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January 9, 2023

Ryan Kilby
VIA Email: ryan@williamsinvest.com

Geotechnical Engineering Evaluation
Rose Hill 12-Unit Mixed-Use Building
730 – 3rd Street
Mukilteo, Washington
NGA File No. 8797B22

Dear Ryan:

We are pleased to submit the attached report titled ***"Geotechnical Engineering Evaluation – Rose Hill 12-Unit Mixed-Use Building – 730 - 3rd Street – Mukilteo, Washington."*** This report summarizes our observations of the existing surface and subsurface conditions within the site and provides general recommendations for the proposed site development. Our services were completed in general accordance with our proposal, which you signed on December 5, 2022.

We previously issued a geotechnical report for the property on August 9, 2013. This evaluation involved two drilled borings and two supplemental hand-augered explorations, where we concluded the site was suitable for the development of a multi-unit mixed use building, after we encountered suitable soils at relatively shallow depths. We revisited the site on November 22, 2022 and December 7, 2022 to re-evaluate site conditions. Our explorations indicated that the site was generally underlain by native glacial soils at depths of 1.0 to 4.0-feet below the existing ground surface.

We have concluded that the site was generally compatible with the planned development. The building could be supported on shallow spread footings placed on the competent glacial soils. These soils should generally be encountered below the existing ground surface, based on our explorations. We should note, however, that deeper areas of loose, undocumented fill could exist in unexplored portions of the site especially in the area of the existing building which could require the removal of such soils and replacement with structural fill. We understand the proposed mixed-use building will be a 22-unit structure with a partial subsurface parking level. Shoring will likely be required to retain the cut for the retaining wall along 3rd Street.

In the attached report, we have included recommendations for foundation support, a shoring wall, erosion control, and surface drainage.

We appreciate the opportunity to provide service to you on this project. Please contact us if you have any questions regarding this report or require further information.

Sincerely,

NELSON GEOTECHNICAL ASSOCIATES, INC.

Khaled M. Shawish, PE
Principal

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Rose Hill 12-Unit Mixed-Use Building
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INTRODUCTION

This report presents the results of our geotechnical engineering investigation and evaluation of the Rose Hill Mixed-use Building project located at **730 – 3rd Street in Mukilteo, Washington,** as shown on the Vicinity Map in **Figure 1.** The purpose of this study is to explore and characterize the site's surface and subsurface conditions, and to provide geotechnical recommendations for the planned site development. For our use in preparing this report, we were provided with the following documents:

For our use in preparing this updated geotechnical evaluation, we were provided with a planset titled "Williams Investments – Third and Park," dated July 17, 2022 and drawn by Dykeman Architecture.

The property is rectangular in shape and covers approximately 0.26 acres in area. It is currently vacant. The property is bordered by existing commercial properties to the west, by a side street to the north, by Park Avenue to the east, and by 3rd Street to the south. Topographically, the site slopes gently to the northwest, with isolated steep slopes mapped to the northeast. We previously issued a geotechnical report for the property on August 9, 2013. During this previous evaluation we performed two drilled boreholes at the site, as well as two supplemental hand-augered explorations, and concluded the site was suitable for the development of a multi-unit mixed-use building with subsurface parking levels, after encountering native soils at relatively shallow depths.

We understand that no significant change has occurred on the lot since issuing our previous geotechnical report, except for the removal of a small structure that occupied the property. We have been requested to provide this updated geotechnical report to address the construction of a building that is approximately the same as the building that was previously proposed, which will be a 12-unit mixed-use building with subsurface parking. We have been requested to provide this report to verify subsurface conditions and provide an update to our original report. The existing site layout and the locations of our explorations are shown on the Site Plan in **Figure 2.**

As a part of this project, we also understand that onsite infiltration systems are being considered. We were requested to evaluate the infiltration capacity of the site soils within the property. The City of Mukilteo utilizes the 2019 Department of Ecology (DOE) Stormwater Management in Western Washington Manual to determine the design of infiltration or detention facilities. We attempted to perform one PIT within the site.

SCOPE

The purpose of this study is to explore and characterize the site surface and subsurface conditions and provide an updated report for the site.

Specifically, our scope of services includes the following:

1. Reviewing available soil and geologic maps of the area, as well as our previous report.
2. Reconnoitering existing conditions on the site and verify the subsurface soil and groundwater conditions within the proposed building area with hand-tool explorations, where possible.
3. Performing an onsite small PIT test and calculate long term infiltration rates per the 2019 SMMWW. Excavator and water truck provided by the client.
4. Evaluating the minor steep slopes mapped in the northeast corner of the site.
5. Performing laboratory grain-size sieve analysis on soil samples, as necessary.
6. Providing recommendations for earthwork and foundation support.
7. Providing recommendations for shoring.
8. Providing recommendations for retaining walls.
9. Providing recommendations for temporary and permanent slopes.
10. Providing recommendations for subsurface utilities and pavement subgrade preparation.
11. Providing general recommendations for site drainage and erosion control.
12. Documenting the results of our findings, conclusions, and recommendations in a written updated geotechnical report for the proposed building.

SITE CONDITIONS

Surface Conditions

The site is composed of two rectangular lots and is bound by Park Avenue to the east, 3rd Street to the south, an alley to the north, and a 2-story commercial building to the west. In general, the overall site topography slopes down to the north from 3rd Street towards the alley in the back of the property. The western half of the property is mostly covered with crushed rock. The eastern half is mostly covered with weeds and historically, and older structure is located within the northeastern portion of the property but has since been removed. We did not observe surface water within the site during our site visit on July 23, 2013, or on our subsequent revisit to the site on November 22, 2022 and December 7, 2022.

Subsurface Conditions

Geology: The geologic units for this area are shown on the Distribution and Description of Geologic Units in the Mukilteo Quadrangle, Washington, by James P. Minard (USGS, 1982). The site is mapped as Qtb (Transitional Beds) and Qw (Whidbey Formation). The Transitional Beds deposits are described as thick beds of gray clay, silt, and fine- to very fine sand, however generally contain fine sands and gravels in the lower portions of the deposit. The Whidbey Formation is described as medium- to coarse-grained mostly cross bedded sand. Our explorations generally encountered fine to medium sand with silt underlain by silt and sand with silt layers generally consistent with the description of the lower portion of the transition beds deposit.

Explorations: The subsurface conditions within the site were initially explored on July 23, 2013 by drilling two borings to depths ranging from approximately 21.5 to 24.0 feet below the existing ground surface, using a limited-access drill rig. We also conducted two hand augers within the north-central portion of the property. We revisited the site on November 22, 2022 and December 7, 2022 to evaluate most recent subsurface conditions. The approximate locations of our explorations are shown on the Site Plan in **Figure 2.** A geologist from NGA was present during the explorations, examined the soils and geologic conditions encountered, obtained samples of the different soil types, and maintained logs of the borings and hand augers. For the borings, a Standard Penetration Test (SPT) was performed on each of the samples during drilling to document soil density at depth. The SPT consists of driving a 2-inch outer-diameter, split-spoon sampler 18 inches using a 140-pound hammer with a drop of 30 inches. The number of blows required to drive the sampler the final 12 inches is referred to as the "**N**" value and is presented on the boring logs. The **N** value is used to evaluate the strength and density of the deposit.

The soils were visually classified in general accordance with the Unified Soil Classification System, presented in **Figure 3.** The logs of our explorations are attached to this report and are presented as **Figures 4 and 5.** The logs of our borings are presented in **Figures 6 and 7.** We present a brief summary of the subsurface conditions in the following paragraphs. For a detailed description of the subsurface conditions, the boring and hand auger logs should be reviewed.

July 23, 2013 Explorations: **Boring 1** was located within the southwestern portion of the site. Below approximately 1.5 feet of modified ground, we encountered 12 feet of stiff, silty fine to medium sand grading to stiff silt with sand. Below the silt, we encountered gray, fine to medium sand with silt. We interpreted this material to be native glacial material. **Boring 1** was terminated in the sand with silt at a depth of 24.0 feet.

Boring 2 was located in the southeastern portion of the site. Below the surficial weeds and two feet of silty sand interpreted as fill/modified ground, we encountered about eight feet of medium dense, light orange-brown, fine to coarse sand with silty and gravel to silty sand with gravel. Below this material, we encountered layers of stiff to very stiff silt and silty sand. We interpreted this material to be native glacial material. **Boring 2** was terminated in the sand with silt at a depth of 21.5 feet.

Hand Auger 1 was excavated in the northwestern portion of the site. Below a surficial layer of crushed rock, we encountered approximately 1.7 feet of brown to brown-gray, silty fine to medium sand with varying amounts of gravel. We interpreted this material as fill. Below the fill, we encountered medium dense, silty fine to medium sand which we interpreted as native glacial soil. **Hand Auger 1** was terminated in the silty sand layer at a depth of 1.8 feet.

Hand Auger 2 was excavated in the northeastern portion of the site. Below a surficial layer of weeds and grasses, we encountered approximately 1.3 feet of light brown to orange-brown, silty fine to medium sand with varying amounts of gravel. We interpreted this material as fill. Below the fill, we encountered medium dense, silty fine to medium sand which we interpreted as native glacial soil. **Hand Auger 2** was terminated in the silty sand layer at a depth of 1.8 feet.

November 22, 2022 and December 7, 2022 Explorations: Test Pits 3, 4 and 5, as well as Infiltration **Test Pit One,** were excavated in the northern portion of the site. Within these test pits, 1.0- to 4.5-feet of surficial topsoil and/or undocumented fill was encountered bearing organics and roots and was encountered in a loose to medium dense condition. Underlying this layer, in **Test Pits Three and Four,** we encountered a more granular gray brown fine to medium sand with iron oxide staining was found in a medium dense condition. In most explorations, we broke through this more granular material to encounter a layer of gray to orange-gray to gray silty fine to medium sand with trace gravel in a medium dense or better condition at depth in every exploration. The encountered material generally showed an interbedding of siltier and more granular layers, which matches the description of transitional beds at depth.

Test Pits One and Two were excavated in the southern portion of the site. Here, we encountered 2.0-feet of surficial topsoil and/or undocumented fill with organics. At depth, we encountered a similar interbedding of gray brown to gray fine sand and silty fine to medium sand with trace gravel, matching the description of transition beds at depth.

Our most recent test pits were excavated to depths between 4.5- and 10.0-feet of depth throughout the site.

Hydrogeologic Conditions

Groundwater seepage was encountered in the borings and was measured with a groundwater reader after drilling. We measured the groundwater in **Boring 1** at 12.3 feet and in **Boring 2** at 12.6 feet below the existing ground surface during the dry season and our July 23, 2013 explorations. We returned to the site, and while we did not find groundwater seepage on November 22, 2022 at up to 6.5-feet of depth, we returned to the site on December 7, 2022 to evaluate infiltration and discovered significant perched groundwater at 6.0-feet of depth. Based on these findings, it is likely that if construction takes place during the wet season, groundwater could be encountered within the excavation. We interpreted the water seepage to be perched water. Perched water occurs when surface water infiltrates through less dense, more permeable soils, and accumulates on top of a relatively low permeability material such as the dense silty sand. Perched water does not represent a regional groundwater "table" within the upper soil horizons. Perched water tends to vary spatially and is dependent upon the amount of rainfall. We would expect the amount of perched groundwater to decrease during drier times of the year and increase during wetter periods.

SENSITIVE AREA EVALUATION

Seismic Hazard

We reviewed the 2018 International Building Code (IBC) for seismic site classification for this project. Since dense soils are interpreted to underlie the site at depth, the site best fits the IBC description for Site Class D. **Table 1** below provides seismic design parameters for the site that are in conformance with the 2018 IBC, which specifies a design earthquake having a two percent probability of occurrence in 50 years (return interval of 2,475 years), and the 2008 USGS seismic hazard maps.

Table 1 – 2018 IBC Seismic Design Parameters

Site Class	Spectral Acceleration at 0.2 sec. (g) S_s	Spectral Acceleration at 1.0 sec. (g) S_1	Site Coefficients		Design Spectral Response Parameters	
			F_a	F_v	S_{DS}	S_{D1}
D	1.405	0.500	1.0	null	0.936	null

The spectral response accelerations were obtained from the OSHPD Seismic Design Maps website (ASCE 7-16 data) for the project latitude and longitude. Hazards associated with seismic activity include liquefaction potential and amplification of ground motion. Liquefaction is caused by a rise in pore pressures in a loose, fine sand deposit beneath the groundwater table. It is our opinion that the dense glacial deposits interpreted to underlie the site have a low potential for liquefaction or amplification of ground motion.

Erosion Hazard

The criteria used for determination of the erosion hazard for affected areas include soil type, slope gradient, vegetation cover, and groundwater conditions. The erosion sensitivity is related to vegetative cover and the specific surface soil types, which are related to the underlying geologic soil units. The Soil Survey of Snohomish County Area, Washington, by the Soil Conservation Service (SCS), was reviewed to determine the erosion hazard of the on-site soils. The surface soils for this site were mapped as Kitsap silt loam, 0 to 8 percent slopes. The erosion hazard for this material is listed as slight. It is our opinion that the erosion hazard for site soils should be low in areas where the site is not disturbed.

LABORATORY ANAYLYSIS

We performed one grain-size sieve analysis on a soil sample obtained from Hand Auger 1 at 1.8 feet below the existing ground surface, on June 23, 2013. The results are presented as **Figure 8**.

CONCLUSIONS AND RECOMMENDATIONS

General

It is our opinion from a geotechnical standpoint that the site is compatible with the planned development of a two-story structure with daylight-basement parking. Our explorations indicated that the site is generally underlain by competent glacial soils below a surficial layer of approximately two to four and a half feet of topsoil or undocumented fill. The native soils underlying the site at depth, below this surficial layer should provide adequate support for foundation, slab, and pavement loads. The new building is planned to occupy the vast majority of the site. We recommend that the building be designed utilizing shallow foundations; however, we understand the parking level will be a daylight basement style, where the opening of the garage is proposed to the north, and the level becomes subsurface to the south. Footings should extend through the undocumented fill or loose soil and be founded on the underlying medium dense or better native soil, or structural fill extending to these soils. The medium dense or better soil should typically be encountered approximately two to four and a half feet below the existing surface, based on our explorations. We should note that deeper areas of unsuitable soils and/or undocumented fill could be encountered in the unexplored areas of the site, especially in the area of the old structure that was removed. This condition, if encountered, would require deeper excavations in foundation and slab areas to remove the unsuitable soils.

Cuts up to approximately 10- to 12-feet are planned along the southern and portions of the western and eastern sides of the property for the construction of the parking garage. Since these cuts cannot be sloped back due to site constraints, we recommend that the cuts be shored with a soldier pile retaining wall. This wall could be designed as a permanent wall and incorporated into the building. We provide recommendations for temporary and permanent cut slopes in the Temporary and Permanent Slopes section of this report. We also provide recommendations for the soldier pile wall in the Shoring Wall subsection of this report.

Infiltration capacity of the site soils were re-evaluated in our most recent site visits per the 2019 Department of Ecology (DOE) Stormwater Management Manual of Western Washington. In general, due to the dense silty nature of the site soils, results from testing in the wet season, and the fact that seasonal high groundwater extends at least within 6-feet from the existing ground surface, an infiltration gallery underlying the structure will not be feasible. Drainage should be retained onsite (detention) or connected to a City system, if possible.

The site soils are generally silty in nature and are considered highly moisture sensitive. We recommend that construction take place during the drier summer months, if possible. If construction is to take place during the rainy months, the soils exposed in the excavation will disturb and additional expenses and delays may be expected due to the wet conditions. Moderate to severe groundwater seepage may be encountered in cuts as well, if construction takes place in winter. Additional expenses could include the need for placing a blanket of rock spalls on exposed subgrades. The on-site soils are generally considered unsuitable for use as structural fill. NGA should be retained to determine if the native on-site soils can be used as structural fill material during construction.

Control of groundwater during and after construction will be important for a successful outcome. Most seepage during construction should be able to be controlled using sump-and-pump systems. For the permanent conditions, ample drainage and waterproofing systems should be incorporated into the design. Such systems could include foundation drains, under slab drainage systems, heavy-duty waterproofing of the basement walls, and other systems.

Erosion Control Measures

The erosion hazard for the on-site soils is considered to be slight, but actual erosion potential will be dependent on how the site is graded and how water is allowed to concentrate. Best Management Practices (BMPs) should be used to control erosion. Areas disturbed during construction should be protected from erosion. Erosion control measures may include diverting surface water away from the stripped areas and erecting silt fences and/or straw bales to prevent muddy water from leaving the site. We also recommend that stockpiles and excavation walls be covered with plastic sheeting. The erosion potential of areas not disturbed should be low.

Site Preparation and Grading

After erosion control measures are implemented and the existing structure is removed, site preparation should consist of removing topsoil, fills, and loose soils and undocumented fill from the building area to expose medium dense or better native soils. The excavation for the building should only be attempted after the shoring wall is installed. The stripped soil should be removed from the site. Based on our observations, we anticipate medium dense or better soil to be encountered approximately 2.0- to 4.5-feet across the site, but this depth could increase in unexplored areas of the site and in the vicinity of the existing structure.

The soldier pile wall should be installed prior to cutting along the southern property line down to the planned elevation for the lower level of the building.

After site preparation, if the exposed subgrade is deemed loose, it should be compacted to a non-yielding condition as approved by NGA. Areas observed to pump or weave during compaction should be reworked to structural fill specifications or over-excavated and replaced with properly compacted structural fill or rock spalls. If loose soils are encountered in the pavement areas, the loose soils should be removed and replaced with rock spalls or granular structural fill. If significant surface water flow is encountered during construction, this flow should be diverted around areas to be developed, and the exposed subgrades should be maintained in a semi-dry condition.

If wet conditions are encountered, alternative site grading techniques might be necessary. These could include using large excavators equipped with wide tracks and a smooth bucket to complete site grading and covering exposed subgrade with a layer of crushed rock for protection. If wet conditions are encountered or construction is attempted in wet weather, the subgrade should not be compacted as this could cause further subgrade disturbance. In wet conditions it may be necessary to cover the exposed subgrade with a layer of crushed rock as soon as it is exposed to protect the moisture sensitive soils from disturbance by machine or foot traffic during construction. The prepared subgrade should be protected from construction traffic and surface water should be diverted around areas of prepared subgrade.

We recommend that construction take place during dry weather, if possible. However, if construction takes place during wet weather, additional expenses and delays should be expected due to the wet conditions. Additional expenses could include the need for placing a blanket of rock spalls on exposed subgrades and construction traffic areas. Wet weather grading will also require additional erosion control and site drainage measures. The on-site soils are generally not suitable for use as structural fill. NGA should be retained to evaluate the suitability of all on-site and imported structural fill material during construction.

Temporary and Permanent Slopes

Temporary cut slope stability is a function of many factors, including the type and consistency of soils, depth of the cut, surcharge loads adjacent to the excavation, length of time a cut remains open, and the presence of surface water or groundwater. It is exceedingly difficult under these variable conditions to estimate a stable, temporary, cut slope angle. Therefore, it should be the responsibility of the contractor to maintain safe slope configurations since he is continuously at the job site, able to observe the soil and groundwater conditions encountered, and able to monitor the nature and condition of the cut slopes.

The following information is provided solely for the benefit of the owner and other design consultants and should not be construed to imply that Nelson Geotechnical Associates, Inc. assumes responsibility for job site safety. Job site safety is the sole responsibility of the project contractor.

For planning purposes, we recommend that temporary cuts in the on-site soils be no steeper than 1.5 Horizontal to 1 Vertical (1.5H:1V). If significant groundwater seepage or surface water flow were encountered, we would expect that flatter inclinations would be necessary. We recommend that cut slopes be protected from erosion. The slope protection measures may include covering cut slopes with plastic sheeting and diverting surface runoff away from the top of cut slopes. We do not recommend vertical slopes for cuts deeper than four feet, if worker access is necessary. We recommend that cut slope heights and inclinations conform to appropriate OSHA/WISHA regulations.

We recommend permanent vertical cuts are planned to be supported by a soldier pile wall as discussed in the **Shoring Wall** subsection. Other permanent cuts and/or fill slopes should be no steeper than 2H:1V, unless specifically approved by NGA. Also, flatter inclinations may be required in areas where loose soils are encountered. Permanent slopes should be vegetated and the vegetative cover maintained until established. This can be discussed with the designers and we can provide recommendations, as needed.

Foundation Support

Conventional shallow spread foundations for the planned building should be placed on medium dense or better native soils, or be supported on structural fill or rock spalls extending to those soils. Medium dense or better soils should be encountered approximately 2.0- to 4.5-feet below the ground surface based on our explorations. Where undocumented fill or less dense soils are encountered at footing bearing elevation, the subgrade should be over-excavated to expose suitable bearing soil. The over-excavation may be filled with structural fill, or the footing may be extended down to the native bearing soils. If footings are supported on structural fill, the fill zone should extend outside the edges of the footing a distance equal to one-half of the depth of the over-excavation below the bottom of the footing.

Footings should extend at least 18 inches below the lowest adjacent finished ground surface for frost protection and bearing capacity considerations. Foundations should be designed in accordance with the 2018 IBC. Footing widths should be based on the anticipated loads and allowable soil bearing pressure. Water should not be allowed to accumulate in footing trenches. All loose or disturbed soil should be removed from the foundation excavation prior to placing concrete.

For foundations constructed as outlined above, we recommend an allowable design bearing pressure of not more than 3,000 pounds per square foot (psf) be used for the design of footings founded on the medium dense or better native soils or structural fill extending to the competent native material. The foundation bearing soil should be evaluated by a representative of NGA. We should be consulted if higher bearing pressures are needed. Current IBC guidelines should be used when considering increased allowable bearing pressure for short-term transitory wind or seismic loads. Potential foundation settlement using the recommended allowable bearing pressure is estimated to be less than one-inch total and ½-inch differential between adjacent footings or across a distance of about 20 feet, based on our experience with similar projects.

Lateral loads may be resisted by friction on the base of the footing and passive resistance against the subsurface portions of the foundation. A coefficient of friction of 0.35 may be used to calculate the base friction and should be applied to the vertical dead load only. Passive resistance may be calculated as a triangular equivalent fluid pressure distribution. An equivalent fluid density of 200 pounds per cubic foot (pcf) should be used for passive resistance design for a level ground surface adjacent to the footing. This level surface should extend a distance equal to at least three times the footing depth. These recommended values incorporate safety factors of 1.5 and 2.0 applied to the estimated ultimate values for frictional and passive resistance, respectively. To achieve this value of passive resistance, the foundations should be poured “neat” against the native medium dense soils or compacted fill should be used as backfill against the front of the footing. We recommend that the upper one foot of soil be neglected when calculating the passive resistance.

Structural Fill

General: Fill placed beneath foundations, pavement, or other settlement-sensitive structures should be placed as structural fill. Structural fill, by definition, is placed in accordance with prescribed methods and standards, and is monitored by an experienced geotechnical professional or soils technician. Field monitoring procedures would include the performance of a representative number of in-place density tests to document the attainment of the desired degree of relative compaction. The area to receive the fill should be suitably prepared as described in the **Site Preparation and Grading** subsection of this report prior to beginning fill placement.

Materials: Structural fill should consist of a good quality, granular soil, free of organics and other deleterious material, and be well graded to a maximum size of about three inches. All-weather structural fill should contain no more than five-percent fines (soil finer than U.S. No. 200 sieve, based on that fraction passing the U.S. 3/4-inch sieve). The use of on-site soils as structural fill is not recommended. We should be retained to evaluate proposed structural fill material prior to placement.

Fill Placement: Following subgrade preparation, placement of structural fill may proceed. All fill placements should be accomplished in uniform lifts up to eight inches thick. Each lift should be spread evenly and be thoroughly compacted prior to placement of subsequent lifts. All structural fill underlying building areas and pavement subgrade should be compacted to a minimum of 95 percent of its maximum dry density. Maximum dry density, in this report, refers to that density as determined by the ASTM D-1557 Compaction Test procedure. The moisture content of the soils to be compacted should be within about two percent of optimum so that a readily compactable condition exists. It may be necessary to over-excavate and remove wet soils in cases where drying to a compactable condition is not feasible. All compaction should be accomplished by equipment of a type and size sufficient to attain the desired degree of compaction.

Shoring Wall

General: We recommend that a soldier pile wall be used to support the cuts on the southern and portions of the eastern and western sides of the building. This wall can be used as a permanent wall and incorporated into the building. We anticipate cuts up to approximate 10- to 12-feet that would be supported by this wall.

A solider pile wall typically consists of a series of steel H-beams placed vertically at a certain spacing from one another (typically six to ten feet). The beams are usually placed in drilled shafts that are filled with structural concrete or a lean mix. The concrete shafts are typically embedded below the bottom of the planned excavation a distance equals one to two times the exposed height of the wall. The steel beams are extended above finished ground surface to provide shoring capabilities for the area to be retained. The beams are typically spanned by pressure treated timber lagging or concrete panels. The H-beam size, shaft diameter, shaft embedment, and pile spacing are dependent on the nature of the soils anticipated to be retained by the wall and the soils at depth, wall height, drainage conditions, and the final geometry. A schematic detail of the wall is shown on the Conceptual Soldier Pile Wall Detail in **Figure 9.**

Wall Design: The shoring wall should be designed by an experienced structural engineer licensed in the State of Washington. The lateral earth pressure acting on the shoring wall will be dependent on the nature and density of the soil behind the wall, structure and traffic loads on the wall, and the amount of lateral wall movement that may occur as material is excavated from the front of the wall. If the shoring wall is free to yield at least one-thousandth of the retained height, an "active" loading condition develops. If the wall is restrained from movement by stiffness or bracing, the wall is considered in an "at-rest" loading condition. Active and at-rest earth pressure can be calculated based on equivalent fluid densities.

The shoring wall should be designed to resist a lateral load resulting from a fluid with a unit weight of 40 and 60 pounds per cubic foot (pcf) for the active and at-rest loading conditions, respectively. These loads should be applied across the pile spacing above the excavation line. These loads can be resisted by a passive pressure of 200 pcf for the medium dense/stiff or better soils. The passive pressure should be applied on two-pile diameters under the excavation line. These values of the passive pressure incorporate a factor of safety of 2.0. The upper one-foot of wall embedment should be neglected when calculating the passive resistance.

The above load should be applied on the full center-to-center pile spacing above the base of the exposed portion of the wall. A 50 percent reduction of this value can be applied for the purpose of designing the wall lagging. The below-grade portion of the wall should not be shorter than 1.5 times the wall stick-up height.

The above pressures assume that the on-site soils retained by the shoring wall are mostly granular in nature and that hydrostatic forces are not allowed to build up behind the wall. These values do not include the effects of surcharges; such as due to foundation loads, traffic, or other surface loads. Surcharge effects should be considered where appropriate. The retained soils should be readily drained and collected water should be routed into a permanent storm system. Adequate gaps should be maintained between the lagging elements to allow for water seepage through the wall.

The wall designer should calculate the predicted wall deflection, including deflection resulting from the below-grade movement of the piles. The predicted deflection values should be confirmed in the field through a monitoring program. Also, existing surrounding structures and roads should be monitored for any adverse effects resulting from shoring wall installation. We should be retained to discuss wall and surrounding structure monitoring plans.

Shoring Wall Installation: The shoring wall should be installed by a shoring contractor experienced with this type of system. We anticipate that an open-hole drilling method may prove difficult to achieve for installing the soldier piles in the on-site soils, and therefore we recommend that the shoring contractor be capable of casing the holes as sloughing and/or water seepage will likely be encountered. It might be prudent to perform one or more “test” holes to confirm installation conditions prior to finalizing budget and work plans. Any sloughing or water that may collect in the drilled holes should be removed prior to pouring grout. Grout should be readily available on site at the time the holes are drilled and cased.

If groundwater seepage is encountered, we recommend that water be pumped out of the holes and the concrete be tremied from the bottom of the excavations to displace the groundwater to the surface. Extra Portland Cement may also be placed in the bottom of the excavations to reduce the effects of seepage. The spoils from the soldier pile excavations are expected to be moisture-sensitive materials and should be removed from the site along with all slide debris found on the downhill side of the wall. We should be retained to monitor onsite activities during the shoring wall installation on a full-time basis.

The wall should be lagged using pressure-treated timber. Adequate gaps, typically by placing lagging nails between the boards, should be maintained between the lagging elements to allow water flow through the face of the wall.

Other Retaining Walls

If the soldier pile wall is not designed as a permanent wall, separate retaining walls will need to be constructed. For those walls and other retaining walls, the lateral pressure acting on subsurface retaining walls is dependent on the nature and density of the soil behind the wall, the amount of lateral wall movement which can occur as backfill is placed, wall drainage conditions, the inclination of the backfill, and other possible surcharge loads. For walls that are free to yield at the top at least one thousandth of the height of the wall (active condition), soil pressures will be less than if movement is limited by such factors as wall stiffness or bracing (at-rest condition). We recommend that walls supporting horizontal backfill and not subjected to hydrostatic forces be designed using a triangular earth pressure distribution equivalent to that exerted by a fluid with a density of 40 pcf for yielding (active condition) walls, and 60 pcf for non-yielding (at-rest condition) walls.

These recommended lateral earth pressures are for a drained granular backfill and are based on the assumption of a horizontal ground surface behind the wall for a distance of at least the subsurface height of the wall, and do not account for surcharge loads. Additional lateral earth pressures should be considered for surcharge loads acting adjacent to subsurface walls and within a distance equal to the subsurface height of the wall. This would include the effects of surcharges such as traffic loads, floor slab, foundation loads, slopes, or other surface loads. Also, hydrostatic and buoyant forces should be included if the walls could not be drained. We could consult with the structural engineer regarding additional loads on retaining walls during final design, if needed.

The lateral pressures on walls may be resisted by friction between the foundation and subgrade soil and by passive resistance acting on the below-grade portion of the foundation. Recommendations for frictional and passive resistance to lateral loads are presented in the Foundation Support subsection of this report.

All wall backfill should be well-compacted as outlined in the **Structural Fill** subsection of this report. Care should be taken to prevent the building up of excess lateral soil pressures due to over-compaction of the wall backfill. This can be accomplished by placing wall backfill in thin loose lifts and compacting it with small, hand-operated compactors within a distance behind the wall equal to at least one-half the height of the wall. The thickness of the loose lifts should be reduced to accommodate the lower compactive energy of the hand-operated equipment. The recommended level of compaction should still be maintained.

Permanent drainage should be installed for retaining walls. Recommendations for these systems are found in the **Subsurface Drainage** subsection of this report. We recommend that we be retained to evaluate the proposed wall drain backfill material and drainage system installation.

Pavements

The pavement subgrade should be prepared as recommended in the **Site Preparation and Grading** and **Structural Fill** subsections of this report, including proof-rolling the subgrade with a loaded dump truck and repairing areas observed to pump or weave during the proof-roll test. Also, all fill placed within the pavement areas, including utility trench backfill, should be compacted to 95 percent of the Maximum Dry Density (Modified Proctor). We should be retained to observe the proof-roll test. Any areas observed to pump or weave under the wheels of the loaded dump truck should be over-excavated and replaced with crushed rock.

Slab-on-Grade

Slab-on-grade should be supported on subgrade soils prepared as described in the **Site Preparation and Grading** subsection of this report. We recommend that all floor slabs be underlain by at least six inches of free-draining gravel with less than three percent by weight of the material passing Sieve #200 for use as a capillary break. We recommend that the capillary break be hydraulically connected to the footing drain system to allow free drainage from under the slab. A suitable vapor barrier, such as heavy plastic sheeting (6-mil minimum), should be placed over the capillary break material. An additional 2-inch thick moist sand layer may be used to cover the vapor barrier. This sand layer is optional and is intended to protect the vapor barrier membrane during construction.

Site Drainage

Infiltration Testing: We attempted to perform a small PIT test per the 2019 Stormwater Management Manual for Western Washington. We excavated the test hole down to 7.0-feet of depth and encountered silty fine sand with trace gravel in a dense condition with a generally high moisture content. We also excavated Test Pit Five nearby to a depth of 10.0 feet to observe groundwater conditions. We encountered moderate groundwater seepage at a depth of 6.0 feet. At the start of the day, we filled the Infiltration Test Pit with 12-inches of water for the soaking period. After waiting an hour, and adding no additional water to the hole, the water level had increased by ¼ inch. The test was terminated prematurely due to water infiltrating into the hole, resulting in the water level rising instead. It is our opinion that stormwater infiltration within the site is not feasible due to the seasonal high groundwater table being relatively shallow, and due to the silty and compact nature of the transition beds deposit. We recommend that the water be detained onsite or routed to a nearby City of Mukilteo storm system, if feasible.

Surface Drainage: The finished ground surface should be graded such that runoff is directed to an appropriate stormwater collection system. Water should not be allowed to collect in any areas where footings, slabs, or pavements are to be constructed. Final site grades should allow for drainage away from the structure. We suggest that the finished ground be sloped at a minimum gradient of three percent, for a distance of at least 10 feet away from the structures. Surface water generated from paved areas and roof drains should be collected by permanent catch basins and drain lines and be routed into an appropriate discharge system.

Subsurface Drainage: If groundwater is encountered during construction, we recommend that the contractor slope the bottom of the excavation and collect the water into ditches and small sump pits where the water can be pumped from the excavation and routed to a suitable discharge point. We recommend the use of footing drains around the structure and behind all retaining walls. Footing drains should be installed at least one foot below the planned finished floor elevation. The drains should consist of a minimum 4-inch-diameter, rigid, slotted or perforated, PVC pipe surrounded by free-draining material wrapped in a filter fabric. We recommend that the free-draining material consist of an 18-inch-wide zone of clean (less than three-percent fines), granular material placed along the back of walls. Washed rock is an acceptable drain material, or a drainage composite may be used instead. The free-draining material or the drainage composite should extend up the wall to one foot below the finished surface. The top foot of backfill should consist of low permeability soil placed over plastic sheeting or building paper to minimize the migration of surface water or silt into the footing drain. Footing drains should discharge into tightlines leading to an appropriate collection and discharge point with convenient cleanouts to prolong the useful life of the drains. Roof drains should not be connected to wall or footing drains.

The shoring wall should be drained by maintaining suitable gaps between the lagging elements to allow water seepage through the wall. Depending on final wall configuration, a drainage composite should be placed along the face of the wall to collect water seeping through the wall face. The collected water would be routed down to the bottom of the wall where a perforated drainpipe should be placed to transmit the collected water into the drainage system. The garage walls can be cast directly on the drainage composite. This concept is shown in **Figure 9.**

Depending on the amount of subsurface water encountered on this site, it may be prudent to install a system of underslab drains underneath the entire building footprint. This system would consist of 4-inch perforated PVC pipes placed within the capillary break layer at roughly 20-foot spacings which are sloped to drain into a main 6-inch solid collector pipe. The main collector pipe would be connected to the drainage system outside the building footprint. Also, ample heavy-duty waterproofing of the basement walls should be incorporated into the project plans.

CONSTRUCTION MONITORING

We recommend that we be retained to provide construction monitoring services to evaluate conditions encountered in the field with respect to anticipated conditions, to provide recommendations for design changes should the conditions differ from anticipated, and to evaluate whether construction activities comply with contract plans and specifications.

USE OF THIS REPORT

NGA has prepared this report for **Ryan Kilby of Williams Investments,** and associated agents, for use in the planning and design of the development planned on this site only. The scope of our work does not include services related to construction safety precautions and our recommendations are not intended to direct the contractors' methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design. There are possible variations in subsurface conditions between the explorations and also with time. Our report, conclusions, and interpretations should not be construed as a warranty of subsurface conditions. A contingency for unanticipated conditions should be included in the budget and schedule.

We recommend that NGA be retained to review project plans as they are being developed. We also recommend that we be retained to provide monitoring and consultation services during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether or not earthwork and foundation installation activities comply with contract plans and specifications. We should be contacted a minimum of one week prior to construction activities and could attend pre-construction meetings if requested.

Within the limitations of scope, schedule, and budget, our services have been performed in accordance with generally accepted geotechnical engineering practices in effect in this area at the time this report was prepared. No other warranty, expressed or implied, is made. Our observations, findings, and opinions are a means to identify and reduce the inherent risks to the owner.

o-o-o

It has been a pleasure to provide service to you on this project. If you have any questions or require further information, please call.

Sincerely,

NELSON GEOTECHNICAL ASSOCIATES, INC.

Katelyn S. Brower, GIT
Project Geologist

1.09.2023

Khaled M. Shawish, PE
Principal

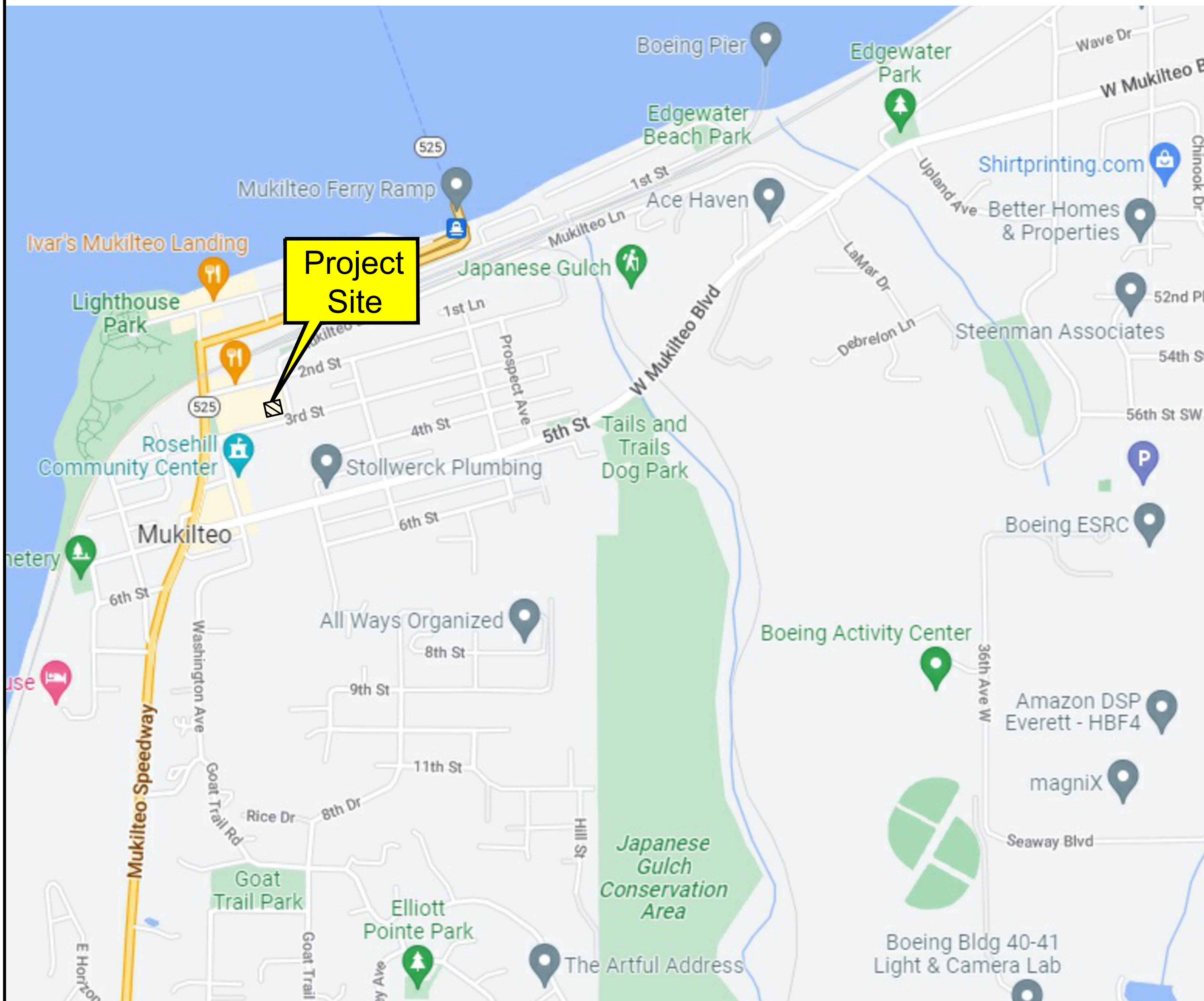
KSB:KMS:dy

Nine Figures Attached

VICINITY MAP

Not to Scale

N



Mukilteo, WA

Project Number 8797B22	Rose Hill Mixed-Use Building Development Vicinity Map	NGA NELSON GEOTECHNICAL ASSOCIATES, INC	No.	Date	Revision	By	CK
Figure 1		Woodinville Office 17311-135th Ave. NE, A-500 Woodinville, WA 98072 (425) 486-1669 / Fax: 481-2510 — Wenatchee Office 105 Palouse St. Wenatchee, WA 98801 (509) 665-7696 / Fax: 665-7692 — www.nelsongeotech.com	1	12/1/22	Original	FKS	DJO

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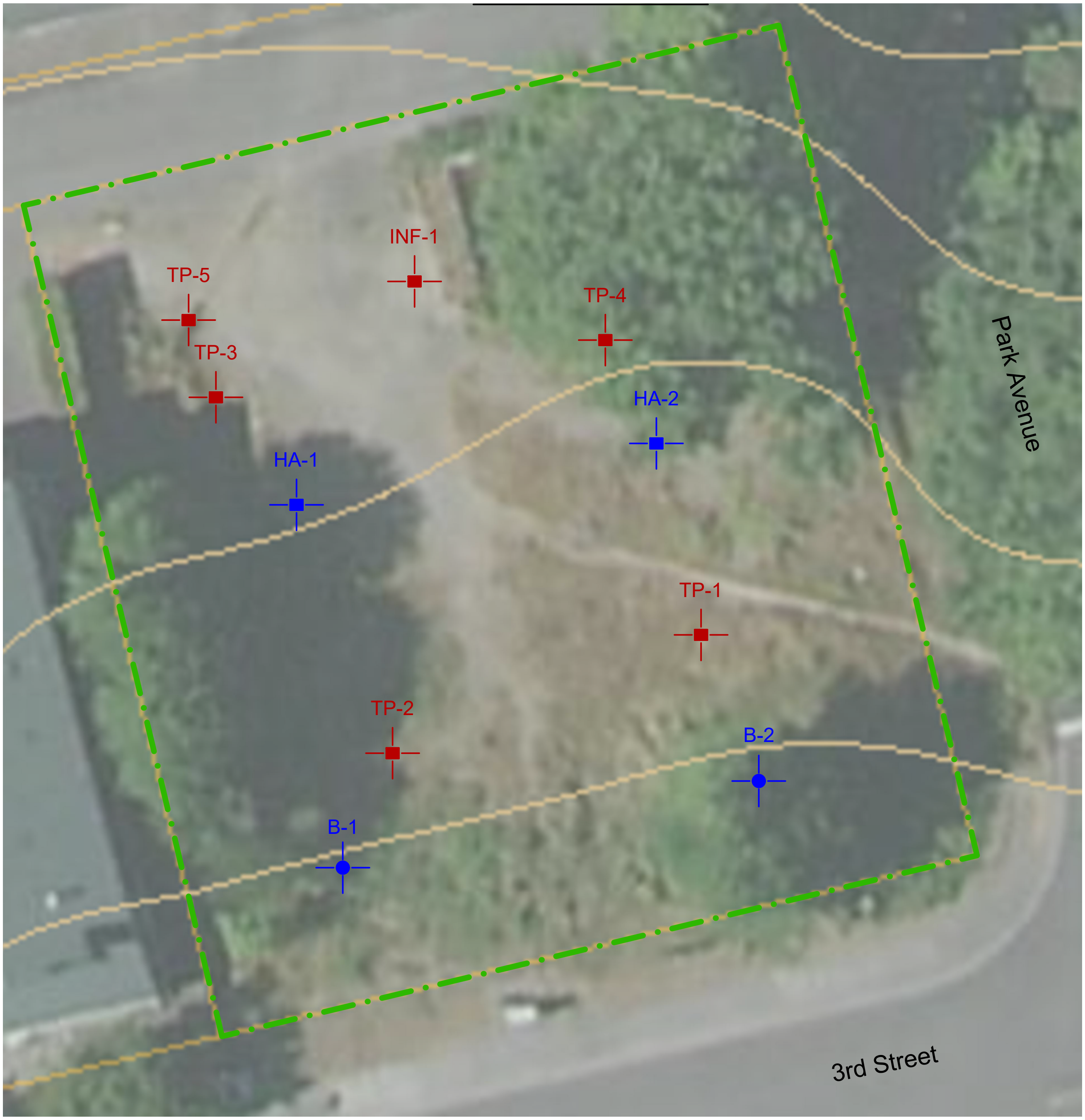
Site Plan



0 20 40

Scale: 1 inch = 20 feet

LEGEND

Property line

TP-1 — Number and approximate location of test pit

INF-1 — Number and approximate location of infiltration test pit

B-1 — Number and approximate location of boring on 7/23/2013

HA-1 — Number and approximate location of hand auger on 7/23/2013

Reference: Site Plan based on field measurements, observations, and aerial parcel map review.

Project Number	Rose Hill Mixed-Use Building Development Site Plan	NELSON GEOTECHNICAL ASSOCIATES, INC	No.	Date	Revision	By	CK
8797B22			1	12/1/22	Original	FKS	DJO
Figure 2							

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UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE - GRAINED SOILS MORE THAN 50 % RETAINED ON NO. 200 SIEVE	GRAVEL MORE THAN 50 % OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVEL	GW	WELL-GRADED, FINE TO COARSE GRAVEL
			GP	POORLY-GRADED GRAVEL
		GRAVEL WITH FINES	GM	SILTY GRAVEL
			GC	CLAYEY GRAVEL
	SAND MORE THAN 50 % OF COARSE FRACTION PASSES NO. 4 SIEVE	CLEAN SAND	SW	WELL-GRADED SAND, FINE TO COARSE SAND
			SP	POORLY GRADED SAND
		SAND WITH FINES	SM	SILTY SAND
			SC	CLAYEY SAND
FINE - GRAINED SOILS MORE THAN 50 % PASSES NO. 200 SIEVE	SILT AND CLAY LIQUID LIMIT LESS THAN 50 %	INORGANIC	ML	SILT
			CL	CLAY
		ORGANIC	OL	ORGANIC SILT, ORGANIC CLAY
	SILT AND CLAY LIQUID LIMIT 50 % OR MORE	INORGANIC	MH	SILT OF HIGH PLASTICITY, ELASTIC SILT
			CH	CLAY OF HIGH PLASTICITY, FAT CLAY
		ORGANIC	OH	ORGANIC CLAY, ORGANIC SILT
HIGHLY ORGANIC SOILS			PT	PEAT

NOTES:

1) Field classification is based on visual examination of soil in general accordance with ASTM D 2488-93.

2) Soil classification using laboratory tests is based on ASTM D 2488-93.

3) Descriptions of soil density or consistency are based on interpretation of blowcount data, visual appearance of soils, and/or test data.

SOIL MOISTURE MODIFIERS:

Dry - Absence of moisture, dusty, dry to the touch

Moist - Damp, but no visible water.

Wet - Visible free water or saturated, usually soil is obtained from below water table

Project Number 8797B22	Rose Hill Mixed-Use Building Development Soil Classification Chart	NELSON GEOTECHNICAL ASSOCIATES, INC
Figure 3		

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LOG OF EXPLORATION

DEPTH (FEET)	USCS	SOIL DESCRIPTION
TEST PIT ONE		
0.0 – 2.0		BROWN TO DARK BROWN, SILTY, FINE TO MEDIUM SAND WITH ORGANICS, ROOTS, GRAVEL, AND IRON-OXIDE WEATHERING (LOOSE TO MEDIUM DENSE, DRY TO MOIST) (**FILL**)
2.0 – 5.0	SP	GRAY-BROWN, FINE TO MEDIUM SAND WITH TRACE ROOTS AND IRON-OXIDE STAINING (MEDIUM DENSE TO DENSE, DRY TO MOIST)
5.0 – 6.0	SM	LIGHT GRAY TO TAN, SILTY, FINE TO MEDIUM SAND WITH ROOTS AND IRON-OXIDE STAINING (MEIDUM DENSE TO DENSE, DRY TO MOIST)
		SAMPLE WAS COLLECTED AT 5.5 FEET GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 6.0 FEET ON 11/22/2022
TEST PIT TWO		
0.0 – 2.0		**TOPSOIL / FILL**
2.0 – 4.5	SP	GRAY-BROWN, FINE TO MEDIUM SAND WITH TRACE ROOTS AND IRON-OXIDE STAINING (MEDIUM DENSE TO DENSE, MOIST)
4.5 – 5.5	SM	LIGHT GRAY TO GRAY, SILTY, FINE SANDWITH TRACE ROOTS AND IRON-OXIDE STAINING (MEDIUM DENSE TO DENSE, MOIST)
5.5 – 6.5	SP	GRAY TO GRAY-BROWN, FINE TO COARSE SAND WITH GRAVEL (MEDIUM DENSE TO DENSE, MOIST)
		SAMPLE WAS COLLECTED AT 6.5 FEET GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 6.5 FEET ON 11/22/2022
TEST PIT THREE		
0.0 – 2.0		**TOPSOIL / FILL**
2.0 – 4.0	SP	GRAY-BROWN, FINE TO MEDIUM SAND WITH TRACE ROOTS AND IRON-OXIDE STAINING (MEDIUM DENSE TO DENSE, MOIST)
4.0 – 6.5	SM	GRAY TO ORANGE-GRAY, SILTY, FINE SAND WITH TRACE ROOTS AND TRACE GRAVEL (MEDIUM DENSE TO DENSE, MOIST)
		SAMPLE WAS COLLECTED AT 5.0 FEET GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 6.5 FEET ON 11/22/2022
TEST PIT FOUR		
0.0 – 3.0		**TOPSOIL / FILL**
3.0 – 4.5	SP	GRAY-BROWN, FINE TO MEDIUM SAND WITH TRACE ROOTS AND IRON-OXIDE STAINING (MEDIUM DENSE TO DENSE, MOIST)
		SAMPLES WERE NOT COLLECTED GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 4.5 FEET ON 11/22/2022

LOG OF EXPLORATION

DEPTH (FEET)	USCS	SOIL DESCRIPTION
TEST PIT FIVE		
0.0 – 4.5		DARK BROWN TO BLACK, SILTY, FINE TO MEDIUM SAND WITH GRAVEL, ROOTS, AND METAL (LOOSE TO MEDIUM DENSE, MOIST) (**FILL**)
4.5 – 6.0	SM	ORANGE-GRAY TO GRAY, SILTY, FINE SAND WITH IRON-OXIDE STAINING, TRACE GRAVEL, AND TRACE ROOTS (MEDIUM DENSE, MOIST)
6.0 – 10.0	SM	GRAY, SILTY, FINE SAND WITH TRACE GRAVEL (MEDIUM DENSE TO DENSE, MOIST)
		SAMPLES WERE NOT COLLECTED GROUNDWATER SEEPAGE WAS ENCOUNTERED AT 6.0 FEET TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 10.0 FEET ON 12/7/2022
INFILTRATION TEST PIT ONE		
0.0 – 1.0		**TOPSOIL / FILL**
1.0 – 2.0	SM	ORANGE-BROWN, SILTY, FINE TO MEDIUM SAND WITH ROOTS, IRON-OXIDE WEATHERING, AND TRACE GRAVEL (LOOSE TO MEDIUM DENSE, MOIST)
2.0 – 5.0	SP-SM	GRAY TO GRAY-BROWN, FINE TO MEDIUM SAND WITH SILT AND TRACE GRAVEL (MEDIUM DENSE TO DENSE, MOIST)
5.0 – 7.0	SM	GRAY, SILTY, FINE SAND WITH TRACE GRAVEL (MEDIUM DENSE TO DENSE, MOIST)
		SAMPLES WERE NOT COLLECTED GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 7.0 FEET ON 12/7/2022
HAND AUGER ONE		
0.0 – 0.9		DARK BROWN, FINE TO MEDIUM SAND WITH SILT AND TRACE GRAVEL (LOOSE TO MEDIUM DENSE, MOIST) (**FILL**)
0.9 – 1.2		LIGHT BROWN, FINE TO MEDIUM SAND WITH SILT AND TRACE GRAVEL (MEDIUM DENSE, MOIST) (**FILL**)
1.2 – 1.7		ORANGE-BROWN, FINE TO MEDIUM SAND WITH SILT AND TRACE GRAVEL (MEDIUM DENSE, MOIST) (**FILL**)
1.7 – 1.9	SP-SM	LIGHT ORANGE-BROWN, FINE TO MEDIUM SAND WITH SILT AND TRACE GRAVEL (MEDIUM DENSE, MOIST)
		SAMPLES WERE COLLECTED AT 0.5, 1.0, 1.5, AND 1.8 FEET GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED HAND AUGER CAVING WAS NOT ENCOUNTERED HAND AUGER WAS COMPLETED AT 1.9 FEET ON 7/23/2013
HAND AUGER TWO		
0.0 – 0.9		LIGHT BROWN, FINE TO MEDIUM SAND WITH SILT (LOOSE, DRY TO MOIST) (**FILL**)
0.9 – 1.3	SP-SM	BROWN, FINE TO MEDIUM SAND WITH SILT (MEDIUM DENSE, MOIST)
1.3 – 1.8	SP-SM	DARK BROWN TO ORANGE-BROWN, FINE TO MEDIUM SAND WITH SILT AND TRACE GRAVEL (MEDIUM DENSE, MOIST)
		SAMPLES WERE COLLECTED AT 0.5, 1.0, AND 1.5 FEET GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED HAND AUGER CAVING WAS NOT ENCOUNTERED HAND AUGER WAS COMPLETED AT 1.8 FEET ON 7/23/2013

BORING LOG

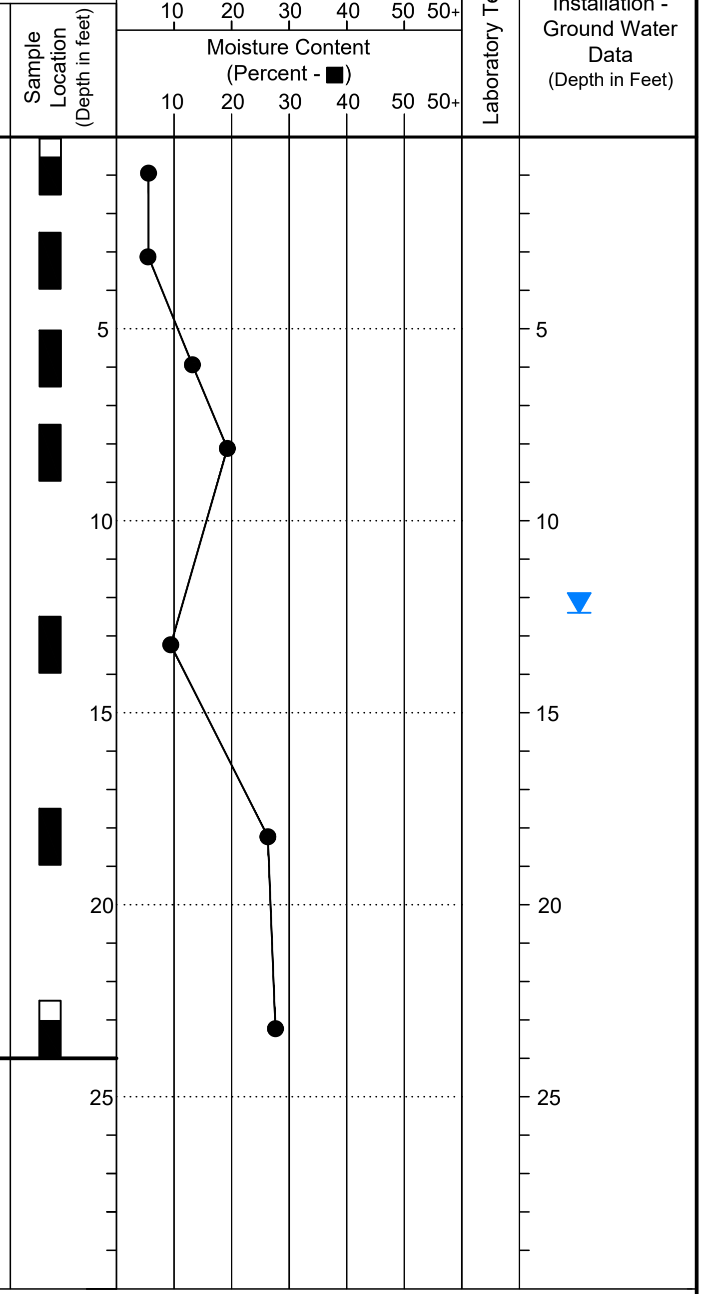
B-1

Logged by: BD on 7/23/2013

Approximate Ground Surface Elevation: ~72 ft

Soil Profile: Description	Graphic Log	Group Symbol	Sample Data: Blow Count
Gray-brown, fine sand, topsoil/modified ground			6
Orange-brown, fine to medium sand (loose, moist)		SP	
Brown-gray, silty fine to medium sand (loose, moist)		SM	6
Brown-gray silt with trace iron-oxide staining (medium stiff, moist) -becomes stiff		ML	12
Dark brown silt with fine sand (very stiff, moist)		ML	19
Gray silt (stiff, moist) -with fine sand		ML	9
Gray, fine to medium sand with silt to silty sand (medium dense, moist to wet)		SP-SM /SM	27
Gray, fine sand with silt to silty fine sand with fine sand lenses (medium dense, moist to wet)		SP-SM /SM	28

Boring terminated below existing grade at 24.0 feet on 7/23/13. Groundwater seepage was encountered at 12.3 feet during drilling.



LEGEND

- Depth Driven and Amount Recovered with 2-inch O.D. Split-Spoon Sampler
- Depth Driven and Amount Recovered with 3-inch Shelby Tube Sampler
- Solid PVC Pipe
- Slotted PVC Pipe
- Monument/ Cap to Piezometer
- * Liquid Limit
- \+ Plastic Limit
- Concrete
- Bentonite
- Native Soil
- Silica Sand
- Water Level

M	Moisture Content
A	Atterberg Limits
G	Grain-size Analysis
DS	Direct Shear
PP	Pocket Penetrometer Readings, tons/ft
P	Sample Pushed
T	Triaxial

NOTE: Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis and judgement. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

Project Number	Rose Hill Mixed-Use Building Development Boring Log
8797B22	
Figure 6	
Page 1 of 1	

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BORING LOG

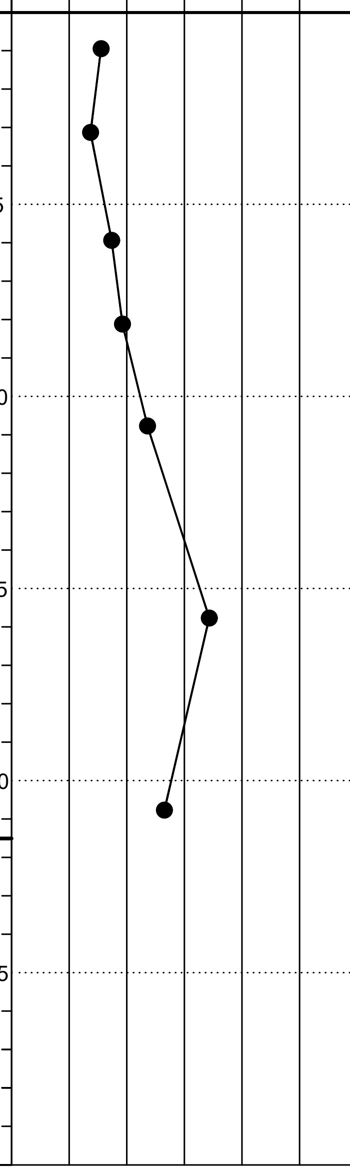
B-2

Approximate Ground Surface Elevation: ~73 ft

Description	Graphic Log	Group Symbol	Blow Count	Sample Location (Depth in feet)
Light brown, silty fine to coarse sand (medium dense, dry) (**FILL** / modified ground)			16	
Light orange-brown, fine to coarse sand with silt and gravel to silty sand (medium dense, dry to moist)		SP-SM / SM	13	
			18	5
			19	
Brown-gray silt (very stiff, moist)		ML		10
Brown-gray, silty fine to medium sand with trace iron-oxide staining (medium dense, wet)		SM	23	
Brown-gray silt with fine sand (very dense, moist)		ML		15
Brown-gray, fine sand with silt (dense, wet)		SP	34	
Brown-gray silt and silt with fine sand layers (very stiff, moist to wet) Gray sand with silt in tip		ML	27	20
Boring terminated below existing grade at 21.5 feet on 7/23/13. Groundwater seepage was encountered at 12.6 feet during drilling.				25

Penetration Resistance (Blows/foot - ●) 10 20 30 40 50 50+

Moisture Content (Percent - ■) 10 20 30 40 50 50+



Laboratory Testing

Piezometer Installation - Ground Water Data (Depth in Feet): 5, 10, 15, 20, 25

LEGEND

- Depth Driven and Amount Recovered with 2-inch O.D. Split-Spoon Sampler
- Depth Driven and Amount Recovered with 3-inch Shelby Tube Sampler
- Solid PVC Pipe
- Slotted PVC Pipe
- Monument/ Cap to Piezometer
- * Liquid Limit
- + Plastic Limit
- Concrete
- Bentonite
- Native Soil
- Silica Sand
- Water Level
- M Moisture Content
- A Atterberg Limits
- G Grain-size Analysis
- DS Direct Shear
- PP Pocket Penetrometer Readings, tons/ft
- P Sample Pushed
- T Triaxial

NOTE: Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis and judgement. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

Project Number 8797B22	Rose Hill Mixed-Use Building Development Boring Log
Figure 7	
Page 1 of 1	

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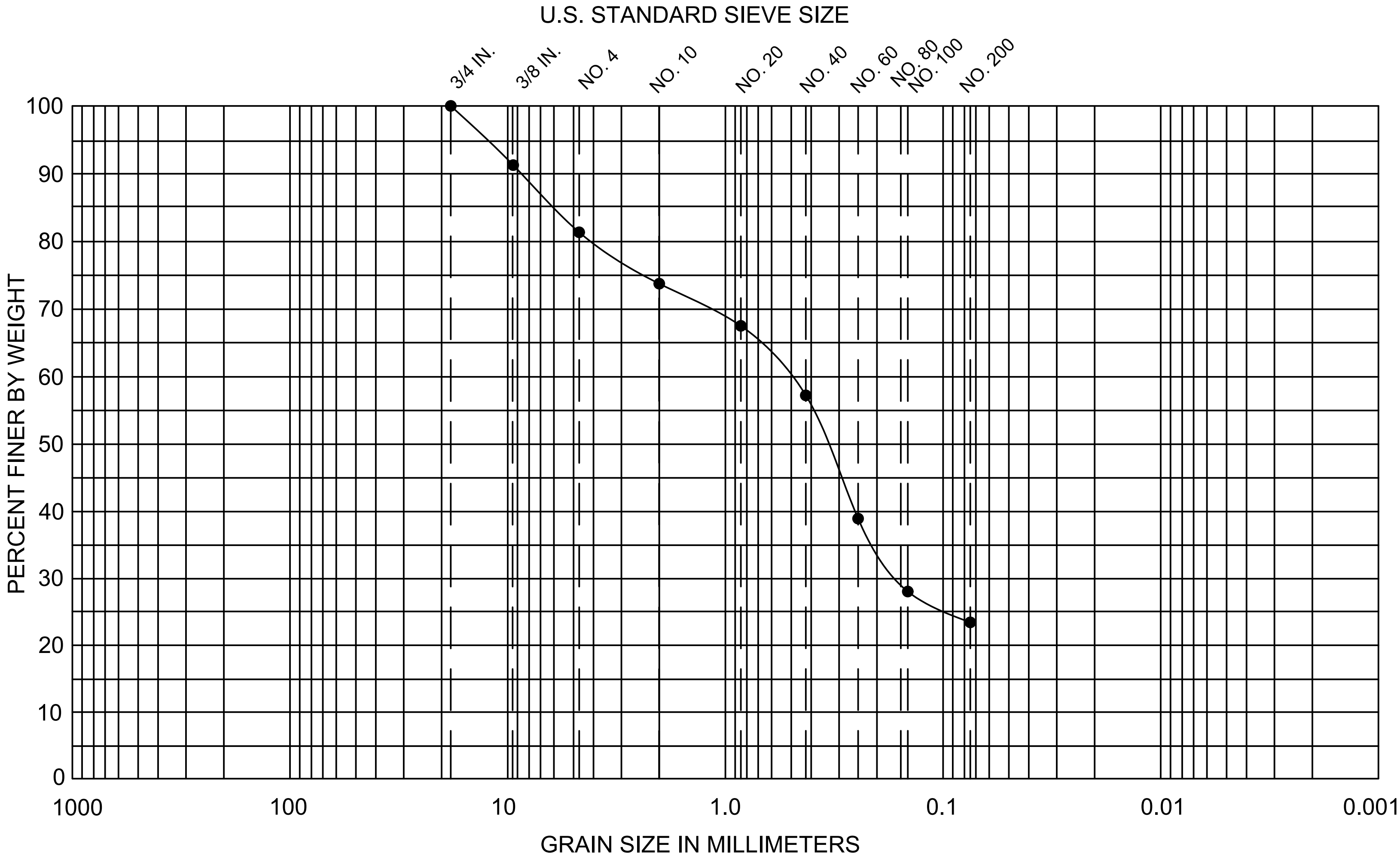

COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

U.S.C. SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION	SOIL DISTRIBUTION
●SM	HA-1	1.8 feet	Light brown, silty fine to medium sand with gravel	Gravel = 19% Sand = 58% Silt/Clay = 23%

Project Number
8797B22
Figure 8

Rose Hill Mixed-Use Building Development
Sieve Analysis

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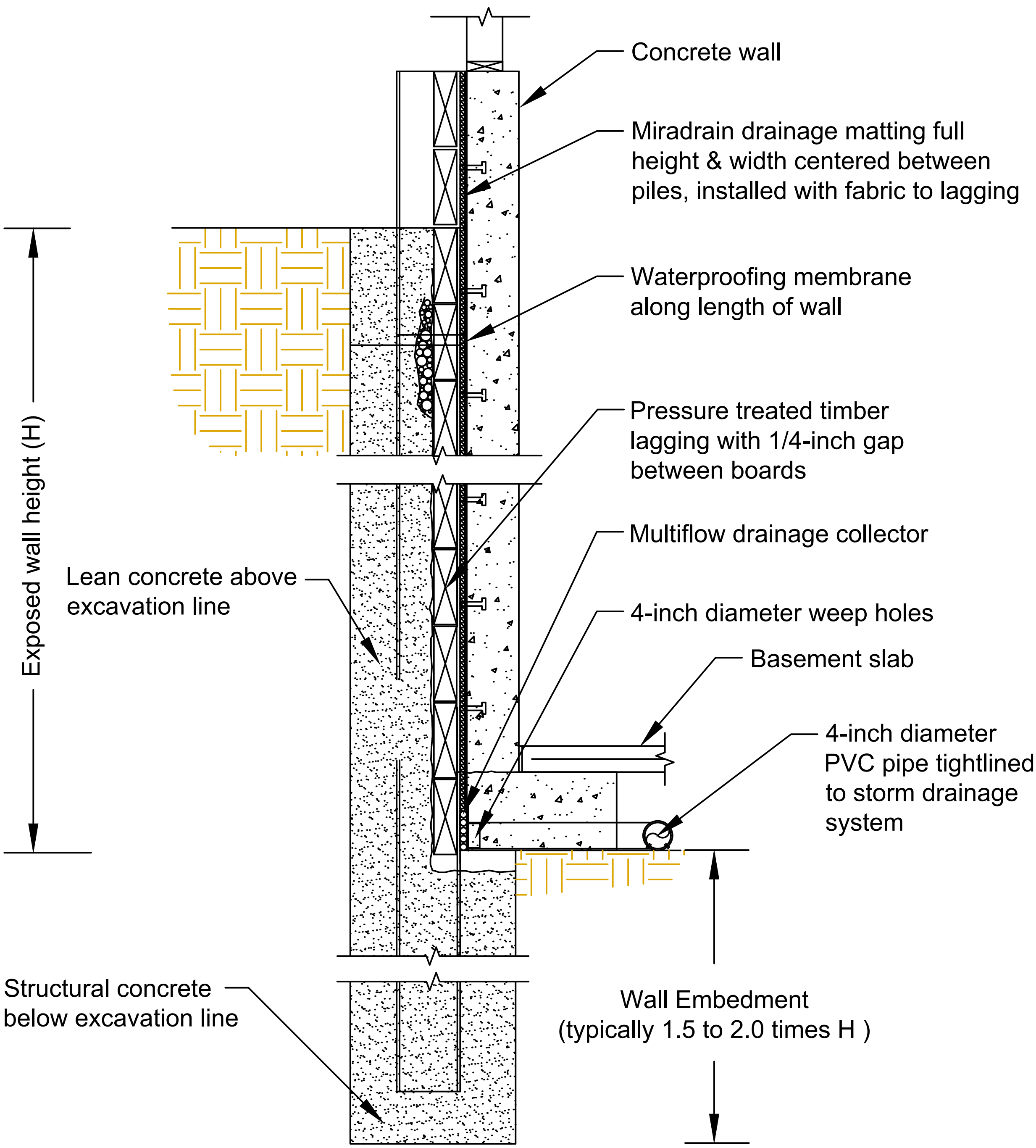
Conceptual Soldier Pile Wall Detail

NOT FOR CONSTRUCTION USE



NOT TO SCALE

Project Number	Rose Hill Mixed-Use Building Development Soldier Pile Wall Detail	NELSON GEOTECHNICAL ASSOCIATES, INC	No.	Date	Revision	By	CK
8797B22			1	1/5/23	Original	FKS	DJO
Figure 9							

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