DRAFT

GEOTECHNICAL REPORT

Covenant Living at the Shores
9150 Fortuna Drive
Mercer Island, Washington

Project No. T-8879

Terra Associates, Inc.

Prepared for:

Covenant Living Communities & Services
Skokie, Illinois

May 18, 2023

TERRA ASSOCIATES, Inc.
Consultants in Geotechnical Engineering, Geology
and
Environmental Earth Sciences

May 18, 2023
Project No. T-8879

Mr. Randy Gross
Covenant Living Communities & Services
5700 Old Orchard Road
Skokie, Illinois 60077

DRAFT

Subject: Geotechnical Report
Covenant Living at the Shores
9150 Fortuna Drive
Mercer Island, Washington

Dear Mr. Gross:

As requested, we have conducted a geotechnical engineering study for the subject project. The attached report presents our findings and recommendations for the geotechnical aspects of project design and construction.

The native soils observed in the test borings generally consist silt and silty fine sand that are generally very loose to medium dense in the upper 10 to 13 feet, becoming medium dense to very dense below those depths. Wet soils indicative of perched groundwater were encountered in all six test borings.

In our opinion, the subsurface conditions are suitable for the proposed development provided the recommendations presented in this report are incorporated into project design and construction. We trust the information presented in this report is sufficient for your current needs. If you have any questions, or require additional information, please call.

Sincerely yours,
TERRA ASSOCIATES, INC.

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Senior Engineering Geologist

DRAFT

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Senior Principal Engineer

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1.0 PROJECT DESCRIPTION

The proposed project will consist of demolishing the existing Fortuna Lodge structure and adjacent parking lot and driveway areas and constructing a new multi-story building with below-grade parking in its place. Site development and building plans are currently not available.

We were provided with an undated, unreferenced conceptual building section that shows the new structure consisting of two floor levels over a daylight basement in the northern building area, three floor levels over a basement parking level in the central building area, and one level of below-grade parking in the southern portion of the planned development area. However, it is our understanding that the conceptual design has been revised to include two levels of below-grade parking in the central and southern building areas.

Proposed building elevations are not shown on the conceptual drawing. For the purpose of this study, we have assumed the floor elevation of the lower parking level and the daylight basement in the northern portion of the building area will be about Elev. 30, which is approximately existing grade at the northern end of the proposed building. Based on this lower floor elevation, excavation depths of about 10 to 22 feet will be required to achieve foundation elevations in the proposed parking garage area.

The recommendations in the following sections of this report are based on our understanding of the preceding design features. We should review design drawings as they become available to verify that our recommendations have been properly interpreted and to supplement them, if required.

2.0 SCOPE OF WORK

We explored subsurface conditions at the site in six test borings drilled to maximum depths ranging between 26.5 and 31.5 feet with a limited-access track-mounted drill rig using hollow-stem auger drilling methods. Using the information obtained from the subsurface exploration and laboratory test results, we performed analyses to develop geotechnical recommendations for project design and construction. Specifically, our report addresses the following:

- Soil and groundwater conditions.
- Geologic Hazards per the City of Mercer Island Municipal Code.
- Seismic site class per the current International Building Code (IBC).
- Site preparation and grading.
- Excavation and temporary shoring.
- Foundations.

- Slab-on-grade floors.
- Lateral earth pressures for wall design.
- Infiltration feasibility.
- Drainage.
- Utilities.
- Pavements.

It should be noted that discussions in this report regarding drainage are associated with soil strength, design earth pressures, erosion, and stability. Design and performance issues with respect to moisture as it relates to the structure environment (i.e., humidity, mildew, mold) are beyond Terra Associates' purview. A building envelope specialist or contractor should be consulted to address these issues, as needed.

3.0 SITE CONDITIONS

3.1 Surface

The project site is a 12.44-acre waterfront parcel located between North Mercer Way and the Lake Washington shoreline, immediately west of Fortuna Drive in Mercer Island, Washington. The site location is shown on Figure 1.

The planned development area is located in the northeastern portion of the site. The northern portion of the planned development area is occupied by Fortuna Lodge, which is a single-story, wood-frame structure with a daylight basement. Asphalt-paved parking and driveway areas border the southern side of the building. Areas located west and north of the building generally consist of landscaped yard and patio areas. A maintenance driveway and service area is located between the eastern side of the building and eastern site margin. The approximate footprint of the planned development area is shown on Figure 2.

In general, site topography slopes gently down to the north to the Lake Washington shoreline. A topographic survey by Lanktree Land Surveying, Inc. (Lanktree), dated September 28, 2022, shows a topographic relief of about 20 feet from Elev. 50 at the southern end of the planned development area to about Elev. 30 on the north side of Fortuna Lodge. Maximum slope gradients typically range between about 6 and 7 percent before steepening to about 14 percent about 55 feet north of Fortuna Lodge.

3.2 Soils

The native soils observed in the test borings generally consist of about 9.5 to 13 feet of very loose to medium dense, moist to wet, silt and silty fine sand with trace amounts of fine organics overlying fine grained deposits consisting predominantly of medium dense to very dense, moist to locally wet, silt and silty fine sand. Very dense, moist to wet, silty, fine- to coarse-grained sand with gravel underlies the dense silt below a depth of 35 feet in Test Boring B-1. The very loose to medium dense deposits were not observed in Test Borings B-3 and B-4, which were located in the central portion of the planned development area near the southwestern and southeastern ends of the Fortuna Lodge building, respectively. We observed fill consisting of medium dense sandy silt to silty sand in the upper approximately 8 feet of Test Boring B-3. Based on its proximity to the Fortuna Lodge building, the fill in Test Boring B-3 is likely associated with building and patio retaining wall backfilling, as well as landscape grading.

The *Geologic Map of Mercer Island, Washington* by K.G. Troost & A.P. Wisher (2006) shows surficial geology in the planned development area mapped as Vashon recessional lacustrine deposits (Qvrl). Older (pre-Olympia age) coarse-grained deposits (Qpoc) and non-glacial fine-grained deposits (Qponf) are mapped nearby to the south and east of the site. The very loose to medium dense silt, clayey silt, and silty fine sand soils observed in the upper approximately 9.5 to 13 feet of Test Borings B-1, B-2, B-5, and B-6 are generally consistent with this geologic map unit. The underlying medium dense to very dense silt and silty fine sand is interpreted to be Lawton clay (Qvlc) based on our observation of scattered coarse sand and fine gravel dropstones within the formation. Lawton clay deposits are mapped about 1,200 feet south-southwest of the site.

Detailed descriptions of the subsurface conditions we observed in the test borings are presented on the Test Boring Logs in Appendix A. The approximate test boring locations are shown on Figure 2.

3.3 Groundwater

Wet soils indicative of perched groundwater were encountered in all six test borings. The observed groundwater conditions consist of an upper perched zone that typically occurred between depths of about 2.5 and 13 feet within the very loose to medium dense recessional lacustrine deposits, and below depths ranging from 35 feet at Test Boring B-1 to 20 feet at Test Boring B-5. A deep perched zone was not observed in Test Boring B-3.

We expect that the shallow perched groundwater levels observed in the upper 10 to 13 feet at the site will fluctuate seasonally with the highest levels observed during the winter and early spring months. Considering the time of year we conducted our site explorations, we expect that the observed groundwater conditions are near seasonal high levels. Based on our field observations, we do not anticipate that the deeper perched groundwater zone observed in the test borings would be subject to significant seasonal fluctuations.

Two-inch diameter groundwater monitoring wells were constructed in Test Borings B-1, B-4, and B-5 subsequent to drilling and sampling. The B-1 and B-4 wells were screened in the lower perched groundwater zone between depths of 35 and 25 feet. The well screen in Test Boring B-5 was constructed in the upper perched groundwater zone between 15 and 5 feet. Well construction schematics are shown on the Test Boring Logs.

The groundwater levels measured in the wells on May 11, 2023 are shown below:

Depth to Groundwater (ft BGS)	
Well ID	**11-May-23**
B-1	7.47
B-4	6.30
B-5	3.21

ft BGS – feet below ground surface

The groundwater levels measured in Test Borings B-1 and B-4 are potentiometric surfaces that have stabilized approximately 15.5 to 16.5 feet above the well screen sand pack, indicating that the lower perched groundwater zone is a pressurized aquifer confined by about 20 to 25 feet of medium dense to very dense silt. A geologic section showing our interpretation of the subsurface conditions across the planned development area is presented as Figure 3.

<u>3.4 Geologic Hazards</u>

We evaluated site conditions for the presence of geologic hazards. Mercer Island City Code (MICC) Section 19.07.160.A defines geologically hazardous areas as "…lands that are susceptible to erosion, landslides, seismic events, or other factors as identified by WAC 365-190-120." Geologically hazardous areas shown mapped on or adjacent to the planned development area by the City of Mercer Island Information & Geographic Services (IGS) map portal website include erosion hazard, potential landslide hazard, and seismic hazard areas. The mapped geologically hazardous areas are shown on Figure 4.

3.4.1 Erosion Hazard Areas

Section 19.16.010 of the MICC defines erosion hazard areas as "those areas greater than 15 percent slope and subject to a severe risk of erosion due to wind, rain, water, slope and other natural agents including those soil types and/or areas identified by the U.S. Department of Agriculture's Natural Resources Conservation Service as having a "severe" or "very severe" rill and inter-rill erosion hazard.."

The Natural Resources Conservation Service (NRCS) has mapped the site soils as *Kitsap Silt Loam, 2 to 8 percent slopes (KpB)*, which is described as having a moderate erosion hazard rating. Based on the NRCS soil mapping and the existing grades, site conditions do not meet the above criteria defining erosion hazard areas.

Although the criteria defining erosion hazard areas is not met, the site soils will be susceptible to erosion when exposed during construction. In our opinion, proper implementation and maintenance of Best Management Practices (BMPs) for erosion prevention and sedimentation control would adequately mitigate the erosion potential at the site. All BMPs for erosion prevention and sedimentation control should conform to City of Mercer Island requirements.

3.4.2 Landslide Hazard Areas

Section 19.16.010 of the MICC defines a landslide hazard area as "areas subject to landslides based on a combination of geologic, topographic, and hydrologic factors, including:

1. Areas of historic failures.
2. Areas with all three of the following characteristics:
    a. Slopes steeper than 15 percent.
    b. Hillsides intersecting geologic contacts with relatively permeable sediment overlying a relatively impermeable sediment or bedrock.
    c. Springs or groundwater seepage.
3. Areas that have shown evidence of past movement or that are underlain or covered by mass wastage debris from past movements.
4. Areas potentially unstable because of rapid stream incision and stream bank erosion.
5. Steep Slope. Any slope of 40 percent or greater calculated by measuring the vertical rise over any 30-foot horizontal run."

We did not observe site conditions meeting the above criteria. Therefore, potential landslide hazards do not exist at the site.

3.4.3 Seismic Hazard Areas

Per WAC 365-190-120(7), seismic hazard areas include areas subject to severe risk of damage as a result of earthquake-induced ground shaking, slope failure, settlement or subsidence, soil liquefaction, surface faulting, or tsunamis. Our evaluation of site susceptibility to these earthquake-induced conditions are discussed below:

Surface Faulting

The Washington State Department of Natural Resources (DNR) Geologic Information Portal website (https://www.dnr.wa.gov/geologyportal) shows the nearest fault suspected of Quaternary activity (designated "Class B") as a strand of the Seattle Fault Zone. The inferred trace of this feature trends northeast-southwest about 300 feet south of the subject site. Accordingly, the risk of ground rupture (surface faulting) along a fault at the site is low.

Soil Liquefaction

Due to presence of relatively loose, saturated soil zones between depths of about 2.5 and 13 feet in the test borings, we completed a liquefaction analysis using the computer program LiquefyPro published by CivilTech Corporation. The analysis was completed for the existing conditions observed in Test Borings B-5 and B-6 using a horizontal acceleration of 0.715g. This seismic acceleration is the site-modified peak ground acceleration (PGA_M) value for the ASCE 7-16 maximum considered earthquake (MCE) determined for the site (Latitude 47.58158191 and Longitude -122.21447498) using the Structural Engineers Association of California (SEAOC) U.S. Seismic Design Maps website (https://seismicmaps.org/) accessed on May 12, 2023.

The results of our analysis indicate soil liquefaction could occur during the design earthquake event resulting in total settlements between about 1.0 to 1.3 inches, half of which could be differential in nature. Based on these results, the site is considered a seismic hazard area with respect to soil liquefaction. The liquefaction analysis results are presented in Appendix B.

In our opinion, this amount of settlement would not structurally impair the building and is consistent with tolerable building settlements associated with static loading. Cosmetic damage to the structure in the form of misaligned doors and windows, cracking, and floor settlement could occur. We expect that liquefaction-susceptible soils in the central and southern portions of the planned building area will be removed by the excavation required to achieve the lower floor level. Excavation and removal of potentially-liquefiable soils from beneath the building foundations in the northern area and restoring grade with compacted granular structural fill could be considered if the owner is not willing to accept the risk for building damage due to liquefaction.

Settlement or Subsidence

Settlement and subsidence during a severe seismic event is typically a result of soil liquefaction or landslide-related ground movement. In our opinion, the site conditions are not susceptible to slope failure during a severe seismic event. The risk of potentially-damaging liquefaction-induced settlement is discussed above. The site location is not susceptible to regional settlement resulting from large subduction zone earthquakes.

<u>*Tsunamis*</u>

Risks associated with tsunami inundation of the Lake Washington shoreline are unknown. Although the site is located within the Seattle Fault Zone, no historic earthquakes (within the last 150 years) have been caused by or associated with deformation or surface rupture along a fault or fold in Washington State. Accordingly, it is our opinion that the risk of damage as a result of an earthquake-induced tsunami is low.

<u>*Ground Shaking*</u>

Based on the subsurface conditions observed at the site, it is our opinion that design in accordance with local building codes for determining seismic forces would adequately mitigate impacts associated with ground shaking. As discussed, localized soil zones at the site will be subject to the soil liquefaction phenomenon during a severe seismic event. Because of this condition, per the current International Building Code (IBC), subsurface conditions would be assigned site class "F" which would require performing a site-specific seismic analysis to determine seismic forces for structural design. However, the IBC allows for using code derived seismic values for the soil conditions indicated if the building's fundamental period is equal to or less than 0.5 seconds. If the proposed structure falls into this category, based on the site soil conditions and our knowledge of the area geology, site class "D" can be used to determine seismic design forces.

<u>*Slope Failure*</u>

In our opinion, the site conditions are not susceptible to slope failure during a severe seismic event.

3.4.4 Summary of Geologic Hazard Evaluation and Statement of Risk

In accordance with the requirements of MICC Section 19.07.160.3.a, we submit the following statement of risk for the subject site: As discussed, some of the site soils are susceptible to seismically-induced soil liquefaction. However, it is our opinion that the risk of potentially damaging building settlement due to soil liquefaction would be adequately mitigated with stone column ground improvements or eliminated by excavation. Site conditions do not meet the criteria defining landslide or erosion hazard areas.

3.5 Seismic Site Class

As discussed above, subsurface conditions at the site would be categorized as site class "F", which requires performing a site-specific seismic analysis to determine seismic forces for structural design. However, the IBC allows for using code derived seismic values for the soil conditions indicated if the building's fundamental period is equal to or less than 0.5 seconds. If the proposed structure falls into this category, based on the site soil conditions and our knowledge of the area geology, site class "D" can be used to determine seismic design forces.

4.0 DISCUSSION AND RECOMMENDATIONS

4.1 General

Based on our study, there are no geotechnical conditions that would preclude the planned development. The structure can be supported on conventional spread footings bearing on a subgrade of competent native soil, or structural fill that is placed on a competent native soil subgrade. Floor slabs and pavements can be similarly supported.

The site soils consist predominantly of silt and will require careful control of the soil moisture content to adequately compact as structural fill. As such, the ability to use soils from site excavations as structural fill will depend upon the natural soil moisture content, the prevailing weather conditions at the time of construction, and the ability of the contractor to properly moisture condition the soil. During the normally dry summer months, the contractor should be prepared to dry soils that are wet of optimum by aeration. Alternatively, stabilizing the moisture in the native soil with cement or lime can be considered. Moisture conditioning of soils that are dry of optimum would require the addition of water to the soils and thoroughly blending the material prior to compaction. If grading activities will take place during the winter season, the contractor should be prepared to import clean granular material for use as structural fill and backfill.

Undisturbed bearing surfaces composed of native soil or structural fill derived from the fine-grained native soils would provide suitable support for conventional spread footing foundations, floor slabs, and pavements; however, the soils will be easily disturbed by normal construction activity, particularly when wet. If disturbed, the soil will not be suitable for support and the affected material would need to be removed with the foundations lowered to obtain support on an undisturbed soil subgrade. Alternatively, the soils can be removed and grade restored with structural fill. To reduce the potential for subgrade disturbance, particularly during wet weather, consideration should be given to placing a six-inch layer of one- to two-inch sized crushed rock or a four-inch layer of lean concrete on completed subgrades to serve as a working surface.

Detailed recommendations regarding these issues and other geotechnical design considerations are provided in the following sections of this report. These recommendations should be incorporated into the final design drawings and construction specifications. Terra Associates, Inc. should review proposed building and grading plans for the project when available to verify that our geotechnical recommendations have been properly interpreted and incorporated into the project design, and to provide additional or alternate recommendations, if needed.

4.2 Site Preparation and Grading

To prepare the site for construction, all vegetation, organic surface soils, and other deleterious materials should be removed from areas of planned construction. Soils containing organic material will not be suitable for use as structural fill but may be used for limited depths in nonstructural areas. Demolition of the existing structure should include removal of existing foundations, floor slabs and abandoned buried utilities. Abandoned existing utilities that fall outside of the new building footprint can remain in place provided they are sealed to prevent groundwater and soil intrusion.

A representative of Terra Associates, Inc. should examine all bearing surfaces to verify that conditions encountered are as anticipated and are suitable for placement of structural fill or direct support of building and pavement elements. Our representative may request proofrolling exposed surfaces with a heavy rubber-tired vehicle to determine if any isolated soft and yielding areas are present. If unstable yielding areas are observed, they should be cut to firm bearing soil and filled to grade with structural fill. In pavement areas, if the depth of excavation to remove unstable soils is excessive, use of geotextile fabric such as Mirafi 500X or equivalent in conjunction with clean granular structural fill can be considered in order to limit the depth of removal.

Our study indicates the native soils consist predominantly of silt and will be difficult to compact as structural fill when too wet or too dry. Accordingly, the ability to use these native soils from site excavations as structural fill will depend on their moisture content and the prevailing weather conditions when site grading activities take place. Native soils that are too wet to properly compact could be dried by aeration during dry weather conditions or mixed with an additive such as lime or Portland cement to reduce and stabilize the soil's moisture content. If soil amendment products are used, additional Temporary Erosion and Sedimentation Control (TESC) BMPs will need to be implemented to mitigate potential impacts to stormwater runoff associated with possible elevated pH levels.

If grading activities are planned during the wet winter months, or if they extend into fall and winter, the owner should be prepared to import wet-weather structural fill. For this purpose, we recommend importing a granular soil that meets the following grading requirements:

U.S. Sieve Size	Percent Passing
3 inches	100
No. 4	75 maximum
No. 200	5 maximum*

*Based on the 3/4-inch fraction.

Prior to use, Terra Associates, Inc. should examine and test all materials planned to be imported to the site for use as structural fill.

Structural fill should be placed in uniform loose layers not exceeding 12 inches and compacted to a minimum of 95 percent of the soil's maximum dry density, as determined by American Society for Testing and Materials (ASTM) Test Designation D-698 (Standard Proctor). The moisture content of the soil at the time of compaction should be within two percent of its optimum, as determined by this ASTM standard. In nonstructural areas, the degree of compaction can be reduced to 90 percent.

4.3 Excavation and Temporary Shoring

Based on the Washington State Department of Labor and Industries current occupational safety and health regulations, existing fills and the very loose to medium dense native silt and silty fine sand soils would fall into the Type C category. The underlying dense to very dense silt and silty fine sand would be categorized as Type B soils. Accordingly, for excavations deeper than 4 feet up to a maximum of 20 feet, excavation side slopes should be inclined at a gradient of 1.5:1 (Horizontal:Vertical) or flatter for Type C soils. Type B soils may be inclined at a gradient of 1:1 or flatter. All exposed temporary slope faces that will remain open for an extended period of time should be covered with a durable reinforced plastic membrane during construction to prevent slope raveling and rutting during periods of precipitation.

We expect that perched groundwater seepage will be encountered in the upper approximately 2.5 to 13 feet of site excavations, particularly during the wet winter months. Based on our field observations, the volume of water and rate of flow into excavations should be relatively minor, and would not be expected to impact the stability of excavations when completed as described above. In our opinion, a system of collection trenches and conventional sump pumping procedures within the excavation and/or the use of interceptor drains located upgradient of the excavation should be capable of maintaining a relatively dry excavation for construction purposes.

This information is provided solely for the benefit of the owner and other design consultants and should not be construed to imply that Terra Associates, Inc. assumes responsibility for job site safety. It is understood that job site safety is the sole responsibility of the project contractor.

Temporary Shoring

Temporary shoring will be required where site constraints preclude the use of an open-cut excavation. We recommend shoring the excavations using conventional cantilevered or tieback-anchored soldier piles with timber lagging. The shoring system would then be used as a back-form for construction of the permanent below-grade walls. Soldier pile walls should be designed to resist lateral loads imposed by the adjacent soils and surcharge loadings that will be imposed. Recommended design earth pressure diagrams are presented on Figures 5 and 6.

We recommend soldier piles have a maximum center-to-center spacing of eight feet. For pile spacing of 8 feet and less, the lateral soil pressure uniformly distributed over the width of the lagging can be reduced by 50 percent to account for soil arching between the soldier piles. Unshored excavation heights should not exceed five feet during the excavation. No excavation should remain unsupported for more than 24 hours.

Because the anticipated pile depths will approach and possibly penetrate the pressurized lower perched groundwater zone, the contractor should be prepared to maintain a water head during drilling of the soldier piles. Caving or collapse of opened drilled shafts may also occur particularly within the upper loose to medium dense soils and localized wet soil layers. The contractor must be prepared to case the drilled shafts as needed to prevent collapse and maintain a relatively clean, open hole. If the shafts are relied upon to carry large vertical components of the tieback anchor loading, the shaft bottoms must be relatively free of loose soil prior to insertion of the soldier pile beams and pouring concrete.

Over-break or gaps between the excavated soil face and the back of the lagging must be filled following each excavation lift. Filling with crushed rock or grouting with control density fill (CDF) is recommended. This will be an important consideration in limiting movement of the adjacent ground.

Tieback Anchors

Tieback anchors should be installed in the soil behind the excavation to a sufficient distance to allow the desired lateral load resistance. The recommended configuration of the no load zone is shown on Figure 7. The minimum horizontal spacing between anchors should be four feet to ensure that group effects between adjacent ground anchors are minimized and that anchor intersection is avoided. Group effects will reduce the load-carrying capacity of individual ground anchors.

We recommend using an allowable design adhesion value of 0.5 ksf for properly installed non-pressure grouted anchors that derive their capacity within the upper loose to medium dense soils. Anchors that derive their capacity within the underlying dense to very dense silt can be designed for an allowable adhesion value of 1.5 ksf. Higher anchor adhesion values will be developed if the anchors are constructed using pressure or secondary grouting techniques. The actual value should be based on the results of pullout tests conducted in the early phases of construction.

As with the drilled shafts the upper fill soils and loose to medium dense native soils may be unstable in an open hole condition particularly if groundwater seepage is encountered. The contractor should be prepared to drill with continuous flight augers or use casing to reduce the potential for ground loss.

The contractor should note the presence of existing facilities adjacent to the subject site, including buried utilities, as they may affect the location or extent of the anchor holes.

Monitoring Program

A monitoring program must be implemented to verify the performance of the shoring system and possible excavation effects on adjacent properties. The first step of this program should consist of documenting the existing conditions of the adjacent properties and pavements. The documentation should include a visual survey and a pictorial record.

We recommend monitoring be conducted by the owner and include the measurement of horizontal and vertical movements of:

1. The surface of the adjacent streets.
2. The shoring system.
3. Adjacent buildings.

To monitor potential vertical and horizontal movements of the shoring, monitoring points should be established at the top of every other soldier pile. Surface reference points should also be established and monitored for elevation at distances of 5 or 10 feet from the back of the shoring at spacing of 25 feet at the excavation perimeter.

4.4 Foundations

Ground Improvement

As discussed, the very loose to medium dense soils observed in the northern portion of the planned building area are susceptible to soil liquefaction during a severe seismic event and would not be suitable for immediate support of conventional spread footings. In order to mitigate the potential for soil liquefaction and allow the use of conventional spread footing foundations for building support, ground improvements will be necessary. In our opinion, ground improvement may consist of over excavating the loose soils to expose competent native material and restoring grade with mechanically compacted granular structural fill that meets grading recommendations for wet weather structural fill outlined in Section 4.2 of this report.

Spread Footings

Following successful excavation and replacement of the loose soils in the northern portion of the site the building can be supported on conventional spread footing foundations. Footings exposed to the weather should bear at a minimum depth of 18-inches for frost protection considerations.

Spread footing foundations obtaining support on compacted granular structural fill or the deeper medium dense to very dense native soils in the central and southern portion of the building area can be dimensioned for an allowable bearing capacity of 4,000 psf. For short-term loads, such as wind and seismic, a one-third increase in the allowable capacity can be used in design. With the anticipated loads and these bearing stresses applied, building settlements should be less than one-half inch total and one-fourth inch differential.

For designing foundations to resist lateral loads, a base friction coefficient of 0.35 can be used. Passive earth pressure acting on the sides of the footings may also be considered. We recommend calculating this lateral resistance using an equivalent fluid weight of 300 pounds per cubic foot (pcf). We recommend not including the upper 12 inches of soil in this computation because they can be affected by weather or disturbed by future grading activity. This value assumes the foundations will be constructed neat against competent native soil or the excavations are backfilled with structural fill, as described in Section 4.2 of this report. The recommended passive and friction values include a safety factor of 1.5.

4.5 Slab-on-Grade Floors

Slab-on-grade floors may be supported on a subgrade prepared as recommended in Section 4.2 of this report. As discussed, similar precautions regarding the susceptibility of the subgrade soils to disturbance must also be considered for slab-on-grade support. Immediately below the floor slab, we recommend placing a 4-inch thick capillary break layer composed of clean, coarse sand or fine gravel that has less than five percent passing the No. 200 sieve. This material will reduce the potential for upward capillary movement of water through the underlying soil and subsequent wetting of the floor slab.

The capillary break layer will not prevent moisture intrusion through the slab caused by water vapor transmission. Where moisture by vapor transmission is undesirable, such as covered floor areas, a common practice is to place a durable plastic membrane on the capillary break layer and then cover the membrane with a layer of clean sand or fine gravel to protect it from damage during construction, and aid in uniform curing of the concrete slab. It should be noted that if the sand or gravel layer overlying the membrane is saturated prior to pouring the slab, it will be ineffective in assisting uniform curing of the slab and can actually serve as a water supply for moisture transmission through the slab that can subsequently affect floor coverings. Therefore, in our opinion, covering the membrane with a layer of sand or gravel should be avoided if floor slab construction occurs during the wet winter months and the layer cannot be effectively drained. We recommend floor designers and contractors refer to the current ACI Collection of Concrete Codes, Specifications, and Practices for further information regarding vapor barrier installation below slab-on-grade floors.

4.6 Lateral Earth Pressures

Lower Level Building Walls

Lower-level building walls can be designed for lateral earth pressures shown on Figure 8. The walls should also be provided with adequate drainage and waterproofed. Typically, for walls constructed using temporary soldier pile/timber lagging, wall drainage is provided by attaching prefabricated drainage panels to the shoring, such as Miradrain G100N. Drainpipes are attached to the Miradrain panels at the wall base and tightlined to discharge through the permanent wall. A typical drainage detail for permanent lower-level walls next to a solider pile wall is shown on Figure 9.

In addition to the drainage panels, if moisture transmission through the basement walls is not desired, the walls should be waterproofed by installation of volclay panels or similar materials. A building envelope specialist should be consulted to design the lower building grade waterproofing if moisture intrusion through the walls is not considered acceptable.

Cantilevered Retaining Walls

The magnitude of earth pressure development on retaining walls will partly depend upon the quality and compaction of the wall backfill. We recommend placing and compacting wall backfill as structural fill, as described in Section 4.2 of this report. To prevent overstressing the walls during backfilling, heavy construction machinery should not be operated within five feet of the wall. Wall backfill in this zone should be compacted with hand-operated equipment. To prevent hydrostatic pressure development, wall drainage must also be installed. A typical wall drainage detail is shown on Figure 10. All drains should be routed to the storm sewer system or other approved point of controlled discharge.

With drainage properly installed, we recommend designing unrestrained walls for an active earth pressure equivalent to a fluid weighing 35 pounds per cubic foot (pcf). For restrained walls, an additional uniform load of 100 psf should be included in the wall design. To account for typical traffic surcharge loading, the walls can be designed for an additional imaginary height of two feet (two-foot soil surcharge). For evaluation of wall performance under seismic loading, a uniform pressure equivalent to 8H psf, where H is the height of the below-grade portion of the wall should be applied in addition to the static lateral earth pressure. These values assume a horizontal backfill condition and that no other surcharge loading, sloping embankments, or adjacent buildings will act on the wall. If such conditions exist, then the imposed loading must be included in the wall design. Friction at the base of foundations and passive earth pressure will provide resistance to these lateral loads. Values for these parameters are provided in Section 4.4 of this report.

4.7 Infiltration Feasibility

Based on the low hydraulic conductivity of the fine-grained native soils and the presence of shallow perched groundwater, it is our opinion that the use of onsite infiltration, including low-impact development (LID) natural drainage practices, will not be feasible for management of development stormwater.

4.8 Drainage

Surface

Final exterior grades should promote free and positive drainage away from the building perimeter. If a positive gradient cannot be provided, provisions for collection and disposal of surface water adjacent to the structure should be provided.

Subsurface

In addition to the drainage for the walls, we recommend installing perimeter foundation drains adjacent to shallow foundations. The drains can be laid to grade at an invert elevation equivalent to the bottom of footing grade. The drains can consist of four-inch diameter perforated PVC pipe that is enveloped in washed pea gravel-sized drainage aggregate. The aggregate should extend six inches above and to the sides of the pipe. Roof and foundation drains should be tightlined separately to the storm drains. All drains should be provided with cleanouts at easily accessible locations.

4.9 Utilities

Utility pipes should be bedded and backfilled in accordance with American Public Works Association (APWA) or local jurisdictional requirements. At minimum, trench backfill should be placed and compacted as structural fill, as described in Section 4.2 of this report. As noted, the site soils are moisture sensitive and will require careful control of moisture to facilitate proper compaction. If utility construction takes place during the winter or if it is not feasible to properly moisture condition the excavated soil at the time of construction, it may be necessary to import suitable wet-weather fill for utility trench backfilling.

4.10 Pavements

Pavement subgrades should be prepared as described in the Section 4.2 of this report. Regardless of the degree of relative compaction achieved, the subgrade must be firm and relatively unyielding before paving. The subgrade should be proofrolled with heavy rubber-tire construction equipment such as a loaded ten-yard dump truck to verify this condition.

The pavement design section is dependent upon the supporting capability of the subgrade soils and the traffic conditions to which it will be subjected. For the proposed self-storage facility, we expect traffic loading will be light to moderate. With a stable subgrade prepared as recommended, we recommend the following pavement sections:

- Three inches of hot mix asphalt (HMA) over four inches of crushed rock base (CRB).
- Four inches full depth HMA over prepared subgrade.

The paving materials used should conform to the Washington State Department of Transportation (WSDOT) specifications for ½-inch class HMA and CRB.

Long-term pavement performance will depend upon surface drainage. A poorly-drained pavement section will be subject to premature failure as a result of surface water infiltrating into the subgrade soils and reducing their supporting capability. For optimum pavement performance, we recommend surface drainage gradients of at least two percent. Some degree of longitudinal and transverse cracking of the pavement surface should be expected over time. Regular maintenance should be planned to seal cracks when they occur.

5.0 ADDITIONAL SERVICES

Terra Associates, Inc. should review the final design drawings and specifications in order to verify that earthwork and foundation recommendations have been properly interpreted and implemented in project design. We should also provide geotechnical service during construction to observe compliance with our design concepts, specifications, and recommendations. This will allow for design changes if subsurface conditions differ from those anticipated prior to the start of construction.

6.0 LIMITATIONS

We prepared this report in accordance with generally accepted geotechnical engineering practices. No other warranty, expressed or implied, is made. This report is the copyrighted property of Terra Associates, Inc. and is intended for specific application to the Covenant Living at the Shores project, and for the exclusive use of Covenant Living Communities & Services and their authorized representatives.

The analyses and recommendations present in this report are based on data obtained from the onsite test borings. Variations in soil conditions can occur, the nature and extent of which may not become evident until construction. If variations appear evident, Terra Associates, Inc. should be requested to reevaluate the recommendations in this report prior to proceeding with construction.

REFERENCE: KING COUNTY IMAP

NOT TO SCALE

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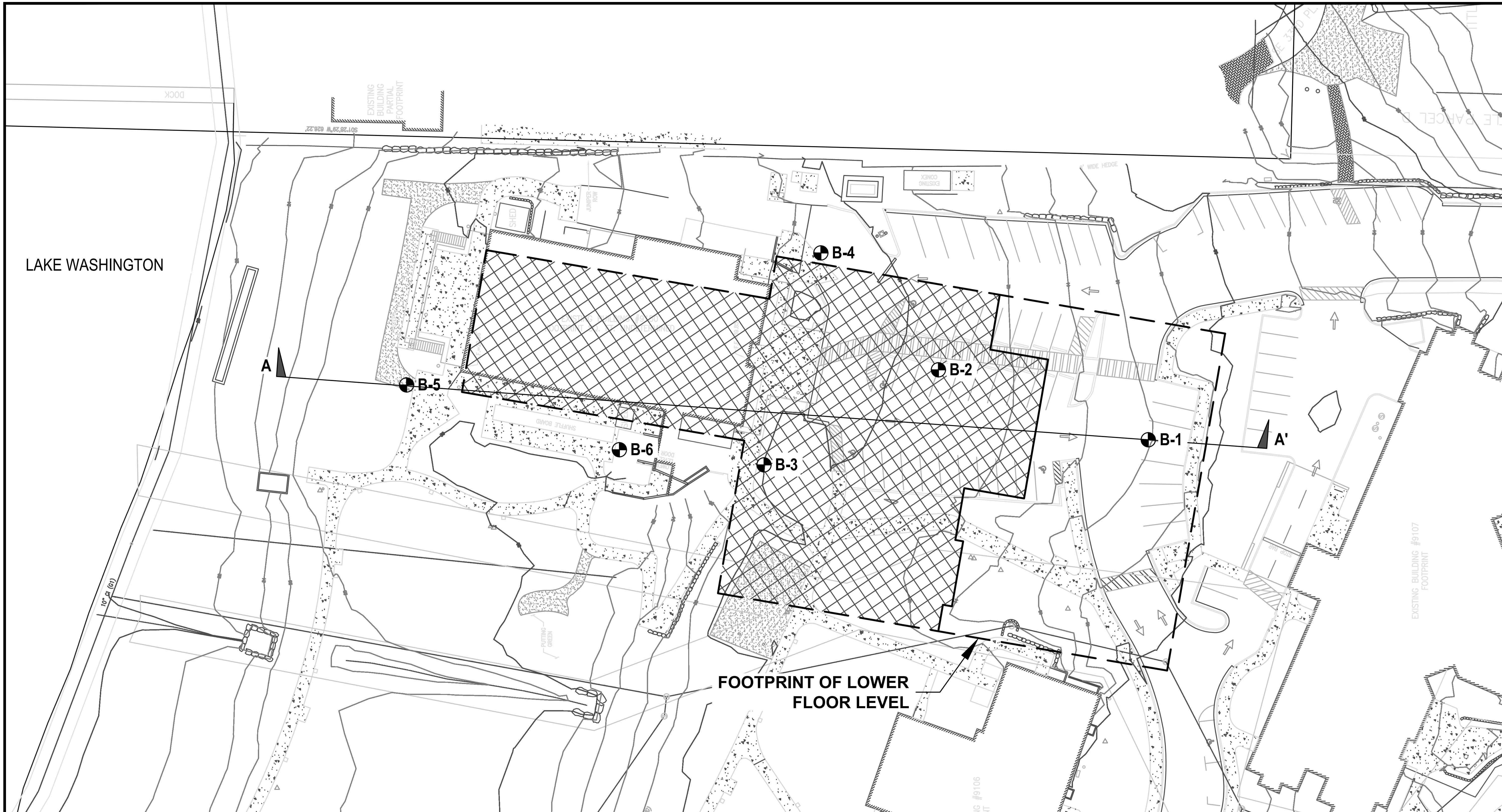
VICINITY MAP
COVENANT LIVING AT THE SHORES
MERCER ISLAND, WASHINGTON

Proj. No.T-8879	Date MAY 2023	Figure 1



NOTE:

THIS SITE PLAN IS SCHEMATIC. ALL LOCATIONS AND DIMENSIONS ARE APPROXIMATE. IT IS INTENDED FOR REFERENCE ONLY AND SHOULD NOT BE USED FOR DESIGN OR CONSTRUCTION PURPOSES.

REFERENCE:

LANKTREE LAND SURVEYING, INC.

LEGEND:

- APPROXIMATE BORING LOCATION
- GEOLOGIC SECTION
- PROPOSED ABOVE-GRADE STRUCTURE

0 30 60

APPROXIMATE SCALE IN FEET

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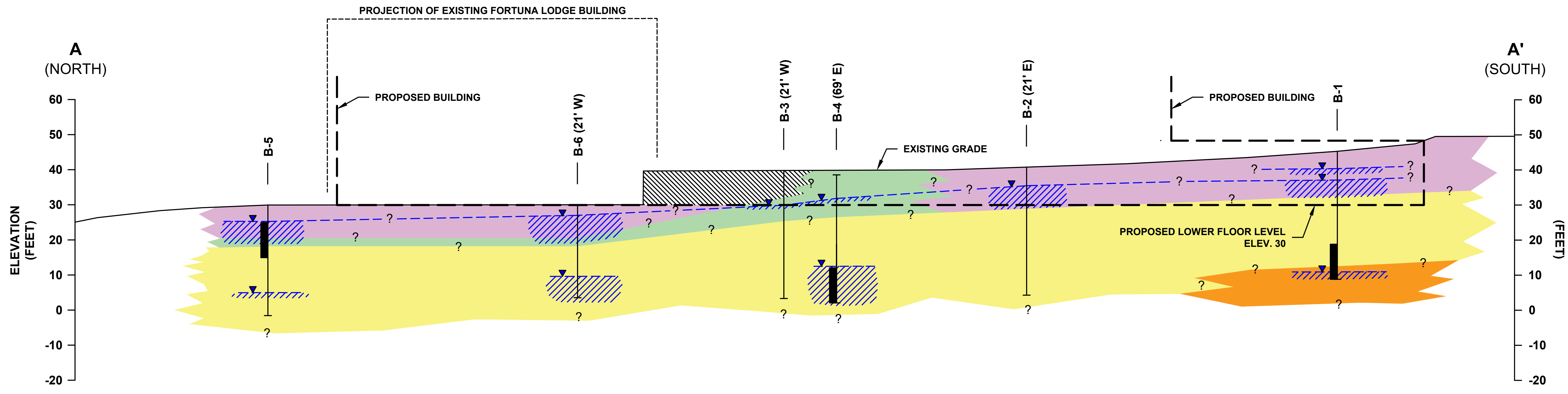
EXPLORATION LOCATION PLAN
COVENANT LIVING AT THE SHORES
MERCER ISLAND, WASHINGTON

Proj. No.T-8879	Date MAY 2023	Figure 2



0 30 60

APPROXIMATE SCALE IN FEET (HORIZ. & VERT.)

LEGEND:

- FILL
- VERY LOOSE TO MEDIUM DENSE SILT AND SILTY FINE SAND
- MEDIUM DENSE TO VERY DENSE SILTY SAND TO SANDY SILT
- MEDIUM DENSE TO VERY DENSE SILT
- VERY DENSE SILTY SAND WITH GRAVEL
- WET SOILS / GROUNDWATER
- TEST BORING / MONITORING WELL SCREEN INTERVAL

NOTE:

THIS SITE PLAN IS SCHEMATIC. ALL LOCATIONS AND DIMENSIONS ARE APPROXIMATE. IT IS INTENDED FOR REFERENCE ONLY AND SHOULD NOT BE USED FOR DESIGN OR CONSTRUCTION PURPOSES.

REFERENCE:

TOPOGRAPHY FROM LANKTREE LAND SURVEYING, INC.

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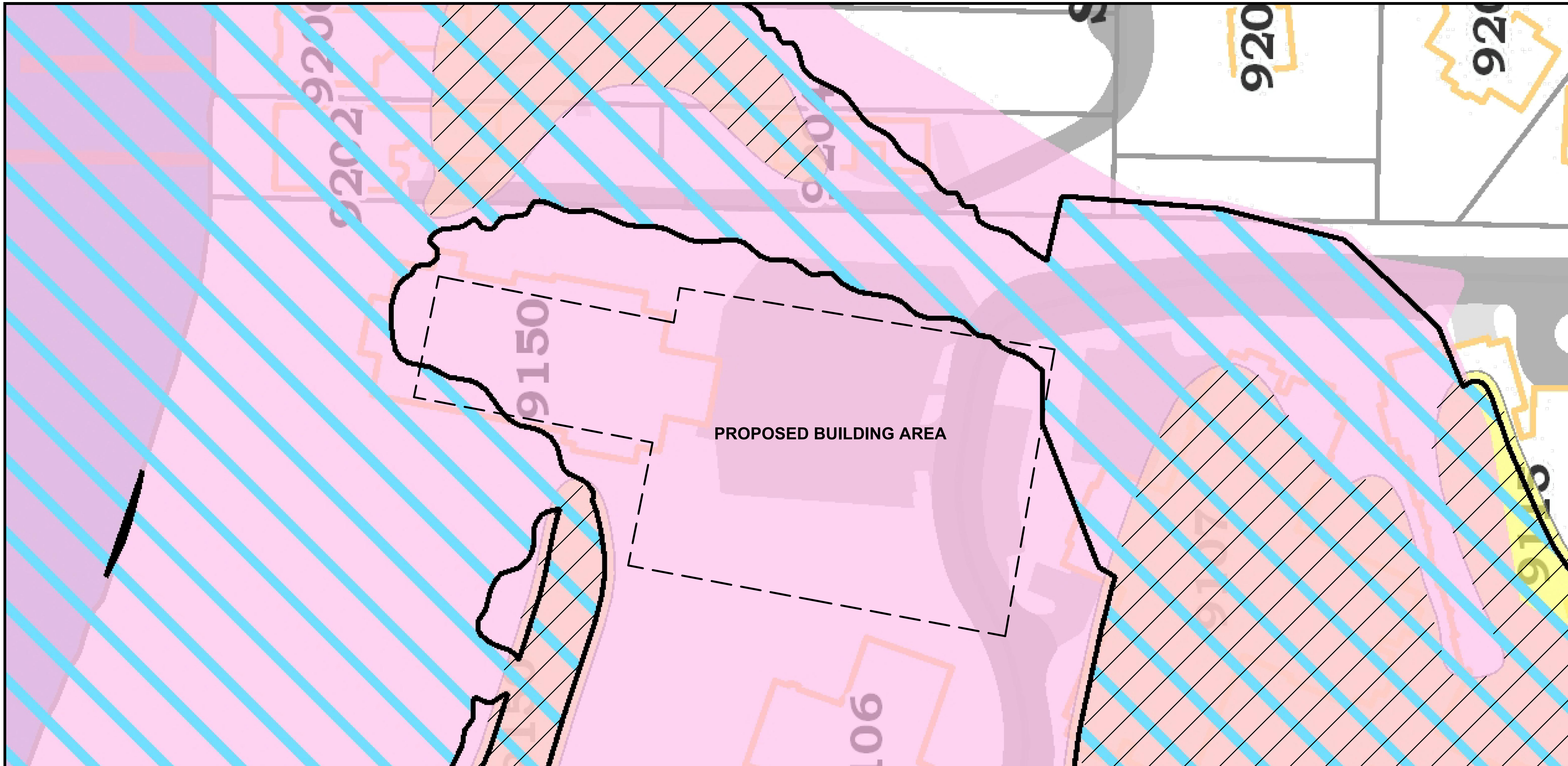
GEOLOGIC SECTION A-A'
COVENANT LIVING AT THE SHORES
MERCER ISLAND, WASHINGTON

Proj. No.T-8879	Date MAY 2023	Figure 3



LEGEND:

- EROSION HAZARD AREA PER MERCER ISLAND IGS MAPPING
- POTENTIAL LANDSLIDE HAZARD AREA PER MERCER ISLAND IGS MAPPING
- SEISMIC HAZARD AREA PER MERCER ISLAND IGS MAPPING

REFERENCE: MERCER ISLAND IGS

N

0 40 80
APPROXIMATE SCALE IN FEET

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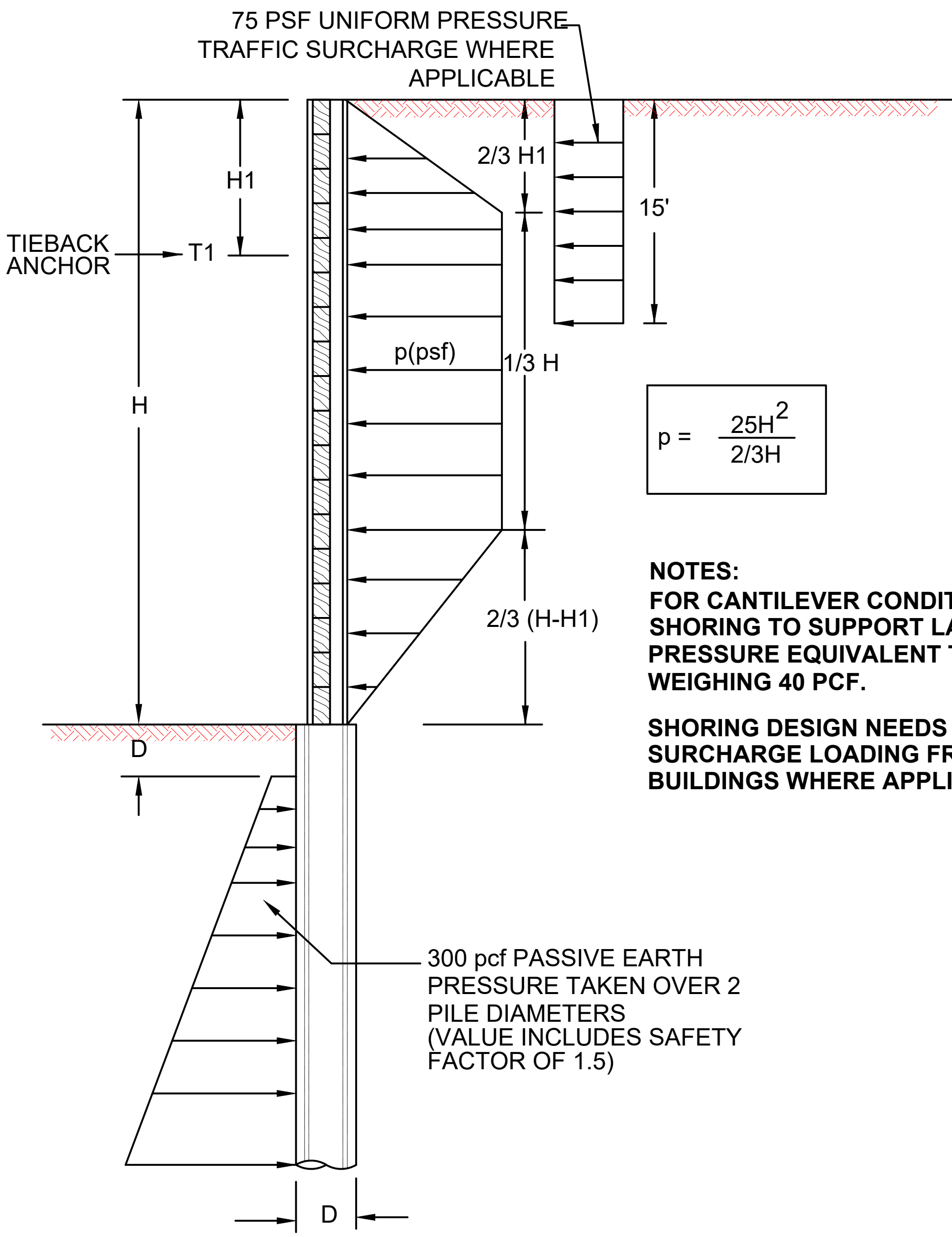
GEOLOGIC HAZARD AREAS MAP
COVENANT LIVING AT THE SHORES
MERCER ISLAND, WASHINGTON

Proj. No.T-8879	Date MAY 2023	Figure 4

EARTH PRESSURE LOADING DIAGRAM
SINGLE TIEBACK



$$p = \frac{25H^2}{2/3H}$$

NOTES:

FOR CANTILEVER CONDITION DESIGN SHORING TO SUPPORT LATERAL EARTH PRESSURE EQUIVALENT TO A FLUID WEIGHING 40 PCF.

SHORING DESIGN NEEDS TO INCLUDE SURCHARGE LOADING FROM ADJACENT BUILDINGS WHERE APPLICABLE.

NOT TO SCALE

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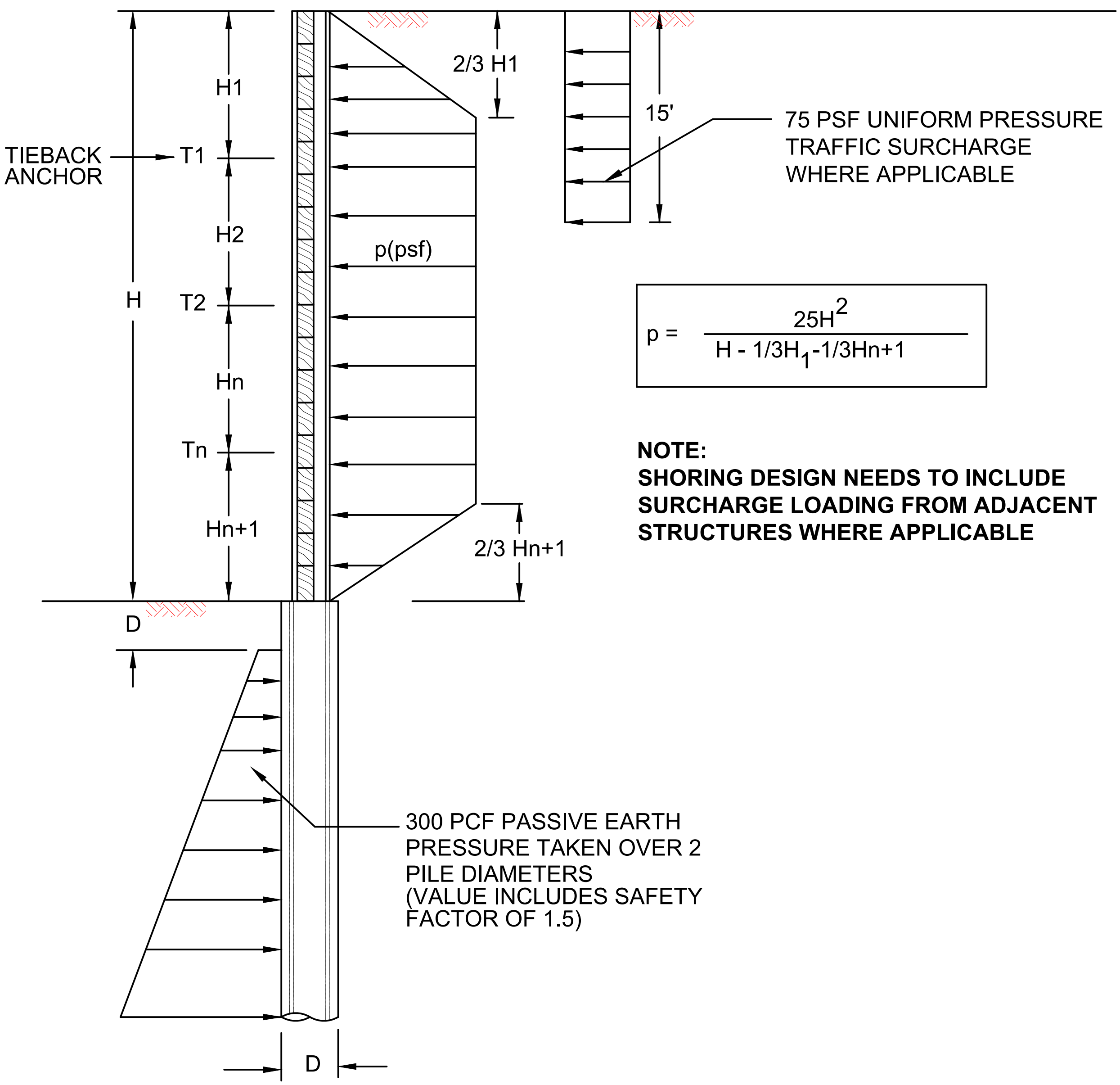
EARTH PRESSURE LOADING DIAGRAM
COVENANT LIVING BY THE SHORES
MERCER ISLAND, WASHINGTON

Proj. No.T-8879	Date MAY 2023	Figure 5

EARTH PRESSURE LOADING DIAGRAM
MULTIPLE TIEBACK



$$p = \frac{25H^2}{H - 1/3H_1 - 1/3Hn+1}$$

NOTE:
SHORING DESIGN NEEDS TO INCLUDE SURCHARGE LOADING FROM ADJACENT STRUCTURES WHERE APPLICABLE

NOT TO SCALE

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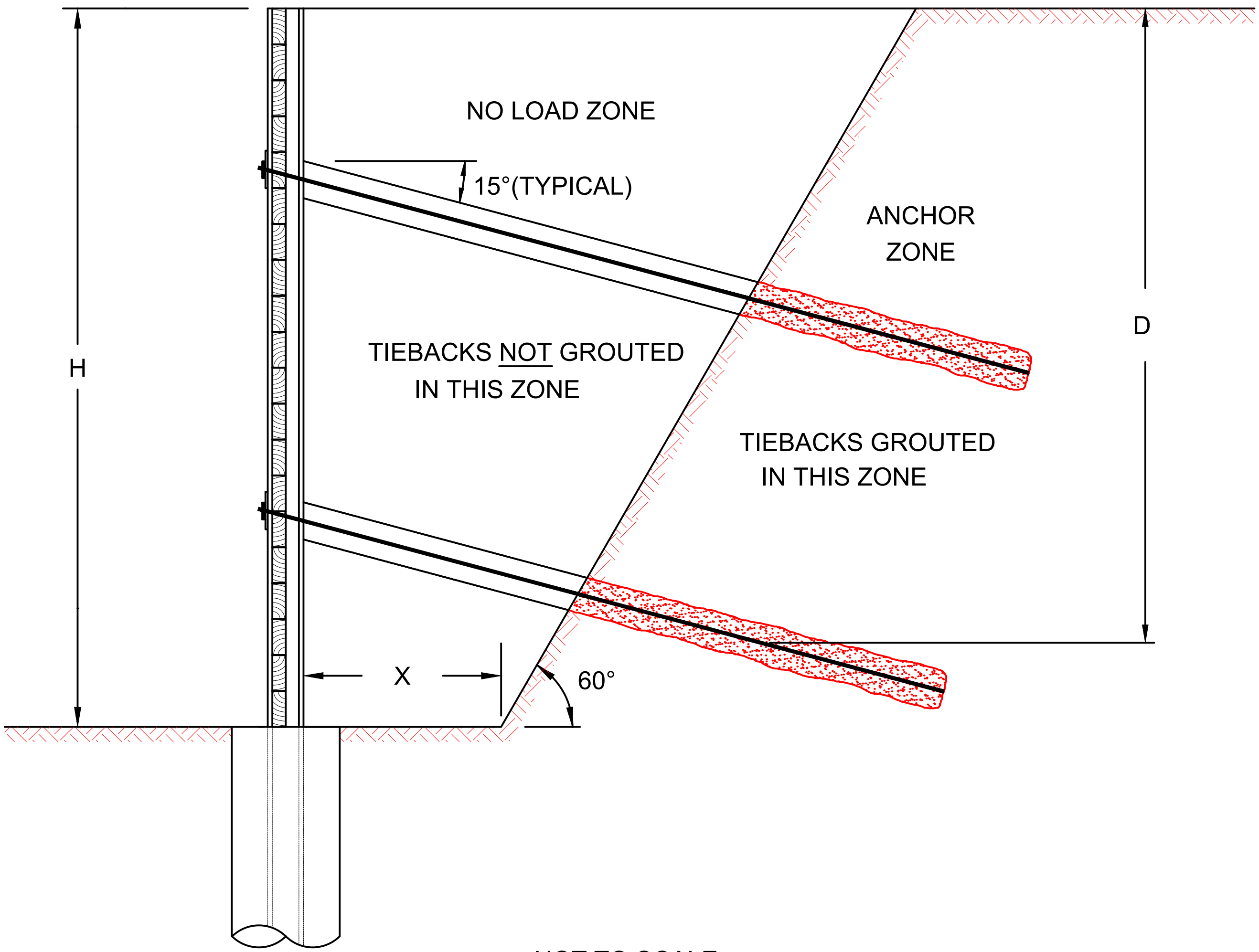
EARTH PRESSURE LOADING DIAGRAM
COVENANT LIVING BY THE SHORES
MERCER ISLAND, WASHINGTON

Proj. No.T-8879	Date MAY 2023	Figure 6

TIEBACK SOLDIER PILE / LAGGING SHORING WALL

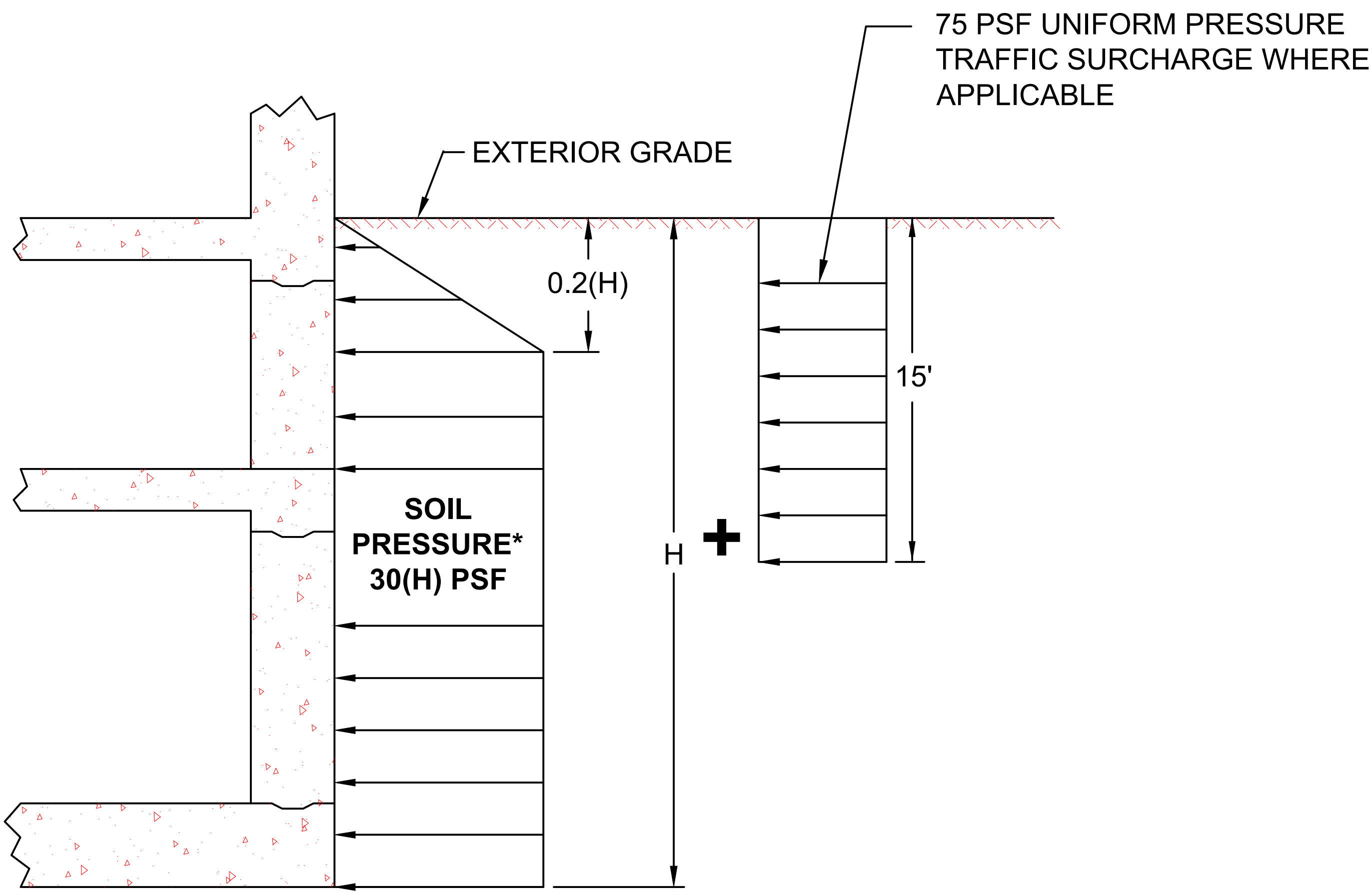


NOT TO SCALE

NOTES:

1. **SEE REPORT NARRATIVE FOR ALLOWABLE TIEBACK ADHESION CAPACITY IN ANCHOR ZONE.**
2. **X = 5 FEET OR 0.2H, WHICHEVER IS GREATER.**

Terra Associates, Inc. Consultants in Geotechnical Engineering Geology and Environmental Earth Sciences	LOAD / NO-LOAD ZONE DIAGRAM COVENANT LIVING BY THE SHORES MERCER ISLAND, WASHINGTON		
	Proj. No.T-8879	Date MAY 2023	Figure 7



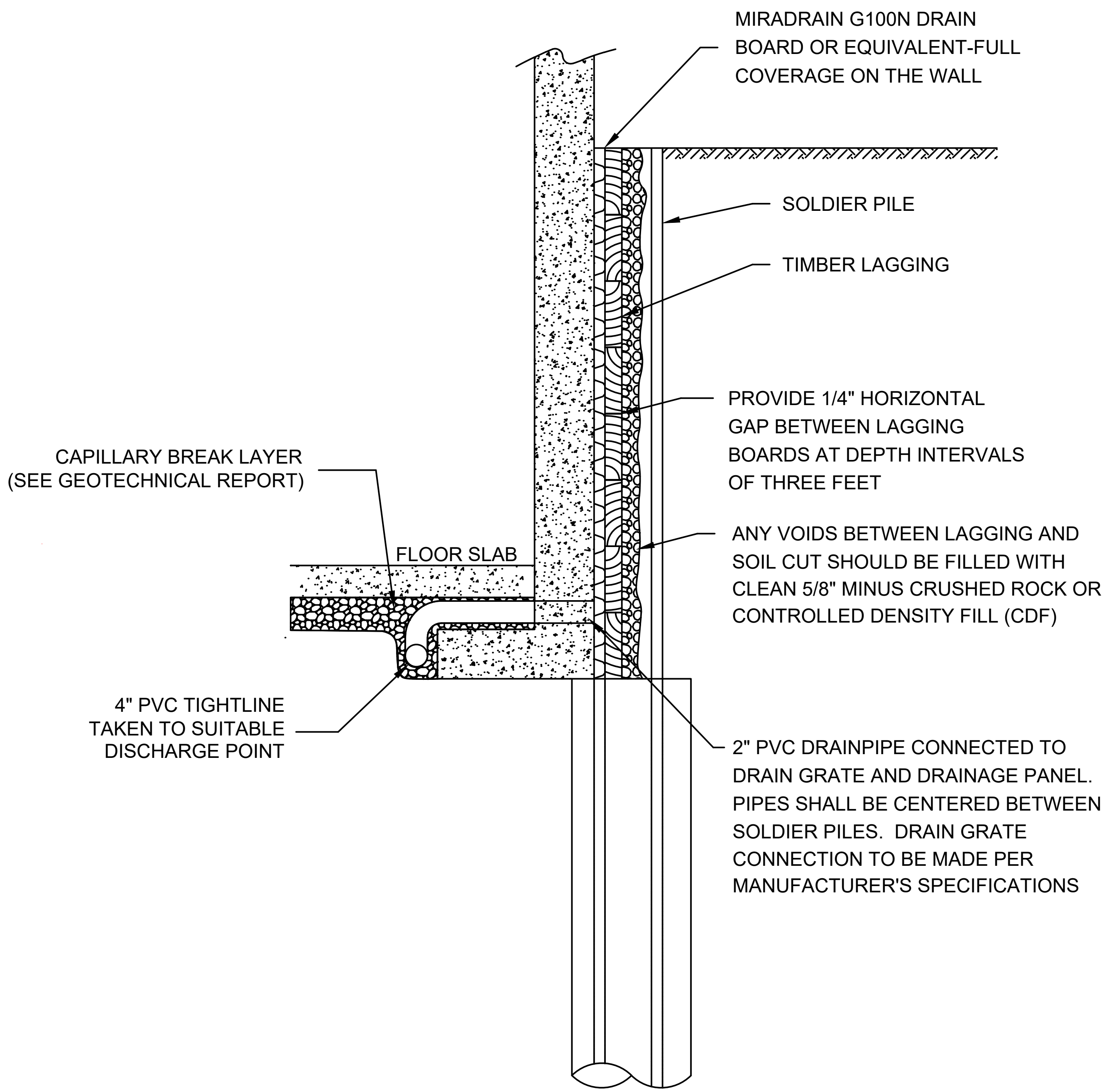
NOT TO SCALE

NOTES:

1. **INCLUDE SURCHARGE LOADING FROM ADJACENT STRUCTURES WHERE APPLICABLE.**

2. *** INCLUDE AN ADDITIONAL UNIFORM EARTH PRESSURE OF 8H PSF FOR SEISMIC LOADING CONSIDERATIONS.**

Terra Associates, Inc. Consultants in Geotechnical Engineering Geology and Environmental Earth Sciences	LATERAL EARTH PRESSURE ON BASEMENT WALLS COVENANT LIVING BY THE SHORES MERCER ISLAND, WASHINGTON	
Proj. No.T-8879	Date MAY 2023	Figure 8



NOT TO SCALE

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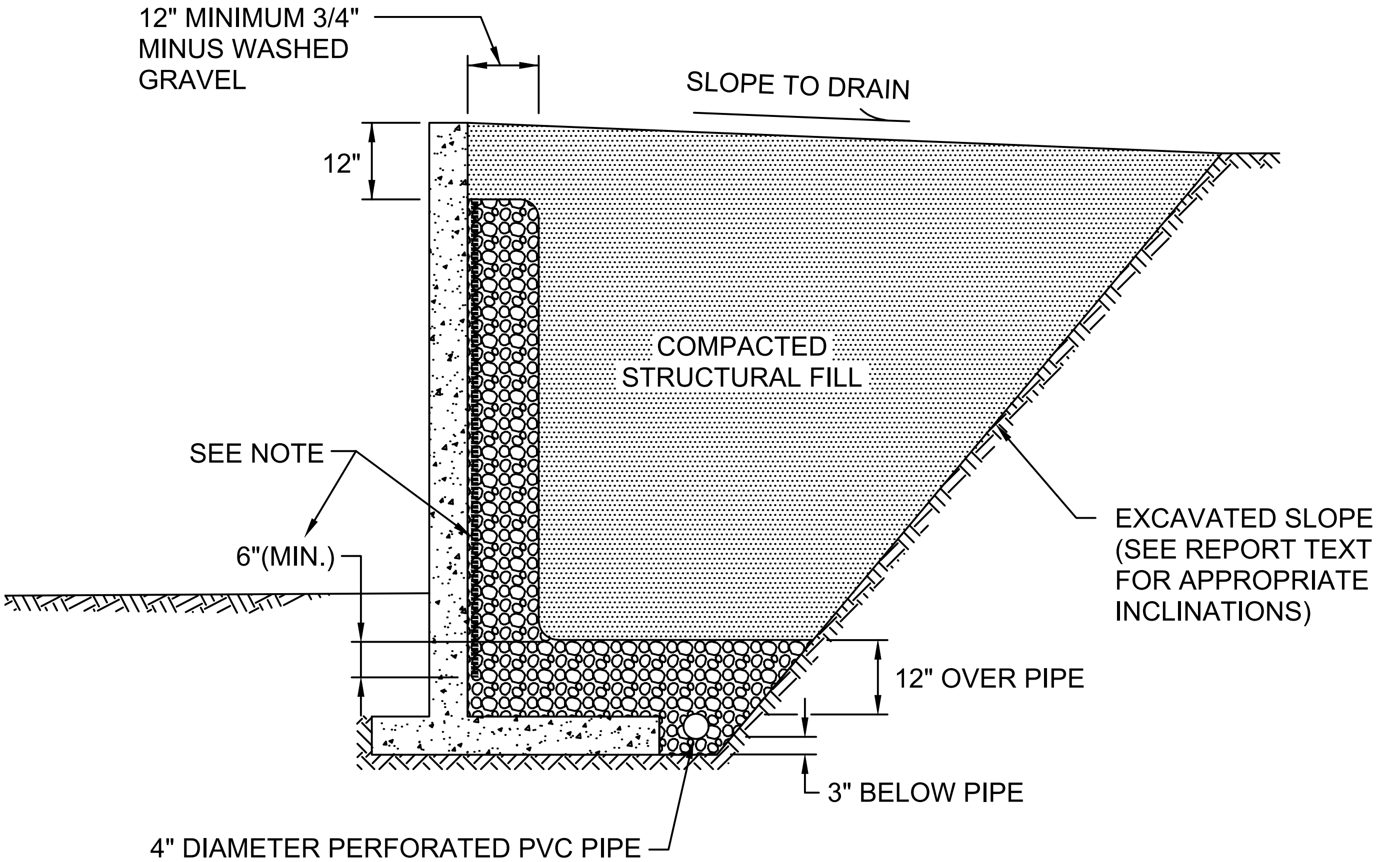
SOLDIER PILE WALL DRAINAGE DETAIL
COVENANT LIVING BY THE SHORES
MERCER ISLAND, WASHINGTON

Proj. No.T-8879	Date MAY 2023	Figure 9



NOT TO SCALE

NOTE:

MIRADRAIN G100N PREFABRICATED DRAINAGE PANELS OR SIMILAR PRODUCT CAN BE SUBSTITUTED FOR THE 12-INCH WIDE GRAVEL DRAIN BEHIND WALL. DRAINAGE PANELS SHOULD EXTEND A MINIMUM OF SIX INCHES INTO 12-INCH THICK DRAINAGE GRAVEL LAYER OVER PERFORATED DRAIN PIPE.

Terra Associates, Inc. Consultants in Geotechnical Engineering Geology and Environmental Earth Sciences	TYPICAL WALL DRAINAGE DETAIL COVENANT LIVING BY THE SHORES MERCER ISLAND, WASHINGTON

Proj. No.T-8879	Date MAY 2023	Figure 10

APPENDIX A
FIELD EXPLORATION AND LABORATORY TESTING

Covenant Living at the Shores
Mercer Island, Washington

We explored subsurface conditions at the site by drilling six test borings to maximum depths of about 10.3 to 21.5 feet with a limited-access track-mounted drill rig using hollow-stem auger drilling methods. The boring locations were determined in the field by pacing, measuring, and sighting from known surface features. The approximate test boring locations are shown on Figure 2. The Test Boring Logs are attached as Figures A-2 through A-7.

An engineering geologist from our office conducted the field exploration, classified the observed soils, maintained a log of each test boring, obtained representative soil samples, and performed a visual reconnaissance of the site. Soil samples were collected during drilling in general accordance with ASTM Test Designation D-1586. Standard Penetration Resistance (N) values obtained for each sampling interval are recorded on the Test Boring Logs. All soil samples were visually classified in accordance with the Unified Soil Classification System (USCS) described on Figure A-1.

Representative soil samples obtained from the test borings were placed in closed containers and taken to our laboratory for further examination and testing. Laboratory testing included determining the moisture content of all soil samples, and grain size distributions on twelve select samples. The soil moisture contents are reported on the Test Boring Logs. The results of the grain size analyses are shown on Figures A-8 through A-11.

MAJOR DIVISIONS			LETTER SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS More than 50% material larger than No. 200 sieve size	**GRAVELS** More than 50% of coarse fraction is larger than No. 4 sieve	Clean Gravels (less than 5% fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines.
		Gravels with fines	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines.
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines.
	SANDS More than 50% of coarse fraction is smaller than No. 4 sieve	Clean Sands (less than 5% fines)	SW	Well-graded sands, sands with gravel, little or no fines.
			SP	Poorly-graded sands, sands with gravel, little or no fines.
		Sands with fines	SM	Silty sands, sand-silt mixtures, non-plastic fines.
			SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS More than 50% material smaller than No. 200 sieve size	**SILTS AND CLAYS** Liquid Limit is less than 50%		ML	Inorganic silts, rock flour, clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity. (Lean clay)
			OL	Organic silts and organic clays of low plasticity.
	SILTS AND CLAYS Liquid Limit is greater than 50%		MH	Inorganic silts, elastic.
			CH	Inorganic clays of high plasticity. (Fat clay)
			OH	Organic clays of high plasticity.
HIGHLY ORGANIC SOILS			PT	Peat.

DEFINITION OF TERMS AND SYMBOLS

	Density	Standard Penetration Resistance in Blows/Foot
COHESIONLESS	Very Loose	0-4
	Loose	4-10
	Medium Dense	10-30
	Dense	30-50
	Very Dense	>50

	Consistancy	Standard Penetration Resistance in Blows/Foot
COHESIVE	Very Soft	0-2
	Soft	2-4
	Medium Stiff	4-8
	Stiff	8-16
	Very Stiff	16-32
	Hard	>32

Symbol	Definition
I	2" OUTSIDE DIAMETER SPILT SPOON SAMPLER
II	2.4" INSIDE DIAMETER RING SAMPLER OR SHELBY TUBE SAMPLER
▼	WATER LEVEL (Date)
Tr	TORVANE READINGS, tsf
Pp	PENETROMETER READING, tsf
DD	DRY DENSITY, pounds per cubic foot
LL	LIQUID LIMIT, percent
PI	PLASTIC INDEX
N	STANDARD PENETRATION, blows per foot

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UNIFIED SOIL CLASSIFICATION SYSTEM
COVENANT LIVING AT THE SHORES
MERCER ISLAND, WASHINGTON

Proj. No.T-8879	Date MAY 2023	Figure A-1

LOG OF BORING NO. 1

Figure No. A-2

Project: Covenant Living at the Shores **Project No:** T-8879 **Date Drilled:** April 10, 2023

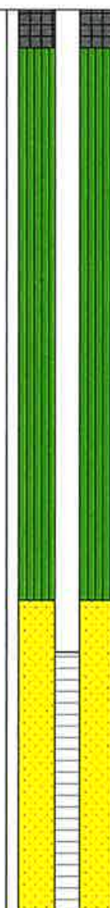
Client: Covenant Living Communities & Services **Driller:** Boretec **Logged By:** JCS

Location: Mercer Island, Washington **Depth to Groundwater:** 5', 8.2', 35' **Approx. Elev:** 46

Depth (ft)	Soil Description	Consistency/ Relative Density
0 – 0.5	6 inches Asphalt Pavement.	Medium Dense
0.5 – 5	Gray silty SAND to sandy SILT, fine grained, moist, trace of wood fragments. (SM/ML) (Possible fill)	Medium Dense
5 – 8	Gray silty SAND, fine grained, trace of fine to coarse gravel, wet, trace of fibrous organics. (SM)	Loose (5–7.5); Medium Dense (7.5–10)
8 – 10.5	Gray-brown sandy SILT, fine grained, moist. (ML)	Medium Dense / Loose
10.5 – 13.5	Gray-brown to gray silty SAND, fine grained, wet, faint mottling between 10 and 10.5 feet. (SM)	Loose
13.5 – 33	Gray SILT to sandy SILT, fine grained, moist. (ML) Pp=4.0 tons/sf - Scattered light gray partings between 20 and 21.5 feet. - Scattered fine to medium sand layers and seams, and trace of fine gravel between 30 and 31.5 feet.	Medium Dense (to 23); Dense (23–33)
33 – 36.5	Gray silty SAND with gravel, fine to coarse sand, fine to coarse gravel, moist to wet. (SM)	Very Dense
36.5	Boring terminated at 36.5 feet. Wet soils encountered between 5 and 6.5 feet, between 8.2 and 13.2 feet, and below 35 feet. Installed 2-inch diameter monitoring well to 35 feet. DOE Well ID: BNV 593 Groundwater measured at 7.47 feet below ground surface on 11-May-23.	

Sample Depth (ft)	SPT (N) Blows / foot	Moisture Content (%)
2.5	11	20.1
5 (ATD)	7	19.2
7.5 (ATD)	15	17.4 19.5
10	9	19.1
12.5	14	21.0
15	21	24.4
20	29	29.3
25	32	30.3
30	45	13.1 25.0
35 (ATD)	80	11.9

Observ. Well



NOTE: This borehole log has been prepared for geotechnical purposes. This information pertains only to this boring location and should not be interpeted as being indicative of other areas of the site

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LOG OF BORING NO. 2

Figure No. A-3

Project: Covenant Living at the Shores **Project No:** T-8879 **Date Drilled:** April 10, 2023

Client: Covenant Living Communities & Services **Driller:** Boretec **Logged By:** JCS

Location: Mercer Island, Washington **Depth to Groundwater:** 5 ft, 30 ft **Approx. Elev:** 40.5

Depth (ft)	Soil Description	Consistency/ Relative Density	SPT (N) Blows/foot	Moisture Content (%)
0	Gray silty SAND, fine grained, moist. (SM)	Loose	5	26.8
5 (ATD)	Gray-brown to gray silty CLAY to clayey SILT, trace of fine sand, wet. (CL-ML/ML) LL=27, PI=6 - Mottled below 7.5 feet. Pp=1-3 tons/sf	Soft to Medium Stiff	3 8	27.1 25.4
10	Gray-brown sandy SILT, fine grained, wet, scattered fine to coarse sand with gravel layers. (ML)	Medium Dense	29	9.8
	Gray SILT, moist, non-plastic to low plasticity. (ML)	Medium Dense	15 25	25.1 22.7
20		Dense	34	26.1
25	- Locally wet between 25 and 26.5 feet.	Dense	40	27.8
30 (ATD)	- Wet below 30 feet.	Dense	41	29.1
35	- Trace of fine to medium sand layers below 35 feet.	Dense	34	28.7
	Boring terminated at 36.5 feet. Wet soils encountered between 5 and 11.5 feet, and below 30 feet.			

NOTE: This borehole log has been prepared for geotechnical purposes. This information pertains only to this boring location and should not be interpeted as being indicative of other areas of the site

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LOG OF BORING NO. 3

Figure No. A-4

Project: Covenant Living at the Shores **Project No:** T-8879 **Date Drilled:** April 10, 2023

Client: Covenant Living Communities & Services **Driller:** Boretec **Logged By:** JCS

Location: Mercer Island, Washington **Depth to Groundwater:** 10 ft **Approx. Elev:** 40

Depth (ft)	Soil Description	Consistency/ Relative Density	SPT (N) Blows/foot	Moisture Content (%)
0–7	Fill: Gray-brown, gray, and brown sandy SILT to silty SAND, fine grained, moist, scattered fine organics, trace of wood fragments. (ML/SM)	Medium Dense	16	18.7
5			11	19.3
7–14.5	Gray silty SAND to silty SAND with gravel, fine sand, fine to coarse gravel, , moist (locally wet between 10 and 10.9 feet). (SM) - Overstated blow counts at 10 feet due to coarse gravel.	Dense	37	10.6
10 (ATD)		Very Dense	50/5"	16.8
		Medium Dense	29	16.0
15	Gray SILT to SILT with fine sand, moist, scattered fine sand partings and seams. (ML)	Dense	32	21.8
20	- Scattered fine gravel between 20 and 21 feet.	Dense	31	19.4
25		Very Dense	51	21.1
30		Very Dense	60	24.2
35	- Scattered light gray silt partings below 35 feet.	Dense	47	28.1
36.5	Boring terminated at 36.5 feet. Wet soils encountered between 10 and 10.9 feet.			

NOTE: This borehole log has been prepared for geotechnical purposes. This information pertains only to this boring location and should not be interpeted as being indicative of other areas of the site

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LOG OF BORING NO. 4

Figure No. A-5

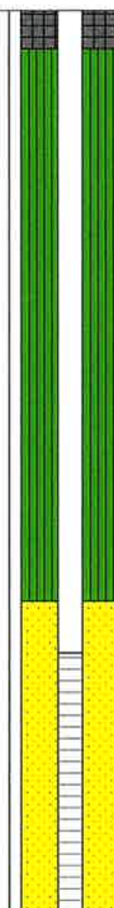
Project: Covenant Living at the Shores **Project No:** T-8879 **Date Drilled:** April 11, 2023

Client: Covenant Living Communities & Services **Driller:** Boretec **Logged By:** JCS

Location: Mercer Island, Washington **Depth to Groundwater:** 7.5 ft, 26 ft **Approx. Elev:** 39

Depth (ft)	Soil Description	Consistency/ Relative Density	SPT (N) Blows / foot	Moisture Content (%)
0	Gray-brown sandy SILT, fine grained, trace of fine to coarse gravel, moist, mottled above 4 feet. (ML)	Medium Dense	21	20.6
5			22	17.3
	Gray silty SAND, fine grained, wet. (SM)	Dense	35	16.6 11.8
10	Gray-brown sandy SILT, fine grained, moist, mottled below 10 feet. (ML)	Medium Dense	24	23.3
	Gray SILT to SILT with fine sand, moist, non-plastic to low plasticity, trace of mottling and iron-oxide stained partings above 14 feet. (ML)	Dense	30	22.3
15	- Scattered light gray silt partings.between 15 and 16.5 feet.		38	23.9
20	- Locally wet between 20 and 21.5 feet.		40	27.0
25		Very Dense	56	27.1 17.9
	Gray SILT to SILT with sand, fine to coarse sand, scattered fine to coarse gravel, wet. (ML)			
30	- Becomes moist with localized wet seams and layers below 30 feet.		69/11"	26.7
35	- Trace of brown organic pockets below 35 feet.		95	23.3
40	Boring terminated at 36.5 feet. Wet soils encountered between 7 and 7.9 feet and below 26.1 feet. Installed 2-inch diameter monitoring well to 35 feet. DOE Well ID: BNV 594 Groundwater measured at 6.3 feet below ground surface on 11-May-23.			
45				

Observ. Well



NOTE: This borehole log has been prepared for geotechnical purposes. This information pertains only to this boring location and should not be interpeted as being indicative of other areas of the site

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LOG OF BORING NO. 5

Figure No. A-6

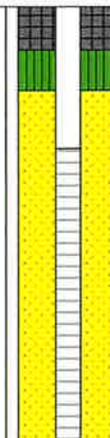
Project: Covenant Living at the Shores **Project No:** T-8879 **Date Drilled:** April 11, 2023

Client: Covenant Living Communities & Services **Driller:** Boretec **Logged By:** JCS

Location: Mercer Island, Washington **Depth to Groundwater:** 5 ft, 25 ft **Approx. Elev:** 29.5

Depth (ft)	Soil Description	Consistency/ Relative Density	SPT (N) Blows / foot	Moisture Content (%)
0	Interbedded gray SAND, and silty SAND to sandy SILT, fine grained, moist. (SP,SM/ML)	Loose	6	24.5
5 (ATD)	Gray silty SAND to sandy SILT, fine grained, wet, trace of coarse sand seams, organic pockets, and root fibers. (SM/ML) - Scattered charcoal and wood fragments below 7.5 feet.	Loose	4	26.3
		Very Loose	3	24.5
10	Gray silty SAND with gravel, fine sand, fine to coarse gravel, wet. (SM)	Dense	36	10.2
				14.8
	Gray-brown to gray sandy SILT, fine grained, moist, mottled. (ML) - Trace of fine to coarse gravel below 12.5 feet.	Medium Dense	15	19.0
15		Medium Dense	25	14.6
	Gray silty SAND to sandy SILT, fine grained, trace of coarse sand and fine to coarse gravel dropstones, moist to wet (moist below 20 feet), faint mottling above 16.5 feet. (SM/ML)	Medium Dense		
20			16	20.2
25 (ATD)	Gray SILT, wet. (ML)	Dense	36	24.5
30	- Becomes moist with scattered light gray silt partings below 30 feet.	Dense	34	28.9
35	Boring terminated at 31.5 feet. Wet soils encountered between 5 and 11.2 feet and between 25 and 26.5 feet. Installed 2-inch diameter monitoring well to 15 feet. DOE Well ID: BNV 596 Groundwater measured at 3.21 feet below ground surface on 11-May-23.			
40				

Observ. Well



NOTE: This borehole log has been prepared for geotechnical purposes. This information pertains only to this boring location and should not be interpeted as being indicative of other areas of the site

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LOG OF BORING NO. 6

Figure No. A-7

Project: Covenant Living at the Shores **Project No:** T-8879 **Date Drilled:** April 11, 2023

Client: Covenant Living Communities & Services **Driller:** Boretec **Logged By:** JCS

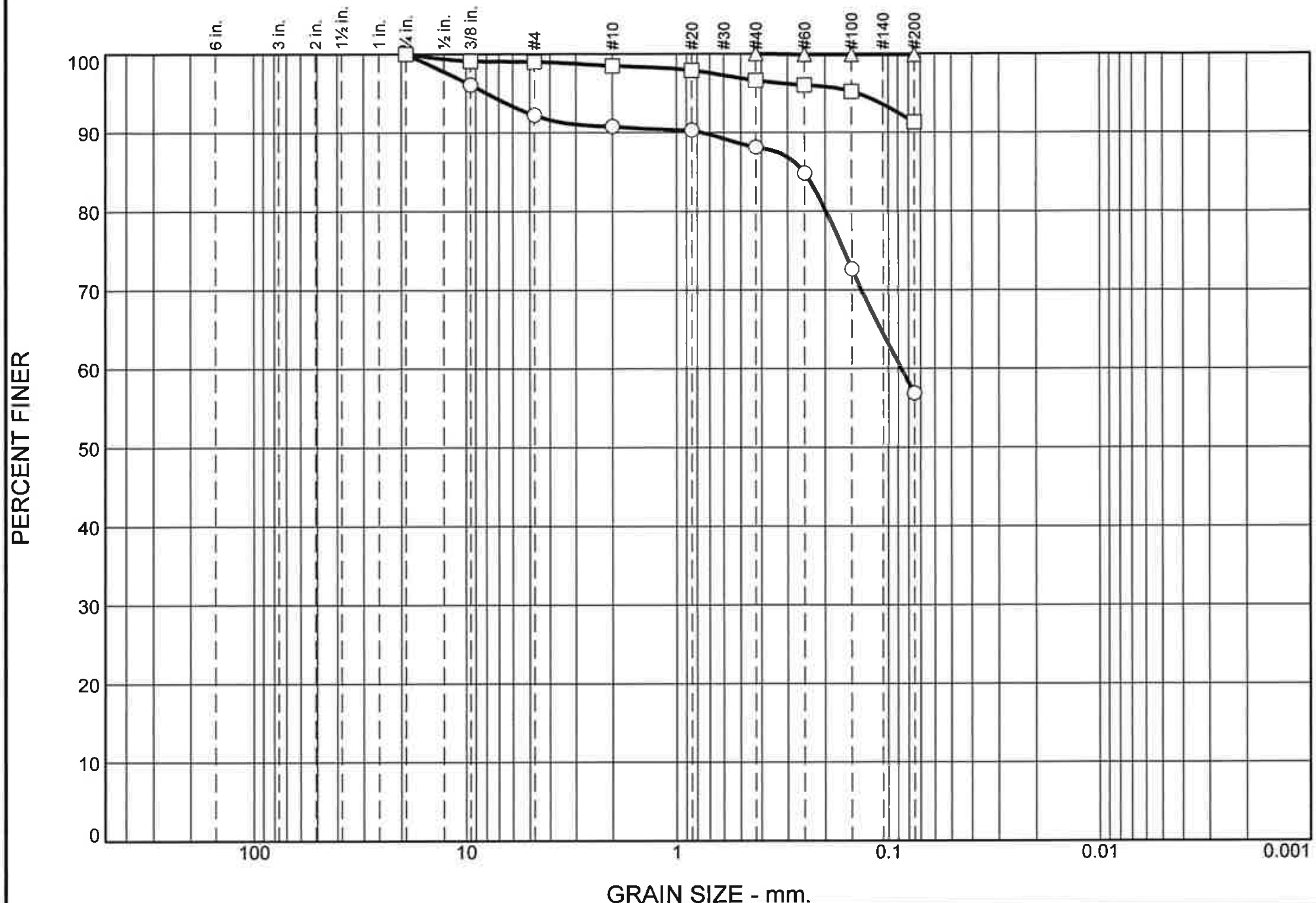
Location: Mercer Island, Washington **Depth to Groundwater:** 2.5 ft, 20 ft **Approx. Elev:** 30

Depth (ft)	Sample Interval	Soil Description	Consistency/ Relative Density	SPT (N) Blows/foot (10 30 50)	Moisture Content (%)
0					
2.5 (▽ ATD)	I	Gray-brown silty SAND to sandy SILT, fine grained, scattered coarse sand and fine to coarse gravel, wet, mottled, scattered root fibers. (SM/ML)	Loose	6	25.6
5	I			4	20.3
	I		Medium Dense	11	17.5
10	I	Gray silty SAND with gravel, fine to coarse sand, fine to coarse gravel, wet. (SM)	Dense	34	11.0
	I	Gray sandy SILT with gravel to silty SAND with gravel, fine sand, fine to coarse gravel, moist. (ML/SM)	Very Dense	63	13.5
15	I	Gray sandy SILT, fine grained, scattered coarse sand dropstones, moist to wet. (ML)	Dense	32	17.2
20 (▽ ATD)	I	- Becomes wet below 20 feet.	Very Dense	53	22.1
25	I		Dense	38	24.8
		Boring terminated at 26.5 feet. Wet soils encountered between 2.5 and 11.5 feet, and below 20 feet.			
30					
35					

NOTE: This borehole log has been prepared for geotechnical purposes. This information pertains only to this boring location and should not be interpeted as being indicative of other areas of the site

Terra Associates, Inc.
Consultants in Geotechnical Engineering
Geology and
Environmental Earth Sciences

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.0	7.8	1.4	2.7	31.2	56.9	
□	0.0	0.0	1.0	0.5	1.9	5.3	91.3	
△	0.0	0.0	0.0	0.0	0.0	0.1	99.9	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			0.2533	0.0869						
□										
△										

Material Description	USCS	AASHTO
○ sandy SILT	ML	
□ SILT	ML	
△ SILT	ML	

Project No. T-8879 **Client:** Covenant Living Communities & Services

Project: Covenant Living at the Shores

○ **Location:** B-1 **Depth:** 8.5'

□ **Location:** B-1 **Depth:** 15'

△ **Location:** B-1 **Depth:** 25'

Remarks:

○ Tested April 27, 2023

□ Tested April 27, 2023

△ Tested April 27, 2023

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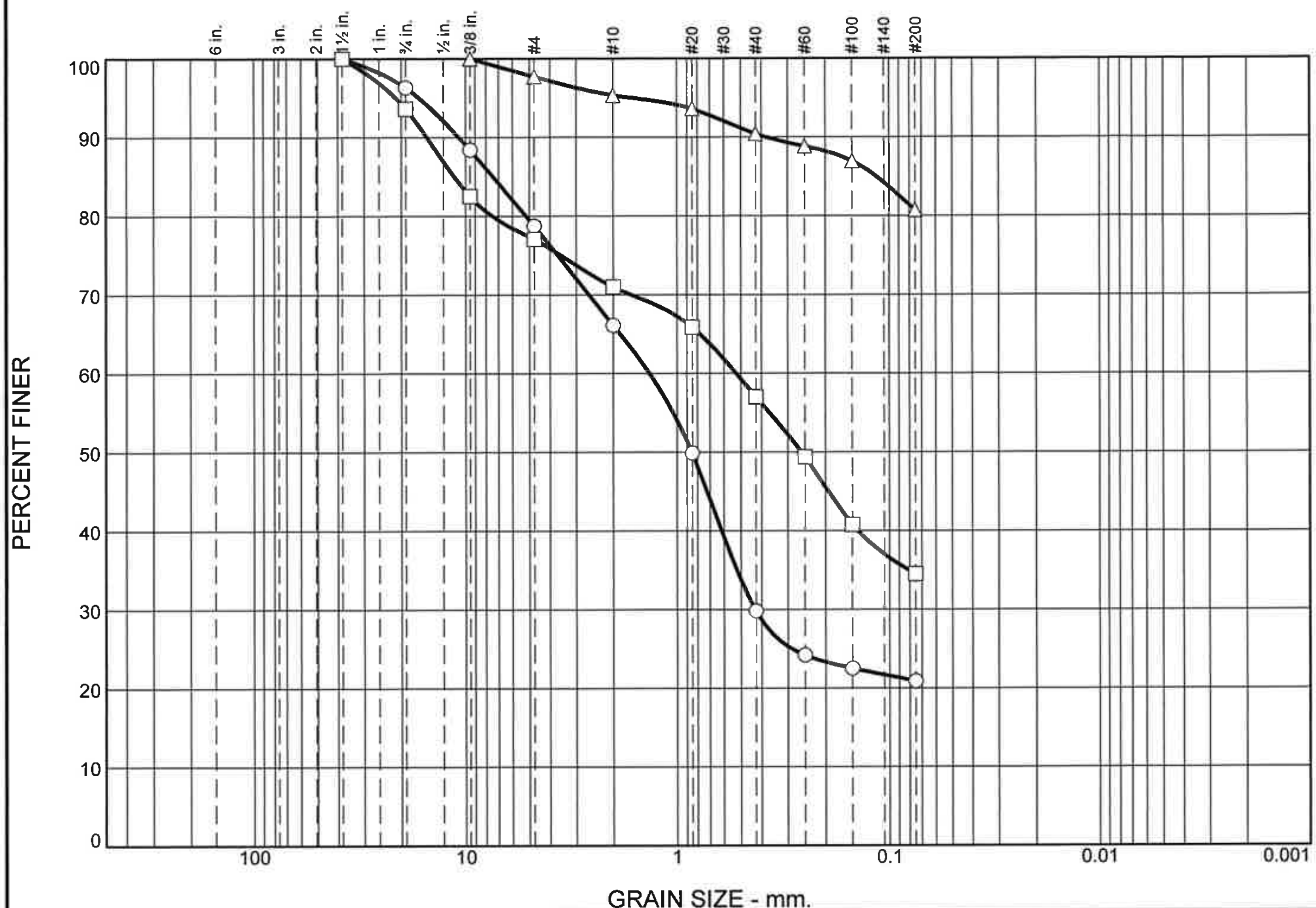
Kirkland, WA

Figure A-8

Tested By: KJ

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	3.7	17.6	12.6	36.3	8.9	20.9	
□	0.0	6.4	16.6	6.0	14.0	22.5	34.5	
△	0.0	0.0	2.3	2.4	4.9	9.7	80.7	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			7.4175	1.3479	0.8531	0.4291				
□			11.3050	0.5266	0.2595					
△			0.1161							

Material Description	USCS	AASHTO
○ silty SAND with gravel	SM	
□ silty SAND with gravel	SM	
△ SILT with sand	ML	

Project No. T-8879 **Client:** Covenant Living Communities & Services

Project: Covenant Living at the Shores

○ **Location:** B-1 **Depth:** 35'

□ **Location:** B-3 **Depth:** 8'

△ **Location:** B-3 **Depth:** 20'

Remarks:

○ Tested April 27, 2023

□ Tested April 27, 2023

△ Tested April 27, 2023

Terra Associates, Inc.

Kirkland, WA

Figure A-9

Tested By: KJ

Particle Size Distribution Report

	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.0	6.7	5.0	20.2	39.1	29.0	
□	0.0	0.0	0.0	0.0	0.1	9.4	90.5	
△	0.0	3.8	0.4	2.0	9.0	35.5	49.3	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			1.0147	0.3152	0.2268	0.0856				
□										
△			0.4373	0.1470	0.0809					

Material Description	USCS	AASHTO
○ silty SAND	SM	
□ SILT	ML	
△ silty SAND	SM	

Project No. T-8879 **Client:** Covenant Living Communities & Services

Project: Covenant Living at the Shores

○ **Location:** B-4 **Depth:** 7.5'

□ **Location:** B-4 **Depth:** 30'

△ **Location:** B-5 **Depth:** 5'

Remarks:

○ Tested April 27, 2023

□ Tested April 27, 2023

△ Tested April 27, 2023

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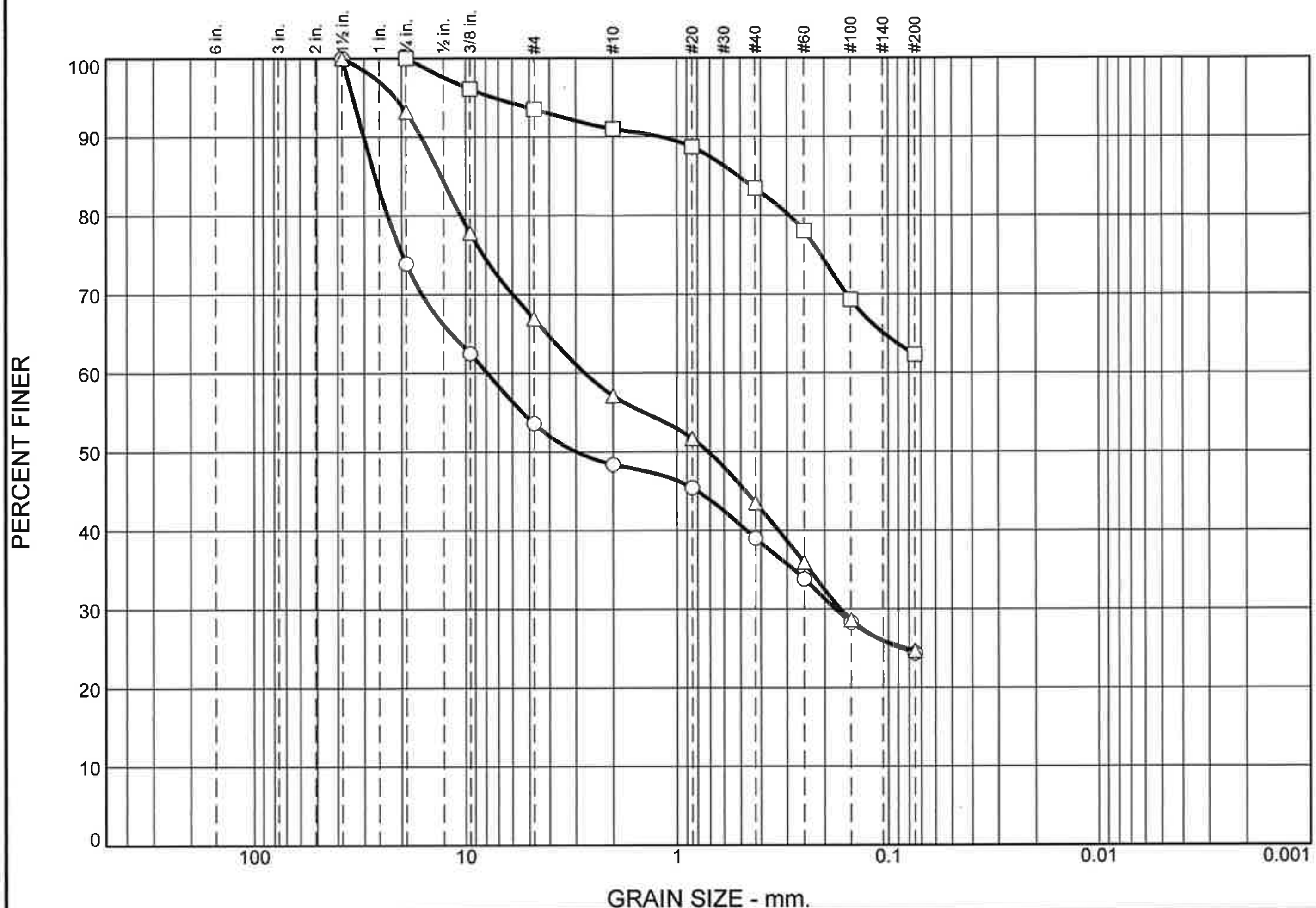
Kirkland, WA

Figure A-10

Tested By: KJ

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	26.1	20.3	5.2	9.4	14.6	24.4	
□	0.0	0.0	6.5	2.5	7.6	21.1	62.3	
△	0.0	6.8	26.3	9.8	13.6	18.8	24.7	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			26.5853	7.8774	2.9891	0.1779				
□			0.5135							
△			13.0234	2.7172	0.7125	0.1685				

Material Description	USCS	AASHTO
○ silty GRAVEL with sand	GM	
□ sandy SILT	ML	
△ silty SAND with gravel	SM	

Project No. T-8879 **Client:** Covenant Living Communities & Services

Project: Covenant Living at the Shores

○ **Location:** B-5 **Depth:** 10'

□ **Location:** B-6 **Depth:** 5'

△ **Location:** B-6 **Depth:** 10'

Remarks:

○ Tested April 27, 2023

□ Tested April 28, 2023

△ Tested April 28, 2023

Terra Associates, Inc.

Kirkland, WA

Figure A-11

Tested By: KJ

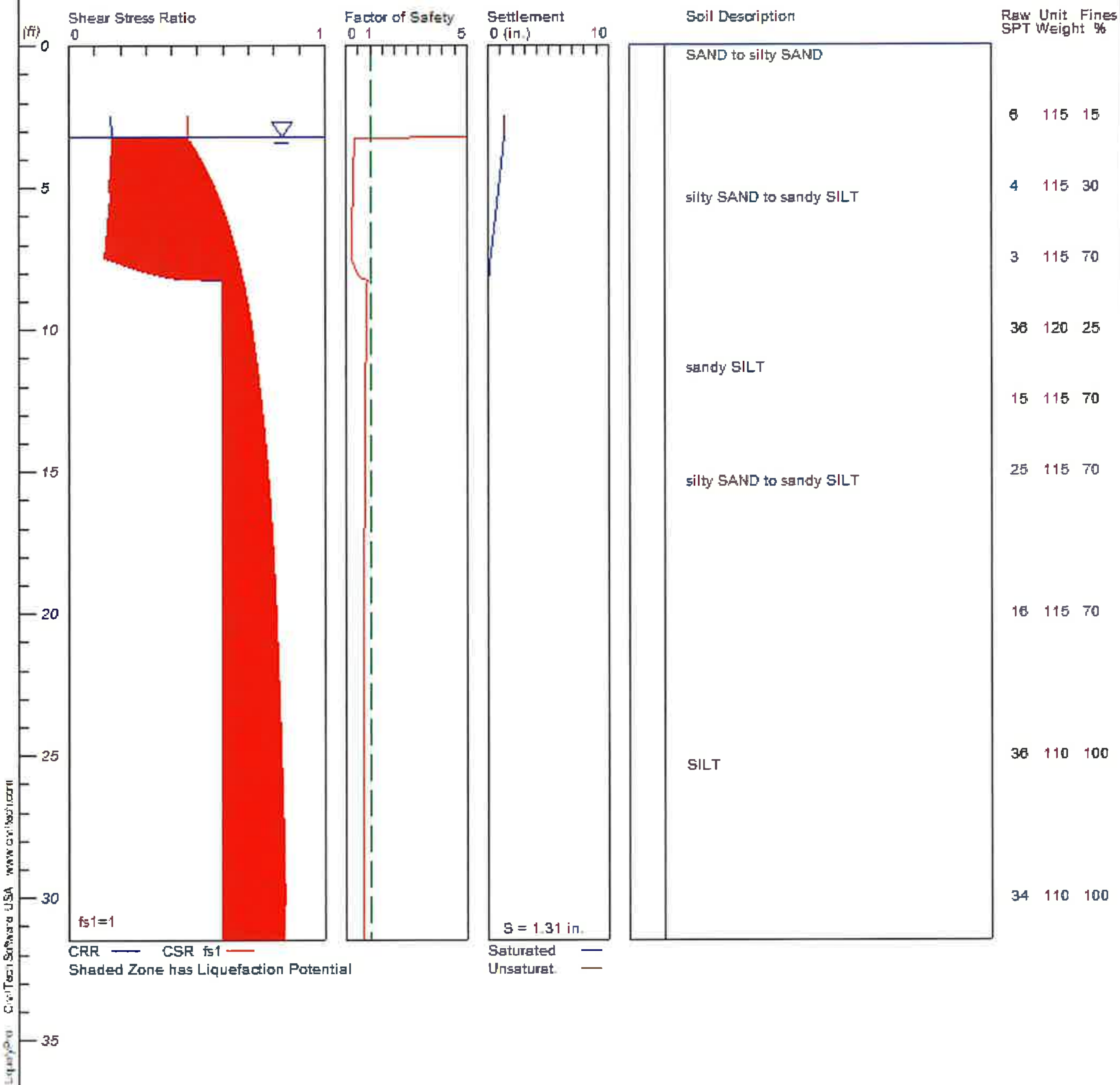
APPENDIX B
LIQUEFACTION ANALYSES

LIQUEFACTION ANALYSIS

Covenant Living

Hole No.=B-5 Water Depth=3.2 ft Surface Elev.=30

Magnitude=7
Acceleration=0.715g



LIQUEFACTION ANALYSIS

Covenant Living

Hole No.=B-6 Water Depth=2.5 ft Surface Elev.=30

Magnitude=7
Acceleration=0.715g