Laurel Village
SEPA Consistency Checklist

Purpose

The purpose of this consistency memorandum and checklist is to document the relationship of the proposed Laurel Village project with the SEPA EIS prepared for the University of Washington 2019 Seattle Campus Master Plan (Final EIS issued on July 5, 2017), and to inform the University of Washington's decision on SEPA compliance as SEPA Lead Agency.

Background

Published on July 5, 2017, the *2018 Seattle Campus Master Plan Final EIS* evaluated environmental conditions associated with development on a total of 86 potential development sites with a development capacity of approximately 12 million gross square feet (gsf) of net new building space. However, during the 10-year planning horizon of the Seattle Campus Master Plan, the University would develop a total of 6 million gsf of building space to meet the anticipated growth in demand for building space. Therefore, only a portion of the 86 potential development sites would be developed over the planning horizon.

The Final EIS analyzed environmental conditions under 17 elements of the environment, including: Earth; Air Quality; Wetlands/Plants & Animals; Energy Resources; Environmental Health; Land Use/Relationship to Plans and Policies; Population; Housing; Light, Glare and Shadows; Aesthetics; Recreation and Open Space; Cultural Resources; Historic Resources; public Services; Utilities; Transportation; and Construction.

For each element of the environment analyzed in the EIS a "sensitivity map" is provided that identifies portions of the campus that have a "High", "Medium", or "Low" potential to encounter sensitive environmental conditions. Specific mitigation or additional studies associated with High, Medium, and Low sensitivity areas on campus are defined for each element of the environment. The following elements of the environment were studied per scoping and comments received on the Draft EIS:

- Earth
- Air Quality
- Wetlands/Plants and Animals
- Energy Resources
- Environmental Health
- Land Use/Relationship to Plans and Policies
- Population
- Housing
- Light, Glare and Shadows
- Aesthetics
- Recreation and Open Space
- Cultural Resources
- Historic Resources

- Public Services
- Utilities
- Transportation
- Construction

Project Description

The Laurel Village project is being proposed in development sites E83 and E84 (see **Exhibits A** and **B**) of the campus to provide additional student apartment housing, including larger units for students with families. The project would be approximately 369,000 square feet, taking the place of the existing Laurel Village apartments. The development would include student resident apartments, student social space, supporting offices, approximately 12,000 square feet for a childcare facility, and storage. It is anticipated that approximately 33 units will be set at 50% of Average Median Income (AMI) rates. Parking would be provided onsite below one of the buildings and in surface lots. **Exhibit C** illustrates a potential option for configuration on the site.

Project Consistency with the Campus Development Agreement

The project is consistent with the allowed uses and development regulations as set forth in the 2019 Seattle Campus Master Plan. The project would not exceed the 65' and 30' maximum height limits and will meet design guidance including streetscape improvements pedestrian improvements for the crossing of Mary Gates Memorial Drive NE from NE Clark Road and through the interior of the site.

Project Consistency with the EIS

The following provides a summary of the relationship of the proposed project to the analysis for each element of the environment presented in the Final EIS (i.e., including if there are any potential environmental impacts associated with the proposed project that were not considered in the EIS). The following provides review of the proposed project by element of the environment:

<u>Earth</u> – According to City of Seattle online GIS mapping (SDCI GIS 2021), the project site is mapped within four Environmentally Critical Areas (ECAs): liquefaction prone soils, landfill 1000' methane buffer area, potential peat settlement, and a small steep stope. The project will address each of these ECAs through the geotechnical analysis and building practices to mitigation these potential conditions. See **Exhibit D** for the supporting geotechnical report.

<u>Air Quality</u> – Building demolition and construction would be conducted in compliance with Seattle Municipal Code Section 15.22.060B. During construction, dust and equipment emissions have the potential to impact adjacent housing uses. The site was identified as "Low" potential to encounter sensitive conditions.

<u>Wetlands/Plants and Animals</u> – Siting of the proposed buildings was chosen to work with the topography of the site and to avoid existing mature vegetation along the north and east edges. The existing vegetation is located in close proximity to the existing buildings and within the proposed footprint of the new building will be removed. **Exhibit E** depicts the proposed tree removal of approximately 12 Tier 2 trees and 71 Tier 3 and 4 trees identified for potential removal. Trees removed will be replaced at a 2:1 ratio; although a greater number of replacement trees (~200) is being considered.

Energy Resources – Decreases in electricity and fossil fuel demand per square foot are anticipated as the new buildings will be more efficient than the existing buildings. Overall use in energy resources may rise due to the increase in the number of units. The site was identified as "Low" potential to encounter sensitive conditions.

Environmental Health – Potential noise impacts would be primarily associated with construction of the buildings. Short-term vibration is anticipated when construction activities occur. Removal of existing hazardous materials will be abated and disposed of in approved waste sites designated for such materials. A portion of the site contains contaminated soil and groundwater. The majority of this area will not be disturbed. Where development is anticipated to come into contact with this area, cleanup and disposal will occur under a Soil Media Management Plan in compliance with MTCA and the Hazardous Waste Management Act. See **Exhibit F** for the supporting report.

Land Use/Relationship to Plans and Policies – The project is consistent with the 2019 Seattle Campus Master Plan.

Population – Occupancy of the proposed buildings would represent a portion of the projected increase in UW campus student, faculty and staff population, consistent with the Final EIS. The existing structure on site currently houses students, whereas the proposed buildings would house a larger number of students.

Housing – Construction and operation of the buildings would increase housing on campus.

Light, Glare and Shadows – The buildings would comply with the University's design review process and design standards, including a review of potential factors that could influence glare. New light sources associated with the proposed facility would be like those described for East Campus in the Final EIS.

Aesthetics – The buildings would be sited and designed in respect to the neighborhood and Burke-Gilman Trail. The site is lower than residences to the north and east and across the trail making the height of the structures diminished. Along the southern boundary is the backside of the U-Village garage and retail structures. The proposed development is similar in scale to the Nordheim Court development to the west. There are no protected view corridors on this site per the 2019 Seattle Campus Master Plan.

Recreation and Open Space – No recreation impacts are anticipated due to the recreation and open spaces available throughout campus.

Cultural Resources – No cultural resource impacts are anticipated.

Historic Resources – The site was identified as "Low" potential to encounter sensitive conditions. The existing buildings were deemed ineligible for historic listing.

Public Services – An increase in demand for public services would represent a portion of the projected increase consistent with the Final EIS.

Utilities – There is the potential for an increase in demand for water, sewer, stormwater, and solid waste with the increase in number of student residents. However, the buildings are anticipated to be more efficient compared to the existing buildings.

Transportation – The project will increase the number of parking stalls located onsite to approximately 185 total stalls to accommodate the proposed development. The proposed project anticipates 9 parking stalls for ADA stalls.

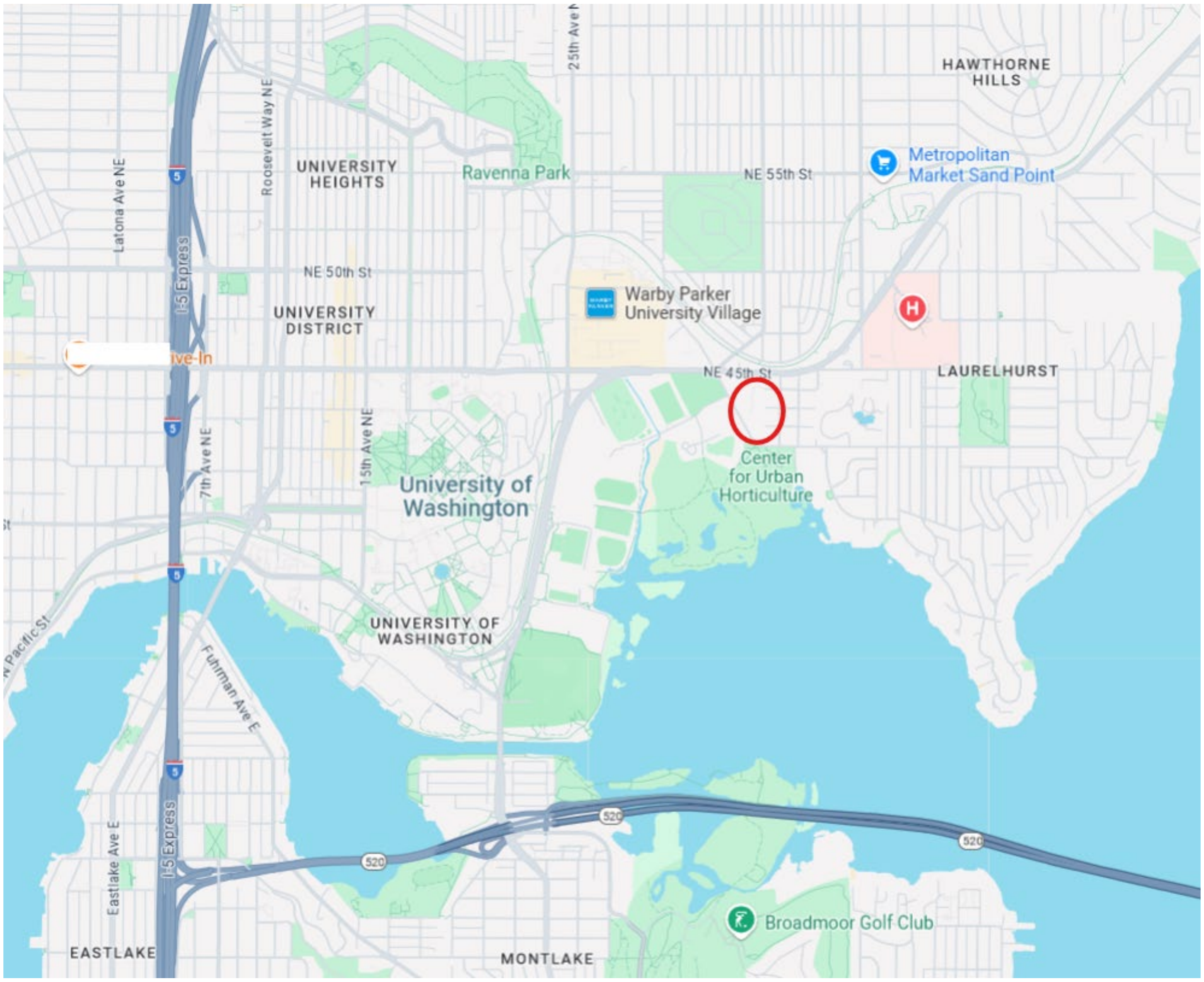
Construction – Construction activities including short-term localized traffic congestion, noise, dust, erosion, and increased street maintenance requirements associated with the removal of dirt tracked onto campus streets are anticipated. The construction of the buildings may temporarily and intermittently disturb occupants of buildings in the vicinity of the development site.

Determination

The UW Seattle adopts the *2018 Seattle Campus Master Plan Final EIS* for the Campus Master Plan for the University of Washington Seattle for the Blakeley Village project for purposes of SEPA. The relevant content has been briefly described above. The EIS may be reviewed at the following website address: https://facilities.uw.edu/files/media/uw-cmp-final-eis-volume-1.pdf

As indicated by the analysis above, the proposed project is within the range of impacts analyzed in the Final EIS. No new mitigation measures are required beyond those identified in the EIS and there are no significant impacts anticipated.

Exhibit A – Site Vicinity



Note: red circle identifies the proposed project site

Exhibit B – 2019 Seattle Campus Master Plan Development Sites E83 and E84

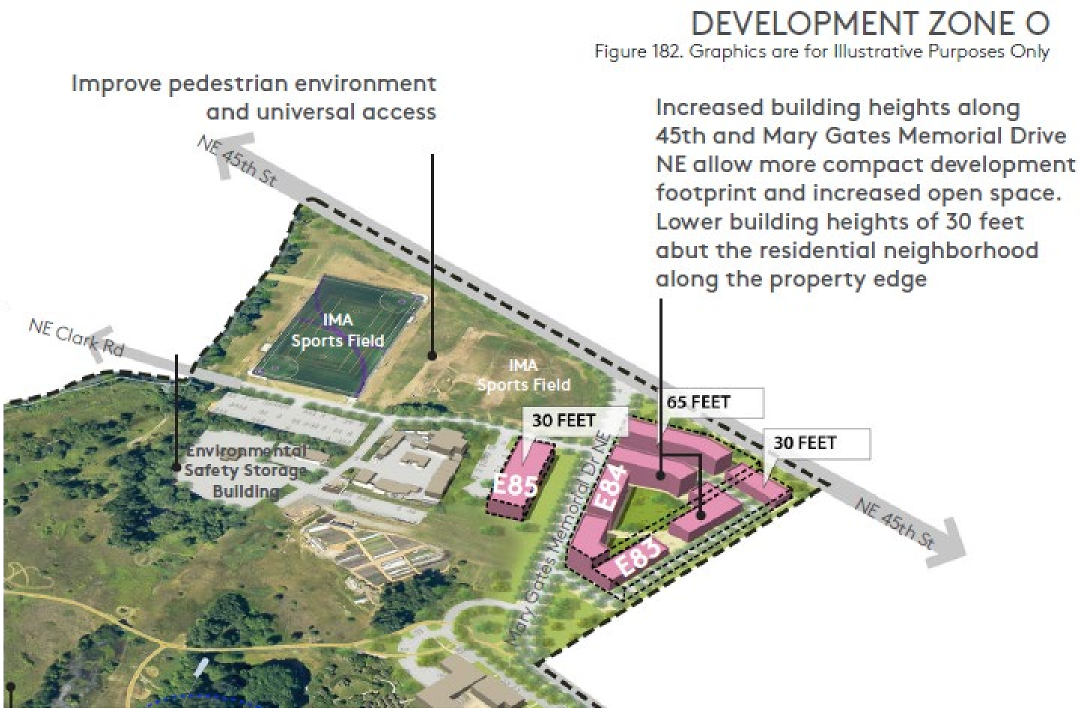


Exhibit C – Proposed Site Configuration

Exhibit D – Geotechnical Report

Appendices available upon request.

Geotechnical Engineering Services
Final Geotechnical Report

UH4 Laurel Village – Buildings A and B
Seattle, Washington

for
GDSU Washington, LLC

September 9, 2024

GeoEngineers

Geotechnical Engineering Services
Final Geotechnical Report

UH4 Laurel Village – Buildings A and B
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Geotechnical Engineering Services
Final Geotechnical Report

UH4 Laurel Village – Buildings A and B
Seattle, Washington

File No. 20449-013-00

September 9, 2024

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GEOENGINEERS

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1.0 INTRODUCTION

This report summarizes the results of GeoEngineers, Inc. (GeoEngineers) geotechnical engineering services for the proposed Buildings A and B of the UH4 Laurel Village development project located in Seattle, Washington. The site and planned buildings are shown relative to surrounding physical features in Figure 1, Vicinity Map, and Figure 2, Site Plan.

The purpose of this report is to provide geotechnical engineering conclusions and recommendations for the design and construction of the planned Buildings A and B. The site consists of one King County Parcel (parcel number 162504-9002) and covers approximately 7 acres. The planned buildings encompass approximately 54,000 square feet. GeoEngineers' services have been completed in accordance with our consultant agreement with GDSU Washington, LLC (Greystar) executed on November 14, 2023 and contract amendment executed on April 25, 2024. GeoEngineers' scope of services includes:

- Reviewing available reports and studies for the subject property and surrounding area available from our files;
- Completing explorations at the site to further characterize subsurface soil and groundwater conditions;
- Providing recommendations for seismic design in accordance with the 2018 International Building Code (IBC);
- Providing deep foundation final design, including axial capacity and lateral capacity analyses, temporary shoring, slab-on-grade and permanent below-grade wall recommendations;
- Evaluating suitability of on-site materials or requirement for off-site materials for compacted fills under building slabs, along with a recommended specification for compacted fill material;
- Providing recommendations for temporary dewatering and permanent below-grade drainage and groundwater seepage estimates;
- Providing consultation to the project team; and
- Preparing this report.

2.0 PROJECT DESCRIPTION

GeoEngineers understands that Greystar plans to redevelop the existing property with new student housing facilities as part of the University of Washington's UH4 project. The site is currently occupied by the existing Laurel Village student family housing, which consists of several two-story at-grade residential buildings constructed in the 1980s. The project will consist of demolishing the existing buildings and constructing 320 new student apartments. Based on review of the conceptual plans prepared by Weber Thompson, the planned development will include two new six-story wood-framed buildings (Buildings A and B) and new townhome/flats structures (Buildings C-1 to C-5) to be constructed at-grade. The proposed building layout is shown on Figure 2. Buildings A and B will have finished floor levels of Elevation 42 and 40 feet, respectively. This report is for the design and construction of Buildings A and B; recommendations for Buildings C-1 to C-5 is provided under separate cover.

Temporary shoring is anticipated to be required to complete the planned excavation. Based on review of exploration logs from our investigation and in the site vicinity, we anticipate that the planned buildings will be supported on shallow foundations where bearing soils are within 5 feet of the planned subgrade. Where the depth to bearing soils is greater than 5 feet, buildings will be supported on deep foundations.

3.0 FIELD EXPLORATIONS AND LABORATORY TESTING

3.1. Field Explorations

Subsurface conditions at the site were evaluated by drilling nine borings (GEI-1 through GEI-8, and GEI-2A). The boring GEI-2A was drilled to accommodate the installation of a monitoring well to a depth of 23 feet in the vicinity of GEI-2. The other borings extended to depths between 16-3/4 and 36-1/2 feet below site grades. The approximate locations of the explorations are shown in Figure 2. Descriptions of the field exploration program and the boring logs are presented in Appendix A, Field Explorations.

3.2. Laboratory Testing

Soil samples were obtained during drilling and were taken to GeoEngineers' laboratory for further evaluation. Selected samples were tested for moisture content, percent fines (material passing the U.S. No. 200 sieve), and grain size distribution (sieve analysis). A description of the laboratory testing and the test results are presented in Appendix B, Laboratory Testing.

3.3. Geophysical Testing

We completed non-invasive geophysical testing on site consisting of two active-source multichannel analysis of surface waves (MASW) surveys and one passive-source microtremor array method (MAM) surveys The geophysical testing report is provided in Appendix C.

3.4. Previous Site Evaluations

The logs of selected explorations from previous site evaluations in the project vicinity were reviewed and are presented in Appendix D, Boring Logs from Previous Studies. The approximate locations of these explorations are also shown on Figure 2.

4.0 SITE CONDITIONS

4.1. Surface Conditions

The UH4 Laurel Village site is bounded by NE 45th Street to the north, existing single-family residences to the east, and Mary Gates Memorial Drive NE to the southwest. The site is currently occupied by a multifamily student housing complex with several wood-framed buildings that were constructed in 1981. Existing site grades slope moderately down from northeast to southwest, from approximately Elevation 72 feet at the northeast corner down to Elevation 36 feet at the southwestern edge.

The subject property is designated as an Environmentally Critical Area (ECA) for steep slopes, a liquefaction-prone area, and a peat settlement-prone area (Category 2) in accordance with the Seattle Municipal Code (SMC) Chapter 25.09. The approximate extents of the ECA zones are shown on Figure 2. The subject property lies along the eastern shoreline of the former Union Bay, which was a peat marshland. The approximate extent of the former shoreline is also shown on Figure 2. Beginning in 1926, the City of Seattle used Union Bay as a public dump which then became the Montlake Landfill.

Buried utilities consisting of sanitary sewer, storm drain, gas, water, electric and telecommunications fiber are anticipated in the right-of-way adjacent to the site.

4.2. Subsurface Soil Conditions

GeoEngineers' understanding of subsurface conditions is based on the results of our investigation as well as our review of existing geotechnical information in the vicinity of the project site. Interpreted subsurface soil and groundwater conditions are illustrated in cross sections presented in Figure 3, Cross Section A-A'; Figure 4, Cross Section B-B'; Figure 5, Bearing Contour Map; and Figure 6, Groundwater Contour Map.

The soils encountered at the site consist of shallow fill and recent deposits overlying glacially consolidated till-like deposits and cohesionless sand and gravel. The fill generally consists of medium dense sand with variable silt and gravel content. The thickness of the fill encountered in the vicinity of Buildings A and B ranges up to 13 feet.

The recent deposits generally consist of peat and medium dense sand with little to no silt. Recent deposits were encountered within borings GEI-1, GEI-4, GEI-5, GEI-6, GEI-7, GEI-8, SW-1, and SW-2. Peat was only encountered in GEI-1 and is approximately 1.5 feet thick. In general, the recent deposits encountered in borings completed on the project site range up to 15 feet thick.

The glacially consolidated soils were encountered below the fill and/or recent deposits and extended to the depths explored. The till-like deposits consist of very stiff to hard clay and silt and dense to very dense silty sand with gravel. The cohesionless sand and gravels consist of dense to very dense sand and gravel with varying amounts of silt. Glacially consolidated soils were encountered at shallower depths in the eastern portion of the site and at deeper depths in the western portion of the site. The estimated elevation of the top of the glacially consolidated/bearing soil layer is shown on Figure 5.

Although not encountered during our investigation, occasional cobbles and boulders are typical of glacially consolidated soils. Occasional cobbles and boulders may be present at the site and have been encountered in nearby construction projects.

4.3. Groundwater Conditions

Groundwater has been measured between 2.0 to 24.4 feet below grade in monitoring wells installed as part of our investigation. The elevation of the groundwater levels observed is presented in Table 1 and on Figure 6. The groundwater measured in the monitoring wells is interpreted to be the regional groundwater table.

TABLE 1. GROUNDWATER MEASUREMENTS

Well ID	Ground Surface Elevation[1] (feet, NAVD 88)	Top of Casing Elevation (feet, NAVD 88)	Date of Measurement	Depth to Groundwater (feet)	Groundwater Elevation (feet, NAVD 88)
GEI-1	40	39.55	December 26, 2023	2.0	37.55
			December 29, 2023	2.6	36.95
			April 4, 2024	2.9	36.65
			August 28, 2024	3.3	36.25

Well ID	Ground Surface Elevation[1] (feet, NAVD 88)	Top of Casing Elevation (feet, NAVD 88)	Date of Measurement	Depth to Groundwater (feet)	Groundwater Elevation (feet, NAVD 88)
GEI-2A	37	36.75	December 26, 2023	2.6	34.15
			December 29, 2023	2.6	34.15
			April 4, 2024	2.77	33.98
			August 28, 2024	3.4	33.35
GEI-3	68	67.75	December 26, 2023	23.85	43.90
			December 29, 2023	24.4	43.35
			April 4, 2024	23.94	43.81
			August 28, 2024	23.6	44.15

Notes:

[1] Measurements based on ALTA Survey data, December 21, 2023.

5.0 ENVIRONMENTALLY CRITICAL AREAS

GeoEngineers has reviewed the ECA maps available online through the City of Seattle Department of Construction and Inspections (SDCI) geographic information system (GIS) website. Based on our review of the SDCI GIS maps, the site is located within a mapped steep slopes area, liquefaction-prone area, and peat settlement prone area.

5.1. Steep Slope Assessment

Based on our review, the area mapped as a steep slope ECA meets the requirements for relief from prohibition on steep slope development per SDCI Tip 327A, which states the relief can be granted (subject to ECA review) when the "development is located on steep slope areas that have been created through previous legal grading activities, including rockeries or retaining walls resulting from rights-of-way improvements, if no adverse impact on the steep slope area will result."

The proposed development at the site will consist of demolishing the existing buildings, which are set back from the steep slope area, and constructing new student housing buildings. The existing steep slope areas were created during the existing site development (as part of legal grading). Given that the existing buildings are set back from the steep slope area, we judge there will be no adverse impacts to the planned development or existing adjacent improvements.

5.2. Liquefaction-Prone Area Assessment

We evaluated the potential for liquefaction at the site. Our analysis indicates that the medium dense fill soils and recent deposits below the groundwater table have a high potential for liquefaction during the design earthquake event. Liquefaction will be mitigated by supporting the portions of the buildings underlain by liquefiable soils on deep foundations which will transfer the building loads to the competent non-liquefying glacially consolidated soils below the liquefiable layer. The deep foundations will be designed for both downdrag due to liquefaction settlement and the seismic loading.

5.3. Landfill Historical Assessment

The project is mapped within a Landfill (Historical) 1,000-foot Methane Buffer Area related to the former Montlake Landfill. Project design and construction may be subject to certain development standards, including barriers or ventilation, to mitigate accumulation of hazardous levels of methane (SMC 25.09.220).

The University of Washington Environmental Health and Safety division is conducting on-going methane monitoring at locations around the perimeter of the former Montlake Landfill. The monitoring network includes two locations within parking areas of Laurel Village (MP-8 in the northwest portion and MP-9 in the southeast portion). Methane concentrations have exceeded the action limit of 100 parts per million (ppm) for University of Washington off-site buildings at MP-8 during the monitoring since the early 2000s but the most recent quarterly data available (from 2022) has not indicated a concentration greater than the action level since February 2022. Monitoring point MP-9 has not indicated a concentration greater than the action limit in the available sampling data back to 2011. The University of Washington is managing the methane at Laurel Village through ventilation and monitoring consistent with their sampling and action plans. Monitoring for methane during our recent drilling and sampling for the redevelopment project has not detected methane in the boreholes on the Laurel Village property.

Based on the project location and the available monitoring data, methane mitigation should be evaluated as part of project design and construction. This could include passive venting and/or use of a methane geomembrane beneath the slab.

5.4. Peat Settlement Prone Area Assessment

In order to avoid negative impacts from the planned development, the City of Seattle will require that the planned development be designed to prevent or accommodate settlement and that the project does not cause settlement off-site through modification of the groundwater table. Modification of the groundwater table through lowering or redirecting groundwater, even for a short period of time, may lead to off-site settlement. Ideally, no excavations should extend below the groundwater table in order to prevent modification of the groundwater table. If the project will require localized excavation below the groundwater table (such as for elevator pits, foundation elements, stairwells/ramps or limited sidewalk setbacks), the excavation will be required to be completed in a manner that does not adversely lower the groundwater table off-site.

6.0 CONCLUSIONS AND RECOMMENDATIONS

A summary of the geotechnical considerations is provided below. The 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 Site Class F per the 2018 IBC due to the liquefiable soils below the site. A site-specific seismic response analysis was completed to develop site-specific response spectra in accordance with the IBC and by reference the American Society of Civil Engineers (ASCE) 7-16. The results of the site-specific seismic response are included in Appendix E, Site-Specific Ground Response Analysis.

- Significant temporary dewatering is not anticipated given the lowest finished floor elevations of the planned Buildings A and B will be located above the groundwater table elevation. Localized dewatering for small excavations that extend below the groundwater table (for instance elevator pits, foundation elements, stairwells/ramps or limited sidewalk setbacks) may be required locally and is anticipated to be completed using sumps and pumps.
- Where space allows, excavations can be temporarily sloped to accommodate planned construction. Where space is limited, excavation support can be completed using soldier pile and tieback shoring. Because the ground anchors may extend into the public right-of-way, these elements would be required to be temporary. The permanent below-grade building walls will be required to resist the permanent lateral earth pressures. The City of Seattle requires that tieback anchors extending into the public right-of-way be de-stressed once the temporary shoring is no longer required, and the below-grade building walls should be designed and constructed to facilitate de-stressing of temporary tieback anchors.
- Due to the variable soils present at the foundation subgrade elevation, Buildings A and B will need to be supported on both shallow foundations bearing on grade and deep foundations. The recommended zones of shallow bearing are shown on Figure 5. Ground improvement consisting of removal and replacement of the non-bearing soils with structural fill may be feasible where the groundwater table is located below the non-bearing soils and depths of removal and replacement are less than about 5 feet.
- Shallow foundations may be used where undisturbed glacially consolidated soils are present at the foundation subgrade elevation or where the non-bearing soils can be removed and replaced with properly compacted structural fill, as presented on Figure 5. For shallow foundations bearing directly on undisturbed dense to very dense glacially consolidated soils or properly compacted structural fill extending down to undisturbed dense to very dense glacially consolidated soils, we recommend an allowable soil bearing pressure of 8 kips per square foot (ksf).
- Augercast piles are the preferred deep foundation system. For design, we recommend 18-inch-diameter augercast piles with a minimum embedment of 10 feet into the glacially consolidated soils. The contractor should use drilling equipment capable of measuring and displaying torque during augercast pile installation. The torque measurement can be used as an indication of the transition from fill or recent deposits to denser glacially consolidated soils, which will be important for evaluating pile embedment in glacially consolidated soils during construction.
- Conventional slabs-on-grade are considered appropriate where shallow foundations are used for Buildings A and B and should be underlain by a 6-inch-thick layer of clean crushed rock (for example, City of Seattle Mineral Aggregate Type 22).
- Where the building is supported on deep foundations, a structural slab is recommended to mitigate long-term settlement from the peat soils and/or liquefaction-induced settlement. The structural slab should be underlain by a 6-inch-thick layer of clean crushed rock (for example, City of Seattle Mineral Aggregate Type 22).

Our specific geotechnical recommendations are presented in the following sections of this report.

6.1. Earthquake Engineering

6.1.1. Liquefaction

Liquefaction refers to the condition by which vibration or shaking of the ground, usually from earthquake forces, results in the development of excess pore pressures in saturated soils with subsequent loss of strength. In general, soils that are susceptible to liquefaction include very loose to medium dense, clean to silty sands that are below the water table.

The results of our analyses indicate that the very loose to medium dense fill soils and recent deposits below the groundwater table have a high potential for liquefaction during the design earthquake event.

The evaluation of liquefaction potential is a complex procedure and is dependent on numerous site parameters, including soil grain size, soil density, site geometry, static stress, and the design ground acceleration. Typically, the liquefaction potential of a site is evaluated by comparing the cyclic stress ratio (CSR), which is the ratio of the cyclic shear stress induced by an earthquake to the initial effective overburden stress, to the cyclic resistance ratio (CRR), which is the soils resistance to liquefaction. We evaluated the liquefaction triggering potential (Youd et al. 2001; Idriss and Boulanger 2008) and liquefaction-induced settlement (Tokimatsu and Seed 1987; Ishihara and Yoshimine 1992; Idriss and Boulanger 2008) for soil conditions in each of the borings we completed at the site. We estimate ¼ to 2 inches of liquefaction-induced settlement across the site for free field conditions. Liquefaction will be mitigated by supporting buildings on deep foundations that transfer the building loads to the competent non-liquefying glacially consolidated soils below the liquefiable layer. The deep foundations will be designed for both downdrag due to liquefaction settlement and the seismic loading.

6.1.2. Other Seismic Hazards

Due to the location of the site and the site's topography, the risk of adverse impacts resulting from seismically induced slope instability, differential settlement, or surface displacement due to faulting is considered to be low.

6.1.3. Site-Specific Seismic Design Information

The project site is Site Class F due to the presence of liquefaction. A site-specific seismic response analysis was completed to develop site-specific response spectra in accordance with the IBC, and by reference ASCE 7-16, and the results are presented in Appendix E.

6.2. Temporary Dewatering

Significant temporary dewatering should be assumed to be not permissible at this site given the presence of the Peat Settlement Prone ECA. Localized dewatering for relatively small excavations that extend below the groundwater table (for instance elevator pits, foundation elements, stairwells/ramps or limited sidewalk setbacks) is permissible if completed in a manner that does not cause adverse impacts to existing improvements located offsite. In such instances, casual dewatering using sumps and pumps is anticipated. We recommend a survey monitoring program be established prior to construction dewatering.

6.3. Excavation Support

We understand that the planned buildings will be constructed at-grade. The northern portions of Buildings A and B will extend partially below grade due to sloping site conditions. Excavations will be completed using a combination of temporary cut slopes, and temporary shoring consisting of soldier pile and tiebacks. The soldier pile and tieback shoring will be used along the northern portion of Building A, where excavations will be on the order of 15 to 18 feet deep.

Ground anchors should be designed to maintain an acceptable clearance from buried utilities in the right-of-way. The ground anchors will be required to be temporary if the ground anchors will extend into the City of Seattle right-of-way. The following section highlights specific considerations for each shoring wall.

We provide recommendations for conventional soldier pile and tieback walls below. Recommendations for temporary cut slopes are provided in Section 6.7.3.

6.3.1. Excavation Considerations

Site soils may be excavated with conventional excavation equipment, such as trackhoes or dozers. It may be necessary to rip the glacially consolidated soils locally to facilitate excavation. The contractor should be prepared for occasional cobbles and boulders in the site soils. Likewise, surficial fill may contain foundation elements and/or utilities from previous site development, debris, rubble and/or cobbles and boulders. We recommend that project specifications identify procedures for measurement and payment of work associated with obstructions.

6.3.2. Soldier Pile and Tieback Walls

Soldier pile walls consist of steel beams that are concreted into drilled vertical holes located along the wall alignment, typically about 8 feet on center. After excavation to specified elevations, tiebacks are installed, if necessary. Once the tiebacks are installed, the pullout capacity of each tieback is tested, and the tieback is locked off to the soldier pile at or near the design tieback load. Tiebacks typically consist of steel strands that are installed into pre-drilled holes and then either tremied or pressure grouted. Timber lagging is typically installed behind the flanges of the steel beams to retain the soil located between the soldier piles. Geotechnical design recommendations for each of these components of the soldier pile and tieback wall system are presented in the following sections.

6.3.2.1. Soldier Piles

We recommend that soldier pile walls be designed using the earth pressure diagram presented in Figure 7, Earth Pressure Diagrams — Temporary Soldier Pile & Tieback Walls. The earth pressures presented in Figure 7 are for cantilever soldier pile walls or soldier pile walls with single or multiple levels of tiebacks, and the pressures represent the estimated loads that will be applied to the wall system for various wall heights.

Earth pressures presented in Figure 7 include the loading from traffic surcharge. Other surcharge loads, such as cranes, construction equipment or construction staging areas, should be applied to the shoring system as recommended in Figure 8, Recommended Surcharge Pressure. No seismic pressures have been included in Figure 7 because it is assumed that the shoring will be temporary.

We recommend that the embedded portion of the soldier piles be at least 2 feet in diameter and extend a minimum distance of 10 feet below the base of the excavation to resist “kick-out.” The axial capacity of the soldier piles must resist the downward component of the anchor loads and other vertical loads, as appropriate. We recommend using an allowable end bearing value of 30 ksf for piles supported on glacially consolidated 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. The allowable end bearing value assumes that the shaft bottom is cleaned out immediately prior to concrete placement. If necessary, an allowable pile skin friction of 1.0 ksf may be used on the embedded portion of the soldier piles to resist the vertical loads.

6.3.2.2. Lagging

The following table presents GeoEngineers' recommended lagging thicknesses (roughcut) as a function of soldier pile clear span and depth.

TABLE 2. LAGGING THICKNESS

Depth (feet)	Recommended Lagging Thickness (roughcut) 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
25 to 50	3 inches	3 inches	3 inches	4 inches	4 inches	5 inches

Lagging should be installed promptly after excavation, especially in areas where perched groundwater or 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 backfilled as soon as practicable. The voids should be backfilled immediately or within a single shift, depending on the selected method of backfill. Placement of this material will help reduce the risk of voids developing behind the wall and damage to existing improvements behind the wall.

Controlled density fill (CDF) is a suitable option for backfill behind the wall, as it will reduce the volume of voids. Full-depth CDF backfill is recommended for the walls located near adjacent buildings, for improved deflection control.

6.3.2.3. Tiebacks

Tieback anchors can be used for wall heights where cantilever soldier pile walls are not cost-effective. Tieback anchors should extend far enough behind the wall to develop anchorage beyond the "no-load" zone (defined in Figure 7) and within a stable soil mass. The anchors should be inclined downward at 15 to 25 degrees below the horizontal. Corrosion protection will not be required for the temporary tiebacks.

Centralizers should be used to keep the tieback in the center of the hole during grouting, and structural grout or concrete should be used to fill the bond zone of the tiebacks. A bond breaker, such as plastic sheathing, should be placed around the portion of the tieback located within the no-load zone if the shoring contractor plans to grout both the bond and unbonded zones of the tiebacks in a single stage. If the shoring contractor does not plan to use a bond breaker to isolate the no-load zone, GeoEngineers should be contacted to provide recommendations.

Loose soil and slough should be removed from the holes drilled for tieback anchors prior to installing the tieback. The contractor should take necessary precautions to minimize loss of ground and prevent disturbance to previously installed anchors and existing improvements in the site vicinity. Drilled tieback holes should be grouted/filled promptly to reduce potential ground loss.

Tieback anchors should develop anchorage in the glacially consolidated soils. We recommend that the spacing between tiebacks be at least three times the diameter of the anchor hole to minimize group interaction. We recommend a design load transfer value between the anchor and soil of 3 kips per foot for glacially consolidated soils and 1.5 kips per foot for fill deposits.

Tieback anchors should be verification- and proof-tested to confirm that the tiebacks have adequate pullout capacity. The pullout resistance of tiebacks should be designed using a factor of safety of 2. The pullout resistance should be verified by completing at least two successful verification tests in each soil type and a minimum of four total tests for the project. Each tieback should be proof tested to 133 percent of the design load. Verification and proof tests should be completed as described in Appendix F.

Tieback layout and inclination should be checked to confirm that the tiebacks do not interfere with adjacent buried utilities. The City of Seattle minimum clearances between ground anchors and existing utilities should be maintained.

6.3.2.4. Drainage

Drainage for soldier pile and lagging walls is achieved through seepage through the timber lagging. Seepage flows at the bottom of the excavation should be contained and controlled to prevent loss of soil from behind the lagging.

6.3.2.5. Construction Considerations

Shoring construction shall be completed by a qualified shoring contractor. A shoring contractor is qualified if they have successfully completed at least 10 projects of similar size and complexity in the Seattle/Bellevue area during the previous 5 years. Interested shoring contractors should prepare a submittal documenting their qualifications, unless this requirement is waived by GeoEngineers. The shoring contractor's superintendent shall have a minimum of 3 years' experience supervising soil nail/soldier pile and tieback shoring construction and the drill operators and on-site supervisors shall have a minimum of 3 years' experience installing soil nails/soldier piles and tiebacks. The personnel experience shall be included in the qualification's submittal.

Temporary casing or drilling fluid will be required to install the soldier piles and casing will be necessary for tiebacks where:

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

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

6.3.3. Shoring Wall Performance

Temporary shoring walls typically move up to 1 inch. Deflections and settlements are usually highest at the excavation face and decrease to negligible amounts beyond a distance behind the wall equal to the height of the excavation. Deflections of the shoring system can be affected by local variations in soil conditions (such as around side sewers) or may be the result of the workmanship of the construction for the shoring wall (completed by the shoring contractor). Given that some movement is expected, existing improvements located adjacent to the temporary shoring system will also experience movement. The deformations discussed above are not likely to cause structural damage to structurally sound existing improvements; however, cosmetic damage is possible (for instance, cracks in drywall finishes; widening of existing cracks; minor cracking of slabs-on-grade/hardscapes; cracking of sidewalks, curbs/gutter, and pavements/ pavement panels; etc.). For this reason, it is important to complete pre-construction survey and photo documentation of existing buildings and nearby improvements prior to shoring construction. Refer to Appendix F for more detailed recommendations for shoring monitoring and preconstruction surveying.

6.4. Foundation Support

Based on the data obtained from the borings completed at the site, review of previous explorations completed at the project site and the anticipated finished floor levels, the soils at the anticipated foundation elevation vary across the planned footprints of Buildings A and B.

Due to the variable soils present at the foundation subgrade elevation, shallow foundations are recommended where the depth to bearing soil is less than 5 feet. Where the depth to bearing soil is greater than 5 feet, deep foundations are recommended for the western portion of the project site. Ground improvement consisting of removal and replacement of the non-bearing soils with structural fill may be feasible where the groundwater table is located below the non-bearing soils.

GeoEngineers has prepared a map with the estimated elevation of the top of bearing soils (Figure 5) to assist the project team with determining where shallow foundations and deep foundations should be used. Our interpretation of where shallow foundations should be used is also presented on Figure 5.

6.4.1. Shallow Foundations

6.4.1.1. Allowable Bearing Pressure

Shallow foundations may be used where undisturbed glacially consolidated soils are present at the foundation subgrade elevation or where the non-bearing soils can be removed and replaced with properly compacted structural fill. For shallow foundations bearing directly on undisturbed dense to very dense glacially consolidated soils or properly compacted structural fill extending down to undisturbed dense to very dense glacially consolidated soils, we recommend an allowable soil bearing pressure of 8 ksf.

The allowable soil bearing pressure applies to the total of dead and long-term live loads and may be increased by up to one-third for wind or seismic loads. The allowable soil bearing pressures are net values.

6.4.1.2. Modulus of Subgrade Reaction

For foundations designed as a beam on an elastic foundation, a static modulus of subgrade reaction of 55 pounds per cubic inch (pci) may be used for foundations bearing on glacially consolidated soils or on structural fill extending down to glacially consolidated soils. GeoEngineers should review the structural engineer's estimated deformation and applied bearing pressures to confirm that this subgrade modulus is appropriate and is consistent with our foundation design.

6.4.1.3. Settlement

Provided that all loose soil is removed and that the subgrade is prepared as recommended under "Construction Considerations" below, we estimate that the total settlement of the foundations will be about 1 inch or less. The settlements will occur rapidly, essentially as loads are applied. Differential settlements across the mat foundations could be half of the total settlement. Note that smaller settlements will result from lower applied loads.

6.4.1.4. Lateral Resistance

Lateral foundation loads may be resisted by passive resistance on the sides of footings and by friction on the base of the shallow foundations. For shallow foundations supported on native soils, CDF, or structural fill, the allowable frictional resistance may be computed using a coefficient of friction of 0.5 applied to vertical dead-load forces.

The allowable passive resistance may be computed using an equivalent fluid density of 400 pounds per cubic foot (pcf) (triangular distribution) above the groundwater table and an equivalent fluid density of 250 pcf (triangular distribution) below the groundwater table. These values are appropriate for foundation elements that are poured directly against undisturbed glacially consolidated soils or surrounded by structural fill.

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

6.4.1.5. Construction Considerations

We recommend that the condition of all subgrade areas be observed by GeoEngineers to evaluate whether the work is completed in accordance with our recommendations and whether the subsurface conditions are as anticipated.

During wet weather conditions or when wet weather is forecasted, the foundation subgrades are recommended to be protected with a rat slab consisting of 2 to 4 inches of lean or structural concrete to prevent deterioration of the subgrade during mat foundation steel and concrete placement.

If soft areas are present at the footing subgrade elevation, the soft areas should be removed and replaced with properly compacted structural fill, lean concrete/CDF, or structural concrete at the direction of GeoEngineers.

We recommend that the contractor consider leaving the subgrade for the foundations as much as 6 to 12 inches high, depending on soil and weather conditions, until excavation to final subgrade is required for foundation reinforcement. Leaving subgrade high will help reduce damage to the subgrade resulting from construction traffic or other activities on site.

6.4.1.5.1. Footing Drains

Where the excavation for below-grade walls and foundations has been temporarily sloped, we recommend that perimeter footing drains be installed. Perimeter footing drains should be installed at the base of the exterior footings. The perimeter drains should consist of at least 4-inch-diameter perforated pipe surrounded by a minimum of 6 inches of Mineral Aggregate Type 22 (¾-inch crushed gravel) or Type 5 (1-inch washed gravel), City of Seattle Standard Specification 9-03.14, or an alternative approved by GeoEngineers. The Type 22 or Type 5 material should be wrapped with a geotextile filter fabric meeting the requirements of construction geotextile for underground drainage, WSDOT Standard Specification 9-33. The footing drains should be routed to a sump or gravity drain. The footing drains should be installed at least 18 inches below the top of the adjacent floor slab. We recommend the drainpipe consist of perforated

Schedule 40 polyvinyl chloride (PVC) pipe. We recommend against using flexible tubing for footing drainpipes.

Drainpipes should be laid with minimum slopes of one-quarter percent (if possible). The pipe installations should include a cleanout riser with cover located at the upper end of each pipe run. The cleanouts could be placed in flush mounted access boxes. Roof downspouts must not discharge into the perforated pipes intended for providing drainage for walls or foundations.

6.4.2. Deep Foundations

Augercast piles are constructed using a continuous-flight, hollow-stem auger attached to a set of leads supported by a crane or installed with a fixed-mast drill rig. The first step in the pile casting process consists of drilling the auger into the ground to the specified tip elevation of the pile. Grout is then pumped through the hollow stem during steady withdrawal of the auger, replacing the soils on the flights of the auger. The final step is to install a steel reinforcing cage and typically a center bar into the column of fresh grout. One benefit of using augercast piles is that the auger provides support for the soils during the pile installation process, thus eliminating the need for temporary casing or drilling fluid. Installation of augercast piles also produces minimal ground vibrations, which is beneficial given the proximity of many existing buildings and improvements in the vicinity.

We recommend 18--inch-diameter augercast piles with a minimum embedment of 10 feet into the glacially consolidated soils based on the bearing soil contours presented on Figure 5.

6.4.2.1. Axial Capacity

Axial pile capacity is developed from side frictional resistance and end bearing for loads in compression. Uplift pile capacity is development from side frictional resistance.

We developed axial capacities for 18-inch diameter augercast piles below in Table 3. Axial pile capacities were evaluated for three conditions:

1. Before earthquake (static conditions);
2. During earthquake; and
3. After earthquake.

The pile capacities were evaluated using allowable stress design (ASD) procedures and are for combined dead plus long-term live loads. Each of the three cases includes a factor of safety of 2, per the Seattle Building Code. The allowable post-earthquake capacities include the effects of downdrag from liquefaction-induced settlement in the liquefiable fill and recent deposits around the pile.

Augercast pile capacities for static and seismic conditions are summarized in the following table. The pile lengths can be determined by the embedment depths needed to develop the required axial capacity in compression and tension. Pile embedment starts at the bearing soil elevation contours shown on Figure 5.

TABLE 3. 18-INCH-DIAMETER AUGERCAST PILE ALLOWABLE AXIAL CAPACITIES

Embedment Depth in Bearing Soils (feet)	Static Conditions		During Earthquake		Post-Earthquake	
	Compression (kips)	Uplift (kips)	Compression (kips)	Uplift (kips)	Compression (kips)	Uplift (kips)
15	190	125	185	120	120	N/A
20	245	175	240	170	175	N/A
25	310	230	305	225	240	N/A

Notes:

[1]See Figure 5 for bearing soil elevation contours.

[2]Post-earthquake condition considers liquefaction and the effect of downdrag.

The capacities apply to single piles. If piles are spaced at least three pile diameters on center, as recommended, no reduction of axial capacity for group action is needed. The structural characteristics of pile materials and structural connections may impose limitations on pile capacities and should be evaluated by the structural engineer.

6.4.2.2. Lateral Capacity

Lateral loads can be resisted by soil pressure on the vertical piles and by the passive soil pressures on the pile cap. Because of the potential separation between the pile-supported foundation components and the underlying soil from settlement, base friction along the bottom of the pile cap should not be included in calculations for lateral capacity.

Based on discussions with the structural engineer, we used the software LPILE to complete analyses assuming both free and fixed pile-head conditions. LPILE results for deflection, bending moment and shear are presented in Appendix G. The allowable lateral pile capacity for a given pile is taken as half of the lateral load that mobilizes 1 inch of movement at the top of pile, per the Seattle Building Code requirements. Allowable lateral pile capacities are summarized in Table 4 for the four cases that represent the variable depths to bearing soils.

TABLE 4. 18-INCH DIAMETER AUGERCAST PILE ALLOWABLE LATERAL CAPACITIES

Pile Case	Pile Head Condition	Pile Head Elevation (feet, NAVD88)	Bearing Soil Elevation (feet, NAVD88)	Pile Tip Elevation (feet, NAVD88)	Lateral Load at 1-inch of Pile Displacement (kips)	Allowable Lateral Capacity (kips)
1	Free	37.2	20	0	9	4.5
	Fixed		20	0	24	12
2	Free	37.2	27	0	14.5	7
	Fixed		27	0	35	17.5
3	Free	39	27	0	23	11.5
	Fixed		27	0	48	24

We recommend that the passive soil pressure acting on the pile cap be estimated using equivalent fluid density of 400 pcf (triangular distribution) above the groundwater table and an equivalent fluid density of 250 pcf (triangular distribution) below the groundwater table. This passive resistance value includes a factor of safety of 1.5 and assumes a minimum lateral deflection of 1 inch to fully develop the passive resistance. Deflections that are less than 1 inch will not fully mobilize the passive resistance in the soil.

Piles spaced closer than five pile diameters apart will experience group effects that will result in a lower lateral load capacity for trailing rows of piles with respect to leading rows of piles for an equivalent deflection. We recommend that the lateral load capacity for trailing piles in a pile group spaced less than five pile diameters apart be reduced in accordance with the factors in Table 5.

TABLE 5. PILE P-MULTIPLIERS, P_M, FOR MULTIPLE ROW SHADING

Pile Spacing[1] (in terms of shaft diameter)	P-Multipliers, P_m[2, 3]		
	Row 1 (leading row)	Row 2 (1st trailing row)	Row 3 and higher (2nd trailing row)
3D	0.8	0.4	0.3
5D	1.0	0.85	0.7

Notes:

1. The P-multipliers in the table above are a function of the center to center spacing of piles in the group in the direction of loading expressed in multiples of the pile diameter, D.
2. The values of P_m were developed for vertical pile only per 2017 ASHTO LRFD Table 10.7.4-1.
3. The P-multipliers are dependent on the pile spacing and the row number in the direction of the loading to establish values of P_m for other pile spacing values, interpolation between values should be conducted.

6.5. Slab Design

The new building slabs are not anticipated to extend below the groundwater table and therefore will not need to consider hydrostatic/uplift pressures; however, slab design in areas supported on deep foundations should consider the estimated liquefaction-induced settlement 1½ inches. If the slab cannot accommodate this estimated settlement, the slab should be designed as a structural slab. Where bearing soils are less than 5 feet from finished floor level, the weak soils should be overexcavated and replaced with structural fill. The slab may be designed as bearing on grade.

6.5.1. Subgrade Preparation

Where the new structure will be supported on-grade, the exposed subgrade should be evaluated after site grading is complete. Probing should be used to evaluate the subgrade. The exposed soil should be firm and unyielding, and without significant groundwater. Disturbed areas should be recompacted if possible or removed and replaced with compacted structural fill.

In areas with structural slabs, the subgrade only needs to be prepared sufficiently to support the structural slab during curing.

6.5.2. Design Parameters

For slabs-on-grade designed as a beam on an elastic foundation, a modulus of subgrade reaction of 150 pci may be used for subgrade soils prepared as recommended.

We recommend that the slab-on-grade and structural slab floors be underlain by a 6-inch-thick capillary break consisting of material meeting the requirements of Mineral Aggregate Type 22 (¾-inch crushed gravel), City of Seattle Standard Specification 9-03.14.

Provided that loose soil is removed and the subgrade is prepared as recommended, we estimate that slabs-on-grade will not settle appreciably.

6.5.3. Below-Slab Drainage

We expect the static groundwater level to be located below the slab-on-grade level for the proposed building, and perched groundwater may be present above the slab subgrade elevation. Conventional below-slab drainage and below grade wall drainage is recommended and flow rates are anticipated to be less than 5 gallons per minute (gpm).

We recommend installing an underslab drainage system to remove water from below the slab-on-grade. The underslab drainage system should include an interior perimeter drain. The 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 underslab drainage system pipes should have adequate slope to allow positive drainage to the 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. The underslab drainage system trenches should be backfilled with Mineral Aggregate Type 22 or Type 5 (1-inch washed gravel), City of Seattle Standard Specification 9-03.14, or an alternative approved by GeoEngineers. The Type 22 or Type 5 material should be wrapped with a geotextile filter fabric meeting the requirements of construction geotextile for underground drainage, Washington State Department of Transportation (WSDOT) Standard Specification 9-33. The 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.

If no special waterproofing measures are taken, leaks and/or seepage may occur in localized areas of the below-grade portion of the building, even if the recommended wall drainage and below-slab drainage provisions are constructed. If leaks or seepage is undesirable, below-grade waterproofing should be specified. A moisture and methane vapor barrier should be used below slab-on-grade floors located in occupied portions of the building. Specification of the vapor barrier requires consideration of the performance expectations of the occupied space, the type of flooring planned and other factors, and is typically completed by other members of the project team.

6.6. Below-Grade Walls

6.6.1. Permanent Subsurface Walls

Permanent below-grade walls constructed adjacent to temporary shoring walls should be designed using the earth pressure diagram presented in Figure 9. Foundation surcharge loads and traffic surcharge loads should be incorporated into the design of the below-grade walls using the surcharge pressures presented in Figure 8. Other surcharge loads, such as from construction equipment or construction staging areas, should be considered on a case-by-case basis.

The soil pressures recommended above assume that wall drains will be installed to prevent the buildup of hydrostatic pressure behind the walls and tied to permanent drains to remove water to suitable discharge points.

6.6.2. Other Cast-in-Place Walls

Conventional cast-in-place walls may be necessary for retaining structures located on-site. 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 that the walls are backfilled and drainage is provided as outlined in the following paragraphs, we recommend that yielding walls supporting horizontal backfill be designed using an equivalent fluid density of 35 pcf (triangular distribution), while non-yielding walls supporting horizontal backfill be designed using an equivalent fluid density of 55 pcf (triangular distribution). For seismic loading conditions, a rectangular earth pressure equal to 7H pounds per square foot (psf, where H is the height of the wall in feet) should be added to the active/at-rest pressures. Other surcharge loading should be applied as appropriate.

Lateral resistance for conventional cast-in-place walls can be provided by frictional resistance along the base of the wall and passive resistance in front of the wall. For walls founded on native soils, the allowable frictional resistance may be computed using a coefficient of friction of 0.5 applied to vertical dead-load forces. The allowable passive resistance may be computed using an equivalent fluid density of 400 pcf (triangular distribution) above the design groundwater table and using an equivalent fluid density of 250 pcf (triangular distribution) below the design groundwater table. The above coefficient of friction and passive equivalent fluid density values incorporate a factor of safety of about 1.5.

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

6.6.3. Drainage

Drainage behind the permanent below-grade walls is typically provided using prefabricated drainage board attached to the temporary shoring walls. Weep pipes that extend through the permanent below-grade wall should be installed around the perimeter of the building at the footing elevation. The weep pipes should have a minimum diameter of 2 or 4 inches. The weep pipes through the permanent below-grade wall should be spaced no more than 10 feet on center (2-inch-diameter weep pipes) or 20 feet on center (4-inch-diameter weep pipes) and should be hydraulically connected to the sump or gravity storm system.

The earth pressures for permanent below-grade walls assume that adequate drainage is provided behind the wall. Prefabricated vertical geocomposite drainage material, such as Aquadrain 15X, should be installed vertically to the face of the timber lagging. The vertical drainage material should extend to the bottom of foundation elevation. The weep pipes that penetrate the basement wall should be connected to the vertical drainage material with a drain grate. For soldier pile shoring walls, the drainage material should be installed on the excavation side of the timber lagging, with the fabric adjacent to the timber lagging.

Full wall face coverage is recommended to minimize seepage and/or wet areas at the face of the permanent wall. Full wall face coverage should extend from the bottom of foundation elevation up to about 3 to 5 feet below site grades to reduce the potential for surface water to enter the wall drainage system. 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, the potential still exists for these conditions to occur. If this is a concern, waterproofing should be specified.

Positive drainage should be provided behind cast-in-place retaining walls by placing a minimum 2-foot-wide zone of Mineral Aggregate Type 17 (bank run gravel), City of Seattle Standard Specification 9-03.14, with the exception that the percent passing the U.S. No. 200 sieve is to be less than 3 percent. A perforated drainpipe should be placed near the base of the retaining wall to provide drainage. The drainpipe should be surrounded by a minimum of 6 inches of Mineral Aggregate Type 22 (¾-inch crushed gravel) or Type 5 (1-inch washed gravel), City of Seattle Standard Specification 9-03.14, or an alternative approved by GeoEngineers. The Type 22 or Type 5 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 drainpipe should be connected to a header pipe and routed to a sump or gravity drain. Appropriate cleanouts for drainpipe maintenance should be installed.

6.7. Rockeries

We understand that rockeries may be used for grade transitions at the site. The primary purpose of a rockery is to protect the slope face from erosion and raveling, while providing limited soil retention. Rockeries with a 15-degree minimum batter (from vertical) and horizontal backslope should be limited to 6 feet exposed height. The height is measured as the vertical distance from the ground surface in front of the toe of the rockery to the top of the rockery. Recommendations for rockeries at cut slopes are presented herein.

The base of rockeries should be embedded at least 12 inches below the adjacent ground surface. Rockeries should be supported on firm, undisturbed native soils or compacted structural fill. The rockery should be constructed using rock sizes specified by the Association of Rockery Contractors and (WSDOT) Standard Specifications, Section 9-13.7(1), and procedures specified the 2024 WSDOT Standard Specifications, Sections 8-24 and the 2023 City of Seattle Standard Plans for Municipal Construction Standard Plan 141 for the required rockery heights.

Rockeries should be installed by a qualified contractor experienced in rockery construction.

If rockeries are to be terraced, the subgrade for the upper rockery should extend at least 3 feet horizontally in front of the rockery before making the cut for the lower rockery. In addition, we recommend that the top of the rockery include a horizontal setback of 5 feet from the adjacent property line. Rockery construction is an art and depends largely on the skill of the builder.

Although rockeries offer some lateral restraint, it is largely indeterminate, and they are not normally intended to provide significant lateral support. Even when the foundation and retained material are satisfactory, and the rockery materials and construction are satisfactory, there is some risk of movement or failure.

6.7.1. Rockery Drainage

The rockery design assumes drained conditions and does not allow for hydrostatic pressure buildup behind the rockeries. We recommend that a perforated drainpipe with a minimum diameter of 4 inches be placed at the back of the rockeries, below the ground surface elevation in front of the rockery. We recommend using either heavy-wall solid pipe (SDR-35 PVC) or rigid corrugated polyethylene pipe (ADS N-12, or equivalent) for the collector pipe. We recommend against using flexible tubing for drainage.

A 12-inch-wide drainage backfill layer should be constructed as a drainage layer immediately behind the rockery facing with the drainpipe placed at the base of this layer. The drainage zone should consist of clear 1-½ to 3/8-inch crushed rock; smaller aggregate may have the potential to erode or pipe through the rockery face. The drainpipe should be routed to a suitable discharge point with suitable erosion protection.

6.8. Earthwork

6.8.1. Subgrade Preparation

Exposed subgrade in structure, hardscape, and pavement areas should be evaluated after site excavation is complete. Foundation subgrades should be prepared as recommended in "Shallow Foundations" above. Where hardscape and pavement subgrade soils consist of disturbed soils, it will likely be necessary to remove and replace the disturbed soil with approved structural fill unless the soil can be adequately moisture-conditioned and compacted.

6.8.2. Structural Fill

Fill placed to support structures or foundations, placed behind retaining structures, for foundation drainage, and/or placed below pavements and sidewalks shall consist of structural fill as specified below:

- If structural fill is necessary beneath shallow foundations, the fill should consist of Mineral Aggregate Type 2 (1¼-inch minus crushed rock), City of Seattle Standard Specification 9 03.14, controlled density fill, or structural concrete.
- If structural fill is necessary beneath building slabs, the fill should consist of Mineral Aggregate Type 2 or Type 17 (1¼-inch minus crushed rock or bank run gravel), City of Seattle Standard Specification 9-03.14.
- Structural fill placed behind retaining walls should meet the requirements of Mineral Aggregate Type 17 (bank run gravel), City of Seattle Standard Specification 9-03.14.
- Structural fill placed as capillary break material should meet the requirements of Type 22 (¾ inch crushed gravel), City of Seattle Standard Specification 9-03.14.
- Structural fill placed around perimeter footing drains, underslab drains and cast-in-place wall drains should meet the requirements of Mineral Aggregate Type 5 (1-inch washed gravel) or Type 22 (¾-inch crushed gravel), City of Seattle Standard Specification 9-03.14.

- Structural fill placed within utility trenches and below pavement and sidewalk areas should meet the requirements of Mineral Aggregate Type 17 (bank run gravel), City of Seattle Standard Specification 9-03.14.
- Structural fill placed as crushed surfacing base course below pavements and sidewalks should meet the requirements of Mineral Aggregate Type 2 (1¼-inch minus crushed rock), City of Seattle Standard Specification 9-03.14.

6.8.2.1. On-site Soils

On-site soils are moisture-sensitive and have natural moisture contents higher than the anticipated optimum moisture content for compaction. In addition, the fines content for the on-site soils generally ranges from 9 to 35 percent. As a result, on-site soils will likely require moisture conditioning to meet the required compaction criteria during dry weather conditions and will 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 do not meet these gradation requirements. Therefore, imported structural fill meeting the requirements described above should be used where structural fill is necessary.

It may be feasible to reuse on-site soils with the addition of cement treatment. If cement treatment is considered, GeoEngineers can work with the contractor to determine the soil/cement ratio and placement procedures.

6.8.2.2. Fill Placement and Compaction Criteria

Structural fill should be mechanically compacted to a firm, non-yielding condition and placed in loose lifts not exceeding 1 foot in thickness. Each lift should be conditioned to the proper moisture content and compacted to the specified density before placing subsequent lifts. Structural fill should be compacted to meet the following criteria:

- Structural fill placed in building areas (including around foundations and supporting slab-on-grade floors), pavement and sidewalk areas (including utility trench backfill) should be compacted to at least 95 percent of the maximum dry density (MDD) estimated in general accordance with ASTM International (ASTM) D 1557.
- Structural fill placed against retaining walls should be compacted to between 90 and 92 percent. Care should be taken when compacting fill against retaining walls to avoid overcompaction and, hence overstressing the walls.

We recommend that 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 to verify compliance with compaction specifications, and advise on any modifications to the procedures that may be appropriate for the prevailing conditions.

6.8.2.3. Weather Considerations

On-site soils contain a sufficient percentage of fines (silt and clay) 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 equipment operation becomes difficult. Additionally, disturbance of near-surface soils should be expected if earthwork is completed during periods of wet weather. During wet weather, we recommend the following:

- The ground surface in and around the work area should be sloped so that surface water is directed away from the work area. The ground surface should be graded such that areas of ponded water do not develop. The contractor should take measures to prevent surface water from collecting in excavations and trenches. Measures should be implemented to remove surface water from the work area.
- Slopes with exposed soils should be covered with plastic sheeting or similar means.
- Site soils should not be left uncompacted and exposed to moisture. Sealing the surficial soils by rolling with a smooth-drum roller prior to periods of precipitation will reduce the extent to which these soils become wet or unstable.
- Construction traffic should be restricted to specific areas of the site, preferably areas that are surfaced with materials not susceptible to wet weather disturbance.
- Construction activities should be scheduled so that the length of time that soils are left exposed to moisture is reduced to the extent practicable.

6.8.3. Temporary Slopes

Temporary slopes may be used around the site to facilitate early installation of shoring or in the transition between levels at the base of the excavation. We recommend that temporary slopes constructed in the fill and recent deposits be inclined at 1½H:1V (horizontal to vertical) and that temporary slopes in the glacially consolidated soils be inclined at 1H:1V. Flatter slopes may be necessary if seepage is present on the face of the cut slopes or if localized sloughing occurs. For open cuts at the site, we recommend that:

- No traffic, construction equipment, stockpiles or building supplies be allowed at the top of the 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 that 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.

6.9. Pavement Design

6.9.1. Pavement Subgrade Preparation

Prior to placing new fill or pavement base course materials, subgrade areas should be proof rolled to locate soft or pumping soils. Prior to proof rolling, unsuitable soils should be removed from below 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 determine the extent of soft soils. If soft or pumping soils are observed, they should be removed and replaced with structural fill.

6.9.2. New Hot-Mix Asphalt Pavement

In light-duty pavement areas (e.g., automobile parking), we recommend a pavement section consisting of at least 3 inches of hot-mix asphalt (HMA) over 4 inches of densely compacted aggregate base. In heavy-duty pavement areas (such as driveways, truck traffic lanes, materials delivery), we recommend a pavement section consisting of at least 4 inches HMA over 6 inches of densely compacted aggregate base.

Structural fill placed as crushed surfacing base course below pavements should meet the requirements of Mineral Aggregate Type 2 (1¼-inch minus crushed rock), City of Seattle Standard Specification 9-03.14 and should be compacted to at least 95 percent of the MDD obtained using ASTM D 1557. We recommend that proof rolling of the subgrade and compacted aggregate base be observed by a representative from our firm prior to paving. Soft or yielding zones observed during proof rolling may require over-excavation and replacement with compacted structural fill.

The pavement sections recommended above are based on our experience. Thicker asphalt sections may be needed based on the actual traffic data, truck loads and intended use. Paved and landscaped areas should be graded so that surface drainage is directed to appropriate catch basins.

6.9.3. Portland Cement Concrete Pavement

Portland cement concrete (PCC) sections may be considered for areas where concentrated heavy loads may occur, including trash enclosures. We recommend that these pavements consist of at least 6 inches of PCC over 6 inches of aggregate base. A thicker concrete section may be needed based on the actual load data for use of the area. If the concrete pavement will have doweled joints, we recommend that the concrete thickness be increased by an amount equal to the diameter of the dowels. The base course should be compacted to at least 95 percent of the MDD.

We recommend PCC pavements incorporate construction joints and/or crack control joints spaced at maximum distances of 12 feet apart, center-to-center, in both the longitudinal and transverse directions. Crack control joints may be created by placing an insert or groove into the fresh concrete surface during finishing, or by saw cutting the concrete after it has initially set-up. We recommend the depth of the crack control joints be approximately one fourth the thickness of the concrete; or about 1½ inches deep for the recommended concrete thickness of 6 inches. We also recommend the crack control joints be sealed with an appropriate sealant to help restrict water infiltration into the joints.

6.10. Infiltration Evaluation

The site is partially mapped as “Infiltration Evaluation Not Required for On-Site Stormwater Management.” Given the shallow groundwater condition, the site does not meet the minimum vertical separation requirement per the 2021 City of Seattle Stormwater Code. We therefore conclude that infiltration is infeasible for the project.

6.11. Recommended Additional Geotechnical Services

During construction, GeoEngineers should observe the installation of the shoring system; review/collect shoring monitoring data; evaluate the suitability of the foundation subgrades; observe installation of deep foundations, 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 that the subsurface conditions are consistent with those observed in the explorations and other reasons described in Appendix H, Report Limitations and Guidelines for Use.

7.0 LIMITATIONS

We have prepared this report for the exclusive use of GDSU Washington, LLC. and their authorized agents for the UH4 Laurel Village project in Seattle, 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 H for additional information pertaining to use of this report.

8.0 REFERENCES

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FIGURES

GeoEngineers

Source(s):
- ESRI

Coordinate System: NAD 1983 UTM Zone 10N

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Vicinity Map

UH4 Laurel Village - Building A and B
Seattle, Washington

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Figure 1

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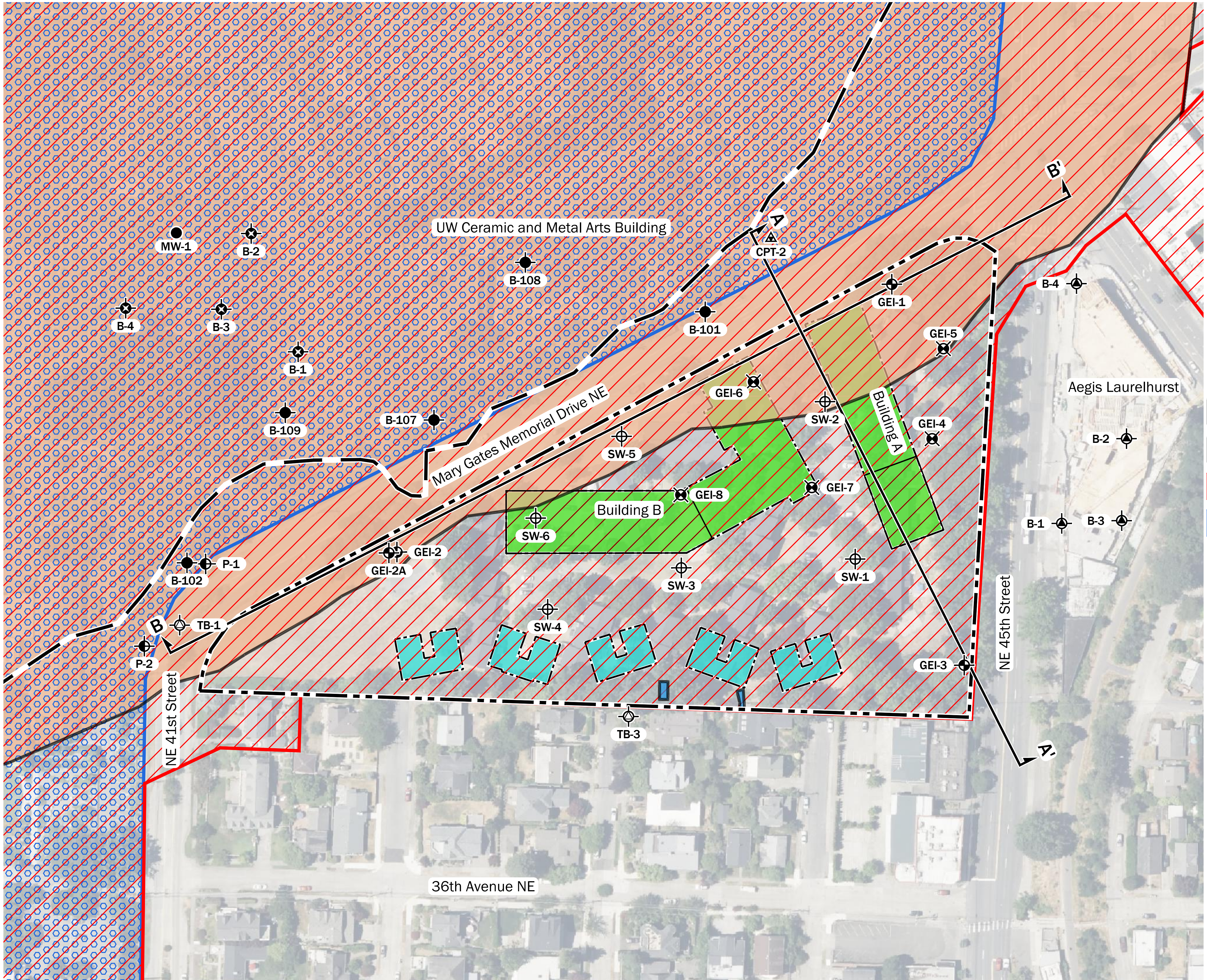

Legend

Site Boundary

Aproximate Extent of Former Union Bay Shorline 1912 (Baist's surveys of Seattle)

Proposed 6-Story Structure

Proposed Townhome and Flats Strctures

GEI-4 Boring by GeoEngineers, Inc., 2024

GEI-1 Monitoring Well by GeoEngineers, Inc., 2023

SW-1 Boring by Shannon & Wilson, 2022

B-1 Boring by PSI Intertek, 2016

B-1 Boring by GeoEngineers, Inc., 2016

MW-1 Monitoring Well by GeoEngineers, Inc., 2016

P-1 Boring by Shannon & Wilson, 2002

CPT-2 Cone Penetration Test by AGRA, 1996

TB-1 Boring by Pacific Testing Laboratories, 1996

B-102 Boring by Shannon & Wilson, 1966

Steep Slope ECA1 per SMC Chapter 25.09

Liquefaction Prone Area ECA5 per SMC Chapter 25.09

Peat Settlement Prone Areas ECA11 per SMC Chapter 25.09

Historic Landfill ECA7per SMC Chapter 25.09

A A' Cross Section Location

Source(s):

- Aerial from Microsoft Bing

Projection: WA State Plane, North Zone, NAD83, US Foot

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N

0 120
Feet

Site Plan

UH4 Laurel Village - Building A and B
Seattle, Washington

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Figure 2

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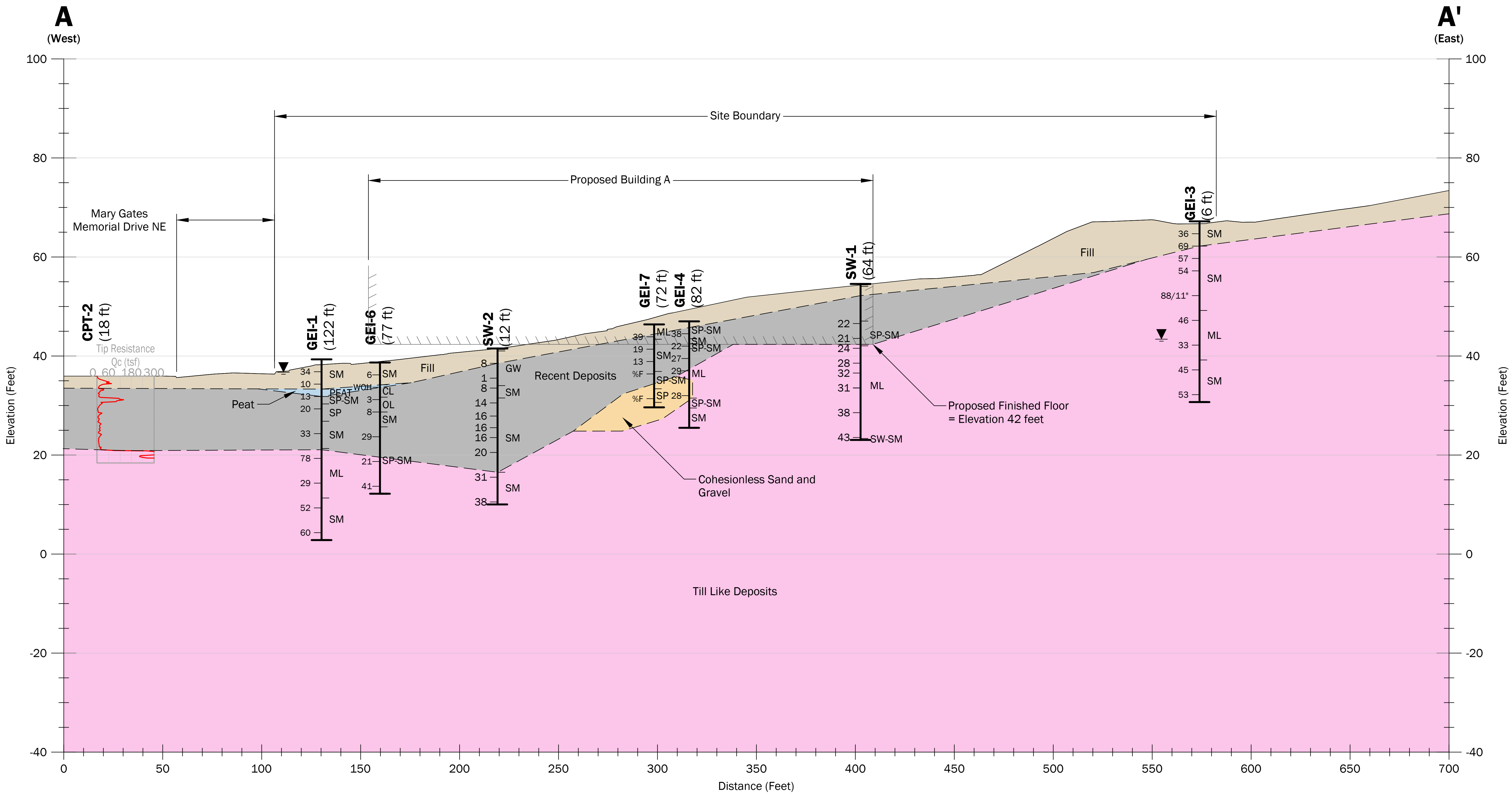

Note(s):

1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.

Datum: NAVD88

Disclaimer: This figure was created for a specific purpose and project. Any use of this figure for any other project or purpose shall be at the user's sole risk and without liability to GeoEngineers. The locations of features shown may be approximate. GeoEngineers makes no warranty or representation as to the accuracy, completeness, or suitability of the figure, or data contained therein. The file containing this figure is a copy of a master document, the original of which is retained by GeoEngineers and is the official document of record.

Legend

Boring ID (Offset)

Boring

— —?— —?— or – – – – – Inferred Soil Contact

SM Soil Classification

Groundwater Measured in Piezometer

20 Blow Count

Fill

Recent Deposits

Cohesionless Sand and Gravel

Till Like Deposits

Peat

0 50
Horizontal Feet

0 20
Vertical Feet
Vertical Exaggeration =2.5X

Cross Section A-A'

UH4 Laurel Village - Building A and B
Seattle, Washington

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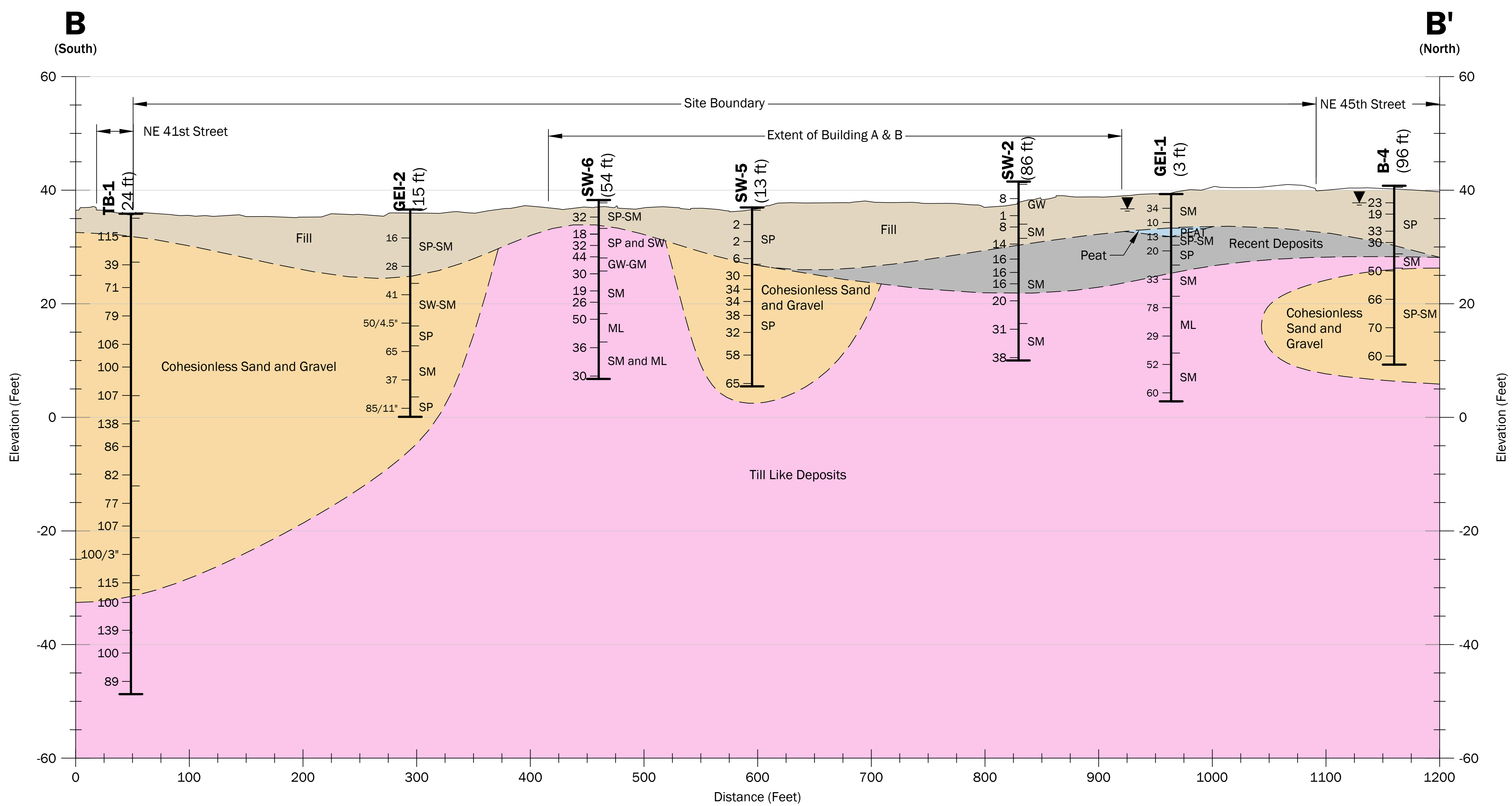
Figure 3

P:\20\20449013\CAD\00\Geotech\20449013000_F02-F06_Site Plans.dwg 3 Date Exported:9/4/2024 2:35 PM - by Jackson N. Fellows



Note(s):

1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.

Datum: NAVD88

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Legend

Boring ID (Offset)

Boring

— —?— —?— or — — — — Inferred Soil Contact

SM Soil Classification

Groundwater Measured in Piezometer

20 Blow Count

Fill

Recent Deposits

Cohesionless Sand and Gravel

Till Like Deposits

Peat

0 100 Horizontal Feet

0 20 Vertical Feet

Vertical Exaggeration = 5X

Cross Section B-B'

UH4 Laurel Village - Building A and B
Seattle, Washington

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Figure 4

P:\20\20449013\CAD\00\Geotech\2044901300_F02-F06_Site Plans.dwg 4 Date Exported:6/12/2024 4:20 PM - by Jackson N. Fellows

Legend

- Site Boundary
- Aproximate Extent of Former Union Bay Shoreline 1912 (Baist's surveys of Seattle)
- 2 — Bearing Contours (Feet, NAVD 88)
- 50 — Ground Surface Elevation (Feet, NAVD 88)
- Proposed 6-Story Structure
- Proposed Townhome and Flats Structures
- GEI-4 — Boring by GeoEngineers, Inc., 2024
- GEI-1 — Monitoring Well by GeoEngineers, Inc., 2023
- SW-1 — Boring by Shannon & Wilson, 2022
- B-1 — Boring by PSI Intertek, 2016
- B-1 — Boring by GeoEngineers, Inc., 2016
- MW-1 — Monitoring Well by GeoEngineers, Inc., 2016
- P-1 — Boring by Shannon & Wilson, 2002
- CPT-2 — Cone Penetration Test by AGRA, 1996
- TB-1 — Boring by Pacific Testing Laboratories, 1996
- B-102 — Boring by Shannon & Wilson, 1966
- A A' — Cross Section Location
- Structure to be Supported on Shallow Foundations Directly on Bearing Soil

Source(s):
- Aerial from Microsoft Bing

Projection: WA State Plane, North Zone, NAD83, US Foot

N

0 120
Feet

Bearing Contour Map

UH4 Laurel Village - Building A and B
Seattle, Washington

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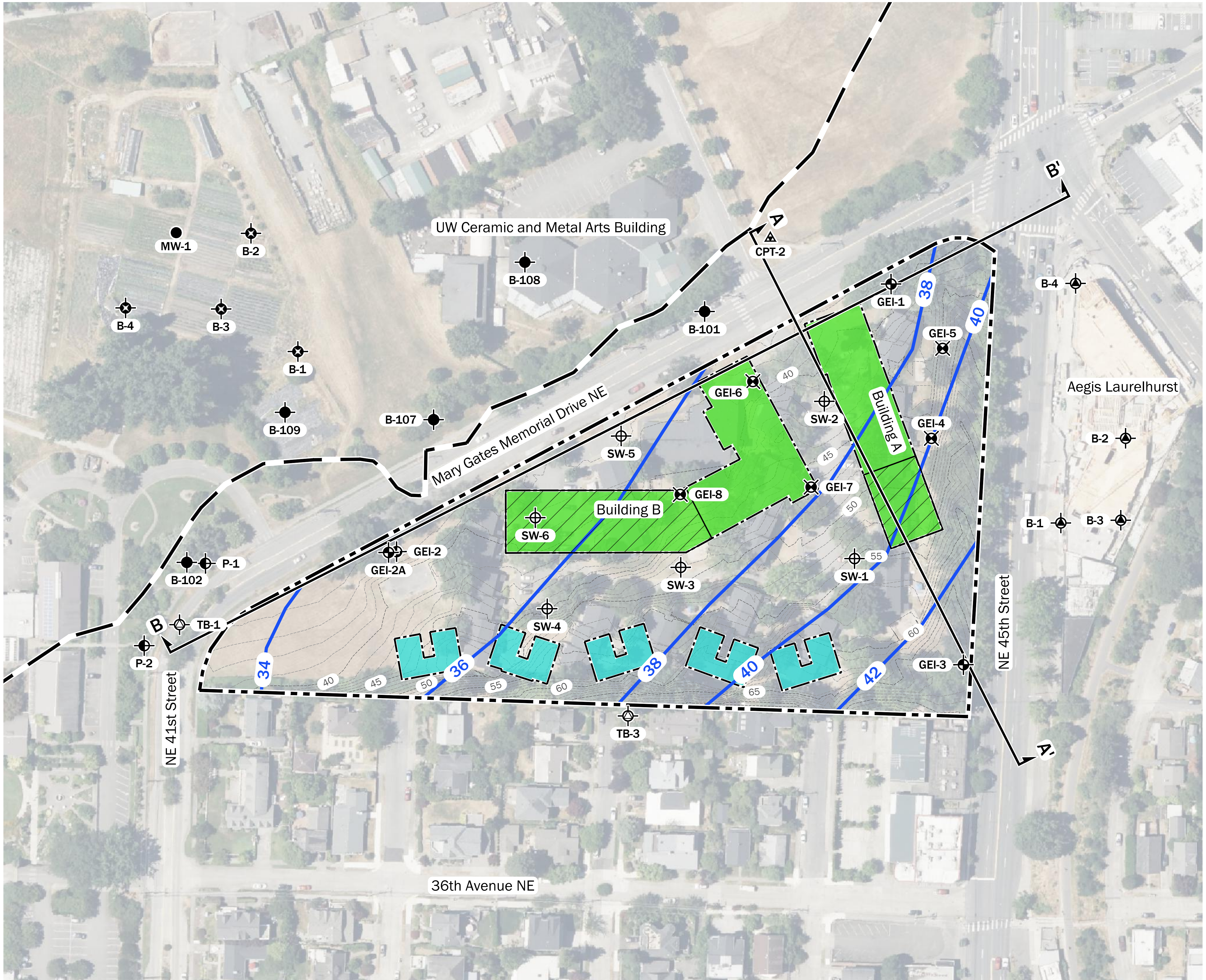
Figure 5

P:\20\20449013\CAD\00\Geotech\2044901300_F02-F06_Site Plans.dwg 5 Date Exported:6/12/2024 4:15 PM - by Jackson N. Fellows



Legend

- Site Boundary
- Aproximate Extent of Former Union Bay Shoreline 1912 (Baist's surveys of Seattle)
- 34 Groundwater Contour (Feet, NAVD 88)
- 50 Ground Surface Elevation (Feet, NAVD 88)
- Proposed 6-Story Structure
- Proposed Townhome and Flats Strctures
- GEI-4 Boring by GeoEngineers, Inc., 2024
- GEI-1 Monitoring Well by GeoEngineers, Inc., 2023
- SW-1 Boring by Shannon & Wilson, 2022
- B-1 Boring by PSI Intertek, 2016
- B-1 Boring by GeoEngineers, Inc., 2016
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- CPT-2 Cone Penetration Test by AGRA, 1996
- TB-1 Boring by Pacific Testing Laboratories, 1996
- B-102 Boring by Shannon & Wilson, 1966
- A A' Cross Section Location

Source(s):

- Aerial from Microsoft Bing

Projection: WA State Plane, North Zone, NAD83, US Foot

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N

0 120

Feet

Groundwater Contour Map

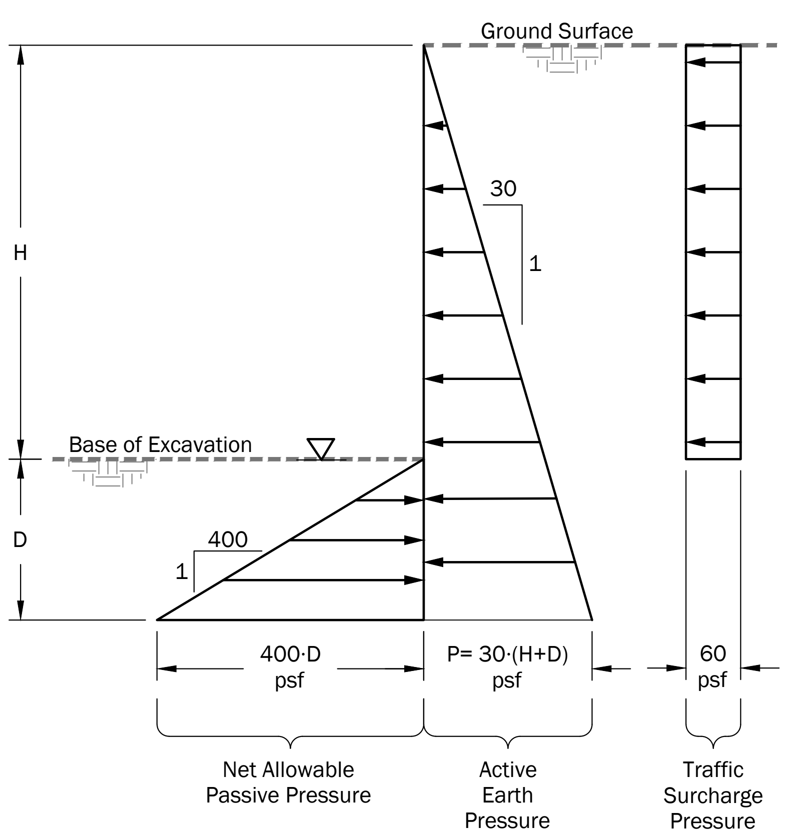
UH4 Laurel Village - Building A and B
Seattle, Washington

GeoEngineers

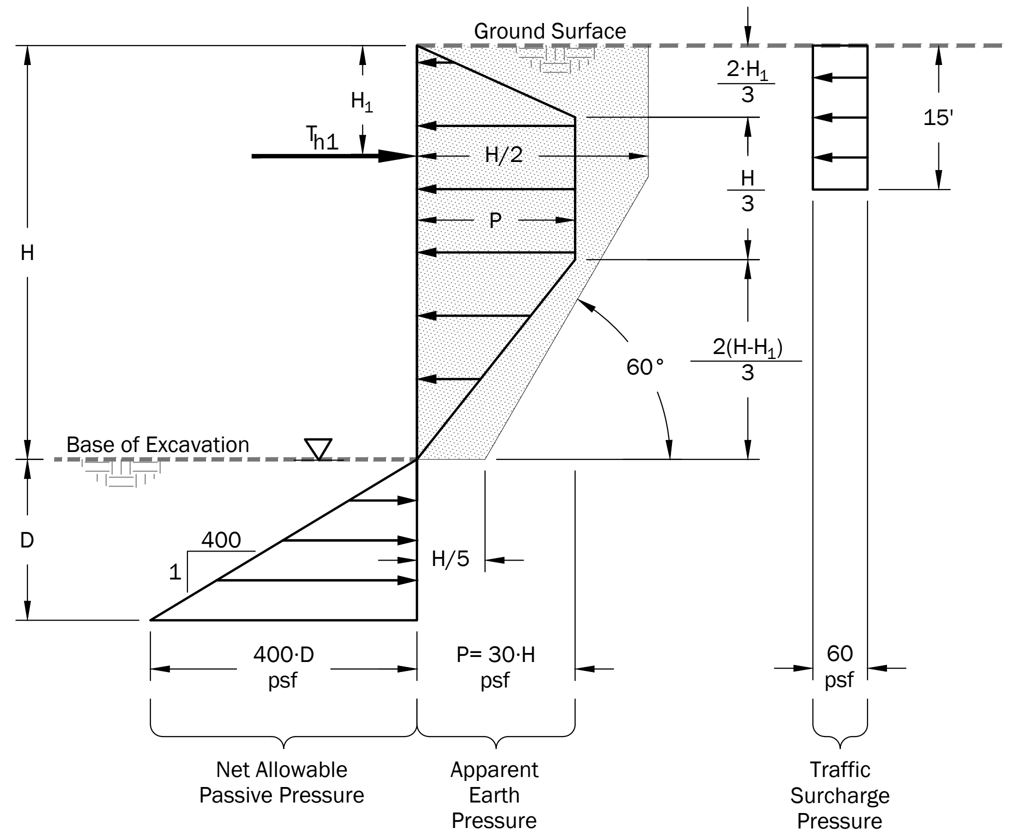
Figure 6

P:\20\20449013\CAD\00\Geotech\2044901300_F02-F06_Site Plans.dwg 6 Date Exported:6/12/2024 4:14 PM - by Jackson N. Fellows

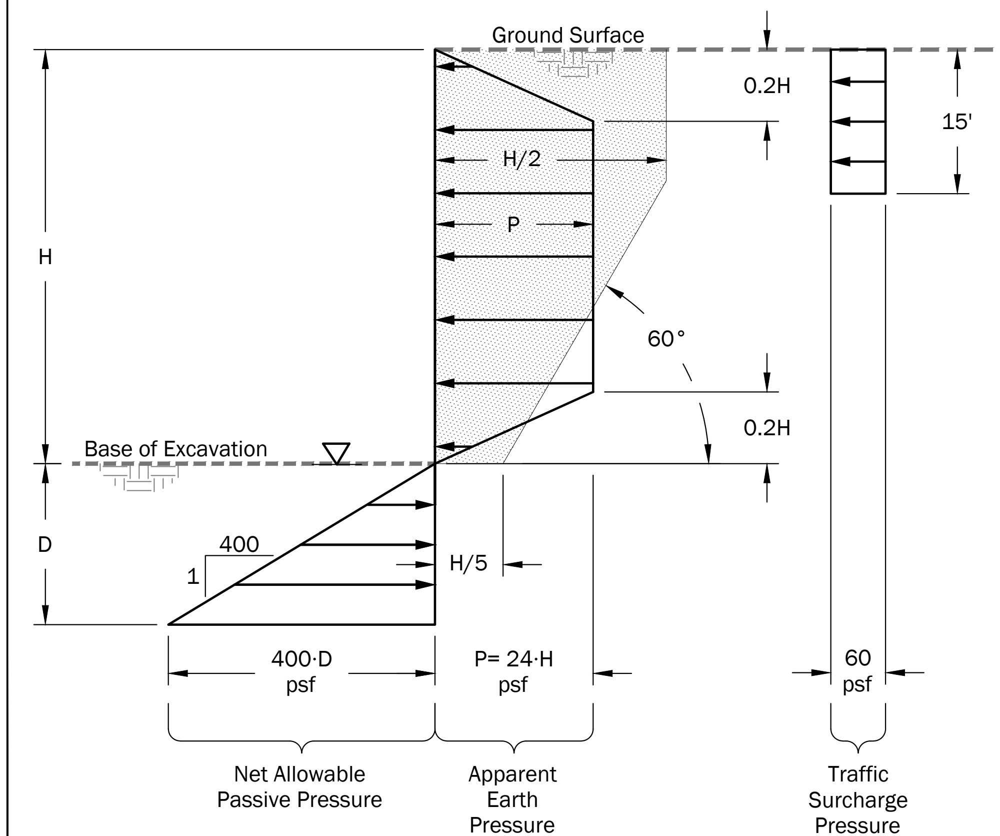
Cantilever Soldier Pile



Conventional Soldier Pile Wall with One Level of Tiebacks



Conventional Soldier Pile Wall with Multiple Levels of Tiebacks



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 3 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 6.
5. This pressure diagram is appropriate for temporary soldier pile and tieback 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.

Legend

No Load Zone

H = Height of Excavation, Feet

D = Soldier Pile Embedment Depth, Feet

H_1 = Distance From Ground Surface to Uppermost Tieback, Feet

T_{h1} = Horizontal Load in Uppermost Ground Anchor

P = Maximum Apparent Earth Pressure Pounds per Square Foot

▽ Assumed Groundwater Elevation during Temporary Shoring.

X = Earth Pressure Surcharge Factor

Not To Scale

Earth Pressure Diagrams
Temporary Soldier Pile & Tieback Walls

UH4 Laurel Village - Buildings A and B
Seattle, Washington

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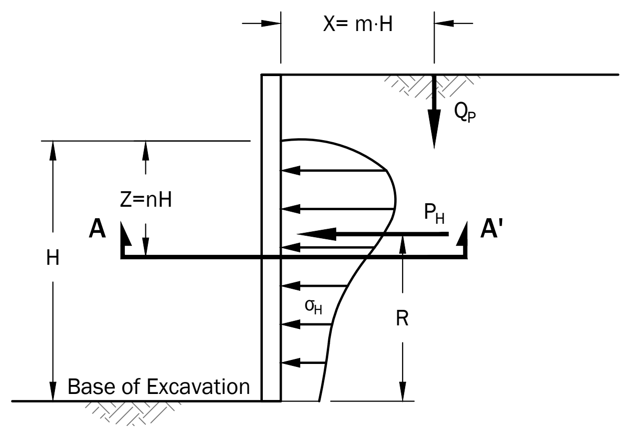
Figure 7

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



For $m \le 0.4$

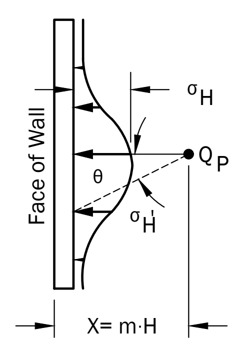
$$\sigma_H = \frac{K \cdot 0.28 Q_P n^2}{H^2(0.16+n^2)^3}$$

For $m > 0.4$

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

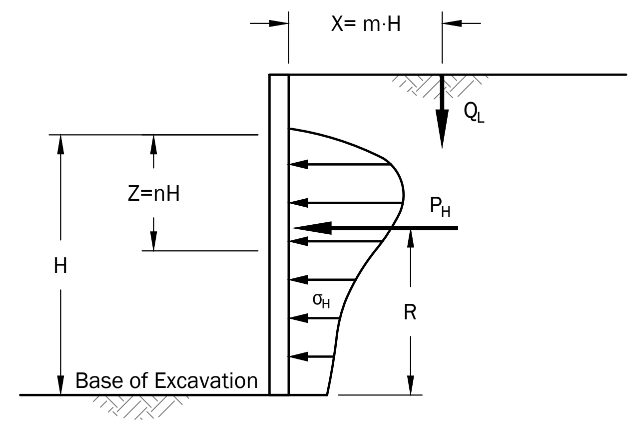
$$\sigma'_H = \sigma \, COS^2 (1.1\theta)$$

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 $m \le 0.4$

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

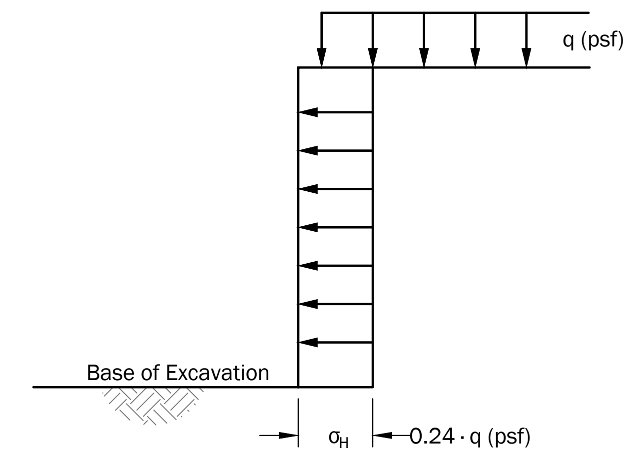
For $m > 0.4$

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

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

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

Wall Type	K
Rigid	1.0
Flexible	0.5

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 5 and 7.
3. See report text for where surcharge pressures are appropriate.
4. Determination of surcharge factor (k). Flexible is for a system that allows small movements (temporary shoring, retaining walls, etc.) and rigid is for a system that does not allow small movements (permanent basement walls, below grade utility structures, etc.). If permanent basement walls are cast/poured directly against temporary shoring, then the lateral surcharge factor should be assumed as flexible when analyzing lateral surcharges.

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
K = Surcharge Factor

Not To Scale

Recommended Surcharge Pressure

UH4 Laurel Village - Buildings A and B
Seattle, Washington

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Figure 8

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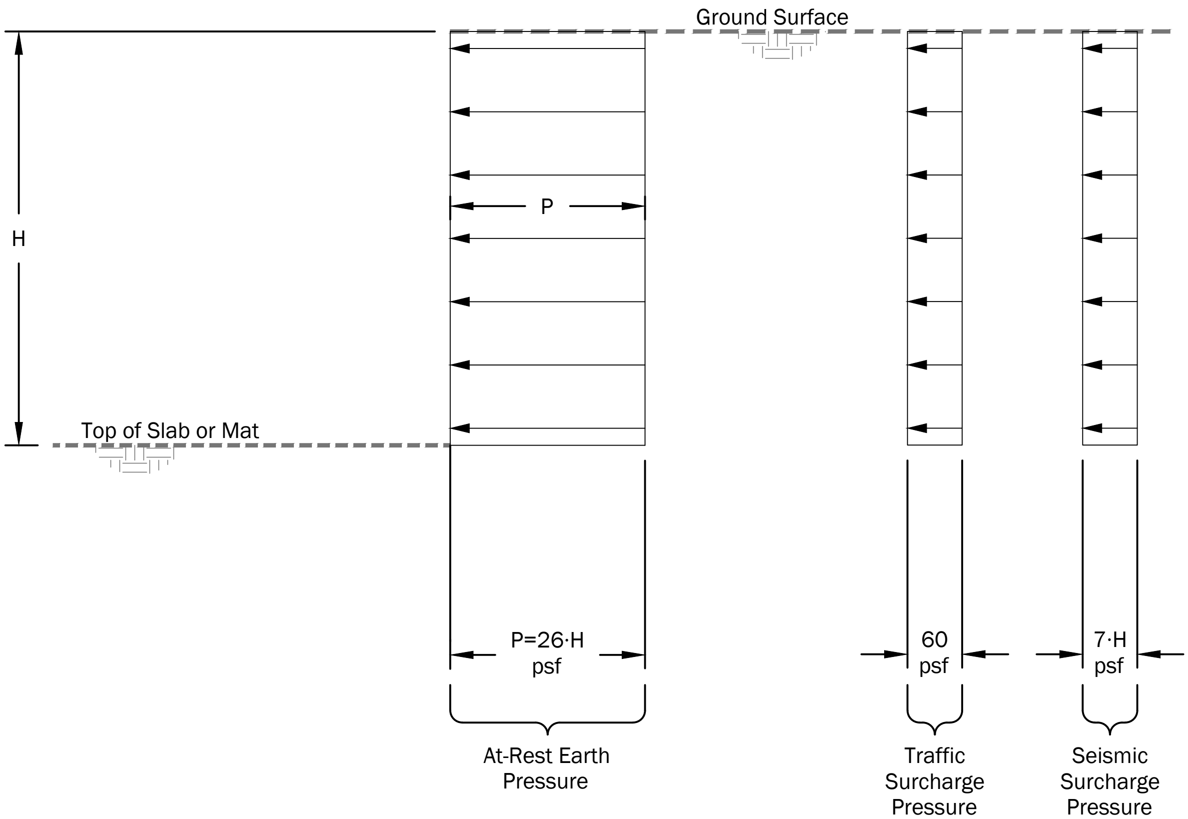
Permanent Basement Wall Against Temporary Shoring



Legend

H = Height of Basement Wall, Feet

P = Maximum At-Rest Earth Pressure, Pounds per Square Foot

Not To Scale

Notes:

1. Additional surcharge(s) from footings of adjacent buildings should be included in accordance with recommendations provided on Figure 6.
2. This pressure diagram is appropriate for permanent basement walls constructed in front of temporary shoring walls with tieback or soil nail anchors. 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.
3. The at-rest earth pressure does not include a factor of safety and represents the actual anticipated static earth pressure.

Earth Pressure Diagram **Permanent Below Grade Walls**	
UH4 Laurel Village - Building A and B Seattle, Washington	
GEOENGINEERS	**Figure 9**

P:\20\20449013\CAD\00\Geotech\2044901300_F09_EPD - Permanent Below Grade Walls.dwg TAB:9 Date Exported: 05/06/24 - 13:21 by JFellows

Geotechnical Engineering Services
Final Geotechnical Report – Revision 1

UH4 Laurel Village – Buildings C-1 to C-5
Seattle, Washington

for
GDSU Washington, LLC

September 9, 2024

GeoEngineers
Earth Science + Technology

Geotechnical Engineering Services
Final Geotechnical Report – Revision 1

UH4 Laurel Village – Buildings C-1 to C-5
Seattle, Washington

for
GDSU Washington, LLC

September 9, 2024

GeoEngineers

17425 NE Union Hill Road, Suite 250
Redmond, Washington 98052
425.861.6000

Geotechnical Engineering Services
Final Geotechnical Report – Revision 1

UH4 Laurel Village– Buildings C-1 to C-5
Seattle, Washington

File No. 20449-013-00

September 9, 2024

Prepared for:

GDSU Washington, LLC
450 Sansome Street, Suite 500
San Francisco, California 94111

Attention: Chad Winters

Prepared by:

GeoEngineers, Inc.
17425 NE Union Hill Road, Suite 250
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35217
REGISTERED
PROFESSIONAL ENGINEER
9/9/24

Erik C. Ellingsen, PE
Senior Geotechnical Engineer

Matthew W. Smith, PE
Senior Principal

ECE:MWS:NLD

GEOENGINEERS

Table of Contents

LIST OF FIGURES

APPENDICES

1.0 INTRODUCTION

This report summarizes the results of GeoEngineers' geotechnical engineering services for the proposed Buildings C-1 to C-5 of the UH4 Laurel Village development project located in Seattle, Washington. The site and planned buildings are shown relative to surrounding physical features in Figure 1, Vicinity Map, and Figure 2, Site Plan.

The purpose of this report is to provide geotechnical engineering conclusions and recommendations for the design and construction of the planned Buildings C-1 to C-5. The site consists of one King County Parcel (parcel number 162504-9002) and covers approximately 7 acres. The planned buildings encompass approximately 20,000 square feet. GeoEngineers' services have been completed in accordance with our consultant agreement with GDSU Washington, LLC executed on November 14, 2023 and contract amendments #1 through 4. GeoEngineers' scope of services includes:

- Reviewing available reports and studies for the subject property and surrounding area available from our files;
- Completing explorations at the site to further characterize subsurface soil and groundwater conditions;
- Providing recommendations for seismic design in accordance with the 2018 International Building Code (IBC);
- Providing foundation, slab-on-grade, and site retaining wall recommendations;
- Evaluating suitability of on-site materials or requirement for off-site materials for compacted fills under building slabs, along with a recommended specification for compacted fill material;
- Providing recommendations for temporary dewatering and groundwater seepage estimates;
- Providing consultation to the project team; and
- Preparing this report.

2.0 PROJECT DESCRIPTION

GeoEngineers understands that GDSU Washington, LLC (Greystar) plans to redevelop the existing property with new student housing facilities as part of the University of Washington's UH4 project. The site is currently occupied by the existing Laurel Village student family housing, which consists of several two-story at-grade residential buildings constructed in the 1980s. The project will consist of demolishing the existing buildings and constructing 320 new student apartments. Based on review of the conceptual plans prepared by Weber Thompson, the planned development will include two new six-story wood-framed buildings (Buildings A and B) and new townhome/flats structures (Buildings C-1 to C-5) to be constructed at-grade. The proposed building layouts are shown on Figure 2. The finished floor for each building is listed below:

- Building C-1: Elevation 58 feet
- Building C-2: Elevation 57 feet
- Building C-3: Elevation 52 feet
- Building C-4: Elevation 48 feet

- Building C-5: Elevation 42 feet

Overall site grading will require retaining walls along the eastern property line. We understand these will likely consist of rockery type walls.

This report is for the design and construction of Buildings C-1 to C-5; recommendations for Buildings A and B will be provided under separate cover.

Based on review of exploration logs from our investigation and in the site vicinity, we anticipate that the planned buildings will be supported on shallow foundations.

3.0 FIELD EXPLORATIONS AND LABORATORY TESTING

3.1. Field Explorations

Subsurface conditions at the site were evaluated by drilling nine borings (GEI-1 through GEI-8, and GEI-2A). The boring GEI-2A was drilled to accommodate the installation of a monitoring well to a depth of 23 feet in the vicinity of GEI-2. The other borings extended to depths between 16-3/4 and 36-1/2 feet below site grades. The approximate locations of the explorations are shown in Figure 2. Descriptions of the field exploration program and the boring logs are presented in Appendix A, Field Explorations.

3.2. Laboratory Testing

Soil samples were obtained during drilling and were taken to GeoEngineers' laboratory for further evaluation. Selected samples were tested for moisture content, percent fines (material passing the U.S. No. 200 sieve), and grain size distribution (sieve analysis). A description of the laboratory testing and the test results are presented in Appendix B, Laboratory Testing.

3.3. Geophysical Testing

We completed non-invasive geophysical testing on site consisting of two active-source multichannel analysis of surface waves (MASW) surveys and one passive-source microtremor array method (MAM) surveys. The geophysical testing report is provided in Appendix C.

3.4. Previous Site Evaluations

The logs of selected explorations from previous site evaluations in the project vicinity were reviewed and are presented in Appendix D, Boring Logs from Previous Studies. The approximate locations of these explorations are also shown on Figure 2.

4.0 SITE CONDITIONS

4.1. Surface Conditions

The UH4 Laurel Village site is bounded by NE 45th Street to the north, existing single-family residences to the east, and Mary Gates Memorial Drive NE to the southwest. The site is currently occupied by a multifamily student housing complex with several wood-framed buildings that were constructed in 1981. Existing site grades slope moderately down from northeast to southwest, from approximately Elevation 72 feet at the northeast corner down to Elevation 36 feet at the southwestern edge.

The subject property is designated as an Environmentally Critical Area (ECA) for steep slopes, a liquefaction-prone area, historic landfill buffer, and a peat settlement-prone area (Category 2) in accordance with the Seattle Municipal Code (SMC) Chapter 25.09. The approximate extents of the ECA zones are shown on Figure 2. The liquefaction-prone area is mapped within the vicinity of Buildings A and B and is addressed under separate cover. The subject property lies along the eastern shoreline of the former Union Bay, which was a peat marshland. The approximate extent of the former shoreline is also shown on Figure 2. In 1926, the City of Seattle used Union Bay as a public dump which then became the Montlake Landfill.

Buried utilities consisting of sanitary sewer, storm drain, gas, water, electric and telecommunications fiber are anticipated in the right-of-way adjacent to the site.

4.2. Subsurface Soil Conditions

GeoEngineers' understanding of subsurface conditions is based on the results of our investigation as well as our review of existing geotechnical information in the vicinity of the project site.

The soils encountered at the site consist of shallow fill overlying glacially consolidated till-like deposits and cohesionless sand and gravel. The fill generally consists of medium dense sand with variable silt and gravel content. The thickness of the fill encountered in the vicinity of Buildings C-1 to C-5 ranges from 1 to 5 feet.

The glacially consolidated soils were encountered below the fill and extended to the depths explored. The till-like deposits consist of very stiff to hard clay and silt and dense to very dense silty sand with gravel. The cohesionless sand and gravels consist of dense to very dense sand and gravel with varying amounts of silt. Glacially consolidated soils were encountered at shallower depths in the eastern portion of the site and at deeper depths in the western portion of the site. The estimated elevation of the top of the glacially consolidated/bearing soil layer is shown on Figure 3.

Although not encountered during our investigation, occasional cobbles and boulders are typical of glacially consolidated soils. Occasional cobbles and boulders may be present at the site and have been encountered in nearby construction projects.

4.3. Groundwater Conditions

Groundwater has been measured between 2.0 to 24.4 feet below grade in monitoring wells installed as part of our investigation. The elevation of the groundwater levels observed is presented in Table 1 and on Figure 4. The groundwater measured in the monitoring wells is interpreted to be regional groundwater table.

TABLE 1. GROUNDWATER MEASUREMENTS

Well ID	Ground Surface Elevation[1] (feet, NAVD 88)	Top of Casing Elevation (feet, NAVD 88)	Date of Measurement	Depth to Groundwater (feet)	Groundwater Elevation (feet, NAVD 88)
GEI-1	40	39.55	12/26/2023	2.0	37.55
			12/29/2023	2.6	36.95
			4/4/2024	2.9	37.10
GEI-2A	37	36.75	12/26/2023	2.6	34.15
			12/29/2023	2.6	34.15
			4/4/2024	2.77	34.23
GEI-3	68	67.75	12/26/2023	23.8	43.95
			12/29/2023	24.4	43.35
			4/4/2024	23.94	44.06

Notes:

[1] Measurements based on ALTA Survey data, December 21, 2023.

5.0 ENVIRONMENTALLY CRITICAL AREAS

GeoEngineers has reviewed the ECA maps available online through the City of Seattle Department of Construction and Inspections (SDCI) geographic information system (GIS) website. Based on our review of the SDCI GIS maps, the Building C-1 to C-5 development area is located within a mapped steep slopes area, historical landfill buffer area, and peat settlement prone area.

5.1. Steep Slope Assessment

Based on our review, the area mapped as a steep slope ECA meets the requirements for relief from prohibition on steep slope development per SDCI Tip 327A, which states the relief can be granted (subject to ECA review) when the "development is located on steep slope areas that have been created through previous legal grading activities, including rockeries or retaining walls resulting from rights-of-way improvements, if no adverse impact on the steep slope area will result."

The proposed development at the site will consist of demolishing the existing buildings, which are set back from the steep slope area, and constructing new student housing buildings. The existing steep slope areas were created during the existing site development (as part of legal grading). Given that the existing buildings are set back from the steep slope area, we judge there will be no adverse impacts to the planned development or existing adjacent improvements.

5.2. Landfill Historical Assessment

The project is mapped within a Landfill (Historical) 1,000-foot Methane Buffer Area related to the former Montlake Landfill. Project design and construction may be subject to certain development standards, including barriers or ventilation, to mitigate accumulation of hazardous levels of methane (SMC 25.09.220).

The University of Washington Environmental Health and Safety division is conducting on-going methane monitoring at locations around the perimeter of the former Montlake Landfill. The monitoring network includes two locations within parking areas of Laurel Village (MP-8 in the northwest portion and MP-9 in the southeast portion). Methane concentrations have exceeded the action limit of 100 parts per million (ppm) for UW offsite buildings at MP-8 during the monitoring since the early 2000s but the most recent quarterly data available (from 2022) has not indicated a concentration greater than the action level since February 2022. Monitoring point MP-9 has not indicated a concentration greater than the action limit in the available sampling data back to 2011. The University of Washington is managing the methane at Laurel Village through ventilation and monitoring consistent with their sampling and action plans. Monitoring for methane during our recent drilling and sampling for the redevelopment project has not detected methane in the boreholes on the Laurel Village property.

Based on the project location and the available monitoring data, methane mitigation will be included as part of project design and construction. This will likely include passive venting and/or use of a methane geomembrane beneath the slab.

5.3. Peat Settlement Prone Area Assessment

In order to avoid negative impacts from the planned development, the City of Seattle will require that the planned development be designed to prevent or accommodate settlement and that the project does not cause settlement off-site through modification of the groundwater table. Modification of the groundwater table through lowering or redirecting groundwater, even for a short period of time, may lead to off-site settlement. Ideally, no excavations should extend below the groundwater table in order to prevent modification of the groundwater table. If the project will require localized excavation below the groundwater table (such as for elevator pits, foundation elements, stairwells/ramps or limited sidewalk setbacks), the excavation will be required to be completed in a manner that does not adversely lower the groundwater table offsite.

6.0 CONCLUSIONS AND RECOMMENDATIONS

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

- The average shear wave velocity of the upper 100 meters at the site is approximately 1,162 feet per second, which corresponds to Site Class D per the 2018 IBC, and by reference, ASCE 7-16.
- Significant temporary dewatering is not anticipated for the planned Buildings C-1 to C-5. Localized dewatering for small excavations that extend below the groundwater table (for instance elevator pits, foundation elements, stairwells/ramps or limited sidewalk setbacks) are permissible if completed in a manner that does not adversely lower the groundwater table off site.
- Excavations for the planned buildings will generally be less than 5 feet, and can be temporarily sloped to accommodate the planned construction.
- Shallow foundations may be used where undisturbed glacially consolidated soils are present at the foundation subgrade elevation or where the non-bearing soils can be removed and replaced with properly compacted structural fill, as presented on Figure 3. For shallow foundations bearing directly

on undisturbed dense to very dense glacially consolidated soils or properly compacted structural fill extending down to undisturbed dense to very dense glacially consolidated soils, we recommend an allowable soil bearing pressure of 8 kips per square foot (ksf).

- Conventional slabs-on-grade are considered appropriate for this site and should be underlain by a 6-inch-thick layer of clean crushed rock (for example, City of Seattle Mineral Aggregate Type 22).

Our specific geotechnical recommendations are presented in the following sections of this report.

6.1. Earthquake Engineering

6.1.1. Liquefaction

Liquefaction refers to the condition by which vibration or shaking of the ground, usually from earthquake forces, results in the development of excess pore pressures in saturated soils with subsequent loss of strength. In general, soils that are susceptible to liquefaction include very loose to medium dense, clean to silty sands that are below the water table.

Groundwater levels at the site are generally within the dense to very dense glacially consolidated soils. Our analysis indicates that the soils that underlie the proposed building area have a low risk of liquefying because of the density and gradation of these soils.

6.1.2. Other Seismic Hazards

Due to the location of the site and the site's topography, the risk of adverse impacts resulting from seismically induced slope instability, differential settlement, or surface displacement due to faulting is considered to be low.

6.1.3. Code-Based Seismic Design Information

Based on the shear wave velocity measurements of the upper 30 meters at the site, the project site is Site Class D in accordance with the 2018 IBC.

We recommend using the following 2018 IBC, and by reference ASCE 7-16 parameters based on Site Class D, 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 as presented in Table 2.

TABLE 2. ASCE 7-16 MAPPED SEISMIC DESIGN PARAMETERS

ASCE 7-16 Parameter[1,2]	Recommended Value
Site Class	D
Mapped MCE_R spectral response acceleration at short period, S_S (g)	1.302
Mapped MCE_R spectral response acceleration at 1-second period, S_1 (g)	0.452
Short-period site coefficient, F_a	1.00
Long-period site coefficient, F_v	1.85[2]
MCE_R spectral response acceleration at short period adjusted or site class effects, S_{MS} (g)	1.302[2]

ASCE 7-16 Parameter[1,2]	Recommended Value
MCE_R spectral response acceleration at 1-second period adjusted or site class effects, S_{M1} (g)	0.835[2]
Design spectral response acceleration at short period adjusted or site class effects, S_{DS} (g)	0.868[2]
Design spectral response acceleration at 1-second period adjusted or site class effects, S_{D1} (g)	0.557[2]

Notes:

[1] Parameters developed based on latitude 47.659829 and longitude - 122.29085 using the ASCE 7 Hazards online tool (https://asce7hazardtool.online/).

MCE_R – risk-targeted maximum-considered earthquake

6.2. Temporary Dewatering

Temporary dewatering, such as sumps and pumps, may be required where excavations encounter perched water; significant temporary dewatering is not anticipated.

6.3. Foundation Support

Based on the data obtained from the borings completed at the site, review of previous explorations completed at the project site and the anticipated finished floor levels, the soils at the anticipated foundation elevation consist of either fill or glacially consolidated deposits.

Shallow foundations are recommended to support the planned buildings. Where existing fill is exposed at foundation subgrade, the fill should be recompacted to a firm and unyielding condition.

GeoEngineers has prepared a map with the estimated elevation of the top of bearing soils (Figure 3) to assist the project team with determining the need for structural fill.

6.3.1. Shallow Foundations

6.3.1.1. Allowable Bearing Pressure

Shallow foundations may be used where undisturbed glacially consolidated soils are present at the foundation subgrade elevation or where the non-bearing soils can be removed and replaced with properly compacted structural fill. For shallow foundations bearing directly on undisturbed dense to very dense glacially consolidated soils or properly compacted structural fill extending down to undisturbed dense to very dense glacially consolidated soils, we recommend an allowable soil bearing pressure of 8 ksf.

The allowable soil bearing pressure applies to the total of dead and long-term live loads and may be increased by up to one-third for wind or seismic loads. The allowable soil bearing pressures are net values.

6.3.1.2. Modulus of Subgrade Reaction

For foundations designed as a beam on an elastic foundation, a static modulus of subgrade reaction of 55 pounds per cubic inch (pci) may be used for foundations bearing on glacially consolidated soils or on structural fill extending down to glacially consolidated soils.

6.3.1.3. Settlement

Provided that all loose soil is removed and that the subgrade is prepared as recommended under “Construction Considerations” below, we estimate that the total settlement of the foundations will be about 1 inch or less. The settlements will occur rapidly, essentially as loads are applied. Differential settlements across the mat foundations could be half of the total settlement. Note that smaller settlements will result from lower applied loads.

6.3.1.4. Lateral Resistance

Lateral foundation loads may be resisted by passive resistance on the sides of footings and by friction on the base of the shallow foundations. For shallow foundations supported on native soils, CDF, or structural fill, the allowable frictional resistance may be computed using a coefficient of friction of 0.5 applied to vertical dead-load forces.

The allowable passive resistance may be computed using an equivalent fluid density of 400 pounds per cubic foot (pcf) (triangular distribution). These values are appropriate for foundation elements that are poured directly against undisturbed glacially consolidated soils or surrounded by structural fill.

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

6.3.1.5. Construction Considerations

We recommend that the condition of all subgrade areas be observed by GeoEngineers to evaluate whether the work is completed in accordance with our recommendations and whether the subsurface conditions are as anticipated.

During wet weather conditions or when wet weather is forecasted, the foundation subgrades are recommended to be protected with a rat slab consisting of 2 to 4 inches of lean or structural concrete to prevent deterioration of the subgrade during mat foundation steel and concrete placement.

If soft areas are present at the footing subgrade elevation, the soft areas should be removed and replaced with properly compacted structural fill, lean concrete/CDF, or structural concrete at the direction of GeoEngineers.

We recommend that the contractor consider leaving the subgrade for the foundations as much as 6 to 12 inches high, depending on soil and weather conditions, until excavation to final subgrade is required for foundation reinforcement. Leaving subgrade high will help reduce damage to the subgrade resulting from construction traffic or other activities on site.

6.3.1.5.1. Footing Drains

We recommend that perimeter footing drains be installed around the buildings. Perimeter footing drains should be installed at the base of the exterior footings. The perimeter drains should be provided with cleanouts and should consist of at least 4-inch-diameter perforated pipe placed on a 3-inch bed of, and surrounded by, 6 inches of drainage gravel enclosed in a non-woven geotextile fabric such as Mirafi 140N (or approved equivalent) to prevent fine soil from migrating into the drainage gravel. The footing drainpipe should be installed at least 18 inches below the top of the adjacent floor slab. The drainage gravel should consist of Mineral Aggregate Type 22 or Type 5 (1-inch washed gravel), City of Seattle Standard Specification 9-03.14, or an alternative approved by GeoEngineers. We recommend the drainpipe consist of perforated Schedule 40 polyvinyl chloride (PVC) pipe. We recommend against using flexible tubing for footing drainpipes.

Drainage pipes should be laid with minimum slopes of one-quarter percent (if possible) and discharge into the stormwater collection system to convey the water off site. The pipe installations should include a cleanout riser with cover located at the upper end of each pipe run. The cleanouts could be placed in flush mounted access boxes. Roof downspouts must not discharge into the perforated pipes intended for providing drainage for walls or foundations.

6.4. Slab Design

The new building slabs are not anticipated to extend below the groundwater table and therefore will not need to consider hydrostatic/uplift pressures. The slabs may be designed as bearing on grade.

6.4.1. Subgrade Preparation

The exposed slab subgrade should be evaluated after site grading is complete. Probing should be used to evaluate the subgrade. The exposed soil should be firm and unyielding, and without significant groundwater. Disturbed areas should be recompacted if possible or removed and replaced with compacted structural fill.

6.4.2. Design Parameters

For slabs-on-grade designed as a beam on an elastic foundation, a modulus of subgrade reaction of 150 pci may be used for subgrade soils prepared as recommended.

We recommend that the slab-on-grade and structural slab floors be underlain by a 6-inch-thick capillary break consisting of material meeting the requirements of Mineral Aggregate Type 22 (¾-inch crushed gravel), City of Seattle Standard Specification 9-03.14.

Provided that loose soil is removed and the subgrade is prepared as recommended, we estimate that slabs-on-grade will not settle appreciably.

6.4.3. Below-Slab Drainage

The planned buildings are anticipated to be constructed without the need for temporary excavation support. Given this, we anticipate that foundation drainage can be provided by means of an exterior perimeter footing drain and that below-slab drainage is not required.

If no special waterproofing measures are taken, leaks and/or seepage may occur in localized areas of the on-grade portion of the building, even if the recommended exterior perimeter footing drain provisions are constructed. If leaks or seepage is undesirable, waterproofing should be specified. A vapor barrier should be used below slab-on-grade floors located in occupied portions of the building. Specification of the vapor barrier requires consideration of the performance expectations of the occupied space, the type of flooring planned and other factors, and is typically completed by other members of the project team.

6.5. Rockeries

We understand that rockeries may be used for grade transitions at the site. The primary purpose of a rockery is to protect the slope face from erosion and raveling, while providing limited soil retention. Rockeries with a 15 degree minimum batter (from vertical) and horizontal backslope should be limited to 6 feet exposed height. The height is measured as the vertical distance from the ground surface in front of the toe of the rockery to the top of the rockery. Recommendations for rockeries at cut slopes are presented herein.

The base of rockeries should be embedded at least 12 inches below the adjacent ground surface. Rockeries should be supported on firm, undisturbed native soils or compacted structural fill. The rockery should be constructed using rock sizes specified by the Association of Rockery Contractors and (WSDOT) Standard Specifications, Section 9-13.7(1), and procedures specified the 2024 Washington State Department of Transportation (WSDOT) Standard Specifications, Sections 8-24 and the 2023 City of Seattle Standard Plans for Municipal Construction Standard Plan 141 for the required rockery heights.

Rockeries should be installed by a qualified contractor experienced in rockery construction.

If rockeries are to be terraced, the subgrade for the upper rockery should extend at least 3 feet horizontally in front of the rockery before making the cut for the lower rockery. In addition, we recommend that the top of the rockery include a horizontal setback of 5 feet from the adjacent property line. Rockery construction is an art and depends largely on the skill of the builder.

Although rockeries offer some lateral restraint, it is largely indeterminate, and they are not normally intended to provide significant lateral support. Even when the foundation and retained material are satisfactory, and the rockery materials and construction are satisfactory, there is some risk of movement or failure.

6.5.1. Rockery Drainage

The rockery design assumes drained conditions and does not allow for hydrostatic pressure buildup behind the rockeries. We recommend that a perforated drainpipe with a minimum diameter of 4 inches be placed at the back of the rockeries, below the ground surface elevation in front of the rockery. We recommend using either heavy-wall solid pipe (SDR-35 polyvinyl chloride [PVC]) or rigid corrugated polyethylene pipe (ADS N-12, or equivalent) for the collector pipe. We recommend against using flexible tubing for drainage.

A 12-inch-wide drainage backfill layer should be constructed as a drainage layer immediately behind the rockery facing with the drainpipe placed at the base of this layer. The drainage zone should consist of clear 1-½ to 3/8-inch crushed rock; smaller aggregate may have the potential to erode or pipe through the rockery face. The drainpipe should be routed to a suitable discharge point with suitable erosion protection.

6.6. Earthwork

6.6.1. Subgrade Preparation

Exposed subgrade in structure, hardscape, and pavement areas should be evaluated after site excavation is complete. Foundation subgrades should be prepared as recommended in “Shallow Foundations” above. Where hardscape and pavement subgrade soils consist of disturbed soils, it will likely be necessary to remove and replace the disturbed soil with approved structural fill unless the soil can be adequately moisture-conditioned and compacted.

6.6.2. Structural Fill

Fill placed to support structures or foundations, placed behind retaining structures, for foundation drainage, and/or placed below pavements and sidewalks shall consist of structural fill as specified below:

- If structural fill is necessary beneath shallow foundations, the fill should consist of Mineral Aggregate Type 2 (1¼-inch minus crushed rock), City of Seattle Standard Specification 9 03.14, controlled density fill, or structural concrete.
- If structural fill is necessary beneath building slabs, the fill should consist of Mineral Aggregate Type 2 or Type 17 (1¼-inch minus crushed rock or bank run gravel), City of Seattle Standard Specification 9-03.14.
- Structural fill placed behind retaining walls should meet the requirements of Mineral Aggregate Type 17 (bank run gravel), City of Seattle Standard Specification 9-03.14.
- Structural fill placed as capillary break material should meet the requirements of Type 22 (¾ inch crushed gravel), City of Seattle Standard Specification 9-03.14.
- Structural fill placed around perimeter footing drains, underslab drains and cast-in-place wall drains should meet the requirements of Mineral Aggregate Type 5 (1-inch washed gravel) or Type 22 (¾-inch crushed gravel), City of Seattle Standard Specification 9-03.14.
- Structural fill placed within utility trenches and below pavement and sidewalk areas should meet the requirements of Mineral Aggregate Type 17 (bank run gravel), City of Seattle Standard Specification 9-03.14.
- Structural fill placed as crushed surfacing base course below pavements and sidewalks should meet the requirements of Mineral Aggregate Type 2 (1¼-inch minus crushed rock), City of Seattle Standard Specification 9-03.14.

6.6.2.1. On-site Soils

On-site soils are moisture-sensitive and have natural moisture contents higher than the anticipated optimum moisture content for compaction. In addition, the fines content for the on-site soils generally ranges from 9 to 30 percent. As a result, on-site soils will likely require moisture conditioning to meet the required compaction criteria during dry weather conditions and will 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 do not meet these gradation requirements. Therefore, imported structural fill meeting the requirements described above should be used where structural fill is necessary.

It may be feasible to reuse on-site soils with the addition of cement treatment. If cement treatment is considered, GeoEngineers can work with the contractor to determine the soil/cement ratio and placement procedures.

6.6.2.2. Fill Placement and Compaction Criteria

Structural fill should be mechanically compacted to a firm, non-yielding condition and placed in loose lifts not exceeding 1 foot in thickness. Each lift should be conditioned to the proper moisture content and compacted to the specified density before placing subsequent lifts. Structural fill should be compacted to meet the following criteria:

- Structural fill placed in building areas (including around foundations and supporting slab-on-grade floors), pavement and sidewalk areas (including utility trench backfill) should be compacted to at least 95 percent of the maximum dry density (MDD) estimated in general accordance with ASTM International (ASTM) D 1557.
- Structural fill placed against retaining walls should be compacted to between 90 and 92 percent. Care should be taken when compacting fill against retaining walls to avoid overcompaction and, hence overstressing the walls.

We recommend that 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 to verify compliance with compaction specifications, and advise on any modifications to the procedures that may be appropriate for the prevailing conditions.

6.6.2.3. Weather Considerations

On-site soils contain a sufficient percentage of fines (silt and clay) 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 equipment operation becomes difficult. Additionally, disturbance of near-surface soils should be expected if earthwork is completed during periods of wet weather. During wet weather, we recommend the following:

- The ground surface in and around the work area should be sloped so that surface water is directed away from the work area. The ground surface should be graded such that areas of ponded water do not develop. The contractor should take measures to prevent surface water from collecting in excavations and trenches. Measures should be implemented to remove surface water from the work area.
- Slopes with exposed soils should be covered with plastic sheeting or similar means.
- Site soils should not be left uncompacted and exposed to moisture. Sealing the surficial soils by rolling with a smooth-drum roller prior to periods of precipitation will reduce the extent to which these soils become wet or unstable.
- Construction traffic should be restricted to specific areas of the site, preferably areas that are surfaced with materials not susceptible to wet weather disturbance.
- Construction activities should be scheduled so that the length of time that soils are left exposed to moisture is reduced to the extent practicable.

6.6.3. Temporary Slopes

Temporary slopes may be used around the site to facilitate early installation of shoring or in the transition between levels at the base of the excavation. We recommend that temporary slopes constructed in the fill be inclined at 1½H:1V (horizontal to vertical) and that temporary slopes in the glacially consolidated soils be inclined at 1H:1V. Flatter slopes may be necessary if seepage is present on the face of the cut slopes or if localized sloughing occurs. For open cuts at the site, we recommend that:

- No traffic, construction equipment, stockpiles or building supplies be allowed at the top of the 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 that 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.

6.7. Pavement Design

6.7.1. Pavement Subgrade Preparation

Prior to placing new fill or pavement base course materials, subgrade areas should be proof rolled to locate soft or pumping soils. Prior to proof rolling, unsuitable soils should be removed from below 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 determine the extent of soft soils. If soft or pumping soils are observed, they should be removed and replaced with structural fill.

6.7.2. New Hot-Mix Asphalt Pavement

In light-duty pavement areas (e.g., automobile parking), we recommend a pavement section consisting of at least 3 inches of hot-mix asphalt (HMA) over 4 inches of densely compacted aggregate base. In heavy-duty pavement areas (such as driveways, truck traffic lanes, materials delivery), we recommend a pavement section consisting of at least 4 inches HMA over 6 inches of densely compacted aggregate base.

Structural fill placed as crushed surfacing base course below pavements should meet the requirements of Mineral Aggregate Type 2 (1¼-inch minus crushed rock), City of Seattle Standard Specification 9-03.14 and should be compacted to at least 95 percent of the MDD obtained using ASTM D 1557. We recommend that proof rolling of the subgrade and compacted aggregate base be observed by a representative from our firm prior to paving. Soft or yielding zones observed during proof rolling may require over-excavation and replacement with compacted structural fill.

The pavement sections recommended above are based on our experience. Thicker asphalt sections may be needed based on the actual traffic data, truck loads and intended use. Paved and landscaped areas should be graded so that surface drainage is directed to appropriate catch basins.

6.7.3. Portland Cement Concrete Pavement

Portland cement concrete (PCC) sections may be considered for areas where concentrated heavy loads may occur, including trash enclosures. We recommend that these pavements consist of at least 6 inches of PCC over 6 inches of aggregate base. A thicker concrete section may be needed based on the actual load data for use of the area. If the concrete pavement will have doweled joints, we recommend that the concrete thickness be increased by an amount equal to the diameter of the dowels. The base course should be compacted to at least 95 percent of the MDD.

We recommend PCC pavements incorporate construction joints and/or crack control joints spaced at maximum distances of 12 feet apart, center-to-center, in both the longitudinal and transverse directions. Crack control joints may be created by placing an insert or groove into the fresh concrete surface during finishing, or by saw cutting the concrete after it has initially set-up. We recommend the depth of the crack control joints be approximately one fourth the thickness of the concrete; or about 1½ inches deep for the recommended concrete thickness of 6 inches. We also recommend the crack control joints be sealed with an appropriate sealant to help restrict water infiltration into the joints.

6.8. Recommended Additional Geotechnical Services

GeoEngineers should be retained to review the project plans and specifications when complete to confirm that our design recommendations have been implemented as intended.

During construction, GeoEngineers should evaluate the suitability of the foundation 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 that the subsurface conditions are consistent with those observed in the explorations and other reasons described in Appendix E, Report Limitations and Guidelines for Use.

7.0 LIMITATIONS

We have prepared this report for the exclusive use of GDSU Washington, LLC. and their authorized agents for the UH4 Laurel Village project in Seattle, 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 E for additional information pertaining to use of this report.

8.0 REFERENCES

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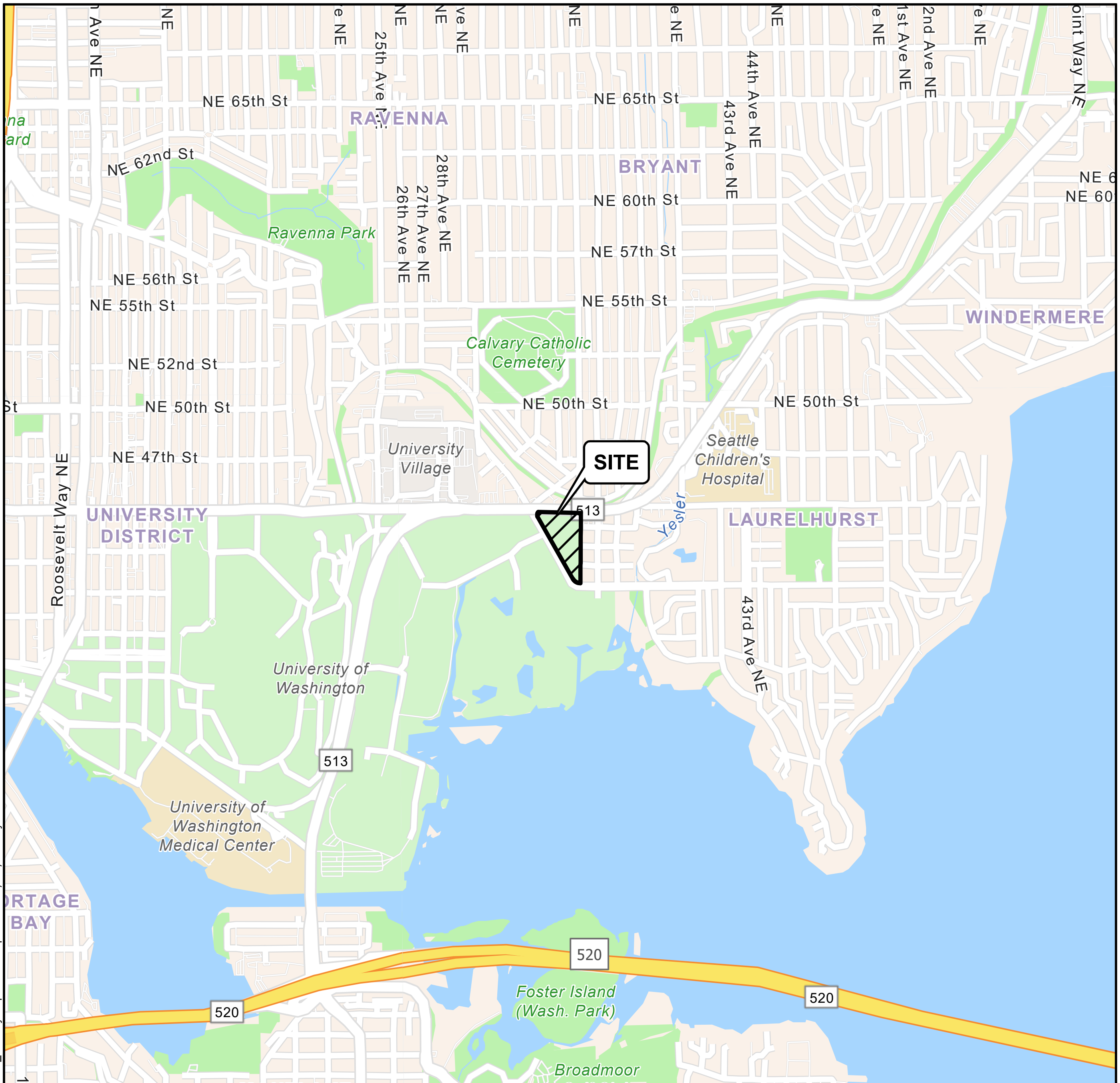
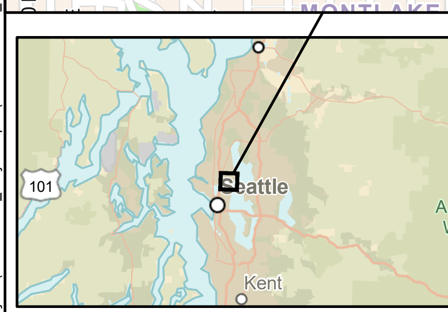
FIGURES





N

0 2,000

Feet

Source(s):
- ESRI

Coordinate System: NAD 1983 UTM Zone 10N

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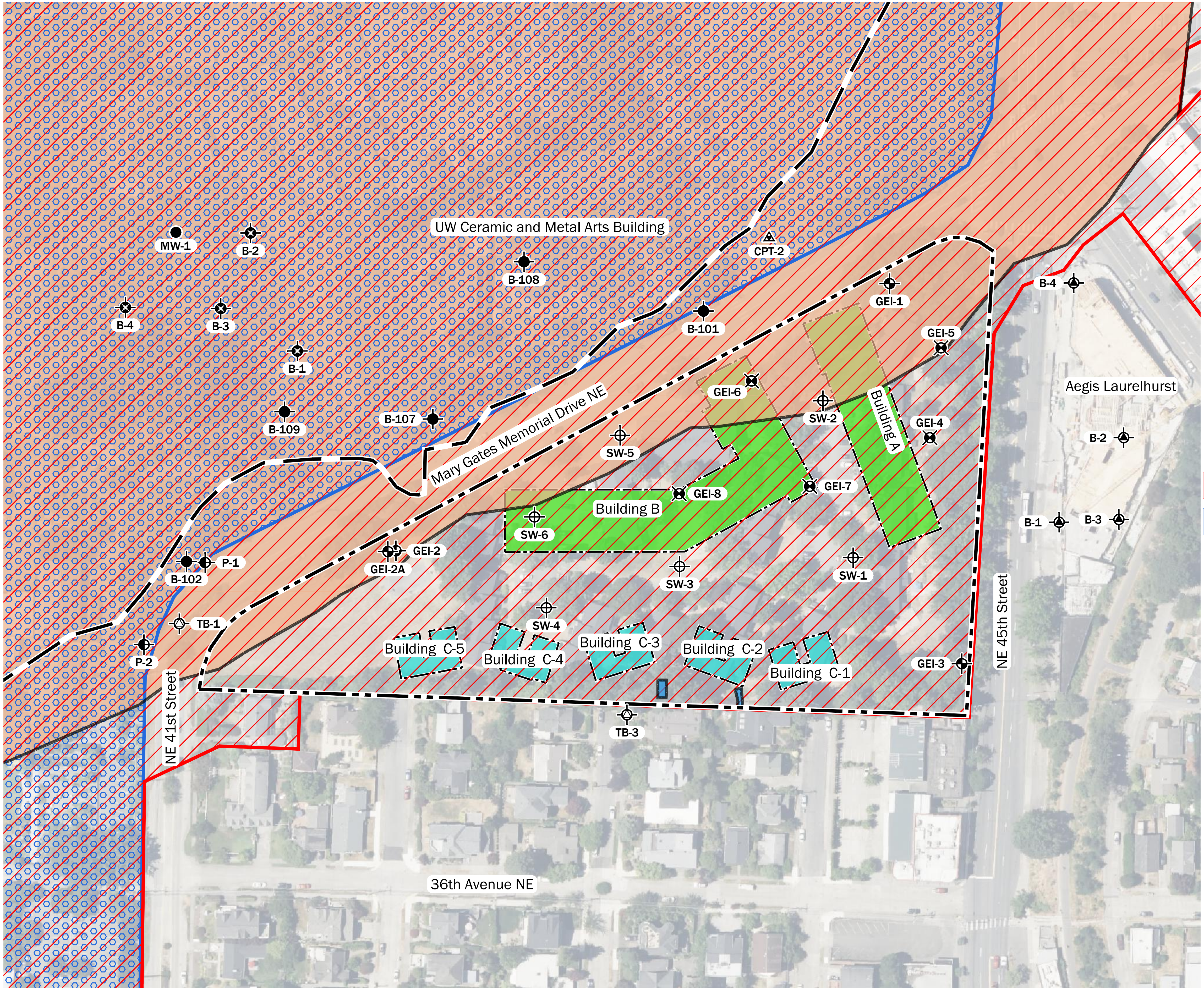
Vicinity Map

UH4 Laurel Village - Buildings C-1 to C-5
Seattle, Washington

GEOENGINEERS

Figure 1



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Site Plan

UH4 Laurel Village - Buildings C-1 to C-5
Seattle, Washington

GeoEngineers

Figure 2

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Legend

- Site Boundary
- Aproximate Extent of Former Union Bay Shoreline 1912 (Baist's surveys of Seattle)
- Bearing Contours (Feet, NAVD 88)
- Ground Surface Elevation (Feet, NAVD 88)
- Proposed 6-Story Structure
- Proposed Townhome and Flats Structures
- GEI-4 Boring by GeoEngineers, Inc., 2024
- GEI-1 Monitoring Well by GeoEngineers, Inc., 2023
- SW-1 Boring by Shannon & Wilson, 2022
- B-1 Boring by PSI Intertek, 2016
- B-1 Boring by GeoEngineers, Inc., 2016
- MW-1 Monitoring Well by GeoEngineers, Inc., 2016
- P-1 Boring by Shannon & Wilson, 2002
- CPT-2 Cone Penetration Test by AGRA, 1996
- TB-1 Boring by Pacific Testing Laboratories, 1996
- B-102 Boring by Shannon & Wilson, 1966

Source(s):
- Aerial from Microsoft Bing

Projection: WA State Plane, North Zone, NAD83, US Foot

N

0 120
Feet

Bearing Contour Map

UH4 Laurel Village - Buildings C-1 to C-5
Seattle, Washington

GeoEngineers

Figure 3

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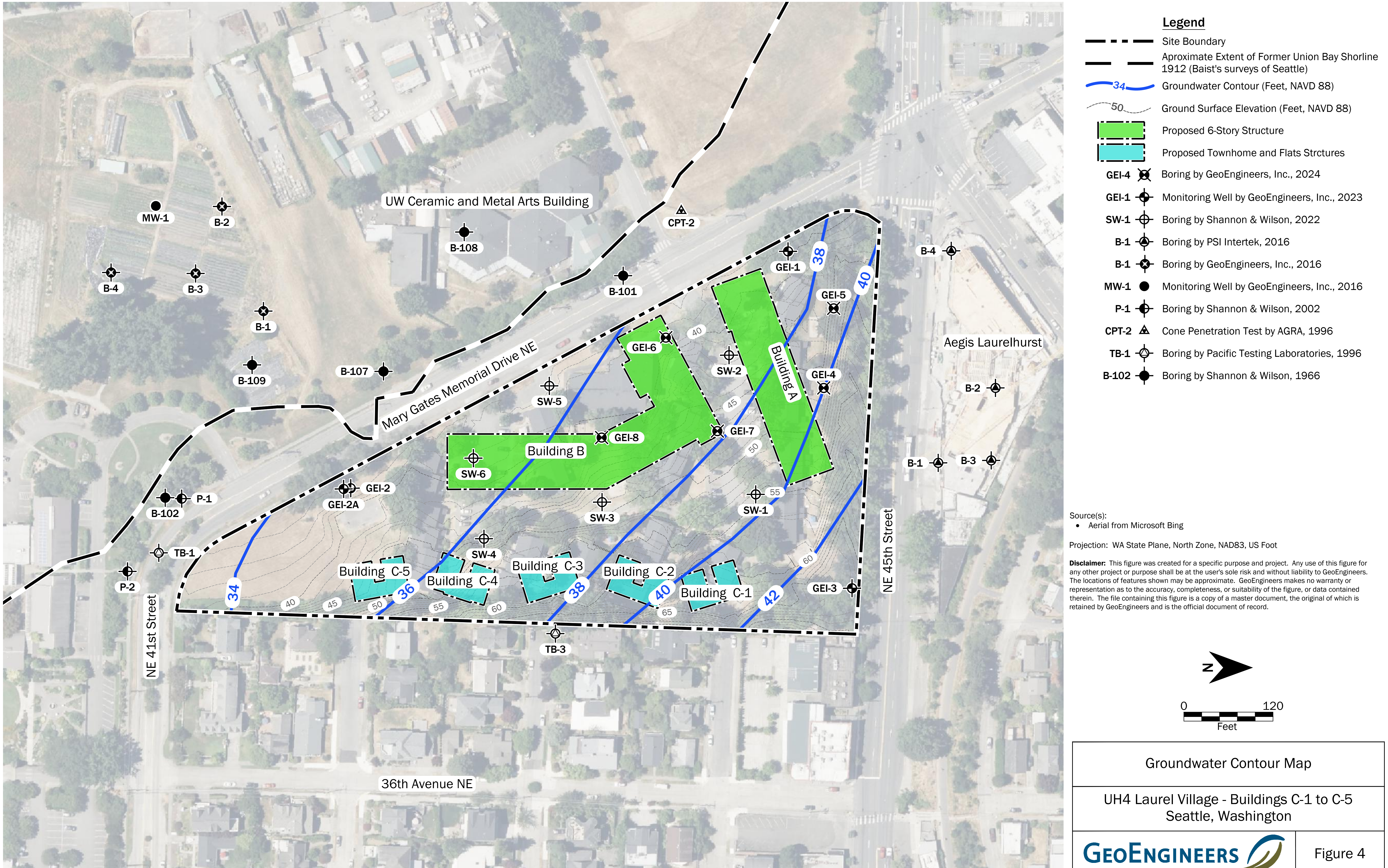

Source(s):
- Aerial from Microsoft Bing

Projection: WA State Plane, North Zone, NAD83, US Foot

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Groundwater Contour Map

UH4 Laurel Village - Buildings C-1 to C-5
Seattle, Washington

GEOENGINEERS

Figure 4

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APPENDICES

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Exhibit E – Tree Preservation and Removal

Key:
Diamond shape = preservation
X demarcation = removal

Exhibit F – Phase II Environmental Assessment

Appendices available upon request.

Phase II Environmental Site Assessment Addendum

UH4 Project – Laurel Village Property
4200 Mary Gates Memorial Drive NE
Seattle, Washington

for
GDSU Washington, LLC

June 20, 2024

2101 4th Avenue, Suite 950
Seattle, Washington 98121
206.728.2674

GeoEngineers

Phase II Environmental Site Assessment Addendum

UH4 Project – Laurel Village Property
4200 Mary Gates Memorial Drive NE
Seattle, Washington

File No. 20449-013-01
June 20, 2024

Prepared for:

GDSU Washington, LLC
450 Sansome Street, Suite 500
San Francisco, California 94111

Attention: Chad Winters

Prepared by:

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GeoEngineers

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Executive Summary

This report is an addendum to the supplemental Phase II Environmental Site Assessment (ESA) conducted for GDSU Washington, LLC (i.e., Greystar) as part of the proposed redevelopment of the property located at 4200 Mary Gates Memorial Drive Northeast in Seattle, Washington (subject property). We understand that Greystar is conducting due diligence for planning purposes associated with a long-term lease and redevelopment of the subject property. Plans for redevelopment include construction of new student housing facilities requiring potential grading and excavation of soil.

The initial Phase II ESA was conducted in December 2023 to evaluate the recognized environmental conditions, including an east-adjoining former dry cleaner property, identified in the 2024 Phase I ESA for the subject property by GeoEngineers, and to provide information regarding shallow subsurface conditions at the subject property. The initial Phase II included observations during the drilling of monitoring well GEI-3 during the concurrent geotechnical investigation, and during the soil sampling at borings DP-1 through DP-8 and groundwater sampling of wells GEI-1, GEI-2A and GEI-3 (Soil Chemical Analytical Results, Figure 2). The laboratory chemical analytical data identified the presence of the chlorinated solvent tetrachloroethene (PCE) at concentrations greater than the Washington State Department of Ecology (Ecology) Model Toxics Cleanup Act (MTCA) Method A cleanup levels in both soil and groundwater in the northeast portion of the subject property (Figure 2; soil borings DP-3 and DP-4, and groundwater monitoring well GEI-3). The lateral extent of the PCE in both media was not documented to the west and south of this portion of the subject property, which is area adjacent to the west of a former dry cleaner property that was identified during the Phase I ESA. Based on the results of the initial Phase II ESA and communications with Greystar, additional supplemental sampling and analysis of soil and groundwater was recommended to further characterize the nature and extent of the PCE in soil and groundwater as part of planning for construction excavation, and for the appropriate management and disposal of contaminated media removed during project construction.

This supplemental Phase II ESA was conducted at the subject property to further characterize the nature and extent of PCE in both soil and groundwater to the west and south of the northeast portion of the subject property. The supplemental scope of work included soil sampling at borings DP-9 through DP-16 and the collection of reconnaissance groundwater samples from borings DP-11 and DP-13. The supplemental Phase II ESA soil and groundwater analytical results identified the following:

- Volatile organic compounds were detected in soil and groundwater in the northern portion of the subject property.
  - PCE was detected at concentrations ranging from 0.0138 to 0.0450 milligrams per kilogram (mg/kg), which are all less than the MTCA Method A cleanup level for unrestricted land use, in the soil samples collected from borings DP-12, DP-14, and DP-15 at depths ranging from 5½ to 15 feet below ground surface.
  - Groundwater flow direction was interpreted to be to the south-southwest, generally toward Union Bay, which is consistent with the local topography, placing the historical Laurelhurst Cleaners upgradient of the subject property.
  - PCE was not detected at concentrations greater than the laboratory reporting limits in the groundwater samples collected from borings located downgradient of GEI-3 where PCE had previously been detected in groundwater.

The extent of the PCE concentrations in soil is bounded laterally by non-detect analytical results for the soil samples collected to the west at boring DP-5 and to the south at borings DP-11, DP-13, and DP-16. The extent of the PCE concentrations in groundwater is bounded laterally by non-detect analytical results for the reconnaissance groundwater sample collected from boring DP-13. The vertical extent of PCE concentrations in soil at borings DP-12, DP-14, and DP-15 has not been fully delineated based on the sampling results to date.

This Executive Summary should be used only in the context of the full report for which it is intended and the associated initial Phase II ESA Report.

1.0 Introduction

This supplemental Phase II Environmental Site Assessment (ESA) was completed by GeoEngineers, Inc. (GeoEngineers) for GDSU Washington, LLC (i.e., Greystar) as part of the evaluation of potential environmental liabilities associated with leasing and redevelopment of the subject property (Vicinity Map, Figure 1). The subject property is located in the University District neighborhood of Seattle and is bounded by NE 45th Street on the north, Mary Gates Memorial Drive NE on the west, NE 41st Street on the south, and private property on the east (Soil Chemical Analytical Results, Figure 2).

This supplemental Phase II ESA was completed to provide additional project site-specific data regarding soil and groundwater conditions to support project design and pre- construction planning associated with the planned redevelopment project. In addition to the environmental services for the subject property, GeoEngineers is also providing geotechnical services to Greystar regarding the subject property (GeoEngineers 2024a).

We understand that the proposed redevelopment of the subject property includes demolition of the existing buildings and constructing several new six-story student housing buildings and townhome/flats structures. The new buildings will generally be constructed at-grade with minimal excavation. However, temporary cut slopes and/or temporary shoring and grading are anticipated to be required to complete the planned localized excavation.

1.1 PREVIOUS ENVIRONMENTAL INVESTIGATION

GeoEngineers conducted a Phase II ESA in December 2023 to evaluate the recognized environmental conditions that were identified in the Phase I ESA and provide details regarding the shallow subsurface conditions (GeoEngineers, 2024c). The Phase II included observations during the drilling of monitoring well GEI-3 during the geotechnical investigation that was conducted by GeoEngineers, and during the soil sampling at borings DP-1 through DP-8 and groundwater sampling of wells GEI-1, GEI-2A and GEI-3. The Phase II ESA soil and groundwater analytical results identified the following:

- Volatile organic compounds (VOCs) were detected in soil and groundwater in the northeast portion of the subject property.
  - Tetrachloroethene (PCE) was detected in soil at concentrations greater than the Washington State Model Toxics Control Act (MTCA) Method A cleanup level for unrestricted land use at boring DP-1 at a depth ranging from 14.0 to 15.0 feet below ground surface (bgs) and at boring DP-4 at a depth ranging from 13.0 to 14.0 feet bgs at concentrations of 0.0501 and 0.0511 milligrams per kilogram (mg/kg), respectively. PCE was detected in soil at concentrations less than the MTCA cleanup level at boring DP-1 at depths ranging from 4.0 to 7.0 feet bgs, and at boring DP-2 at a depth ranging from 12.0 to 13.0 feet bgs at concentrations ranging from 0.0230 to 00457 mg/kg, which are less than the MTCA Method A cleanup level.
  - PCE was detected in the groundwater sample collected from monitoring well GEI-3 at a concentration greater than the MTCA Method A cleanup level protective of groundwater (5.58 micrograms per liter [µg/L]). This concentration was very close to the MTCA Method A cleanup level of 5 µg/L, and less than the MTCA Method B long-term potential vapor intrusion screening level for cancer of 25 µg/L for PCE.
  - Groundwater flow direction was interpreted to be to the south-southwest, generally toward Union Bay, which is consistent with the local topography, placing the historical Laurelhurst Cleaners upgradient of the subject property.

2.0 Supplemental Phase II Environmental Site Assessment

GeoEngineers conducted a supplemental Phase II ESA at the subject property on April 4, 2024 as part of planning related to the proposed redevelopment.

2.1 PURPOSE AND SCOPE OF SERVICES – APRIL 2024

The objective of the supplemental Phase II ESA was to further characterize the nature and extent of the PCE in soil and groundwater at the subject property that could be encountered during construction for the planned redevelopment.

Figure 2 shows the approximate exploration locations for the supplemental Phase II ESA. Exploration locations were selected to document the lateral extent of the PCE in both soil and groundwater to the west and south of northeast portion of the subject property, which is adjacent to a former dry cleaner property that was identified during the Phase I ESA. Following pre-field coordination with a private utility locator to mark and clear the proposed boring locations for underground utilities and magnetic anomalies, GeoEngineers completed the services outlined below on April 4, 2024.

2.1.1 Field Exploration and Sampling

1. Drilling and sampling of eight (8) soil borings (DP-9 through DP-16) using direct-push drilling equipment operated by Holocene Drilling of Puyallup, Washington. Exploration depths extend to 15 feet bgs.
2. During drilling, discrete soil samples from the borings were field screened for evidence of contamination using visual, water sheen, and headspace vapor screening methods (measured with a photoionization detector). Soil from the borings was visually classified in general accordance with ASTM International (ASTM) D 2488, and a detailed log of each exploration was prepared. Soils were saturated near surface to 15 feet bgs in borings DP-9 through DP-11 and DP-13 through DP-16. Field methods and boring logs are presented in Field Methods and Boring Logs, Attachment A. Selected soil samples were submitted for laboratory chemical analysis.
3. Collecting reconnaissance groundwater samples from borings DP-11 and DP-13. Additional reconnaissance groundwater sampling was attempted at borings DP-10 and DP-12; however, due to insufficient recharge, reconnaissance groundwater samples were not collected. Prior to sampling, each temporary well was purged until turbidity in groundwater appeared to decrease or the temporary well was purged dry and sufficient recharge had occurred. Selected groundwater samples were submitted for laboratory chemical analysis.
4. Collecting depth-to-water measurements from monitoring wells GEI-1, GEI-2A, and GEI-3 to evaluate groundwater elevations and inferred groundwater flow direction at the subject property.

Potential contaminants in soil and groundwater were identified as those associated with dry-cleaning operations including VOCs. Supplemental Phase II ESA Explorations, Table A summarizes the supplemental Phase II ESA sampling and analysis completed at the subject property. The exploration locations are shown in Figure 2.

TABLE A. SUPPLEMENTAL PHASE II ESA EXPLORATIONS – APRIL 2024

LOCATION IDENTIFICATION	ANALYSES COMPLETED
	VOCS
DP-9	S
DP-10	S
DP-11	S
DP-12	S
DP-13	S, W
DP-14	S
DP-15	S
DP-16	S

Notes:

MTCA = Model Toxics Control Act

S = Soil Sample Analyzed

W = Water Sample Analyzed

The groundwater sample collected from DP-11 was damaged during handling at the analytical laboratory and could not be analyzed.

Chemical analytical results for the soil and groundwater samples obtained during this supplemental Phase II ESA were compared to the respective MTCA Method A cleanup levels (Ecology, 2023). MTCA Method B cleanup levels were used for analytes where MTCA Method A cleanup levels are not established. The supplemental Phase II ESA soil and groundwater chemical analytical results are summarized in Tables 1 and 2. The analytical results for the detected contaminants are shown in Figures 2 and 3.

2.1.2 Soil

The 41 soil samples collected from the direct-push borings DP-9 through DP-16 were submitted for laboratory chemical analysis of VOCs.

2.1.2.1 VOCS

PCE was detected in soil samples DP-12-5.0 and DP-12-12.5 (boring DP-12 at depths of 5 and 12½ feet bgs), DP-14-10.0 and DP-14-15.0 (boring DP-14 at depths of 10 and 15 feet bgs), and DP-15-10.0 (boring DP-15 at a depth of 10 feet bgs) at concentrations ranging from 0.0138 to 0.450 mg/kg, which are all less than the MTCA Method A cleanup level for unrestricted land use (Figure 2; Table 1).

PCE and associated degradation compounds including, trichloroethene (TCE), cis-1,2-dichloroethene (DCE), trans-1,2-DCE, and vinyl chloride; and remaining VOCs were not detected at concentrations greater than the laboratory reporting limits in the remaining soil samples analyzed (Table 1).

2.1.3 Reconnaissance Groundwater

The two reconnaissance groundwater samples collected from direct-push borings DP-11 and DP-13 were submitted for laboratory chemical analysis of VOCs. As noted previously, the groundwater sample from boring DP-11 was damaged during handling and could not be analyzed.

Groundwater elevations ranged from 33.98 feet North American Vertical Datum of 1988 (NAVD 88) at monitoring well GEI-2A to 43.81 feet NAVD 88 at monitoring well GEI-3. The groundwater measured in monitoring wells GEI-1, GEI-2A, and GEI-3 is interpreted to be the regional groundwater table and the interpreted groundwater flow direction is to the south-southwest. The measured elevation of the groundwater levels observed, and the interpreted groundwater flow are presented on Figure 3.

2.1.3.1 VOCS

PCE and associated degradation compounds including, TCE cis-1,2-DCE, trans-1,2-DCE, and vinyl chloride and remaining VOCs were not detected at concentrations greater than the laboratory reporting limits in the reconnaissance groundwater sample collected from boring DP-13.

3.0 Discussion and Conclusions

The supplemental Phase II ESA field and chemical analytical data provide further detail regarding shallow subsurface conditions at the subject property. The findings regarding subsurface conditions at the subject property are as follows:

- VOCs were detected in soil in the north and northeast portion of the subject property.
  - PCE was detected in soil at borings DP-12, DP-14, and DP-15 at depths ranging from 5.0 to 15.0 bgs, at concentrations less than the MTCA Method A cleanup level.

There is no information to indicate that the concentrations of PCE detected in soil are related to historical operations/sources on or from the subject property. PCE was not detected in the soil samples collected in the central and west portions of the subject property. The source of the PCE contamination in the soil samples collected from borings DP-12, DP-14, and DP-15, is likely related to the upgradient historical/former dry cleaner operation that was located on the adjacent property to the northeast of the subject property.

The extent of the PCE concentrations in soil is bounded laterally by non-detect analytical results for the soil samples collected to the south at borings DP-11, DP-13, and DP-16 and to the west at boring DP-5. The extent of PCE concentrations in groundwater is bounded laterally by non-detect analytical results for the reconnaissance groundwater sample collected at DP-13.

Other VOCs were not detected at concentrations greater than the laboratory reporting limits in any of the soil or groundwater samples analyzed during the supplemental Phase II ESA.

3.1 CONSTRUCTION SOIL MANAGEMENT

Based on our understanding of the current plans for redevelopment, future construction at the subject property will encounter areas of PCE-impacted and -contaminated soil as shown on Figure 2. Soil removed from northeastern portion of the subject property during property redevelopment will require handling and disposal separate from other site soils during construction excavation.

Chlorinated solvents such as PCE are among contaminants listed by Washington State Department of Ecology (Ecology) as “dangerous wastes” requiring special documentation and approval for handling and disposal. Management of PCE containing soil encountered during excavation for property redevelopment should be conducted under the Ecology Contained-In policy which allows for soil with low concentrations of PCE to be managed as solid (non-hazardous) waste at reduced disposal costs.

Two potential approaches for disposal of chlorinated solvent-impacted soil are available:

- Dangerous Waste Disposal, under which soil with any amount of an Ecology-listed dangerous waste is transported and disposed of, at premium cost, at a facility specifically designated to manage a dangerous waste;
- Contained-In Determination, under which an applicant may be able to reduce disposal costs by submitting representative data for Ecology review to show that it is below established toxicity thresholds, and thereby manage it as solid (nonhazardous) waste and disposed of at a conventional, permitted landfill.

Based on the results of the supplemental Phase II ESA, the estimated volume of soil is 650 cubic yards (1,105 tons). Assuming maximum estimated extents of PCE impacts in soil and construction excavation anticipated by current design (Figure 2), Rough Order of Magnitude (ROM) cost estimates for each type of soil disposal are shown below.

SCENARIO	DISPOSAL TYPE	ESTIMATED COST PER TON	ROM COST ESTIMATE
Estimated Extent of Contaminants	Subtitle D Landfill	$70 per ton	$77,400
	Private facility	$35 per ton	$38,700

4.0 References

GeoEngineers, Inc., (GeoEngineers 2024a). "Geotechnical Due Diligence Services, UH4 Laurel Village, Seattle Washington" dated January 12, 2024.

GeoEngineers, Inc., (GeoEngineers 2024b). "Phase I Environmental Site Assessment, UH4 Project – 4200 Mary Gages Memorial Drive, Seattle, Washington" dated January 19, 2024.

GeoEngineers, Inc., (GeoEngineers 2024c). "Phase II environmental Site Assessment, UH4 Laurel Village Property, 4200 Mary Gates Memorial Drive, Seattle Washington" dated February 13, 2024.

Washington State Department of Ecology (Ecology, 2023). Model Toxics Control Act Regulation and Statute. Publication No. 94-06. Revised August 2023.

5.0 Limitations

We have prepared this report for the exclusive use of GDSU Washington, LLC, their authorized agents and regulatory agencies. This report is not intended for use by others and the information contained herein is not applicable to other sites. No other party may rely on the product of our services unless we agree in advance, and in writing, to such reliance. This is to provide our firm with reasonable protection against open-ended liability claims by third parties with whom there would otherwise be no contractual limits to their actions.

Our conclusions are based on our site observations, field screening results and chemical analysis of a limited number of discrete soil and groundwater samples obtained from the subject property. It is always possible that contaminants are present in locations that were not observed, sampled or tested.

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

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

Tables

Table 1

Soil Field Screening and Chemical Analytical Data (Petroleum Hydrocarbons and VOCs)

Laurel Village Property

4200 Mary Gates Memorial Drive

Seattle, Washington

Exploration Location[1]	Sample ID	Sample Date	Depth (feet bgs)	Field Screening[2]		Petroleum Hydrocarbons (mg/kg)			VOCs[5] (mg/kg)						
				Sheen	Headspace (ppm)	Gasoline Range[3]	Diesel Range[4]	Heavy Oil Range[4]	Benzene	Tetrachloroethene	Trichloroethene	cis-1,2-Dichloroethene	trans-1,2-Dichloroethene	1,1-Dichloroethene	Vinyl Chloride
Hollow stem auger boring sampled by GeoEngineers December 6, 2024															
GEI-3	GEI3-2.5-4	12/6/2023	2.5 - 4.0	--	--	--	--	--	--	< 0.0133	< 0.0133	< 0.0133	< 0.00889	< 0.0889	< 0.0222
	GEI3-25-26.5		25.0 - 26.5	--	--	--	--	--	--	< 0.0160	< 0.0160	< 0.0160	< 0.0107	< 0.107	< 0.0266
	GEI3-30-31.5		30.0 - 31.5	--	--	--	--	--	--	< 0.0155	< 0.0155	< 0.0155	< 0.0103	< 0.103	< 0.0259
Direct-push borings sampled by GeoEngineers December 21, 2023															
DP-1	DP1-4-5	12/21/2023	4.0 - 5.0	NS	<1	--	--	--	< 0.0197	**0.0426**	< 0.0169	< 0.0169	< 0.0112	< 0.112	< 0.0281
	DP1-6-7		6.0 - 7.0	NS	<1	--	--	--	< 0.0184	**0.0457**	< 0.0157	< 0.0157	< 0.0105	< 0.105	< 0.0262
	DP1-14-15		14.0 - 15.0	NS	<1	< 4.35	< 51.1	< 102	< 0.0152	**0.0501**	< 0.0130	< 0.0130	< 0.00870	< 0.0870	< 0.0217
DP-2	DP2-2-3	12/21/2023	2.0 - 3.0	SS	< 1	--	--	--	--	< 0.00664	< 0.00664	< 0.00664	< 0.00442	< 0.0442	< 0.0111
	DP2-12-13		12.0 - 13.0	NS	< 1	< 5.42	< 55.1	<110	< 0.0190	**0.0230**	< 0.0163	< 0.0163	< 0.0108	< 0.108	< 0.0271
DP-3	DP3-3-4	12/21/2023	3.0 - 4.0	NS	< 1	< 5.91	< 52.0	< 104	--	< 0.0177	< 0.0177	< 0.0177	< 0.0118	< 0.118	< 0.0295
	DP3-14-15		14.0 - 15.0	NS	< 1	< 6.56	< 55.4	< 111	< 0.0230	< 0.0197	< 0.0197	< 0.0197	< 0.0131	< 0.131	< 0.0328
DP-4	DP4-4-5	12/21/2023	4.0 - 5.0	SS	< 1	< 4.47	< 51.0	< 102	--	< 0.0134	< 0.0134	< 0.0134	< 0.00894	< 0.0894	< 0.0224
	DP4-8-9		8.0 - 9.0	NS	< 1	--	--	--	--	< 0.0151	< 0.0151	< 0.0151	< 0.0101	< 0.101	< 0.0252
	DP4-13-14		13.0 - 14.0	NS	< 1	< 5.61	< 48.7	< 97.5	< 0.0196	**0.0511**	< 0.0168	< 0.0168	< 0.0112	< 0.112	< 0.0280
DP-5	DP5-9-10	12/21/2023	3.0 - 4.0	SS	< 1	< 4.80	< 53.1	< 106	< 0.0168	< 0.0144	< 0.0144	< 0.0144	< 0.00961	< 0.0961	< 0.0240
DP-6	DP6-9-10	12/21/2023	9.0 - 10.0	NS	< 1	< 4.70	< 50.2	< 100	--	--	--	--	--	--	--
	DP6-11-12		11.0 - 12.0	NS	< 1	< 4.81	< 53.9	< 108	--	--	--	--	--	--	--
DP-7	DP7-1-2	12/21/2023	1.0 - 2.0	NS	< 1	--	< 44.5	< 88.9	--	--	--	--	--	--	--
	DP7-7-8		7.0 - 8.0	SS	< 1	<4.73	< 46.6	< 93.3	--	--	--	--	--	--	--
	DP7-14-15		14.0 - 15.0	NS	< 1	< 5.96	< 57.9	< 116	< 0.0209	< 0.0179	< 0.0179	< 0.0179	< 0.0119	< 0.119	< 0.0298
DP-8	DP8-4-5	12/21/2023	4.0 - 5.0	SS	1.0	< 5.95	< 50.0	< 99.9	--	--	--	--	--	--	--
	DP8-12-13		12.0 - 13.0	NS	< 1	< 4.69	< 52.1	< 104	--	--	--	--	--	--	--
Direct-push borings sampled by GeoEngineers April 4, 2024															
DP-9	DP-9-3.0	4/4/2024	3.0	NS	<1	--	--	--	--	<0.0126	<0.0126	<0.0126	<0.00840	--	<0.0210
	DP-9-8.0		8.0	NS	1.2	--	--	--	--	<0.0110	<0.0110	<0.0110	<0.00732	--	<0.0183
DP-10	DP-10-5.0	4/4/2024	5.0	NS	<1	--	--	--	--	<0.0131	<0.0131	<0.0131	<0.00871	--	<0.0218
DP-11	DP-11-5.0	4/4/2024	5.0	NS	1.0	--	--	--	--	<0.0126	<0.0126	<0.0126	<0.00840	--	<0.0210
	DP-11-8.0		8.0	NS	<1	--	--	--	--	<0.0217	<0.0217	<0.0217	<0.0144	--	<0.0361
DP-12	DP-12-5.0	4/4/2024	5.0	NS	<1	--	--	--	--	**0.0150**	<0.0132	<0.0132	<0.00879	--	<0.0220
	DP-12-12.5		12.5	NS	<1	--	--	--	--	**0.0450**	<0.0128	<0.0128	<0.00850	--	<0.0213
DP-13	DP-13-5.0	4/4/2024	5.0	NS	<1	--	--	--	--	<0.0113	<0.0113	<0.0113	<0.00756	--	<0.0189
	DP-13-10.0		10.0	NS	<1	--	--	--	--	<0.0129	<0.0129	<0.0129	<0.00862	--	<0.0216
DP-14	DP-14-5.0	4/4/2024	5.0	NS	<1	--	--	--	--	<0.0127	<0.0127	<0.0127	<0.00849	--	<0.0212
	DP-14-10.0		10.0	NS	1.2	--	--	--	--	**0.0207**	<0.0142	<0.0142	<0.00947	--	<0.0237
	DP-14-15.0		15.0	NS	1.3	--	--	--	--	**0.0234**	<0.0126	<0.0126	<0.00837	--	<0.0209
DP-15	DP-15-5.0	4/4/2024	5.0	NS	1.0	--	--	--	--	<0.0117	< 0.0117	< 0.0117	<0.00778	--	<0.0194
	DP-15-10.0		10.0	NS	<1	--	--	--	--	**0.0138**	<0.0126	<0.0126	<0.00841	--	<0.0210
DP-16	DP-16-5.0	4/4/2024	5.0	NS	<1	--	--	--	--	<0.0117	< 0.0117	< 0.0117	< 0.00780	--	<0.0195
MTCA Method A or Method B Cleanup Levels for Unrestricted Land Use[6]						30/100[7]	2,000	2,000	0.03	0.05	0.03	160	1,600	4,000	0.67

Notes:

[1]Approximate exploration locations shown on Figure 2.

[2]Field screening methods are described in Appendix A.

[3]Gasoline-range hydrocarbons analyzed by Northwest Method NWTPH-Gx.

[4]Diesel- and heavy oil-range hydrocarbons analyzed by Northwest Method NWTPH-Dx.

[5]Volatile organic compounds (VOCs) analyzed by U.S. Environmental Protection Agency (EPA) Method 8260.

[6]Model Toxics Cleanup Act (MTCA) Method A and B cleanup levels derived from Ecology's "CLARC Master Spreadsheet.xlsx" dated August 2023.

[7]When benzene is present, the gasoline range cleanup level is 30 mg/kg. When benzene is not present the gasoline range cleanup level is 100 mg/kg.

bgs = below pre-construction ground surface.

mg/kg = milligrams per kilogram

NS = no sheen

SS = slight sheen

ppm = parts per million

< = Analyte not detected at a concentration greater than the indicated laboratory reporting limit.

– = not tested

Bolding indicates analyte was detected.

Shading indicates that concentration exceeded Model Toxics Control Act (MTCA) cleanup level.

Chemical analytical testing by Fremont Analytical in Seattle, Washington. Laboratory analytical reports in Appendix B.

Table 2

Groundwater Chemical Analytical Data (Petroleum Hydrocarbons and VOCs)

Laurel Village Property

4200 Mary Gates Memorial Drive

Seattle, Washington

Sample Location[1]	Sample ID	Sample Date	Depth to Groundwater (from TOC)	Groundwater Elevation	Petroleum Hydrocarbons (µg/L) Gasoline Range[2]	Diesel Range[3]	Heavy Oil Range[3]	VOCs[4] (µg/L) Benzene	Tetrachloroethene	Trichloroethene	cis-1,2-Dichloroethene	trans-1,2-Dichloroethene	1,1-Dichloroethene	Vinyl Chloride	Chloroform	Other VOCs[5]
Sampled December 29, 2023																
GEI-1	GEI-1-231229	12/29/2023	2.51	37.04	< 50	< 100	< 100	< 0.440	< 0.350	< 0.400	< 0.500	< 0.350	< 0.500	< 0.200	< 0.500	ND
GEI-2A	GEI-2-231229	12/29/2023	2.51	34.24	< 50	< 100	< 100	< 0.440	< 0.350	< 0.400	< 0.500	< 0.350	< 0.500	< 0.200	< 0.500	ND
GEI-3	GEI-3-231229	12/29/2023	24.70	43.05	< 50	< 100	< 100	< 0.440	**5.58**	< 0.400	< 0.500	< 0.350	< 0.500	< 0.200	**1.58**	ND
Sampled April 4, 2024																
DP-13	DP-13-040424	4/4/2024	--	--	--	--	--	--	<0.500	<0.500	<0.500	<0.500	--	<0.200	--	--
MTCA Method A or Method B Cleanup Level[6]					800/1,000[7]	500	500	5	5	5	16	160	400	0.20	80	Varies

Notes:

[1]Monitoring well location shown on Figure 2.

[2]Gasoline-range hydrocarbons analyzed by Northwest Method NWTPH-Gx.

[3]Diesel- and heavy oil-range hydrocarbons analyzed by Northwest Method NWTPH-Dx.

[4]Volatile Organic Compounds (VOCs) analyzed by U.S. Environmental Protection Agency (EPA) Method 8260D. Refer to laboratory report for individual analytes and detection limits.

[5]Only selected VOCs are shown; refer to laboratory reports in Appendix C for complete list of method analytes and detection limits.

[6]Model Toxics Cleanup Act (MTCA) Method A and B cleanup levels derived from Ecology's "CLARC Master Spreadsheet.xlsx" dated August 2023.

[7]When benzene is present, the gasoline range cleanup level is 800 µg/L; when benzene is not present the gasoline range cleanup level is 1,000 µg/L.

µg/L = micrograms per liter

ND = Not Detected

TOC = top of casing

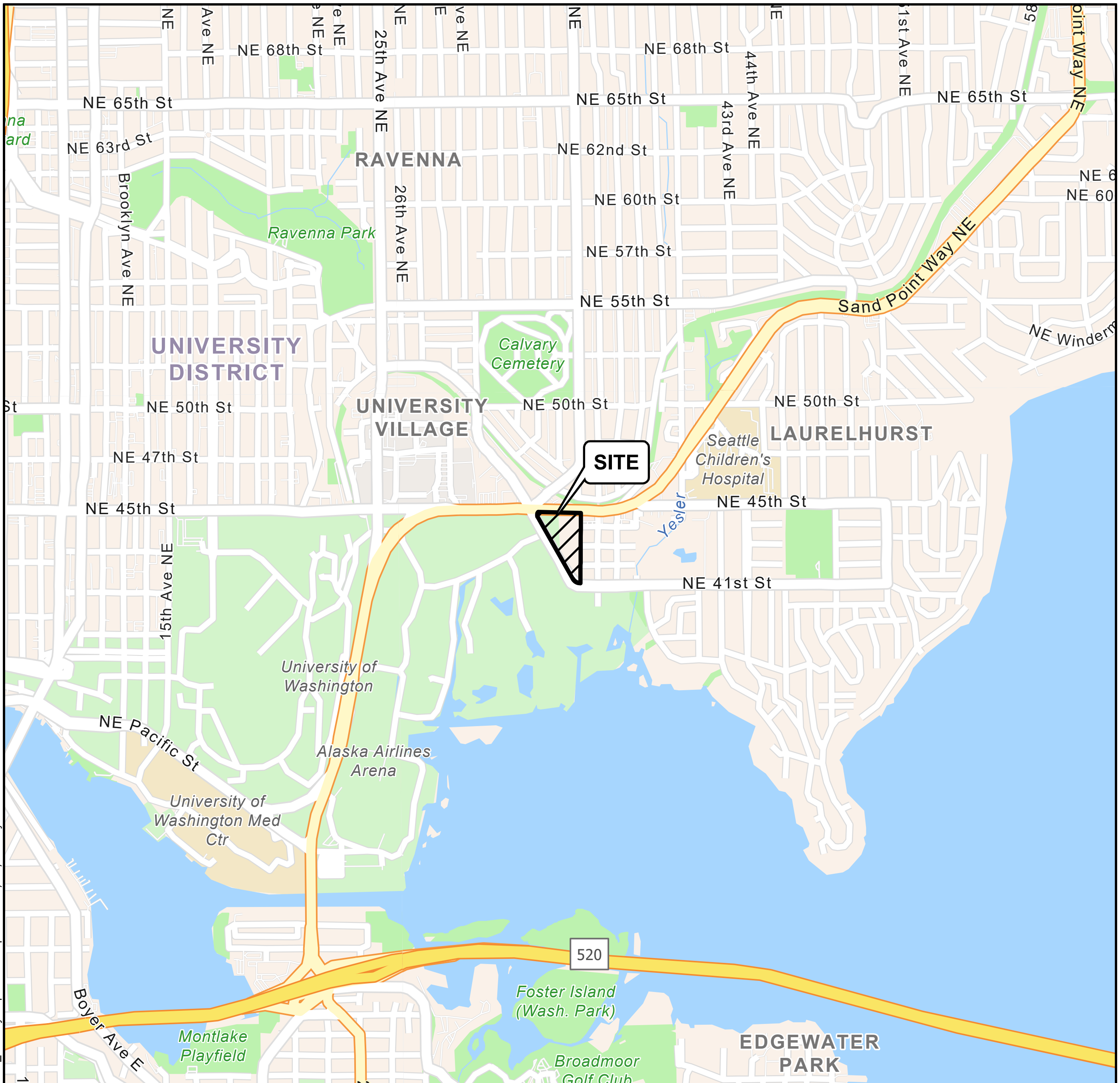
< = Analyte not detected at a concentration greater than the indicated laboratory reporting limit.

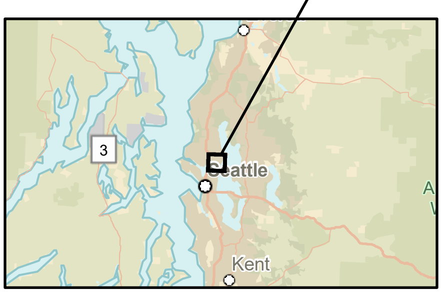
Bolding indicates analyte was detected.

Shading indicates exceedance of Model Toxics Control Act (MTCA) cleanup value.

Chemical analytical testing by Fremont Analytical in Seattle, Washington. Laboratory analytical reports in Appendix C.

Figures





N

0 2,000
Feet

Source(s):
- ESRI

Coordinate System: NAD 1983 UTM Zone 10N

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Vicinity Map

UH4 Laurel Village
Seattle, Washington

GeoEngineers

Figure 1

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Legend

Site Boundary

Aproximate Extent of Former Union Bay Shorline 1912 (Baist's surveys of Seattle)

50 Ground Surface Elevation (Feet, NAVD 88)

Proposed 6-Story Structure

Proposed Townhome and Flats Strctures

DP-9 Direct Push Boring by GeoEngineers, Inc., 2024

DP-1 Direct Push Boring by GeoEngineers, Inc., 2023

GEI-3 Monitoring Well by GeoEngineers, Inc., 2023

Soil Containing Concentration of PCE Greater Than MTCA Method A Cleanup Level

Soil Containing Concentration of PCE Greater than Laboratory Reporting Limit

Notes:

1. All concentrations expressed in milligrams per kilogram (mg/kg)
2. All additional HVOCs, including TCE, cis-1,2-DCE, trans-1,2-DCE, and vinyl chloride were less than the laboratory reporting limits.
3. < = Analyte not detected at a concentration greater than the indicated laboratory reporting limit.
4. -- = Analyte not tested.
5. Bolding indicates analyte was detected.
6. Shading indicates that concentration is Greater than the Model Toxics Control Act (MTCA) cleanup level.

Source:

- Aerial from Microsoft Bing

Projection: WA State Plane, North Zone, NAD83, US Foot

N

0 120

Feet

Soil Chemical Analytical Results

UH4 Laurel Village
Seattle, Washington

GeoEngineers

Figure 2

P:\20\20449013\CAD\01\Groundwater analysis - 2024\2044901301_F02-F03_Site Plans.dwg 2 Date Exported:6/19/2024 5:48 PM - by Mike C. Rau

GEI-1	
Sample Date	Dec. 29, 2023
TPH-G	<50
TPH-D	<100
TPH-O	<100
Benzene	<0.440
PCE	<0.350
TCE	<0.400
cis-1,2-DCE	<0.500
trans-1,2-DCE	<0.350
Vinyl chloride	<0.200

GEI-2A	
Sample Date	Dec. 29, 2023
TPH-G	<50
TPH-D	<100
TPH-O	<100
Benzene	<0.440
PCE	<0.350
TCE	<0.400
cis-1,2-DCE	<0.500
trans-1,2-DCE	<0.350
Vinyl chloride	<0.200

DP-13-040424	
Sample	April, 4, 2024
PCE	<0.500
TCE	<0.500
cis-1,2-DCE	<0.500
trans-1,2-DCE	<0.500
Vinyl chloride	<0.200

GEI-3	
Sample Date	Dec. 29, 2023
TPH-G	<50
TPH-D	<100
TPH-O	<100
Benzene	<0.440
PCE	**5.58**
TCE	<0.400
cis-1,2-DCE	<0.500
trans-1,2-DCE	<0.350
Vinyl chloride	<0.200

Legend

- Site Boundary
- Aproximate Extent of Former Union Bay Shorline 1912 (Baist's surveys of Seattle)
- 34 Groundwater Contour (Feet, NAVD 88) (April 4, 2024)
- 50 Ground Surface Elevation (Feet, NAVD 88)
- Proposed 6-Story Structure
- Proposed Townhome and Flats Strctures
- DP-1 Direct Push Boring by GeoEngineers, Inc., 2023
- GEI-3 Monitoring Well by GeoEngineers, Inc., 2023
- Flow Direction

Notes:
1. All concentrations expressed in micrograms per liter (µg/L)
2. < = Analyte not detected at a concentration greater than the indicated laboratory reporting limit.
3. -- = Analyte not tested.
4. Bolding indicates analyte was detected.
5. Shading indicates that concentration exceeded Model Toxics Control Act (MTCA) cleanup level.

Source:
- Aerial from Microsoft Bing

Projection: WA State Plane, North Zone, NAD83, US Foot

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N

0 120
Feet

Groundwater Analytical Results and Groundwater Contours

UH4 Laurel Village
Seattle, Washington

GEOENGINEERS

Figure 3

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Appendices