CAMPUS ARCHITECTURE & PLANNING
UNIVERSITY of WASHINGTON
Facilities

November 5, 2021

City of Seattle
Department of Construction & Inspections
700 5th Ave, Suite 2000
Seattle WA 98104

RE: SEPA Lead Agency
Interdisciplinary Engineering Building

Per RCW 43.21C, WAC 197-11 and WAC 478-324-020 through 210, the University of Washington is the Lead Agency responsible for compliance with the State Environmental Policy Act (SEPA) for projects which the University initiates. These rules state that when an agency initiates a proposal, it is the lead agency for the proposal and defines lead agency as the agency with the main responsibility for complying with SEPA's procedural requirements.

Per the SEPA Guidelines, as the SEPA lead agency, the University of Washington has the authority to prepare determinations of exemption, threshold determinations, scoping, preparing and issuance of environmental impact statements, etc.

The SEPA review has been completed for the Interdisciplinary Engineering Building as noted in the SEPA consistency paper stating how the project site has been reviewed with the 2018 Campus Master Plan Final EIS.

Sincerely,

Julie Blakeslee, AICP
University Environmental & Land Use Planner
SEPA Responsible Official

UNIVERSITY OF WASHINGTON
DETERMINATION OF NON-SIGNIFICANCE
ADOPTION OF EXISTING DOCUMENT

Date: November 5, 2021

Lead Agency: University of Washington

Description of Proposal: The proposed University of Washington Interdisciplinary Engineering Building is intended to create a new academic building in Central Campus for student and faculty collaboration spaces, laboratories, classrooms and offices.

Location of proposal, including address, if any: 4000 East Stevens Way NE. The site is generally bounded by the UW Club to the north, UW plant operation buildings and Mason Road NE to the east, Jefferson Road NE to the south, and East Stevens Way NE to the west.

Title of document being adopted by reference: University of Washington 2018 Seattle Campus Master Plan Final Environmental Impact Statement

Date adopted document was prepared: July 2017

Description of document being adopted by reference: The Seattle Campus Master Plan guides development on the Seattle Campus and includes guidelines and policies for new development on the campus. It is formulated to maintain and enhance the fundamental mission of the University; its multiple important roles in undergraduate and professional education, and its dedication to research and public service. The Draft and Final EIS for the master plan analyzed the potential impacts of all identified development sites.

The adopted document is available at: https://facilities.uw.edu/files/media/uw-cmp-final-eis-volume-1.pdf

As lead agency, we have identified and adopted this document as being appropriate for this proposal after independent review. This proposal and site is consistent with the Campus Master Plan. It has been determined that it does not have a probable significant adverse impact on the environment. An environmental impact statement (EIS) is not required under RCW 43.21C.030(2)(c). This decision was made after adoption of the 2018 Seattle Campus Master Plan EIS for the project and preparation and review of a SEPA consistency checklist.

This DNS is issued under WAC 197-11-340(2). The comment period will end November 30, 2021.

Responsible Official: Julie Blakeslee, AICP, Environmental & Land Use Planner
jblakesl@uw.edu

Interdisciplinary Engineering Building

SEPA Consistency Memorandum

Purpose

The purpose of this consistency memorandum and checklist is to document the relationship of the proposed Interdisciplinary Engineering Building 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 Interdisciplinary Engineering Building project is being proposed in development site C11 (**See Exhibit A)** of the campus to provide space that is flexible and adaptable to meet the evolving needs of the College of Engineering. The project would be a new approximately 72,000 square foot building taking the place of two existing UW Facility Buildings and two office trailers. **See Exhibit B.** The building would include a mix of classrooms, project space, student social space, and offices. It would be the primary academic hub for freshmen and sophomore students for engineering education and collaboration.

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 105' maximum height nor the total maximum gross square feet of 85,000. The proposed project does not have ground level building structure setbacks, mid-block corridors, or open space commitments.

Project Consistency with the EIS

The following provides a summary of the relationship of the proposed Interdisciplinary Engineering Building 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 two Environmentally Critical Areas (ECAs): Peat Settlement Prone Area and Historical Landfill (1000-foot buffer). According to the GIS map, the entirety of the University of Washington Seattle campus east of 15th Avenue NE and south of NE 45th Street is mapped as Peat Settlement Prone without much discrimination. Based on our understanding of the geologic setting, topography, and review of soil borings, we did not encounter compressible peat deposits at this site and believe the site is misclassified as a Peat Settlement Prone Area. The site is also mapped within the 1000-foot methane buffer zone from the Montlake Landfill (University of Washington 2017). Because the site is underlain by glacial till over consolidated soils and is significantly upslope from the known extents of the historic landfill, we do not expect project development to be significantly impacted by the site's proximity to this historic landfill and do not recommend any methane mitigation requirements. **See Exhibit C.**

<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 building was chosen to work with the hillside and existing vegetation to retain as many large and Exceptional trees as possible. **Exhibit D** depicts the proposed tree removal and protection plan and shows up to three Exceptional trees identified for potential removal. The reason from removal of each tree is indicated in the exhibit.

Energy Resources – Decreases in electricity and fossil fuel demand is anticipated as the new building will be more efficient than the existing buildings. 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 building. Short-term vibration is anticipated when construction activities occur. 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 building would represent a portion of the projected increase in UW campus student, faculty and staff population, consistent with the Final EIS. The existing four structures on site currently house UW Facilities staff, whereas the proposed building would house a larger number of people in total, primarily students and faculty, and a substantially lower number of staff.

Housing – Construction and operation of the building would not remove nor increase housing on campus.

Light, Glare and Shadows – The building 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 Central Campus in the Final EIS.

Aesthetics – The building would be sited and designed in respect to East Stevens Way NE with a front porch approach that translates into a connector to the east side of central campus. There is a protected view corridor from the UW Club (2019 Seattle Campus Master Plan View Corridor 4) such that views east and southeast looking towards Union Bay must remain unobstructed. The proposed building would be outside of the view corridor. No impacts anticipated. **See Exhibit E.**

Recreation and Open Space – No recreation impacts nor demand for open space is anticipated from the project because it would not increase housing on campus nor substantially increase the campus population.

Cultural Resources – No cultural resource impacts are anticipated. The site was identified as "Low" potential to encounter sensitive conditions.

Historic Resources – Two of the four structures on the development site are older than 50 years and have been reviewed for historic resource eligibility; Facilities Services Administration Building (FAB) and University Facilities Building (UFB). Historic reviews determined the properties do not to meet any of the National Register criteria. **See Exhibit F.**

Public Services – No increase in demand for public services is anticipated due to operation of the building as the project would not substantially increase the population on campus.

Utilities – There is no anticipated increase in demand for water, sewer, stormwater, and solid waste as the project is anticipated to be efficient compared to the existing buildings. Because the project would not propose to use infiltration or stormwater re-use systems, the amount of stormwater leaving the site would not change substantially.

Transportation – The project will reduce the parking capacity adjacent to the UW Club (Lot C19) and vehicular circulation between the C19 parking lot and East Stevens Way NE. ADA parking for the project will be accommodated via assigned stalls within the C17 parking lot across East Stevens Way NE (southwest of the site).

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 building 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 Interdisciplinary Engineering Building 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 Interdisciplinary Engineering Building 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
Vicinity Map

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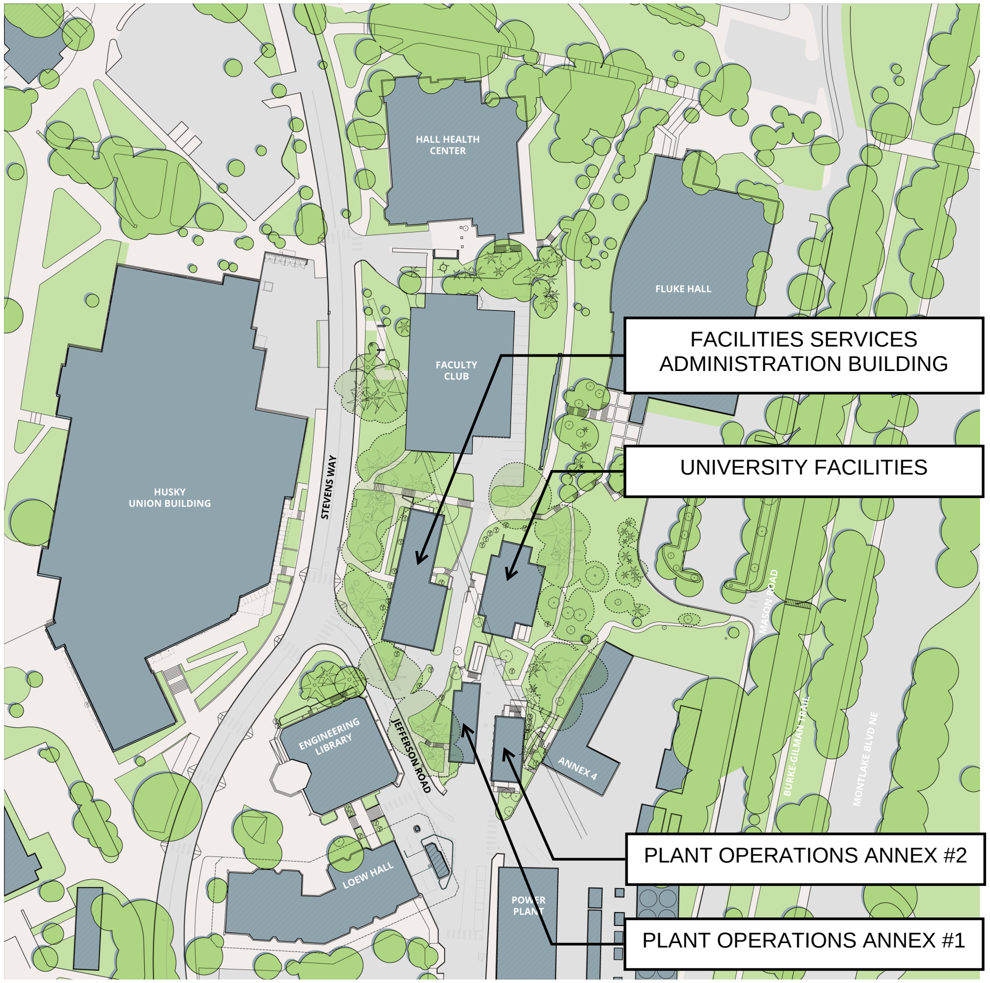
Exhibit B
General Site Plan



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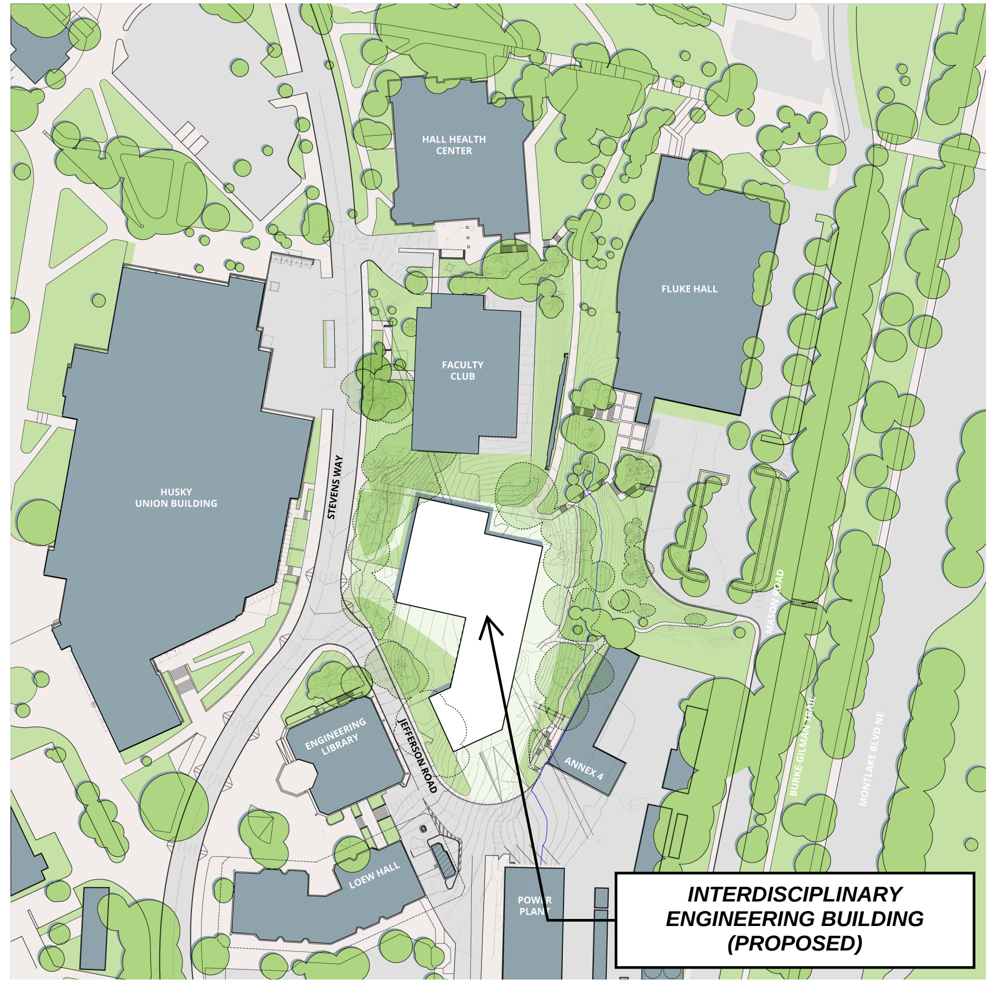

PROPOSED

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Exhibit C
GeoTech Report

DRAFT Rev1

Geotechnical Engineering Design Study

UW Interdisciplinary Education and Research Building

Seattle, Washington

Prepared for
University of Washington

October 29, 2021
0202944-000 (19574-00)

HARTCROWSER
A division of Haley & Aldrich

DRAFT Rev1

Geotechnical Engineering Design Study

UW Interdisciplinary Education and Research Building

Seattle, Washington

Prepared for
University of Washington

October 29, 2021
0202944-000 (19574-00)

Prepared by
Hart Crowser, a division of Haley & Aldrich

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Geotechnical Engineering Design Study

UW Interdisciplinary Education and Research Building

Seattle, Washington

This report presents our geotechnical engineering design study for the University of Washington Interdisciplinary Education and Research (UW IER) Building project in Seattle, Washington (Figure 1).

Our scope of services for this study included:

- Reviewing existing subsurface information on the project site;
- Completing five subsurface soil boring explorations;
- Performing laboratory tests on soil samples obtained from the borings;
- Providing geotechnical engineering recommendations for temporary shoring support, foundations, basement walls, subsurface drainage, earthwork, and other considerations; and
- Preparing this geotechnical engineering design report.

This report is for the exclusive use of the University of Washington and their design consultants for specific application to this project and site. This report was prepared in accordance with our contract dated August 27, 2020 and signed September 15, 2020. We completed this study in accordance with generally accepted geotechnical practices for the nature and conditions of work completed in the same or similar localities, at the time the work was performed. We make no other warranty, express or implied.

All elevations noted below are in reference to the North American Vertical Datum of 1988 (NAVD 88), unless stated otherwise.

PROJECT UNDERSTANDING

The proposed site is located at the northeast corner of the intersection of E Stevens Way and Jefferson Road on the University of Washington campus in Seattle, Washington (Figure 2). The site is bound by the UW Faculty Club to the north, the UW power plant facilities to the east, Jefferson Road to the south, and Stevens Way to the west. Construction site work will include demolition of the Facilities Administration and University Facilities Buildings. The proposed improvements on the site will include regrading the existing slope to accommodate the IER Building, terraces, and walkway networks between and through the facilities. The realigned and regraded path network will connect with the future Phase II building site and the larger campus network.

The existing site slopes down from west to east with the west boundary of the site at an approximate elevation of 132 feet and the east boundary at an approximate elevation of 108 feet. We understand the

IER Building will be five to six stories tall with one to two below-grade levels that daylight to the south and the east. The building will step down to accommodate site grades, with finished floor elevations at approximately 120 and 105 feet on the western and eastern portions of the building, respectively. Shallow footing foundations are anticipated to bear approximately 5 feet lower than the finished floor; at elevations of approximately 115 and 100 feet.

Several existing utilities are located within the building footprint. Some of these utilities connect to buildings that will be removed as part of the site development. However, others will remain in service during and after construction of the IER Building. Most significant of these is a steam tunnel segment that runs beneath the proposed building footprint, oriented approximately NNW to SSE. This brick-lined, arch-shaped tunnel was constructed circa 1950s, and is approximately 10 feet high and 6 feet wide. The top of the existing tunnel arch ranges in elevation across the proposed building footprint from approximately an elevation of 82 to 89 feet, with about 30 feet of soil cover. Proposed building excavations will reduce the soil cover, resulting in a new minimum cover of about 14 feet. Along each side of the tunnel alignment, deep foundation elements (i.e., drilled shafts or augercast piles) are planned, to transfer a portion of the building loads below the tunnel.

The IER Building will be supported on a combination of shallow and deep foundations. Below-grade levels will be constructed using conventional shoring systems. Our understanding of this project is based on information provided by and discussions with the University of Washington and our experience in the area. Our understanding of the site and subsurface conditions is based on our work to date at the site and on multiple sites nearby.

MAPPED ENVIRONMENTALLY CRITICAL AREAS

According to City of Seattle online GIS mapping (SDCI GIS 2021), the project site is mapped within two Environmentally Critical Areas (ECAs): Peat Settlement Prone Area and Historical Landfill (1000-foot buffer). According to the GIS map, the entirety of the University of Washington Seattle campus west of 15th Avenue NE and south of NE 45th Street is mapped as Peat Settlement Prone without much discrimination. Based on our understanding of the geologic setting, topography and review of soil borings, we did not encounter compressible peat deposits at this site and believe the site is misclassified as a Peat Settlement Prone Area. The site is also mapped within the 1000-foot methane buffer zone from the Montlake Landfill (University of Washington 2017). Because the site is underlain by glacially over consolidated soils and is significantly upslope from the known extents of the historic landfill, we do not expect project development to be significantly impacted by the site's proximity to this historic landfill and do not recommend any methane mitigation requirements.

Although not mapped within the development footprint, an ECA for Steep Slopes (40 percent average) is located downslope to the east of the project site. We will perform global stability analyses for temporary construction and final building conditions, and provide the results of these analyses under a separate cover.

SUBSURFACE CONDITIONS

Our interpretation of the subsurface conditions is based on conditions encountered in our borings as well as our review of historical geotechnical data near the site, our previous experience in the area, and published regional geologic maps. Hart Crowser completed five borings (HC-1 through HC-5) drilled to a depth of 50 feet on September 16 through 18, 2020. We also reviewed historical borings completed to the north for Fluke Hall (Roger Lowe Associates 1977 and Shannon & Wilson 1985) and for the Hall Health Center Addition (Shannon & Wilson 1973), to the east for the University Facilities Building (Rittenhouse-Zeman & Associates 1980), to the south for the Engineering Library and Loew Hall (Shannon & Wilson 1966), and to the west for the Student Union Building (Dames and Moore 1975). Locations of our borings for this project and nearby historical borings are shown on Figure 2. Generalized subsurface cross sections A-A' through C-C' are shown on Figures 3 through 5, respectively.

Soil and groundwater conditions are summarized in the following sections. The conditions encountered in our explorations are presented in boring logs in Appendix A. The results of associated laboratory tests on selected samples are presented in Appendix B. Boring logs in the nearby areas considered generally relevant for the project site are included in Appendix C.

The explorations referenced in this study reveal subsurface conditions only at discrete locations across the project site and that the actual conditions in other areas will vary. Furthermore, the nature and extent of any such variations would not become evident until additional explorations are performed or until construction activities are underway. If significant variations are observed at that time, we may need to modify our conclusions and recommendations accordingly to reflect actual site conditions.

Soil Conditions

In general, the subsurface soil consists of very dense glacially overridden soils near to the ground surface. These glacial soils are suitable for the shallow foundation support. In general, the soils observed in the explorations consist of the following soil units, described in the order they were encountered from the ground surface down.

- **Fill – Very Loose to Medium Dense Silty Sand and Sand with Silt.** Borings indicate between 5 and 7.5 feet of fill consisting of very loose to medium dense, moist, silty sand or sand with silt with occasional organics. Fill was identified in all borings, but may be encountered to variable and deeper depths due to historical development activity across the site.

- **Glacial Till – Very Dense Silty Gravelly Sand.** Below the Fill, the borings indicated very dense, moist, silty, gravelly sand. This unit is a glacially overridden Glacial Till material and appears to extend down to an elevation of 90 to 95 feet, based on nearby borings. Glacial Till is a suitable bearing unit for shallow foundations.

- **Outwash – Very Dense Poorly-Graded Sand.** Below the Glacial Till, the borings encountered a very dense, moist, clean to slightly silty, fine to medium sand and appears to extend down to an elevation of 60 to 65 feet. Two of the five borings (HC-1 and HC-4) terminated in this soil unit. Outwash is a suitable bearing unit for shallow foundations.

- **Lacustrine – Hard Silt.** Below the Outwash, borings encountered a hard, moist, slightly sandy, silt. Three of the five borings (HC-2, HC-3, and HC-5) terminated in this soil unit. Lacustrine deposits are a suitable bearing unit for shallow foundations; however, they are relatively deep and not expected to be encountered within the planned depth of excavation.

In some of our borings, low sample recovery or rough gravelly drilling was encountered at various depths, indicating oversized material such as cobbles and boulders. Such large materials could make drilling and/or excavation difficult. Therefore, the contractor should be prepared to deal with hard drilling or large obstructions. In addition, the native soils may contain relatively clean sand and/or gravel zones, where groundwater may accumulate and be more prone to caving when exposed in a vertical face or encountered in a drilled hole. Provisions should be made in contract documents to account for the possibility of these conditions.

Groundwater Conditions

Our understanding of groundwater conditions at the site is based on observations during our explorations and conditions described in existing historical borings around the site (Figure 2 and Appendices A and C).

Static groundwater was not observed in our explorations or in the historical borings in the vicinity of the site. Based on historical information, the anticipated groundwater level is well below the planned base of excavation elevation for this project.

Isolated perched water-bearing zones may exist in the upper soils and should be anticipated during construction. A perched water layer was encountered at approximately an elevation of 70 feet in boring HC-3. Fluctuations in groundwater conditions including depth and volume may be caused by variations in rainfall, temperature, season, and other factors.

SEISMIC CONSIDERATIONS

Seismic Setting

The seismicity of Western Washington is dominated by the Cascadia Subduction Zone, in which the offshore Juan de Fuca Plate is subducting beneath the continental North American Plate. Three main types of earthquakes are typically associated with subduction zones: crustal, interface subduction, and intraslab subduction earthquakes.

Crustal Sources. Recent fault trenching and seismic records in the Puget Sound area clearly indicate a distinct shallow zone of crustal seismicity, the Seattle Fault, which may have surficial expressions and can extend 25 to 30 kilometers deep.

Subduction Zone Sources. The offshore Juan de Fuca Plate is subducting below the North American Plate. This causes two distinct types of events. Large-magnitude interface earthquakes occur at shallow depths near the Washington coast (e.g., the 1700 earthquake with a magnitude of 8 to 9) at the interface between the two plates. A deeper zone of seismicity is associated with bending the Juan de Fuca Plate below the

Puget Sound region that produces intraslab earthquakes at depths of 40 to 70 kilometers (e.g., the 1949, 1965, and 2001 earthquakes).

Design Response Spectrum

We provide code-based seismic design parameters for use on elements designed to ASCE 7-16 (ASCE 2017) which is referenced by the 2018 IBC (International Code Council 2018). The mapped response spectra are based on Site Class B (rock) conditions. Seismic parameters are adjusted according to the actual site conditions. The class for this project location is Site Class C (very dense soil). IBC defines the design spectral acceleration parameters at short periods (S_{DS}) and at the one-second period (S_{1D}) as two-thirds of the corresponding site-class-adjusted MCE_R parameters (S_{MS} and S_{M1}). Similarly, ASCE 7 requires MCE_G peak ground acceleration adjusted for site effects (PGA_M) to be used for evaluation of liquefaction, lateral spreading, seismic settlements, and other soil-related issues. The resulting seismic design parameters are shown in Table 1 for ASCE 7-16.

Table 1 – ASCE 7-16 Seismic Parameters

Parameter	Value
Latitude	47.655
Longitude	–122.304
Site class	C
Risk category	I, II, or III
Peak ground acceleration, PGA	0.556 g
Spectral response acceleration at short periods, S_S	1.310 g
Spectral response acceleration at the 1-second period, S_1	0.455 g
Seismic site coefficient, F_{PGA}	1.2
Seismic site coefficient, F_a	1.2
Seismic site coefficient, F_v	1.5

Seismically Induced Geotechnical Hazards

Our assessment of the seismically induced geotechnical hazards at the project site is based on the existing soil explorations presented in this report, regional experience, and our knowledge of local seismicity. The potential hazards include surface rupture, liquefaction and subsidence, and lateral spreading.

Surface Rupture. The Seattle Fault Zone consists of multiple east-trending, north-verging reverse thrust faults located in the Puget Lowlands of western Washington. The northernmost splay of the Seattle Fault is estimated to be approximately 7 miles south of the site. Because there are not any known faults underlying the site, the hazard associated with surface rupture at the site during the life of the structure is considered low.

Landslides. The near-surface soils are dense and the groundwater level is relatively deep therefore, the hazard associated with landslides is low.

Liquefaction and Subsistence. When cyclic loading occurs during a seismic event, the shaking can increase the pore pressure in loose to medium dense saturated sands and cause liquefaction, or temporary loss of soil strength. This can lead to surface settlement. We did not encounter saturated soil in a loose to medium dense condition in the borings conducted for this project. The soils below the anticipated groundwater table at this site are generally very dense silty, gravelly sand or hard sandy silt. The risk of liquefaction, seismically induced settlement, or significant ground deformation as a result of liquefaction from the design earthquakes is very low.

Lateral Spreading. Lateral spreading is typically associated with lateral movement on sloping ground caused by liquefaction or a reduction of shear strength of soil within or under the slope. Lateral spreading could impact the proposed project by increasing the lateral force exerted on the subsurface walls. However, because the liquefaction hazard is low, the lateral spreading hazard is also very low.

GEOTECHNICAL ENGINEERING DESIGN RECOMMENDATIONS

Our recommendations are based on our current understanding of the project and the subsurface conditions revealed by relatively recent and historical borings. If the nature or location of the proposed project facilities change, Hart Crowser should be notified so that we can change or confirm our recommendations.

General Considerations

Based on the current design plans and our discussions with the design team, the primary geotechnical aspects of this project are as follows:

- **Temporary Shoring.** Excavation depths up to about 20 feet are planned to accommodate the below-grade levels. A soil nail shoring system is a suitable and cost-effective solution to support the proposed excavation; however, recommendations for a soldier pile and tieback shoring system are also provided. The need for specific shoring systems at specific locations should be determined by the shoring designer.

- **Shallow Foundations.** The dense native soils (Glacial Till and Outwash) are suitable bearing materials for shallow footing foundations. The existing granular fill is relatively loose and not a suitable bearing material for the anticipated building loads. Where existing fill is encountered, it should be overexcavated and replaced with structural fill or lean concrete.

- **Deep Foundations.** Due to the proximity of the steam tunnel to the anticipated shallow foundation bearing elevations, deep foundations are required adjacent to the tunnel to avoid surcharging the tunnel from the new building loads. These should be installed at least 5 feet (edge-to-edge) from the tunnel, within the building area encompassed by a 1H:1.5V projection from the bottom edges of the tunnel. We anticipate drilled shafts or augercast piles will be used for these deep foundations.

- **Drainage.** The regional groundwater table is expected to be below the bottom of the excavation; however, perched groundwater may be encountered during excavation. Drainage of perched water will need to be accommodated by the shoring system during excavation. Similarly, a suitable foundation wall drainage system will be needed.

Support of Excavation Using Shoring

Temporary shoring walls will be required to support the vertical sides of the excavation. The shoring system should be designed to provide temporary lateral support for the excavation while ensuring safety and stability of the buildings, utilities, and other infrastructure adjacent to the excavation.

Based on the subsurface soil conditions and the potential need to support adjacent buildings, it is our opinion that the project excavation could be supported using conventional soldier piles with tieback anchors, a soil nail shoring system, or a combination of the two. The selection of a suitable temporary shoring system for earth retention will depend on numerous factors, including contractor experience and cost. The advantages and disadvantages of each system should be carefully weighed to account for cost and construction benefits that may be lost or gained with alternate retaining systems.

General Considerations

We recommend that shoring should be designed by a professional engineer registered in Washington State. We also recommend that we be given the opportunity to review the proposed shoring design before construction, which is required by the City of Seattle.

This report is not intended to provide specific criteria for the contractor's construction means and methods. It should be the responsibility of the shoring contractor to verify actual ground conditions at the site and determine the construction means, methods, and procedures needed to install an appropriate shoring system.

Adjacent building foundation surcharges will need to be checked for application to the shoring and permanent foundation wall design. The shoring designer and contractor will need to confirm the location of adjacent building walls and other obstructions such as utilities, rights of way, etc.

Shoring elements may extend into Seattle Department of Transportation (SDOT) right of way. Temporary shoring elements can extend into public rights of way, but tiebacks must be destressed and piles must be cut off when no longer needed for wall stability. A private property easement must be obtained from property owners for shoring elements extending into private property. Internal bracing may be used where easement agreements cannot be obtained.

Soil Nail Support of Excavation

In our opinion, the site is generally conducive to the use of soil nailing. It is a more cost-effective alternative than a soldier pile and tieback system if used with conventional construction techniques. Soil nail walls consist of a series of small-diameter (typically 6- to 8-inch) holes drilled in a rectangular or diamond pattern, filled with reinforcing steel and structural grout, and connected to a shotcrete facing or "wall." The pattern and length of the nails (i.e., the reinforcing steel/grout installations) vary depending on the soil type, the depth of cut, and other factors. The nails and shotcrete are installed sequentially as the excavation proceeds downward.

Soil Nail Design

We provide preliminary soil nail recommendations within this section. A final design for a soil nail system is not part of this study, and is best completed after the owner and design team have finalized the proposed excavation geometry. A soil nail and shotcrete shoring system is typically designed using a limit equilibrium analysis approach. Design is based on an assumed pullout capacity for the soil nails that depends on their size, anticipated subgrade conditions, and local experience with similar soils. During construction, the assumed capacity is verified by a testing program to confirm that nail diameter, lengths, and installation techniques are suitable to meet the design assumptions.

A typical soil nail design includes the following elements:

- Design methods should be in accordance with Federal Highway Administration "Geotechnical Engineering Circular No. 7, Soil Nail Walls" (FHWA 2015).
- Soil nail wall design should consider surface loading from traffic, site equipment, and loads from adjacent structures. Vertical elements may be needed in the upper soils to improve face stability and reduce the risk of raveling or sloughing where fill is encountered.
- Permanent wall drainage should be incorporated to relieve potential hydrostatic pressure, intercept and divert water away from the wall and toe of the wall, and convey water to the permanent drainage system. This drainage and pressure relief is provided by Miradrain (or equivalent) strips affixed to the soil behind the shotcrete. Surface water runoff should be directed away from the top of the wall.
- Soil nails should be steel bars without couplers, splices, or welds, and should be installed with centralizers.
- Soil nails should be between 3 and 6 feet apart horizontally and 3 to 5 feet apart vertically.
- Temporary wall facing may consist of a 6-inch-thick steel reinforced shotcrete wall. Reinforcement may include a single mat of 4 inch by 4 inch, W4.0 x W4.0, welded wire fabric, as well as vertical and horizontal reinforcing bars. Actual facing design would be determined during the comprehensive design of the soil nail system.
- Soil nail lengths should be plotted, and their layout compared with existing utilities and adjacent underground foundations to minimize interference.

The soil nail system should be designed to performance specifications, and the designer should be able to demonstrate that:

- No failure surface that has a factor of safety less than 1.35 against sliding exists through or outside the nails;
- The nails are not allowed to be stressed to more than 80 percent of their yield strength; and
- The mobilized bond stress is less than half the ultimate adhesion between the grout and the soil. Ultimate adhesion is determined by the soil shear strength and must be justified by both pullout testing before nail installation and by limited production nail testing.

We recommend the soil parameters in Table 2 for preliminary evaluation of soil nail feasibility. Final nail adhesion should be determined by the shoring designer and contractor based on the planned installation method(s) and verified with pullout tests conducted before shoring production.

Table 2 – Soil Nail Design Parameters

Depth (feet)	Soil Type	Unit Weight (pcf)	Soil Friction Angle (°)	Soil Cohesion (psf)	Service Nail Pullout [a] (kips/foot)
0 – 10	Fill	120	32	0	1.5
10 – 50	Glacial Deposits	135	40	250	4

Notes:

a. Assumes pressure grouting.

Soil Nail Wall Construction and Installation

Construction sequencing is especially important in soil nail construction. Soil nail wall systems are designed so that the excavation must proceed in staged lifts (a lift is a single row of nails). For vertical cuts, we recommend:

- Test each material type to demonstrate that the unsupported face will be stable over the required “stand-up” time;
- Ensure that all surface water is controlled during construction;
- Excavate the initial cut so it is a few feet below the first row of nails; and
- Limit excavation height to the minimum amount necessary for practical and timely application of shotcrete, typically no more than an unsupported height of about 4 feet. In caving ground, provide an initial stabilizing layer of shotcrete (flashcoat) and/or steel-reinforced flashcoat as soon as possible; in firm ground, the nails may be installed first.

For soil nail wall installation, we recommend:

- Close excavation sections before the end of a work day, unless prior approval is given by the shoring designer and Hart Crowser.
- Advance drill holes using rotary methods with air flush, dry auger, and cased methods (for less stable grounds). Drill the soil nail holes using equipment and techniques that will minimize caving and loss of ground. Drilling with a casing will reduce the potential for ground loss. Ensure that the hole is clean of disturbed material.
- Do not leave holes open overnight.
- Pump structural grout into the hole through the auger (wet bar installation method) or through a tremie tube extended to the bottom of the hole.
- Grout the hole as soon as possible after drilling to prevent caving.
- Require that nails consist of reinforced steel bars without couplers, splices, or welds, and that they be installed with centralizers.
- Minimize the duration of unsupported cuts and limit the total area of wall constructed during one shift to preserve face stability. We recommend that the initial duration of unsupported cuts be limited to one shift unless the contractor's demonstration test for each soil type shows that longer stand-up times are possible, and as approved by the shoring designer and Hart Crowser.
- Expect cobbles, boulders, debris, and/or groundwater seepage to be encountered.
- Take care not to "mine out" large cavities in granular soil if drilling with a continuous-flight auger.
- Maintain continuous cutting return if using pneumatic drilling techniques so that air pressure, which may damage subgrade structures, is not "channeled" to nearby utility vaults, corridors, or subgrade slabs.
- The shoring contractor should particularly note the presence of existing facilities adjacent to the project site, including buried utilities and foundations, as these may affect the location or extent of the anchor holes.
- Monitor potential movement of the shoring system and potential ground settlement adjacent to the excavation.

It is the responsibility of the contractor to verify actual ground conditions at the site and to determine appropriate construction methods and procedures for installing a suitable shoring system. Cobbles, boulders, or debris may be encountered and could impact construction.

For shotcrete wall construction, we recommend:

- Before production, shotcrete application test panels should be applied by each nozzleman under field conditions at the site, and the panels should be cored and examined for defects;
- Require that preparations for shotcrete include installation of drainage material, installation of soil nails, and placement of approved reinforcement; and
- If sloughing occurs, shorten the time a cut is left open, reduce the height of the cut, use a stabilizing berm, place a flashcoat of shotcrete, or place or complete the cut in sections or stages.

Soil Nail Testing

We recommend that selection of the materials and the installation technique be left to the shoring contractor. The selected soil nail installation method must be subject to field verification with performance testing and proof testing.

Soil nails should be tested to confirm the design friction (adhesion) value and to verify that suitable installation has been achieved. Soil nail adhesion is highly dependent on soil conditions encountered during construction and on installation techniques. We recommend using performance-based specifications and that the shoring contractor be responsible for the installation techniques to achieve the design soil nail adhesion.

- Soil nail specifications should include an appropriate number of verification load tests (200 percent) and proof load tests (130 percent) on production nails. We recommend a minimum of two successful verification tests for each soil type. Proof testing is required on at least 5 percent of the production nails.
- Verification test nails should have an unbonded length of at least 3 feet, but not longer than a maximum length such that the nail load does not exceed 90 percent of the nail bar tensile allowable load. The nail hole should be fully grouted after testing. Perform verification tests in at least 2 soil nails per soil type.
- A load reaction system must be provided by the contractor and approved by the shoring designer.

We recommend the shoring contractor and Hart Crowser coordinate on selecting the test locations based on observation of the soil conditions as the excavation proceeds.

Deflections

In theory, a soil nail system should deflect more than a soldier pile/tieback system since the nails are not pre-stressed. However, observations of soil nail wall deflections in the Puget Sound area indicate that, if constructed in favorable soil conditions, deflections of the two systems tend to be similar. Typical horizontal movement for properly designed and constructed soil nail walls is anticipated to be on the order of 0.001 to 0.005 times the excavation depth and is highly dependent on construction practices.

Soldier Pile/Tieback Support of Excavation

The basic geotechnical criteria for the design of a conventional soldier pile/tieback shoring wall system are: (1) lateral earth pressure, and (2) vertical bearing capacity of soldier piles. Figures 6, 7, and 8 provide recommended parameters for the design of temporary soldier pile walls (cantilevered or supported by tieback anchors). Additional surcharge loads should be calculated as shown on Figure 9 and added to the lateral earth pressure diagrams. Tied-back/braced shoring must be designed by a professional structural engineer registered in the State of Washington.

Lateral Earth Pressures

Lateral earth pressures for the shoring design depend on the type of shoring and its ability to deform. If the top of the shoring is allowed to deform about 0.001 to 0.002 times the shoring height, and if no settlement-sensitive structures or utilities are within the potential zone of deformation behind the shoring wall, the shoring may be designed using active earth pressures. If settlement-sensitive structures or utilities exist within the potential zone of deformation, or where the shoring system is too stiff to allow sufficient lateral movement for development of an active condition, at-rest earth pressures should be used for the shoring design.

Depending on the excavation and construction phases, and specific shoring wall configurations to be considered for design, use of equivalent fluid unit weights resulting in a triangular distribution of lateral earth pressure may be used for the shoring wall under cantilevered condition, as shown on Figure 6. Single-braced and multiple-braced walls should be designed using a trapezoidal apparent earth pressure distribution as shown on Figures 7 and 8, respectively. These lateral earth pressure recommendations assume a level ground surface behind the walls, and a fully drained condition behind the walls so that hydrostatic pressures do not act on the walls above the excavation base. Based on our current understanding, sloping ground conditions behind shoring walls is not anticipated, but any minor sloping or higher ground conditions existing behind the walls may be treated as an additional surcharge load. Lateral earth pressures for cantilevered wall and single-braced wall will need to be considered in the design of multiple-braced wall to model conditions of the early construction stages.

Based on the assumed loading conditions and the applied loads, we generally expect the shoring system to deflect up to 0.5 to 1 inch into the excavation when designed considering active earth pressures. Individual soldier piles may deflect more than 1 inch or deflect away from the excavation under tieback stressing loads. As recommended herein, Hart Crowser should review the results of shoring monitoring. Based on this review, any soldier piles that deflect more than 1/2 inch will be identified for review by the project team to assess the cause of the deflection and to determine whether remedial measures are required.

Lateral pressures due to surcharge loads such as building foundations, footings, heavy equipment, sloped ground, and large material stockpiles should be calculated using the methods shown on Figure 9 and added as additional design loads. Any additional lateral pressure due to existing and applicable surcharge loads should be added to the load calculated for the shoring walls' design. We recommend that Hart Crowser review or calculate the lateral earth pressures when surcharge loads, footprints, and configurations become available based on as-built foundation plans of adjacent structures and assessment of foundation loads by structural engineer, any potholing work, and contractor's plan for locating heavy equipment, vehicles, baker tanks, material stockpiles etc.

Soldier Pile Design and Installation

Soldier piles must be designed to carry bending stresses from lateral earth pressure resulting from various sources in combination with applicable vertical loads. Also, the embedded portion of the piles must be deep enough to resist lateral kickout and vertical loads.

- Design soldier piles according to recommendations on Figures 6, 7, and 8.
- For design against kickout, compute the lateral resistance on the basis of passive pressure acting over twice the diameter of the soldier pile section or the pile spacing, whichever is less. Use a factor of safety no less than 1.5 to calculate allowable passive resistance.
- Embed soldier piles at least 10 feet below the deepest point of the excavation within 15 feet of the pile location or the required structural depth to resist applied vertical tieback loads, whichever is greater.

Conditions such as caving soil or groundwater can loosen soil at the bottom of the soldier pile borehole, thereby reducing the bearing capacity. Tieback destressing and shoring failure could occur if soldier pile bearing capacity is inadequate and soldier piles settle under the vertical component of the inclined tieback load. We recommend that a Hart Crowser representative closely monitor soldier pile installation for these conditions so that construction methods can be adjusted accordingly. The contractor should be prepared to:

- If substantial perched water is encountered during soldier pile installation, the use of casing or drilling mud may be needed. Although the actual need for casing and/or drilling mud can be determined in the field at the time of installation, the Contractor should be prepared for such requirements.
- Tremie concrete from the bottom of the hole to displace groundwater or drilling mud used to maintain an open hole. Drilling mud should not be used unless the mix is reviewed and approved by the geotechnical and structural engineer.
- Excavate the soldier piles in a manner that prevents "heave" or "boiling" at the bottom of the soldier pile excavation. It may be necessary to over-drill the borehole and backfill the bottom of the borehole with structural concrete bearing on undisturbed soil.

Soldier pile shoring system construction may be difficult if boulders, cobbles, or loose sand and gravel are encountered in the excavation. If these conditions are encountered, substantial soil raveling could occur.

Temporary Lagging Design

Timber lagging is often used to prevent ground loss between the soldier piles. The lagging is inserted between the webs of the soldier piles and is designed for some fraction of the applied pressure on the wall. Limiting the exposed lift height and prompt and careful installation of lagging to support the exposed height is particularly important for seepage areas and loose soil areas to be anticipated for this project to ensure and maintain the integrity of the excavation. The shoring contractor should be responsible for installing lagging to prevent soil failure, sloughing, and ground loss, and to provide safe working conditions.

We recommend the rough-cut timber lagging thickness shown in Table 3. These values are based on recommendations in FHWA Geotechnical Engineering Circular No. 4 (FHWA 1999) and our experience from similar excavations in Seattle.

Table 3 – Recommended Lagging Thickness

Excavation Depth in Feet	Recommended Lagging Thickness (rough-cut) for Clear Spans of:					
	5 feet	6 feet	7 feet	8 feet	9 feet	10 feet
0 to 25	2 inches	3 inches	3 inches	3 inches	4 inches	4 inches
25 to 60	3 inches	3 inches	3 inches	4 inches	4 inches	5 inches

The contractor should be prepared to place lagging in small vertical increments and to backfill voids caused by ground loss behind the shoring system during construction.

We make the following recommendations concerning lagging:

- Backfill voids greater than 1 inch using sand or a porous slurry. Backfill the void spaces progressively as the excavation deepens. The backfill must not allow potential hydrostatic pressure buildup behind the wall. Drainage behind the wall must be maintained or hydrostatic pressure should be added to the recommended lateral earth pressures.

- Install extra lagging above the shoring wall if there is a slope above the wall to provide a partial barrier for material that could ravel down from the slope face and fall into the excavation.

Tieback Anchor Design and Construction

Our tieback anchor design recommendations are based on the assumption that cased boreholes at least 6 inches in diameter and pressure grouting will be used. We anticipate the anchors will be installed by single stage, high-pressure grouting as the casings are withdrawn. An allowable load transfer (adhesion) of 4 kip/ft in the native (very dense sand or hard silt) soils may be used for the planning and design of pressure-grouted anchors described herein. The shoring contractor is responsible to achieve these adhesions, which could require secondary grouting if the initial pressure grouting is insufficient. Alternatively, the anchor zone could be lengthened. We can provide separate recommendations if anchors are to be grouted under gravity using tremie methods.

Pressure-grouted tieback anchors used for external lateral support of the soldier pile walls should be designed using the following recommendations.

- For planning purposes, design anchors in accordance with Figures 7 and 8. Note the required anchor lengths should be provided behind and below the line defining the “no load zone” on Figures 7 and 8.

- Locate tieback anchors no closer to each other than 4 feet.

- Tieback unbond lengths should have a minimum length of 10 feet for bars and 15 feet for strands (FHWA 1999).
- Install a bond breaker such as plastic sheathing or a PVC pipe around the tie rods/cables within the no-load zone.
- Grout and backfill drilled installations immediately after drilling; do not leave holes open overnight. This will help minimize possibilities of collapse of the holes, loss of ground, and surface subsidence.
- Take care not to mine out large cavities in granular soil.
- Maintain continuous cutting return if using pneumatic drilling techniques so that air pressure, which may damage subgrade structures, is not channeled to nearby utility vaults, corridors, or subgrade slabs.
- Plot and compare anchor lengths with any underground support elements of adjacent structures to avoid interference.
- Note the presence of existing facilities adjacent to the project site, including buried utilities and foundations of adjacent buildings, as these may affect the location, orientation, and the length of the anchor holes.
- Install the anchor holes in a manner that will minimize ground loss and not disturb previously installed anchors. During tieback drilling, wet or saturated zones may be encountered, and soil could cave in, especially within the advance outwash layer. Drilling with a casing reduces the potential for soil caving and ground loss.

We recommend selection of the materials and the installation technique be left to the shoring contractor. The shoring contractor shall be made contractually responsible for the design of the tieback anchors, as tieback capacity is largely a function of the means and methods of installation. The selected tieback anchor installation method must be subject to field verification with verification testing and proof testing.

Hart Crowser should review the design for anchor locations, capacities, and related criteria before construction begins. We recommend a factor of safety of at least 2.0 against anchor pullout. This factor of safety provides for a reasonable additional load capacity should an unforeseen increase in load develop during excavation and construction. The variable soil conditions and unit friction values mean that some field changes in anchor length may be necessary.

The tieback anchor testing program should include verification testing of tiebacks at selected locations, and proof testing of all production tiebacks. We recommend testing tiebacks in general accordance with the recommendations in the publication "Recommendations for Prestressed Rock and Soil Anchors" by the Post Tensioning Institute (PTI 2014) and the recommendations in the following sections.

Verification Tests

We recommend a minimum of two verification tests for each installation method and soil unit before installation of production anchors to validate the design pullout value. Hart Crowser will select the testing locations with input from the shoring subcontractor. Hart Crowser or shoring designer may require additional verification tests when creep susceptibility is suspected, or when varying ground conditions are encountered.

Tiebacks for verification tests should be installed using the same methods, personnel, material, and equipment as are used for the production tiebacks. Deviations may require additional verification testing as determined by the engineer.

Verification tests load the tieback to 200 percent of the design load (DL) and include a 60-minute creep test at 150 percent of the design load (DL). The tieback design loads should be clearly shown on the shoring plans/drawings. The tieback load should not exceed 80 percent of the steel's ultimate tensile strength. Verification test tiebacks should be incrementally loaded and unloaded using the schedule in Table 4, and as recommend below.

- The alignment load (AL) should be the minimum required to align the testing assembly and should be less than 5 percent of the design load (DL). The dial gauge should be zeroed after the alignment load has stabilized.
- The verification test will measure anchor stress and displacement incrementally to values of unit skin friction (adhesion) to 200 percent of the design adhesion.
- The soldier piles, vertical elements, shotcrete facing, and/or anchor tendon may require extra reinforcement to permit stressing to 2.0DL, as required for the performance test.
- Perform tests without backfill in front of the bonded anchor zone, if the hole will remain open, to avoid any contributory resistance by the backfill. If the hole will not remain open during testing, provide a bond breaker on the no-load zone specified on the plans.
- Load the anchor in increments of 0.25DL and unload to the aligning load (AL) before incrementally loading to the next load increment (e.g., AL, 0.25DL, AL; 0.25DL, 0.50DL, AL; 0.25DL, 0.50DL, 0.75DL, AL; etc.). Ensure that deflection readings stabilize for intermediate load increments (e.g., 0.25DL and 0.50DL) before increasing the load to the next increment (e.g., 0.75DL).
- Load levels at during hold intervals should be held constant to within 50 pounds per square inch (psi), and deflection measurements shall be made to minimum accuracy of 0.01 inch.
- Obtain and record deflection measurements for loading at intervals of 1, 2, 3, 5, 7, and 10 minutes for 10-minute hold interval.
- The creep test at 1.5DL should be performed by and recording deflections at 1, 2, 3, 5, 7, 10, 20, 30, 50, and 60 minutes.

Table 4 – Tieback Verification Test Incremental Load and Hold Time

Load Level	Hold Time
AL	Until stable
0.25DL	10 min
0.5DL	10 min
0.75DL	10 min
1.0DL	10 min
1.25DL	10 min
1.5DL	**60 min**
1.75DL	10 min
2.0DL	10 min
1.5DL	Until Stable
1.0DL	Until Stable
0.5DL	Until Stable
AL	Until Stable

The acceptance criteria for a verification test are as follows.

- Exhibits a linear or near-linear relationship between unit stress and movement over the percent stress range during loading from AL to 2.0DL.
- Holds the maximum test unit stress at 1.5DL without noticeable creep. Noticeable creep is defined as a rate of movement of more than 0.04 inch between the 1- and 10-minute readings, or more than 0.08 inches between the 6- and 60-minute readings. If the reading does not stabilize to 0.08 inch or less per log cycle of time, the test shall be considered as failing the creep criteria.
- Satisfies the apparent free tendon length criteria. Apparent free length criteria are as follows:
  - Minimum apparent free length, based on the measured elastic and residual movement, should be greater than 80 percent of the designed free length plus the jack length; and
  - Maximum apparent free length, based on the measured elastic and residual movement, should be less than 100 percent of the designed free length plus 50 percent of the bond length plus the jack length.
- The anchor does not pull out under repeated loading or at 2.0DL.

Proof Tests

Proof tests load all the production tiebacks to 1.33DL and include a 10-minute hold time at 1.33DL. The purpose of proof test is to quickly and economically determine the acceptability of each anchor for adequate performance after lock-off at design load (DL). The tieback design loads should be on the shoring drawings. The tieback load should not exceed 80 percent of the steel's ultimate tensile strength. Proof tests should be incrementally loaded and unloaded using the schedule in Table 5.

Table 5 – Tieback Proof Test Schedule

Load Level	Hold Time
AL	Until stable
0.25DL	1 minute
0.5DL	1 minute
0.75DL	1 minute
1.0DL	1 minute
1.33DL	10 minutes
AL	Until stable

The AL should be the minimum load required to align the testing assembly and should be less than 5 percent of the design load. The dial gauge should be zeroed after the alignment load has stabilized.

The load should be held constant to within 50 psi and deflections recorded at 1, 2, 3, 5, 6, and 10 minutes. If the tieback deflection between 1 and 10 minutes at 1.33DL exceeds 0.04 inches, the load should be held for an additional 50 minutes and deflections recorded at 20, 30, 50, and 60 minutes.

The acceptance criteria for a proof test are:

- The creep rate at 1.33DL is less than 0.04 inches between 1 and 10 minutes or less than 0.08 inches between 6 and 60 minutes and the creep rate is linear or decreasing during the creep test;
- Satisfies the apparent free tendon length criteria as described above for verification test; and
- The anchor does not show pull out failure or tendency to failure during loading.

Shoring Monitoring

A shoring monitoring program provides early warning if the shoring does not perform as expected (e.g., excessive movement or impacts to surrounding nearby structures and utilities). The monitoring program should include a preconstruction survey of existing conditions, periodic surveys during construction, and a post-construction survey.

The contractor and the shoring subcontractor should be familiar with the existing site conditions, including surrounding nearby structures and utilities. They should be allowed to review the inspection data gathered by the owner and may also choose to complete a survey on their own. The contract should clearly define the responsibilities of the owner, contractor, and shoring contractor in making inspections, reviewing data, and repairing possible damage.

Preconstruction Survey

A preconstruction survey documents the condition of existing streets, utilities, and buildings. The survey should include video and/or photograph documentation. The size and location of existing cracks in streets and buildings should receive special attention and may be monitored with a crack gauge.

Construction Survey

We recommend including adjacent building surveys and optical surveys in the shoring monitoring program during construction.

All monitoring data should be submitted to Hart Crowser for review. The data will be included in our weekly field transmittals to the project team and the City during construction. Details of our expectations for shoring monitoring are included below.

Adjacent Building Surveys. We recommend surveying adjacent buildings before, during, and after construction. The pre-construction survey will establish the baseline of existing conditions (e.g., identifying the size and locations of any cracks). The surveys should consist of a video and/or photographs of the interior and exterior of adjacent buildings and detailed mapping of all cracks. Any existing cracks could be monitored with a crack gauge.

Optical Surveying. We recommend optical surveys of horizontal and vertical movements of (1) the surface of the adjacent streets, (2) buildings on and adjacent to the site, and (3) the shoring system itself. The contractor, in coordination with the geotechnical engineer, should establish two reference lines adjacent to the excavation at horizontal distances back from the excavation face of about 1/3 H and H, where H is the final excavation height. Typically, these lines will be established near the curb line and across the street from the excavation face. The points on the adjacent buildings can be set either at the base, or on the roof, or both. Shoring system monitoring should include measuring vertical and horizontal movement at the top of every other soldier pile at the minimum. Shoring monitoring should also include geotechnical instrumentation (i.e., inclinometers) on each shoring wall.

The measuring system for the shoring monitoring should have an accuracy of at least 0.01 foot. All reference points on the ground surface should be installed and read before excavation begins. The frequency of readings will depend on the results of previous readings and the rate of construction. At a minimum, readings on the external points should be taken twice a week through construction until below-grade structural elements such as floors, decks, and columns are completed, or as specified by the structural engineer or shoring designer. Readings on the top of soldier piles and the face of existing buildings on or adjacent to the property should be taken at least twice a week during this time. We recommend that the owner hire an independent surveyor to record the data at least once per week, and that the surveyor or contractor take the other reading.

All monitoring data should be submitted to Hart Crowser for review.

Automation of Optical Surveying. The City of Seattle requires Hart Crowser to review and submit survey data to the project team. As an alternate to manual surveying, Hart Crowser has subcontractor agreements with firms that can instal automated total stations to monitor the shoring and adjacent points. Please reach out to us if automated monitoring is desirable for this project. The benefits of automated monitoring include:

- Near real-time data acquisition and hosting on the cloud;
- Automatic data acquisition and reporting helps minimize reporting delays and human error;

- Improved safety with reduced congestion on site with fewer people and equipment accessing the site;
- More frequent collection of data; and
- Rapid, automated detection of anomalies and movement.

Inclinometers. Inclinometers are typically used to monitor lateral earth movement below the ground surface. This device consists of a hollow casing placed in a borehole that is typically placed behind the shoring wall or on the backside of a soldier pile at selected locations around the excavation. Inclinometers are monitored regularly during construction. An instrument is lowered down the casing to measure casing deflections at discrete elevations for the entire profile of the casing. Inclinometer casings should extend below the base of the excavation so the bottom is fixed in soil that will not deform due to the shoring system, typically at least about 15 feet below the lowest point of excavation for soil nail walls and attached to the back side of soldier piles for soldier pile walls. Based on the soils, setting, and depth expected for this project, we recommend two inclinometers. One located on the west side of the excavation and the other on the north side.

Shallow Foundations

Outside the tunnel alignment, we expect the structure will be supported on a mat foundation or a combination of continuous and spread footings. For a majority of the building footprint, we anticipate shallow foundations will bear directly on undisturbed, very dense, Glacial Till. However, there appears to be a portion of the upper (western) building level that is underlain by existing fill. For this condition, we recommend overexcavating the fill materials, and replacing with structural fill or lean concrete.

At the time of this report footing sizes and loads are unknown. As design progresses and loads are determined, we should be notified to verify or change our recommendations.

We recommend the following for all shallow foundations:

- Foundations should be founded outside of an imaginary 1H:1V plane projected upward from the bottom edge of adjacent footings or utility trenches. From the bottom edges of the steam tunnel, a 1H:1.5V projection is considered adequate.

- Refer to the **Spring Constants for Foundations** section to design shallow foundation using soil springs.

Mat Foundation

For the design of mat foundations, we recommend:

- Use a maximum allowable bearing pressure of 10 kips per square foot (ksf) for mat foundations bearing on undisturbed, very dense, Glacial Till, or lean concrete that extends to the very dense glacial soils.

- Use a maximum allowable bearing pressure of 6 kips per square foot (ksf) for mat foundations bearing on no more than 3 feet of structural fill placed over the very dense glacial soils.

- Increase allowable bearing pressures by one-third for infrequently applied loads such as seismic or wind forces, as needed.

We expect most of the anticipated settlement will not be time dependent and will occur as the loads are applied. Once the foundations are designed and the design loads are known, we recommend that we be allowed to analyze and estimate post-construction settlements.

It is possible that the result of the structural engineer's analysis for the mat foundation may realize peak edge and corner stresses in the mat that exceed the recommended allowable bearing capacity. We recommend that we be afforded the opportunity to work with the structural engineer to resolve any issues.

Spread Footings

For spread footings bearing on undisturbed, very dense, Glacial Till, or lean concrete that extends to the very dense glacial soils, we recommend:

- For isolated spread footings at least 12 feet by 12 feet in plan dimensions and bearing at least 3 feet below the lowest adjacent grade, use a maximum allowable bearing pressure of 10 ksf.
- For isolated footings as small as 4 feet by 4 feet in plan dimensions and bearing at least 3 feet below the lowest adjacent grade, use a maximum allowable bearing pressure of 6 ksf.
- For strip footings at least 4 feet wide and embedded at least 3 feet below the lowest adjacent grade, use a maximum allowable bearing pressure of 6 ksf.

For spread footings bearing on no more than 3 feet of structural fill placed over the very dense glacial soils, we recommend:

- For isolated spread footings at least 12 feet by 12 feet in plan dimensions and bearing at least 3 feet below the lowest adjacent grade, use a maximum allowable bearing pressure of 6 ksf.
- For isolated footings as small as 4 feet by 4 feet in plan dimensions and bearing at least 3 feet below the lowest adjacent grade, use a maximum allowable bearing pressure of 4 ksf.
- For strip footings at least 4 feet wide and embedded at least 3 feet below the lowest adjacent grade, use a maximum allowable bearing pressure of 4 ksf.

For either bearing condition, we also recommend:

- Linearly interpolate the allowable bearing pressure for footing sizes between 4 and 12 feet.
- Use an increase in the allowable soil bearing pressure of up to one-third for loads of short duration, such as those caused by wind or seismic forces.

Assuming proper subgrade preparation (as described in this report), we expect total settlement of the footings to be less than about 1 inch. Differential settlement is expected to be on the order of half the total settlement. Most of the settlement is expected to occur essentially as the loads are applied.

It may be desirable to size and lay out the footings in a manner that would reduce the potential for differential settlement between adjacent foundation elements. Relatively large individual footings tend to settle more than smaller footings that are loaded to the same bearing pressure. Because of superposition effects of the footing pressures on the supporting soil, footings near the middle of the building will tend to settle more than those near the edges.

Once the foundations are designed and the design loads are known, we recommend that we be allowed to analyze and estimate post-construction settlement.

Lateral Load Resistance

For resistance to lateral loads, use an equivalent fluid density to represent the passive resistance of the soil. For a typical footing poured against *in situ* glacial till above the groundwater table, we recommend an allowable passive equivalent fluid density of 350 pounds per cubic foot (pcf) in a triangular pressure distribution (includes a factor of safety of 1.5). For footings backfilled against with structural fill, we recommend an allowable passive equivalent fluid density of 250 pounds per cubic foot (pcf) in a triangular pressure distribution (includes a factor of safety of 1.5). The equivalent fluid pressure should be applied using triangular pressure distribution, ignoring the passive resistance 2 feet below the adjacent ground surface.

Use an allowable coefficient of friction of 0.30 for footings poured neat on the glacial till for resistance on the base of foundations (includes a factor of safety of 1.5).

Spring Constants for Foundations

Modeling foundation behavior under vertical loads will require modulus of subgrade reaction (vertical spring constant) applicable to the soils on which the foundations bear. Depending on the elevation of the foundation elements, the underlying soil may vary in density and consistency. Loading type, such as static or dynamic loading, has a dramatic effect on the stiffness of the springs. Determining the subgrade modulus value to be used depends on:

- The structural and geotechnical engineer's experience designing similar foundations in similar soil conditions;
- The quantity, magnitude, and area of the mat foundation under various loads; and
- Back-checking settlement predicted from structural modeling with geotechnical settlement estimates for given foundation geometries.

Spring for Static Loading

Mat Foundation. For static loading conditions, we recommend the use of a vertical subgrade modulus (K_s) of 70 pounds per cubic inch (pci) for the mat foundation bearing on glacial till and 50 pci for mat foundations bearing on no more than 3 feet of structural fill. We consider these values a reasonable starting point for an iterative design process. Hart Crowser should review the displacement estimates from the structural model and perform settlement evaluations of the specific geometry and loading for compatibility. Based on these settlement evaluations, modifications to the subgrade modulus used in the structural model may be required.

Spread Footings. For rectangular and strip footings under static loading conditions, we can provide recommended spring constant (vertical subgrade modulus) after the footing sizes are determined, if needed.

Shallow Foundation Preparation and Construction

Careful preparation and protection of the exposed subgrade should occur before concrete placement. Any loosening of the materials during construction could result in larger than estimated settlements. It is important that foundation excavations be cleaned of loose or disturbed soil before placing any concrete and there is no standing water in any foundation excavation. These conditions should be documented before construction.

Based on the design groundwater level and the proposed range of excavation depths, we do not expect the regional groundwater table to affect the excavation. However, perched groundwater may seep into the excavation and collect on the excavation bottom. Wet subgrades are particularly susceptible to loosening and disturbance from foot and equipment traffic.

Hart Crowser's geotechnical engineer or geologist should observe exposed subgrades before footing construction to verify suitable bearing surfaces.

Any loose to medium dense sand or gravel or soft to medium stiff silt present at the subgrade should be overexcavated and replaced with structural fill or lean concrete. Any visible organic or other unsuitable material should be removed from the exposed subgrade.

The foundation settlement estimated herein assumes that careful preparation and protection of the exposed subgrade will occur before concrete placement. Any loosening of the subgrade during construction could result in greater settlement. It is important that all foundation excavations be cleaned of loose or disturbed soil prior to placing any concrete and that there be no standing water in any foundation excavation. Also, groundwater should be controlled such that heave or boiling of the foundation subgrades does not occur. These conditions should be documented before construction.

Considering the conditions anticipated at the bottom of the excavation, it will be necessary to place a nominal 2- to 4-inch-thick "mud slab" consisting of lean concrete immediately after the excavation has been checked by the geotechnical engineer.

These recommendations are based on expected conditions and need to be confirmed in the field.

Deep Foundations

Deep foundations should be installed beneath the building adjacent to the steam tunnel, to transfer the new building loads below the tunnel. We anticipate deep foundations will consist of drilled shafts or augercast piles. The foundations within a 1H:1.5V projection from the bottom of the tunnel should be supported on deep foundations. The side resistance should be neglected above this imaginary plane. We recommend maintaining at least 5 feet clearance between deep foundations and outer edges of the steam tunnel.

Drilled Shafts and Augercast Piles

Axial Capacity and Settlement

For axial capacity of individual drilled shafts and augercast piles, we recommend using the following:

- Allowable unit skin friction (compression or uplift)
  - In existing fill or within a 1H:1.5V projection from the steam tunnel: Neglect
  - In dense/hard native glacial soils: 1.5 ksf
- Allowable unit end bearing in dense/hard native glacial soils: 40 ksf

The total deep foundation settlement consists of elastic settlement that includes compression of the soil beneath the shaft tip and along the shaft as well as elastic shortening of the shaft itself. Settlement at the top of the deep foundation is expected to approach about 1.5 percent of the diameter as the compression load approaches the axial capacity calculated from the recommended allowable resistances.

Group Effects on Axial Capacity

When deep foundations are grouped together, the resulting axial capacity is not necessarily the sum of the capacity of the individual foundation elements. Group effects must be considered for center-to-center spacing of less than 4 diameters. The axial capacity of the group must be reduced by an efficiency factor that depends on the spacing of the foundation elements relative to their diameter. The FHWA (FHWA-NHI-10-016) recommends a group efficiency factor (η) of:

$\eta = 0.65$ for a center-to-center spacing of 2.5 diameters, and

$\eta = 1.0$ for a center-to-center spacing of 4 diameters or more.

Actual value of η applicable to the drilled shafts foundations may be determined by linear interpolation between spacings of 2.5 and 4 diameters. We recommend a center-to-center spacing of at least 3 times the diameter.

Lateral Capacity

Lateral loads, which may be imposed on the piles by wind or earthquake forces, can be resisted by horizontal bearing support of soil adjacent to the piles. The lateral resistance of a deep foundation depends on its length, stiffness in the direction of loading, proximity to other shafts, and degree of fixity at the head, as well as on the engineering properties of the soil. The computer program LPILE is often used to calculate lateral load capacity and deflection for deep foundations. LPILE uses lateral soil reaction (p) and lateral deflection (y) curves generalized from field load tests, along with soil input properties, to approximate lateral pile deflections and moments for piles subjected to an axial load. We recommend using the LPILE soil input parameters in Table 6 for deep foundations.

Table 6 – Soil Parameters for LPILE Input

Soil Layer / Zone Description	Unit Weight (pcf)	Soil Model	Friction Angle (Degrees)	Cohesion (psf)	Static Slope of Soil Modulus, k_s (pci)	Cyclic Slope of Soil Modulus, k_c (pci)
Existing Fill or Above 1H:1.5V Projection Line	Neglect lateral resistance					
Glacial Till and Outwash	135	API Sand	40	--	225	225
Lacustrine	130	Stiff Clay (No Free Water)	--	4,000	1,500	600

For full lateral capacity, we recommend spacing deep foundation at least 6D center-to-center. Deep foundations spaced 5D and closer should be adjusted for group effects, according to Table 7. Interpolation should be used for spacing values other than 3D and 5D.

Table 7 – Group Effects for LPILE Analysis

Shaft Center-to-Center Spacing in the Direction of Loading	P-Multipliers Applicable to LPILE		
	Row 1	Row 2	Row 3 and Higher
3D	0.8	0.4	0.3
5D	1.0	0.85	0.7

Testing of Deep Foundations

We recommend that deep foundations be tested to confirm the integrity of the concrete column and verify that a suitable installation is achieved. This should be completed using thermal integrity profiling (TIP). TIP uses the heat generated by curing cement (hydration energy) to assess the quality of cast-in-place concrete foundations. It can help identify necks, inclusions, poor concrete quality, or bulges in the pile. It is important to conduct integrity testing to ensure the quality of the construction method used.

We recommend at least 10 percent of the production shafts or piles be tested using TIP methods. Each element tested should include a minimum of four thermal wire cables installed the full length of the steel. The wires should be mounted on rebar spaced evenly around the cage.

The pile installation contractor should submit a plan for TIP testing for approval by Hart Crowser. This plan should include the pile locations proposed for testing.

Drilled Shaft Installation

Installation may be challenging because of obstructions, dense soil, or unexpected groundwater. The contractor should review our recommendations and be prepared to address the construction considerations below. If significant variations are observed at any time, we may need to modify our conclusions and recommendations. For drilled shaft installation, we recommend the following.

- Installation of permanent casing is required in the zone above the plane projected at 1H:1.5V from the bottom edges of the steam tunnel to ensure soils are not mobilized adjacent to the tunnel. The inside of the permanent casing may be coated with a bond breaker, such as bitumen, to further reduce load transfer between the shaft and the tunnel.
- Have the contractor review the boring logs thoroughly and choose appropriate drilling methods.
- Difficult drilling may be encountered in glacial soils that are very dense with relatively high cementation and that may contain cobbles, boulders, and gravel or cobble lenses, or in fill materials that may contain buried concrete or other large construction debris. The contractor should be prepared to deal with large obstructions that may be encountered during excavation in these conditions.
- Have the contractor clean slough and other loose material from the bottom of all drilled shafts before placing concrete.
- Tremie the concrete from the bottom of the shaft.
- Clean out the shaft toe no more than 6 hours before placing concrete so suspended solids do not have much time to settle to the toe and reduce its geotechnical stiffness.
- Where multiple drilled shafts are planned within 5 diameters of each other, consider the timing of excavation and concrete placement of the adjacent shafts. Provide the adjacent drilled shaft with adequate cure time, at least 24 hours, before starting to excavate the next drilled shaft. This will not only minimize the potential for communication between adjacent shafts but will also reduce the likelihood of disturbing the set and cure of the concrete in the recently poured shaft.

We recommend having a representative from Hart Crowser on site full time for special inspection of drilled shaft installation. The on-site geotechnical representative should verify that soil conditions encountered during drilled shaft excavation match those assumed during design before concrete is placed. The geotechnical representative should also verify that the shaft is installed according to the project plans and specifications.

Augercast Pile Installation

We recommend that the installation of augercast piles be observed by a Hart Crowser representative to evaluate the contractor’s operation and collect and interpret the installation data. As the completed pile is below the ground surface and cannot be observed during construction, judgment and experience must be used to aid in determining the acceptability of the pile. This also requires use of an augercast pile contractor who is familiar with such installations. We recommend close monitoring of installation procedures, such as installation sequence, auger withdrawal rate, grouting pressure, and quantity of grout used per pile. Variations from the established pattern, such as low grout pressure, excessive settlement of grout in a completed pile, etc., would make the pile susceptible to rejection.

We make the following recommendations for augercast pile installation:

- Do not install two piles within five pile diameters of each other in a single 24-hour period. This is intended to prevent interconnection of grout between piles. Our experience indicates that this minimum 24-hour period may need to be increased, because of the very soft peat layer. This should be evaluated at the time of construction.
- Require the contractor to provide a pressure gauge in the grout line.
- Minimum pressures should be those required to maintain a steady flow of grout to the auger. A typical value of 100 psi should be used for this purpose.
- Rapid drops in the grout pressure of 50 psi or more occurring when otherwise accepted procedures are used should be specified as a possible cause for reconstructing the pile.
- The rate of grout injection and rate of auger withdrawal from the soils should be able to maintain a positive grout head of at least 10 feet above the bottom of the auger. Note that a larger head may be required to counteract the high groundwater table and water pressure at the site.
- Withdraw auger from hole at a slow rate so pressure on the grout column is maintained.
- Require contractor to provide a means of monitoring quantity of grout used per pile. A stroke counter on the group pump is the most efficient means to obtain grout quantity.
- Require the contractor to rotate the auger after initial grout pumping (about 2 cubic feet) prior to the beginning of auger withdrawal.
- At completion of grouting, require the contractor to install a full-length rebar (aka center bar) down the pile center to ensure a properly constructed pile.

Augercast piles will generate soil spoils that will likely need to be disposed of off site. Any environmental considerations that affect disposal of the spoils should be identified before construction.

Tower Cranes and Other Temporary Structures

Design recommendations in this report should not be used for the design of foundations for tower cranes or any other temporary structure to be used during construction. Tower cranes and temporary structures should be the responsibility of the contractor.

Floor Slabs

The lowest floor slab (outside of the mat foundation) may be constructed as slab-on-grade above a drainage layer. We recommend the following for the design of floor slabs with permanent drainage and a passive sump system:

The drainage layer should be at least 6 inches thick. This layer serves as a capillary break and drainage layer and is intended to reduce the potential build-up of hydrostatic pressure beneath the slab and to provide permanent control of groundwater beneath the floor slab and behind the perimeter walls.

We recommend the following for floor slabs:

- Compact the drainage layer to the criteria of structural fill;
- A modulus of subgrade reaction of 150 pci may be used where appropriate for the design of the slab-on-grade using a beam on elastic foundation analysis;
- Any soil that is to be considered as capillary break or drainage material should be submitted to Hart Crowser for gradational analysis and approval; and
- If the bottom of the excavation is soft, wet, or disturbed, the contractor should be prepared to place a temporary working surface (which should not be considered part of the drainage layer).

Methane Mitigation

From our review of the geologic setting, topography, and subsurface explorations, it is our opinion that methane releases associated with the nearby historic Montlake Landfill and surrounding low-lying peat areas do not have a reasonable subsurface pathway to our site. Therefore, a methane mitigation system for the building does not appear necessary from a geotechnical perspective.

Permanent Basement Walls

Permanent walls constructed flush with soldier piles with or without tiebacks temporary shoring systems should be designed for the same active or at-rest lateral soil pressures recommended for shoring design based on the local standard of practice. Figures 6, 7, and 8 provide typical lateral soil pressures for the design of a permanent foundation wall constructed against soldier pile temporary shoring walls. In addition, a uniform seismic load should be applied as discussed below. The structural engineer will need to coordinate with the shoring engineer as final design earth pressures are based on the configuration of the shoring system and adjacent surcharge loads.

Permanent walls constructed flush with soil nail wall temporary shoring systems should be designed for the active or at-rest lateral soil pressures recommended in Figure 6. The seismic increment must be added to these static loads as discussed below.

We do not anticipate retaining walls that are backfilled on one side only to be used at the site; however, if there are such walls, the structural engineer can estimate the lateral load and resistance on the walls using an equivalent fluid to represent the soil. For typical granular fill soil, active and at-rest pressures may be determined using the equivalent fluid unit weights in Table 8. The equivalent fluid soil density does not include any surface loading conditions or loading due to groundwater hydrostatic groundwater pressure; also, the ground surface behind the wall is assumed to be horizontal.

The use of active and passive pressure is appropriate if the wall is allowed to yield a minimum 0.001 times the wall height. For a non-yielding wall, at-rest pressures should be used instead of active pressures.

Table 8 – Soil Equivalent Fluid Unit Weights for Walls Backfilled with Structural Fill

Soil Type	Earth Pressure	Value (pcf)
Structural fill	Active	35
	At-rest	55
	Passive[a]	300

a. Includes a factor of safety of 1.5.

The lateral earth pressures presented herein are based on drained/dewatered conditions so that hydrostatic pressure does not act on the walls.

Seismic Loading against Permanent Foundation Walls

Depending upon the design approach used by the project structural engineers, the lateral earth pressures for permanent foundation walls described above may need to be increased to account for seismic earth pressures. This additional lateral earth pressure can be approximated as a rectangular uniform load. We have assumed level ground conditions for the backslope. We recommend a seismic surcharge against permanent basement walls of 8.5H (where H is the total wall height and the surcharge is in psf) for design to ASCE 7-16.

Surcharge Pressures on Walls

The design of the permanent basement walls should include permanent surcharges in the calculation. Surcharges should include traffic loads, adjacent building foundations and floor slabs, or any other permanent features and should be calculated using the equations on Figure 9.

We recommend Hart Crowser review or complete the estimated surcharge loads when surcharge loads, footprints, and foundation plans of adjacent structures are available.

Permanent Site Retaining Walls

We understand short retaining walls (less than approximately 8 feet in height) may be used to support planned cuts and fills. Because the soils on the site are granular, concrete cantilever (typically cast-in-place) and mechanically stabilized earth (MSE) retaining walls are feasible. In the following sections, we present recommendations for lateral earth pressures, foundations, backfill, and drainage as they relate to cantilevered concrete and MSE walls.

Foundations for Site Retaining Walls

We recommend supporting concrete cantilever walls on shallow foundations. We recommend the following for design of shallow retaining wall footings:

- Design footings to bear on glacial till or on compacted structural fill placed immediately above these natural soils.
- Design footings with a minimum width of 3 feet and a minimum embedment depth (between the bottom of the footing and the adjacent ground surface) of 18 inches. The recommended minimum embedment protects against frost effects.
- Use a maximum allowable bearing pressure of 4.0 ksf. Allowable bearing pressure may be increased by up to one-third for loads of short duration (e.g., wind or seismic loads).
- We expect the bearing soils to behave elastically, with settlement occurring as the design loads are applied or shortly thereafter, and with total settlement of less than 1.0 inch for foundation subgrade prepared as recommended in this report. Differential settlement is likely to be approximately one-half of the total settlement.
- To resist lateral forces, use an allowable coefficient of friction against sliding of 0.30 for footings poured directly on dense granular soil. This value includes a factor of safety of 1.5.
- Require compaction of all exposed footing subgrades to a dense non-yielding condition during construction.
- Hart Crowser should assess and document the suitability of the subgrade during construction, prior to steel rebar and concrete placement.

Lateral Earth Pressures for Site Retaining Walls

Lateral earth pressures depend on the ability of a retaining wall to deform. If the top of the wall is allowed to yield on the order of 0.001 to 0.002 times the height, and if no settlement-sensitive structures or utilities are located in the zone of deformation, the wall may be designed using active earth pressures. If settlement-sensitive structures or utilities exist within the potential zone of deformation, or where the wall system is too stiff to allow sufficient lateral movement to develop an active condition, at-rest earth pressures should be used to design the wall. Theoretically, little movement should occur behind walls properly designed and installed for at-rest conditions.

The following recommendations apply to backfilled retaining walls where drainage is provided behind the wall such that there will be no hydrostatic pressure buildup behind the wall. To estimate lateral pressure on the wall, we recommend the following:

- For yielding backfilled walls, use an equivalent active fluid density of 35 pcf for areas where the ground is level at the top of the wall. For sloped areas above the wall, use equivalent active fluid density of

35(H+h/2) pcf, where H is the height of the wall and h represents the height of the slope above the wall and h is no more than 6 feet.

- For non-yielding backfilled walls and the permanent building wall (i.e., walls for which allowable deflection is less than 0.001 times the height of the wall), use an equivalent fluid density of 55 pcf to compute at-rest earth pressures.

- For seismic loading conditions, use a rectangular seismic surcharge of 8.5H in psf, where H is the height of the wall the surcharge is acting over. This surcharge is based on the design seismic event as described in the **Seismic Considerations** section of this report.

The active or at-rest pressures should extend to the base of the wall system.

- If construction or vehicular traffic is present above the wall, a 2-foot surcharge should be included in the design.

Backfill for Site Retaining Walls

Backfill soil should consist of a well-graded structural fill that meets WSDOT Standard Specification 9-03.12(2), Gravel Backfill for Walls. It should be placed in 8- to 10-inch loose lifts and compacted to at least 95 percent of modified Proctor maximum dry density as determined by the American Society for Testing and Materials (ASTM) D 1557 test procedure. Compaction within 2 feet of the walls should be performed with small hand-operated equipment to avoid imparting excess horizontal stresses on the wall due to compaction. Within this zone, compaction criteria may be reduced to 92 percent.

Onsite soil may be reused as backfill for cast-in-place walls provided that it conforms to the definition outlined in the **Structural Fill** section of this report.

Drainage for Site Retaining Walls

Lateral earth pressures recommended in this section do not consider hydrostatic pressure. Therefore, we recommend providing drainage behind the wall. To prevent lateral hydrostatic pressure buildup against the wall, a free-draining granular material (less than 3 percent passing the US No. 200 sieve based on the minus 3/4-inch fraction) should be used within an 18-inch-wide zone immediately behind the wall. The drainage material should meet WSDOT Standard Specification 9-03.12(3), Gravel Backfill for Drains. The fill should be continuous and hydraulically connected to a drainage system at the base of footings supporting the wall. The system should incorporate a perforated drain pipe with a minimum diameter of 4 inches. The pipe should be surrounded by at least 6 inches of free-draining material. Drain pipes should include cleanouts and the drain holes or slots in the pipe should be compatible with the surrounding drainage material.

Mechanically Stabilized Earth Walls

Construction of MSE walls generally consists of compacting a block or mass of soil in lifts, placing reinforcing strips between the lifts, and placing wall-facing panels or vegetation on the face of the wall.

Numerous systems using this basic design principal are available. Successful construction and performance of MSE walls depends on several factors, such as:

- Suitability of supporting subgrade soils;
- Presence and quantity of suitable drainage that is able to prevent water from building up behind the wall;
- Type, length, and spacing of reinforcement strips used;
- Type and installation method of wall facing;
- Surcharge loads and compaction effort near the wall face during construction;
- Consistency of the fill soil; and
- Attention to construction details, especially the facing's connection to the reinforcement strips.

MSE Wall Design

In our experience, it is typically more economical for the vendor of the MSE wall materials to design the MSE wall for internal stability with our input and review. We verify adequate global stability and compound stability once a vendor has a general idea of reinforcement strip geometry. We recommend the following:

- Design the MSE walls in general accordance with "Mechanically Stabilized Earth Walls and Reinforced Soil Slope Design & Construction Guidelines," dated March 2001 (FHWA-NHI-00-043; link http://www.fhwa.dot.gov/engineering/geotech/library_listing.cfm).
- Design the length and spacing of reinforcing layers in the MSE wall so the wall is stable against sliding, overturning, bearing capacity failure, overall slope instability, and internal instability (i.e., breaking reinforcement and pullout of reinforcement).
- Design the MSE wall for stability in both static and seismic events.
- Use a friction (phi) angle of 34 degrees and a unit weight of 125 pounds per cubic foot for the compacted structural fill that makes up the MSE wall. These values assume use of granular fill free of organic material, placed and compacted to the degree presented in the **Structural Fill** section of this report. The frictional strength of the fill material will need to be determined early in the MSE design stage. Soil properties should be confirmed and the design modified, if necessary, once actual fill materials are identified. Backfill used within the reinforced zone of an MSE wall should meet WSDOT Standard Specification 9-03.14(4), Gravel Borrow for Structural Earth Wall.
- Account for lateral pressures on the wall due to a seismic event as described in the **Seismic Considerations** section of this report.

- Involve Hart Crowser in the bid process to assist with the selection of qualified MSE designers and the means of bidding the MSE design to obtain economical bids.
- Involve Hart Crowser in the design process as soon as possible to assist the design team and expedite MSE design and permitting.
- Retain Hart Crowser to review the MSE wall designer's design calculations, specifications, and plans to check for conformance with geotechnical recommendations.

We understand that MSE wall design is applicable for reinforced soil with face slopes equal to or steeper than 70 degrees. Face slopes flatter than 70 degrees can generally be designed more economically as reinforced soil slopes.

Construction Dewatering

The regional groundwater table is expected to be well below the bottom of the excavation; however, limited amounts of perched groundwater may be encountered during excavation. Drainage of perched water will need to be accommodated by the shoring system during excavation. Similarly, a suitable drainage system will need to be considered.

Most perched zones will produce minor amounts of seepage into the excavation. The primary source of water in the perched zones is from precipitation, but leaking utilities can also act as a source. The amount of seepage is also expected to vary seasonally; less seepage is expected during summer and fall and more during winter and spring.

Perched groundwater should drain by seeping out through the shoring system, and sumps and ditches in the bottom of the excavation are expected to be sufficient for construction excavation.

Geotechnical Impacts of Dewatering

Geotechnical impacts of dewatering are primarily related to dewatering-induced settlement outside of the excavation. Given the dense nature of the soils underlying the site, and the deep regional groundwater table, we expect dewatering-induced settlement to be negligible. The amount of settlement that occurs depends on the soil conditions, as well as on the amount and duration of dewatering.

Permanent Drainage

As noted above, the groundwater table sits below the lowest proposed footing elevation; however, limited amounts of perched groundwater may be encountered and should be considered in the permanent drainage design. Rainfall, surface water, and groundwater from adjacent utility trenches can also increase short-term water discharge rates; therefore, we recommend installing a permanent drainage system to accommodate a minimal discharge of approximately 5 gallons per minute (gpm).

Drainage for Walls Installed Against Shoring

We recommend the following for permanent drainage behind the basement walls, perimeter of walls, and foundations. We make the following recommendations in case a drained system is adopted:

- Miradrain-type composite panels should be laid flush on the shoring wall (on the outside of the permanent wall) and connected to a collector pipe that runs along the foundation, at an elevation lower than the bottom of the floor slab. This will allow water collected outside the wall to be tight-lined beneath the slab and into a central drainage sump. We recommend installing a minimum of one strip of paneling between each soldier pile from the top of the wall down the full face of the wall to drain any perched water.

- Perimeter drains should be installed near the base of the perimeter wall foundation. The perimeter drains should be a minimum 4-inch-diameter perforated pipe and should be surrounded by 6 inches of drainage material. All pipes should be sloped to drain.

- As noted above, all slabs-on-grade should be underlain by a capillary break/drainage layer at least 6-inches thick that is hydraulically connected to the perimeter drains. This layer should consist of well graded, free-draining sand and gravel with less than 3 percent fines. To prevent fines from clogging the drainage layer, a non-woven geotextile fabric is required at the base of the layer. The drainage layer is intended to reduce the potential buildup of hydrostatic pressures beneath the slab.

- All wall and perimeter drainage pipes should be connected to a central underslab sump, complete with an appropriate sump pump.

Backfilled Retaining Walls

Walls with soil backfilled on only one side will require drainage. We recommend:

- Backfill immediately behind the wall with a minimum thickness of 18 inches of well graded, free-draining sand or sand and gravel.

- Install drains behind any backfilled subgrade walls. Drains, with cleanouts, should consist of a minimum 4-inch-diameter perforated pipe placed on a bed of, and surrounded by, 6 inches of free-draining (less than 3 percent passing the U.S. No. 200 mesh sieve based on minus 3/4-inch fraction), well-graded sand or sand and gravel. The drains should be sloped to carry the water to a sump or other suitable discharge.

- Wall drainage can also consist of Miradrain-type composite panels laid flush on the outside of the permanent wall and connected to a collector pipe that runs along the footing, at an elevation lower than the bottom of the floor slab. This will allow water collected outside the wall to be tight-lined beneath the slab and into the central drainage sump.

- The drainage backfill should be continuous and envelop the drainage pipe behind the wall.

Subslab Drainage

As previously mentioned in **Floor Slabs**, all slabs should be underlain directly, everywhere, by a 6-inch-thick drainage layer hydraulically connected to the perimeter drains. This layer serves as a capillary break and drainage layer and is intended to reduce the potential buildup of hydrostatic pressure beneath the slab and to provide permanent control of groundwater beneath the floor slab and behind the perimeter walls. We also recommend the following for subslab drainage:

- The drains with cleanouts should consist of 4-inch-diameter perforated pipe wrapped in filter fabric and placed on a bed of, and surrounded by, 6 inches of clean free-draining sand and gravel. The drains should be sloped to carry the water to a sump or other suitable discharge.

- Compact the drainage layer to the criteria of structural fill with less than 3 percent by weight passing the U.S. No. 200 mesh sieve based on the material passing the 3/4-inch sieve.

- Submit any soil considered for use as capillary break or drainage material to Hart Crowser for gradational analysis.

- Provide subslab drainage by using perimeter drains around the building.

Site Drainage

Final grades should be sloped to carry surface water runoff away from structures to prevent water from infiltrating near foundation walls. Roof drainage and new pavement drainage should be tied into the storm drainage system and should not be tied into the subdrain system or discharge onto the site slopes.

Stormwater Infiltration

On-site soils generally have a high fines content and are unlikely to allow water to infiltrate quickly. We do not recommend using a stormwater infiltration system for the proposed development, since this could cause a buildup of water above the fine-grained glacial soils and potentially decrease the stability of the slope in the project area or downslope of the project area.

Permanent Slopes

Permanent cut and fill slopes should be adequately inclined and revegetated to minimize long-term raveling, sloughing, and erosion. A vegetative groundcover should be established as soon as possible following grading to further protect the slope from runoff water erosion. We generally recommend that permanent slopes not be steeper than 2H:1V to minimize long-term erosion and to facilitate revegetation. Final grading near the top of permanent slopes should be such that surface water is directed away from the slope face.

Site Paving

New site paving is planned, which is expected to include asphalt paving for vehicular traffic and pedestrian walkways/sidewalks consisting of concrete or asphalt pavement sections. Based on our observations at the boring locations, pavement at the site consists of 2- to 4-inch-thick asphalt.

Pavement Subgrade Preparation

All pavement sections should be supported on a minimum thickness of 12 inches of well-compacted subgrade. Subgrade may consist of compacted structural fill, natural soils that have been compacted in-place or dense, natural soil. In general, the pavement sections can be supported on the existing near-surface soil provided: (1) it is compacted to a dense, non-yielding condition by a heavy vibratory roller; (2) all unsuitable material including organic matter, construction debris, boulders, etc., is removed; and (3) the final surface is proof-rolled with a fully loaded dump truck. When structural fill is required, the upper 2 feet of material beneath the pavement section should be compacted to at least 95 percent of the modified Proctor maximum dry density (ASTM D1557).

We recommended that Hart Crowser observe pavement subgrade proof-rolling prior to placement of the pavement section to confirm that the surface is firm and non-yielding.

Pavement Design

We recommend the minimum sections in Table 9 for full-depth concrete or asphalt pavements.

Table 9 – Recommended Minimum Pavement Sections

Pavement Location / Use	Compacted Subgrade Thickness [a]	Base Course Thickness [b]	Pavement Thickness [c, d]
Vehicular Paving	Min. 12 inches	Min. 6 inches	Min. 8 inches PCC, or Min. 4 inches AC
Pedestrian Walkways / Sidewalks	Min. 12 inches	Min. 6 inches	Min. 4 inches PCC, or Min. 3 inches AC

Notes:

a. See **Pavement Subgrade Preparation**.

b. WSDOT Specification 9-03.9(3) Crushed Surfacing Base Course (WSDOT 2020), compacted to a minimum of 95 percent of the maximum dry density as determined by the modified Proctor (ASTM D1557) test method. Material specification provided as a suggestion; alternative materials may be acceptable.

c. PCC: Portland Cement Concrete

d. AC: Asphalt Concrete

e. Reinforcing elements should be considered to limit cracking in PCC pavements, if desired. We only considered pavement thickness in our evaluations.

Recommended pavement sections are based on our experience on site, our understanding of past performance of existing pavements, and review of City of Seattle construction standard minimums. A formal pavement design was not completed; this would require traffic volume and design life information. Hart Crowser can perform a formal pavement design, if desired, when this information becomes available.

Earthwork

Site Preparation and Grading

Site preparation for the building footprint will involve demolishing existing buildings and foundations, removing pavement, removing of obstructions in the fill that may interfere with construction, and excavating to the foundation level. We recommend all site grading, paving, and any utility trenching be conducted during relatively dry weather.

It may be necessary to relocate or abandon some utilities. Excavation of these utility lines will occur through fill materials. Abandoned underground utilities should be removed or completely grouted. Remaining abandoned utility lines should be sealed to prevent piping of soil or water into the utility pipe. Soft or loose backfill materials should be removed, and excavations should be backfilled with structural fill. Coordination with the utility owners is generally required in addressing existing utilities.

Temporary Open Cuts

The stability and safety of cut slopes depends on a number of factors, including:

- The type and density of the soil;
- The presence and amount of any seepage;
- Depth of cut;
- Proximity of the cut to any surcharge loads near the top of the cut, such as stockpiled material, traffic loads, structures, etc., and the magnitude of these surcharges;
- Duration of the open excavation; and
- Care and methods used by the contractor.

The Occupational Safety and Health Administration (OSHA) classification of the site soils is Type C. We make the following recommendations regarding open cuts for Type C soils.

- The maximum allowable slope for excavations less than 20 feet deep is 1.5H:1V.
- Protect the slope from erosion by using plastic sheeting.
- Limit the maximum duration of the open excavation to the shortest time period possible.
- Place no surcharge loads (equipment, materials, etc.) within 10 feet of the top of the slope.

Because of the variables involved, actual slope angles required for stability in temporary cut areas can only be estimated prior to construction. We recommend that stability of the temporary slopes used for construction be the responsibility of the contractor, since the contractor is in control of the construction operation and is continuously at the site to observe the nature and condition of the subsurface. All excavations should be made in accordance with all local, state, and federal safety requirements.

Structural Fill

Backfill placed within the building area or below paved areas should be considered structural fill. We make the following recommendations for structural fill:

- For imported soil to be used as structural fill, use a clean, well-graded sand or sand and gravel with less than 5 percent by weight passing the No. 200 mesh sieve (based on the minus 3/4-inch fraction). Compaction of soil containing more than approximately 5 percent fines may be difficult if the material is wet or becomes wet during rainy weather.

- Place and compact all structural fill in lifts with a loose thickness no greater than 10 inches. For hand-operated "jumping jack" compactors, loose lifts should not exceed 6 inches. For small vibrating plate/sled compactors, loose lifts should not exceed 3 inches.

- Compact all structural fill to at least 95 percent of the modified Proctor maximum dry density (as determined by ASTM D1557 test procedure).

- Control the moisture content of the fill to within 2 percent of the optimum moisture. Optimum moisture is the moisture content corresponding to the maximum Proctor dry density.

- In wet subgrade areas, clean material with a gravel content of at least 30 to 35 percent may be necessary. Gravel is material coarser than a US No. 4 sieve.

- At least one week before filling begins, provide samples of the structural and drainage fill for laboratory testing. Laboratory testing will include a Proctor test and gradation for structural fill and a gradation for drainage fill. Field testing with a nuclear density gauge uses the maximum dry density determined from a Proctor test, therefore it is important to complete the laboratory testing as soon as possible in order to not delay backfilling.

Use of On-Site Soil as Structural Fill

The suitability of excavated site soils for compacted structural fill will depend upon the gradation and moisture content of the soil when it is placed. As the amount of fines (that portion passing the No. 200 sieve) increases, the soil becomes increasingly sensitive to small changes in moisture content and adequate compaction becomes more difficult to achieve. Soil containing more than approximately 5 percent fines cannot be consistently compacted to a dense non-yielding condition when the water content is greater than approximately 2 percent above or below optimum. Reusable soil must also be free of organic and other deleterious material.

In our opinion, the on-site glacial till soils are generally unsuitable for reuse as structural fill. Depending upon weather conditions (i.e., if construction occurs during the summer months) it may be possible to use the fill and glacial outwash from the cuts as backfill in areas of landscaping. These materials should not be used as fill below structures, pavements, and utilities.

In the proposed building areas, much of the upper soils appear to be a silty sand fill with a relatively moderate percentage of fines. It may be possible, depending upon weather conditions, to reuse these soils as structural fill but only if they meet the specific requirements of structural fill as outlined above. For planning purposes, the use of import material should be assumed.

Utilities

Utility trench cut design should generally be the contractor's responsibility. For shallow trench excavations (less than 4 feet deep), open cutting may be used provided the side walls are stable. Use of trench boxes or temporary shoring may be necessary for unstable side wall conditions or if deeper excavations are required for placement of utilities. The contractor should verify the conditions of the side slopes during construction and slope back trench cuts as necessary to conform to current standards of practice and safety requirements.

Our recommendations for bedding and trench backfill materials are summarized in Table 10 and described in the following section. The minimum dry densities recommended are a percentage of the modified Proctor maximum dry density as determined by the ASTM D1557 test procedure.

Table 10 – Material Specifications for Utility Trenching and Installation

Use	Material Specification [a]
Structural Fill	See **Structural Fill** section
Pipe/utility vault bedding	WSDOT 9-03.12(3) Gravel Backfill for Pipe Zone Bedding
Pipe zone backfill	WSDOT 9-03.12(3) Gravel Backfill for Pipe Zone Bedding
Trench/vault backfill	WSDOT 9-03.15 Native Material for Trench Backfill
Trench/vault backfill (settlement sensitive areas)	WSDOT 9-03.19 Bank Run Gravel for Trench Backfill

Note:

a. Material specifications are provided as a suggestion. Alternative materials may be acceptable.

Pipe and Utility Vault Bedding

At least 4 inches of bedding material is recommended for all utility pipes. For bedding material beneath catch basins and manholes, we recommend at least 6 inches. The bedding materials should meet requirements of WSDOT Specification 9-03.12(3), Gravel Backfill for Pipe Zone Bedding (WSDOT 2020), except the amount passing the No. 200 sieve should be less than 3 percent (based on the minus 3/4-inch fraction). The bedding materials should be compacted to at least 90 percent.

Pipe Zone Backfill

The pipe zone extends from the top of the bedding to 6 inches above the top of the utility pipe. The pipe zone backfill should meet the requirements recommended for bedding material. The backfill material used should meet the specific gradation requirements associated with the utility being installed.

Utility Trench/Vault Backfill

The recommendations for the trench backfill (extending from the top of the pipe zone) depend on the location of the utility trenches. Utility trenches outside of the roadway prism or building footprint can be backfilled with compacted on-site native material as long as it meets the requirements of WSDOT Specification 9-03.15, Native Material for Trench Backfill (WSDOT 2020). Utility trenches inside the roadway prism or building footprint can be backfilled with a compacted import gravel material meeting the requirements of WSDOT Specification 9-03.19, Bank Run Gravel for Trench Backfill (WSDOT 2020).

In settlement-sensitive areas (such as paved areas), the upper 2 feet of backfill should be compacted to at least 95 percent. Below the upper 2 feet, backfill should be compacted to at least 90 percent.

Compaction Equipment

We recommend using hand-operated compaction equipment within 12 inches of any pipe, catch basin, or similar structure to reduce risk of damage. The contractor should be responsible for selecting appropriate compaction equipment and adjusting the lift thickness of the backfill as needed to avoid damage to the pipe.

RECOMMENDEDATIONS FOR CONTINUING GEOTECHNICAL SERVICES

Recommendations discussed in this report should be reviewed and modified, if necessary, as project elements progress through final design. As part of final design, we recommend that Hart Crowser:

- Continue to meet with the design team as needed to address geotechnical questions that may arise as the design progresses;
- Review anticipated foundation settlements and foundation springs based on the structural engineer's actual foundation plan and loads;
- Prepare a final geotechnical engineering design study report; and
- Review geotechnical aspects of the final design plans and earthwork specifications to see that our recommendations were properly interpreted and implemented in the design documents.

During the construction phase of the project, we recommend that Hart Crowser review contractor submittals and provide a representative to observe:

- Excavation and installation of the shoring system;
- Excavation and installation of deep foundations;
- Excavation and preparation of the subgrade for shallow foundations and slabs-on-grade;
- Installation and commissioning of a temporary construction dewatering system, if required;
- Installation of the permanent subslab and wall drainage system;
- Utility installation;
- Placement and testing of compacted material; and
- Other geotechnical engineering considerations that may arise during the course of construction.

The purpose of our observations is to verify compliance with geotechnical design concepts and recommendations and to allow design changes or evaluation of appropriate construction methods in the event that subsurface conditions differ from those anticipated prior to the start of construction.

REFERENCES

ASCE 2017. Minimum Design Loads and Associated Criteria for Buildings and Other Structures, ASCE Standard ASCE/SEI 7-16.

FHWA 1999. Geotechnical Engineering Circular No. 4 – Ground Anchors and Anchored Systems, June 1999. Report No. FHWA-IF-99-015.

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PTI 2014. Recommendations for Prestressed Rock and Soil Anchors, Fifth Edition, Post Tensioning Institute, PTI DC35.1-14.

SDCI GIS 2021. Seattle Department of Construction & Inspections GIS, accessed June 3, 2021. <https://seattlecitygis.maps.arcgis.com/apps/webappviewer/index.html?id=f822b2c6498c4163b0cf908e2241e9c2>

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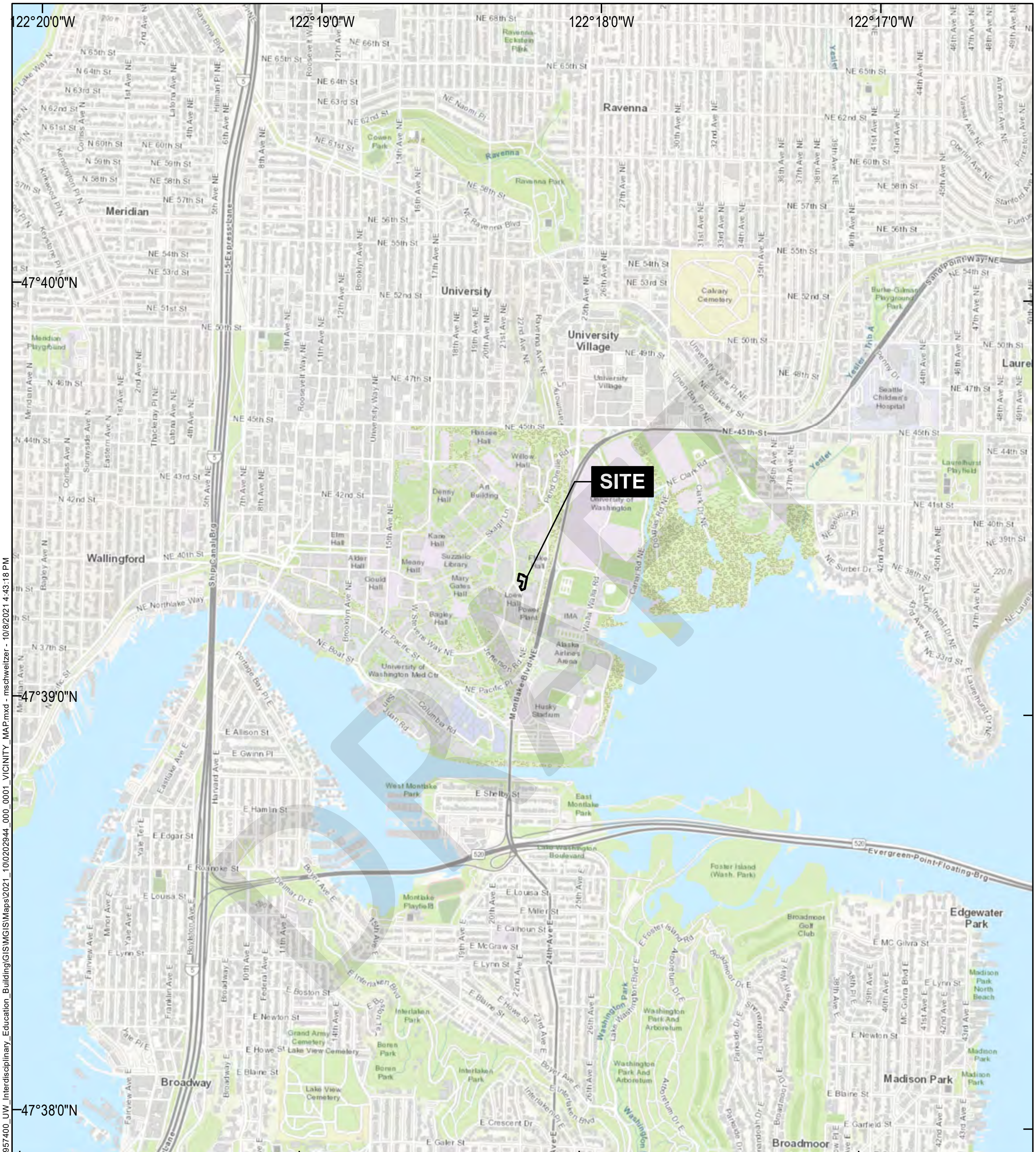
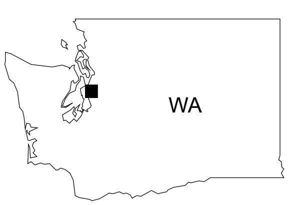
\\haleyaldrich.com\share\sea_projects\Notebooks\1957400_UW_Interdisciplinary_Education_Building\Deliverables\Reports\Geotech Draft Report Rev1\2021_1029_HCHA_UW_IERBDraftGeotechnicalDesignReport_D_Rev1.docx





MAP SOURCE: ESRI
SITE COORDINATES: 47°39'18"N, 122°18'14"W

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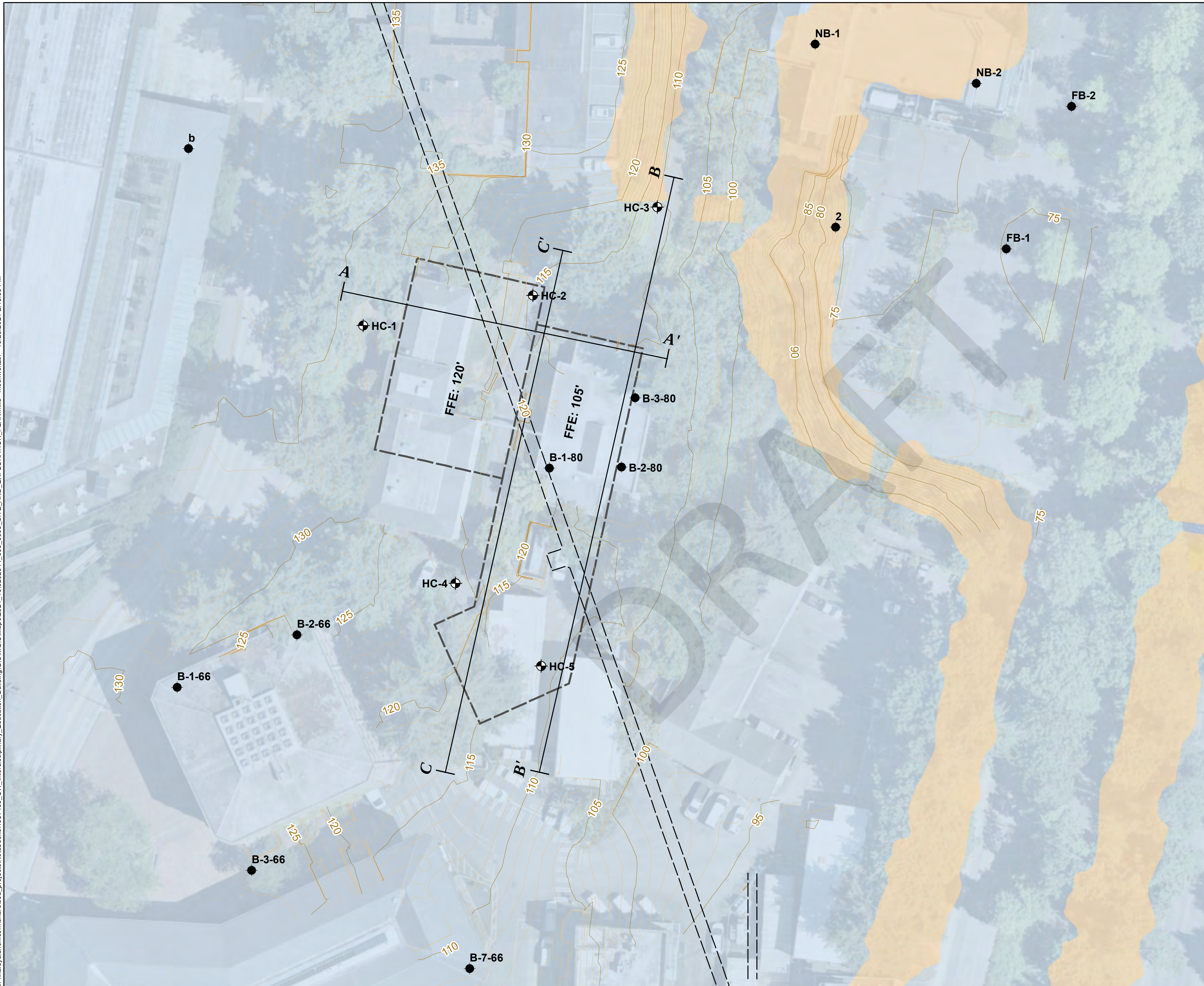
VICINITY MAP

APPROXIMATE SCALE: 1 IN = 2000 FT
OCTOBER 2021

FIGURE 1

GIS: \\haleyaldrich.com\share\sea_projects\Notebooks\1957400_UW_Interdisciplinary_Education_Building\GIS\MGIS\Maps\2021_10\0202944_000_0001_VICINITY_MAP.mxd - mschweitzer - 10/8/2021 4:43:18 PM



LEGEND

- BORING
- HISTORICAL EXPLORATION
- STEAM TUNNEL
- CROSS SECTION
- ELEVATION CONTOUR (NAVD 88)
- PROPOSED BUILDING
- STEEP SLOPE (40% AVERAGE)
- 1000' METHANE BUFFER

NOTES

1. AERIAL IMAGERY SOURCE: NEARMAP, 01 JUNE 2021
2. SURVEY ELEVATION CONTOUR SOURCE: OTAK, 20 JANUARY 2021
3: STEEP SLOPE AND MATHANE BUFFER SOURCE: SEATTLE CITY GIS OPEN DATA

N

0 50 100
SCALE IN FEET

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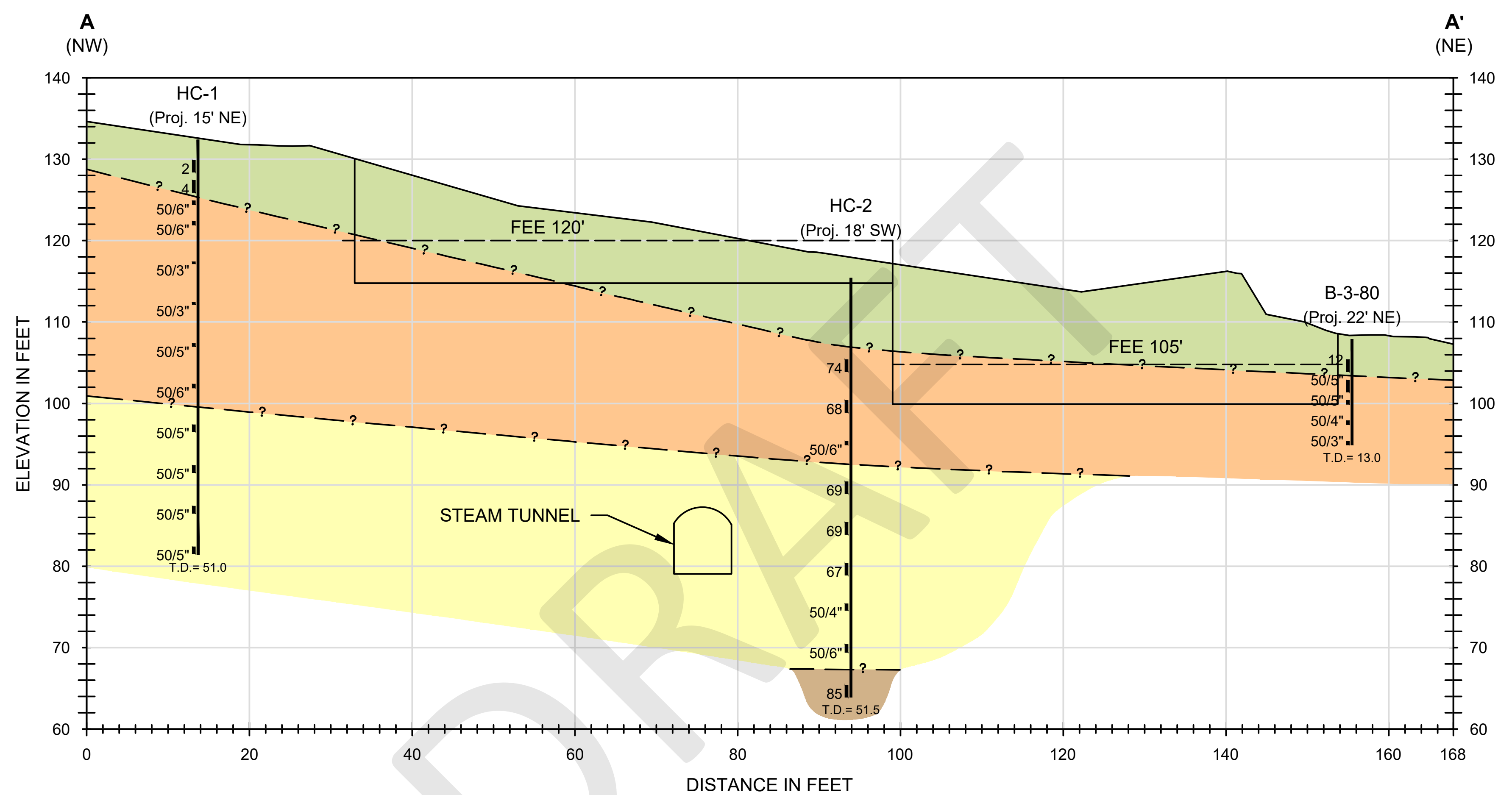
SEATTLE, WASHINGTON

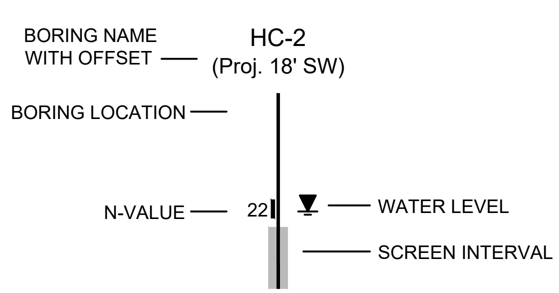
SITE AND EXPLORATION PLAN

OCTOBER 2021

FIGURE 2

GIS:\\haleyaldrich.com\share\sea_projects\Notebooks\1957400_UW_Interdisciplinary_Education_Building\GIS\MGIS\Maps\2021_10\0202944_000_0003_SITE_AND_EXPLORATION_PLAN.mxd - mschweitzer - 10/25/2021 2:15:09 PM





LEGEND

- FILL
- VERY DENSE SAND AND GRAVEL (GLACIAL TILL)
- VERY DENSE SAND (OUTWASH)
- HARD SILT AND CLAY (LACUSTRINE)
- APPROXIMATE LIMITS OF EXCAVATION
- PROPOSED FINISHED FLOOR ELEVATION

NOTES

1. SURFACE PROFILE LINE SOURCE: S33313B190.DWG PROVIDED BY OTAK, 30 SEPTEMBER 2021.
2. THIS SUBSURFACE PROFILE IS GENERALIZED FROM MATERIALS OBSERVED IN SOIL BORINGS. VARIATIONS MAY EXIST BETWEEN PROFILE AND ACTUAL CONDITIONS.
3. FEATURE LOCATIONS ARE APPROXIMATE.

0 20 40
SCALE IN FEET

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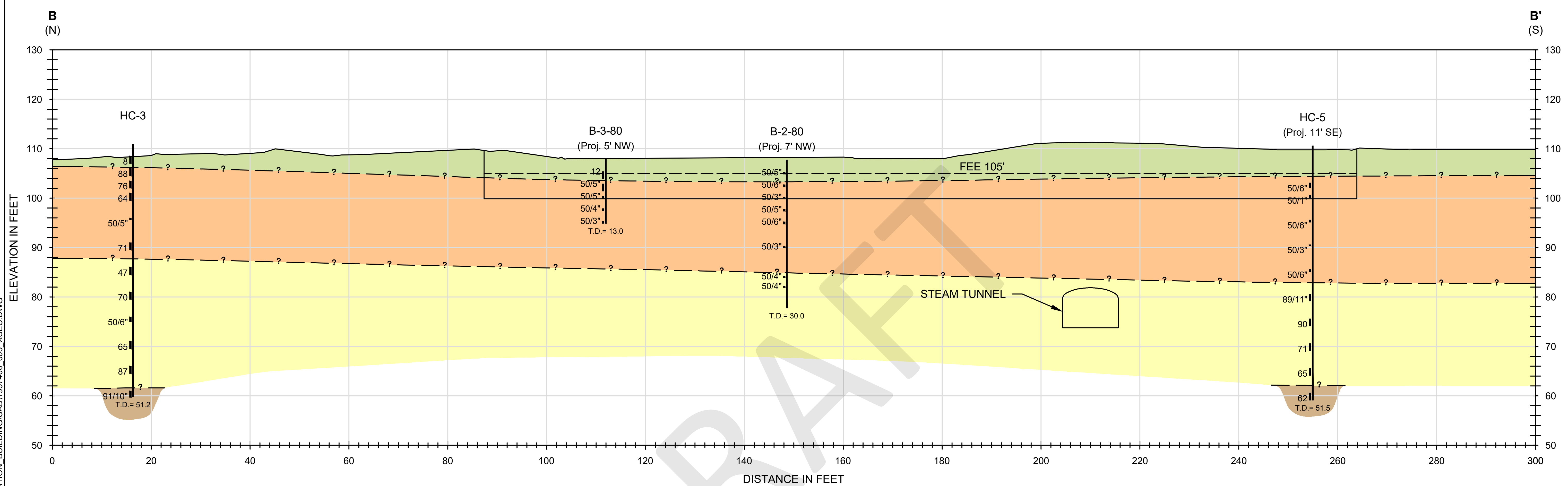
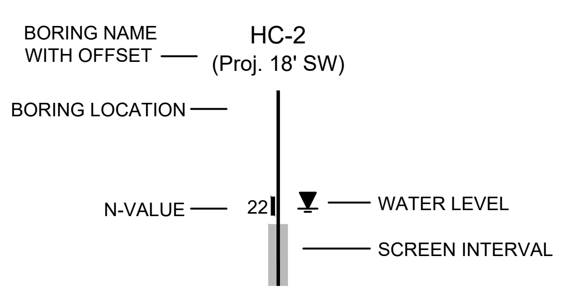
SUBSURFACE PROFILE A-A'

OCTOBER 2021

FIGURE 3





LEGEND

- FILL
- VERY DENSE SAND AND GRAVEL (GLACIAL TILL)
- VERY DENSE SAND (OUTWASH)
- HARD SILT AND CLAY (LACUSTRINE)
- APPROXIMATE LIMITS OF EXCAVATION
- PROPOSED FINISHED FLOOR ELEVATION

NOTES

1. SURFACE PROFILE LINE SOURCE: S33313B190.DWG PROVIDED BY OTAK, 30 SEPTEMBER 2021.
2. THIS SUBSURFACE PROFILE IS GENERALIZED FROM MATERIALS OBSERVED IN SOIL BORINGS. VARIATIONS MAY EXIST BETWEEN PROFILE AND ACTUAL CONDITIONS.
3. FEATURE LOCATIONS ARE APPROXIMATE.

0 20 40
SCALE IN FEET

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SUBSURFACE PROFILE B-B'

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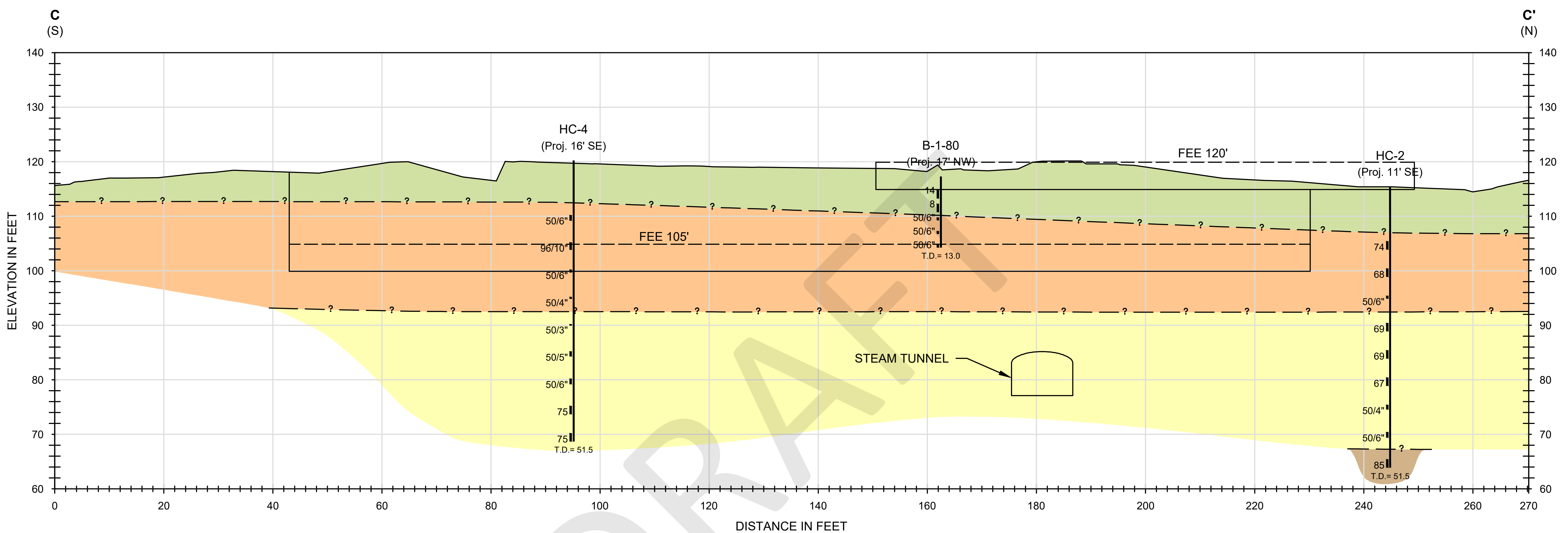
FIGURE 4



BORING NAME WITH OFFSET — HC-2 (Proj. 18' SW)

BORING LOCATION

N-VALUE — 22

WATER LEVEL

SCREEN INTERVAL

LEGEND

- FILL
- VERY DENSE SAND AND GRAVEL (GLACIAL TILL)
- VERY DENSE SAND (OUTWASH)
- HARD SILT AND CLAY (LACUSTRINE)
- APPROXIMATE LIMITS OF EXCAVATION
- PROPOSED FINISHED FLOOR ELEVATION

NOTES

1. SURFACE PROFILE LINE SOURCE: S33313B190.DWG PROVIDED BY OTAK, 30 SEPTEMBER 2021.
2. THIS SUBSURFACE PROFILE IS GENERALIZED FROM MATERIALS OBSERVED IN SOIL BORINGS. VARIATIONS MAY EXIST BETWEEN PROFILE AND ACTUAL CONDITIONS.
3. FEATURE LOCATIONS ARE APPROXIMATE.

0 20 40
SCALE IN FEET

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SUBSURFACE PROFILE C-C'

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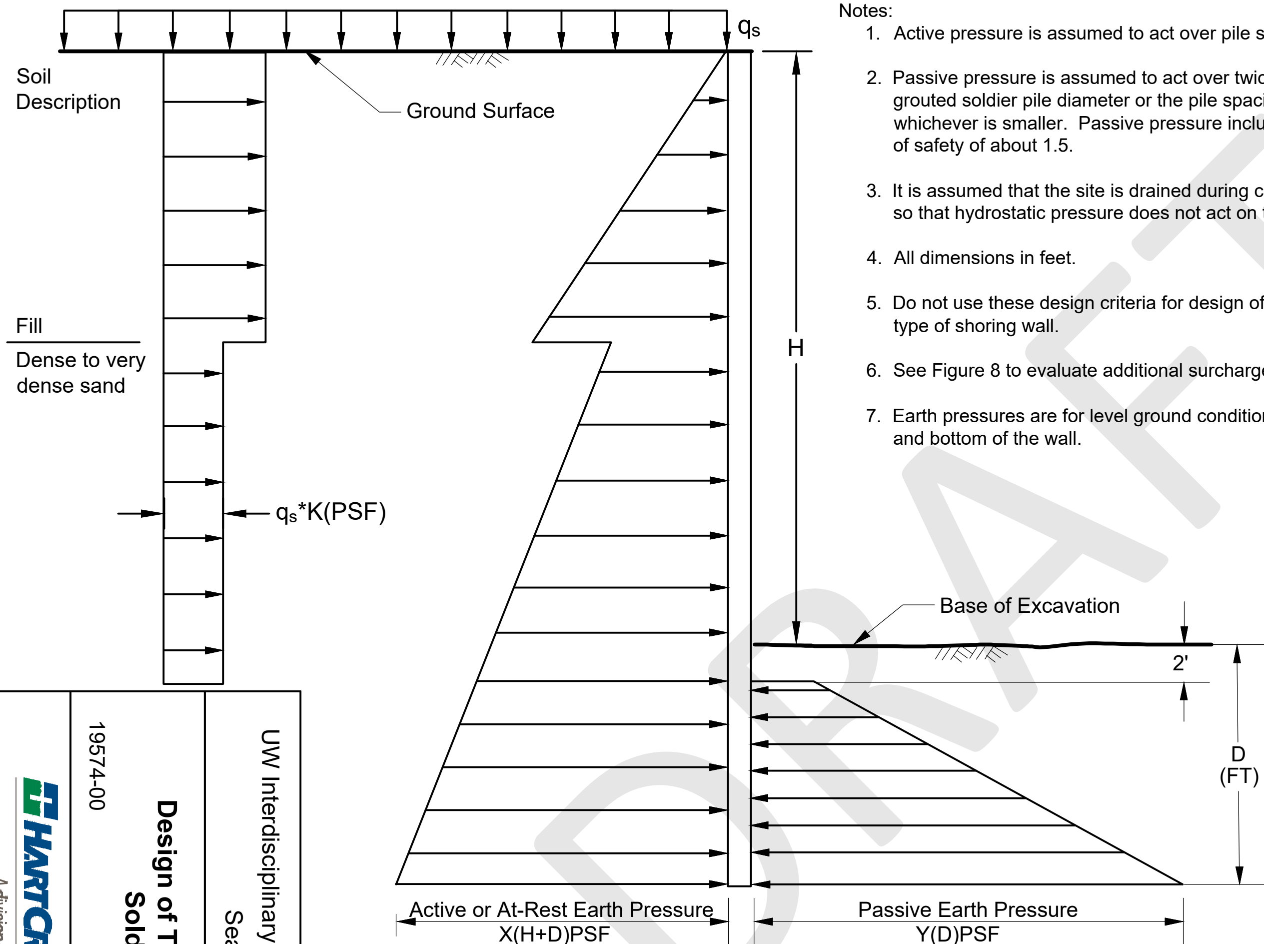
FIGURE 5

A.Lateral Soil Pressure - Temporary Cantilevered Shoring



Notes:

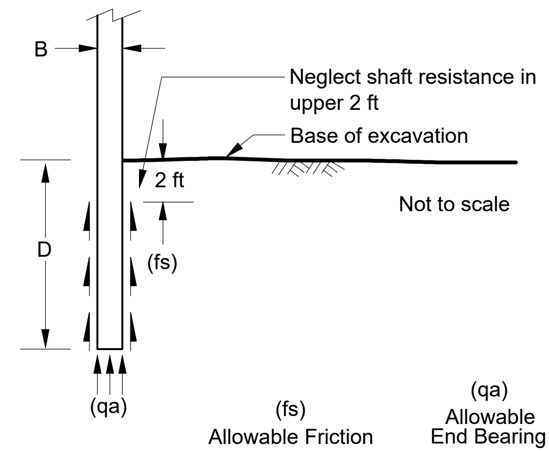
1. Active pressure is assumed to act over pile spacing.
2. Passive pressure is assumed to act over twice the grouted soldier pile diameter or the pile spacing, whichever is smaller. Passive pressure includes factor of safety of about 1.5.
3. It is assumed that the site is drained during construction so that hydrostatic pressure does not act on the walls.
4. All dimensions in feet.
5. Do not use these design criteria for design of any other type of shoring wall.
6. See Figure 8 to evaluate additional surcharge.
7. Earth pressures are for level ground conditions at the top and bottom of the wall.

Recommended Values of X, PSF		
Soil Unit	**Active**	**At-Rest**
Fill	40	60
Dense to very dense sand	30	50

Dense to very dense sand, qs = 250 psf (Traffic and Temporary Loads) + Additional Surcharges

Not to Scale

B.Vertical Capacity of Soldier Pile



	(fs) Allowable Friction	(qa) Allowable End Bearing
Dense to very dense sand	1.5 KSF	$\frac{8D}{B} \le 40$ KSF

Recommended Minimum Embedment Depth
10 Feet below Base of Excavation

Recommended Values of K		
Soil Unit	**Active**	**At-Rest**
Fill	0.33	0.5
Dense to very dense sand	0.22	0.35

Recommended Values of Y, PCF	
Soil Unit	**Passive**
Dense to very dense sand	400

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Design of Temporary Cantilevered Soldier Pile Shoring

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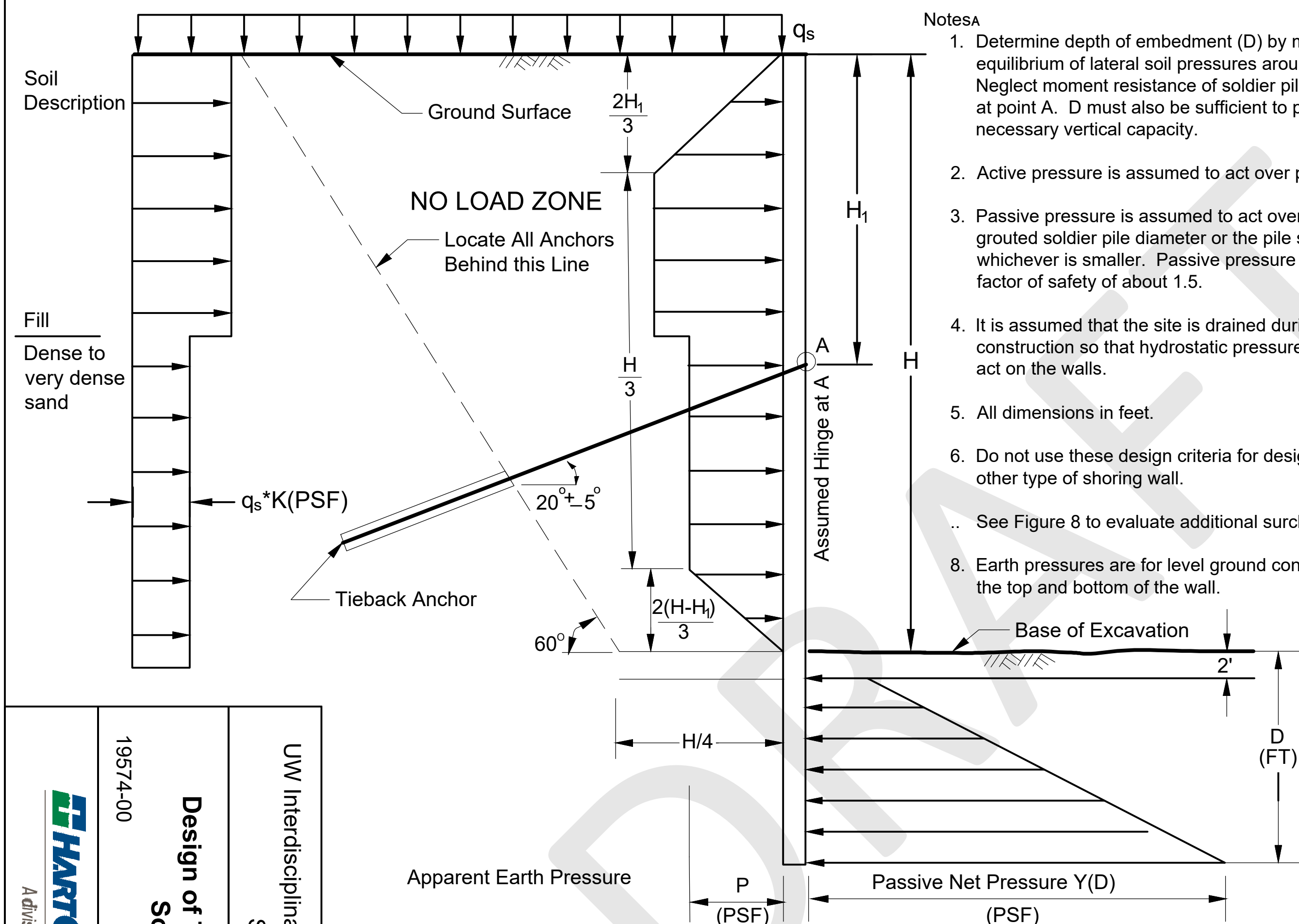
Figure
6

A. Lateral Soil Pressures - Temporary Shoring with One Row of Tiebacks



NotesA

1. Determine depth of embedment (D) by moment equilibrium of lateral soil pressures around point A. Neglect moment resistance of soldier pile member at point A. D must also be sufficient to provide necessary vertical capacity.
2. Active pressure is assumed to act over pile spacing.
3. Passive pressure is assumed to act over twice the grouted soldier pile diameter or the pile spacing, whichever is smaller. Passive pressure includes factor of safety of about 1.5.
4. It is assumed that the site is drained during construction so that hydrostatic pressure does not act on the walls.
5. All dimensions in feet.
6. Do not use these design criteria for design of any other type of shoring wall.

.. See Figure 8 to evaluate additional surcharge.

8. Earth pressures are for level ground conditions at the top and bottom of the wall.

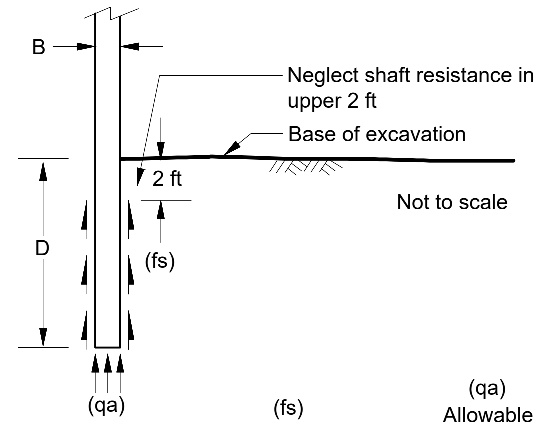
Not to Scale

Recommended Values of P, PSF		
Soil Unit	**Active**	**At-Rest**
Fill	40H	60H
Dense to very dense sand	30H	50H

Dense to very dense sand, qs = 250 psf (Traffic and Temporary Loads) + Additional Surcharges

Recommended Values of K		
Soil Unit	**Active**	**At-Rest**
Fill	0.33	0.5
Dense to very dense sand	0.22	0.35

B. Vertical Capacity of Soldier Pile



	(fs) Allowable Friction	(qa) Allowable End Bearing
Dense to very dense sand	1.5 KSF	$\frac{8D}{B} \leq 40$ KSF

Recommended Minimum Embedment Depth
10 Feet below Base of Excavation

C. Tentative Temporary Anchor Pullout Resistance

	Friction (Adhesion)
Dense to very dense sand	4.0 KLF
Hard silt and clay	3.0 KLF

Verify with Load Test to 200% of Design Stress Level.

(KLF: Kips per Lineal Foot)

Recommended Values of Y, PCF	
Soil Unit	**Passive**
Dense to very dense sand	400

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Design of Temporary Single-Support Soldier Pile Shoring

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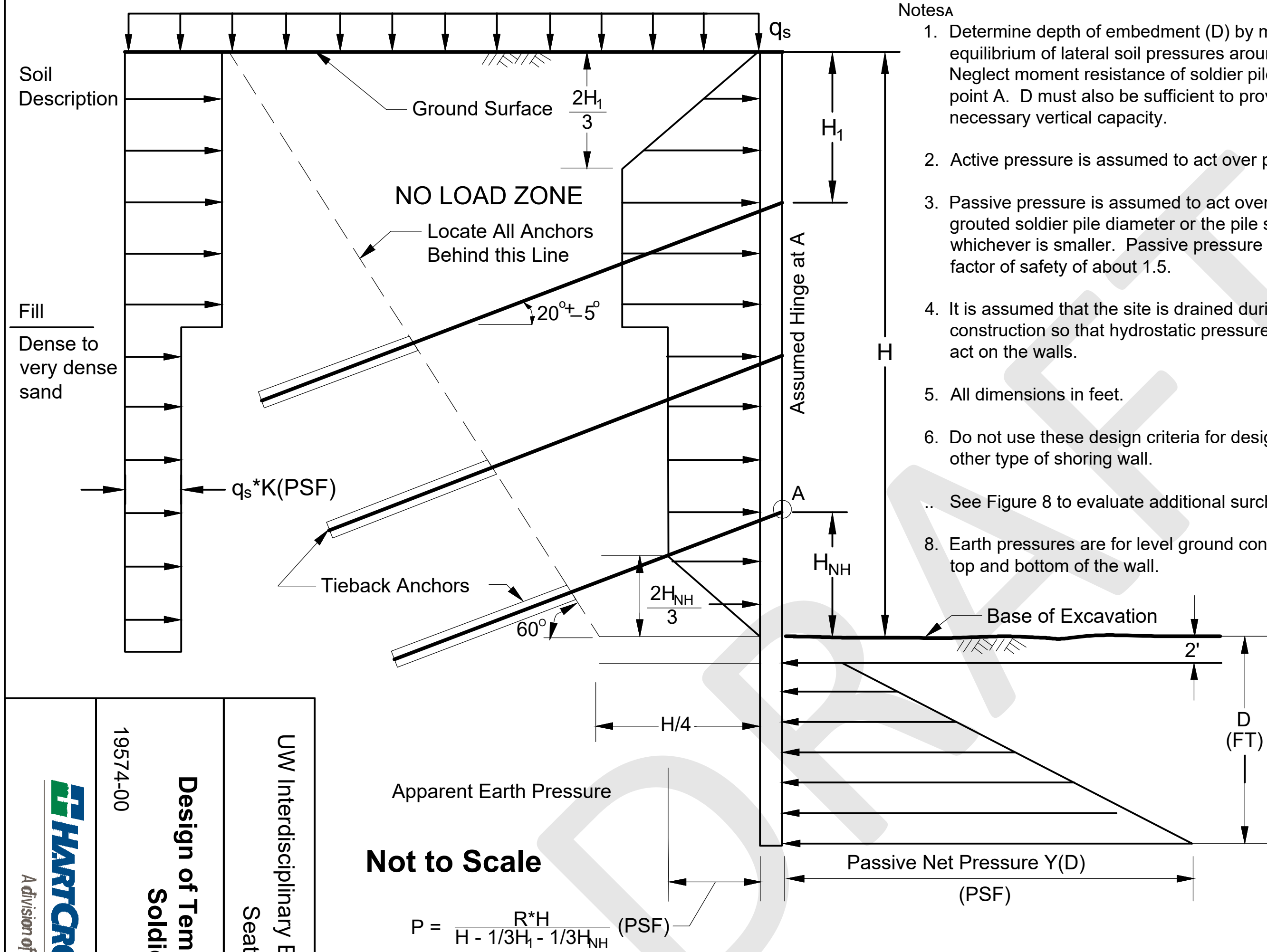
Figure
7

A. Lateral Soil Pressures - Temporary Shoring with Multiple Rows of Tiebacks



$$P = \frac{R*H}{H - 1/3H_1 - 1/3H_{NH}} \text{ (PSF)}$$

NotesA

1. Determine depth of embedment (D) by moment equilibrium of lateral soil pressures around point A. Neglect moment resistance of soldier pile member at point A. D must also be sufficient to provide necessary vertical capacity.
2. Active pressure is assumed to act over pile spacing.
3. Passive pressure is assumed to act over twice the grouted soldier pile diameter or the pile spacing, whichever is smaller. Passive pressure includes factor of safety of about 1.5.
4. It is assumed that the site is drained during construction so that hydrostatic pressure does not act on the walls.
5. All dimensions in feet.
6. Do not use these design criteria for design of any other type of shoring wall.

.. See Figure 8 to evaluate additional surcharge.

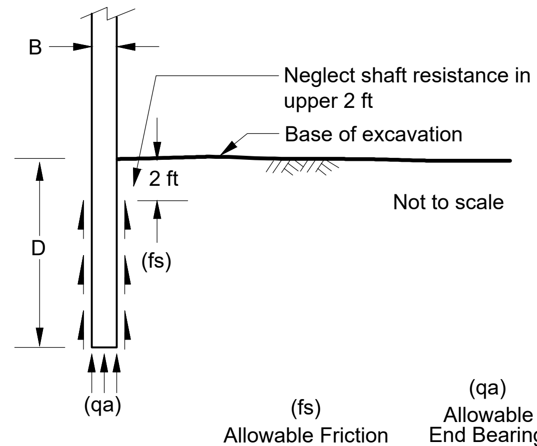
8. Earth pressures are for level ground conditions at the top and bottom of the wall.

Recommended Values of R, PSF		
Soil Unit	**Active**	**At-Rest**
Fill	28H	42H
Dense to very dense sand	22H	35H

Dense to very dense sand, qs = 250 psf (Traffic and Temporary Loads) + Additional Surcharges

Recommended Values of K		
Soil Unit	**Active**	**At-Rest**
Fill	0.33	0.5
Dense to very dense sand	0.22	0.35

B. Vertical Capacity of Soldier Pile



	(fs) Allowable Friction	(qa) Allowable End Bearing
Dense to very dense sand	1.5 KSF	$\frac{8D}{B} \leq 40$ KSF

Recommended Minimum Embedment Depth
10 Feet below Base of Excavation

C. Tentative Temporary Anchor Pullout Resistance

	Friction (Adhesion)
Dense to very dense sand	4.0 KLF
Hard silt and clay	3.0 KLF

Verify with Load Test to 200% of Design Stress Level.

(KLF: Kips per Lineal Foot)

Recommended Values of Y, PCF	
Soil Unit	**Passive**
Dense to very dense sand	400

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Design of Temporary Multiple-Support Soldier Pile Shoring

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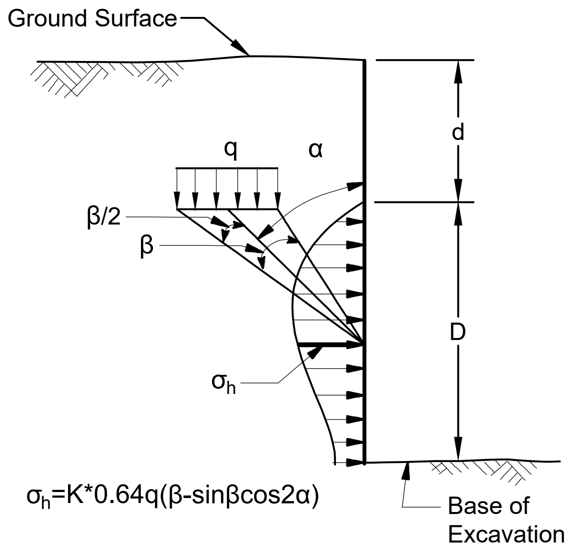
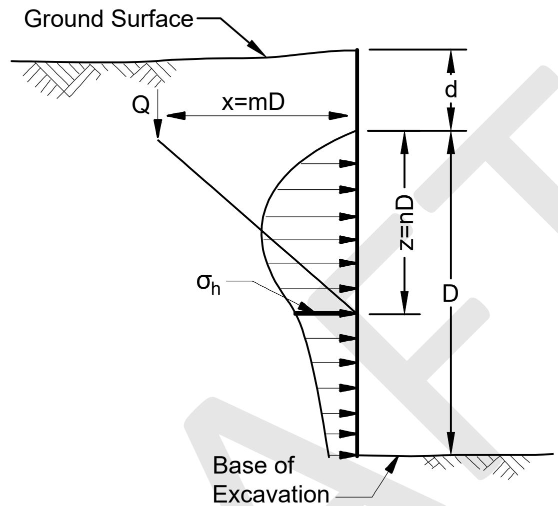
Figure
8

A. Strip Footing Cross Section View



$\sigma_h = K * 0.64q(\beta - \sin\beta \cos 2\alpha)$

B(1). Small Isolated Footing Cross Section View



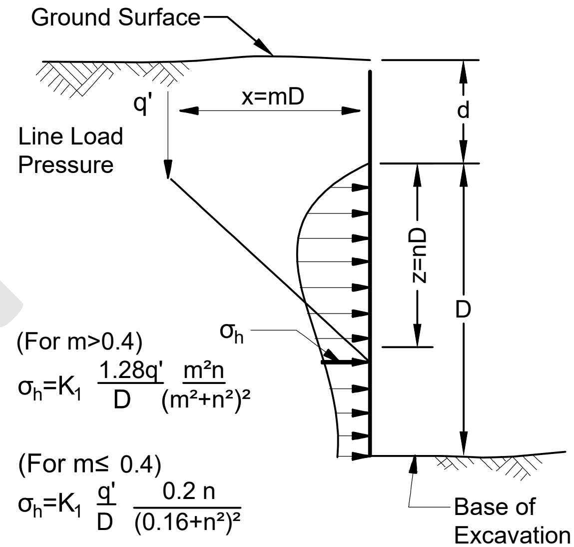
(For m>0.4)

$$\sigma_h = K_1 \frac{1.77Q}{D^2} \frac{m^2 n^2}{(m^2+n^2)^3}$$

(For m≤ 0.4)

$$\sigma_h = K_1 \frac{0.28Q}{D^2} \frac{n^2}{(0.16+n^2)^3}$$

C. Continuous Wall Footing Parallel to Excavation Cross Section View



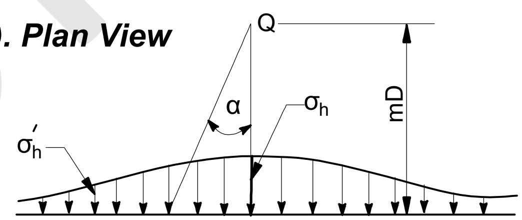
(For m>0.4)

$$\sigma_h = K_1 \frac{1.28q'}{D} \frac{m^2 n}{(m^2+n^2)^2}$$

(For m≤ 0.4)

$$\sigma_h = K_1 \frac{q'}{D} \frac{0.2\,n}{(0.16+n^2)^2}$$

B(2). Plan View



$\sigma_h' = \sigma_h \cos^2 (1.1\alpha)$

Definition and Units

Q	Footing Load in Pounds
D	Excavation Depth below Footing in Feet
d	Depth to Base of Footing in Feet
σ_h	Lateral Soil Pressure in PSF
q	Unit Loading Pressure in PSF
q'	Footing Load in Pounds per Foot
α, β	Radians

K_1	Conditions
0.35	Active earth pressure on a flexible wall (e.g., shoring)
0.5	At-rest conditons, where surcharge loads exist prior to excavation
1.0	At-rest conditions, where surcharge loads are applied after construction on permanent wall

Notes:
1. Lateral pressures from adjacent structures should be added to lateral pressures on Figures 5, 6, and 7.
2. Wall footings acting other than parallel to the excavation can be treated as series of discrete point loads, using Diagram B.
3. Contact Hart Crowser for surcharge recommendations, if necessary.

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Surcharge Pressures Determination of Lateral Pressure Acting on Adjacent Shoring

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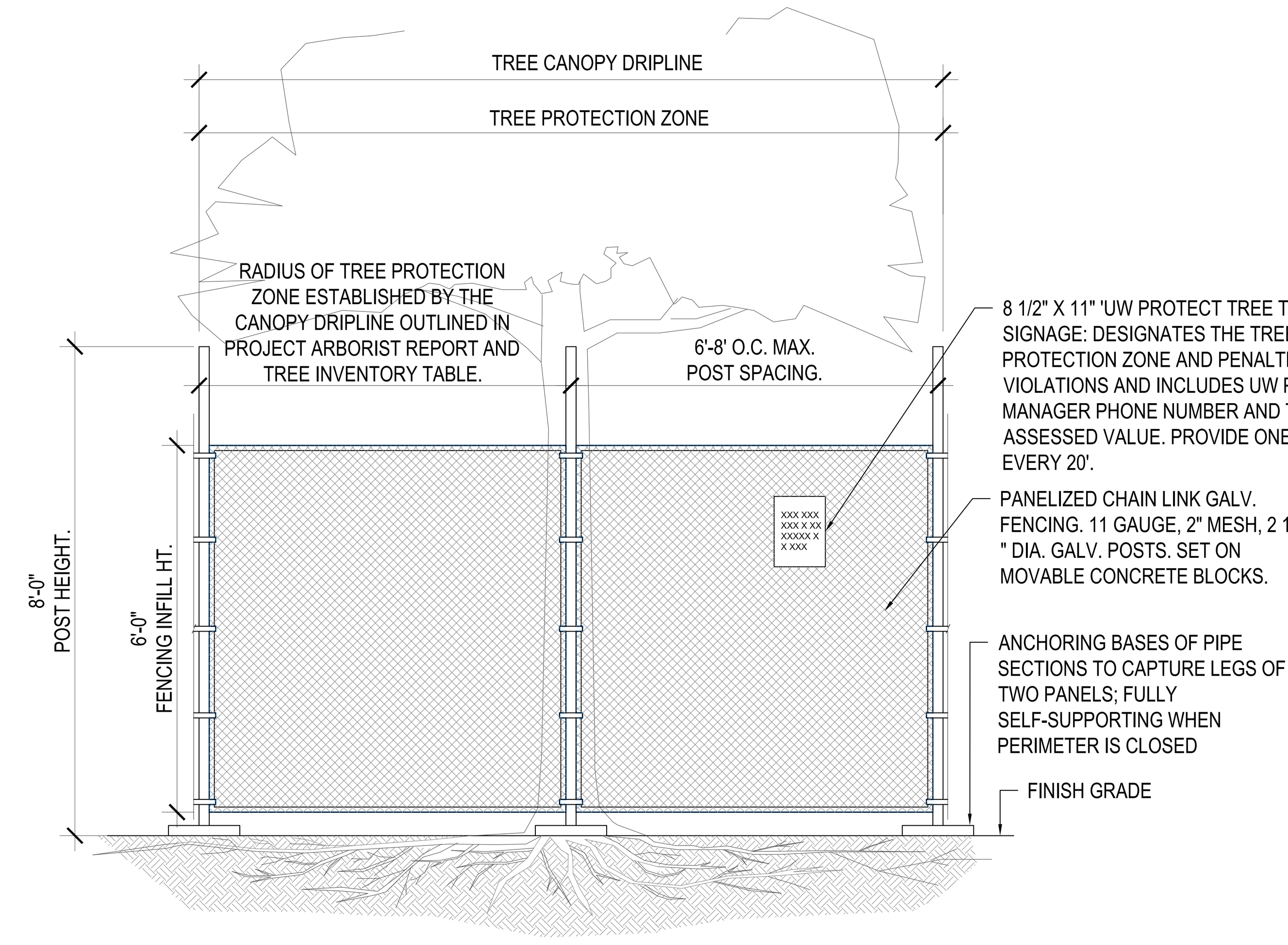
Figure
9

Geotechnical Report Appendices A-C

Available Upon Request

Exhibit D
Tree Removal Plan



NOTES:

1) NO SOIL GRADE CHANGES, COMPACTION OR STORAGE OF MATERIALS SHALL BE ALLOWED WITHIN THE TREE PROTECTION ZONE. THE FOLLOWING CONSTRUCTION ACTIVITIES SHALL NOT BE ALLOWED WITHIN THE TREE PROTECTION ZONE:
   - STORAGE OR STAGING OF MATERIALS OR EQUIPMENT
   - DUMPING OF REFUSE
   - VEHICLE PARKING
   - EXCAVATION, TRENCHING, OR FILL
2) TREE PROTECTION FENCING SHALL BE INSTALLED PRIOR TO DEMOLITION OR GROUND DISTURBANCE AND KEPT IN PLACE FOR THE DURATION OF CONSTRUCTION.
3) TREE PROTECTION ZONE MAY EXTEND BEYOND DRIPLINE. ARBORIST'S RECOMMENDATIONS MAY EXTEND PROTECTION MEASURES BEYOND THE SCOPE OF THIS DETAIL.
4) EXISTING CONDITIONS AND TOPOGRAPHY OF THE SITE VARY. ALTERNATIVE ANCHORING METHODS MAY BE CONSIDERED, INCLUDING HEAVY-DUTY ORANGE PLASTIC MESH FENCING FABRIC, 48" WIDTH - ANCHORED IN PLACE WITH NO SAGGING AT STEEP SLOPES OR FENCE PANELS ANCHORED WITH STAKES DRIVEN INTO THE GROUND, IN LIEU OF FREE-STANDING BLOCKS, AT SPECIFIC SITE ADJACENCIES. ALTERNATE PROTECTION SHALL ONLY BE IMPLEMENTED AT THE RECOMMENDATION AND APPROVAL OF THE ARBORIST.
5) LOCKABLE, 3' WIDE GATES FOR EACH FENCE TYPE AND EACH SEPARATE FENCED AREA ARE REQUIRED.
6) PROTECTIVE FENCING IS REQUIRED WHEN THE WORK AREA IS WITHIN THE TREE DRIPLINE, EXCEPT WHERE PORTIONS OF THE ROOT ZONE ARE COVERED WITH PAVEMENT SUCH AS STREETS OR WALKS.
7) WORK DONE WITHIN THE TREE DRIPLINE MUST MINIMIZE ROOT DISTURBANCE. SPECIAL CARE SHALL BE TAKEN DURING EXCAVATION AND REMOVAL OF EXISTING CURB, GUTTER, AND SIDEWALKS TO AVOID DAMAGE TO TREE ROOTS. LOCATE EXISTING TREE ROOTS USING HAND TOOLS OR OTHER APPROVED METHODS SUCH AS AIRSPADE.
8) NO ROOT OVER 2" SHALL BE CUT WITHOUT APPROVAL OF AN APPROVED ARBORIST. ROOTS SHALL BE CUT WITH APPROVED SAWS. NO ROOTS OVER 2" SHALL BE CUT OR TORN DURING TRENCHING WITH POWER EQUIPMENT SUCH AS BACKHOES AND TRENCHERS. UTILITY LINES AND IRRIGATION OR OTHER PIPES SHALL BE INSTALLED BY HAND DIGGING OR TUNNELING UNDER ROOTS AS NECESSARY TO AVOID CUTTING ROOTS 2" AND LARGER.
9) USE 3 INCHES OR DEEPER WOOD CHIP MULCH OUTSIDE FENCED AREAS TO PROTECT FEEDER ROOTS.
10) FINAL TREE PROTECTION REQUIREMENTS WILL BE DETAILED IN THE FINAL ARBORIST REPORT AND INCORPORATED INTO THE PLAN SET.

TREE PROTECTION AND REMOVAL SCHEDULE

XXXX — TREE — XXXX= TREE I.D. NUMBER PER SURVEY

XXXX — TREE TO BE REMOVED — XXXX= TREE I.D. NUMBER PER SURVEY

XXXX — EXCEPTIONAL TREE TO BE REMOVED — XXXX= TREE I.D. NUMBER PER SURVEY

XXXX — TREE TO BE RETAINED — XXXX= TREE I.D. NUMBER PER SURVEY

XXXX — EXCEPTIONAL TREE TO BE RETAINED — XXXX= TREE I.D. NUMBER PER SURVEY

TREE DRIPLINE / INNER ROOT ZONE — TREE ROOT PROTECTION ZONE

TREE PROTECTION FENCING

CONSTRUCTION FENCE

APPROXIMATE LIMITS OF WORK

APPROXIMATE LIMITS OF WORK OF ASSOCIATED PERMIT

L-001 SHEET NOTES:

1. ALL VEGETATION WITHIN TREE PROTECTION FENCING TO BE PRESERVED TEMPORARILY. ANY VEGETATION TO BE REMOVED/PRESERVED WITHIN FENCING WILL BE ADDRESSED WITH PROJECT PLANTING PLAN AND SPECIFICATIONS.
2. REFER TO ARBORIST REPORT DATED JULY 19, 2021.

N
SCALE: 1" = 20'
0 10 20 40

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PROJECT NO.: KT 932

UNIVERSITY of WASHINGTON

UW INTERDISCIPLINARY ENGINEERING BUILDING (IEB)

UNIVERSITY OF WASHINGTON
4002 NORTHEAST STEVENS WAY
SEATTLE, WA 98195
UW PROJECT NUMBER
UW FACNUM

SDCI PERMIT NUMBER: 6857016-DM
4002 EAST STEVENS WAY NE

REVISIONS

NO.	DATE	DESCRIPTION

DEMOLITION PERMIT

NOT FOR CONSTRUCTION

DATE: 05 NOVEMBER 2021
DRAWN BY: DM
CHECKED BY: PB

TREE PROTECTION & REMOVAL PLAN

L-001

11/4/2021 11:07:02 AM

Exhibit D
Tree Removal Plan

ANALYSIS PROVIDED IN ARBORIST REPORT DATED 210719 BY TREE SOLUTIONS, INC.

TREE PROTECTION PLAN DATED 11/05/2021

Tree ID	Scientific Name	Common Name	DSH (inches)	DSH Multistem	Health Condition	Structural Condition	Dripline Radius (feet) N	E	S	W	Exceptional Threshold	Exceptional by Size	Exceptional Grove	Notes	Tree Remove/Retain Notes
6267	*Gleditsia triacanthos*	Honeylocust	10.2		Good	Good	27.4	18.4	12.4	13.9	*20.0*	-		Limited soil volume	Outside Limit of Work
6281	*Cedrus deodara*	Deodar cedar	40.0		Good	Good	29.2	46.2	35.2	29.7	*30.0*	Exceptional			Outside Limit of Work
6282	*Tsuga heterophylla*	Western hemlock	24.7		Good	Good	24.5	29.5	15.5	12.5	*24.0*	Exceptional		Compacted soils near base, slight lean that is corrected	To be retained
6283	*Ulmus procera*	English elm	26.0		Good	Good	28.6	20.6	22.1	26.1	*30.0*	-		Leaf miner, ivy at base, compacted soils	To be retained
6284	*Castanea sativa*	European chestnut	18.7		Good	Good	15.3	14.3	24.3	25.3	*30.0*	-		Corrected lean, shared canooy with tree 6283	To be retained
6285	*Tilia cordata*	Littleleaf linden	24.1	16.5,17.6	Good	Good	17.0	21.0	27.0	19.0	*30.0*	-		Codominant at base with inckuded bark, uniin looks stable	To be retained
6286	*Chamaecyparis pisifera*	Sawara cypress	23.0		Good	Good	16.5	12.5	11.5	18.5	*26.9*	-		Slightly suppressed, ivy at base	To be retained
6287	*Thuja plicata*	Western redcedar	16.3		Good	Good	7.7	10.2	13.2	16.7	*30.0*	-			To be retained
6289	*Thuja plicata*	Western redcedar	9.4		Fair	Good	12.9	10.9	6.4	15.9	*30.0*	-		Compacted soils, surface roots, suppressed	To be retained
6290	*Thuja plicata*	Western redcedar	19.9		Good	Good	12.8	12.8	14.8	18.8	*30.0*	-			To be retained
6291	*Thuja plicata*	Western redcedar	20.0		Fair	Fair	20.8	15.8	15.8	4.8	*30.0*	-		Limited growing space, unlikely to survive demo, large diameter pruning wounds	To be removed due to site improvements
6292	*Prunus serrulata*	Flowering cherry	7.8		Fair	Fair	12.8	10.3	7.3	3.3	*23.0*	-		Dieback in canopy	To be removed due to site improvements
6293	*Ginkgo biloba*	Gingko biloba	10.1		Good	Good	7.9	14.4	16.4	19.9	*24.0*	-		Growing under building, not likely to survive demo, not a good longterm spot	To be removed due to site improvements
6294	*Prunus serrulata*	Flowering cherry	8.9		Fair	Fair	4.4	10.4	6.4	9.4	*23.0*	-		Large pruning wounds and dieback, in decline	To be removed due to site improvements
6295	*Gleditsia triacanthos*	Honeylocust	12.2		Fair	Good	19.5	16.0	12.0	21.5	*20.0*	-		Dieback in canopy, ivy at base, limited soil volume, large diameter pruning cut on previous	To be removed due to building construction
6298	*Thuja plicata*	Western redcedar	32.8		Fair	Fair	13.9	21.4	19.6	17.2	*30.0*	Exceptional		Decay at base, ivy at base, previous failures and reiterating branches, low retention value	To be removed due to building construction
6299	*Cedrus deodara*	Deodar cedar	26.0		Good	Good	8.1	27.1	27.6	12.1	*30.0*	-			To be removed due to building construction
6300	*Cedrus deodara*	Deodar cedar	27.6		Good	Good	29.2	19.7	13.2	26.2	*30.0*	-		One reiterating branch, canopy over existing building, heavy ivy at base	To be removed due to building construction
6301	*Cedrus deodara*	Deodar cedar	26.0		Fair	Good	12.1	19.1	24.6	28.1	*30.0*	-		Heavy ivy at base, slightly thin canopy	To be removed due to building construction
6302	*Cedrus deodara*	Deodar cedar	25.3		Good	Fair	31.1	12.6	16.1	35.1	*30.0*	-		Heavy ivy, slightly thin canopy. codominant trunks near top of canopy	To be removed due to building construction
6303	*Fraxinus pennsylvanica*	Green ash	6.8		Poor	Good	10.8	15.3	10.8	13.8	*30.0*	-		Ash dieback, leaf disease	To be removed due to poor health
6305	*Chamaecyparis lawsoniana*	Lawson cypress	15.3		Good	Good	8.1	7.1	5.6	10.4	*30.0*	-		Ivy climbimg trunk	To be removed due to building construction
6306	*Acer palmatum*	Japanese maple	9.4	4.6,3.6,5.2, 5.3	Good	Good	8.9	8.9	10.4	10.9	*12.0*	-		Tag embeded in trunk	To be removed due to building construction
6524	*Arbutus menziesii*	Pacific madrone	5.6		Good	Good	7.2	6.7	6.2	9.7	*6.0*	-		Not regulated	To be removed due to site improvements
6307	*Acer palmatum*	Japanese maple	9.2	6.9,6.1,	Good	Good	6.4	7.4	12.4	10.4	*12.0*	-			To be removed due to building construction
6311	*Pinus strobus*	Eastern white pine	10.4		Fair	Good	15.4	12.9	7.9	6.4	*30.0*	-		Suppressed and some chlorotic needles, heavy ivy	To be removed due to potential Jefferson Road improvements
6312	*Pseudotsuga menziesii*	Douglas-fir	25.5		Good	Fair	21.1	28.6	19.1	12.1	*30.0*	-		Previously broken top with new leader establishing, ivy at base and in pavement island	To be removed due to potential Jefferson Road improvements
6313	*Pseudotsuga menziesii*	Douglas-fir	23.4		Fair	Good	22.5	10.0	20.5	27.5	*30.0*	-		Compacted soils and ivy, near sideal k, shared canooy with adjacent Douglas-fir, no tag	To be removed due to potential Jefferson Road improvements
6314	*Acer circinatum*	Vine maple	6.5	3,3.3,3.4,3. 3,	Good	Good	8.3	4.3	11.3	7.3	*8.0*	-			To be removed due to building construction
6315	*Acer circinatum*	Vine maple	6.7	3.2,3.7,3.3, 3.1	Good	Good	3.3	6.3	10.8	5.3	*8.0*	-			To be removed due to building construction
6316	*Acer circinatum*	Vine maple	5.3	3.1,3.5,2.5	Good	Good	3.2	8.2	9.2	6.2	*8.0*	-			To be retained
6317	*Acer circinatum*	Vine maple	8.0	3.4,3.8,3.3, 4,3.4	Good	Good	6.3	9.8	16.3	15.8	*8.0*	Exceptional		Foliage appears to have been impacted by heat	To be retained
6319	*Acer circinatum*	Vine maple	5.0	3.1,3.9	Fair	Good	12.2	13.2	12.2	13.2	*8.0*	-		Minor dieback and ivy climbing trunk	To be removed due to site improvements: Accessible Entry
6320	*Pseudotsuga menziesii*	Douglas-fir	39.0		Good	Good	20.1	24.6	25.1	24.6	*30.0*	Exceptional		Ivy at base	To be retained
6321	*Cornus nuttallii*	Pacific dogwood	19.0		Fair	Good	19.8	15.3	19.3	23.3	*6.0*	Exceptional	Exceptional Grove	Top dieback and anthracnose, ivy at base	To be removed due to site improvements: Accessible Path
6323	*Pseudotsuga menziesii*	Douglas-fir	31.1		Good	Good	20.3	19.3	21.3	18.8	*30.0*	Exceptional	Exceptional Grove		To be retained
6325	*Cornus nuttallii*	Pacific dogwood	9.6		Good	Good	8.4	7.4	8.9	11.4	*6.0*	Exceptional		Tag embeded into trunk, ivy at base	To be retained
6326	*Pseudotsuga menziesii*	Douglas-fir	26.8		Good	Fair	18.1	25.1	25.1	23.1	*30.0*	-	Exceptional Grove	Previously broken top with new leader establishing	To be removed due to poor health
6327	*Pseudotsuga menziesii*	Douglas-fir	19.6		Good	Good	14.8	14.3	13.8	17.8	*30.0*	-	Exceptional Grove	Ivy at base	To be retained
6328	*Prunus lusitanica*	Portuguese cherry laurel	9.3	6.2,5.1,4.6	Good	Good	10.4	9.4	8.9	10.4	*24.7*	-			To be retained
6329	*Pseudotsuga menziesii*	Douglas-fir	27.2		Good	Good	27.6	25.6	19.6	17.6	*30.0*	-	Exceptional Grove	Swollen base, asphalt up to trunk, ivy at base, corrected phototropic lean	To be retained
6332	*Pseudotsuga menziesii*	Douglas-fir	35.0		Good	Fair	24.5	26.5	15.5	24.5	*30.0*	Exceptional		Ivy at base, top previously broken with new leaders establishing	To be retained
6333	*Pseudotsuga menziesii*	Douglas-fir	28.7		Good	Good	17.2	20.2	13.7	19.2	*30.0*	-		Ivy climbing trunk	To be retained
6334	*Ilex aquifolium*	English holly	11.9	3.6,6.7,9.1	Good	Good	8.5	3.5	7.5	16.0	*18.8*	-		No tag	To be removed due to site improvements
6335	*Pseudotsuga menziesii*	Douglas-fir	36.0		Good	Good	14.0	20.5	24.4	24.5	*30.0*	Exceptional		Previously broken top, new leader established with bend in trunk	To be retained
6337	*Prunus serrulata*	Flowering cherry	13.4	5.6,7.8,6.1, 7.1	Fair	Fair	16.1	11.6	21.1	15.6	*23.0*	-		Dieback in canopy, compacted soils and blackberry at base	To be removed due to site improvements
6338	*Prunus serrulata*	Flowering cherry	10.9	6.3,4.2,5.2, 5.8	Fair	Fair	14.0	13.5	10.5	16.5	*23.0*	-		Dieback in canopy	To be removed due to site improvements
6339	*Gleditsia triacanthos*	Honeylocust	12.7		Fair	Good	24.5	12.5	11.5	11.0	*20.0*	-		Some dieback in canopy	To be removed due to site improvements
6340	*Betula pendula*	European white birch	16.7		Fair	Good	13.2	12.7	15.2	14.2	*24.0*	-		Symptoms of bronze birch borer, ivy at base	To be retained
6341	*Pinus densiflora*	Japanese red pine	7.1		Poor	Good	4.8	6.3	4.3	2.8	*20.0*	-		Dieback, on slope with ivy	Outside Limit of Work
6342	*Pinus densiflora*	Japanese red pine	7.6		Poor	Good	8.3	4.8	4.6	5.7	*20.0*	-		Dieback, on slope with ivy	Outside Limit of Work
6343	*Pinus densiflora*	Japanese red pine	11.2		Fair	Fair	9.0	8.0	5.5	5.0	*20.0*	-		Narrow codominant, with minor dieback	Outside Limit of Work
6344	*Pinus densiflora*	Japanese red pine	7.4	4.1,6.2	Fair	Fair	6.8	5.3	5.3	5.3	*20.0*	-		Codominant near base, minor dieback	Outside Limit of Work
6345	*Betula pendula*	European white birch	10.8		Fair	Good	8.5	12.5	9.0	6.5	*24.0*	-		Corrected lean, ivy at base, not on survey, symptoms of bronze birch borer	Outside Limit of Work
6346	*Pinus densiflora*	Japanese red pine	10.5	9.4,4.6	Fair	Fair	4.4	5.4	4.4	3.4	*20.0*	-		Not on survey, codominant near base	Outside Limit of Work
6347	*Pinus densiflora*	Japanese red pine	8.7		Fair	Good	10.4	13.4	7.4	5.4	*20.0*	-		Not on survey, some dieback	Outside Limit of Work
6348	*Pinus densiflora*	Japanese red pine	9.0		Poor	Good	9.4	7.4	9.4	8.4	*20.0*	-		Not on survey, major dieback	Outside Limit of Work
6349	*Pinus densiflora*	Japanese red pine	10.0		Fair	Good	6.4	4.4	9.4	8.4	*20.0*	-		Not on survey, some dieback	Outside Limit of Work
6351	*Acer palmatum*	Japanese maple	9.3	4.9,5,4.1,4. 5	Good	Good	11.4	14.9	9.9	12.4	*12.0*	-		Ivy at base	Outside Limit of Work
6353	*Pinus thunbergii*	Japanese black pine	12.5		Fair	Fair	9.5	16.0	8.5	2.5	*15.8*	-		Phototropic canopy, blackberry at base	Outside Limit of Work
6355	*Malus domestica*	Apple	7.8		Good	Good	5.3	11.3	6.8	8.8	*20.0*	-			Outside Limit of Work
6421	*Pinus densiflora*	Japanese red pine	14.8		Good	Good	9.1	15.6	9.6	5.1	*20.0*	-		Not on survey, adjacent to dead pine	Outside Limit of Work
6428	*Pseudotsuga menziesii*	Douglas-fir	38.0		Good	Good	31.6	32.1	29.1	22.1	*30.0*	Exceptional		Ivy climbing trunk	To be retained
6523	*Sorbus aucuparia*	European mountain ash	7.5		Good	Good	4.8	8.3	9.3	5.3	*29.0*	-			To be removed due to site improvements
6525	*Pseudotsuga menziesii*	Douglas-fir	23.6		Good	Good	17.5	14.5	18.0	19.0	*30.0*	-	Exceptional Grove		To be removed due to site improvements
6527	*Cornus nuttallii*	Pacific dogwood	7.5		Fair	Good	9.3	6.8	14.3	15.3	*6.0*	Exceptional		Canopy dieback and anthracnose	To be retained
6528	*Cedrus deodara*	Deodar cedar	35.4		Good	Good	20.5	26.5	27.5	36.0	*30.0*	Exceptional	Exceptional Grove	Not on survey but in tree study area, part of exceptional grove and overlaps adjacent firs	To be retained
6530	*Pseudotsuga menziesii*	Douglas-fir	32.8		Good	Good	27.4	24.4	18.4	17.4	*30.0*	Exceptional	Exceptional Grove	Asphalt to base of trunk, lots of ivy, minor pitchjng on trunk	To be retained
6531	*Pseudotsuga menziesii*	Douglas-fir	22.8		Good	Good	17.0	13.0	14.0	16.0	*30.0*	-	Exceptional Grove	Ivy at base	To be retained
6532	*Pseudotsuga menziesii*	Douglas-fir	20.7		Good	Good	6.9	8.4	12.4	14.9	*30.0*	-		Not on survey, ivy at base	Outside Limit of Work
6533	*Pseudotsuga menziesii*	Douglas-fir	19.6		Good	Good	17.3	12.3	9.3	22.8	*30.0*	-		Not on survey, ivy at base	Outside Limit of Work
6534	*Pseudotsuga menziesii*	Douglas-fir	22.1		Good	Good	10.9	12.9	20.9	19.9	*30.0*	-		Not on survey, ivy at base	Outside Limit of Work
6535	*Pseudotsuga menziesii*	Douglas-fir	38.4		Good	Good	27.6	25.6	26.6	20.6	*30.0*	Exceptional		Not on survey, ivy at base	Outside Limit of Work
9238	*Laurus nobilis*	Bay laurel	12.7	8.8,9.1	Good	Fair	9.0	12.0	11.0	13.0	*6.0*	Exceptional			To be retained
A	*Magnolia tripetala*	Umbrella magnolia	7.4		Good	Good	8.3	9.3	11.3	12.3	*16.6*	-		No tag, very close to the building probably wont survuve demo	To be removed due to building construction
B	*Magnolia virginiana*	Sweetbay magnolia	7.2	5.4,4.8,	Good	Good	9.3	13.3	6.3	6.3	*14.3*	-		No tag	To be removed due to building construction
C	*Prunus laurocerasus*	Cherry laurel	9.4	4.5,3.8,5.4, 4.9	Good	Good	14.4	12.9	10.4	10.9	*26.2*	-		No tag	To be removed due to building construction
D	*Magnolia insignis*	Red lotus tree	11.9	8.1,7.1,5	Good	Good	12.5	9.5	5.5	9.0	*30.0*	-		No tag	To be removed due to building construction
E	*Prunus laurocerasus*	Cherry laurel	14.5	7,4.6,7.9,4. 8,5.6,4.9,	Good	Good	30.6	14.6	17.6	18.6	*26.2*	-		No tag, large spread near path	To be removed due to site improvements: Accessible Path
F	*Tilia cordata*	Littleleaf linden	10.3	3.8,3.9,4.8, 4.6,4.7,3.2	Good	Fair	10.4	12.4	13.4	12.4	*30.0*	-		No tag, many leaders, previously hedged or topped, between trees 6337 and 6339	To be removed due to site improvements

DSH (Diameter at Standard Height) is measured 4.5 feet above grade, or as specified in the Guide for Plant Appraisal, 10th Edition , published by the Council of Tree and Landscape Appraisers.

DSH for multi-stem trees are noted as a single stem equivalent, which is calculated using the method defined in the Director's Rule 16-2008.

Letters are used to identify trees without UW metal tags.

Dripline is measured from the center of the tree to the outermost extent of the canopy.

L-002 SHEET NOTES:

1. REFER TO ARBORIST REPORT DATED JULY 19, 2021.

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PROJECT NO.: KT 932

UNIVERSITY of WASHINGTON

UW INTERDISCIPLINARY ENGINEERING BUILDING (IEB)
UNIVERSITY OF WASHINGTON

4002 NORTHEAST STEVENS WAY
SEATTLE, WA 98195
UW PROJECT NUMBER
UW FACNUM

SDCI PERMIT NUMBER: 6857016-DM
4002 EAST STEVENS WAY NE

REVISIONS

NO.	DATE	DESCRIPTION

DEMOLITION PERMIT

NOT FOR CONSTRUCTION

DATE: 05 NOVEMBER 2021
DRAWN BY: DM
CHECKED BY: PB

TREE PROTECTION & REMOVAL TABLE

L-002

Exhibit E
View Corridor #4, East/Southeast from UW Faculty Club
(reference Final EIS page 252)

Exhibit F
Historic Resources - FAB

Historic Property Report

Resource Name: Facilities Services Administration Building - University of Washington

Property ID: 708387

Location

Address: Jefferson Rd, Seattle, Washington, USA

Geographic Areas: King Certified Local Government, Seattle Certified Local Government, King County, T25R04E16, SEATTLE NORTH Quadrangle

Information

Number of stories: N/A

Construction Dates:

Construction Type	Year	Circa
Built Date	1940	☑
Remodel	1961	☐
Addition	1961	☐

Historic Use:

Category	Subcategory
Education	Education - College

Historic Context:

Category
Education
Architecture

Architect/Engineer:

Category	Name or Company

Historic Property Report

Resource Name: Facilities Services Administration Building - University of Washington

Property ID: 708387

Thematics:

Local Registers and Districts

Name	Date Listed	Notes

Project History

Project Number, Organization, Project Name	Resource Inventory	SHPO Determination	SHPO Determined By, Determined Date
041212-22-NPS, NPS, SR 520 Bridge Replacement and MOA for Bryant Site 6(f)	5/12/2017		

Photos

Facility Svc Admin_1176_1.JPG

Facility Svc Admin_1176_5.JPG

Facility Svc Admin_1176_4.JPG

Facility Svc Admin_1176_3.JPG

Facility Svc Admin_1176_2.JPG

Historic Property Report

Resource Name: Facilities Services Administration Building - University of Washington

Property ID: 708387

Inventory Details - 5/12/2017

Common name:	
Date recorded:	5/12/2017
Field Recorder:	Sonja Molchany
Field Site number:	
SHPO Determination	

Detail Information

Characteristics:

Category	Item
Foundation	Concrete - Poured
Roof Type	Mansard
Roof Material	Asphalt/Composition - Shingle
Cladding	Wood - Drop Siding
Structural System	Wood - Platform Frame
Plan	Irregular
Form Type	Utilitarian

Surveyor Opinion

Property appears to meet criteria for the National Register of Historic Places: No

Property is located in a potential historic district (National and/or local): Yes

Property potentially contributes to a historic district (National and/or local): No

Significance narrative:

NRHP ELIGIBILITY
This building is recommended not eligible for listing in the National Register of Historic Places, as it does not meet any of the listing criteria. The original building was utilitarian and has been extensively altered over the years. Additionally, it does not appear to contribute to the recommended Central Campus Historic District, as it lacks sufficient integrity to convey any significance within the context of the larger district.

OVERVIEW
The Facilities Services Administration Building is located in the east-central portion of campus, in an area of campus that has long contained utilitarian functions, dating back to early development associated with the AYPE. No original or early drawings for the building are on file in Facilities Services Records. The earliest record on file is a hardware schedule for “Engineer’s Residence” that dates from 1940. Records dating from 1956 refer to the building as the Building & Grounds Administration Building, “formerly Engineer’s Residence.” Numerous alterations and additions have been made over the years as the building was updated to serve the needs of its occupants.

Physical description:

The Facilities Services Administration Building is situated on the east side of E Stevens Way NE, set back and down from the sidewalk, and on the north side of Jefferson Road NE. It is also heavily screened by trees and shrubs. The site slopes down steeply to the east, resulting in a daylight basement level on the east side. A series of concrete steps and retaining walls along the north side of the building provides access along the north end of the site.

The wood-framed building is comprised of a central two-story portion that has a relatively compact footprint, with one-story wings on the north and south sides. The wings are significantly larger than the central portion of the building. While the two-story portion is likely the earliest/original part of the building, no clear drawings were discovered to verify this. All portions of the building have mansard roofs clad with composition shingle. The main entry is situated on the primary west façade, at the north end of the south wing. It is emphasized by a projecting flat canopy and a brick wing wall. The building has utilitarian finishes and elements, and is characterized by its horizontal wood lapped siding with cornerboards, contemporary aluminum windows with operable awning units at the bottom, and flat wood trim at openings. All finishes appear to be non-original. Many likely date from a 1961 “Alterations & Additions” project.

INTEGRITY

The building has been extensively altered and added on to over time and does not retain architectural integrity.

Bibliography:

University of Washington Facilities Services Records.

NW VERNACULAR
historic preservation

MEMO

TO: Julie Blakeslee
FROM: Northwest Vernacular
DATE: May 14, 2019
SUBJECT: University Facilities Building

The following information stems from review of aerials and drawings in the Facilities Information Library - Facility Records and aerial images from King County and the US Geological Society (USGS).

Built in 1982 and designed by The Mithun Associates, PS, the building replaced the former green house (demolished ca. 1979) and provided expanded physical plant office space. All windows and the front entrance door were replaced in 2015.

The one-story office building with a full, partially day lighted basement features a rectangular plan and is built out on a sloped site. Both floor levels are day lighted on the east, south, and partial north sides, with only the upper floor above grade on the west. A service road runs along the west side of the building, between it and the Facility Services Administration Building (FSA). A wood clad connecting beam extends west from the building to the FSA Building.

The platform frame structure stands on a reinforced concrete foundation. A flat roof with a central shed roof clerestory and perimeter parapets shelters interior spaces. A wood frieze wraps the top of the parapet. Wood lap siding clads the building exterior (outer corners and parapet) with tongue and groove siding below the bands of windows on each facade, and above the first story east facade windows. A wood belt course wraps the building at the basement to first story transition. The windows were replaced in 2015 and consist of aluminum with a large horizontal upper lite over two smaller lites. One of the smaller lites is fixed and the other an awning sash. Two of the windows on the north facade are two-lite windows with a horizontal slider comprising one of the sash due to grade height. Clerestory windows consist of groupings of four awning sash. The main building entrance is centrally placed on the west facade. The recessed entrance consists of a concrete landing with an aluminum two-lite door providing access to the interior. The door was installed in 2015 to replace the previous wood door. A secondary entrance on the south end of the basement level provides staff access.

The basement consists of an inner loop corridor with offices along the east and north sides, with a conference room (former drafting room, remodeled in 2012) in the south end of the floor. Support spaces, including restrooms and mechanical spaces are along the west side of the floor. The central volume of the floor contains the main half-turn stairway at the south end with open office space in the central and north portions of the area.

The first floor consists of an inner loop corridor with offices around the perimeter and within the central portion. The main half-turn stairway is located at the south end of the central volume across from the main receiving counter.

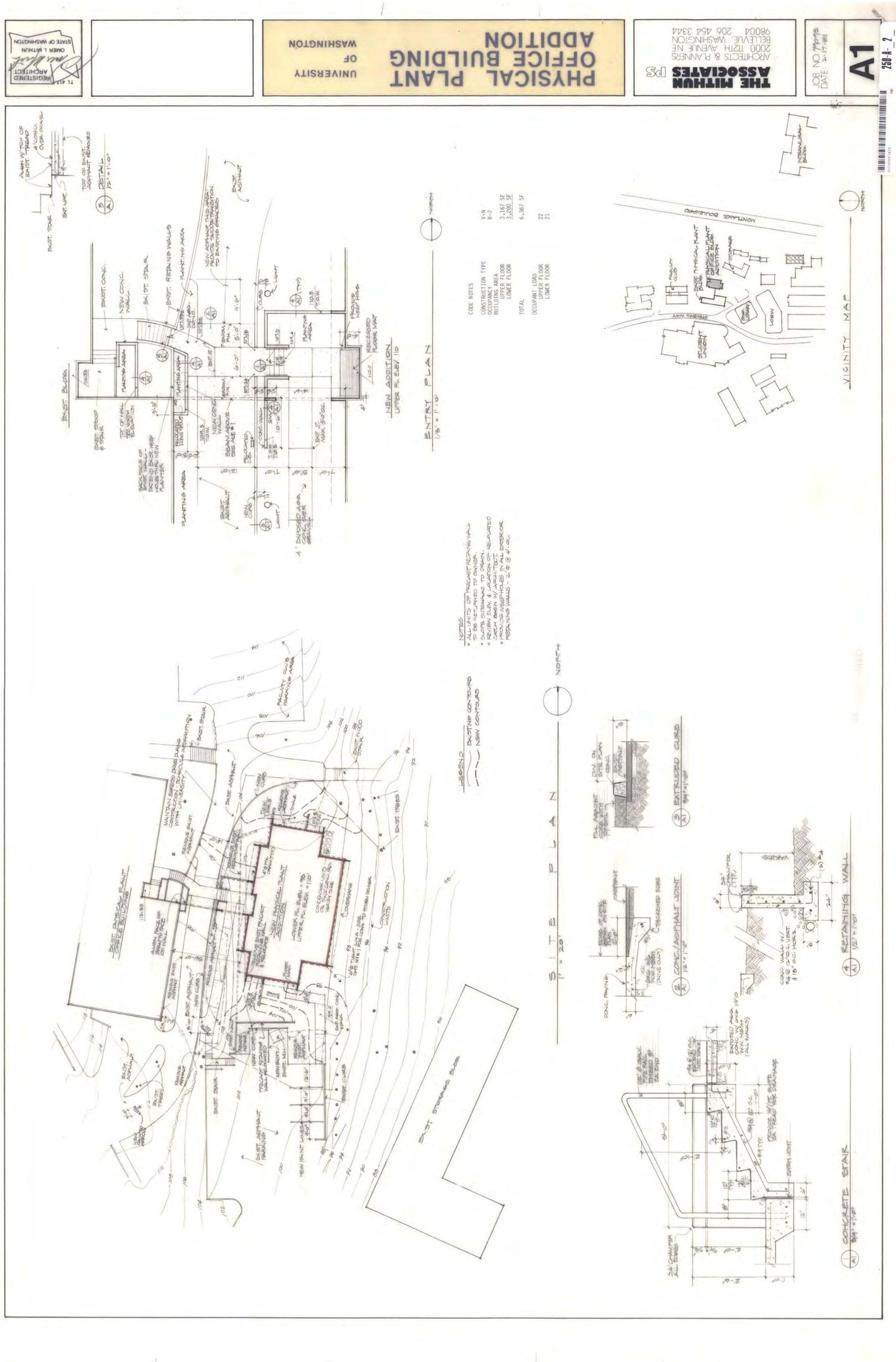

Fig. 1. 1981 drawing showing the University Facilities Building outlined in red, courtesy of UW Facilities Information Library - Facility Records.

Fig. 2. 2019 view, west facade, north end.

Fig. 3. 2019 view, north facade, looking east.

Fig. 4. 2019 view, east facade, looking southwest.

Fig. 5. 2019 view, east facade, looking northwest.

Fig. 6. 2019 view, south facade, looking west.

Fig. 7. 2019 view, basement level, looking southeast at the main stairwell.

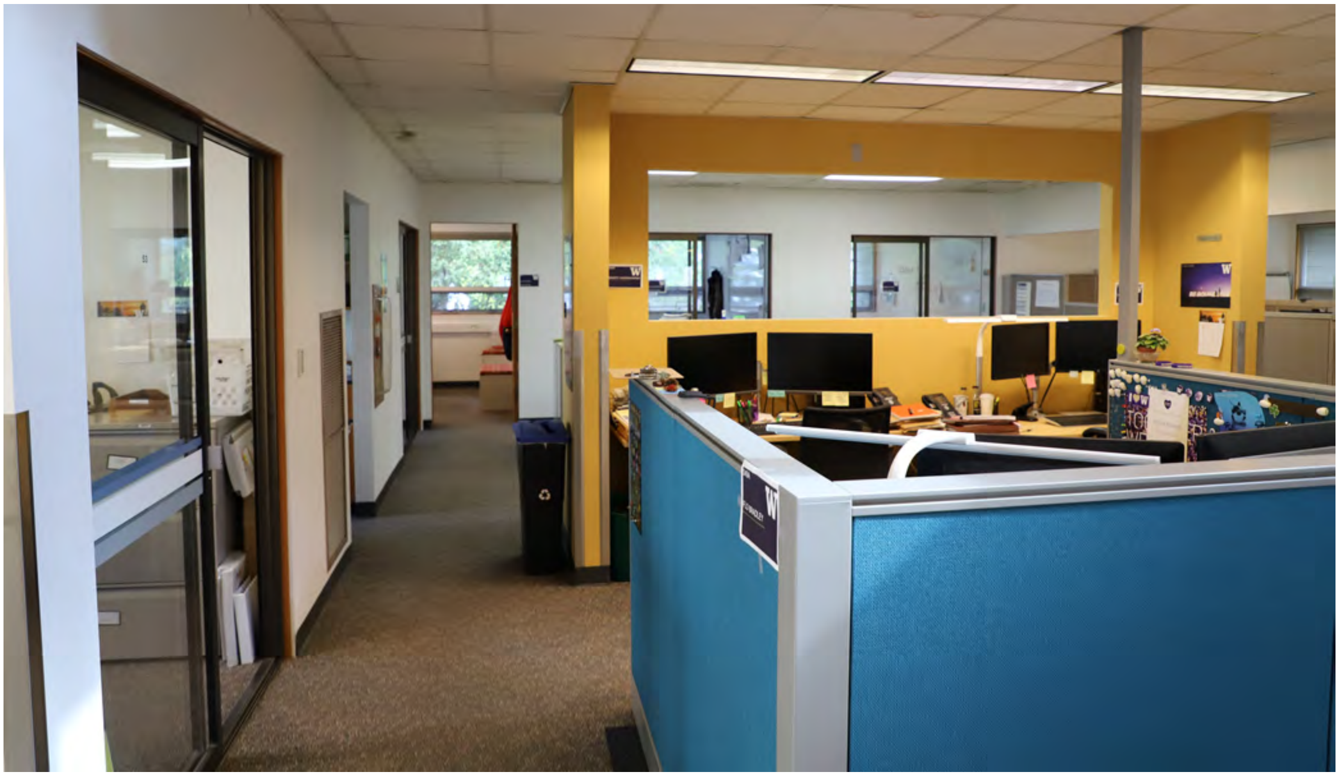

Fig. 8. 2019 view, first floor, looking north.

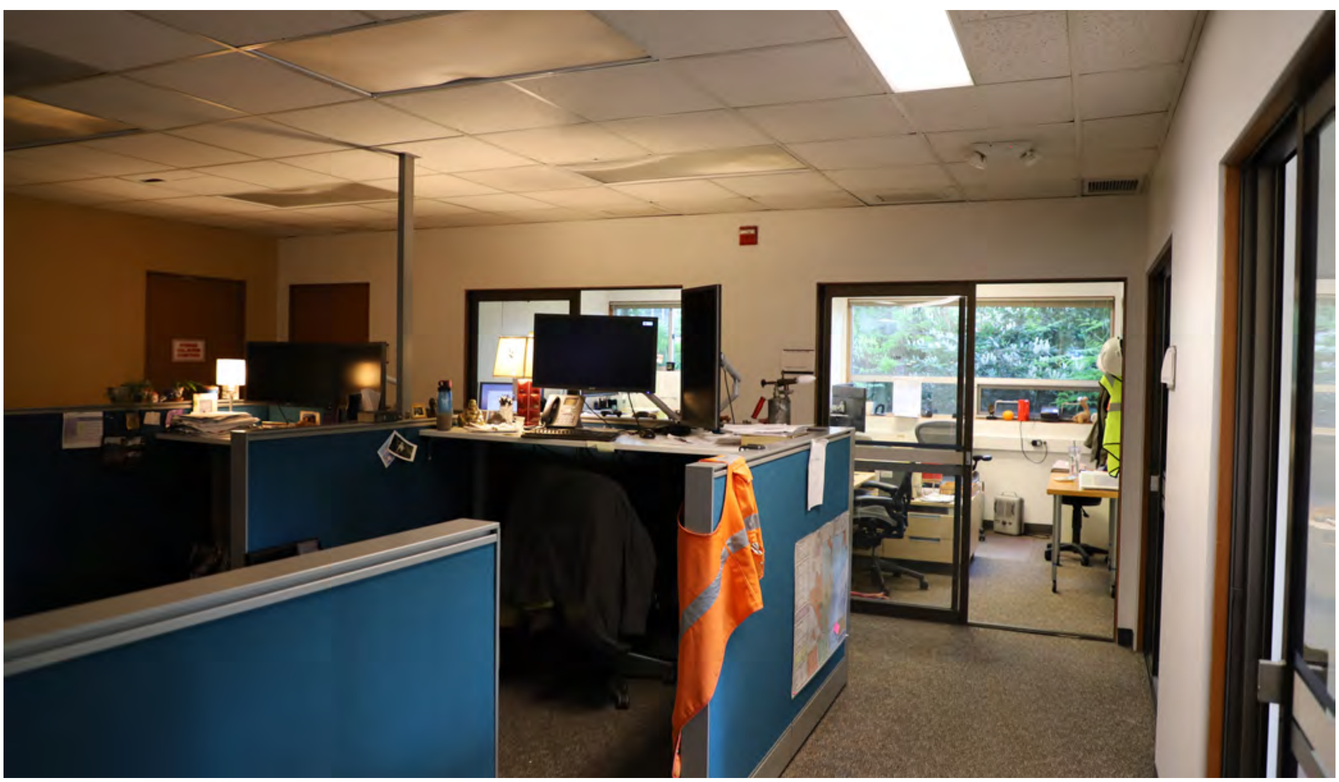

Fig. 9. 2019 view, basement level, looking north towards the northeast corner of the floor.