Blakeley Village
SEPA Consistency Checklist

Purpose
The purpose of this consistency memorandum and checklist is to document the relationship of the proposed Blakeley 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 Blakeley Village project is being proposed in development sites E81 and E82 (see **Exhibits A** and **B**) of the campus to provide additional student apartment housing. The project would be approximately 460,000 square feet, taking the place of the existing Blakeley Village apartments and the Gilman Building for a net increase of approximately 367,340 square feet. The building would include student resident apartments, student social space, supporting offices, and storage. Parking would be provided by the adjacent underutilized Nordheim Court parking garage. **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' maximum height and building setbacks from the Burke-Gilman Trail will meet design guidance. The total maximum gross square feet of 225,000 will be supplemented by transferring approximately 142,340 square feet from other available development sites in the East Campus Sector as allowed in the Campus Master Plan by an exempt change. The proposed project does not have mid-block corridors or open space commitments.

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 one Environmentally Critical Areas (ECAs): Steep Slopes. The project will not disturb the existing steep slope adjacent to the Burke-Gilman Trail. 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 academic and student 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 Burke-Gilman Trail. 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 shows up to 30 Tier 2 trees and 52 Tier 3 and 4 trees identified for potential removal. Trees removed will be replaced at a 2:1 ratio; as many as practical on the site and the rest elsewhere on campus.

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 – No risk to human health from the project is anticipated. 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. The site was identified as "Low" potential to encounter sensitive conditions.

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 eliminate approximately 87 of the existing 92 parking stalls located onsite. The proposed project anticipates 5 parking stalls for ADA and load/unload stalls. The residents choosing to bring a vehicle would use some of the available capacity in the adjacent Nordheim Court garage.

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

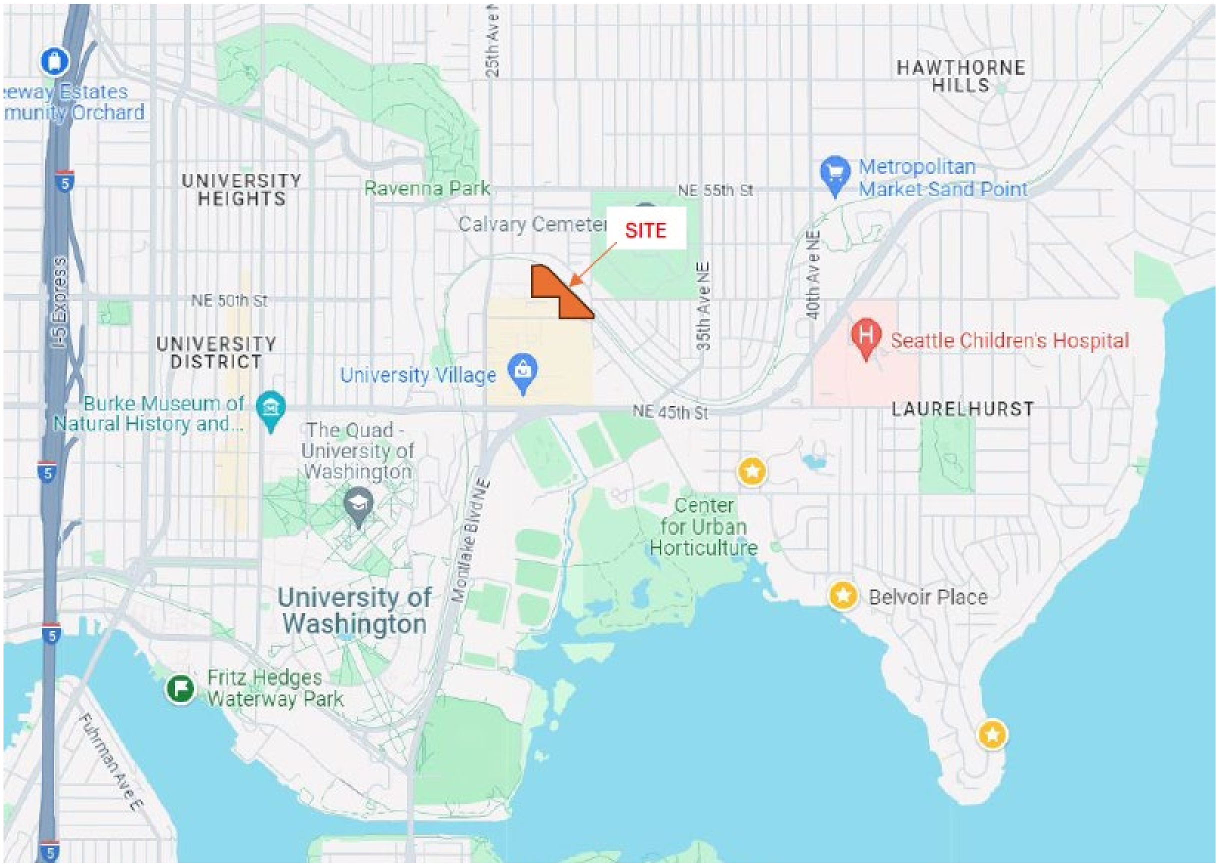

Exhibit B – 2019 Seattle Campus Master Plan Development Sites E81 and E82

CAMPUS ARCHITECTURE & PLANNING
UNIVERSITY of WASHINGTON
Facilities

Exhibit C – Proposed Site Configuration

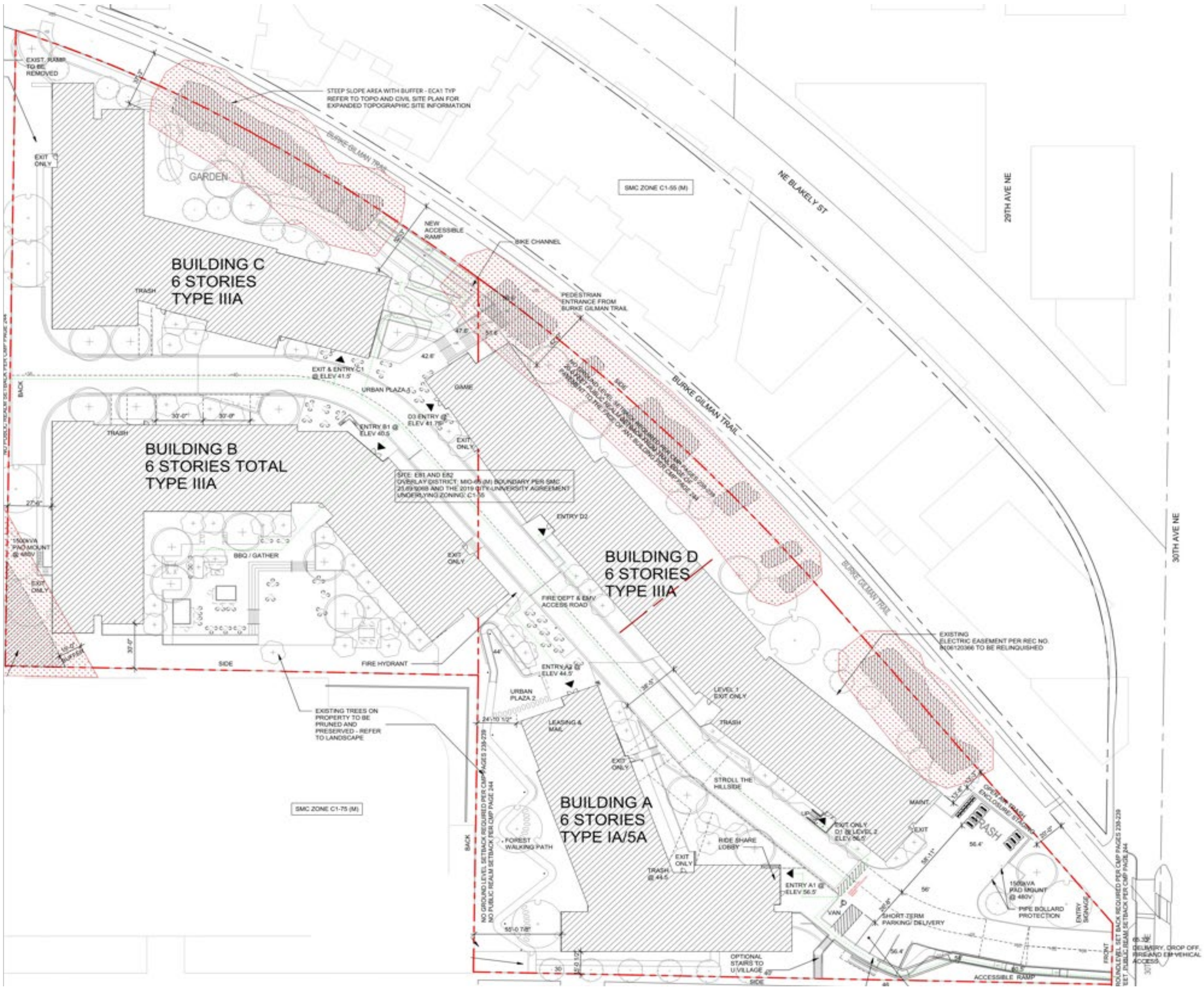

Exhibit D – Geotechnical Report

Geotech Report (Draft)

Geotechnical Due Diligence Services

UH4 Blakeley Village
Seattle, Washington

for
GDSU Washington, LLC

January 17, 2024

GeoEngineers
Earth Science + Technology

Geotechnical Due Diligence Services

UH4 Blakeley Village
Seattle, Washington

for
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GeoEngineers

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Geotechnical Due Diligence Services

UH4 Blakeley Village
Seattle, Washington

File No. 20449-011-00

January 17, 2024

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1.0 INTRODUCTION

This report summarizes the results of GeoEngineers' geotechnical due diligence services for the proposed UH4 Blakeley Village development project located in Seattle, Washington. The site is shown relative to surrounding physical features in Figure 1, Vicinity Map, and Figure 2, Site Plan.

The purpose of this report is to provide preliminary geotechnical engineering conclusions and recommendations for the design and construction of the planned development. The site consists of two (2) King County Tax Parcels (Nos. 092504-9439 and 092504-9438) and covers approximately 5 acres. GeoEngineers' services have been completed in accordance with our consultant agreement with GDSU Washington, LLC executed on November 14, 2023. GeoEngineers' scope of services includes:

- Review 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 preliminary recommendations for seismic design in accordance with the 2018 or 2021 International Building Code (IBC).
- Providing preliminary foundation, 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 preliminary recommendations for temporary dewatering and permanent below-grade drainage and groundwater seepage estimates.
- Provide consultation to the project team, as needed.
- Preparing this report.

2.0 PROJECT DESCRIPTION

GeoEngineers understands that GDSU Washington, LLC (Greystar) is interested in redeveloping 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 Blakeley 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 several new 6-story student housing buildings. The new buildings will generally be constructed at-grade. The ground floor finished floor level was not available at the time this report was prepared.

There is a gentle slope across the site which will require some excavation into the hillside. Temporary cut slopes and/or temporary shoring is anticipated to be required to complete the planned excavations. Based on review of exploration logs from our investigation and in the site vicinity, we anticipate that the planned buildings can be supported on shallow foundations where bearing soils are within five feet of the planned subgrade. Where the depth to bearing soils is greater than five feet, buildings will need to be supported on deep foundations. The relatively shallow groundwater at the site may preclude overexcavation of weak soils.

The need for and extent of deep foundations will be confirmed during design once the foundation and lowest finished floor elevations have been determined.

3.0 FIELD EXPLORATIONS AND LABORATORY TESTING

3.1. Field Explorations

Subsurface conditions at the site were evaluated by drilling five borings (GEI-1 through GEI-5). The borings extended to depths between 35½ 36½ 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 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. Previous Site Evaluations

We reviewed the logs of selected explorations from previous site evaluations in the project vicinity, which are presented in Appendix C, 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 Blakeley Village site is bounded by the Burke Gilman Trail along the north and northeast, 30th Avenue NE on the east, University Village on the south and southwest, and the Nordheim Court Apartments and Silver Cloud Hotel on the west. The site is currently occupied by a multifamily student housing complex with several wood-framed buildings that were constructed in the 1980s. On the eastern edge of the subject property is the former University of Washington Center for Leadership in Athletics at the Gilman Building, which is a three-story wood-framed building constructed in the 1960s. The site also includes asphalt paved parking and drive aisles along with landscaped areas with play structures. Existing site grades slope moderately down from northeast to southwest, from approximately Elevation 60 to 64 feet along the Burke Gilman Trail down to Elevation 28 to 34 feet near the University Village parking garage.

The subject property is designated as an Environmentally Critical Area (ECA) for steep slopes and a liquefaction-prone area in accordance with the Seattle Municipal Code (SMC) Chapter 25.09. The approximate extents of the ECA zones are shown on Figure 2. The site is immediately adjacent to the University Village mall, which is designated as a peat settlement prone ECA. The site is north of the former Union Bay, which was a peat marshland. 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 and recent deposits overlying glacially consolidated soils. The fill generally consists of medium dense sand with variable silt and gravel content. The thickness of the fill encountered at the site ranges from 4.5 to 14.5 feet.

The recent deposits generally consist of medium dense sand with little silt/clay. Recent deposits were encountered within borings GEI-4 and SW-1. In general, the recent deposits are 4 to 9 feet thick. Within Boring GEI-4, the recent deposits extend down approximately 13.5 feet, which corresponds to Elevation 21.5 feet.

The glacially consolidated soils were encountered below the fill and/or recent deposits and extend to the depths explored. The glacially consolidated soils consist of dense to very dense cohesionless sand and gravel and till-like deposits. 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 conditions observed are representative of the regional groundwater table. Groundwater appears to be confined locally by the till-like deposits, as the top of the water bearing soils is lower than the measured potentiometric surface. Groundwater monitoring should continue during the design phase of the project to observe seasonal fluctuations.

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	45	44.62	12/26/2023	1.0	43.62
			12/29/2023	1.0	43.62
GEI-2	34	N/A	12/26/2023	8.0	26.0
			12/29/2023	3.95	30.05
GEI-3	45	44.5	12/26/2023	1.32	43.18
GEI-4	35	34.57	12/26/2023	2.63	31.94
GEI-5	52	51.60	12/26/2023	16.50	35.10
			12/29/2023	16.57	35.03

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, and liquefaction-prone area. The site is not designated as a peat settlement prone area, but is immediately adjacent to University Village, which has the peat ECA.

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 steep slopes are mapped along the Burke Gilman Trail, which was formerly the Northern Pacific railway line originally constructed in 1885. The railway embankment and resulting steep slope condition was created using legal grading. 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. 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.

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 or 2021 IBC due to the liquefiable soils below the site. We expect that the planned structures will have a fundamental period of vibration of less than 0.5 seconds, and therefore, the exception of ASCE 7-16 Section 20.3.1 can apply for this project. If the building period is greater than 0.5 seconds, a site response analysis will be required by the provisions of ASCE 7-16 Chapter 11.
- Based on the groundwater levels measured during our investigation and as presented on Figure 4, groundwater slopes down from the northeast to the southwest towards University Village. GeoEngineers recommends that the design groundwater table elevation be taken as the contours presented on Figure 4. We understand that the planned development will generally be constructed at-grade above

the design groundwater table with limited cuts into the hillside. There will likely be some seepage for these hillside cuts which we expect can be managed using sumps and pumps. Permanent drainage is not likely permissible; therefore, the structure would need to be designed to resist hydrostatic pressures below the design groundwater table elevation. Careful consideration will be needed if portions of the planned development extend below the design groundwater table and are in close proximity to University Village, which is designated as having a peat settlement prone ECA.

- 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, shallow foundations are recommended at the northeastern portion of the project site and deep foundations are recommended for the southwestern 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.
- 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 kips per square foot (ksf).
- Augercast piles are the preferred deep foundation system. For design, we preliminarily 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 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).
- Where the building is supported on deep foundations, a structural slab is recommended to mitigate liquefaction-induced settlement. The structural slab should also be underlain by a 6-inch-thick layer of clean crushed rock.

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 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 1¾ 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. 2018 or 2021 IBC Seismic Design Information

The project site is Site Class F due to the presence of liquefaction. Per American Society of Civil Engineers (ASCE) 7-16, Site Class F requires performing a site-specific site response analysis. However, we expect that the proposed structures will have a fundamental period of vibration of less than 0.5 seconds. Therefore, the exception of ASCE 7-16 Section 20.3.1 can apply for this project. If the building period is greater than 0.5 seconds, a site response analysis will need to be performed.

We recommend using the following 2018 and 2021 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. Please note that the Site Class F designation and associated requirements of ASCE 7-16 Chapter 12 still apply.

TABLE 2. ASCE 7-16 MAPPED SEISMIC DESIGN PARAMETERS

ASCE 7-16 Parameter[1,2]	Recommended Value
Site Class	F
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]
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 47.665137and longitude - 122.2982 using the ASCE 7 Hazards online tool (https://asce7hazardtool.online/).

[2] These values are valid for structures with fundamental periods less than 0.5 seconds.

MCE_R – risk-targeted maximum-considered earthquake

6.2. Temporary Dewatering

Localized dewatering for relatively small excavations that extend below the groundwater table (for instance elevator pits, foundation elements, stairwells/ramps or limited sidewalk setbacks) will likely be needed for planned development. Temporary dewatering should be completed in a manner that does not cause adverse impacts to existing improvements located offsite. In such instances, casual dewatering using sumps and pumps or a localized vacuum wellpoint system is anticipated. Temporary dewatering should be reviewed with the project team during the design after excavation depths and locations are more fully defined.

6.3. Excavation Support

We understand that the planned buildings will either be constructed at-grade or extend partially below grade due to sloping site conditions. For preliminary design, excavations are anticipated to be completed using temporary cut slopes, where feasible, or by using temporary shoring consisting of soldier pile and tiebacks.

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 preliminary 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.

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 5, Earth Pressure Diagrams — Temporary Soldier Pile & Tieback Walls. The earth pressures presented in Figure 5 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 5 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 6, Recommended Surcharge Pressure. No seismic pressures have been included in Figure 5 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 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 3. 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 5) 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.

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/recent 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 D.

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 D 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 project site.

Due to the variable soils present at the foundation subgrade elevation, shallow foundations are recommended at the northeastern portion of the project site and deep foundations are recommended for the southwestern 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 3) to assist the project team with determining where shallow foundations and deep foundations should be used.

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 mat 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 structural mat foundations bearing on 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 core mat(s) 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.

6.4.2. 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 many existing buildings and improvements in the vicinity.

For planning purposes, we suggest that the project team consider 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 3. For 18-inch-diameter piles, we preliminarily recommend an ultimate pile capacity of 200 kips. We can assess allowable pile axial and lateral capacities (including assessment of downdrag and seismic loading) during the final stage of design.

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 should consider the estimated liquefaction-induced settlement along the southwestern portion of the site of up to 1¾ inches. If the slab cannot accommodate this estimated settlement, the slab should be designed as a structural slab. Along the northern and eastern portions of the site, the slab may be designed as bearing on grade.

6.5.1. Subgrade Preparation

If 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 understand that the planned building finished floor will be above the annual high static groundwater level. We recommend installing a capillary break layer to limit the potential for capillary rise below the slabs. The capillary break layer should consist of a 6-inch layer of Mineral Aggregate Type 22, City of Seattle Standard Specification 9-03.14, or an alternative approved by GeoEngineers.

If no special waterproofing measures are taken, leaks and/or seepage may occur in localized areas of the below-grade portion of the building. If leaks or seepage is undesirable, below-grade 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.

6.6. 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 7. 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 6. 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.

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, Washington State Department of Transportation (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. Earthwork

6.7.1. Subgrade Preparation

Exposed subgrade in structure and hardscape areas should be evaluated after site excavation is complete. Foundation subgrades should be prepared as recommended in "Shallow Foundations" above. Where hardscape 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.

6.7.2. 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.

6.7.2.1. 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 8 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 as structural fill 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. On-site soils may be used as general fill outside building footprints and planned flatwork.

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.

6.7.2.2. 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.

6.7.2.3. 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.

6.7.3. 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.

6.8. Recommended Additional Geotechnical Services

GeoEngineers will complete a design-level engineering report for the project during the design phase of the project. GeoEngineers should also 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 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 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 Blakeley 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

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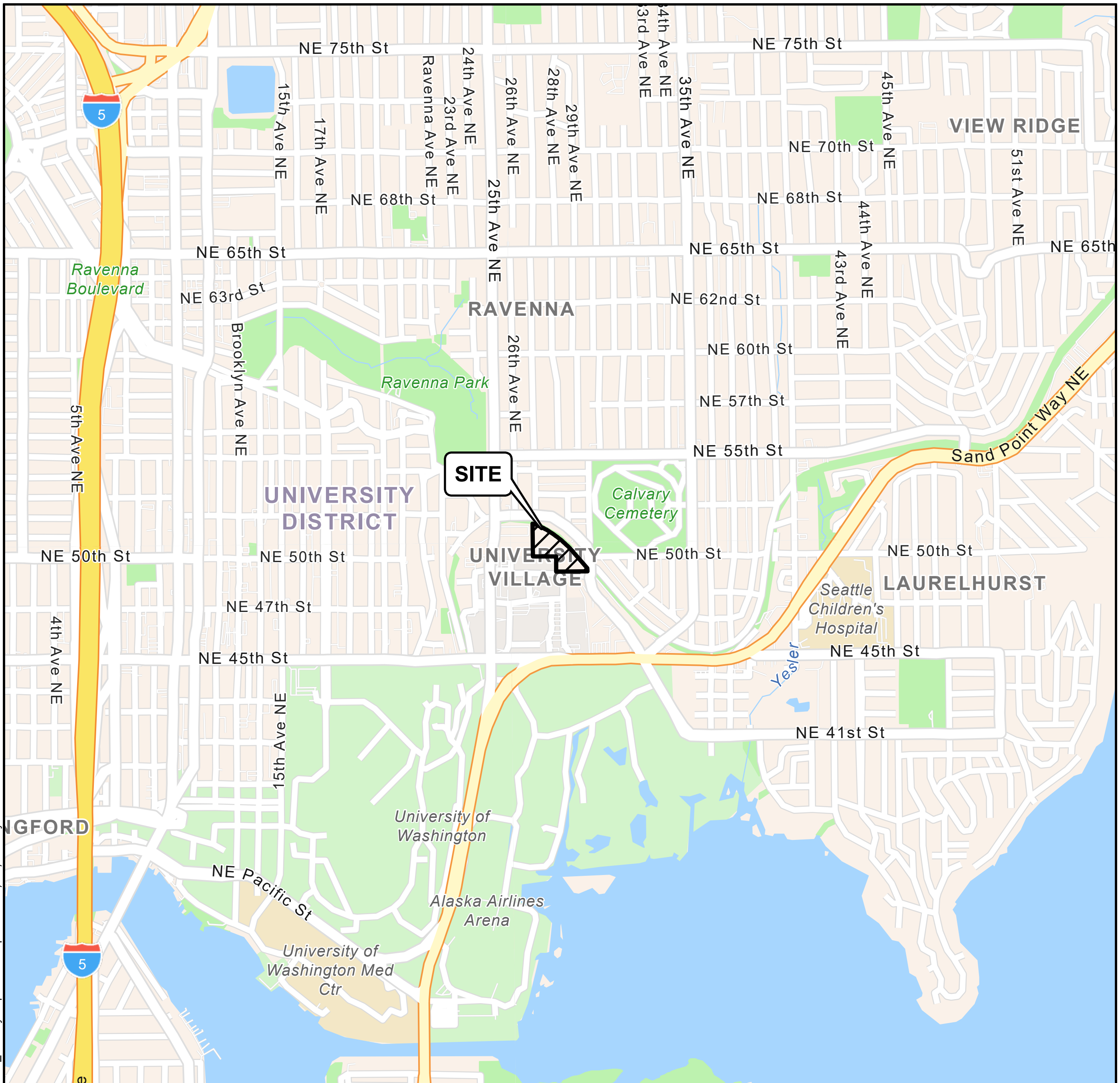

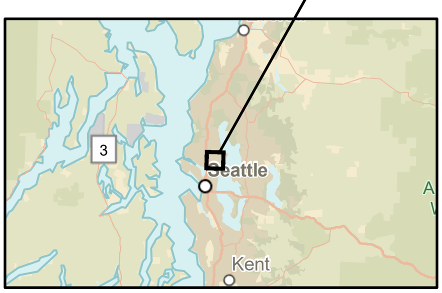

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Feet

Source(s):
- ESRI

Coordinate System: NAD 1983 UTM Zone 10N

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Vicinity Map

UH4 Blakeley Village
Seattle, Washington

GeoEngineers

Figure 1

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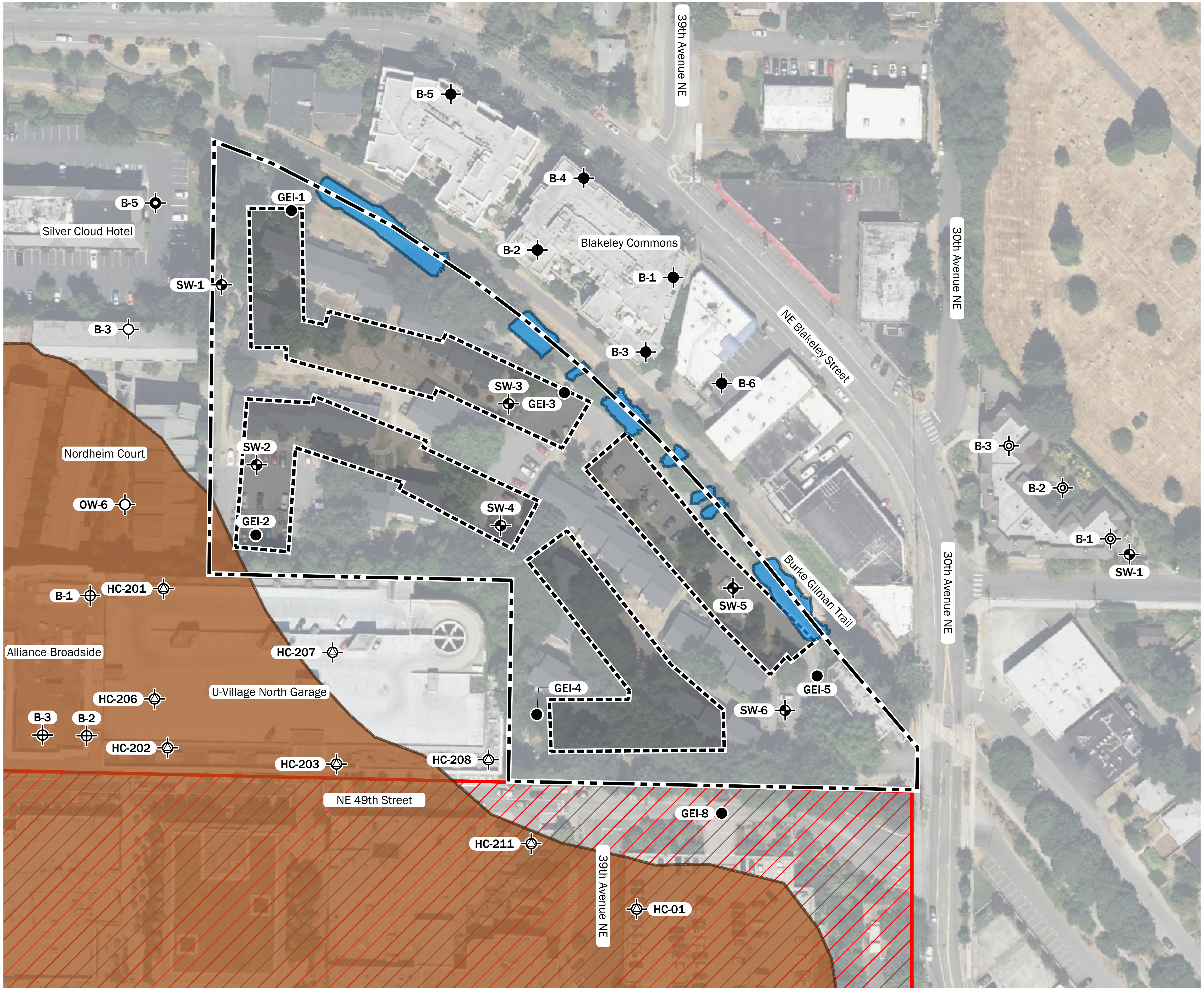

Legend

- Site Boundary
- Proposed Building Footprint
- GEI-1 Monitoring Well by GeoEngineers, Inc., 2023
- SW-1 Boring by Shannon and Wilson, 2022
- B-1 Boring by Pangeo, 2019
- GEI-1 Boring by GeoEngineers, Inc., 2012
- HC-111 Boring by Hart Crowser, 2001
- B-1 Boring by Earth Consultants, 2001
- B-3 Boring by AMEC, 2001
- B-1 Boring by Earth Consultants, 1997
- B-5 Boring by Earth Consultants, 1987
- 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

Source(s):

- Aerial from Microsft Bing

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

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Site Plan

UH4 Blakeley Village
Seattle, Washington

GEOENGINEERS

Figure 2

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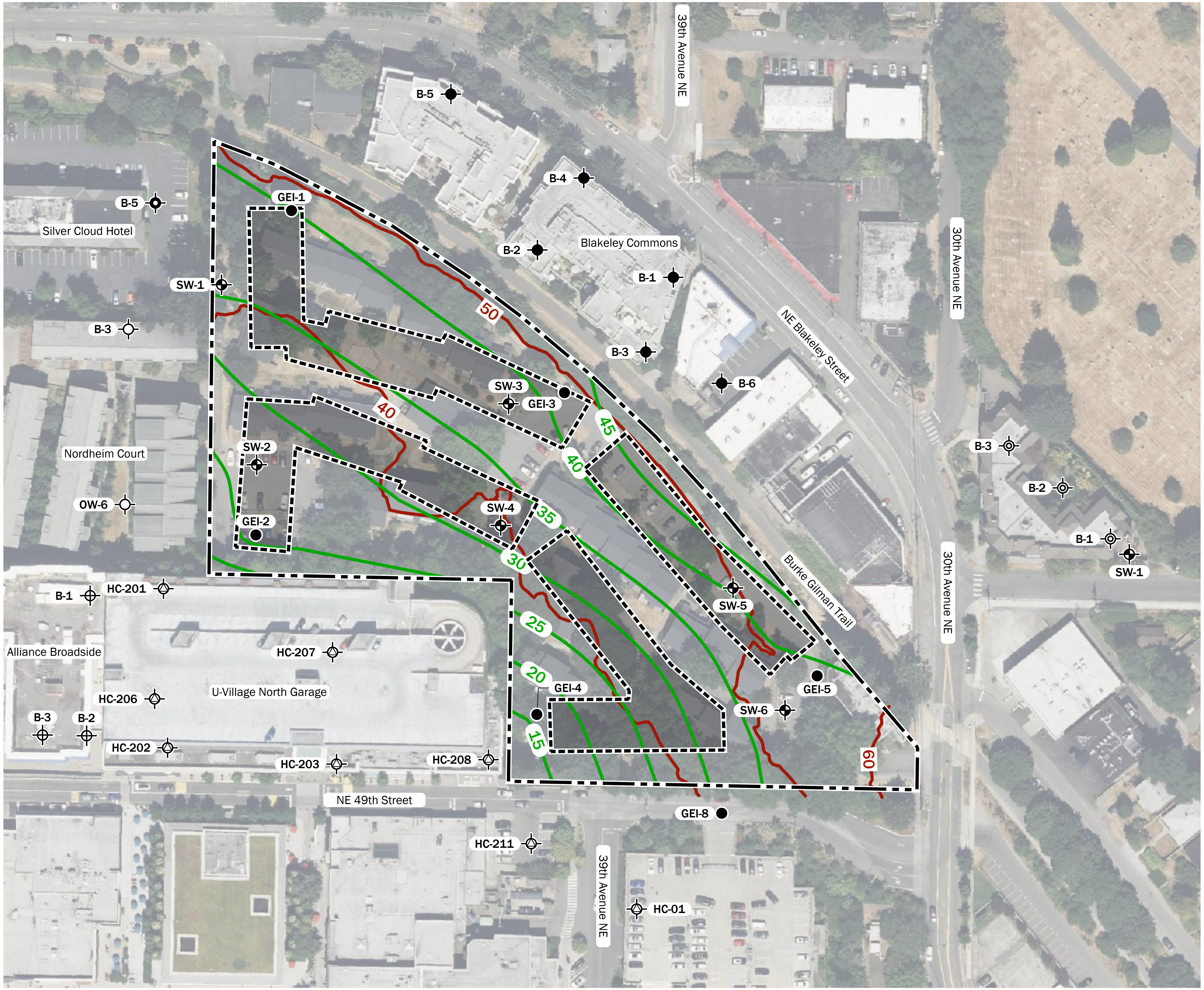

Legend

- Site Boundary
- 15 Bearing Contours (Feet, NAVD 88)
- 40 Ground Surface Elevation Contours (Feet, NAVD 88)
- Proposed Building Footprint
- GEI-1 Monitoring Well by GeoEngineers, Inc., 2023
- SW-1 Boring by Shannon and Wilson, 2022
- B-1 Boring by Pangeo, 2019
- GEI-1 Boring by GeoEngineers, Inc., 2012
- HC-111 Boring by Hart Crowser, 2001
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Source(s):

- Aerial from Microsft Bing

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

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Feet

Bearing Contour Map

UH4 Blakeley Village
Seattle, Washington

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Figure 3

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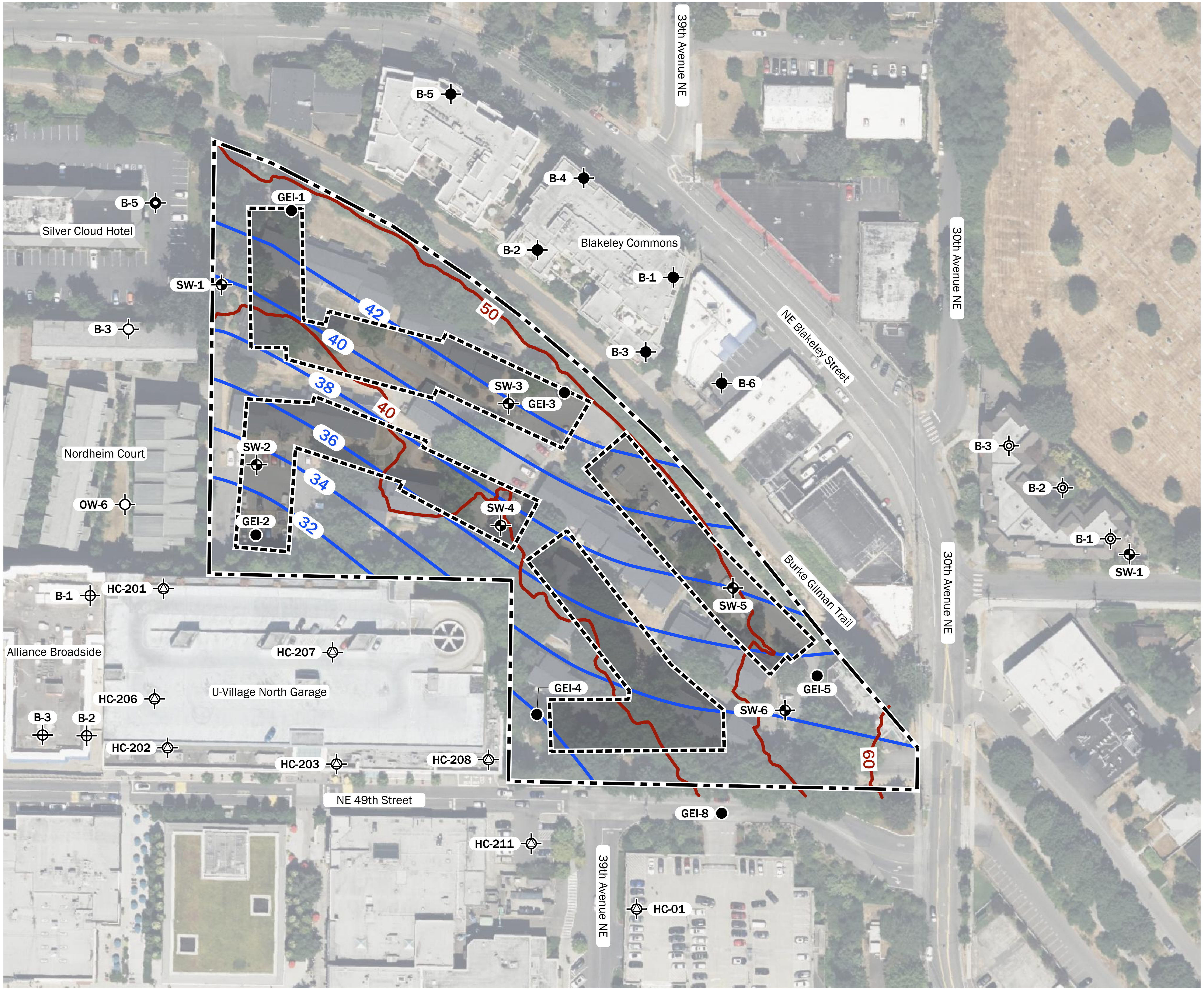

Legend

- Site Boundary
- 34 Groundwater Contour (Feet, NAVD 88)
- 40 Ground Surface Elevation Contours (Feet, NAVD 88)
- Proposed Building Footprint
- GEI-1 Monitoring Well by GeoEngineers, Inc., 2023
- SW-1 Boring by Shannon and Wilson, 2022
- B-1 Boring by Pangeo, 2019
- GEI-1 Boring by GeoEngineers, Inc., 2012
- HC-111 Boring by Hart Crowser, 2001
- B-1 Boring by Earth Consultants, 2001
- B-3 Boring by AMEC, 2001
- B-1 Boring by Earth Consultants, 1997
- B-5 Boring by Earth Consultants, 1987

Source(s):
- Aerial from Microsft Bing

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

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.

N

0 100
Feet

Groundwater Contour Map

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Seattle, Washington

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Figure 4

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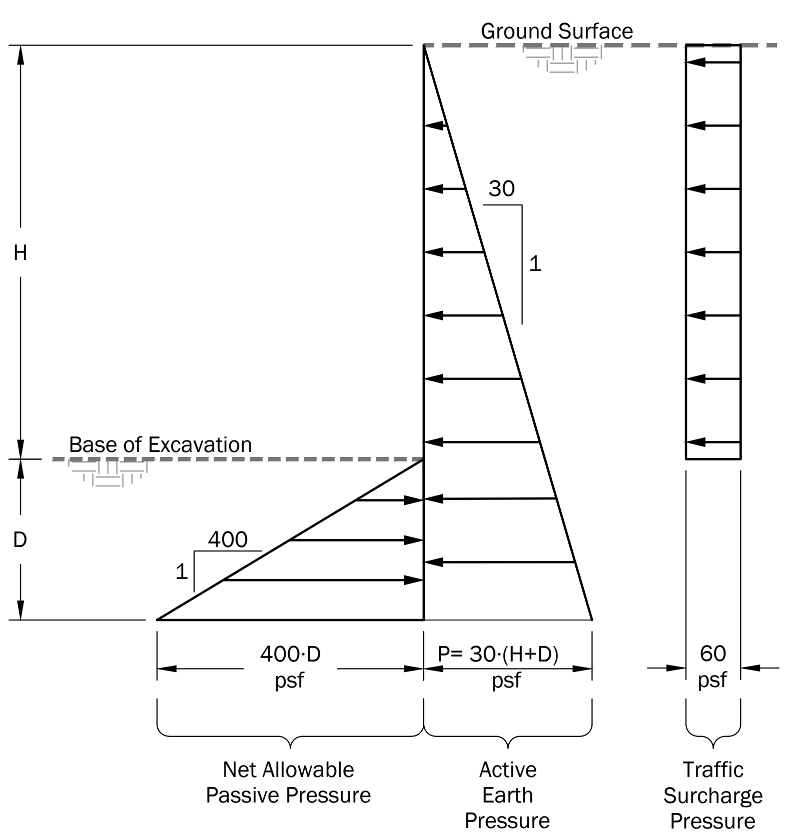
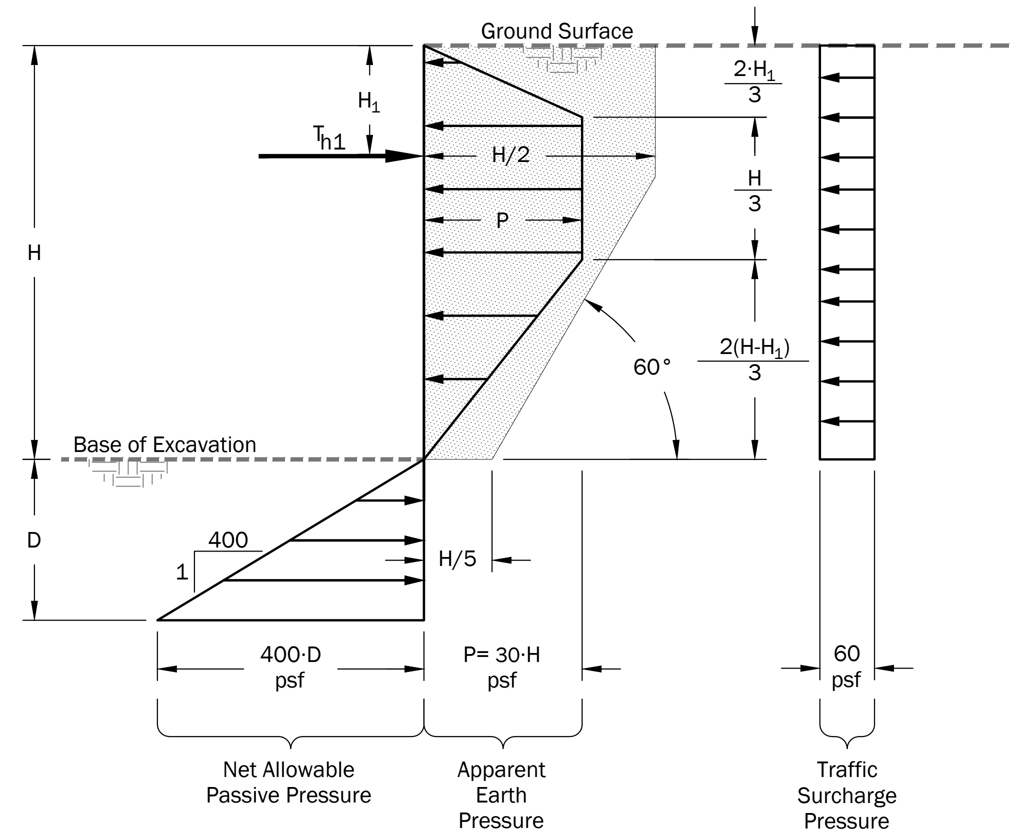
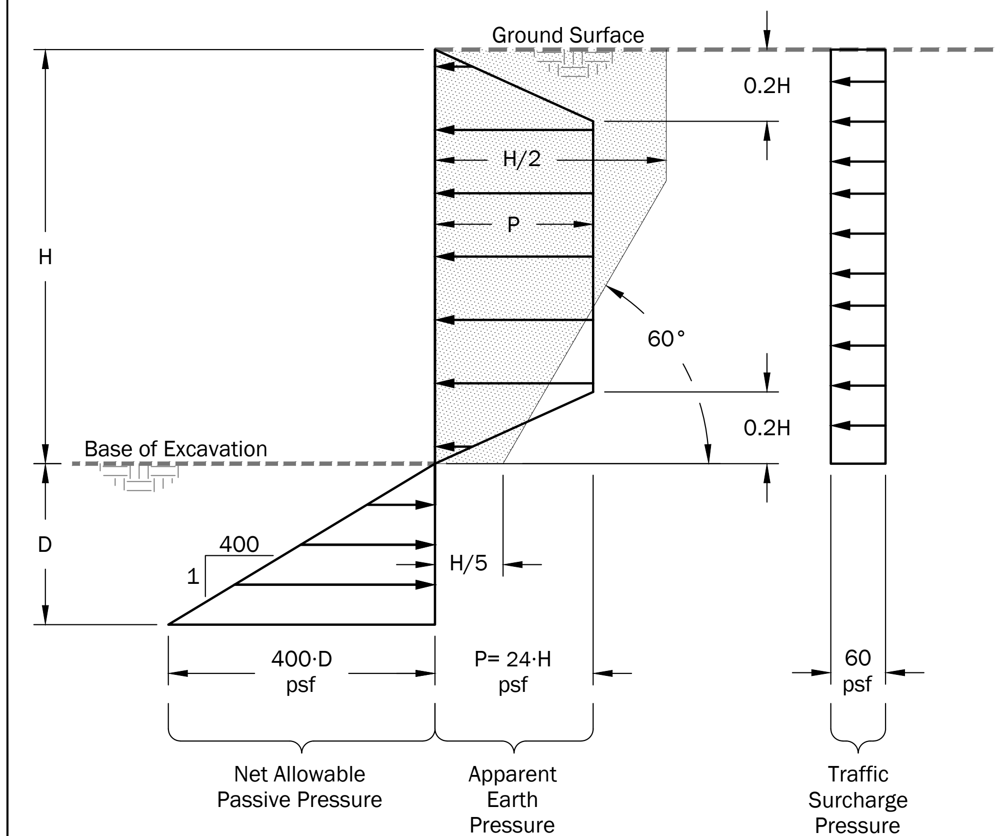
Cantilever Soldier Pile



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

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Seattle, Washington

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Figure 5

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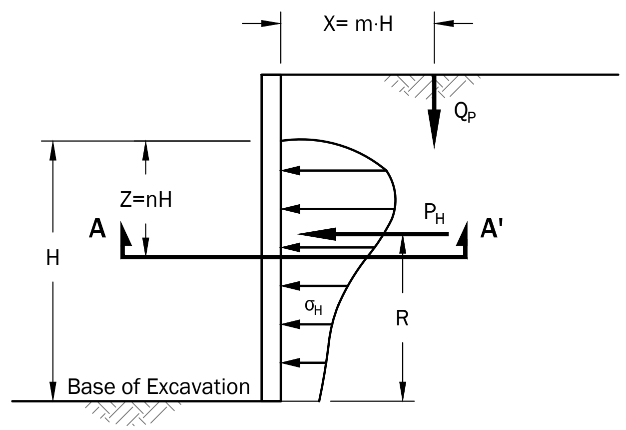
Lateral Earth Pressure from Point Load, Q_P (Spread Footing)



For $m \le 0.4$

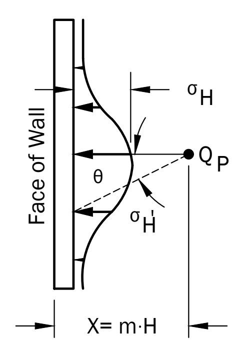
$$\sigma_H = \frac{K \cdot 0.28 Q_P n^2}{H^2(0.16+n^2)^3}$$

For $m > 0.4$

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

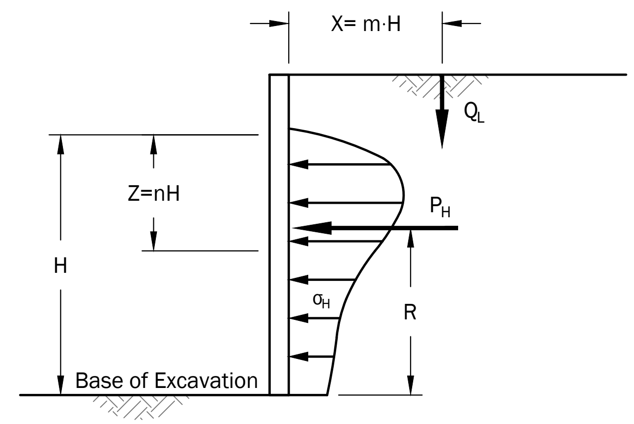
$$\sigma'_H = \sigma \, COS^2 (1.1\theta)$$

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 $m \le 0.4$

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

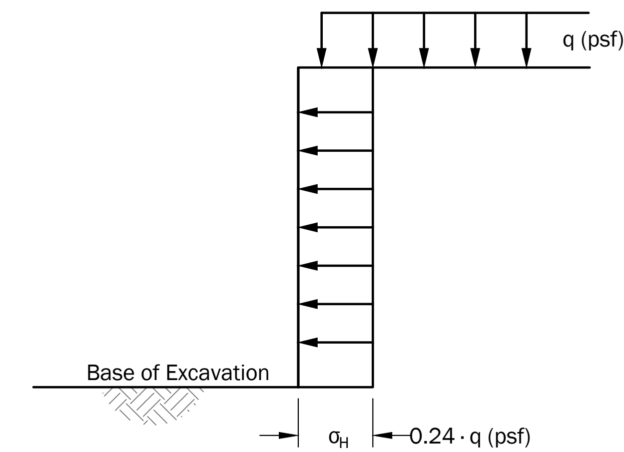
For $m > 0.4$

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

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

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

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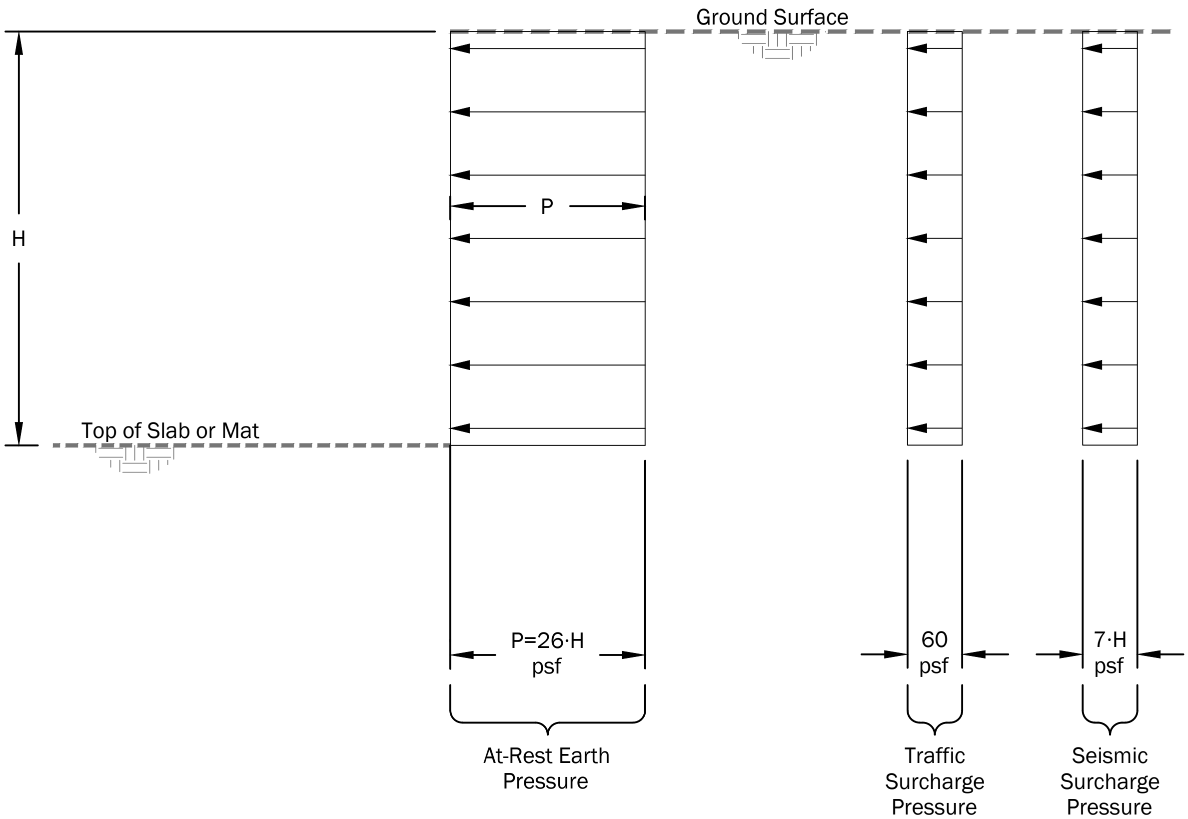
Figure 6

Permanent Basement Wall Against Temporary Shoring



Legend

H = Height of Basement Wall, Feet

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

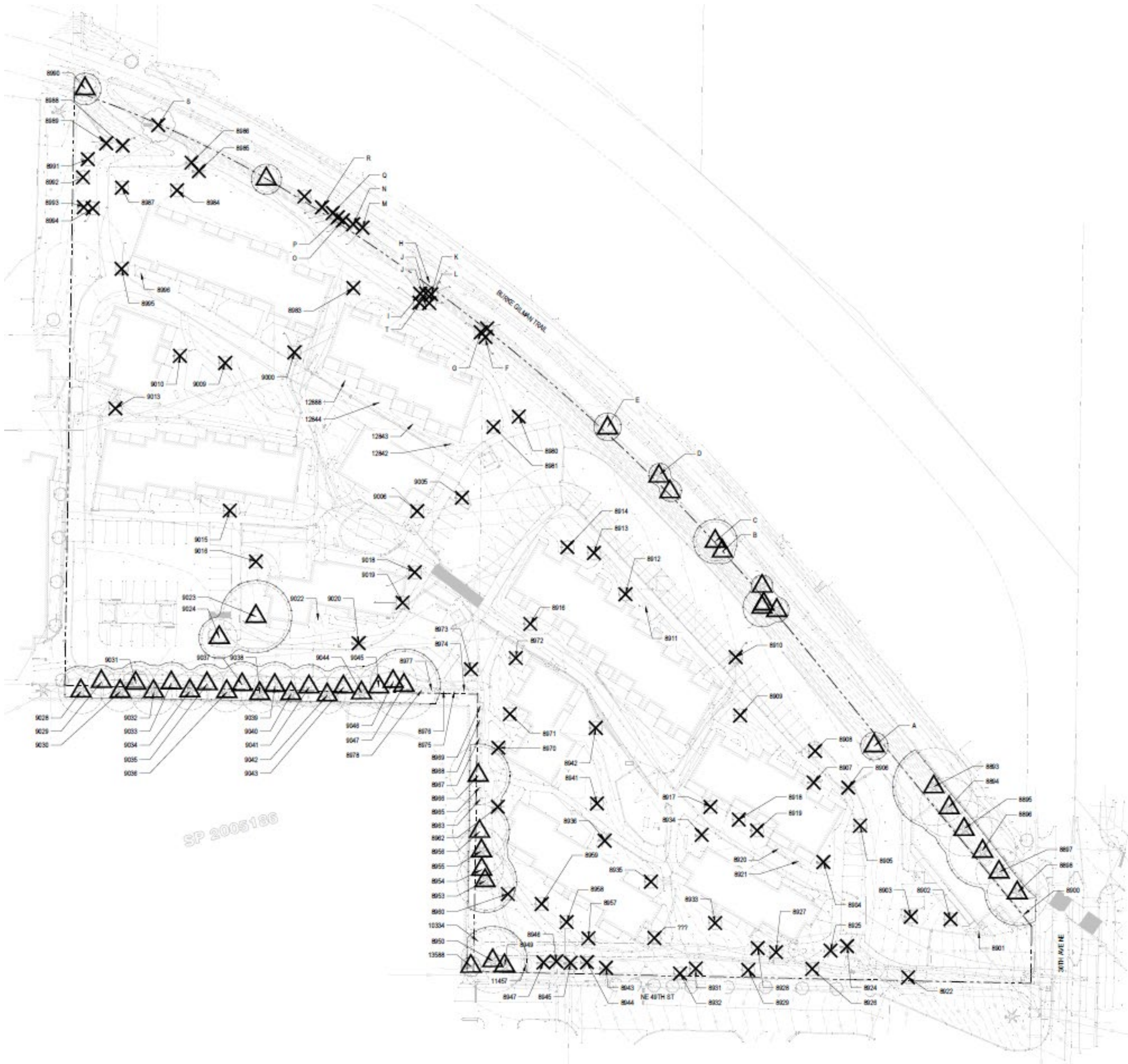
Not To Scale

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 Blakeley Village Seattle, Washington	
GEOENGINEERS	**Figure 7**

Exhibit E – Tree Preservation and Removal



Key:
Diamond shape = preservation
X demarcation = removal