Geotechnical Engineering Services Report

159th Avenue NE Vault in Purchased Parcels
Tosh Creek Watershed Restoration
Redmond, Washington

for
City of Redmond and
HDR Engineering, Inc.

July 6, 2016

GeoEngineers

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File No. 0500-204-00, Task 0500

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INTRODUCTION

This report presents the results of our geotechnical engineering services for the 159th Avenue NE Vault in Purchased Parcels project in Redmond, Washington. This project is part of the larger Tosh Creek Watershed Restoration project. The site is shown relative to surrounding physical features on Figure 1, Vicinity Map, and Figure 2, Site Plan.

GeoEngineers, Inc. (GeoEngineers) previously provided a draft geotechnical engineering report for the project, dated December 4, 2015, which provided preliminary geotechnical engineering design recommendations for a vault located within the right-of-way (ROW) of 159th Avenue NE. A secondary vault location has been proposed, which will move the vault onto purchased properties to the east of 159th Avenue NE. The purpose of this study is to evaluate subsurface conditions as a basis of developing geotechnical recommendations in support of the design of the proposed 159th Avenue NE vault located on the purchased parcels east of 159th Avenue NE.

Our geotechnical engineering services were completed in general accordance with the Subconsultant Agreement between GeoEngineers and HDR Engineering, Inc. (HDR) executed on July 27, 2015, and the Scope and Fee estimate provided to HDR on March 15, 2016. The conclusions and recommendations provided in this report are draft and are subject to modification pending further discussion with the project team. Our scope of services summarized in the report includes:

- Review geologic maps and available geotechnical reports for the site and nearby properties.
- Coordinate and complete an exploration program to include four borings at the site; two of the borings completed with standpipe piezometers (monitoring wells).
- Perform geotechnical laboratory tests on samples obtained from the borings.
- Evaluate pertinent physical and engineering characteristics of the soils based on the results of the field exploration, laboratory testing and our experience, and provide conclusions and geotechnical recommendations for the following:
  - Site preparation and grading, including underground utility installation.
  - Placement of backfill and structural fill, including fill type and compaction requirements and the reuse of on-site soils.
  - Potential areas of settlement and vibration impacts during construction.
  - Geotechnical considerations for temporary excavation support.
  - Vault wall and foundation recommendations and appropriate design capacities and lateral restraint.
  - Critical areas and slope evaluation of the existing slopes east of the site and appropriate setback distances.
  - Geotechnical considerations related to groundwater conditions including anticipated seasonal fluctuations, and buoyancy and drainage considerations for buried structures.
  - Guideline recommendations for construction dewatering, if necessary.
  - Recommendations for sedimentation and erosion control during and following construction, and permanent site drainage.

- Preparing this report presenting our geotechnical design conclusions and recommendations.

PROJECT DESCRIPTION

Our understanding of the project is based on discussions with, and information provided by, HDR and the City of Redmond (City). We understand that the project will construct a new vault within City-owned parcel number 856293-0710 and two parcels to be purchased by the City, 4612 and 4618 159th Avenue NE. The current vault concept would construct a concrete vault, with approximate dimensions of 70 feet wide by 240 feet long by 9 feet deep. The bottom of the vault will be approximately 15 feet below existing grade, with localized areas extending up to 20 feet below grade (e.g. sump areas).

FIELD EXPLORATIONS AND LABORATORY TESTING

Field Explorations

Subsurface soil and groundwater conditions were evaluated by drilling borings B-2 and B-2A (previous study) and B-5 through B-8 (this study). The borings were drilled to depths ranging to 41.5 feet below the ground surface (bgs). Borings B-5 and B-7 were completed with standpipe piezometers (monitoring wells). The approximate locations of the borings are shown on the Site Plan. Details of the exploration program, boring logs, and details of the piezometer installations are presented in Appendix A, Field Explorations.

Laboratory Testing

Soil samples were collected during drilling and taken to GeoEngineers' laboratory for further evaluation. Selected samples were tested for the determination of moisture content, grain size distribution (sieve analysis), and plasticity characteristics (Atterberg limits). A description of the laboratory testing and the test results are presented in Appendix B, Laboratory Testing.

SITE CONDITIONS

Geologic Setting

The project area is located within the central portion of the Puget Lowland Physiographic Province. The Puget Lowland Physiographic Province consists of a broad, low-lying region of subdued topography situated between the Cascade Range to the east and the Olympic Mountains and Willapa Hills to the west.

Starting in the early Pleistocene, the Puget Lowland was subject to at least six periods of extensive glaciation. The Puget Lowland owes many of its present-day geomorphic features to the last continental glacier that covered the region during the Vashon Stade of the Fraser Glaciation. The Puget Lobe of the Cordilleran ice sheet advanced from British Columbia about 18,000 years ago to just south of Olympia, covering the Puget Lowland with glacial ice. As during previous glaciations, streams and rivers draining the Cascades were dammed by the Cordilleran ice sheet. By 14,000 years ago the ice had retreated northward past Seattle. Large areas south of Seattle were being covered by recessional outwash sands and gravels. These sands and gravels are an important source of raw materials for concrete and other construction uses. Approximately 10,000 years ago, after a short-lived re-advance in the northern Puget Lowland, the Cordilleran ice sheet disappeared from the area (paragraph adapted from Lasmanis (1991)).

Site Geology

Geologic information for the project vicinity includes a Pacific Northwest Center for Geologic Mapping Studies map, "Geologic Map of King County, Washington" (Booth et al. 2007). The mapped surficial geology in the project vicinity consists of Vashon age glacial till deposits.

Glacial till was deposited and consolidated by several thousand feet of ice and is mapped in the upland portion of the Tosh Creek project area, including the 159th Avenue NE area. Glacial till consists of dense to very dense, non-sorted mixture of clay, silt, sand, gravel, cobbles, and boulders. The upper 2 to 5 feet of glacial till is typically weathered, and the density ranges from medium dense to dense. This weathered zone is moderately well-drained, whereas the underlying unweathered till is generally a barrier to vertical drainage. Water percolating into the weathered till will generally pond and migrate laterally between the weathered and unweathered layers. In its native undisturbed state, glacial till soils are competent load bearing soils.

In contrast, differing surficial geologic units have been mapped by others (City of Redmond et al. 2015) in the vicinity of Tosh Creek that include glacial lakebed deposits and ice contact deposits. Both of these geologic units typically consist of medium stiff to hard silt and clay.

Fill soils associated with site development are also present at the site.

Geologically Hazardous Areas Delineation

We reviewed the available geologically hazardous areas maps for the City of Redmond (Redmond 2016) and completed a geologic site reconnaissance to determine if the project site and immediate vicinity are located within geologically hazardous areas pursuant to the Critical Areas Ordinance (CAO) as outlined in Redmond Zoning Code (RZC) 21.64.060. Geologically hazardous areas as defined by the CAO include areas susceptible to erosion, sliding, earthquake, or other geological events, and are generally classified as follows:

- Erosion Hazard Areas – areas underlain by soils identified by the U.S. Department of Agriculture (USDA) Soil Conservation Service as having "severe" or "very severe" rill and inter-rill erosion hazards.
- Landslide Hazard Areas – areas potentially subject to signification or severe risk of landslides based on a combination of geologic, topographic, and hydrogeologic features and include any area with a slope 40 percent or steeper with a vertical relief of 10 feet or more.
- Seismic Hazard Areas – areas subject to severe risk of damage as a result of earthquake-induced ground shaking, slope failure, settlement, soil liquefaction, or surface faulting.

Review of Geologic Hazard Maps

The current City of Redmond hazard areas maps show geologic hazard areas as defined by the CAO, which include erosion, seismic and landslide hazard areas. A landslide hazard area is mapped by the City along the eastern margin of the project site. Neither erosion nor seismic hazard areas are mapped by the City in the immediate project vicinity. It should be noted that the intent of the maps is for use as a general guide representing the approximate locations of hazard areas and do not necessarily ensure the presence or absence of said hazards and where the maps and the CAO shall prevail (i.e., field observations by a qualified professional prevail over the published hazard areas maps).

We reviewed the USDA Natural Resources Conservation Service (NRCS) soils classification for the site, and the entire site is mapped as Alderwood gravelly sandy loam, 8 to 15 percent slopes. Erosion hazard for this unit is designated as "slight to moderate."

Geologic Field Reconnaissance

A geologic field reconnaissance was completed by a licensed geologist from our firm on May 17, 2016. The purpose of the reconnaissance was to evaluate the site conditions with respect to determining the presence or absence of geologically hazardous areas at the project site and immediate vicinity. A field report documenting the observations and evaluations made during the reconnaissance is included as Appendix C, Field Report – Geologic Reconnaissance, of this report. The following key observations were made during the geologic field reconnaissance:

- The head of Tributary B consists of an inclined convergent swale with slopes typically inclined to between 30 and 35 percent, and sideslopes locally inclined up to 45 percent for up to 10 vertical feet in height; two 10-inch-diameter culverts discharge at the head of the tributary onto coarse boulder rip-rap.
- The head of Tributary C also consists of an inclined convergent swale with slopes typically inclined to between 30 and 35 percent, and sideslopes locally inclined up to 35 percent for 10 vertical feet in height; a perched 10-inch-diameter culvert discharges at the head of the tributary.
- Tributary B contains a small stream with a channel incised between 2 and 4 feet into weathered till overlying hard glacially consolidated silts and clays.
- Tributary C contains a small stream with a channel incised between 1 and 2 feet into colluvium overlying hard glacially consolidated silts and clays.
- In general, no evidence was observed of recent or historical slope movement or seepage on the sloped areas within the heads of Tributaries B or C, and no evidence of significant soil erosion was observed outside of the main channels of the respective tributary streams.

Critical Areas Assessment

Based on our field observations and review of available critical areas maps, and geologic, geotechnical, and hydrogeologic information, we conclude that a portion of the left (north) bank of Tributary B qualifies as a borderline landslide hazard area on the basis of slope angle and height, as defined by RZC 21.64.060. However, we observed no evidence of recent or historic instability or movement of this slope and we conclude that the planned improvements can be successfully completed without negatively impacting the stability of this slope, provided the geotechnical considerations presented in this report are incorporated into the project planning and design. The remaining portions of the site, including the slopes adjacent to the eastern perimeter of the proposed vault, do not qualify as geologically hazardous areas as defined by RZC 21.64.060, nor was evidence observed that indicates potential instability of these areas.

Surface Conditions

The project site is located on three parcels east of 159th Avenue NE. The southern two parcels both have single-family residences with driveways, landscaping, and grass lawns. The northernmost parcel is undeveloped and serves as the ROW for the Tributary B stormwater outfall. The project area not currently developed is wooded with a moderately dense understory vegetation of sword fern, English ivy, devil's club, vine maple, and hazelnut. The overstory consists of western hemlock, western red cedar, big leaf maple, and alder with diameter at breast heights up to 2 feet. The ground surface at the current vault location is generally flat at about Elevation 250 feet to moderately sloping downward to the east to about Elevation 250 feet at the south and Elevation 240 feet at the north ends of the vault. Beyond the planned vault footprint, the topography slopes downward toward the east. Immediately east of the vault footprint, the heads of two sub-parallel east-draining tributaries, Tributary B (north tributary) and Tributary C (south tributary) of Tosh Creek, descend a generally east-facing slope into the Sammamish Valley.

Subsurface Conditions

Soil

The soils encountered in the explorations are generally consistent with the mapped geology of the region and consist of fill overlying glacially consolidated soils. Each of these units is discussed below.

Fill was encountered in each of the borings, is variable, and consists of layers of very loose to medium dense sand and gravel with varying amounts of silt and soft to medium silt. The fill extends to depths ranging up from 6 to about 14 feet below ground surface (bgs). The pavement section at the boring B-2 location is 4 inches of asphalt concrete.

In borings B-5 and B-6, a layer of loose/soft native soils was encountered below the fill. These soils consist of silty sand with organic matter and sandy organic silt. These soils extended to depths of 13 to 14 feet bgs.

Glacially consolidated soils below the fill and loose/soft native soils consist of layers of stiff to hard silt and clay with varying amounts of sand. The top of these stiff to hard silt and clay soils varies from 6 to 14 feet bgs. These soils are interpreted to be glacial lakebed deposits and were encountered to the maximum depth explored of 41.5 feet bgs.

Groundwater

Groundwater levels have been collected regularly from the project standpipe piezometers (monitoring wells). Discrete readings are shown on the boring logs in Appendix A. Pressure transducer readings are being reported as part of our quarterly data download under separate memoranda to the project team.

Based on the measured groundwater levels and site soils, the groundwater table is expected to be encountered at a depth of about 12 to 17 feet bgs, with capillary rise extending the presence of saturated soils to shallower depths within vadose zone, which is composed mainly of low permeability fine-grained soils (silt and clay). Some seasonal variation of the groundwater table is expected in response to changes in recharge from infiltration rainfall between the wet winters and dry summers.

The deeper well completion for the previous vault location, screened between 75 and 95 feet bgs, shows a lower piezometric level that is consistent with the regional hydrogeologic setting of the site within an elevated upland above Lake Sammamish. The low-permeability fine-grained soils likely form a regional aquitard that limits the rate of downward percolation and maintains high levels of soil saturation.

Additionally, during the wet season, perched groundwater is expected to develop at the contact between the fill and glacially consolidated soils. The majority of the soils above the static groundwater table are expected to be saturated due to these conditions, with perched zones likely diminishing during drier summer periods.

CONCLUSIONS AND RECOMMENDATIONS

Summary of Geotechnical Considerations

We conclude that the planned improvements can be successfully completed from a geotechnical perspective, provided the considerations presented in this report are incorporated into the project planning and design. A summary of the primary site preparation and design considerations for the proposed project is provided below. This summary is presented for introductory purposes and should be used in conjunction with the complete recommendations presented in this report.

- Subsurface conditions are anticipated to consist of an upper layer of fill and/or loose/soft native soils overlying stiff to hard silt and clay soils. Excavation can be accomplished in these soils with conventional excavation equipment.
- We conclude that the planned vault location has an adequate setback from adjacent slopes and can be successfully constructed without negatively impacting the stability of the slopes in the project vicinity, provided the geotechnical considerations presented in this report are incorporated into the project planning and design.
- Temporary shoring will be required along the west side and portions of the north and south sides of the excavation due to space constraints.
- With the current planned vault bottom of slab at Elevation 235.5 feet, we anticipate some overexcavation will be required along the east side of the vault to reach undisturbed glacially consolidated soils for foundation support. Based on our explorations, overexcavation depths are anticipated to be up to 5 feet.
- The planned vault excavation will extend into saturated soils and below the anticipated groundwater table. However, based on the low permeability of the fine-grained soils below the groundwater table, we anticipate that seepage into the planned excavation will be minor and that dewatering, if needed, can be accomplished with sumps and pumps.
- The existing fill and native soils at the site contain sufficient fines (silt and clay) such that they are moisture-sensitive soils that will become easily disturbed when wet. We recommend site development be accomplished during extended periods of dry weather when the site soils will be less susceptible to disturbance due to rain and runoff and when the groundwater seepage is less. If construction is completed during the wet season, additional excavation and replacement of portions of the vault subgrade soils and an increased dewatering effort may be necessary.
- Consideration should be given to placing a gravel layer below the vault subgrades to provide a working surface, especially if construction will extend into the wet season. The gravel layer will help prevent subgrade deterioration under the disturbance of equipment and foot traffic during construction. We recommend a minimum 12-inch-thick clean crushed gravel layer under the foundation elements. This gravel layer can also be incorporated into the underslab drainage system for the vault, if drainage is incorporated into the design.
- Where open cuts may be feasible, we recommend temporary slopes be inclined at 1.5H:1V (horizontal to vertical) or flatter at the site. These slopes may need to be modified depending on the excavation depth, seepage conditions, and stability.

- The project site is best designated as seismic Soil Profile Type D per the 2012 International Building Code (IBC).

These and other geotechnical considerations are discussed further, and recommendations pertaining to the geotechnical aspects of the project are presented in the following sections of this report.

Earthwork

Earthwork Considerations

Fill and glacially consolidated deposits were observed in the explorations. While cobbles and/or boulders were not noted during drilling the borings, cobbles and boulders are commonly encountered in glacially consolidated soils. The contractor should be prepared to deal with debris in the fill, and cobbles and boulders in the glacially consolidated soils.

Clearing and Grubbing

Significant clearing and grubbing is not expected within the project area and these activities are expected to be limited to the demolition and removal of the two single-family homes, pavements, concrete hardscapes (sidewalks, curbs, gutters, etc.), utilities, and landscape areas.

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. The project's impact on erosion-prone areas can be reduced by implementing an erosion and sedimentation control plan. The plan should be designed in accordance with applicable City and/or county standards. The plan should incorporate basic planning principles including:

- Scheduling grading and construction to reduce soil exposure,
- Retaining existing vegetation whenever feasible,
- Revegetating or mulching denuded areas,
- Directing runoff away from denuded areas,
- Minimizing the length and steepness of slopes with exposed soils,
- Decreasing runoff velocities,
- Confining sediment to the project site,
- Inspecting and maintaining control measures frequently,
- Covering soil stockpiles, and
- Implementing proper erosion control best management practices (BMPs).

Temporary erosion protection should be used and maintained in areas with exposed or disturbed soils to help reduce the potential for erosion and reduce transport of sediment to adjacent areas. Temporary erosion protection should include the construction of a silt fence around the perimeter of the work area prior to the commencement of grading activities. Permanent erosion protection should be provided by re-establishing vegetation using hydroseeding and/or landscape planting.

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

Subgrade Preparation

A geotechnical engineer should monitor the subgrade preparation to help determine the condition of the exposed subgrade, and if necessary, the depth of removal of soft or pumping soils, and to evaluate whether subgrade disturbance or progressive deterioration is occurring. Subgrade disturbance or deterioration could occur if the subgrade is wet and cannot be dried. Prior to constructing foundations, placing new fill, subbase or base course materials, subgrade areas should be proof-rolled to locate areas of loose, soft, or pumping soils. Proof-rolling can be completed using a piece of heavy tire-mounted equipment or a loaded dump truck. Probing should be used to evaluate the subgrade during periods of wet weather or if access is not feasible, for construction equipment. If soft or pumping soils are observed, such unsuitable subgrade soils should be recompacted or overexcavated and replaced. The depth of overexcavation should be determined by a geotechnical engineer.

Based on our explorations, we anticipate some overexcavation will be required along the east side of the vault to reach undisturbed glacially consolidated soils for foundation support. Overexcavation depths are anticipated to be up to 5 feet, based on the explorations.

If deep pockets of soft or pumping soils are encountered, it may be possible to limit the depth of overexcavation by placing a woven Geotextile for Separation or Soil Stabilization (Washington State Department of Transportation [WSDOT] Standard Specification 9-33) on the overexcavated subgrade and covering the geotextile with structural fill. We recommend using the specified woven fabric for soil stabilization. The geotextile will provide additional support by bridging over the soft material, and will help reduce fines contamination into the structural fill. The need for geotextile fabric and overexcavation should be evaluated by a geotechnical engineer.

Backfill in old utility trenches or previously placed fill that exhibits surface settlement should be tested and recompacted if necessary. The uppermost 2 feet supporting the new pavement should be compacted to at least 95 percent of the maximum dry density (MDD) estimated in general accordance with ASTM International (ASTM) D 1557. Material below this level should be compacted to at least 90 percent.

Structural Fill

Materials

Materials used to construct roadways, placed to support buried structures or foundations, or placed behind retaining or buried structures are classified as structural fill for the purpose of this report. Structural fill material quality varies depending upon its use, as described below:

1. As a minimum, structural fill placed to construct embankments and roadways, to backfill utility trenches and to support foundations should meet the criteria for common borrow, WSDOT 9-03.14(3). Common borrow will be suitable for use as structural fill during dry weather conditions only. If structural fill is placed during wet weather, the structural fill should consist of gravel borrow, WSDOT 9-03.14(1).
2. Structural backfill for walls should meet the criteria for gravel backfill for walls, WSDOT 9-03.12(2).

3. Structural fill placed for underslab drainage should consist of 1.5-inch minus clean crushed gravel, WSDOT 9-03.1(4)C, Grading No. 57.
4. Structural fill placed to surround collector pipe (drain rock) should meet the criteria for gravel backfill for drains, WSDOT 9-03.12(4).
5. Structural fill placed as crushed surfacing base/top course below pavements should conform to WSDOT 9-03.9(3).

On-site Soils

The on-site soils have a high percentage of fines (silt and clay). These soils will not be suitable for use as structural fill during wet weather conditions, but may selectively be used for fill during periods of extended dry weather, provided the material meets the criteria for structural fill and can be conditioned to a suitable moisture content to be adequately compacted. The on-site soils generally meet the criteria for common borrow, WSDOT 9-03.14(3), with the exception of the organic silt layer encountered in boring B-6.

If wet weather construction is planned, the on-site soils will not be suitable for use as structural fill. For wet weather construction, we recommend that structural fill consist of imported sand and gravel with little or no silt, WSDOT 9-03.14(1).

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 1 foot 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 behind retaining walls and buried structures should be compacted to at least 90 percent of the MDD in general accordance with ASTM D 1557. Care should be taken when compacting fill near the face of structures to avoid overcompaction and hence overstressing the walls.
2. Structural fill in embankment and new pavement areas, including utility trench backfill, should be compacted to 90 percent of the MDD (ASTM D 1557), except that the upper 2 feet of fill below final subgrade should be compacted to 95 percent of the MDD (ASTM D 1557).
3. Structural fill placed to support foundations should be compacted to 95 percent of the MDD (ASTM D 1557).
4. Structural fill placed as crushed rock base course below pavements should be compacted to 95 percent of the MDD (ASTM D 1557).

We recommend that a geotechnical engineer be present during proof-rolling and/or probing of the exposed subgrade soils and during placement of structural fill. The geotechnical engineer will evaluate the adequacy of the subgrade soils and identify areas needing further work, perform in-place moisture-density tests in the fill to evaluate whether the work is being done in accordance with the compaction specifications, and advise on any modifications to procedure that may be appropriate for the prevailing conditions.

Weather Considerations

The majority of the on-site soils contain a moderate to high percentage of fines (silt and clay) and are moisture-sensitive. When the moisture content of these soils is more than a few percent above the optimum moisture content, these soils become muddy and unstable, operation of equipment on these soils will be difficult, and it will be difficult or impossible to meet the required compaction criteria. Additionally, disturbance of near-surface soils should be expected if earthwork is completed during periods of wet weather. The contractor will need to take precautions to protect the subgrade during periods of wet weather.

The wet weather season in western Washington generally begins in October and continues through May; however, periods of wet weather may occur during any month of the year. The optimum earthwork period for these types of soils is typically June through September. If wet weather earthwork is unavoidable, we recommend that:

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

Temporary Slopes

We recommend that temporary unsupported cut slopes greater than 4 feet deep be inclined no steeper than 1.5H:1V. This applies to fully dewatered conditions. Flatter slopes may be necessary if seepage is present on the cut face. Temporary cut slopes should encroach no closer than 5 feet laterally from roadways, pavements, structures, or other improvements. Some sloughing and raveling of the cut slopes should be expected. Temporary covering, such as heavy plastic sheeting, should be used to protect these slopes during periods of rainfall. Surface water runoff from above cut slopes must be prevented from flowing over the slope face by using curbs, berms, drainage ditches, swales, or other appropriate methods.

If temporary cut slopes experience excessive sloughing or raveling during construction, it may become necessary to modify the cut slopes to maintain safe working conditions and protect adjacent facilities or structures. Slopes experiencing excessive sloughing or raveling can be flattened or can be regraded to add intermediate slope benches, or additional dewatering can be provided if the poor slope performance is related to groundwater seepage.

Permanent Slopes

We recommend that permanent cut and fill slopes be constructed no steeper than 2H:1V. To achieve uniform compaction, we recommend that fill slopes be overbuilt slightly (several feet) and subsequently cut back to expose properly compacted fill. We recommend that the finished slope faces be compacted by track walking with the equipment running perpendicular to the slope contours so that the track grouser marks help provide an erosion-resistant slope texture.

To reduce erosion, newly constructed slopes should be planted or hydroseeded shortly after completion of grading. Until the vegetation is established, some sloughing and raveling of the slopes should be expected. This may require localized repairs and reseeding. Temporary covering, such as clear heavy plastic sheeting, jute fabric, loose straw, or excelsior or straw/coconut matting, should be used to protect the slopes during periods of rainfall.

Temporary Shoring Support of Excavations

Shoring and temporary slope inclinations must conform to the provisions of Title 296 Washington Administrative Code (WAC), Part N, "Excavation, Trenching, and Shoring." Excavation, shoring, trench boxes, or sloped sidewalls will be required under Washington Industrial Safety and Health Act (WISHA). The contract documents should specify that the contractor is responsible for selecting excavation and dewatering methods, monitoring the excavations for safety and providing shoring, as required, to protect personnel and structures. Based on our experience and the site conditions, soldier pile walls are considered to be the appropriate shoring system for the north, west and south sides of the vault excavation.

Vertical excavations deeper than 3 feet should be shored or laid back at a stable slope if workers are required to enter. Because of the diversity of available shoring systems and construction techniques, the design of temporary shoring is most appropriately left up to the contractor proposing to complete the installation. However, we recommend that the shoring be designed by a Professional Engineer (P.E.) licensed in the State of Washington, and that the P.E.-stamped shoring plans and calculations be submitted to the City and the Engineer for review prior to construction. The following paragraphs present general recommendations for the type of shoring system and design parameters that we conclude are appropriate for the subsurface conditions at the project site.

The lateral soil pressures acting on temporary supports will depend on the type and density of the soil behind the wall, the inclination of the ground surface behind the wall, and groundwater. For shoring walls that are free to yield at the top at least one thousandth of the height of the wall (i.e., wall height times 0.001), soil pressures will be less than if movement is restrained. The design of temporary shoring should allow for lateral pressures exerted by the adjacent soil, and for surcharge loads resulting from structures, traffic, construction equipment, temporary stockpiles adjacent to the excavation, etc. Lateral load resistance can be mobilized through the use of braces, tiebacks, anchor blocks, and passive pressures on members that extend below the bottom of the excavation. Temporary shoring used to support trench excavations typically uses internal bracing such as hydraulic shoring or trench boxes.

Recommended earth pressures for the design of temporary shoring are presented in Figure 3, Temporary Shoring Earth Pressure Diagram. We recommend that yielding walls retaining on-site soils be designed using active earth pressures. For non-yielding (i.e., braced) systems, we recommend that the shoring be designed using at-rest earth pressure. The earth pressure values presented in Figure 3 assume that the ground behind the shoring is sufficiently drained (dewatered) such that the hydrostatic pressures do not develop behind the shoring. If the shoring system is not adequately drained, then full hydrostatic conditions should be assumed by the shoring designer. We can provide additional earth pressure recommendations appropriate for undrained shoring, if requested.

The soil pressure available to resist lateral loads against shoring is a function of the passive resistance that can develop on the face of below-grade elements of the shoring as those elements move horizontally into the soil. The allowable passive resistance presented on Figure 3 assumes the native soils are below the groundwater table and includes a factor of safety of about 1.5.

Construction Dewatering

We understand that the bottom of the vault floor slab will be at Elevation 235.5 feet. At this depth we do not expect the excavation to complete installation of the vault to extend more than 5 feet below the static groundwater level, and the quantity and inflow rate of groundwater into the excavation is expected to be less than 5 gallons per minute (gpm) because of the relatively low permeability of the glacially consolidated soils. Although soils appear to be uniformly similar within the four borings, glacial terrain is notoriously variable and more permeable soils could be present between the boring locations. Such features, if encountered in the excavation, may increase potential groundwater inflows to 10 or 20 gpm but may be short-lived if the permeable features are of limited extent and allowed drain. In our opinion, these expected construction inflow rates should be manageable with a system of drainage trenches and pumped sumps. We recommend that construction be completed in the late summer or early fall months when the groundwater level is typically at its lowest elevation.

Settlement and Vibration Potential during Construction

Peak particle velocity (PPV) is the generally accepted vibration component for assessing the potential for damaging vibrations produced by a wide variety of energy sources, including construction equipment. Empirical studies show that the PPV associated with ground vibrations is inversely and exponentially proportional to the distance for the source vibration. In other words, the PPV decreases very rapidly with distance from the source vibration. For example, the PPV measured at a distance of 100 feet from the source will be approximately 0.1 percent of the PPV measured at the source for typical construction-related vibrations. Because of the exponential rate of energy decay with distance from the source, variations in subsurface material type have only a minor effect on PPV, as compared to source distance.

A PPV of 2 inches per second is generally considered a threshold value for inducing damage to residential structures located near construction sites and quarry blasting operations (International Society of Explosives Engineers [ISEE] 1998). A PPV of 0.5 inches per second has been proposed as a threshold value of "old residential structures in very poor condition" (Wiss 1981). Similarly, Hudson and Harrison (1997) report the tolerable PPV limit of 0.5 to 2 inches per second for residential masonry buildings. By way of comparison, a PPV of 0.02 inches per second is considered the threshold for human perception of motion. ISEE (1998) reports that a PPF of 5.4 inches per second would be expected to cause minor damage to an average house subjected to quarry blasting vibrations, and a PPV of 20 inches per second would be expected to cause damage to nearly all houses. It has been demonstrated that an upper PPV limit of 12 inches per second is adequate to protect buried steel pipelines in most circumstances (Oriard 2002).

For preliminary risk assessment purposes, we evaluated the distance expected to produce PPV values of 0.5 and 2 inches per second for the anticipated construction activities/equipment as summarized in Table 1 below.

TABLE 1. SUMMARY OF ANTICIPATED CONSTRUCTION VIBRATIONS

Construction Equipment/Activity	Distance (feet)	
	PPV = 0.5 in/sec	PPV = 2.0 in/sec
Caisson Drilling and Large Bulldozers	8	3
Trucks	7	< 3
Jack Hammering	4	< 2
Crane Idling	< 2	< 1

Using available published information, ground vibrations produced by the anticipated construction activities are expected to be much less than damage threshold values at the nearby homes. However, human perception of vibration is very sensitive and is much lower than the level to damage residential structures. Therefore, we recommended the following be considered to manage the risk associated with potential homeowner claims of damage from the planned construction activities:

- Complete outreach to the community informing them of the construction activities and what to expect by way of vibrations in addition to schedule, noise, street closures, etc.
- Document the exiting conditions of the adjacent homes prior to construction with photographs and/or video. It is not uncommon for homeowners to notice preexisting cracks in their home's foundation, concrete finishes, and masonry until after a construction project is underway.
- Obtain seismographic test data early on during construction to record the real-time vibration intensity with distance for the various construction activities/equipment and to document that construction-included vibrations are below damage thresholds.

We do not anticipate measurable settlement of the near surface soils adjacent to the construction from construction vibrations at the site.

Infiltration Evaluation

Based on the soils encountered in the borings, the site soils are too low in permeability and are not considered favorable for stormwater infiltration. It is our opinion that an infiltration alternative to the vault is not appropriate for the site.

Buried Structure Design Parameters

We understand that the dimensions of the proposed concrete vault will be on the order of approximately 70 feet wide by 240 feet long by 9 feet deep. The current planned vault bottom of slab is Elevation 235.5 feet. The following sections provide geotechnical recommendations for the vault design.

Allowable Bearing Pressure

The soils explored for this study at anticipated foundation depths generally consist of glacially consolidated materials that will have relatively high allowable bearing capacities in the undisturbed condition. The exception is along the east side of the vault, where up to 5 feet of loose/soft soils were encountered below planned slab elevation in borings B-5 and B-6. These loose/soft soils will need to be overexcavated and replaced with structural fill compacted to at least 95 percent of the MDD per ASTM D 1557. The zone of structural fill should extend laterally beyond the buried structure foundation edges a horizontal distance at least equal to the thickness of the fill.

Foundation designs for the buried structures supported on glacially consolidated soils or on structural fill placed over glacially consolidated soils can be evaluated using an allowable soil bearing capacity of 4 kips per square foot (ksf). Retaining wall foundations that are subject to overturning loads can be evaluated using an allowable maximum toe bearing pressure of 4.5 ksf. These allowable soil bearing values apply to the total of dead and long-term live loads and may be increased by up to one-third for seismic loads.

Buried structures that do not extend down to bottom out in glacially consolidated soils should be designed for lesser bearing capacities. Foundation capacities for these shallower structures should be evaluated independently.

A modulus of subgrade reaction equal to 80 pounds per cubic inch (pci) can be used for design of structural slab/mat foundations founded on glacially consolidated soils.

Settlement Performance

Provided all loose/soft soil is removed and the subgrade is prepared as recommended under "Construction Considerations" below, we estimate the total settlement of buried structure foundations will be on the order of 1 inch or less. The settlements will occur rapidly, essentially, as loads are applied. Differential settlements measured along 25 feet of the structure are expected to be less than 0.5 inch.

Lateral Pressures

The lateral earth pressure values presented in Figure 4, Permanent Vault Earth Pressure Diagram, and discussed in this section, assume relatively level backfill conditions. We can provide additional earth pressure recommendations for specific site geometry (sloping back and foreslopes) to the design team, if necessary.

If the buried structure walls are rigid (restrained against rotation), we recommend that the walls be designed for an at-rest earth pressure. Rigid walls are walls that deflect less than about H/1000 under the at-rest pressure loading, where H is the height of the wall measured from the bottom of the structure to the ground surface. Once the wall moves approximately H/1000, the active pressure state is achieved. Walls that are allowed to deflect more than about H/1000 under loading may be designed for the active earth pressure. Figure 4 presents both the at-rest and active pressures for drained and submerged conditions.

To account for traffic surcharge loading, we recommend that the buried structure walls be designed for a uniform surcharge pressure determined by increasing the height of the fill behind the walls by 2 feet (250 pounds per cubic foot [pcf]) as shown on Figure 4. Other surcharge loads, such as heavy construction equipment or soil stockpiles, should be included as appropriate.

If drainage is not provided around the buried structures, the structure should be designed for submerged conditions and the uplift forces on the structures will need to be considered. Uplift forces may be resisted by widening the structure's footings such that the weight of the backfill over the footing exceeds the uplift force.

Seismic earth pressure should be used as a check in the buried structure wall design. We recommend that a rectangular seismic earth pressure distribution equal to 7H in pounds per square foot (psf) (where H is the wall height in feet) be added to the static lateral earth pressures presented above for the rigid wall or active earth pressure condition, whichever is appropriate.

Lateral wall loads on buried structures can be resisted by a combination of friction between the footing and the supporting soil and by the passive lateral resistance of the soil surrounding the embedded portion of the footing. A coefficient of friction between concrete and soil of 0.4 and a passive lateral resistance corresponding to an equivalent fluid density of 162 pcf may be used in the design. We recommend that the upper 2 feet of passive resistance be ignored. The friction coefficient and passive lateral resistance are allowable values and include a suitable factor of safety of about 1.5. The passive resistance value assumes submerged (undrained) conditions.

Permanent Drainage

At the current planned vault bottom of slab of Elevation 235.5 feet, we anticipate that the vault will extend below the static groundwater table. In addition, perched water should be expected as discussed in the "Groundwater" section of this report. Because of the permeable nature of backfill soils around the vault compared to the adjacent on-site soils, a bathtub effect can result; therefore, hydrostatic pressures, buoyancy and uplift are concerns. We recommend providing drainage around the vault to help manage groundwater; otherwise the vault should be designed for submerged conditions.

We recommend that the permanent drainage system include underslab drainage and wall drainage. Underslab drainage should consist of a 12-inch-thick layer of 1.5-inch minus clean crushed gravel, WSDOT 9-03.1(4)C, Grading No. 57. The gravel layer should be placed below the entire footprint of the buried structure and extend at least 18 inches beyond the footprint to connect with the wall drainage. A geotextile filter fabric should be placed between the 12-inch-thick gravel layer and the native soil to maintain separation and reduce piping for the fine-grained soil up into the gravel. We recommend using a non-woven geotextile for underground drainage in accordance with WSDOT 9-33.2(1), Moderate Survivability with Class A filtration properties.

For walls constructed adjacent to shoring walls, drainage is typically provided using full-coverage sheets of drainage material (such as Miradrain G100, Amerdrain 500, or equivalent) between the shoring wall and the vault wall. For backfilled walls, wall drainage should consist of an 18-inch-wide zone of drainage material such as gravel backfill for walls (WSDOT 9-03.12(2)) placed against the vault walls. Water drained from the drainage material or gravel backfill can be collected and discharged by means of a perforated collector pipe installed at the base of the drainage zone material. The pipe should be a minimum of 6 inches in diameter, enveloped with a minimum thickness of 6 inches of gravel backfill for drains (WSDOT 9-03.12(4)). A non-woven geotextile fabric such as described above for the underslab drainage, should be placed between the gravel drain backfill and the native soils.

We recommend using either heavy-wall solid pipe (SDR-35 PVC) or rigid corrugated polyethylene pipe (ADS N-12, or equal) for the perforated collector pipes. The wall drains should be discharged through a tightline pipe connected into stormwater system downstream of the vault.

Construction Considerations

Subgrade disturbance may occur if foundation excavations are completed during wet weather. A working mat of crushed rock should be placed over the buried structure foundation subgrade immediately following excavation to reduce softening and disturbance of the subgrade if construction occurs during wet weather.

If soft areas are present at the subgrade elevation, the soft areas should be removed and replaced with structural fill at the direction of a geotechnical engineer. In such instances, the zone of structural fill should extend laterally beyond the buried structure foundation edges a horizontal distance at least equal to the thickness of the fill.

The condition of the foundation subgrades should be observed by a geotechnical engineer to evaluate if the work is completed in accordance with these recommendations and that the subsurface conditions are as anticipated.

Earthquake Engineering

2012 IBC Seismic Design Parameters

We recommend the 2012 IBC parameters for Site Class, short period spectral response acceleration (S_S), 1-second period spectral response acceleration (S_1), and Seismic Coefficients F_a and F_v presented below. The values presented below in Table 2 are based on the 2008 United States Geologic Survey (USGS) National Seismic Hazards Mapping project for determining a peak ground (bedrock) acceleration coefficient for design for an earthquake that has a 2 percent probability of exceedance in a 50-year period (approximate 2,475-year return period).

TABLE 2. 2012 IBC SEISMIC DESIGN PARAMETERS

2012 IBC Parameter	Recommended Value
Site Class	D
Short Period Spectral Response Acceleration, S_S (percent g)	125.8
1-Second Period Spectral Response Acceleration, S_1 (percent g)	48.1
Seismic Coefficient, F_a	1.000
Seismic Coefficient, F_v	1.519

Seismic Hazards

We evaluated the site conditions for seismic hazards including: liquefaction, lateral spreading, seismically induced landsliding, and surface fault rupture. Our evaluation indicates that the site soils have a low potential for liquefaction. Because there is a low risk of liquefaction, the site has a low risk of liquefaction-induced ground disturbance. There is also a low potential for seismically induced landsliding and/or lateral spreading resulting from liquefaction. Based on USGS maps of active faults in the Puget Sound region, the site is located approximately 4.8 miles north of the Seattle Fault Zone. Because of the distance to the nearest mapped fault and the lack of fault displacement evidence in the area, potential for surface fault rupture is considered very remote.

Pavement Design

Subgrade Preparation

We recommend that the subgrade soils in new pavement areas be prepared and evaluated as described in the "Earthwork" section of this report. We recommend that the subgrade be compacted to at least 95 percent of the MDD per ASTM D 1557 prior to placing pavement section materials. If the subgrade soils are loose or soft, it may be necessary to excavate the soils and replace them with structural fill. A layer of suitable woven geotextile fabric may be placed over soft subgrade areas to limit the thickness of structural fill required to bridge soft, yielding areas.

New Hot Mix Asphalt Pavements

We recommend that new pavement sections be designed in accordance with City minimum surfacing requirements, as presented in the City of Redmond Standard Specifications and Details, 2012. We recommend that the pavement section for parking areas with automobile and light truck parking only be consistent with the Parking Lot section given in Redmond Standard Detail SD301. We recommend that the pavement section for access driveways and for truck parking areas be consistent with the Commercial/Industrial section given in Redmond SD301. These pavement sections are presented in Table 3 below.

TABLE 3. RECOMMENDED MINIMUM PAVEMENT SECTION

Type	Section Material	Section Thickness (inches)	
		Automobile and Light Truck Parking	Access Drives and Truck Parking
HMA	HMA Class ½-inch pg 64-22	3	7
	HMA Class 1-inch pg 64-22	0	5
Crushed Surfacing Base Course (WSDOT 9-03.9(3))		4	0

Notes:

HMA = hot mix asphalt

Consideration can be given to an alternative pavement section for the access drives and truck parking areas consisting of 4 inches of hot mix asphalt (HMA) Class ½-inch pg 64-22 over 6 inches of Crushed Surfacing Base Course (WSDOT 9-03.9(3)).

LIMITATIONS

We have prepared this report for the use of the City of Redmond, HDR Engineering, Inc. and other project team members for the Tosh Creek Watershed Restoration project in Redmond, Washington. The data should be provided to prospective contractors for their bidding or estimating purposes, but our report and interpretations should not be construed as a warranty of the subsurface conditions.

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

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

Please refer to Appendix D titled "Report Limitations and Guidelines for Use" for additional information pertaining to use of this report.

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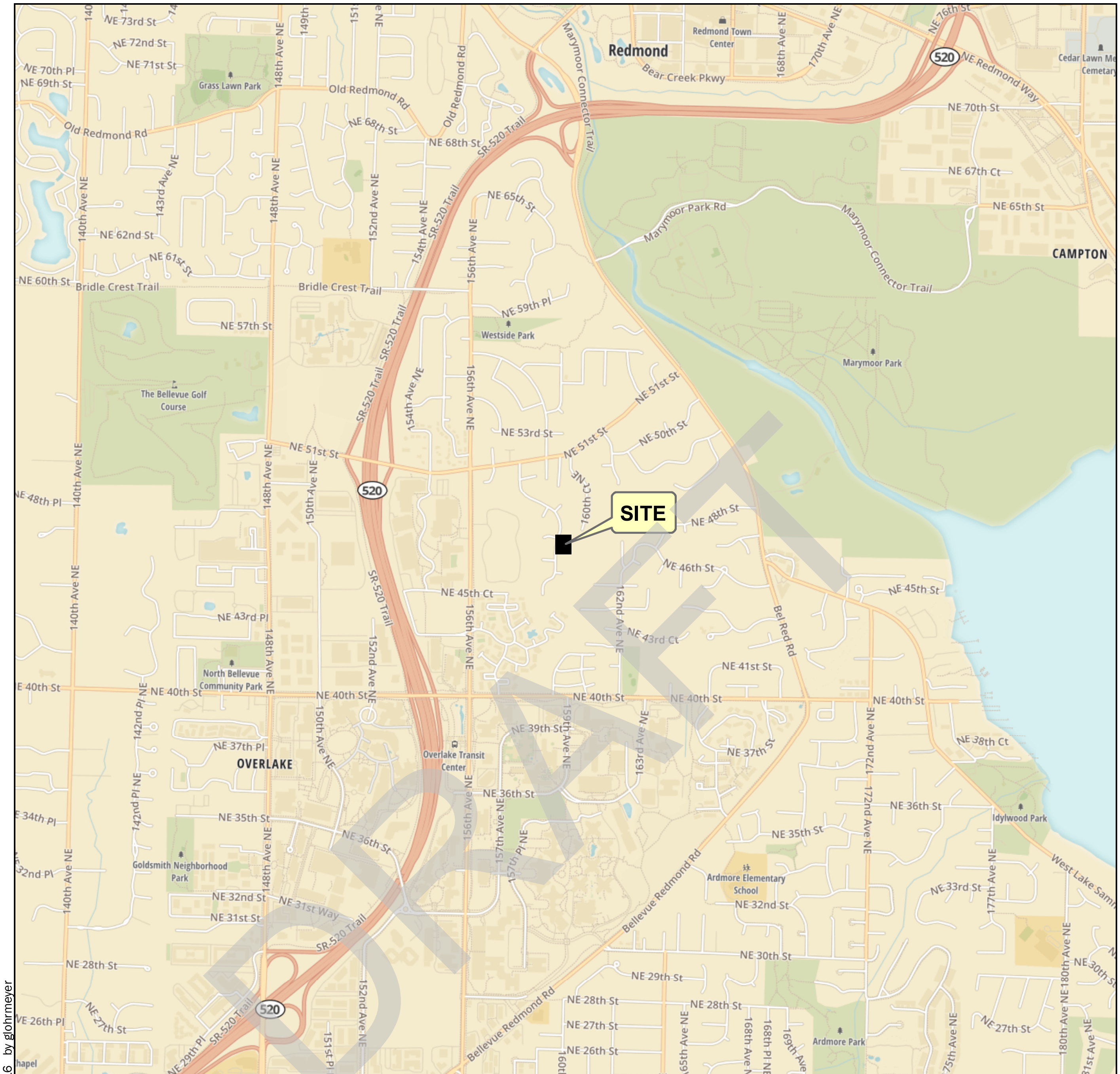

N
W E
S

2,000 0 2,000

Feet

Notes:

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

Data Source: Mapbox Open Street Map, 2015

Projection: NAD 1983 StatePlane Washington North FIPS 4601 Feet

Vicinity Map

159th Avenue NE Vault in Purchased Parcels
Tosh Creek Watershed Restoration
Redmond, Washington

GEOENGINEERS

Figure 1

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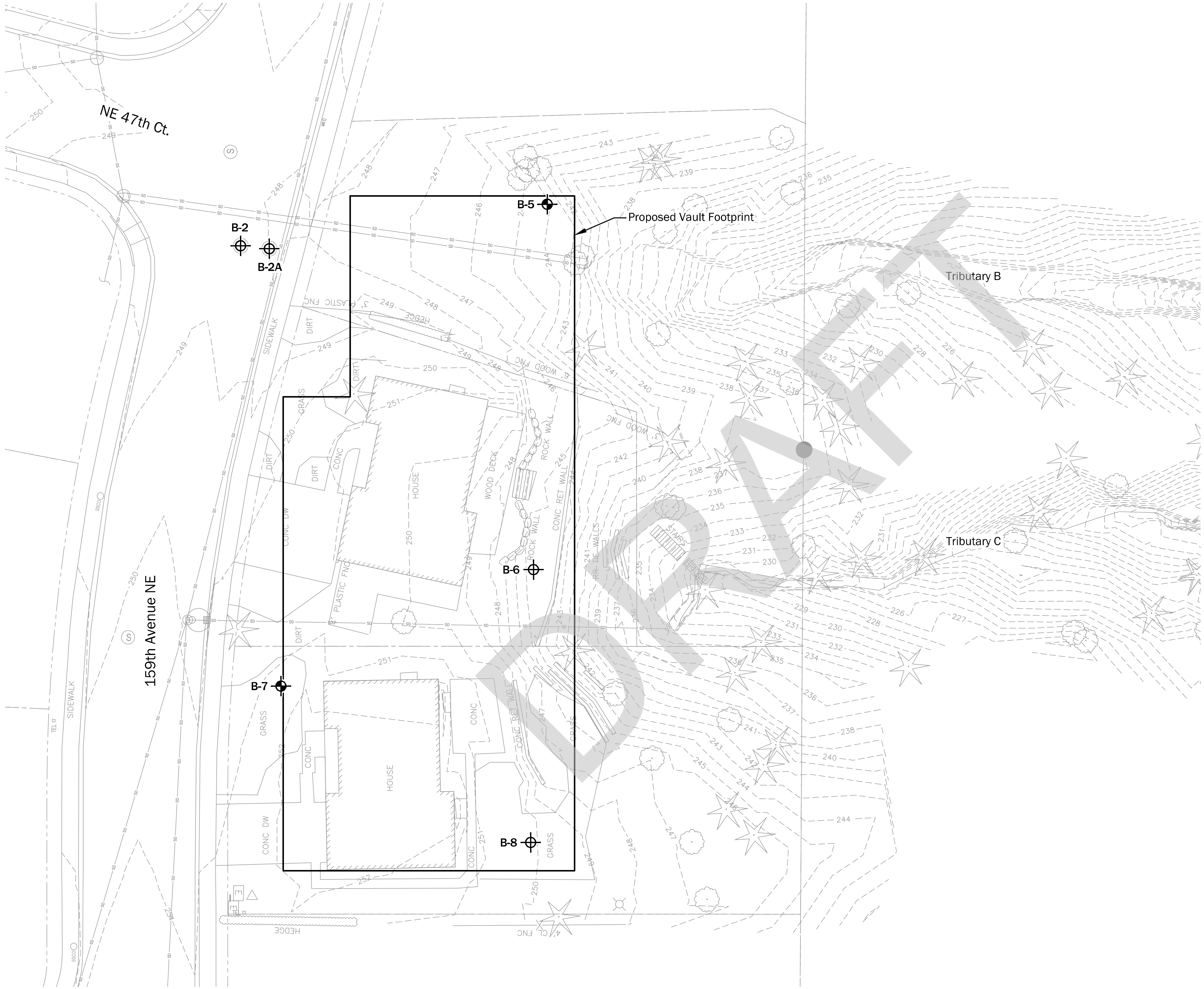

Legend

B-5 Monitoring Well completed by GeoEngineers Inc., 2015, 2016

B-2 Boring completed by GeoEngineers Inc., 2015, 2016

N E S W

30 0 30

Feet

Notes:

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

Data Source:
Base drawing provided by HDR Engineering, Inc., dated 05/25/16.

Vertical Datum: MLLW (NAVD 88).

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

Site Plan

159th Avenue NE Vault in Purchased Parcels
Tosh Creek Watershed Restoration
Redmond, Washington

GEOENGINEERS

Figure 2

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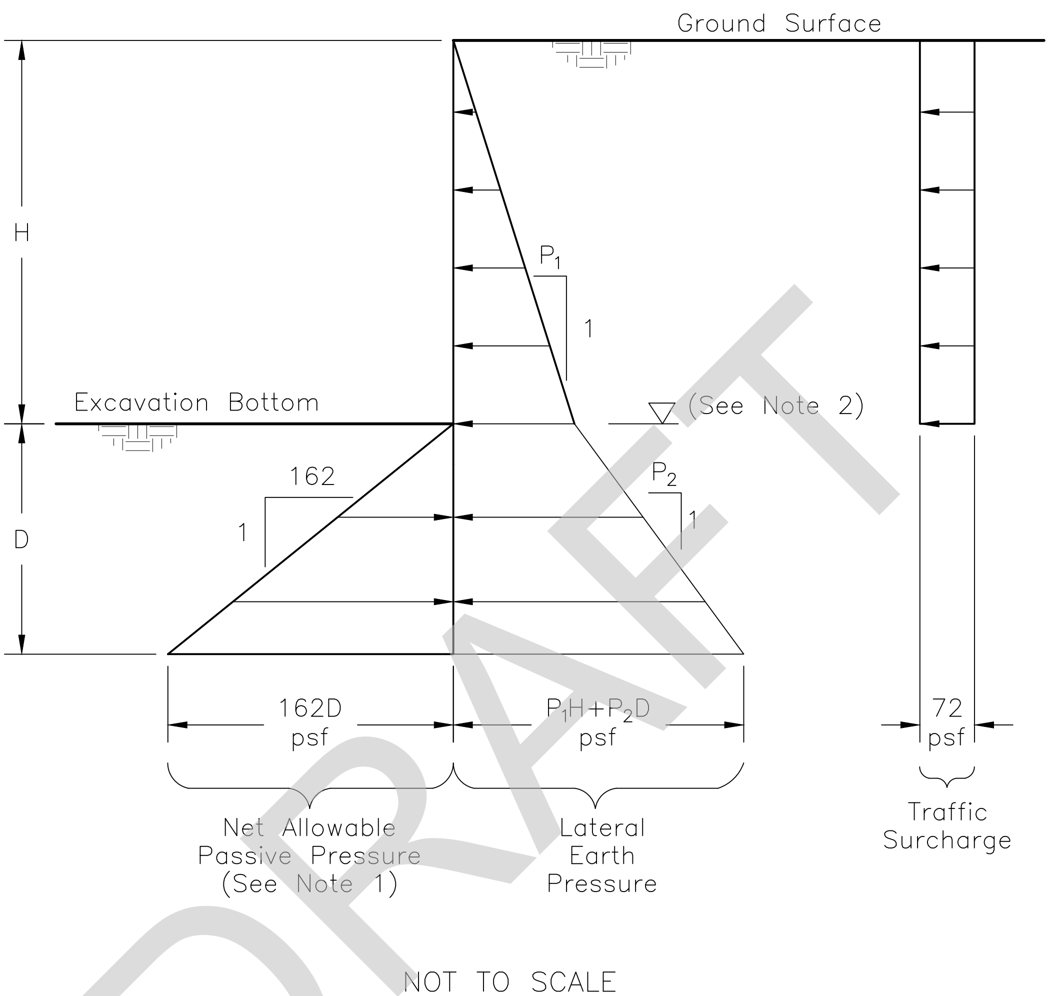


Legend

H = Depth Below Finished Ground Surface, Feet

D = Foundation Embedment, Feet

P = Equivalent Fluid Density (pcf)

P (pcf)	P_1	P_2
Active	36	80
At-Rest	59	91

Notes:
1. Net allowable passive pressure includes a factor of safety of 1.5 and acts over 2.5 times the shaft diameter, but no more than the pile spacing.
2. Earth pressure diagram assumes drained conditions above the excavation bottom and submerged conditions below the excavation bottom.
3. If additional surcharge (such as from soil stockpiles, excavators, dumptrucks, cranes, or concrete trucks) loading is anticipated, GeoEngineers should be consulted to provide revised surcharge pressures.

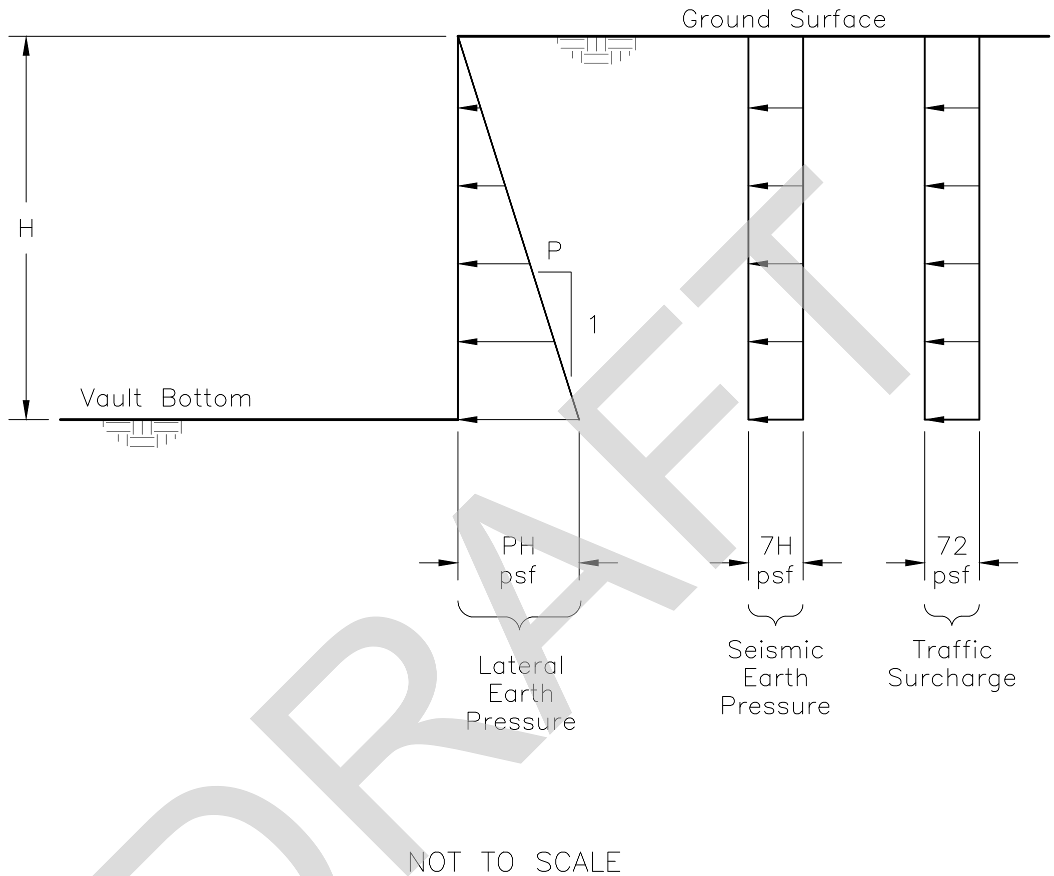
Temporary Shoring Earth Pressure Diagram

159th Avenue NE Vault in Purchased Parcels
Tosh Creek Watershed Restoration
Redmond, Washington

GEOENGINEERS

Figure 3

P:\0\0500204\CAD\0500204-00 Figure 3 & 4 EPD.dwg TAB:Fig 3 Date Exported: 06/29/16 - 10:14 by hmara



Legend

H = Depth Below Finished Ground Surface, Feet

P = Equivalent Fluid Density (pcf)

P (pcf)	Active	At-Rest
Drained	36	59
Submerged	80	91

(See Note 1)

Notes:

1. For vaults constructed without wall drainage use submerged values.
2. If additional surcharge (such as from soil stockpiles, excavators, dumptrucks, cranes, or concrete trucks) loading is anticipated, GeoEngineers should be consulted to provide revised surcharge pressures.

Permanent Vault Earth Pressure Diagram

159th Avenue NE Vault in Purchased Parcels
Tosh Creek Watershed Restoration
Redmond, Washington

GEOENGINEERS

Figure 4

P:\0\0500204\CAD\0500204-00 Figure 3 & 4 EPD.dwg TAB:Fig 4 Date Exported: 06/29/16 - 10:15 by hmara

APPENDIX A
Field Explorations

APPENDIX A
FIELD EXPLORATIONS

Subsurface conditions at the site were explored by drilling borings B-2 and B-2A (previous study) and B-5 through B-8 (this study). The borings were completed to depths ranging to 41.5 feet below the existing ground surface. The drilling was performed by Holocene Drilling, Inc. and Geologic Drill, Inc. under subcontract to GeoEngineers on September 14 and 21, 2015, and May 23 and 24, 2016.

The locations of the explorations were estimated by taping/pacing from existing site features. The exploration locations are shown on Figure 2, Site Plan.

The borings were completed using continuous-flight, hollow-stem auger drilling equipment. The borings were continuously monitored by an engineer or geologist from our firm who examined and classified the soils encountered, obtained representative soil samples, observed groundwater conditions, and prepared a detailed log of the explorations.

The soils encountered in the borings were sampled at 2.5- or 5-foot vertical intervals with a 2-inch outside diameter split-barrel standard penetration test (SPT) sampler. The samples were obtained by driving the sampler 18 inches into the soil with a 140-pound hammer free-falling 30 inches. The number of blows required for each 6 inches of penetration was recorded. The blow count ("N value") of the soil was calculated as the number of blows required for the final 12 inches of penetration. This resistance, or N value, provides a measure of the relative density of granular soils and the relative consistency of cohesive soils. Where very dense soil conditions precluded driving the full 18 inches, the penetration resistance for the partial penetration was entered on the logs. The blow counts are shown on the boring logs at the respective sample depths.

Soils encountered in the borings were visually classified in general accordance with the classification system described in Figure A-1, Key to Exploration Logs. A key to the boring log symbols is also presented in Figure A-1. The logs of the borings are presented in Figures A-2 through A-7. The boring logs are based on our interpretation of the field and laboratory data and indicate the various types of soils and groundwater conditions encountered. The logs also indicate the depths at which these soils or their characteristics change, although the change may actually be gradual. If the change occurred between samples, it was interpreted. The densities noted on the boring logs are based on the blow count data obtained in the borings and judgment based on the conditions encountered.

Observations of groundwater conditions were made during drilling and are included on the boring logs. These observations represent a short-term condition and may or may not be representative of the long-term groundwater conditions at the site. Groundwater conditions observed during drilling should be considered approximate. Standpipe piezometers (monitoring wells) were installed in borings B-5 and B-7. Installation details of the monitoring well and measured groundwater levels in the monitoring wells are included in Figures A-4 and A-6.

SOIL CLASSIFICATION CHART

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

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

Sampler Symbol Descriptions

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

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

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

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

ADDITIONAL MATERIAL SYMBOLS

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

Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata

Approximate contact between soil strata

Material Description Contact

Contact between geologic units

Contact between soil of the same geologic unit

Laboratory / Field Tests

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

Sheen Classification

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

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

KEY TO EXPLORATION LOGS

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FIGURE A-1

Rev. 02/16

Drilled	Start	End	Total Depth (ft)	Logged By	Checked By	Driller	Drilling Method
	9/21/2015	9/21/2015	36.5	EJC	CMK	Holocene Drilling, Inc.	Hollow-Stem Auger

Surface Elevation (ft) / Vertical Datum	248.34 / NAVD88	Hammer Data	Automatic 140 (lbs) / 30 (in) Drop	Drilling Equipment	Mobile B-59 Truck-Mounted
Easting (X) / Northing (Y)	1321400.197 / 240388.692	System Datum	WA State Plane,North NAD83 (feet)	Groundwater	

Notes: Auger Data: 4-inch I.D.; 8½-inch O.D.

Groundwater Date Measured	Depth to Water (ft)	Elevation (ft)
	Not observed	

FIELD DATA

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0								AC	4 inches asphalt concrete			
									CR	3 inches base course			
									SM	Brown silty fine to coarse sand with gravel (medium dense, moist) (fill)			Rough drilling at 1 to 2 feet
245			9	13		1 / SA					8	19	
									GM	Brown silty coarse gravel with sand (loose, moist)			
	5		9	6		2 / SA					12	34	
									SM	Gray silty fine to medium sand with gravel (medium dense, moist) (glacially consolidated soils)			
240			8	29		3 / SA					13	22	
									ML	Gray silt with sand (very stiff, moist)			
	10		18	35		4 / AL					24		AL (LL = 29; PI = 6)
235			18	46		5							
									CL	Gray lean clay with sand (stiff to hard, moist)			
	15		18	56		6							
230													
	20		18	44		7 / AL					29		AL (LL = 36; PI = 14)
225													
	25		18	20		8							

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

Log of Boring B-2

GEOENGINEERS

Project: 159th Ave. NE Vault in Purchased Parcels
Project Location: Redmond, Washington
Project Number: 0500-204-00

Figure A-2
Sheet 1 of 2

Elevation (feet)	Depth (feet)	Interval Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
220												
	30		27		9							
215												
	35		38		10				Becomes moist to wet			

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

Log of Boring B-2 (continued)

GeoEngineers

Project: 159th Ave. NE Vault in Purchased Parcels
Project Location: Redmond, Washington
Project Number: 0500-204-00

Drilled	Start 9/14/2015	End 9/14/2015	Total Depth (ft) 7.5	Logged By EJC / Checked By CMK	Driller Holocene Drilling, Inc.	Drilling Method Hollow-Stem Auger

Surface Elevation (ft) 247.98 / Vertical Datum NAVD88	Hammer Data: Automatic, 140 (lbs) / 30 (in) Drop	Drilling Equipment	Mobile B-59 Truck-Mounted
Easting (X) 1321408.77 / Northing (Y) 240387.8154	System Datum: WA State Plane,North NAD83 (feet)	Groundwater	Date Measured / Depth to Water (ft) / Elevation (ft)

Notes: Auger Data: 4-inch I.D.; 8½-inch O.D.

Elevation (feet)	Depth (feet)	FIELD DATA: Interval / Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							AC	4 inches asphalt concrete			
								CR	3 to 4 inches base course			
								SM	Gray silty sand with gravel (loose, moist) (fill)			
245		9	8		1							
								SM	Brownish gray silty sand with gravel (loose, moist)			
	5	2	5		2							

Boring terminated due to presence of storm drain

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

DRAFT

Log of Boring B-2a

GeoEngineers

Project: 159th Ave. NE Vault in Purchased Parcels
Project Location: Redmond, Washington
Project Number: 0500-204-00

Drilled	Start 5/23/2016	End 5/23/2016	Total Depth (ft) 41.5	Logged By DTM / Checked By CMK	Driller Geologic Drill, Inc.	Drilling Method Hollow-Stem Auger

Hammer Data	Automatic 140 (lbs) / 30 (in) Drop	Drilling Equipment	Track-Mounted Diedrich D-50
Surface Elevation (ft) / Vertical Datum	244.11 / NAVD88	Top of Casing Elevation (ft)	243.67
Easting (X) / Northing (Y)	1321491.737 / 240401.0894	Horizontal Datum	WA State Plane,North NAD83 (feet)

DOE Well I.D.: BIK-349
A 2 (in) well was installed on 5/23/2016 to a depth of 39.3 (ft).

Groundwater

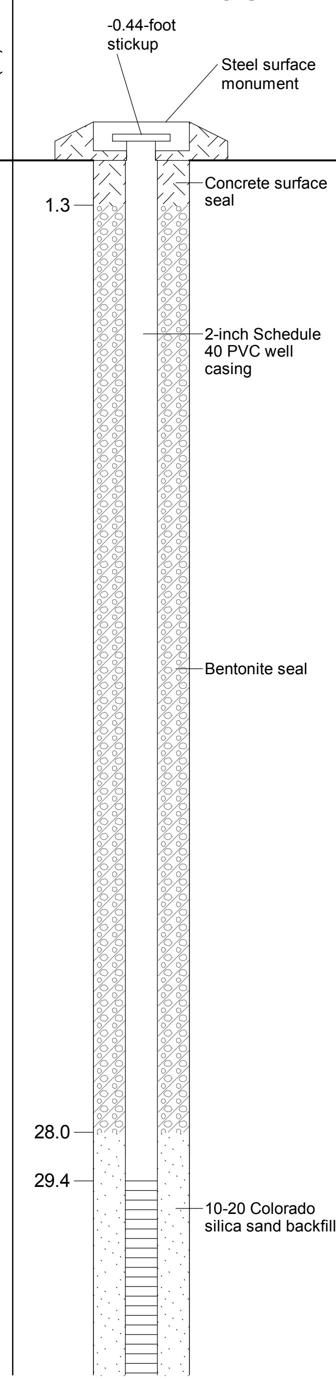
Date Measured	Depth to Water (ft)	Elevation (ft)
5/26/2016	12.1	231.6

Notes: Auger Data: 4¼-inch I.D.; 8-inch O.D.

Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
0				TS	3 inches forest duff and topsoil		
				SM	Brown-gray and dark brown silty fine to medium sand with roots and occasional gravel (medium dense, moist) (fill)		
	8	18	1				
5	18	6	2	SM	Brown and dark brown silty fine to medium sand with gravel and roots (very loose to loose, moist)		
	15	3	3 %F	SM	Gray silty fine to medium sand with occasional gravel and organic matter (very loose, moist) (fill/native?)	17	37
10	15	3	4		Becomes wet		
					(Drilling becomes harder)		
15	18	39	5 AL	CL	Gray lean clay with occasional sand (hard, moist) (glacially consolidated soils) AL (LL = 33; PI = 10)	22	
20	18	49	6				
				CL	Sandy lean clay (hard, moist to wet)		
25	9	51	7				
30	15	35	8 AL		AL (LL = 33; PI = 10)	25	
				ML	Silt with occasional sand (hard, moist to wet)		
35							

Elevation (feet): 240, 235, 230, 225, 220, 215, 210

WELL LOG



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

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Log of Monitoring Well B-5

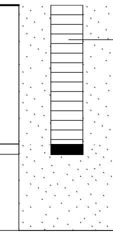
GEOENGINEERS

Project: 159th Avenue NE Vault in Purchased Parcels
Project Location: Redmond, Washington
Project Number: 0500-204-00

Figure A-4
Sheet 1 of 2

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
	35		18	35		9 AL				AL (LL = 34; PI = 8)	27	
205												
	40		18	37		10				Becomes wet		

WELL LOG



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

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Log of Monitoring Well B-5 (continued)

GeoEngineers

Project: 159th Avenue NE Vault in Purchased Parcels
Project Location: Redmond, Washington
Project Number: 0500-204-00

Drilled	Start 5/24/2016	End 5/24/2016	Total Depth (ft)	41.5	Logged By DTM Checked By CMK	Driller	Geologic Drill, Inc.	Drilling Method	Hollow-Stem Auger

Surface Elevation (ft) Vertical Datum	244.76 NAVD88	Hammer Data	Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment	MT52 Mini-Track
Easting (X) Northing (Y)	1321487.634 240291.9779	System Datum	WA State Plane,North NAD83 (feet)	Groundwater	Date Measured / Depth to Water (ft) / Elevation (ft)
Notes: Auger Data: 2¼-inch I.D.; 5 3/8-inch O.D.					Not observed

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0				SOD	3 inches sod			
					SM	Brown silty fine to medium sand with occasional gravel and organic matter (wood fragments) (loose, moist to wet) (fill)			
		10	5	1A 1B	ML	Gray silt with sand and occasional gravel (soft to medium stiff, moist to wet)			
240	5	18	3	2					
		3	7	3	GP-GM	Fine to coarse gravel with silt and sand (loose, moist to wet)			
					OL	Dark brown with gray and black mottling sandy organic silt with occasional gravel and wood debris (medium stiff, moist to wet) (old topsoil horizon?)			
235	10	16	10*	4 OC			46		*Blow count overstated due to driving on wood debris OC = 9.4%
		<1	50/6"	5	ML	Gray sandy silt (hard, moist) (glacially consolidated soils)			
230	15	17	91/11"	6 %F AL			21	68	AL (LL = 27; PI = 4)
225	20	18	53	7					
220	25	18	46	8 AL			27		AL (LL = 28; PI = 2)
215	30	18	27	9		Becomes very stiff			
					ML	Gray silt (hard, wet)			
210	35								

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

Log of Boring B-6

GeoEngineers

Project: 159th Avenue NE Vault in Purchased Parcels
Project Location: Redmond, Washington
Project Number: 0500-204-00

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	35		18	32		10 / AL					44		AL (LL = 32; PI = 7)
205	40		14	42		11							

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

Drilled Start 5/23/2016 End 5/23/2016	Total Depth (ft) 41.5	Logged By DTM Checked By CMK	Driller Geologic Drill, Inc.	Drilling Method Hollow-Stem Auger

Hammer Data	Automatic 140 (lbs) / 30 (in) Drop	Drilling Equipment	Track-Mounted Diedrich D-50	DOE Well I.D.: BIK-348 A 2 (in) well was installed on 5/23/2016 to a depth of 39.7 (ft).
Surface Elevation (ft) Vertical Datum	251.43 NAVD88	Top of Casing Elevation (ft)	250.97	
Easting (X) Northing (Y)	1321412.248 240257.1859	Horizontal Datum	WA State Plane,North NAD83 (feet)	

Groundwater

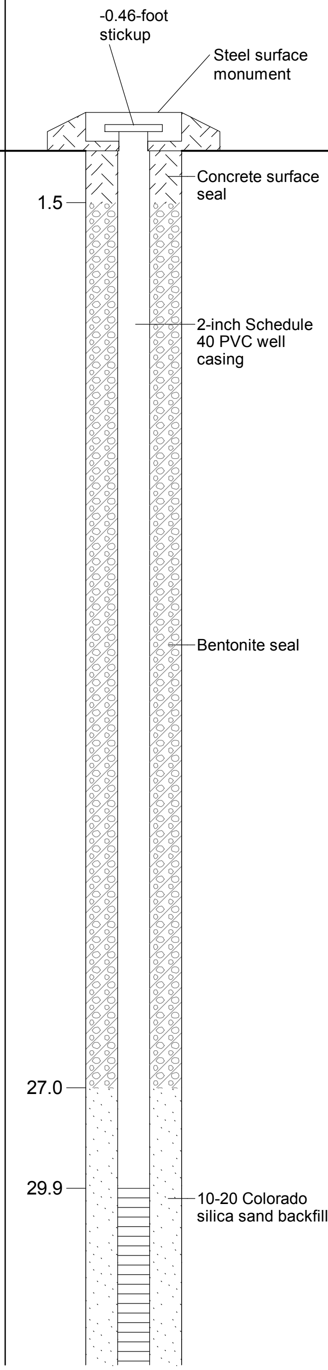
Date Measured	Depth to Water (ft)	Elevation (ft)
5/26/2016	17.2	233.8

Notes: Auger Data: 4¼-inch I.D.; 8-inch O.D.

FIELD DATA

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Water Level	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
	0					SOD	2 inches sod		
250						SM	Brown silty fine to medium sand with occasional gravel and roots (loose to medium dense, moist) (fill)		
		18	11*	1A 1B		SM	Brown and gray silty fine to medium sand with occasional gravel and organic matter (loose to medium dense, moist)		
245	5	18	5	2 SA			(*Blow count may be ovserstated due to roots)	15	43
		9	4	3					
240	10	18	10	4			Becomes dark brown with organic material		
							Perched groundwater encountered between 12 and 14 feet during drilling		
						GM	Silty gravel (loose, moist to wet)		
						ML	Gray sandy silt (hard, moist) (glacially consolidated soils)		
235	15	18	34	5 %F AL	▼		AL (LL = 28; PI = 5)	21	68
						CL	Gray lean clay (hard, moist)		
230	20	18	36	6 AL			AL (LL = 35; PI = 13)	25	
225	25	18	35	7 AL			AL (LL = 36; PI = 14)	26	
220	30	18	34	8					
	35								

WELL LOG



DRAFT

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

Redmond: Date:6/29/16 Path:W:\PROJECTS\0\0500204\GINT\050020400.GPJ DBTemplate/LibTemplate:GEOENGINEERS_DF_STD_US.GDT/GEI8_GEOTECH_WELL_%F

Log of Monitoring Well B-7

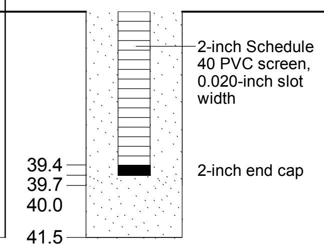
GeoEngineers

Project: 159th Avenue NE Vault in Purchased Parcels
Project Location: Redmond, Washington
Project Number: 0500-204-00

Figure A-6
Sheet 1 of 2

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
215	35		18	43		9				Becomes moist to wet		
210	40		18	30		10 / AL				Becomes stiff AL (LL = 38; PI = 14)	26	

WELL LOG



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

Drilled	Start 5/24/2016	End 5/24/2016	Total Depth (ft) 41.5	Logged By DTM / Checked By CMK	Driller Geologic Drill, Inc.	Drilling Method Hollow-Stem Auger

Surface Elevation (ft) / Vertical Datum	250.12 / NAVD88	Hammer Data	Automatic 140 (lbs) / 30 (in) Drop
Drilling Equipment	MT52 Mini-Track		
Easting (X) / Northing (Y)	1321486.793 / 240210.4854	System Datum	WA State Plane,North NAD83 (feet)
Notes: Auger Data: 2¼-inch I.D.; 5 3/8-inch O.D.			

Groundwater Date Measured	Depth to Water (ft)	Elevation (ft)
	Not observed	

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0				SOD	3 inches sod			
					ML	Brown silt with sand and occasional roots (medium stiff, moist) (fill)			
		18	11	1A 1B					
					SM	Brownish gray silty fine to medium sand with occasional gravel (loose, moist)			
245	5	18	14	2 SA		Becomes moist to wet	14	44	
					ML	Brownish gray sandy silt with occasional gravel (hard, moist) (glacially consolidated soils)			
		12	50/6"	3 SA			11	57	
240	10	9	50/3"	4		Increasing silt content and decreasing gravel content			
					CL-ML	Gray silty clay with occasional sand (hard, moist)			
235	15	18	48	5 AL			20		AL (LL = 29; PI = 7)
230	20	18	38	6 %F			26	73	
					ML	Gray silt with sand (very stiff to hard, moist)			
225	25	18	33	7 AL			27		AL (LL = 32; PI = 6)
220	30	18	34	8					
	35								

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

Log of Boring B-8

GeoEngineers

Project: 159th Avenue NE Vault in Purchased Parcels
Project Location: Redmond, Washington
Project Number: 0500-204-00

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	35		18	38		9 AL					29		AL (LL = 30; PI = 6)
210	40		18	33		10				Becomes wet			

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

Log of Boring B-8 (continued)

GeoEngineers

Project: 159th Avenue NE Vault in Purchased Parcels
Project Location: Redmond, Washington
Project Number: 0500-204-00

APPENDIX B
Laboratory Testing

APPENDIX B
LABORATORY TESTING

Soil samples obtained from the explorations were transported to our laboratory and examined to confirm or modify field classifications, as well as to evaluate index properties of the soil samples. Representative samples were selected for laboratory testing consisting of moisture content, sieve analyses (grain size distribution), and Atterberg limits (plasticity characteristics). The tests were performed in general accordance with test methods of the American Society for Testing and Materials (ASTM) or other applicable procedures.

Moisture Content Testing

Moisture content tests were completed in general accordance with 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.

Percent Passing U.S. No. 200 Sieve

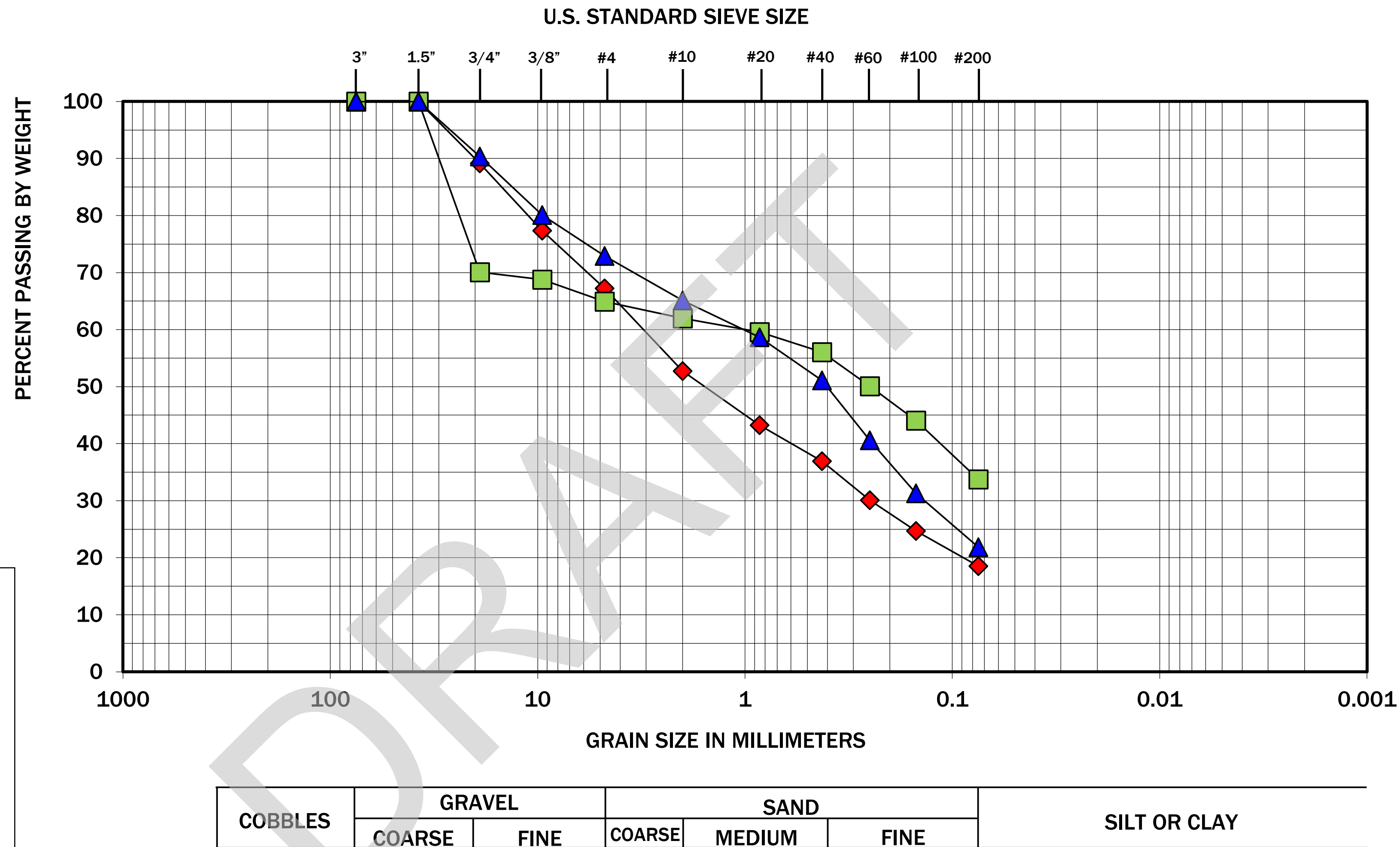
Selected samples were "washed" through the No. 200 mesh sieve to estimate the relative percentages of coarse and fine-grained particles in the soil. The percent passing value represents the percentage by weight of the sample finer than the U.S. No. 200 sieve. These tests were conducted to verify field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

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.

Atterberg Limits Testing

Atterberg limits testing was performed on selected fine-grained soil samples. The tests were used to classify the soil as well as to evaluate index properties. The liquid limit and the plastic limit were estimated through a procedure performed in general accordance with ASTM D 4318. The results of the Atterberg limits testing are summarized in Figures B-3 through B-7.



Symbol	Boring Number	Depth (feet)	Moisture (%)	Soil Description
◆	B-2	2.5	8	Silty fine to coarse sand with gravel (SM)
■	B-2	5	12	Silty coarse gravel with sand (GM)
▲	B-2	7.5	13	Silty fine to medium sand with gravel (SM)

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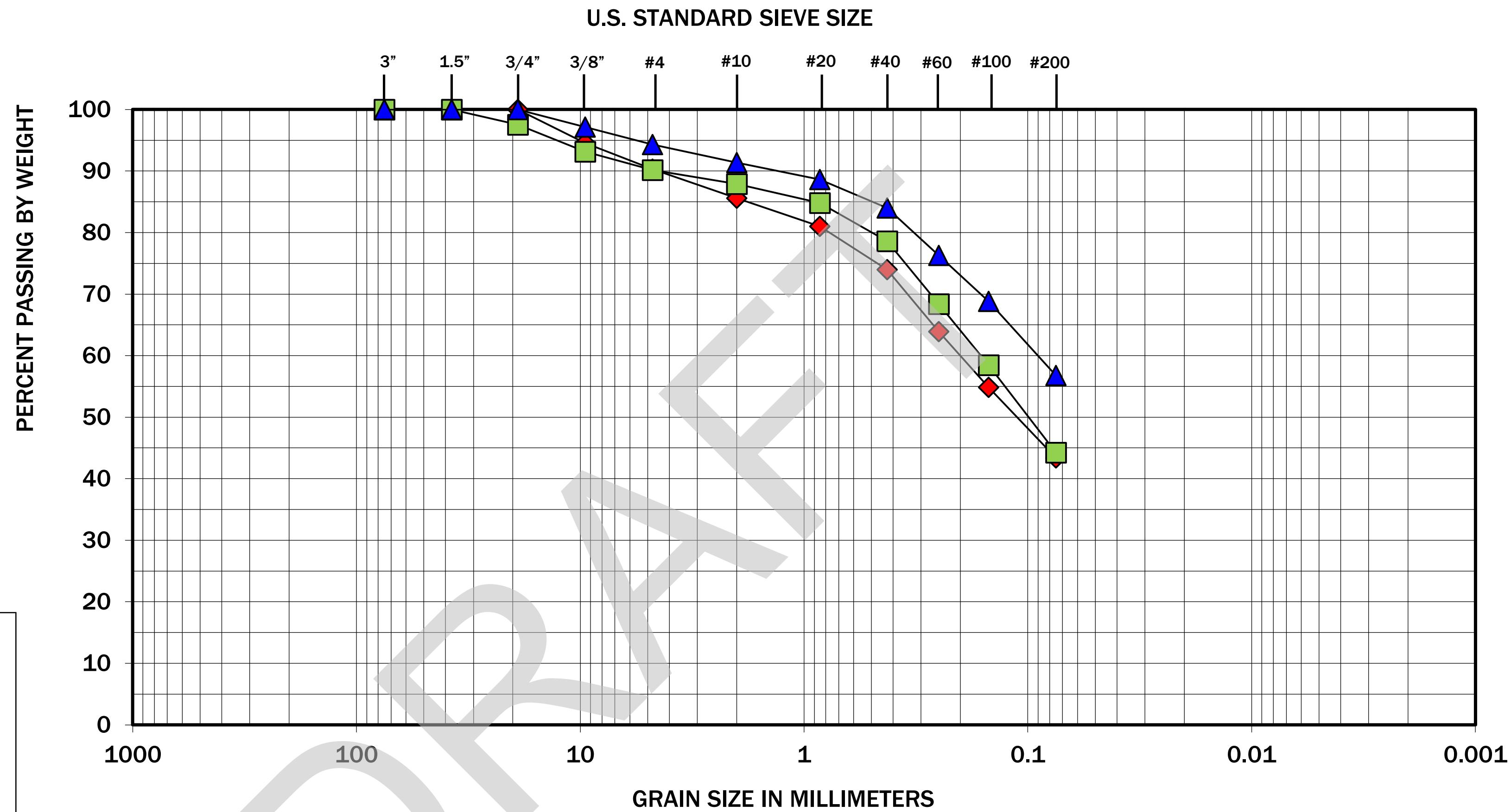
The grain size analysis results were obtained in general accordance with ASTM D 6913.

Sieve Analysis Results

159th Avenue NE Vault in Purchased Parcels
Tosh Creek Watershed Restoration
Redmond, Washington

GeoEngineers

Figure B-1



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Symbol	Boring Number	Depth (feet)	Moisture (%)	Soil Description
◆	B-7	5	15	Silty fine to medium sand with occasional gravel (SM)
■	B-8	5	14	Silty fine to medium sand with occasional gravel (SM)
▲	B-8	7½	11	Sandy silt with occasional gravel (ML)

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

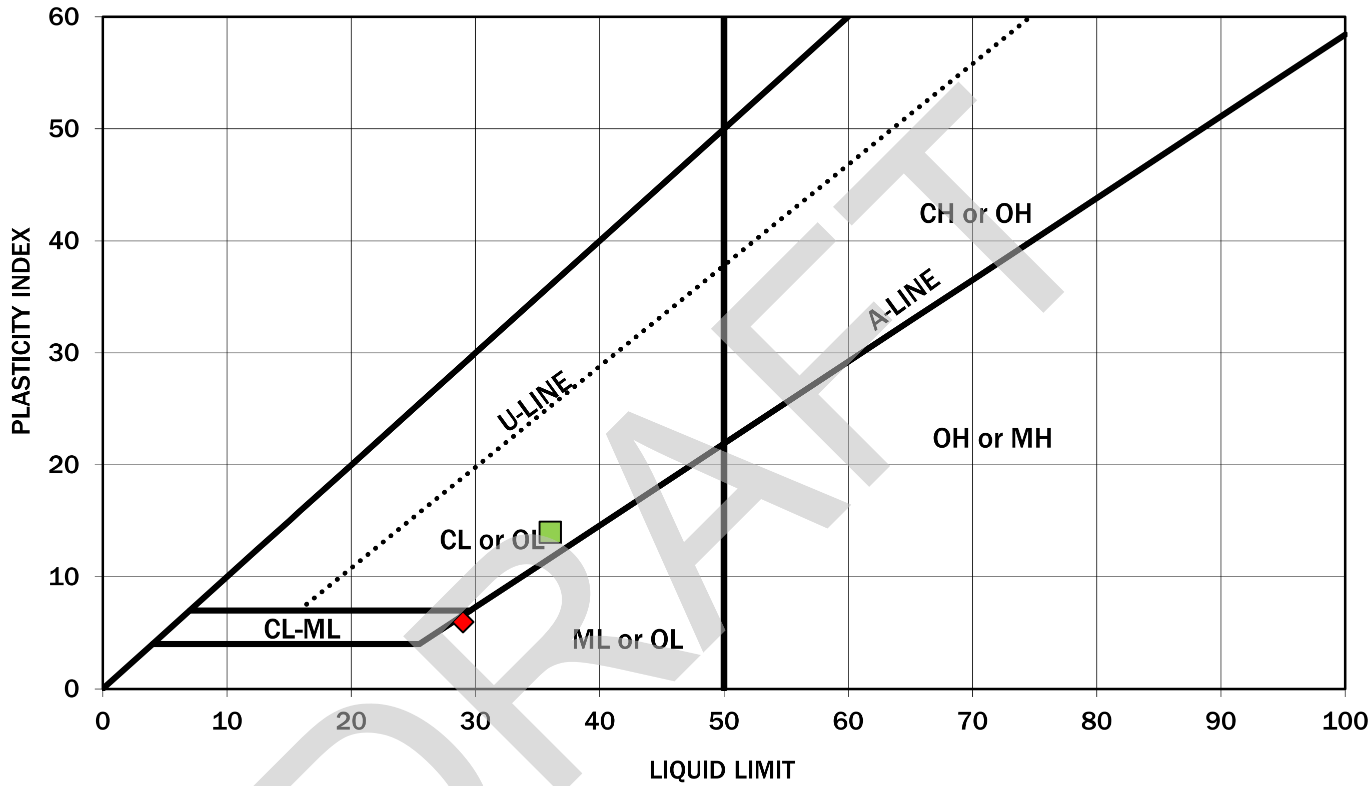
Sieve Analysis Results

159th Avenue NE Vault in Purchased Parcels
Tosh Creek Watershed Restoration
Redmond, Washington

GEOENGINEERS

Figure B-2

PLASTICITY CHART



Symbol	Boring Number	Depth (feet)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Soil Description
◆ (red diamond)	B-2	10	24	29	6	Silt with sand (ML)
■ (green square)	B-2	20	29	36	14	Lean clay with sand (CL)

The liquid limit and plasticity index were obtained in general accordance with ASTM D 4318.

Atterberg Limits Test Results

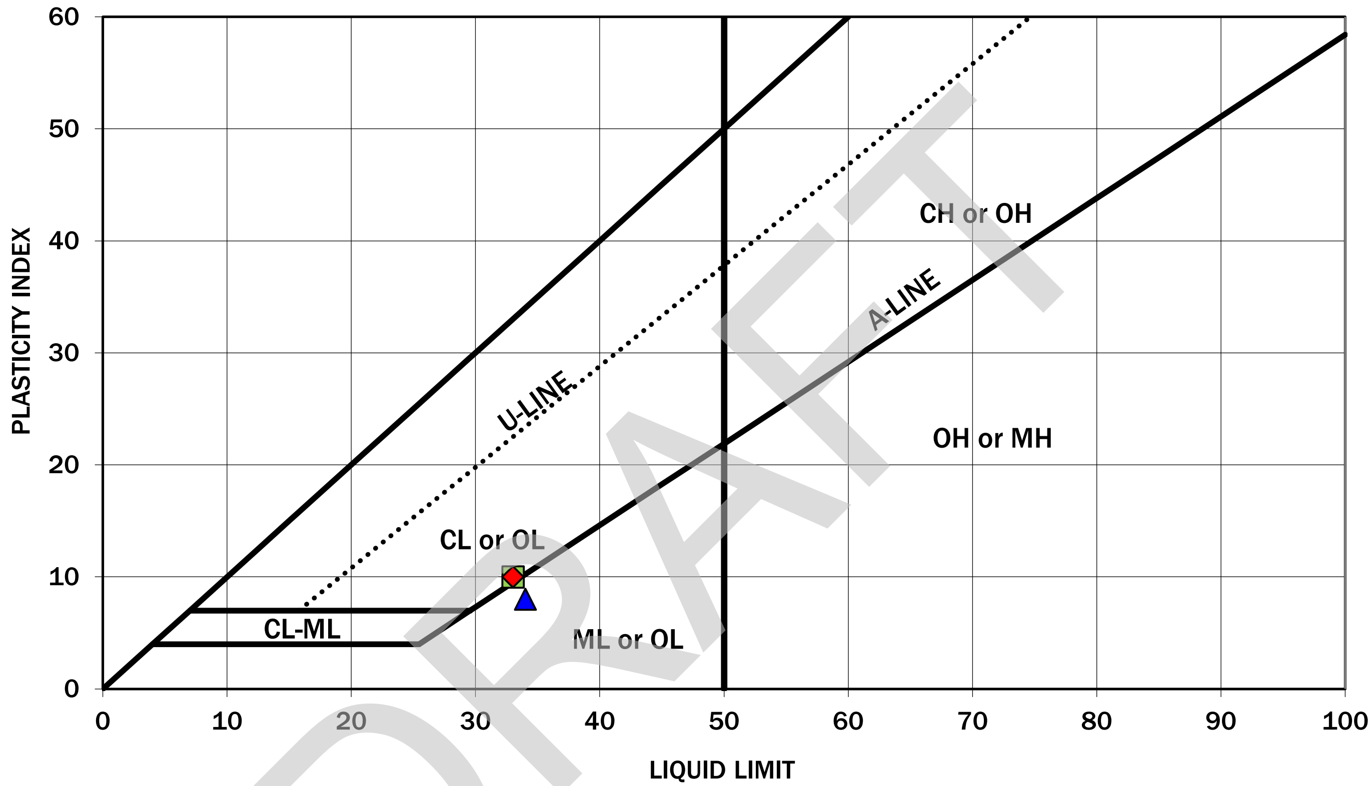
159th Avenue NE Vault in Purchased Parcels
Tosh Creek Watershed Restoration
Redmond, Washington

GEOENGINEERS

Figure B-3

0500-204-00 Date Exported: 10/22/15

PLASTICITY CHART



Symbol	Boring Number	Depth (feet)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Soil Description
◆	B-5	15	22	33	10	Lean clay with occasional sand (CL)
■	B-5	30	25	33	10	Sandy lean clay (CL)
▲	B-5	35	27	34	8	Silt with occasional sand (ML)

The liquid limit and plasticity index were obtained in general accordance with ASTM D 4318.

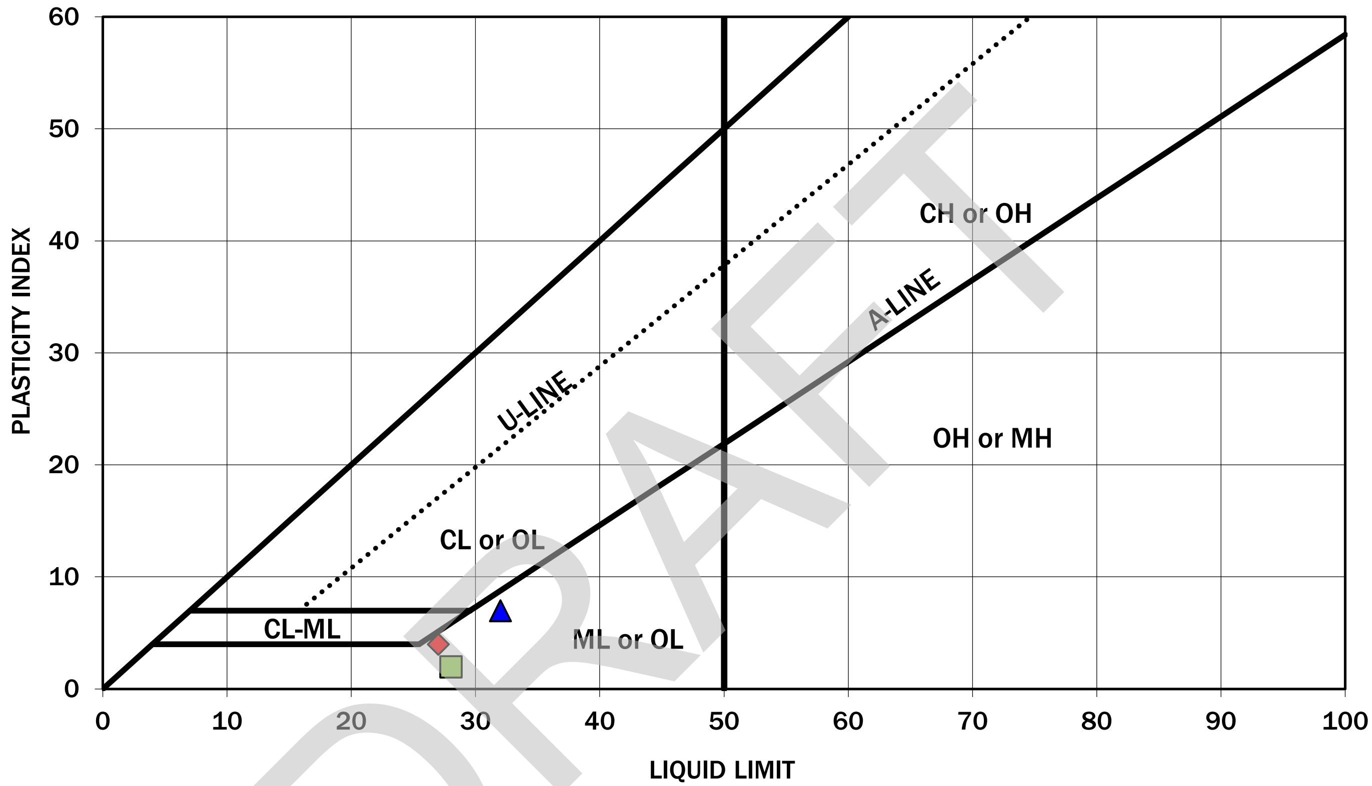
Atterberg Limits Test Results

159th Avenue NE Vault in Purchased Parcels
Tosh Creek Watershed Restoration
Redmond, Washington

GEOENGINEERS

Figure B-4

PLASTICITY CHART



Symbol	Boring Number	Depth (feet)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Soil Description
◆ (red diamond)	B-6	15	21	27	4	Sandy silt (ML)
■ (green square)	B-6	25	27	28	2	Sandy silt (ML)
▲ (blue triangle)	B-6	35	44	32	7	Silt (ML)

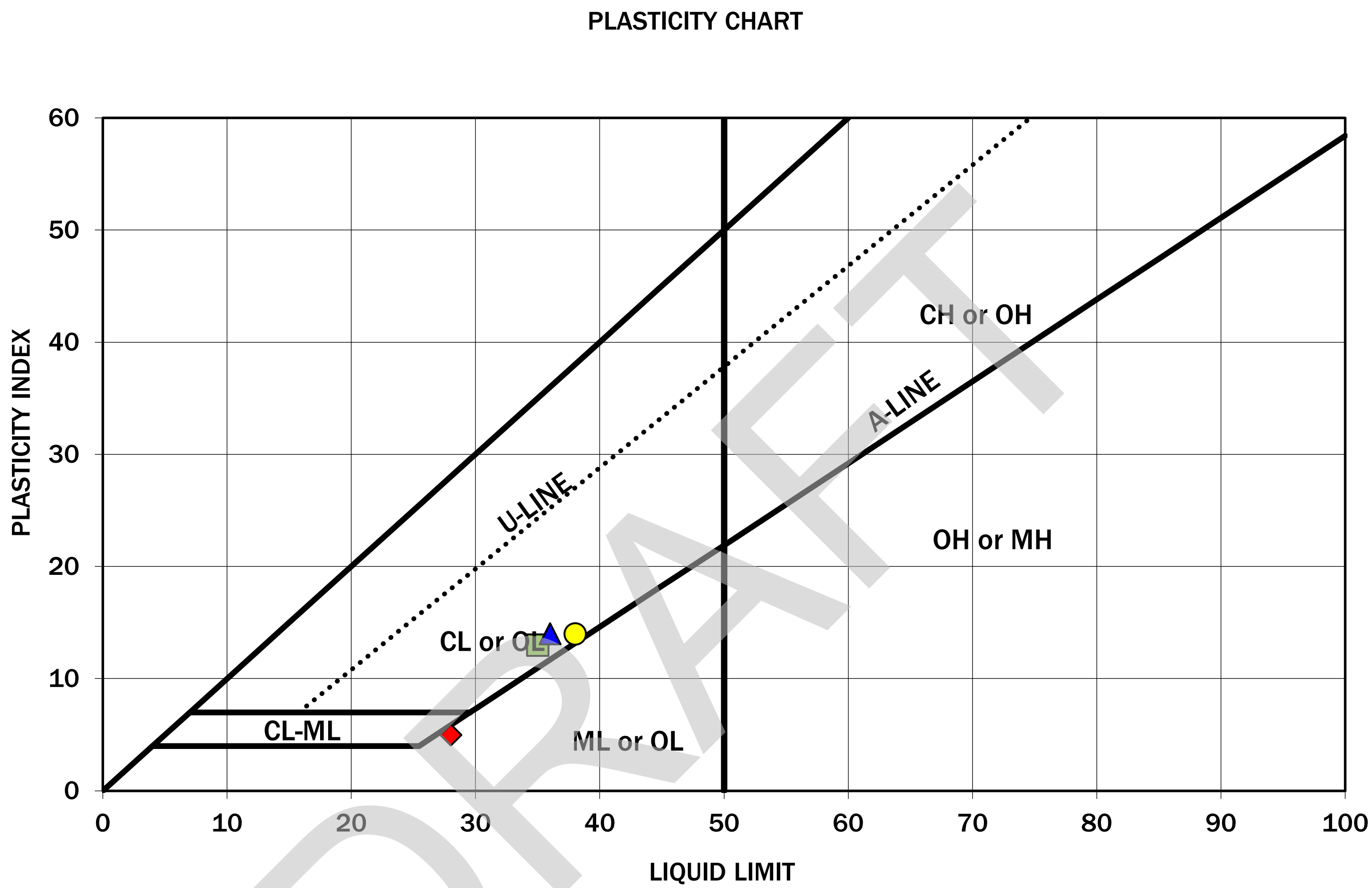
The liquid limit and plasticity index were obtained in general accordance with ASTM D 4318.

Atterberg Limits Test Results

159th Avenue NE Vault in Purchased Parcels
Tosh Creek Watershed Restoration
Redmond, Washington

GEOENGINEERS

Figure B-5



Symbol	Boring Number	Depth (feet)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Soil Description
◆ (red diamond)	B-7	15	21	28	5	Sandy silt (ML)
■ (green square)	B-7	20	25	35	13	Lean clay (CL)
▲ (blue triangle)	B-7	25	26	36	14	Lean clay (CL)
● (yellow circle)	B-7	40	26	38	14	Lean clay (CL)

The liquid limit and plasticity index were obtained in general accordance with ASTM D 4318.

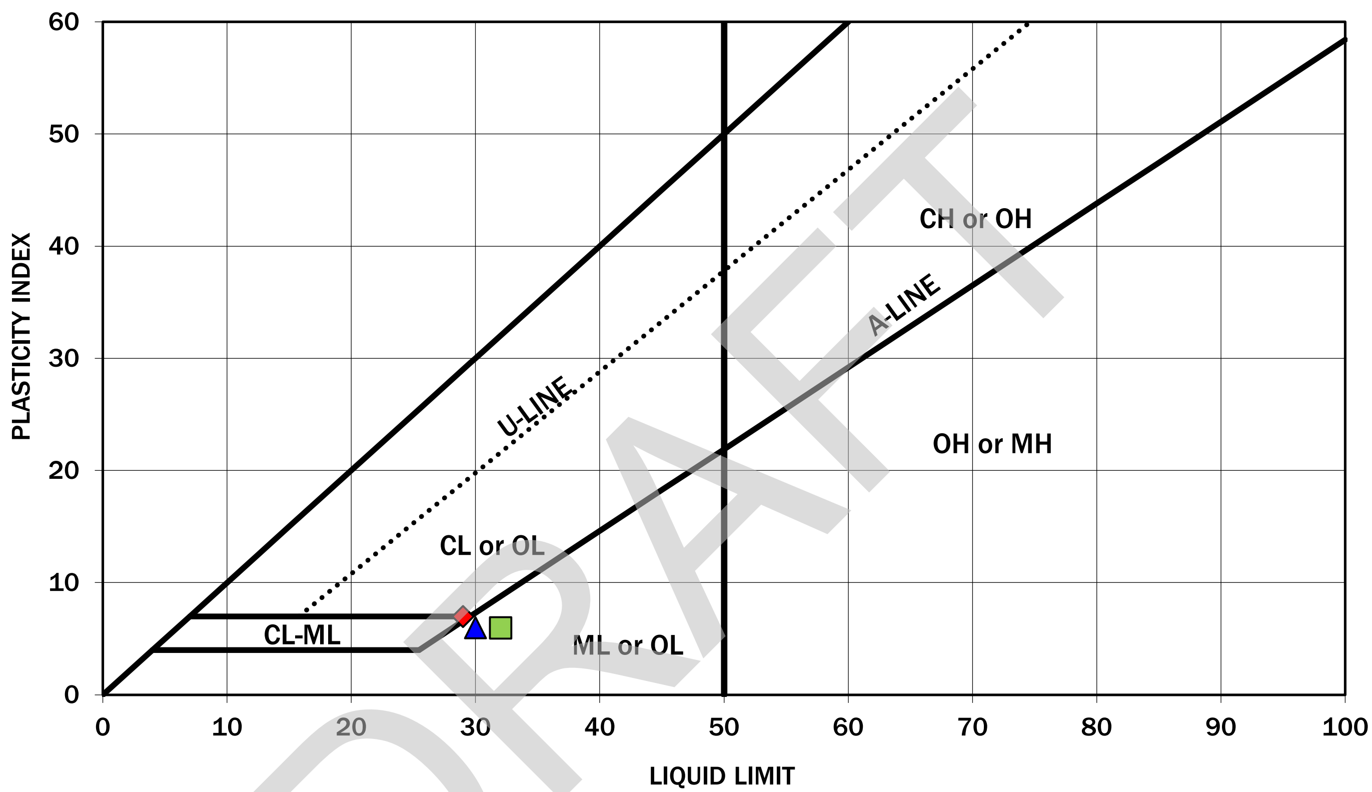
Atterberg Limits Test Results

159th Avenue NE Vault in Purchased Parcels
Tosh Creek Watershed Restoration
Redmond, Washington

GeoEngineers

Figure B-6

PLASTICITY CHART



Symbol	Boring Number	Depth (feet)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Soil Description
◆ (red diamond)	B-8	15	20	29	7	Silty clay (CL-ML)
■ (green square)	B-8	25	27	32	6	Silt with sand (ML)
▲ (blue triangle)	B-8	35	29	30	6	Silt with sand (ML)

The liquid limit and plasticity index were obtained in general accordance with ASTM D 4318.

Atterberg Limits Test Results

159th Avenue NE Vault in Purchased Parcels
Tosh Creek Watershed Restoration
Redmond, Washington

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Figure B-7

0500-204-00 Date Exported: 05/27/16

APPENDIX C
Field Report – Geologic Reconnaissance

GeoEngineers 8410 154th Avenue NE Redmond, WA 98052 425.861.6000	**Field Report**		File Number: 0500-204-00
	Project: Tosh Creek Watershed --159th Street Vault Alt. Location		Date: 5/17/2016
	Owner: City of Redmond	Time of Arrival: 0945	Report Number: FR-1
Prepared by: Benjamin Cashman, LG	Location: 159th Place NE, Redmond, WA	Time of Departure: 1145	Pages: 3
Purpose of visit: Geologic Reconnaissance	Weather: 70°F, overcast	Travel Time: 15 minutes	Permit Number:

Upon arrival to the site I assessed personal safety hazards: ☐ Yes or ☐ Referred to Site Safety Plan and Safety Tailgate if applicable

Safety Hazards Were Addressed by : ☐ Staying Alert to Construction and Equipment Hazards ☐ Other (describe

I visited the site on May 17, 2016 to complete a reconnaissance to screen for geologically hazardous areas that could have the potential to impact the proposed stormwater vault, per the City of Redmond's critical areas regulations (Redmond Zoning Code 21.64.060). The reconnaissance focused on the sloped areas along the eastern portion of the project site, where the heads of two sub-parallel east-draining tributaries of Tosh Creek descend a generally east-facing slope into the Sammamish Valley. The two tributaries are referred to herein as Tributary B (north tributary) and Tributary C (south tributary). During the reconnaissance, I observed and documented slope conditions including soils, drainage, vegetation, and potential evidence of instability. Pertinent site features are shown on the attached site plan.

Tributary B Area

Tributary B flows through a short, relatively straight valley eastward from 159th Place NE to its confluence with Tosh Creek. The stream channel begins at the outlets of two ten-inch diameter CMP culverts that daylight from under a lawn area between two residences. Stream flow at the time of our reconnaissance was estimated to be approximately 2 cubic feet per second (cfs).

The head of Tributary B consists of a convergent inclined swale with an apron of large-diameter, angular boulder rip rap placed at the culvert outlets. The rip rap armors the upper portion of the stream channel and extends about 25 feet downstream from the culvert outlets. The slope at the head of the convergent swale is inclined to about 30 percent and is generally eastward-draining. Downstream from the rip rap the channel is incised with near vertical banks that are typically 2 to 4 feet tall. Glaciolacustrine (cohesive, hard silt) is exposed in the banks, superimposed by weathered glacial till (cobbles, gravels and sand). Above the crest of the banks, the side slopes of Tributary B are approximately 8 feet tall (in addition to the 2 to 4 foot tall bank height) and are locally inclined up to 45 percent on the left bank and 35 percent on the right bank.

The moderately dense understory vegetation includes sword fern, English ivy, devil's club, vine maple and hazelnut. The overstory consists of western hemlock, western red cedar, big leaf maple and alder with diameter at breast heights (dbh) up to 2 feet. The stems of the trees are generally in the vertical growth position, with a small number of trees that have slightly pistol-butted stems.

Separating Tributaries B and C is a relatively flat upland area that is approximately 15 feet wide between the crests of the sideslopes of the two drainages.

☑ **THIS FIELD REPORT IS PRELIMINARY** A preliminary report is provided solely as evidence that field observation was performed. Observations and/or conclusions and/or recommendations conveyed in the final report may vary from and shall take precedence over those indicated in a preliminary report.	**FIELD REPRESENTATIVE** Benjamin Cashman, LG	**DATE** 5/17/2016
☑ **THIS FIELD REPORT IS FINAL** A final report is an instrument of professional service. Any conclusions drawn from this report should be discussed with and evaluated by the professional involved.	**REVIEWED BY** Christopher Kokesh, PE	**DATE** 5/18/2016

Attachments: Figure 2 – Site Plan

Distribution: GeoEngineers File

Slope and rip rap in channel at head of Tributary B

Left bank sideslope and incised channel of Tributary B

In general, I observed no evidence of recent or incipient slope instability, no evidence of seepage or significantly distressed vegetation on the slopes of Tributary B, and no evidence of erosion outside of the main stream channel of Tributary B.

Tributary C Area

Tributary C flows east through a short, mostly straight valley from 159th Place NE to its confluence with Tosh Creek. The stream channel begins at the outlet of a perched 10-inch-diameter corrugated metal pipe (CMP) culvert. The culvert is the outlet from the Tamarack Lane stormwater pond. Stream flow at the time of our reconnaissance was less than 1 cfs.

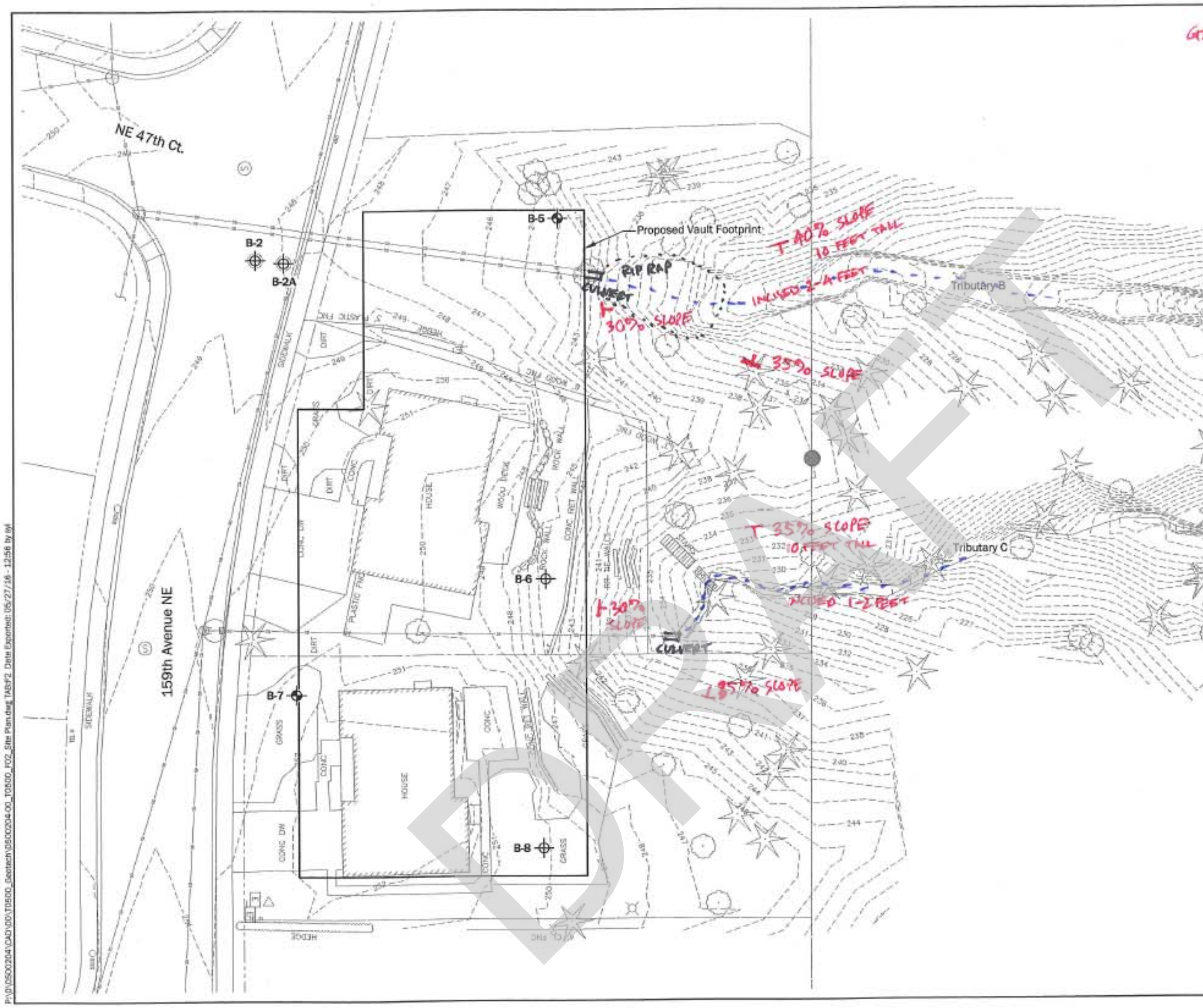
The head of Tributary C consists of a convergent inclined swale that contains a well-defined stream channel. The slope in this area is inclined to about 30 percent and is generally eastward draining. The channel slope is about 14 percent and is slightly incised with near vertical banks that are typically 1 to 2 feet tall. Hard glaciolacustrine silt armors the channel floor and lag deposits of gravel, cobbles, and occasional bounders rest on top of the silt. The banks are nearly

Sloped areas at head of Tributary C

vertical and consist of glaciolacustrine silt overlain by colluvium. Above the crests of the near vertical banks, the sideslopes of the drainage are approximately 10 feet tall and are typically inclined to 35 percent.

The sideslopes are well vegetated with trees generally less than 1-foot-diameter at breast height (dbh) including western red cedar, alder and big leaf maple and an understory of sword fern and hazel nut. The stems of the trees are generally in the vertical growth position, with a small number of trees that have slightly pistol-butted stems.

In general, I observed no evidence of recent or incipient slope instability, no evidence of seepage or significantly distressed vegetation on the adjacent slopes, and no evidence of erosion outside of the main channel of Tributary C.



Legend

B-5 — Monitoring Well completed by GeoEngineers Inc., 2015, 2016

B-2 — Boring completed by GeoEngineers Inc., 2015, 2016

N / W / E / S

30 0 30
Feet

Notes:

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

Data Source:
Base drawing provided by HDR Engineering, Inc., dated 05/25/16.

Vertical Datum: MLLW (NAVD 88).

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

Site Plan

159th Avenue NE Vault in Purchased Parcels
Tosh Creek Watershed Restoration
Redmond, Washington

GEOENGINEERS

Figure 2

APPENDIX D
Report Limitations and Guidelines for Use

APPENDIX D
REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

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

Geotechnical Services Are Performed for Specific Purposes, Persons and Projects

This report has been prepared for the exclusive use of the City of Redmond, HDR Engineering, Inc., and other project team members for the 159th Avenue NE Vault in Purchased Parcels project, which is part of the Tosh Creek Watershed Restoration 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.

A Geotechnical Engineering or Geologic Report Is Based on a Unique Set of Project-specific Factors

This report has been prepared for the 159th Avenue NE Vault in Purchased Parcels project, which is part of the Tosh Creek Watershed Restoration project 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

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

- project ownership.

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

Subsurface Conditions Can Change

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

Most Geotechnical and Geologic Findings Are Professional Opinions

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

Geotechnical Engineering Report Recommendations Are Not Final

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

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

A Geotechnical Engineering or Geologic Report Could Be Subject to Misinterpretation

Misinterpretation of this report by other design team members can result in costly problems. You could lower that risk by having GeoEngineers confer with appropriate members of the design team after submitting the report. Also retain GeoEngineers to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering or geologic report. Reduce that risk by having GeoEngineers participate in pre-bid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Exploration Logs

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

Give Contractors a Complete Report and Guidance

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

Contractors Are Responsible for Site Safety on Their Own Construction Projects

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

Read These Provisions Closely

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

Geotechnical, Geologic and Environmental Reports Should Not Be Interchanged

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

Biological Pollutants

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

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