until July 16, 2016, with a five year extension potential to July 16, 2021.

C. Built Green™

1. A two-star or three-star rating for the project site under the Built Green™ "Green Communities" Program must be achieved. This note shall be shown on the face of the final plat.

   Condition acknowledged; the note is reflected on the recorded Plat of Windrose (Recording No. 20190502000861) as note 3.5., sheet 5 of 9.

2. Design and construct the block CV4 building to meet a two-star rating under the Built Green™ Program for multifamily construction. This note shall be shown on the face of the final plat.

   Not applicable.

D. The plat shall have a minimum density of 744 units and a maximum of 1,100 units. All dimensions of the lots shall be shown on the face of the approved preliminary plat. Minor revisions to the plat may be approved at the discretion of DDES as described in Condition 5. The R-6 zoned portion (block W13) shall have a maximum of 7 units.

The overall build-out of Greenbridge will comply with the minimum and maximum number of dwelling units as identified.

Greenbridge Administrative Minor Modification #6, and Wind Rose Administrative Minor Modification #1, were approved 11/20/2014 demonstrating compliance with applicable density provisions.

2. Site Development Standards

The site development standards specified below apply to all developments within Greenbridge. Where applicable, these standards supersede and modify Title 21A Zoning Code development standards for density and dimensions, parking, landscaping, and signs. Title 21A development standards which are not specifically modified herein shall continue to apply. Further modifications to the site development standards may be approved pursuant to Conditions 5, 6 and 7.

A. Density and Dimensions
The density and dimension standards are provided in Attachment M to the DDES staff report. A note shall be placed on the final plat requiring conformance with these standards.

B. Design Requirements
The design requirements are provided in Attachment N to the DDES staff report. A note shall be placed on the final plat requiring conformance with these standards.

C. Landscaping
The landscape requirements are provided in Attachment O to the DDES staff report. A note shall be placed on the final plat requiring conformance with these standards.

D. Parking and Circulation
The parking and circulation requirements are provided in Attachment P to the DDES staff report. A note shall be placed on the final plat requiring conformance with these standards.

E. Signs
The sign requirements are provided in Attachment Q to the DDES staff report. A note shall be placed on the final plat requiring conformance with these standards.

F. Permitted Uses
The permitted uses are provided in Attachment R to the DDES staff report. A note shall be placed on the final plat requiring conformance with these standards.

None are applicable to this application.

3. Other Development Standards

Except as modified in this approval, all County codes and regulations adopted and in effect on the date of complete application for the preliminary plat (September 4, 2003) shall apply to Greenbridge.

Condition acknowledged, regarding applicable codes and regulations not specifically described/conditioned per the Greenbridge Hearing Examiner Decision. For Wind Rose at Greenbridge (Lot 32, Plat of Windrose) the site development and future building permit applications for the project will meet the 2016 King County Surface Water Design Manual (KCSWDM) stormwater design standards.

An Authorized Modification Request is submitted with the Site Development Permit application (SDP) requesting confirmation of acceptance and inclusion of

these conditions, the Wind Rose Administrative Minor Modification #1 conditions and requested modifications contained therein. See the Request for Authorization or Administrative Modification under separate cover with the formal SDP Application.

4. Vesting of Development Standards, Mitigation, and Fees

All development within Greenbridge shall be governed by the development standards specifically approved in this permit, notwithstanding any conflicting or different development standards or requirements elsewhere in County code. These standards shall be implemented through plats, binding site plans, building and grading permits and other permits and approvals from the County. During the buildout period, the County shall neither modify nor impose new or additional conditions or impact fees beyond those set forth in this permit, nor apply subsequently adopted ordinances or other regulations, except as follows:

A. Building permit applications shall be subject to building codes in effect at the time of application for each given building permit

B. Application and review fees for subsequent permits and approvals shall be those fees in effect at the time of such future applications.

C. King County may determine that subsequently adopted standards are necessary to address imminent public health and safety hazards, and new conditions may be imposed to facilitate or mitigate a major permit modification.

Condition acknowledged; building codes, application and review fees, and standards necessary to address imminent public health and safety hazards may be imposed as new adoptions occur, beyond those set for in the Greenbridge Preliminary Plat approval.

5. Flexibility and Modification of Standards

K.C.C. 21A.55.060(I) provides that after preliminary approval the applicant may request that site plan elements or conditions of approval be amended or modified. This section provides a framework for distinguishing various types of future modifications and the process necessary to review such modifications.

Three categories or levels of preliminary plat modifications have been identified and are provided below: Authorized Modifications, Minor Modifications and Major Modifications. The Director of DDES, or his/her designee, shall have the authority to review and render decisions on Authorized Modifications and Administrative Minor Modifications. Major modifications shall be treated as new Type 3 applications.

A. Authorized Modifications:

1. Changes in the location and number of overall dwelling units, provided:

a) the total number of dwelling units is no greater than 1,100 (the maximum number approved); and

b) the total number of dwelling units is no less than 744 (the minimum density required).

2. Changes in the location and number of lots (i.e. consolidate, alter, reconfigure or relocate lots), provided that the total number of lots identified for construction of buildings is not increased above 721.

3. Changes in the location and number of dwelling units for the development blocks, provided:

   a) the number of dwelling units does not increase above the maximum number of units proposed for the identified block; and

   b) the number of lots does not increase above the maximum number of lots proposed for the identified block, except that within the Neighborhood Core any number of lots may be allowed if the total number of lots within the development does not exceed 721.

4. Minor changes in the location and size of recreational tracts, recreation facilities or trails for the overall Greenbridge site, provided the total area of improvements proposed in recreational tracts complies with the requirements set forth in K.C.C. 21A.14.180 as calculated based on the number and type of units achieved and documented through the final plat process and no per unit reduction of recreation space occurs in the areas lying east of 4th Avenue Southwest or west of 8th Avenue Southwest.

5. Minor changes in the location of the residential building types (i.e. single-family detached, cottage, townhouse, "over/unders", and apartments) provided that apartments over 3 stories are not permitted within the Residential Area.

6. Changes in the location of buildings and uses within the Neighborhood Core as depicted on the approved preliminary plat Unit Range Plan.

7. Minor changes in lot size, lot configuration and internal road patterns resulting from changes in the density or intensity described above.

8. Other amendments or modifications requested by the applicant that are authorized by KCC 19A.12.030B.

The director/designee shall review each requested modification to verify that it is within the scope of the changes identified above and to verify that no other regulated feature will be altered or impacted by approval of the modification. If these verifications are made, the request will be granted. If these verifications are not made, the request may be considered as a minor or major modification.

Greenbridge Administrative Minor Modification #6, and Wind Rose Administrative Minor Modification #1, were approved 11/20/2014 demonstrating compliance with applicable density provisions.

An Authorized Modification Request is submitted with the Site Development Permit application (SDP) requesting confirmation of acceptance and inclusion of these conditions, the Wind Rose

Administrative Minor Modification #1 conditions and requested modifications contained therein. See the Request for Authorization or Administrative Modification under separate cover with the formal SDP Application.

B. Administrative Minor Modifications:

1. Modifications which convert streets proposed as public to private streets or which convert streets proposed as private to public streets, provided the primary streets serving the project (SW Roxbury Street, 4th Avenue SW, 8th Avenue SW, SW 100th Street and SW 102nd Street) shall not be converted to private streets except as a major modification.

2. Minor changes to the location and design of roads. However, changes to the design standards which are not consistent with the provisions of this permit or the King County Road Standards will be subject to approval by the County Road Engineer.

3. Changes in the number of dwelling units proposed for an identified block, provided the overall number of dwelling units shall not exceed 1,100.

4. Code modifications submitted in conjunction with either an authorized or minor modification as defined herein.

5. The applicant's election to comply with a county standard adopted subsequent to the approval of this project, if the director/designee determines that no interdependency or critical relationship to other development standards exist.

6. Other amendments or modifications to the preliminary plat or preliminary plat conditions which DDES determines to be consistent with the purpose of the approved uses and development standards for the Greenbridge demonstration project and are not major modifications as defined in condition 5C below.

The director/designee may approve, or approve with conditions, a requested minor modification upon determining that the proposed modification reasonably meets or exceeds the protections provided by the original requirement; otherwise, it shall be denied. No separate variance or other revision procedure is required hereunder, except as may be required by the County Road Engineer. The decision shall be provided in writing, and King County shall maintain a cumulative list of all approved administrative minor modifications. The time period for DDES review shall be consistent with the time period established for the underlying permit. Determinations by DDES on administrative minor zoning code modification requests shall be appealable in the manner provided for Type 2 decisions.

Greenbridge Administrative Minor Modification #6, and Wind Rose Administrative Minor Modification #1, were approved 11/20/2014 demonstrating compliance with applicable density provisions.

An Authorized Modification Request is submitted with the Site Development Permit application (SDP) requesting confirmation of

acceptance and inclusion of these conditions, the Wind Rose Administrative Minor Modification #1 conditions and requested modifications contained therein. See the Request for Authorization or Administrative Modification under separate cover with the formal SDP Application.

C. Major modifications:

Proposed major modifications shall be reviewed through the public hearing process. For vesting purposes, a major modification is considered to be a new application. The following are designated as major modifications:

1. Changes in the number of residential units where the change will result in an increase above the approved maximum number of units (1,100)

2. Changes in the number of residential units where the change will result in a decrease below the required minimum number of units (744)

3. Changes in the number of lots where the change will result in an increase above the approved maximum number of lots (721)

4. Changes in the maximum number of residential units for the following blocks: Block W13, Block E10, and the north portion of Block W1.

5. An increase in the neighborhood core area or an alteration of its boundaries

6. A reduction in the ratio of parking spaces required for each unit type as set forth in Attachment P.

7. An increase in the total floor area limit for non-residential uses (100,000 square feet) or the floor area limit for retail uses (25,000 square feet) or an expansion of the list of permitted retail, manufacturing or regional land uses.

8. A reduction in the minimum lot area below 1200 square feet.

9. Any other change which does not qualify as an administrative minor modification.

Greenbridge Administrative Minor Modification #6, and Wind Rose Administrative Minor Modification #1, were approved 11/20/2014 demonstrating compliance with applicable density provisions.

An Authorized Modification Request is submitted with the Site Development Permit application (SDP) requesting confirmation of acceptance and inclusion of these conditions, the Wind Rose Administrative Minor Modification #1 conditions and requested modifications contained therein. See the Request for Authorization or Administrative Modification under separate cover with the formal SDP Application.

D. Proposed major modifications shall be reviewed using the procedures and requirements for a Type 3 land use decision. For vesting purposes, a major modification is considered to be a new application. However, the new vesting date shall only apply to those aspects of the development approval being proposed for major modification.

Acknowledged.

6. Review Process for Future Code Modifications and Waivers

A. Except as provided in Condition 5.B.5 above, the subject matter of proposed future modifications and waivers to development regulations shall include only the following King County code regulations and related public rules:

1. Drainage review requirements pursuant to K.C.C. chapter 9.04 and the Surface Water Design Manual.

2. King County road standards pursuant to K.C.C. 14.42.010 and the county road standards, 1993 update.

3. Density and dimension standards established by K.C.C. chapter 21A.12, except as otherwise specifically provided in this condition.

4. Design requirements established by K.C.C. chapter 21A.14, which are not authorized changes.

5. Landscaping and water use requirements established by K.C.C. chapter 21A.16 which are not authorized changes.

6. Parking and circulation requirements established by K.C.C. chapter 21A.18 which are not authorized changes.

7. Sign requirements established by K.C.C. chapter 21A.20.

8. Sensitive area requirements established by K.C.C. chapter 21A.24, provided such modifications and waivers must comply with the requirement of K.C.C. 21A.55.060(D)(8).

9. Uses established by K.C.C. 21A.55.060E, including modifications and waivers of requirements of K.C.C. 21A.08.030, 21A.08.040, 21A.08.050, 21A.08.060, 21A.08.070, 21A.08.080 and 21A.08.100.

An Authorized Modification Request is submitted with the Site Development Permit application (SDP) requesting confirmation of acceptance and inclusion of these conditions, the Wind Rose Administrative Minor Modification #1 conditions and requested modifications contained therein. See the Request for Authorization or Administrative Modification under separate cover with the formal SDP Application.

B. The procedure for review of future code modifications and waivers shall be as follows:

1. The applicant shall submit a written request for the waiver or modification, together with supporting documentation. The request shall be submitted either before or in conjunction with an application for one of the following permits, including implementation approvals (e.g. final plat approval):

   a) a site development permit,

   b) a binding site plan,

   c) a building permit,

   d) a short subdivision,

   e) a subdivision.

2. Except for an applicant's request for a modification or waiver that accompanies or is designated hereunder a Type 3 permit application, zoning modification or waiver applications shall be Type 2 decisions for appeal purposes. Drainage adjustments and road variances shall be processed pursuant to KCC 9.04 and KCC 14.42 respectively. All modifications or waiver requests shall be reviewed subject to the standards set forth below in condition no. 7. If a request for a modification or waiver is associated with a permit application that requires notice, a public hearing or other administrative processes, the request shall be consolidated with the underlying permit and associated procedural requirements shall apply. If the request is not associated with a permit application that would otherwise require notice or a public hearing, the only components of the Type 2 or 3 process applicable shall be the provisions related to appeals.

3. If the reviewing department determines that the request complies with the standards set forth below, the modification or waiver shall be approved.

4. Any appeal regarding a requested modification or waiver shall be consolidated with any concurrent appeal of the underlying permit.

Acknowledged. An Authorized Modification Request is submitted with the Site Development Permit application (SDP) requesting confirmation of acceptance and inclusion of these conditions, the Wind Rose Administrative Minor Modification #1 conditions and requested modifications contained therein. See the Request for Authorization or Administrative Modification under separate cover with the formal SDP Application.

7. **Review Criteria for Code Modifications and Waivers**

Proposals to modify or waive development regulations for a development application must be consistent with general health, safety and public welfare standards, and must not violate state or federal law.

A. Applications shall demonstrate how the proposed project overall, including all approved preliminary plat/subdivision modifications and any new proposed modifications or waivers to the code, will comply with all the criteria listed in this subsection 7.A in a manner comparable to development without the modification or waiver, including achievement of higher-quality urban development; enhancement of infill, redevelopment and greenfield development; optimization of site utilization; stimulation of neighborhood redevelopment; and enhancement of pedestrian experience and sense of place and community.

Acknowledged.

B. The proposed project overall, including the new proposed waivers and modifications, shall also meet the following performance standards. In addition, each individual request for modification or waiver, when considered together with the approved preliminary plat/subdivision modifications (e.g., modifications approved in Attachment M), must meet at least two of the following criteria:

1) uses the natural site characteristics to protect natural systems;

2) addresses stormwater and drainage safety, function, appearance, environmental protection and maintainability based upon sound engineering judgment;

3) contributes to achievement of a two-star or a three-star rating for the project site under the Built Green™ "Green Communities" program recognized by the Master Builders Association of King and Snohomish Counties; and

4) where applicable, reduces housing costs for future project residents or tenants without decreasing environmental protection.

Acknowledged..

C. The criteria of this section supersede other variance, modification or waiver criteria and the provisions contained in K.C.C. Title 9 and Title 21A.

Acknowledged.

8. Community-Oriented Uses

There shall be a maximum of 100,000 square feet of non-residential buildings in the project, excluding the elementary school on-site. The retail portion shall not exceed 25,000 square feet.

Acknowledged.

9. Submittals

All submittals (building permits, final plats, etc.) shall stand on their own regarding compliance with all requirements. All final plats shall demonstrate the following:

A. Compliance with all platting provisions of Title 19 of the King County Code.

B. All persons having an ownership interest in the subject property shall sign on the face of the final plat a dedication which includes the language set forth in King County Council Motion No. 5952.

Acknowledged. The Windrose Final Plat was recorded on May 2, 2019 (Recording #20190502000861).

10. The applicant must obtain the approval of the King County Fire Protection Engineer certifying the adequacy of the fire hydrant, water main, and fire flow to meet the standards of Chapter 17.08 of the King County Code.

Fire Marshall approval will be obtained with future building permit applications and approval of the required water plans by Seattle Public Utilities.

11. Final plat approval shall require full compliance with the drainage provisions set forth in King County Code 9.04. Compliance may result in reducing the number and/or location of lots as shown on the preliminary approved plat. Preliminary review has identified the following conditions of approval, which represent portions of the drainage requirements. All other applicable requirements in KCC 9.04 and the King County Surface Water Design Manual (KCSWDM) must also be satisfied during engineering and final review.

Acknowledged. The Windrose Final Plat was recorded on May 2, 2019 (Recording #20190502000861).

An Authorized Modification Request is submitted with the Site Development Permit application (SDP) requesting confirmation of acceptance and inclusion of these conditions, the Wind Rose Administrative Minor Modification #1 conditions and requested modifications contained therein. See the Request for Authorization or Administrative Modification under separate cover with the formal SDP Application.

A. Drainage plans and analysis shall comply with the 1998 King County Surface Water Design Manual. DDES approval of the drainage and roadway plans is required prior to any construction.

Acknowledged.

B. Current standard plan notes and ESC notes, as established by DDES Engineering Review shall be shown on the engineering plans.

Standard plan notes / ESC notes / and drainage notes as required will be included in the engineering plans to be submitted in the future with the building permit, as applicable.

C. The following note shall be shown on the final recorded plat:

"All building downspouts, footing drains, and drains from all impervious surfaces such as patios and driveways shall be connected to the permanent storm drain outlet as shown on the approved construction drawings #__________ on file with DDES and/or the Department of Transportation. This plan shall be submitted with the application of any building permit. All connections of the drains must be constructed and approved prior to the final building inspection approval. For those lots that are designated for individual lot infiltration systems, the systems shall be constructed at the time of the building permit and shall comply with the plans on file."

The note is included on the Plat of Windrose, (Recording No. 20190502000861), General Note 1.1, Sheet 5 of 9.

Standard plan notes / ESC notes / and drainage notes as required will be included in the engineering plans to be submitted in the future with the building permit, as applicable.

D. Core Requirement No. 1: Discharge at the Natural Location.

The applicant has received approval for a requested diversion of surface water within the project; see Drainage Adjustment File L03V0066 (Attachment 3). The conditions for adjustment approval shall be satisfied during design and review of the project's engineering plans.

The conditions of this Drainage Adjustment are met by the current design.

E. Core Requirement No. 2: Offsite Analysis

The applicant has provided a Level 1 Offsite analysis that describes the existing drainage conditions and conceptual drainage plan. If the applicant wants to reduce the potential flow control requirement in the Salmon Creek subbasin from Level Two (described below) to modified Level One, a Level 2 Offsite Analysis is required to determine when the existing capacity of the downstream system is exceeded.

Noted; Level Three Flow Control is proposed for the Hamm Creek (Duwamish) Basin.

F. Core Requirement No. 3: Flow Control.

The conceptual drainage plan anticipates that post-developed flows from the Salmon Creek subbasin of the site will match pre-developed conditions. If post-developed flow will exceed the pre-developed runoff, Level Two flow control shall be required for the increase. The applicant can reduce this requirement from Level Two flow control to modified Level One flow control if a Level 2 Offsite Analysis is provided as described in the previous condition. Storm water runoff control for the Hamm Creek/Duwamish River sub-basin shall be provided using the Level Three flow control detention standard outlined in the 1998 KCSWDM, or as approved in Drainage Adjustment, L03V0066.

Noted; Level Three for the Hamm Creek Basin.

The size of the proposed drainage tracts may need to be increased to accommodate the required detention storage volumes and/or water quality facilities. The runoff control facilities shall be located in separate tracts and dedicated to King County unless portions of a drainage tract are also used for required recreation space in accordance with KCC 21A.14.180.

Not applicable.

Sub-basin ML-1 is exempt from flow control requirements based on exemption #4, Peak Flow Exemption for Urban Redevelopment Projects, but it needs to implement flow control Best Management Practices as defined in the exemption criteria.

Not applicable.

Due to the size and proximity of the RD-DR-2A/B drainage facilities to the eastern steep slopes, a supplemental soils report is required for submittal with the engineering plans.

Not applicable.

G. Core Requirement No. 4: Conveyance System.

The outlet pipe from the eastern detention facilities (RD-DR-2A/B) is proposed to convey water over steep slopes before connecting to the existing 30-inch pipe at the eastern site boundary. Due to concerns for potential impacts by drainage discharge onto the steep slopes, storm water shall be conveyed down the steep slopes in an enclosed system constructed of high density polyethylene pipe (e.g. Driscopipe). The pipe shall be placed at a location presenting the least potential for erosion and which minimizes disturbance to natural vegetation. Requirements as specified in Section 4.3.6 of the SWM Manual shall be used for design purposes. In addition, the following specific considerations shall be addressed with the applicant's construction plans:

1. The pipe system shall be located on the ground surface within a King County drainage easement sufficient in width to provide for proper location and maintenance.

2. The method of construction and structural attachment of the system to the ground shall be addressed on the plans. Adequate energy dissipation shall be provided at the connection point to the existing 30-inch pipe that extends east off-site.

3. The detention pond or vault emergency overflow system shall be designed for piped conveyance, rather than open spillways. The overflow structure and conveyance pipe shall be designed to accommodate flows for the 100-year storm underdeveloped site conditions. As described in adjustment L03V0066, justification to daylight emergency overflows in non-steep slope areas on-site (Tract U-2) can be presented during engineering review.

4. A redundant interconnect system shall be provided between the RD-DR-2A and RD-DR-2B facilities of the eastern stormwater facility in case blockage occurs. This redundancy can be eliminated if a separate emergency overflow system is provided for each facility.

   None are applicable to this application.

H. Core Requirement No. 5: Erosion and Sediment Control.

The Temporary Erosion and Sediment Control (TESC) plan shall address the annual phasing of construction that will occur across the site, or a separate TESC plan shall be submitted for each construction phase. DDES shall require the applicant to implement a Temporary Erosion and Sedimentation Control Plan which will not result in an increase in phosphorus loading to the Lake Garrett watershed. The TESC Plan shall be submitted to DDES and approved prior to the commencement of any earthwork.

TESC will be addressed and submitted with the future building permit and engineering plan submittal.

I. Core Requirement No. 6: Maintenance and Operations.

As described in 1998 KCSWDM adjustment L03V0066, King County Water and Land Resource Division shall assume ownership and maintenance responsibilities of all formal stormwater facilities for the Greenbridge project.

Acknowledged; permanent stormwater control facilities / tracts have been conveyed to King County per the Plat of Windrose (Recording No. 20190502000861).

J. Core Requirement No. 8: Water Quality.

The project is required to meet the Basic water quality requirements of the 1998 KCSWDM.

For Wind Rose at Greenbridge (Lot 32, Plat of Windrose) the site development and future building permit applications for the project will meet the 2016 King County Surface Water Design Manual (KCSWDM) stormwater design standards. This standard requires that Enhanced treatment be provided for the project site.

An Authorized Modification Request is submitted with the Site Development Permit application (SDP) requesting confirmation of acceptance and inclusion of these conditions, the Wind Rose Administrative Minor Modification #1 conditions and requested modifications contained therein. See the Request for Authorization or Administrative Modification under separate cover with the formal SDP Application.

K. Special Requirement #4: Source Control.

Because the Greenbridge development project contains commercial and multi-family elements, the project must provide water quality source controls in accordance with the King County Stormwater Pollution Control Manual and King County Code 9.12, where applicable.

Source control will be addressed and submitted with the future building permit and engineering plan submittal.

12. The proposed subdivision shall comply with the 1993 King County Road Standards (KCRS), including the following requirements:

A. The following conditions state the required improvements for on-site roads, except as may be provided in Condition 22:

1. Engineering plans shall be prepared in accordance with the design requirements outlined in the County Road Engineer's conditional approval of Variance L03V0060 shown in Attachment 2 (design criteria included).

   Acknowledged; the design has been prepared based on the approval of Variance L03V0060.

2. Street illumination shall be provided pursuant to the requirements in KCRS 5.05.

   Acknowledged; Street illumination will be included on the future building permit plans, as applicable per the requirements of KCRS 5.05.

3. The proposed road improvements shall address the requirements for road surfacing outlined in KCRS Chapter 4. As noted in Section 4.01F, full width pavement overlay is required where widening existing asphalt unless waived during the inspection process. Pavement designs shall be provided for arterials and commercial access streets as required by KCRS Section 4.03.

   Not applicable to this application.

4. The road classification map shows that the intersection of SW 99th Street and 9th Avenue SW does not comply with the provisions of KCRS Section 2.10.A.1 "Angle of Intersection". Due to the existing site constraints, this angle of intersection may not be able to be revised to comply with the above-noted KCRS requirements. The Applicant shall provide, however, with the submittal of engineering plans, documentation (construction plans) describing whether the requirements of KCRS Section 2.10.A.1 cannot be met as a result of topographic constraints.

   Not applicable to this application.

5. The Applicant shall execute a reimbursement agreement with the King County Department of Transportation to fully cover the Department's cost of the following items:

   a. The manufacture, installation and inspection of the required street signage, which may include street name signs, regulatory signage (including, but limited to "STOP" signs, "Yield" signs, and "No Parking" signs) or other signage related to the public roadways;

   b. Inspection of the installation of all required channelization (including but not limited to channelization of the mini-roundabouts on SW 100th Street, STOP bars/lines as determined to be required either during engineering plan approval or during inspection, crosswalks, parking stall striping, etc.) within the public right-of-way;

   c. Inspection and required hardware associated with the installation of street illumination that may be proposed within the public right-of-way of the project roadways, or that may be required by the provisions of KCRS Section 5.05;

d. Inspection and required hardware of modifications of the signalization system at 8th Avenue SW/SW Roxbury.

Acknowledged; KCHA has executed a reimbursement agreement with KCDOT.

6. The following conditions outline the required improvements for the fronting roadways, where not already addressed in the County Road Engineer's conditional approval of Variance L03V0060.

a. FRONTAGE SW Roxbury Street (east of 4th Avenue SW):

The frontage along SW Roxbury, east of 4th Avenue SW (between 4th Avenue SW and the extension of 97th Place SW) shall be improved to the urban subaccess street standard (south side only). West of these frontage improvements, to the intersection of 4th Avenue SW, these improvements may require a widening to provide a minimum 20-foot wide traveled way and a 5-foot wide shoulder if sufficient existing right-of-way is available to accommodate such improvements.

Acknowledged; plans to be submitted with future building permit and engineering plans.

b. FRONTAGE SW 102nd Street:

The frontage along SW 102nd Street, between 4th Avenue SW and 5th Avenue SW, shall be widened and reconstructed as necessary in accordance with the site plan to the urban neighborhood collector standard, including any additional paving to provide an 18-foot wide paved section on the north side of the right-of-way centerline, together with the construction of concrete curb, gutter and sidewalk along the northerly side of the roadway

Not applicable to this application.

c. FRONTAGE 4th Avenue SW:

Outside of the limits of the construction/reconstruction of the intersections of SW 97th Place/SW 98th Street, SW 98th Place, SW 100th Street and SW 102nd Street the Applicant shall reconstruct, as required, any existing damaged sections of sidewalk along 4th Avenue SW, between SW 102nd Street and SW Roxbury Street (west side only) and between SW 100th Street and SW Roxbury (east side).

Not applicable to this application

d. FRONTAGE SW 100th Street:

Outside of the limits of the construction/reconstruction of the intersections of 10th Avenue SW and 11th Avenue SW the applicant shall reconstruct, as required, any existing damaged sections of sidewalk along the northerly side of SW 100th Street from 11th Avenue SW to the westerly project boundary.

Not applicable to this application.

7. Non-roadway widening-related transportation improvements:

The transportation mitigation measures that follow are required as a condition of plat approval to provide adequate accommodation for the public health, safety and welfare of the residents of the community. Design plans, as required, for these improvements shall be included with the first set of engineering plans submitted for King County review, together with a schedule for construction to be provided, reviewed, and approved by King County DOT.

a. 8th Avenue SW Project Trail Crossing between SW 97th Street and SW 99th Street (near community center)

1) The Applicant shall install an actuated pedestrian flasher at the proposed trail crossing location on 8th Avenue SW, between SW 97th Street and SW 99th Street.

This flasher shall include the use of a steel pole and mast arm-type installation, related signage, pole foundation, and any/all underground conduits, pedestrian indications, push buttons, wiring, and related appurtenances necessary to achieve the desired operation.

2) It is recommended that a high contrast pavement treatment, e.g., textured concrete (preferred) or colored textured asphalt, should be provided for the trail crosswalk area itself – rather than the standard thermoplastic material ‘piano key’ type crosswalk markings.

3) Bulb-outs of the proposed curb and gutter improvements on 8th Avenue SW shall be provided at the trail crossing location. The curbline of the bulb-out section shall not encroach any closer to the centerline of 8th Avenue SW than the adjacent parking lanes, i.e. no more than 21-feet towards the centerline of the street from the nominal curb line where located adjoining the back-in in-street angle parking, no more than 8-feet towards the centerline of the street from the nominal curb line where located adjoining the in-street parallel parking spaces. In no event shall this reduce the curb-to-curb width (i.e. effective pedestrian crossing distance) to less than the 24 feet required for the northbound and southbound travel lanes on 8th Avenue SW.

b. Variance decision *L03V0060* granted by the County Road Engineer on April 13, 2004, conditionally deleted a requirement to reconstruct SW 100th Street to achieve a roadway profile that meets the requirements of the 1993 KCRS. In accordance with the Variance decision, the Applicant shall construct mini-roundabouts (18-foot diameter central island) meeting applicable design criteria (see Attachment 2) as specified by King County DOT, at the intersections of (1) 9th Avenue SW and (2) 10th Avenue SW on SW 100th Street.

Plans for these roundabouts and required splitter islands, related channelization and any required intersection modifications (street illumination, modification of/construction of off-site curbs and gutters, sidewalks and ADA ramps, for example) to the existing public improvements on SW 100th Street, shall be submitted to KCDOT for review and approval with the first submittal of the road improvement plans.

Not applicable to this application.

Acknowledged; this was addressed in a prior plan set.

B. Condition Missing / Skipped in original condition

C. On-street parking along 8th Avenue SW

1. Channelization plans for the proposed on-street parking: (1) back-in angle parking along the westerly side, and (2) parallel parking along the easterly side, on 8th Avenue SW shall be submitted to King County DOT for review and approval.

   Acknowledged; this was addressed in a prior plan set.

2. Bulb-outs of the proposed curb and gutter improvements on 8th Avenue SW shall be provided at the ends of each proposed parking area. The curbline of the bulb-out section shall not encroach any closer to the centerline of 8th Avenue SW than the adjacent parking lanes, i.e., no more than 21-feet towards the centerline of the street from the nominal curb line where located adjoining the back-in in-street angle parking, no more than 8-feet towards the centerline of the street from the nominal curb line where located adjoining the in-street parallel parking spaces. In no event shall this reduce the curb-to-curb width to less than the 24 feet required for the northbound and southbound travel lanes on 8th Avenue SW.

   Not applicable to this application.

D. On-street parking along 7th Avenue SW

1. Channelization plans for the proposed on-street parking: (1) angle parking along the westerly side, and (2) parallel parking along the easterly side, on 7th Avenue SW near the Wiley Center, shall be submitted to King County DOT for review and approval.

   Not applicable to this application.

2. Bulb-outs of the proposed curb and gutter improvements on 7th Avenue SW shall be provided in between the two proposed parking areas. The curbline of the bulb-out section shall not encroach any closer to the centerline of 7th Avenue SW than the adjacent parking lanes, i.e., no more than 21-feet towards the centerline of the street from the nominal curb line where located adjoining the in-street angle parking.

   Not applicable to this application.

E. All construction and upgrading of public and private roads shall be done in accordance with the King County Road Standards established and adopted by Ordinance No. 11187, as amended (1993 KCRS) or as approved by Road Variance L03V0060 .

   Acknowledged.

F. SW Roxbury Street, 4th and 8th Avenues SW and SW 100th and 102nd Streets are designated as either arterials or neighborhood collector streets which may require designs for bus zones and turn outs. As specified in KCRS 2.16, the designer shall contact Metro and the local school district to determine specific requirements.

   Not applicable to this application.

G. Modifications to the above road conditions may be considered by King County pursuant to the variance procedures in KCRS 1.08.

   Acknowledged.

H. There shall be no direct vehicular access to or from SW Roxbury Street and 4th Avenue SW from those lots which abut them. A note to this effect shall appear on the engineering plans and final plat.

   Not applicable to this application.

I. Modifications to the above road conditions may be considered by King County pursuant to the variance procedures in KCRS 1.08 or by the procedures in Condition 5, as applicable.

   Acknowledged.

13. All utilities within proposed rights-of-way must be included within a franchise approved by the King County Council prior to final plat recording.

   Acknowledged. Not applicable to this application.

14. The applicant or subsequent owner shall comply with King County Code 14.75, Mitigation Payment System (MPS), by paying the required MPS fee and administration fee as determined by the applicable fee ordinance. The applicant has the option to either: (1) pay the MPS fee at final plat recording, or (2) pay the MPS fee at the time of building permit issuance. If the first option is chosen, the fee paid shall be the fee in effect at the time of plat application and a note shall be placed on the face of the plat that reads, "All fees required by King County Code 14.75, Mitigation Payment System (MPS), have been paid." If the second option is chosen, the fee paid shall be the amount in effect as of the date of building permit application. The applicants may request an exemption, through King County Housing and Community Development, for the Roads MPS fee for qualifying low- and moderate-income homes.

Acknowledged. Not applicable to this application.

15. Lots within this subdivision are subject to King County Code 21A.43, which imposes impact fees to fund school system improvements needed to serve new development. As a condition of final approval, fifty percent (50%) of the impact fees due for the plat shall be assessed and collected immediately prior to recording, using the fee schedules in effect when the plat receives final approval. The balance of the assessed fee shall be allocated evenly to the dwelling units in the plat, based on the dwelling unit type, and shall be collected prior to building permit issuance.

Acknowledged. Not applicable to this application.

16. The proposed subdivision shall comply with the Sensitive Areas Code as outlined in KCC 21A.24. Permanent survey marking, and signs as specified in KCC 21A.24.160 shall also be addressed prior to final plat approval. Temporary marking of sensitive areas and their buffers (e.g., with bright orange construction fencing) shall be placed on the site and shall remain in place until all construction activities are completed.

Not applicable to this application.

17. Preliminary plat review has identified the following specific sensitive areas requirements which apply to this project. All other applicable requirements from KCC 21A.24 shall also be addressed by the applicant.

A. Wetlands & Streams

1. Class 2 wetlands are required to have a buffer width of 50 feet as measured from the wetland edge (K.C.C.21A.24.320). The wetland and its buffer must be accurately identified on site plans.

2. Class 3 streams are required to have buffer width of 25 feet as measured from the ordinary high water mark (K.C.C.21A.24.360). Streams and their buffers must be accurately identified on site plans.

3. Sensitive Area Tracts (SAT) shall be used to delineate and protect sensitive areas and buffers on the proposal site and shall be recorded on all documents of title of record for all affected lots (K.C.C.21A.24.180).

4. A 15-foot BSBL shall be established from the edge of the Sensitive Areas Tract (K.C.C.21A.24.200).

5. A Temporary Erosion and Sedimentation Control Plan (TESCP) in accordance with Appendix D of the King County Surface Water Design Manual for temporary protection of exposed soils and receiving surface water bodies shall be submitted to the Department for review and approval prior to final plat recording.

6. Prior to commencing construction activities on the site, the applicant shall mark sensitive areas tracts in a highly visible manner, and these areas

must remain so marked until all development proposal activities in the vicinity of the sensitive areas are completed (K.C.C.21A.24.150).

7. The sensitive area buffer / tract shall be identified using permanent sensitive area boundary signs installed between the sensitive area buffer / tract and the 15 foot BSBL. Signs shall be posted on all weather backing on 4”x4” (or equivalent) posts. Signs are available for sale at the DDES cashier.

8. To the extent practicable, landscaping of developed areas shall use native plant species to provide ground cover as nesting and feeding sites for birds and small mammals.

9. Prior to final recording and/or final engineering review of the plat, the plan set shall be routed to the DDES Critical Areas Section for approval.

10. A spill control and prevention plan shall be submitted to the Department for review and approval final to final plat recording.

None are applicable to this application.

B. Geotechnical

1. Determine the top, toe, and sides of 40% slopes by field survey. Provide a *10*-foot buffer from these slopes. The building setback shall be a minimum of 15 feet from the outer edge of the buffer. For those lots adjacent to the steep slope buffer, the need for additional building setbacks or slope mitigation measures shall be evaluated in project specific geotechnical engineering reports subject to review and approval by DDES geologist, prior to building permit approval (per the Geotechnical Analysis prepared by GeoEngineers and received by DDES on January 27, 2004).

2. The applicant shall delineate all on-site erosion hazard areas on the final engineering plans (erosion hazard areas are defined in KCC 21A.06.415). The delineation of such areas shall be approved by a DDES geologist. The requirements found in KCC 21A.24.220 concerning erosion hazard areas shall be met, including seasonal restrictions on clearing and grading activities.

3. Steep slopes and related buffers greater than one acre in size following development shall be placed in sensitive area tracts. The Tracts shall be recorded on all documents of title of record for all affected lots (K.C.C.21A.24.180).

4. A minimum 15-foot BSBL shall be established from the edge of the Sensitive Areas Tract (K.C.C.21A.24.200). Additional building setback requirements may be established by King County DDES upon review of the applicant’s project specific geotechnical engineering recommendations during individual building permit review.

5. The storm water management system shall be designed in a manner that will protect the steep slopes on and immediately adjacent to the site. The design of the system shall be prepared in consultation with the project geotechnical engineer. In particular the engineer must evaluate the adequacy and location of the proposed outfall design, the emergency overflow conveyance and any required linings to prevent seepage related impacts from the detention facilities.

   The Storm Water Management System was designed and constructed per the approved Wind Rose engineering plans (STRV15-0006) and the Plat of Windrose (Recording 20190502000861*).* GeoEngineers report provided for reference in the TIR Appendix.

C. The following note shall be shown on the final engineering plan and recorded plat:

RESTRICTIONS FOR SENSITIVE AREA TRACTS AND SENSITIVE AREAS AND BUFFERS

Dedication of a sensitive area tract/sensitive area and buffer conveys to the public a beneficial interest in the land within the tract/sensitive area and buffer. This interest includes the preservation of native vegetation for all purposes that benefit the public health, safety and welfare, including control of surface water and erosion, maintenance of slope stability, and protection of plant and animal habitat. The sensitive area tract/sensitive area and buffer imposes upon all present and future owners and occupiers of the land subject to the tract/sensitive area and buffer the obligation, enforceable on behalf of the public by King County, to leave undisturbed all trees and other vegetation within the tract/sensitive area and buffer. The vegetation within the tract/sensitive area and buffer may not be cut, pruned, covered by fill, removed or damaged without approval in writing from the King County Department of Development and Environmental Services or its successor agency, unless otherwise provided by law.

Not applicable to this application.

18. Suitable recreation space shall be provided consistent with the requirements of KCC 21A.14.180 and KCC 21A.14.190 (i.e., sport court[s], children's play equipment, picnic table[s], benches, etc.).

A. A phased detailed recreation space plan (i.e., landscape specs, equipment specs, etc.) consistent with the overall conceptual plan shall be submitted for review and approval by DDES and King County Parks prior to or concurrent with the submittal of the final plat documents.

B. A performance bond, if necessary, for recreation space improvements shall be posted prior to recording of the plat.

C. The proposed trail shall be a minimum 8 feet wide with asphalt or pervious concrete surfacing and 5 feet of landscaping on each side. Compliance with trail standards in the KCRS and KCC 21A.14.240 is not required.

D. Regarding any amenity at White Center Heights Elementary School for which the applicant seeks to obtain recreational credit, the applicant shall submit an agreement executed by the Highline School District stating that the residents, successors, and assigns of Greenbridge may use the school's recreation space improvements and parking lot during non-school hours.

None are applicable to this application.

19. If the King County Housing Authority proposes not to own and maintain any of the following amenities, a homeowners' association or other workable organization shall be established to the satisfaction of DDES which provides for the ownership and continued maintenance of such recreation, open space and/or sensitive area tract(s).

Not applicable to this application.

20. Street trees shall be provided as follows (per KCRS 5.03 and KCC 21A.16.050). See attachment O for additional conditions:

A. Trees located within the street right-of-way shall be planted in accordance with Drawing No. 5-009 of the 1993 King County Road Standards.

B. If King County determines that the required street trees should not be located within the right-of-way, they shall be located no more than 20 feet from the street right-of-way line.

C. The applicant shall submit a street tree plan and bond quantity sheet for review and approval by DDES prior to engineering plan approval or building permit submittal, whichever is first.

D. The applicant shall contact Metro Service Planning at 684-1622 to determine if SW Roxbury Street, 8th Avenue SW, 4th Avenue SW, SW 100th Street, or any other internal or adjacent road is on a bus route. If any of the applicable roads are a bus route, the street tree plan shall also be reviewed by Metro.

E. The street trees must be installed and inspected, or a performance bond posted (if applicable) prior to recording of the plat or per building permit requirements, whichever is first. If a performance bond is posted, the street trees must be installed and inspected within one year of recording of the plat or building permit issuance, whichever is first. At the time of inspection, if the trees are found to be installed per the approved plan, a maintenance bond must be submitted or the performance bond replaced with a maintenance bond, and held for one year. After one year, the maintenance bond may be released after DDES has completed a second inspection and determined that the trees have been kept healthy and thriving.

A landscape inspection fee shall also be submitted prior to plat recording or building permit issuance, whichever is first. The inspection fee is subject to change based on the current county fees.

None are applicable to this application.

21. Areas used as regional utility corridors shall be contained in separate tracts and meet the setback requirements of King County Code 21A.12.140 – Setbacks from regional utility corridors.

The regional utility corridor on the eastern portion of the site has been platted in a separate tract as part of the Greenbridge Master Plat.

22. The following conditions have been established under SEPA authority as requirements necessary to mitigate the adverse environmental impacts of this development. The applicant shall demonstrate compliance with these items, where applicable, prior to final approval.

A. Transportation Mitigation required under SEPA

The traffic mitigation contained within the FEIS for the Greenbridge HOPE VI FEIS shall set the standards and methods applicable to off-frontage or direct access intersection transportation improvements, and shall be reviewed by the King County Department of Transportation and DDES. Design plans, as required, for the roadway improvements shall be included with the first set of engineering plans submitted for King County review.

As identified in the FEIS for the Greenbridge HOPE VI project, the following mitigation measures shall be implemented by the Applicant.

1. The applicant shall submit plans to KCDOT for the edgeline channelization recommended by the KCDOT HAL/HARS report for the project's impacts on the High Accident Road Segment [HARS] on SW 116th Street (HARS #50 from the July 2003 HARS list), and install this channelization.

2. 8th Avenue SW at SW Roxbury Street

   a. The Applicant shall dedicate the full right of way, namely 50-feet from centerline or 15 feet, as required for the construction of a westbound left turn lane of storage length and transitions (including to the west of the intersection of 8th Avenue SW) complying with the requirements of the KCRS.

   b. With the first submittal of engineering plans, the Applicant shall submit plans for the following revisions to the traffic signalization system. This shall include all hardware and other appurtenances, which may include but not be limited to: poles and mast arm, signal controller, related signage, pole foundation, any/all underground conduits, signal head indications, pedestrian indications, push buttons, wiring, and related appurtenances necessary to achieve the desired operation:

      1) Revise the current phasing of the signal system to provide for a leading protected/permissive movement for westbound left turns from SW Roxbury Street to southbound 8th Avenue SW

2) Provide for advance warning (preferably actuated by traffic) indication: signage, amber flashing lights, of westbound traffic at the 8th Avenue SW/ SW Roxbury Street intersection.

3) Restrict northbound right turns to prohibit north-to-east right turning movements during the northbound red signal indication phase.

c. As identified in the FEIS, an re-evaluation will be made of the operational efficiency and accident history at the intersection of 8th Avenue SW/SW Roxbury at a point in time when approximately 67% (2/3rds) of the units (based upon a representative mix) have been constructed and occupied (i.e. generating traffic).

Based upon the results of that operational evaluation, additional improvements to SW Roxbury Street may be required to address the impacts of this development. If additional mitigation is determined to be warranted, the County and/or City of Seattle may propose, and the applicant shall participate in, a multi-jurisdictional or corridor-wide capital improvement project to mitigate any ongoing safety problem. In this case, the applicant would contribute its proportionate share to any needed mitigation project. The monitoring program identified above will be used to help establish the applicant's proportionate share of any additional mitigation. If additional mitigation is determined to be warranted and no County and/or City of Seattle project exists, the applicant shall provide mitigation that is proportionate to its identified impacts.

Not applicable to this site.

B. NOISE

1. **Construction:** Construction noise impacts shall be mitigated pursuant to best management mitigation measures identified in the Greenbridge EIS, including:

a. Construction equipment shall be properly sized and maintained including mufflers, engine intake silencers, engine enclosures, turning off idle equipment, and confining construction activities to daytime hours as specified in the King County Noise Ordinance.

b. Construction contracts shall specify that mufflers be in good working order and that engine enclosures be used on equipment when the engine is the dominant source of noise.

c. Stationary equipment shall be placed as far away or shielded from sensitive receiving locations as possible.

d. Condition Missing / Skipped in original condition

e. Where feasible, equipment operators shall drive forward rather than backward to minimize noise from back-up alarms.

f. Where feasible, noise from material handling shall be minimized by requiring operators to lift rather than drag materials.

g. Where possible, contractors should make efforts to keep construction equipment greater than 100 feet from or to shield the nearest on and offsite residences and the school to comply with King County Noise Ordinance noise limits and to minimize impacts to these sensitive receivers.

None are applicable to this application.

C. HISTORIC AND CULTURAL RESOURCES

1. A qualified archeologist shall implement a formal monitoring and discovery plan during construction.

2. In the event that historic or prehistoric cultural remains are exposed during construction, the State Historic Preservation Offices and concerned tribes shall be contacted.

3. The King County Sheriff and Medical Examiner's Office shall be notified immediately of any accidental discovery of human remains.

4. If remains were determined to be Native American, all concerned tribes shall be contacted immediately.

Not applicable to this application.

D. AIR QUALITY

1. Natural Gas units are required in place of wood-burning appliances.

Acknowledged.

2. In order to mitigate for air quality:

a. Use equipment and trucks that are maintained in good operational condition.

b. Require off-road equipment to be retrofitted with emission reduction equipment (i.e., require participation in Puget Sound regional diesel solutions)

c. Implement restrictions on construction truck idling

d. Locate construction equipment away from sensitive receptors

e. Locate construction staging zones where diesel emissions won't be noticeable to the public or near sensitive populations such as the elderly and the young.

None are applicable to this application.

E. PLANTS AND ANIMALS

1. As required by Condition 11, the applicant shall implement “built-green” and low-impact design principles to limit effective impervious surface area and provide biofiltration of stormwater runoff.

   Conditions acknowledged; Landscape Plans for implementation consistent with this condition will be submitted with a future building permit.

2. Where landscaped areas abut native growth areas, landscape with native plant species to provide ground cover as nesting and feeding sites for birds and small mammals.

   Conditions acknowledged; Landscape Plans for implementation consistent with this condition will be submitted with a future building permit.

F. FISH RESOURCES

As required by Condition 11, mitigation measures include BMPs to improve and protect water quality and benefit fish and their habitat to include a roadside biofiltration BMP.

See responses to Condition 11.

G. EARTH

Steep Slope Hazards. Typical mitigation of impacts in or near steep slope and landslide hazard areas resulting from project development should include the following:

1. Reduce clearing to the minimum extent necessary.
2. Constrain earthwork to dry weather.
3. Revegetate disturbed areas as soon as practicable.
4. No fill will be placed on or near the crest of steep slope areas without approval from the project geotechnical engineer and King County DDES.

   None are applicable to this application.

23. Prior to site demolition or construction, a plan for controlling construction noise impacts as required above in condition 22.B shall be submitted to and approved by DDES.

Not applicable to this application.

24. Prior to redevelopment of the maintenance facility site, the soils associated with the dry well that were contaminated with waste oil shall be removed in compliance with state Department of Ecology requirements.

Not applicable to this application.

KCSWDM – Greenbridge Adjustment L03V0066 Conditions of Approval.

1. The release rates for the detention facility serving the Hamm Creek subbasin will be based on only that portion of the site that naturally drains to the east in the pre-developed conditions.

   Not applicable to this application.

2. The volume for the eastern detention facility will be based on all flows directed to the facility at full development under current zoning. The allowed release rate will be reduced by any undetained flows that would bypass the proposed drainage facility. The detention volume shall be sized using the Level Three flow control standard in the 1998 KCSWDM or a modified Level One flow control with amended soil bioswales. A 10 percent volumetric factor of safety must be applied to all storm event requiring detention.

   Not applicable to this application.

3. A tightline shall be provided for the eastern detention pond to convey regular releases and undetained emergency overflows through the officially designated steep slope portion of the on-site ravine area. Regular releases are required to be connected to the existing 30-inch storm drain system that conveys stormwater to Myers Way SW. Emergency overflows may be allowed to daylight on project property between the steep slope and existing 30-inch pipe with justification during engineering review.

   Not applicable to this application.

4. All roadside bioswale landscape plans shall be approved by King County WLRD prior to engineering plan approval. Planting of shrubbery and deciduous trees shall be reviewed for Recommended Design Features found on page 6-47 of the 1998 KCSWDM.

   Not applicable to this application.

5. A Special Use Permit should be obtained to allow the King County Housing Authority or the Home Owners Association to provide additional landscape maintenance on drainage facility tracts owned by King County.

   Acknowledged; a Special Use Permit will be obtained if applicable.

6. If the option of modified Level One flow control with amended bioswales is implemented, then the East Pond's outflow state / discharge shall be continuously monitored for three years starting at 75% buildout of that portion of the site. The data and interpreted results shall be presented to King County WLRD Stormwater Services Section at the end of each full year of monitoring.

   Acknowledged; Level 3 Flow Control is proposed; therefore, monitoring is not required.

7. Additional storm drainage requirements identified by SEPA or the plat hearing review will apply to this project.

   Noted.

3. Offsite Analysis

The following downstream analysis for evaluating potential site impacts from the proposed Wind Rose at Greenbridge (Lot 32, Plat of Windrose) development area should be based on site and downstream conditions that existed prior to any redevelopment activity on the associated Windrose Final Plat (Recording #20190502000861) in 2019. Please note that references to the "project site" in this chapter refer to the overall Wind Rose and Greenbridge EO1 site.

The Level 1 Downstream Analysis was prepared for the Wind Rose Short Plat (L10S0013) and included in the "Wind Rose (incl. a Division of Greenbridge) Technical Information Report" by Goldsmith (Rev. October 2017) which was approved with the Wind Rose engineering plans (STRV15-0006).

On May 4, 2020, a request for relevant drainage complaints within a one mile radius of the site was submitted to the King County Stormwater Services Section. Upon receiving and reviewing the list of complaints on May 6, 2020, it was determined that none of the potential complaints were applicable to the site.

PLEASE NOTE – The original FIGURES referenced in this section have been revised / updated for consistency with this TIR.

Task 1. Study Area Definition

Existing Land Use / Surface Cover

The project site is ± 8.0 acres in size and is shown in the attached aerial photo (Figure 5). The existing commercial development and Greenbridge EO1 area are shown on the aerial photo. Until approximately six years ago, these uses were comprised of 2,600± square-foot retail / convenience market and 2,800± square-foot automobile care center. One single-family residence is located in the northeastern corner of the property. The single family residence in the northeast corner of the site will remain temporarily. As part of the site demolition existing pavement surfaces were removed along with any hazardous materials. A large portion of the site is covered with blackberries; these will be removed prior to the entire site being stabilized.

Site Soils / Geology

There is no SCS mapping available for the project area; however, a soil investigation has been completed for the Wind Rose site (*Summary of Subsurface Conditions and Preliminary Geotechnical Considerations, GeoEngineers, Aug, 2010*).

Soils generally consisted of Vashon ice contact deposits (classified as Alderwood series soils) were observed to a depth of 20 to 30 ft. below the site. Alderwood soils are "characterized by moderately well drained, undulating to hilly soils underlain by very slowly permeable glacial till". The Alderwood soils are underlain by advanced outwash (Everett Series) soils. Outwash materials are identified as generally very dense sand with variable amounts of gravel. Outwash materials have been observed east of the project site at a depth of 30 ft.

Groundwater

Localized zones of shallow perched groundwater were identified on-site in areas underlain by low permeability soils. These areas were observed at depths ranging from 10 to 33 ft below the ground surface and are anticipated to exist at various depth intervals in response to season rainfall patterns (*Summary of Subsurface Conditions and Preliminary Geotechnical Considerations, GeoEngineers, Aug, 2010*).

Existing Site Drainage Patterns

The existing site topography is shown on Figure 6. The site slopes towards the east with ground slopes of 2% within the areas of the existing commercial development (west side of preliminary short plat site). Within the east half of the site ground slopes increase to approximately 15%. Site runoff sheet flows toward the east draining to the road and drainage infrastructure located within the adjacent Greenbridge EO1 development area surrounding the Wind Rose site to the east and south. The existing Greenbridge site topography and drainage infrastructure are shown on the Existing Conditions Basin Plan (Figure 6). The Greenbridge development area has a closed conveyance system that drains to one of two existing outfalls along the slopes located east of the project site. These outfalls drain to the steep sloped area before reaching a shallow and flat bottomed swale. The swale drains southerly towards the fill embankment. This fill embankment is located beneath the Seattle City Light power lines.

Drainage routes for the areas fronting the Wind Rose site are shown on Figure 6. West of the site, 4^{th} Ave. SW drains to the South via an existing closed conveyance system. This closed system ultimately drains to the Greenbridge Stormwater Control facility DR-2. North of the site, SW Roxbury St. drains to the East before reaching the storm drainage system within the Greenbridge site previously described.

Drainage continues downstream via a mainly closed drainage system ultimately reaching the Duwamish River. This drainage path is considered the North Fork of the Hamm Creek sub-basin (WRIA 09-0002). To be consistent with prior work completed for the Preliminary Plat of Greenbridge, the Wind Rose preliminary plat site drainage basin is labeled as sub-basin DR-3. The Hamm Creek basin is a heavily urbanized drainage basin approximately 1300 acres in size draining to the Duwamish River (River Mile 5 in the vicinity of Turning Bay No. 3). Details of the drainage system downstream of the swale described above are given in Task 4 of this report.

AERIAL MAP

GOLDSMITH
LAND DEVELOPMENT SERVICES

FIG. 5

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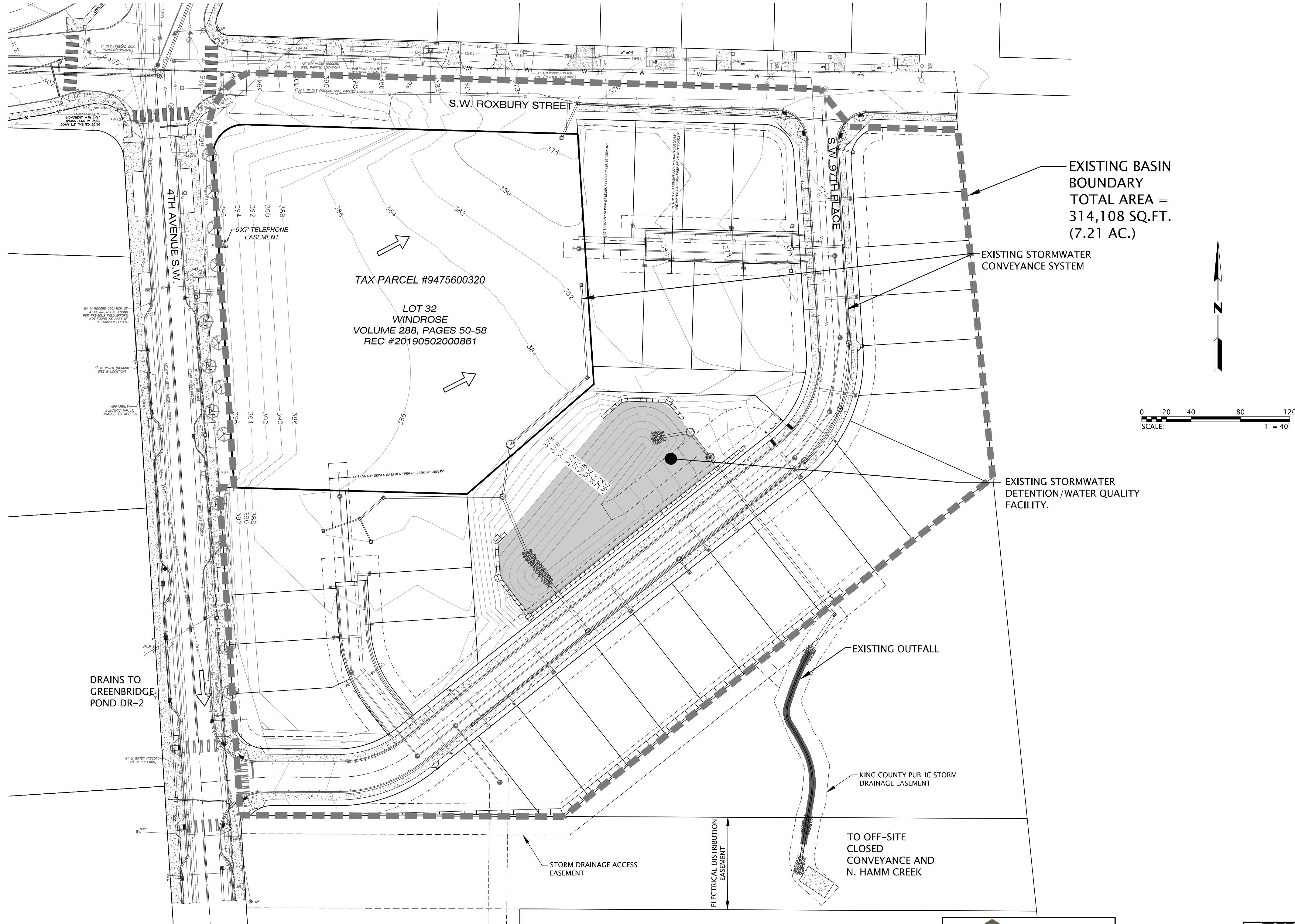
NE 1/4, NE 1/4, SECTION 6, TOWNSHIP 23 N, RANGE 4 E, W.M.
KING COUNTY, WASHINGTON

EXISTING CONDITIONS NOTE

VERTICAL DATUM: NAVD 1988.

EXISTING CONDITIONS LEGEND

AVR	AIR VAC RISER
CI	CAST IRON
CWB	CROSS WALK BOX
DI	DUCTILE IRON
EB	ELECTRIC BOX
FH	FIRE HYDRANT
IP	IRON PIPE
LP	LIGHT POLE
MB	MAILBOX
PED SIG	PEDESTRIAN SIGNAL
PVLT	POWER VAULT
SCL	SEATTLE CITY LIGHT
TMH	TELEPHONE MANHOLE
TER	TELEPHONE RISER
TSB	TRAFFIC SIGNAL BOX
TSR	TRAFFIC SIGNAL RISER
UG	UNDERGROUND CONNCTION
UP	UTILITY POLE
	GUY ANCHOR
	WATER MANHOLE
	WATER METER (SURVEYED)
	WATER METER (DESIGN)
	WATER VALVE
	CATCH BASIN (SURVEYED)
	CATCH BASIN (DESIGN)
	STORM DRAIN MANHOLE
	SANITARY SEWER MANHOLE
	GAS VALVE
	DESIGN STREET TREE
	CONCRETE AREA
	OVERHEAD UTILITY LINE
	GAS LINE
	SEWER SERVICE LINE
	WATER LINE



King County Housing Authority

811 Know what's below. Call before you dig

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APPROVED: LNYQUIST

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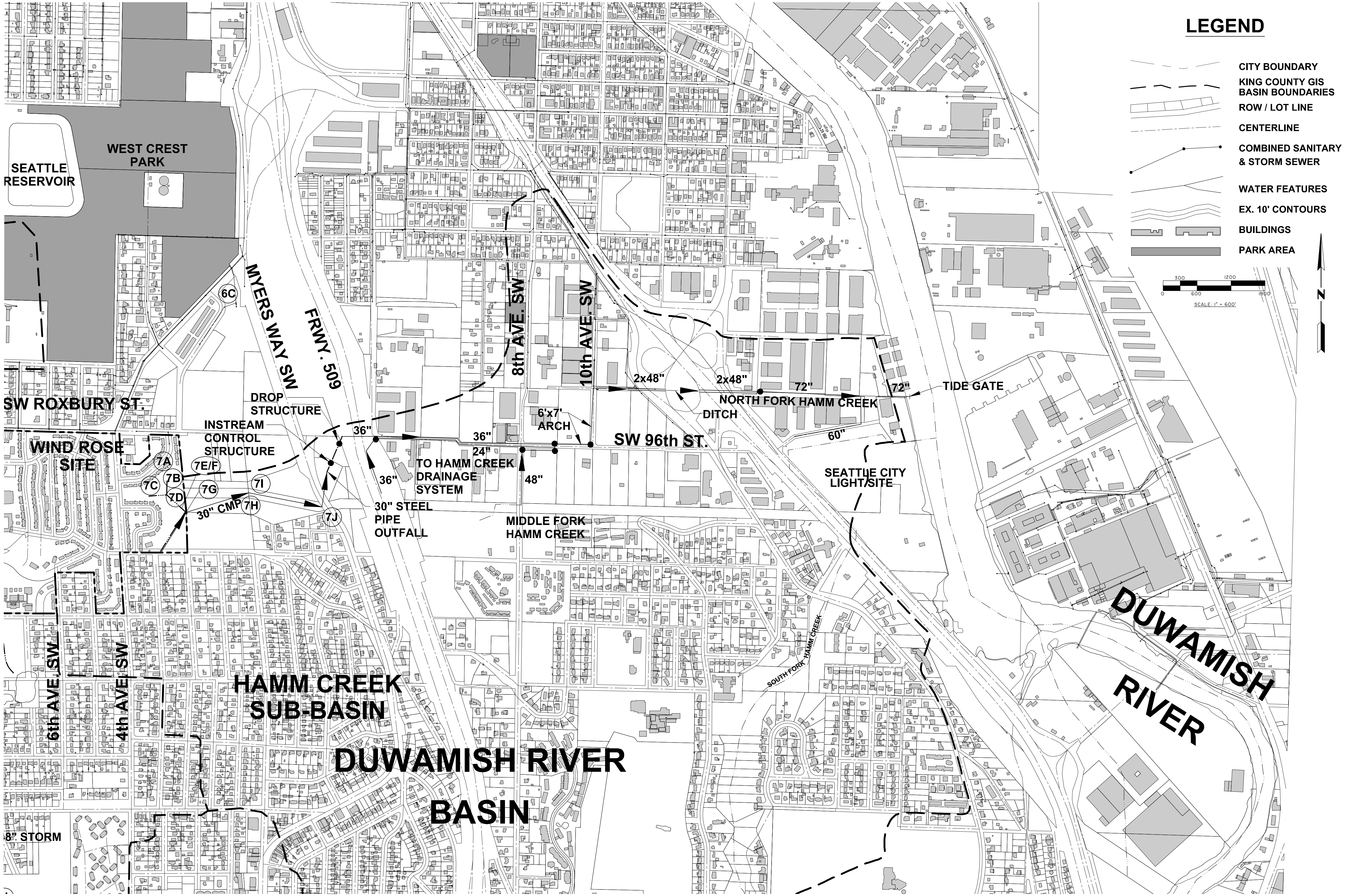
KEITH J. GOLDSMITH, STATE OF WASHINGTON, 24007, REGISTERED PROFESSIONAL ENGINEER — 7/17/2020
MARK A. BARBER, STATE OF WASHINGTON, 36000, REGISTERED PROFESSIONAL ENGINEER — 7/17/2020

KING COUNTY HOUSING AUTHORITY

EXISTING CONDITIONS DRAINAGE PLAN
FOR
WIND ROSE AT GREENBRIDGE
(LOT 32, PLAT OF WINDROSE)

KING COUNTY WASHINGTON

JOB NO. 20067
SHEET
FIG.6



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KING COUNTY HOUSING AUTHORITY

JOB NO. 20067

OFF-SITE DRAINAGE MAP
FOR
WIND ROSE

KING COUNTY WASHINGTON

FIG.7

Task 2. Resource Review

The following is a summary of the resources and documents reviewed for this downstream analysis. Relevant maps from these references have been included in the attached appendices as referenced below.

Sensitive Area Folios

King County iMAP sensitive area mapping was reviewed to identify any potential sensitive areas within, adjacent to, and downstream of the proposed site.

- Wetlands: No wetlands are identified on-site or downstream of the project site.
- Streams and 100-year Floodplains: There are no streams or floodplains identified on the project site; nor is any portion of the downstream drainage system located in a flood plain as identified by iMAP data.
- Erosion Hazard Areas: Mapping indicates no erosion hazard areas on-site or downstream of the project study area.
- Landslide Hazard Areas: Mapping identifies no landslide hazard areas on the project site.
- Seismic Hazard Areas: No seismic hazard areas are identified on the project site. The nearest hazard area identified is located 1.2 miles downstream of the site in the Duwamish Drainage Basin directly east of Highway 99 and continues to the Duwamish River. This area is not impacted by drainage from the project site. However previous geotechnical investigations (GeoEngineers, 2002) indicate that the landslide hazard area directly east of the site is also a seismic hazard area.
- Coal Mine Hazard Area: No coal mine hazard areas are identified on, or downstream of the project study area.
- Critical Aquifer Recharge Area (CARA): iMAP data indicates a Category 2 CARA beneath a portion of the site.
- Basin Condition: iMAP data indicates the basin condition as low.
- Areas Susceptible to Groundwater Contamination: A portion of the project site, the area also identified as being in a CARA, is identified as having a high susceptibility to groundwater contamination.

USDA SCS King County Soils Survey

There is no SCS mapping available for the project area; however, surficial soils have been identified according to the sites mapped geologic units (GeoEngineers, 2010).

Washington State 303d List

There are no listed water bodies located within a mile of the Wind Rose site.

Floodplain / Floodway (FEMA) Maps

There are no floodplains identified on-site or adjacent to the project site.

Adopted Basin Plans and Finalized Drainage Studies

There is no adopted basin plan for the project site or for drainage systems downstream of the site.

Downstream Drainage Complaints

Downstream drainage complaints as received from King County Water and Land Resources (WLR) Division – Drainage Services Section for the area around and downstream of the project site were reviewed (Appendix B). These complaints are predominately water quality related, although a few localized flooding complaints are listed.

The drainage complaint list was further screened to identify relevant complaints located along the conveyance system downstream of the site. The screening eliminated further review of many complaints based on the physical address of the complaint and based on the comments description. Based on this review, the flooding complaints are isolated on-lot nuisance flooding problems unrelated to the conveyance system capacity or are maintenance / code enforcement issues. The water quality complaints are mainly source control problems or problems related to past construction activities.

None of the complaints indicate restrictions in downstream conveyance capacity. However, flooding of the Hamm Creek conveyance system has been observed in the past. Observations by King County staff indicate that flooding has occurred in the past downstream of the site in the lower reaches of the basin along South 96th St. During one of the largest storms on record for the Seattle area (November 20, 2003), peak runoff within this basin coincided with the high tide. During high tide a tidal gate at the outfall from the basin to the Duwamish River closes preventing any discharge. The flooding observed during this event would be classified as a Type 3 flooding problem. A discussion of the appropriate flow control for this sub-basin is given in the following report section.

Other Reports and Information reviewed include:

- *Summary of Subsurface Conditions and Preliminary Geotechnical Considerations, GeoEngineers, Aug, 2010*
- *South 96th Street Water Quality Engineering Report, Non-point Source Pollution Controls for the Hamm Creek Watershed – Volume 1*, Herrera Environmental Consultants, April 1994.
- Additional information on the drainage system downstream of the site was also collected through numerous discussions with King County and City of Seattle Staff. Discussions included information on drainage flow paths, infrastructure condition, both completed and on-going CIP projects (i.e. Lake Garrett pump station and force main upgrades, and Hamm Creek water quality and stream rehabilitation studies and improvement projects).

Topographic and Site Survey Information

Field survey data for the project site was collected by Goldsmith in July 2010 and included collection of site topography, utilities, buildings, trees, etc.

Task 3. Field Inspection

A field inspection of the project site and downstream systems were conducted during August 2010. Inspections were completed using the guidelines for a downstream analysis as given in Section 2.3.1.1 of the 2009 KCSWDM. The results of this investigation have been combined with prior downstream investigations completed with the Preliminary Plat of Greenbridge (winter 2002/2003).

A Level 1 inspection was completed for the downstream system. The basin boundaries were verified, along with an examination of on-site and off-site drainage conditions and systems. Ground cover, slopes, soil types, and other topographic features were also observed. As required by the 2009 KCSWDM, the downstream system has been investigated to a minimum distance of at least a quarter mile (1320 ft.) downstream of the site or where the project site area comprises less than 15% of the basin. For this project the distance criteria governs the downstream inspection distance. The downstream drainage system in the Duwamish River Basin was investigated to a distance of 2300 ft. downstream of the site as detailed on Table 2-1 with locations noted on Figure 7. Further observations were made between Hwy 99 and the Duwamish River 2300 ft. and 7500 ft. downstream of the site respectively. The off-site drainage basins and downstream systems are also shown on the enclosed Figure 7.

Task 4. Drainage System Description and Problem Description

The following gives a detailed description of the drainage system downstream of the site where the existing conveyance system discharges to the off-site swale and includes a description of the drainage features and conditions beyond the limit of the inspection requirements.

Off-site Swale

A 12 inch concrete pipe discharges to the north end of a short swale system, north of the power line easement crossing the basin. This swale is shallow and flat bottomed sloping at about 4%. Evidence of flow is exhibited in areas where leaf litter is cleared exposing the soil surface of the swale bottom. At a distance of 150 ft. downstream the swale bottom is about 2 ft. wide and flows through an area of tire debris. At this point the swale bottom widens to about 3 to 4 ft. in width and has a slope of about 8%. At a distance of 300 ft. downstream, signs of additional flow entering the swale can be seen. At this point the swale bottom widens and the slope flattens before reaching an area where runoff ponds on the upslope of a fill embankment beneath the Seattle City Light power lines. This embankment allows only seepage in this portion of the north swale to drain to the swale south of the power lines.

South of the power lines the swale is significantly steeper and shows signs of significant erosion and further erosion potential. The swale continues to the south for about 300 ft. before discharging to a flat area near an existing 30” CMP storm pipe draining to the east. As flow spreads out in this flat area flow velocities are reduced causing a large area to be inundated with sediment. The manhole is at the upstream end of the 30 inch storm system (400 ft. downstream of the power line easement). Directly north of the manhole is further evidence of instabilities. This manhole has an 18 inch storm drain entering from the southwest, draining off-site areas to the south, and a 12 inch line entering from the north in the direction of the sink hole and the outfall from the Greenbridge detention facility to the west.

The 30 inch storm system continues for 1500 ft. to the east towards Myers Way S. The storm system is poorly maintained with signs of grading activities in the vicinity of the storm system that have exposed the manhole structures with the top of manholes up to 8.5 ft. above the surrounding ground elevation. Lack of maintenance is indicated by the past grading activities in the vicinity of this line and with the exception of the manholes close to Myers Way S. these manhole structures are missing their lids. The poorly maintained condition does not restrict the capacity of this system or the potential to convey site discharge.

Runoff collects in the flat area along with drainage from a south swale area below before flowing to the east in the swale/ ditch system between the site boundary and Myers Way S. 1500 ft. to the east of the property line. This swale is heavily vegetated and passes through two separate culverts where a dirt road crosses the swale. Flow then enters the 30 inch storm system crossing Myers Way S. See below for a description of the drainage system downstream of Myers Way S.

East of Myers Way S.

Just prior to crossing Myers Way S. the swale enters the closed conveyance system described above. Prior to crossing Myers Way S. the storm drain discharges to a drop structure of unknown depth. This closed system then discharges through a 30 inch steel pipe to a heavily riprapped channel upstream of an in-stream control structure (concrete dam and pool) about 300 ft. east of Myers Way S. This riprapped slope and storm drain crossing were installed following a slope failure circa 1996. This dam discharges through a drain hole in the face of the structure or by overtopping to a pool directly downstream. This pool is formed by a rock weir. Flow proceeds downstream by either flowing over the weir or by seeping through the weir. The series of pools formed by the concrete dam and the weir provide a level of energy dissipation and sediment deposition prior to flowing to the channel downstream.

Downstream a channel flows east for 400 ft. towards Hwy 509. Directly east of Hwy 509 the channel re-enters a closed storm drain system flowing towards the east through a “bird cage” drop structure of unmeasured (but deep) depth. Prior to entering the drop structure the flow enters a concrete channel. This channel has a series of baffles and a flume-like component. The function of this channel is not specifically known but it is most likely to dissipate energy, slowing flow down to a level that it can enter the drop structure in a controlled manner.

East of Hwy 509

A 36 inch storm drain crosses Hwy 509 2200 ft. downstream of the site boundary. This storm drain combines with 36” storm drain that collects Hwy 509 drainage to the south. Downstream, where the two 36” pipes are combined, a parallel 36 inch and 24 inch storm drainage system flows to the east along S. 96th St. for approximately 1900 ft. to a point approximately 300 ft. east of 8th Ave S. At this point the North and Middle Forks of Hamm Creek join together along with drainage from a ditch system flowing along 8th Ave S. A 6 ft. x 7 ft. arch pipe continues east (390 ft.) before heading north along 10th Ave S for 600 ft. (5300 downstream of the site) where flow enters a ditch flowing straight east for 500 ft. to the SR 99 cloverleaf. Twin 48 inch pipes continue for 275 ft. discharging to a ditch for about 275 ft. before crossing SR-99 in two 48 inch pipes for 550 ft. to 15th Ave S. A 72 inch pipe continues for 900 ft. at which point a 60 inch pipe joins from the south. Prior to the habitat restoration project on the Seattle City Light site to the south, the 60 inch storm drain was part of the South Fork of Hamm Creek.

East of the 72 inch storm drain is a 60 ft. long section stream reach (channel slope less than 1%, 8 ft. deep with bank width of 10 ft. flows between buildings and paved surfaces) upstream of a 72 inch pipe (*Habitat Limiting Factors and Reconnaissance Assessment Report, Dec 2000*). This pipe flows for 200 ft. before discharging through a tide gate to the Duwamish River (8500 ft. or 1.6 miles downstream of the existing Wind Rose site). Flow enters the Duwamish River in the vicinity of Turning Bay No.3 (Duwamish River Mile 5).

OFF-SITE ANALYSIS DRAINAGE SYSTEM TABLE 2-1
Surface Water Design Manual, Core Requirement #2

Basin: Duwamish River Basin **Sub-basin Name:** Duwamish River **Sub-basin Number:DR-3**

Symbol	Drainage Component Type, Name, and Size	Drainage Component Description	Slope	Distance from site discharge	Existing Problems	Potential Problems	Observations of field inspector, resource reviewer, or resident
See Figure 7	Type: sheet flow, swale, stream, channel, pipe, pond; Size: diameter, surface area	Drainage basin, vegetation, cover, depth, type of sensitive area, volume	%	¼ ml = 1,320 ft.	constrictions, under capacity, ponding, overtopping, flooding, habitat or organism destruction, scouring, bank sloughing, sedimentation, incision, other erosion		tributary area, likelihood of problem, overflow pathways, potential impacts
	North end of Swale			0 ft.	________	________	
7A	12” Conc. outfall			0 ft.	________	________	
7B	Swale			0-100 ft.	None	Erosion	Poorly defined, flat bottom
	Swale		4%	100 ft. to 125 ft.	None	Erosion	2 ft. wide
	Swale	Passes through tire and concrete debris		125 ft. to 150 ft.	None	Erosion	
7C	Roof drain	Swale flowing from up slope roof drains	8%	300 ft.	None	Erosion	3-4 ft. wide
7D	U/S of fill embankment	Low ponding area		450 ft.	None	Erosion	Water seeps through fill slope towards Swale south of powerline
7D	D/S side of power easement			450 ft.	None	Erosion	
7E	Swale	Steep swale		450-750 ft.	Erosion	Increased Erosion	
7F	Swale	Flat area		750-800 ft.	Erosion	Increased Erosion	Large sediment inundated area
7G	Manhole			800 ft.	Sink Hole	________	
7H	30” CMD Storm S.			800-2300 ft.	No LIDs Poor cover	________	
7I	Swale	Wide swale		800-2300 ft.	________	________	Parallels 30” CMP
7J	Myer’s Way			2300 ft.	________	________	

Task 5. Mitigation of Existing or Potential Problems

Flooding in the Hamm Creek system downstream of the Wind Rose site occurs in the valley bottom during periods of heavy rainfall during high tide conditions when the tide gate is closed and water backs up into the drainage system. This has been observed by King County Staff during the November 20, 2003 storm event when peak runoff coincided with high tide. During high tide the tidal gate was closed causing flooding of the upstream area. This flooding overtopped roadways and would be classified as a Type 3 flooding problem by the definition given in the 2005 KCSWDM. For this type of flooding problem in a conservation flow control application area, Level 3, detention would be required with no duration control up to the storm event return period that flooding occurs, with the return period determined through detailed hydrologic/hydraulic modeling.

Given that the South Fork of Hamm Creek has been separated from the North and Middle Fork system, effectively increasing the capacity of the North Fork, and that the size of the proposed development area and diversion is small relative to the entire basin area, the flooding potential downstream will not likely increase as a result of the Wind Rose site development with application of Level 3 flow control. In addition to the relative size of the developed sub-basin area compared to the tributary area upstream of the flooding, there is currently very little flow control in the Hamm Creek Basin. Level 3 flow control will delay peak runoff rates compared to the rapid runoff response that would be experienced in a basin characterized as heavily industrialized and having little flow control such as the Hamm Creek Sub-basin. This lagging of peaks will likely not cause a measurable increase in downstream peak flow rates.

From east of Hwy 509 to the Duwamish River the North Fork of Hamm Creek flows through a heavily industrialized area. Although referred to as the North Fork of Hamm Creek, no portion of what is identified as the creek is a natural channel. Poor water quality in this basin is attributable to runoff generated in this portion of the sub-basin. From Hwy 509 to the Duwamish River only about 835 ft. of the 6300 ft. of Hamm Creek system is not in a closed pipe conveyance system. Of the North, Middle, South and Lost Forks of Hamm Creek, the South Fork was identified as having the greatest potential for habitat improvements. Any increases in site runoff volume in the Hamm Creek basin will likely help the degraded water quality condition downstream of the site. The reason for this is that site runoff will receive treatment prior to release. Under existing conditions there is no water quality treatment on the Wind Rose site. Work completed by the Duwamish coalition as part of the Hamm Creek regional stormwater and habitat proposal indicated that the basin would benefit from additional “clean” water from the upper basin.

Existing runoff discharges to the downstream drainage system via the outfalls along the slopes east of the site. To maintain existing drainage patterns drainage easement will be needed across the adjacent (Greenbridge) property for the developed site runoff to reach the eastern swale system. Swale runoff would then be directed into the existing 30” conveyance system described previously.

As described previously the site has a low potential for infiltration. This is due to near surface soil conditions and also due to the location of a CARA below the site. Therefore, the use of infiltration BMPs is restricted.

Based on the aforementioned drainage conditions a drainage control plan has been developed that addresses any existing and potential drainage problems. The following section outlines the proposed Preliminary Drainage Control Plan.

4. Flow Control and Water Quality Facility Analysis and Design

The following provides the stormwater control plan proposed for Wind Rose at Greenbridge (Lot 32, Plat of Windrose), the subject project. The project site is known as Lot 32 located in the northwest corner of the Plat of Windrose (Recording No. 20190502000861).

The stormwater control plan described below ensures impacts related to future building permits are mitigated in accordance with the 2016 KCSWDM. The hydrologic analysis provided herein verifies that the existing off-site stormwater control facility constructed with the Plat of Windrose (Greenbridge Development Parcel E01) in 2019 is adequately sized to provide flow control for the subject project meeting the 2016 KCSWDM standards without modification.

Stormwater Control Plan

The stormwater control plan developed for the site encompasses all available information about the site and its downstream drainage system. This includes site topography, geology, detailed field investigations, and drainage complaints and observations. The stormwater control plan addresses the identified flooding problems and potential erosion problems downstream of the site. Flow control and water quality treatment BMPs have been selected to ensure any potential adverse impacts from the development will be prevented.

The proposed developed conditions drainage plan is shown on Figure 3 and the preliminary stormwater plan for the subject project is shown on Figure 8. The Wind Rose at Greenbridge (Lot 32, Plat of Windrose) project will utilize the existing off-site stormwater control facility constructed in 2019 with the Plat of Windrose (Greenbridge Development Parcel E01) for flow control. This existing facility is a combined detention/ wetpond designed and constructed to provide flow control for the subject project as demonstrated herein.

Water quality treatment meeting the 2016 KCSWDM standard for enhanced treatment standard is required for the subject project. The off-site wetpond was sized to provide basic level treatment for the project site consistent with the 2009 KCSWDM; therefore, additional treatment will be required for the Wind Rose project to provide enhanced treatment. This treatment BMP will be located on-site discharging treated water to the existing off-site stormwater infrastructure. Since treated stormwater from the subject project will be mixed with untreated off-site stormwater prior to reaching the existing combined detention facility off-site it is necessary to demonstrate that basic water quality treatment is provided for runoff directed to the facility. As demonstrated herein, the existing wetpond is adequately sized.

Flow control BMPs are required by the 2016 KCSWDM for the subject project. As a high-density development site, the use of dispersion BMPs is limited. In addition, based on the near-surface soil conditions, the proximity of the site to the steep slopes east of the site, and the presence of more easily erodible materials along these slopes, the potential to infiltrate site runoff is limited. These conditions also limit the potential for infiltration dependent flow control BMPs. The final selection of a BMP will likely involve a BMP that utilizes limited infiltration via bio-retention or through permeable surfaces. This final selection will be deferred to detailed engineering for a future building permit allowing the final selection to be integrated with future site development.

Part A. Existing Site Hydrology

The following is a summary of the existing state hydrologic input parameters and the basin areas used as the basis for evaluating the proposed stormwater control plan and for the verification that the existing off-site stormwater facility is adequately sized. As described in Part C, the flow control standards required for the subject project and the existing Plat of Windrose (Greenbridge E01) development area are different. Except for Lot 32 of the Plat of Windrose (subject project) the plat was part of the Plat of Greenbridge (development block E01) vested to the 1998 KCSWDM. The subject project must meet the requirements of the 2016 KCSWDM. Although these standards require that each site will provide Flood Problem (Level 3) flow control, each standard has a different requirement for how the existing condition is modeled. For the subject project area, the 2016 KCSWDM requires that projects located in conservation flow control areas are to assume a forested historical site condition for the existing site conditions. For the remaining area, the 1998 KCSWDM allows the existing condition to be based on the existing condition. An extensive analysis was completed and reviewed with the Greenbridge Plat development. This analysis found the existing condition to be 50% impervious with 78% of the impervious area considered effective.

These assumptions have been used for calculating allowable release rates and flow durations from the existing conditions area shown in Figure 3. The subject project area site hydrology has been modeled using the hydrologic analysis software MGSFlood. The MGSFlood report file is included at the end of Section 4.

Existing Basin Area	**=**	**7.21 Ac.** (includes frontage improvement area)
Till Forest	=	1.89 Ac. (Subject project on-site area)
Till Grass	=	3.25 Ac.
Impervious Area	=	2.07 Ac. (2.66 Ac. At 78% EIA)

Part B. Developed Site Hydrology

The following is a summary of the developed state sub-basin areas used for simulating the developed conditions runoff tributary to the existing off-site stormwater BMPs serving the subject project. The maximum impervious area for the Wind Rose at Greenbridge (Lot 32, Plat of Windrose) site area and the Plat of Windrose (Greenbridge E01) have been looked at separately.

- For the subject project, the project area the maximum impervious area used is based on the maximum allowed by King County zoning (90%).
- For the adjacent Plat of Windrose (Greenbridge E01) residential area, the impervious area is based on the platted site plan with the right-of-way area at 85% and the maximum allowed lot impervious area (85%).
- For the SW Roxbury St. right-of-way, the amount of impervious area is based on the proposed frontage and road improvements.

As previously described flow control is not required for frontage improvements along 4th Avenue SW as they drain south and have been accounted for in the sizing of previous existing Greenbridge stormwater facility DR-2. Frontage improvements along SW Roxbury St. will be captured and conveyed to the existing off-site stormwater facility.

Developed Basin Total Area	**=**	**7.21 Ac.**
Wind Rose Impervious (1.89 Ac.@90%)	=	1.70 Ac.
SW Roxbury St. ROW Impervious	=	0.26 Ac.
Plat of Greenbridge ROW (2.03 Ac.@85%)	=	1.74 Ac.
Plat of Greenbridge On-lot Impervious (2.85 Ac.@85%)	=	2.42 Ac.
Total Impervious	**=**	**6.12 Ac.**
Till Grass	**=**	**1.09 Ac.**

Part C. Performance Standards

As described below, there are two different KCSWDMs that are used for evaluating flow control and water quality treatment for the subject project. As noted, stormwater from the subject property is directed to an existing off-site stormwater facility. The portion of the facility drainage basin within the Plat of Windrose (Greenbridge E01) is vested to the 1998 KCSWDM, and the subject project basin area will utilize the 2016 KCSWDM. The following describes the required performance standards for flow control, water quality treatment, and for flow control BMPs for each project area.

Flow Control Standard

Under both the 1998 and 2016 KCSWDM Flow Control Applications Maps the project site is located in a Conservation Flow Control Area (Level 2 flow control). However, drainage conditions downstream of the site dictate that a more stringent flow control standard be used. The project site will discharge to an area that experiences Type 3 flooding problems with overtopping of roadways in the Lower Hamm Creek Basin. This flooding has been observed during past storm events. Flooding in the Lower Hamm Creek Basin can be exacerbated if significant rainfall occurs during high tide conditions when the outfalls to the Duwamish River are closed. Type 3 flooding problems downstream will make it necessary to use Flood Problem (Level 3) flow control for this project. The application of Level 3 flow control is consistent with the drainage control plan implemented for prior Greenbridge development in the Hamm Creek Basin. The following outlines the Level 3 flow control requirements for each of the two project areas.

- Wind Rose at Greenbridge (Lot 32, Plat of Windrose) – 2016 KCSWDM

  For the Wind Rose project area, as required by the 2016 KCSWDM post-developed flow rates and durations are required to be based on historical site conditions (i.e. a forested site).

- Plat of Windrose (Greenbridge E01) – 1998 KCSWDM

  The project area in the Plat of Greenbridge is vested to the 1998 KCSWDM. The 1998 manual that allows post-developed peak flow release rates and durations to be based on the existing site condition prior to development.

Flow Control BMPs

- Wind Rose at Greenbridge (Lot 32, Plat of Windrose) – 2016 KCSWDM

  - Per the 2016 KCSWDM, since the proposed subject project site is larger than 22,000sf, and will have impervious site coverage in excess of 65%, *flow control BMPs must be applied to an impervious area equal to 10% of the site area or 20% of the target impervious area, whichever is less.*" (2016 KCSWDM 1.2.9.2.2 pg. 1-88).

  - It should be noted that even though a portion of the project is located in a Critical Aquifer Recharge Area (CARA) no additional BMP requirements or restrictions apply to the proposed project since the project is classified as a Large Lot High Impervious site. The final selection of the flow control BMPs will also be completed with future building permit submittals. Perforated stub-out connections will be included with future engineering plans for the subject project.

- Plat of Windrose (Greenbridge E01) – 1998 KCSWDM

  The 1998 KCSWDM does not require flow control BMPs.

<u>Water Quality Standard</u>

- Wind Rose at Greenbridge (Lot 32, Plat of Windrose) – 2016 KCSWDM
  - The project property is located in a Basic Water Quality Control Area as shown by the King County Water Quality Applications Map; but since the project has a density of greater than 8 du/ac, Enhanced Treatment is required.
- Plat of Windrose (Greenbridge E01) – 1998 KCSWDM
  - The project area within the Plat of Greenbridge is vested to the 1998 KCSWDM and the project's conditions of approval that require the project to provide Basic Water Quality Treatment.

Part D. Flow Control System

The following gives detailed information used to verify that the existing off-site flow control facility as constructed can meet the required 2016 KCSWDM performance standard for the subject project without modifications. The facility has been modeled using the MGSFlood as required by the 2016 KCSWDM.

Detention Facility Design Summary

Modeled 100-Year Detention Volume = 44,980 cu.-ft.
Required 100- Year Detention Volume= 49.478 cu.-ft. (with required 10% FS)
Designed 100 -Year Detention Volume= 51,277 cu.-ft.

Orifice Sizing Table:

Orifice	Diameter	Height	Elev. (ft)
1	2.60 in.	-0.50 ft.	366.50
2	2.10 in	2.60 ft	369.60
3	2.40 in.	2.90 ft	369.90

Outlet Elevation	=	366.50
Riser Height	=	4.85 ft. (Above Outlet)
Riser Elevations	=	371.85 ft
Riser Diameter	=	18 inches
Pond Bot.	=	360.00
W.Q. W.S.	=	367.00
100 Year W.S.	=	371.91

Flow Control Evaluation Summary

The following provides a comparison of existing and developed peak flows and flow durations showing the Level 3 flow control criteria for both Wind Rose and the Plat of Greenbridge as described in Part C has been met.

<u>Peak Flow Rate Comparison:</u>

For Level 3 flow control the post-developed peak flow rates shall be equal to or less than the pre-developed peak rates for the 2, 10, and 100-year return periods. As shown in the following table the peak flow rate criteria have been met.

Discharge Rate Comparison

Event	Flow Rate (cfs)	
	Existing	Total Developed
2-year	1.08	0.37
10-year	1.87	0.66
100-year	3.80	1.16

Flow Duration Control Evaluations

For Level 3 flow control the post-developed flow durations must meet the following criteria.

1. Post developed flow durations between 50% of the 2-year (0.33 cfs) and the 2-year flow rate (0.65 cfs) must be equal to or less than the pre-developed condition. As shown in the flow duration comparison and the MGSFlood report the flow duration is 24% below the existing condition.

2. At any duration within the range of control, the post-developed flow is to be no more than 10% above the pre-developed flow. As shown in the MGSFlood report, following development, the maximum deviation is 87% below the pre-developed flow.

3. The target duration curve may not be exceeded along more than 50% of the range of control. As shown on the flow duration comparison graph below and in the MGSFlood report this criteria has been met.

4. The peak flow rate at the upper end of the range of control (25-year) may not exceed the predeveloped flow rate by more than 10%. The predeveloped 25-year peak flow rate is 2.549 cfs and the post-developed peak flow rate is 0.796 cfs thus meeting this criterion.

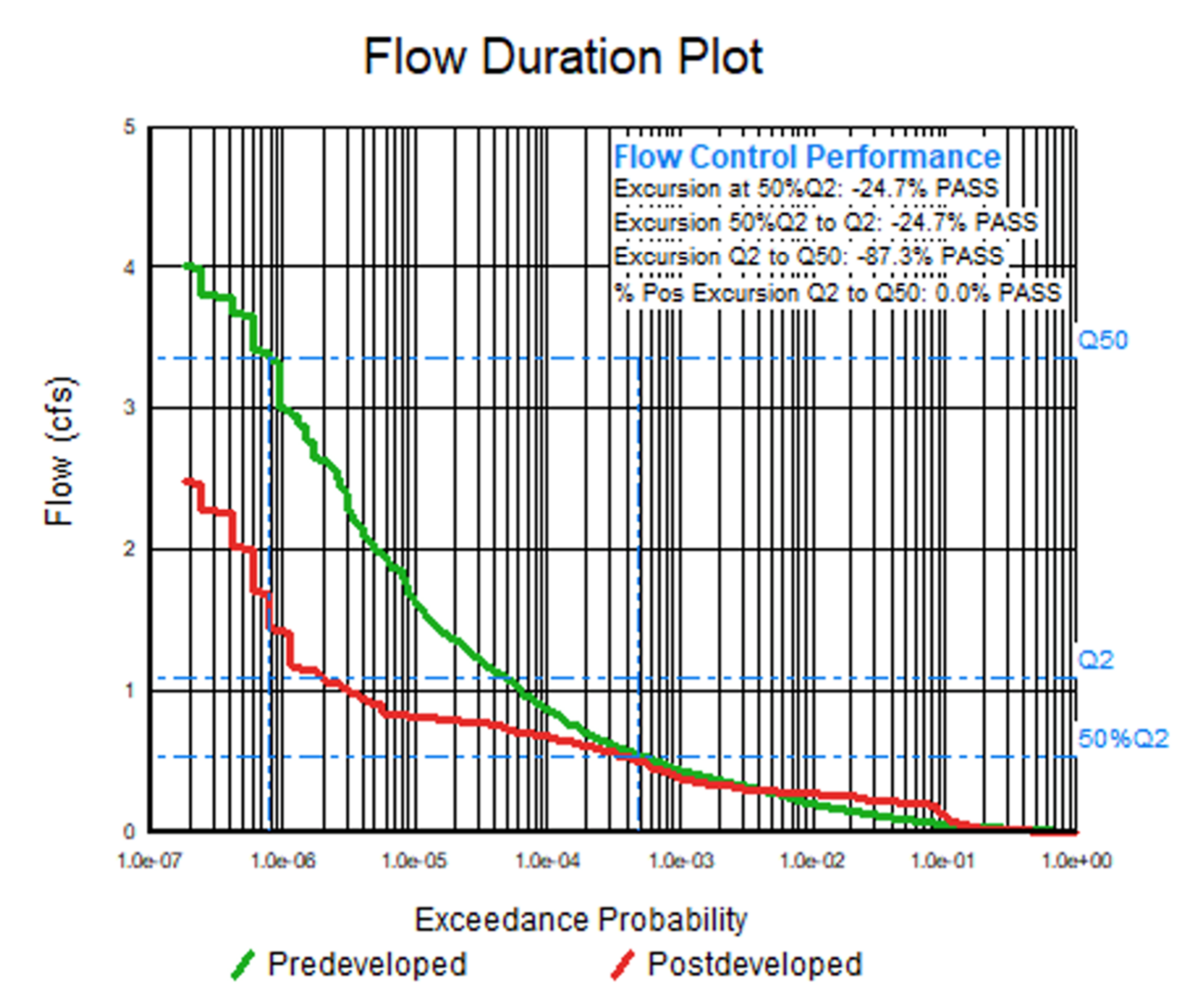
Pond Overflow and Discharge Swale

The existing off-site pond discharges to a swale through a conventional conveyance system as shown on the design plans and Figure 3. The pond outlet and outfall were designed for both the 100-year controlled discharge and potential emergency overflows. Both of the design rates used for the design of this outfall system are larger than anticipated with the proposed subject project development as summarized below. No further analysis is, therefore, necessary

Peak Pond Discharge Design Rate (100-year pond discharge) = 1.58 cfs
Revised Peak Pond Discharge Rate (100-year pond discharge) = 1.16 cfs

Emergency Overflow Design Rate = 7.69 cfs (100 year - 15 min peak pond inflow)
Revised Emergency Discharge Rate (100-year pond inflow) = 6.81 cfs

Comparison of Existing and Developed Flow Durations



NE 1/4, NE 1/4, SECTION 6, TOWNSHIP 23 N, RANGE 4 E, W.M.
KING COUNTY, WASHINGTON

LEGEND

- PROJECT BOUNDARY
- BUILDING SETBACK LINE
- ACCESS CENTERLINE AND FLOW LINE
- EDGE OF TRAVEL LANE
- 380 PROPOSED MAJOR 10' CONTOUR
- 382 PROPOSED MINOR 2' CONTOUR
- 2% ROAD SLOPE
- DRAINAGE FLOW ARROW
- PROPOSED CATCH BASIN
- SD PROPOSED 12" STORM DRAIN
- RD PROPOSED ROOF DRAIN
- YD PROPOSED PERFORATED BUILDING FOOTING DRAIN
- 390 EXIST. MAJOR 10' CONTOUR
- 398 EXIST. MINOR 2' CONTOUR
- EXIST. CATCH BASIN
- EXIST. DETENTION MANHOLE
- SD EXIST. 12" STORM DRAIN

GENERAL NOTES

1. EXISTING LINEWORK REFLECTS A COMPOSITE OF TOPOGRAPHIC DATA USING EXISTING SURVEY AND DESIGNED LINEWORK OF 4TH AVENUE SW, IN ADDITION TO AS-BUILT DATA AND DESIGNED LINEWORK OF THE NEIGHBORING WIND ROSE PLAT.

PERFORMANCE STANDARDS

FLOW CONTROL:	FLOOD PROBLEM (LEVEL 3)
WATER QUALITY:	ENHANCED

SITE QUANTITIES

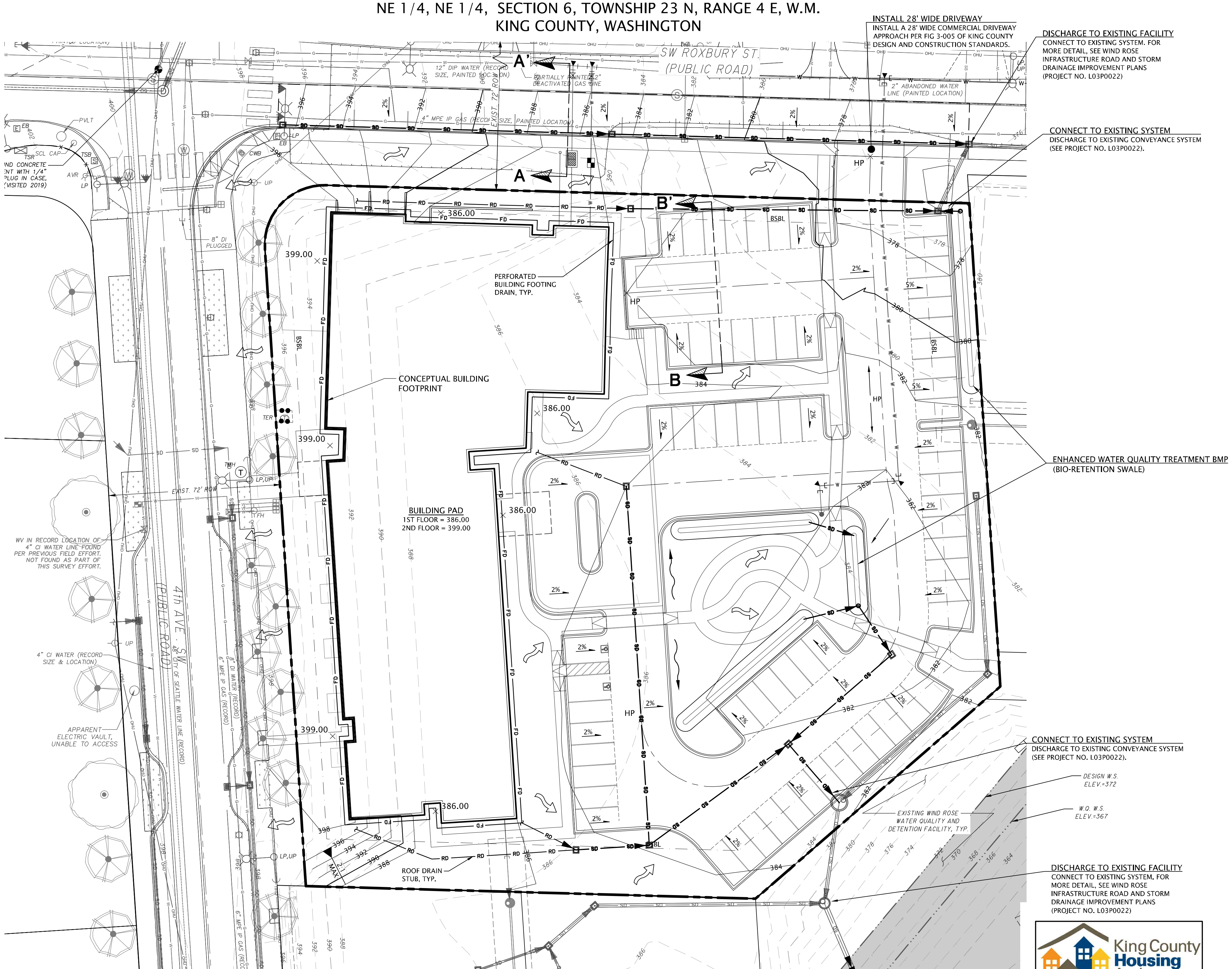
GENERAL:
TOTAL SITE AREA: 82,449 SF (1.89 AC)

DRAINAGE:
EXISTING IMPERVIOUS: 0 SF
NEW/REPLACED IMPERVIOUS: 74,204 SF
(ASSUMED MAXIMUM OF 90% OF SITE)
NEW/REPLACED IMPERVIOUS (OFFSITE): 8,012 SF

EARTHWORK:
CUT: 6,292 BCY
FILL: 1,729 BCY
TOTAL: 4,563 BCY (CUT)

STRIPPING[1]: 0 BCY

[1] STRIPPING VOLUMES ARE ESTIMATE ASSUMING REMOVAL OF APPROXIMATELY 12" FROM EXISTING GRADE.
[2] EARTHWORK IS FOR PERMITTING PURPOSES ONLY AND SHOULD NOT BE USED FOR BIDDING.



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SCALE: 1" = 20'

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KING COUNTY HOUSING AUTHORITY

CONCEPTUAL GRADING AND STORMWATER PLAN
FOR
WIND ROSE AT GREENBRIDGE
(LOT 32, PLAT OF WINDROSE)

KING COUNTY WASHINGTON

JOB NO. 20067

SHEET FIG.8

Existing Off-Site Pond :Stage Storage-Discharge Table

	Orifice #1	Orifice #2	Orifice #3	Riser
Stage	**-0.5**	**2.6**	**2.9**	**4.85**
Elev (ft)	**366.50**	**369.60**	**369.90**	**371.85**
Diam(in)	**2.60**	**2.10**	**2.40**	**18**
Area(SF)	0.037	0.024	0.031	N/A

Outlet Elevation = **367.00**

Factor of Safety = **14%**

Elev (ft)	Stage (ft)	Design Area (SF)	Area with F.S. (SF)	Volume (CF)	Cumm Vol. (CF)	Cumm Vol. (ac.ft)	ΔH1 (ft)	Q1 (cfs)	ΔH2 (ft)	Q2 (cfs)	ΔH3 (ft)	Q3 (cfs)	ΔHriser (ft)	Qriser (cfs)	Qtotal (cfs)
367.00	**0.00**	28	25	0	-	0.00	0.50	0.13						0.00	0.130
367.10	**0.10**	28	25	2	2	0.00	0.60	0.14						0.00	0.142
367.49	**0.49**	28	25	10	12	0.00	0.99	0.18						0.00	0.183
367.50	**0.50**	**9,452**	8,291	42	54	0.00	1.00	0.18	-	-	-	-	-	0.00	0.183
367.55	**0.55**	9,503	8,336	416	469	0.01	1.05	0.19	-	-	-	-	-	0.00	0.188
367.60	**0.60**	9,554	8,380	418	887	0.02	1.10	0.19	-	-	-	-	-	0.00	0.192
367.70	**0.70**	9,655	8,469	842	1,730	0.04	1.20	0.20	-	-	-	-	-	0.00	0.201
367.80	**0.80**	9,757	8,559	851	2,581	0.06	1.30	0.21	-	-	-	-	-	0.00	0.209
367.90	**0.90**	9,858	8,648	860	3,441	0.08	1.40	0.22	-	-	-	-	-	0.00	0.217
368.00	**1.00**	9,960	8,737	869	4,311	0.10	1.50	0.22	-	-	-	-	-	0.00	0.225
368.10	**1.10**	10,062	8,826	878	5,189	0.12	1.60	0.23	-	-	-	-	-	0.00	0.232
368.20	**1.20**	10,163	8,915	887	6,076	0.14	1.70	0.24	-	-	-	-	-	0.00	0.239
368.30	**1.30**	10,265	9,004	6,490	6,959	0.16	1.80	0.25	-	-	-	-	-	0.00	0.246
368.40	**1.40**	10,366	9,093	905	7,864	0.18	1.90	0.25	-	-	-	-	-	0.00	0.253
368.50	**1.50**	**10,468**	9,182	914	8,777	0.20	2.00	0.26	-	-	-	-	-	0.00	0.259
368.75	**1.75**	10,712	9,396	2,322	11,100	0.25	2.25	0.28	-	-	-	-	-	0.00	0.275
368.96	**1.96**	10,919	9,578	2,015	13,115	0.30	2.46	0.29	-	-	-	-	-	0.00	0.288
369.25	**2.25**	11,200	9,825	2,791	15,905	0.37	2.75	0.30	-	-	-	-	-	0.00	0.304
369.50	**2.50**	**11,444**	10,039	2,483	18,388	0.42	3.00	0.32	-	-	-	-	-	0.00	0.318
369.75	**2.75**	11,680	10,246	2,536	20,924	0.48	3.25	0.33	0.15	0.05	-	-	-	0.00	0.377
370.11	**3.11**	12,017	10,542	3,714	24,638	0.57	3.61	0.35	0.51	0.09	0.21	0.07	-	0.00	0.505
370.25	**3.25**	12,152	10,660	1,512	26,150	0.60	3.75	0.36	0.65	0.10	0.35	0.09	-	0.00	0.544
370.50	**3.50**	**12,388**	10,867	2,691	28,841	0.66	4.00	0.37	0.90	0.11	0.60	0.12	-	0.00	0.602
370.74	**3.74**	12,616	11,067	2,672	31,513	0.72	4.24	0.38	1.14	0.13	0.84	0.14	-	0.00	0.649
371.00	**4.00**	12,857	11,278	2,864	34,377	0.79	4.50	0.39	1.40	0.14	1.10	0.16	-	0.00	0.695
371.25	**4.25**	13,091	11,483	2,845	37,222	0.85	4.75	0.40	1.65	0.15	1.35	0.18	-	0.00	0.735
371.50	**4.50**	**13,325**	11,689	2,896	40,118	0.92	5.00	0.41	1.90	0.16	1.60	0.20	-	0.00	0.773
371.85	**4.85**	13,653	11,976	4,141	44,260	1.02	5.35	0.42	2.25	0.18	1.95	0.22	-	0.00	0.822
371.91	**4.91**	13,709	12,025	720	44,980	1.03	5.41	0.43	2.31	0.18	2.01	0.22	0.06	0.21	1.045
372.35	**5.35**	14,121	12,386	5,371	50,350	1.16	5.85	0.44	2.75	0.20	2.45	0.24	0.50	5.16	6.052
372.50	**5.50**	**14,261**	12,510	1,867	52,217	1.199	6.00	0.45	2.90	0.20	2.60	0.25	0.65	6.86	7.766

Part E. Water Quality System

Water quality treatment meeting the design objectives of the 2016 KCSWDM standard for enhanced treatment standard is required for the subject project. The selected BMP will be located on-site, and discharge treated water to the existing off-site stormwater infrastructure. Since treated stormwater from the subject project will be mixed with untreated off-site stormwater prior to reaching the existing combined detention facility off-site it is necessary to demonstrate that basic water quality treatment is provided for total runoff directed to the existing off-site facility. The following provides design details and analysis for the proposed Bio-retention BMPs and the off-site wetpond.

Bio-Retention BMP Sizing Analysis

There are 2 bio-retention facilities proposed for the project site as shown on Figures 3 and 8. These facilities will treat proposed parking lot impervious areas and adjacent landscaped areas within the facility drainage sub-area. Requests for modification and waivers have been submitted to King County for the subject project. These requests are pursuant to the flexibility established in compliance with King County's Demonstration Ordinance (King County Code 21A.55.060).

The demonstration ordinance allows for the use of non-standard stormwater BMPs that are not in the KCSWDM. For this project bio-retention cells have been selected that are designed consistent with the WA DOE 2019 Stormwater Management Manual for Western Washington bio-retention BMP design requirements. As described by the DOE manual this BMP will provide enhanced treatment, removing both >80% of TSS and removal of >30% dissolved copper and >60% of dissolved zinc.

The 2019 DOE SWMM requires that a minimum of 91% of runoff be filtered through the bio-retention media. These bio-retention cells have been designed with 18" of bio-retention soil mix and underdrains and will filter over 91% of the annual runoff as demonstrated herein based on the following Bio-retention BMP designs. Overflows and discharges from these BMP will be directed to the off-site stormwater facility previously described.

Modeling results presented below indicate that bio-retention swales as shown on Figures 3 and 8 can provide the required amount of treatment for the projects PGIS areas.

Bio-Retention BMP #1

Treatment Areas:

Water Quality Basin Area	= 0.29 Ac
PGIS	= 0.22 Ac
Landscape Area	= 0.07 Ac.

Facility Dimensions

Bottom Length: 75 ft.
Bottom Width: 1 ft.
Side Slopes 3:1
Bio-retention Bed Material thickness of the first layer: 1.5 (18 in.)
Under Drain: Included
Overflow Riser
Riser Height: 0.5 ft.
Riser Diameter: 6 in.

Required Treatment Volume = 91%
Design Treatment Volume = 91.89%

Bio-Retention BMP #2

Treatment Areas:

Water Quality Basin Area	= 0.71 Ac
PGIS	= 0.57 Ac
Landscape Area	= 0.14 Ac.

Facility Dimensions

Bottom Length: 145 ft.
Bottom Width: 2 ft.
Side Slopes 3:1
Bio-retention Bed Material thickness of the first layer: 1.5 (18 in.)
Under Drain: Included
Overflow Riser
Riser Height: 0.5 ft.
Riser Diameter: 6 in.

Required Treatment Volume = 91%
Design Treatment Volume = 91.18%

Off-Site Wetpond Sizing Analysis

The MGSFlood report file verifies that the off-site water quality control facility maintains an adequate wetpond volume for the facility's tributary drainage basin. The facility is a basic wet pond with a required wetpond volume of 28,171 cu.ft, less than the design wetpond volume of 34,343 cu.ft.

MGS FLOOD
PROJECT REPORT

Program Version: MGSFlood 4.50
Program License Number: 201810008
Project Simulation Performed on: 07/16/2020 11:14 PM
Report Generation Date: 07/16/2020 11:15 PM

Input File Name: WQ Windrose.fld
Project Name: Wind Rose
Analysis Title:
Comments:

PRECIPITATION INPUT

Computational Time Step (Minutes): 15

Extended Precipitation Time Series Selected

Climatic Region Number: 15

Full Period of Record Available used for Routing
Precipitation Station : 96004005 Puget East 40 in_5min 10/01/1939-10/01/2097
Evaporation Station : 961040 Puget East 40 in MAP
Evaporation Scale Factor : 0.750

HSPF Parameter Region Number: 1
HSPF Parameter Region Name : USGS Default

********** Default HSPF Parameters Used (Not Modified by User) ***************

*********************** WATERSHED DEFINITION ***********************

Predevelopment/Post Development Tributary Area Summary

	Predeveloped	Post Developed
Total Subbasin Area (acres)	7.210	8.210
Area of Links that Include Precip/Evap (acres)	0.000	0.008
Total (acres)	7.210	8.218

---------------------SCENARIO: PREDEVELOPED
Number of Subbasins: 1

---------- Subbasin : Ex. Cond ----------

	-------Area (Acres) --------
Till Forest	1.890
Till Grass	3.250
Impervious	2.070
Subbasin Total	7.210

---------------------SCENARIO: POSTDEVELOPED
Number of Subbasins: 3

---------- Subbasin : Dev. Cond ----------

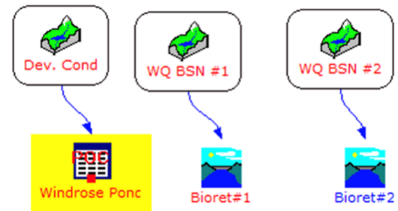
	-------Area (Acres) --------
Till Grass	1.090
Impervious	6.120
Subbasin Total	7.210

---------- Subbasin : WQ BSN #2 ----------

	-------Area (Acres) --------
Till Grass	0.140

Impervious 0.570

Subbasin Total 0.710



---------- Subbasin : WQ BSN #1 ----------
-------Area (Acres) --------
Till Grass 0.070
Impervious 0.220

Subbasin Total 0.290

************************** **LINK DATA** ********************************

----------------------SCENARIO: PREDEVELOPED
Number of Links: 0

************************** **LINK DATA** ********************************

----------------------SCENARIO: POSTDEVELOPED
Number of Links: 3

--
Link Name: Windrose Pond
Link Type: User Rating Table
Downstream Link: None

Elev (ft)	Area (ac)	Storage (ac-ft)	Discharge (cfs)	Infilt Discharge (cfs)
0.000	0.000	0.000	0.000	0.000
0.100	0.001	0.000	0.142	0.000
0.490	0.001	0.000	0.183	0.000
0.500	0.190	0.001	0.183	0.000
0.550	0.191	0.011	0.188	0.000
0.600	0.192	0.020	0.192	0.000
0.700	0.194	0.040	0.201	0.000
0.800	0.197	0.059	0.209	0.000
0.900	0.199	0.079	0.217	0.000
1.000	0.201	0.099	0.225	0.000
1.100	0.203	0.119	0.232	0.000
1.200	0.205	0.140	0.239	0.000
1.300	0.207	0.160	0.246	0.000
1.400	0.209	0.181	0.253	0.000
1.500	0.211	0.202	0.259	0.000
1.750	0.216	0.255	0.275	0.000
1.960	0.220	0.301	0.288	0.000
2.250	0.226	0.365	0.304	0.000
2.500	0.231	0.422	0.318	0.000
2.750	0.235	0.480	0.377	0.000
3.110	0.242	0.566	0.505	0.000
3.250	0.245	0.600	0.544	0.000
3.500	0.250	0.662	0.602	0.000

3.740	0.254	0.723	0.650	0.000
4.000	0.259	0.789	0.695	0.000
4.250	0.264	0.855	0.735	0.000
4.500	0.268	0.921	0.773	0.000
4.850	0.275	1.016	0.822	0.000
5.000	0.278	1.058	1.691	0.000
5.350	0.284	1.156	6.052	0.000
5.500	0.287	1.199	7.766	0.000

--
Link Name: Bioret#2
Link Type: Bioretention Facility
Downstream Link: None

Base Elevation (ft) : 100.00
Riser Crest Elevation (ft) : 100.50
Storage Depth (ft) : 0.50
Bottom Length (ft) : 145.0
Bottom Width (ft) : 2.0
Side Slopes (ft/ft) : L1= 3.00 L2= 3.00 W1= 3.00 W2= 3.00
Bottom Area (sq-ft) : 290.
Area at Riser Crest El (sq-ft) : 740.
(acres) : 0.017
Volume at Riser Crest (cu-ft) : 344.
(ac-ft) : 0.008

Infiltration on Bottom and Sideslopes Selected

Soil Properties
Biosoil Thickness (ft) : 1.50
Biosoil Saturated Hydraulic Conductivity (in/hr) : 2.00
Biosoil Porosity (Percent) : 20.00
Maximum Elevation of Bioretention Soil : 101.00
Native Soil Hydraulic Conductivity (in/hr) : 0.02

Underdrain Present
Orifice NOT Present in Under Drain

Riser Geometry
Riser Structure Type : Circular
Riser Diameter (in) : 6.00
Common Length (ft) : 0.000
Riser Crest Elevation : 100.50 ft

Hydraulic Structure Geometry

Number of Devices: 0

--
Link Name: Bioret#1
Link Type: Bioretention Facility
Downstream Link: None

Base Elevation (ft) : 100.00

Riser Crest Elevation (ft) : 100.50
Storage Depth (ft) : 0.50
Bottom Length (ft) : 75.0
Bottom Width (ft) : 1.0
Side Slopes (ft/ft) : L1= 3.00 L2= 3.00 W1= 3.00 W2= 3.00
Bottom Area (sq-ft) : 75.
Area at Riser Crest El (sq-ft) : 312.
(acres) : 0.007
Volume at Riser Crest (cu-ft) : 118.
(ac-ft) : 0.003

Infiltration on Bottom and Sideslopes Selected

Soil Properties
Biosoil Thickness (ft) : 1.50
Biosoil Saturated Hydraulic Conductivity (in/hr) : 2.00
Biosoil Porosity (Percent) : 20.00
Maximum Elevation of Bioretention Soil : 101.00
Native Soil Hydraulic Conductivity (in/hr) : 0.02

Underdrain Present
Orifice NOT Present in Under Drain

Riser Geometry
Riser Structure Type : Circular
Riser Diameter (in) : 6.00
Common Length (ft) : 0.000
Riser Crest Elevation : 100.50 ft

Hydraulic Structure Geometry

Number of Devices: 0

************************FLOOD FREQUENCY AND DURATION STATISTICS**********************

---------------------SCENARIO: PREDEVELOPED
Number of Subbasins: 1
Number of Links: 0

---------------------SCENARIO: POSTDEVELOPED
Number of Subbasins: 3
Number of Links: 3

********** Link: Windrose Pond ********** Link WSEL Stats
WSEL Frequency Data(ft)
(Recurrence Interval Computed Using Gringorten Plotting Position)

Tr (yrs)	WSEL Peak (ft)
1.05-Year	1.848
1.11-Year	1.922
1.25-Year	2.172
2.00-Year	2.722
3.33-Year	3.100

5-Year	3.412
10-Year	3.796
25-Year	4.667
50-Year	4.794
100-Year	4.908

*************Groundwater Recharge Summary ***************
Recharge is computed as input to Perlnd Groundwater Plus Infiltration in Structures

Total Predeveloped Recharge During Simulation

Model Element	Recharge Amount (ac-ft)
Subbasin: Ex. Cond	723.075
Total:	723.075

Total Post Developed Recharge During Simulation

Model Element	Recharge Amount (ac-ft)
Subbasin: Dev. Cond	133.210
Subbasin: WQ BSN #2	17.109
Subbasin: WQ BSN #1	8.555
Link: Windrose Pond	0.000
Link: Bioret#2	0.000
Link: Bioret#1	0.000
Total:	158.874

Total Predevelopment Recharge is Greater than Post Developed
Average Recharge Per Year, (Number of Years= 158)
Predeveloped: 4.576 ac-ft/year, Post Developed: 1.006 ac-ft/year

*************Water Quality Facility Data ***************

---------------------SCENARIO: PREDEVELOPED

Number of Links: 0

---------------------SCENARIO: POSTDEVELOPED

Number of Links: 3

********** Link: Windrose Pond **********

Infiltration/Filtration Statistics--------------------
Inflow Volume (ac-ft): 2963.10
Inflow Volume Including PPT-Evap (ac-ft): 2963.10
Total Runoff Infiltrated (ac-ft): 0.00, 0.00%
Total Runoff Filtered (ac-ft): 0.00, 0.00%
Primary Outflow To Downstream System (ac-ft): 2957.97
Secondary Outflow To Downstream System (ac-ft): 0.00
Percent Treated (Infiltrated+Filtered)/Total Volume: 0.00%

********** Link: Bioret#2 **********

Infiltration/Filtration Statistics--------------------
Inflow Volume (ac-ft): 283.71
Inflow Volume Including PPT-Evap (ac-ft): 287.94
Total Runoff Infiltrated (ac-ft): 0.00, 0.00%
Total Runoff Filtered (ac-ft): 262.53, 91.18%
Primary Outflow To Downstream System (ac-ft): 288.30
Secondary Outflow To Downstream System (ac-ft): 0.00
Percent Treated (Infiltrated+Filtered)/Total Volume: 91.18%

********** Link: Bioret#1 **********

Infiltration/Filtration Statistics--------------------
Inflow Volume (ac-ft): 112.71
Inflow Volume Including PPT-Evap (ac-ft): 114.28
Total Runoff Infiltrated (ac-ft): 0.00, 0.00%
Total Runoff Filtered (ac-ft): 105.02, 91.89%
Primary Outflow To Downstream System (ac-ft): 114.57
Secondary Outflow To Downstream System (ac-ft): 0.00
Percent Treated (Infiltrated+Filtered)/Total Volume: 91.89%

*************Compliance Point Results** *************

Scenario Predeveloped Compliance Subbasin: Ex. Cond

Scenario Postdeveloped Compliance Link: Windrose Pond

*** **Point of Compliance Flow Frequency Data** ***
Recurrence Interval Computed Using Gringorten Plotting Position

Predevelopment Runoff		Postdevelopment Runoff	
Tr (Years)	Discharge (cfs)	Tr (Years)	Discharge (cfs)
2-Year	1.082	2-Year	0.370
5-Year	1.439	5-Year	0.581
10-Year	1.873	10-Year	0.660
25-Year	2.549	25-Year	0.796
50-Year	3.360	50-Year	0.825
100-Year	3.797	100-Year	1.158
200-Year	3.945	200-Year	2.037
500-Year	4.129	500-Year	3.210

** Record too Short to Compute Peak Discharge for These Recurrence Intervals

**** **Flow Duration Performance** ****

Excursion at Predeveloped 50%Q2 (Must be Less Than or Equal to 0%):	-24.7%	PASS
Maximum Excursion from 50%Q2 to Q2 (Must be Less Than or Equal to 0%):	-24.7%	PASS
Maximum Excursion from Q2 to Q50 (Must be less than 10%):	-87.3%	PASS
Percent Excursion from Q2 to Q50 (Must be less than 50%):	0.0%	PASS

MEETS ALL FLOW DURATION DESIGN CRITERIA: PASS
----

5. Conveyance System Analysis and Design

The conveyance system design and analysis is not required for this permit application.

A detailed on-site conveyance analysis will be provided with future engineering plans submittals. The on-site conveyance system will be designed to meet the required 2016 KCSWDM design standards. Under both existing and developed conditions stormwater runoff from the project site is directed to the off-site conveyance system constructed with the Plat of Windrose to the east of the site as shown on the Preliminary Stormwater Plan. The off-site conveyance system was previously designed assuming 90% impervious surface coverage for the Wind Rose at Greenbridge (Lot 32, Plat of Windrose) site.

6. Special Reports and Studies

- "Wind Rose (incl. a Division of Greenbridge) Technical Information Report" by Goldsmith, Rev. October 2017
- "Wind Rose Preliminary Short Plat Level 1 Downstream Analysis and Preliminary Drainage Control Plan" by Goldsmith, Rev. December 2010
- "Greenbridge Level 1 Downstream Analysis" by Goldsmith, March 2004
- "Geotechnical Engineering Services, Wind Rose Neighborhood Development" by GeoEngineers, April 2016
- "Summary of Subsurface Conditions and Preliminary Geotechnical Considerations, KCHA Notch Properties near the Greenbridge Redevelopment Project", by GeoEngineers, August 2010
- "Soils and Geotechnical Report" by GeoEngineers, 2004

7. Other Permits

Additional future permits listed below are anticipated to be required with future building permit submittal.

- Commercial Building Permit
- Grading Permit
- Fire Hydrant Permit
- Water Permits
- Sewer Permit
- NPDES Permit
- Dry Utility Permits
- Traffic Plan Permit
- ROW Landscape Permit
- Lighting Permit
- Right of Way Use Permit – City of Seattle
- Sign Permit
- Urban Design and Art Plan Permit

8. ESC Analysis and Design

Not Applicable. No construction is proposed with the Site Development Permit application associated with this Technical Information Report.

9. Bond Quantities, Facility Summaries, and Declaration of Covenant

Not applicable. To be submitted with future building permit.

10. Operations and Maintenance Manual

Not Applicable. No construction is proposed with the Site Development Permit application associated with this Technical Information Report.

Appendix A

"Summary of Subsurface Conditions and Preliminary Geotechnical Considerations, KCHA Notch Properties near the Greenbridge Redevelopment Project", by GeoEngineers, August 18, 2010

GeoEngineers
2924 Colby Avenue
Everett, Washington 98201
425.252.4565

August 18, 2010

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

Attention: John Eliason

Subject: Wind Rose NEPA EA
Summary of Subsurface Conditions and Preliminary Geotechnical Considerations
KCHA Notch Properties near the Greenbridge Redevelopment Project
Southeast Corner 4th Avenue Southwest and SW Roxbury Street
Unincorporated King County, Washington
File No. 1329-003-14 Task 0600

INTRODUCTION AND PROJECT UNDERSTANDING

This letter presents a summary of the surface and subsurface conditions observed at the site based on information previously developed by GeoEngineers for the King County Housing Authority (KCHA). This letter also provides preliminary geotechnical considerations for future building support. Documents used in preparation of this letter are summarized in the following section.

The future Wind Rose mixed use redevelopment site was previously occupied by an approximate 2,800-square foot auto repair building, an approximately 2,600-square foot convenience store (previous service station), and the associated paved and unpaved parking and storage areas associated with these commercial uses. In addition, one single family residence is situated in the northeastern corner of the property with its surrounding landscaped and hardscape areas. Figure 1 (Vicinity Map) presents the approximate site location and Figure 2 illustrates the current site layout.

We understand that the site will be regraded for development. Future construction on the property is intended to consist of multi-family and/or high density single-family residences (maximum 80 units) with potential for a maximum of up to 10,000 square feet of retail/commercial development. Cuts on the order of 10 feet may be needed for below-grade structures. On-site infiltration of storm water is not planned at this time.

SUMMARY OF PRIOR STUDIES

GeoEngineers has provided geologic and geotechnical engineering reports for KCHA properties surrounding the Wind Rose site. GeoEngineers also provided environmental engineering services for the Wind Rose site. These reports include:

- “Report of Environmental Services, Underground Storage Tanks Removal, Former Altayar Auto Repair Also Known as KCHA “Notch” Property, 9606 4th Avenue Southwest”, dated July 2, 2010.
- “Phase I Environmental Site Assessment and Phase II Environmental Site Assessment, Lucky 7 Property, 9618 4th Avenue SW”, dated March 12, 2009.
- “Phase II Environmental Site Assessment, Altayar Auto Repair Property, 9606 4th Avenue Southwest”, dated December 18, 2008.
- “Focused Environmental Soil Sampling, KCHA Property Acquisition Near Greenbridge, 301 SW Roxbury”, dated May 31, 2007 and addendum memorandum titled “Revision to Table 1”, dated August 22, 2007.
- “Update Report, Geotechnical Engineering Services, Greenbridge Hope VI Redevelopment Project, King County, Washington”, dated January 12, 2007.
- “Report, Geotechnical Engineering Services, Greenbridge Development, Phase 1, King County, Washington”, dated July 21, 2004.
- “Final Revised Report, Preliminary Engineering Geologic and Geotechnical Engineering Services, Greenbridge Redevelopment, Unincorporated White Center, King County, Washington”, dated January 26, 2004.

TOPOGRAPHY

The Wind Rose site gently descends from about Elevation 398 feet along 4th Avenue SW to about Elevation 377 feet along the east property line. Slopes generally range between 2 and 5 percent; except for an east facing 15 to 25 percent slope that bisects roughly the middle of the site. There is a relatively small high spot near the middle of the site having 50 percent slopes. This area was created by recent demolition and excavation work and will be regraded during future development.

GEOLOGY

Geologic information indicates that the Wind Rose site is underlain by Vashon ice contact deposits. However Vashon ice contact and/or recessional outwash deposits are mapped along the eastern edge of the property. The ice contact/recessional outwash was deposited during the most recent glaciation of the region, which occurred 13,000 to 15,000 years ago, and is known as the Vashon stade of the Frasier glaciation. Recently placed fill (artificially placed soil) is present over the glacial deposits at several locations throughout the site.

Soils interpreted as Vashon ice contact deposits at the site generally consist of a medium dense to dense mixture of silt and sand that is commonly stratified. These sediments were deposited by meltwater on or adjacent to glacial ice. The on-site Vashon ice contact deposits are interpreted to have a maximum thickness of approximately 20 to 30 feet.

Vashon ice contact/recessional outwash deposits may include the ice contact unit described above, but may also include significant amounts of gravel. These sediments were deposited by meltwater on or adjacent to glacial ice, or by meltwater streams from the retreating ice during the later part of the Vashon stade. We interpret that ice contact deposits and recessional outwash interfinger and the distinction between the two types are unclear. The base of these deposits beneath the eastern portion of the site is interpreted to occur at an approximate elevation of 320 to 350 feet, where it is likely underlain by Vashon advance outwash.

Vashon advance outwash beneath the site generally consists of very dense sand with variable amounts of gravel and occasional layers of silty sand. These sediments were deposited by streams from the advancing ice sheet during the early part of the Vashon stade. Vashon advance outwash is not exposed at the surface within the site boundaries, but it was encountered in a deep boring located about 200 feet east of the site at a depth of about 30 feet.

SOILS

Soils of the upland plateau in the site vicinity are generally classified as part of the Alderwood Association, which is characterized by moderately well drained, undulating to hilly soils underlain by very slowly permeable glacial till. The Soil Conservation Service did not map specific soil series in the vicinity of the site; therefore, we have assigned specific soil series according to the mapped on-site geologic units as shown in the following table.

CORRELATION OF SOIL SERIES TO GEOLOGIC UNIT

Soil Series	Geologic Unit
Alderwood Series	Vashon Ice Contact Deposits
Indianola Series	Vashon Ice Contact Deposits and Vashon Ice Contact/ Recessional Outwash Deposits
Everett Series	Vashon Ice Contact/Recessional Outwash Deposits

SUBSURFACE SOIL CONDITIONS

The subsurface information summarized below is based on the geotechnical explorations adjacent to the site and environmental explorations located on site. Four general soil units were encountered in the explorations: topsoil, fill, glacial ice contact deposits, and advance outwash. Demolition of the existing commercial buildings and pavements has already taken place. There may be remnants of the demolition activities in the near surface soils, such as construction debris including concrete and asphalt rubble. The soils observed on site consist of the following.

- Sod/Topsoil. Six to 12 inches of sod and topsoil were encountered in several of the explorations.
- Fill. Fill soils were observed in all of the explorations. The fill ranged from 3 to 11½ feet thick but is typically 3 to 8 feet thick across the site. The fill consists of loose to medium dense sand with varying amounts of silt and gravel and trace organic debris. Fill is an unpredictable engineering material and may require reworking before it can be used to support loads.

- **Ice Contact Deposits.** Ice contact deposits were observed below the fill and consists of dense to very dense sand with silt and variable gravel and occasional layers of silty sand. Ice contact deposits appear to extend approximately 30 feet below the ground surface. Medium dense weathered ice contact deposits likely overlie the denser deposits in some areas. The contact with the denser materials is gradational. Although not overridden, these soils are competent support materials, and can generally be reused as fill across the site. Because of the high fines content, the ice contact may be difficult material to use as fill during wet weather.
- **Advance Outwash.** Advance outwash deposits exist below the ice contact deposits and were encountered approximately 30 feet below the ground surface. The outwash extended at least 76 feet below the ground surface as observed in a boring drilled about 200 feet east of the subject site. Advance outwash generally consists of very dense sand, sand with silt, and silty sand deposits with occasional medium stiff to stiff silt layers. Advance outwash deposits are overridden and provide excellent stability and support for building loads. Advance outwash will be good fill in all weather conditions.

HYDROGEOLOGY

Perched Groundwater

Localized zones of shallow groundwater were encountered within the ice contact deposits. These shallow zones of groundwater were typically encountered at depths of 10 to 33 feet beneath ground surface, in lenses of sand or silty sand that were underlain by sediments with low permeability. These shallow groundwater zones are perched on localized deposits of low permeability soils. Lateral movement of groundwater within these shallow perched zones may travel in topographically down-slope directions, but is generally expected to be limited because of their isolated occurrence. We anticipate that perched groundwater may exist at various intervals in response to seasonal changes in precipitation.

Regional Aquifers

Based on existing studies of the area, the shallowest regional or laterally extensive aquifer beneath the site occurs within the Vashon advance outwash sediments at an inferred elevation of 230 feet. Groundwater within the advance outwash aquifer beneath the site is inferred to flow to the northeast, toward the Duwamish valley. Deeper aquifers are also present below the site.

A review of records on file with the Washington State Department of Ecology (2002) did not identify any drinking water wells completed in the regional aquifers within a one mile radius of the site. The nearest wells extracting groundwater from the regional aquifers appear to be associated with the Highline wellfield, located approximately 1 to 4 miles south-southeast of the site.

GEOLOGIC HAZARDS

Erosion Hazards

Erosion of soils is a natural, ongoing physical process by which sediment is removed from topographic high points and transported down gradient by a variety of geomorphic processes. Removal of vegetation, modification of topography and unmanaged storm water runoff commonly contribute to increased erosion rates, although construction controls to limit erosion are routinely used on graded sites. In general, soils on steep slopes are more susceptible to erosion than soils on flat ground.

Areas potentially susceptible to erosion based on soil type and topography and include all soils on slopes greater than 40 percent, and all Alderwood series soils on slopes greater than or equal to 15 percent. There are no on-site erosion hazard areas identified in the King County Sensitive Areas Map Folio (1990), indicating that any such "hazard" is a localized condition.

In our opinion, while uncontrolled site work can result in localized erosion, the conditions at this site are typical of the region and the development plans are not likely to result in widespread erosion.

Steep Slope Hazards

Steep slope hazard areas are defined as those areas 40 percent or steeper with a vertical change of at least 10 feet (King County Code KZ21A, 2001). Steep slopes over 20 feet in vertical height are more severely regulated than those less than 20 feet in vertical height (King County Code KZ21A, 2001). The steep slope hazard areas are delineated based on the base survey map provided by Goldsmith and Associates (August 13, 2010).

There are no mapped steep slope hazard areas within or immediately adjacent to the Wind Rose site. There is an isolated slope located near the middle of the site that has approximately 50 percent slope, but it is less than 20 feet high and will be removed as part of the development plans.

Landslide Hazards

Landsliding is the slow to rapid, downslope movement of a mass that includes rock, soil and/or vegetative cover. The failures may occur as planar slides, block slides, rotational slumps, debris avalanches and mudflows. Landsliding usually occurs on steep slopes and is often initiated during periods of intense rainfall when the water table is high. Landsliding also can be initiated by removing lateral support from the toe of a slope or by surcharging the slope near the top.

Landslide hazard areas are defined as (1) any areas with slopes greater than 15 percent that are underlain by impermeable soils and that include springs or groundwater seepage; (2) landslides that have moved during the last 10,000 years; (3) areas "potentially unstable as a result of rapid stream incision, stream bank erosion or undercutting by wave action"; (4) areas showing "evidence of or is [sic] at risk from snow avalanches"; or (5) areas located on alluvial fans that are "presently subject to or potentially subject to inundation by debris flows or deposition of stream-transported sediments." (King County Code 21A, 2001).

No landslide hazards are identified on or adjacent to the site in the Sensitive Areas Map Folio (King County, 1990).

Seismic Hazards

The Puget Sound area is a seismically active region and has experienced thousands of earthquakes in historical time. Seismicity in this region is attributed primarily to the interaction between the Pacific, Juan de Fuca and North American plates. The Juan de Fuca plate is subducting beneath the North American Plate. Each year 1,000 to 2,000 earthquakes occur in Oregon and Washington. However, few of these are typically felt because the majority of the earthquakes are relatively minor, smaller than Richter magnitude 3. The site is assigned Site Class C base on the International Building Code.

Seismic hazards represent risk of injury or damage to humans and property resulting directly from earthquakes. Seismic hazard mechanisms include surface fault rupture, ground shaking and associated ground failure such as liquefaction and landsliding. Liquefaction is the loss of strength by loose, saturated soil when subjected to vibration or shaking.

Seismic hazard areas are defined as those areas "subject to severe risk of earthquake damage as a result of soil liquefaction in areas underlain by cohesionless soils of low density and usually in association with a shallow groundwater table or of other seismically induced settlement" (King County Code 21A, 2001). The fill and ice contact deposits along with intermittent perched groundwater creates a low potential for liquefaction at the site.

Recent maps of the east-west trending Seattle fault zone indicate that the nearest fault is located approximately 0.5 to 1 mile north of the site. Recent scientific articles suggest that fault movement may have occurred between 500 and 1,500 years ago. Based on the available data, surface fault rupture is, in our opinion, unlikely at the site.

The U.S. Geological Survey data indicate that seismic triggering of landslides is less common in the Pacific Northwest than in other seismically active areas partly because of the typically greater focus depth of earthquakes in the Pacific Northwest. In our opinion, seismic triggering of landslides on the site is unlikely.

Coal Mine Hazards

Coal mine hazard areas are defined as those areas "underlain or directly affected by operative or abandoned subsurface coal mine workings" (King County Code 21A, 2001). The principal issues regarding public safety and property damage related to abandoned coal mines include: (1) sinkholes and related gas emissions or concentrations; (2) trough subsidence; and (3) coal spoils. The Sensitive Areas Map Folio (King County, 1990) shows no coal mine hazard areas in the vicinity of the site.

PRELIMINARY GEOTECHNICAL CONSIDERATIONS

Foundations

We anticipate that development on the site can be supported on shallow spread footings founded on dense to very dense native glacial deposits or on recompacted structural fill overlying dense native soils. For shallow foundation support, we recommend that a preliminary allowable bearing value of 6,000 pounds per square foot (psf) be used for footings supported on the dense glacial deposits and 3,000 psf for footings supported on structural fill overlying dense native soils. The glacial soils may be excavated using conventional heavy duty excavators and dozers.

We recommend that perimeter footing drains be installed around the buildings.

Below-Grade Walls

Below-grade walls should be provided with backdrainage to reduce the potential for hydrostatic water pressure buildup. Backdrainage can be achieved by using free draining material against the walls with a perforated pipe adjacent to the footings to discharge collected water.

Temporary Cut Slopes

For planning purposes, excavations for buildings may be made using temporary cut slopes. We anticipate that temporary cut slopes may be made at 1.5H:1V (horizontal:vertical) in the dense to very dense ice contact deposits and at 2H:1V in overlying fill soil, unless significant groundwater seepage is encountered or if soil conditions are not as expected.

Reuse of On-site Native Soils

Existing clean fill and dense to very dense ice contact deposits is expected to be suitable for structural fill in areas requiring compaction to at least 95 percent of the maximum dry density (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. The ice contact deposits and other on-site soils planned for use as structural fill must be protected from moisture and soil stockpiles should be covered with plastic sheeting. It may be necessary to import sand and gravel with low fines content, such as WSDOT Gravel Borrow, to achieve adequate compaction for support of pavement areas, floor slabs and structures for wet weather construction. We therefore recommend that for planning purposes the project include importing all structural fill for wet weather construction where compaction to at least 90 percent of MDD is required.

Infiltration

The capacity for onsite infiltration of storm water is low based on the near surface soil conditions observed in our explorations. Additional explorations are needed to characterize the infiltration characteristics of the subsurface soils.

LIMITATIONS

This summary letter has been prepared for use by King County Housing Authority and their authorized agents for preliminary geotechnical engineering planning purpose for the subject site. Our services were performed in accordance with the terms of our contract with KCHA. Within the limitations of scope, schedule and budget, our services have been executed in accordance with the generally accepted geotechnical engineering practices in this area at the time this report was prepared. No warranty or other conditions, express or implied, should be understood.

Sincerely,
GeoEngineers, Inc.

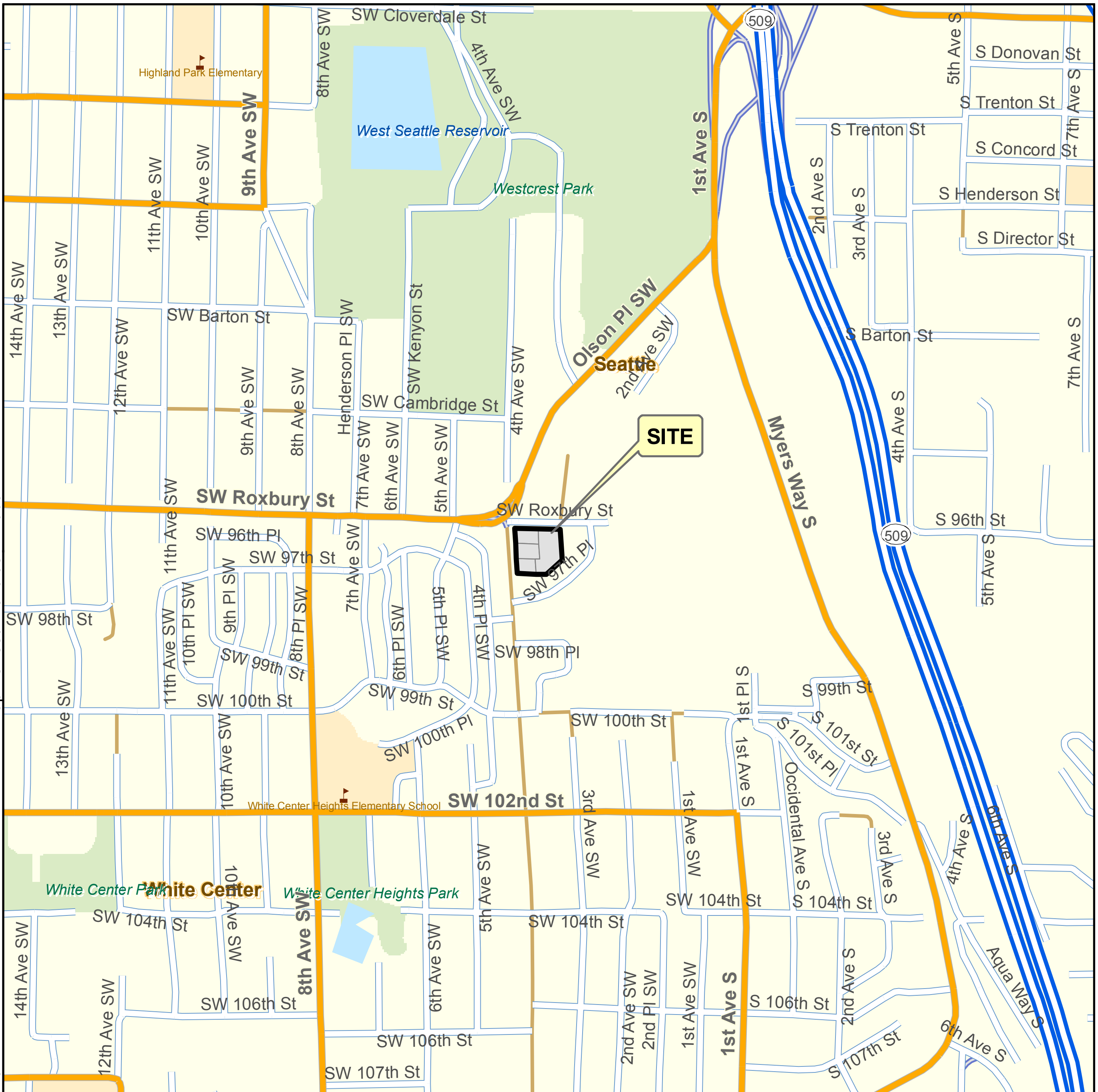
Robert C. Metcalfe, PE, LEG
Associate

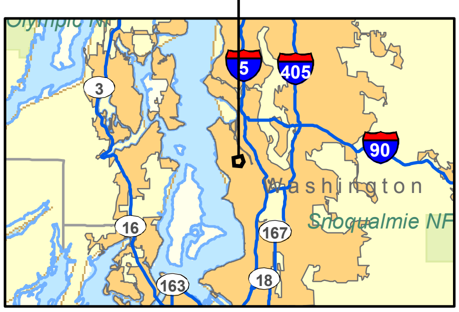
ROBERT C. METCALFE
STATE OF WASHINGTON
32807
REGISTERED PROFESSIONAL ENGINEER

8/13/10

Attachments: Figure 1. Vicinity Map
Figure 2. Site Layout

DPC:RCM:ta
https://projects.geoengineers.com/sites/0132900314/Final/Geotechnical Summary Letter.docx





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Feet

Notes:
1. The locations of all features shown are approximate.
2. This drawing is for infomation 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.
Reference: ESRI Data & Maps, Street Maps 2005; King County parcels.

Vicinity Map

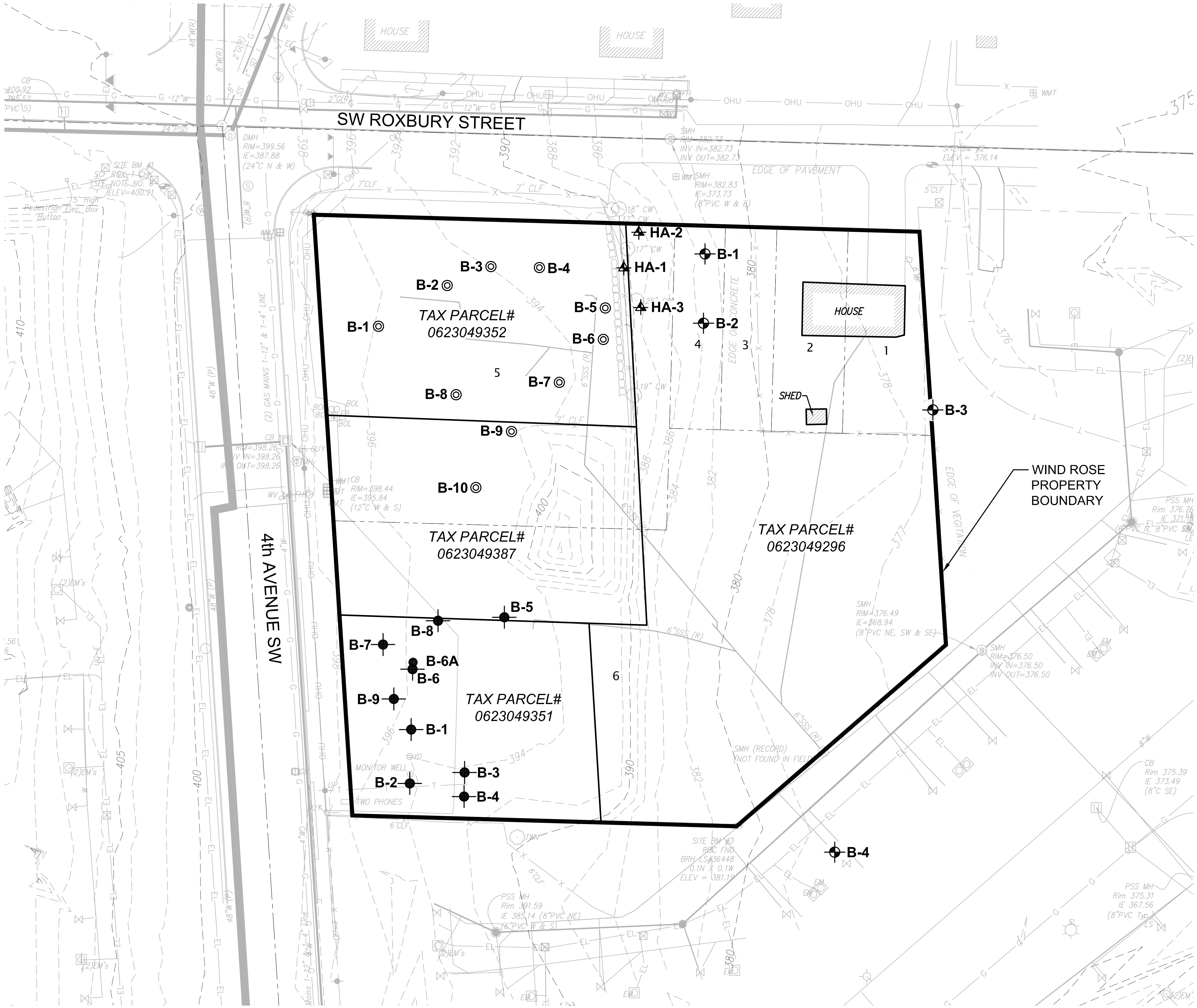
King County Housing Authority
Wind Rose Preliminary Short Plat
King County, Washington

GEOENGINEERS

Figure 1

Map Revised: 2010.08.12 -dbc

Path: \\evt\Projects\1\1329003\14\GIS\132900314_F1_VM.mxd



Legend

B-1	Geoprobe by GeoEngineers (November 2008)
B-6A	Boring by GeoEngineers (November 2008)
B-1	Geoprobe by GeoEngineers (November 2007)
B-1	Geoprobe by GeoEngineers (May 2007)
HA-1	Hand Auger by GeoEngineers (May 2007)
	Existing Parcel
	Proposed Parcel

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FEET

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. can not 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.

Reference: Base CAD file "02075-15X01.dwg" provided by Goldsmith & Associates on 8/13/2010.

Site Layout

KCHA Wind Rose
King County, Washington

GEOENGINEERS

Figure 2

W:\REDMOND\PROJECTS\13290031\4 TASK 600\CAD\132900314 TASK 600 FIGURE 2 MODIFIED BY TANDERSON ON AUGUST 17, 2010 - 11:46

Appendix B

Geotechnical Report, “Wind Rose Neighborhood Development, King County, Washington,” for King County Housing Authority, by GeoEngineers, April 28, 2016

Geotechnical Engineering Services

Wind Rose Neighborhood Development
King County, Washington

for
King County Housing Authority

April 28, 2016

GeoEngineers
Earth Science + Technology

Geotechnical Engineering Services

Wind Rose Neighborhood Development
King County, Washington

for
King County Housing Authority

April 28, 2016

GeoEngineers

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Redmond, Washington 98052
425.861.6000

Geotechnical Engineering Services

Wind Rose Neighborhood Development
King County, Washington

File No. 1329-009-01

April 28, 2016

Prepared for:

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

Attention: John Eliason, Kevin Preston

Prepared by:

GeoEngineers, Inc.
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Herbert R. Pschunder, PE

Herbert R. Pschunder, PE
Senior Geotechnical Engineer

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CEB:HRP:RCM:nld

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LIST OF FIGURES

APPENDICES

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 though 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 recompacted 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 ⅜ 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 ½ 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 recompacted 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)	WSDOT[1] 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.

Rockeries

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.

Rockeries may be used in both cut and fill areas. Rockeries 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.

Rockeries 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

International Code Council, 2012, International Building Code.

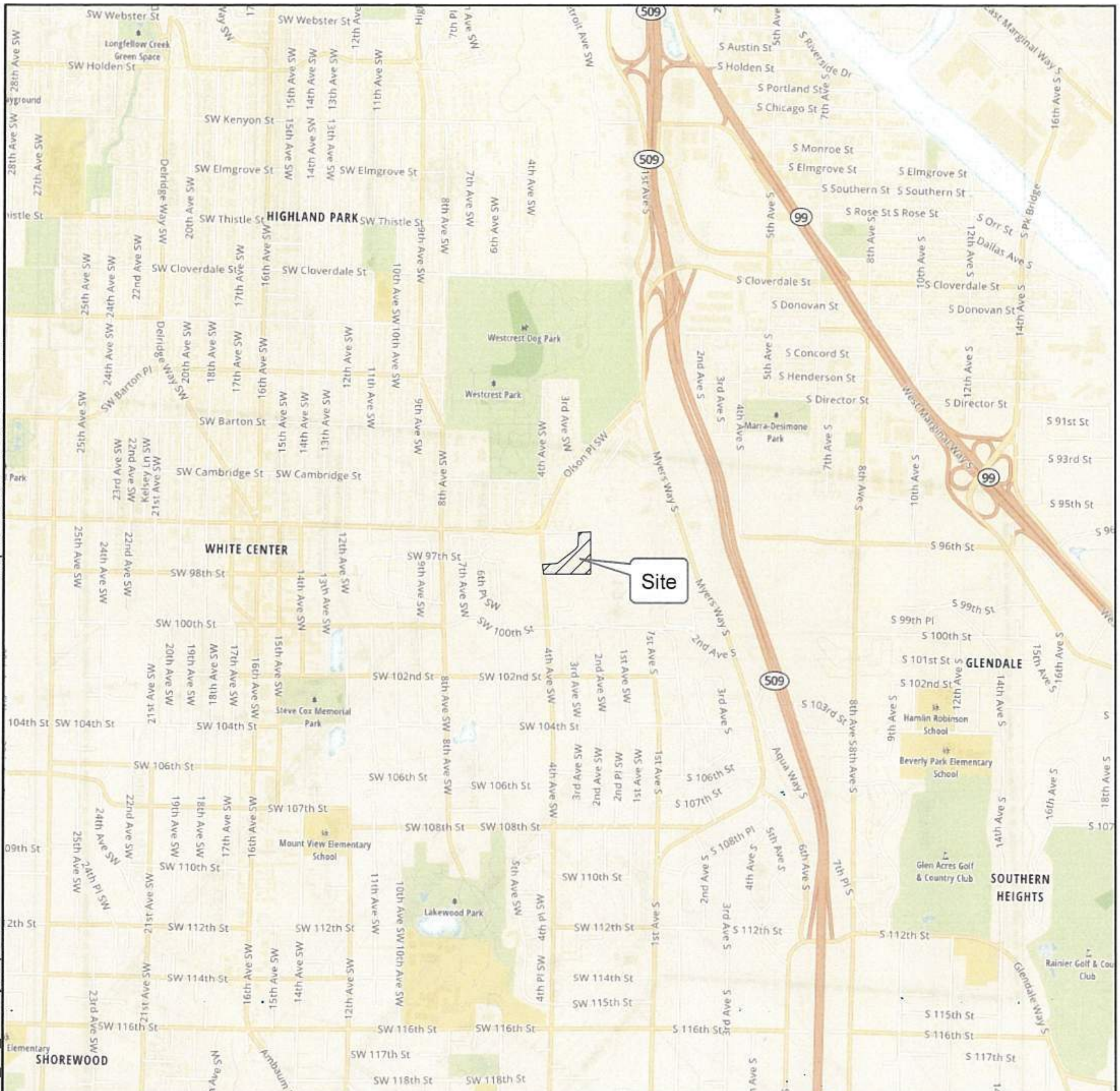
Puget Sound Action Team, 2005, "Low Impact Development Technical Guidance Manual for Puget Sound."

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.

GeoEngineers



N
W E
S

2,000 0 2,000

Feet

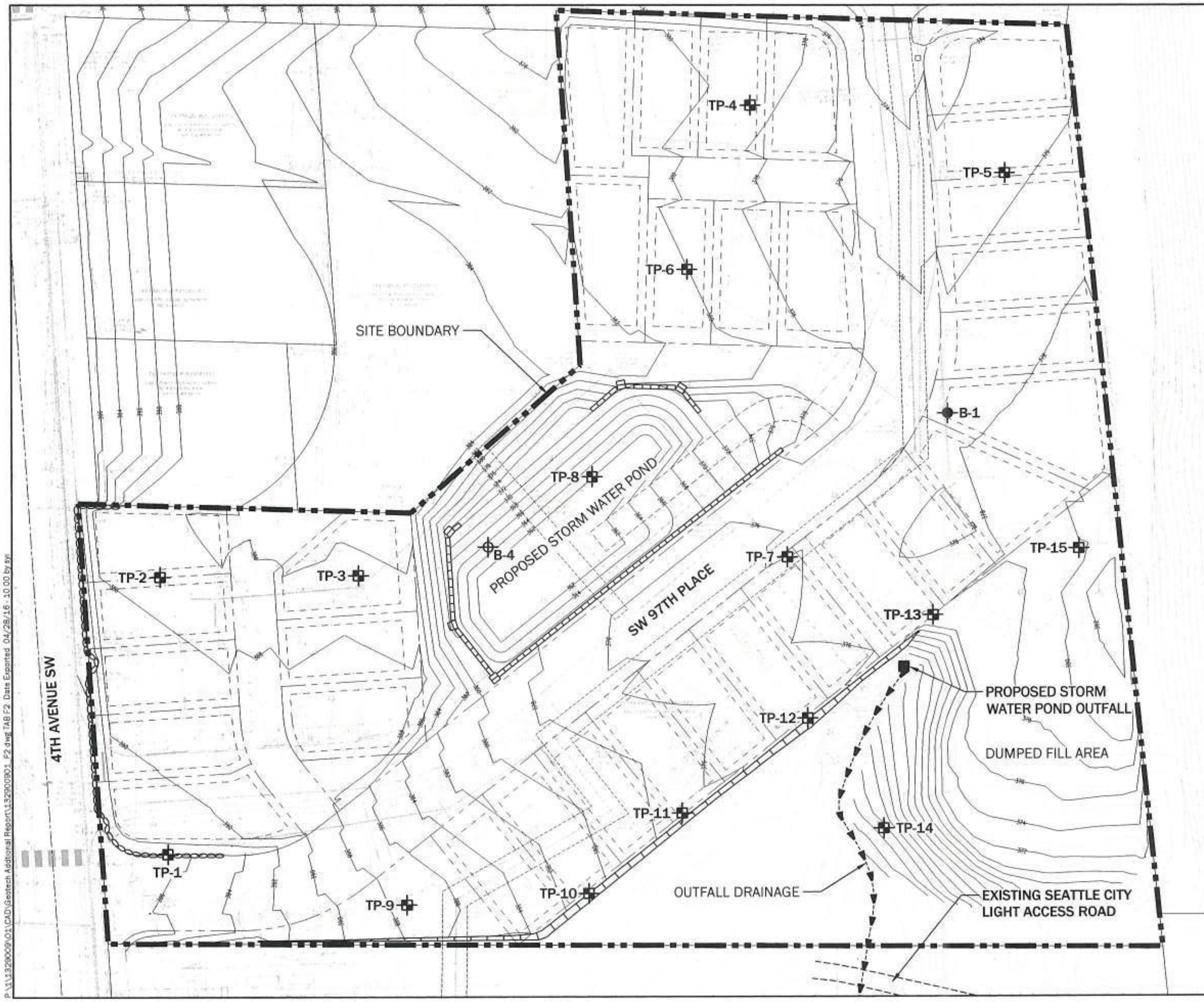
Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers,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

Office: Redmond Path: P:\1\1329009\GIS\132900901_F1_VicinityMap.mxd Map Revised: 4/28/2016 EL



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

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.

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

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Legend

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

N W E S

100 0 100

Feet

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.

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

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Figure 3

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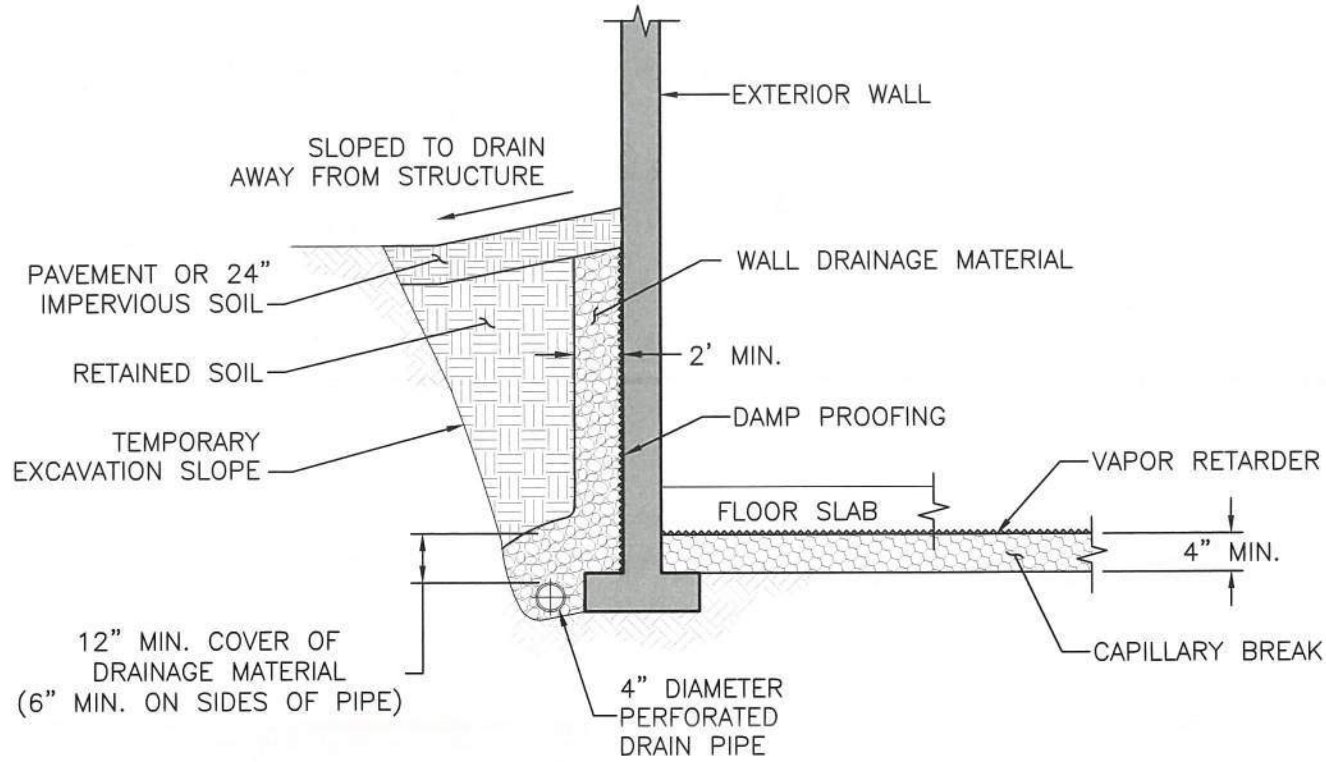


NOT TO SCALE

Materials:

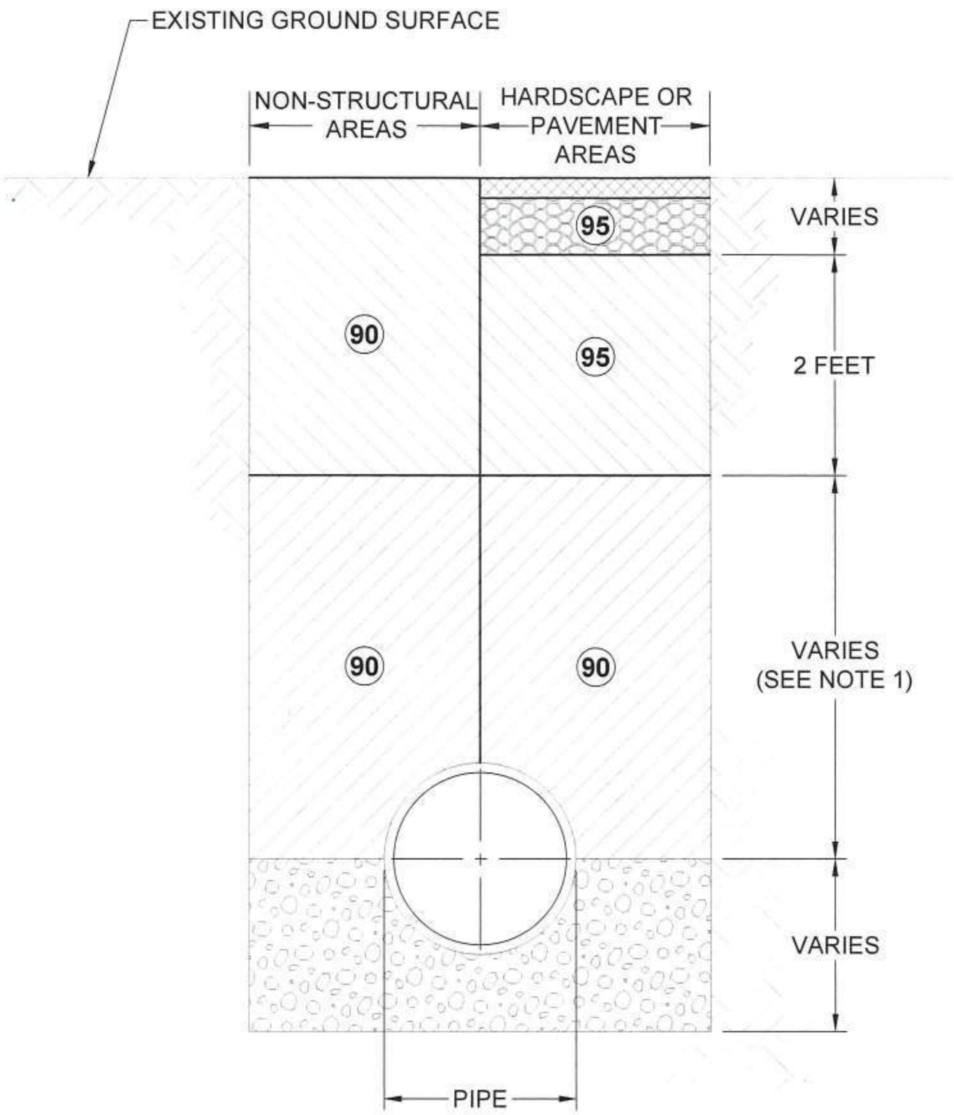
A. □ ALL DRAINAGE MATERIAL: Ma□ consist of □ashed □□" to No. □ pea gra□el. Alternati□el□ the □all drainage material ma□ consist of "Gra□el Backfill for Drains" per □ SDOT Standard Specification 9-□□.12(□), surrounded □ith a non-□o□en geote□tile such as Mirafi 1□□N (or appro□ed e□ui□alent).

B. RETAINED SOIL: Should consist of structural fill, either on-site soil or imported. The backfill should be compacted in loose lifts not e□ceeding □ inches. □ all backfill supporting building floor slabs should consist of imported sand and gra□el per □ SDOT Standard Specification 9-□□.1□ or Cit□ of Seattle T□pe 17 compacted to at least 95 percent ASTM D1557. Backfill not supporting building floor slabs, side□alks, or pa□ement should be compacted to 9□ to 92 percent of the ma□imum dr□ densit□, per ASTM D1557. Backfill supporting side□alks or pa□ement areas should be compacted to at least 95 percent in the upper t□o feet. Onl□ hand-operated e□uipment should be used for compaction □ithin 5 feet of the □alls and no hea□□ e□uipment should be allo□ed □ithin 5 feet of the □all.

C. CAPILLARY BREAK: Should consist of at least □ inches of clean crushed gra□el □ith a ma□imum si□e of 1-1□2 inches and negligible sand or fines.

D. PERFORATED DRAIN PIPE: Should consist of a □-inch diameter perforated hea□□-□all solid pipe (SDR-□5 PVC) or rigid corrugated pol□eth□lene pipe (ADS N-12) or e□ui□alent. Drain pipes should be placed □ith □.5 percent minimum slopes and discharge to the storm □ater collection s□stem.

Wall Drainage and Backfill	
Wind Rose Development King County, Washington	
GEOENGINEERS	**Figure 4**



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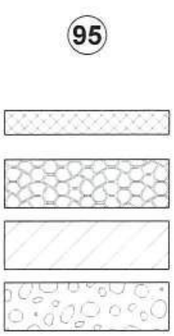
LEGEND:



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

GEOENGINEERS

Figure 5

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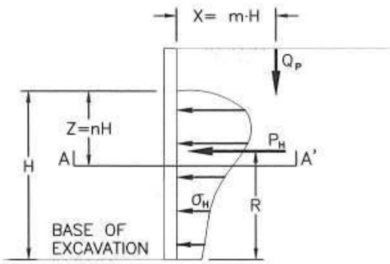
DEFINITIONS:

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.[illegible]2 Naval Facilities Engineering Command, September 19[illegible] (NAVFAC DM 7.[illegible]2).
2. Lateral earth pressures from surcharge should be added to earth pressures presented on Figures [illegible] and 9 as appropriate.
[illegible]. See report text for where surcharge pressures are appropriate.

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



FOR $m \le 0.4$

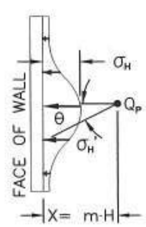
$$\sigma_H = \frac{0.28Q_P n^2}{H^2(0.16+n^2)^3}$$

FOR $m > 0.4$

$$\sigma_H = \frac{1.77Q_P m^2 n^2}{H^2(m^2+n^2)^3}$$

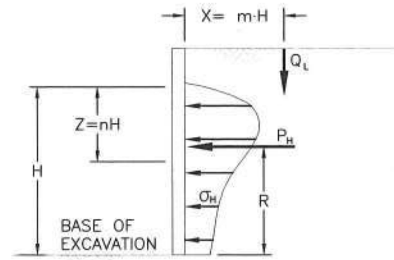
$$\sigma_H' = \sigma_H \cos^2(1.1\theta)$$

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 $m \le 0.4$

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

$$P_H = 0.55Q_L$$

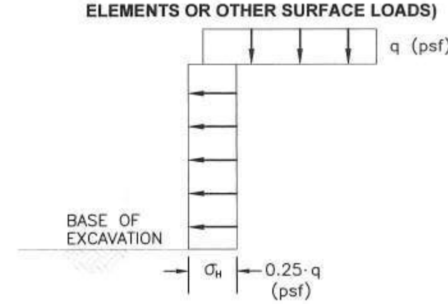
FOR $m > 0.4$

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

$$P_H = \frac{0.64Q_L}{(m^2+l)}$$

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

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Figure 6

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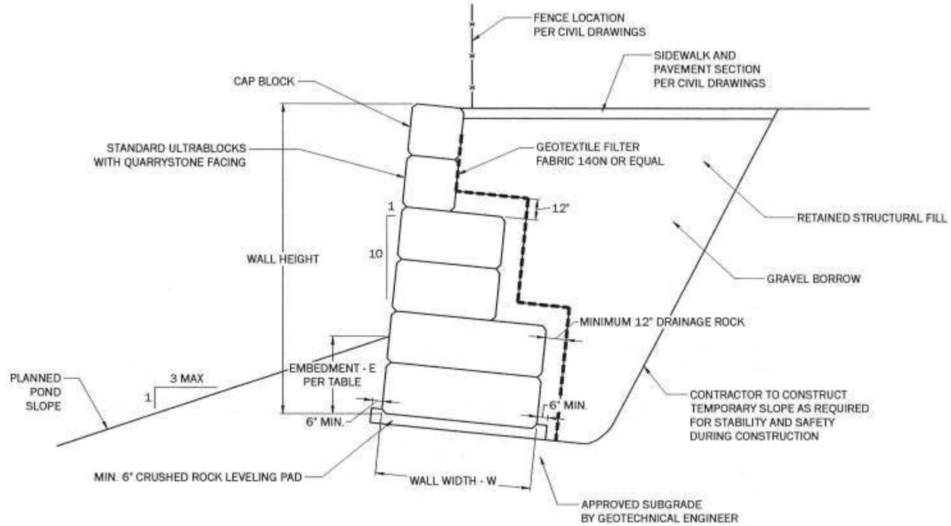
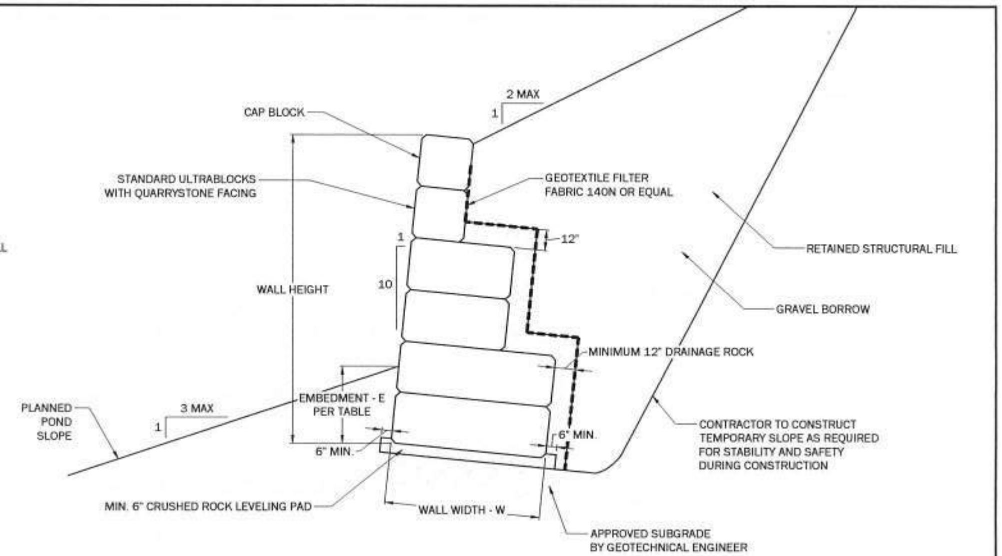


WALL CROSS SECTION FOR DETENTION POND (LEVEL BACKSLOPE)

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

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)

Typical Gravity Block Retaining Walls Section and Notes

Wind Rose Development
King County, Washington

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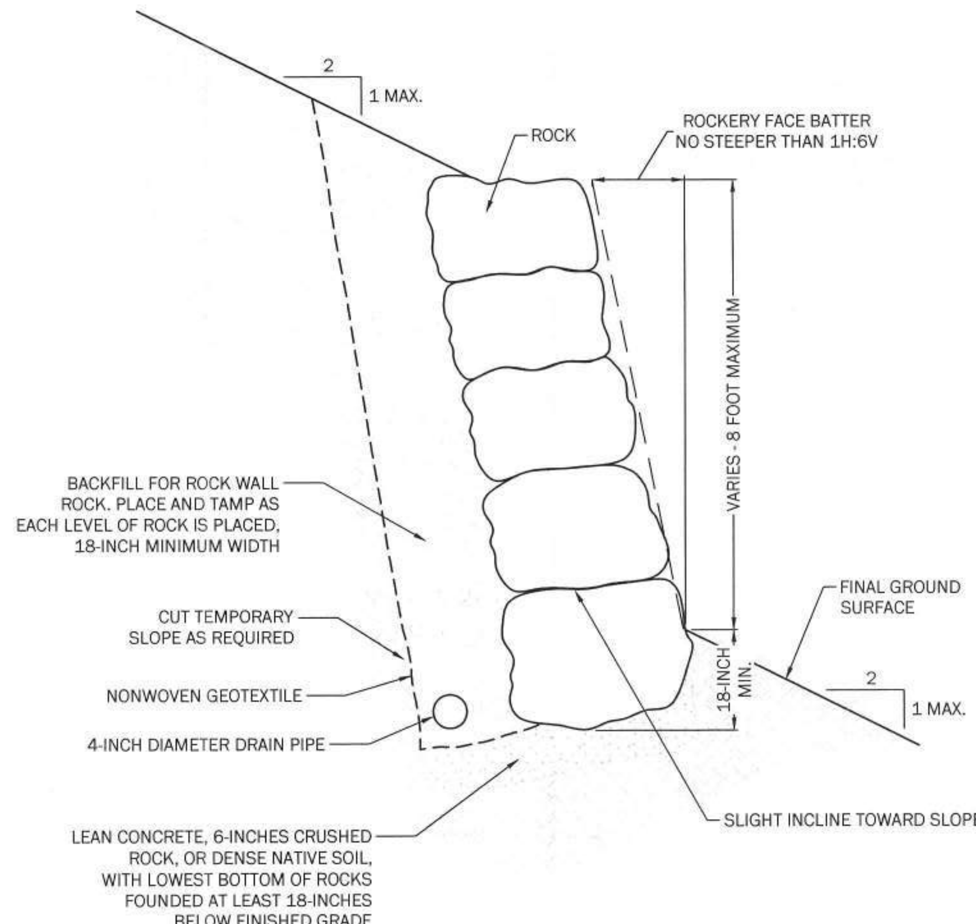
Figure 7

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 serves 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
NO SCALE

Rockery at Cut Slope	
Wind Rose Development King County, Washington	
GeoEngineers	Figure 8

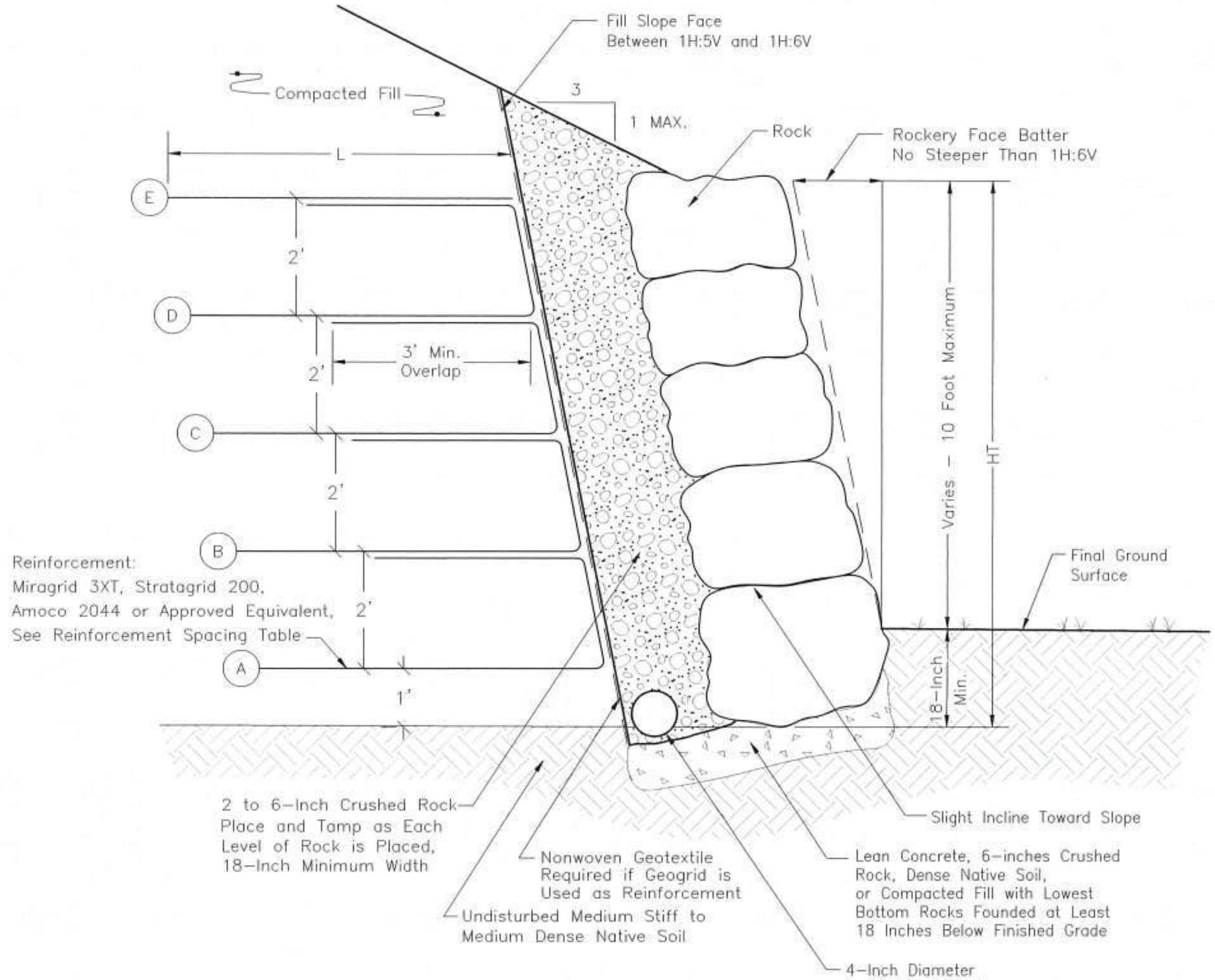
REINFORCEMENT SPACING TABLE

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

1. See the Rockery Construction Specifications on Figure 1 for additional requirements for Rockeries.
2. 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.
3. 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.
4. 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.
5. 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.



ROCKERY AT FILL SLOPE

NO SCALE

Rockery at Fill Slope

Wind Rose Development
King County, Washington

GEOENGINEERS | Figure 9

APPENDICES

GeoEngineers

Appendix B

Geotechnical Report, “Wind Rose Neighborhood Development, King County, Washington,” for King County Housing Authority, by GeoEngineers, April 28, 2016

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 put 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 GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% RETAINED ON NO. 200 SIEVE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	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
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS
				SP	POORLY-GRADED SANDS, GRAVELLY SAND
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% PASSING NO. 200 SIEVE	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
	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.

ADDITIONAL MATERIAL SYMBOLS

SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
	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

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.

KEY TO EXPLORATION LOGS

GEOENGINEERS

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 Testing Sample	Sample Name Testing	Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
					TS		Approximately 6 inches topsoil (very loose, moist)		Probe depth = 12 to 14 inches
395	1				SM		Brown silty fine to medium sand with occasional gravel, roots (medium dense, moist) (fill)		
394	2	X	1 MC					15	
393	3				SP-SM		Brown-gray fine to medium sand with silt and gravel (dense, moist) (outwash deposits)		
392	4	X	2 MC					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.

Redmond: Date:3/3/16 Path:P:\1\1329009\GINT\132900901.GPJ DBTemplate/LibTemplate:GEOENGINEERS8.GDT/GEI8_TESTPIT_1P_GEOTEC

Log of Test Pit TP-1

GEOENGINEERS

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: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
	0				SM		Gray silty fine to medium sand, grass and roots (medium dense, moist) (topsoil)		Probe depth = 2 to 4 inches
393	1				SM		Dark brown silty fine to medium sand, roots (loose, moist) (fill)		
392	2	☒	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	☒	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	☒	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

GEOENGINEERS

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

Date Excavated: 2/10/2016

Equipment: Komatsu PC 120 Excavator

Logged By: DTM

Total Depth (ft) 5.5

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
					SM		Dark brown silty fine to medium sand, roots (loose, moist) (fill)		Probe depth = 8 to 10 inches
383	1				SM		Brown silty fine to medium sand, occasional roots (loose, moist)		
382	2	☒	1 / SA					21	Probe depth = 6 to 12 inches %F = 29
381	3								
380	4	☒	2 / MC		SP-SM		Gray fine to medium sand with silt and gravel (dense, moist) (outwash deposits)	11	Probe depth = ½ 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.

Redmond: Date:3/3/16 Path:P:\1\1329009\GINT\132900901.GPJ DBTemplate/LibTemplate:GEOENGINEERS8.GDT/GEI8_TESTPIT_1P_GEOTEC

Log of Test Pit TP-3

GEOENGINEERS

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

GeoEngineers

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

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

Logged By: DTM
Total Depth (ft) 7.5

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
					SM		Gray silty fine to medium sand with gravel, roots (medium dense, moist) (fill)		Probe depth = 1 to 3 inches
375	1				SM		Brown silty fine to medium sand with occasional gravel (loose to medium dense, moist)		
374	2	X	1 / SA		SM		Grayish, brownish orange silty fine to medium sand (medium dense, moist) (outwash deposits)	20	Probe depth = 4 to 6 inches %F = 34
373	3								
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.

Redmond: Date:3/3/16 Path:P:\1\1329009\GINT\132900901.GPJ DBTemplate/LibTemplate:GEOENGINEERS8.GDT/GEI8_TESTPIT_1P_GEOTEC

Log of Test Pit TP-5

GEOENGINEERS

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

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

Logged By: DTM
Total Depth (ft) 4

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

GEOENGINEERS

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 Testing Sample	SAMPLE Sample Name / Testing	Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
					SM		Dark brown silty fine to medium sand, roots (loose, moist) (fill)		Probe depth = 6 to 8 inches
375	1								
					SM		Light brown silty fine to medium sand with occasional gravel, roots (loose, moist)		
374	2	⊠	1 / MC					15	Probe depth = 8 to 10 inches
373	3								
372	4	⊠	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	⊠	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

GEOENGINEERS

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

Date Excavated: 2/10/2016

Equipment: Komatsu PC 120 Excavator

Logged By: DTM

Total Depth (ft) 17

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
	0				SM		Gray silty fine to medium sand, roots (loose to medium dense, moist) (fill)		Probe depth = 3 to 5 inches
376	1				SM		Gray-brown/orange silty fine to medium sand (loose to medium dense, moist)		
375	2	X	1 / MC		SM		Brown silty fine to medium sand (dense, moist) (outwash deposits)	15	Probe depth = 1 to 2 inches
373	4	X	2 / SA		SM		Gray silty fine sand (dense, moist)	16	Probe depth = ½ to 1 inch %F = 47
364	13	X	3 / SA		SP-SM		Gray fine sand with silt (dense, wet) Moderate groundwater seepage observed from 13 to 17 feet	19	%F = 10
362	15						Becomes wet		
360	17	X	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

GEOENGINEERS

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

GeoEngineers

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

Equipment: Komatsu PC 130 Excavator

Logged By: DTM

Total Depth (ft) 4

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
					SM		Dark brown silty fine to medium sand, roots (loose, moist) (fill)		Probe depth = 8 to 10 inches
367	1				SM		Gray silty fine to medium sand (medium dense, moist) (outwash deposits)		
366	2	X	1 MC					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

GEOENGINEERS

Project: Wind Rose Development

Project Location: King County, Washington

Project Number: 1329-009-01

Figure A-11

Date Excavated: 2/10/2016

Equipment: Komatsu PC 130 Excavator

Logged By: DTM

Total Depth (ft) 6

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
	0				SM		Dark brown silty fine to medium sand (loose, moist) (fill)		Probe depth = 4 to 8 inches
366	1				SM		Grayish brown silty fine to medium (medium dense, moist)		
					SM		Gray silty fine to medium sand (medium dense, moist) (outwash deposits)		
365	2	☒	1 / MC					15	Probe depth = 2 to 5 inches
364	3								
363	4	☒	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

GeoEngineers

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: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
	0				SM		Dark brown silty fine to medium sand (loose, moist) (fill)		Probe depth = 12 to 16 inches
361	1				SM		Grayish orange/brown silty fine to medium sand with occasional gravel (medium dense, moist)		
360	2	X	1 MC					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

GEOENGINEERS

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

Date Excavated: 2/10/2016

Equipment: Komatsu PC 120 Excavator

Logged By: DTM

Total Depth (ft) 11

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
368	0				SM		Dark brown silty fine to medium sand, roots (loose, moist) (fill)		Probe depth = 10 to 16 inches
367	1				SM		Orangish gray silty fine to medium sand, roots (loose, moist)		
366	2	X	1 SA					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

GEOENGINEERS

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

Date Excavated: 2/10/2016

Equipment: Komatsu PC 120 Excavator

Logged By: DTM

Total Depth (ft) 4

Elevation (feet)	Depth (feet)	SAMPLE Testing Sample	SAMPLE Sample Name Testing	Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
					TS		Dark brown topsoil with roots (loose, moist)		
349	1				SP-SM		Orange fine to medium sand with silt, roots (medium dense, moist) (outwash deposits)		
348	2	☒	1 MC					8	
347	3				SP-SM		Gray silty fine to medium sand with silt (dense, moist)		
346	4	☒	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

GEOENGINEERS

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
Equipment: Komatsu PC 120 Excavator

Logged By: DTM
Total Depth (ft) 4

Elevation (feet)	Depth (feet)	SAMPLE Testing Sample	SAMPLE Sample Name / Testing	Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
					SM		Dark brown silty fine to medium sand (loose, moist) (topsoil)		
377	1				SM		Grayish orange silty fine to medium sand (medium dense, moist) (outwash deposits)		
376	2				SM		Gray silty fine to medium sand (dense, moist)		
375	3								
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

GeoEngineers

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

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

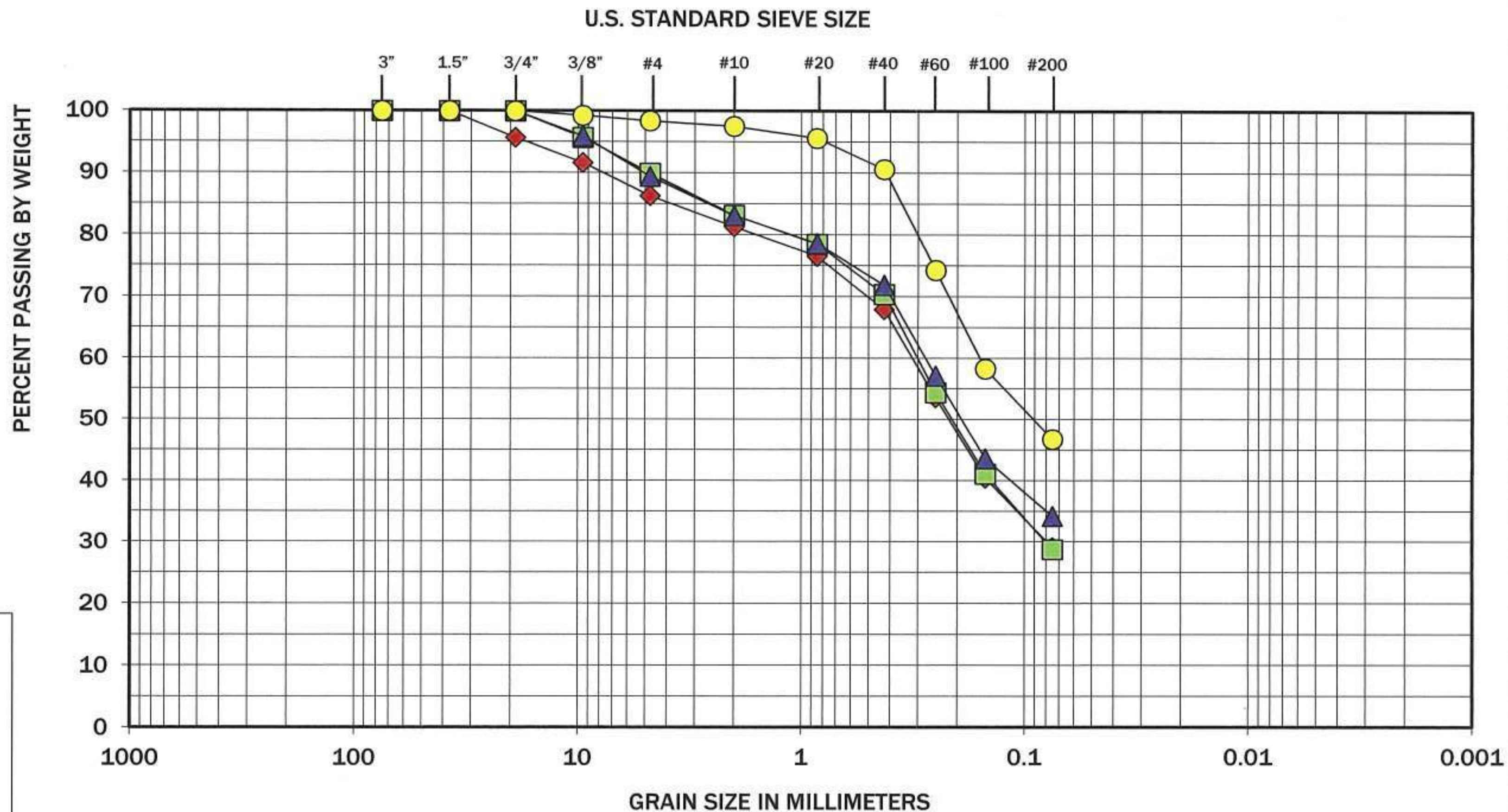
All soil samples obtained from the test pits were visually classified in the field and/or in our laboratory using a system based on the Unified Soil Classification System (USCS) and ASTM classification methods. ASTM test method D 2488 was used to visually classify the soil samples, while ASTM D 2487 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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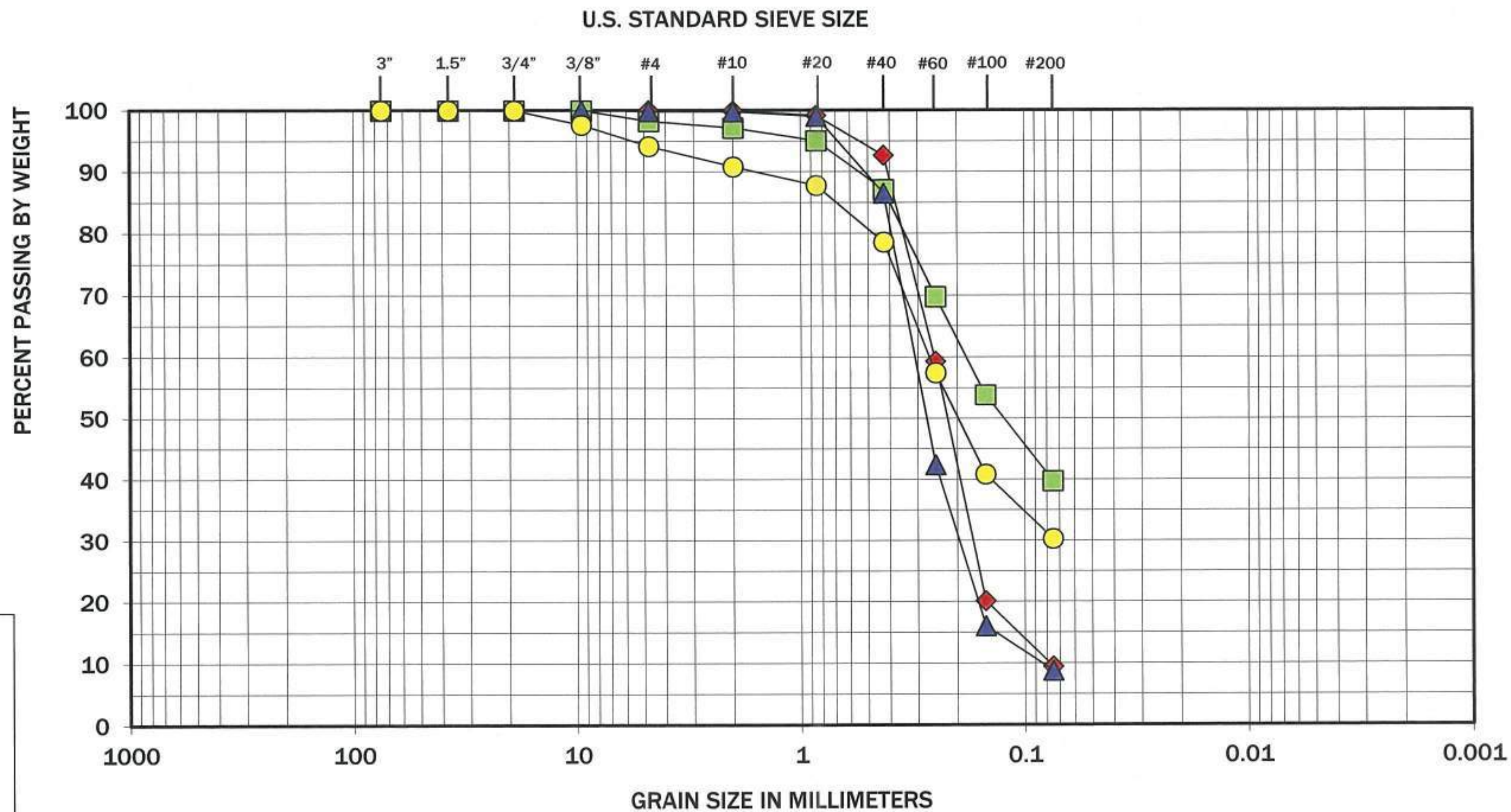
The grain size analysis results were obtained in general accordance with ASTM D 6913.

Sieve Analysis Results

Wind Rose Development
King County, Washington

GEOENGINEERS

Figure B-1



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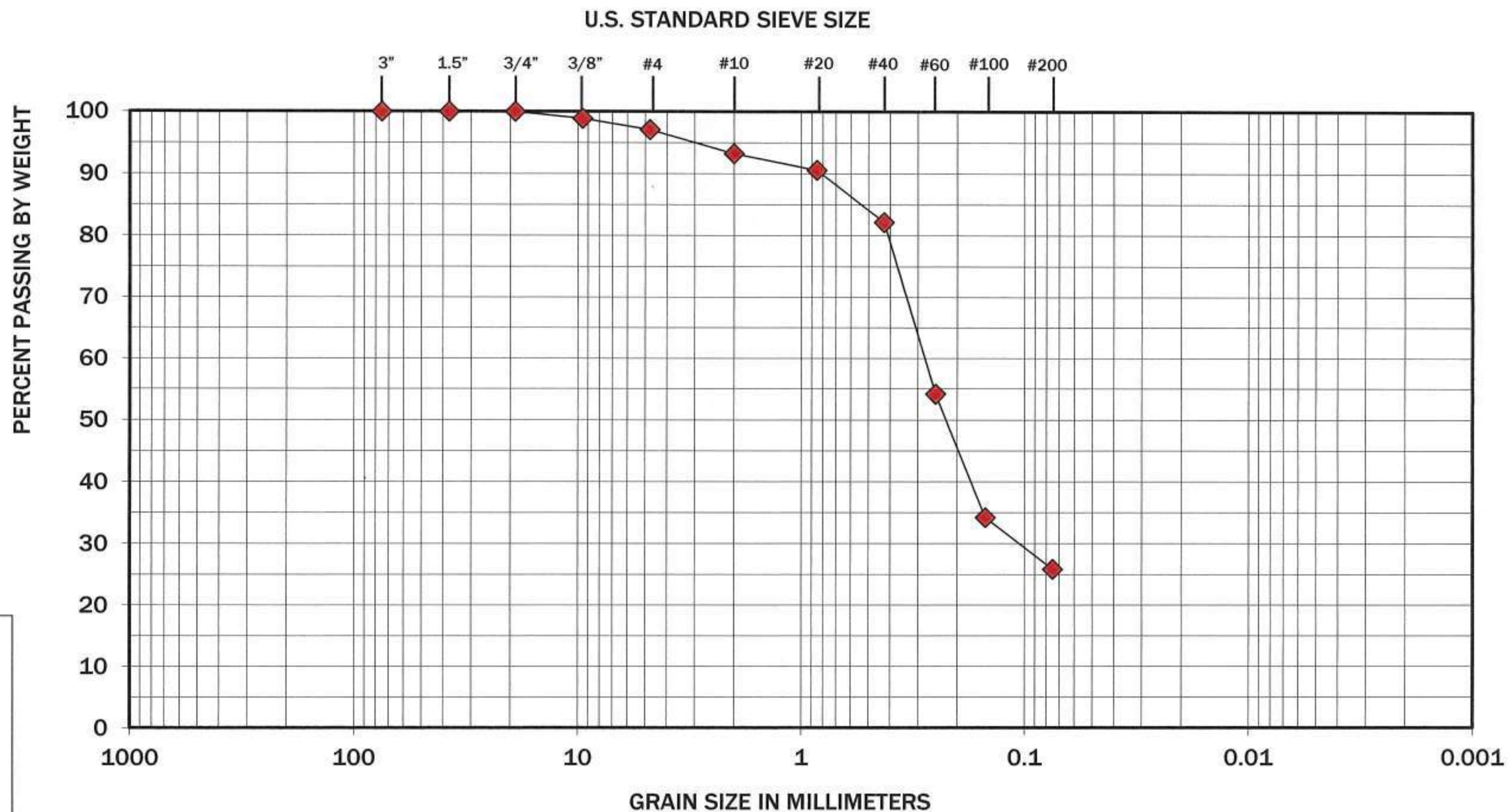
The grain size analysis results were obtained in general accordance with ASTM D 6913.

Sieve Analysis Results

Wind Rose Development
King County, Washington

GEOENGINEERS

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.

Sieve Analysis Results

Wind Rose Development
King County, Washington

GEOENGINEERS

Figure B-3

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 Interval	Number	Recovered (in)	Blows/foot	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, lbs/ft^3	OTHER TESTS AND NOTES
375	0							SM	Brown silty fine sand (loose, moist) (fill?)			
								SP-SM	Brown fine sand with silt (dense, moist) Becomes dense			
370	5		1	12	39					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			
								SP	Gray brown fine sand (very dense, moist)			
	35											

Note: See Figure A-2 for explanation of symbols

LOG OF BORING B-1

GeoEngineers

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

Figure: A-3

1329-003-00 GEI_GTBORING 2.1.0 P:\1\1329003\00\FINALS\1329003.GPJ GEIV2_2.GDT 8/27/03

Elevation feet	Depth feet	Interval	Number	Recovered (in)	Blows/foot	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
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)

GeoEngineers

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: Interval	SAMPLES: Recovered (in)	SAMPLES: Blows/foot	SAMPLES: Sub-Sample Number	SAMPLES: Testing	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Sheen	Headspace Vapor PID (ppm)	NOTES
0								SM	Brown to dark brown silty fine sand with occasional gravel and bricks (moist) (fill)			
					CA					SS	2.2	
										SS	1.8	
								ML	Brown silt with fine sand (moist)			
5												
									becomes mottled brown to gray	NS	1.9	
									becomes wet			
								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										NS	2.4	
								ML	Brown sandy silt (moist)			
										NS	1.7	
								SM	Brown silty fine to medium sand (moist)			
					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

GeoEngineers

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

GEOENGINEERS
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.

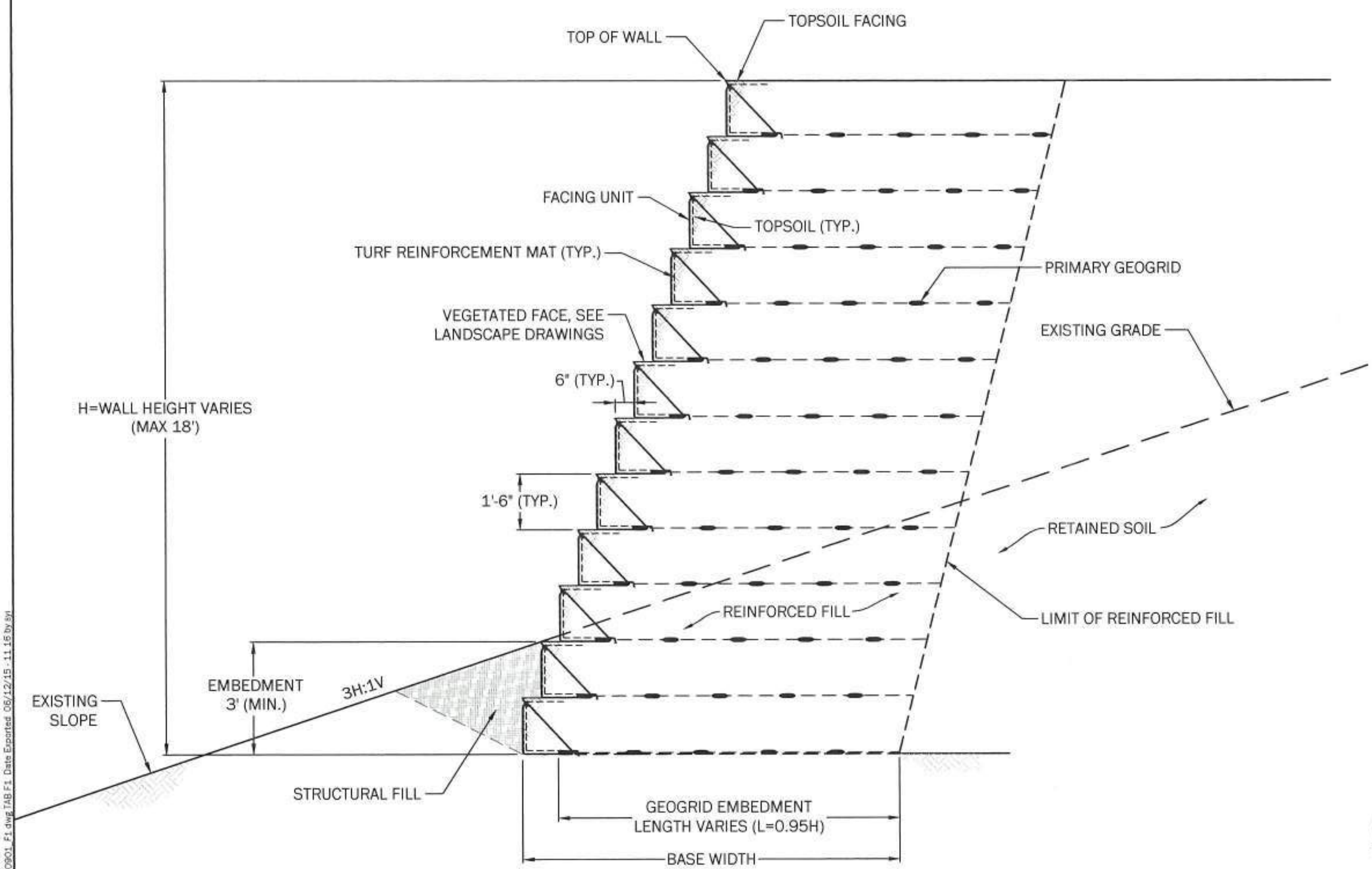
Herbert R. Pschunder, PE
Senior Geotechnical Engineer

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)



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

TYPICAL MSE WALL SECTION, LOTS 15-24

NOT TO SCALE

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.

MSE Wall Section

Wind Rose Development

King County, Washington

GEOENGINEERS

Figure 1

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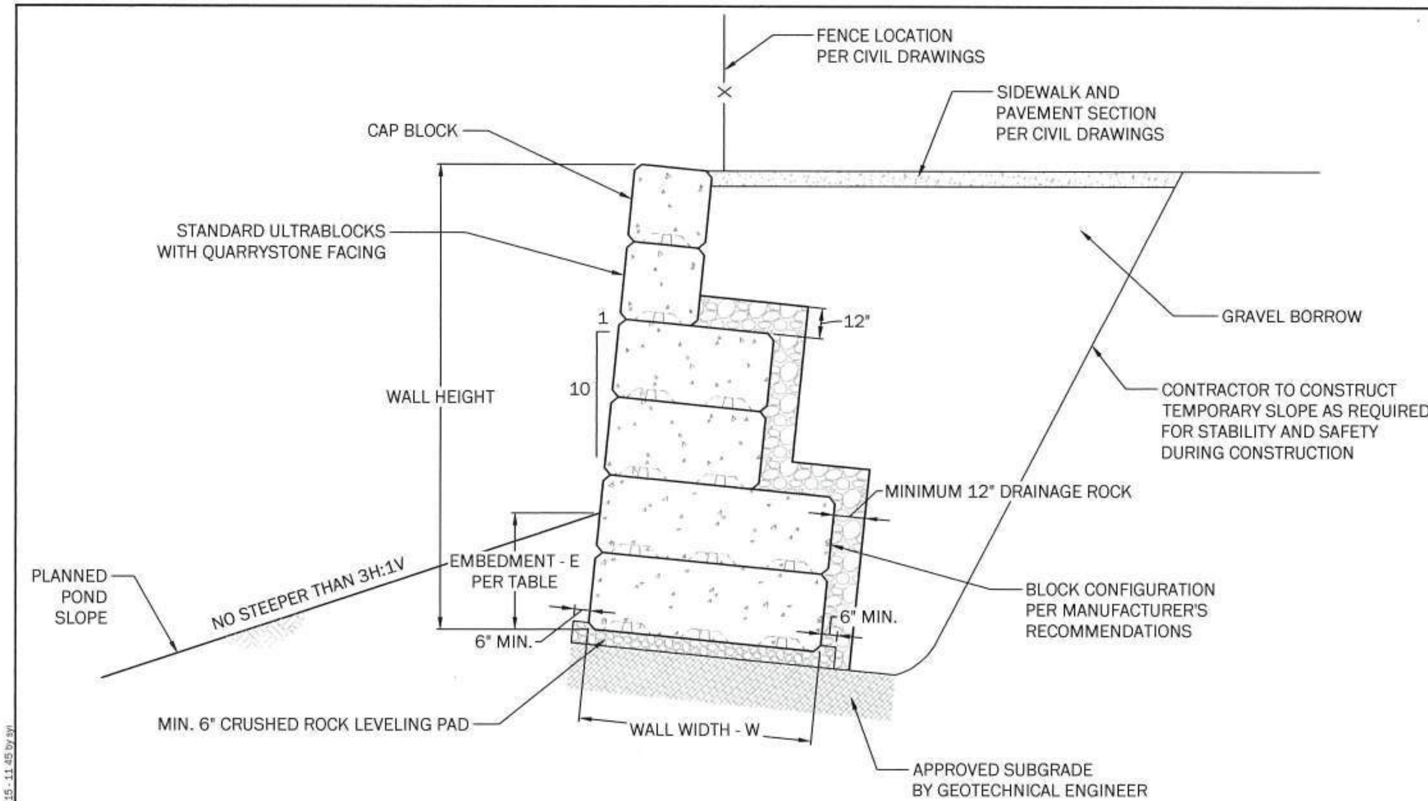


TYPICAL GRAVITY BLOCK WALL CROSS SECTION DETENTION POND
NOT TO SCALE

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

GEOENGINEERS

Figure 2

USGS 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] $S_S = 1.543$ g

From Figure 22-2 [2] $S_1 = 0.585$ 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 $\bar{s}_u < 500$ 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				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
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				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
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):	$S_{MS} = F_aS_S = 1.000 \times 1.543 = 1.543$ g
Equation (11.4-2):	$S_{M1} = F_vS_1 = 1.500 \times 0.585 = 0.878$ g

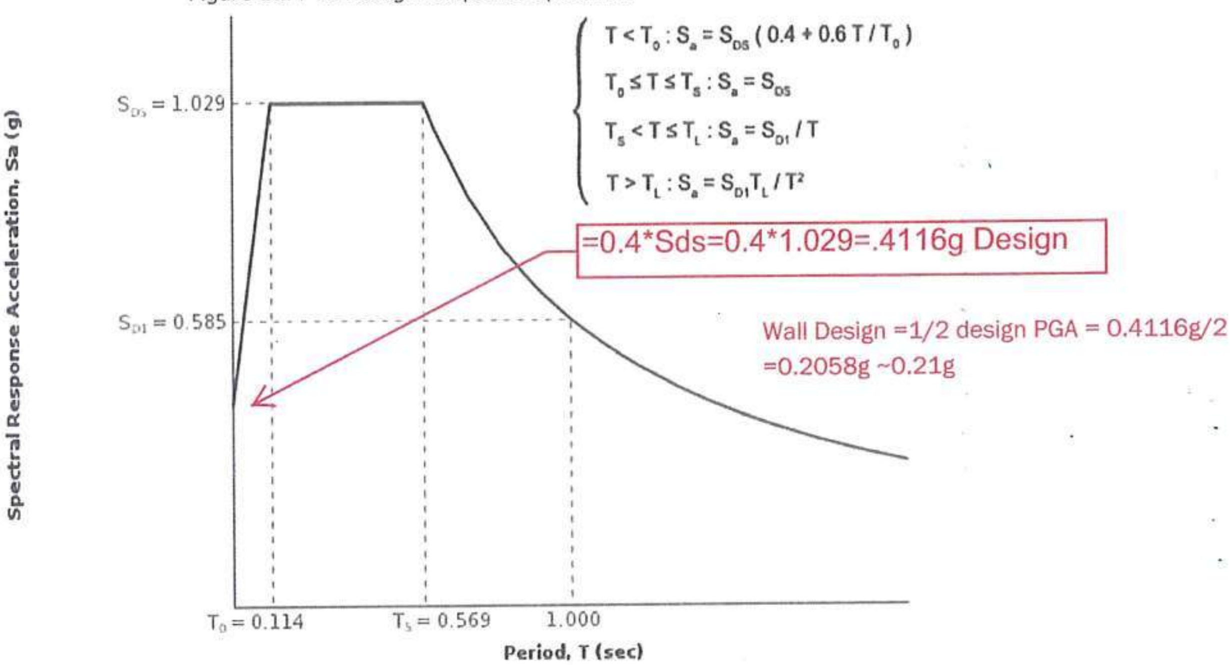
Section 11.4.4 — Design Spectral Acceleration Parameters

Equation (11.4-3):	$S_{DS} = ⅔\, S_{MS} = ⅔ \times 1.543 = 1.029$ g
Equation (11.4-4):	$S_{D1} = ⅔\, S_{M1} = ⅔ \times 0.878 = 0.585$ g

Section 11.4.5 — Design Response Spectrum

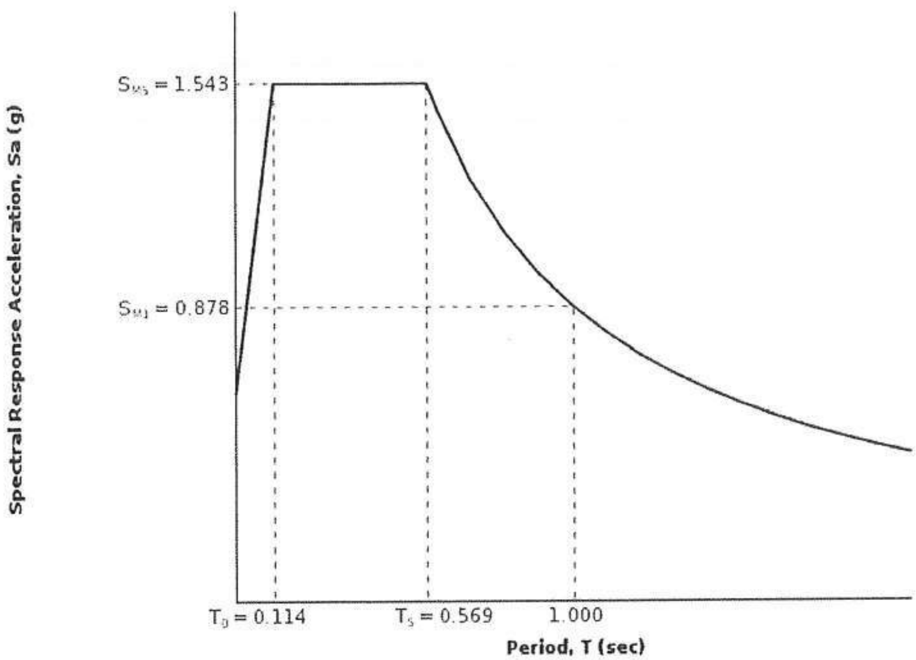
From Figure 22-12 [3]	$T_L = 6$ seconds

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): $PGA_M = F_{PGA}PGA = 1.000 \times 0.647 = 0.647$ 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
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 1.029 g, 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
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.585 g, 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 ≡ "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

AASHTO 2002 ASD DESIGN METHOD
Wind Rose Development
MSEW(3.0): Update # 14.94

PROJECT IDENTIFICATION

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

Description:

Company's information:

Name: GeoEngineers
Street: 600 Dupont

Bellingham, WA 98225
Telephone #:
Fax #:
E-Mail:

Original file path and name: C:\Users\ahartvigsen\Documents\FlexMSE-Geogrid wall Kin.....
.....ousing Authority.BEN

Original date and time of creating this file:

PROGRAM MODE: ANALYSIS
of a SIMPLE STRUCTURE
using GEOGRID as reinforcing material.

SOIL DATA

REINFORCED SOIL
it weight, γ — 125.0 lb/ft³
ign value of internal angle of friction, ϕ — 34.0 °

RETAINED SOIL
Unit weight, γ — 125.0 lb/ft³
Design value of internal angle of friction, ϕ — 34.0 °

FOUNDATION SOIL (Considered as an equivalent uniform soil)
Equivalent unit weight, $\gamma_{equiv.}$ — 125.0 lb/ft³
Equivalent internal angle of friction, $\phi_{equiv.}$ — 36.0 °
Equivalent cohesion, $c_{equiv.}$ — 0.0 lb/ft²

Water table does not affect bearing capacity

LATERAL EARTH PRESSURE COEFFICIENTS

Ka (internal stability) = 0.2827 (if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Inclination of internal slip plane, ψ = 62.00° (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): Nc = 0.00 N γ = 26.71

SEISMICITY

Maximum ground acceleration coefficient, A = 0.210
Design acceleration coefficient in Internal Stability: Kh = Am = 0.260
Design acceleration coefficient in External Stability: Kh_d = 0.260 => Kh = Am = 0.260

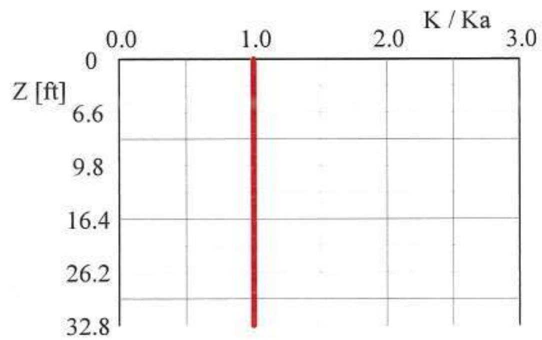
Kae (Kh > 0) = 0.4208 Kae (Kh = 0) = 0.2463 Δ Kae = 0.1745
smic soil-geogrid friction coefficient, F* is 80.0% of its specified static value.

INPUT DATA: Geogrids
(Analysis)

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



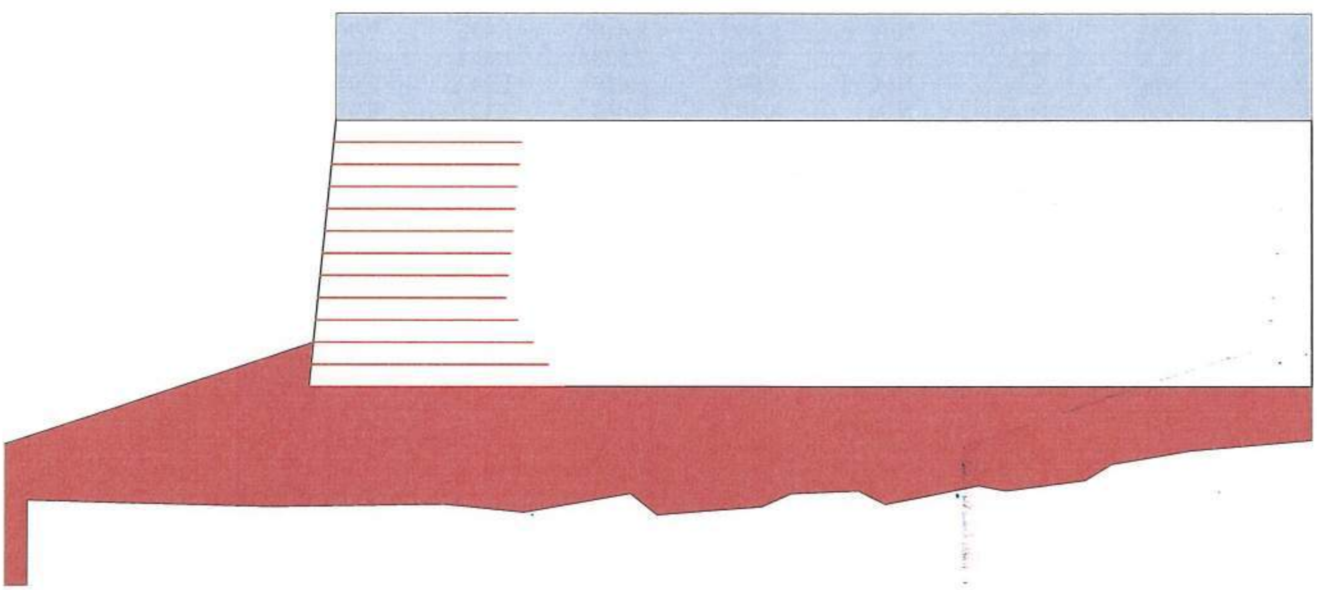
INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)

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 }
...r, 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, Fs = 10.35, Meyerhof stress = 2550 lb/ft².

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

GEOGRID #	Elevation [ft]	Length [ft]	Type #	CONNECTION 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)

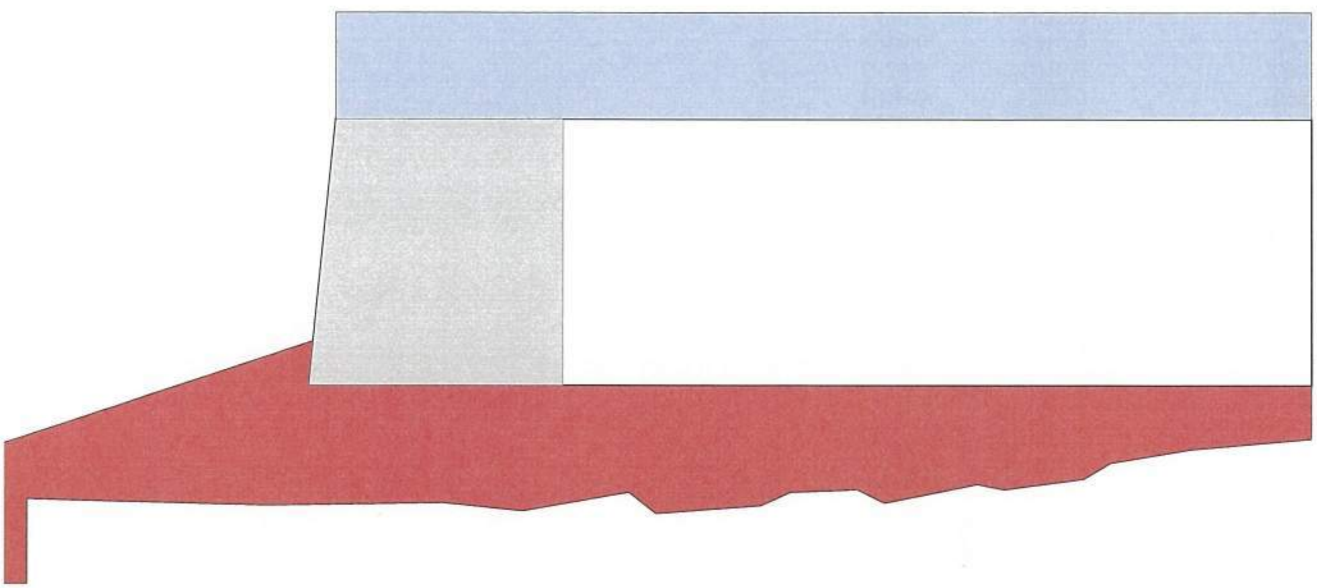
Bearing capacity, Fs = 6.81, Meyerhof stress = 3143 lb/ft².

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

GEOGRID #	Elevation [ft]	Length [ft]	Type #	CONNECTION 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]



SCALE:

0 2 4 6 8 10 [ft]

DIRECT SLIDING for GIVEN LAYOUT (for GEOGRID reinforcements)

Along reinforced and foundation soils interface: Fs-static = 3.518 and Fs-seismic = 1.822

#	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	---

ECCENTRICITY for GIVEN LAYOUT

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

#	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

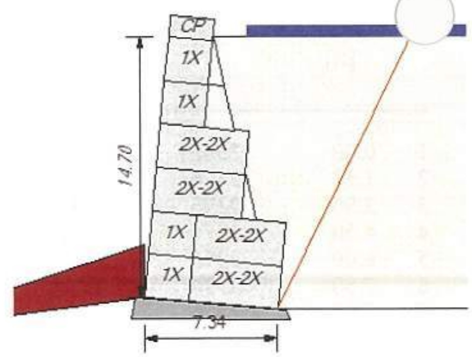
Live Load included in calculating Tmax

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
[illegible]	1.50	1.000	980.7	291.5	15.28	0.65	22234.9	N/A	22.673	17788.0	N/A	13.982
[illegible]	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

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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 and the results may come out to 140 pcf, as shown in the table. The units with more gravel may have lower effective unit weights based on the calculations.

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

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

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

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 Thickeness =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

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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(φ) [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 φ) + 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
τ = N tan(0.00) + 17796.00 ppf

Unit to Leveling Pad Shear
τ = N tan(34.00) + 0.00 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
τ = N tan(0.00) + 17796.00 ppf

Unit to Leveling Pad Shear
τ = N tan(33.80) + 0.00 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 Pa = 5040 ppf
 Pa_h (Force H) = Pa cos(batter + δ) = 5040 x cos(12.7 + 27.0) = 3875 ppf
  Y-Arm = 5.13 ft
 Pa_v (Force V) = Pa sin(batter + δ) = 5040 x sin(12.7 + 27.0) = 3223 ppf
  X-Arm = 6.18 ft

Live Load Pq = 1069 ppf
 Pq_h (Force H) = Pq cos(batter + δ) = 1069 x cos(12.7 + 27.0) = 822 ppf
  Y-Arm = 7.70 ft
 Pq_v (Force V) = Pq sin(batter + δ) = 1069 x sin(12.7 + 27.0) = 683 ppf
  X-Arm = 5.59 ft

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CALCULATION RESULTS

OVERVIEW

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

EARTH PRESSURES

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

EXTERNAL EARTH PRESSURES

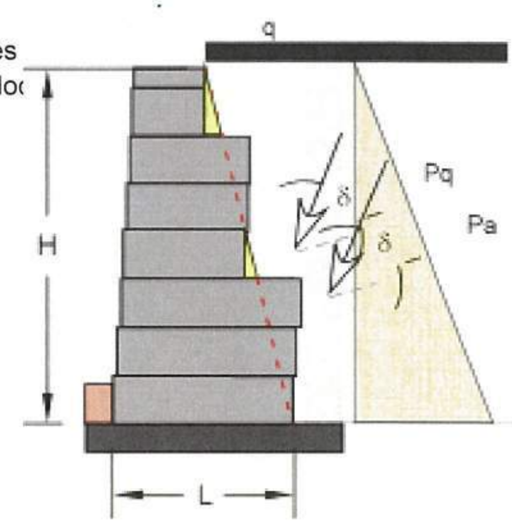
Effective δ angle (3/4 retained phi)	δ =27.0 deg
Coefficient of active earth pressure	ka =0.340
External failure plane	ρ = 63 deg
Effective Angle from horizontal	Eff. Angle =77.25 deg
Coefficient of passive earth pressure: kp = (1 + sin(φ)) / (1 - sin(φ))	kp =4.60

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

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

FORCES AND MOMENTS

The program resolves all the geometry into simple geometric shapes
coordinates are referenced to a zero point at the front toe of the base bloc



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.

Forces Resisting sliding = W1 + W2 + Pav + Pqv	
10576 + 533 + 3223 + 683	N =15015 ppf
Resisting force at pad = (N tan(slope) + intercept x L)	
15015 x tan(33.8) + 0.0 x 7.4	Rf1 =11391
where L is the base block width	
Friction angle is the lesser of the leveling pad and Fnd	φ =40.00 deg
N1 includes N (the leveling pad) + leveling pad (LP)	
15015 + 891	N1 = 15906 ppf
Passive resistance is calculated using kp = (1 + sin(40))/(1 - sin(40))	kp = 4.60
Force at top of resisting trapezoid, d1 = 3.00	Fp1 = 863.68 ppf
Force at base of resisting trapezoid, d2 = 4.20	Fp2 = 1208.32 ppf
Depth of trapezoid	depth = 1.20 ft
Pp = [(Fp1 + Fp2) / 2] * depth	Pp = 1240.24
Resisting force at fnd = (N1 tan(phi) + c L) + Pp	
15906 x tan(40) + 0 x 7.6 + 1240	Rf2 = 14587
where LP = lvl pad thickness * 130pcf * (L + lvl pad thickness/2)	
Driving force is the horizontal component of	
Pah + Pqh	
3875 + 822	Df =4697
FSsl = Rf1/Df / Rf2/Df	FSsl =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 ft-lbs

Moments causing eccentricity = MPah + MPqh
19888 + 6325 Mo =26212 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 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

SumV = (W1 + W2 + LL + Pa_v + Pq_v)

e = L/2 - (SumMr + M_LL - SumMo)/(SumV + LL)

e =7.38/2 - (34431 /15015.13) e =1.394 ft

Calculation of Bearing Pressures

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

where:

Nc =75.31

Nq =64.20

Ng =109.41

c =0.00 psf

q = 219.10 psf

B' = B - 2e + lvlpad = 5.09 ft

Gamma =63 pcf

Calculate Ultimate Bearing, Qult Qult =31483 psf

Bearing Pressure = (SumVert / B') + (LP width * gamma) sigma =3036.24 psf

Calculated Factors of Safety for Bearing 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 occuring 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) + intercept x L	
14908.60 + tan(33.8) + 0.00 x 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

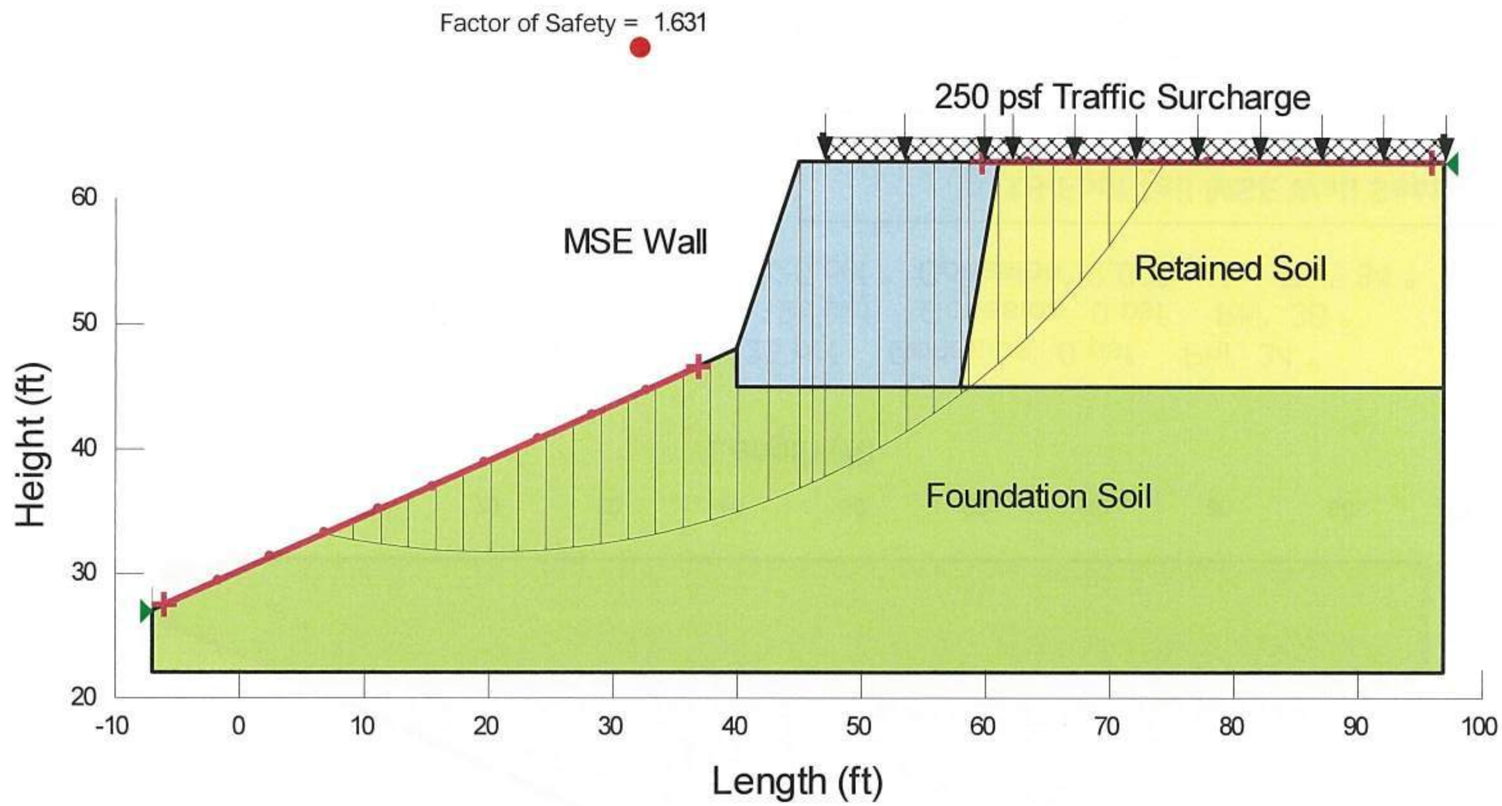
Bearing is the ability of the foundation to support the mass of the structure.

Qult = c*Nc + q*Nq + 0.5*gamma*(B')*Ng

where:

Nc = 75.31
Nq = 64.20
Ng =109.41
c = 0.00 psf
q = 219.10 psf

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



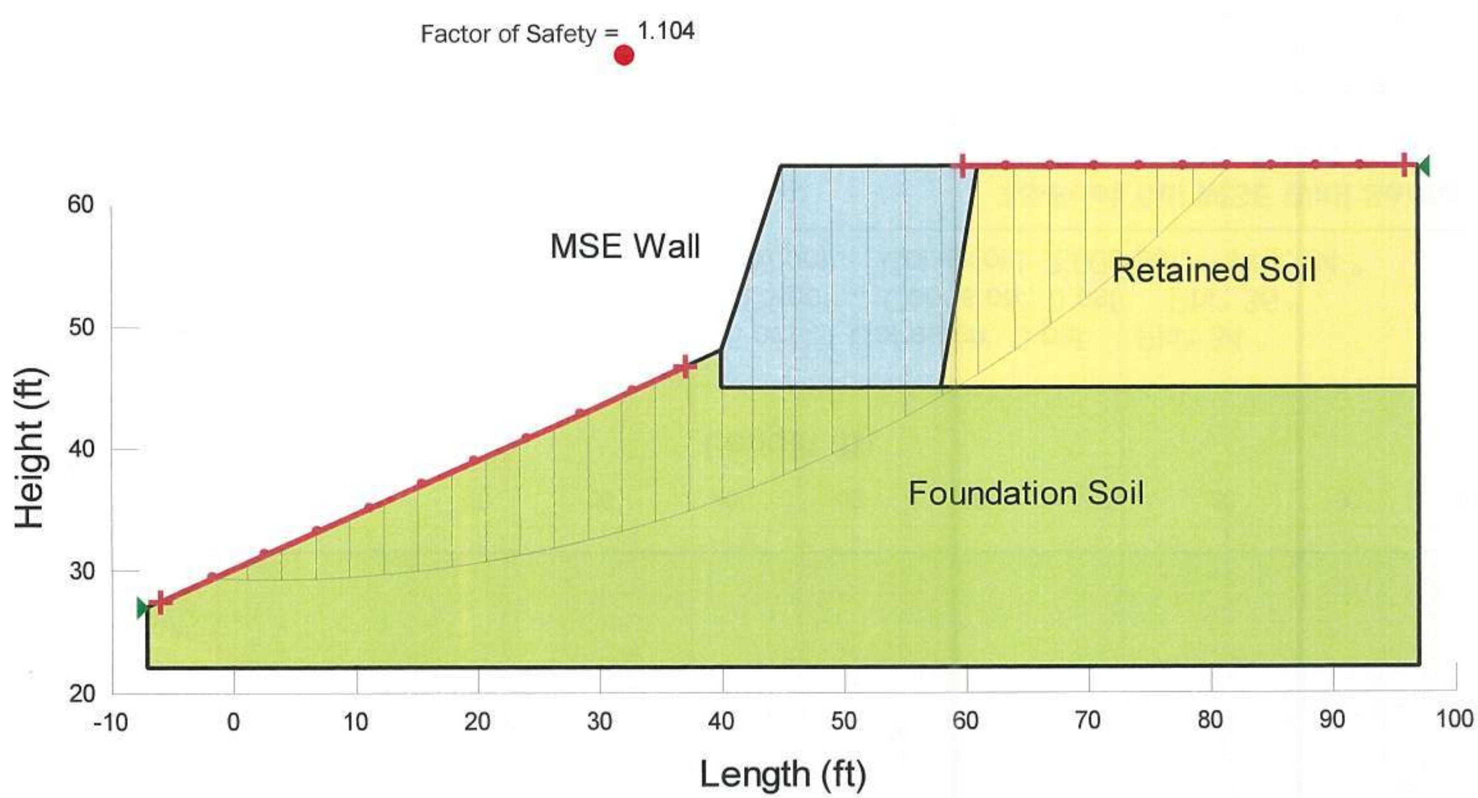
Name: Retained Soil	Unit Weight: 125 pcf	Cohesion': 0 psf	Phi': 34 °
Name: Foundation Soil	Unit Weight: 125 pcf	Cohesion': 0 psf	Phi': 36 °
Name: Reinforced Soil	Unit Weight: 125 pcf	Cohesion': 2,000 psf	Phi': 34 °

18-Foot Tall MSE Wall Section
Static Slope Stability

King County Housing Authority
Wind Rose Development
King County, Washington

GEOENGINEERS

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



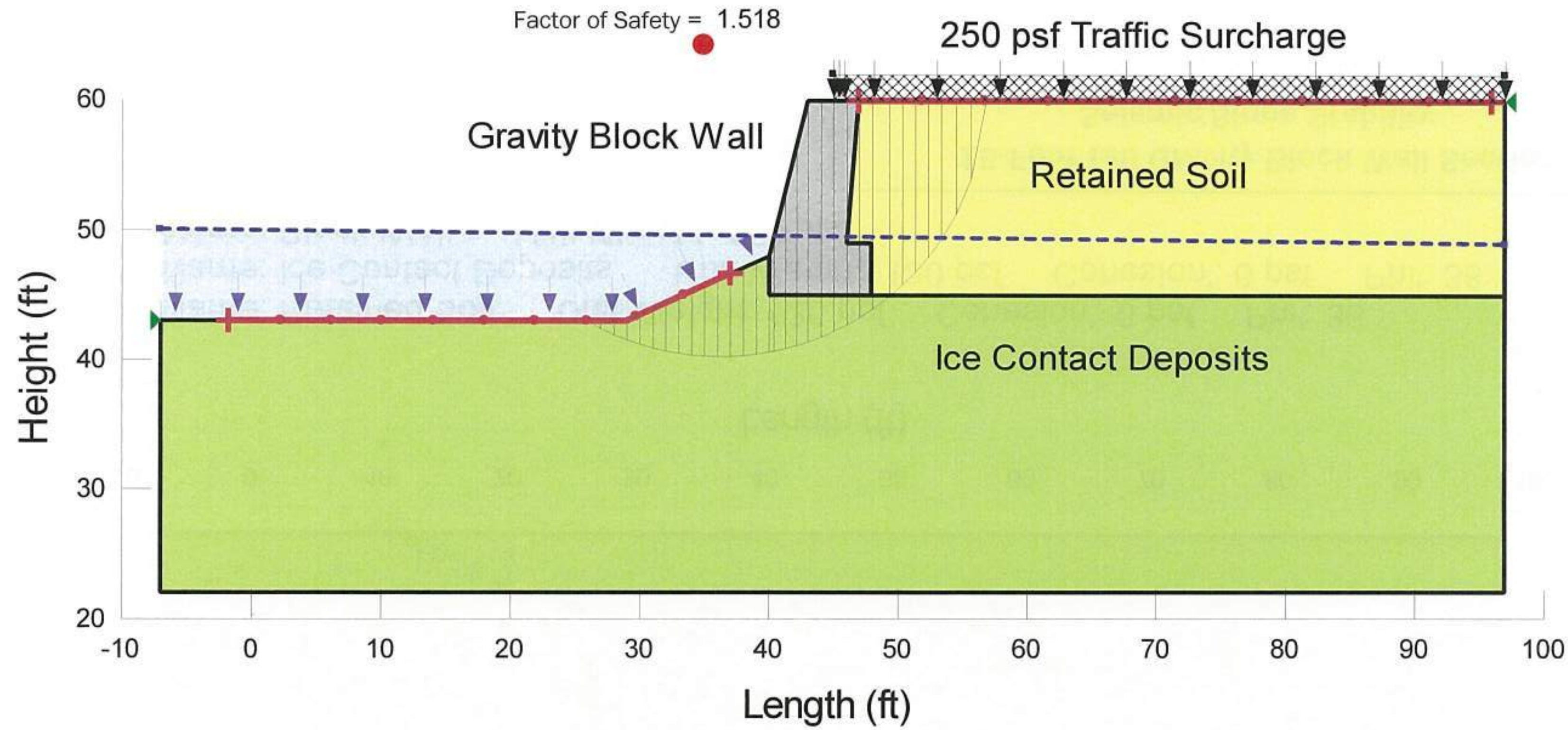
Name: Retained Soil Unit Weight: 125 pcf Cohesion': 0 psf Phi': 34 °
Name: Foundation Soil Unit Weight: 125 pcf Cohesion': 0 psf Phi': 36 °
Name: Reinforced Soil Unit Weight: 125 pcf Cohesion': 2,000 psf Phi': 34 °

18-Foot Tall MSE Wall Section
Seismic Slope Stability

King County Housing Authority
Wind Rose Development
King County, Washington

GeoEngineers

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



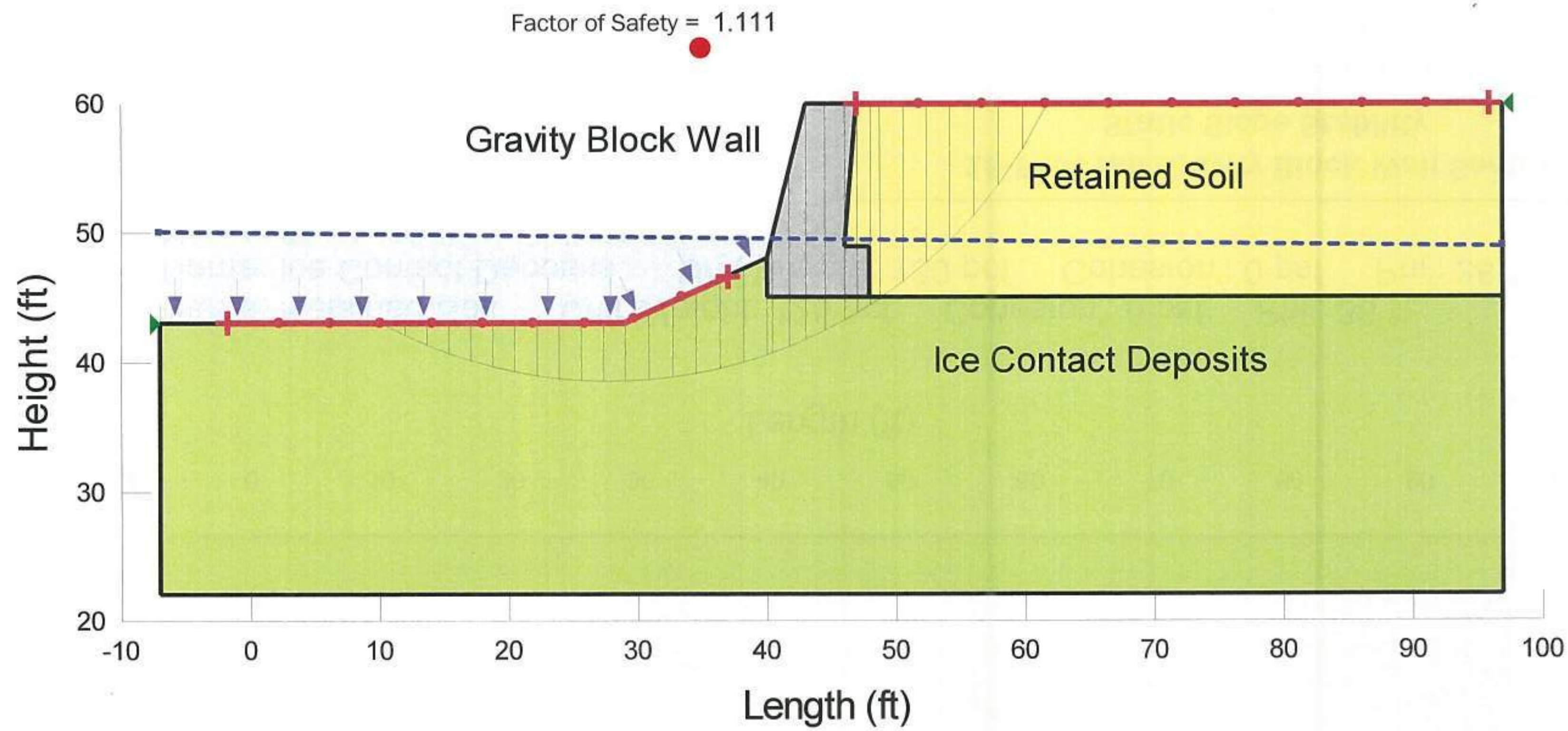
Name: Retained Soil Unit Weight: 125 pcf Cohesion': 0 psf Phi': 36 °
Name: Ice Contact Deposits Unit Weight: 130 pcf Cohesion': 0 psf Phi': 38 °
Name: Block Wall Unit Weight: 150 pcf

15-Foot Tall Gravity Block Wall Section
Static Slope Stability

King County Housing Authority
Wind Rose Development
King County, Washington

GEOENGINEERS

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



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

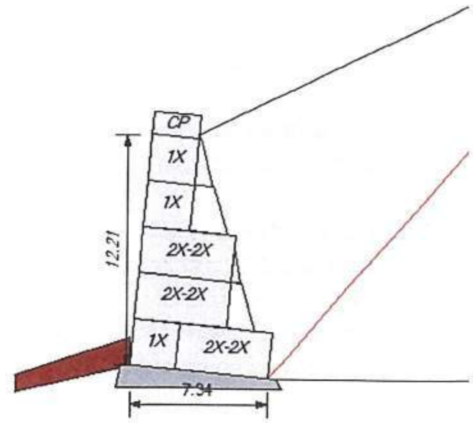
GEOENGINEERS

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	Overturning:	1.50 / 1.125
Bearing:	2.00 / 1.5		

RESULTS (Static / Seismic)

FoS Sliding:	2.00 (lvlpd) / 1.49	FoS Overturning:	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

ULTRABLOCK, INC.

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 and the results may come out to 140 pcf, as shown in the table. The units with more gravel may have lower effective unit weights based on the calculations.

Hollow Units

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

Rounding Errors

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

Result Rounding

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

ULTRABLOCK, INC.

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 Thickeness =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

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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(φ) [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 φ) + 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

τ = N tan(0.00) + 17796.00 ppf

Unit to Leveling Pad Shear

τ = N tan(34.00) + 0.00 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

τ = N tan(0.00) + 17796.00 ppf

Unit to Leveling Pad Shear

τ = N tan(33.80) + 0.00 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 Pa = 6882 ppf

Pa_h (Force H) = Pa cos(batter + δ) = 6882 x cos(16.1 + 25.5) = 5149 ppf

Y-Arm = 4.30 ft

Pa_v (Force V) = Pa sin(batter + δ) = 6882 x sin(16.1 + 25.5) = 4567 ppf

X-Arm = 6.10 ft

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CALCULATION RESULTS

OVERVIEW

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

EARTH PRESSURES

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

EXTERNAL EARTH PRESSURES

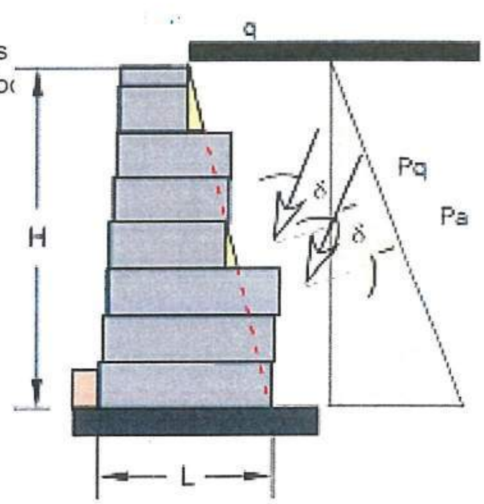
Effective δ angle (3/4 retained phi)	δ =25.5 deg
Coefficient of active earth pressure	ka =0.661
External failure plane	ρ = 49 deg
Effective Angle from horizontal	Eff. Angle =73.93 deg
Coefficient of passive earth pressure: kp = (1 + sin(φ)) / (1 - sin(φ))	kp =3.85

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

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

FORCES AND MOMENTS

The program resolves all the geometry into simple geometric shapes coordinates are referenced to a zero point at the front toe of the base bloc



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.

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BASE SLIDING

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

Forces Resisting sliding = W1 + W2 + Pav	
8038 + 796 + 4567	N =13401 ppf
Resisting force at pad = (N tan(slope) + intercept x L)	
13401 x tan(33.8) + 0.0 x 7.4	Rf1 =10256
where L is the base block width	
Friction angle is the lesser of the leveling pad and Fnd	φ =36.00 deg
N1 includes N (the leveling pad) + leveling pad (LP)	
13401 + 891	N1 = 14292 ppf
Passive resistance is calculated using kp = (1 + sin(36))/(1 - sin(36))	kp = 3.85
Force at top of resisting trapezoid, d1 = 1.50	Fp1 = 361.69 ppf
Force at base of resisting trapezoid, d2 = 2.70	Fp2 = 650.35 ppf
Depth of trapezoid	depth = 1.20 ft
Pp = [(Fp1 + Fp2) / 2] * depth	Pp = 605.78
Resisting force at fnd = (N1 tan(phi) + c L) + Pp	
14292 x tan(36) + 0 x 7.6 + 606	Rf2 = 10990
where LP = lvl pad thickness * 130pcf * (L + lvl pad thickness/2)	
Driving force is the horizontal component of Pah	
5149	Df =5149
FSsl = Rf1/Df / Rf2/Df	FSsl =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 ft-lbs

Moments causing eccentricity = MPah
22152 — Mo =22152 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

SumV = (W1 + W2 + Pa_v)

e = L/2 - (SumMr - SumMo)/(SumV)

e =7.38/2 - (32729 /13400.82) e =1.245 ft

Calculation of Bearing Pressures

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

where:

Nc =50.59

Nq =37.75

Ng =56.31

c =0.00 psf

q = 125.20 psf

B' = B - 2e + lvlpad = 5.38 ft

Gamma =63 pcf

Calculate Ultimate Bearing, Qult	Qult =14217 psf
Bearing Pressure = (SumVert / B') + (LP width * gamma)	sigma =2568.94 psf
Calculated Factors of Safety for Bearing	Qult/sigma =5.53

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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 = 8037.73 *0.080

Pif =642.48 ppf

SEISMIC THRUST

Kae	Kae =0.838
D_Kae = Kae - Ka = (0.838 - 0.661)	D_Kae =0.177
Pae = 0.5*gamma*(H)^2*D_Kae	Pae =1843.19 ppf
Pae_h = Pae*cos(δ)	Pae_h =1378.94 ppf
Pae_v = Pae*sin(δ)	Pae_v =1223.05 ppf

TABLE OF RESULTS FOR SEISMIC REACTIONS

Name	Force (V)	Force (H)	X-len	Y-len	Mo	Mr
Face Blocks(W1)	8037.726	--	2.9	--	--	23313.35
Face Soil(W2)	796.47	--	4.66	--	--	3711.65
Pa_h	--	5148.694	--	4.302	22151.78	--
Pa_v	4566.625	--	6.1	--	--	27855.83
Pif	--	642.479	--	7.744	4975.57	--
Pae_h	--	1378.944	--	7.744	10679.0	--
Pae_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 occuring 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 = 14624
Resisting force = SumVs * tan(phi) + intercept x L	
14623.87 + tan(33.8) + 0.00 x 7.38	FRe =10664 lbf
Driving force = Pa_h + Pae_h + Pif	
5149 +1379 +642	FDr =7170 lbf
FOS = FRe/FDr [leveling pad / foundation]	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	SumMrS =62341 ft lbf
Driving Moment = MPah + MPaeh +MPif	SumMoS =37806.35 ft lbf
Factor of Safety = SumMrS/SumMoS	FoS =1.65

SEISMIC BEARING

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

Qult = c*Nc + q*Nq + 0.5*gamma*(B')*Ng

where:

Nc = 50.59
Nq = 37.75
Ng = 56.31
c = 0.00 psf
q = 125.20 psf

Calculate Ultimate Bearing, Qult (seismic)	Qult = 14217.16 psf
eccentricity (e)	e =2.010
Equivalent Footing Width, B' = L - 2e + lvl pad	B' =4 ft
Bearing Pressure = sumVs/B'	sigma =3891 psf
Factor of Safety for Bearing = Qult/Bearing	FoS =3.65

APPENDIX E
Report Limitations and Guidelines for Use

APPENDIX E
REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

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.