COUGHLINPORTERLUNDEEN
STRUCTURAL CIVIL SEISMIC ENGINEERING

ENGINEERING REPORT

DRAINAGE REPORT

Microsoft Red West South Fields
Redmond, WA
May 24, 2019

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Drainage Report

Microsoft Red West South Fields
Coughlin Porter Lundeen
Project No. P18BSB-001
May 24, 2019

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I. PROJECT OVERVIEW

GENERAL PROJECT DESCRIPTION

The following Drainage Report provides the design analysis for the Red West South Fields drainage plan. The storm water design for the project was based on the requirements set forth in the 2017 City of Redmond Stormwater Technical Notebook (STN) which reference Ecology's 2012 Stormwater Management Manual for Western Washington as amended in 2014 (2014 SWMMWW).

The Red West South Fields project is located within the City of Redmond. The site is bounded by the Microsoft Red West Access Road to the north, State Route 520 to the east, NE 51st Street to the south, and 148th Avenue NE to the west, as shown in Figure 1-Vicinity Map. The parcel (#2182500120) is located in Section 14, Township 25 North, Range 5 East, Willamette Meridian. The area within the property boundary is approximately 26.5 acres, of which approximately 8.5 acres will be redeveloped. The area that will be redeveloped is in Basin 2 of the North Overlake Drainage Area.

The project proposes to add two artificial turf athletic fields as well as a basketball court and volleyball court. Associated site improvements include sidewalks and ADA access to the fields, a storm conveyance system, water quality systems, and site grading. The drainage design and analysis covered in this report includes storm water mitigation for the developed area of the project.

EXISTING DRAINAGE CONDITIONS

The site for the Red West South Fields project is currently partly developed, having undergone some site improvement projects in the past. There is an asphalt parking lot on the north side of the site that drains to a detention pond that was sized and permitted in 2009. The detention pond provides flow control and water quality treatment for the parking lot area. See Appendix C for the drainage report for the 2009 Microsoft Red West South Parking Lot Modifications. There are three wetlands on site that were improved to accommodate future development during a pre-mitigation project in 2012. The project will only affect the drainage basin for the large wetland in the northeast (Wetland F) so that will be the only wetland modeled and discussed in this drainage design. This drainage report addresses the wetland hydrology design and will comply with the requirements set forth in the 2017 STN.

The project site for the Red West South Fields currently consists of lawn area with a gravel pedestrian path/access road running from the existing parking lot on the north side of the site to the south and east sides of the site. There is also a large forested area in the northeast of the site that contains Wetland F and another smaller wetland that will not be affected by this project. The site slopes from west to east with approximately 20 feet of elevation drop over the area that is to be developed. The runoff from the project area is currently collected through a series of ditches, french drains, catch basins, and closed pipes and conveyed to Wetland F. This wetland contains an overflow pipe that discharges the excess stormwater to the SR520 conveyance system at the northeast corner of the site. The existing conditions are analyzed further in Section IV of this report, including a level I downstream analysis of the project site.

Aside from the wetlands previously mentioned, there are no critical areas within the proposed development area of the site. See Figures 2A-2G for City of Redmond critical area maps.

PROPOSED DRAINAGE CONDITIONS

Storm water runoff from the developed site area will be collected and conveyed to either Wetland F or straight to the site outfall to the SR-520 conveyance system. On-site conveyance will consist of overland flow, catch basins, and underground pipes. Detention is not required for the project and there will be no modifications to the existing

parking lot detention pond that will remain. Water quality treatment facilities will be used to treat runoff from the artificial turf fields, that is considered pollution generating. Downstream of the water quality facilities, the project will use flow splitters to match developed wetland flows to existing wetland flows within guidelines set forth in the STN. The systems are described in further detail in Sections IV-VII of this report.

II. MINIMUM REQUIREMENTS

This section will address the nine minimum requirements set forth in the STN. All nine minimum requirements apply as shown in Figure 3-Flow Chart for Determining Requirements for New Development.

MR #1: PREPARATION OF STORM WATER SITE PLANS

Stormwater plans and reports that address each of the applicable minimum requirements will be prepared by a licensed civil engineer in accordance with City Requirements.

MR #2: CONSTRUCTION STORM WATER POLLUTION PREVENTION PLAN

A temporary erosion and sediment control (TESC) plan has been prepared and submitted with the plan set. A SWPPP will be prepared and submitted at a later date.

MR #3: SOURCE CONTROL OF POLLUTION

There will be no post-development pollution generating activities on site that will require source control BMPs.

MR #4: PRESERVATION OF NATURAL DRAINAGE SYSTEM AND OUTFALLS

Stormwater will continue to flow to the current outfall that discharges to the SR520 conveyance system at the northeast corner of the site.

MR #5: ON-SITE STORM WATER MANAGEMENT

The project will employ On-site Stormwater Management BMPs to infiltrate, disperse, and retain stormwater runoff on site to the maximum extent feasible without causing flooding or erosion impacts. See Section V “Low Impact Development” for implementation of stormwater BMPs.

MR #6: RUNOFF TREATMENT

The proposed project contains more than 5,000 square feet of pollution generating surface. Therefore, water quality facilities are required for new and replaced pollution generating surfaces. The receiving water body is Sammamish River, which requires enhanced treatment. Enhanced water quality treatment will be provided by modular wetland systems or an approved equivalent. The water quality system is described further in Section IV.

MR #7: FLOW CONTROL

The project site is located in the North Overlake Drainage Basin and is subject to the Alternative Flow Control Standard as described in Section 2.57 of the 2017 STN. The North Overlake Drainage Basin is classified as direct discharge to Sammamish River and flow rates are only limited by the capacity of the WSDOT conveyance system under SR-520. To comply with the alternative flow control standard, the 50-year max flow rate leaving the site must remain below 0.28 CFS/Acre (6.28 CFS total for the 22.45 Acres of site flowing to the site outfall). One-half of the ROW area must be included (0.90 Acres) but does not count towards additional allowable maximum flow rate. Compliance with the flow control standard is described in greater detail in Section IV.

MR #8: WETLAND PROTECTION

There is one wetland on site that receives runoff from the project area, labeled Wetland F. The project will follow protection guidelines listed and match the pre- and post-project discharge volumes as described in Appendix 1-D of the 2014 SWMMWW. Wetland protection is described in further detail in Section VI. A Critical Areas Report has been prepared by Tallasea and submitted as well.

MR #9: OPERATION AND MAINTENANCE

An operation and maintenance manual for all proposed stormwater facilities and BMPs consistent with Appendix L of the City's STN will be prepared and included at a later date.

III. DRAINAGE ANALYSIS

FIELD INSPECTION

A site visit has been made to gather information about the existing drainage system, including a Level 1 Downstream Analysis. This field visit took place July 17, 2018. The analysis is described below.

DRAINAGE SYSTEM PROBLEM DESCRIPTIONS

There are no known or observed existing drainage system problems on site. As such, no drainage problems are anticipated due to previously existing problems.

UPSTREAM ANALYSIS

The proposed site will match the existing grade at the project limits as to not alter flow paths or drainage basins. There are no significant upstream areas that drain onto the project site. Therefore, no drainage problems are anticipated due to upstream flows.

DOWNSTREAM ANALYSIS

Stormwater from the project area currently discharges to Wetland F in the northeast corner of the site. The overflow from the wetland discharges at the northeast corner of the property to a large rock pad outfall that then enters a 24'' culvert that connects to the SR-520 conveyance system. There is an existing storm system installed adjacent to the access road to the east of Wetland F that catches and conveys runoff to the outfall to rock pad outfall as well. All pipes and structures look to be in good condition.

The SR-520 conveyance system discharges directly to Sammamish River under the supervision of WSDOT. The downstream SR-520 conveyance system is under-sized and the City of Redmond is diligently monitoring the discharge from the North Overlake area. Public Works is working on repairing at least one section and there are other problem locations downstream. The City of Redmond regulates the maximum 50-year flow rate to be discharged to the SR-520 conveyance system using the North Overlake alternative flow control standard. The project will adhere to the North Overlake alternative flow control standard and no further drainage problems are anticipated due to project flows.

IV. FLOW CONTROL AND WATER QUALITY FACILITY ANALYSIS AND DESIGN

EXISTING SITE HYDROLOGY (PART A)

The project area for the Red West South Fields is currently mostly lawn area with a gravel pedestrian path/access road running from the parking lot in the north to the south and east sides of the site. The runoff from the project area is collected through a series of ditches, french drains, catch basins, and closed pipes and conveyed to Wetland F. This wetland contains an overflow pipe that discharges the excess stormwater to the SR520 conveyance system at the northeast corner of the site. Table 1 summarizes the land cover characteristics of the existing site. These areas are shown in Figure 4 – Existing Conditions.

TABLE 1 – EXISTING SITE CONDITIONS AREA BREAKDOWN

Land Cover	Area
Impervious Area	3.79 acres
Pervious Area	9.25 acres
Forested Area	9.41 acres
Total Site Area	22.45 acres
ROW Area	0.90 acres
Total Basin Area	23.35 acres
Percent Impervious	17%

*ROW area must be included in flow control calculations but does not count towards additional allowable flow.

DEVELOPED SITE HYDROLOGY (PART B)

Storm water mitigation will be required for construction of all new and replaced impervious and pervious surfaces. This includes the new artificial turf fields, new basketball and volleyball courts, new concrete sidewalks and paving, and new and replaced landscaping. Table 2 summarizes the land cover characteristics of the proposed redevelopment. These areas are shown in Figure 5 – Proposed Conditions.

TABLE 2 –DEVELOPED SITE CONDITIONS AREA BREAKDOWN

Land Cover	Area
Impervious Area	8.58 acres
Pervious Area	4.46 acres
Forested Area	9.41 acres
Total Site	22.45 acres
ROW Area	0.90 acres
Total Basin Area	23.35 acres
Percent Impervious	38%

*ROW area must be included in flow control calculations but does not count towards additional allowable flow.

PERFORMANCE STANDARDS AND FLOW CONTROL SYSTEM (PART C AND D)

Flow control will conform to the North Overlake Basin alternative flow control standard as described in Appendix J of the 2017 STN. The North Overlake Drainage Basin is classified as direct discharge to Sammamish River and flow rates are only limited by the capacity of the WSDOT conveyance system under SR-520. To comply with the alternative flow control standard, the 50-year max flow rate leaving the site must remain below 0.28 CFS/Acre (6.28 CFS total for the 22.45 Acres of site (+0.90 Acres of ROW) flowing to the site outfall).

In accordance with the 2017 STN and the 2014 DOE manual, WWHM2012, a continuous-modeling software, will be used to model the proposed site conditions. New and replaced impervious surfaces will be modeled as impervious area. New and replaced landscape areas will be modeled as pasture (as they will meet the compost-amended soils BMP).

The runoff from the developed site is below the 50-year max flow rate prescribed by the North Overlake alternative flow control standard so detention is not required for the project. Flow splitters will be used to direct the required amount of runoff to satisfy wetland requirements for the two individual outfalls to Wetland F. The excess water not required for wetland hydration will be routed to the conveyance system that is to be installed with the Hackathon project to route the water around Wetland F. See Figure 5 and Appendix B1 demonstrating compliance with the North Overlake alternative flow control standard.

WATER QUALITY SYSTEM (PART E)

The Red West South Fields project will construct more than 5,000 square feet of pollution-generating impervious surface, triggering water quality treatment requirements in accordance with the STN. The analysis of the required treatment facilities follows Chapter V-2 of the 2014 SWMMWW:

Step 1: The receiving water body is Sammamish River.
Step 2: Oil control facilities are not required because the site is not considered “high-use.”
Step 3: The site is unsuitable for infiltration based on the geotechnical analysis.
Step 4: Phosphorous control is not required for discharge to Sammamish River.
Step 5: Enhanced treatment is required as Sammamish River is a fish-bearing water body.

In addition to requirements in Chapter V-2, the project contains artificial turf playfields with standard crumb rubber infill that have been identified to leach heavy metals into runoff so the runoff from any turf area must be treated for heavy metals.

Enhanced water quality and heavy metal treatment will be provided by proprietary modular wetland systems (or approved equivalents). The modular wetland system has a General Use Level Designation from the Washington State Department of Ecology for enhanced water quality treatment, heavy metals, and many additional water quality categories. The location of the proposed facilities are shown in Figure 6 – Site Map.

Modular wetland systems are biofiltration systems that utilize horizontal flow. Sediment and hydrocarbons are removed from the stormwater in a pre-treatment chamber before reaching the biofiltration chamber. The modular wetland system will be designed and sized by BioClean based upon the water quality design flow rate (91% treatment volume) as modeled by WWHM. The modular wetland system will include an internal bypass system designed for the 100-year mitigated flowrate. The final sizing and design of the Water Quality Facility will be completed at a later date.

V. LOW IMPACT DEVELOPMENT

Low impact development (LID) on the project site was assessed to meet the minimum requirements set forth in Section 2.5.5 of the 2017 STN. This project is required to evaluate the feasibility of BMPs in List #2 of Section 2.5.5 but is not required to meet the LID Performance Standard.

The following LID BMP's were considered in an effort to implement On-Site Stormwater Management and to meet LID Performance Standards. See Figure 7 – LID Site Plan for the layout of proposed LID BMPs.

Post-Construction Soil Quality and Depth: Post-construction soil quality and depth will be implemented for all new and replaced landscape areas. (2.59 Acres)

Full Dispersion: Full dispersion has been deemed infeasible for the project because the project area currently flows to the SR-520 conveyance system and there is no feasible flow path to the existing outfall that contains a native vegetation dispersion path.

Downspout Full Infiltration: Downspout full infiltration has been deemed infeasible for the project because the soils onsite do not meet the minimum required depth of five feet to the hardpan layer. The depth to the hardpan layer is three to four feet in the project area. See the Geotechnical Report provided in Appendix D for soil report.

Bioretention Cells, Swales, and Planter Boxes: Bioretention has been deemed infeasible for the project because the vertical separation between the hardpan layer and the bottom of the bioretention would be less than the required three feet. The bottom of the bioretention would be as deep as the hardpan layer in parts of the project area. See the Geotechnical Report provided in Appendix D for soil report.

Downspout Dispersion Systems: Downspout dispersion has been deemed infeasible for the project because the project area currently flows to the SR-520 conveyance system and there is no feasible flow path to the existing outfall that contains a native vegetation dispersion path.

Perforated Stub-Out Connections: Perforated stub-out connections have been deemed infeasible because the depth to the seasonal water table is expected to be less than one foot below the trench bottom. While the water table was not observed during wet season, it can be assumed that the water table will be above the hardpan layer which would only be one foot below the trench bottom in many places.

Permeable Pavements: The City of Redmond functional equivalent infiltration sidewalk will be used for sidewalk areas. A strip of drain rock will be placed adjacent to the sidewalk to infiltrate runoff into the base layer. The remaining area consists mostly of artificial turf fields, which allow water to infiltrate through to the subgrade – similar to permeable pavement. However, the artificial turf fields have an underdrain system designed by the manufacturer and located at the bottom of the base layer, preventing the fields from receiving LID credit. (0.09 Acres)

Due to site constraints and infeasibility criteria provided above the project is proposing to provide Onsite Stormwater Management and LID to the maximum extent feasible.

The following are the responses to the site assessment minimum requirements as noted in Section 8.7.5:

1. *A survey prepared by a registered land surveyor showing existing public and private development, including utility infrastructure, on and adjacent to the site, major and minor hydrologic features, including seeps, springs, closed depression areas, drainage swales, and 2 foot contours up to 10 percent slope and 5 foot contours for slopes above 10 percent. Spot elevations shall be at 25 foot intervals.*

   A survey has been prepared by a registered land surveyor meeting the required criteria.
2. *Location of all existing lot lines, lease areas and easements.*

   A survey has been prepared by a registered land surveyor meeting the required criteria.

3. *A soils report prepared by a licensed geotechnical engineer or licensed engineering geologist. The report shall identify:*

   a. Underlying soils on the site utilizing soil pits and soil grain analysis to assess infiltration capability on site. The frequency and distribution of test pits shall be adequate to direct placement of the roads and structures away from soils that can most effectively infiltrate stormwater;

   b. Percolation tests if appropriate or requested by the Stormwater Engineer;

   c. Topographic and geologic features that may act as natural stormwater storage or conveyance and underlying soils that provide opportunities for storage and partial infiltration;

   d. Depth to wet season high groundwater;

   e. Geologic hazard areas and associated buffer requirements as defined in RZC 21.64.060;

   f. Distance from site boundaries to any areas within 200 feet of the site identified as landslide hazard areas or having a slope of 40 percent or steeper with a vertical relief of 10 feet or more; [Note: the City may require the applicant to expand the 200 feet to encompass a larger area if there are concerns for downstream geological hazards.]

   g. Identification of Wellhead Protection Zone(s); and

   h. For previously cleared or graded sites, analysis of topsoil according to the soil

   i. requirements in the City of Redmond Standard Specifications, Section 9.14.1.

   A geotechnical report was previously prepared by a licensed geotechnical engineer in 2008 (see Appendix D, *Geotechnical Report*). From the conditions stated in the geotechnical report, it is expected that infiltration is infeasible onsite due to the presence of very dense glacially consolidated till soils. Infiltration testing will be performed at a later date to determine if infiltration is feasible or confirm that our assumptions were correct. A topographic survey has been prepared by a licensed surveyor to meet the remaining required criteria.

4. *A survey of existing native vegetation cover and wildlife habitat by a qualified biologist identifying any forest areas on the site, species and condition of ground cover and shrub layer, and tree species, seral stage, and canopy cover.*

   A survey has been prepared by a registered land surveyor meeting the required criteria.

5. *A streams, wetland, and water body survey and classification report by a qualified biologist*

   showing wetland and buffer boundaries consistent with the requirements of RZC 21.64.030 and Critical Areas Reporting Requirements (RZC Appendix 1).

   A survey has been prepared by a registered land surveyor meeting the required criteria.

6. *Flood hazard areas on or adjacent to the site.*

   A survey has been prepared by a registered land surveyor. There are no flood hazards on or adjacent to the site.

7. *A preliminary drainage report providing analysis of the existing site hydrologic conditions on*

   the site and recommendations for type, location, and restrictions on LID BMPs.

   See Appendix B, Drainage Calculations, for a hydrologic analysis of the project site.

8. *Other studies as deemed necessary by the Stormwater Engineer.*

   No other documents have been identified as being necessary for LID assessment.

VI. STORMWATER DISCHARGES TO WETLANDS

Wetland Guidelines:

The project area currently flows to a Category IV wetland (labeled Wetland F) according to the Advanced Wetland Mitigation project performed by wetland consultant Talasaea and civil engineering firm Coughlin Porter Lundeen in 2012. The STN refers to the 2014 SWMMWW Appendix 1-D: Guidelines for Wetlands when Managing Stormwater for projects discharging to a classified wetland. The 2014 SWMMWW lists the following requirements:

The project does not propose to utilize the wetland as a treatment or flow control BMP/facility so those requirements are not applicable. To maintain the current function and value of the wetland, the project will not perform any work within the wetland buffer.

The project will protect the wetlands from impacts of changes in water flows using the guidelines in Guide Sheet 3B:

- Criterion 1: Total volume of water into a wetland during a single precipitation event should not be more than 20% higher or lower than the pre-project volumes.
- Criterion 2: Total volume of water into a wetland on a monthly basis should not be more than 15% higher or lower than the pre-project volumes.

Wetland Design:

Wetland F currently contains one rock pad outfall (labeled as west outfall to Wetland F) and one level spreader (labeled as east outfall to Wetland F) that discharge runoff to the wetland. At Tallasea's recommendation, each outfall has been analyzed separately as they discharge to separate areas of the wetland with differing flow capabilities. There is an existing overflow drain system on the north side of the wetland that carries excess water from the wetland to the SR 520 conveyance system.

The post-project flows to Wetland F will match existing flow volumes within the prescribed tolerances (20% daily, 15% monthly). While some of the existing area will continue to flow to the Wetland F outfalls, the developed condition of the site will reduce the drainage basin flowing to both Wetland F outfalls. A flow splitter will be used to direct the prescribed amount of runoff to each Wetland F outfall to satisfy DOE criteria. The turf field area must be treated for water quality before routing to the wetland so the field area will be routed through water quality systems upstream of the flow splitters. The excess runoff from the flow splitters will be directed to a conveyance system installed with the previous Hackathon project that will route the water around Wetland F to an existing storm system that connects to the site outfall.

The full design of the flow splitters and post-project wetland volumes will be completed in the CCR process, after the site plan is closer to being finalized.

VII. CONVEYANCE SYSTEM ANALYSIS AND DESIGN

New conveyance systems and existing conveyance systems with altered flows will adhere to requirements set forth in the STN. The STN requires that conveyance systems be designed to convey the 50-year peak flow rate without overtopping and the 100-year peak flow rate without flooding roads or structures. The STN refers to the WSDOT Hydraulics Manual for conveyance design requirements. The WSDOT Hydraulics Manual requires that conveyance systems be designed using an approved continuous modeling program.

EXISTING CONDITIONS

There is currently a storm system that collects and conveys the runoff from the parking lot, access roads, and lawn area to Wetland F. The parking lot area sheet flows to the east edge of the lot where a series of closed pipes convey it to the detention pond adjacent to the parking. The detention pond discharges through a closed pipe system to a rock pad outfall at Wetland F (called the west outfall of Wetland F in this report). The access road and lawn areas are collected and conveyed through open and closed pipe systems that discharge to Wetland F at the west outfall or at a separate level spreader outfall (called the east outfall of Wetland F in this report). Wetland F contains a 12'' PVC overflow pipe that conveys excess runoff to a large rock pad outfall at the entrance to the SR-520 conveyance system.

There is a separate 12'' conveyance system that is to be installed with the Hackathon project to convey runoff around Wetland F during the temporary Hackathon event. The conveyance system collects runoff from the gravel surface and tents installed for the event and will be relatively inactive when the gravel surfaces and tents have been removed after the event. The Hackathon conveyance system connects to the existing storm system that flows to the site outfall, downstream of the Wetland F overflow pipe.

DEVELOPED STORM SYSTEM DESCRIPTION

Storm water runoff from the proposed development will be collected through underdrain systems or area drains then conveyed through closed pipes to either Wetland F or directly to the site outfall. A flow splitter will be used to direct only the required runoff volume to Wetland F and the overflow will be routed to the storm system installed during Hackathon that will convey the water around Wetland F to the site outfall through a 12'' closed pipe system. The new conveyance systems will be designed to convey the 50-year peak flow rate as described above. Conveyance calculations will be submitted in CCR when the site and flow splitter design are more complete.

OUTFALLS

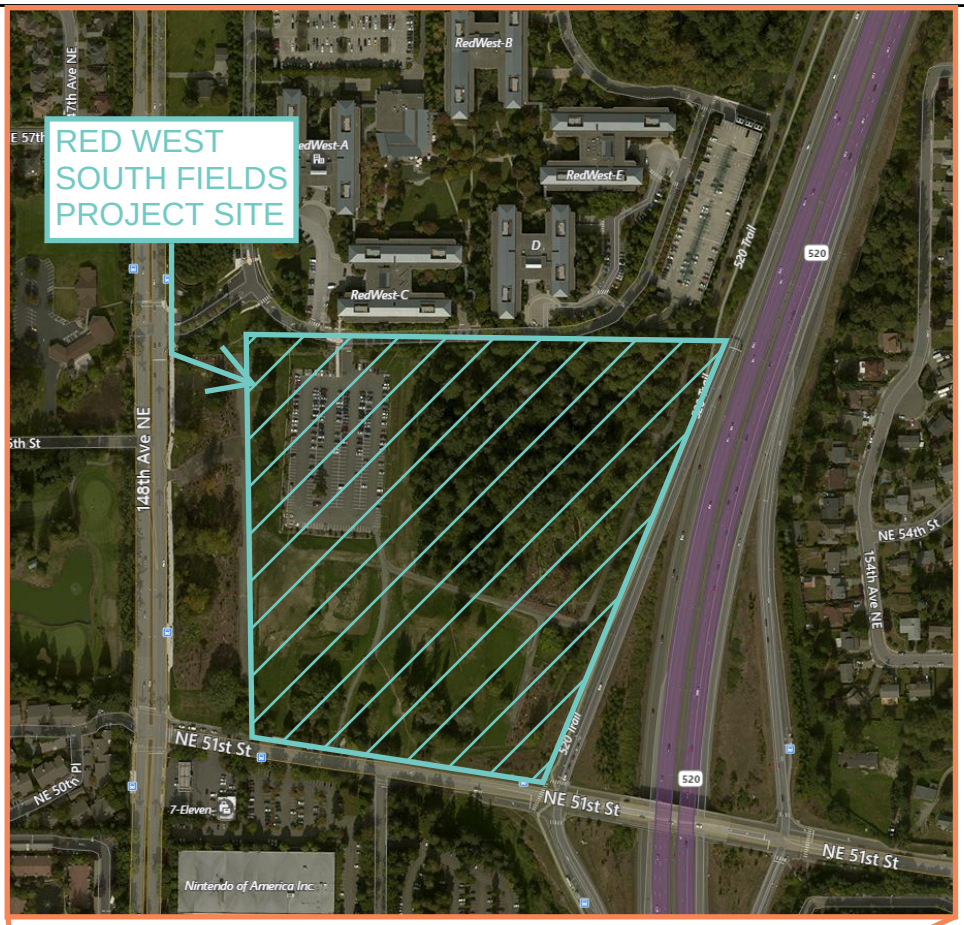
The proposed development will utilize the existing rock pad at the outfall to the SR 520 conveyance system.

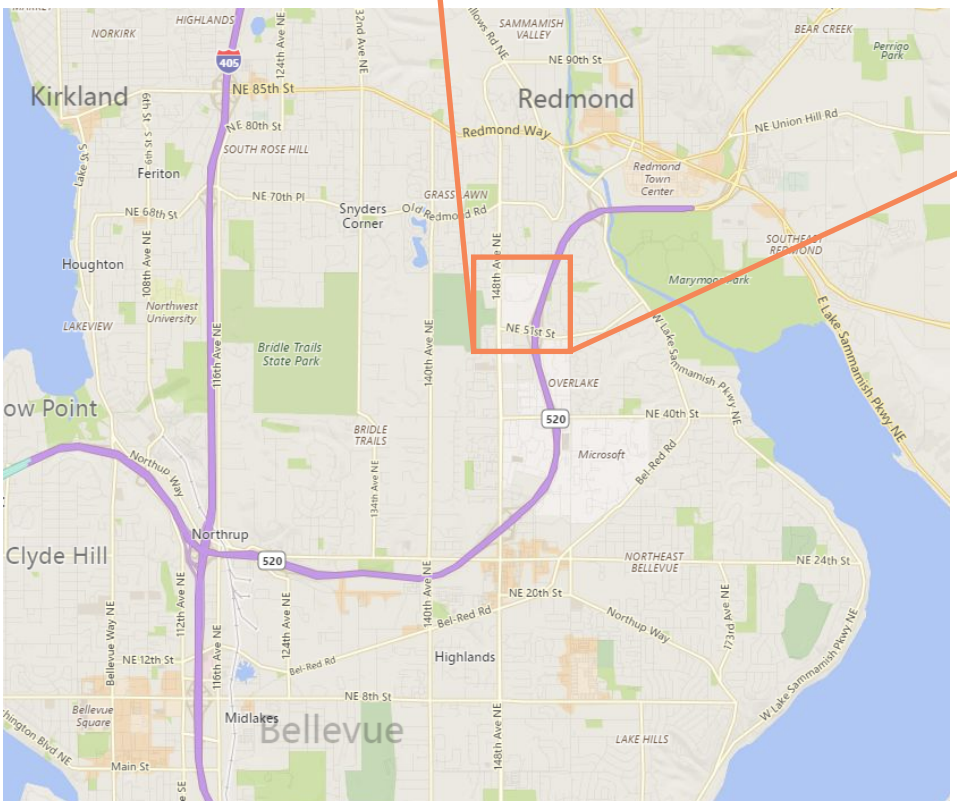
VIII. SUPPLEMENTAL REPORTS AND REQUIRMENT SUMMARY

Geotechnical Engineering Services Report–Microsoft RedWest South Project, prepared by GeoEngineers, Inc. on November 13, 2008.

APPENDIX A: FIGURES

Figure 1 – Vicinity Map
Figure 2A-G – City of Redmond Maps
Figure 3 – Flow Chart for Determining Requirements for New Development
Figure 4 – Existing Conditions
Figure 4b – Existing Wetland Basins
Figure 4c – Existing Impervious Area
Figure 5 – Proposed Conditions
Figure 5b – Proposed Wetland Basins
Figure 5c – Proposed Impervious Area
Figure 5d – New/Replaced Impervious Area
Figure 6 – Site Map
Figure 7 – LID Site Plan





SCALE: NTS

VICINITY MAP

Project: MICROSOFT RED WEST SOUTH FIELDS

Project No: Date:

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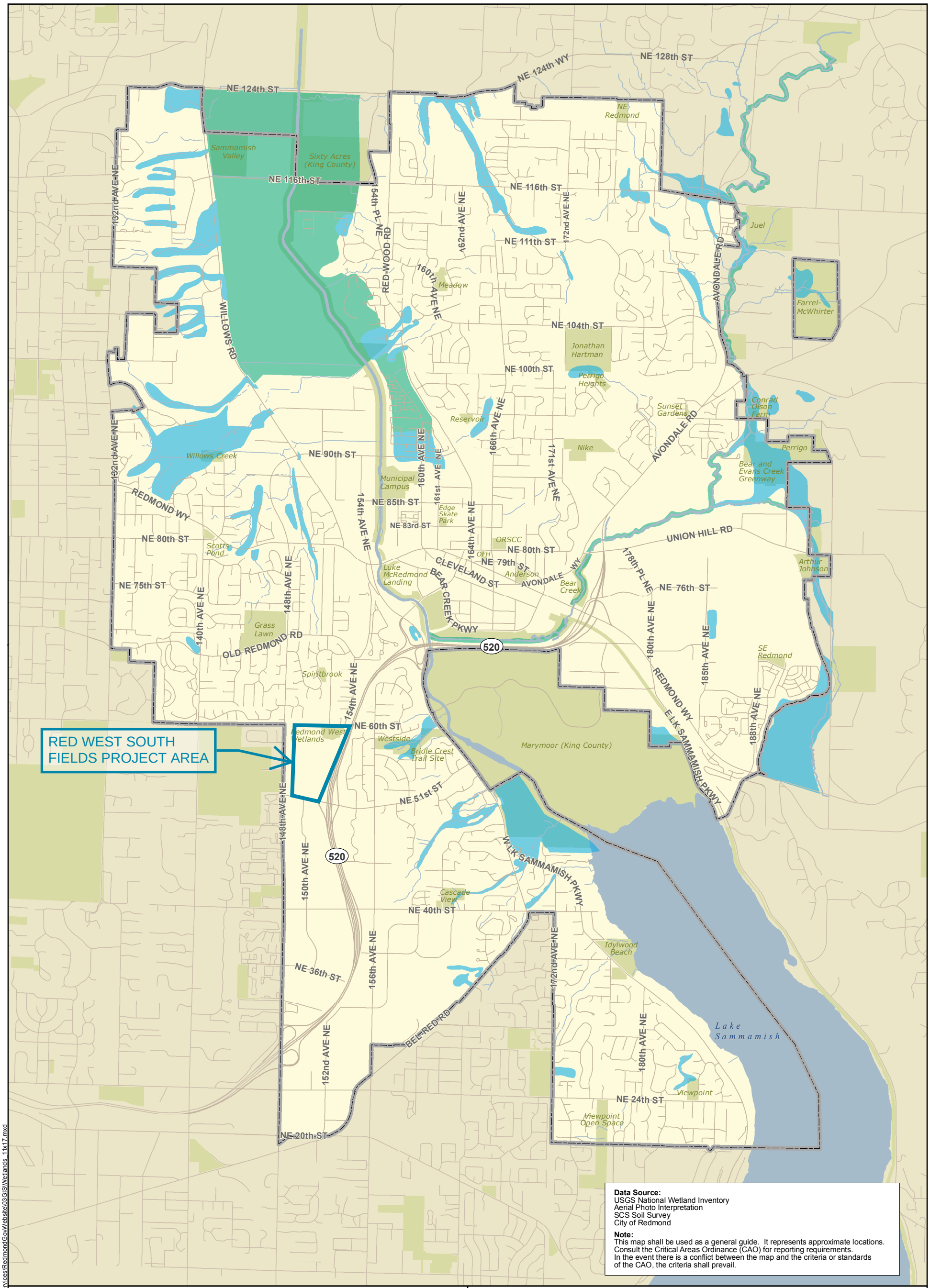

Data Source:
USGS National Wetland Inventory
Aerial Photo Interpretation
SCS Soil Survey
City of Redmond

Note:
This map shall be used as a general guide. It represents approximate locations.
Consult the Critical Areas Ordinance (CAO) for reporting requirements.
In the event there is a conflict between the map and the criteria or standards of the CAO, the criteria shall prevail.

Wetlands

Critical Areas Map

City of Redmond, Washington

Effective: 05/28/2005

0 0.5 1
Miles

N

Mixed Wetland/Upland
Wetland
City Limit
Park and Open Space
Water

Figure 2A

Disclaimer: This map is created and maintained by GIS Services Group,Finance and Information Services, City of Redmond, Washington, for reference purposes only.

The City makes no guarantee as to the accuracy of the features shown on this map.

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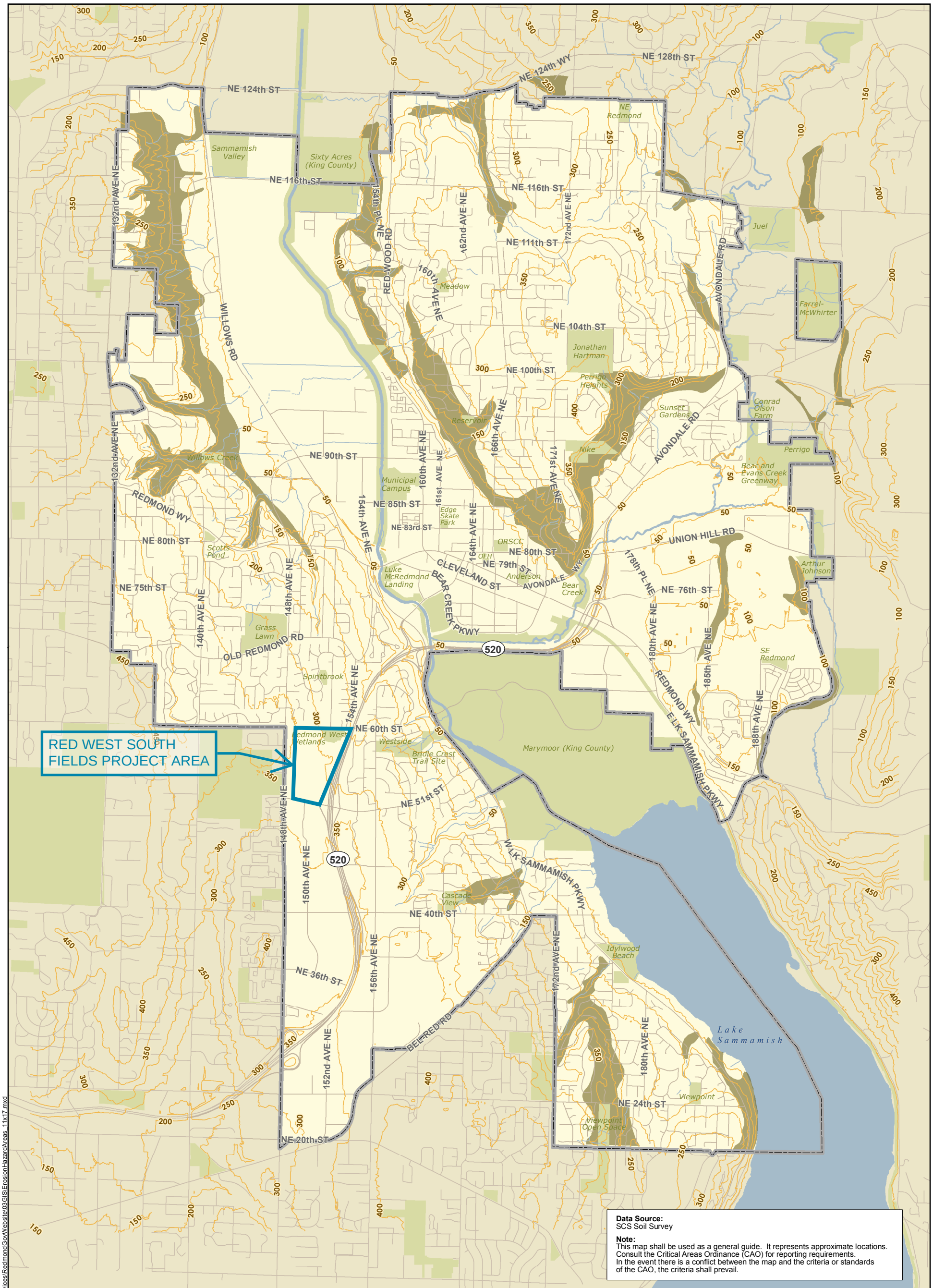

Data Source:
SCS Soil Survey

Note:
This map shall be used as a general guide. It represents approximate locations. Consult the Critical Areas Ordinance (CAO) for reporting requirements. In the event there is a conflict between the map and the criteria or standards of the CAO, the criteria shall prevail.

Erosion Hazard Areas

Critical Areas Map

City of Redmond, Washington

Effective: 05/28/2005

0 0.5 1
Miles

N

Contour
Erosion Hazard Area
City Limit
Park and Open Space
Water

Figure 2B

Disclaimer: This map is created and maintained by GIS Services Group,Finance and Information Services, City of Redmond, Washington, for reference purposes only.

The City makes no guarantee as to the accuracy of the features shown on this map.

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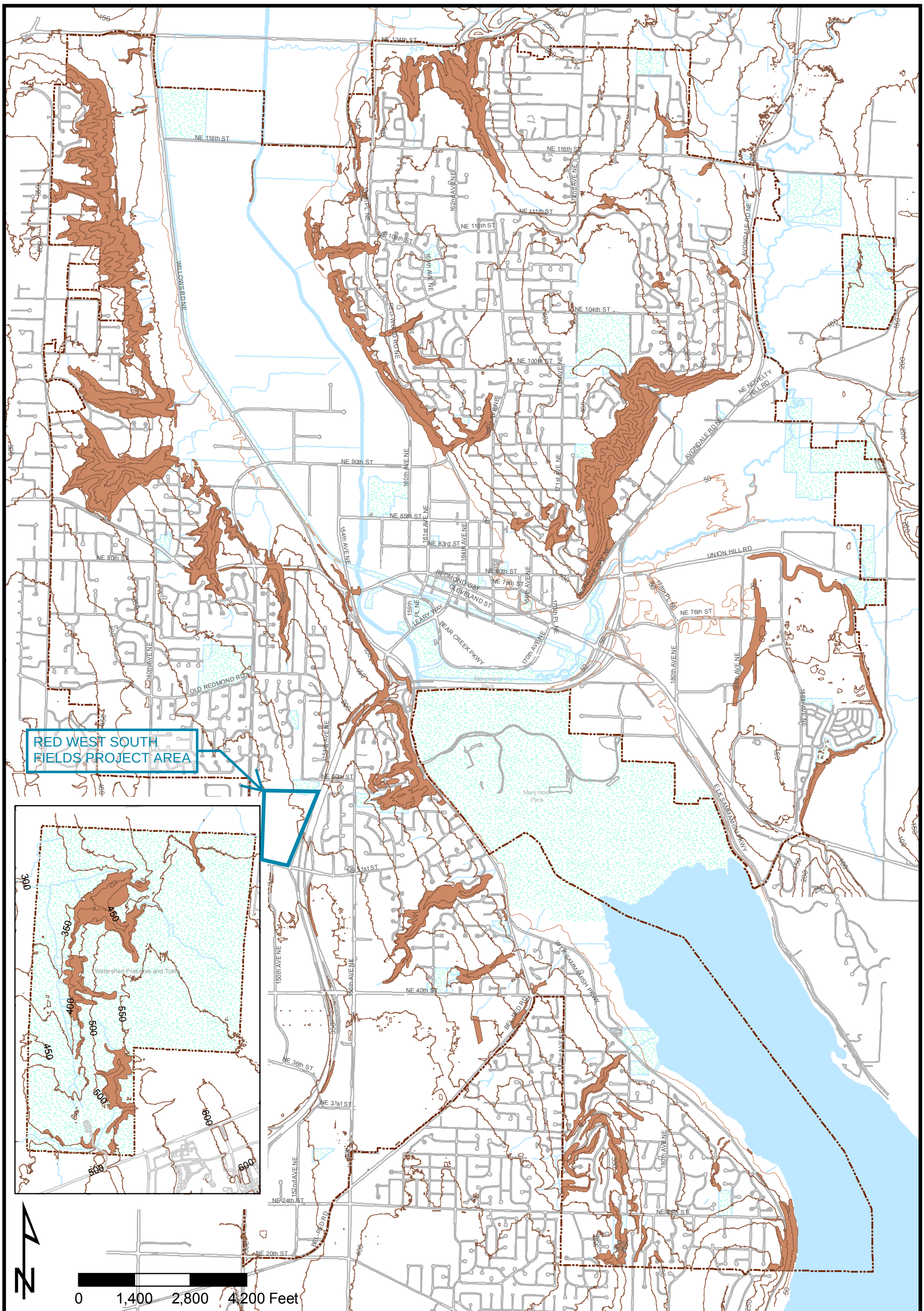

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City of Redmond

Critical Areas Map
Effective: 03/12/2016

Map 64.7 Landslide Hazards

Legend

- Landslide Hazards Areas
- Contours
- Park

Sources:
City of Redmond GIS Services
King County GIS

Note: This map shall be used as a general guide representing the approximate location of steep slopes, per RZC 21.64.060(A)(1)b.vii. The map does not necessarily ensure the presence or absence of landslide hazards. In the event of a conflict between the map and the criteria of the Critical Areas Ordinance (CAO), the criteria shall prevail. Consult the CAO (RZC 21.64) for reporting requirements.

Figure 2C

Frequently Flooded Areas

Critical Areas Map

City of Redmond, Washington

Effective: 05/28/2005

0 0.5 1
Miles

N

100 Year Floodplain

FEMA Floodway

City Limit

Park and Open Space

Water

Figure 2D

Disclaimer: This map is created and maintained by GIS Services Group,Finance and Information Services, City of Redmond, Washington, for reference purposes only.

The City makes no guarantee as to the accuracy of the features shown on this map.

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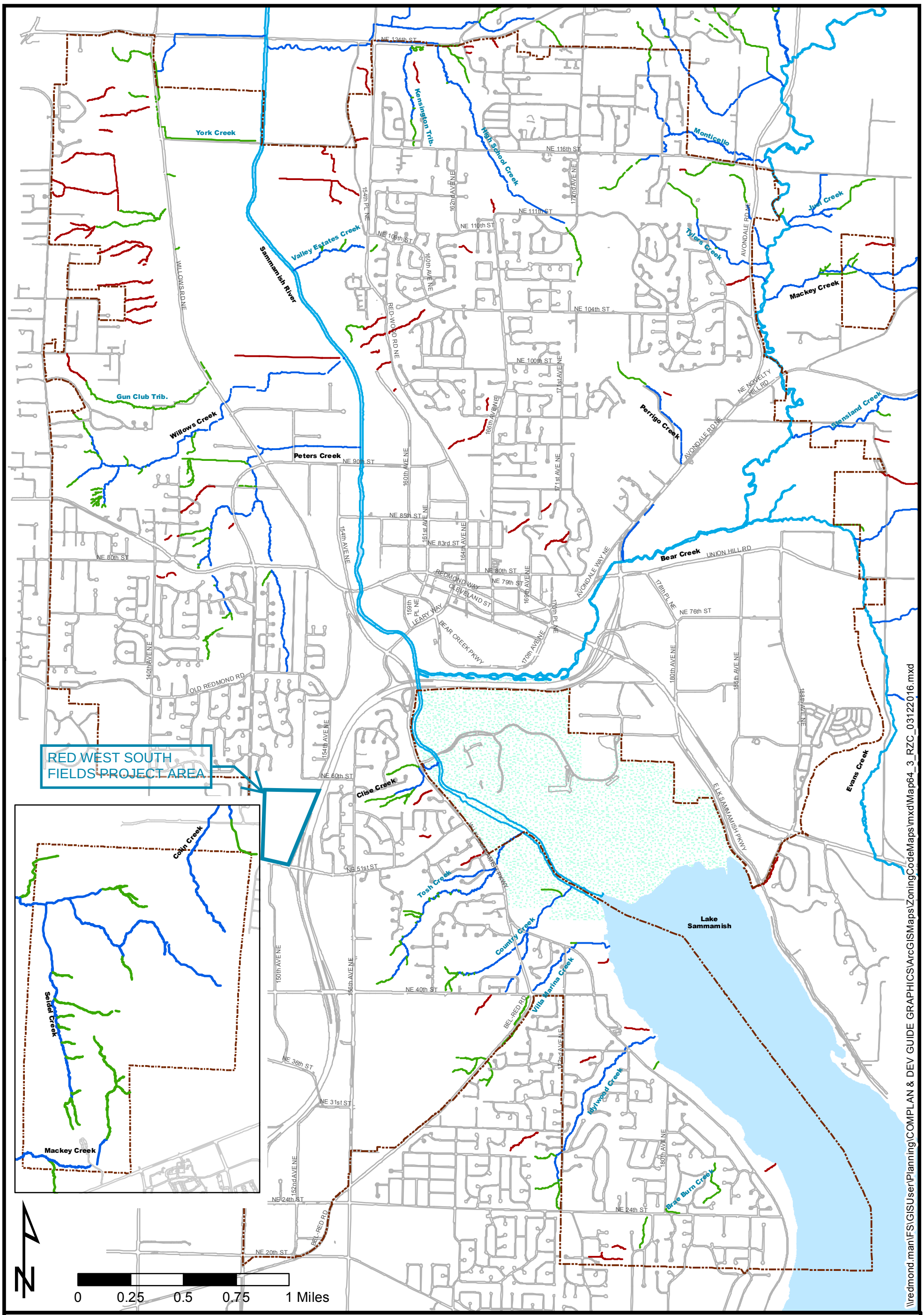


\\redmond.man\FS\GISUser\Planning\COMPLAN & DEV GUIDE GRAPHICS\ArcGISMaps\ZoningCodeMaps\mxd\Map64_3_RZC_03122016.mxd

City of Redmond

Figure 2E

Critical Areas Map
Effective: 3/12/2016

Stream Official USGS Stream Name
Stream Informal Stream Name

Class I Stream
Class II Stream
Class III Stream
Class IV Stream

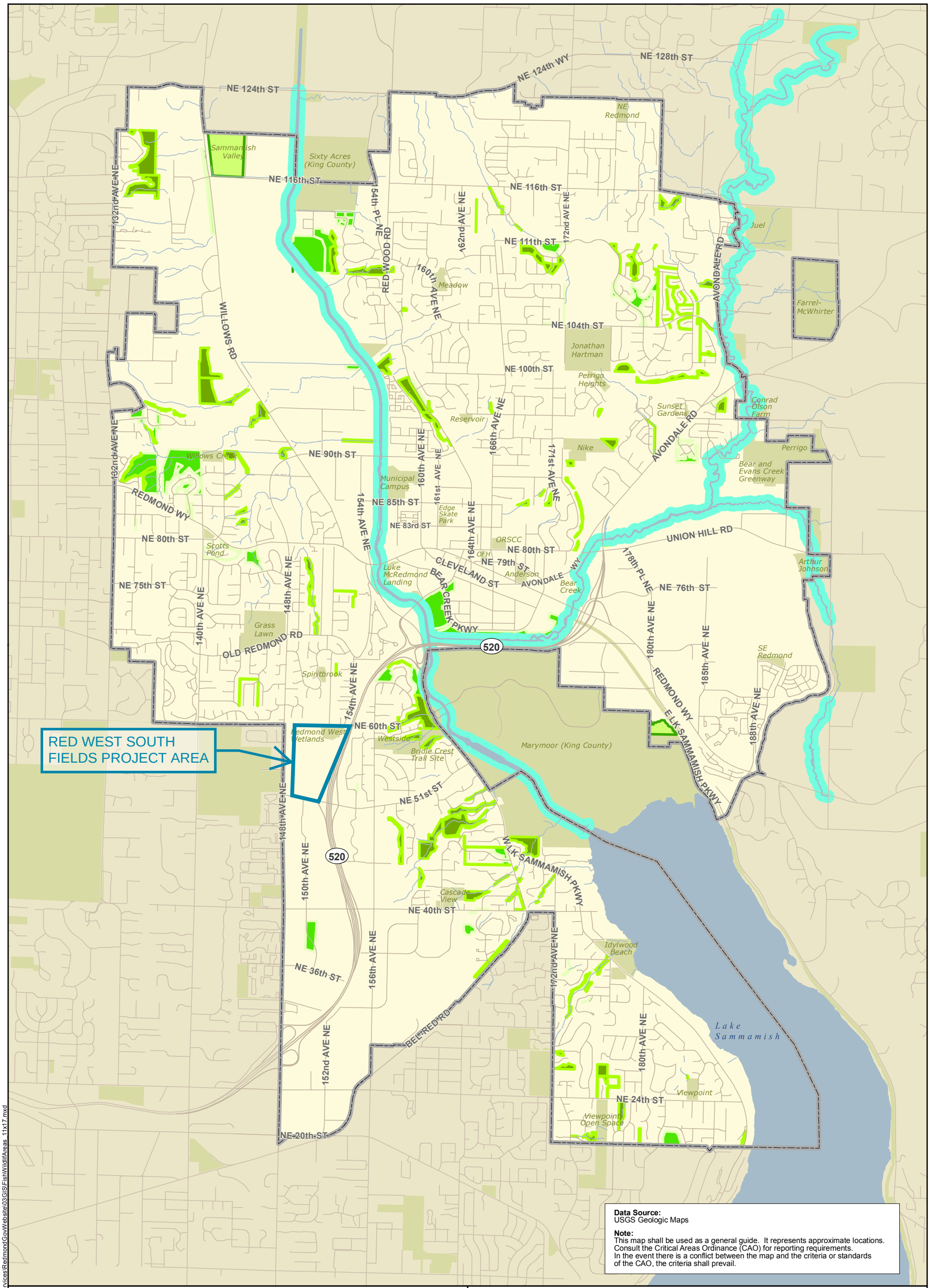
Map 64.3 Streams Classification

Sources:
City of Redmond Public Works, Natural Resources Division
City of Redmond GIS Services
Washington Trout / Wild Fish Conservancy
King County GIS

Note: This map shall be used as a general guide representing the approximate location of streams, per RZC 21.64.010(E)(2). The map does not necessarily ensure the presence or absence of streams. In the event of a conflict between the map and the criteria of the Critical Areas Ordinance (CAO), the criteria shall prevail. Consult the CAO (RZC 21.64) for reporting requirements

Note: Gaps in illustrated streams may indicate culverts, pipes, etc.

Note: Informal stream names may not conform to USGS policies and may change in the future.



Data Source:
USGS Geologic Maps

Note:
This map shall be used as a general guide. It represents approximate locations. Consult the Critical Areas Ordinance (CAO) for reporting requirements. In the event there is a conflict between the map and the criteria or standards of the CAO, the criteria shall prevail.

Fish and Wildlife Habitat Conservation Areas (Core Preservation Areas)

Critical Areas Map

City of Redmond, Washington

Effective: 05/28/2005

0 0.5 1
Miles

N

Class 1 Streams and Buffers

Native Growth Protection Easements

Open Space Easements

Transfer Development Rights Easements

City Limit

Park and Open Space

Water

Figure 2F

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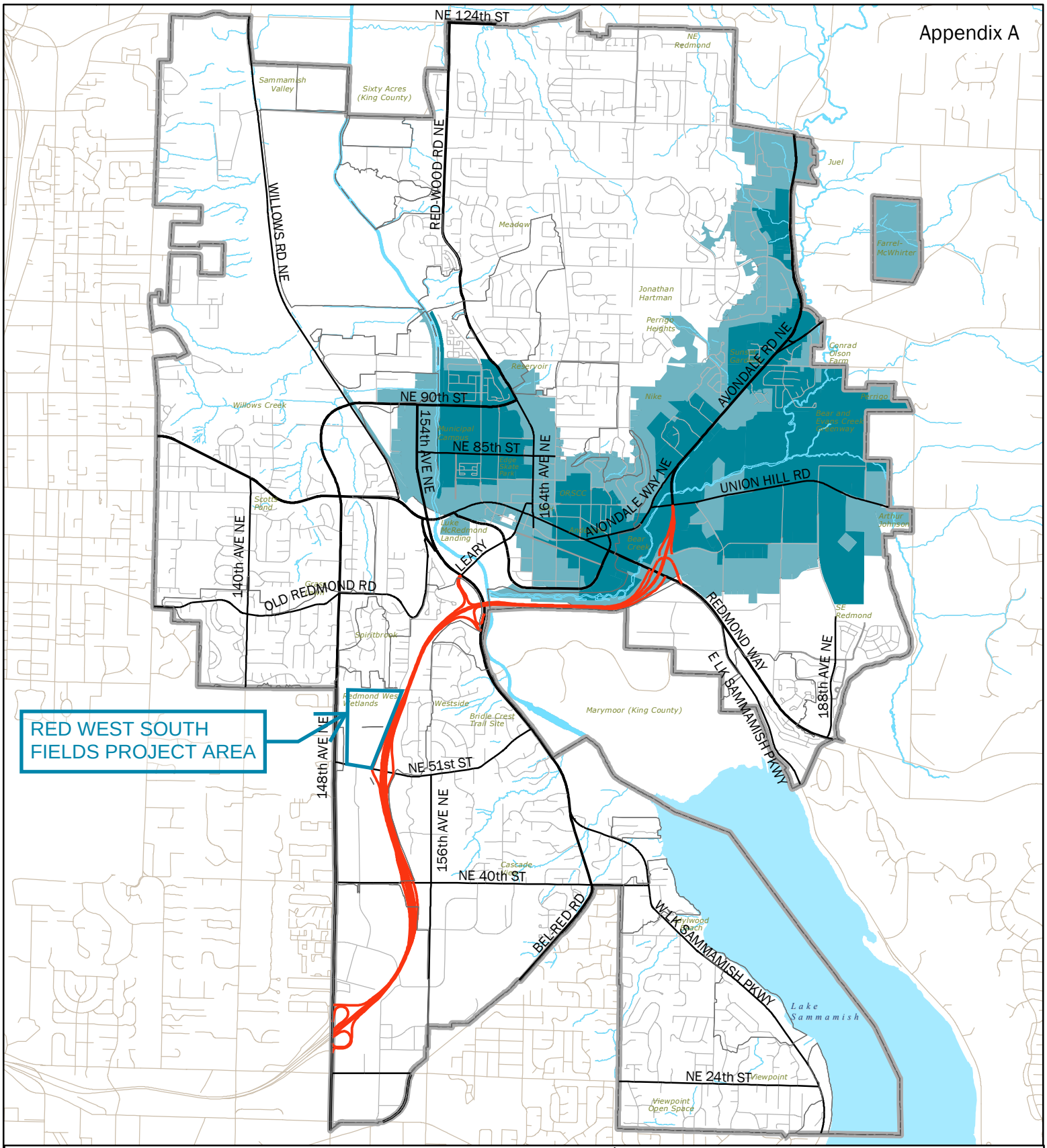
File Name:\\redmond.man\fs\GISUser\GIS\GISServicesProject\GISServices\RedmondGovWebsite\03GIS\FishWildlifAreas_11x17.mxd

Appendix A



Wellhead Protection Zones

City of Redmond, Washington

08/10/2016

Wellhead Zone 1

Wellhead Zone 2

N

0 0.5 1 Miles

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Figure 2G

File Name:Q:\Q:\NRDivision\NRFunctions\04MappingGISDatabase\SECTION

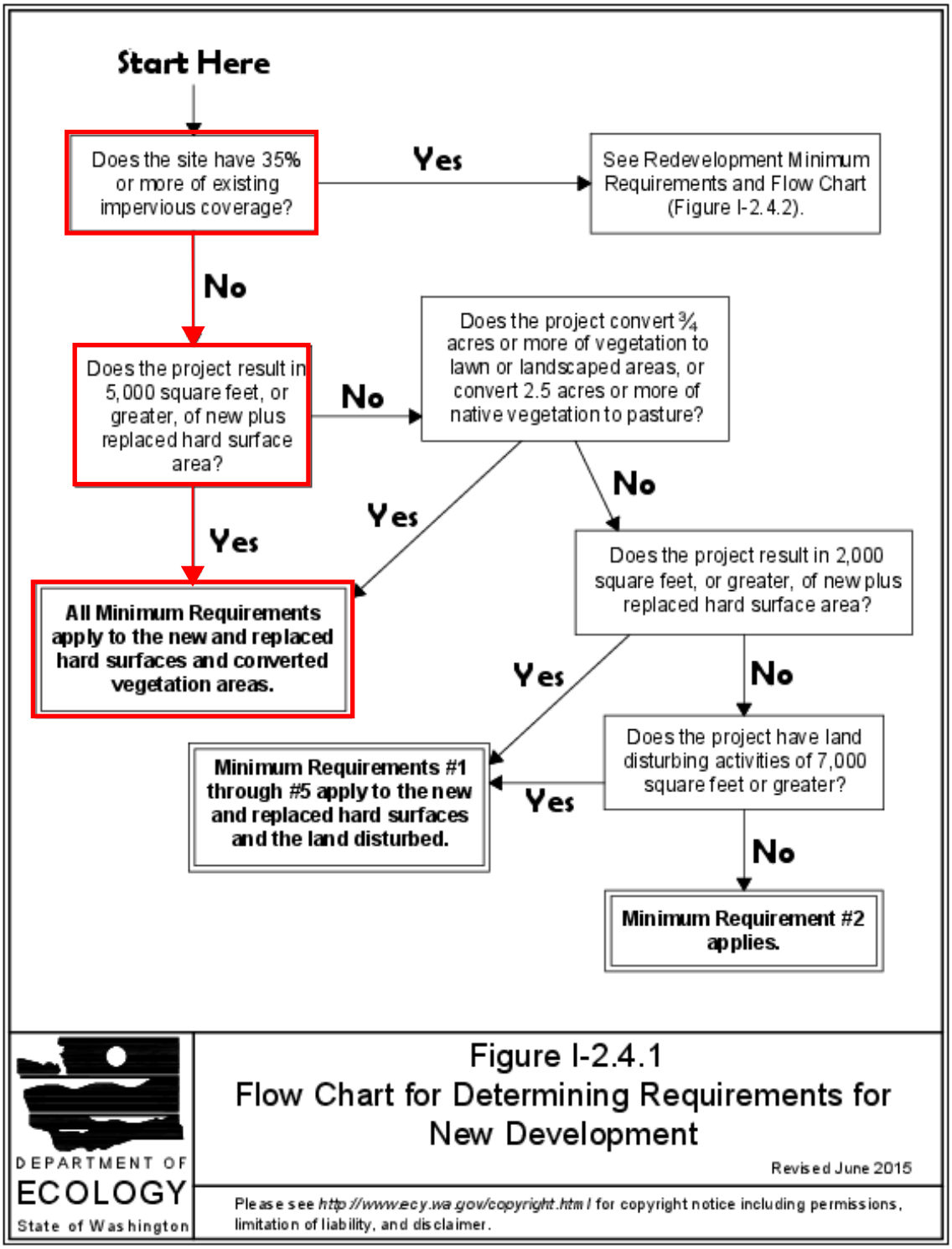
Start Here
Does the site have 35% or more of existing impervious coverage?
Yes
See Redevelopment Minimum Requirements and Flow Chart (Figure I-2.4.2).
No
Does the project result in 5,000 square feet, or greater, of new plus replaced hard surface area?
No
Does the project convert ¾ acres or more of vegetation to lawn or landscaped areas, or convert 2.5 acres or more of native vegetation to pasture?
Yes
No
Yes
All Minimum Requirements apply to the new and replaced hard surfaces and converted vegetation areas.
Does the project result in 2,000 square feet, or greater, of new plus replaced hard surface area?
Yes
No
Minimum Requirements #1 through #5 apply to the new and replaced hard surfaces and the land disturbed.
Does the project have land disturbing activities of 7,000 square feet or greater?
Yes
No
Minimum Requirement #2 applies.
DEPARTMENT OF ECOLOGY
State of Washington
Figure I-2.4.1
Flow Chart for Determining Requirements for New Development
Revised June 2015
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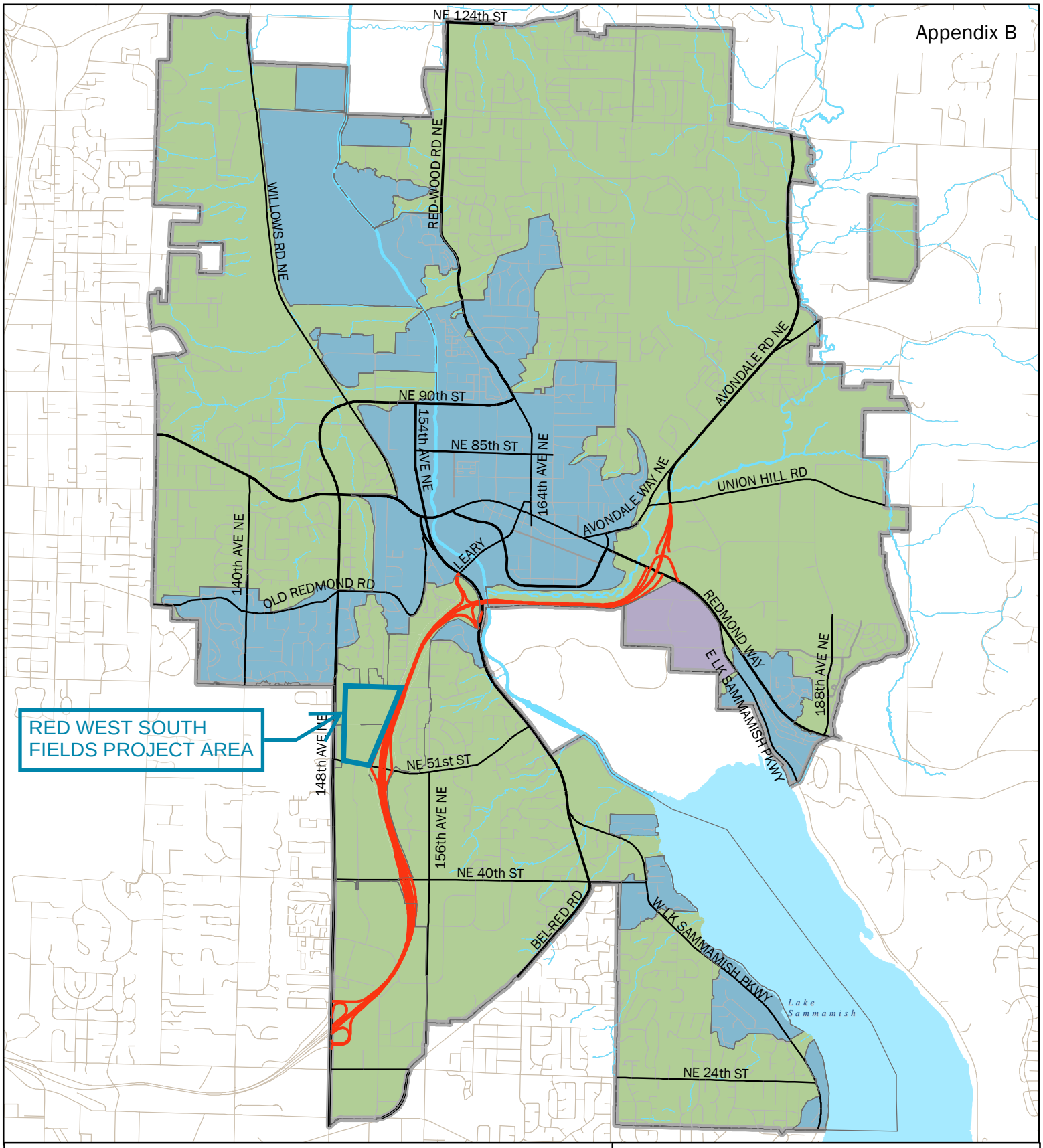
FIGURE 3

Appendix B



MR 5: Onsite Stormwater Management Custom Areas

City of Redmond, Washington

03/01/2017

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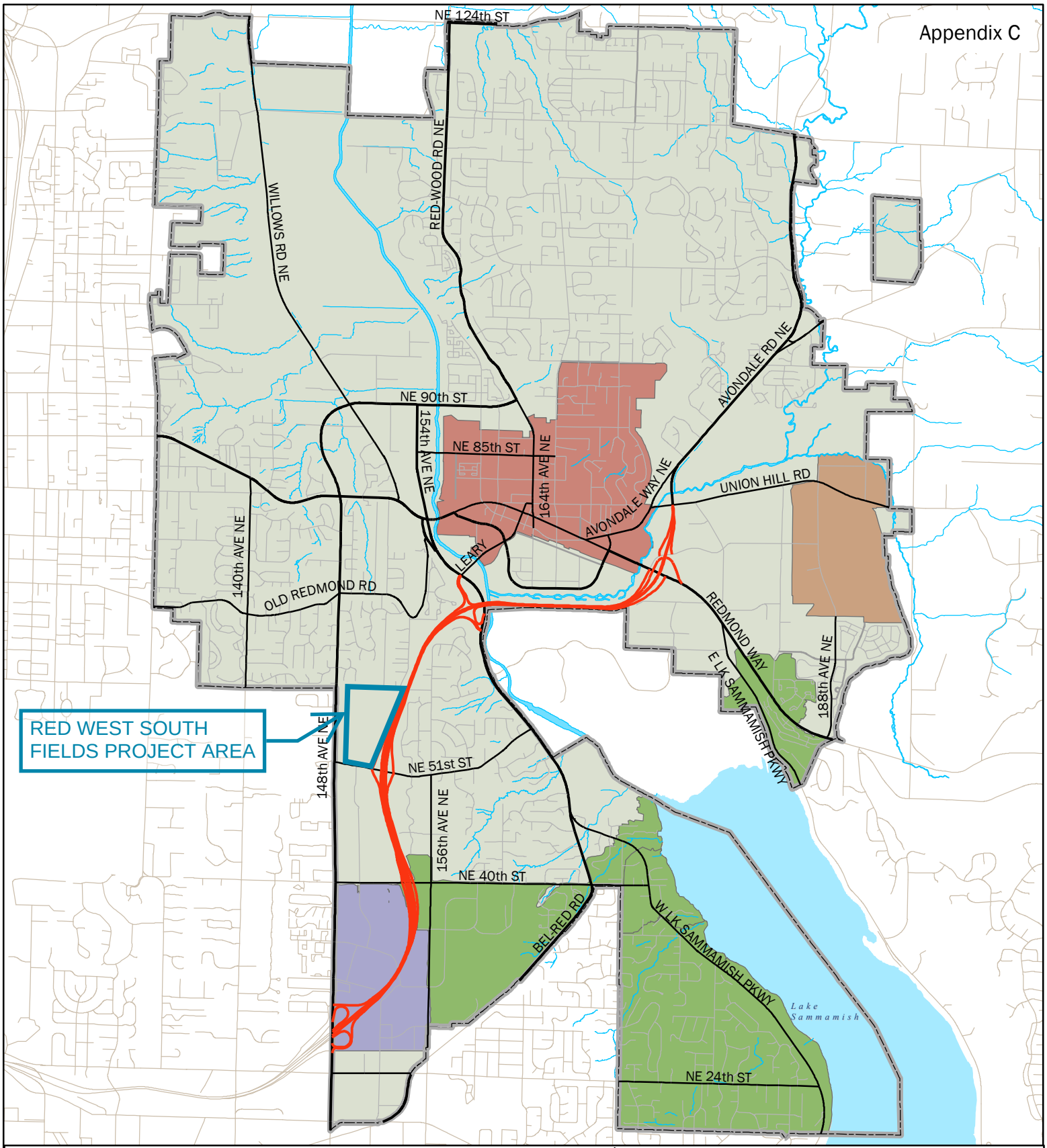
N

0 0.5 1 Miles

- Standard LID
- Flow Control Exempt
- Marymoor - 100% Infiltration

File Name:Q:\Q:\NRDivision\NRFunctions\04MappingGISDatabase\SECTION

Appendix C



MR 6: Runoff Treatment Custom Areas

City of Redmond, Washington

08/10/2016

CityofRedmond WASHINGTON

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N

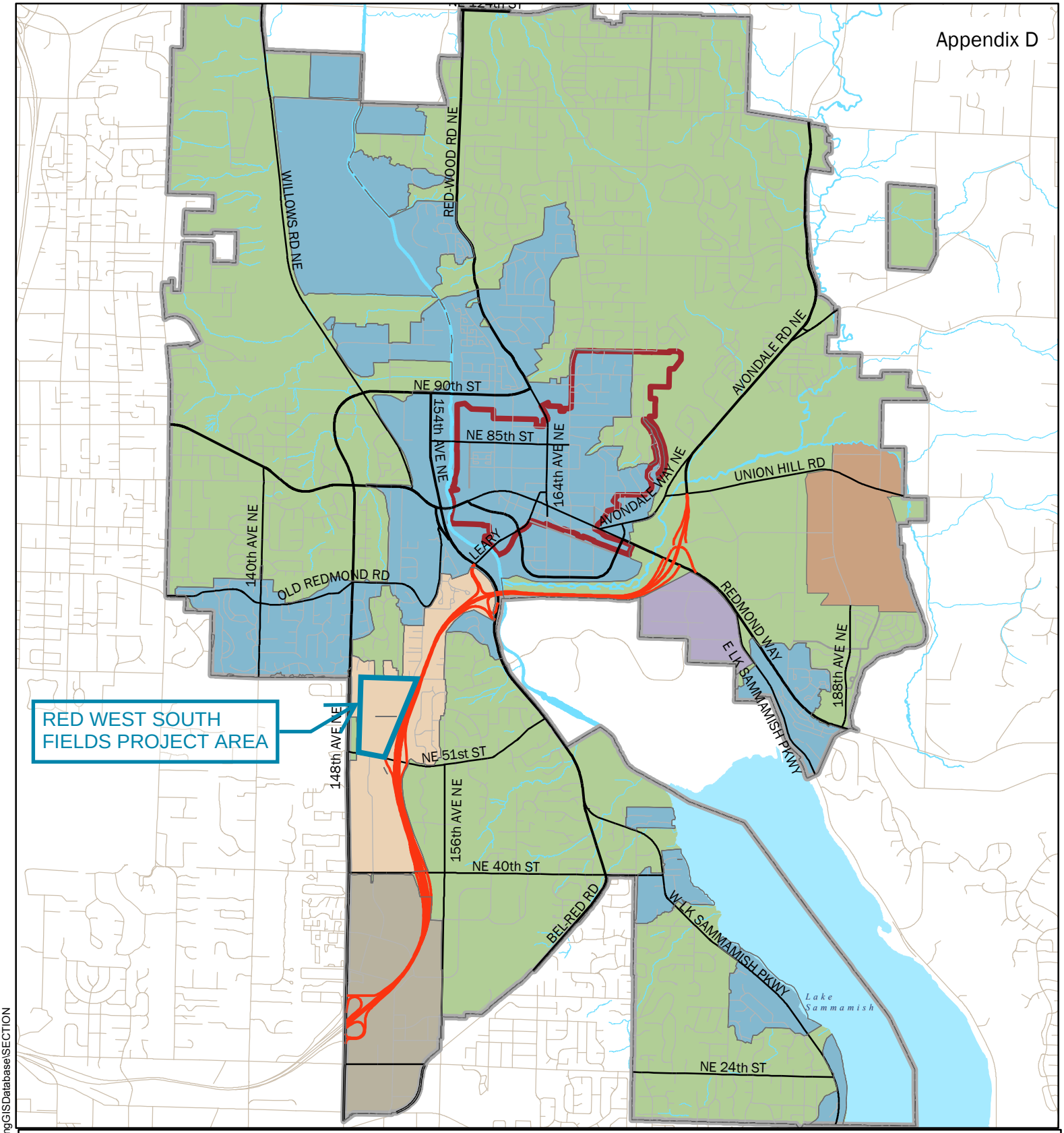
0 0.5 1 Miles

- Downtown Regional Facility Surcharge Area
- Overlake Regional Facility Surcharge Area
- Phosphorous Control
- SE Redmond
- Basic or Enhanced

Note: Oil/water separator applies where required city wide

File Name:Q:\Q:\NRDivision\NRFunctions\04MappingGISDatabase\SECTION

Appendix D



MR 7: Flow Control Requirements Custom Areas

City of Redmond, Washington

08/10/2016

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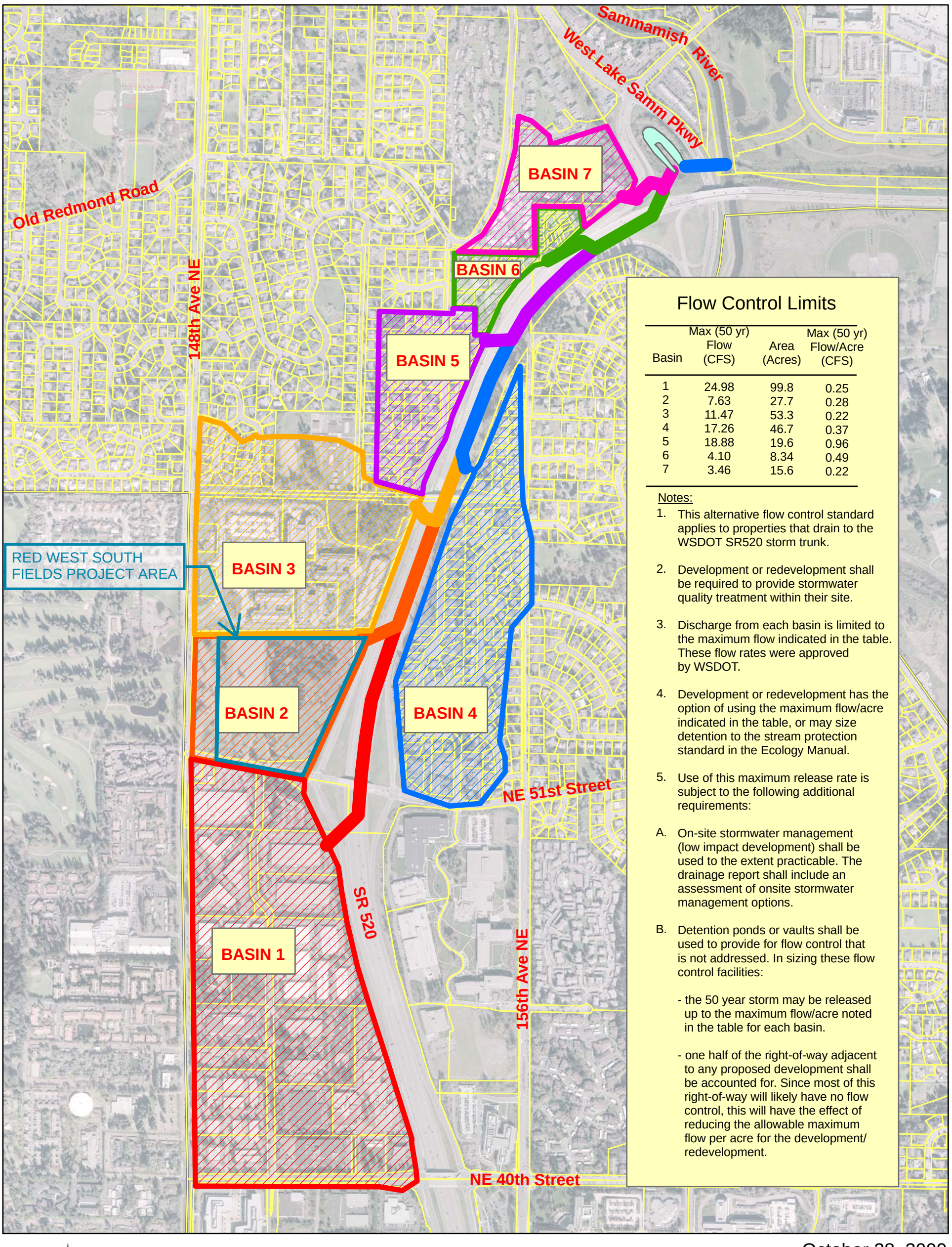
0 0.5 1 Miles

N

- Downtown Regional Facility Surcharge Area
- SE Redmond
- 520 Drainage Basin
- Flow Control Required
- Flow Exempt
- Overlake Regional Facility Surcharge Area
- Marymoor - 100% Infiltration

File Name:Q:\Q:\NRDivision\NRFunctions\04MappingGISDatabase\SECTION

CITY OF REDMOND PUBLIC WORKS
NORTH OVERLAKE DRAINAGE AREA
ALTERNATIVE FLOW CONTROL STANDARD



Flow Control Limits

Basin	Max (50 yr) Flow (CFS)	Area (Acres)	Max (50 yr) Flow/Acre (CFS)
1	24.98	99.8	0.25
2	7.63	27.7	0.28
3	11.47	53.3	0.22
4	17.26	46.7	0.37
5	18.88	19.6	0.96
6	4.10	8.34	0.49
7	3.46	15.6	0.22

Notes:

1. This alternative flow control standard applies to properties that drain to the WSDOT SR520 storm trunk.

2. Development or redevelopment shall be required to provide stormwater quality treatment within their site.

3. Discharge from each basin is limited to the maximum flow indicated in the table. These flow rates were approved by WSDOT.

4. Development or redevelopment has the option of using the maximum flow/acre indicated in the table, or may size detention to the stream protection standard in the Ecology Manual.

5. Use of this maximum release rate is subject to the following additional requirements:

A. On-site stormwater management (low impact development) shall be used to the extent practicable. The drainage report shall include an assessment of onsite stormwater management options.

B. Detention ponds or vaults shall be used to provide for flow control that is not addressed. In sizing these flow control facilities:

- the 50 year storm may be released up to the maximum flow/acre noted in the table for each basin.

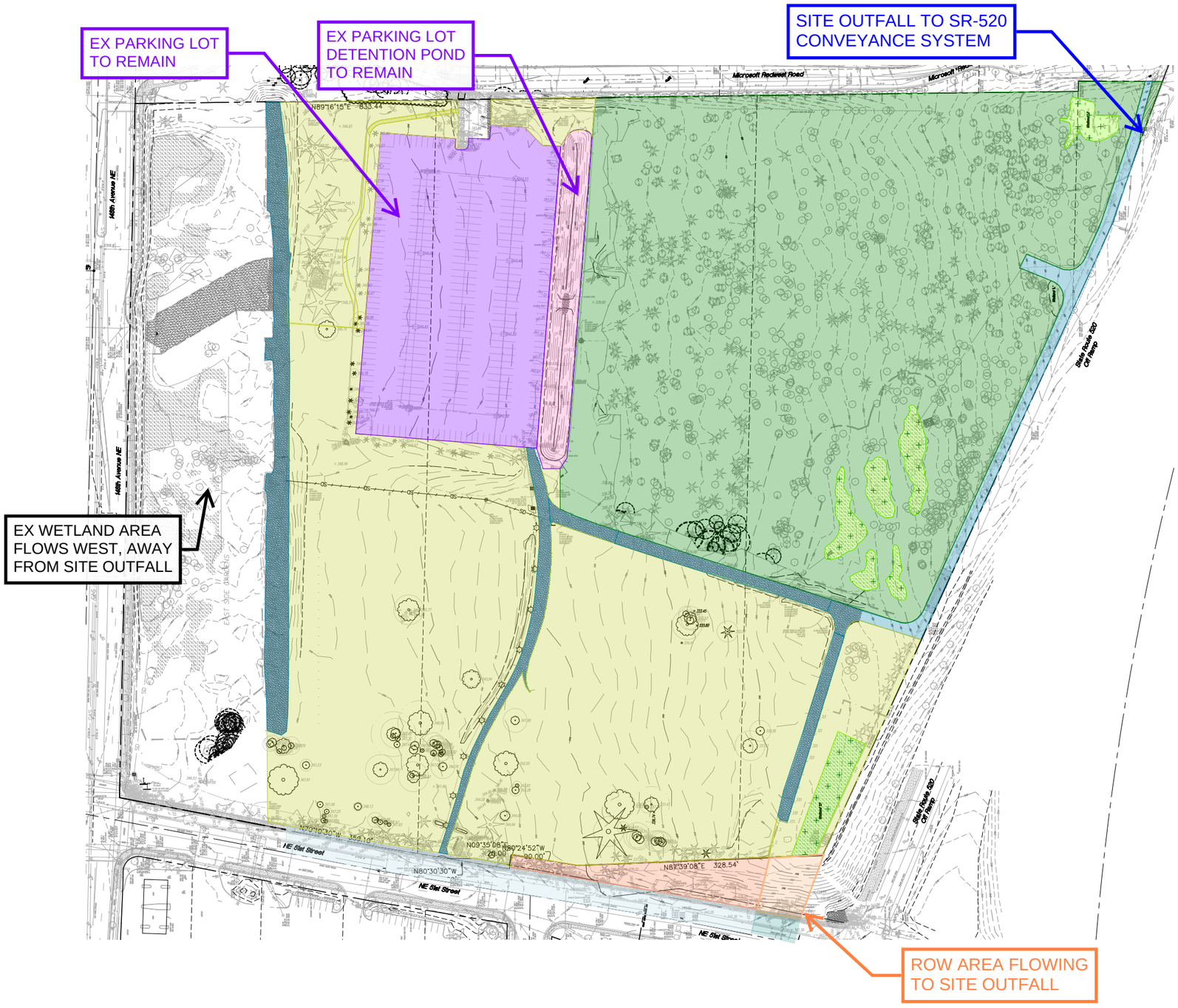
- one half of the right-of-way adjacent to any proposed development shall be accounted for. Since most of this right-of-way will likely have no flow control, this will have the effect of reducing the allowable maximum flow per acre for the development/ redevelopment.

October 28, 2009

tyofRedmond
ASHINGTON

N
W E
S

0 360 720 1,440 2,160 2,880
Feet



LEGEND

Description	Quantity
Impervious Area	1.25 Ac (54,450 SF)
Pervious Area	9.25 Ac (402,930 SF)
Forested Area	8.97 Ac (390,733 SF)
Saturated Forest Area (Wetlands)	0.44 Ac (19,166 SF)
Ex Parking Lot to Remain	2.28 Ac (99,317 SF)
Ex Parking Lot Det Pond to Remain	0.26 Ac (11,326 SF)
ROW Impervious Area	0.49 Ac (21,344 SF)
ROW Pervious Area	0.41 Ac (17,860 SF)
Total Basin Area	23.35 Ac (1.02M SF)

Scale: 1"=150'
0' 75' 150'

EXISTING CONDITIONS

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Project: Microsoft Red West South Fields

Project No: C182071-04 Date: 04/23/2019

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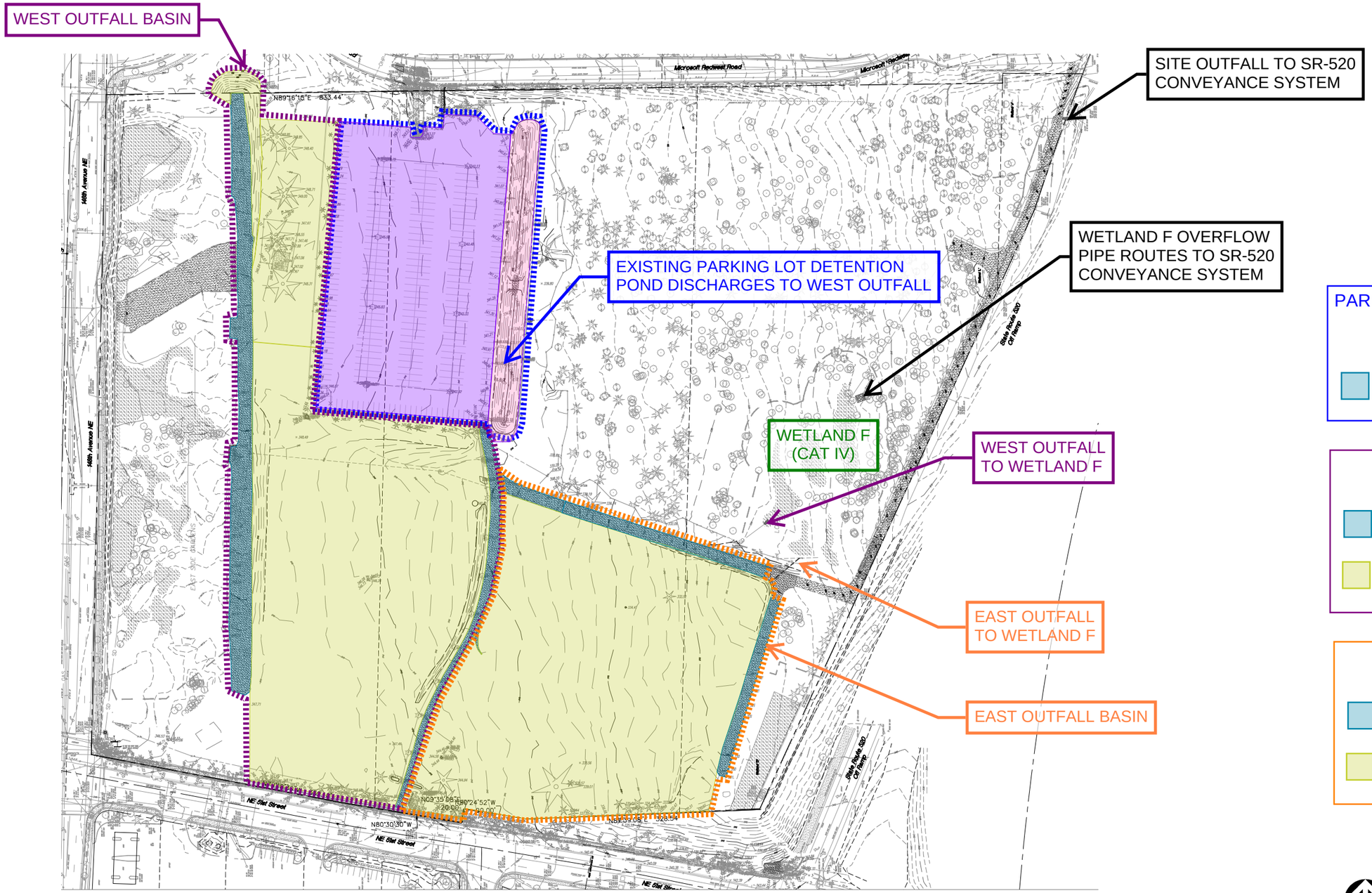

PARKING LOT DETENTION FLOWING TO WEST OUTFALL

Description	Quantity
Impervious Area	2.28 Ac (99,317 SF)

WEST OUTFALL BASIN

Description	Quantity
Impervious Area	0.71 Ac (30,928 SF)
Pervious Area	4.69 Ac (204,296 SF)

EAST OUTFALL BASIN

Description	Quantity
Impervious Area	0.24 Ac (10,454 SF)
Pervious Area	3.70 Ac (161,172 SF)

Scale: 1"=150'
0' 75' 150'

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EXISTING WETLAND F BASINS

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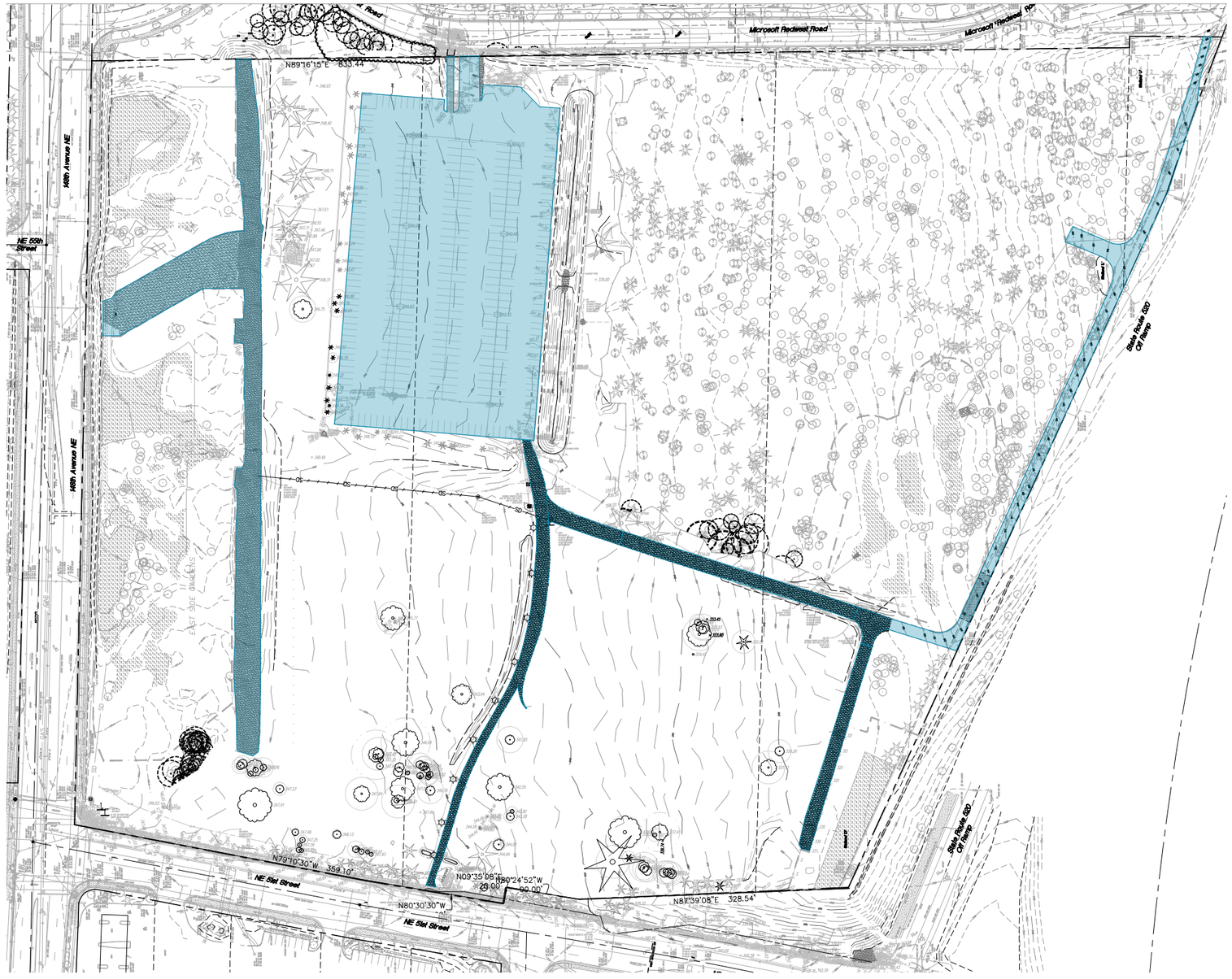

LEGEND

Description	Quantity
Impervious Area	3.51 Ac (152,896 SF)

Scale: 1"=150'

0' 75' 150'

EXISTING OVERALL IMPERVIOUS

Project: Microsoft Red West South Fields

Project No: C182071-04 Date: 04/23/2019

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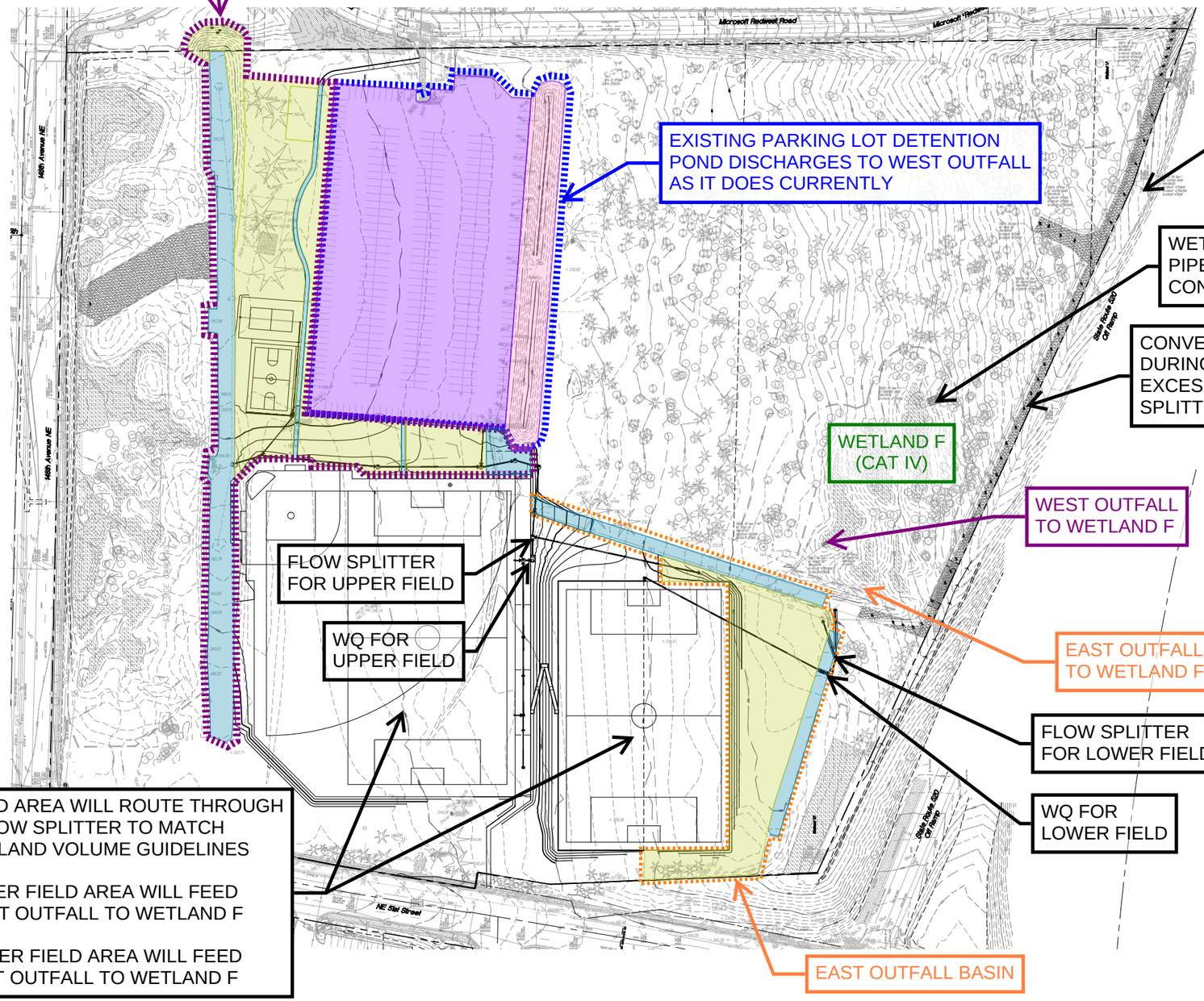

FLOW SPLITTER DESIGN AND WETLAND VOLUME CALCULATIONS TO BE SUBMITTED WITH CCR

FIELD AREA WILL ROUTE THROUGH A FLOW SPLITTER TO MATCH WETLAND VOLUME GUIDELINES

UPPER FIELD AREA WILL FEED WEST OUTFALL TO WETLAND F

LOWER FIELD AREA WILL FEED EAST OUTFALL TO WETLAND F

PARKING LOT DETENTION POND FLOWING TO WEST OUTFALL

Description	Quantity
Ex Parking Lot to Remain	2.28 Ac (99,317 SF)
Ex Parking Lot Det Pond to Remain	0.26 Ac (11,326 SF)

WEST OUTFALL BASIN

Description	Quantity
Impervious Area	0.61 Ac (26,572 SF)
Pervious Area	1.44 Ac (62,726 SF)

EAST OUTFALL BASIN

Description	Quantity
Impervious Area	0.24 Ac (10,454 SF)
Pervious Area	0.77 Ac (33,541 SF)

Scale: 1"=150'
0' 75' 150'

PROPOSED WETLAND F BASINS

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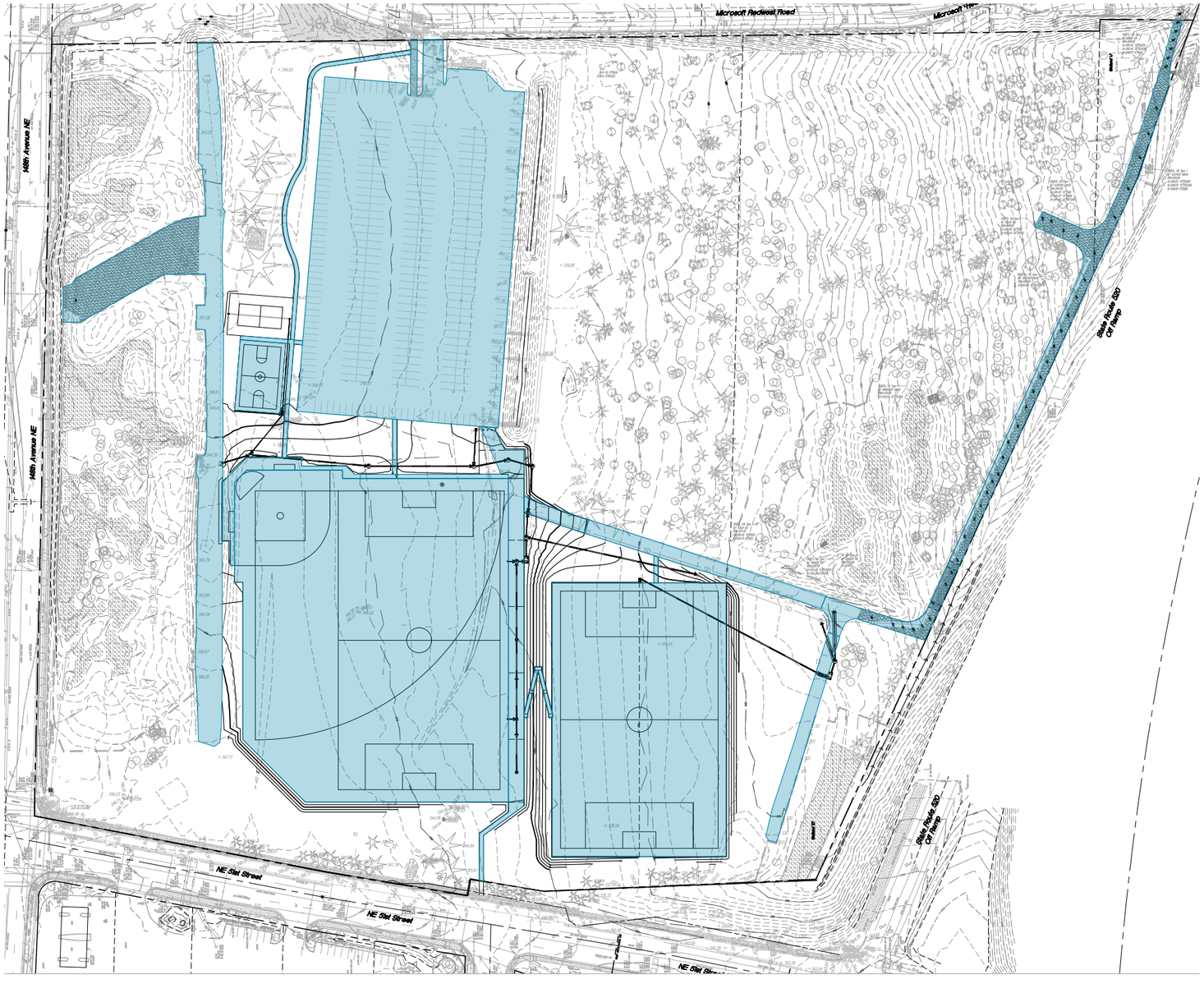

Note: Impervious area shown represents the overall impervious area on the parcel in the final condition and includes both existing and new/replaced impervious area

LEGEND

Description	Quantity
Impervious Area	8.60 Ac (374,616 SF)

Scale: 1"=150'

0' 75' 150'

PROPOSED OVERALL IMPERVIOUS

Project: Microsoft Red West South Fields

Project No: C182071-04 Date: 04/23/2019

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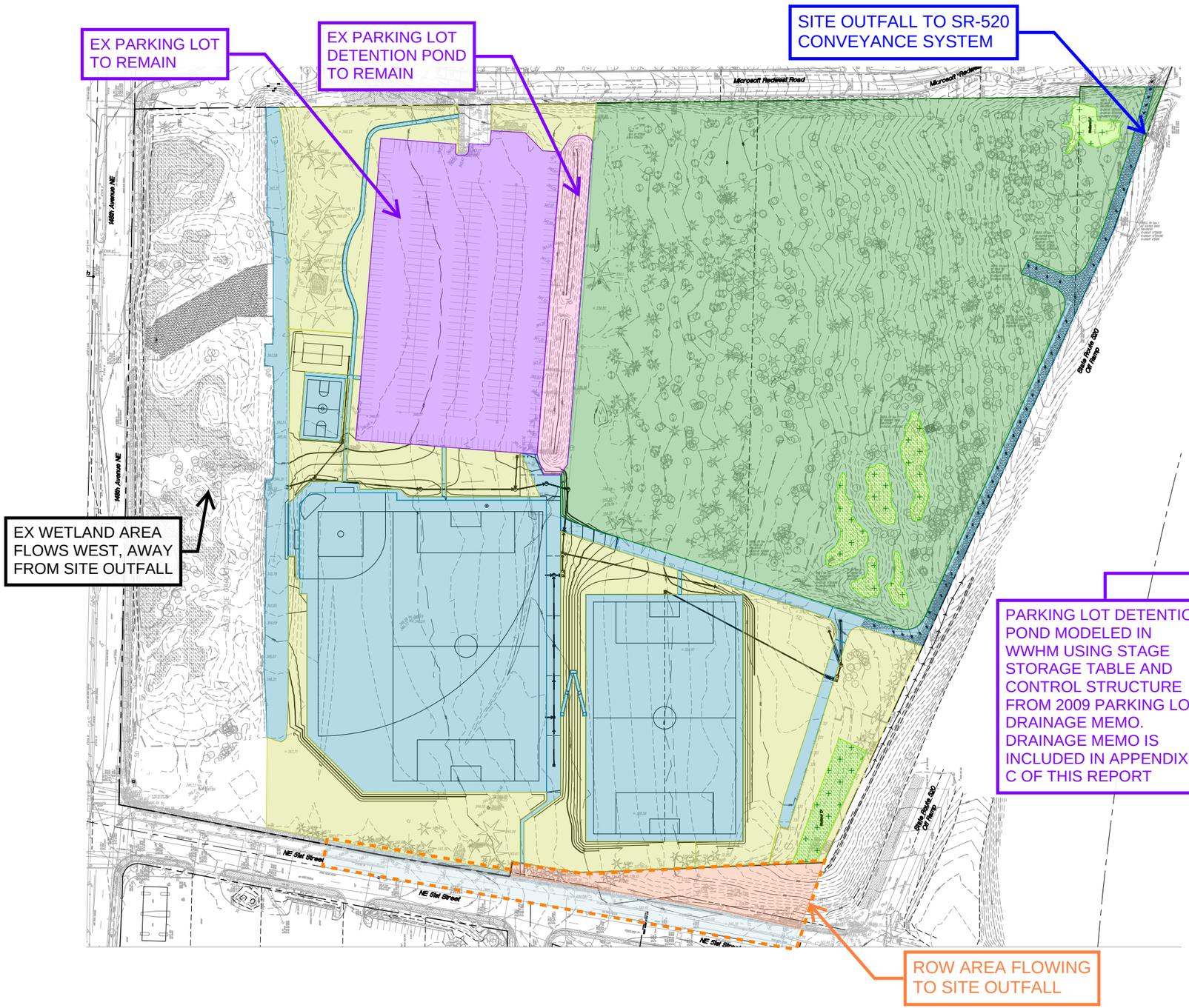

NORTH OVERLAKE ALT FLOW CONTROL STANDARD

Project area directly discharges to Sammamish River through the SR-520 conveyance system. Flow control for the project is regulated by the North Overlake Alternative Flow Control Standard, which limits the max 50-year flow rate based on the capacity of the SR-520 conveyance system. The project area makes up Basin 2 of the North Overlake Flow Control Area, which regulates the Developed Max 50-year peak flow to 0.28 CFS/Acre.

OUTFALL BASIN AREA

Description	Quantity (Ac)	Developed 50-year Peak Flow (CFS)
Impervious Area	6.04 Ac (263,102 SF)	
Pervious Area	4.46 Ac (194,278 SF)	
Forested Area	8.97 Ac (390,733 SF)	
Saturated Forest Area (Wetlands)	0.44 Ac (19,166 SF)	
Ex Parking Lot to Remain	2.28 Ac (99,317 SF)	
Ex Parking Lot Det Pond to Remain	0.26 Ac (11,326 SF)	
Total Site Area	22.45 Ac (977,922 SF)	**6.07 CFS**
Allowable 50-year Max Flow (CFS)	22.45 Acres @ 0.28 CFS/Ac =	**6.28 CFS**

ROW AREA (SEE NOTE BELOW)

Description	Quantity
Impervious Area	0.49 Ac (21,344 SF)
Pervious Area	0.41 Ac (17,860 SF)

*ROW area is included in developed 50-year peak flow calculation but is not included in Total Area for calculating Allowable 50-year Max Flow per flow control standard

Scale: 1"=150'
0' 75' 150'

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Project: Microsoft Red West South Fields

Project No: C182071-04 Date: 05/24/2019

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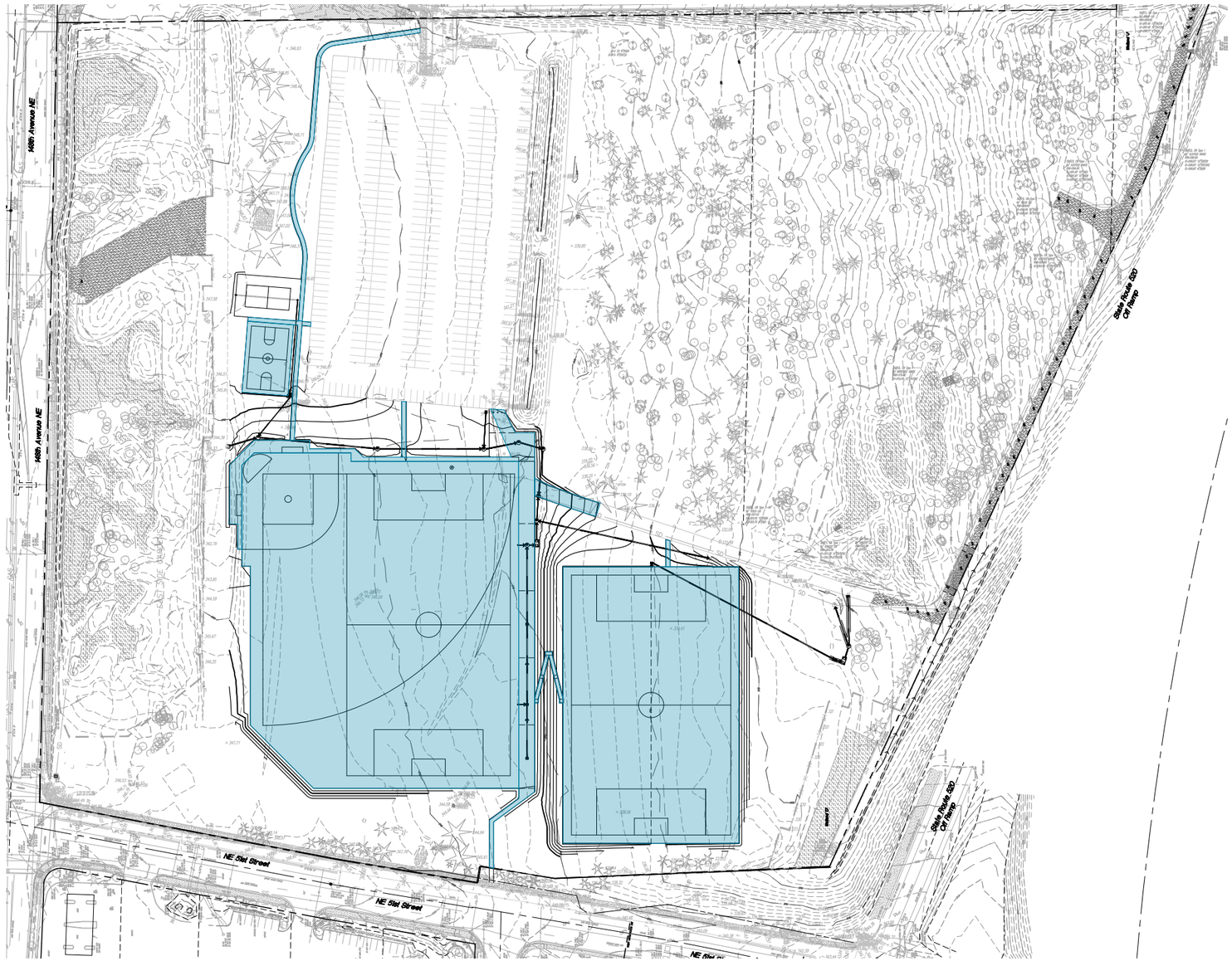
FIGURE 5
PROPOSED CONDITIONS



Note: Impervious area shown represents the impervious area that is being added or replaced with this project.

LEGEND

Description	Quantity
Impervious Area	4.99 Ac (217,364 SF)

Scale: 1"=150'

0' 75' 150'

NEW/REPLACED IMPERVIOUS

Project: Microsoft Red West South Fields

Project No: C182071-04 Date: 04/23/2019

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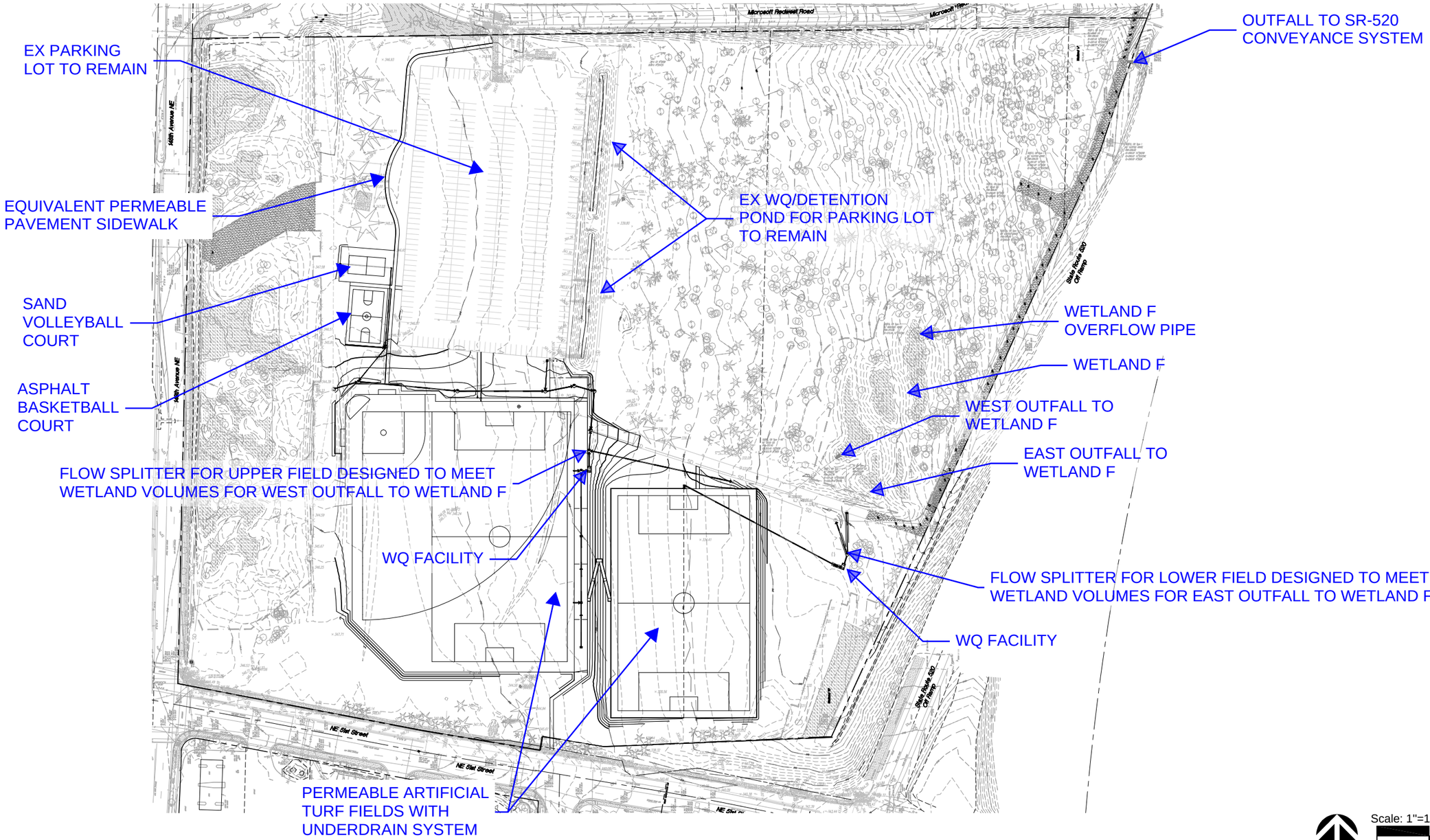

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Project: MICROSOFT RED WEST SOUTH FIELDS

Project No: C182071-04 Date: 04/23/2019

SITE MAP

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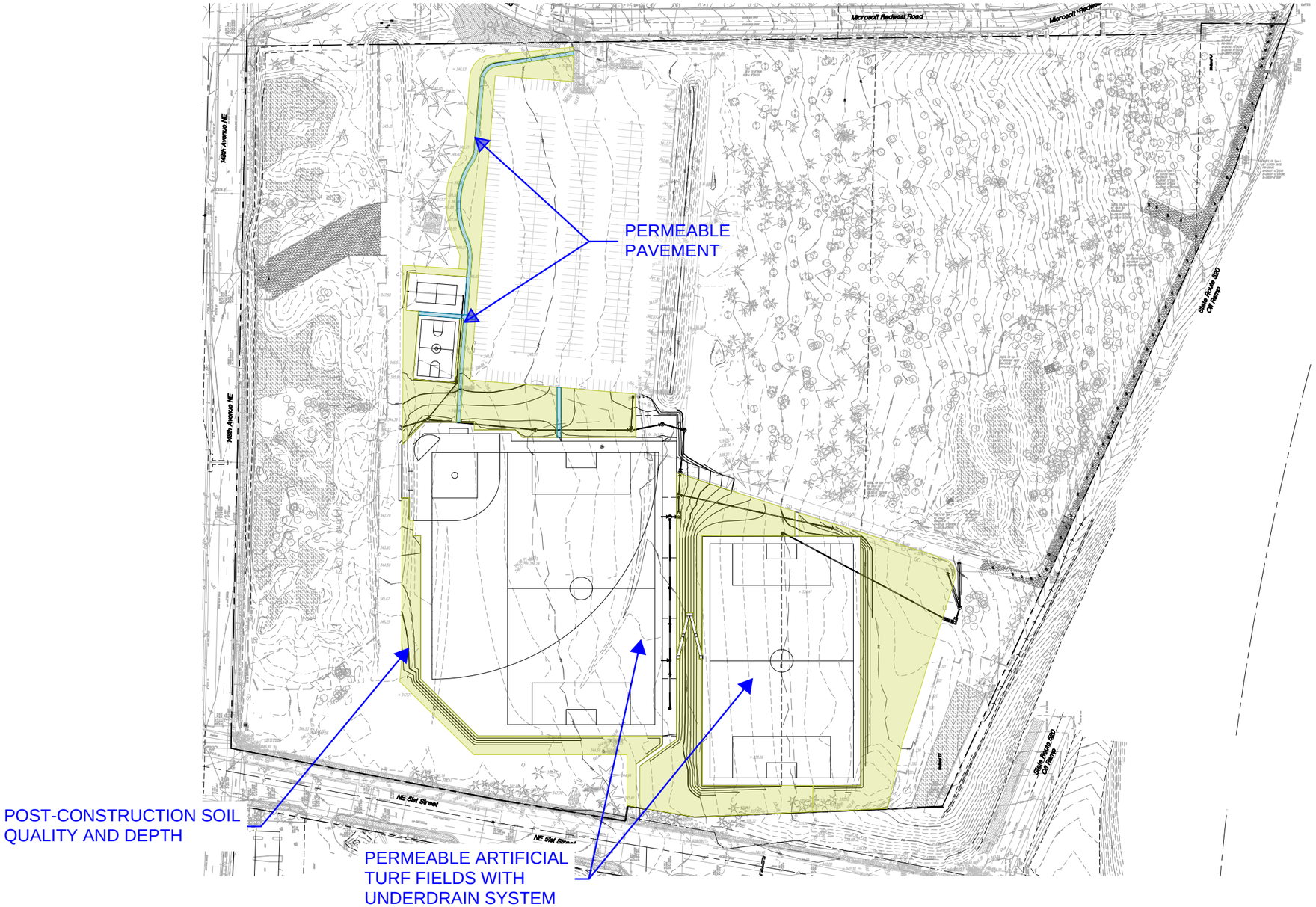

LEGEND

Description	Quantity
Permeable Pavement	0.09 Acres
Post-Construction Soil Quality and Depth	2.59 Acres

Scale: 1"=150'
0' 75' 150'

LID SITE PLAN

Project: MICROSOFT RED WEST SOUTH FIELDS

Project No: C182071-04 Date: 04/23/2019

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APPENDIX B: ENGINEERING CALCULATIONS

Flow Control Calculations

WWHM2012

PROJECT REPORT

DETENTION CALCULATIONS PER
NORTH OVERLAKE ALTERNATIVE
FLOW CONTROL STANDARD

General Model Information

Project Name:	WWHM Full Site with Parking Lot
Site Name:	Red West South
Site Address:	148th Ave & 51st St
City:	Redmond
Report Date:	5/23/2019
Gage:	Seatac
Data Start:	1948/10/01
Data End:	2009/09/30
Timestep:	15 Minute
Precip Scale:	1.00
Version Date:	2016/02/25
Version:	4.2.12

POC Thresholds

~~Low Flow Threshold for POC1:~~	~~50 Percent of the 2 Year~~
~~High Flow Threshold for POC1:~~	~~50 Year~~

PROJECT MUST ADHERE TO NORTH OVERLAKE BASIN ALTERNATIVE FLOW CONTROL STANDARD

POC THRESHOLD: MITIGATED 50 YEAR < 0.28 CFS/ACRE

Landuse Basin Data
Predeveloped Land Use

Historic Conditions

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
C, Forest, Mod	23.35
Pervious Total	23.35
Impervious Land Use	acre
Impervious Total	0
Basin Total	23.35

Element Flows To:

Surface	Interflow	Groundwater

Mitigated Land Use

Site Area

Bypass:	Yes
GroundWater:	No
Pervious Land Use	acre
C, Pasture, Mod	4.46
C, Forest, Mod	8.97
SAT, Forest, Mod	0.44
Pervious Total	13.87
Impervious Land Use	acre
PARKING FLAT	6.04
Impervious Total	6.04
Basin Total	19.91

Element Flows To:

Surface	Interflow	Groundwater

ROW Area

Bypass:	Yes
GroundWater:	No
Pervious Land Use	acre
C, Pasture, Steep	0.41
Pervious Total	0.41
Impervious Land Use	acre
ROADS MOD	0.49
Impervious Total	0.49
Basin Total	0.9

ROW AREA INCLUDED IN FLOW MODEL BUT DOES NOT COUNT AS ADDITIONAL AREA FOR ALLOWABLE PEAK FLOW *PER NORTH OVERLAKE FLOW CONTROL STANDARD

Element Flows To:

Surface	Interflow	Groundwater

Parking Lot Area

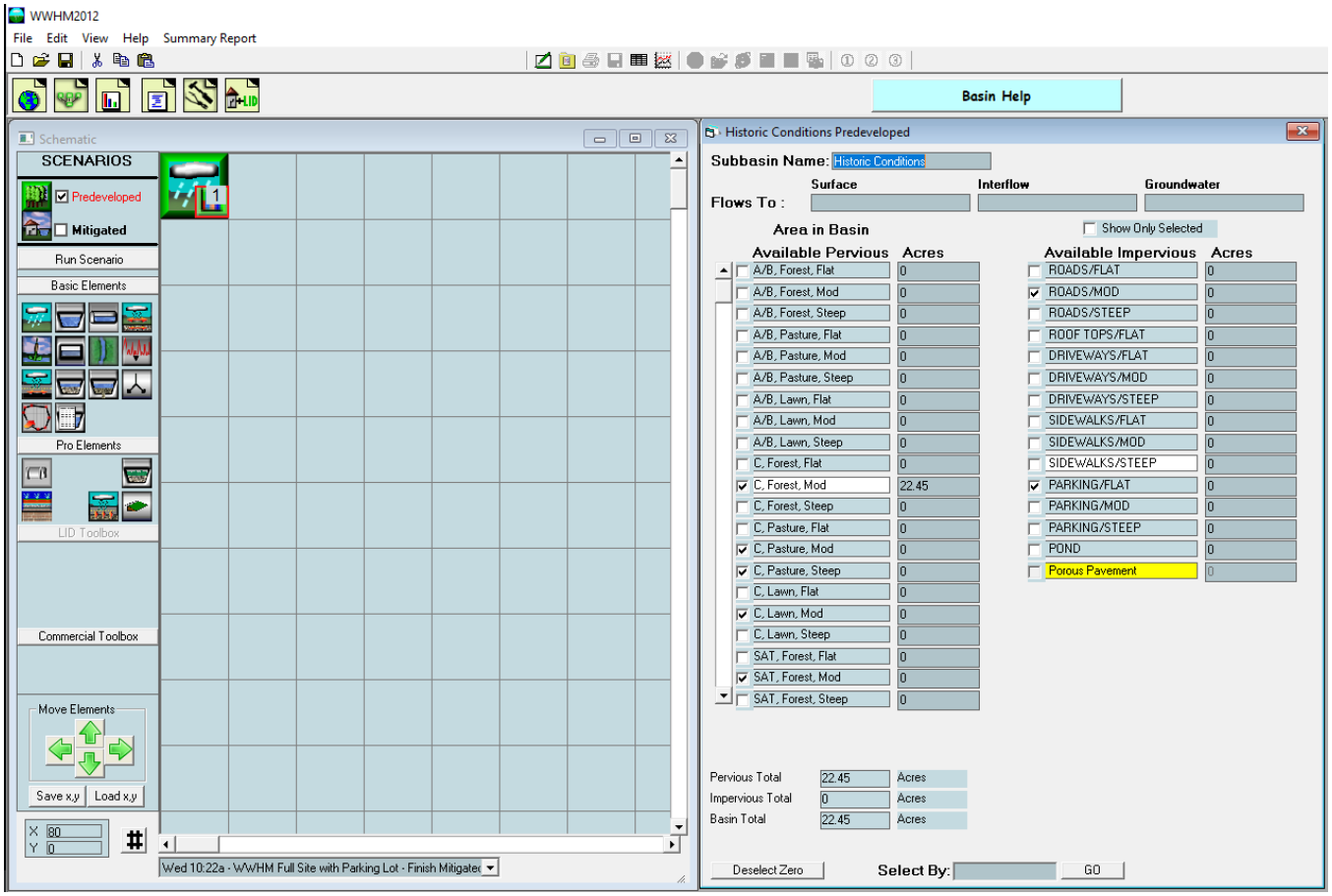
Bypass:	No
GroundWater:	No
Pervious Land Use	acre
Pervious Total	0
Impervious Land Use	acre
PARKING FLAT	2.54
Impervious Total	2.54
Basin Total	2.54

INCLUDES:
2.28 ACRE PARKING LOT
0.26 ACRE POND AREA (MODELED AS IMPERVIOUS)

Element Flows To:

Surface	Interflow	Groundwater
Parking Lot Detention Pond	Parking Lot Detention Pond	

Routing Elements
Predeveloped Routing



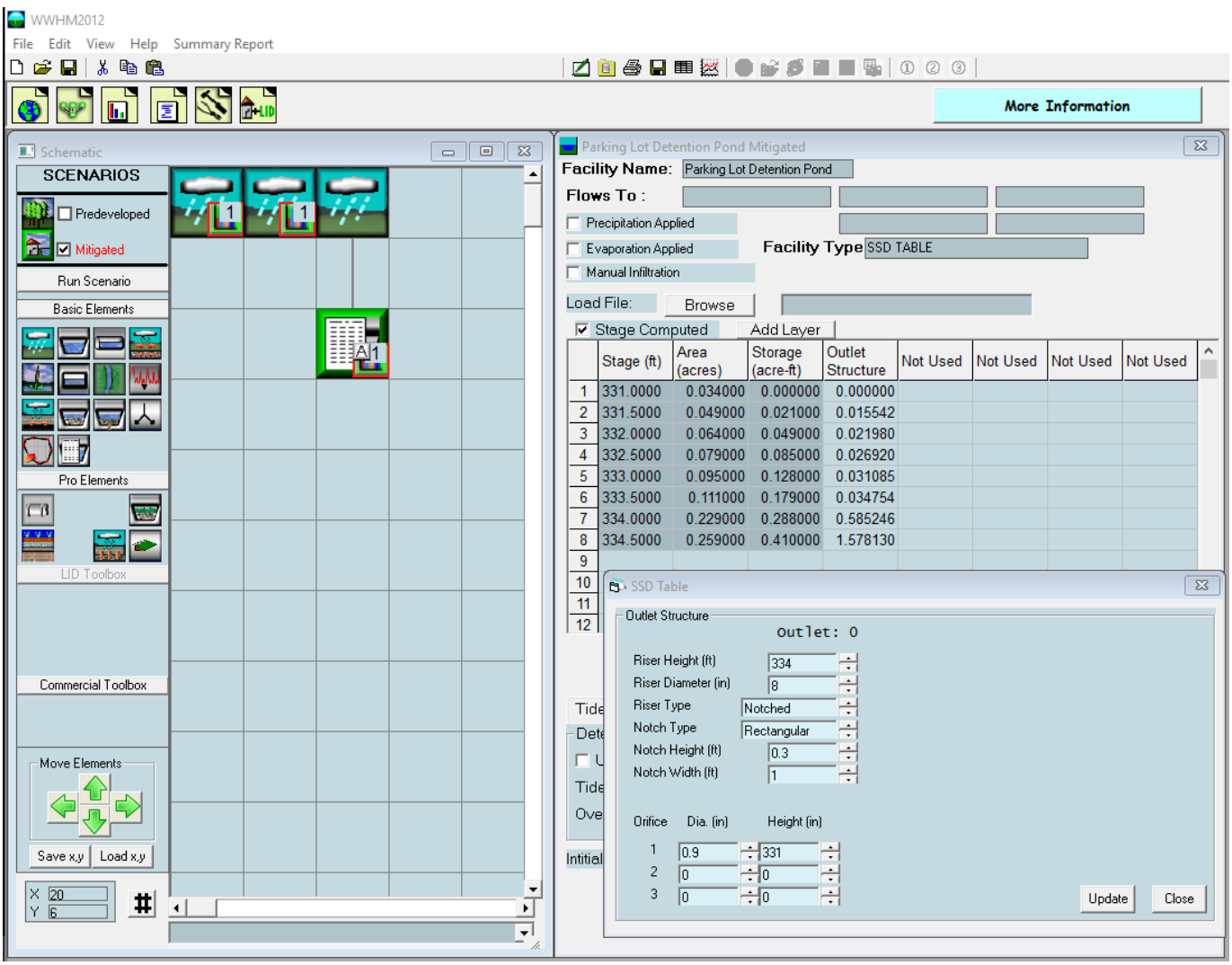
Mitigated Routing

Parking Lot Detention Pond

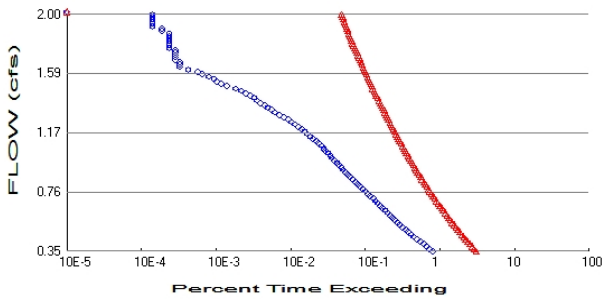
Depth: 334.5 ft.
Element Flows To:
Outlet 1 Outlet 2

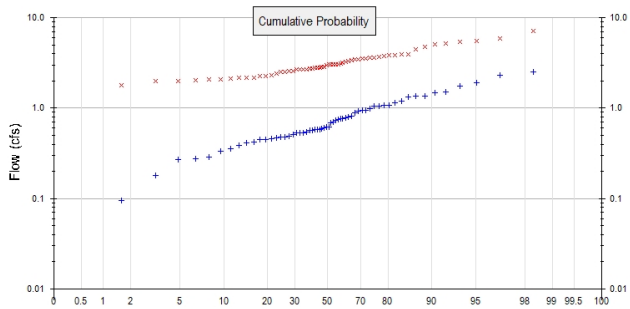
SSD Table Hydraulic Table

Stage (feet)	Area (ac.)	Volume (ac-ft.)	Outlet Struct	NotUsed	NotUsed	NotUsed	NotUsed
331.0	0.034	0.000	0.000	0.000	0.000	0.000	0.000
331.5	0.049	0.021	0.016	0.000	0.000	0.000	0.000
332.0	0.064	0.049	0.022	0.000	0.000	0.000	0.000
332.5	0.079	0.085	0.027	0.000	0.000	0.000	0.000
333.0	0.095	0.128	0.031	0.000	0.000	0.000	0.000
333.5	0.111	0.179	0.035	0.000	0.000	0.000	0.000
334.0	0.229	0.288	0.585	0.000	0.000	0.000	0.000
334.5	0.259	0.410	1.578	0.000	0.000	0.000	0.000



Analysis Results
POC 1





+ Predeveloped x Mitigated

~~Predeveloped Landuse Totals for POC #1~~
~~Total Pervious Area: 23.35~~
~~Total Impervious Area: 0~~

PREDEVELOPED FLOWS NOT APPLICABLE IN NORTH OVERLAKE ALTERNATIVE FLOW CONTROL STANDARD

Mitigated Landuse Totals for POC #1
Total Pervious Area: 14.28
Total Impervious Area: 9.07

Flow Frequency Method: Log Pearson Type III 17B

~~Flow Frequency Return Periods for Predeveloped. POC #1~~

~~**Return Period**~~	~~**Flow(cfs)**~~
~~2 year~~	~~0.695251~~
~~5 year~~	~~1.139231~~
~~10 year~~	~~1.424701~~
~~25 year~~	~~1.76428~~
~~50 year~~	~~1.999495~~
~~100 year~~	~~2.219197~~

PREDEVELOPED FLOWS NOT APPLICABLE IN NORTH OVERLAKE ALTERNATIVE FLOW CONTROL STANDARD

Flow Frequency Return Periods for Mitigated. POC #1

Return Period	**Flow(cfs)**
2 year	2.975363
5 year	3.902245
10 year	4.550476
25 year	5.410656
50 year	6.082274
100 year	6.781042

NORTH OVERLAKE ALTERNATIVE FLOW CONTROL STANDARD

DEVELOPED 50-YEAR PEAK FLOW RATE = 6.08 CFS

ALLOWABLE 50-YEAR PEAK FLOW RATE FOR BASIN = 6.28 CFS
(22.45 ACRES X 0.28 CFS/ACRE)

DEVELOPED 50-YEAR PEAK FLOW RATE < ALLOWABLE MAX 50-YEAR FLOW RATE

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APPENDIX C: 2009 PARKING LOT DRAINAGE MEMO & CALCULATIONS

Appendix C : Existing Parking Lot Drainage Memo (from 2009)

MEMORANDUM

DATE:	June 5, 2009		
TO:	Lisa Rigg	FROM:	Bart Balko
COMPANY:	City of Redmond Public Works Department	PROJECT:	Microsoft Red West South Parking Lot Modifications
ATTACHMENTS:	Drainage Calculations – Lot with asphalt surfacing Drainage Memorandum – Entranco Engineers 4/12/99		

STEPHEN C. PORTER
STATE OF WASHINGTON
26577
REGISTERED
PROFESSIONAL ENGINEER
6/05/09

Project Background

This memo represents the technical information supporting the detention and water quality systems required for the Microsoft Red West temporary parking lot, located south of the Microsoft Red West campus.

The temporary parking lot was originally designed by Entranco Engineers in 1999. Original conditions were designed and constructed with a gravel surface parking lot, however subsequent to initial construction the parking lot has been paved with asphalt concrete pavement. The lot was intended to be a short-term, temporary parking lot; however it has been realized that the lot will now be used longer than originally anticipated. The City of Redmond is requiring that the parking lot storm drainage systems be improved such that the impervious asphalt paving surface is accommodated. It is noted that the owner desires that the parking lot may continue to be considered temporary. Because of this, the City has indicated that the calculations to determine improvements to the drainage systems can be made according to the original design standards with asphalt surfacing, rather than requiring the stormwater facilities be upgraded to current standards.

A combination detention and water quality pond was constructed as part of the original project. The calculations were performed using a stormwater runoff curve number for gravel surfacing. Asphalt surfacing produces a larger amount of stormwater runoff and has a higher runoff curve number used in calculations. Thus, designing a pond for asphalt surfacing produces a larger storage requirement than a pond designed for gravel surfacing.

The existing pond consists of two cells. The initial pond designed by Entranco was modified slightly several years ago to what is now the current condition. Currently, stormwater runoff from the parking lot is piped to the first (south) cell which is designed for water quality dead storage. During rainfall events, stormwater flows over the center dividing berm into the second (north) cell where it is stored for flow control live storage (detention). An outlet flow control structure is located at the north end of the pond.

The proposed pond revisions will mimic the existing behavior of the pond, though the outlet control structure will be revised to provide the required volumes for detention. Information is provided below for the design of both the existing and proposed stormwater systems.

Stormwater Detention

EXISTING

The initial detention pond calculations were designed according to Department of Ecology Stormwater Management Manual requirements (current edition at the time of design). For the project, the 2-year, 10-year, and 100-year storm frequencies were used to size the detention pond. The WaterWorks Hydrographic Modeling program was utilized for detention system sizing. The detention pond and flow control structure were designed such that discharge runoff rates from the developed area wouldn't exceed: ½ the 2-year existing for the 2-year post developed, the 10-year existing for the 10-year post developed and the 100-year existing for the 100-year post developed. The design detention volume for the original gravel parking lot was approximately 7,053 cubic feet. The provided detention volume was approximately 7,555 cubic feet. The detention storage exists entirely within the second cell of the existing pond.

PROPOSED

Equivalent design constraints and modeling software have been used to size a detention system for the parking lot with asphalt surfacing. The required detention volume for the asphalt parking lot is 11,173 cubic feet. This volume is approximately a 4,120 cubic foot increase over the original gravel parking lot design. By maintaining the existing pond length and width, the storage depth increases to obtain the new storage volume.

The existing pond has sufficient depth to the top of the eastern berm to allow for this additional storage depth. The second cell has approximately 7,813 cubic feet available for detention storage. At peak rainfall events, detention ponding depth will exceed the depth of the second cell. When this occurs, detention ponding will overtop the cell dividing berm and pond within both cells. Calculations (provided) show that the maximum ponding depth will be approximately 0.41 feet above the cell dividing berm. See the pond plan and section cut in Figures 1 and 2.

The pond outlet control structure will be modified such that the additional ponding occurs and the required stormwater detention volume is provided. See Detail 3 for the revised control structure configuration. No further detention pond modifications are necessary to meet stormwater requirements.

Water Quality

EXISTING

The initial water quality pond calculations were designed according to Department of Ecology requirements (current edition at the time of design), and was sized to contain the entire volume of runoff for the 6-month, 24-hour storm event. The design volume for the original gravel parking lot was approximately 4,739 cubic feet. The required detention pond volume and other design constraints dictated the pond length and width geometry. Due to the pond's linear shape, long length, and the required water quality depth of 3 feet, the provided water quality dead storage volume was much larger than required. The provided water quality volume within the pond first cell is approximately 7,260 cubic feet.

PROPOSED

The required water quality storage volume for the asphalt parking lot is 6,970 cubic feet, sized using the same code requirements as the original gravel parking lot design. This required volume is less than the volume which is currently provided within the first cell of the combination pond, so no pond improvements are necessary for water quality.

Conveyance

The Red West temporary parking lot detention pond discharges through a large forest area on site, to the east of the parking lot, and discharges to the WSDOT stormwater conveyance system within SR520. The pond discharge pipe does not directly connect to the SR520 system. Rather it discharges within the existing forested land to the east of the pond and sheet flows to the SR520 conveyance system. The SR520 conveyance system consists entirely of below grade conveyance pipes until discharge to a regional water quality pond near the SR520 / West Lake Sammamish Parkway interchange. Stormwater from this regional facility discharges directly to the Sammamish River. Recent drainage studies performed for the City of Redmond by Roth Hill Engineers, the SR520 drainage system is flowing near

capacity, however does not appear to be experiencing any flow problems with the currently contributing volumes. No pond outfall or conveyance system modifications are proposed at this time.

Calculations are provided for detention and water quality for the asphalt surface condition for the temporary parking lot. In addition, original design calculations are also provided. It is our understanding that the proposed upgrades stated in this memorandum are sufficient for this development. It is also understood that the parking lot can continue to function as is in the future with no additional upgrades. Please contact our office if there are any questions, or if additional information required.

Microsoft

COUGHLIN PORTER LUNDEEN

A CONSULTING STRUCTURAL AND CIVIL ENGINEERING CORPORATION

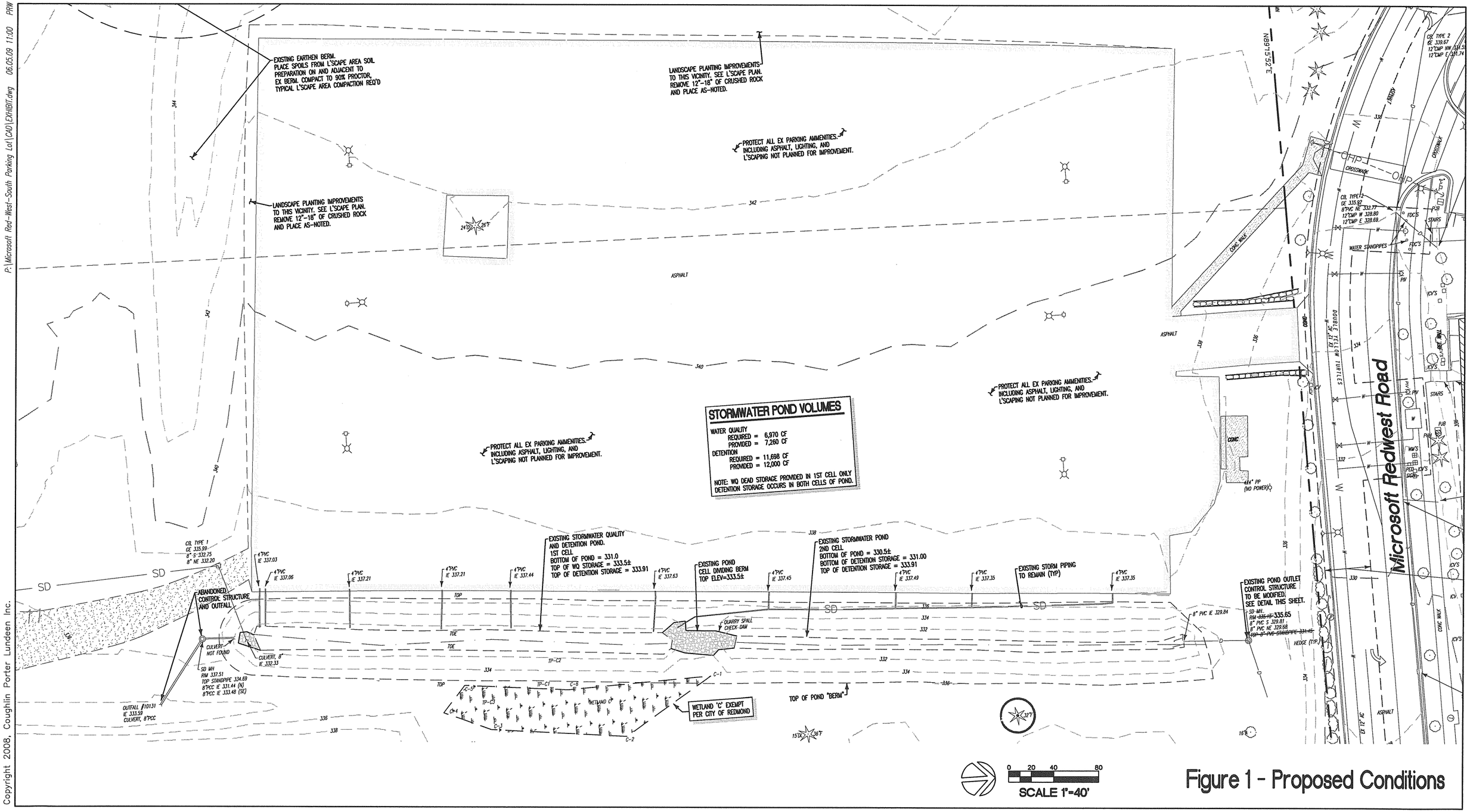

Figure 1 - Proposed Conditions

PROJECT: MS Red West South Parking Lot		DESIGNED BY: BSB	DATE: 06-05-09
PROJECT NO. C090158-02	CLIENT: Microsoft	CHECKED BY: PRW	SHEET 1 OF 1
413 PINE STREET - SUITE 300	SEATTLE, WA 98101	P: 206/343-0460	F: 206/343-5691

P:\Microsoft Red-West-South Parking Lot\CAD\EXHIBIT.dwg 06.05.09 11:00 PRW
Copyright 2008, Coughlin Porter Lundeen Inc.

Microsoft

COUGHLIN PORTER LUNDEEN

A CONSULTING STRUCTURAL AND CIVIL ENGINEERING CORPORATION

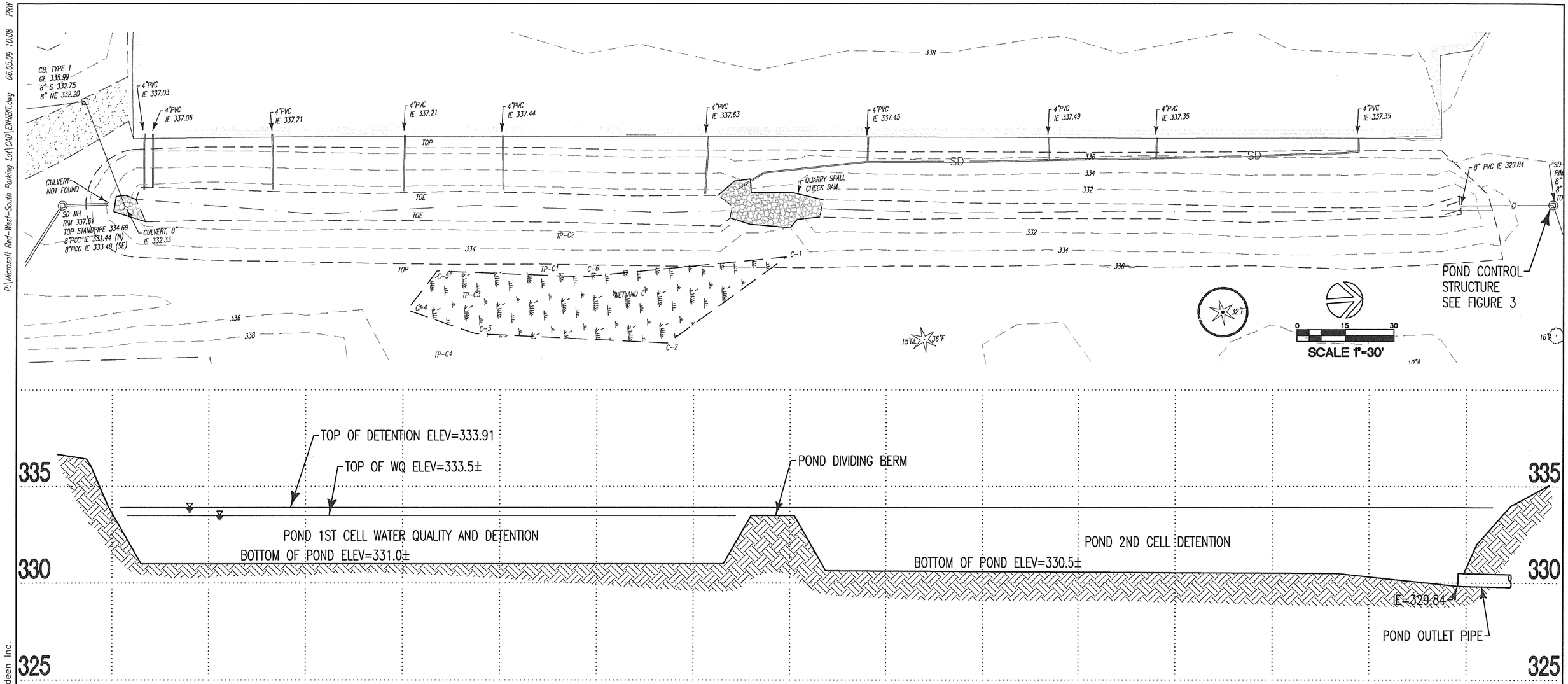

Figure 2 - Plan and Profile

PROJECT: MS Red West South Parking Lot		DESIGNED BY: BSB	DATE: 06-05-09
PROJECT NO. C090158-02	CLIENT: Microsoft	CHECKED BY: PRW	SHEET 1 OF 1
413 PINE STREET - SUITE 300	SEATTLE, WA 98101	P: 206/343-0460	F: 206/343-5691

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Copyright 2008, Coughlin Porter Lundeen Inc.

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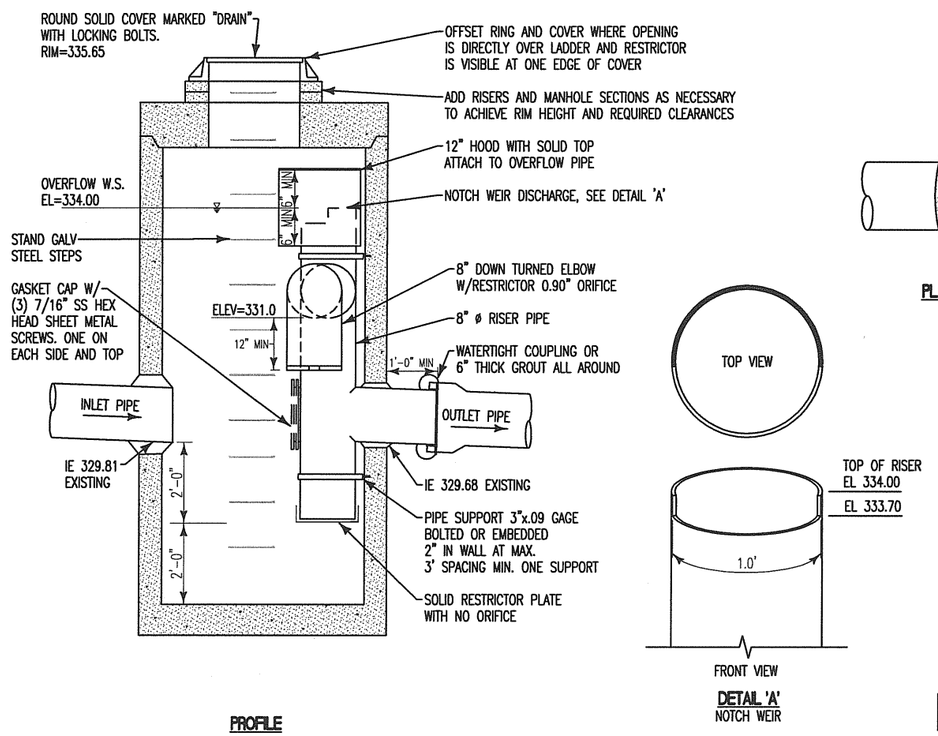

NOTES:

1. CONTROL STRUCTURE SHALL BE IN ACCORDANCE WITH CITY OF REDMOND STANDARDS AND STD DWG #610
2. METAL PARTS; CORROSION RESISTANT, GALVANIZED PIPE TO HAVE ASPHALT TREATMENT 1.
3. FRAME AND LADDER OR STEPS SHALL BE OFFSET SO;
   A. CLEANOUT GATE IS VISIBLE FROM TOP
   B. CLIMB-DOWN SPACE IS CLEAR OF RISER AND CLEANOUT GATE
   C. FRAME IS CLEAR OF CURB
4. IF METAL OUTLET PIPE CONNECTS TO CEMENT CONCRETE PIPE: OUTLET PIPE TO HAVE SMOOTH O.D. EQUAL TO CONCRETE I.D LESS 1/4". USE 6" THICK GROUT ALL AROUND OUTSIDE CONNECTION (TYPE 5 - 3/4")

Pond Control Structure

Figure 3 - Pond Control Structure

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PROJECT: MS Red West South Parking Lot		DESIGNED BY: BSB	DATE: 06-05-09
PROJECT NO. C090158-02	CLIENT: Microsoft	CHECKED BY: PRW	SHEET 1 OF 1

413 PINE STREET - SUITE 300 SEATTLE, WA 98101 P: 206/343-0460 F: 206/343-5691

MS RedWest Temporary Parking Lot
Detention & WQ Calculations
SBUH Method for 2,10,100 Year Storms

==========

BASIN SUMMARY

BASIN ID: D10 NAME: DEVELOPED 10 YEAR
SBUH METHODOLOGY
TOTAL AREA.....: 1.96 Acres BASEFLOWS: 0.00 cfs
RAINFALL TYPE....: TYPE1A PERV IMP
PRECIPITATION....: 2.82 inches AREA..: 0.00 Acres 1.96 Acres
TIME INTERVAL....: 10.00 min CN....: 0.00 98.00
TC....: 0.00 min 5.00 min
ABSTRACTION COEFF: 0.20
PEAK RATE: 1.14 cfs VOL: 0.42 Ac-ft TIME: 480 min

BASIN ID: D100 NAME: DEVELOPED 100 YEAR
SBUH METHODOLOGY
TOTAL AREA.....: 1.96 Acres BASEFLOWS: 0.00 cfs
RAINFALL TYPE....: TYPE1A PERV IMP
PRECIPITATION....: 3.80 inches AREA..: 0.00 Acres 1.96 Acres
TIME INTERVAL....: 10.00 min CN....: 0.00 98.00
TC....: 0.00 min 5.00 min
ABSTRACTION COEFF: 0.20
PEAK RATE: 1.55 cfs VOL: 0.58 Ac-ft TIME: 480 min

BASIN ID: D2 NAME: DEVELOPED 2 YEAR
SBUH METHODOLOGY
TOTAL AREA.....: 1.96 Acres BASEFLOWS: 0.00 cfs
RAINFALL TYPE....: TYPE1A PERV IMP
PRECIPITATION....: 1.85 inches AREA..: 0.00 Acres 1.96 Acres
TIME INTERVAL....: 10.00 min CN....: 0.00 98.00
TC....: 0.00 min 5.00 min
ABSTRACTION COEFF: 0.20
PEAK RATE: 0.73 cfs VOL: 0.27 Ac-ft TIME: 480 min

BASIN ID: D6MO NAME: DEVELOPED 6 MONTH ← WATER QUALITY STORM EVENT
SBUH METHODOLOGY
TOTAL AREA.....: 1.96 Acres BASEFLOWS: 0.00 cfs
RAINFALL TYPE....: TYPE1A PERV IMP
PRECIPITATION....: 1.18 inches AREA..: 0.00 Acres 1.96 Acres
TIME INTERVAL....: 10.00 min CN....: 0.00 98.00
TC....: 0.00 min 5.00 min
ABSTRACTION COEFF: 0.20
PEAK RATE: 0.44 cfs VOL: 0.16 Ac-ft TIME: 480 min

REQUIRED WQ VOLUME

MS RedWest Temporary Parking Lot
Detention & WQ Calculations
SBUH Method for 2,10,100 Year Storms

==========

BASIN SUMMARY

BASIN ID: E10 NAME: EXISTING 10 YEAR
SBUH METHODOLOGY
TOTAL AREA.....: 1.96 Acres BASEFLOWS: 0.00 cfs
RAINFALL TYPE....: TYPE1A PERV IMP
PRECIPITATION....: 2.82 inches AREA..: 1.96 Acres 0.00 Acres
TIME INTERVAL....: 10.00 min CN....: 81.00 0.00
TC....: 68.01 min 0.00 min
ABSTRACTION COEFF: 0.20
PEAK RATE: 0.22 cfs VOL: 0.19 Ac-ft TIME: 540 min

BASIN ID: E100 NAME: EXISTING 100 YEAR
SBUH METHODOLOGY
TOTAL AREA.....: 1.96 Acres BASEFLOWS: 0.00 cfs
RAINFALL TYPE....: TYPE1A PERV IMP
PRECIPITATION....: 3.80 inches AREA..: 1.96 Acres 0.00 Acres
TIME INTERVAL....: 10.00 min CN....: 81.00 0.00
TC....: 68.01 min 0.00 min
ABSTRACTION COEFF: 0.20
PEAK RATE: 0.41 cfs VOL: 0.32 Ac-ft TIME: 490 min

BASIN ID: E2 NAME: EXISTING 2 YEAR
SBUH METHODOLOGY
TOTAL AREA.....: 1.96 Acres BASEFLOWS: 0.00 cfs
RAINFALL TYPE....: TYPE1A PERV IMP
PRECIPITATION....: 1.85 inches AREA..: 1.96 Acres 0.00 Acres
TIME INTERVAL....: 10.00 min CN....: 81.00 0.00
TC....: 68.01 min 0.00 min
ABSTRACTION COEFF: 0.20
PEAK RATE: 0.08 cfs VOL: 0.08 Ac-ft TIME: 550 min

BASIN ID: HALFE2 NAME: HALF EXISTING 2 YEAR
SBUH METHODOLOGY
TOTAL AREA.....: 0.98 Acres BASEFLOWS: 0.00 cfs
RAINFALL TYPE....: TYPE1A PERV IMP
PRECIPITATION....: 1.85 inches AREA..: 0.98 Acres 0.00 Acres
TIME INTERVAL....: 10.00 min CN....: 81.00 0.00
TC....: 68.01 min 0.00 min
ABSTRACTION COEFF: 0.20
PEAK RATE: 0.04 cfs VOL: 0.04 Ac-ft TIME: 550 min

MS RedWest Temporary Parking Lot
Detention Calculations
SBUH Method for 2,10,100 Year Storms

LEVEL POOL TABLE SUMMARY

<--------DESCRIPTION--------->	MATCH (cfs)	INFLOW (cfs)	-STO- --id-	-DIS- --id-	<-PEAK-> <-STAGE>	id	STORAGE VOL (cf)	DISCHARGE (cfs)
2 YEAR	0.04	0.73	SSPond	COMBO2	333.65	1	9273.73 cf	0.04
10 YEAR	0.22	1.14	SSPond	COMBO2	333.82	2	10864.49 cf	0.19
100 YEAR	0.41	1.55	SSPond	COMBO2	333.91	3	11698.11 cf	0.37

REQUIRED STORAGE VOLUME

100-YEAR PEAK FLOW RATE LEAVING DETENTION POND=0.37 CFS
-USED FOR 50-YEAR PEAK FLOW RATE FOR EXISTING PARKING LOT AND PARKING LOT DETENTION POND

THE 50-YEAR PEAK FLOW RATE WILL BE LESS THAN 0.37 SO THIS IS A CONSERVATIVE NUMBER

RWS Parking Lot Pond

Stage	Pond Area	Storage	Total Storage	
331.0	1490	0	0	STORAGE WITHIN SECOND CELL OF POND
331.5	2122	903	903	
332.0	2773	1224	2127	
332.5	3442	1554	3681	
333.0	4128	1893	5573	
333.5	4831	2240	7813	
333.5	8950	0	7813	STORAGE WITHIN SECOND CELL PLUS STORAGE ABOVE TOP OF CELL DIVIDING BERM
334.0	9954	4726	12539	
334.5	11298	5313	17852	
335.0	12656	5989	23840	
335.5	14029	6671	30512	

MS RedWest Temporary Parking Lot
Detention Calculations
SBUH Method for 2,10,100 Year Storms

STAGE STORAGE TABLE

CUSTOM STORAGE ID No. SSPond
Description: Stage Storage Pond

STAGE (ft)	<----STORAGE----> ---cf---	--Ac-Ft-	STAGE (ft)	<----STORAGE----> ---cf---	--Ac-Ft-	STAGE (ft)	<----STORAGE----> ---cf---	--Ac-Ft-	STAGE (ft)	<----STORAGE----> ---cf---	--Ac-Ft-
331.00	0.0000	0.0000	332.20	2749	0.0631	333.40	7365	0.1691	334.60	19050	0.4373
331.10	180.60	0.0041	332.30	3059	0.0702	333.50	7813	0.1794	334.70	20247	0.4648
331.20	361.20	0.0083	332.40	3370	0.0774	333.60	8758	0.2011	334.80	21445	0.4923
331.30	541.80	0.0124	332.50	3681	0.0845	333.70	9703	0.2228	334.90	22642	0.5198
331.40	722.40	0.0166	332.60	4059	0.0932	333.80	10649	0.2445	335.00	23840	0.5473
331.50	903.00	0.0207	332.70	4438	0.1019	333.90	11594	0.2662	335.10	25174	0.5779
331.60	1148	0.0263	332.80	4816	0.1106	334.00	12539	0.2879	335.20	26509	0.6086
331.70	1393	0.0320	332.90	5195	0.1193	334.10	13602	0.3122	335.30	27843	0.6392
331.80	1637	0.0376	333.00	5573	0.1279	334.20	14664	0.3366	335.40	29178	0.6698
331.90	1882	0.0432	333.10	6021	0.1382	334.30	15727	0.3610	335.50	30512	0.7005
332.00	2127	0.0488	333.20	6469	0.1485	334.40	16789	0.3854			
332.10	2438	0.0560	333.30	6917	0.1588	334.50	17852	0.4098			

MS RedWest Temporary Parking Lot
Detention Calculations
SBUH Method for 2,10,100 Year Storms

==========

STAGE DISCHARGE TABLE

COMBINATION DISCHARGE ID No. COMBO2
Description: TEMP PARKING CONTROL STANDPIPE
Structure: ORIFICE2 Structure:
Structure: NOTCH2 Structure:
Structure: RISER2

STAGE (ft)	<--DISCHARGE---> ---cfs-- -------	STAGE (ft)	<--DISCHARGE---> ---cfs-- -------	STAGE (ft)	<--DISCHARGE---> ---cfs-- -------	STAGE (ft)	<--DISCHARGE---> ---cfs-- -------
331.00	0.0000	331.90	0.0209	332.80	0.0295	333.70	0.0361
331.10	0.0070	332.00	0.0220	332.90	0.0303	333.80	0.1423
331.20	0.0098	332.10	0.0231	333.00	0.0311	333.90	0.3411
331.30	0.0120	332.20	0.0241	333.10	0.0319	334.00	0.6049
331.40	0.0139	332.30	0.0251	333.20	0.0326	334.10	1.4554
331.50	0.0155	332.40	0.0260	333.30	0.0333	334.20	2.0437
331.60	0.0170	332.50	0.0269	333.40	0.0341	334.30	2.6252
331.70	0.0184	332.60	0.0278	333.50	0.0348	334.40	3.2207
331.80	0.0197	332.70	0.0287	333.60	0.0354	334.50	3.8363

MS RedWest Temporary Parking Lot
Detention Calculations
SBUH Method for 2,10,100 Year Storms

STAGE DISCHARGE TABLE

NOTCH WEIR ID No. NOTCH2
Description: notch weir 2
Weir Length: 1.0000 ft. Weir height (p): 0.3000 ft.
Elevation : 333.70 ft. Weir Increm: 0.10

STAGE (ft)	<--DISCHARGE---> ---cfs-- -------	STAGE (ft)	<--DISCHARGE---> ---cfs-- -------	STAGE (ft)	<--DISCHARGE---> ---cfs-- -------	STAGE (ft)	<--DISCHARGE---> ---cfs-- -------
333.70	0.0000	334.00	0.5669	334.30	1.6646	334.50	2.6066
333.80	0.1055	334.10	0.8852	334.40	2.1171		
333.90	0.3037	334.20	1.2526	334.50	2.6066		

STAGE DISCHARGE TABLE

MULTIPLE ORIFICE ID No. ORIFICE2
Description: TEMP POND CONTROL ORIFICE 2
Outlet Elev: 331.00
Elev: 329.00 ft Orifice Diameter: 0.9000 in.

STAGE (ft)	<--DISCHARGE---> ---cfs-- -------	STAGE (ft)	<--DISCHARGE---> ---cfs-- -------	STAGE (ft)	<--DISCHARGE---> ---cfs-- -------	STAGE (ft)	<--DISCHARGE---> ---cfs-- -------
331.00	0.0000	331.90	0.0209	332.80	0.0295	333.70	0.0361
331.10	0.0070	332.00	0.0220	332.90	0.0303	333.80	0.0368
331.20	0.0098	332.10	0.0231	333.00	0.0311	333.90	0.0374
331.30	0.0120	332.20	0.0241	333.10	0.0319	334.00	0.0381
331.40	0.0139	332.30	0.0251	333.20	0.0326	334.10	0.0387
331.50	0.0155	332.40	0.0260	333.30	0.0333	334.20	0.0393
331.60	0.0170	332.50	0.0269	333.40	0.0341	334.30	0.0399
331.70	0.0184	332.60	0.0278	333.50	0.0348	334.40	0.0405
331.80	0.0197	332.70	0.0287	333.60	0.0354	334.50	0.0411

STAGE DISCHARGE TABLE

RISER DISCHARGE ID No. RISER2
Description: SS POND CONTROL RISER

Riser Diameter (in):	8.00	elev:	334.00 ft
Weir Coefficient...:	9.739	height:	334.50 ft
Orif Coefficient...:	3.782	increm:	0.10 ft

STAGE (ft)	<--DISCHARGE---> ---cfs-- -------	STAGE (ft)	<--DISCHARGE---> ---cfs-- -------	STAGE (ft)	<--DISCHARGE---> ---cfs-- -------	STAGE (ft)	<--DISCHARGE---> ---cfs-- -------
334.00	0.0000	334.10	0.5315	334.30	0.9207	334.50	1.1886
334.00	0.0000	334.20	0.7517	334.40	1.0631		

APPENDIX D: GEOTECHNICAL REPORT

November 13, 2008

Microsoft Corporation
c/o CB Richard Ellis
Global Corporate Services
One Microsoft Way
Redmond, Washington 98052

Attention: Steve Williamson

We are pleased to submit four copies of our "Geotechnical Engineering Services Report, Proposed Microsoft RedWest South Project, Redmond, Washington." Our services were completed in general accordance with our proposal dated May 14, 2008.

Preliminary conclusions and recommendations were provided to you and other members of the design team during a meeting on June 12, 2008. A preliminary draft report was issued on June 24, 2008. The conclusions and recommendations presented in this report are consistent with those given previously, and have been updated to reflect the project changes which have occurred since our initial draft report was issued.

We appreciate the opportunity to provide these services to you on this project. Please contact us if you have any questions regarding this report or if we can provide further assistance.

DRAFT

Sincerely,

GeoEngineers, Inc.

Bo McFadden, PE, LEG, LG
Principal

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cc: NBBJ
223 Yale Avenue North
Seattle, WA 98109
Attn: Ms Amy Williams

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

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GEOTECHNICAL ENGINEERING SERVICES REPORT
MICROSOFT REDWEST SOUTH PROJECT
REDMOND, WASHINGTON

NOVEMBER 13, 2008

FOR
MICROSOFT CORPORATION

DRAFT

File No. 0694-019-00

Geotechnical Engineering Services Report
Microsoft RedWest South Project
Redmond, Washington
File No. 0694-019-00

November 13, 2008

Prepared for:

Microsoft Corporation
c/o CB Richard Ellis
Global Corporate Services
One Microsoft Way

Redmond, Washington 98052

Attention: Steve Williamson

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GEOTECHNICAL ENGINEERING SERVICES REPORT
MICROSOFT REDWEST SOUTH PROJECT
REDMOND, WASHINGTON
FOR
MICROSOFT CORPORATION

INTRODUCTION

This report presents the results of our geotechnical engineering services for the proposed Microsoft RedWest South project (P.11083) in Redmond, Washington. The project site is located northeast of the intersection of 148th Avenue NE and NE 51st Street, and immediately west of State Route (SR) 520. The site is shown relative to surrounding physical features on the Vicinity Map, Figure 1 and the Site Plan, Figure 2. We have completed our services in accordance with the scope of services presented in our proposal dated May 14, 2008.

PROJECT UNDERSTANDING

The proposed site encompasses approximately 23 acres, with most of the site currently undeveloped with the exception of a parking lot in the northern portion of the site and other pathways and drainage features constructed across the central portion of the site. We understand that Phase 1 of the proposed project will include construction of:

- A four-story, L-shaped, below-grade parking garage with one four-story office building (Building 245) situated on top of the northern portion of the garage,
- A four-story office building (Building 244) situated immediately south of the parking garage, and
- A two-story building (Building 246) situated east of the garage in the central portion of the site.

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The below-grade parking garage will extend to about Elevation 300 feet, and we understand that the excavation will be accomplished using a temporary soil nail shoring system. Buildings 244 and 246 will be situated at grade, with the first floor level at about Elevation 338 to 340 feet. The northernmost foundations for Building 244 will be situated very close to the edge of the parking garage. We understand that all of the buildings will be constructed almost simultaneously. The proposed building layouts are shown on the Site Plan, Figure 2.

Main access to the site will be from a new at-grade roadway extending across the center of the site (east of the parking garage and Building 246) from NE 51st Street to the existing Microsoft RedWest campus to the north. In addition, access will be provided to the western side of the parking garage via a new driveway extending from 148th Avenue NE east to the parking garage. Retaining walls will be constructed along both sides of this access driveway as it descends to the first parking level of the garage.

Stormwater facilities will include a treatment/detention vault in the lower northeast portion of the site, as shown in Figure 2. The vault will be about 150 feet wide by 350 feet long and include 4 feet of depth for dead storage/treatment plus 6 feet of depth for live storage. The excavation for the detention vault will be 15 to 20 feet deep along the upslope side. An existing wetland area along the western portion of the site will be retained and enhanced.

FIELD EXPLORATIONS AND LABORATORY TESTING

FIELD EXPLORATIONS

Subsurface conditions at the site were evaluated by drilling 11 borings (GEI-1 through GEI-11), excavating 14 test pits (TP-1 through TP-14), and reviewing borings and test pits completed previously by Terra Associates across the site. The borings were drilled to depths of about 39 to 60 feet between May 27 and May 30, 2008. The test pits were excavated to depths of 4 to 9 feet on September 11, 2008, in the vicinity of Building 244. The locations of the borings and test pits completed for this project and the previous borings that we reviewed are shown on the Site Plan, Figure 2.

Monitoring wells were installed in four of the borings (GEI-5 through GEI-7, and GEI-11) to obtain more accurate information regarding groundwater conditions, including static water levels and hydraulic testing to calculate in-situ permeability values. Slug tests were completed in GEI-6, GEI-7 and GEI-11. One well (GEI-5) was installed near a previous monitoring well completed by Terra Associates, so that a short aquifer pumping test could be conducted to generate interference drawdown in the previous monitoring well. Analysis of this test provided additional large-scale verification of in-situ permeability values. Details of the subsurface exploration program, including logs of the borings and analysis of the hydraulic testing, are presented in Appendix A.

LABORATORY TESTING

Soil samples were obtained during drilling and taken to our laboratory for further evaluation. Selected samples were tested for moisture content, fines content, grain size distribution (sieve analysis) and maximum dry density (MDD) determination. A description of the laboratory testing and the test results are presented in Appendix B. Portions of the test results are also included on the boring logs in Appendix A.

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SITE CONDITIONS

GEOLOGY

We reviewed the "Geologic Map of the Kirkland Quadrangle, Washington" by the United States Geological Survey, (Minard,1983). The project site is located in an area where the surface soils are mapped as glacial till deposited during the Vashon stade of the Fraser glaciation (about 13,000 years ago). The glacial till generally consists of a non-sorted, non-stratified mixture of clay, silt, sand and gravel with occasional cobbles and boulders, but can also contain minor amounts of interbedded stratified sand and gravel. The upper portion of the glacial till is typically weathered to medium dense at shallow depths, with very dense, typically unweathered till below.

The Vashon glacial deposits are in turn underlain by pre-Fraser deposits (Booth et al. 2006) that include fine-grained lacustrine silts and clays.

We also reviewed logs of water supply wells in the vicinity, using the Washington State Department of Ecology's Well Log Viewer website,[1] to assess the lithology encountered and reported water levels. Wells in the area that penetrated through the glacial till encountered sandy deposits below that are described as Vashon advance outwash (Liesch et al. 1963). The Vashon advance outwash forms a regional aquifer underlying the glacial till, with reported static groundwater levels that are typically within the overlying till, and showing a regional hydraulic gradient toward the east.

[1] Universal Resource Locator (URL) address: http://apps.ecy.wa.gov/welllog/

SURFACE CONDITIONS

The approximately 23-acre site is located at the northeast corner of the intersection of 148th Avenue NE and NE 51st Street in Redmond, Washington. The site is bounded by NE 51st Street to the south, 148th Avenue NE to the west, SR 520 to the east and Microsoft's RedWest Campus to the north.

The site is currently vacant, with an asphalt parking lot situated in the north-central portion of the site. A relatively deep drainage ditch is present along the east side of the parking lot. Shallower drainage ditches are present along the south side of the parking lot, and from the southeastern corner of the parking lot extending to the south about 200 feet. An existing gravel road extends from the southeast corner of the parking lot to NE 51st Street. The gravel road is currently used as a walking path between different Microsoft campuses. A small detention pond is situated east of the gravel road/pathway about 200 feet north of NE 51st Street. An approximately 6-foot-high berm, which appears to be man-made, is present west of the gravel road about 150 feet south of the parking lot. A rubble/rock pile was observed southwest of the berm.

The topography of the western portion of the site is relatively level. This portion of the site is vegetated mainly with grass and scattered clusters of deciduous and a few conifer trees. The eastern portion of the site slopes downward to the east toward SR 520. Across the southeast portion of the site, the slope is fairly gentle and vegetated mainly with grass and scattered clusters of deciduous trees. From the gravel road to SR 520, the ground surface loses about 15 to 20 feet in elevation. Across the northeast portion of the site, the slope is gentle for a distance of about 300 to 400 feet from the eastern edge of the parking lot, gradually changing to a moderate slope within 300 to 500 feet of the eastern property line. Along the northern property line, the ground surface loses about 50 to 60 feet in elevation. The northeastern section of the site is forested with a mixture of deciduous and conifer trees, with undergrowth consisting of native bushes and groundcovers, blackberry bushes, and nettles. A landscaped strip with ornamental bushes and trees is located along the northern property line, and a sidewalk connecting the parking lot to the adjacent Microsoft RedWest campus to the north.

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Some of the grassy areas across the site have been designated as wetlands. A few scattered areas of standing water, mainly across the northwest quadrant of the site, were observed at the time the drilling was completed. We also observed some standing water in the small detention pond located east of the gravel drive.

SUBSURFACE CONDITIONS

In general, soil conditions encountered in the test pits and borings are consistent with those shown on the geologic maps. Based on the subsurface soil conditions encountered in our explorations, it appears that the site is underlain by three general soil units: glacial till, advance outwash deposits and older glacially consolidated lacustrine deposits. These soil units are discussed in more detail below. Figures 3 through 6 present generalized subsurface profiles along each side of the proposed below-grade parking garage and transverse to the long direction of the parking garage, illustrating the variability of subsurface conditions. The locations of the profiles are shown in Figure 2.

Limited amounts of fill were encountered in some of the test pits and borings, and previous test pits completed by Terra Associates. Boring GEI-4 encountered 2 inches of asphalt pavement and 4 inches of crushed rock base course underlain by about 5 to 6 feet of silty sand that appears to be fill derived from native glacial till. Some of the test pits also encountered 2 to 3 feet of fill that appears to be derived from the native weathered till deposits. Test pit TP-5, located near the northeast corner of proposed Building 244 and on the side of the berm that appears to be fill, encountered about 4 feet of fill with some debris.

Two test pits completed by Terra Associates south of the existing parking lot, in the vicinity of the berm, encountered 5 feet to possibly more than 8 feet of fill. The fill generally consisted of silty sand with quarry spalls and small amounts of boulders, organic matter and asphalt. With the exception of the berm and the nearby area where Terra Associates encountered fill, and possibly along the east side of the existing parking lot, there are no features that suggest that the depth of possible fill across most of the site would be thick (more than 2 to 4 feet). The other borings completed by GeoEngineers did not encounter identifiable quantities of fill.

With the exception of the northeast quadrant, most of the borings encountered a surficial layer of grass and rootmass. The rootmass was typically 2 to 5 inches in thickness. Borings GEI-7, GEI-8 and GEI-11, which were completed in the forested northeast quadrant of the site, encountered a forest duff and topsoil layer to depths of about 1½ to 2½ feet. This upper layer consists of loose silty sand with organic matter and some gravel, and occasional cobbles and boulders.

The following sections describe the general characteristics of the three major native soil units underlying the upper sod, topsoil or fill, as well as the observed groundwater conditions.

Glacial Till

Glacial till was generally encountered beneath the upper surficial grass and rootmass, topsoil, or fill in all of the borings and test pits. Most of the explorations encountered weathered glacial till consisting of medium dense silty sand with gravel, cobbles and boulders immediately underlying the grass and rootmass layer. The weathered zone extended to a depth of about 6 to 8 feet across most of the site, becoming denser with depth. The glacial till becomes very dense below the weathered zone. Portions of the glacial till consist of stiff to hard sandy silt, also containing gravel, cobbles and boulders.

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Boulders were encountered while drilling in the glacial till deposits across the site, and appear to be more numerous near the surface and also immediately above the underlying advance outwash deposits. Boring GEI-4 met refusal at a depth of 39 feet because of the presence of a boulder. A few of the other borings had to use a solid cone point to push past boulders. Boulders should be expected in any excavations completed at the site.

The test pits were completed with the purpose of identifying the depth of dense unweathered glacial till across the limits of Building 244. A simplified graphical representation of the elevation of the surface of the dense glacial till across the Building 244 area and along a portion of the west side of the parking garage is presented in Figure 7.

Advance Outwash

Dense to very dense advance outwash soils were encountered in most of our borings below the glacial till at depths ranging from approximately 31 to 45 feet. The advance outwash soils encountered in our borings generally consist of dense to very dense fine- to medium-grained sand with silt and occasional gravel. Interlayered zones of silty sand and clean fine- to coarse-grained sand with varying amounts of gravel were also encountered within the advance outwash soils. Borings GEI-2 and GEI-3, located along the western side of the proposed garage, did not encounter a defined advance outwash layer, although some thin lenses of cleaner sand were noted below depths of about 35 to 40 feet. Boring GEI-10, situated in the eastern portion of the site, also did not encounter a definable layer of advance outwash. The advance outwash appears to become thinner toward the east and my pinch out where the till comes into contact with the underlying lacustrine deposits. Boring GEI-11, located in the vicinity of the proposed detention tank, encountered a sand layer or pocket that appears to be separate and isolated from the main

layer of advance outwash sand in the western and central portions of the site, based on soil conditions encountered in the other borings.

Older Lacustrine Deposits

Borings GEI-6 and GEI-7 encountered very dense silty sand and hard sandy silt underlying the outwash deposits at a depth of about 48 feet. Borings GEI-9 and GEI-10 encountered deposits of hard silt and organic silt underlying the glacial till at depths of about 35 feet. These deposits are interpreted to be older glacially consolidated lacustrine deposits.

Groundwater Conditions

Groundwater was encountered during drilling in all of the borings with the exception of borings GEI-2 and GEI-10. Groundwater was encountered at depths varying from about 6 to 43 feet during drilling. Monitoring wells were installed in borings GEI-5, GEI-6, GEI-7 and GEI-11. Boring GEI-5 had 3-inch-diameter screen and casing installed, with the screen placed from a depth of about 40 to 60 feet, to place the screen near the same depth as Terra Associate's boring B-1 located nearby. The remaining wells had 2-inch-diameter screen and casing installed, with the bottom 10 feet of the well screened within the advance outwash deposits. Water levels were measured in the monitoring wells at different times after installation. A summary of the measured groundwater levels is presented in Table 1.

Table 1. Groundwater Measurements

Well Number	Ground Surface Elevation (feet)*	Depth to Top of Well Casing (feet)	Date	Groundwater Depth (feet)	Groundwater Elevation (feet)
GEI-5	336	0.05	06/03/2008	10.45	325.50
			06/04/2008	10.39	325.56
GEI-6	341	0.38	06/03/2008	13.32	327.30
			06/04/2008	13.31	327.31
GEI-7	332	0.3	06/05/2008	6.34	325.36
GEI-11	301	0.4	06/05/2008	14.92	285.68
Terra B-1	336.5	0.2	06/04/2008	10.29	326.01
Terra B-3	343	0	06/05/2008	14.51	328.49

Note:
*Ground surface elevations at the wells estimated from the site topographic contours.

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Based on the groundwater level measurements, the groundwater in the advance outwash aquifer is confined by the glacial till and is under artesian pressure, such that the groundwater elevations listed in Table 2 above represent a potentiometric surface (natural water table) for the outwash over most of the site, as illustrated on the cross section in Figure 6. The potentiometric surface represents the elevation at which groundwater contained in the advance outwash sands rises under pressure, in a boring or monitoring well that penetrates through the till and into the advance outwash. It differs from the natural water table within the till, which appears to vary based on water levels encountered while drilling through the till, and exists mainly within disconnected pockets or lenses of sandy or gravelly material within the till.

The exception is in the eastern part of the site, where we believe the advance outwash pinches out and the till mantle extends downslope to much lower elevations. The water level measured at GEI-11 is in a monitoring well completed below the base of the till, in a sand layer or pocket that appears to be separate

and isolated from the main layer of advance outwash sand in the western and central portions of the site. The potentiometric surface at this location is lower by around 40 feet. The lower pressure likely reflects lateral drainage and dissipation of the higher groundwater pressure present in the main advance outwash layer to the west, with groundwater likely seeping through the till and draining toward SR 520 and Marymoor Slough.

For deeper excavations, it is possible that removal of a significant portion of the till cap could risk potential uplift or heave of the excavation subgrade resulting from artesian pressure acting as an uplift force on the overlying partially excavated glacial till remaining beneath the excavation. Pressure relief wells (in the form of dewatering wellpoints) are recommended to address this issue, as described in the "Conclusions and Recommendations" section of this report.

Perched groundwater exists within the upper glacial till deposits where lenses of sand may be present. Perched groundwater was encountered in most of the borings at depths varying from 9 to 20 feet. Therefore, even if the excavations are completed within the upper glacial till deposit, minor amounts of groundwater seepage should therefore be expected, especially if the excavations are completed during the wet season.

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

Based on our explorations and evaluation, we conclude that:

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1. The proposed office buildings and parking garage may be supported on conventional shallow spread footings bearing on the native dense to very dense glacial till or advance outwash sand or on compacted structural fill bearing on these soils.
2. The planned excavation for the below-grade parking garage can be completed using the soil nail shoring method along with temporary cut slopes. Dewatering will be necessary prior to constructing soil nail walls for temporary excavation support in the underlying advance outwash sand deposits. The soil nail wall design will need to account for surcharges from nearby footings along the south side of the garage and possibly along other areas where footing loads will be present prior to completion of the permanent garage walls.
3. The amount of groundwater encountered in the garage excavation will depend on the final depth of the garage and the time of year the excavation proceeds, but the volumes should be manageable. We estimate that dewatering volumes up to about 30 gallons per minute (gpm) could be generated for the garage excavation. Perched groundwater that will be present within the till that can be controlled by means of sumps and pumps.
4. Drainage should be provided behind exterior below-grade walls, including those for the parking garage, access driveway and detention vault. We recommend that an underslab drainage system be constructed underneath the slab of the lowest garage parking level.
5. The on-site glacial till soils encountered in our borings contain relatively high fines content and may be used as structural fill during dry weather conditions. The on-site advance outwash soils encountered in our borings were relatively clean and should be appropriate to be used as structural fill during dry or wet weather conditions, provided the excavated soils are drained of excess water prior to use.

6. Because the site soils are susceptible to disturbance when wet, we recommend that all surface water entering site construction areas be collected and tightlined or otherwise channeled to a suitable discharge location.

The detailed conclusions and recommendations for the project are presented in the following report sections.

EARTHWORK

Excavation Considerations

Fill, glacial till and advance outwash were observed in the explorations. We anticipate that these soils may be excavated with conventional excavation equipment, such as trackhoes or dozers. It may be necessary to rip the glacial till locally to facilitate excavation. Cobbles and boulders typically exist within the glacial till and advance outwash soils, and the contractor should be prepared to deal with them. Boulders were encountered in the explorations. We recommend that procedures be identified in the project specifications for measurement and payment of work associated with removal of cobbles and boulders.

Clearing and Grubbing

The site is currently occupied by a paved parking lot, existing drainage features and vegetation ranging from grass to trees. We anticipate that the existing improvements will be demolished prior to constructing the proposed development. Any remaining foundations or other below-grade elements from previous site development should be removed, if encountered. All existing utilities should be removed from the planned building footprints and rerouted if needed. All existing utility trenches leading into the areas of planned construction should be backfilled with structural fill.

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Areas to be developed or graded should be cleared of surface and subsurface deleterious matter including organic-rich topsoil, debris, shrubs, trees and associated stumps and roots. Vegetation, including the root mass and organic-rich topsoil, should be stripped and removed from the building and paving areas.

Subgrade Preparation

Prior to placing new fills, pavement base course materials or gravel to support on-grade floor slabs, subgrade areas should be evaluated by proof-rolling or probing to locate any soft or pumping soils. Prior to proof-rolling, all unsuitable soils should be removed from below building areas. Proof-rolling can be completed using a piece of heavy tire-mounted equipment such as a loaded dump truck. During wet weather, the exposed subgrade areas should be probed to determine the extent of soft soils. If soft or pumping soils are observed, they should be removed and replaced with structural fill.

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

After the proof-rolling is complete, 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 the construction is performed. If the work is performed during dry weather conditions, we recommend that all subgrade areas be recompacted to at least 95 percent of the MDD estimated in general accordance with

the American Society for Testing and Materials (ASTM) D 1557 test procedure (modified Proctor). If the work is performed during wet weather conditions, 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 proof-rolling or compaction, it may become necessary to modify the proof-rolling or compaction criteria or methods.

If it is difficult to maintain a dry subgrade in the base of the excavations, we recommend that the contractor be prepared to place a gravel pad of suitable thickness (at least 12 inches in traffic areas) over the exposed subgrade to minimize disturbance of the underlying soils and to provide a stable working surface for construction labor and equipment. All collected water should be routed to suitable discharge points.

Structural Fill

General

All fill placed to support structures, placed behind retaining structures, and placed below pavements and sidewalks will need to be specified as structural fill as described below:

- Structural fill placed around perimeter footing drains and underslab drains should meet the requirements of Washington State Department of Transportation (WSDOT) Standard Specifications Section 9-03.12(4), Gravel Backfill for Drains, or should consist of pea gravel.
- Structural fill placed for cast-in-place wall drains should meet the requirements of WSDOT Standard Specifications Section 9-03.12(2), Gravel Backfill for Walls.
- Structural fill placed as capillary break material below slab-on-grade floors should meet the requirements of WSDOT Standard Specifications Section 9-03.17, Foundation Material Class A.
- Structural fill placed as crushed surfacing base course below pavements and sidewalks should meet the requirements of WSDOT Standard Specifications Section 9-03.9(3), Crushed Surfacing Base Course.

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The suitability of soil for use as structural fill will depend on its gradation, moisture content and the weather conditions during construction. All material used for structural fill should be free of debris, organic contaminants, other deleterious material and rock fragments or cobbles larger than 6 inches.

Imported Structural Fill

We recommend imported structural fill for general use during wet weather. The fill should consist of either crushed or well-graded sand and gravel containing less than 3 percent fines (material passing U.S. Standard No. 200 sieve) by weight relative to the fraction of the material passing the ¾-inch sieve. This imported fill material should be free of rock fragments larger than 4 inches, debris and organic material. We recommend that the suitability of structural fill material from proposed borrow sources be evaluated by the geotechnical engineer before the earthwork contractor is allowed to transport any material to the site.

On-Site Soils

The existing fill, native glacial till and portions of the advance outwash have a relatively high fines content (silt and clay) and are, therefore, moisture-sensitive. These soils are suitable for use as structural fill only if construction takes place during the drier summer months and the soil has a moisture content near optimum. The glacial till and advance outwash in its undisturbed condition possesses a moisture

content near or slightly higher than the optimum for compaction, based on our laboratory test results and experience with compaction of till. Zones within the till where perched groundwater occurs will likely have a moisture content significantly greater than optimum. Therefore, it may be necessary to moisture-condition the till soil by aerating it before placement and compaction as structural fill. Cleaner layers of the advance outwash that contain approximately 5 percent fines will be suitable for use as structural fill during dry and wet weather conditions, provided the material is drained of excess water prior to use. We recommend that all stockpiled soils be covered with plastic sheeting during periods of wet weather.

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 8 to 10 inches in thickness. Each lift should be conditioned to the proper moisture content and compacted to the specified density before placing subsequent lifts. Structural fill should be compacted to the following criteria:

1. Structural fill placed in building areas (supporting foundations or slab-on-grade floors) and within the top 2 feet below the pavement and sidewalk areas (including utility trench backfill) should be compacted to at least 95 percent of the MDD estimated in general accordance with ASTM D 1557. Structural fill placed within pavement and sidewalk areas that are below 2 feet should be compacted to at least 90 percent of the MDD.
2. Structural fill placed against subgrade walls should be compacted to between 90 and 92 percent of the MDD. Care should be taken when compacting fill against subsurface walls to avoid overcompaction and hence overstressing the walls.

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Permanent Slopes

We recommend a maximum permanent slope inclination of 2H:1V (horizontal to vertical) in the native soils or in structural fill placed in accordance with our recommendations. The structural fill should be compacted to at least 90 percent of MDD. Fill should be carefully compacted out to the slope face, or the fill embankment should be overbuilt and cut back. A maximum permanent slope of 2H:1V for cuts in the dense glacial till or advance outwash sand may be used; however, it may be difficult to establish permanent vegetation to prevent erosion. Some sloughing and raveling should be expected at this steeper inclination. Permanent slopes should be hydroseeded or otherwise protected from erosion as soon as possible. Temporary erosion control measures may be necessary until permanent vegetation is established.

Erosion and Sedimentation Control

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's impact on erosion-prone areas and nearby sensitive areas. The plan should be designed in accordance with the City of Redmond standards. The plan should incorporate basic planning principles including:

- All work and materials should be per City of Redmond standards.
- Off-site streets should be kept clean at all times. Flushing streets should not be allowed. All streets should be swept.
- Existing vegetation outside of clearing areas should be retained.
- Denuded areas should be revegetated or mulched when work is stopped or completed.

- Runoff should be directed away from denuded areas.
- The length and steepness of slopes with exposed soils should be reduced.
- Slopes with exposed soils should be covered with plastic sheeting or similar means.
- Drainage ways and outlets should be prepared to handle concentrated or increased runoff.
- Sediment should be confined to the project site.
- The temporary erosion and sedimentation controls should be installed, inspected and operating before any grading or extensive land clearing occurs. These controls should be satisfactorily maintained until construction and landscaping are complete.

During wet weather, we recommend that:

- 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 reduce the extent to which these soils become wet or unstable.
- Construction traffic should be restricted to specific areas of the site, preferably areas that are surfaced with 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 practicable.

In addition to the above measures, it will be necessary to prepare and implement a stormwater turbidity monitoring plan in accordance with the City of Redmond Stormwater Technical Notebook. We are prepared to develop a specific turbidity monitoring plan upon your request and provide monitoring during construction.

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Construction Observations

We recommend that GeoEngineers be present during proof-rolling and/or probing of the exposed subgrade soils in building and pavement areas, and during 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.

EXCAVATION SUPPORT

General

At this time, we anticipate that the below-grade parking garage excavation will be on the order of 45 feet in depth. Excavations for the western access driveway to the parking garage will likely range from 2 to 12 feet in depth. We anticipate that the detention vault excavation will be 15 to 20 feet in depth on the upslope side.

Based on the subsurface conditions encountered in the explorations, it is our opinion that a combination of temporary cut slopes and soil nail walls will be feasible for excavation support. If the lowest level of the garage extends into the underlying advance outwash sand, dewatering will be necessary prior to constructing soil nail walls for temporary excavation support in this deposit.

Soil conditions encountered within the planned excavation depth for the parking garage generally consist of dense to very dense glacially consolidated soils. The upper 5 to 8 feet is typically weathered and in a

medium dense to dense condition. Minor depths (2 to 3 feet) of fill may be encountered along portions of the site. Fill depths on the order of 4 to 6 feet were encountered in boring GEI-4 and test pit TP-5. We do not know if this fill was compacted and may vary in density. The use of vertical support elements in addition to the soil nails may be necessary if a significant depth of fill is encountered along portions of the excavation cut face. At this time, we do not anticipate that the depth of fill encountered will require vertical support elements, with the possible exception of the east side of the excavation (at the east end of Building 245) and in the southeast corner in the vicinity of TP-5. Alternatively, the upper portion of the excavation could be sloped back.

For soil nail wall construction, special construction techniques (including flash coating, staggered excavations and berming) and dewatering may be required in order to control spalling of the advance outwash sand at the site. At this time, we anticipate that the soil nail walls will be used only for temporary support of the excavation. General soil nail techniques are discussed below.

Temporary Cut Slopes

We recommend temporary cut slopes of 1H:1V in the dense to very dense glacial till and advance outwash sand, and 1.25H:1V where fill is encountered. These inclinations may have 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 should be allowed at the top of the cut slopes within a distance of at least 5 feet 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 should be scheduled so that the length of time the temporary cut is left open is minimized.
- Erosion control measures should be constructed as appropriate such that runoff from the site is minimized.
- Surface water should be diverted away from the excavation.
- The general conditions of the slopes should be observed periodically by GeoEngineers to identify problems.

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Because 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. All shoring and temporary slopes must conform to applicable local, state and federal safety regulations.

Soil Nail Walls

General

We anticipate that the excavations will be completed using a combination of open cuts and temporary soil nail walls. The soil nail wall system consists of drilling and grouting rows of steel bars or "nails" behind the excavation face as it is excavated and then covering the face with reinforced shotcrete. The walls are typically constructed from the top down. It may be necessary to utilize vertical elements, post-tensioned nails and/or temporary cut slopes in areas where looser soils are present and/or where wall deflections must be limited to protect existing improvements.

Soil nail walls are typically constructed using the following sequence:

1. Install vertical elements (if needed) into vertical drilled holes and grout the hole with lean concrete.
2. Excavate the soil at the wall face to between 1 and 3 feet below the row of soil nails to be installed. Depending upon the soil conditions at the wall face, the excavation may be completed with a vertical cut or a temporary berm can be utilized. A temporary 1H:IV berm (native cut or fill) can be left on the face of the excavation to reduce face instability.
3. Drill, install and grout soil nails.
4. Excavate berm, if present, located within about 3 feet below the elevation of the soil nail.
5. Place drainage strips, steel wire mesh and/or reinforcing bars in front of excavated soil.
6. Install shotcrete and place steel plates and nuts over soil nails.
7. Complete nail pullout capacity testing on approximately 1 out of every 20 nails in the installed row.
8. Repeat steps 2 through 7 for each row of nails located below the completed row.

Soil nails typically consist of #6 to #12 threaded steel bars (¾- to 1½-inch diameter). The steel bars are placed in 4- to 12-inch-diameter holes drilled at angles typically ranging from 10 to 25 degrees below horizontal. Centralizers are used to center the steel bars in the holes. Once the steel bars are installed, the holes are grouted using cement grout or concrete. Post grout tubes can be installed with the steel bars to increase the bond strength between the grout and the soil. Post grouting consists of injected grout under high pressure through holes placed in the post grout tube one to two days after initial grout placement.

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The soils typically are required to have an adequate standup time (to allow placement of the steel wire mesh and/or reinforcing bars to be installed and the shotcrete to be placed). Soils that have short standup times are problematic for soil nailing and may require the use of vertical elements.

The subsurface soil and groundwater conditions at the site are generally expected to be suitable for soil nailing. However, difficulties may be experienced during construction where the excavation extends into the dense to very dense advance outwash sand encountered at the site. Where the excavation extends into the underlying advance outwash, we recommend dewatering this deposit prior to installing nails. Difficulties may also be encountered where perched groundwater is present within the cleaner layers of the glacial till and where fill is present. Spalling and raveling of the cut face may occur at these locations during installation of the soil nail walls. Construction techniques used to mitigate spalling and raveling include:

- Flash coating the cut face with 2 to 4 inches of shotcrete immediately after the cut face is excavated to final line and drainage material is installed. This typically provides enough standup time to allow for the installation of the reinforcing steel and final shotcrete.
- Excavating in half-height (2- to 3-foot) lifts, rather than full-height (6-foot) lifts.
- Excavating and leaving a 1H:1V earth berm in front of the wall. The soil nails are installed by drilling through the soil berm. The soil berm is then removed to allow for installation of the drainage material and reinforcing steel and shotcrete.
- Shortening the length of wall drilled and shotcreted using a staggered excavation approach.

Preliminary Design Recommendations

We recommend the following for preliminary design purposes:

- The soil nail grid pattern should be about 6 feet by 6 feet.

- The soil nail length should be two-thirds of the wall height, inclined at about 15 to 20 degrees from the horizontal.
- The preliminary allowable adhesion value should be 2,500 pounds per square foot (psf) for a 6- to 12-inch-diameter grouted nail installed in the native glacial till and advance outwash soils.
- Wick drains or strips of drainage material should be installed behind the shotcrete to relieve hydrostatic pressures. Additional drainage provisions may be necessary if significant zones of perched groundwater are encountered during the excavation.
- It may be necessary to install vertical elements within the upper portion of the soil nail wall in order to improve face stability where fill is encountered. Vertical elements typically consist of vertical steel bars or small beams placed in drilled holes located along the wall alignment and backfilled with lean concrete. The spacing of vertical elements typically ranges from 2 to 4 feet on center. The need for vertical elements should be further evaluated during the design of the soil nail wall, if selected.
- Along the southern garage wall, the soil nail wall will need to be designed to withstand surcharge loading from the adjacent Building 244 foundation, if that foundation is supported on spread footings. Figure 9 presents general surcharge lateral pressures from point, line and uniform loads. Alternatively, the northern foundations for Building 244 could be supported on vertical elements that transfer the loads below the depth of the soil nail wall.

Contractors experienced in the soil nailing method should be able to mitigate the spalling and raveling anticipated in the excavation.

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With soil nailing, the permanent basement walls can be constructed top-down during soil nail wall installation or can be cast-in-place (bottom-up) in front of a temporary 4-inch-thick wall. A top-down permanent wall is often less expensive but has a higher risk of instability because higher vertical cuts and longer standup times are required to install the reinforcement and shotcrete. One potential disadvantage with permanent top-down walls is that full-face drainage or waterproofing cannot be installed behind a permanent top-down wall. At this time, we anticipate that the soil nail wall will be temporary and that cast-in-place concrete walls will be constructed against the temporary soil nail walls.

Drainage

A suitable drainage system must be installed to prevent the buildup of hydrostatic groundwater pressures behind the soil nail walls. Drainage behind soil nail walls typically consists of prefabricated geocomposite drainage strips, such as Miradrain, installed vertically between the soil nails. The drainage strips are typically a minimum of 16 inches wide and extend the entire height of the wall. Horizontal drainage strips may also be used in areas where perched groundwater is observed, at cold joints in permanent top-down basement wall construction, or for other reasons. We recommend that drainage strips be connected to a tightline pipe installed along the base of the wall and routed to a suitable discharge point as described in the “Drainage Considerations” section of this report.

Soil Nail Wall Performance

A soil nail wall is a passive shoring system that requires deflections for load to be applied to the soil nails. We recommend that the soil nail walls be designed such that average wall deflections will be limited to 1 inch and ground surface settlements behind the wall will be less than about 1 inch. The deflections and settlements are usually highest at the excavation face and decrease to negligible amounts beyond a distance behind the wall equal to the excavation height. Wall deflections can be reduced by post-tensioning the upper row(s) of soil nails. Localized deflections may exceed the above estimates and may reflect local variations in soil conditions (such as around abandoned utilities) or may be the result of the workmanship used to construct the wall.

Monitoring of the shoring system should be completed as described in Appendix C, Ground Anchor Load Tests and Shoring Monitoring Program.

Soil Nail Wall Contract

Typically the bid on the shotcrete wall is based on an assumed shotcrete wall thickness. Spalling and raveling of the cut face during construction generally result in change orders from the soil nail wall contractor. Accordingly, we recommend that the following items be considered in order to reduce the cost impact of change orders and to limit the owner's liability with regard to change orders. The contract should include unit prices for shotcrete overbreak (cubic yards) and flash coating (square-foot of wall face). The unit prices should include all costs associated with materials, placement and labor. We recommend that GeoEngineers review the contract on behalf of the owner to see that the risk and liability of potential change orders are addressed appropriately.

We are prepared to develop specific designs for the soil nail walls for the below-grade parking garage upon your request. We have not included recommendations for conventional cantilevered or tied-back soldier pile and timber lagging walls because of the suitability of the on-site soils for support using soil nail walls and the increased cost in using soldier pile and lagging walls. Recommendations for other support systems can be provided upon your request to address specific excavation issues.

CONSTRUCTION DEWATERING

This section summarizes our dewatering recommendations for construction of the below-grade parking garage that extends below the potentiometric surface. These recommendations are based on subsurface soil and groundwater information obtained from borings, wells, aquifer slug tests and aquifer pumping tests completed during June 2008. Our recommendations are also based on our review of previous studies completed on and near the site by others.

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Parking Garage Evaluation

Our construction dewatering evaluation is based on the following assumptions:

- The bottom of the excavation will have lateral dimensions of approximately 500 feet (north-south) by 250 feet (east-west) with a 200-foot by 200-foot extension to the east at the north end of the excavation. The faces of the excavation will be vertical walls that are supported using soil nails.
- Excavation for four levels of underground parking will extend to approximately subgrade Elevation 290 feet.
- The static (pre-pumping) groundwater level in the parking garage area is at approximate Elevation 325 to 327 feet. This water level also applies to the groundwater encountered within the glacial till, which likely occurs in perched or isolated pockets that may not be laterally continuous. However, toward the base of the glacial till, more permeable sandy zones within the till may be hydraulically connected to the underlying advance outwash sands.
- The underlying advance outwash deposits contain groundwater that is under potentially artesian pressure relative to the base of the proposed excavations. Pressure relief will be required to ensure that the excavation subgrade is not compromised by uplift or heave that could rupture or soften the subgrade soils if it is not managed appropriately.
- The design recommendations for excavation, shoring and construction assume that groundwater levels and artesian pressures will be lowered to two feet below the lowest point of the excavation to facilitate construction. This means that groundwater will need to be lowered by approximately 35 feet during construction.

- The upper portion of the excavation above Elevation 320 feet is expected to encounter limited quantities of perched water and incidental seepage. Inflowing seepage above this elevation should be manageable by passive dewatering from within the excavation using trenches and sumps. However, excavation must not proceed below approximate Elevation 315 feet without adequate depressurization of the artesian advance outwash aquifer, in consideration of the significant risk for piping, softening and basal heave caused by unrelieved artesian pressure beneath the subgrade.
- For extending the excavation below Elevation 320 feet, an active system of dewatering is required. The construction dewatering system should consist of vacuum wellpoints installed in two tiers from within the excavation. Alternatively, a single system of eductor wells installed outside of the excavation may be more cost-effective and provide less interference within the excavation. However, the potential for conflict between soil nails and eductor wells may preclude their use on this project.
- For vacuum wellpoints, the first tier of wellpoints and associated header pipe should be installed no deeper than Elevation 320, with wellpoints extending into the top of the Advance Outwash and screened sections set between Elevation 295 and 300 feet.
- For four levels of parking, the structure will extend into the Vashon outwash sands and a second tier of vacuum wellpoints will be required to lower the artesian pressure in the Vashon outwash sands underlying the glacial till. The second tier of vacuum, wellpoints should be installed from Elevation 300 feet, approximately 20 feet below the first wellpoint system header, with wellpoints screened in permeable soils between Elevation 275 and 285 feet.
- The average hydraulic conductivity (permeability) of the aquifer to be dewatered is $2x10^{-4}$ centimeters per second (cm/s). This value is based on the results of our aquifer pumping and slug tests.
- The well spacing will be determined during dewatering system design.

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Parking Garage Recommendations

Our preliminary construction dewatering recommendations for the parking garage are as follows:

- We recommend installing the primary vacuum well point system with well points on 10-foot centers. Well points should be installed along the north, west and south walls. Well points should capture and intercept most groundwater movement coming onto the site from the west, and may not be necessary on the east wall.
- Monitoring wells installed in geotechnical exploration borings must be preserved during excavation to allow the actual water level drawdown to be evaluated by measuring water levels as dewatering and excavation proceed.
- Vacuum well points should be installed to penetrate through the base of the Vashon till and extend at least 5 feet into the top of the advance outwash formation.
- The well points should be completed using 2- to 3-inch-diameter polyvinyl chloride (PVC) casing and well screen surrounded with a clean, rounded, medium sand filter pack. The well screens should be 5 feet in length. The sand pack should extend from the bottom of the well point to a depth of approximately 5 feet below the header level. The upper portion of each well point annulus should be sealed with 5 feet of bentonite chips, hydrated to manufacturer's specification.

- Dewatering flow rates are expected to be highest in the first weeks of pumping and then decrease significantly as water is removed from storage over time. The primary well point dewatering system will need to be pumped continuously to achieve and maintain adequate water level drawdown in the excavation. After approximately two weeks of pumping, we expect the system to produce approximately 15 to 25 gpm.
- We anticipate that the long-term, steady state dewatering discharge rate should not exceed 30 gpm during construction. However, long-term dewatering discharge rates are uncertain because they will reflect variations in aquifer hydraulic conductivity in different parts of the site that may be greater than encountered at GEI-5 and GEI-6, where hydraulic testing was completed. For example, an average hydraulic conductivity value that is 50 percent higher than our assumed value will yield a discharge rate that is also approximately 50 percent higher. If clean coarser sands or sand and gravel are encountered, they may give rise to inflow rates that are higher than anticipated.
- The construction dewatering system will need to be operated continuously after the excavation reaches approximately Elevation 325 feet. A wall drainage system that is integral to the soil nail wall design, plus a permanent underslab drainage system will need to be installed before turning off the construction dewatering system. Recommendations for the wall drainage system were discussed previously and for the permanent underslab drainage system are provided in a subsequent section of this report.
- We recommend installing shallow, gravel-filled trenches with sumps around the perimeter of the completed excavation to collect stormwater runoff, perched groundwater and groundwater that seeps between the dewatering well points. A 1-foot-thick layer of 1½-inch minus clean crushed rock (Section 9-03.1(4)C, Grading No. 57 of the 2008 WSDOT Standard Specifications, or Type 21 aggregate per Section 9-03.16 of the 2008 City of Seattle Standard Specifications) that drains to these shallow, perimeter water collection trenches is recommended to provide a relatively stable and dry working surface at the base of the excavation. This material can be used as underslab drainage material provided that it is not contaminated with fines during construction.
- Arrangement for discharge permitting will be required prior to and during discharge of water generated during construction dewatering. A point of discharge should be designated for the water produced from the temporary well point dewatering system. Water discharged to the storm sewer will need to meet the surface water quality standards for the state of Washington (Chapter 173-291A Washington Administrative Code [WAC]). Water quality testing will be required, and the water may initially require treatment for turbidity.
- A Baker tank will be required to store turbid (muddy) water pumped from the well points during development (surging and bailing) and the first several days of pumping. Budget for temporary water storage and treatment should be included in the cost estimate for construction dewatering.
- We recommend a backup electric power source (or sources) for the dewatering system pumps in the event of a power failure involving the primary power source.
- We recommend having contingency budget available for installation of additional well points if water level monitoring during the early stages of construction dewatering or excessive seepage from the face of the excavation indicates that additional drawdown is required.

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Other Excavation Dewatering Requirements

We anticipate that the excavations for the detention tank and the access driveway walls will be located within the upper glacial till deposits. Therefore, we anticipate that only perched zones of water are likely to be encountered within these excavations. For planning purposes, we recommend that the contractor

plan to use sumps and pumps located within these excavations for temporary dewatering. If necessary, a 6- to 12-inch-thick granular working pad consisting of crushed rock, quarry spalls or the contractor's preferred material can be used if groundwater or stormwater needs to be managed throughout the interior of the site.

SEISMICITY

General

Seismicity in this region is attributed primarily to the interaction of the Pacific, Juan de Fuca and North American plates. The Juan de Fuca Plate is subducting beneath the North American Plate. Each year, numerous earthquakes occur in Oregon and Washington. However, only few of these are typically felt because the majority of recorded earthquakes are smaller than magnitude 3. Because of the thick overburden of glacial sediments, no active surface faults are evident. In addition, the distribution of the recorded seismic epicenters is scattered and does not define a mapable fault zone.

GeoEngineers evaluated the site for seismic hazards including liquefaction, lateral spreading, fault rupture and earthquake-induced landsliding. Our evaluation indicates that the site does not have liquefiable soils present and therefore also has a very low risk of liquefaction-induced settlement or lateral spreading. In addition, the site has a low risk of fault rupture and a low risk of earthquake-induced landsliding because of the moderate topography at the site and the density of the underlying soils.

International Building Code (IBC) Site Coefficients

We recommend the IBC seismic design parameters for the average field standard penetration resistance, 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 Table 2.

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Table 2. IBC Seismic Parameters

IBC Parameter	Recommended Value
Average Field Standard Penetration Resistance	N > 50
Site Class	C
Short Period Spectral Response Acceleration, S_S (percent g)	133
1-Second Period Spectral Response Acceleration, S_1 (percent g)	45
Seismic Coefficient, F_A	1.0
Seismic Coefficient, F_V	1.35

SHALLOW FOUNDATION SUPPORT

General

The type of subgrade soils encountered at the foundation level for the buildings will be dependent on the depth of excavation and finish floor elevation. For the parking garage, the finish floor will be around Elevation 300 feet. Therefore, we anticipate that subgrade soils across the garage will vary from very dense silty sand (glacial till) to very dense sand with silt (advance outwash). Some areas of the parking garage subgrade may expose pockets of cleaner sand. For Buildings 244 and 246, which will be situated at-grade, we anticipate that subgrade soils may consist of medium dense to very dense silty sand (weathered and non-weathered glacial till). Some areas of fill may be encountered along portions of the

buildings. We have therefore developed the recommendations presented in the following paragraphs for allowable foundation bearing pressures based on the anticipated soil conditions and depth of foundations.

Conventional Spread and Continuous Footings

We recommend that individual column footings and continuous wall footings should be at least 24 and 16 inches wide, respectively. Exterior footings should be founded at least 18 inches below the lowest adjacent grade. Interior footings should be founded a minimum of 12 inches below the top of the slab.

We recommend that foundations be supported on undisturbed medium dense to very dense glacial till and advance outwash or on structural fill extending down to the glacial till or advance outwash. Any existing fill encountered should be removed and replaced with structural fill, or the depth of the footing should be extended to bear on the medium dense to very dense native soils. Footings founded on native, undisturbed, very dense glacial till or advance outwash sand may be designed using an allowable soil bearing pressure of 8,000 psf; footings founded at least 15 feet below existing grades may be designed using an allowable soil bearing pressure of 12,000 psf. Table 3 presents a summary of the estimated elevation at which the different bearing pressures are present at the boring locations.

Table 3. Estimated Elevations for Different Soil Bearing Pressures

Boring Number (Surface Elevation)	Elevation at Allowable Bearing Pressure (feet)[1]		
	4000 psf	8000 psf	12000 psf
GEI-1 (344)	342	338	329
GEI-2 (345)	343	339	330
GEI-3 (344)	342	337-338	329
GEI-4 (339)	331-332	326-327	324
GEI-5 (336)	334	330-331	321
GEI-6 (341)	337-338	333-334	326
GEI-7 (332)	328-329	324	317
GEI-8 (333)	329-330	324-325	318
GEI-9 (326)	324	321	311
GEI-10 (316)	311	305-306	301
GEI-11 (301)	297-298	296	286

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Note:
Boring surface elevations were interpreted from site topography and should only be considered approximate.

No structural fill should be placed below footings designed for a bearing value of 8,000 psf or higher. Footings founded on properly compacted structural fill over native glacial deposits or on the medium dense weathered glacial till may be designed using an allowable soil bearing pressure of 4,000 psf. The allowable soil bearing pressures apply to the total of dead and long-term live loads and may be increased by up to one-third for wind or seismic loads.

We anticipate that foundations for the parking garage will be supported on very dense glacial till or outwash and therefore may be designed for an allowable bearing pressure of 12,000 psf. Foundations for Buildings 244 and 246 may be designed for an allowable bearing pressure of 8,000 psf if they extend to into the underlying dense glacial till. A summary of the elevation at which very dense glacial till was observed across the Building 244 area is presented in Table 4. A simplified graphical representation of the

elevation of the surface of the dense glacial till across the Building 244 area and along a portion of the west side of the parking garage is presented in Figure 7.

Table 4. Elevation to Very Dense/Hard Glacial Till – Building 244

Test Pit	Surface Elevation (feet)	Elevation of very dense/hard glacial till (bearing value of 8,000 psf)
1	343	339.5
2	343	336
3	343	338
4	343	340
5	341	334
6	336	333
7	343	339
8	342	339
9	343	339
10	344	340
11	342	339
12	344	340
13	344	341
14	343	340

Note: psf = pounds per square foot

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Based on explorations completed near Building 246 we expect that building foundations will be supported on structural fill or medium dense native soil if situated only 2 to 3 feet below existing grades and should therefore be designed for an allowable bearing pressure of 4,000 psf. Alternatively, the footing elevations could be deepened such that the footing subgrade exposes very dense glacial till, or the footing excavations extend to this depth and controlled density fill (CDF) placed from the exposed very dense till to the design footing grade.

We estimate that the total settlement of footings founded on compacted structural fill or medium dense weathered glacial till will be ¾ inch or less. Settlement of footings supported on undisturbed glacial till or advance outwash sand will be less than ½ inch. We estimate that less than ½ inch of differential settlement will occur between comparably loaded footings. Loose fill or disturbed soil not removed from footing excavations prior to pouring concrete may result in greater settlements.

It should be noted that the footing subgrades will be susceptible to disturbance when wet. It may be necessary to pour a lean concrete "mud mat" or place a layer of crushed rock on the bottom of the footing excavations to protect the footing subgrade from water and/or wet weather during reinforcement bar placement and preparation for placing concrete. We recommend that the condition of all footing excavations be observed by a representative from our firm prior to placement of concrete to confirm that the bearing soils are undisturbed and are consistent with our recommendations contained in this report.

Lateral Resistance

The soil resistance available to resist lateral foundation loads is a function of the friction that can develop on the base and the passive resistance that can develop on the face of below-grade elements of the

structure as these elements tend to move into the soil. For footings and on-grade floor slabs supported on native soils or on structural fill placed and compacted in accordance with our recommendations, 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 on the face of footings or other embedded foundation elements may be computed using an equivalent fluid density of 350 pounds per cubic foot (pcf) (triangular distribution) if these elements are poured directly against undisturbed dense to very dense glacial till or advance outwash sand, or surrounded by structural fill. The structural fill should extend out from the face of the foundation element for a distance at least equal to two and one-half times the height of the element and should be compacted to at least 95 percent of MDD, estimated in general accordance with ASTM D 1557. The above coefficient of friction and passive equivalent fluid density values include a factor of safety of about 1.5.

FLOOR SLABS

Slabs may be supported on-grade, provided that the subgrade soils are prepared as recommended above in the "Earthwork" section of this report. We recommend that the slabs be founded on either undisturbed native soils or on structural fill placed over these same native soils.

We recommend that the slab be supported on a base course consisting of at least 6 inches of ¾-inch minus crushed gravel containing negligible sand and silt. The base course should be placed and compacted as described previously. The base course will provide uniform support and serve as a capillary break to reduce moisture migration through the slab. For slabs designed as a beam on an elastic foundation, a modulus of subgrade reaction of 200 pounds per cubic inch (pci) may be used for subgrade soils prepared as recommended above.

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We recommend that an underslab drainage system, as described in the "Drainage Considerations" section of this report, be installed beneath the garage floor slab. It will not be necessary to install underslab drainage systems below Building 244 and 246. Where moisture-sensitive floor coverings or moisture-sensitive equipment will be used (such as in elevator vestibules in the lowest garage level), we recommend that a suitable vapor retarder be installed below the slab to reduce the potential for migration of moisture. The vapor retarder should be a heavy-duty sheeting intended for this purpose, such as Stego Wrap 15-mil vapor retarder from Stego Industries, Moistop from Fortifibre Building Product Systems, or Vapor Block Plus from Raven Industries. The contractor should be made responsible for maintaining the integrity of the vapor retarder during construction. A 2-inch-thick layer of sand may be placed over the vapor retarder to protect it during construction and to aid in uniform curing of the concrete, provided the sand is kept dry prior to pouring the slab. This should be completed in accordance with American Concrete Institute standards (Committee Report 302.1R-96, or current guidance). The structural engineer should be consulted regarding the need for the 2-inch-thick layer of sand. It may also be prudent to apply a sealer to the slab to further retard the migration of moisture through the floor.

Based on our experience with other projects, it is unlikely that it will be possible to prevent the work pad from becoming contaminated with fines during the installation of foundations and/or below-slab utilities such as plumbing. Therefore, it will likely not be possible to use the work pad as the underslab drainage layer. We recommend that the excavation be extended sufficiently below the slab subgrade elevation to accommodate the work pad as well as the underslab drainage layer.

An alternative to the use of a work pad is to leave the subgrade elevation approximately 1 foot higher than final grade during foundation construction and installation of below-slab utilities. After completion of

this work, the remaining approximately 1 foot of soil will be removed and immediately replaced with the underslab drainage layer. With this alternative, it still may be necessary to construct temporary access roads for construction traffic, especially during periods of wet weather. The decision to utilize a work pad or to leave the subgrade high until capillary break placement should be made by the general contractor.

BELOW-GRADE WALLS

Permanent Below-Grade Garage Walls

Below-grade garage walls are anticipated to be about 45 feet high. The lateral soil pressures acting on the permanent garage walls will depend on the nature, density and configuration of the soil behind the wall and the type of restraint system for the wall. We anticipate that most of the permanent garage walls will be constructed immediately adjacent to the temporary soil nail shoring walls. The permanent below-grade walls will be internally braced by floor slabs.

Where the permanent garage walls will be constructed in front of the temporary shoring walls, we recommend that the earth pressure diagram presented in Figure 8 be used in the design. The static and seismic earth pressures presented in Figure 8 represent the best estimate of actual loads and do not include a factor of safety. These best estimate loads should be factored in accordance with the load combinations in Section 1605 of the 2006 IBC. Other surcharge loading should be estimated using the procedures presented in Figure 9. These soil pressures assume that wall drains will be installed to prevent the buildup of hydrostatic pressure behind the walls, as discussed below, and tied to permanent drains to remove water to suitable discharge points.

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Other Cast-in-Place Walls

We anticipate that the detention vault and access driveway retaining walls will be constructed using conventional cast-in-place walls. In addition, there may be areas where temporary open cuts are used for a portion of the parking garage. The lateral soil pressures acting on conventional cast-in-place walls will depend on the nature, density and configuration of the soil behind the wall and the amount of lateral wall movement that can occur as backfill is placed. We understand that it will likely not be possible to drain the lower 4 to 6 feet of the detention vault walls.

For walls that are free to yield at the top at least 0.1 percent 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). For unrestrained walls with backfill sloping up at 3H:1V, the design lateral earth pressure should be increased to 45 pcf.

At this time, we assume that the detention vault walls may be constructed such that they will be restrained from movement during backfill. Also, the lower portion of the walls will be subjected to permanent pressures from groundwater as it will not be feasible to provide drainage to the base of the walls. We recommend that non-yielding walls supporting horizontal backfill be designed using an equivalent fluid density of 55 pcf (triangular distribution). For restrained walls with a 3H:1V sloping backfill, the design lateral earth pressure should be designed using an equivalent fluid density of 70 pcf. For portions of the walls subject to hydrostatic pressures, we recommend that the walls with level and sloping backfill be designed using an equivalent fluid density of 28 and 35 pcf, respectively, plus hydrostatic water pressure.

The seismic and surcharge pressures presented in Figures 8 and 9 should also be used in the design, as appropriate.

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. For walls founded on native soils, 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). This passive value assumes that the minimum embedment depth for walls founded on sloping ground results in a theoretical horizontal bench equal to twice the depth of the footing between the face of the wall and the sloping ground in front of the wall. If footings are situated closer to the adjacent slope, we should evaluate if the allowable passive pressures should be reduced. 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 drainage will be installed to prevent the buildup of hydrostatic pressure behind the walls, as discussed below in the "Drainage Considerations" section of this report.

DRAINAGE CONSIDERATIONS

Perimeter Footing Drains and Exterior Below-Grade Building Walls

We recommend that the exterior below-grade building walls and perimeter footings be constructed with drains. The drains should consist of a perforated collector pipe, a minimum of 6 inches in diameter, enveloped within a minimum thickness of 6 inches of gravel backfill for drains conforming to Section 9-03.12(4) of the 2008 WSDOT Standard Specifications. A non-woven geotextile fabric, such as Mirafi 140N or other as approved by the Geotechnical Engineer, should be placed between the gravel backfill and the native soils to prevent movement of the soils into the drainage material.

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We recommend using either heavy-wall solid pipe (SDR-35 PVC) or rigid corrugated polyethylene pipe (ADS N-12, or equivalent) for the collector pipe. We recommend against using flexible tubing for wall drainpipe.

The footing drains should be discharged through a tightline pipe to a sump or to a gravity drain. The tightline pipes should also be heavy-wall or rigid pipe.

Underslab Drainage

Construction of the underground parking structure will extend below the potentiometric surface (natural water table) that is at approximately Elevation 327 feet in the garage area. For long-term groundwater drainage and lowering of the groundwater table, we recommend that an underslab drainage system be installed to prevent the buildup of hydraulic pressure beneath the lowest floor of parking. The underslab drainage system should consist of at least 12 inches of 1½-inch minus clean crushed gravel with negligible sand or silt conforming to the gradation presented in Section 9-03.1(4)C, Grading No. 57 of the 2008 WSDOT Standard Specifications or Type 21 aggregate as described in Section 9-03.16 of the 2008 City of Seattle Standard Specifications. The gravel layer should be placed across the entire slab area between spread footings or mat foundations, but should not be placed below these foundation elements. The 12-inch-thick gravel layer will also provide the necessary capillary break recommended above in the "Floor Slabs" section of this report.

A network of minimum 6-inch-diameter perforated pipes should be placed 2 inches above the base of the gravel and should be spaced 20 feet on center. The drainage pipes should be similar to those described above for the perimeter drains. The drainage pipe should be either machine slotted or perforated. The slots should be a maximum of ⅛-inch wide with four slots per inch and extend the lower 60-degree perimeter of the pipe. Perforated pipe should have two rows of ⅛-inch holes spaced 120 degrees apart and at 4 inches on center. The pipe may be laid flat or sloped at 0.5 percent. If sloped, trenches will be required to extend beneath the drainage layer for the pipes to maintain adequate cover separation of drainage layer material (4 inches) above and below. The underslab drainage system pipes should be connected to a header pipe and routed to a sump or gravity drain. To limit overall length of pipe runs, the sump should be centrally located, or more than one sump should be used.

A filter fabric or filter material may be required between the 12-inch-thick gravel layer and the native soil to maintain separation and reduce piping of the fine-grained soil up into the gravel. We can determine if a filter is required once a proposed gravel gradation is provided, and once the floor slab subgrade has been exposed. Generally, a well-graded gravel would more likely be self-filtering than a poorly-graded (single size) gravel.

The amount of groundwater flow entering the drainage system behind the exterior below-grade walls and the underslab drainage system may fluctuate as a function of season and precipitation. We estimate that the flow from this passive drainage system will typically range from 15 to 30 gpm, depending on the depth of the final structure configuration. The expected flow from the underslab drainage system can be reevaluated during construction using actual flows pumped by the active dewatering system.

Wall Drainage

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Drainage behind permanent below-grade walls for parking garages is typically provided using drainage material installed behind the facing for the temporary soil nail wall. Drainage material such as AmerDrain® 500 is typically installed as vertical strips before the shotcrete facing is placed. The strip drains should be connected to weep pipes that extend through the soil nail wall and the permanent building wall at the footing elevation. The weep pipes should be connected to a tightline that is routed to the sump or gravity drain separately from the interior perimeter drain.

The recommendations presented in this report assume that adequate drainage is provided behind the wall. However, we expect that permanent waterproofing may be included on the inside face of the soil nail wall before the permanent wall is cast against it. We expect that the building envelope consultant will be developing the recommendations for permanent waterproofing or damp-proofing for all of the below-grade walls.

If drainage strips are used, additional drainage strips may be necessary in wet areas. Although the use of full wall face coverage will reduce spotting or leaking at the face of the permanent wall, there is still a potential for seepage. If this is a concern, waterproofing should be specified.

In areas where temporary cut slopes are used for excavations, positive drainage should also be provided behind cast-in-place retaining walls by placing a minimum 18-inch-wide zone of free-draining gravel backfill immediately adjacent to the walls. Gravel backfill for walls should conform to Section 9-03.12(2) of the 2008 WSDOT Standard Specifications. The gravel backfill zone should extend from the base of the wall to within 1 foot of the finished ground surface behind the wall. A perforated or slotted drainpipe should be placed near the base of the wall to provide drainage. The wall drainpipe should be connected to a header pipe and routed to a sump or gravity drain. Appropriate cleanouts for drainpipe maintenance should be installed. A larger-diameter pipe will allow for easier maintenance of

drainage systems. The top 1-foot of fill should consist of relatively impermeable soil to prevent infiltration of surface water into the wall drainage zone.

Other Considerations

The soils exposed in the base of the excavations may be moisture-sensitive and susceptible to disturbance from construction activities, especially when water is present on the subgrade. A system of curbs, berms, drainage ditches and swales should be installed around the perimeter of the excavation to intercept and collect surface water.

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 into the storm sewer disposal system. The roof drains should be kept separate from the footing, wall or underslab drainpipes.

PAVEMENTS

Prior to paving, we recommend that the subgrade in pavement areas be probed or proof-rolled with heavy, rubber-tired construction equipment. Any soft, loose or otherwise unsuitable areas should be recompacted, if practical, or removed and replaced with structural fill. We recommend that the probing or proof-rolling of subgrade areas be observed by a representative of our firm to identify areas needing remedial work and to assess the adequacy of subgrade conditions.

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We recommend that the design pavement section in automobile parking areas consist of 2 inches of ½-inch hot mix asphalt (HMA) (PG 58-22) per WSDOT Standard Specifications Sections 5-04 and 9-03, 4 inches of crushed rock base course, and at least a 6-inch thickness of sand and gravel or crushed rock subbase. The subbase material should contain less than 5 percent fines (that portion of material passing the U.S. No. 200 sieve) by weight relative to the fraction of the material passing the ¾-inch sieve. In truck traffic areas, the thickness of HMA and crushed rock base course should be increased to 3 inches and 6 inches, respectively. An appropriate thickness of asphalt-treated base (ATB) can be substituted for a portion of the crushed rock base course, if desired. The substitution of ATB for crushed rock base course provides a similar structural section but improves trafficability over the areas to be paved prior to placement of the HMA pavement. The base course and subbase should both be compacted to at least 95 percent of MDD, estimated in general accordance with ASTM D 1557. The sand and gravel or crushed rock subbase is intended to act as a drainage layer for the pavement when the pavement section is constructed.

Our recommendations for the thickness of sand and gravel or crushed rock subbase beneath new pavement are based on dry weather construction conditions. If the pavement areas are constructed during extended periods of wet weather, a thicker layer of subbase may be required. In areas of extreme disturbance, the subbase many need to be up to 18 inches thick. A separation layer of geotextile fabric may be used between the subgrade soils and the subbase layer to minimize the thickness of selected material required in areas of soft or disturbed subgrade soils.

WETLAND CUTOFF BERM AND TRENCH CONSTRUCTION

The existing wetlands west of the parking garage and Building 244 will be enhanced, in part, by using groundwater collected from the garage drainage system. A fire lane that will be constructed between the

wetland area and the buildings will be about 3 to 6 feet lower in grade than the existing grades across the wetland area. In addition, the west access driveway will cut through the wetland area. The current design includes use of impermeable material in surficial berms and in trenches to form cutoff walls to reduce the likelihood of surface water from the wetland areas seeping into lower cut slopes or retaining wall backfill in this area.

In general, we recommend that cutoff trenches constructed along portions of the perimeter of the wetland area be sufficiently wide to allow for adequate compaction of the material, and extend a minimum of 12 inches into the underlying dense to very dense glacial till deposits. We recommend that the material for the berms and cutoff trenches consist of very silty glacial till compacted to at least 90 percent of the MDD, estimated in general accordance with ASTM D 1557. To the extent possible, the material in the trench and overlying berm should be consistent in character and placed in a continuous operation (for example, weeks or months should not be allowed to pass between the placing and compacting of the trench material and the construction of the berm). In addition, the upper surface of each compacted lift of soil should be scarified before placing subsequent lifts. This is to reduce the risk of preferential pathways for groundwater flow developing within the cutoff berm or trench.

In general, the glacial till soils encountered across the western portion of the Building 244 area are siltier than the soils across the remainder of the building areas and would be best suited for this purpose. The upper very weathered zones of glacial till that contain organic matter should not be used for the cutoff trenches or berms. If the material is placed during wet weather such that the compaction criteria can not be achieved, it may be necessary to use controlled density fill (CDF).

Where retaining walls or other subgrade structures will be present adjacent to wetland areas, the cutoff trenches will have to be located beyond the wall backfill zone. We also recommend that the upper 2 feet of wall backfill consist of the same material used for the cutoff trenches to reduce the risk of significant seepage near the structures.

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LIMITATIONS

We have prepared this report for use by Microsoft Corporation, and their design consultants in design of the proposed Microsoft RedWest South project. The data and report should be provided to prospective contractors for their bidding or estimating purposes, but our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions.

When the design has been finalized, we recommend that we be retained to review the final design drawings and specifications to see that our recommendations have been interpreted and implemented as intended.

The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences or procedures, except as specifically described in our report for consideration in design.

Variations in subsurface conditions are possible between the explorations and may also occur with time. A contingency for unanticipated conditions should be included in the budget and schedule. Sufficient monitoring, testing and consultation should be provided by our firm 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 and foundation installation activities comply with contract plans and specifications.

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.

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

REFERENCES

Booth, D.B., Troost, K.A., and Wisher, A.P. (2006) Geologic Map of King County, Scale 1:100,000. GeoMapNW, Earth and Space Sciences, University of Washington.

International Code Council, "International Building Code", 2006.

Liesch, Bruce A., Charles E. Price and Kenneth L. Walters, (1963). Geology and Ground-Water Resources of Northwestern King County, Washington, WA Dept of Ecology Water Supply Bulletin 20.

Minard, J.P., 1983, United States Geologic Survey, Miscellaneous Field Map MF-1543, "Geologic Map of the Kirkland Quadrangle, Washington."

Terra Associates, 2007, "Due Diligence Review, Geotechnical Considerations, Nintendo Parcel, Redmond, Washington," May 2007.

Terra Associates, 1994, "Geotechnical Report, Microsoft Redmond West, 148th NE and NE 57th Street, Redmond, Washington," May 1994.

U.S. Department of Transportation, Federal Highway Administration, 1999, "Geotechnical Engineering Circular No. 4, Ground Anchors and Anchored Systems," FHWA Report No. FHWA-IF-99-015.

U.S. Geological Survey – National Seismic Hazard Mapping Project – Interactive Deaggregations. URL: *http://eqint.cr.usgs.gov/eq/html/deaggint.html*

WAC (Washington Administrative Code), Title 296, Part N, "Excavation, Trenching and Shoring."

WSDOT, 2008, "Standard Specifications for Road, Bridge and Municipal Construction."

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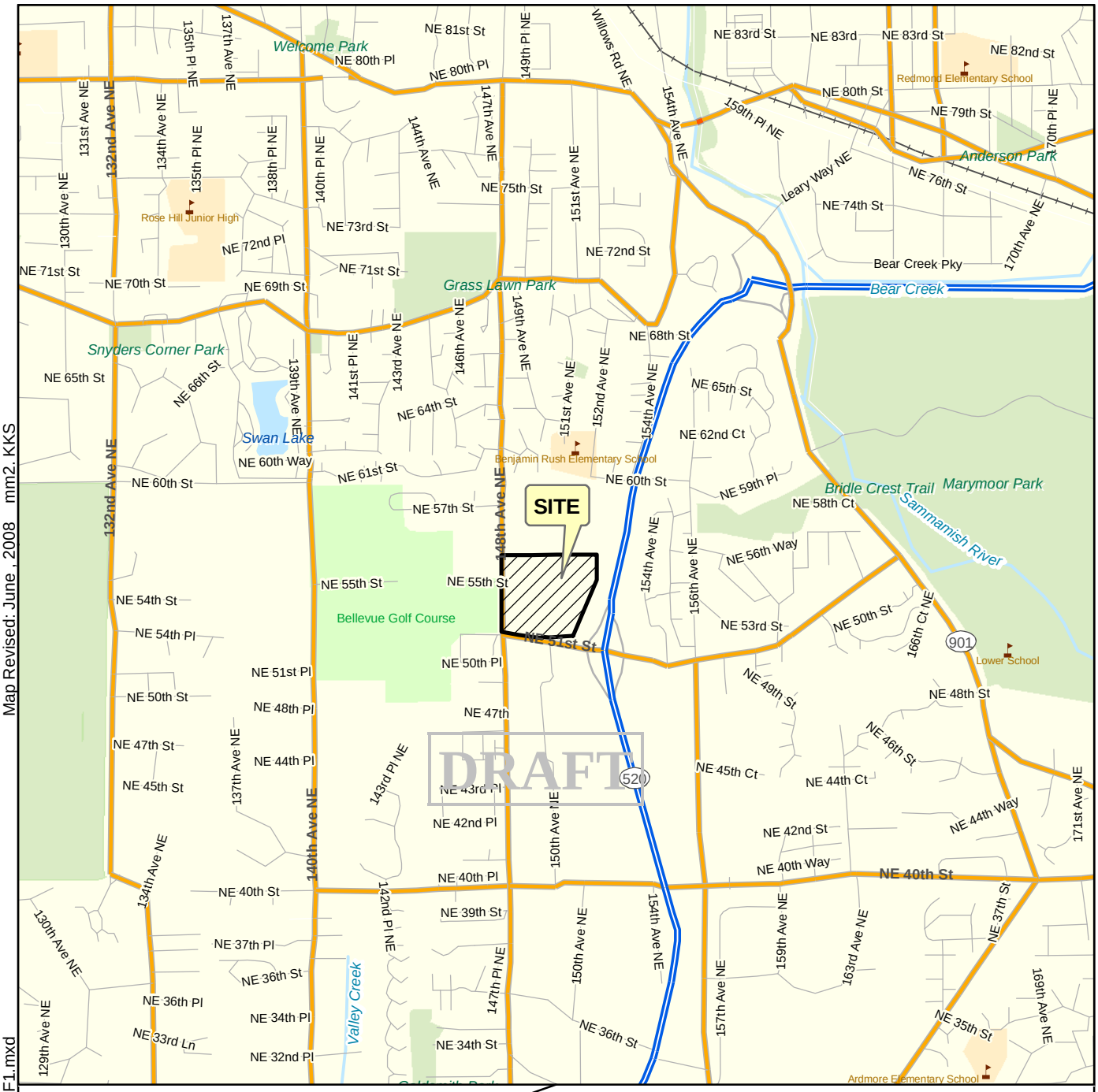

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Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. can not guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.
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Data Sources: ESRI Data & Maps, Street Maps 2005

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

Vicinity Map

Microsoft Redwest South
Redmond, Washington

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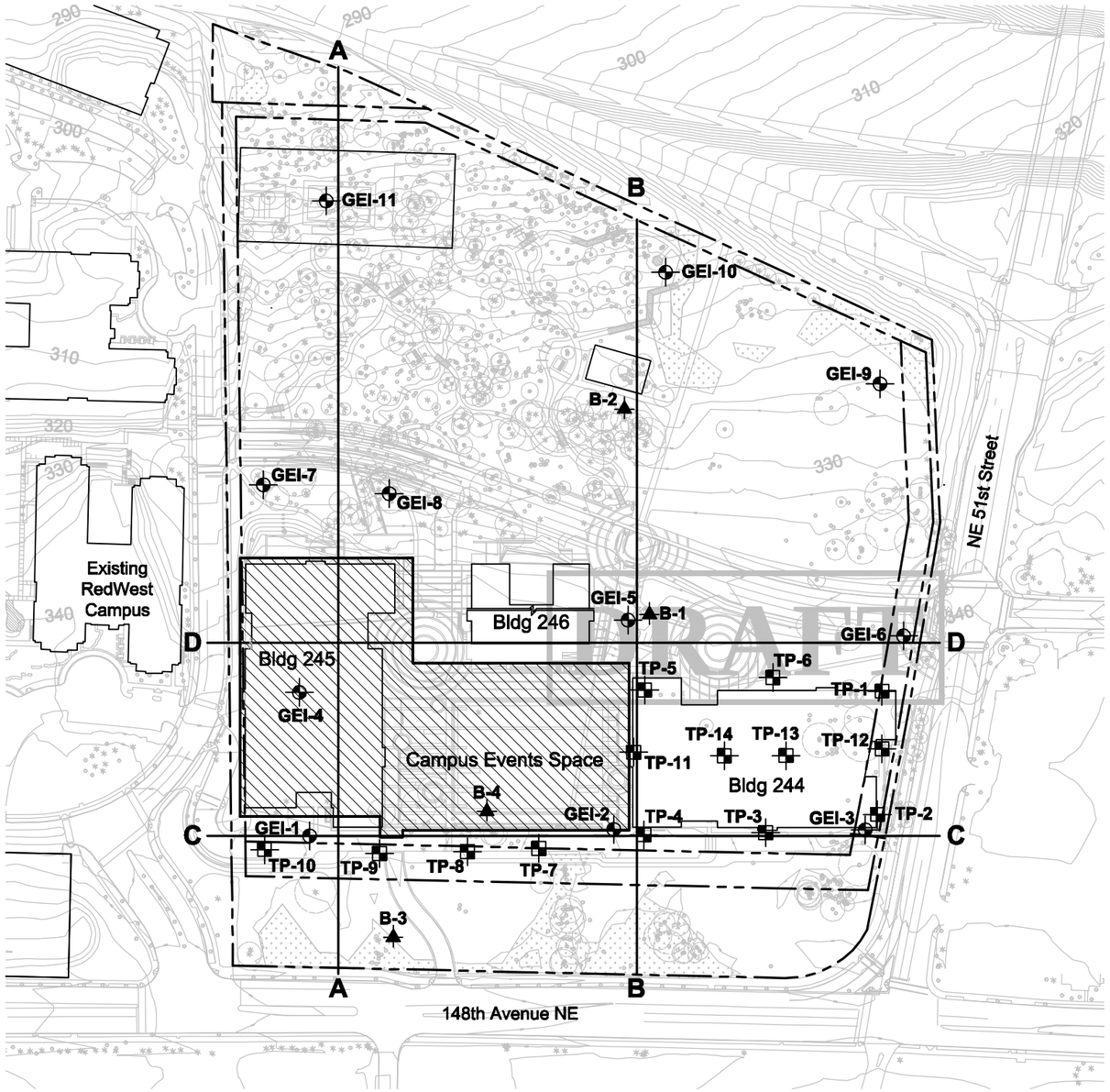
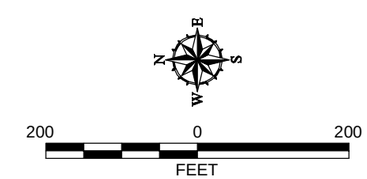
Figure 1

Office: Red Path: P:\0\0694019\GIS\069401900_VM_F1.mxd Map Revised: June , 2008 mm2. KKS





Legend

GEI-1	Boring by GeoEngineers
B-1	Boring by Terra Associates
TP-1	Test Pit by GeoEngineers
	Proposed Parking Garage
A——A	Cross Section Location

Notes

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Reference: Drawing "BaseSitePlan_080818.dwg, provided by Microsoft, dated 8/18/08.

Site Plan

Microsoft RedWest South
Redmond, WA

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Figure 2

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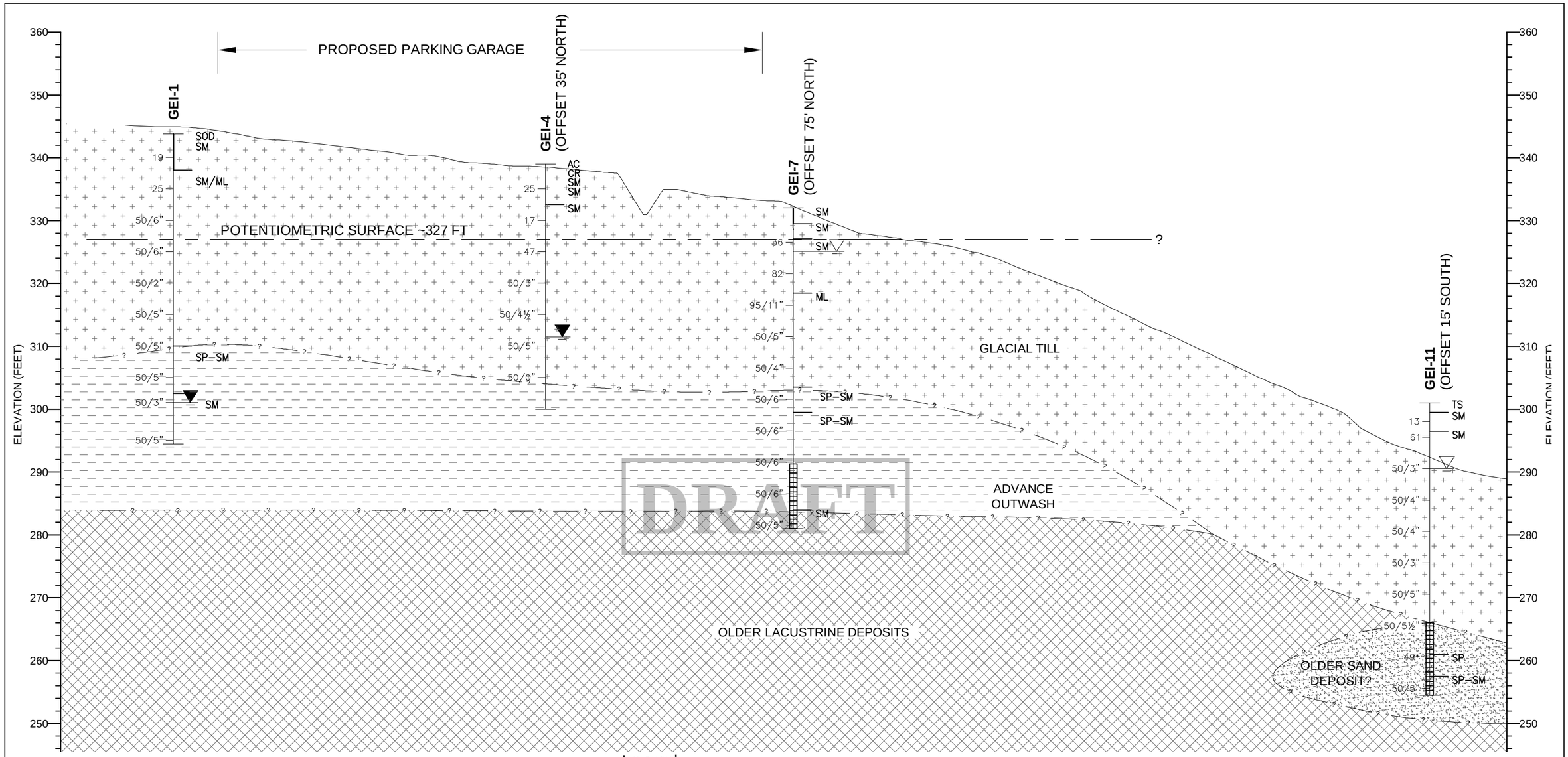

Notes

1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
2. Refer to Figure 2 for location of cross sections.
3. This figure is for informational purposes only. It is intended to assist in the identification of features discussed in a related document. Data were compiled from sources as listed in this figure. The data sources do not guarantee these data are accurate or complete. There may have been updates to the data since the publication of this figure. This figure is a copy of a master document. The hard copy is stored by GeoEngineers, Inc. and will serve as the official document of record.
4. Ground surface elevations based on 2008 site survey by W&H Pacific.

Legend

GEI-1 — BORING

INFERRED GEOLOGIC CONTACT

GROUNDWATER LEVEL OBSERVED DURING DRILLING

MEASURED GROUNDWATER LEVEL

SM — SOIL CLASSIFICATION

MONITORING WELL SCREEN

25 — BLOW COUNT

HORIZONTAL SCALE: 1"= 80'
VERTICAL SCALE: 1"= 16'
VERTICAL EXAGGERATION: 5X

Cross Section A - A'

Microsoft RedWest South
Redmond, Washington

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Figure 3

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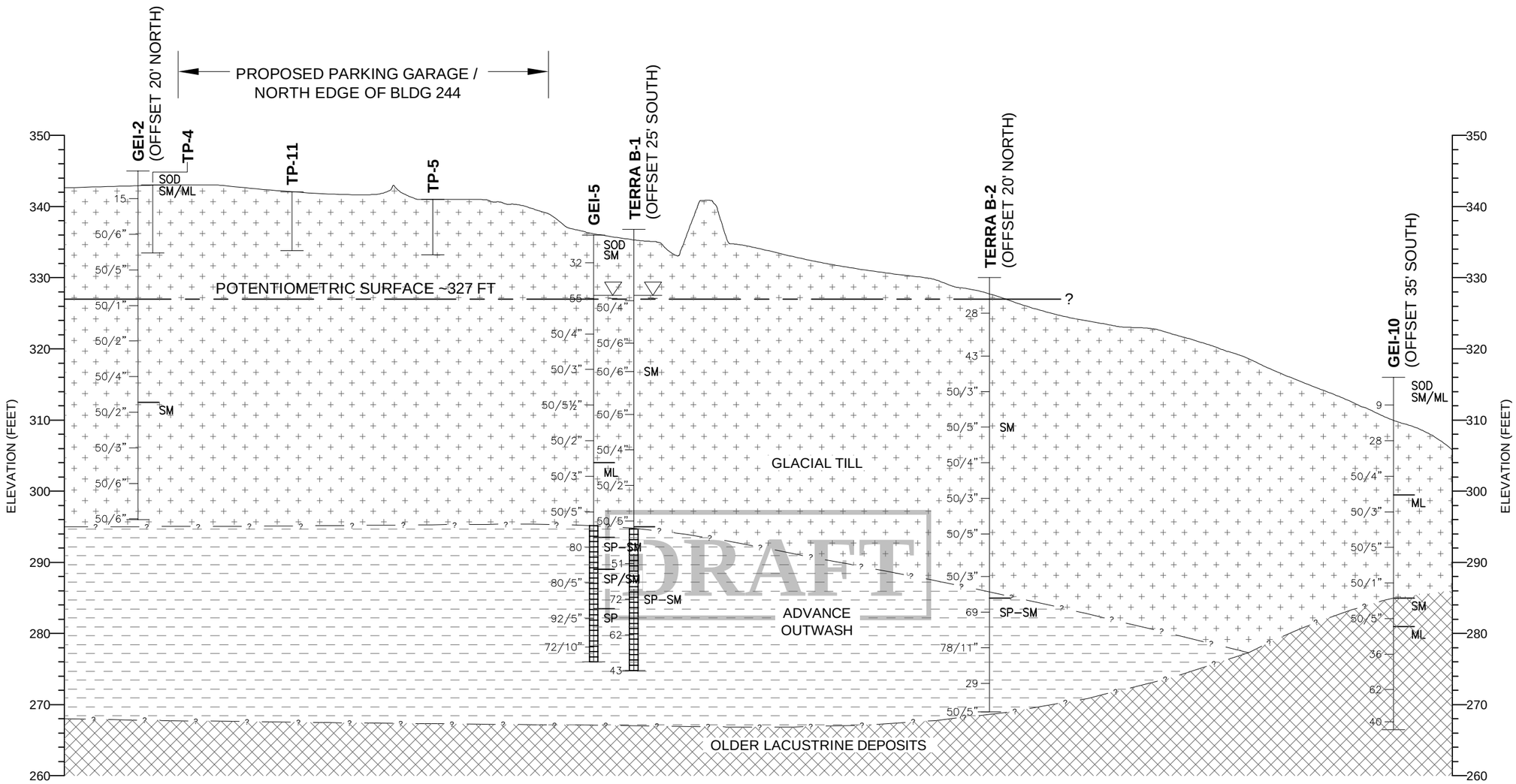

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4. Ground surface elevations based on 2008 site survey by W&H Pacific.

Legend

GEI-1 — BORING

INFERRED GEOLOGIC CONTACT

GROUNDWATER LEVEL OBSERVED DURING DRILLING

MEASURED GROUNDWATER LEVEL

SM — SOIL CLASSIFICATION

MONITORING WELL SCREEN

25 — BLOW COUNT

HORIZONTAL SCALE: 1"= 80'
VERTICAL SCALE: 1"= 16'
VERTICAL EXAGGERATION: 5X

Cross Section B - B'

Microsoft RedWest South
Redmond, Washington

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Figure 4

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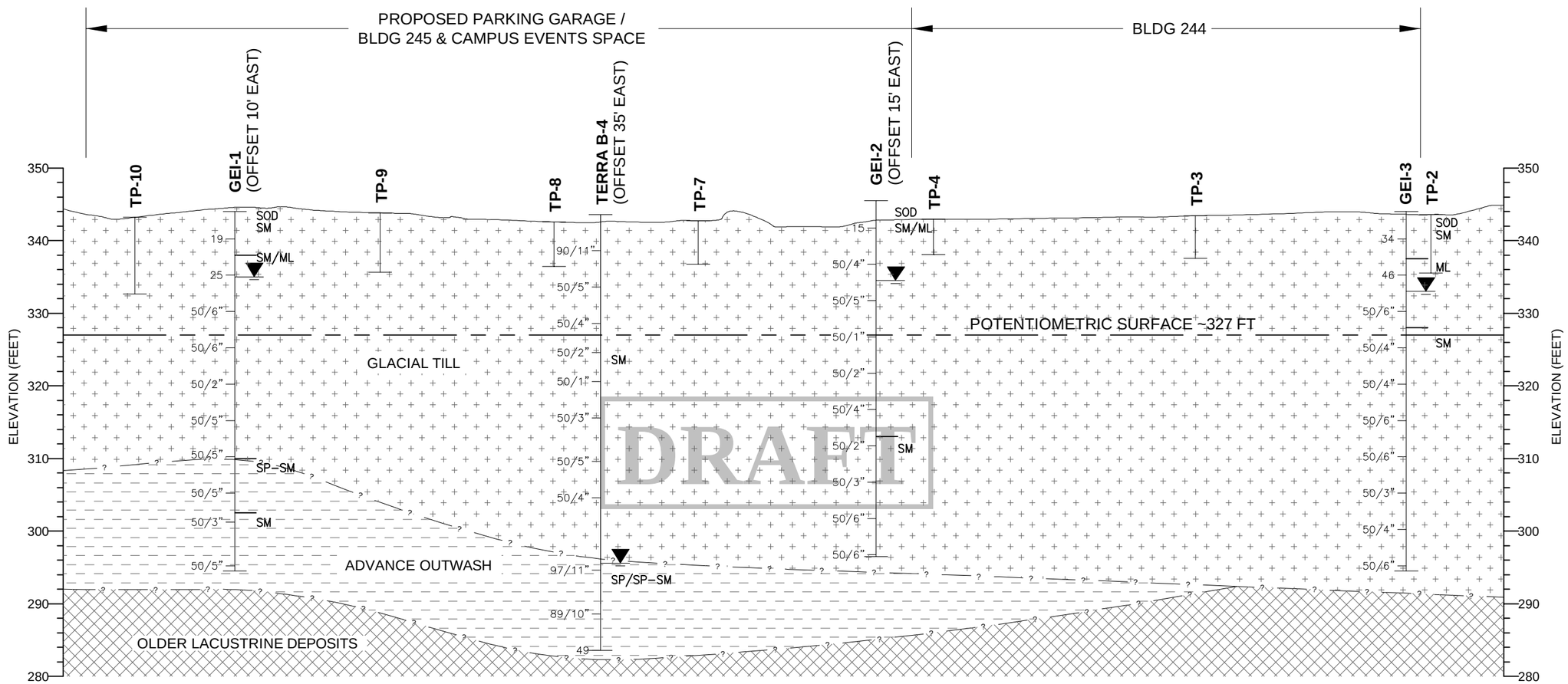

Notes

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4. Ground surface elevations based on 2008 site survey by W&H Pacific.

Legend

GEI-1 — BORING

INFERRED GEOLOGIC CONTACT

GROUNDWATER LEVEL OBSERVED DURING DRILLING

MEASURED GROUNDWATER LEVEL

SM — SOIL CLASSIFICATION

MONITORING WELL SCREEN

25 — BLOW COUNT

HORIZONTAL SCALE: 1"= 80'
VERTICAL SCALE: 1"= 16'
VERTICAL EXAGGERATION: 5X

Cross Section C - C'

Microsoft RedWest South
Redmond, Washington

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Figure 5

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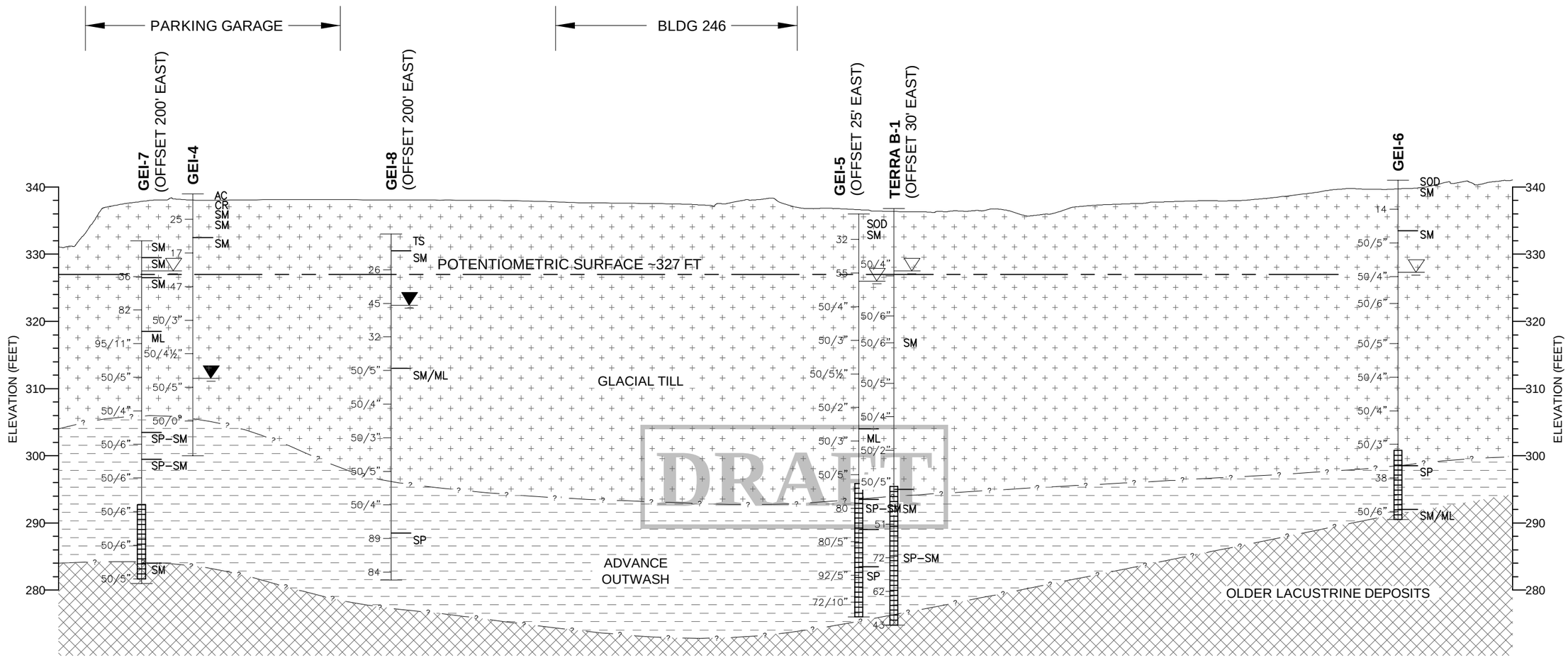

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4. Ground surface elevations based on 2008 site survey by W&H Pacific.

Legend

GEI-1 — BORING

INFERRED GEOLOGIC CONTACT

GROUNDWATER LEVEL OBSERVED DURING DRILLING

MEASURED GROUNDWATER LEVEL

SM — SOIL CLASSIFICATION

MONITORING WELL SCREEN

25 — BLOW COUNT

HORIZONTAL SCALE: 1"= 80'
VERTICAL SCALE: 1"= 16'
VERTICAL EXAGGERATION: 5X

Cross Section D - D'

Microsoft RedWest South
Redmond, Washington

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Figure 6

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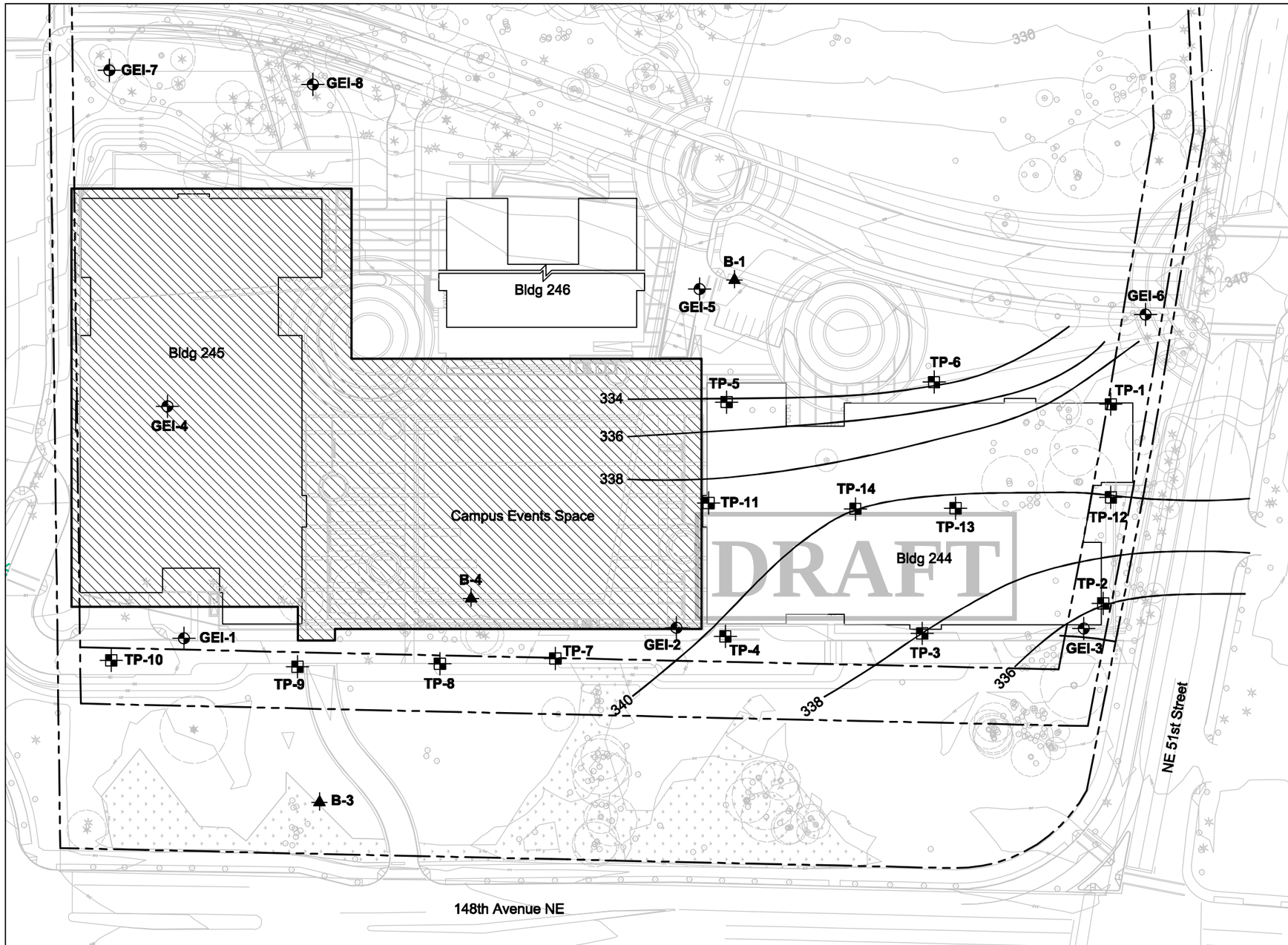

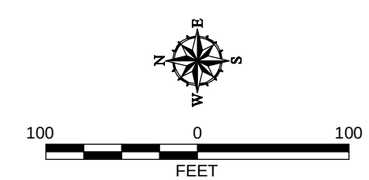

Legend

GEI-1	Boring by GeoEngineers
B-1	Boring by Terra Associates
TP-1	Test Pit by GeoEngineers
	Proposed Parking Garage
340	Very Dense Glacial Till Elevation

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: Drawing "BaseSitePlan_080818.dwg, provided by Microsoft, dated 8/18/08.

Elevation of Very Dense Glacial Till
Building 244

Microsoft RedWest South
Redmond, WA

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Figure 7

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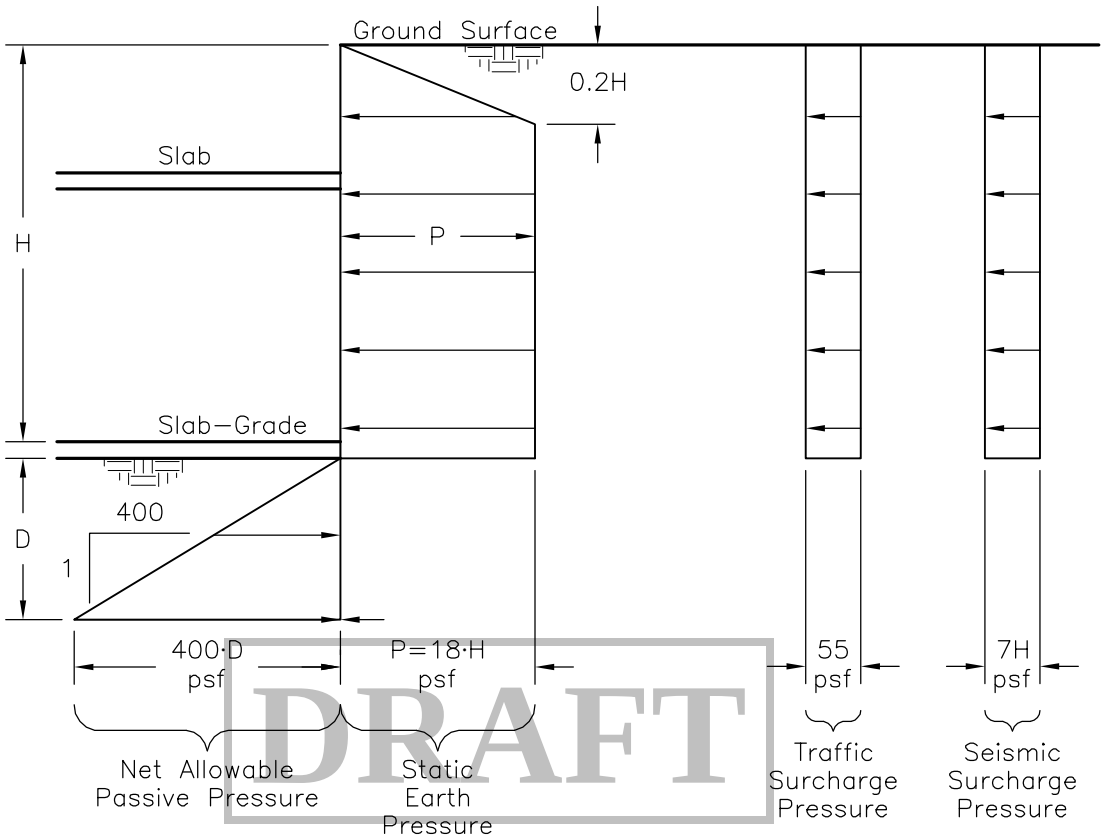
PERMANENT BASEMENT WALL AGAINST TEMPORARY SHORING



DRAFT

NOT TO SCALE

Legend

H = Height of Basement Wall, Feet

D = Foundation Embedment Depth Below Slab, Feet

P = Maximum Static Earth Pressure
Pounds per Square Foot

Notes:
1. Passive pressure includes a factor of safety of 1.5
2. The static earth pressure does not include a factor of safety and represents the actual anticipated static earth pressure.
3. Additional surcharge from footings of adjacent buildings should be included in accordance with recommendations provided in the geotechnical report.
4. This pressure diagram is appropriate for permanent basement walls constructed in front of temporary shoring walls. If additional surcharge loading (such as from soil stockpiles, excavators, dumptrucks, cranes, or concrete trucks) is anticipated, GeoEngineers should be consulted to provide revised surcharge pressures.

Earth Pressure Diagram
Permanent Below Grade Walls

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Figure 8

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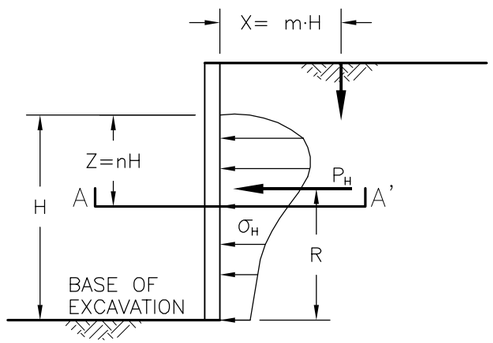
LATERAL EARTH PRESSURE FROM POINT LOAD, Q_P (SPREAD FOOTING)



FOR $m \leq 0.4$

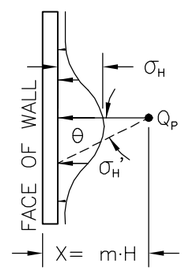
$$\sigma_H = \frac{0.28 Q_P n^2}{H^2 (0.16+n^2)^3}$$

FOR $m > 0.4$

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

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

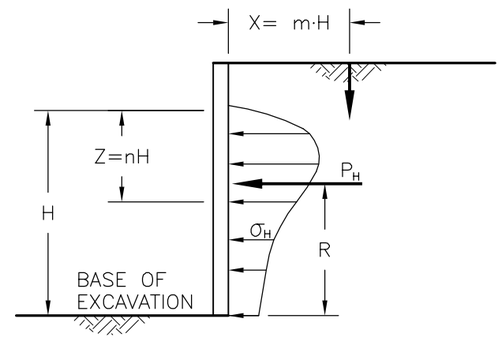
M	$P_H\left(\frac{H}{Q_P}\right)$	R
0.2	0.78	0.59H
0.4	0.78	0.59H
0.6	0.45	0.48H



SECTION A–A'

Pressures from Point Load Q_P

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



FOR $m \leq 0.4$

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

FOR $m > 0.4$

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

RESULTANT $P_H = \dfrac{0.64 Q_L}{(m^2+1)}$

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

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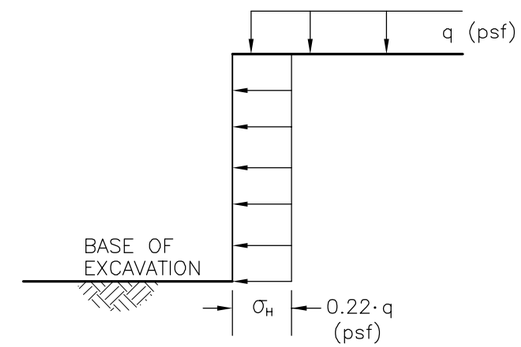
UNIFORM SURCHARGES, q (FLOOR LOADS, LARGE FOUNDATION ELEMENTS OR TRAFFIC LOADS)



σ_H = LATERAL SURCHARGE PRESSURE FROM UNIFORM SURCHARGE

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 7.02 Naval Facilities Engineering Command, September 1986 (NAVFAC DM 7.02).
2. Lateral earth pressures from surcharge should be added to earth pressures presented on Figures 3 and 5.
3. See report text for where surcharge pressures are appropriate.

Recommended Surcharge Pressures

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Figure 9

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APPENDIX A
FIELD EXPLORATIONS AND AQUIFER TESTS

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APPENDIX A
FIELD EXPLORATIONS
AND AQUIFER TESTS

FIELD EXPLORATIONS

Subsurface soil and groundwater conditions at the project site were explored by drilling 11 borings to depths ranging from 39 to 60 feet and excavating 14 test pits to depths ranging from 4 to 9 feet. The borings were drilled between May 27 and May 30, 2008, by Holocene Drilling of Fife, Washington, using track-mounted, hollow-stem auger drilling equipment, and by Geologic Drill of Nine Mile Falls, Washington, using truck-mounted, continuous-flight, hollow-stem auger drilling equipment. The test pits were excavated on September 11, 2008, using a rubber-tired backhoe provided by GLY Construction. Locations of the explorations were determined in the field by measuring distances from existing site features and by using a global positioning system (GPS) locating device. The approximate locations of the explorations are shown on the Site Plan, Figure 2.

The borings and test pits were continuously monitored by a geotechnical engineer from our firm who examined and classified the soils encountered, obtained representative soil samples, observed groundwater conditions, and prepared a detailed log of each exploration. Relatively undisturbed samples were obtained from the borings at selected depths using a 2½-inch-inside-diameter split-barrel sampler. The sampler was driven into the soil a distance of 18 inches or other specified distance using a cathead operated 140-pound hammer free-falling a vertical distance of 30 inches. The number of hammer blows needed to drive the sampler the final 12 inches or other indicated distance is recorded on the boring logs. Bag samples were obtained from the test pit excavations.

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Soils encountered in the explorations were visually classified in general accordance with the system described in Figure A-1. A key to the boring log symbols is also presented in Figure A-1. Logs of the borings are presented in Figures A-2 through A-12. Logs of the test pits are presented in Figures A-13 through A-26. These boring and test pit logs are based on our interpretation of the field and laboratory data and indicate the various types of soils and groundwater conditions encountered. They also indicate the depths at which these soils or their characteristics change, although the change might actually be gradual. If the change occurred between samples, it was interpreted.

Observations of groundwater conditions were made as the explorations were completed. Monitoring wells were also installed in four of the borings (GEI-5, GEI-6, GEI-7 and GEI-11) after completion of the drilling. The monitoring wells installed in borings GEI-6, GEI-7 and GEI-11 consisted of 2-inch-diameter schedule 40 polyvinyl chloride (PVC) pipe. The lower 10 feet of the pipe was machine-slotted (0.01-inch slot width) to allow entry of water into the piezometer. Boring GEI-5 had 3-inch-diameter screen and casing installed, with the screen placed from a depth of about 40 to 60 feet, to place the screen near the same depth as existing boring B-1, installed previously by Terra Associates. Clean sand was placed in the borehole annulus surrounding the slotted portion of the PVC pipe. Bentonite chips were placed above the sand pack to form a surface seal. The piezometers are protected by at-grade steel monuments. Groundwater levels were measured in the piezometers on June 3, 4 and 5, 2008, and the results are presented in the report text.

AQUIFER SLUG TESTS

General

Aquifer slug tests were completed in monitoring well GEI-6 on June 4, 2008. This monitoring well was selected to estimate the hydraulic conductivity (permeability) of the Vashon advance outwash deposits beneath the proposed subgrade elevation for the planned underground parking garage.

Field Procedures

Initial water levels were measured in the monitoring well using an electric water level indicator before the slug was introduced into the well. After the initial water levels were measured, the slug tests were performed using the following procedure:

1. A 5 pounds per square foot (psi) pressure transducer was placed in the well and was connected to an electronic data logger.
2. A slug (weighted 18-foot length of sealed PVC casing) of known volume was rapidly lowered into the well to a point below the initial static water level, causing the water level to rise by displacement. The increased water level in the well caused groundwater to flow out through the well screen and into the surrounding formation, as a falling-head slug test. The declining groundwater level was monitored until it returned to the approximate initial water level. The rate at which the water level declined is proportional to the hydraulic conductivity of the surrounding formation.
3. The slug was then rapidly removed from the well to initiate a rising-head slug test. The reduced water level in the well caused groundwater to flow in through the well screen and into the surrounding formation. The rising groundwater level was monitored until it returned to the approximate initial water level. The rate at which the water level rose is proportional to the hydraulic conductivity of the surrounding formation.
4. The data logger recorded the water level in the well at predetermined intervals (between 2 and 5 seconds) as the water level rose after the slug was removed. Measurements continued until the water level returned to the approximate initial water table level.
5. The pressure transducer was removed from the well.

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Data Analysis

The hydraulic data from the two slug tests were plotted as a transient decline in excess head against elapsed time for each slug test (Figure A-27). The hydraulic head is expressed as a ratio of the observed head to the maximum head difference caused by slug displacement, plotted on a logarithmic scale. The appropriate method of data analysis was chosen based on the shape of the data plots:

- If the data plots as a straight line, the horizontal hydraulic conductivity (K) values were calculated using the Bouwer-Rice method (Bouwer and Rice 1976).
- If the data plots as a curve, the horizontal hydraulic conductivity values were calculated using the Papadopulos method (Cooper et al. 1967; Papadopulos et al. 1973).

Based on the curved shape of the data plots in Figure A-27, the results were re-plotted and analyzed using the Papadopulos method. Well construction details were obtained from the well logs. The Papadopulos method, as applied for this project, is based on the following assumptions:

- The aquifer is confined, homogeneous, isotropic and fully penetrated by the monitoring wells.

- The aquifer and initial piezometric surface are horizontal and extend infinitely in the radial direction.
- Groundwater temperature, density and viscosity are constant.
- Groundwater flow can be described by Darcy's Law.
- A slug of known volume is inserted/extracted instantaneously at the start of the test.
- Water levels are above the screened portion of the well.
- Head losses through the well screen and filter material are negligible.
- Changes in the piezometric surface are small compared to the saturated aquifer thickness.
- The compressibility of the soil matrix and the groundwater are combined in the aquifer storativity term.

To apply the Papadopulos method, the declining head ratio is plotted against the elapsed time since slug injection or withdrawal, with the time plotted on a logarithmic scale, as shown in Figure A-29 for the falling head test and in Figure A-30 for the falling head test. The data are analyzed by matching the shape of the resulting curve to one of a series of published type curves. We have used a slug test spreadsheet published by the United States Geological Survey (USGS) that allows both the aquifer transmissivity and storativity to be adjusted to generate the type-curve that provides the best fit to test data.

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The Papadopulos method yields values for aquifer transmissivity and storativity, as summarized in the following table:

Slug Test	Transmissivity (ft^2/d)	Hydraulic Conductivity (cm/s)	Storativity (dimensionless)
GEI-6 Rising Head	7.2	2.5×10^{-4}	0.014
GEI-6 Falling Head	9.4	3.3×10^{-4}	0.003

Notes:
ft^2/d = square feet per day
cm/s = centimeters per second

A value for hydraulic conductivity is calculated from the transmissivity based on the aquifer thickness at the well, which was approximately 10 feet, based on the boring log (Figure A-7).

The course-fitting process is somewhat subjective, and there are some factors that can change the shape of the curve, so the accuracy of the conductivity determination is limited. Also, the shape of the type curves changes slightly for large changes in storativity, so the value calculated for this parameter is approximate at best. The average K-value calculated from the slug tests at GEI-6 is 3×10^{-4} cm/s.

References

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

Cooper, H.H., Jr., Bredehoeft, J.D., and Papadopulos, I.S., 1967. Response of a finite-diameter well to an instantaneous charge of water. *Water Resources Research*, v. 3, no. 1, p. 263-269.

Papadopulos, I.S., Bredehoeft, J.D., and Cooper, H.H., Jr., 1973. On the analysis of "slug test" data. *Water Resources Research*, v. 9, no. 4, p. 1087-1089.

AQUIFER PUMPING TEST

General

An aquifer pumping test was completed in monitoring well GEI-5 on June 4, 2008. Boring B-1, installed previously by Terra Associates and completed as a monitoring well, was used as an observation well during the test to measure interference drawdown within the cone of depression created while pumping groundwater from GEI-5. Data from the test were used to estimate the hydraulic conductivity and storativity (storage coefficient) of the Vashon advance outwash deposits beneath the site.

Field Procedures

Pumping

The static water levels were measured in GEI-5 and B-1 using an electric water level indicator before the pump was installed in GEI-5. A Grundfos RediFlo 3-inch electric submersible pump was installed at a depth of 30 feet. After the static water levels were measured, the pump was turned on. The pumping rate initially spiked at 5.69 gallons per minute (gpm) and caused over 25 feet of drawdown in the GEI-5 well. The pump was throttled back to about 1.5 gpm, and pumping continued for approximately 3¼ hours.

Water levels were measured in GEI-5 and B-1 throughout the test using pressure transducers with periodic manual checks using an electric water level indicator. A plot of pumping rate and drawdown for GEI-5 is included as Figure A-30. The pumping rate declined exponentially during the duration of the test, in which the drawdown in GEI-5 remained relatively constant. Water discharged from GEI-5 was diverted into a nearby drainage trench. Pumping rates were measured with an in-line totalizer. Drawdown was observed in B-1 after a few minutes and increased throughout the test, but at a progressively decreasing rate. The data are plotted in Figure A-31.

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Recovery

The pump was turned off after 3 hours and 20 minutes of pumping. Rising water levels were measured in the wells during the subsequent recovery period for two hours after the pump was turned off. The recovery data are plotted in Figures A-33 and A-34.

Data Analysis

A summary of the maximum water level drawdowns measured in the wells near the end of the 10-hour pumping period is presented in the following table.

Well	Approximate Distance from Pumping Well	Maximum Drawdown (feet)
GEI-5	0 feet	25
B-3	20.5 feet	8.3

Virtually constant drawdown of around 25 feet was measured in GEI-5 throughout the test. During the pumping period, the measured flowrate declined to a little more than 1 gpm by the end of the pumping phase, as plotted in Figure A-32(a). The test data were analyzed using the method of Jacob and Lohman (1952), which was developed for artesian wells flowing at the ground surface.

Constant-Drawdown Analysis

In accordance with the Jacob and Lohman method, if the specific drawdown (defined as drawdown divided by discharge) is plotted against the adjusted time value plotted on a logarithmic scale, the data should conform to an approximate straight line. The resulting plot is shown in Figure A-32(b), with some scatter because of the difficulty of measuring low flow rates accurately, or to variation in the rate of

pumping. The slope of the straight line is proportional to the aquifer transmissivity, which is calculated by this method to be 6.7 ft^2/d. The intercept of the line with the adjusted time axis is proportional to the storativity of the aquifer, which is calculated to be 0.0016.

Observation Well Drawdown Analysis

The drawdown response in the observation well would normally plot as a straight line when graphed against the logarithm of elapsed time if the well had been pumped at a constant rate. However, the drawdown data from B-1 shows substantial curvature (see Figure A-33). This is caused in part by the declining flowrate.

The second line drawn in Figure A-33 is a simulated drawdown curve taking account of the constant drawdown, rather that constant discharge at monitoring well GEI-5. The simulated drawdown curve shown in Figure A-33 was developed using the solution developed by Mishra and Guyonnet (1992) for drawdown in an observation well during a constant-drawdown test. This has some curvature but not as much as the B-1 data, suggesting the data are affected by other factors. The simulated curve was developed using an aquifer transmissivity of 6.7 ft^2/d and a storativity of 0.0005. The transmissivity value is very similar to that developed from the constant drawdown analysis of the pumping rate, and a much smaller value of storativity is more typical for a confined aquifer.

Recovery Analysis

The recovery data are plotted in Figure A-34 to facilitate analysis using the Theis recovery method. In this method, the recovery data is plotted against the time ratio (t/t') plotted on a logarithmic scale. This results in the data plotting "backwards" with the late-time recovery data as small values of residual drawdown on the left side of the chart. The common slope of this portion of the recovery plot is used in the Theis recovery method to calculate the transmissivity for the aquifer; in this case, the resulting value is 4.6 ft^2/d. Storativity cannot be determined from the recovery analysis.

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Summary

The calculated aquifer parameters using the three different analytical methods are summarized in the following table:

Methodology	Transmissivity (ft^2/d)	Hydraulic Conductivity K (cm/s)	Storativity (dimensionless)
Jacob & Lohman	6.7	1.6×10^{-4}	0.0016
Mishra and Guyonnet	6.7	1.6×10^{-4}	0.0005
Theis Recovery	4.6	1.1×10^{-4}	NA

The calculated K values from the aquifer pumping test are similar to but generally lower than those calculated from the slug tests. The hydraulic conductivity values are calculated from the transmissivity assuming the aquifer is 15 feet thick in the vicinity of the test wells.

References

Jacob, C.E. (1963). The Recovery Method for Determining the Coefficient of Transmissibility. USGS Water-Supply Paper 1536-I; P283-292

Jacob, C.E., and Lohman, S.W. (1952). Non-steady flow to a well of constant drawdown in an extensive aquifer. Trans. AGU, Vol 33, pp 559-569.

Mishra, S., and Guyonnet, D., (1992). Analysis of Observation-Well Response During Constant-Head Testing. Ground Water, Vol 30, No 4, pp523-528.

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

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ADDITIONAL MATERIAL SYMBOLS

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

Measured groundwater level in exploration, well, or piezometer

Groundwater observed at time of exploration

Perched water observed at time of exploration

Measured free product in well or piezometer

Stratigraphic Contact

Distinct contact between soil strata or geologic units

Gradual change between soil strata or geologic units

Approximate location of soil strata change within a geologic soil unit

Sampler Symbol Descriptions

2.4-inch I.D. split barrel

Standard Penetration Test (SPT)

Shelby tube

Piston

Direct-Push

Bulk or grab

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.

Laboratory / Field Tests

%F	Percent fines
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
PP	Pocket penetrometer
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

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Figure A-1

Date(s) Drilled	05/28/08	Logged By	BP/JS	Checked By	NLT
Drilling Contractor	Geologic Drill	Drilling Method	Hollow-stem Auger	Sampling Methods	SPT
Auger Data	3¼-inch I.D., 7¼-inch O.D.	Hammer Data	140 lb hammer/30 in drop rope and cathead	Drilling Equipment	Deep Rock XL
Total Depth (ft)	49.5	Surface Elevation (ft)	344	Groundwater Elevation (ft)	301
Vertical Datum		Datum/ System		Easting(x): Northing(y):	1317870.241 242534.9873

Elevation feet	Depth feet	SAMPLES Interval	Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
	0							SOD	2 inches sod layer			
								SM	Brown silty fine to medium sand with gravel, cobbles and occasional boulders (medium dense,moist to wet) (weathered glacial till)			
340	5		18	19	1							
								SM/ML	Brownish gray silty fine sand to sandy fine silt with trace of gravel and occasional boulders (dense to very dense/hard, wet) (glacial till)			
335	10		15	25	2					12		SA; %F = 42 Perched water at 9 feet
									Grades to moist			
330	15		6	50/6"	3							
									Grades to gray in color			
325	20		6	50/6"	4							
320	25		2	50/2"	5							Rough drilling
315	30		5	50/5"	6							

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

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V6_GTBORING P:\0694019\00\FINALS\06940190O.GPJ GEIV6_1.GDT 11/7/08

LOG OF BORING GEI-1

GEOENGINEERS

Project: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-2
Sheet 1 of 2

Elevation feet	Depth feet	SAMPLES Interval Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft^3	OTHER TESTS AND NOTES
	30										
310	35	5	50/5"	7			SP-SM	Sample wet at 34 feet Gray fine to medium sand with silt and trace gravel (very dense, moist to wet) (advance outwash)	7		
305	40	17	50/5"	8					10		
							SM	Grades to more sand and gravel Gray silty fine sand (very dense, wet) (advance outwash?)			
300	45	4	50/3"	9	▼						Free water in hole
295	50	11	50/5"	10				Grades to moist			
290	55										
285	60										
280	65										

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LOG OF BORING GEI-1 (continued)

GEOENGINEERS

Project: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-2
Sheet 2 of 2

Date(s) Drilled	05/28/08	Logged By	BP/JS	Checked By	NLT
Drilling Contractor	Geologic Drill	Drilling Method	Hollow-stem Auger	Sampling Methods	SPT
Auger Data	3¼-inch I.D., 7¼-inch O.D.	Hammer Data	140 lb hammer/30 in drop rope and cathead	Drilling Equipment	Deep Rock XL
Total Depth (ft)	49	Surface Elevation (ft)	345	Groundwater Elevation (ft)	Not Encountered
Vertical Datum		Datum/ System		Easting(x): Northing(y):	1317862.734 242093.6846

Elevation feet	Depth feet	SAMPLES Interval	Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
345	0							SOD	2 to 3 inches sod			
								SM/ML	Light brownish gray silty fine sand to fine sandy silt with gravel and occasional cobbles and boulders (medium dense/medium stiff to stiff, wet) (weathered glacial till)			
			18	15	1							
340	5								Grades to very dense glacial till			
			18	50/6"	2							Possible perched water at 11 to 12 feet
335	10											
			9	50/5"	3					10		
330	15								Grades to gray			
			1	50/1"	4							Cobbles or boulders encountered
325	20											
			2	50/2"	5							Rough drilling
320	25								More gravel			
			4	50/4"	6							
315	30											

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

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LOG OF BORING GEI-2

GEOENGINEERS

Project: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-3
Sheet 1 of 2

V6_GTBORING P:\0694019\00\FINALS\06940190O.GPJ GEIV6_1.GDT 11/7/08

Elevation feet	Depth feet	SAMPLES Interval Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft^3	OTHER TESTS AND NOTES
315	30										
							SM	Gray silty fine sand with gravel and occasional cobbles (very dense, moist)			
		2	50/2"	7							Rough drilling
310	35										
		3	50/3"	8							
305	40										Obstruction (cobble/boulders)
		6	50/6"	9							
300	45										
		6	50/6"	10							
295	50										
290	55										
285	60										
280	65										

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V6_GTBORING P:\0\0694019\00\FINALS\069401900.GPJ GEIV6_1.GDT 11/7/08

LOG OF BORING GEI-2 (continued)

GEOENGINEERS

Project: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-3
Sheet 2 of 2

Date(s) Drilled	05/28/08	Logged By	JS	Checked By	NLT
Drilling Contractor	Geologic Drill	Drilling Method	Hollow-stem Auger	Sampling Methods	SPT
Auger Data	3¼-inch I.D., 7¼-inch O.D.	Hammer Data	140 lb hammer/30 in drop rope and cathead	Drilling Equipment	Deep Rock XL
Total Depth (ft)	49.5	Surface Elevation (ft)	344	Groundwater Elevation (ft)	307.9
Vertical Datum		Datum/ System		Easting(x): Northing(y):	1317856.531 241728.9717

Elevation feet	Depth feet	SAMPLES Interval	Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
	0							SOD	2 to 3 inches sod			
								SM	Brown silty fine to medium sand with gravel and trace organic matter, cobbles and boulders (weathered glacial till)			
340	5		3	34	1							
								SM/ML	Brownish gray silty fine sand to fine sandy silt with gravel and occasional cobbles and boulders (medium dense to dense/stiff, moist to wet) (glacial till)			
335	10		12	46	2							
									Grades to very dense, hard			Possible perched water
330	15		6	50/6"	3							
								SM	Gray silty fine sand with trace gravel and occasional cobbles and boulders (very dense, moist)			
325	20		4	50/4"	4					7		Layer with gravel
320	25		4	50/4"	5							Rough/slow drilling
315	30		6	50/6"	6					7		%F = 28

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

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LOG OF BORING GEI-3

GEOENGINEERS

Project: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-4
Sheet 1 of 2

V6_GTBORING P:\0694019\00\FINALS\06940190O.GPJ GEIV6_1.GDT 11/7/08

Elevation feet	Depth feet	SAMPLES: Interval Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
	30										
310	35	6	50/6"	7							Rough drilling
305	40	3	50/3"	8				Grades to interbedded fine sandy silt and silty fine sand			
300	45	10	50/4"	9				With occasional lenses of fine sand Grades to wet			
295	50	12	50/6"	10							
290	55										
285	60										
280	65										

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LOG OF BORING GEI-3 (continued)

GEOENGINEERS

Project: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-4
Sheet 2 of 2

Date(s) Drilled	05/30/08	Logged By	CMK	Checked By	NLT
Drilling Contractor	Geologic Drill	Drilling Method	Hollow-stem Auger	Sampling Methods	SPT
Auger Data	3¼-inch I.D., 7¼-inch O.D.	Hammer Data	140 lb hammer/30 in drop rope and cathead	Drilling Equipment	Deep Rock XL
Total Depth (ft)	39	Surface Elevation (ft)	339	Groundwater Elevation (ft)	311.5
Vertical Datum		Datum/ System		Easting(x): Northing(y):	1318166.612 242570.0785

Elevation feet	Depth feet	SAMPLES Interval	Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
	0							AC	2 inches asphalt			
								CR	4 inches crushed rock			
								SM	Dark brown silty sand with gravel and construction debris (medium dense, moist) (fill)			
								SM	Brownish gray silty fine to medium sand with occasional gravel (medium dense, moist) (fill?)			
335	5		18	25	1							Cobbles
								SM	Brownish gray with orange staining silty fine to medium sand with occasional gravel and cobbles and boulders (medium dense, moist to wet) (weathered glacial till)			
330	10		18	17	2							
325	15		17	47	3					11		SA; %F = 34
									Grades to dense to very dense glacial till			
									Grades to moist			
320	20		9	50/3"	4							
									Grades to gray			
315	25		4½	50/4½"	5							Rough drilling
310	30		5	50/5"	6							

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

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LOG OF BORING GEI-4

GEOENGINEERS

Project: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-5
Sheet 1 of 2

V6_GTBORING P:\0694019\00\FINALS\06940190 0.GPJ GEIV6_1.GDT 11/7/08

Elevation feet	Depth feet	SAMPLES: Interval Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft^3	OTHER TESTS AND NOTES
	30										
		0	50/0"	7							
305											
	35										Pushing rock/boulder from 35 to 39 feet Possible outwash below 35 to 36 feet
300								Refusal at 39 feet on boulder			
	40										
295											
	45										
290											
	50										
285											
	55										
280											
	60										
275											
	65										

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V6_GTBORING P:\0694019\00\FINALS\06940190 0.GPJ GEIV6_1.GDT 11/7/08

LOG OF BORING GEI-4 (continued)

GEOENGINEERS

Project: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-5
Sheet 2 of 2

Date(s) Drilled	05/29/08	Logged By	RV/CMK	Checked By	NLT
Drilling Contractor	Holocene Drilling	Drilling Method	Hollow-stem Auger	Sampling Methods	SPT
Auger Data	6-inch I.D., 10-inch O.D.	Hammer Data	140 lb hammer/30 in drop automatic	Drilling Equipment	CME 850
Total Exploration Depth (ft)	60	Ground Surface Elevation (ft)	336	Groundwater Elevation (ft)	326
Vertical Datum		Datum/ System		Easting(x): Northing(y):	1318182.824 242073.65

Elevation feet	Depth feet	SAMPLES Interval Recovered (in)	Blows/foot	Sub-Sample Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	WELL CONSTRUCTION
	0					SOD	5 inches sod			BAR 257; Steel surface monument
335						SM	Light brownish gray silty fine sand with gravel and occasional cobbles and boulders (dense, moist to wet) (weathered glacial till)			Concrete surface seal
		10	32	1						
	5									
330							Rough drilling			
							Grades to very dense glacial till			
		17	55	2			Rough drilling			
	10									
325										
		10	50/4"	3						
	15									
320										
							Grades to with thin lenses of fine to medium sand, wet			
		9	50/3"	4						Bentonite seal
	20									
315										
							Grades to silty fine sand			
		5½	50/5½"	5						
	25									
310										
		8	50/2"	6						2-inch Schedule 40 PVC well
	30									

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

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V6_GTWELL P:\0\0694019\00\FINALS\069401900.GPJ GEIV6_1.GDT 11/7/08

LOG OF BORING GEI-5

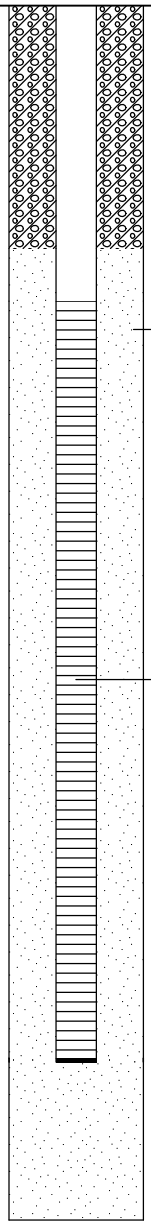
GEOENGINEERS

Project: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-6
Sheet 1 of 2

Elevation feet	Depth feet	Interval Recovered (in)	Blows/foot	Sub-Sample Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³
	30								
305						ML	Gray fine sandy silt with trace clay and gravel (hard, moist)		
		6	50/3"	7					
	35								
300									
		15	50/5"	8					
	40								
295						SP-SM	Gray fine to medium sand with silt and gravel (very dense, wet) (advance outwash) SA; %F = 7	18	
		18	80	9					
	45								
290						SP/SM	Interbedded gray fine sand and silty fine sand (very dense, wet)		
		18	80/5"	10					
	50								
285						SP	Gray fine to medium sand with lenses of silty fine sand (very dense, wet) %F = 9	17	
		18	92/5"	11					
	55								
280									
		18	72/10"	12					
	60								
275									
	65								
270									

WELL CONSTRUCTION



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V6_GTWELL P:\0\0694019\00\FINALS\069401900.GPJ GEIV6_1.GDT 11/7/08

LOG OF BORING GEI-5 (continued)

GEOENGINEERS

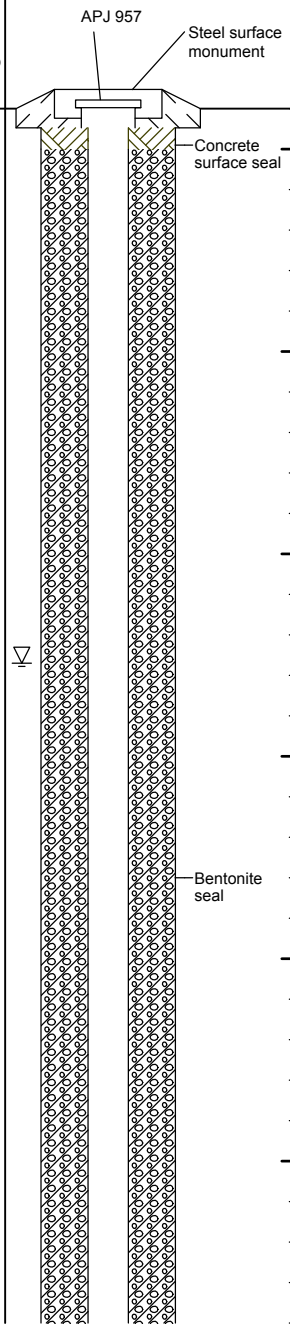
Project: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-6
Sheet 2 of 2

Date(s) Drilled	05/29/08	Logged By	JS	Checked By	NLT
Drilling Contractor	Geologic Drill	Drilling Method	Hollow-stem Auger	Sampling Methods	SPT
Auger Data	4¼-inch I.D., 8½-inch O.D.	Hammer Data	140 lb hammer/30 in drop rope and cathead	Drilling Equipment	Deep Rock XL
Total Exploration Depth (ft)	50.5	Ground Surface Elevation (ft)	341	Groundwater Elevation (ft)	327.3
Vertical Datum		Datum/ System		Easting(x): Northing(y):	1318151.365 241672.8845

Elevation feet	Depth feet	SAMPLES Interval Recovered (in)	Blows/foot	Sub-Sample Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³
	0					SOD	3 to 4 inches sod		
340						SM	Brownish gray silty fine to medium sand with gravel and occasional cobbles and boulders (medium dense, moist to wet) (weathered glacial till?)		
		18	14	1					
	5								
335									
						SM	Brownish gray silty fine sand with gravel and occasional cobbles and boulders (very dense, moist) (glacial till)		
		17	50/5"	2					
	10								
330							Rough drilling observed		
		16	50/4"	3			SA; %F = 28	12	
	15								
325							Grades to gray		
		6	50/6"	4					
	20								
320									
			50/5"	5					
	25						Rough drilling from 25 to 35 feet		
315									
		10	50/4"	6					
	30								

WELL CONSTRUCTION



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

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LOG OF BORING GEI-6

GEOENGINEERS

Project: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-7
Sheet 1 of 2

V6_GTWELL P:\0\0694019\00\FINALS\06940190 0.GPJ GEIV6_1.GDT 11/7/08

Elevation feet	Depth feet	SAMPLES: Interval Recovered (in)	Blows/foot	Sub-Sample Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft^3	WELL CONSTRUCTION
310	30									
		4	50/4"	7						
305	35						Pushing rock/boulder from 35 to 39 feet			2-inch Schedule 40 PVC well casing
		11	50/3"	8						
300	40									10/20 sand backfill
						SP	Gray fine to medium sand with trace silt (dense, wet) (advance outwash)			
		18	38	9			SA; %F = 2			
295	45						Approximately 8 to 10 feet of heave - well installed approximately 5 feet north of boring			2-inch Schedule 40 PVC screen, 0.010-inch slot width
		12	50/6"	10		SM/ML	Gray silty fine sand interbedded with fine sandy silt (very dense/hard, moist to wet) (older lacustrine deposit?)			
290	50									
285	55									
280	60									
275	65									

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V6_GTWELL P:\0\0694019\00\FINALS\069401900.GPJ GEIV6_1.GDT 11/7/08

LOG OF BORING GEI-6 (continued)

GEOENGINEERS

Project: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-7
Sheet 2 of 2

Date(s) Drilled	05/27/08 - 05/28/08	Logged By	CMK	Checked By	NLT
Drilling Contractor	Holocene Drilling	Drilling Method	Hollow-stem Auger	Sampling Methods	SPT
Auger Data	4½-inch I.D., 8-inch O.D.	Hammer Data	140 lb hammer/30 in drop automatic	Drilling Equipment	CME 850
Total Exploration Depth (ft)	51	Ground Surface Elevation (ft)	332	Groundwater Elevation (ft)	326
Vertical Datum		Datum/ System		Easting(x): Northing(y):	1318364.358 242603.2009

Elevation feet	Depth feet	SAMPLES Interval Recovered (in)	Blows/foot	Sub-Sample Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³
	0					SM	Brown silty fine sand with trace gravel (loose, moist) (topsoil/duff)		
330						SM	Brownish gray silty fine to medium sand with trace gravel and organic matter, cobbles and boulders (medium dense, moist) (weathered glacial till)		
	5	18	36	1		SM	Brownish gray silty fine sand with occasional gravel and organic matter, cobbles and boulders (medium dense to dense, moist) (glacial till)		
325							Rough drilling		
	10	16	82	2			Grades to dense to very dense		
320						ML	Gray sandy silt with occasional gravel, cobbles and boulders (hard, moist to wet)		
	15	15	95/11"	3				14	
315									
	20	5	50/5"	4					
310							Rough drilling Cobbles/boulders		
	25	5	50/4"	5					
305									
	30					SP-SM	Gray fine to medium sand with silt (very dense, wet) (advance outwash)		

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

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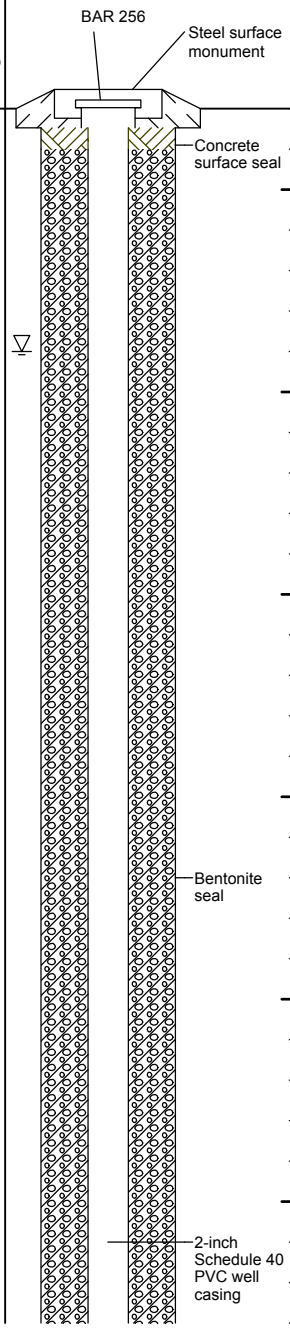
WELL CONSTRUCTION



LOG OF BORING GEI-7

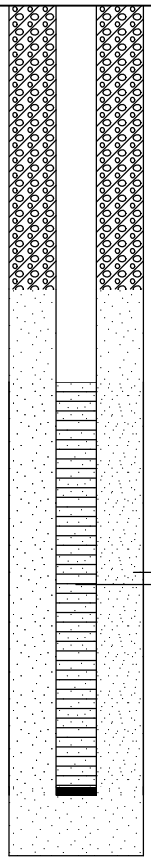
GEOENGINEERS

Project: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-8
Sheet 1 of 2

V6_GTWELL P:\0\0694019\00\FINALS\069401900.GPJ GEIV6_1.GDT 11/7/08

Elevation feet	Depth feet	SAMPLES: Interval Recovered (in)	Blows/foot	Sub-Sample Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft^3	WELL CONSTRUCTION
	30	4	50/6"	6			SA; %F = 14	19		
300						SP/SM	Interbedded gray fine sand with trace silt and silty fine sand (very dense, wet)			
	35	6	50/6"	7			Heave			
295										
	40		50/6"	8						
290										
	45		50/6"	9						
285										
	50		50/5"	10		SM	Gray silty fine sand (very dense, wet) (older lacustrine deposits?)			
280										
	55									
275										
	60									
270										
	65									
265										



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V6_GTWELL P:\0\0694019\00\FINALS\069401900.GPJ GEIV6_1.GDT 11/7/08

LOG OF BORING GEI-7 (continued)

GEOENGINEERS

Project: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-8
Sheet 2 of 2

Date(s) Drilled	05/28/08	Logged By	CMK	Checked By	NLT
Drilling Contractor	Holocene Drilling	Drilling Method	Hollow-stem Auger	Sampling Methods	SPT
Auger Data	4½-inch I.D., 8-inch O.D.	Hammer Data	140 lb hammer/30 in drop ??	Drilling Equipment	CME 850
Total Depth (ft)	51.5	Surface Elevation (ft)	333	Groundwater Elevation (ft)	322.4
Vertical Datum		Datum/ System		Easting(x): Northing(y):	1318361.199 242417.1972

Elevation feet	Depth feet	SAMPLES Interval	Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
	0							TS	Brown silty sand with organic matter (very loose to loose, moist) (topsoil/duff)			
330								SM	Brownish gray silty fine sand with occasional gravel, cobbles and boulders (medium dense, moist to wet) (weathered glacial till)			
	5		18	26	1							
325												
	10		18	45	2	▼			Grades to dense to very dense glacial till			
320												Rough drilling, gravel
	15		18	32	3				Grades to with thin lenses with sand and silt			
315												
	20		11	50/5"	4			SM/ML	Gray silty fine sand to sandy silt with trace of gravel and cobbles and boulders (very dense/hard, moist to wet)			
310												Cobbles
	25		4	50/4"	5							
305									Grades to moist with gravel and cobbles			
	30											

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

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LOG OF BORING GEI-8

GEOENGINEERS

Project: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-9
Sheet 1 of 2

V6_GTBORING P:\0694019\00\FINALS\069401900.GPJ GEIV6_1.GDT 11/7/08

Elevation feet	Depth feet	SAMPLES Interval Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft^3	OTHER TESTS AND NOTES
	30	3	50/3"	6							Hard drilling
300											
	35	5	50/5"	7							
295											
	40	10	50/4"	8							
290											
	45	18	89	9			SP	Gray fine to medium sand with silt and lenses of silty sand (very dense, wet) (advance outwash)	19		%F = 11
285											
	50	18	84	10							
280											
	55										
275											
	60										
270											
	65										

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V6_GTBORING P:\0\0694019\00\FINALS\069401900.GPJ GEIV6_1.GDT 11/7/08

LOG OF BORING GEI-8 (continued)

GEOENGINEERS

Project: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-9
Sheet 2 of 2

Date(s) Drilled	05/29/08	Logged By	JS	Checked By	NLT
Drilling Contractor	Geologic Drill	Drilling Method	Hollow-stem Auger	Sampling Methods	SPT
Auger Data	4¼-inch I.D., 8½-inch O.D.	Hammer Data	140 lb hammer/30 in drop rope and cathead	Drilling Equipment	Deep Rock XL
Total Depth (ft)	49.5	Surface Elevation (ft)	326	Groundwater Elevation (ft)	282.5
Vertical Datum		Datum/ System		Easting(x): Northing(y):	1318521.613 241703.0779

Elevation feet	Depth feet	SAMPLES Interval	Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
	0							SOD	3 to 4 inches sod			
325								SM/ML	Light brownish gray silty fine sand to fine sandy silt with trace of gravel and cobbles and boulders (dense/hard, moist) (weathered glacial till)			
			0	60	1							
	5											
320												
			18	58	2					12		
	10											
315									Grades to gray with lenses of fine sand			
			10	50/4"	3							
	15											
310									Grades to fine sandy silt			
			2	50/2"	4							
	20											
305												
			5	50/5"	5							
	25											
300												
			11	50/5"	6							Rough drilling
	30											

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

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LOG OF BORING GEI-9

GEOENGINEERS

Project: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-10
Sheet 1 of 2

V6_GTBORING P:\0694019\00\FINALS\06940190O.GPJ GEIV6_1.GDT 11/7/08

Elevation feet	Depth feet	SAMPLES: Interval Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
295	30										
		5	50/5"	7							
290	35						SP/OL	Gray fine sand interbedded with organic silt (very dense/hard, moist) (older lacustrine deposit?)			
		18	58	8							
285	40						ML	Gray silt (hard, moist) (older lacustrine deposit)			
		18	50	9	▼						
280	45							Grades to fine sandy silt			
		18	50/5"	10							
275	50										
270	55										
265	60										
260	65										

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V6_GTBORING P:\0\0694019\00\FINALS\069401900.GPJ GEIV6_1.GDT 11/7/08

LOG OF BORING GEI-9 (continued)

GEOENGINEERS

Project: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-10
Sheet 2 of 2

Date(s) Drilled	05/29/08	Logged By	JS	Checked By	NLT
Drilling Contractor	Geologic Drill	Drilling Method	Hollow-stem Auger	Sampling Methods	SPT
Auger Data	3¼-inch I.D., 7¼-inch O.D.	Hammer Data	140 lb hammer/30 in drop rope and cathead	Drilling Equipment	Deep Rock XL
Total Depth (ft)	49.5	Surface Elevation (ft)	316	Groundwater Elevation (ft)	Not Encountered
Vertical Datum		Datum/ System		Easting(x): Northing(y):	1318674.848 242017.8693

Elevation feet	Depth feet	SAMPLES Interval Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
	0						SOD	2 to 3 inches sod			
315							SM/ML	Light brown silty fine to medium sand to fine sandy silt with gravel and occasional cobbles and boulders (medium dense, wet) (weathered glacial till)			
		18	9	1					18		%F = 53
	5										
310											
		18	28	2							
	10										
305								Grades to gray, moist, very dense glacial till			
		16	50/4"	3							
	15										
300							ML	Gray fine sandy silt (hard, moist)			
		9	50/3"	4							
	20										
295											
		11	50/5"	5							
	25										Slow drilling, cobbles?
290											
		1	50/1"	6							
	30										

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

DRAFT

LOG OF BORING GEI-10

GEOENGINEERS

Project: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-11
Sheet 1 of 2

V6_GTBORING P:\0\0694019\00\FINALS\06940190O.GPJ GEIV6_1.GDT 11/7/08

Elevation feet	Depth feet	SAMPLES Interval Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft^3	OTHER TESTS AND NOTES
	30										
285							SM	Gray silty fine sand with trace gravel, cobbles and boulders (very dense, moist)			
		5	50/5"	7							
	35						ML	Gray silt (hard, moist) (older lacustrine deposit?)			
280											
		18	36	8							
	40										
275											
		18	62	9							
	45										
270											
		18	40	10				Grades to light brown fine sandy silt with trace organic matter			
	50										
265											
	55										
260											
	60										
255											
	65										
250											

DRAFT

V6_GTBORING P:\0\0694019\00\FINALS\069401900.GPJ GEIV6_1.GDT 11/7/08

LOG OF BORING GEI-10 (continued)

GEOENGINEERS

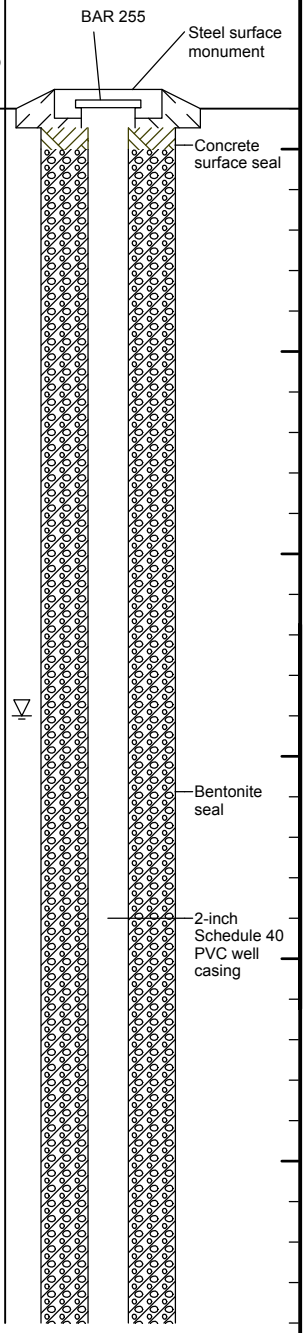
Project: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-11
Sheet 2 of 2

Date(s) Drilled	05/27/08	Logged By	CMK	Checked By	NLT
Drilling Contractor	Holocene Drilling	Drilling Method	Hollow-stem Auger	Sampling Methods	SPT
Auger Data	4½-inch I.D., 8-inch O.D.	Hammer Data	140 lb hammer/30 in drop automatic	Drilling Equipment	CME 850
Total Exploration Depth (ft)	46.5	Ground Surface Elevation (ft)	301	Groundwater Elevation (ft)	286
Vertical Datum		Datum/ System		Easting(x): Northing(y):	1318781.765 242508.5601

Elevation feet	Depth feet	SAMPLES Interval Recovered (in)	Blows/foot	Sub-Sample Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³
	0					TS	Dark brown silty sand with organic matter (very loose to loose, moist) (topsoil/forest duff)		
300						SM	Brown with orange staining silty fine sand with trace gravel and with cobbles and boulders (medium dense, moist) (weathered glacial till)		
		15	13	1			Boulder	84	
	5	18	61	2		SM	Gray silty fine sand with gravel, cobbles and boulders (very dense, moist) (glacial till)	12	
295							%F = 70		
	10	9	50/3"	3			Cobbles	10	
290							%F = 62		
	15	10	50/4"	4					
285									
	20	4	50/4"	5					
280									
	25	9	50/3"	6			Grades sandier		
275									
	30								

WELL CONSTRUCTION



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

DRAFT

LOG OF BORING GEI-11

GEOENGINEERS

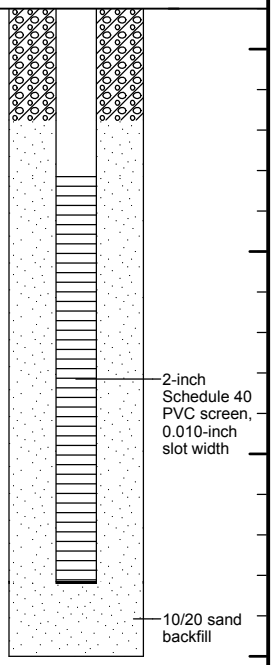
Project: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-12
Sheet 1 of 2

V6_GTWELL P:\0\0694019\00\FINALS\069401900.GPJ GEIV6_1.GDT 11/7/08

Elevation feet	Depth feet	SAMPLES: Interval Recovered (in)	Blows/foot	Sub-Sample Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³
	30	5	50/5"	7					
270									
	35	6	50/5½"	8					
265							Grades siltier		
	40	18	49*	9		SP	%F = 5	13	
260							Dark gray fine to coarse sand (dense, wet) (advance outwash?)		
							*Blowcount may not be representative due to heave		
						SP-SM	Brownish gray fine to coarse sand with silt and organic matter (wood/tree chunks) (very dense, wet)		
	45	4	50/5"	10					
255									
	50								
250									
	55								
245									
	60								
240									
	65								
235									

WELL CONSTRUCTION



DRAFT

LOG OF BORING GEI-11 (continued)

GEOENGINEERS

Project: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-12
Sheet 2 of 2

V6_GTWELL P:\0\0694019\00\FINALS\069401900.GPJ GEIV6_1.GDT 11/7/08

Date Excavated: 09/11/08

Logged by: SH

Equipment: Rubber-Tired Backhoe

Surface Elevation (ft): 343

Elevation feet	Depth feet	Sample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
	0				SOD	3 inches sod		
					SM	Brown silty fine to medium sand with occasional gravel (medium dense, moist) (fill)		
			1					
			2		SM	Gray silty fine to medium sand with occasional gravel (dense to very dense, moist) (glacial till)		
340								Probed <1 inch at 3 feet
						Grades to very dense at approximately 3½ feet		
						Test pit completed at 4 feet No groundwater seepage observed No caving observed		
	5							
335								
	10							

DRAFT

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

LOG OF TEST PIT TP-1

GEOENGINEERS

Project: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-13
Sheet 1 of 1

V6_GTTPIT P:\0\0694019\00\FINALS\069401900.GPJ GEIV6_1.GDT 11/7/08

Date Excavated: 09/11/08

Logged by: SH

Equipment: Rubber-Tired Backhoe

Surface Elevation (ft): 343

Elevation feet	Depth feet	Sample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
	0				SOD	4 inches sod		
					SM	Brown silty fine to medium sand with occasional gravel (medium dense, moist) (weathered till/fill?)		
		X	1		ML	Gray silt with variable amounts of fine sand with occasional gravel (stiff to very stiff, moist) (weathered glacial till)		
340								
	5	X	2					
						Grades to hard at approximately 7 feet		
335		X	3					
						Test pit completed at 8½ feet No groundwater seepage observed No caving observed		
	10							

DRAFT

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: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-14
Sheet 1 of 1

V6_GTTPIT P:\0\0694019\00\FINALS\06940190 0.GPJ GEIV6_1.GDT 11/7/08

Date Excavated: 09/11/08

Logged by: SH

Equipment: Rubber-Tired Backhoe

Surface Elevation (ft): 343

Elevation feet	Depth feet	Sample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
	0				SOD	3 inches sod		
					SM	Brown silty fine to medium sand with occasional gravel (medium dense, moist) (weathered till?)		
					ML	Gray silt with variable amounts of fine sand and occasional gravel (stiff to very stiff, moist) (weathered glacial till)		
340								
	5					Grades to hard at 5 feet		
						Test pit completed at 6 feet No groundwater seepage observed No caving observed		
335								
	10							

DRAFT

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

LOG OF TEST PIT TP-3

GEOENGINEERS

Project: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-15
Sheet 1 of 1

V6_GTTPIT P:\0\0694019\00\FINALS\069401900.GPJ GEIV6_1.GDT 11/7/08

Date Excavated: 09/11/08

Logged by: SH

Equipment: Rubber-Tired Backhoe

Surface Elevation (ft): 343

Elevation feet	Depth feet	Sample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
	0				SOD	3 inches sod		
					SM	Brown silty fine to medium sand with occasional gravel (medium dense, moist) (weathered till?)		
					SM	Gray silty fine to medium sand with gravel and occasional boulder (dense, moist) (glacial till)		
340						Grades to very dense at a depth of approximately 3 feet		
	5	X	1					
						Test pit completed at 5½ feet No groundwater seepage observed No caving observed		
335								
	10							

DRAFT

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: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-16
Sheet 1 of 1

V6_GTTPIT P:\0\0694019\00\FINALS\069401900.GPJ GEIV6_1.GDT 11/7/08

Date Excavated: 09/11/08

Logged by: SH

Equipment: Rubber-Tired Backhoe

Surface Elevation (ft): 341

Elevation feet	Depth feet	Sample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
	0				SOD	3 inches sod		
					SM	Brown silty fine to medium sand with occasional gravel and fabric debris (medium dense, moist) (fill)		
340								
					SM	Gray silty fine to medium sand with gravel (dense, moist) (glacial till)		
	5							
335								
		X	1			Grades to very dense at a depth of approximately 7 feet		
						Test pit completed at 8 feet No groundwater seepage observed No caving observed		
	10							

DRAFT

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

LOG OF TEST PIT TP-5

GEOENGINEERS

Project: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-17
Sheet 1 of 1

V6_GTTPIT P:\0\0694019\00\FINALS\06940190 0.GPJ GEIV6_1.GDT 11/7/08

Date Excavated: 09/11/08

Logged by: SH

Equipment: Rubber-Tired Backhoe

Surface Elevation (ft): 336

Elevation feet	Depth feet	Sample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
	0				SOD	3 inches sod		
					SM	Brown silty fine to medium sand with occasional gravel (medium dense, moist) (weathered till/fill?)		
335								
					SM	Gray silty fine to medium sand with gravel (dense, moist)		
						Grades to very dense at a depth of approximately 3 feet		
	5					Test pit completed at 5 feet No groundwater seepage observed No caving observed		
330								
	10							

DRAFT

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: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-18
Sheet 1 of 1

V6_GTTPIT P:\0\0694019\00\FINALS\069401900.GPJ GEIV6_1.GDT 11/7/08

Date Excavated: 09/11/08

Logged by: SH

Equipment: Rubber-Tired Backhoe

Surface Elevation (ft): 343

Elevation feet	Depth feet	Sample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
	0				SOD	3 to 4 inches sod		
					SM	Brown silty fine to medium sand with occasional gravel (medium dense, moist) (weathered till?)		
340					SM	Gray silty fine to medium sand with gravel (dense to very dense, moist) (glacial till)		
	5							
		X	1					
						Test pit completed at 6 feet No groundwater seepage observed No caving observed		
335								
	10							

DRAFT

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: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-19
Sheet 1 of 1

V6_GTTPIT P:\0\0694019\00\FINALS\069401900.GPJ GEIV6_1.GDT 11/7/08

Date Excavated: 09/11/08

Logged by: SH

Equipment: Rubber-Tired Backhoe

Surface Elevation (ft): 342

Elevation feet	Depth feet	Sample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
	0				SOD	3 to 4 inches sod		
					SM	Brown silty fine to medium sand with occasional gravel (medium dense, moist) (weathered till)		
					SM	Gray silty fine to medium sand with gravel (dense to very dense, moist) (glacial till)		
340								
						Grades to very dense		
	5							
						Test pit completed at 6½ feet No groundwater seepage observed No caving observed		
335								
	10							

DRAFT

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: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-20
Sheet 1 of 1

V6_GTTPIT P:\0\0694019\00\FINALS\069401900.GPJ GEIV6_1.GDT 11/7/08

Date Excavated: 09/11/08

Logged by: SH

Equipment: Rubber-Tired Backhoe

Surface Elevation (ft): 343

Elevation feet	Depth feet	Sample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
	0				SOD	3 to 4 inches sod		
					SM	Brown silty fine to medium sand with occasional gravel (medium dense, moist) (weathered till)		
					SM	Gray silty fine to medium sand with gravel (dense, moist) (glacial till)		
						6 inch lense of fine to medium gravel with silt and sand at 2½ feet		
340		X	1					
						Grades to very dense		
	5							
335								
						Test pit completed at 8½ feet No groundwater seepage observed No caving observed		
	10							

DRAFT

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: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-21
Sheet 1 of 1

V6_GTTPIT P:\0\0694019\00\FINALS\06940190 0.GPJ GEIV6_1.GDT 11/7/08

Date Excavated: 09/11/08

Logged by: SH

Equipment: Rubber-Tired Backhoe

Surface Elevation (ft): 344

Elevation feet	Depth feet	Sample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
	0				SOD	3 to 4 inches sod		
					SM	Brown silty fine to medium sand with occasional gravel (medium dense, moist) (weathered till)		
					SM	Gray silty fine to medium sand with gravel (dense to very dense, moist) (glacial till)		
340								
	5							
		X	1					
335						Test pit completed at 9 feet No groundwater seepage observed No caving observed		
	10							

DRAFT

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: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-22
Sheet 1 of 1

V6_GTTPIT P:\0\0694019\00\FINALS\069401900.GPJ GEIV6_1.GDT 11/7/08

Date Excavated: 09/11/08

Logged by: SH

Equipment: Rubber-Tired Backhoe

Surface Elevation (ft): 342

Elevation feet	Depth feet	Sample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
	0				SOD	3 to 4 inches sod		
					SM	Brown silty fine to medium sand with occasional gravel (medium dense, moist) (weathered till/fill?)		
340					SM	Gray silty fine to medium sand with gravel (dense to very dense, moist) (glacial till)		
						Grades to very dense at a depth of approximately 3 feet		
	5					Test pit completed at 5½ feet No groundwater seepage observed No caving observed		
335								
	10							

DRAFT

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: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-23
Sheet 1 of 1

V6_GTTPIT P:\0\0694019\00\FINALS\06940190 0.GPJ GEIV6_1.GDT 11/7/08

Date Excavated: 09/11/08

Logged by: SH

Equipment: Rubber-Tired Backhoe

Surface Elevation (ft): 344

Elevation feet	Depth feet	Sample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
	0				SOD	3 to 4 inches sod		
					SM	Brown silty fine to medium sand with occasional gravel (medium dense, moist) (fill?)		
					SM	Gray silty fine to medium sand with gravel (dense to very dense, moist) (glacial till)		
340						Very dense below a depth of approximately 4 feet		
	5							
						Test pit completed at 7 feet No groundwater seepage observed No caving observed		
335								
	10							

DRAFT

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: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-24
Sheet 1 of 1

V6_GTTPIT P:\0\0694019\00\FINALS\06940190 0.GPJ GEIV6_1.GDT 11/7/08

Date Excavated: 09/11/08

Logged by: SH

Equipment: Rubber-Tired Backhoe

Surface Elevation (ft): 344

Elevation feet	Depth feet	Sample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
	0				SOD	3 inches sod		
					SM	Brown silty fine to medium sand with occasional gravel (medium dense, moist) (weathered till?)		
					SM	Gray silty fine to medium sand with gravel (dense to very dense, moist) (glacial till)		
						Grades to very dense below a depth of 3 feet		
340								
	5					Test pit completed at 5 feet No groundwater seepage observed No caving observed		
335								
	10							

DRAFT

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: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-25
Sheet 1 of 1

V6_GTTPIT P:\0\0694019\00\FINALS\06940190 0.GPJ GEIV6_1.GDT 11/7/08

Date Excavated: 09/11/08

Logged by: SH

Equipment: Rubber-Tired Backhoe

Surface Elevation (ft): 343

Elevation feet	Depth feet	Sample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
	0				SOD	3 to 4 inches sod		
					SM	Brown silty fine to medium sand with occasional gravel (medium dense, moist) (weathered till?)		
					SM	Gray silty fine to medium sand with gravel (dense to very dense, moist) (glacial till)		
340						Grades to very dense below a depth of approximately 3 feet		
	5					Test pit completed at 5 feet No groundwater seepage observed No caving observed		
335								
	10							

DRAFT

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.

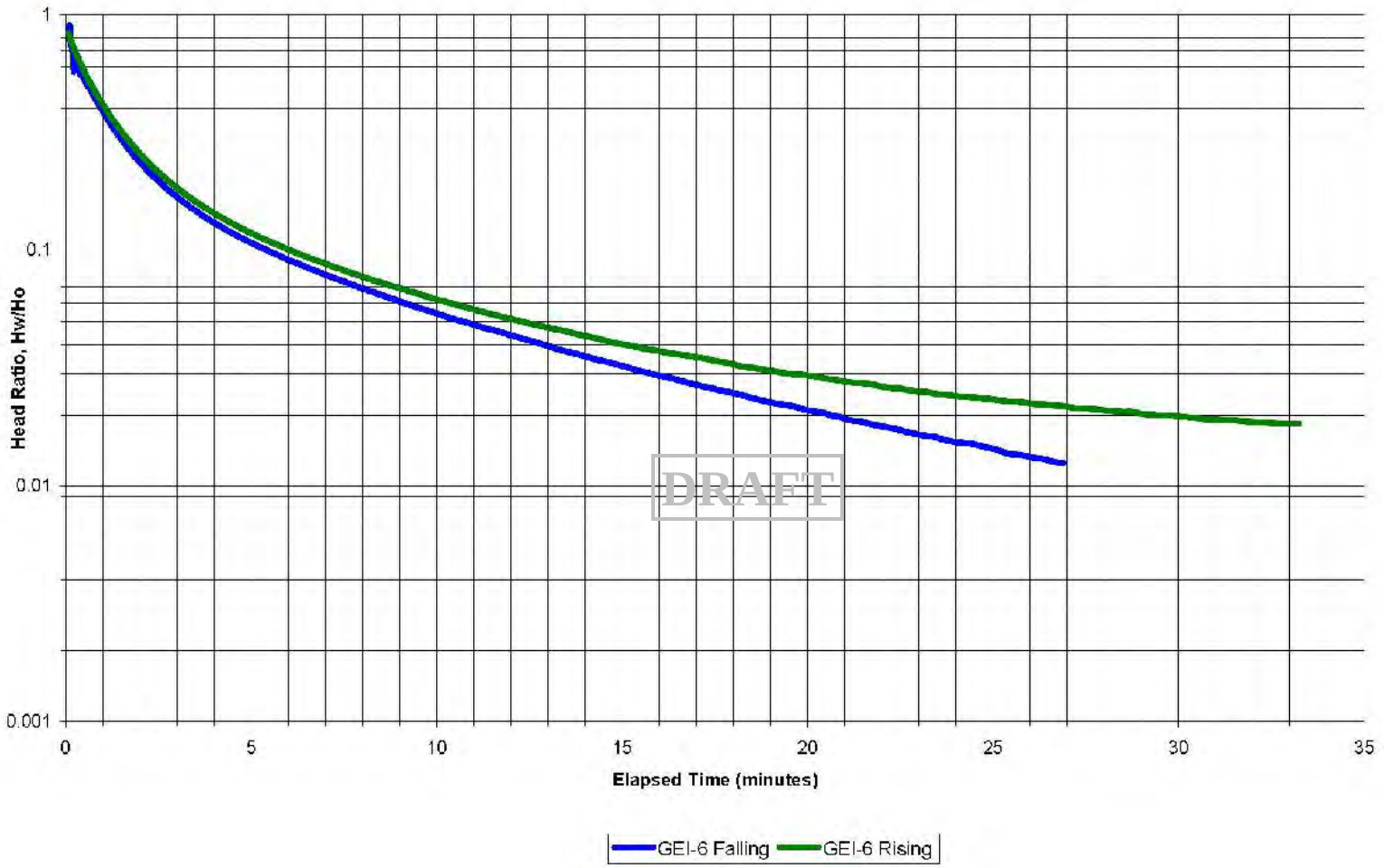
LOG OF TEST PIT TP-14

GEOENGINEERS

Project: Microsoft RedWest South
Project Location: Redmond, Washington
Project Number: 0694-019-00

Figure A-26
Sheet 1 of 1

V6_GTTPIT P:\0\0694019\00\FINALS\069401900.GPJ GEIV6_1.GDT 11/7/08



Aquifer Slug Test at GEI-6

Microsoft RedWest South
Redmond, Washington

GEOENGINEERS

Figure A-27

WELL ID: GEI-6

Local ID:	GEI-6 In
Date:	6/4/2008
Time:	8:44
Fraction:	1.00

INPUT

Construction:		
Casing dia. (d_c)	2	Inch
Annulus dia. (d_w)	5	Inch
Depths to:		
water level (DTW)	13.31	Feet
Top of Aquifer	40	Feet
Base of Aquifer	50	Feet
Annular Fill:		
across screen --	Medium Sand	
above screen --	Bentonite	
Aquifer Material --	Advance Outwash	

COMPUTED

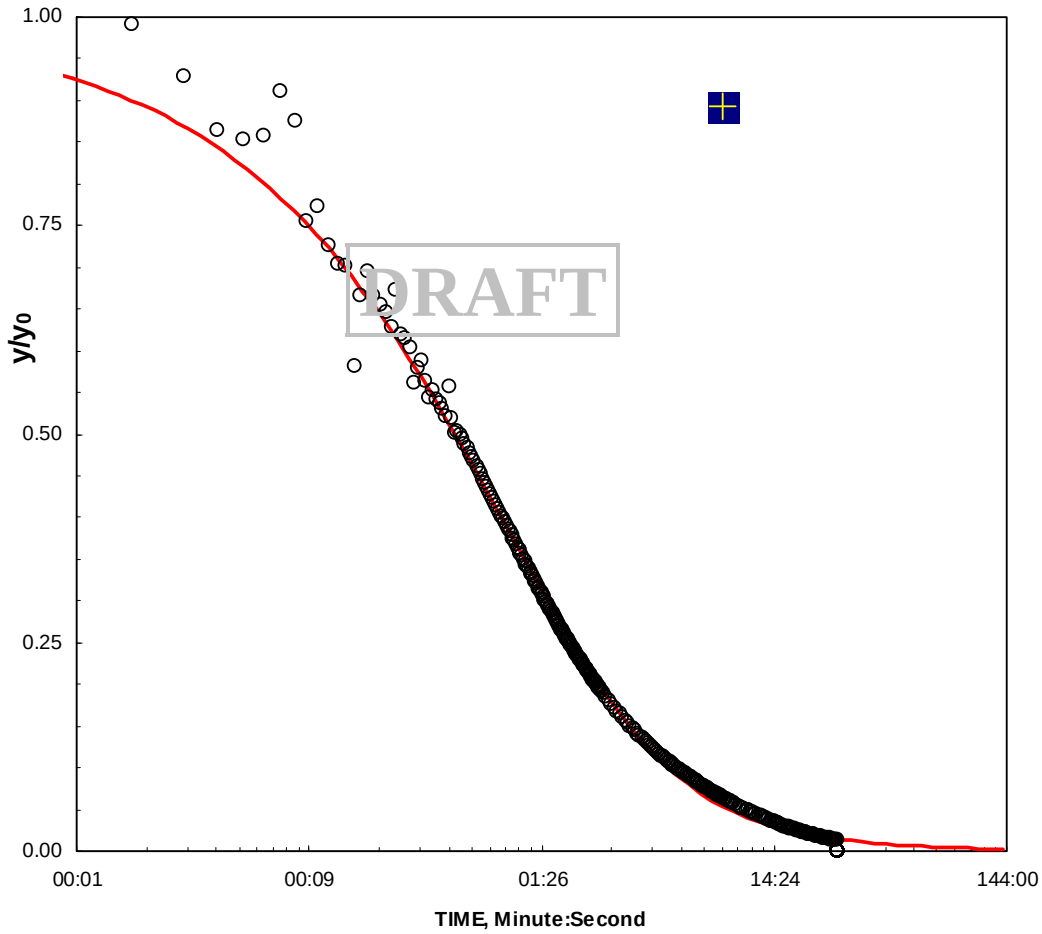
Aquifer Thickness	10	Feet
$y_{0\text{-DISPLACEMENT}}$ =	6.40	Feet
$y_{0\text{-SLUG}}$ =	7.61	Feet

Input is consistent.

S > 10 X Ss of water * thickness.

T =	7.2	Feet²/Day
K =	0.72	Feet/Day
S =	0.0136182	d'less

Adjust position of the match point (✚) to estimate T and S
Left to right changes T. Up and down changes S



Slug Test Analysis; Falling Head, GEI-6

Microsoft RedWest South
Redmond, Washington

GEOENGINEERS

Figure A-28

P:\0\0694019\00\CAD\069401900FA-28.DWG\TAB:A-28 MODIFIED BY LKNOWLTON ON Nov 11, 2008 - 13:40

WELL ID: GEI-6

Local ID:	GEI-6 Out
Date:	6/4/2008
Time:	9:15
Fraction:	1.00

INPUT

Construction:		
Casing dia. (d_c)	2	Inch
Annulus dia. (d_w)	5	Inch
Depths to:		
water level (DTW)	13.31	Feet
Top of Aquifer	40	Feet
Base of Aquifer	50	Feet
Annular Fill:		
across screen --	Medium Sand	
above screen --	Bentonite	
Aquifer Material --	Advance Outwash	

COMPUTED

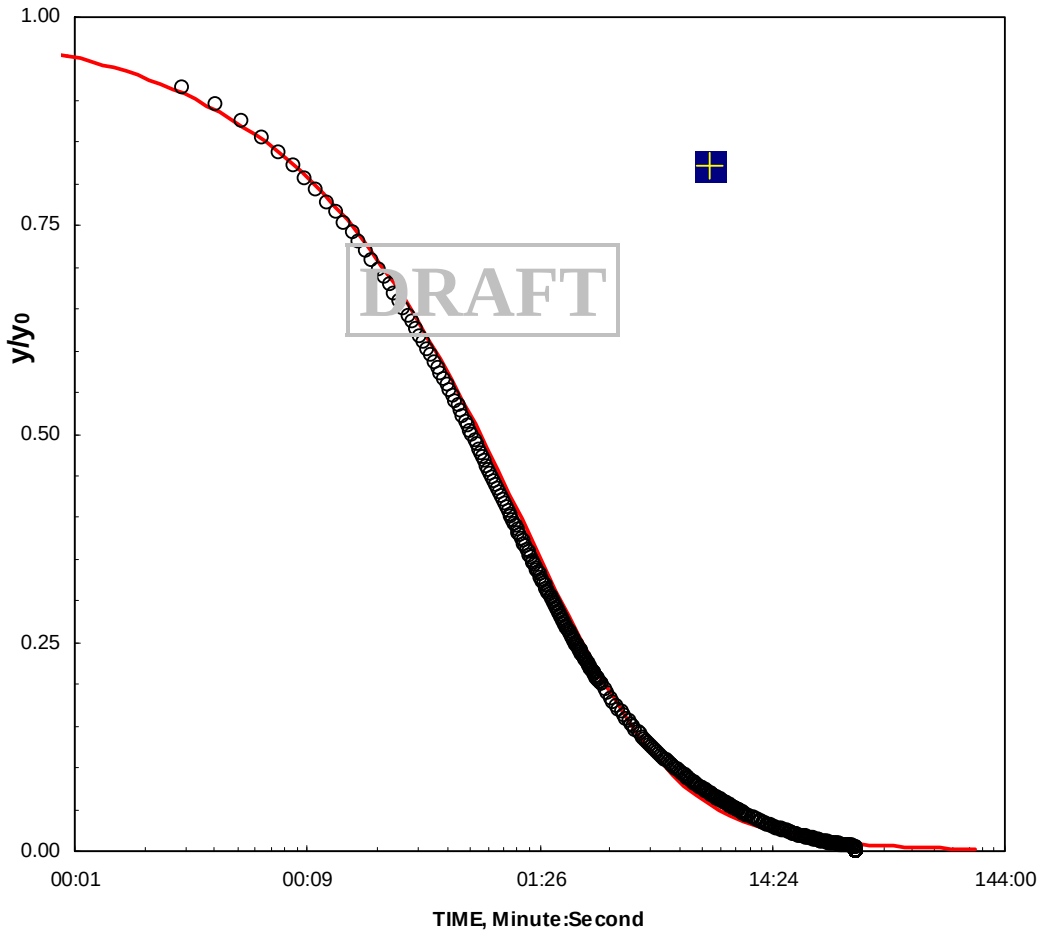
Aquifer Thickness	10	Feet
$y_{0\text{-DISPLACEMENT}}$ =	6.20	Feet
$y_{0\text{-SLUG}}$ =	7.61	Feet

Input is consistent.

S > 10 X Ss of water * thickness.

T =	9.4	Feet²/Day
K =	0.94	Feet/Day
S =	0.0026553	d'less

Adjust position of the match point (✚) to estimate T and S
Left to right changes T. Up and down changes S



Slug Test Analysis; Rising Head, GEI-6

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Figure A-29

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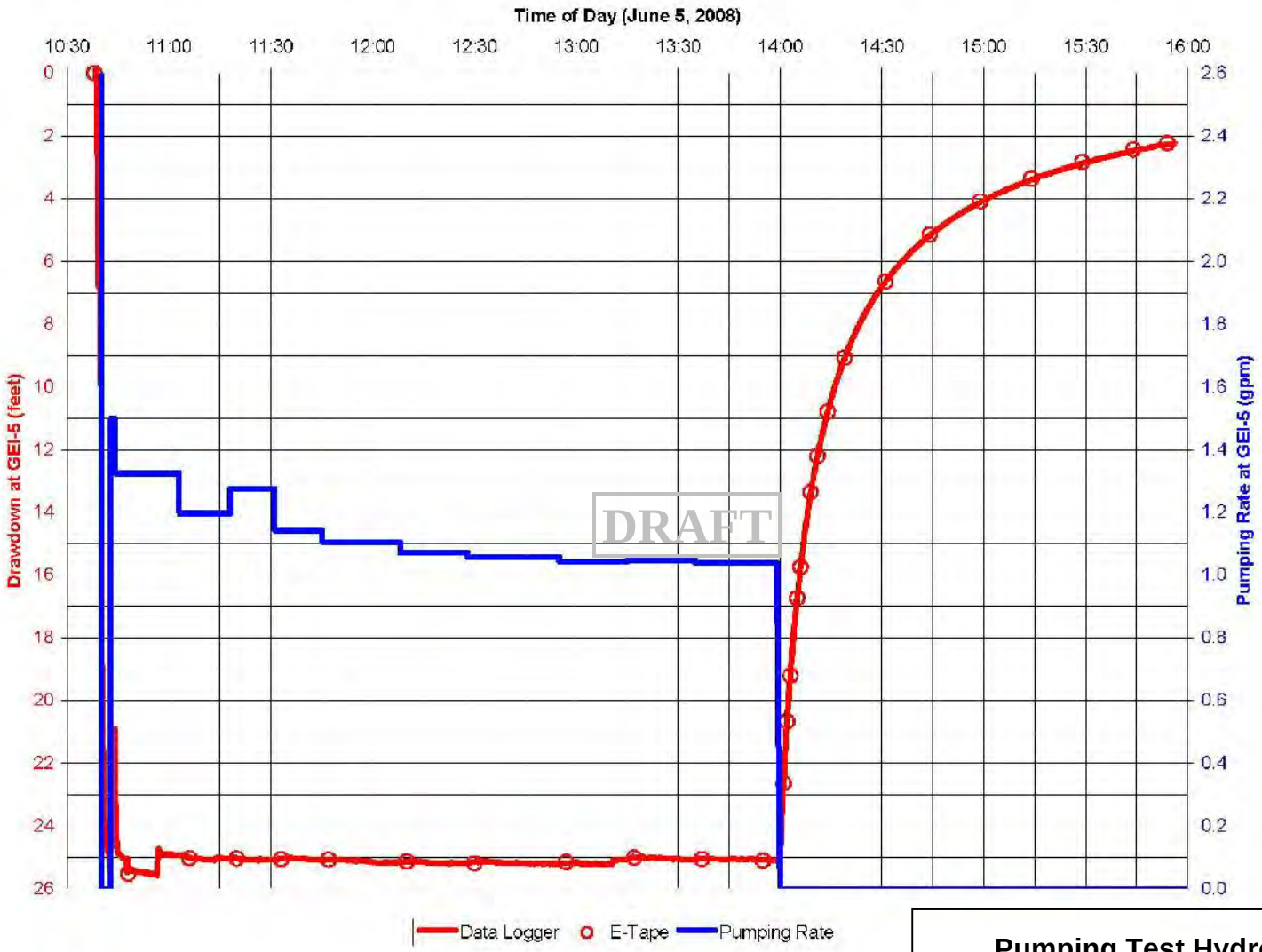

Pumping Test Hydrograph, GEI-5

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Figure A-30

Observation Well Hydrograph, B-3

Microsoft RedWest South
Redmond, Washington

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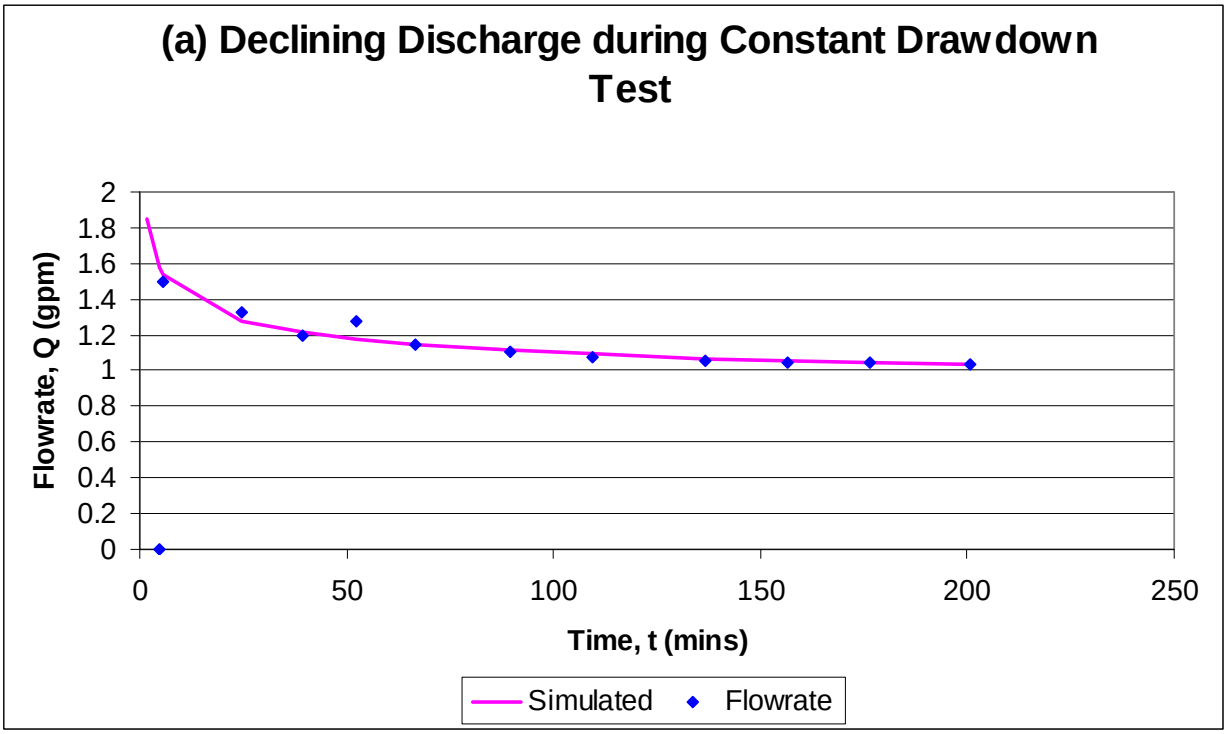
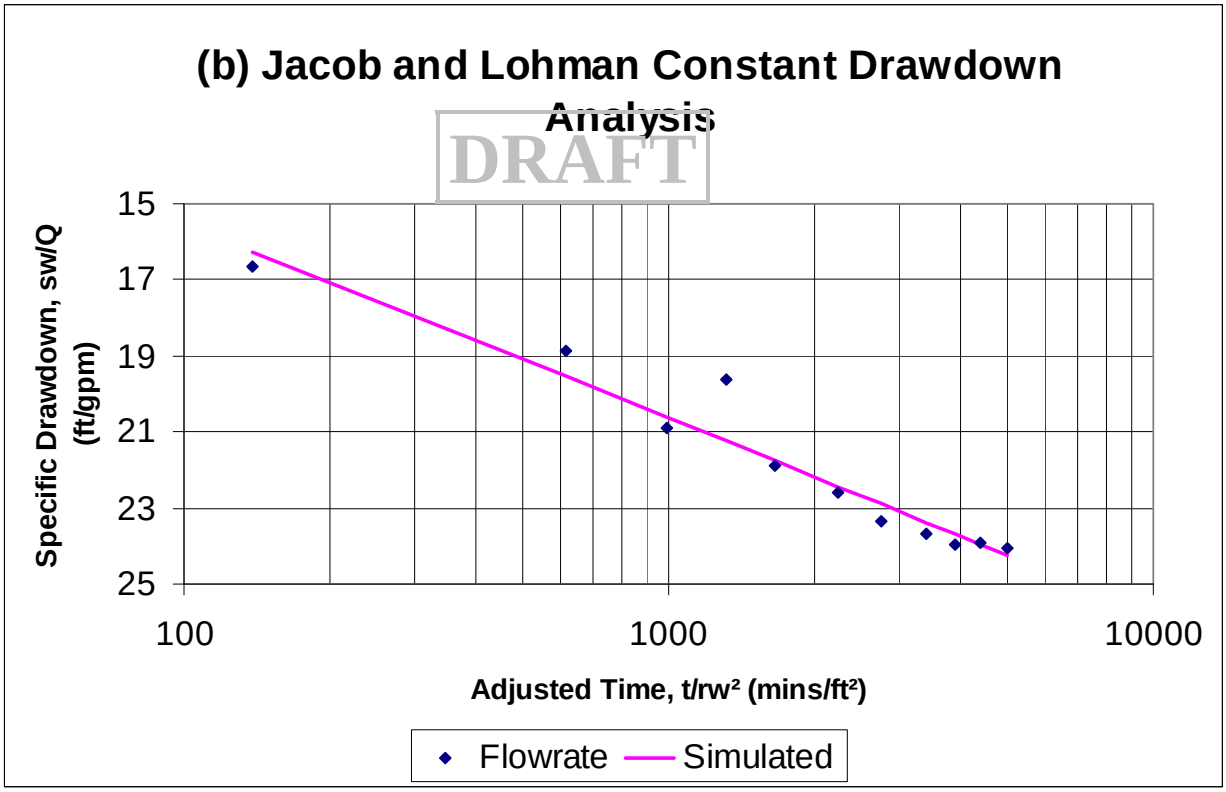
Figure A-31





Constant Drawdown Analysis, GEI-5	
Microsoft RedWest South Redmond, Washington	
GeoEngineers	Figure A-32

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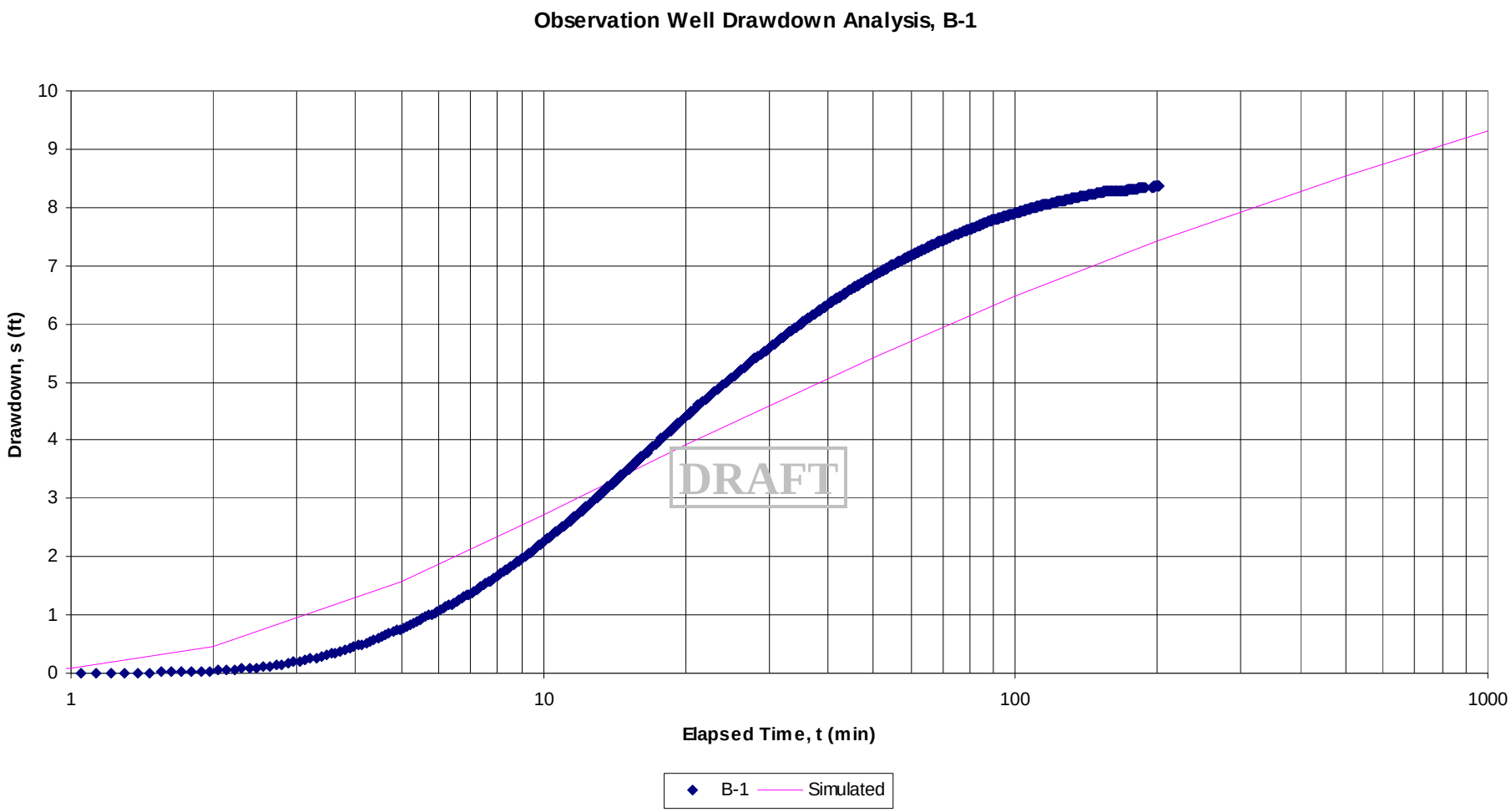

Observation Well Drawdown Analysis, B-1

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Figure A-33

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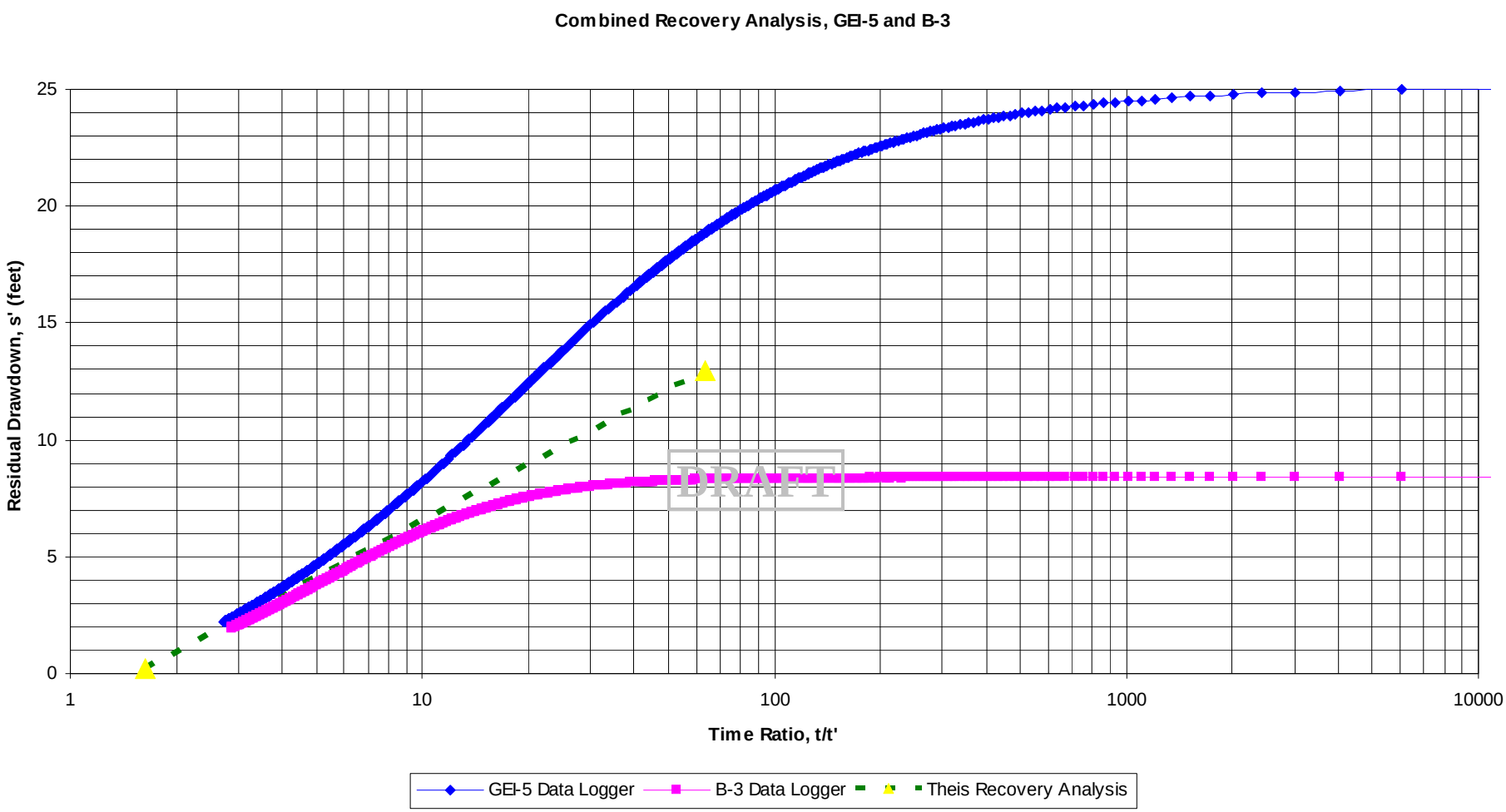

Combined Recovery Analysis, GEI-5 and B-3

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Figure A-34

APPENDIX B

LABORATORY TESTING

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APPENDIX B
LABORATORY TESTING

GENERAL

Soil samples obtained from the explorations were transported to our laboratory and examined to confirm or modify field classifications, as well as to evaluate engineering properties of the soil samples. Representative samples were selected for laboratory testing consisting of moisture content determinations and sieve analysis. The results of the sieve analysis are presented in Figures B-1 through B-5. The results of the moisture content determinations are presented on the exploration logs at the respective sample depth in Appendix A.

MOISTURE CONTENT TESTING

Moisture content tests were completed in general accordance with American Society for Testing and Materials (ASTM) D-2216 for representative samples obtained from the explorations. The results of these tests are presented on the exploration logs in Appendix A at the depths at which the samples were obtained.

SIEVE ANALYSES

Sieve analyses were performed on selected samples in general accordance with ASTM D-422. The wet sieve analysis method was used to determine the percentage of soil greater than the U.S. No. 200 mesh sieve. The results of the sieve analyses were plotted, were classified in general accordance with the Unified Soil Classification System (USCS), and are presented in Figures B-1 and B-2.

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PERCENT PASSING U.S. NO. 200 SIEVE

Selected samples were "washed" through the U.S. No. 200 mesh sieve to determine the relative percentages of coarse- and fine-grained particles in the soil. The percent passing value represents the percentage by weight of the sample finer than the U.S. No. 200 sieve. These tests were conducted to verify field descriptions and to determine the fines content for analysis purposes. The tests were conducted in general accordance with ASTM D-1140, and the results are shown on the exploration logs at the respective sample depths.

MAXIMUM DRY DENSITY

One maximum dry density (MDD) determination was completed in general accordance with ASTM D 1557 from a bulk sample from the west side of the site. The sample was obtained from the weathered glacial till at a depth of about 2 to 3 feet. Results of the MDD determination are summarized in Figure B-3.

0694-019-00 NLT : SAW : saw 6-06-08 (Sieve.ppt)

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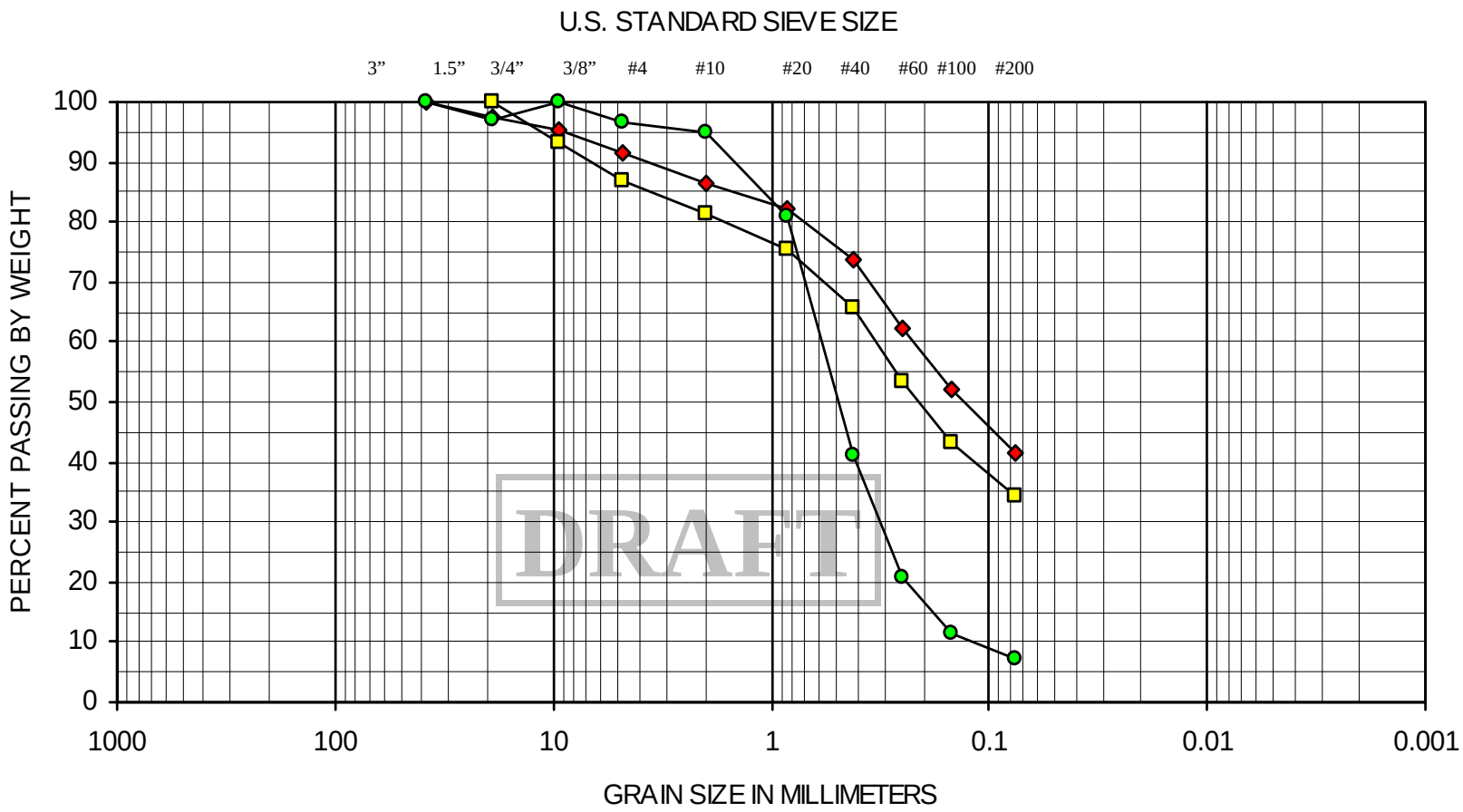

COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

SYMBOL	EXPLORATION NUMBER	DEPTH (ft)	SOIL CLASSIFICATION
◆	GEI-1	9.0	Gray silty sand (SM)
□	GEI-4	14.0	Gray silty sand (SM)
●	GEI-5	44.0	Gray fine to medium sand with silt (SP-SM)

SIEVE ANALYSIS RESULTS

FIGURE B-1

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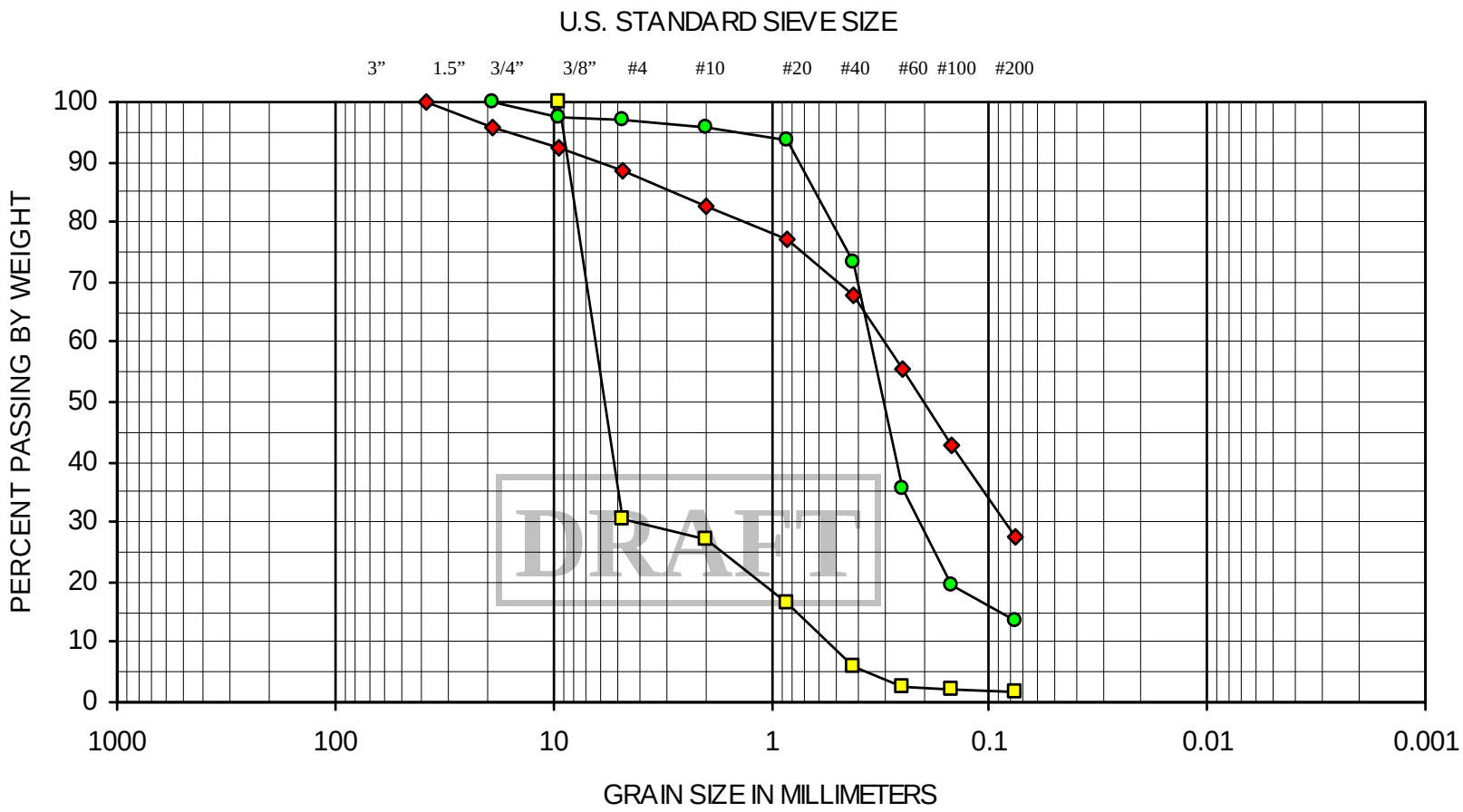
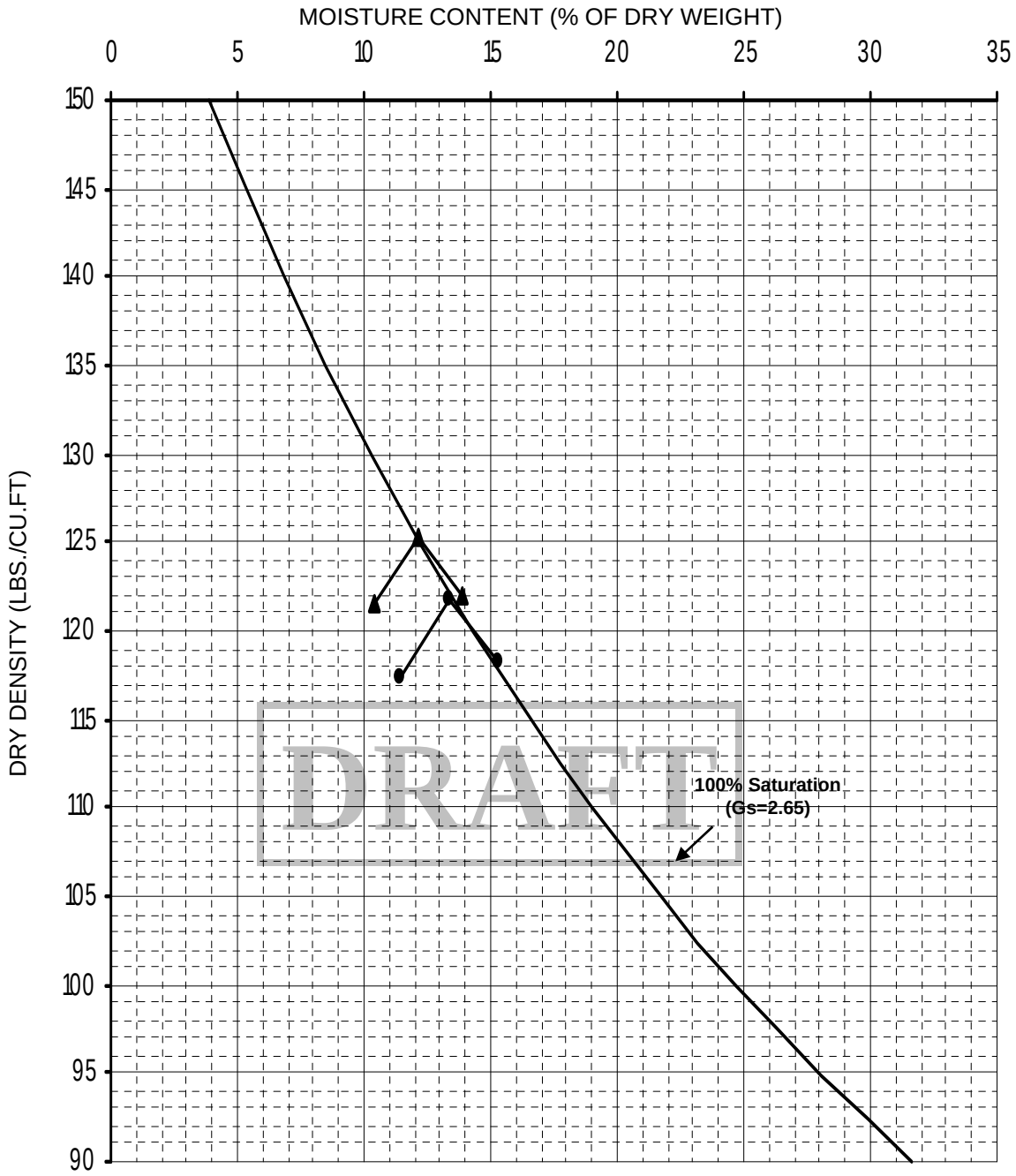


COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

SYMBOL	EXPLORATION NUMBER	DEPTH (ft)	SOIL CLASSIFICATION
◆	GEI-6	14.0	Gray silty sand (SM)
□	GEI-6	44.0	Gray fine sand (SP)
●	GEI-7	30.0	Gray fine to medium sand with silt (SP-SM)

SIEVE ANALYSIS RESULTS

FIGURE B-2



BORING NUMBER	SAMPLE NUMBER	SOIL CLASSIFICATION	OPTIMUM MOISTURE CONTENT	MAXIMUM DRY DENSITY (LBS/FT^3)
	Composite	Brown silty sand with gravel (sm)	12.1	125.4

NOTE: Tests performed in accordance with ASTM D-1557

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COMPACTION TEST RESULTS

FIGURE B-3

0694-019-00 : NLT : SAW :cts : 6-18-08 (MDD.ppt)

APPENDIX C

GROUND ANCHOR LOAD TESTS AND SHORING MONITORING PROGRAM

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APPENDIX C
GROUND ANCHOR LOAD TESTS AND SHORING MONITORING PROGRAM

GROUND ANCHOR LOAD TESTING

General

The locations of the load tests shall be approved by the Engineer and shall be representative of the field conditions. Load tests shall not be performed until the nail/tieback grout and shotcrete wall facing, where present, have attained at least 50 percent of the specified 28-day compressive strengths.

Where temporary casing of the unbonded length of test nails/tiebacks is provided, the casing shall be installed to prevent interaction between the bonded length of the nail/tieback and the casing/testing apparatus.

The testing equipment shall include two dial gauges accurate to 0.001 inch, a dial gauge support, a calibrated jack and pressure gauge, a pump and the load test reaction frame. The dial gauge should be aligned within 5 degrees of the longitudinal nail/tieback axis and shall be independently supported of the load frame/jack and the shoring wall. The hydraulic jack, pressure gauge and pump shall be used to apply and measure the test loads.

The jack and pressure gauge shall be calibrated by an independent testing laboratory as a unit. The pressure gauge shall be graduated in 100 pounds per square inch (psi) increments or less and shall have a range not exceeding twice the anticipated maximum pressure during testing unless approved by the Engineer. The ram travel of the jack shall be sufficient to enable the test to be performed without repositioning the jack.

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The jack shall be independently supported and centered over the nail/tieback so that the nail/tieback does not carry the weight of the jack. The jack, bearing plates and stressing anchorage shall be aligned with the nail/tieback. The initial position of the jack shall be such that repositioning of the jack is not necessary during the load test.

The reaction frame should be designed/sized such that excessive deflection of the test apparatus does not occur and that the testing apparatus does not need to be repositioned during the load test. If the reaction frame bears directly on the shoring wall facing, the reaction frame should be designed so as not to damage the facing.

Verification Tests

Prior to production soil nail/tieback installation, at least two soil nails/tiebacks for each soil type shall be tested to validate the design pullout value. All test nails/tiebacks shall be installed by the same methods, personnel, material and equipment as the production anchors. Changes in methods, personnel, material or equipment may require additional verification testing as determined by the Engineer. At least two successful verification tests shall be performed for each installation method and each soil type. The nails/tiebacks used for the verification tests may be used as production nails/tiebacks if approved by the Engineer.

For soil nails, the unbonded length of the test nails shall be at least 3 feet unless approved otherwise by the Engineer. The bond length of the test nails shall not be less than 10 feet and shall not be longer than the bond length that would prevent testing to 200 percent of the design load while not exceeding the allowable bar load. The allowable bar load during testing shall not exceed 80 percent of the steel ultimate

strength for Grade 150 bars or 90 percent of the steel ultimate strength for Grade 60 and 75 bars. The allowable tieback load should not exceed 80 percent of the steel ultimate strength.

For soil nails, the design test load shall be determined by multiplying the bond length of the nail times the design load pullout resistance (load transfer). Verification test nails/tiebacks shall be incrementally loaded and unloaded in accordance with the following schedule:

Load	Hold Time
Alignment Load	1 minute
0.25 Design Load (DL)	1 minute
0.5DL	1 minute
0.75DL	1 minute
1.0DL	1 minute
1.25DL	1 minute
1.5DL	60 minutes
1.75DL	1 minute
2.0DL	10 minutes

The alignment load shall be the minimum load required to align the testing apparatus and should not exceed 5 percent of the design load. The dial gauge should be zeroed after the alignment load is applied. Nail/tieback deflections during the 1.5DL test load shall be recorded at 1, 2, 3, 5, 6, 10, 20, 30, 50 and 60 minutes.

Proof Tests

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Proof tests shall be completed on approximately 5 percent of the production nails at locations selected by the owner's representative. Additional testing may be required where nail installation methods are substandard. Proof tests shall be completed on each production tieback.

For soil nails, the unbonded length of the test nails shall be at least 3 feet unless approved otherwise by the Engineer. The bond length of the test nails shall not be less than 10 feet and shall not be longer than the bond length that would prevent testing to 200 percent of the design load while not exceeding the allowable bar load. The allowable bar load during testing shall not exceed 80 percent of the steel ultimate strength for Grade 150 bars or 90 percent of the steel ultimate strength for Grade 60 and 75 bars.

For soil nails, the design test load shall be determined by multiplying the bond length of the nail times the design load pullout resistance (load transfer). Proof test nails/tiebacks shall be incrementally loaded and unloaded in accordance with the following schedule:

Load	Hold Time
Alignment Load	1 minute
0.25 Design Load (DL)	1 minute
0.5DL	1 minute
0.75DL	1 minute
1.0DL	1 minute
1.25DL (soil nails)	1 minute
1.3DL (tiebacks)	10 minutes
1.5DL (soil nails)	

The alignment load shall be the minimum load required to align the testing apparatus and should not exceed 5 percent of the design load. The dial gauge should be zeroed after the alignment load is applied. Nail/tieback deflections during the 1.3DL and 1.5DL test loads shall be recorded at 1, 2, 3, 5, 6 and 10 minutes.

Depending upon the nail/tieback deflection performance, the load hold period at 1.3DL (tiebacks) and 1.5DL (soil nails) may be increased to 60 minutes. Nail/tieback movement shall be recorded at 1, 2, 3, 5, 6 and 10 minutes. If the nail/tieback deflection between 1 minute and 10 minutes is greater than 0.04 inches, the 1.3DL/1.5DL load shall be continued to be held for a total of 60 minutes and deflections recorded at 20, 30, 50 and 60 minutes.

Test Nail/Tieback Acceptance

A test nail/tieback shall be considered acceptable when:

1. For verification tests, a nail/tieback is considered acceptable if the creep rate is less than 0.08 inches per log cycle of time between 6 minutes and 60 minutes and the creep rate is linear or decreasing throughout the creep test load hold period.
2. For proof tests, a nail/tieback is considered acceptable if the creep rate is less than 0.04 inches per log cycle of time between 1 and 10 minutes or the creep rate is less than 0.08 inches per log cycle of time between 6 minutes and 60 minutes, and the creep rate is linear or decreasing throughout the creep test load hold period.
3. The total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length.
4. Pullout failure does not occur. Pullout failure is defined as the load at which continued attempts to increase the test load result in continued pullout of the test nail/tieback.

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Acceptable proof test nails/tiebacks may be incorporated as production nails/tiebacks provided that the unbonded test length of the nail/tieback hole has not collapsed and the test nail/tieback length and bar size/number of strands are equal to or greater than the scheduled production nail/tieback at the test location. Test nails/tiebacks meeting these criteria shall be completed by grouting the unbonded length. Maintenance of the temporary unbonded length for subsequent grouting is the contractor's responsibility.

The Engineer shall evaluate the verification test results. Nail/tieback installation techniques that do not satisfy the nail/tieback testing requirements shall be considered inadequate. In this case, the contractor shall propose alternative methods and install replacement verification test nails/tiebacks.

The Engineer may require that the contractor replace or install additional production nails/tiebacks in areas represented by inadequate proof tests.

SHORING MONITORING

Preconstruction Survey

A shoring monitoring program should be established to monitor the performance of the cut section retaining walls and to provide early detection of deflections that could potentially damage nearby improvements. We recommend that a pre-construction survey of adjacent improvements, such as streets, utilities and buildings, be performed prior to commencing construction. The preconstruction survey should include a video or photographic survey of the condition of existing improvements to establish the preconstruction condition, with special attention to existing cracks in streets or buildings.

Optical Survey

The shoring monitoring program should include an optical survey monitoring program. The recommended frequency of monitoring should vary as a function of the stage of construction as presented in the following table:

Construction Stage	Monitoring Frequency
During excavation and until wall movements have stabilized	Twice weekly
During excavation if lateral wall movements exceed 1 inch and until wall movements have stabilized	Daily
After excavation is complete and wall movements have stabilized, and before the floors of the building reach the top of the excavation	Twice monthly

Monitoring should include vertical and horizontal survey measurements accurate to at least 0.01 feet. A baseline reading of the monitoring points should be completed prior to beginning shoring installation. The survey data should be provided to GeoEngineers for review within 24 hours.

For shoring walls, we recommend that optical survey points be established: (1) along the top of the shoring walls, (2) at the curb behind the north and south walls, and (3) on existing buildings located within a horizontal distance of the shoring walls equal to the height of the wall. The survey points should be spaced every 25 feet along the wall face, and the points along the curb line/existing buildings should be located immediately behind the points along the top of the wall. Additional survey points should also be installed at mid-height of the wall as the excavation progresses if lateral movements exceed 1 inch, at the same spacing as the points along the top of the wall. If lateral wall movements are observed to be in excess of ½ inch between successive readings or if total wall movements exceed 1 inch, construction of the shoring walls should be stopped to determine the cause of the movement and to establish the type and extent of remedial measures required.

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APPENDIX D

REPORT LIMITATIONS AND GUIDELINES FOR USE

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APPENDIX D
REPORT LIMITATIONS AND GUIDELINES FOR USE[2]

This attachment 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 Microsoft Corporation and their design consultants in the design of the proposed Microsoft RedWest South project in Redmond, Washington. 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 our Agreement with the Client 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.

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A GEOTECHNICAL ENGINEERING OR GEOLOGIC REPORT IS BASED ON A UNIQUE SET OF PROJECT-SPECIFIC FACTORS

This report has been prepared for the planned Microsoft RedWest South project located in Redmond, 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.

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.

[2] Developed based on material provided by ASFE, Professional Firms Practicing in the Geosciences; www.asfe.org .

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.

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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

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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.