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Subsurface Exploration, Geologic Hazard,
and Preliminary Geotechnical Engineering Report

HERZL-NER TAMID CONSERVATIVE CONGREGATION K-12 EXPANSION

Mercer Island, Washington

Prepared For:

HERZL-NER TAMID CONSERVATIVE CONGREGATION

Project No. 20210371E001
October 27, 2023

Associated Earth Sciences, Inc.

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Herzl-Ner Tamid Conservative Congregation
3700 East Mercer Way
Mercer Island, Washington 98040

Attention: Ms. Audrey Covner

Subject: Subsurface Exploration, Geologic Hazard,
and Preliminary Geotechnical Engineering Report
Herzl-Ner Tamid Conservative Congregation K-12 Expansion
3700 East Mercer Way
Mercer Island, Washington

Dear Ms. Covner:

We are pleased to present our preliminary geotechnical engineering report for the proposed K-12 school expansion project at the Herzl-Ner Tamid Conservative Congregation Synagogue Campus. This report summarizes the results of our subsurface explorations, geologic hazard, infiltration feasibility, and geotechnical engineering studies and offers preliminary design recommendations based on our present understanding of the project. Once project plans are developed and finalized, we should review the plans and confirm or update the recommendations in this report.

We have enjoyed working with you on this study and are confident that the recommendations presented in this report will aid in the successful completion of your project. If you should have any questions or if we can be of additional help to you, please do not hesitate to call.

Sincerely,
ASSOCIATED EARTH SCIENCES, INC.
Kirkland, Washington

G. Bradford Drew, P.E.
Senior Engineer

BD/ld - 20210371E001-003

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SUBSURFACE EXPLORATION, GEOLOGIC HAZARD, AND PRELIMINARY GEOTECHNICAL ENGINEERING REPORT

HERZL-NER TAMID CONSERVATIVE CONGREGATION K-12 EXPANSION

Mercer Island, Washington

Prepared for:
Herzl-Ner Tamid Conservative Congregation
3700 East Mercer Way
Mercer Island, Washington 98040

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October 27, 2023
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I. PROJECT AND SITE CONDITIONS

1.0 INTRODUCTION

This report presents the results of Associated Earth Sciences, Inc.'s (AESI's) subsurface exploration, geologic hazard, and preliminary geotechnical engineering study for the proposed K-12 school expansion at the Herzl-Ner Tamid Conservative Congregation Synagogue Campus in Mercer Island, Washington. Our recommendations are preliminary as the project is in the conceptual design phase at this time. We will provide a final design report that addresses the project details once the type, size, and locations of the new buildings and ancillary structures are finalized. The site location is shown on the "Vicinity Map," Figure 1. The approximate locations of explorations completed for this study are shown on the "Existing Site and Exploration Plan," Figure 2. Copies of the exploration logs for this current study are included in Appendix A.

1.1 Purpose and Scope

The purpose of this study was to provide subsurface soil and groundwater data to be utilized in the design of the project. Our study included reviewing available geologic literature, advancing three exploration borings, and performing a geologic study to assess the type, thickness, distribution, and physical properties of the subsurface sediments and shallow groundwater conditions across the project area. Geotechnical engineering studies were completed to determine the type of suitable foundations, allowable foundation soil bearing pressures, anticipated foundation settlements, drainage considerations, and stormwater infiltration feasibility. This report summarizes our current fieldwork and offers preliminary design recommendations based on our present understanding of the project.

2.0 SITE AND PROJECT DESCRIPTION

The project location is the Herzl-Ner Tamid Conservative Congregation Synagogue Campus located at 3700 East Mercer Way in Mercer Island, Washington. The campus includes several King County parcels covering an approximate area of 3.6 acres with the main synagogue building centrally located across the parcels. To the west of the building is an undeveloped forested area covering approximately 0.6 acres. South of the forested area is a paved parking lot. Topography across the forested portion of the site slopes down gently to the north and gradually steepens beyond the property boundary toward Frontage Road. Overall vertical relief across the forested area ranges from about 5 to 10 feet, and the slope to the north of the forested area is inclined at about 60 percent over a maximum height of about 20 feet. East of the synagogue building topography slopes down gently to the east toward the Lake Washington shoreline.

We understand that a new K-12 school building is being planned within the forested portion of the site, west of the existing synagogue building. A conceptual site plan provided by the project architect, Anjali Grant Design LLC, indicates the new building will contain three levels of above-grade classrooms and rental offices, and a basement level for the gymnasium. The basement level generally spans the eastern half and southern margin of the forested area, extends about 18 feet below existing site grades at its deepest point, and generally daylights along the northwest margin of the site. The concept plans, dated September 6, 2023, are attached in Appendix B for reference.

3.0 SITE EXPLORATION

Our field studies for this phase of work were completed in September 2023 and included advancing three exploration borings (EB-1 through EB-3) across the site to define the general soil and shallow groundwater conditions below the proposed K-12 expansion building. The exploration locations are shown on the "Existing Site and Exploration Plan," Figure 2 and "LIDAR Based Shaded Relief," Figure 3. The various types of sediments, as well as the depths where characteristics of the sediments changed, are indicated on the exploration logs presented in Appendix A. The depths indicated on the logs where conditions changed may represent gradational variations between sediment types in the field. The locations of our field explorations were determined by approximate measurements from known site features.

The conclusions and recommendations presented in this report are based, in part, on the exploration borings completed for this study. The number, locations, and depths of the explorations were completed within site and budgetary constraints. Because of the nature of exploratory work below ground, extrapolation of subsurface conditions between field explorations is necessary. It should be noted that differing subsurface conditions might sometimes be present due to the random nature of deposition and the alteration of topography by past grading and/or filling. The nature and extent of variations between the field explorations may not become fully evident until construction. If variations are observed at that time, it may be necessary to re-evaluate specific recommendations in this report and make appropriate changes.

3.1 Exploration Borings

The exploration borings were completed by Geologic Drill Partners Inc., an independent driller working under subcontract to AESI, by advancing a 6-inch outside-diameter, hollow-stem auger with a track-mounted drill rig. During the drilling process, samples were generally obtained at 2½-foot to 5-foot-depth intervals. After drilling, each borehole was backfilled with bentonite chips, and the surface was patched using cold-mix asphalt in existing pavement areas.

Disturbed, but representative samples were obtained by using the Standard Penetration Test (SPT) procedure in accordance with *ASTM International* (ASTM) D-1586. This test and sampling method consists of driving a standard 2-inch, outside-diameter, split-barrel sampler a distance of 18 inches into the soil with a 140-pound hammer free-falling a distance of 30 inches. The number of blows for each 6-inch interval is recorded, and the number of blows required to drive the sampler the final 12 inches is known as the Standard Penetration Resistance ("N") or blow count. If a total of 50 is recorded within one 6-inch interval, the blow count is recorded as the number of blows for the corresponding number of inches of penetration. The resistance, or N-value, provides a measure of the relative density of granular soils or the relative consistency of cohesive soils; these values are plotted on the attached exploration boring logs.

The borings were continuously observed and logged by a geologist from our firm. The samples obtained from the split-barrel sampler were classified in the field and representative portions placed in watertight containers. The samples were then transported to our laboratory for further visual classification and laboratory testing. The exploration logs presented in Appendix A are based on the N-values, field observations, and drilling action.

4.0 SUBSURFACE CONDITIONS

Subsurface conditions at the project site were inferred from the field explorations accomplished for this study, visual reconnaissance of the site, and review of applicable geologic literature. The following sections describe observed site stratigraphy, regional geology, and local groundwater.

The near-surface native sediments encountered in our explorations generally consisted of a surficial layer of existing fill overlying native nonglacial sediments of pre-Fraser age. The following section presents more detailed subsurface information.

4.1 Stratigraphy

The following subsections summarize our observations and interpretations of different sedimentary units observed in subsurface explorations in order of deposition from most recent to oldest.

Fill

Directly below the ground surface and pavement sections, existing fill soils (those not naturally deposited) were encountered within exploration borings EB-1, EB-2, and EB-3. The fill generally consisted of dry to slightly moist, primarily gray with zones of brown and tan, soft to stiff fine sandy silt ranging to silt with variable gravel and organic content. The existing fill ranged in thickness from about 4 to 5 feet in EB-1 and EB-2, and about 10 feet in EB-3.

Existing fill is not considered suitable for foundation support and will require removal and replacement with structural fill in areas where the building foundation is close to existing grade, such as the northwest portion of the site. Given the west and south portions of the building will be 10 to 18 feet below grade, we anticipate the extent of fill removal will be confined to the northwest portion of the site (although variable conditions can be expected in unexplored areas).

Excavated existing fill material may be suitable for reuse in structural fill applications if such reuse is specifically allowed by project plans and specifications, if excessively organic and any other deleterious materials are removed, and if moisture content is adjusted to allow compaction to the specified level and to a firm and unyielding condition. The silty fill soils may prove difficult to reuse as structural fill due to the high percentage of fine-grained (silt-sized) sediments which make them highly moisture-sensitive and subject to disturbance when wet. Existing fill is not suitable for infiltration of stormwater.

Pre-Fraser Nonglacial Deposits

Directly below the existing fill, explorations EB-1 and EB-2 encountered a deposit of dry grading to moist with depth, stiff grading to hard with depth, sandy silt with variable beds of dense sand and silty sand. These sediments also contained occasional silt and clay interbeds, slightly disturbed textures, sandy laminations, micas, scattered fine organics, and dark orange-brown to black iron-oxide staining. We interpreted these sediments as pre-Fraser nonglacial sediments. These sediments were deposited in a nonglacial environment and overridden by glacial ice during a subsequent glaciation. Due to their stratigraphic position, we infer that these sediments were deposited prior to the Fraser Glaciation that occurred between 12,500 to 15,000 years before present and have been consolidated by at least one glaciation. The pre-Fraser nonglacial sediments extended to depths of approximately 40.5 feet in EB-1 and the termination depth of EB-2 at 21.5 feet.

The stiff to hard pre-Fraser nonglacial sediments are suitable for foundation support with proper preparation as recommended in this report. The pre-Fraser sediments generally contain a significant fraction of fine-grained material (silt and clay) and will be difficult to reuse as structural fill as they are highly moisture-sensitive, difficult to aerate and dry when above optimum moisture, and subject to disturbance when wet. Due to the high percentage of fine-grained material and high density, the pre-Fraser fine-grained sediments are expected to have low-permeability characteristics and are considered unsuitable as a stormwater infiltration receptor.

Pre-Fraser Lacustrine Deposits

Directly below the pre-Fraser nonglacial sediments, EB-1 and EB-3 encountered stiff to hard, bluish gray to light brown silt with trace fine micas and trace to absent sand content. We interpreted these sediments as pre-Fraser lacustrine deposits. The pre-Fraser lacustrine sediments consist of fine sediments that were deposited in a lake environment prior to the Vashon Stade of the Fraser Glaciation, approximately 12,500 to 15,000 years ago. The high relative density characteristic of the pre-Fraser lacustrine deposits is due to its consolidation by the glacial ice that overrode these sediments after their deposition. The pre-Fraser lacustrine deposits extended beyond the maximum depths explored in EB-1 (41.5 feet) and EB-3 (26.5 feet).

While it is not likely that pre-Fraser lacustrine sediments will be handled in substantial quantity during construction due to the depth and location in which they were encountered, these sediments are suitable for support of building foundations with proper preparation. Sand content of the pre-Fraser lacustrine sediments was variable but generally low. Due to the high percentage of fine-grained material and high density, the pre-Fraser lacustrine sediments are expected to have low-permeability characteristics and are considered unsuitable as a stormwater infiltration receptor.

4.2 Regional Geologic Mapping

Review of the geologic map of the project area (*Geologic Map of Mercer Island, Washington,* by Kathy G. Troost and Aaron P. Wisher, GeoMapNW, October 2006) indicates that the site is expected to be underlain by Vashon subglacial till deposits with older pre-Fraser nonglacial deposits just south of the site. Our interpretation of the sediments encountered in our recent explorations is somewhat in agreement with the regional geologic map. We did not encounter Vashon till below the site but did encounter the older pre-Fraser nonglacial deposits beneath the existing fill.

4.3 Hydrology

No groundwater was encountered at the time of drilling within any of the three explorations completed for this study; however, it should be noted that our site explorations were conducted in September when groundwater levels are typically nearing a seasonal low. During wetter periods of the year, zones of perched groundwater may be present within more-permeable strata in existing fill and within sandy zones of the pre-Fraser nonglacial deposits. The occurrence and level of groundwater seepage encountered during construction will largely depend on the soil grain-size distribution, topography, seasonal precipitation, on- and off-site land usage, and other factors.

II. GEOLOGIC HAZARDS AND MITIGATIONS

The following discussion of potential geologic hazards is based on the geologic, slope, and ground and surface water conditions, as observed and discussed herein. The discussion will be limited to landslide, seismic, and erosion hazards. Individual geologic hazard topics are discussed in further detail below.

5.0 LANDSLIDE HAZARDS AND MITIGATIONS

Topography across the forested portion of the site and within the footprint of the proposed K-12 expansion building generally slopes down gently to the north, with localized areas of moderately sloping terrain and hummocky topography, and gradually steepens beyond the property boundary toward Frontage Road. We interpret the hummocky topography to be a result of random fill placement across the site rather than indicative of landslide activity. Areas to the east, south, and west of the proposed building are generally flat to gently sloping. Overall vertical relief across the forested area ranges from about 5 to 10 feet.

The area that steepens to the north toward Frontage Road contains a steep slope inclined at about 60 percent over a maximum height of about 20 feet. The crest of this slope is located about 10 to 25 feet away from the proposed building footprint. This slope classifies as a landslide hazard area in accordance with the *City of Mercer Island Municipal Code*. Based on our site reconnaissance and review of site topography, this slope appears to be the result of grading for Frontage Road that crosses beneath Interstate 90 and could be a combination of cut and fill. The slope is well vegetated with ivy ground cover and mature trees. Some of the trees appeared to be leaning or slightly bowed near the trunk, indicating the slope may be experiencing soil creep; however, we did not observe any obvious signs of landslide activity or groundwater seepage emanating from the slope face.

We also reviewed the 2021 King County Light Detection and Ranging (LIDAR)-based shaded relief map of the site and steep slope area as provided on the Washington State Department of Natural Resources (WADNR), Division of Geology and Earth Resources, LIDAR portal. Shaded relief maps generated from LIDAR data provide a detailed image of the ground surface, even in heavily vegetated areas. Such images can reveal the presence of geomorphic features, such as faults or landslide features, not visible on conventional aerial photos. The LIDAR-based shaded relief map of the project site and vicinity along with topographic contours are shown on Figure 3. The LIDAR imaging does not reveal any indications of recent landsliding activity along the steep slope area.

Provided the slope is comprised of glacially consolidated native sediments at shallow depths, similar to the subsurface conditions encountered within exploration EB-1, and considering that the eastern half of the proposed building will contain a basement level that results in additional setback between the building footings and slope face (up to 50 feet of setback where the basement level finished floor is proposed at an elevation of about 86 feet), the risk of slope movement affecting the proposed development is low, in our opinion. Additional explorations and slope stability evaluation may become necessary as the project design and proposed site grading are finalized.

6.0 SEISMIC HAZARDS AND MITIGATIONS

The following discussion is a general assessment of seismic hazards that is intended to be useful to the project design team in terms of understanding seismic issues, and to the structural engineer for design.

All of Western Washington is at risk of strong seismic events resulting from movement of the tectonic plates associated with the Cascadia Subduction Zone (CSZ), where the offshore Juan de Fuca plate subducts beneath the continental North American plate. The site lies within a zone of strong potential shaking from subduction zone earthquakes associated with the CSZ. The CSZ can produce earthquakes up to magnitude 9.0, and the recurrence interval is estimated to be on the order of 500 years. Geologists infer the most recent subduction zone earthquake occurred in 1700 (Goldfinger et al., 2012[1]). Three main types of earthquakes are typically associated with subduction zone environments: crustal, intraplate, and interplate earthquakes. Seismic records in the Puget Sound region document a distinct zone of shallow crustal seismicity (e.g., the Seattle Fault Zone [SFZ]). These shallow fault zones may include surficial expressions of previous seismic events, such as fault scarps, displaced shorelines, and shallow bedrock exposures. The shallow fault zones typically extend from the surface to depths ranging from 16 to 19 miles. A deeper zone of seismicity is associated with the subducting Juan de Fuca plate. Subduction zone seismic events produce intraplate earthquakes at depths ranging from 25 to 45 miles beneath the Puget Lowland including the 1949, 7.2-magnitude event; the 1965, 6.5-magnitude event; and the 2001, 6.8-magnitude event) and interplate earthquakes at shallow depths near the Washington coast including the 1700 earthquake, which had a magnitude of approximately 9.0. The 1949 earthquake appears to have been the largest in this region during recorded history and was centered in the Olympia area. Evaluation of earthquake return rates indicates that an earthquake of the magnitude between 5.5 and 6.0 is likely within a given 20-year period.

Generally, there are four types of potential geologic hazards associated with large seismic events: 1) surficial ground rupture, 2) seismically induced landslides, 3) liquefaction, and 4) ground

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[1] Goldfinger, C., Nelson, C.H., Morey, A.E., Johnson, J.E., Patton, J.R., Karabanov, E., Gutierrez-Pastor, J., Eriksson, A.T., Gracia, E., Dunhill, G., Enkin, R.J., Dallimore, A., and Vallier, T., 2012, *Turbidite Event History—Methods and Implications for Holocene Paleoseismicity of the Cascadia Subduction Zone*: U.S. Geological Survey Professional Paper 1661–F, 170.

motion. The potential for each of these hazards to adversely impact the proposed project is discussed below.

6.1 Surficial Ground Rupture

Seattle Fault Zone

The site is located within the mapped limits of the SFZ. The SFZ is a broad east-west oriented zone that extends from approximately Issaquah to Alki Beach, and is approximately 2.5 to 4 miles in width from north to south. The SFZ is speculated to contain multiple distinct fault "strands," some of which are well understood and some of which may be poorly understood or unknown. Mapping of individual fault strands is imprecise, as a result of pervasive modification of the land surface by development, which has obscured possible surficial expression of past seismic events. Studies by the U.S. Geological Survey (USGS) and others have provided evidence of surficial ground rupture along strands of the Seattle Fault (USGS, 2010[2]; Pratt et al., 2015[3]; Haugerud, 2005[4]; Liberty et al., 2008[5]). According to USGS studies the latest movement of this fault was about 1,100 years ago when about 20 feet of surficial displacement took place. This displacement can presently be seen in the form of raised, wave-cut beach terraces along Alki Point in West Seattle and Restoration Point at the south end of Bainbridge Island. Based on our review of the WADNR website, inferred fault traces associated with the SFZ are located along the southern boundary of the subject property. Further review of the Mercer Island Seismic Hazard Assessment (Kathy Troost, Aaron Wisher; April, 2009) indicates that the strand is mapped as the Vasa Park Fault, and has no known surface rupture on Mercer Island; however, it does form a scarp east of Mercer Island. It further indicates that the fault is not well defined on Mercer Island. Due to the suspected long recurrence interval, lack of known surficial rupture, and location uncertainty, the potential for surficial ground rupture is considered to be low during the expected life of the proposed structure.

6.2 Seismically Induced Landslides

Similar to the discussion in the "Landslide Hazards and Mitigations" section above, it is our opinion that the potential risk of damage to the proposed improvements by seismically induced slope failures is low provided the slope is comprised of glacially consolidated native sediments at

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[2] U.S. Geological Survey, 2010, *Quaternary Fault and Fold Database for the United States*, accessed November 10, 2010, from USGS web site: http://earthquake.usgs.gov/hazards/qfaults/.

[3] Pratt et al., 2015, *Kinematics of Shallow Backthrusts in the Seattle Fault Zone, Washington State*: Geosphere, v. 11, no. 6, p. 1-27).

[4] Haugerud, R.A., 2005, *Preliminary Geologic Map of Bainbridge Island, Washington*: U.S. Geological Survey Open-File Report 2005-1387, version 1.0, 1 sheet, scale 1:24,000.

[5] Liberty, Lee M.; Pratt, Thomas L., 2008, *Structure of the Eastern Seattle Fault Zone, Washington State - New Insights from Seismic Reflection Data*: Bulletin of the Seismological Society of America, v. 98, no. 4, p. 1681-1695.

shallow depths, similar to the subsurface conditions encountered within exploration EB-1, and considering that the eastern half of the proposed building will contain a basement level that provides additional setback between the building footings and slope face. Additional explorations and slope stability evaluation may become necessary as the project design and proposed site grading are finalized.

6.3 Liquefaction

Liquefaction is a process through which unconsolidated soil loses strength as a result of vibrations, such as those which occur during a seismic event. During normal conditions, the weight of the soil is supported by both grain-to-grain contacts and by the fluid pressure within the pore spaces of the soil below the water table. Extreme vibratory shaking can disrupt the grain-to-grain contact, increase the pore pressure, and result in a temporary decrease in soil shear strength. The soil is said to be liquefied when nearly all of the weight of the soil is supported by pore pressure alone. Liquefaction can result in deformation of the sediment and settlement of overlying structures. Areas most susceptible to liquefaction include those areas underlain by very soft to stiff, non-cohesive silt and very loose to medium dense, non-silty to silty sands with low relative densities, accompanied by a shallow water table.

The project site is generally comprised of a surficial layer of existing fill underlain by glacially consolidated native sediments with a lack of adverse groundwater conditions. In our opinion, the potential risk of damage to the proposed site improvements by liquefaction is low due to the high relative density of the underlying native sediments and the absence of adverse groundwater conditions at the site. No detailed liquefaction hazard analysis was performed for this study, and none is warranted, in our opinion.

6.4 Ground Motion/Seismic Site Class

It is our opinion that earthquake damage to the proposed building, when founded on suitable bearing strata in accordance with the recommendations contained herein, will be caused by the intensity and acceleration associated with the event. We assume that structural design of the buildings will follow the 2018 *International Building Code* standards and the American Society of Civil Engineers (ASCE) *7 - Minimum Design Loads for Buildings and Other Structures*, the current version of which is ASCE 7-16. Based on the subsurface conditions encountered in our explorations within the vicinity of the proposed building, we recommend using Site Class “D” as defined in Table 20.3-1 of ASCE 7-16.

7.0 EROSION HAZARDS AND MITIGATIONS

The sediments underlying the site generally consist of fine sand and silt. These sediments will be susceptible to erosion and off-site sediment transport when exposed during construction. Therefore, the project should follow best management practices (BMPs) to mitigate erosion hazards and potential for off-site sediment transport. To mitigate the potential for off-site sediment transport, we recommend the following:

1. Construction activity should be scheduled or phased as much as possible to reduce the amount of earthwork activity that is performed during the winter months.

2. The winter performance of a site is dependent on a well-conceived plan for control of site erosion and stormwater runoff. The project temporary erosion and sediment control (TESC) should include ground-cover measures, access roads, and staging areas. The contractor must implement and maintain the required measures.

3. TESC measures for a given area, to be graded or otherwise worked, should be installed prior to any activity within that area. The recommended sequence of construction within a given area would be to install sediment traps and/or ponds and establish perimeter flow control prior to starting earthwork.

4. During the wetter months of the year, or when large storm events are predicted during the summer months, each work area should be stabilized so that if precipitation occurs, the work area can receive the rainfall without excessive erosion or sediment transport. The required measures for an area to be “buttoned-up” will depend on the time of year and the duration the area will be left unworked. During the winter months, areas that are to be left unworked for more than 2 days should be mulched or covered with plastic. During the summer months, stabilization will usually consist of seal-rolling the subgrade. Such measures will aid in the contractor’s ability to get back into a work area after a storm event. The stabilization process also includes establishing temporary stormwater conveyance channels through work areas to route runoff to the approved treatment facilities.

5. All disturbed areas should be revegetated as soon as possible. If it is outside of the growing season, the disturbed areas should be covered with mulch, or as recommended in the erosion control plan. Straw mulch provides a cost-effective cover measure and can be made wind-resistant with the application of a tackifier after it is placed.

6. Surface runoff and discharge should be controlled during and following development. Uncontrolled discharge may promote erosion and sediment transport.

7. Soils that are to be reused around the site should be stored in such a manner as to reduce erosion from the stockpile. Protective measures may include, but are not limited to, covering with plastic sheeting, the use of low stockpiles in flat areas, or the use of straw bales/silt fences around pile perimeters. During the local wet season period, between November 1 and March 31, these measures are required.

It is our opinion that with the proper implementation of the TESC plans and by field-adjusting appropriate mitigation elements (BMPs) throughout construction, the potential for adverse impacts from erosion hazards on the project may be mitigated.

III. PRELIMINARY DESIGN RECOMMENDATIONS

8.0 INTRODUCTION

Our explorations indicate that, from a geotechnical engineering standpoint, the proposed project is feasible provided the recommendations contained herein are properly followed. At the locations explored, we encountered a surficial layer of existing fill, ranging in thickness from about 5 to 10 feet, overlying glacially consolidated native sediments consisting of stiff to hard pre-Fraser nonglacial deposits and pre-Fraser lacustrine deposits. The pre-Fraser sediments will provide suitable support for conventional spread and strip footings with proper preparation. The existing fill soils are not considered suitable for direct foundation support and may require remedial measures for support of slabs-on-grade, pavements, and hardscapes.

The following sections provide our preliminary design recommendations for site preparation, temporary and permanent slopes, structural fill, foundation support, slab-on-grade support, lateral earth pressures on below-grade walls, drainage considerations, temporary excavation shoring methods, pavement design, and infiltration feasibility. Our recommendations are preliminary in that site development plans, site grading plans, structural plans, and construction methods have not been finalized.

9.0 SITE PREPARATION

Prior to site work, erosion and surface water control should be established around the perimeter of the site to satisfy City of Mercer Island requirements, as discussed in the “Erosion Hazards and Mitigations” section of this report.

9.1 Clearing and Stripping

Existing pavements, buried utilities, vegetation, topsoil, and any other deleterious materials should be removed where they are located below planned construction areas. Any disturbed soils or depressions, such as those that may be caused by demolition activities or tree removal, below planned final grades should be compacted with a smooth-drum vibratory roller to at least 95 percent of the modified Proctor maximum dry density as determined by the ASTM D-1557 test procedure, and to a firm and unyielding surface. Structural fill should be placed as needed to restore planned grades as discussed under the “Structural Fill” section of this report.

Where excavated existing fill and natural sediments are free of organics and near their optimum moisture content for compaction they can be segregated and considered for reuse as structural fill if allowed by project specifications. Most of the native sediments encountered in our explorations contained significant silt fractions and are considered highly moisture-sensitive; these soils may be difficult to reuse as structural fill.

9.2 Existing Fill

After clearing, stripping, and any planned excavations have been completed, any remaining existing fill should be addressed. Below the planned building, existing fill should be removed and replaced with structural fill as needed to establish the building pad. Below areas of planned pavements, the existing fill should be exposed, compacted, and proof-rolled under the observation of AESI. Any areas that are soft, yielding, or contain excessive organic material or debris should be corrected as needed prior to paving.

Erosion and surface water control should be established around the perimeter of the excavation to satisfy City of Mercer Island requirements. Site preparation should include removal of all existing pavements, structures, buried utilities, and any other deleterious material below the building footprint. Existing fill should be removed from below the building foundations until suitable native soils are exposed, and the fill removal should extend laterally beyond a 1H:1V (Horizontal:Vertical) slope projected down from the footing limits. The resulting surface should then be compacted and proof-rolled before placing structural fill, as necessary, to reach planned grades.

9.3 Site Disturbance

The existing fill and native sediments contain a high percentage of fine-grained material. These sediments are considered to be highly moisture-sensitive and subject to disturbance when wet. The contractor must use care during site preparation and excavation operations so that the underlying soils are not softened. If disturbance occurs, the softened soils should be removed and the area brought to grade with structural fill.

9.4 Wet Weather Considerations

The on-site soils are considered to be highly moisture-sensitive. If construction takes place in, during, or immediately following the wetter periods of the year, we anticipate the on-site soils will become unsuitable for structural fill applications. If earthwork will be completed during wet season months, we recommend budgeting to construct all structural fills with select, imported fill materials. For construction immediately following wet periods, significant, but unavoidable effort will be needed to scarify, aerate, and dry site soils to reduce moisture content prior to compaction in structural fill applications. Care should be taken to seal all earthwork areas during mass grading at the end of each workday by grading all surfaces to drain and sealing them with a smooth-drum roller. Stockpiled soils that will be reused in structural fill applications should be covered whenever rain is possible.

Construction during extended wet weather periods could create the need to overexcavate exposed soils if they become disturbed and cannot be recompacted due to elevated moisture content and/or weather conditions. Even during dry weather periods, soft/wet soils may be encountered in some portions of the site that will require overexcavation. If overexcavation is necessary, it should be confirmed through continuous observation and testing by AESI. Soils that have become unstable may require remedial measures in the form of one or more of the following:

1. Drying and recompaction. Selective drying may be accomplished by scarifying or windrowing surficial material during extended periods of dry and warm weather.

2. Removal of affected soils to expose a suitable bearing subgrade and replacement with compacted structural fill.

3. Mechanical stabilization with a coarse crushed aggregate compacted into the subgrade, possibly in conjunction with a geotextile.

4. Soil/cement admixture stabilization.

Consideration should be given to protecting access and staging areas with an appropriate section of crushed rock or asphalt treated base (ATB). If crushed rock is considered for the access and staging areas, it should be underlain by engineering stabilization fabric (such as Mirafi 500X or approved equivalent) to reduce the potential of fine-grained materials pumping up through the rock during wet weather and turning the area to mud. The fabric will also aid in supporting construction equipment, thus reducing the amount of crushed rock required. We recommend that at least 10 inches of rock be placed over the fabric. Crushed rock used for access and staging areas should be of at least 2-inch size.

9.5 Temporary and Permanent Slopes

In our opinion, stable construction slopes should be the responsibility of the contractor and should be determined during construction. For estimating purposes, however, we anticipate that temporary, unsupported cuts into the existing fill or native soils can be made near vertical to a maximum depth of 4 feet. If excavations greater than 4 feet are required, then temporary, unsupported cut slopes can be planned at maximum inclinations of 1.5H:1V. These slope angles are for areas where groundwater seepage is not present at the faces of the slopes. If groundwater or surface water is present when the temporary excavation slopes are exposed, flatter slope angles on the order of 2H:1V to 3H:1V may be required. As is typical with earthwork operations, some sloughing and raveling may occur, especially if groundwater seepage is present in the excavation cuts, and cut slopes may have to be adjusted in the field. In addition, WISHA/OSHA regulations should be followed at all times.

Permanent cut and structural fill slopes that are not intended to be exposed to surface water should be designed at inclinations of 2H:1V or flatter. All permanent cut or fill slopes should be compacted to at least 95 percent of the modified Proctor maximum dry density, as determined by ASTM D-1557, and the slopes should be protected from erosion by sheet plastic until vegetation cover can be established during favorable weather.

10.0 STRUCTURAL FILL

We anticipate that placement of structural fill may be necessary to establish desired grades at the site and for backfilling within utility trenches and around foundation elements. All references to structural fill in this report refer to subgrade preparation, fill type, and placement and compaction of materials as discussed in this section.

10.1 Subgrade Compaction

After overexcavation/stripping have been performed to the satisfaction of the geotechnical engineer, the upper 12 inches of exposed ground should be recompacted to a firm and unyielding condition. If the subgrade contains too much moisture, suitable recompaction may be difficult or impossible to attain and should probably not be attempted. In lieu of recompaction, the area to receive fill should be blanketed with washed rock or quarry spalls to act as a capillary break between the new fill and the wet subgrade. Where the exposed ground remains soft and further overexcavation is impractical, placement of an engineering stabilization fabric may be necessary to prevent contamination of the free-draining layer by silt migration from below. After recompaction of the exposed ground is tested and approved, or a free-draining rock course is laid, structural fill may be placed to attain desired grades.

10.2 Structural Fill Compaction

Structural fill is defined as non-organic soil, acceptable to the geotechnical engineer, placed in maximum 8-inch loose lifts, with each lift being compacted to at least 95 percent of the modified Proctor maximum dry density using ASTM D-1557 as the standard. Utility trench backfill should be placed and compacted in accordance with applicable municipal codes and standards.

10.3 Use of On-Site Soils as Structural Fill

The existing fill and native soils onsite consisting of sand, silty sand, and sandy silt are suitable for use as structural fill provided they are free of roots or other deleterious materials and have a moisture content suitable for achieving the specified compaction. At the time of our exploration, the moisture content for the majority of the near-surface fill and native sediments encountered in our exploration appeared to be near or slightly below optimum for achieving suitable compaction. It should be noted that our explorations were completed in the dry season when moisture conditions are near a seasonal low. The moisture content of the near-surface soils can be expected to be above optimum during wetter periods of the year.

Soils in which the amount of fine-grained material (smaller than No. 200 sieve) is greater than approximately 5 percent (measured on the minus No. 4 sieve size) should be considered moisture-sensitive. The majority of existing fill and native soils contain a substantial amount of silt and are considered highly moisture-sensitive. These soils may require moisture-conditioning before use as structural fill. Good construction practices and erosion control measures will be necessary to protect the fine-grained soils and prevent over-optimum moisture conditions from developing in the finer-grained soil areas.

If structural fill is placed during wet weather or if proper compaction cannot be obtained, a select import material consisting of a clean, free-draining gravel and/or sand should be used. Free-draining fill consists of non-organic soil, with the amount of fine-grained material (silt and clay) limited to 5 percent by weight when measured on the minus No. 4 sieve fraction, and at least 25 percent retained on the No. 4 sieve.

10.4 Structural Fill Testing

Compaction testing will likely be required by the City of Mercer Island. We recommend that a representative from our firm observe the subgrades and be present during placement of structural fill to observe the work and perform a representative number of in-place density tests. In this way, the adequacy of the earthwork may be evaluated as filling progresses and any problem areas may be corrected at that time.

11.0 FOUNDATIONS

Based on the explorations completed for this study, native glacially consolidated sediments suitable for conventional shallow foundation support were observed at about 5 to 10 feet below the existing ground surface. Spread and strip footings may be used for building support when founded either directly on stiff to hard native sediments properly prepared as described in this report, or on structural fill placed over these materials after removal of existing fill.

For footings founded either directly upon stiff to hard native sediments, or on structural fill placed over these native sediments, we recommend using a maximum allowable bearing pressure of 3,000 pounds per square foot (psf) for design purposes, including both dead and live loads. An increase in the allowable bearing pressure of one-third may be used for short-term wind or seismic loading. If structural fill is placed below footing areas, the structural fill should extend laterally beyond a 1H:1V slope projected down from the footing limits.

Perimeter footings should be buried at least 18 inches into the surrounding soil for frost protection. However, all foundations must penetrate to the prescribed bearing strata, and no foundations should be constructed in or above loose, organic, or existing fill soils. Anticipated settlement of footings founded as recommended should be less than 1 inch with differential settlement one-half of the anticipated total settlement. Most of this movement should occur during initial dead load applications. However, disturbed material not removed from footing trenches prior to footing placement could result in increased settlements. All footing areas should be observed by AESI prior to placing concrete to verify that the foundation subgrades are undisturbed and construction conforms to the recommendations contained in this report. Foundation bearing verification by AESI will likely be required by the City as a condition of permitting. Perimeter footing drains should be provided as discussed under the “Drainage Considerations” section of this report.

It should be noted that the area bounded by lines extending downward at 1H:1V from any footing must not intersect another footing or intersect a filled area that has not been compacted to at least 95 percent of ASTM D-1557. In addition, a 1.5H:1V line extending down and away from any footing must not daylight because sloughing or raveling may eventually undermine the footing. Thus, footings should not be placed near the edges of steps or cuts in the bearing soils.

The contractor must use care during site preparation and excavation operations so that the underlying soils are not softened. If disturbance occurs, the softened soils should be removed and foundations extended down to competent natural soil. If foundation excavation will occur during the wet season, consideration should be given to “armoring” the exposed subgrade with a 6- to 8-inch layer of crushed rock or ballast rock to provide a working surface during foundation construction.

12.0 SLAB-ON-GRADE SUPPORT

Floor slabs can be supported directly on stiff to hard native sediments, or on new structural fill placed above the native sediments after removal of any existing fill. All fill placed beneath the slab must be compacted to at least 95 percent of ASTM D-1557. The floor should be cast atop a minimum of 4 inches of washed crushed “chip rock” to act as a capillary break. It should also be protected from dampness by an impervious, 15-mil (minimum thickness) plastic sheeting placed atop the capillary break specifically designed for use as a moisture barrier.

13.0 BASEMENT WALLS

We have provided preliminary design and construction recommendations for design of the basement walls below.

13.1 Static Lateral Earth Pressures

It is likely that some yielding of the basement walls will occur before the interior floor diaphragms are constructed. As such, the walls can be designed to withstand an appropriate *static active earth pressure*. For restrained walls that deflect less than 0.005 times the wall height, an appropriate *static at-rest earth pressure* should be used instead. In both cases, these pressures act over the entire back of the wall and vary with the backslope inclination. Assuming a level backslope and well-drained conditions, we recommend using the following values, which are given in pounds per cubic foot (pcf) of equivalent fluid pressure.

Static Active Earth Pressure:	35 pcf
Static At-Rest Earth Pressure:	55 pcf

13.2 Static Lateral Surcharge Pressures

Any backslope load located within a 45-degree plane projected upward from the wall base will apply a lateral surcharge on the wall. Possible sources of surcharge loading include parking lots, traffic lanes, and structure footings. These surcharge pressures act over the portion of wall adjacent to the load source. For distributed vertical loads, active and at-rest static lateral surcharge pressures can be approximated by multiplying the vertical pressure “Q” in psf by the appropriate coefficient shown below. We recommend using a vertical pressure of 250 psf to model traffic and parking loads behind the wall.

Static Active Surcharge Pressure:	0.28(Q) psf
Static At-Rest Surcharge Pressure:	0.44(Q) psf

13.3 Undrained Wall Pressures

If groundwater is allowed to build up behind a basement wall, a hydrostatic surcharge pressure will act on the inundated portion of the wall. Because this hydrostatic pressure also reduces the aforementioned static earth pressures, the net effect is diminished relative to full hydrostatic water pressure. We recommend using the following combined soil and hydrostatic surcharge pressures for design of undrained walls.

Static Active Earth Pressure:	80 pcf
Static At-Rest Earth Pressure:	90 pcf

13.4 Seismic Lateral Surcharge Pressures

The total static pressures acting on a wall should be increased to account for seismic surcharge loadings resulting from lateral earthquake motions. These surcharge pressures act over the entire back of the wall and vary with the backslope inclination, the seismic acceleration, and the wall height. For retaining walls with a level backslope, active and at-rest seismic lateral surcharge pressures can be approximated by multiplying the wall height "H" (in feet) by the appropriate coefficient shown below.

Seismic Active Surcharge Pressure:	10(H) psf
Seismic At-Rest Surcharge Pressure:	15(H) psf

13.5 Resisting Forces

Lateral pressures acting on a wall are resisted by a combination of passive lateral earth pressure from the embedded portion of wall foundations system, friction between the foundation and the native soils or supporting structural fill soils, and from resistance from the structural slab. For on-site basement walls, we recommend using the following values. These values incorporate static and seismic safety factors of at least 1.5 and 1.1, respectively. Interface friction acting along the bottom of a wall footing can be combined with passive pressure to resist sliding. The coefficient of friction value incorporates a safety factor of 1.5. The soil under the footings must be recompacted to 95 percent of ASTM D-1557 for this value to apply.

Allowable Static Passive Earth Pressure:	300 pcf
Allowable Seismic Passive Earth Pressure:	400 pcf
Coefficient of Friction:	0.30

14.0 DRAINAGE CONSIDERATIONS

Traffic across the on-site soils when they are damp or wet will result in disturbance of the otherwise firm stratum. Therefore, during site work and construction, the contractor should provide surface drainage and subgrade protection, as necessary.

No groundwater was encountered at the time of drilling within any of the three explorations completed for this study; however, it should be noted that our site explorations were conducted in September when groundwater levels are typically nearing a seasonal low. During wetter periods of the year, zones of perched groundwater may be present within more-permeable strata in existing fill and within sandy zones of the pre-Fraser nonglacial deposits. Therefore, we recommend the contractor be prepared to encounter groundwater seepage during excavation for the basement level. We anticipate that surface and groundwater seepage can be managed during construction with conventional ditches and sumps, and that more complex dewatering systems will not be needed.

All perimeter footings, slabs, and below-grade walls should be provided with a drain at the footing or subgrade elevation. Drains should consist of rigid, perforated, PVC pipe surrounded by washed gravel. The level of the perforations in the pipe should be set at or slightly below the bottom of the footing, and the perforations should be located on the lower portion of the pipe. The drains should be constructed with sufficient gradient to allow gravity discharge away from the structures.

We recommend that curtain drains be installed behind the basement walls in order to prevent hydrostatic pressure from developing behind the walls. A curtain drain is a vertical layer of drainage material placed against the back of a wall to dissipate hydrostatic pressures. The curtain drains should communicate with the perimeter footings drains, have a minimum width of 12 inches, and should be installed along the full height of the wall to a depth of 12 inches below final grade.

To minimize erosion, stormwater discharge or concentrated runoff should not be allowed to flow down any steep slopes. In planning, exterior grades adjacent to walls should be sloped downward away from the structures at an inclination of at least 3 percent to achieve surface drainage. Roof and surface runoff should not discharge into the foundation drain system, but should be handled by a separate, rigid, tightline drain that ties into the site stormwater system.

15.0 TEMPORARY EXCAVATION SHORING SYSTEM

Since the planned basement excavation will extend up to 18 feet below existing site grades, we anticipate that temporary excavation shoring may be required to support portions of the building excavation that are near adjacent structures, streets, and utilities. If an excavation shoring system is needed for the project, we anticipate that cantilever soldier pile walls with timber lagging will be adequate for this project. Tieback anchors may be needed to control horizontal deflections of the shoring wall depending on the proximity of the excavation to existing structures. The shoring system will require a specialty shoring subcontractor(s) to determine the appropriate design details, construction methods, and procedures for installation of the shoring system. We are available to provide recommended soil parameters and earth pressure diagrams to aid in the design of the shoring system as the project design develops.

A monitoring program will likely be required by the City to measure any horizontal or vertical movement of the excavation sidewalls and the installed shoring system. The monitoring should be performed by a licensed surveyor with monitoring points established on settlement-sensitive structures (buildings, manholes, poles, etc.) around the excavation and at regular intervals along the shoring system. Monitoring should be performed at least twice a week and the specifics of the monitoring program should be provided to AESI for review prior to implementation. We recommend the monitoring program be prepared as part of the final shoring wall design.

16.0 PAVEMENT RECOMMENDATIONS

The pavement sections included in this report section are for driveway and parking areas onsite and are not applicable to right-of-way improvements. At this time, we are not aware of any planned right-of-way improvements; however, if any new paving of public streets is required, we should be allowed to offer situation-specific recommendations.

Pavement areas should be prepared in accordance with the "Site Preparation" section of this report. If the stripped native soil or existing fill pavement subgrade can be compacted to 95 percent of ASTM D-1557 and is firm and unyielding, no additional overexcavation is required. Soft or yielding areas should be overexcavated to provide a suitable subgrade and backfilled with structural fill. The upper 2 feet of pavement subgrade should be recompacted to 95 percent of ASTM D-1557. If required, structural fill may then be placed to achieve desired subbase grades.

We anticipate the project will include light-duty pavements for passenger vehicles and heavy-duty pavements for buses, fire trucks, and/or garbage trucks. In light-duty traffic areas, we recommend a pavement section consisting of 3 inches of hot-mix asphalt (HMA) underlain by 4 inches of crushed surfacing base course (CSBC) as the recommended minimum in areas of planned passenger car lanes and parking. In heavy-duty traffic areas, a minimum pavement section consisting of 4 inches of HMA underlain by 6 inches of CSBC is recommended. The CSBC must be compacted to 95 percent of the maximum density, as determined by ASTM D-1557. All paving materials should meet gradation criteria contained in the current Washington State Department of Transportation (WSDOT) Standard Specifications.

Depending on construction staging and desired performance, the crushed base course material may be substituted with asphalt treated base (ATB) beneath the final asphalt surfacing. The substitution of ATB should be as follows: 4 inches of crushed rock can be substituted with 3 inches of ATB, and 6 inches of crushed rock may be substituted with 4 inches of ATB. ATB should be placed over a firm and unyielding subgrade as determined by proof-rolling and a 1½- to 2-inch thickness of crushed rock to act as a working surface. If ATB is used for construction access and staging areas, some rutting and disturbance of the ATB surface should be expected. The general contractor should remove affected areas and replace them with properly compacted ATB prior to final surfacing.

17.0 INFILTRATION FEASIBILITY

Infiltration opportunities appear limited to not feasible with the current building layout and proposed basement level. The project site is generally underlain by existing fill soils and native pre-Fraser sediments that generally consisted of stiff to hard silt and sandy silt. A layer of slightly moist to moist sand with trace silt was encountered within EB-1 from a depth of about 12 to

22 feet that may provide opportunity for infiltration; however, a significant portion of this layer will be removed during construction of the basement level. The fill soils are not considered suitable receptor soils for infiltration due to the relatively high silt content observed and variable composition. The pre-Fraser sediments are also not considered suitable receptor soils for infiltration due to the relatively high silt content and high relative density. Based on our experience with similar soil types in the Puget Sound region, the field infiltration rate of the pre-Fraser sediments is anticipated to be on the order of 1 to 2 inches per month. Therefore, it is our opinion that shallow infiltration is not feasible within the limits of the project site.

18.0 RECOMMENDATIONS FOR ADDITIONAL EXPLORATION

Due to equipment access constraints and utility conflicts, no explorations were completed along the northern margin of the site for this current study. We recommend completing up to three additional explorations with a miniature or portable drill rig that can access the forested area within the central and northwest portions of the site to explore the depth to native sediments. The exploration data in this area would also aid us in evaluating the steep slope that descends to the north of the project site.

19.0 PROJECT DESIGN AND CONSTRUCTION MONITORING

We recommend that AESI perform a geotechnical review of the plans prior to final design completion. In this way, we can confirm that our recommendations have been correctly interpreted and implemented in the design. The City may require a plan review by the geotechnical engineer as a condition of permitting.

The City may also require geotechnical special inspections during construction and preparation of a final summary letter when construction is complete. We are available to provide geotechnical engineering services during construction. The integrity of the earthwork and foundations depends on proper site preparation and construction procedures. In addition, engineering decisions may have to be made in the field in the event that variations in subsurface conditions become apparent.

We have enjoyed working with you on this study and are confident these recommendations will aid in the successful completion of your project. If you should have any questions or require further assistance, please do not hesitate to call.

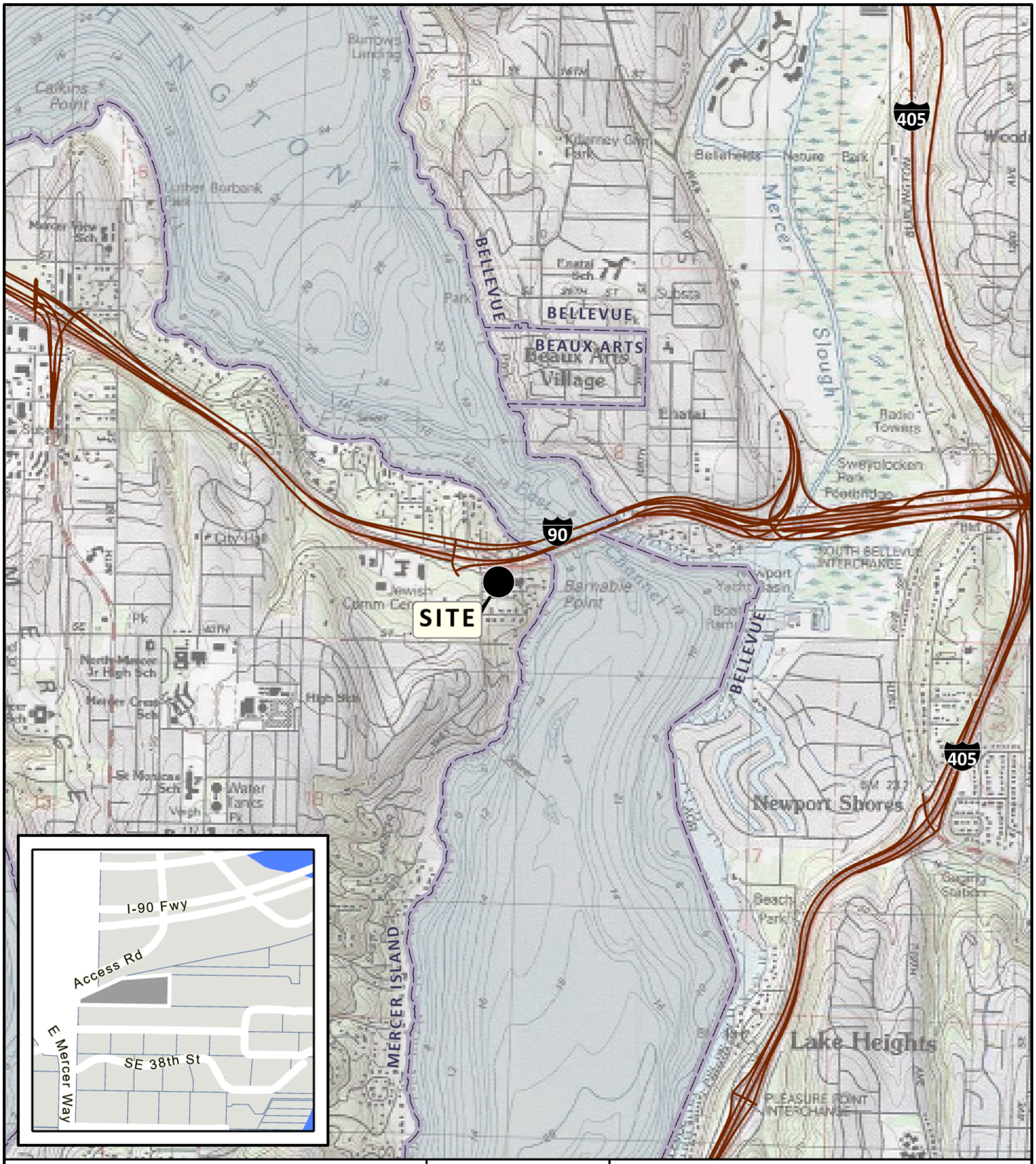
Sincerely,
ASSOCIATED EARTH SCIENCES, INC.
Kirkland, Washington

G. BRADFORD DREW
57948
10/27/2023

Kurt D. Merriman, P.E.
Senior Principal Engineer

G. Bradford Drew, P.E.
Senior Engineer

Attachments: Figure 1: Vicinity Map
Figure 2: Existing Site and Exploration Plan
Figure 3: LIDAR-Based Shaded Relief Map
Appendix A: Exploration Logs
Appendix B: Concept Plans



DATA SOURCES / REFERENCES:
USGS: 7.5' SERIES TOPOGRAPHIC MAPS, ESRI/I-CUBED/NGS 2013
KING CO: STREETS 5/23, PARCELS 4/23, CITY LIMITS, PARKS 5/23
LOCATIONS AND DISTANCES SHOWN ARE APPROXIMATE

NOTE: BLACK AND WHITE REPRODUCTION OF THIS COLOR ORIGINAL MAY REDUCE ITS EFFECTIVENESS AND LEAD TO INCORRECT INTERPRETATION

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VICINITY MAP

HERZL-NER TAMID CONSERVATIVE CONGREGATION K-12
MERCER ISLAND, WASHINGTON

PROJ NO. 20210371E001	DATE: 9/23	FIGURE: 1

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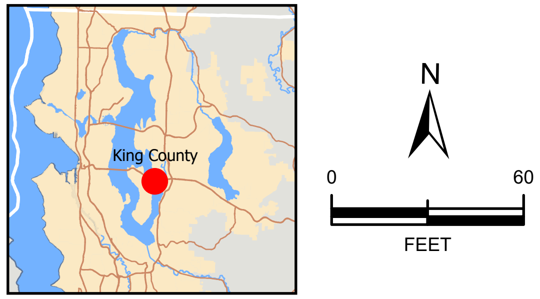
LEGEND

- 🟨 SITE
- ● EXPLORATION BORING
- ∿ CONTOUR 10 FT
- ∿ CONTOUR 2 FT
- ▭ PARCEL

DATA SOURCES / REFERENCES:
WADNR WGS: WA LIDAR PORTAL, KING CO. 2021, USGS 3DEP
GRID CELL SIZE 1.5', FLOWN 4/2021
CONTOURS DERIVED FROM LIDAR

KING CO: STREETS, PARCELS, 4/23
AERIAL PICTOMETRY INT. 2021

LOCATIONS AND DISTANCES SHOWN ARE APPROXIMATE



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EXISTING SITE AND EXPLORATION PLAN

HERZL-NER TAMID CONSERVATIVE CONGREGATION K-12
MERCER ISLAND, WASHINGTON

PROJ NO.	DATE:	FIGURE:
20210371E001	9/23	2

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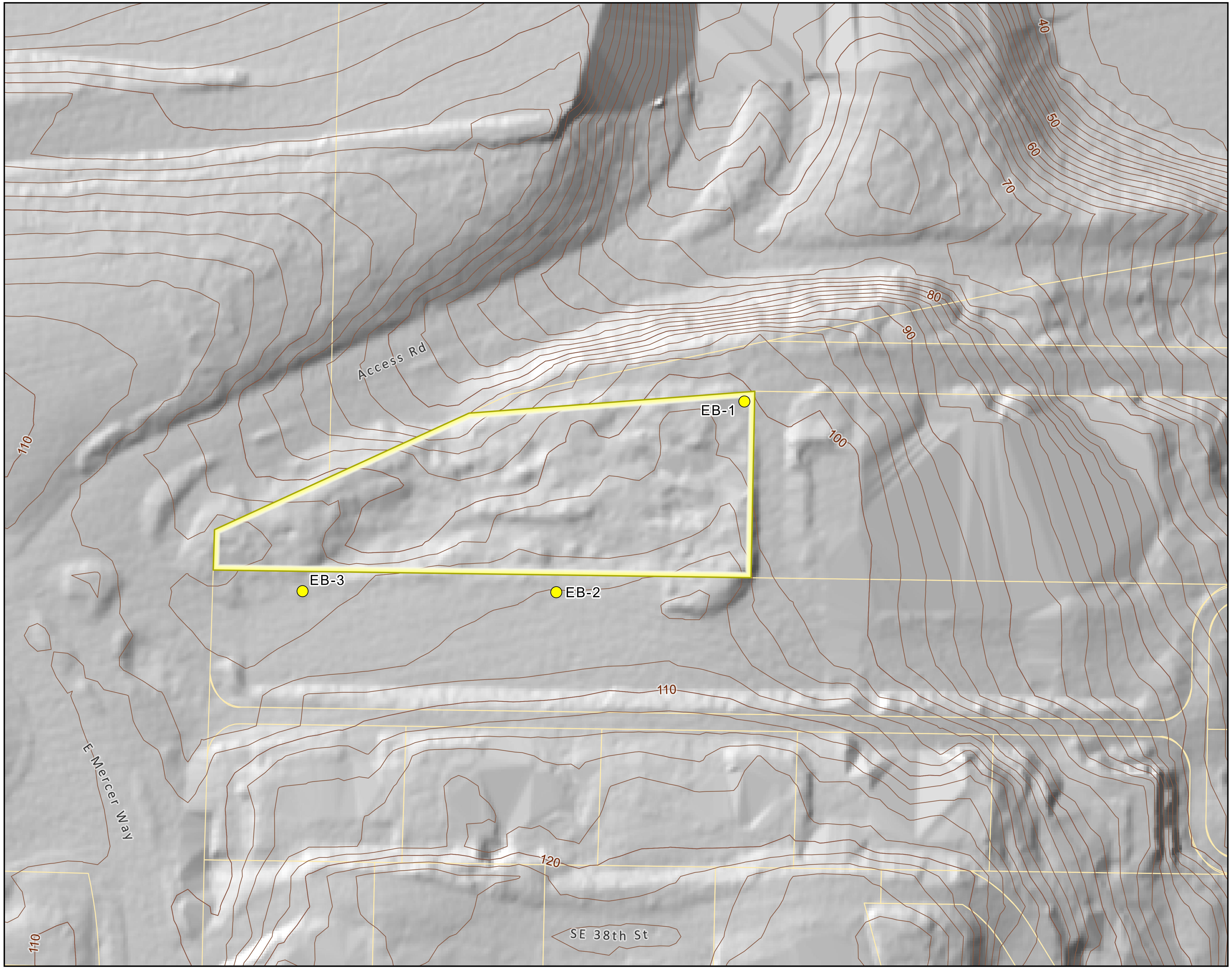


LEGEND

🟨 SITE

● EXPLORATION BORING

CONTOUR 10 FT

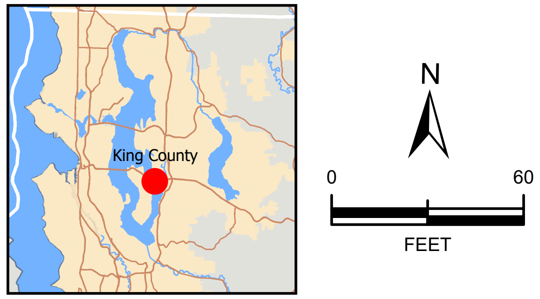
CONTOUR 2 FT

PARCEL

DATA SOURCES / REFERENCES:
WADNR WGS: WA LIDAR PORTAL, KING CO. 2021, USGS 3DEP
GRID CELL SIZE 1.5', FLOWN 4/2021
CONTOURS DERIVED FROM LIDAR

KING CO: STREETS, PARCELS, 4/23

LOCATIONS AND DISTANCES SHOWN ARE APPROXIMATE



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LIDAR BASED SHADED RELIEF

HERZL-NER TAMID CONSERVATIVE CONGREGATION K-12
MERCER ISLAND, WASHINGTON

PROJ NO. 20210371E001	DATE: 9/23	FIGURE: 3

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APPENDIX A

Exploration Logs

Coarse-Grained Soils - More than 50%(1) Retained on No. 200 Sieve	Gravels - More than 50%(1) of Coarse Fraction Retained on No. 4 Sieve	≤5% Fines(2)	**GW**	Well-graded gravel and gravel with sand, little to no fines
			GP	Poorly-graded gravel and gravel with sand, little to no fines
		≥12% Fines(2)	**GM**	Silty gravel and silty gravel with sand
			GC	Clayey gravel and clayey gravel with sand
	Sands - 50%(1) or More of Coarse Fraction Passes No. 4 Sieve	≤5% Fines(2)	**SW**	Well-graded sand and sand with gravel, little to no fines
			SP	Poorly-graded sand and sand with gravel, little to no fines
		≥12% Fines(2)	**SM**	Silty sand and silty sand with gravel
			SC	Clayey sand and clayey sand with gravel
Fine-Grained Soils - 50%(1) or More Passes No. 200 Sieve	Silts and Clays Liquid Limit Less than 50		**ML**	Silt, sandy silt, gravelly silt, silt with sand or gravel
			CL	Clay of low to medium plasticity; silty, sandy, or gravelly clay, lean clay
			OL	Organic clay or silt of low plasticity
	Silts and Clays Liquid Limit 50 or More		**MH**	Elastic silt, clayey silt, silt with micaceous or diatomaceous fine sand or silt
			CH	Clay of high plasticity, sandy or gravelly clay, fat clay with sand or gravel
			OH	Organic clay or silt of medium to high plasticity
Highly Organic Soils			**PT**	Peat, muck and other highly organic soils

Terms Describing Relative Density and Consistency

	Density	SPT(3) blows/foot
Coarse-Grained Soils	Very Loose	0 to 4
	Loose	4 to 10
	Medium Dense	10 to 30
	Dense	30 to 50
	Very Dense	>50
	Consistency	**SPT(3) blows/foot**
Fine-Grained Soils	Very Soft	0 to 2
	Soft	2 to 4
	Medium Stiff	4 to 8
	Stiff	8 to 15
	Very Stiff	15 to 30
	Hard	>30

Test Symbols

G = Grain Size
M = Moisture Content
A = Atterberg Limits
C = Chemical
DD = Dry Density
K = Permeability

Component Definitions

Descriptive Term	Size Range and Sieve Number
Boulders	Larger than 12"
Cobbles	3" to 12"
Gravel	3" to No. 4 (4.75 mm)
Coarse Gravel	3" to 3/4"
Fine Gravel	3/4" to No. 4 (4.75 mm)
Sand	No. 4 (4.75 mm) to No. 200 (0.075 mm)
Coarse Sand	No. 4 (4.75 mm) to No. 10 (2.00 mm)
Medium Sand	No. 10 (2.00 mm) to No. 40 (0.425 mm)
Fine Sand	No. 40 (0.425 mm) to No. 200 (0.075 mm)
Silt and Clay	Smaller than No. 200 (0.075 mm)

(4) Estimated Percentage

Component	Percentage by Weight
Trace	<5
Some	5 to <12
Modifier (silty, sandy, gravelly)	12 to <30
Very *modifier* (silty, sandy, gravelly)	30 to <50

Moisture Content

Dry - Absence of moisture, dusty, dry to the touch
Slightly Moist - Perceptible moisture
Moist - Damp but no visible water
Very Moist - Water visible but not free draining
Wet - Visible free water, usually from below water table

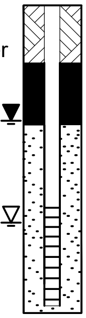
Symbols

Sampler Type and Description

10 15 20 Blows/6" or portion of 6"
Split-Spoon Sampler (SPT)
California Sampler
Ring Sampler
Continuous Sampling
Grab Sample
Portion not recovered

Groundwater depth

▼ ATD At time of drilling
▽ Static water level (date)


Cement grout surface seal
Bentonite seal
Filter pack with blank casing section
Screened casing or Hydrotip with filter pack
End cap

Classifications of soils in this report are based on visual field and/or laboratory observations, which include density/consistency, moisture condition, grain size, and plasticity estimates and should not be construed to imply field or laboratory testing unless presented herein. Visual-manual and/or laboratory classification methods of ASTM D-2487 and D-2488 were used as an identification guide for the Unified Soil Classification System.

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(1) Percentage by dry weight
(2) Combined USCS symbols used for fines between 5% and 12%
(3) (SPT) Standard Penetration Test (ASTM D-1586)
(4) In General Accordance with Standard Practice for Description and Identification of Soils (ASTM D-2488)

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EXPLORATION LOG KEY

FIGURE: A1

Exploration Boring **EB-1**

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Herzl-Ner Tamid Conservative Congregation K-12		Sheet: 1 of 2
Mercer Island, WA	Start Date: 9/7/23	Logged By: JTL
20210371E001	Ending Date: 9/7/23	Approved By: CMM

Driller/Equipment: Geologic Drill/Recon Track Rig
Hammer Weight/Drop: 140#/30"
Hole Diameter (in): 2
▼Groundwater Depth ATD (ft): Not encountered

Total Depth (ft): 41.5
Ground Surface Elevation (ft): ≈101
Datum: NAVD88
▽ Groundwater Depth Post Drilling (ft) (Date): ()

Depth (ft)	Sample Type	Sample	% Recovery	Graphic Symbol	Description	Water Level	Blows/6"	Blows/Foot	Other Tests
0					**Fill**				
					Dry, sandy, GRAVEL to gravelly, SAND; topsoil organics and rootlets (GP/SP).				
		1			Dry, tan, SILT, some fine gravel, trace fine sand; rootlets (ML).		3 3 3	6	
5					**Pre-Fraser Nonglacial**				
		2			Dry, tan with iron oxide staining, fine sandy, SILT, trace fine gravel (ML).		7 9 7	16	
		3			Dry, light tan, fine sandy, SILT, some fine to medium gravel (ML). Becomes moist, less gravel.		3 18 20	38	
10		4			Dry to slightly moist, brown to tan, fine SAND with silt interbeds, trace fine gravel (SP-SM). Gravel increases.		12 13 18	31	
15		5	6		Slightly moist, gray and brown with iron oxide staining, medium SAND, trace fine sand, trace gravel (SP).		50/6"	50/6"	
20		6			Moist, light brown, medium SAND, trace gravel; white silt at tip of spoon (SP).		13 20 31	51	
25		7			Moist to wet, dark brown, fine sandy, SILT; micas present (ML). Becomes sandier at 26.5 feet.		19 40 43	83	
30		8			Slightly moist to moist, tan and brown, fine silty, SAND (SM).		13 20 25	45	
35		9			Moist to wet, tan and brown with iron oxide staining and mottling, fine sandy, SILT, trace gravel; organics present; slightly disturbed texture (ML).		1 18 20	38	

associated earth sciences incorporated

Exploration Boring		**EB-1**
Herzl-Ner Tamid Conservative Congregation K-12		Sheet: 2 of 2
Mercer Island, WA	Start Date: 9/7/23	Logged By: JTL
20210371E001	Ending Date: 9/7/23	Approved By: CMM

Driller/Equipment: Geologic Drill/Recon Track Rig
Hammer Weight/Drop: 140#/30"
Hole Diameter (in): 2
▼ Groundwater Depth ATD (ft): Not encountered

Total Depth (ft): 41.5
Ground Surface Elevation (ft): ≈101
Datum: NAVD88
▽ Groundwater Depth Post Drilling (ft) (Date): ()

Depth (ft)	Sample Type	Sample	% Recovery	Graphic Symbol	Description	Water Level	Blows/6"	Blows/Foot 10 20 30 40 50+	Other Tests
40		10			Moist to wet, tan and brown with iron oxide staining and mottling, fine sandy, SILT, trace gravel; organics present; slightly disturbed texture (ML). As above.		8 16 20	36	
					Pre-Fraser Lacustrine Slightly moist to moist, bluish gray, SILT, trace sand (ML).				
					No groundwater encountered.				
45									
50									
55									
60									
65									
70									
75									

associated earth sciences incorporated

Exploration Boring EB-2

Herzl-Ner Tamid Conservative Congregation K-12 Sheet: 1 of 1

Mercer Island, WA	Start Date: 9/7/23	Logged By: JTL
20210371E001	Ending Date: 9/7/23	Approved By: CMM

Driller/Equipment: Geologic Drill/Recon Track Rig
Hammer Weight/Drop: 140#/30"
Hole Diameter (in): 2
▼Groundwater Depth ATD (ft): Not encountered

Total Depth (ft): 21.5
Ground Surface Elevation (ft): ≈106
Datum: NAVD88
▽ Groundwater Depth Post Drilling (ft) (Date): ()

Depth (ft)	Sample Type	Sample	% Recovery	Graphic Symbol	Description	Water Level	Blows/6"	Blows/Foot 10 20 30 40 50+	Other Tests
0					**Asphalt - 4 inches**				
					Fill				
		1			Slightly moist, mottled gray and brown with some iron oxide staining, SILT, some fine to medium sand, some gravel (ML).		8 17 8	25	
5		2			**Pre-Fraser Nonglacial** Slightly moist to moist, mottled brown with iron oxide staining, silty, fine SAND, some gravel; local silty interbeds; fine micas (SM).		2 5 8	13	
		3			Slightly moist to wet, brown, fine sandy, SILT, some fine sand; trace silt interbeds (ML).		5 8 9	17	
10		4			Slightly moist, light brown, silty, fine SAND to fine sandy, SILT, trace gravel (SM/ML).		6 9 13	22	
15		5			Moist, light brown with slight iron oxide staining, fine sandy, SILT; very fine micas (ML).		11 18 21	39	
					Becomes finer, darker.				
20		6			Moist, brown, SILT, trace fine sand; very fine micas (ML).				
					No groundwater encountered.				
25									
30									
35									

associated earth sciences incorporated	**Exploration Boring**		**EB-3**
	Herzl-Ner Tamid Conservative Congregation K-12		Sheet: 1 of 1
	Mercer Island, WA	Start Date: 9/7/23	Logged By: JTL
	20210371E001	Ending Date: 9/7/23	Approved By: CMM

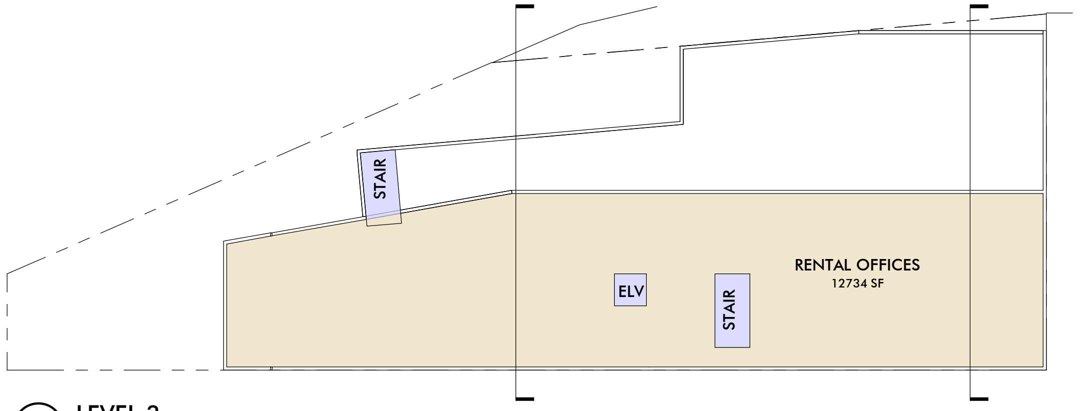
Driller/Equipment: Geologic Drill/Recon Track Rig
Hammer Weight/Drop: 140#/30"
Hole Diameter (in): 2
▼Groundwater Depth ATD (ft): Not encountered

Total Depth (ft): 26.5
Ground Surface Elevation (ft): ≈103
Datum: NAVD88
▽ Groundwater Depth Post Drilling (ft) (Date): ()

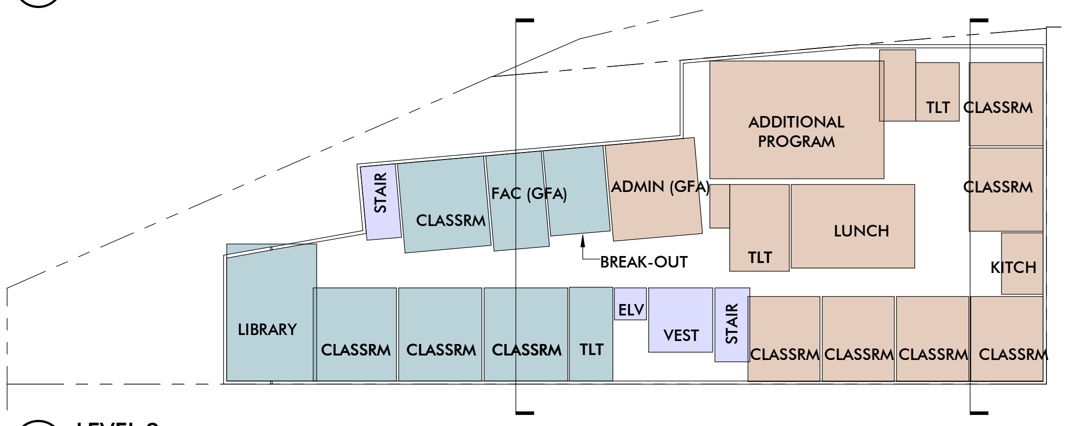
Depth (ft)	Sample Type	Sample	% Recovery	Graphic Symbol	Description	Water Level	Blows/6"	Blows/Foot 10 20 30 40 50+	Other Tests
0					**Asphalt - 6 inches**				
					Fill				
		1			Slightly moist, gray, fine sandy, SILT, trace gravel (ML).		3 5 4	9	
5		2			Slightly moist to moist, gray and bluish gray, fine sandy, SILT, trace gravel; trace organics (ML).		3 4 2	6	
		3			Slightly moist to moist, gray, sandy, SILT; black organics (ML).		2 1 1	2	
10		4			**Pre-Fraser Lacustrine** Slightly moist to moist, blue and gray mottling with iron oxide stains, SILT (ML).		4 7 7	14	
					Grades to tan.				
15		5			Moist, brown with iron oxide stains, SILT; fine micas (ML).		1 5 9	14	
20		6			Moist, light brown with heavy iron oxide staining, SILT, trace gravel (ML).		5 8 12	20	
25		7			Moist, bluish gray, SILT (ML).		4 9 17	26	
					No groundwater encountered.				
30									
35									

APPENDIX B

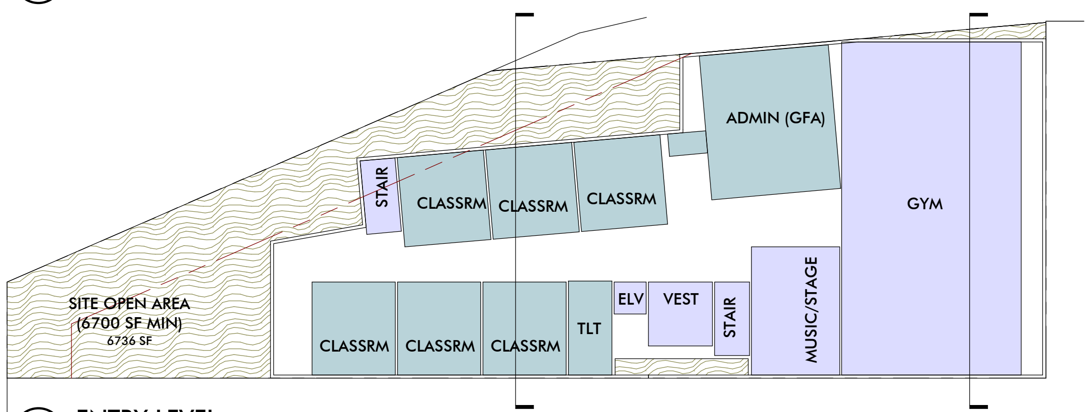
Concept Plans



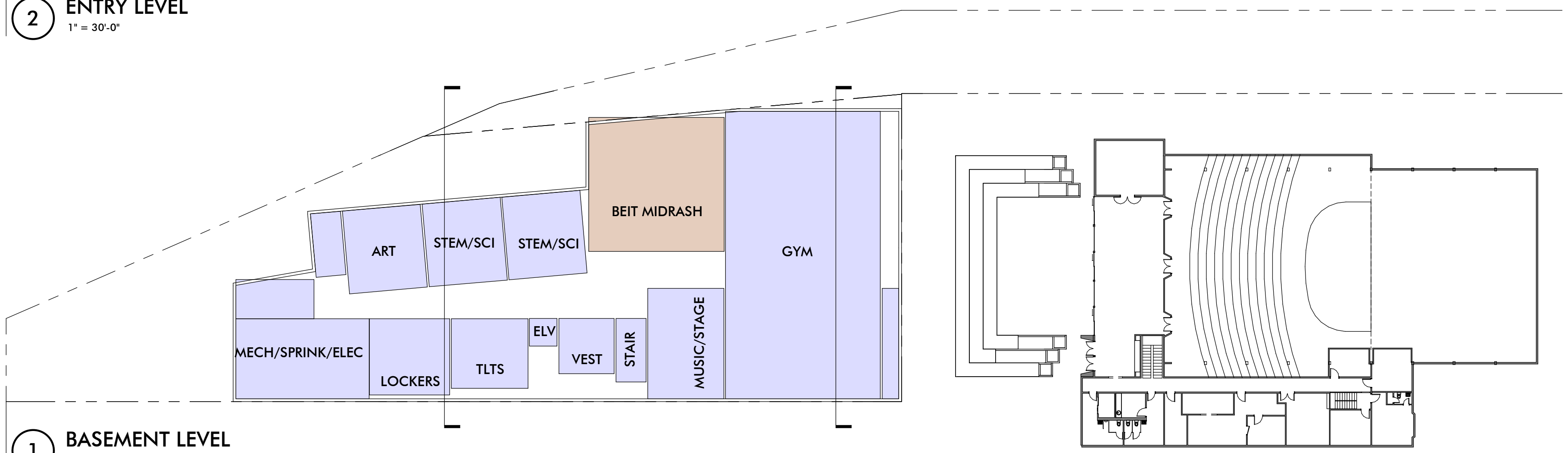
9 LEVEL 3
1" = 30'-0"



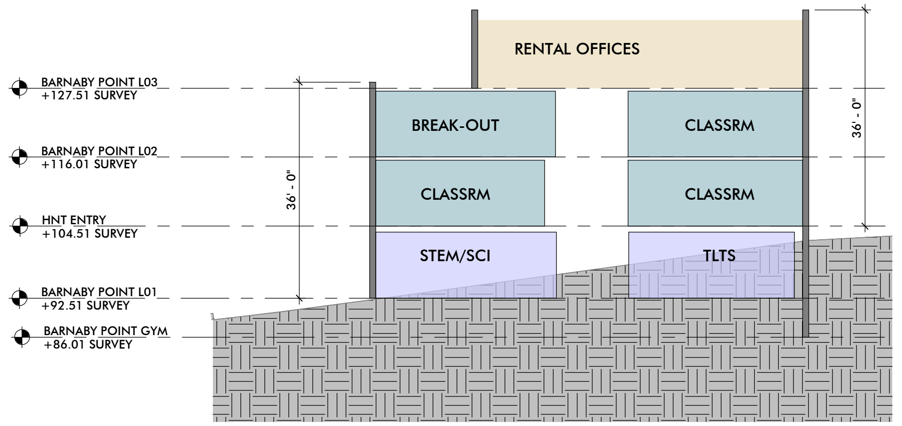
3 LEVEL 2
1" = 30'-0"



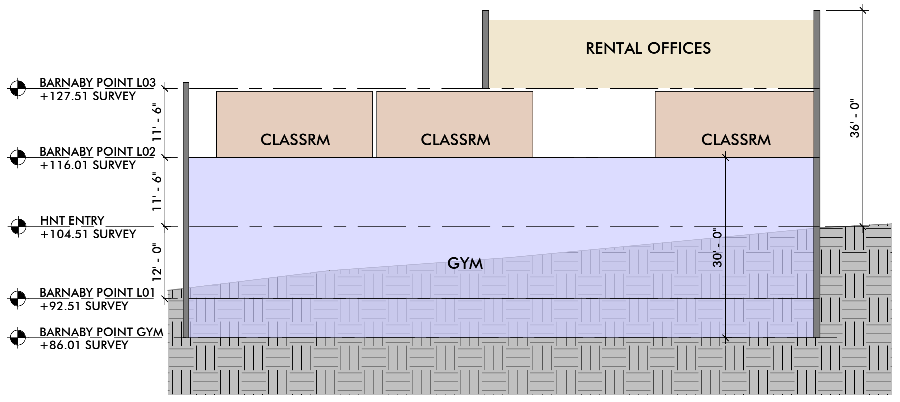
2 ENTRY LEVEL
1" = 30'-0"



1 BASEMENT LEVEL
1" = 30'-0"



5 SECTION DIAGRAM 1 LOOKING WEST
1/16" = 1'-0"



6 SECTION DIAGRAM 2 LOOKING WEST
1/16" = 1'-0"

PROGRAM AREAS

PARTNER	SPACE TYPE	QTY	TOTAL AREA
JDS	ADMIN (GFA)	1	1,822 SF
JDS	BREAK-OUT	1	500 SF
JDS	CLASSRM	10	7,560 SF
JDS	FAC (GFA)	1	518 SF
JDS			10,400 SF
NYHS	ADDITIONAL PROGRAM	1	2,000 SF
NYHS	ADMIN (GFA)	1	837 SF
NYHS	CLASSRM	6	3,600 SF
NYHS	KITCH	1	250 SF
NYHS	LUNCH	1	1,000 SF
NYHS	STAFF TLT	1	84 SF
NYHS	TLT	3	1,000 SF
NYHS			8,771 SF
SHARED	ART	1	850 SF
SHARED	ELV	1	100 SF
SHARED	GYM	1	5,824 SF
SHARED	LOCKERS	1	800 SF
SHARED	MECH/SPRINK/ELEC	2	1,800 SF
SHARED	MUSIC/STAGE	1	1,100 SF
SHARED	STAIR	2	500 SF
SHARED	STEM/SCI	2	1,700 SF
SHARED	TELESCOPIC SEATING	1	240 SF
SHARED	TLTS	1	700 SF
SHARED	VEST	2	800 SF
SHARED			14,414 SF
			33,585 SF

GFA Gym Scheme 1

Comments	Area
ENTRY LEVEL	13,953 SF
L01/GYM	21,557 SF
L02	18,534 SF
L03	13,820 SF
	67,865 SF

GFA Gym Scheme 2

Comments	Area
L01/GYM	20,552 SF
L02	20,811 SF
HNT ENTRY	12,299 SF
L03	13,341 SF
	67,003 SF

CONCEPT PLANS

HERZL FEASIBILITY / 6 SEPTEMBER 2023

ORIGINAL SHEET SIZE 22 x 34; REDUCE DRAWING SCALE BY 50% IF VIEWING AS 11 x 17