Geotechnical Engineering Services
Design Phase

The Osprey
Proposed Residential Development
King County Tax Parcel 9270700080
7440 159th Place NE
Redmond, Washington

for
G.W. Williams Co. and
PemReal Advisors LLC

June 7, 2019

GeoEngineers
Earth Science + Technology

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File No. 23699-001-01

June 7, 2019

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GEOENGINEERS

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1.0 INTRODUCTION

This report presents the results of GeoEngineers, Inc.'s (GeoEngineers) geotechnical engineering services for the proposed residential development (The Osprey) on property located at 7440 159th Place NE in Redmond, Washington. The property is identified as King County Tax Parcel Number 9270700080.

The location of the site is shown on the Vicinity Map, Figure 1. The project site is shown in relation to surrounding features on the Site Plan, Figure 2.

1.1. Purpose and Scope

The purpose of this report is to provide geotechnical engineering conclusions and recommendations for the design and permitting phases of the development. GeoEngineers' geotechnical services have been completed in general accordance with our revised proposal dated May 20, 2019. Written authorization for our services was provided by Sean Williams of G.W. Williams Co. on May 20, 2019.

Our geotechnical scope of services includes:

- Reviewing available geotechnical exploration data for the site and surrounding area available from our files;
- Providing International Building Code (IBC) 2015 seismic design criteria;
- Providing foundation, temporary shoring, slab-on-grade, permanent below-grade wall, and other geotechnical recommendations; and
- Preparing this report.

No additional subsurface explorations were completed for the design phase of this project.

Our firm previously prepared due diligence phase services for this project, the results of which are summarized in our report dated March 14, 2019.

GeoEngineers has prepared a Geologically Hazardous Area (Critical Areas) report which is being submitted separately. A Critical Aquifer Recharge Area (Wellhead Protection) report for this project is also being submitted separately as part of the permitting process.

1.2. Project Description

We understand the property will be redeveloped with a multi-story mixed-use residential building. Based on conceptual plans prepared by the project architect, HKS, Inc., and dated May 30, 2019, the project will consist of a six-story building with one level of below-grade parking. The ground level will include additional parking, retail and residential spaces and the upper five stories will include residential space. The floor slab for the basement is planned to be approximately 10 feet below existing grade. An elevator pit will extend to about 15 feet below existing grade.

The building footprint will occupy nearly the entire 0.62-acre site. An infiltration trench for disposal of site stormwater runoff is planned between the east side of the building and the adjacent east property line.

Temporary shoring will likely be required for the planned building excavation if the building footprint is maximized at the site. Open cuts will only be feasible if there is enough building setback distance from the property lines.

2.0 PREVIOUS STUDIES

GeoEngineers completed geotechnical engineering services in 1988 for improvements to Leary Way, which extends along the south side of the site. Several borings were drilled as part of that project, including a boring (B-7) about 125 feet southwest of the intersection of Leary Way and 159th Place NE (see Figure 2).

Associated Earth Sciences, Inc. (AESI) completed geotechnical engineering services for the adjacent properties to the north (7494 and 7500 159th Place NE) which are summarized in a report dated April 18, 2014. Several borings were drilled for that project, including a boring (EB-4) near the northwest corner of the Evans Automotive site. AESI also completed a hydrogeologic and infiltration assessment for the properties in 2015; the assessment included test pits and additional borings.

A Phase II Environmental Site Assessment (ESA) was completed in 2018 by G-Logics, Inc. and summarized in a report dated June 28, 2018. The Phase II ESA included 11 borings, three of which were completed as groundwater monitoring wells (GLMW-1, -2 and -3), with the remaining eight borings (GLB-1 through GLB-8) being backfilled. The approximate locations of these borings and monitoring wells are shown on Figure 2.

Logs of the previous explorations are included in Appendix A.

3.0 SITE DESCRIPTION

The project site consists of one parcel (King County Tax Parcel Number 9270700080) as shown on Figure 2. The site comprises approximately 0.62 acres and is located at 7440 159th Place NE in downtown Redmond, Washington.

3.1. Geology

The project lies in the downtown Redmond area of the Sammamish River valley. The valley is a major glacial trough between glaciated uplands to the west and east. The valley trends north to south and is underlain by recent alluvium and glacial recessional outwash sediments.

Geologic information for the project vicinity was obtained from the map entitled "Geologic Map of the Kirkland Quadrangle, Washington" (Minard 1983) published by the United States Geological Survey (USGS). The native geologic unit mapped in the site vicinity consists of alluvium.

The alluvium is mapped along and east of the Sammamish River and consists primarily of near-surface organic rich fine sand, silt and clay. Peat layers are often present in the upper few feet of the alluvium. Sand and gravel alluvial deposits underlie the surficial soils.

Fill associated with past grading for the existing building and pavement areas mantles the alluvial deposits.

Recessional outwash deposits are known to underlie the alluvium at depth. The recessional outwash typically consists of sand and gravel with variable silt, cobble and boulder content deposited by meltwater flowing from a receding ice sheet that occupied the Sammamish River valley during the last glacial epoch.

3.2. Critical/Sensitive Areas Delineation

Review of the City of Redmond Critical Areas Maps and King County Sensitive Areas Maps indicate the project area is located within a mapped Seismic Hazard Area. The project area is also within a Critical Aquifer Recharge Area I (CARA I) in accordance with the City of Redmond Zoning Code Section 20D.140.50 and the 2019 edition of the Stormwater Technical Notebook.

There are no mapped Landslide or Erosion Hazard Areas on the project site or in the immediate vicinity of the site.

GeoEngineers has prepared a Geologically Hazardous Area (Critical Areas) report which addresses the Seismic Hazard Area designation and a Critical Aquifer Recharge Area (Wellhead Protection) report. These reports are being submitted separately as part of the permitting process.

3.3. Surface Conditions

The site is bounded on the north by a recently completed apartment building (The Carter), on the east by the wooded Heron Rookery Park, on the south by Leary Way NE, and on the west by 159th Place NE. The property is owned by G.W. Williams Co. and is currently occupied by automotive facilities (A1 Luxury Motors and Harvey's Auto Service). The one-story automobile facility building occupies the east part of the site. Asphalt paved parking and driveway areas are in the north and west parts of the site.

The existing building was constructed in 1968. The property was historically operated as Evans Auto Center. Occupants of the building have included auto repair businesses going back to the first occupants following construction of the building. Prior tenants have also included a feed company, a carpet and interiors company, and an appliance services company.

The ground surface is generally level. The finished floor of the existing building is at about Elevation 43 feet. (Elevations in this report refer to the North American Vertical Datum of 1988 [NAVD 88].) Surface grades outside the building range from about Elevation 41 to 43 feet. Underground power and fiber optic lines extend along the west edge of the site.

3.4. Subsurface Soil Conditions

Based on our review of available subsurface information, the subsurface soils in the vicinity of the site generally consist of varying thicknesses of fill overlying medium dense to dense granular alluvial and recessional outwash deposits, as discussed below:

- **Pavement and Floor Slab Materials:** Several of the borings were drilled within asphalt paved areas and within the existing building. The thicknesses of the pavement and floor slab were not noted on the boring logs.
- **Fill:** Existing fill was apparently encountered in the upper 5 feet of borings GLMW-3 and GLB-8, based on the presence of wood fragments. The fill layer is described as loose sand with gravel. The remaining boring logs did not note the presence of fill.

- **Granular Alluvium/Recessional Outwash:** Medium dense to dense sand and gravel deposits were encountered in all these explorations and extend to the maximum depth explored, 41½ feet. The upper portion of these deposits is alluvium, while the lower portion likely consists of recessional outwash.

While not encountered during drilling of the previous borings, cobbles and boulders are known to be present in the alluvium and recessional outwash and may be encountered during excavation and soldier pile drilling at this site.

3.5. Groundwater Conditions

Groundwater was encountered in the previous explorations and monitoring wells within about 18 to 20 feet of the existing ground surface, based on measurements made in late June 2018. We measured groundwater levels in the wells at similar depths on March 1, 2019. Groundwater levels in the wells are being measured by pressure transducers which are being downloaded on a quarterly basis through mid-2020.

This groundwater represents a shallow aquifer within the near surface alluvial soils that is part of the Redmond Alluvial Aquifer underlying the downtown area. This aquifer is in direct hydraulic communication with the Sammamish River, located within 200 feet of the southern end of the site. Groundwater flows to the north and northwest to the Sammamish River. Based on our recent measurements, the groundwater gradient across the site is approximately 0.004 (0.4 feet of elevation difference over a horizontal distance of 100 feet).

We expect the groundwater level will rise in response to seasonal precipitation and flood stages of the river and could be as high as 7 to 10 feet below the ground surface during flood stage The estimated highest groundwater level (7 feet below ground surface, or at Elevation 35 feet) corresponds to the 100-year flood level in the Sammamish River.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1. Summary

Based on the previous explorations, analyses and our experience on nearby projects in the downtown Redmond area, we conclude the residential project can be satisfactorily completed as planned.

A summary of the geotechnical considerations for the proposed residential development is provided below. This summary is presented for introductory purposes only and should be used in conjunction with the complete recommendations presented in this report.

- The site is designated as seismic Soil Profile Type D per the 2015 IBC.
- Most of the near-surface on-site soils contain a high percentage of fines (silt) and are moisture sensitive. Soils encountered at the site may be used as structural fill during dry weather conditions (typically June through September) provided they are properly moisture conditioned. Imported fill will be necessary during periods of wet weather (typically October through May).
- Excavations to depths ranging from about 12 to 15 feet below existing site grades are planned for the development. Temporary shoring will be required for the planned building excavation if the building footprint is maximized at the site (that is, extending to the property lines). Recommendations for

cantilever soldier pile walls are provided below. Open cuts will only be feasible if there is enough building setback distance and space from the property lines.

- We anticipate construction dewatering will not require extensive measures, particularly if excavation takes place during the normally dry months of the year and when the Sammamish River is not at flood stage. Localized zones of groundwater could be encountered in limited excavations extending below the general basement excavation, such as for elevator pits, depending on the time of year these are made.
- Suitable foundation support for the building can consist of shallow foundations placed directly on the medium dense to dense granular alluvial or recessional outwash soils, or on a zone of crushed rock fill replacing loose soils that may be encountered at footing subgrade level. An allowable bearing pressure of 5,000 pounds per square foot (psf) may be used.
- Conventional slabs-on-grade are appropriate for the lower parking level at this site and should be underlain by a 4-inch-thick layer of clean crushed rock (for example, Washington State Department of Transportation (WSDOT) Standard Specification Section 9-03.1(4)C, Grading No. 57).
- A seasonal high groundwater at Elevation 35 feet corresponding to the 100-year flood level in the Sammamish River should be used for design of the structure.
- A slab underdrain system leading to a sump with pump should be installed to limit water intrusion in the basement level during flood stages of the Sammamish River. Alternatively, the basement slab should be waterproofed and designed for hydrostatic uplift pressures.
- Permanent below-grade building walls should be designed to resist permanent lateral earth pressures and should be provided with drainage to prevent the buildup of hydrostatic pressures. Waterproofing of the below-grade wall will be required along the east side of the building where it is adjacent to the planned infiltration trench. The elevator pit base and walls should also be waterproofed.
- On a preliminary basis, the infiltration trench can be designed using an allowable design infiltration rate of 9 inches per hour. This rate should be confirmed at the beginning of construction after the existing building has been demolished and test pit excavation with soil sampling and grain size testing has been completed.

The following sections of this report present our specific geotechnical recommendations for this project.

4.2. Earthquake Engineering

4.2.1. 2015 IBC Seismic Design Information

We recommend the use of the following 2015 IBC parameters for short period spectral response acceleration (S_S), 1-second period spectral response acceleration (S_1) and seismic coefficients (F_A and F_V) for the project site.

2015 IBC Parameter	Recommended Value
Soil Profile Type	D
Short Period Spectral Response Acceleration, S_S (percent g)	125.5
1-Second Period Spectral Response Acceleration, S_1 (percent g)	48.1
Seismic Coefficient, F_A	1.0
Seismic Coefficient, F_V	1.52

4.2.2. Seismic Hazards

As mentioned above, the site is within a mapped Seismic Hazard area. Potential seismic hazards from earthquakes include ground shaking, surface fault rupture, liquefaction, lateral spreading and landslides. We evaluated the likelihood of each of these hazards at the site, except for landslides, which are very unlikely to occur due to the gentle topography. Ground shaking is addressed through use of the IBC parameters provided above.

Based on our knowledge of regional geology in the vicinity of the site, distance to known active faults, and the substantial thickness of glacial and postglacial sediments beneath the site, we conclude the potential for surface fault rupture is remote.

Liquefaction is a condition where soils experience a rapid loss of internal strength resulting from strong ground shaking. Ground settlement, lateral spreading and sand boils may result from soil liquefaction. Structures supported on large zones of liquefied soils could undergo potentially damaging settlements or lateral movement. Conditions favorable for liquefaction include loose to medium dense sand with a low percentage of silt, and which are below the ground water table.

Based on the previous explorations and our liquefaction analyses, we conclude liquefaction induced settlements at the site will be isolated and minor, estimated to be less than about ½ to 1 inch.

Some lateral spreading may occur immediately adjacent to the Sammamish River banks during a large earthquake. We do not anticipate the lateral spreading would extend to the project site because of the low potential for liquefaction at the site; therefore, the risk of lateral spreading at the site is low.

4.3. Site Preparation

The surficial soils at the site contain a high percentage of fines (particles passing the U.S. Standard No. 200 sieve) and are therefore moisture sensitive and susceptible to disturbance. These soils may be wet during part of the year. It will be difficult to properly compact or operate equipment on these soils when they are wet. Accordingly, we recommend site preparation, shoring, excavation and foundation installation activities be planned for the normally drier late summer to early fall months so that difficulties and costs associated with these activities can be reduced. Dewatering effort, if required, will also be reduced, and the potential for reusing the existing fill and native soils as structural fill may be increased.

Trafficability on the site is not expected to be difficult during dry weather conditions. However, the fill and native soils will be susceptible to disturbance from construction equipment during wet weather conditions, and pumping and rutting of the exposed soils under equipment loads will likely occur. Construction traffic should be limited to existing paved areas whenever feasible, particularly during wet weather.

We anticipate site preparation will largely include demolition of the existing building and removal of existing asphalt pavement and underground utilities. Trees, shrubs and associated stumps and root wads should also be removed. The site should be stripped of any sod or organic soil.

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

Removal and demolition of existing structures and pavements should also include removal of below-grade elements such as underground utilities. Existing voids or new depressions created during site preparation should be cleaned of loose soil or debris and backfilled with compacted structural fill.

The exposed subgrade in structure and hardscape areas should be evaluated after site excavation is completed. Disturbed areas below slabs and foundations should be recompacted if the subgrade soil consists of granular material. It may be necessary to remove and replace the disturbed soil with structural fill if the disturbed soil cannot be adequately moisture-conditioned and compacted.

Prior to placing new fills, pavement base course material or hardscape, subgrade areas should be evaluated by proof rolling or probing to identify zones of soft or pumping soils. Prior to proof rolling, all unsuitable soils should be removed from new fill and pavement areas.

Proof rolling can be completed using a piece of heavy tire-mounted equipment such as a loaded dump truck. During wet weather, the exposed subgrade areas should be probed to evaluate the extent of soft soils. If zones of soft or pumping soils are identified, they should be removed and replaced with structural fill to the depth recommended by a qualified geotechnical representative.

4.4. 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 of Redmond standards. The plan should incorporate basic planning principles including:

- Scheduling grading and construction to reduce soil exposure;
- Retaining existing asphalt paved surfaces for as long as practical;
- 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 reestablishing vegetation using hydroseeding and/or landscape planting and by installing new pavement and hardscape features.

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.

4.5. Earthwork

4.5.1. Excavation

We understand the planned building will have one level of below-grade parking and that the general excavation will extend down to approximately 12 feet below existing site grades. Temporary shoring will be needed to maximize the building footprint or where there is insufficient space to use temporary open cuts. Open cuts will only be feasible if there is enough building setback distance from property lines. Recommendations for temporary shoring are provided below in "Temporary Excavation Shoring."

We recommend excavation for foundation elements, elevator pits, under-slab utilities and other below-grade structures be planned for the normally dry season of the year. Groundwater control and handling will require less effort and cost during the summer months when rainfall is minimal and river levels are typically low.

Based on the subsurface soil conditions encountered in the previous explorations, we anticipate the soils at the site may be excavated with conventional heavy duty construction equipment. Typical soils encountered in the previous explorations include loose to medium dense granular fill and medium dense to dense granular alluvial and recessional outwash soils. The contractor should be prepared to address cobbles and boulders in these soils.

4.5.2. Dewatering

As the site is within the CARA 1 zone, and the aquifer is a source of municipal water supply for the City of Redmond, development projects that need temporary construction dewatering must comply with City of Redmond Ordinance No. 2831, as embodied in Redmond Municipal Code (RMC) Section 13.25.

Under the RMC, projects that involve temporary construction dewatering discharges greater than 500 gallons per minute (gpm) must follow the procedures established under City of Redmond Temporary Construction Dewatering Operating Policy, including preparation and submission of a Temporary Construction Dewatering Feasibility Study. Projects that involve temporary construction dewatering of less than 500 gpm must follow the less restrictive guidelines outlined in Chapter 2 of the City of Redmond's Stormwater Technical Notebook.

Based on review of groundwater level data from our recent measurements, in the previous reports and available as part of the City of Redmond's groundwater monitoring program for the Redmond Alluvial Aquifer, we expect groundwater inflows will be minor and significantly less than 500 gpm, particularly if general excavation for the building and localized deeper excavation for elevator pits takes place during the normally dry months of the year and when the Sammamish River is not at flood stage. There is a risk that partially constructed elements of the project could be inundated by abnormally high groundwater levels, especially during or in response to flood stages of the Sammamish River.

Localized dewatering, for example, from elevator pit excavations, can be completed if necessary, by pumping from localized sumps.

Consideration must also be given to design of subsurface structures given the risk of high groundwater levels in response to flood stages in the Sammamish River. Subgrade structures (basement floors and walls) should be fully waterproofed up to at least 2 feet above the estimated seasonal high groundwater level and should be designed for the worst-case hydrostatic conditions (lateral loading and uplift pressures) created by a high groundwater elevation. This is expected to be a very rare event.

Alternatively, if occasional flooding (probably once every few years) of the basement parking structure can be tolerated, and signs of seepage stains and efflorescence on interior walls below grade are acceptable, then waterproofing can be deleted. However, a slab underdrain system leading to a sump with pump, or pressure relief in the form of flood flaps must be included to allow high groundwater to inundate the basement and balance hydrostatic forces that could otherwise damage floor slab and wall panel elements.

4.5.3. Temporary Cut Slopes

We recommend temporary open cut slopes around excavations be inclined at 1.5H:1V or flatter, depending on whether seepage is encountered in the cut. The amount of seepage will vary seasonally. Cut slopes should be made flatter if significant seepage occurs during excavation.

For open cuts at the site, we recommend that:

- No traffic, construction equipment, stockpiles or building supplies be allowed at the top of cut slopes within a distance of at least 5 feet from the top of the cut;
- Exposed soil along the slope be protected from surface erosion by using waterproof tarps or plastic sheeting;
- Construction activities be scheduled so the length of time the temporary cut is left open is reduced to the extent practicable;
- Erosion control measures be implemented as appropriate such that runoff from the site is reduced to the extent practicable;
- Surface water be diverted away from the slope; and
- The general condition of the slopes be observed periodically by the geotechnical engineer to confirm adequate stability.

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

4.5.4. Structural Fill

4.5.4.1. Materials

We anticipate small amounts of new fill will be required for the project, such as around the perimeter of the building and in floor slab and pavement areas. Structural fill should meet the following criteria:

- As a minimum, structural fill placed in parking areas and to backfill utility trenches should meet the criteria for Common Borrow as described in Section 9-03.14(3) of the WSDOT Standard Specifications. 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 as described in Section 9-03.14(1) of the WSDOT Standard Specifications, with the additional restriction that the fines content be limited to no more than 5 percent.
- Structural fill for support of foundations should consist of crushed rock meeting the criteria in Section 9-03.1(14)C, Grading No. 57 in the WSDOT Standard Specifications.
- Structural fill placed as capillary break material below slabs-on-grade should meet the requirements for Section 9-03.1(4)C, Grading No. 57 of the WSDOT Standard Specifications.
- Structural fill placed behind retaining walls should meet the requirements for Gravel Backfill for Walls, Section 9-03.12(2) of the WSDOT Standard Specifications.
- Structural fill placed around perimeter footing drains and drains behind cast-in-place walls should meet the requirements for Gravel Backfill for Drains, Section 9-03.12(4) of the WSDOT Standard Specifications.
- Structural fill placed as crushed surfacing base course below pavements and sidewalks should meet the requirements of Crushed Surfacing Base Course, Section 9-03.9(3) of the WSDOT Standard Specifications.

4.5.4.2. Reuse of On-site Soils

The on-site soils are moisture-sensitive and generally have natural moisture contents higher than the anticipated optimum moisture content for compaction. Thus, the on-site soils will likely require moisture conditioning to meet the required compaction criteria during dry weather conditions and may not be suitable for reuse during wet weather. Furthermore, most of the fill soils required for the project have specific gradation requirements, and the on-site soils may not meet these gradation requirements. If the contractor desires to use on-site soils for structural fill, GeoEngineers can evaluate whether these soils are suitable for reuse as structural fill, depending on weather conditions and other factors.

4.5.4.3. Fill Placement and Compaction Criteria

Structural fill should be mechanically compacted to a firm, non-yielding condition. Structural fill should be placed in loose lifts not exceeding 12 inches in thickness if using heavy compactors and 6 inches if using hand operated compaction equipment. The actual lift thickness required will be dependent on the structural fill material used and the type and size of compaction equipment.

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:

- Structural fill placed in building areas (below and around foundations or below slab-on-grade floors) should be compacted to at least 95 percent of the maximum dry density (MDD) estimated using the

American Society for Testing and Materials (ASTM) D 1557 test method. Crushed rock fill placed below building foundations should be compacted to a firm and non-yielding condition.

- Structural fill placed in new pavement or hardscape areas, including utility trench backfill, should be compacted to 90 percent of the MDD estimated using ASTM D 1557, except that the upper 2 feet of fill below final subgrade level should be compacted to at least 95 percent of the MDD.
- Structural fill placed as crushed rock base course below pavements should be compacted to at least 95 percent of the MDD estimated using ASTM D 1557.
- Structural fill placed against below-grade or retaining walls within a distance equal to the height of the wall should be compacted to between 90 and 92 percent of the MDD estimated using ASTM D 1557. Care should be taken when compacting fill against retaining walls to avoid over compaction ~~and~~and, hence overstressing the walls.
- Non-structural fill, such as fill placed in landscape areas, should be compacted to at least 85 percent of the MDD.

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

4.5.4.4. Weather Considerations

The near-surface on-site soils contain a sufficient percentage of fines (silt) to be 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, and operation of equipment on these soils is difficult. Additionally, disturbance of near-surface soils should be expected if earthwork is completed during periods of wet weather.

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

- The ground surface in and around the work area should be sloped so that surface water is directed away from the work area and should be graded such that areas of ponded water do not develop.
- Surface water must not be directed toward slopes. We recommend storm water drainage ditches be constructed where needed along the crest of slopes to prevent uncontrolled surface water runoff.
- The contractor should take measures to prevent surface water from collecting in excavations and trenches.
- Earthwork activities should not take place during periods of moderate to heavy precipitation.
- Slopes with exposed soils should be covered with plastic sheeting or similar means.
- The site soils should not be left uncompacted and exposed to moisture. Sealing the surficial soils by rolling with a smooth-drum roller prior to periods of precipitation will reduce the extent to which these soils become wet or unstable.

- The contractor should cover all soil stockpiles that will be used as structural fill with plastic sheeting.
- Construction traffic should be restricted to specific areas of the site, preferably areas that are surfaced with materials not susceptible to wet weather disturbance.
- Construction activities should be scheduled so that the length of time soils are left exposed to moisture is reduced to the extent practicable.

Routing of equipment on the existing fill and native soils during the wet weather months will be difficult and the subgrade will likely become highly disturbed and rutted. In addition, a significant amount of mud can be produced by routing equipment directly on these soils in wet weather. Therefore, to protect the subgrade soils and to provide an adequate wet weather working surface for the contractor's equipment and laborers, we recommend the contractor protect exposed subgrade soils with a layer of sand and gravel with a low percentage of fines or crushed gravel.

4.5.5. Permanent Cut and Fill Slopes

We recommend permanent cut and fill slopes, if required, be constructed no steeper than 2H:1V or flatter and be blended into existing slopes with smooth transitions. To achieve uniform compaction, we recommend fill slopes be overbuilt slightly (1 to 2 feet) and subsequently cut back to expose properly compacted fill. We recommend the finished slope faces be compacted by track walking with the equipment running perpendicular to the slope contours so 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 or erosion control blankets (such as American Excelsior Curlex 1 or North American Green SC150) could be used to protect the slopes during periods of rainfall.

4.6. Temporary Excavation Shoring

We understand the planned building will have one level of below-grade parking and that the excavation will extend down to approximately 12 feet below existing site grades. As discussed above, we anticipate temporary shoring will be needed to maximize the building footprint or where there is insufficient space to utilize temporary open cuts.

The subsurface conditions support the use of conventional soldier pile shoring. If the excavation depth is 12 feet or less, a cantilever soldier pile wall can be economically constructed. The following sections provide geotechnical design and construction recommendations for cantilever soldier pile walls.

Soldier pile walls consist of steel beams concreted into drilled vertical holes located along the wall alignment, typically about 6 to 8 feet on center. Timber lagging is typically installed behind the flanges of the steel beams to retain the soil located between the soldier piles.

4.6.1. Soldier Piles

We recommend soldier pile walls be designed using the earth pressure diagrams presented in Figure 3. The earth pressures presented in Figure 3 are for cantilever soldier pile walls. The pressures represent the estimated loads that will be applied to the wall system for various wall heights.

The earth pressures presented in Figure 3 include the loading from typical traffic surcharge. Other surcharge loads, such as cranes, construction equipment or construction staging areas, should be considered on a case-by-case basis in accordance with the recommendations presented in Figure 4. No seismic pressures have been included in Figure 3 because it is assumed the shoring will be temporary.

We recommend the embedded portion of the soldier piles be at least 2 feet in diameter and extend a minimum distance of 8 feet below the base of the excavation to resist “kick-out.” The axial capacity of the soldier piles may need to resist vertical loads, as appropriate. We recommend using an allowable end bearing value of 15 kips per square foot (ksf) for piles supported on the medium dense to dense granular alluvial or recessional outwash soils. The allowable end bearing value should be applied to the base area of the drilled hole into which the soldier pile is concreted. This value includes a factor of safety of about 2.5. The allowable end bearing value assumes the shaft bottom is cleaned out immediately prior to concrete placement. If necessary, an allowable pile skin friction value of 0.5 ksf in these soils may be used on the embedded portion of the soldier piles to resist the vertical loads.

4.6.2. Lagging

We recommend the temporary timber lagging be sized using the procedures outlined in the Federal Highway Administration’s Geotechnical Engineering Circular No. 4. The site soils are best described as competent soils. The following table presents GeoEngineers’ recommended lagging thicknesses (rough-cut) as a function of soldier pile clear span and depth.

Depth (feet)	Recommended Lagging Thickness (rough-cut) for clear spans of:					
	5 feet	6 feet	7 feet	8 feet	9 feet	10 feet
0 to 25	2 inches	3 inches	3 inches	3 inches	4 inches	4 inches

Lagging should be installed promptly after excavation, especially in areas where perched groundwater is present or where clean sand and gravel soils are present and caving soils conditions are likely. The workmanship associated with lagging installation is important for maintaining the integrity of the excavation.

The space behind the lagging should be filled with soil as soon as practicable. Voids should be backfilled immediately or within a single shift, depending on the selected method of backfilling. Placement of this material will help reduce the risk of voids developing behind the wall and damage to existing improvements located behind the wall.

Material used as backfill in voids located behind the lagging should not cause buildup of hydrostatic pressure behind the wall. Lean concrete is a suitable option for the use of backfill behind the walls. Lean concrete will reduce the volume of voids present behind the wall. Alternatively, lean concrete may be used as backfill behind the upper 5 feet of the excavation to limit caving and sloughing of the upper soils, with on-site soils used to backfill the voids for the remainder of the excavation. Based on our experience, the voids between each lean concrete lift are sufficient for preventing the buildup of hydrostatic pressure behind the wall.

4.6.3. Drainage

A suitable drainage system should be installed to prevent the buildup of hydrostatic groundwater pressures behind the soldier pile and lagging wall. It may be necessary to cut weep holes through the lagging in wet areas. Seepage flows at the bottom of the excavation should be contained and controlled. Drainage should be provided for permanent below-grade walls as described below in the "Below-Grade and Retaining Walls" section of this report.

4.6.4. Construction Considerations

We recommend the soldier piles be installed in a sequence where every other soldier pile along the length of the wall is installed first, followed by installation of the soldier piles in between. Doing this will reduce the potential for caving or interaction of adjacent drilled holes.

Temporary casing or drilling fluid may be required to install the soldier piles where:

- Loose fill is present;
- The native soils do not have adequate cementation or cohesion to prevent caving or raveling; and/or
- Perched groundwater is present.

The granular alluvium and recessional outwash deposits are known to contain cobbles and boulders. The contractor should be prepared to address cobbles and boulders during soldier pile drilling.

GeoEngineers should be allowed to observe and document the installation of the shoring elements to verify conformance with the design assumptions and recommendations.

Monitoring of the shoring system should be completed as described in Appendix B, Shoring Monitoring Program.

4.7. Shallow Foundations

Based on the previous explorations completed at the site and the anticipated depth of excavation, medium dense to dense granular alluvial soils will be present at foundation level for the building. Shallow spread or mat foundations will therefore be suitable for this project. Shallow foundations may also be supported on a pad of compacted crushed rock that partially replaces loose or soft zones of alluvial soils that may be encountered in the building excavation.

4.7.1. Soil Bearing Value

Shallow foundations bearing on undisturbed medium dense granular alluvial deposits or bearing on a pad of compacted crushed rock fill meeting the criteria for WSDOT Standard Specification 903.1(4)C, Grading NO. 57 and placed over the granular alluvial deposits may be designed using an allowable soil bearing pressure of 5,000 psf. The zone of compacted fill should extend laterally beyond the footing edges a horizontal distance at least equal to the thickness of fill.

This bearing pressure applies to the sum of all dead plus long-term live loads, excluding the weight of the footing and any overlying backfill. This value may be increased by one-third when wind or seismic loads are considered.

For mat foundations designed as a beam on an elastic foundation, a modulus of subgrade reaction of 50 pounds per cubic inch (pci) may be used for support on medium dense alluvial soils or on compacted crushed rock fill.

4.7.2. Settlement

Provided all loose soil is removed and the subgrade is prepared as recommended under “Construction Considerations” below, we estimate the total settlement of shallow foundations will be less than 1 inch. The settlements will occur rapidly, essentially as loads are applied. We anticipate differential settlements between footings could be half of the expected total settlement. Differential settlements along continuous wall footings are expected to be less than ½ inch over 25 feet. Note that smaller settlements will result from lower applied loads. As noted above, liquefaction induced settlement of the building is expected to be less than about ½ to 1 inch.

4.7.3. Size and Embedment

We recommend exterior footings be founded a minimum of 18 inches below the lowest adjacent grade. Interior footings should be founded a minimum of 12 inches below the top of the slab-on-grade. Continuous wall footings and individual column footings should have minimum widths of 24 inches.

4.7.4. Lateral Resistance

Lateral foundation loads may be resisted by passive resistance on the sides of footings and by friction on the base of the footings. For shallow foundations supported on medium dense alluvium or on crushed rock fill, the allowable frictional resistance may be computed using a coefficient of friction of 0.4 applied to vertical dead-load forces.

The allowable passive resistance may be computed using an equivalent fluid density of 300 pounds per cubic foot (pcf) (triangular distribution). This value is appropriate for foundation elements surrounded by structural fill.

The above coefficient of friction and passive equivalent fluid density values incorporate a factor of safety of about 1.5.

4.7.5. Construction Considerations

Footing subgrade soils may be susceptible to disturbance when wet. It may be necessary to pour a lean concrete “mud mat” or place a 2- to 4-inch-thick layer of crushed rock on the bottom of the footing excavations to protect the footing subgrade soils from water and/or wet weather during reinforcement bar placement and preparation for concrete pouring.

We recommend the condition of all subgrade areas be observed by a representative of GeoEngineers prior to placement of concrete to confirm the bearing soils are undisturbed and are consistent with the recommendations contained in this report.

4.8. Slab-on-Grade

The exposed subgrade in slab-on-grade areas should be evaluated after site grading is complete. Proof rolling with heavy rubber-tired construction equipment should be used for this purpose during dry weather, if the subgrade is dry, and if access for this equipment is practical. Probing should be used to evaluate the subgrade during periods of wet weather or if access is not feasible for construction equipment. The exposed soil should be firm and nonyielding, and without significant groundwater present. Disturbed areas should be recompacted if possible or removed and replaced with compacted structural fill.

4.8.1. Design Parameters

Conventional slabs may be supported on-grade, provided the subgrade soils are prepared as recommended in the "Subgrade Preparation" section above. We recommend the slab be founded on either undisturbed alluvial soils or on structural fill placed over the undisturbed alluvial soils. For slabs designed as a beam on an elastic foundation, a modulus of subgrade reaction of 90 pci may be used for subgrade soils prepared as recommended.

We estimate settlements of floor slabs supported as recommended and subjected to uniform areal loads in the range of 100 to 200 psf will be approximately ½ inch or less. Abrupt differential settlements are not likely to occur unless highly variable floor loads are placed.

A 4-inch-thick base course layer of clean crushed gravel with negligible sand or silt (Section 9-03.1(4)C, Grading No. 57 of the WSDOT Standard Specifications) should be placed to provide uniform support and form a capillary break beneath the slab. Where moisture-sensitive floor coverings such as vinyl, tile or carpet or where moisture-sensitive equipment will be used, we recommend a vapor retarder consisting of 10-mil plastic sheeting be installed below the slab to reduce the potential for migration of moisture.

4.8.2. Hydrostatic Uplift, Waterproofing and Slab Underdrain Considerations

The design finished floor elevation of the below-grade parking level is close to or slightly below the estimated high groundwater level resulting from flood stages of the Sammamish River. It will be necessary to address the potential for water intrusion into the basement using one of several options:

- Provide waterproofing below the slab and on basement walls up to at least 2 feet above the estimated high groundwater level. The slab and foundation system will need to be designed to resist hydrostatic uplift pressures.
- If occasional flooding (probably once every few years) of the basement parking level can be tolerated, and signs of seepage stains and efflorescence on interior walls below grade are acceptable, then waterproofing can be deleted. However, pressure relief in the form of flood flaps must be included to allow high groundwater to inundate the basement and balance hydrostatic forces that could otherwise damage floor slab and wall panel elements.
- Provide an underslab drain system leading to a sump with pump. Recommendations for an underslab drainage system follow.

The underslab drainage system should include an interior perimeter drain and one to two longitudinal drains. The location(s) of the longitudinal drain(s) will depend on the foundation and below-grade structure design and may need to be modified. The project civil engineer should develop a conceptual foundation drainage plan for GeoEngineers to review.

The drains should consist of perforated Schedule 40 polyvinyl chloride (PVC) pipes with a minimum diameter of 4 inches placed in a trench at least 12 inches deep. The top of the underslab drainage system trenches should coincide with the base of the capillary break layer. The thickness of the capillary break layer for the underslab drainage system should be at least 12 inches. The underslab drainage system pipes should have adequate slope to allow positive drainage to a sump/gravity drain.

The drainage pipe should be perforated. Perforated pipe should have two rows of ½-inch holes spaced 120 degrees apart and at 4 inches on center. If the perimeter underslab drain will also be used to collect weep pipe discharge from the below-grade walls, we recommend the holes of the perforated pipe be oriented up.

The underslab drainage system trenches should be backfilled with WSDOT Gravel Backfill for Drains, Section 9-03.12(4) or an alternative approved by GeoEngineers. The underslab drainage system trench backfill material should be wrapped with a geotextile filter fabric meeting the requirements of Construction Geotextile for Underground Drainage, WSDOT Standard Specification 9-33.

Underslab drainage system pipes should be connected to a header pipe and routed to a sump or gravity drain. Appropriate cleanouts for drainpipe maintenance should be installed. A larger-diameter pipe will allow for easier maintenance of drainage systems.

4.9. Below-Grade and Retaining Walls

4.9.1. Permanent Below-Grade Walls Against Shoring

Permanent below-grade walls and structures such as elevator pits constructed adjacent to shoring walls should be designed using the earth pressures presented in Figure 5. Foundation surcharge loads and traffic surcharge loads should be incorporated in design of the below-grade walls using the surcharge pressures presented in Figure 4. Other surcharge loads, such as from construction equipment or construction staging areas, should be considered on a case-by-case basis. We can provide the lateral pressures from these surcharge loads as the design progresses.

The soil pressures recommended above assume wall drains will be installed to prevent the buildup of hydrostatic pressure behind the walls, as described above in the “Temporary Excavation Shoring” section of this report and tied to permanent drains to remove water to suitable discharge points.

4.9.2. Other Cast-in-Place Walls

Conventional cast-in-place walls may be necessary for small retaining structures located on-site, internal ramps within the parking structures, or where temporary open cuts are used for excavation support. The lateral soil pressures acting on conventional cast-in-place subsurface walls will depend on the nature, density and configuration of the soil behind the wall and the amount of lateral wall movement that can occur as backfill is placed.

For walls that are free to yield at the top at least 0.1 percent of the height of the wall, soil pressures will be less than if movement is limited by such factors as wall stiffness or bracing. Assuming the walls are backfilled and drainage provided as outlined in the following paragraphs, we recommend yielding walls supporting horizontal backfill be designed using an equivalent fluid density of 35 pcf (triangular distribution). Non-yielding walls supporting horizontal backfill should be designed using an equivalent fluid density of 55 pcf (triangular distribution). Permanent walls may be assumed to be yielding walls if constructed in front of temporary shoring walls as enough wall movement will have occurred during construction.

For seismic loading conditions, a rectangular earth pressure equal to 8H psf, should be added to the active/at-rest pressures. Other surcharge loads, such as from foundations, construction equipment or construction staging areas, should be considered on a case-by-case basis, as shown on Figure 4.

We recommend below-grade wall or other retaining wall foundations be designed using the foundation recommendations provided above under the "Shallow Foundations" section in this report.

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

If drainage cannot be provided behind below-grade walls or structures, hydrostatic pressures should be added to the lateral soil pressures. The equivalent fluid densities, which include hydrostatic pressures, for the submerged portion of the backfill should be increased to 85 and 95 pcf, respectively, for yielding and nonyielding walls. In addition, it may be necessary to provide waterproofing of elevator pits. As noted above, waterproofing for below-grade walls should extend up to at least 2 feet above the estimated seasonal high groundwater level.

4.9.3. Drainage

Drainage behind the permanent below-grade walls constructed in front of shoring walls is typically provided using prefabricated drainage board attached to the temporary shoring walls. The drainage board should be connected to weep pipes that extend through the permanent below-grade building walls at the footing elevation. The weep pipes through the permanent below-grade walls should be spaced no more than 12 feet on center and should have a minimum diameter of 2 inches. The weep pipes should be connected to perimeter perforated or slotted drains (described below).

Full wall face coverage is preferable for minimizing seepage and/or wet areas at the face of the permanent wall. As an alternative to full coverage drainage material, strips of drainage material at least 16 inches wide spaced at 6 feet on center can be used and will be sufficient to reduce the buildup of hydrostatic pressures acting on the basement walls. The drainage strips or full wall face coverage should extend from the weep pipe elevation up to about three feet below the top of the wall to reduce the potential for surface water to enter the wall drainage system. The tops of the drainage strips should be sealed to prevent soil and water entry.

Although the use of full wall face coverage will reduce the likelihood of seepage and/or wet areas at the face of the permanent wall, there is still a potential for these conditions to occur. If this is a concern, waterproofing should be specified. For example, waterproofing will be needed along the east side of the building where the planned infiltration trench will be located. We recommend elevator pits also be waterproofed, as discussed below.

For permanent walls constructed in open cut areas, positive drainage should be provided behind cast-in-place retaining walls by placing a minimum 2-foot-wide zone of Gravel Backfill for Walls, Section 9-03.12(2) of the WSDOT Standard Specifications.

A perforated or slotted drain pipe should be placed near the base of the retaining wall to provide drainage. The drain pipe should be surrounded by at least 6 inches of WSDOT Gravel Backfill for Drains, Section 9-03.12(4), or an alternative approved by GeoEngineers. The drainage material should be wrapped with a geotextile filter fabric meeting the requirements of Construction Geotextile for Underground Drainage, WSDOT Standard Specification 9-33.

The wall drain pipe should be connected to a header pipe and routed to a sump or gravity drain. Appropriate cleanouts for drain pipe maintenance should be installed. A larger-diameter pipe will allow for easier maintenance of drainage systems.

4.9.4. Waterproofing

Based on the previous explorations and our experience with similar projects in alluvial and recessional outwash soils, we anticipate waterproofing will generally not be required for this project, except for the east side of the building where the basement wall will be close to the planned infiltration trench. Elevator pits should also be waterproofed. As discussed above, the basement floor slab and the lower portion of the basement walls may also need to be waterproofed.

4.9.4.1. Waterproofing Options

There are many waterproofing options that include a wide range of risks and costs associated with each. Considerations include:

- Ease of implementation with the planned shoring and foundation systems;
- The planned use of the facility (for example, parking space, storage space, or habitable space);
- The consequences of water seepage;
- Options for mitigating water seeping into the facility; and
- Planned heating and ventilation for below-grade portions of the facility.

The considerations presented above along with the experience of the design team with the various waterproofing options should assist in identifying the appropriate waterproofing system for the site, if used.

There are three general types of below-grade waterproofing systems:

- Membranes/panels
- Fluid applied waterproofing
- Concrete additives

4.9.4.2. Membranes/Panels

Exterior building walls and elevator pits may be waterproofed by placing a membrane or a panel behind the walls or surrounding the pits. Available products include, but are not limited to:

- Bentonite panels (Volclay® or similar) consisting of 4-foot by 4-foot corrugated kraft panels filled with sodium bentonite clay;
- Bentonite composite liners (Voltex® or similar) consisting of two geotextile fabric layers encapsulating a layer of sodium bentonite clay;

- Dual waterproofing membranes comprised of a layer of high density polyethylene (HDPE) and a layer of sodium bentonite clay (Paraseal or Swelltite™);
- Rubberized asphalt and HDPE composite membranes (Bituthene®);
- HDPE membrane with a pressure sensitive adhesive that bonds to cast-in-place concrete or slab-on-grade concrete (Preprufe®); and
- Thermoplastic membrane with hot-air welded seams (Sarnafil®).

Bentonite waterproofing systems have been used extensively. One potential disadvantage with bentonite waterproofing systems is that repeated wet-dry cycles may cause the membrane to crack. Dual membranes offer two layers of protection in the event water penetrates the first layer. Membrane/panel waterproofing is relatively easy to apply to vertical surfaces such as temporary shoring; however, tieback heads create local discontinuities that can require special detailing. Where spread footings and utilities are present, membrane/panel waterproofing is more difficult to install. Hot-air welded systems offer more protection against seepage and leaks; however, the costs are relatively high.

4.9.4.3. Fluid Applied Waterproofing

Fluid applied waterproofing, such as Liquid Boot® or Procor®, provides waterproofing protection with the advantage of ease of application in areas where spread footings or other irregularly shaped features are present.

Concrete Additives

Additives, such as Caltite, can be added to the concrete used in below-grade walls and elevator pits as a waterproofing system. The primary advantage with the Caltite system is that minimal additional labor is required to install the waterproofing. Joints and penetrations in the concrete require special attention to prevent seepage and leaks.

4.9.5. Other Considerations

With each of the waterproofing systems described above, special attention should be directed to construction quality assurance and details such as joints and penetrations.

4.10. Pavement Design

4.10.1.1. Subgrade Preparation

We recommend the subgrade soils in new pavement areas be prepared and evaluated as described in the "Site Preparation" section of this report. If the exposed subgrade soils are loose or soft, it may be necessary to excavate localized areas and replace them with structural fill or crushed rock base course. Pavement subgrade conditions should be observed during construction and prior to placing the pavement section materials to evaluate the presence of zones of unsuitable subgrade soils and the need for over-excavation and replacement of these zones.

If necessary, 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.

4.10.1.2. New Hot-Mix Asphalt Pavements

At a minimum, paved areas exposed to automobile parking only should consist of 2 inches of hot-mix asphalt (HMA), Class ½ inch, PG 58-22 over 4 inches of crushed surfacing base course. In driveways and areas of occasional truck traffic, new pavement sections should consist of at least 3 inches of HMA (PG 64-22) per WSDOT Sections 5-04 and 9-03, over a minimum 6-inch thickness of compacted Crushed Surfacing Base Course per WSDOT Section 9-03.9(3). The crushed surfacing base course should be compacted to at least 95 percent of the MDD obtained using ASTM D 1557 prior to HMA placement.

All paved and landscaped areas should be graded so that surface drainage is directed to appropriate catch basins or other suitable disposal points.

4.11. Surface Drainage

We recommend pavement surfaces be sloped away from building areas to promote drainage away from the building. Pavement areas should be graded so that surface runoff does not pond and infiltrate into the pavement section. We recommend all roof drains be connected to a tight line leading to storm drain facilities.

4.12. Infiltration Considerations

We understand an infiltration trench is planned along the east property line and adjacent to the east side of the proposed building. The trench will be about 160 feet long and will have a bottom level at Elevation 39 feet, or about 3 to 4 feet below existing grade. We understand design of the trench is intended to infiltrate 100 percent of the developed site runoff.

We reviewed the results of test pits and grain size analyses on soil samples completed by AESI as part of the hydrogeologic and infiltration assessment for The Carter building to the north of the project site. AESI concluded an infiltration rate of 9 inches per hour could be used for design of an infiltration facility for The Carter. We also reviewed available subsurface information for the Evans site and the results of pilot infiltration tests (PITs) conducted for a nearby City of Redmond project.

On preliminary basis, we estimate a design infiltration rate of 9 inches per hour may be used for the proposed infiltration trench. This rate should be confirmed by test pit exploration and grain size testing of soil samples from the proposed trench area after the existing building is demolished.

4.13. Recommended Additional Geotechnical Services

GeoEngineers should be retained to review the project plans and specifications when complete to confirm our design recommendations have been implemented as intended. As mentioned above, the preliminary design infiltration rate should be confirmed at the start of project construction once the existing building is removed.

During construction, GeoEngineers should observe excavation and installation of the shoring system, review/collect shoring monitoring data, evaluate the suitability of the foundation subgrades, observe installation of subsurface drainage measures, evaluate structural backfill, observe the condition of temporary cut slopes, and provide a summary letter of our construction observation services. The purposes of GeoEngineers construction phase services are to confirm the subsurface conditions are consistent with those observed in the explorations and other reasons described in Appendix C, Report Limitations and Guidelines for Use.

5.0 LIMITATIONS

We have prepared this report for use by G.W. Williams Co., Cleverly Development Consulting, and their authorized agents in the design phase of The Osprey residential development project to be located at 7440 159th Place NE in Redmond, Washington.

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

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

Please refer to Appendix C for additional information pertaining to use of this report.

6.0 REFERENCES

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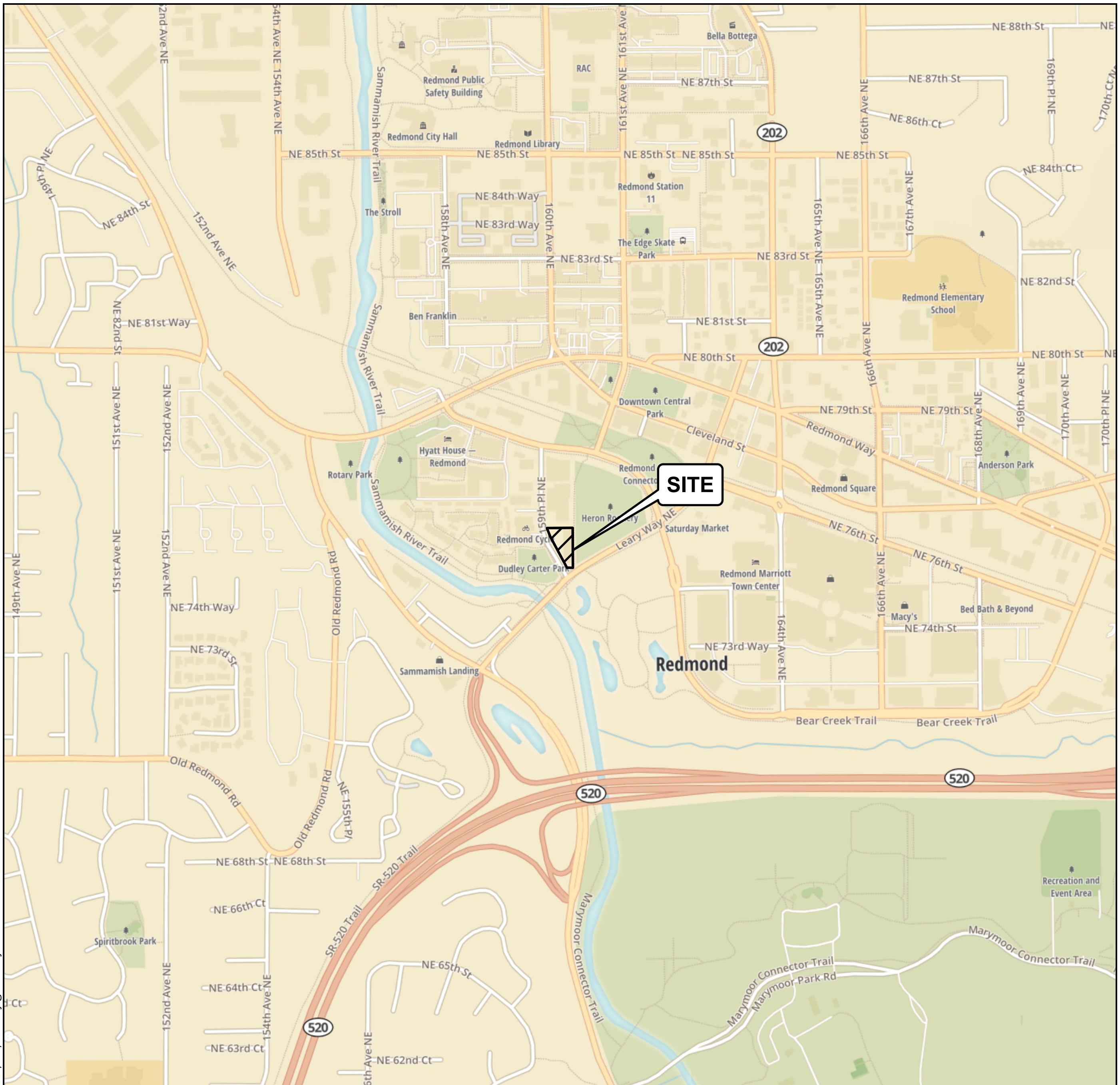
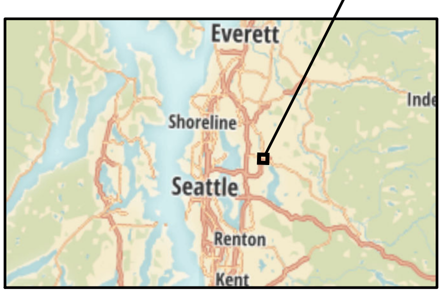
FIGURES





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Feet

Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. 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, 2018

Projection: NAD 1983 UTM Zone 10N

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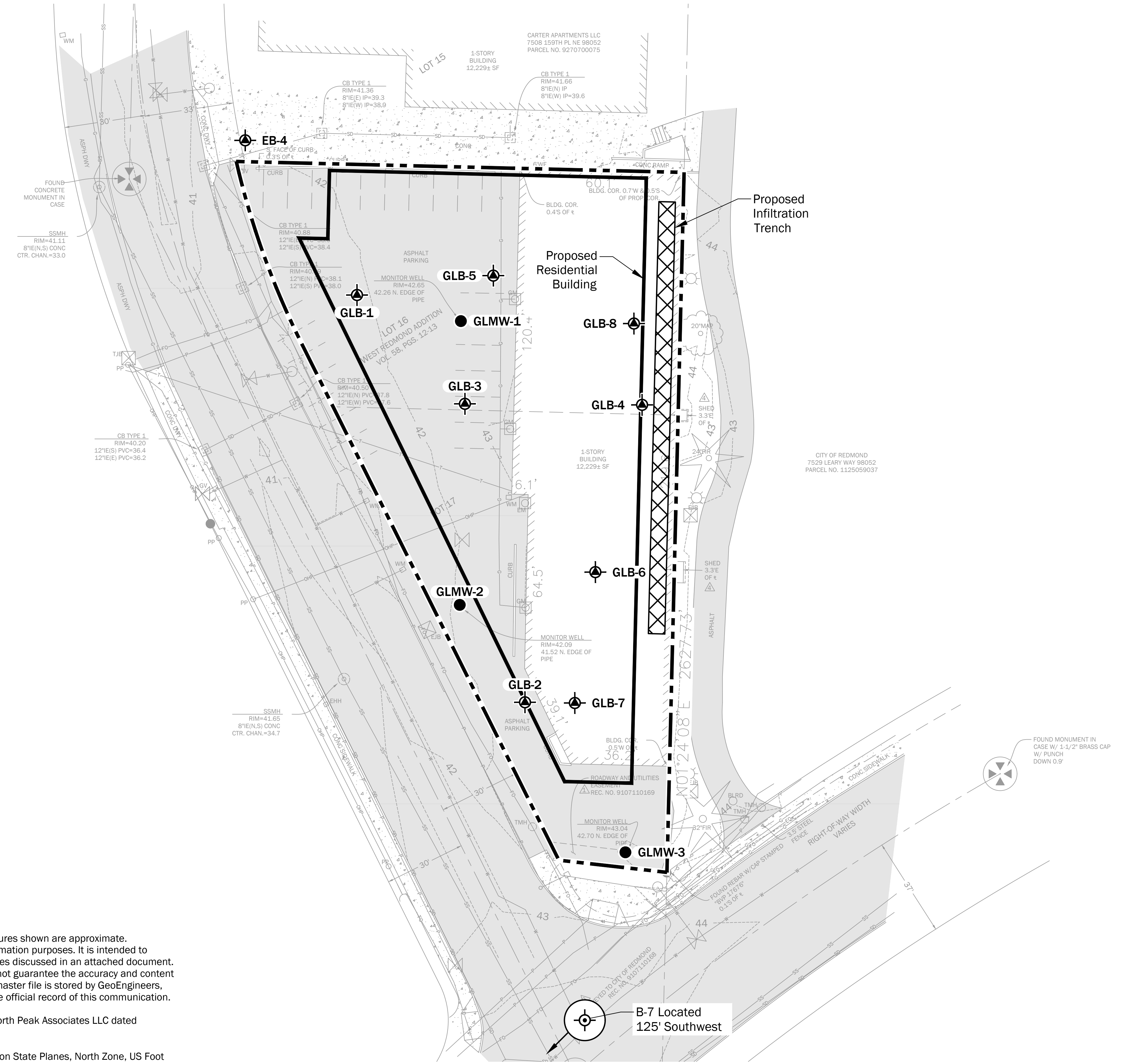
Vicinity Map

Proposed Residential Development
Redmond, Washington

GEOENGINEERS

Figure 1



Legend

- Property Boundary
- B-7 — Boring by GeoEngineers (1998)
- GLMW-1 — Monitoring Well by Others
- GLB-1 — Direct Push Boring by Others

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: Survey from North Peak Associates LLC dated 12/26/2018.

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

Site Plan

Proposed Residential Development
Redmond, Washington

GeoEngineers

Figure 2

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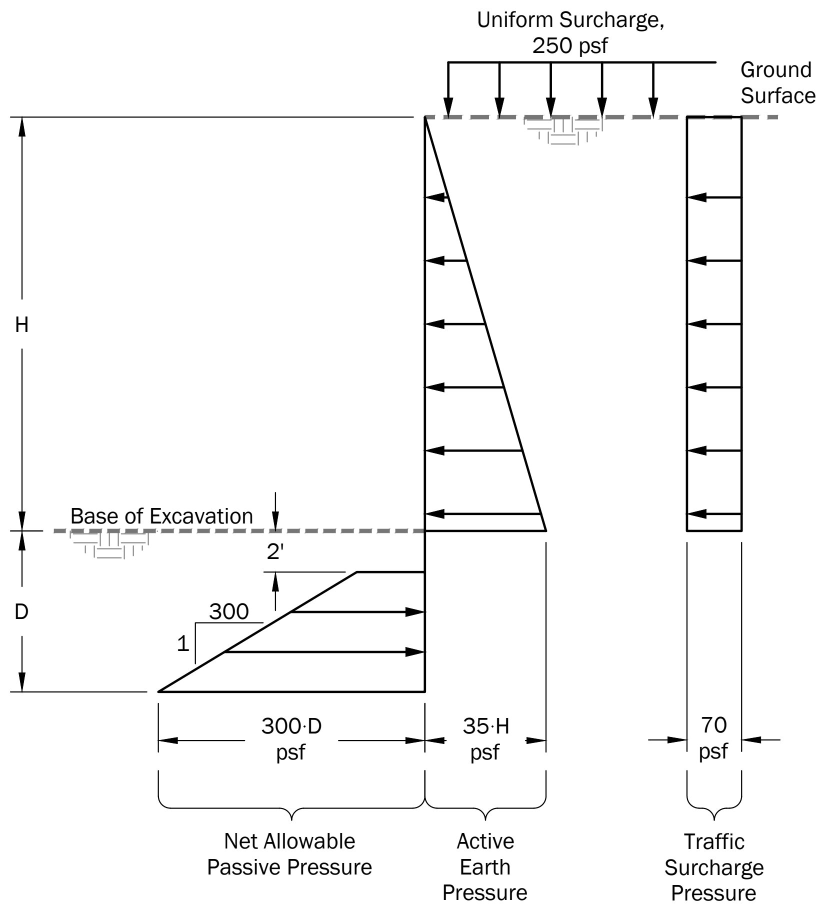
Cantilever Soldier Pile Wall



Legend

H = Height of Excavation, Feet

D = Soldier Pile Embedment Depth, Feet

Notes:

1. Active/apparent earth pressure and traffic surcharge pressure act over the pile spacing above the base of the excavation.
2. Passive earth pressure acts over 2.5 times the concreted diameter of the soldier pile, or the pile spacing, whichever is less.
3. Passive pressure includes a factor of safety of 1.5
4. Additional surcharge from footings of adjacent buildings should be included in accordance with recommendations provided on Figure 4.
5. This pressure diagram is appropriate for temporary soldier pile walls. If additional surcharge loading (such as from soil stockpiles, excavators, dumptrucks, cranes, or concrete trucks) is anticipated, GeoEngineers should be consulted to provide revised surcharge pressures.
6. Seismic pressure does not need to be included in design of temporary shoring walls.

Not To Scale

Earth Pressure Diagram
Temporary Soldier Pile Wall

Proposed Residential Development
Redmond, Washington

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Figure 3

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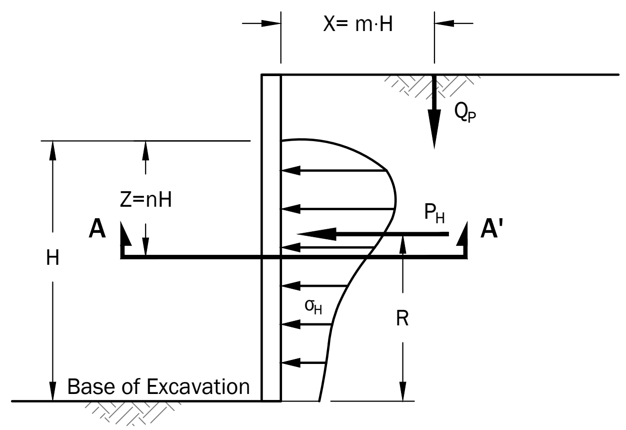
Lateral Earth Pressure from Point Load, Q_P (Spread Footing)



For $m \le 0.4$

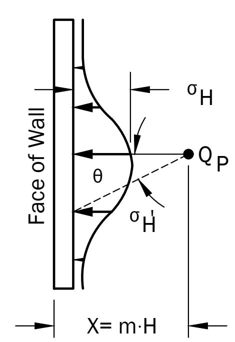
$$\sigma_H = \frac{0.28 Q_P n^2}{H^2(0.16+n^2)^3}$$

For $m > 0.4$

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

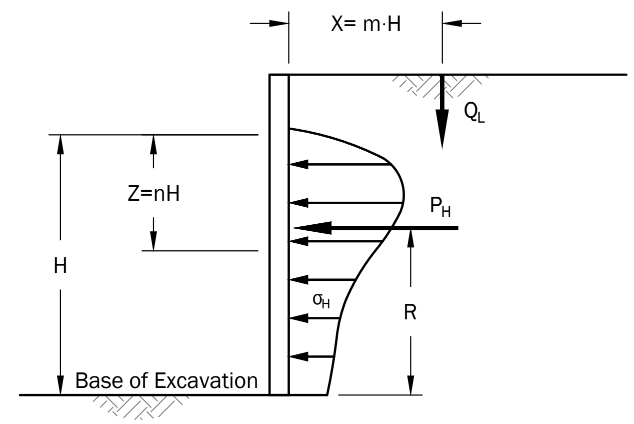
$$\sigma'_H = \sigma \, COS^2 (1.1\theta)$$

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



Section A-A'
Pressures from Point Load Q_P

Lateral Earth Pressure from Line Load, Q_L (Continuous Wall Footing)



For $m \le 0.4$

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

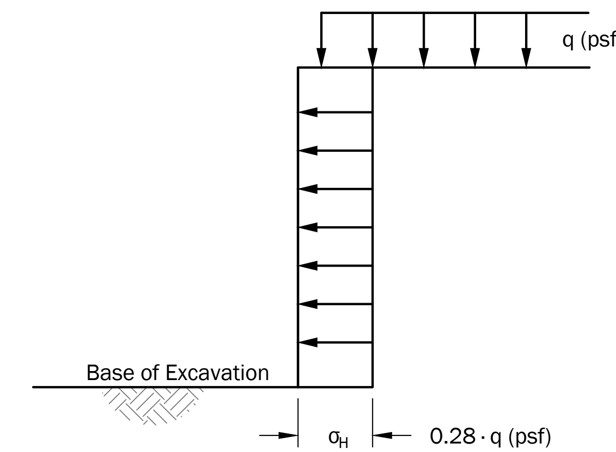
For $m > 0.4$

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

$$\text{Resultant } P_H = \frac{0.64 Q_L}{(m^2+1)}$$

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

Uniform Surcharges, q (Floor Loads, Large Foundation Elements or Traffic Loads)



σ_H = Lateral Surcharge Pressure from Uniform Surcharge

Definitions:

Q_P = Point load in pounds
Q_L = Line load in pounds/foot
H = Excavation height below footing, feet
σ_H = Lateral earth pressure from surcharge, psf
q = Surcharge pressure in psf
θ = Radians
σ'_H = Distribution of σ_H in plan view
P_H = Resultant lateral force acting on wall, pounds
R = Distance from base of excavation to resultant lateral force, feet
X = Resultant lateral force acting on wall, pounds
Z = Depth of σ_H to be evaluated below the bottom of Q_P or Q_L
m = Ratio of X to H
n = Ratio of Z to H

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

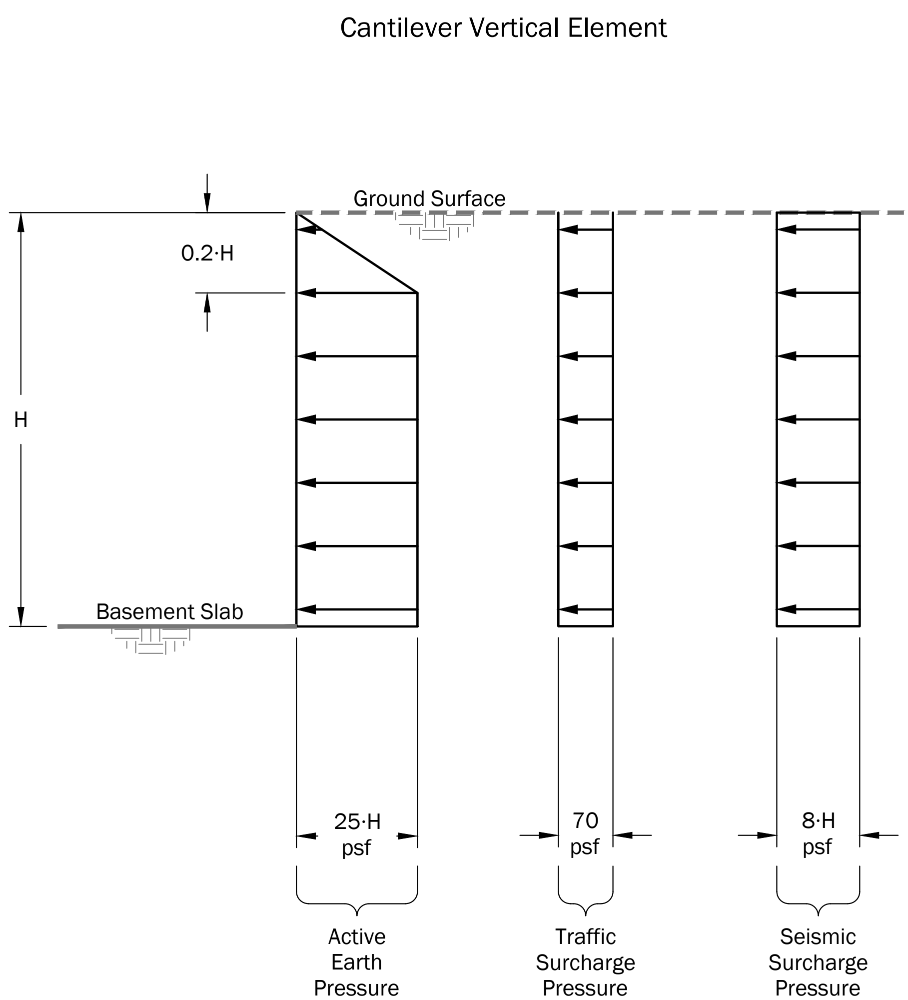
Not To Scale

Recommended Surcharge Pressure

Proposed Residential Development
Redmond, Washington

GEOENGINEERS

Figure 4



Legend

H = Height of Excavation, Feet

D = Vertical Embedment Depth, Feet

▽ Design Groundwater Elevation for Drained Walls/ Passive Resistance Design

Not To Scale

Earth Pressure Diagram
Permanent Walls Against Shoring Wall

Proposed Residential Development
Redmond, Washington

GEOENGINEERS

Figure 5

Notes:
1. Refer to the report text for further discussion.
2. Surcharge from footing loads for adjacent buildings should be added to the pressures shown above. Refer to Figure 4 for guidance.

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APPENDICES

GeoEngineers

APPENDIX A
Previous Explorations

APPENDIX A
PREVIOUS EXPLORATIONS

This appendix presents logs of selected borings completed by GeoEngineers in 1988 and by others in 2014 and 2018 within and near the project site.

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

BORING NO. 7

Lab Tests	Moisture Content	Dry Density	Blow-Count	Samples	Group Symbol	DESCRIPTION Approximate Elevation: 41 feet
						7 INCHES OF ASPHALTIC CONCRETE
MD	3.0%	127	20	■	SM	REDDISH-BROWN SILTY FINE TO MEDIUM SAND WITH GRAVEL AND OCCASIONAL CRUSHED ROCK AND COBBLES (MEDIUM DENSE, DRY TO MOIST) (FILL)
			14	■		
					SP	BROWN FINE TO MEDIUM SAND WITH GRAVEL (MEDIUM DENSE, MOIST) (FILL)
			29	■	SM	DARK BROWN SILTY FINE SAND WITH GRAVEL AND A TRACE OF ORGANIC MATTER AND OCCASIONAL COBBLES (MEDIUM DENSE, DRY) (FILL)
MD	2.5%	117	25	■	SP	BROWN FINE TO MEDIUM SAND WITH A TRACE OF COARSE SAND AND GRAVEL (MEDIUM DENSE, MOIST)
MD, DS	3.7%	105	18	■		
					SW	BROWN FINE TO COARSE SAND WITH A TRACE OF SILT AND GRAVEL, AND OCCASIONAL COBBLES (MEDIUM DENSE, WET)
MD	11.0%	127	19	■		
			24	■		
					SW	GRAY FINE TO COARSE SAND WITH GRAVEL (MEDIUM DENSE TO DENSE, WET)
MD, DS	10.0%	130	34	■		
M, GS	6.8%		37	■		

DEPTH IN FEET: 0, 5, 10, 15, 20, 25, 30, 35, 40

Note: See Figure A-2 for Explanation of Symbols

GeoEngineers Incorporated

LOG OF BORING

FIGURE A-10

BORING NO. 7 (Continued)

Depth in Feet	Lab Tests	Moisture Content	Dry Density	Blow-Count	Samples	Group Symbol	Description
40						SP	GRAY FINE TO MEDIUM SAND (MEDIUM DENSE, WET)
	MD, DS	18.0%	104	20	■	SW	GRAY FINE TO COARSE SAND WITH GRAVEL AND OCCASIONAL COBBLES (MEDIUM DENSE, WET)
45						SP	GRAY FINE TO MEDIUM SAND WITH A TRACE OF SILT (DENSE, WET)
	MD	8.3%	135	41	■		
50							BORING COMPLETED AT 49.0 FEET ON 3/10/88

GROUND WATER NOTED AT A DEPTH OF ABOUT 21 FEET DURING DRILLING

Note: See Figure A-2 for Explanation of Symbols

GeoEngineers Incorporated

LOG OF BORING

FIGURE A-11

Associated Earth Sciences, Inc.	Exploration Log		
	Project Number KE140132A	Exploration Number EB-4	Sheet 1 of 1

Project Name: Queen City Auto
Location: Redmond, WA
Driller/Equipment: Geologic Drill / XL Rig
Hammer Weight/Drop: 140# / 30"

Ground Surface Elevation (ft):
Datum: N/A
Date Start/Finish: 3/31/14,3/31/14
Hole Diameter (in): 8 inches

Depth (ft)	S T	Samples	Graphic Symbol	DESCRIPTION	Well Completion	Water Level	Blows/6"	Blows/Foot (10 20 30 40)	Other Tests
				Asphalt - 2 inches					
				Quaternary Younger Alluvium					
		S-1		Medium dense, moist, brown, medium sandy fractured GRAVEL, few fine sand, few silt; stratified (~3 inches thick) (GP-GM).			11 12 13	▲25	
5		S-2		Medium dense, moist to very moist, brown, fine to medium SAND, little fractured gravel, trace coarse sand, few silt; stratified to thinly stratified (SM-SP).			9 11 13	▲24	
		S-3		Very dense, slightly moist, brown to dark brown, fine to medium SAND, with gravel, trace coarse sand, few to little silt; faintly stratified (SM-SP).			15 18 32	▲50	
10		S-4		Very dense, slightly moist to moist, brown, fine to medium SAND, little gravel, trace coarse sand, few to little silt; faintly stratified (SM-SP).			21 44 25	▲69	
						▼			
15		S-5		Medium dense, wet, brown, fine to medium SAND, with fine gravel, few coarse sand, few silt; faintly stratified (~4 inches thick) (SM-SP).			14 13 12	▲25	
20		S-6		6 inch sample; 7 inches heave. Very dense, wet, brown, gravelly fine to medium SAND, trace coarse sand, few silt; massive (SM-SP).			50/6"	▲50/6"	
25		S-7		9 inch sample; 12 inches heave. Very dense, wet, grayish brown, GRAVEL, with medium to coarse sand, trace fine sand, trace silt; massive (GP).			24 34 33	▲67	
				Bottom of exploration boring at 26.5 feet Blow counts are likely overstated due to high gravel content of soils. Soil densities likely range from loose to medium dense.					
30									
35									

Sampler Type (ST):
- 2" OD Split Spoon Sampler (SPT)
- 3" OD Split Spoon Sampler (D & M)
- Grab Sample
- No Recovery
- Ring Sample
- Shelby Tube Sample
- M - Moisture
- ▽ Water Level ()
- ▼ Water Level at time of drilling (ATD)

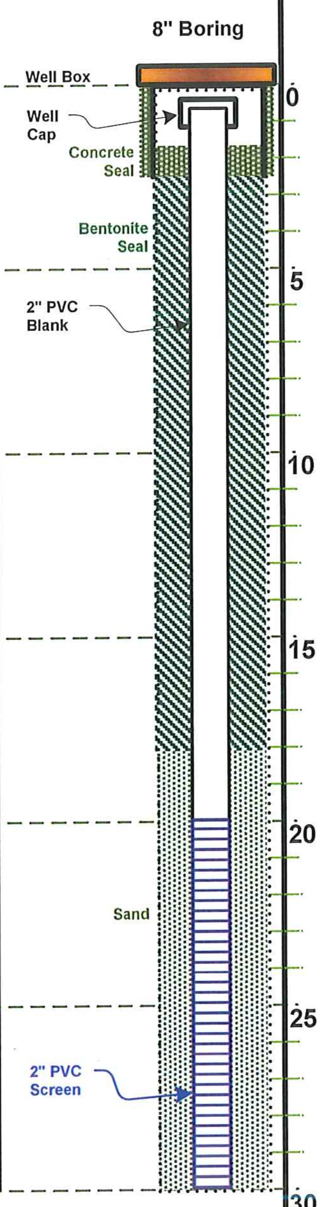
Logged by: DMG
Approved by:

AESIBOR 140132.GPJ April 10, 2014

glmw-1 1.vsd

Depth in feet	BLOWS/6 inches	INTERVAL	SAMPLE NUMBER	SOIL DESCRIPTION	Recovery %	USCS	PID (ppmv in headspace)
0	16 21 25			No Recovery at 2.5'			
5	10 14 15		GLMW-1-5	5-6.5': SAND, medium to coarse grained with fine to coarse gravel, brown, dry, no odor.	5	SW	0.0
10	18 24 16		GLMW-1-10	10-11.5': SAND, medium to coarse grained with fine to coarse gravel, brown, dry, no odor.	30	SW	0.0
15	18 29 27		GLMW-1-15	15-16.5': SAND, medium to coarse grained with fine to coarse gravel, brown, dry, no odor.	20	SW	0.1
20	50/3		GLMW-1-20	20-21.5': SAND, coarse grained with fine to coarse gravel, brown, wet, no odor.	60	SW	0.1
25	8 40 50/3		GLMW-1-25	25-25.75': SAND, coarse grained with fine to coarse gravel, brown, wet, no odor. 25.75-26.5': SAND, fine grained with trace silt, brown, moist to wet, no odor.	100	SW SP	0.1
30							

WELL CONSTRUCTION



Depth in feet

Drilling Method: Hollow-stem auger	**Date:** 6/19/2018	**Other Information:**
Drilling Company: Holocene	**Weather:** Sunny	Well Tag BKZ-663
Boring Diameter: 8"	**Page** 1 **of** 2	
Logged By: H. Carter		

g-logics

Boring/Well Log
Evans Auto Center
7440 159th Place NE
Redmond, Washington

GLMW-1

glmw-1 2.vsd

Depth (ft)	BLOWS/6 inches	INTERVAL	SAMPLE NUMBER	SOIL DESCRIPTION	Recovery %	USCS	PID (ppmv in headspace)	WELL CONSTRUCTION
								8" Boring
30	8 / 14 / 50/6		GLMW-1-30	30-30.75': GRAVEL with trace medium grained sand, brown, moist to wet, no odor.	100	GW	0.0	Bentonite Backfill
				30.75-31.5': SAND, fine grained with trace silt, brown, moist to wet, no odor.		SP		
35	8 / 32 / 50/4		GLMW-1-35	35-36.5': SAND, medium to coarse grained with fine to coarse gravel, brown, moist to wet, no odor.	100	SW	0.1	
40	7 / 21 / 28		GLMW-1-40	40-41.5': SAND, medium to coarse grained with fine to coarse gravel, brown, moist to wet, no odor.	100	SW	0.1	
45								
50								
55								
60								

Depth in feet

Drilling Method: Hollow-stem auger	Date: 6/19/2018	Other Information:
Drilling Company: Holocene	Weather: Sunny	Well Tag BKZ-664
Boring Diameter: 8"	Page 2 of 2	
Logged By: H. Carter		

g-logics

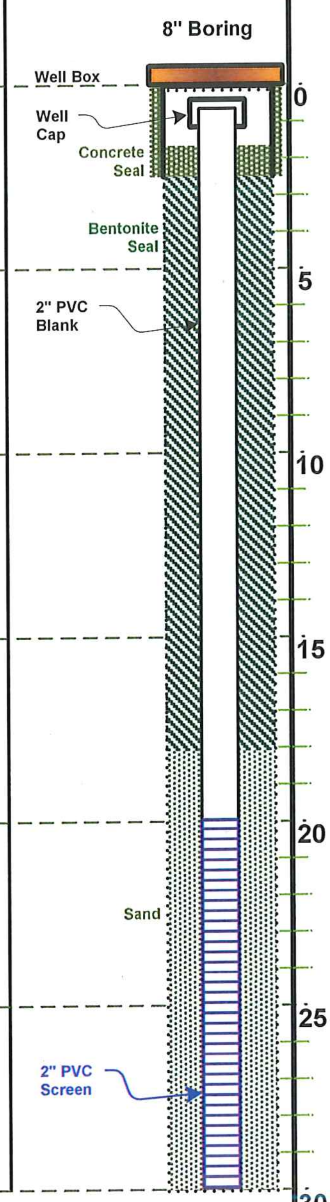
Boring/Well Log
Evans Auto Center
7440 159th Place NE
Redmond, Washington

GLMW-1

glmw-2 1.vsd

Depth in feet	BLOWS/6 inches	INTERVAL	SAMPLE NUMBER	SOIL DESCRIPTION	Recovery %	USCS	PID (ppmv in headspace)
0							
5	5 14 13		GLMW-2-5	5-6.5': SAND, medium to coarse grained with fine to coarse gravel, brown, moist, no odor.	40	SW	0.0
10	6 11 11		GLMW-2-10	10-11.5': SAND, medium to coarse grained with fine to coarse gravel, brown, moist, no odor.	30	SW	0.1
15	15 28 30		GLMW-2-15	15-16.5': SAND, medium to coarse grained with fine to coarse gravel, brown, dry, no odor.	40	SW	0.0
20	24 50/4		GLMW-2-20	20-21.5': SAND, coarse grained with fine to coarse gravel, brown, wet, no odor. Very thin layer of silty sand at 21.5'.	20	SW	0.1
25	20 50/3		GLMW-2-25	25-26.5': SAND, fine to medium grained with trace gravel, brown, wet, no odor.	70	SP	0.0
30							

WELL CONSTRUCTION



Drilling Method: Hollow-stem auger	**Date:** 6/19/2018	**Other Information:** Well Tag BKZ-664
Drilling Company: Holocene	**Weather:** Sunny	
Boring Diameter: 8"	**Page** 1 **of** 2	
Logged By: H. Carter		

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Boring/Well Log
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Redmond, Washington

GLMW-2

glmw-2 2.vsd

Depth	BLOWS/6 inches	INTERVAL	SAMPLE NUMBER	SOIL DESCRIPTION	Recovery %	USCS	PID (ppmv in headspace)	WELL CONSTRUCTION
								8" Boring
30	3 15 27		GLMW-2-30	30-31.5': SAND, fine to medium grained with trace gravel, brown, moist to wet, no odor.	100	SP	0.0	Bentonite Backfill
35	11 14 18		GLMW-2-35	35-36.5': SAND, medium grained with trace gravel, brown, moist to wet, no odor.	90	SP	0.1	
40	3 9 13		GLMW-2-40	40-41.5': SAND, medium grained with trace gravel, brown, wet, no odor.	30	SP	0.1	
45								
50								
55								
60								

Depth in feet

Drilling Method: Hollow-stem auger	Date: 6/19/2018	Other Information: Well Tag BKZ-664
Drilling Company: Holocene	Weather: Sunny	
Boring Diameter: 8"	Page 2 of 2	
Logged By: H. Carter		

g-logics

Boring/Well Log
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Redmond, Washington

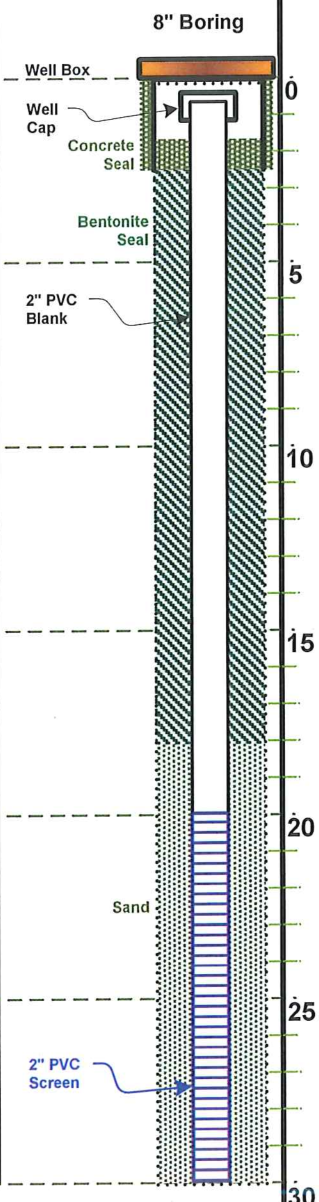
GLMW-2

glmw-3 1.vsd

Depth (feet)	BLOWS/6 inches	INTERVAL	SAMPLE NUMBER	SOIL DESCRIPTION	Recovery %	USCS	PID (ppmv in headspace)
0							
	2 4 4		GLMW-3-2.5	2.5-4': SAND, fine grained with fine gravel, brown, dry, slight petroleum odor, woodchips at 4'.	10	SW	6.1
5	12 18 14		GLMW-3-5	5-6.5': SAND, fine grained with fine to coarse gravel, brown, dry, no odor.	15	SW	2.8
10	10 37 10		GLMW-3-10	10-11.5': SAND, medium grained with fine to coarse gravel, brown, moist to dry, no odor.	50	SW	0.3
15	20 40 41		GLMW-3-15	15-16.5': SAND, medium grained with fine to coarse gravel, brown, moist to dry, no odor.	60	SW	0.1
20	34 17 19		GLMW-3-20	20-21.5': SAND, medium grained with fine to coarse gravel, brown, wet, no odor.	10	SW	0.0
25	2 8 32		GLMW-3-25	25-26.5': SAND, fine to medium grained with trace gravel, gray brown, moist, no odor.	100	SP	0.1
30							

Depth in feet

WELL CONSTRUCTION



Drilling Method: Hollow-stem auger	**Date:** 6/20/2018	**Other Information:**
Drilling Company: Holocene	**Weather:** Sunny	Well Tag BKZ-665
Boring Diameter: 8"	**Page** 1 **of** 2	
Logged By: H. Carter		

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Boring/Well Log
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Redmond, Washington

GLMW-3

glmw-3 2.vsd

Depth (feet)	BLOWS/6 inches	INTERVAL	SAMPLE NUMBER	SOIL DESCRIPTION	Recovery %	USCS	PID (ppmv in headspace)	WELL CONSTRUCTION
								8" Boring
30	14 50/6		GLMW-3-30	30-31.5': SAND, fine to medium grained with trace gravel, brown, moist to wet, no odor.	100	SP	0.0	Bentonite Backfill
35	6 17 47		GLMW-3-35	35-36.5': SAND, medium grained with trace gravel, brown, moist to wet, no odor.	100	SP	0.0	
40	6 17 32		GLMW-3-40	40-41.5': SAND, fine to medium grained with trace gravel, brown, wet, no odor.	90	SP	0.0	
45								
50								
55								
60								

Depth in feet

Drilling Method: Hollow-stem auger	Date: 6/20/2018	Other Information:
Drilling Company: Holocene	Weather: Sunny	Well Tag BKZ-665
Boring Diameter: 8"	Page 2 of 2	
Logged By: H. Carter		

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Boring/Well Log
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Redmond, Washington

GLMW-3

glb-1.vsd

Depth (feet)	BLOWS/6 inches	INTERVAL	SAMPLE NUMBER	SOIL DESCRIPTION	Recovery %	USCS	PID (ppmv in headspace)	WELL CONSTRUCTION
0								Temporary Boring, Backfilled with Bentonite
	5 10 11		GLB-1-2.5	2.5-4': SAND, medium to coarse grained with fine to coarse gravel, brown, moist, no odor.	5	SW	0.3	
5	14 16 20		GLB-1-5	5-6.5': SAND, fine grained with fine to coarse gravel, gray brown, dry, no odor.	<5		0.0	
	16 50/5		GLB-1-7.5	7.5-9': SAND, medium to coarse grained with fine to coarse gravel, brown, moist to dry, no odor.	15	SW		
10	8 11 7		GLB-1-10	10-11.5': SAND, medium to coarse grained with fine to coarse gravel, brown, moist to dry, no odor.	30	SW	0.0	
15				No Recovery				
20	50/5		GLB-1-20	20-21.5': SAND, coarse grained with fine to coarse gravel, gray brown, wet, no odor.	15	SW	0.1	
25								
30								

Depth in feet

Drilling Method: Hollow-stem auger	Date: 6/20/2018	Other Information:
Drilling Company: Holocene	Weather: Sunny	
Boring Diameter: 8"	Page 1 of 1	
Logged By: H. Carter		

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Boring/Well Log
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Redmond, Washington

GLB-1

glb-2.vsd

Depth (feet)	BLOWS/6 inches	INTERVAL	SAMPLE NUMBER	SOIL DESCRIPTION	Recovery %	USCS	PID (ppmv in headspace)	WELL CONSTRUCTION
0								Temporary Boring, Backfilled with Bentonite
	7 11 6		GLB-2-2.5	2.5-4': SAND, medium grained with fine to coarse gravel, brown, dry, no odor.	10	SW	0.2	
5	7 11 18		GLB-2-5	5-6.5': SAND, medium grained with fine to coarse gravel, brown, dry, no odor. No Recovery	5	SW	0.3	
10	13 17 12		GLB-2-10	10-11.5': SAND, fine to medium grained with fine to coarse gravel, brown, dry, no odor.	5	SW	0.3	
15	10 19 19		GLB-2-15	10-11.5': SAND, fine to medium grained with fine to coarse gravel, brown, dry, no odor.	40	SW	0.4	
20	10 7 4		GLB-2-20	20-21.5': SAND, fine to medium grained with fine to coarse gravel, brown, wet, no odor.	10	SW	0.3	
25								
30								

Depth in feet

Drilling Method: Hollow-stem auger	Date: 6/20/2018	Other Information:
Drilling Company: Holocene	Weather: Sunny	
Boring Diameter: 8"	Page 1 of 1	
Logged By: H. Carter		

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Redmond, Washington

GLB-2

glb-3.vsd

Depth in feet	BLOWS/6 inches	INTERVAL	SAMPLE NUMBER	SOIL DESCRIPTION	Recovery %	USCS	PID (ppmv in headspace)	WELL CONSTRUCTION
0				0-4': SAND, very fine to fine grained with fine to coarse gravel and silt, brown, moist, no odor.		SW		Temporary Boring, Backfilled with Bentonite
			GLB-3-4		20		0.0	
5				4-19': SAND, medium grained with fine to coarse gravel, brown, dry, no odor.				
			GLB-3-8		15	SW	0.0	
10			GLB-3-12		15	SW	0.0	
15			GLB-3-16		20	SW	0.0	
					60			
20			GLB-3-20	19-24': SAND, medium to coarse grained with fine to coarse gravel and trace silt, brown, wet, no odor.		SW	0.0	
			GLB-3-24		70		0.0	
25								
30								

Drilling Method: Direct Push	Date: 6/19/2018
Drilling Company: Holocene	Weather: Sunny
Boring Diameter: 2"	Page 1 of 1
Logged By: H. Carter	

Other Information:

g-logics

Boring/Well Log
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Redmond, Washington

GLB-3

glb-4.vsd

Depth in feet	BLOWS/6 inches	INTERVAL	SAMPLE NUMBER	SOIL DESCRIPTION	Recovery %	USCS	PID (ppmv in headspace)	WELL CONSTRUCTION
0				0-9': SAND, very fine to fine grained with fine to coarse gravel and silt, light brown, dry, no odor.		SW		Temporary Boring, Backfilled with Bentonite
			GLB-4-5		70		0.0	
5						SW		
			GLB-4-9	9-12': SAND, very fine to fine grained with fine to coarse gravel and silt, red, dry, slight petroleum odor.	60		0.0	
10			GLB-4-12	12-16': SAND, very fine to fine grained with fine to coarse gravel and silt, light brown, dry, no odor.	30	SW	0.0	
15			GLB-4-16	16-19': SAND, fine grained with fine to coarse gravel and trace silt, red, dry, no odor.	30	SW	0.2	
			GLB-4-19		40	▽	0.6	
20								
25								
30								

Depth in feet

Drilling Method: Direct Push	Date: 6/19/2018	Other Information: Drill tip stuck in ground at 10'. Had to move hole and re-drill.
Drilling Company: Holocene	Weather: Sunny	
Boring Diameter: 2"	Page 1 of 1	
Logged By: H. Carter		

g-logics

Boring/Well Log
Evans Auto Center
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Redmond, Washington

GLB-4

glb-5.vsd

BLOWS/6 inches	INTERVAL	SAMPLE NUMBER	SOIL DESCRIPTION	Recovery %	USCS	PID (ppmv in headspace)	WELL CONSTRUCTION
			No Recovery				Temporary Boring, Backfilled with Bentonite
6 7 9		GLB-5-5	5-6.5': SAND, medium to coarse grained with fine to coarse gravel, brown, moist to dry, no odor.	10		0.0	
11 50/6		GLB-5-7.5	7.5-9': SAND, medium to coarse grained with fine to coarse gravel, brown, moist to dry, no odor.	5	SW	0.0	
14 19 12		GLB-5-10	10-11.5': SAND, medium to coarse grained with fine to coarse gravel, brown, moist to dry, no odor.	5	SW	0.0	
4 12 19		GLB-5-15	10-11.5': SAND, fine to medium grained with trace silt, brown, moist to dry, no odor.	40	SP	0.0	
22 50/3		GLB-5-20	20-21.5': SAND, medium to coarse grained with fine to coarse gravel, brown, wet, no odor.	10	SW	0.0	

Depth in feet: 0, 5, 10, 15, 20, 25, 30

Drilling Method: Hollow-stem auger	Date: 6/20/2018	Other Information:
Drilling Company: Holocene	Weather: Sunny	
Boring Diameter: 8"	Page 1 of 1	
Logged By: H. Carter		

g-logics

Boring/Well Log
Evans Auto Center
7440 159th Place NE
Redmond, Washington

GLB-5

glb-6.vsd

Depth (feet)	BLOWS/6 inches	INTERVAL	SAMPLE NUMBER	SOIL DESCRIPTION	Recovery %	USCS	PID (ppmv in headspace)	WELL CONSTRUCTION
0			GLB-6-2.5	0-2.5': SAND, fine grained with silt and occasional gravel, brown, dry, no odor. 2.5-13': SAND, medium to coarse grained with fine to coarse gravel and silt, brown, dry to moist, no odor.	30	SW	0.0	Temporary Boring, Backfilled with Bentonite
5			GLB-6-7.5			SW	0.0	
			GLB-6-10		50			
10							0.0	
			GLB-6-13	13-15': SAND, coarse grained with silt and trace gravel, brown, slightly moist, no odor.	100	SP	0.0	
15			GLB-6-20	15-20': SAND, medium to coarse grained with fine to coarse gravel, brown, dry to wet (20'), no odor.	40	SW	0.0	
20								
25								
30								

Depth in feet

Drilling Method: Direct Push	Date: 6/26/2018	Other Information:
Drilling Company: Holocene	Weather: Cloudy	
Boring Diameter: 2"	Page 1 of 1	
Logged By: Z. Wall		

g-logics

Boring/Well Log
Evans Auto Center
7440 159th Place NE
Redmond, Washington

GLB-6

glb-7.vsd

BLOWS/6 inches	INTERVAL	SAMPLE NUMBER	SOIL DESCRIPTION	Recovery %	USCS	PID (ppmv in headspace)	WELL CONSTRUCTION
	0–5	GLB-7-5	0-10': SAND, fine grained with silt and trace gravel, brown, dry, no odor.	5	SW	0.0	Temporary Boring, Backfilled with Bentonite
	5–10	GLB-7-10		5	SW	0.8	
	10–15	GLB-7-15	10-15': SAND, fine to coarse grained with fine to coarse gravel, brown, slightly moist, no odor.	75	SW	0.0	
	15–20	GLB-7-20	15-23': SAND, medium to coarse grained with fine to coarse gravel and silt, brown, dry to slightly moist, no odor.	100	SW	0.0	
	20–25	GLB-7-23	23-25': SAND, coarse grained with fine to coarse gravel, brown, wet, no odor.	100	SW	0.0	

Depth in feet: 0, 5, 10, 15, 20, 25, 30

Drilling Method: Direct Push	Date: 6/26/2018	Other Information:
Drilling Company: Holocene	Weather: Cloudy	
Boring Diameter: 2"	Page 1 of 1	
Logged By: Z. Wall		

g-logics

Boring/Well Log
Evans Auto Center
7440 159th Place NE
Redmond, Washington

GLB-7

glb-8.vsd

BLOWS/6 inches	INTERVAL	SAMPLE NUMBER	SOIL DESCRIPTION	Recovery %	USCS	PID (ppmv in headspace)	WELL CONSTRUCTION
	0-5	GLB-8-5	0-5': SAND, fine to medium grained with silt, Gravel, and wood, brown, dry, no odor.	5	SW	0.0	Temporary Boring, Backfilled with Bentonite
	5-10	GLB-8-10	5-8': GRAVEL, fine to coarse grained with sand and silt, light brown, dry, no odor. 8-10': SAND, fine to coarse grained with fine to coarse gravel, brown, dry, no odor.	5	GW	0.0	
	10-15		No Recovery	75	SW		
	15-20		No Recovery	100	SW		
	20-25	GLB-8-20	23-25': SAND, fine to medium grained with fine to coarse gravel, brown, dry, no odor.		SW	0.0	
		GLB-8-25		100		0.0	

Depth in feet: 0, 5, 10, 15, 20, 25, 30

Drilling Method: Direct Push	Date: 6/26/2018	Other Information:
Drilling Company: Holocene	Weather: Cloudy	
Boring Diameter: 2"	Page 1 of 1	
Logged By: Z. Wall		

g-logics

Boring/Well Log
Evans Auto Center
7440 159th Place NE
Redmond, Washington

GLB-8

APPENDIX B
Shoring Monitoring Program

APPENDIX B
SHORING MONITORING PROGRAM

Preconstruction Survey

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

Optical Survey

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

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

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

For shoring walls, we recommend optical survey points be established along the top of the shoring walls, at the curb behind the shoring walls, and along the centerline of adjacent streets. The survey points should be established at the top of every other soldier pile along the wall face for soldier pile walls. The points along the curb line/centerline should be spaced every 25 feet. If lateral wall movements are observed to exceed ½ inch between successive readings or if total wall movements exceed 1 inch, construction of the shoring walls should be stopped to determine the cause of the movement and to establish the type and extent of remedial measures required.

APPENDIX C
Report Limitations and Guidelines for Use

APPENDIX C
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.

Read These Provisions Closely

It is important to recognize that the geoscience practices (geotechnical engineering, geology and environmental science) rely on professional judgment and opinion to a greater extent than other engineering and natural science disciplines, where more precise and/or readily observable data may exist. To help clients better understand how this difference pertains to our services, GeoEngineers includes the following explanatory "limitations" provisions in its reports. Please confer with GeoEngineers if you need to know more how these "Report Limitations and Guidelines for Use" apply to your project or site.

Geotechnical Services are Performed for Specific Purposes, Persons and Projects

This report has been prepared for G. W. Williams Co., Cleverly Development Consulting and members of the design team for the project specifically identified in this report. The information contained herein is not applicable to other sites or projects.

GeoEngineers structures its services to meet the specific needs of its clients. No party other than the party to whom this report is addressed may rely on the product of our services unless we agree to such reliance in advance and in writing. Within the limitations of the agreed scope of services for the Project, and its schedule and budget, our services have been executed in accordance with our revised proposal dated May 20, 2019 and generally accepted geotechnical practices in this area at the time this report was prepared. We do not authorize, and will not be responsible for, the use of this report for any purposes or projects other than those identified in the report.

A Geotechnical Engineering or Geologic Report is Based on A Unique Set of Project-Specific Factors

This report has been prepared for the design phase of The Osprey residential development to be located at 7440 159th Place NE 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.

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

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

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

If changes occur after the date of this report, GeoEngineers cannot be responsible for any consequences of such changes in relation to this report unless we have been given the opportunity to review our interpretations and recommendations. Based on that review, we can provide written modifications or confirmation, as appropriate.

Environmental Concerns Are Not Covered

Unless environmental services were specifically included in our geotechnical scope of services, this report does not provide any environmental findings, conclusions, or recommendations, including but not limited to, the likelihood of encountering underground storage tanks or regulated contaminants.

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 man-made events such as construction on or adjacent to the site, new information or technology that becomes available subsequent to the report date, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations. If more than a few months have passed since issuance of our report or work product, or if any of the described events may have occurred, please contact GeoEngineers before applying this report for its intended purpose so that we may evaluate whether changed conditions affect the continued reliability or applicability of our conclusions and recommendations.

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 the specific subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoEngineers reviewed field and laboratory data and then applied its professional judgment to render an informed opinion about subsurface conditions at other locations. Actual subsurface conditions may differ, sometimes significantly, from the opinions presented in this report. Our report, conclusions and interpretations are not a warranty of the actual subsurface conditions.

Geotechnical Engineering Report Recommendations Are Not Final

We have developed our preliminary recommendations based on data gathered from subsurface exploration(s). These explorations sample just a small percentage of a site to create a snapshot of the subsurface conditions elsewhere on the site. Such sampling on its own cannot provide a complete and accurate view of subsurface conditions for the entire site. Therefore, the recommendations included in this report are preliminary and should not be considered final. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for the recommendations in this report if we do not perform construction observation.

We recommend that you allow sufficient monitoring, testing and consultation during construction by GeoEngineers to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes if the conditions revealed during the work differ from those anticipated, and to evaluate whether earthwork and foundation installation activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective means of managing the risks associated with unanticipated conditions. If another party performs field observation and confirms our expectations, the other party must take full responsibility for both the observations and recommendations. Please note, however, that another party would lack our project-specific knowledge and resources.

A Geotechnical Engineering or Geologic Report Could Be Subject to Misinterpretation

Misinterpretation of this report by members of the design team or by contractors can result in costly problems. GeoEngineers can help reduce the risks of misinterpretation by conferring with appropriate members of the design team after submitting the report, reviewing pertinent elements of the design team's plans and specifications, participating in pre-bid and preconstruction conferences, and providing construction observation.

Do Not Redraw the Exploration Logs

Geotechnical engineers and geologists prepare final exploration logs based upon their interpretation of field logs and laboratory data. The logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Photographic or electronic reproduction is acceptable, but separating logs from the report can create a risk of misinterpretation.

Give Contractors a Complete Report and Guidance

To help reduce the risk of problems associated with unanticipated subsurface conditions, GeoEngineers recommends giving contractors the complete geotechnical engineering or geologic report, including these "Report Limitations and Guidelines for Use." When providing the report, you should preface it with a clearly written letter of transmittal that:

- advises contractors that the report was not prepared for purposes of bid development and that its accuracy is limited; and
- encourages contractors to confer with GeoEngineers and/or to conduct additional study to obtain the specific types of information they need or prefer.

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 adjacent properties.

Biological Pollutants

GeoEngineers' Scope of Services 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.

A Client who desires these specialized services is advised to obtain them from a consultant who offers services in this specialized field.

Information on Water Levels in the Ground May Be Confusing

The groundwater information in this report may appear confusing and could be misunderstood. We try to show the depth at which groundwater was encountered on all our boring logs, but in some soils, this can be very different from the true groundwater level. Monitoring wells installed in borings give the most reliable information, but this may apply only to the soil layer(s) in which the well is screened. If the top of the well screen or sand/gravel pack is more than a few feet below the groundwater level, then that groundwater level may not correspond to the true groundwater elevation. Soils that are described on our logs as "wet" are usually below the groundwater level, but perched groundwater can also make the interpretation of groundwater conditions difficult.

Groundwater levels typically vary seasonally by a few feet to as much as 100 feet or more depending on location, site conditions, recharge, and many other factors. If in any doubt, you should have a hydrogeologist from GeoEngineers confer with appropriate members of the design team to help them interpret groundwater level information and apply it to the project. The consequences of misunderstanding groundwater levels can be serious, which impacts can range from drainage problems and inadequate provision for construction dewatering, to water intrusion, hydrostatic instability of the subgrade and uplift of completed structures.

GeoEngineers