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June 24, 2024

Ms. Heather Cochran
Cade Hill Homes
Via Email: **heather@cadehillhomes.com**

Geotechnical Engineering Evaluation
Cade Hill Homes 86th Avenue SE Residential Development
4332 and 43XX – 86th Avenue SE
Mercer Island, Washington
NGA File No. 1518224

Dear Ms. Cochran:

We are pleased to submit the attached report titled ***"Geotechnical Engineering Evaluation – Cade Hill Homes 86th Avenue SE Residential Development – 4332 and 43XX - 86th Avenue SE – Mercer Island, 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 the proposal signed by you on May 7, 2024.

The northern property (4332 - 86th Avenue SE) is currently occupied by a single-family residence within the central portion of the property. The southern property (43XX – 86th Avenue SE) is currently vacant. The ground surface within the properties is generally relatively level to gently sloping down from the east to the west. We understand that the proposed development will include removal of the existing site structure within the northern property and construction of a new single-family residence within the central portion of both the northern and southern properties.

We explored the subsurface conditions within the site with seven trackhoe excavated test pits, including two infiltration test pits. Our explorations extended to depths in the range of 4.5 to 8.0 feet below the existing ground surface. Our explorations indicated that the site was underlain by surficial topsoil and/or undocumented fill with competent native glacial till soils at depth.

It is our opinion that the proposed site development is feasible from a geotechnical engineering standpoint, provided that our recommendations for site development are incorporated into project plans. In general, the native glacial bearing soils underlying the site should adequately support the planned structure. Foundations should be advanced through any loose surficial and/or undocumented fill soils down to the competent glacial bearing material interpreted to underlie the site, for bearing capacity and settlement considerations. These soils should generally be encountered between approximately 2.0 to 3.0 feet below the existing ground surface, based on our explorations. If deeper areas of loose soils or undocumented fill are encountered in unexplored areas of the site, they should be removed and replaced with structural fill for foundation and pavement support.

Specific grading and stormwater plans had not been finalized at the time this report was prepared. However, we understand that stormwater from the proposed development may be directed into on-site infiltration systems, if feasible. The City of Mercer Island uses the Department of Ecology's 2019 Stormwater Management for Western Washington (2019 SWMMWW) to determine the design infiltration rate and overall system sizing. According to this manual and the City of Mercer Island requirements, system sizing for the proposed infiltration system can be determined by on-site infiltration testing consisting of the Small Pilot Infiltration Test (PIT). As a part of our evaluation, we performed two on-site small-scale PITs. Based on our on-site testing and observations, it is our opinion that stormwater infiltration within the site is not feasible within the native glacial till soils encountered at depth within the proposed development area.

In the attached report, we have also provided general recommendations for site grading, slabs-on-grade, structural fill placement, erosion control, and drainage. We should be retained to review and comment on final development plans and observe the earthwork phase of construction. We also recommend that NGA 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 construction differ from those anticipated, and to evaluate whether or not earthwork and foundation installation activities comply with contract plans and specifications.

It has been a pleasure 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 Engineer

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INTRODUCTION

This report presents the results of our geotechnical engineering investigation and evaluation of the planned residential development project on **Mercer Island, Washington.** The project site is located at the properties with addresses of **4332 – 86th Avenue SE** (northern property) and **43XX – 86th Avenue SE** (southern property), 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.

The northern property is currently occupied by a single-family residence within the central portion of the property. The southern property is currently vacant. The ground surface within the properties is generally relatively level to gently sloping down from the east to the west. We understand that the proposed development will include removal of the existing site structure within the northern property and construction of a new single-family residence within the central portion of both the northern and southern properties. Final development and grading plans had not been prepared at the time this report was issued. Final stormwater plans have also not been developed; however, we understand that stormwater may be directed to on-site infiltration systems, if feasible. The existing site layout is shown on the Schematic Site Plan in Figure 2.

SCOPE

The purpose of this study is to explore and characterize the site surface and subsurface conditions and provide general recommendations for site development. Specifically, our scope of services included the following:

1. Review available soil and geologic maps of the area.
2. Explore the subsurface soil and groundwater conditions within the site with trackhoe-excavated test pits. Trackhoe was provided by NGA.
3. Perform laboratory grain-size sieve analysis on soil samples, if necessary.
4. Provide recommendations for earthwork, foundation support, and slabs-on-grade.
5. Provide recommendations for temporary and permanent slopes.
6. Provide recommendations for pavement subgrade.
7. Provide recommendations for site drainage and erosion control.
8. Provide our opinion on the feasibility of infiltration for the onsite soils.
9. Provide recommendations for infiltration system installation.

10. Provide long-term design infiltration rates based on on-site Pilot Infiltration Testing (PIT) per the 2019 SWMMWW. One test was performed within each property for a total of two tests.
11. Document the results of our findings, conclusions, and recommendations a written geotechnical report.

SITE CONDITIONS

Surface Conditions

The sites consist of rectangular-shaped parcels each covering approximately 0.30 acres. The northern property is currently occupied by a single-family residence within the central portion of the property. The southern property is currently vacant. The ground surface within the properties is generally relatively level to gently sloping down from the east to the west. The properties are generally vegetated with grass, landscaping plants and young to mature trees. The properties are bordered to the north, east and south by existing residential properties, and to the west by 86th Avenue SE. We did not observe surface water within the site during our site visit on May 31, 2024.

Subsurface Conditions

Geology: The geologic units for this area are mapped on the Geologic Map of Mercer Island, Washington by Kathy G. Troost and Aaron P. Wisher, (University of Washington, 2006). The project site is mapped as Quaternary Vashon till (Q_{vt}). Vashon till is typically a mixture of relatively equal parts of silt, sand, and gravel, deposited during the last glaciation period approximately 12,000 to 15,000 years ago. Our explorations generally encountered surficial topsoil and/or undocumented fill soils underlain silty sand with gravel consistent with the description of the Vashon Till at depth throughout the site.

Explorations: The subsurface conditions within the site were explored on May 31, 2024 by excavating seven test pit explorations extending to depths in the range of 4.5 to 8.0 feet below the existing ground surface, two of which were utilized as an infiltration test pit. The approximate locations of our explorations are shown on the Schematic 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 test pits. 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. We present a brief summary of the subsurface conditions in the following paragraph. For a detailed description of the subsurface conditions, the logs of our explorations should be reviewed.

At the surface of all of our explorations, we encountered 1.0 to 3.0 feet of surficial topsoil and/or undocumented fill. Underlying the topsoil and/or undocumented fill in each of our explorations, we encountered medium dense to dense, brown-gray to gray, silty, fine to medium sand with gravel, which we interpreted to as native glacial till soils. All of the test pits were terminated within the native glacial till deposits at depths in the range of 4.5 to 8.0 feet below the existing ground surface.

Hydrogeologic Conditions

We did not encounter groundwater within our explorations. If groundwater is encountered during construction, we would interpret this groundwater seepage to be perched groundwater. Perched water occurs when surface water infiltrates through less dense, more permeable soils and accumulates on top of a relatively low permeability material. 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 2021 International Building Code (IBC) and the ASCE 7-16 for seismic site classification for this project. Since competent glacial till soils were encountered at depth within the subject site, the site conditions best fit the IBC description for Site Class D.

Table 1 below provides seismic design parameters for the site that are in conformance with the 2021 IBC, which specifies a design earthquake having a two percent probability of occurrence in 50 years (return interval of 2,475 years), and the 2014 USGS seismic hazard maps.

Table 1 – ASCE 7-16 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	1.423	0.495	1.0	Null	0.949	Null

The spectral response accelerations were obtained from the ASCE Hazard Tool website (2014 data) for the project address.

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 medium dense or better glacial till deposits interpreted to underlie the site and nearby vicinity have a low potential for liquefaction or amplification of ground motion, due to their high internal strength, grain size distribution, and lack of shallow groundwater conditions.

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 King County Area, Washington, by the Natural Resources Conservation Service (NRCS), was reviewed to determine the erosion hazard of the on-site soils. The surface soils for this site are mapped as Arents, Alderwood material, 6 to 15 percent slopes. The erosion hazard for these materials 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.

CONCLUSIONS AND RECOMMENDATIONS

General

It is our opinion that the planned development within the site is generally feasible from a geotechnical standpoint. Our explorations indicated that the site was generally underlain by competent native glacial till soils at depth within the site. The native glacial bearing soils encountered at depth should provide adequate support for foundation, slab, and pavement loads. We recommend that the planned structures be designed utilizing conventional shallow foundations. Footings should extend through any loose soil or undocumented fill soils and be founded on the underlying medium dense or better native glacial till soils, or structural fill extending to these soils. The medium dense or better native glacial bearing soils should typically be encountered approximately 2.0 to 3.0 feet below the existing surface, based on our explorations. We should note that localized areas of deeper unsuitable soils and/or undocumented fill could be encountered at this site. This condition would require additional excavations in foundation, slab, and pavement areas to remove the unsuitable soils.

Based on the results of our on-site infiltration testing and soil explorations throughout the site, it is our opinion that the onsite native glacial till soils encountered at depth within the proposed development area are not conducive for stormwater infiltration methods. This is further discussed in the **Site Drainage** section of this report.

The surficial soils encountered on this site are considered moisture-sensitive and will disturb easily when wet. We recommend that construction take place during the drier summer months, if possible. If construction is to take place during wet weather, the soils may disturb, and additional expenses and delays may be expected due to the wet conditions. Additional expenses could include the need for placing a blanket of rock spalls to protect exposed subgrades and construction traffic areas. Some of the native on-site soils may be suitable for use as structural fill depending on the moisture content of the soil during construction. NGA should be retained to determine if the on-site soils can be used as structural fill material during construction.

Erosion Control

The erosion hazard for the on-site soils is interpreted to be slight for exposed soils, 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 or disturbed areas. Silt fences and/or straw bales should be erected to prevent muddy water from leaving the site. Disturbed areas should be planted as soon as practical, and the vegetation should be maintained until it is established. The erosion potential of areas not stripped of vegetation should be low.

Site Preparation and Grading

After erosion control measures are implemented, site preparation should consist of stripping the topsoil, undocumented fill and loose soils from foundation, slab, pavement areas, and other structural areas, to expose medium dense or better native bearing glacial soils. The stripped soil should be removed from the site or stockpiled for later use as a landscaping fill. Based on our observations, we anticipate stripping depths of 2.0 to 3.0 feet, depending on the specific locations. However, additional stripping may be required if areas of deeper undocumented fill and/or loose soil are encountered in unexplored areas of the site.

After site stripping, if the exposed subgrade is deemed loose, it should be compacted to a non-yielding condition and then proof-rolled with a heavy rubber-tired piece of equipment. Areas observed to pump or weave during the proof-roll test 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 stripping and 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.

The site soils are considered to be moisture-sensitive and will disturb easily when wet. We recommend that construction take place during the drier summer months if possible. However, if construction takes place during the wet season, 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, construction traffic areas, and paved areas prior to placing structural fill. Wet weather grading will also require additional erosion control and site drainage measures. Some of the native on-site soils may be suitable for use as structural fill, depending on the moisture content of the soil at the time of construction. 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 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 at all times as indicated in OSHA guidelines for 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 upper surficial and/or undocumented fill soils be no steeper than 1.5 Horizontal to 1 Vertical (1.5H:1V). Temporary cuts in the competent native glacial till soils at depth should be no steeper than 1H:1V. We recommend that temporary cut slope excavations be performed as to not disturb the 1H:1V inclination extending down from the base of the neighboring residence foundations to the bottom of the temporary cuts. If temporary cut excavations are not able to achieve the recommended inclinations, we recommend that the cuts be temporarily shored with either an Ultra Block shoring wall or a soldier pile shoring wall as discussed in the **Temporary Ultra Block/Ecology Block Shoring Wall and Soldier Pile Shoring Wall** subsections of this report, respectively. Any temporary cut excavations to be located within a 1H:1V inclination from the neighboring residence foundations should be supported entirely with a soldier pile shoring wall. If a soldier pile shoring wall is utilized to support temporary excavations within this property, we recommend that the soldier piles be installed in drilled shafts filled with concrete due to the relatively dense, compact nature of the native glacial till soils encountered at depth. Due to the soil conditions, we would anticipate that installation of the beams via driven impact methods may prove difficult and adequate embedment depths may not be achieved.

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. Permanent cut and fill slopes should be no steeper than 2H:1V. However, flatter inclinations may be required in areas where loose soils are encountered. Permanent slopes should be vegetated, and the vegetative cover maintained until established.

If planned temporary excavations and shoring systems are to be located within close proximity to the neighboring properties and structures, we recommend that settlement monitoring survey points be installed on the surrounding structures during construction and monitored at least once a week until it is confirmed that no movement is occurring. We should be retained to discuss wall and surrounding structure monitoring plans as project plans are finalized. Additional photographic and visual pre-existing surveys of the project vicinity and neighboring structures prior to construction activities should also be performed to document existing conditions within the vicinity of the property.

Temporary Shoring Walls

General: Specific grading plans were not available at the time this report was prepared. However, we anticipate that tall cuts and retaining walls will likely be needed for the planned structures depending on the final grades. Due to the proposed depth of the anticipated cuts and tight site constraints from existing properties, we anticipate that temporary/permanent shoring walls may be needed to support the cut excavations for structure construction.

Temporary Ultra Block/Ecology Block Shoring Wall: If temporary cut excavations as recommended above cannot be accommodated because of close proximity of the excavations to the property-lines, we recommend that the cuts could be temporarily shored with an Ultra Block and/or ecology block shoring wall. Ultra Blocks typically consist of 2.5-ft by 2.5ft by 5-ft concrete blocks while ecology blocks typically measure 2.0-ft by 2.0-ft by 6.0-ft. The total height of the temporary Ultra Block shoring wall should not exceed 7.5 feet or three blocks tall. The total height of a temporary ecology block shoring wall should not exceed 8.0 feet or four blocks tall. The Ultra Block or ecology block shoring wall should be constructed with a slight 1H:10V inclination back towards the cut and should be supported directly on competent native bearing glacial soils. All vertical joints between blocks should be staggered at each row. Temporary cuts above the temporary Ultra Block shoring wall could be sloped back away from the wall at 1.5H:1V or flatter inclination and should be no greater than four feet in overall height. If ecology blocks are utilized, the ground surface should be level and no additional surcharges from traffic, temporary slopes, building or machinery loads should be located within a 1H:1V inclination extending back from the base of the blocks. All exposed soils above the shoring wall should be protected from erosion. The Ultra Block and ecology block walls are considered only a temporary excavation support measure and should be buried or removed, and permanent support established by the building retaining walls. Schematic wall details utilizing both ecology blocks and Ultra Blocks are shown on Figures 6 and 7, respectively.

Where space is limited for the Ultra Block or ecology block shoring wall between the proposed residence foundation and property lines, the proposed residence foundation can be designed with L-shaped foundations. The block wall materials should be readily available on site prior to beginning excavation of the temporary cuts to be shored. The cut should be sloped or benched as needed for temporary stability, and wall construction should be accomplished immediately after excavation of the temporary slopes. Safe worker access should be maintained at all times during wall construction. We recommend that the construction of the temporary block walls be performed in short segments no greater than 15 feet in length and be entirely completed using machinery. No personnel should be present between the wall and the cut at any time. Gaps between the wall and cuts should be backfilled with clean crushed rock.

Soldier Pile Shoring Walls

General: A soldier pile shoring wall could also be utilized to support cut excavations around the proposed structures. A soldier pile wall typically consists of a series of steel H-beams placed vertically at a certain spacing between H-beams (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 equal to 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 Soldier Pile Wall Detail in Figure 8.

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. An additional uniform surcharge of 8H should be applied to the wall design to account for seismic loading, if the shoring walls are intended to provide permanent support; **H** in this case, is the exposed height of the wall. These loads should be applied across the pile spacing above the excavation line. These loads can be resisted by a passive pressure of 250 pcf on the below grade very stiff or better soils encountered at depth. 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 two feet of pile embedment should be neglected when calculating the passive resistance for the permanent condition.

Also, for the permanent condition, the below-grade portion of the wall should be no less than 1.5 times the wall stick-up height. The above loads 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 the active pressure could be applied for the purpose of designing the wall lagging. The above pressures assume that the on-site soils retained by the shoring wall are not significantly disturbed and that hydrostatic forces are not allowed to build up behind the wall. These values do not include the effects of surcharges other than what is described above.

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 any potential water seepage buildup to flow 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 survey monitoring program. Also, surrounding structures should be monitored for any adverse effects resulting from shoring wall installation.

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 be feasible for installing the soldier piles in the on-site soils but recommend that the shoring contractor have the capability of casing the holes as sloughing and/or water seepage may 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 pumping grout. Grout should be readily available on site at the time the holes are drilled. 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, or other additives, may also be placed in 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. We should be retained to monitor on-site activities during the shoring wall installation on a full-time basis.

Foundations

Conventional shallow spread foundations should be placed on medium dense or better native glacial till soils or be supported on structural fill or rock spalls extending to those soils. Native medium dense or better glacial bearing soils should be encountered approximately 2.0 to 3.0 feet below the existing 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 native bearing soil. The over-excavation may be filled with structural fill, or the footings may be extended down to the competent native soils. If footings are supported on structural fill, the fill zone should extend outside the edges of the footing a distance equal to half of the depth of the over-excavation below the bottom of the footing. In case of excessive undocumented fill thickness, deep foundation options may be required. NGA is available to work with the structural engineer to explore those options.

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 bearing pressure of not more than 2,500 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 bearing 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 1-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 250 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. 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.

Retaining Walls

Specific grading plans for this project were not available at the time this report was prepared, but retaining walls may be incorporated into project plans. In general, the lateral pressure acting on 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, and the inclination of the backfill. 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. A seismic design loading of 8H should also be included in the wall design, where “**H**” represents the total height of the wall.

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 height of the wall, and do not account for surcharge loads. Additional lateral earth pressures should be considered for surcharge loads acting adjacent to walls and within a distance equal to the height of the wall. This would include the effects of surcharges such as traffic loads, floor slab loads, slopes, or other surface loads. 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 **Foundations** 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 buildup of excess lateral soil pressures due to over-compaction of the wall backfill. This can be accomplished by placing wall backfill in 8-inch loose lifts and compacting the backfill with small, hand-operated compactors within a distance behind the wall equal to at least 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 systems 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 observe installation of the drainage systems.

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 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 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). Some of the more granular native on-site soils may be suitable for use as structural fill, but this will be highly dependent on the moisture content of these soils at the time of construction. We should be retained to evaluate all proposed structural fill material prior to placement.

Fill Placement: Following subgrade preparation, placement of structural fill may proceed. All filling 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 and should be tested.

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. 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 may be used to protect the vapor barrier membrane and to aid in curing the concrete.

Pavements

Pavement subgrade preparation and structural filling where required, should be completed as recommended in the **Site Preparation and Grading** and **Structural Fill** subsections of this report. The pavement subgrade should be proof-rolled with a heavy, rubber-tired piece of equipment, to identify soft or yielding areas that require repair. The pavement section should be underlain by a stable subgrade. We should be retained to observe the proof-rolling and recommend repairs prior to placement of the asphalt or hard surfaces.

Utilities

We recommend that underground utilities be bedded with a minimum six inches of pea gravel prior to backfilling the trench with on-site or imported material. Trenches within settlement sensitive areas should be compacted to 95 percent of the modified proctor as described in the Structural Fill subsection of this report. Trench backfill should be compacted to a minimum of 95 percent of the modified proctor maximum dry density. Trenches located in non-structural areas and five feet below roadway subgrade should be compacted to a minimum 90 percent of the maximum dry density. The trench backfill compaction should be tested.

Site Drainage

Infiltration: The 2019 Stormwater Management Manual for Western Washington (2019 SWMMWW) was utilized to determine the appropriate sizing of the proposed on-site infiltration systems. In accordance with this manual, on-site infiltration testing consisting of the Small-Scale Pilot Infiltration Test (Small PIT) was used to determine the long-term design infiltration rates. We conducted two Small PITs within Infiltration Pit 1 and Infiltration Pit 2 within the native glacial till soils as shown on the Schematic Site Plan in Figure 2. Infiltration Pits 1 and 2 measured 4.0-feet long by 3.0-feet wide by 4.5-feet deep. The holes were filled with approximately 12-inches of water and this level was maintained for six hours for the pre-soak period. After the 6-hour soaking period was completed, the water level was maintained at approximately 12-inches and the water flow rate into the hole was monitored with a Great Plains Industries (GPI) TM 050 water flow meter for one hour for the steady-state period of the test.

Infiltration Pit 1: The most conservative flow rate obtained from the steady state portion of the test within Infiltration Test Pit 1 was 0.024 gallons per minute (1.44 gallons per hour), which equates to an approximate infiltration rate of 0.19 inches per hour. The water was shut off after the steady-state period and the water level within the pit was monitored every 15 minutes for one hour. The water level within the pit had dropped 0.125 inches in 60 minutes, resulting in an infiltration rate of 0.125 inches per hour.

Infiltration Pit 2: After the six-hour pre-soak period, we observed that the water level remained at 12 inches with no additional water being added. After the one-hour steady-state portion of the test, the water level did not change resulting in a measured infiltration rate of zero inches per hour. As a result, the infiltration testing was concluded.

Based on the results of both of the small-PIT's and the relatively silty compact nature of the native glacial till soils that underlie the site, it is our opinion that the onsite native glacial till soils are not conducive for stormwater infiltration systems. We recommend that all stormwater generated from proposed structures and other hard surfaces be directed to on-site detention systems and ultimately into an approved point of discharge likely found within the adjacent roadways.

Surface Drainage: The finished ground surface should be graded such that stormwater is directed to an approved stormwater collection system. Water should not be allowed to stand in any areas where footings, slabs, or pavements are to be constructed. Final site grades should allow for drainage away from the residences. We suggest that the finished ground be sloped downward at a minimum gradient of three percent, for a distance of at least 10 feet away from the residences. Surface water should be collected by permanent catch basins and drain lines and be discharged into an approved 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 out and routed into a permanent storm drain. We recommend the use of footing drains around the structures. Footing drains should be installed at least one foot below 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. Pea gravel is an acceptable drain material. The free-draining material should extend up the wall to one foot below the finished surface. The top foot of backfill should consist of impermeable soil placed over plastic sheeting or building paper to minimize surface water or fines migration into the footing drain. Footing drains should discharge into tightlines leading to an approved 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.

CONSTRUCTION MONITORING

We should be retained to provide construction monitoring services during the earthwork phase of the project to evaluate subgrade conditions, temporary cut conditions, fill compaction, and drainage system installation.

USE OF THIS REPORT

NGA has prepared this report for **Ms. Heather Cochran with Cade Hill Homes,** and associated agents, for use in the planning and design of the development 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 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 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.

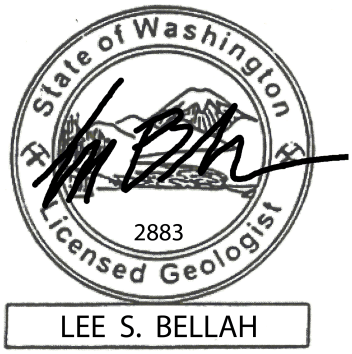
Within the limitations of scope, schedule, and budget, our services have been performed in accordance with generally accepted geotechnical engineering practices in effect in this area at the time this report was prepared. No other warranty, expressed or implied, is made. Our observations, findings, and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

o-o-o

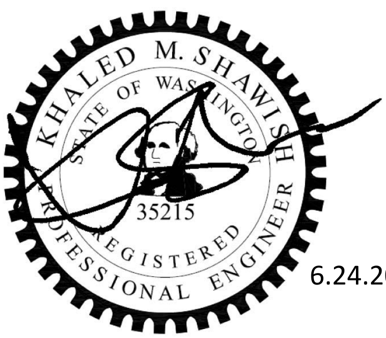
It has been a pleasure to provide service to you on this project. If you have any questions or require further information, please call.

Sincerely,

NELSON GEOTECHNICAL ASSOCIATES, INC.



Lee S. Bellah, LG
Senior Geologist



6.24.2024

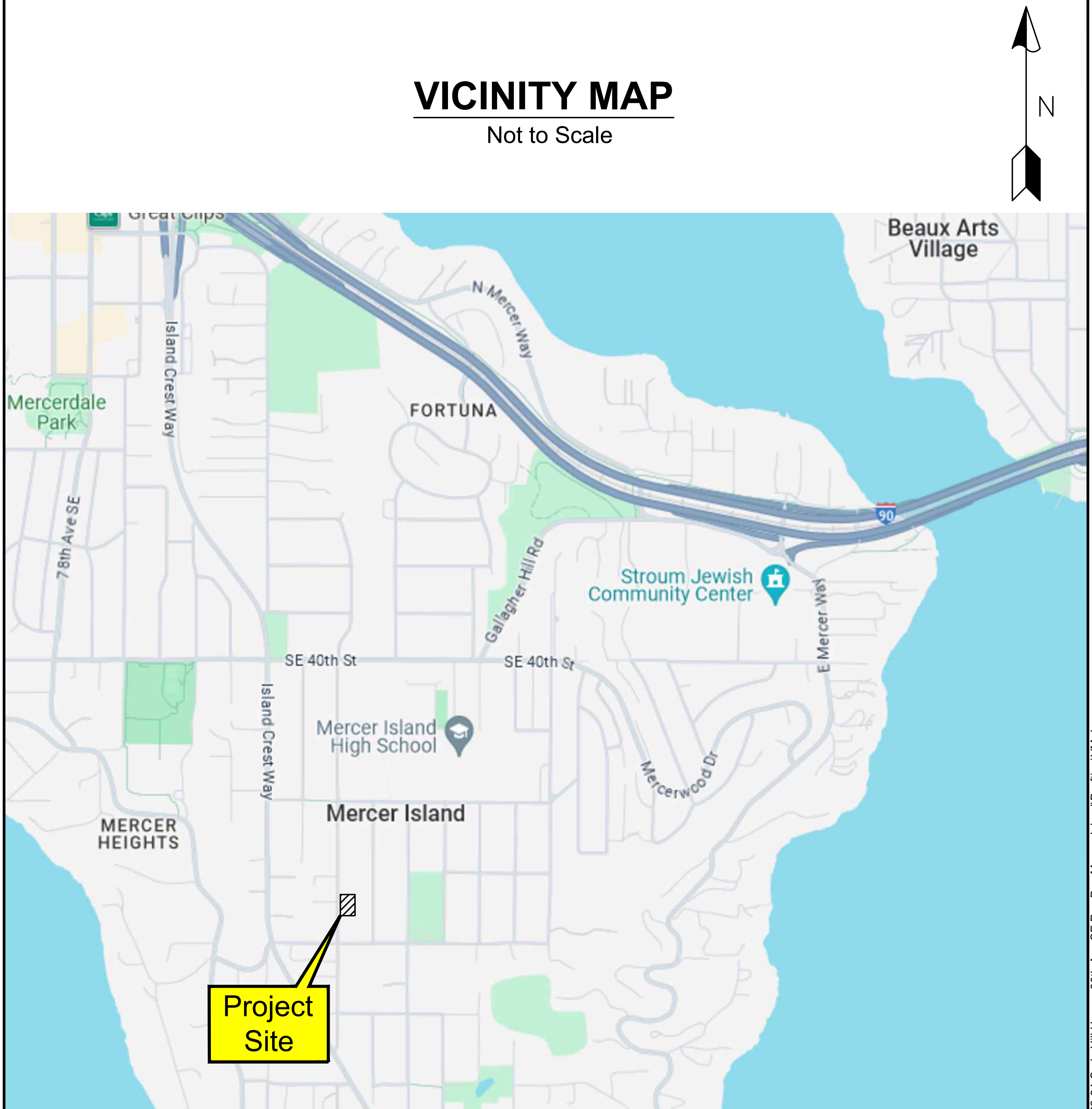
Khaled M. Shawish, PE
Principal

SAM:LSB:KMS:dy

Eight Figures Attached

VICINITY MAP

Not to Scale



Mercer Island, WA

Project Number 1518224	Cade Hill Homes 86th Avenue SE Residential Development Vicinity Map	NELSON GEOTECHNICAL ASSOCIATES, INC 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	No.	Date	Revision	By	CK
Figure 1			1	6/7/24	Original	ABT	LSB

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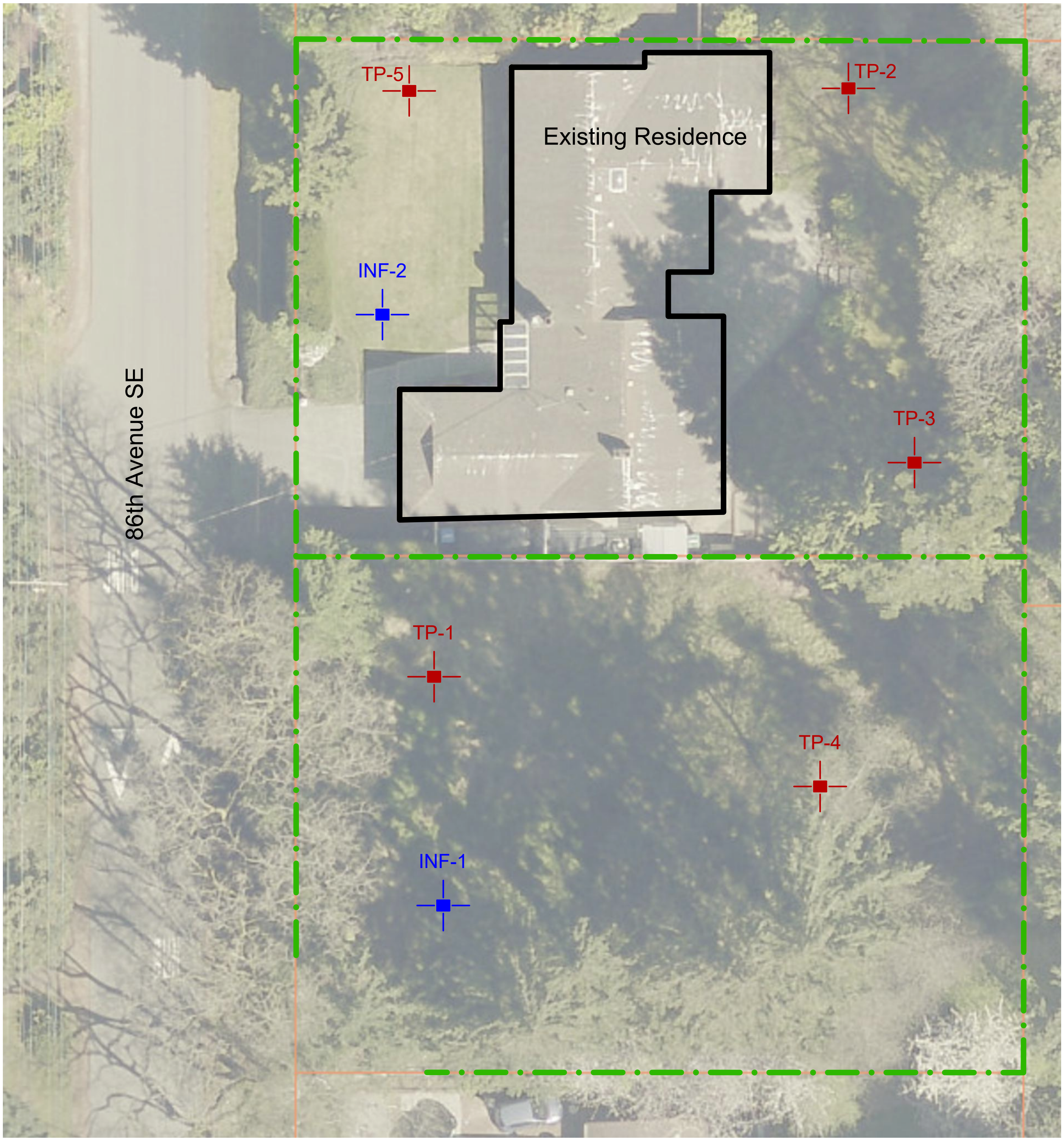
Site Plan

N



LEGEND

Property line

TP-1 Number and approximate location of test pit

INF-1 Number and approximate location of infiltration test pit

0 30 60

Approximate Scale: 1 inch = 30 feet

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

Project Number	Cade Hill Homes	NELSON GEOTECHNICAL ASSOCIATES, INC	No.	Date	Revision	By	CK
1518224	86th Avenue SE	Woodinville Office, 17311-135th Ave. NE, A-500, Woodinville, WA 98072, (425) 486-1669 / Fax: 481-2510	1	6/7/24	Original	ABT	LSB
Figure 2	Residential Development Site Plan	Wenatchee Office, 105 Palouse St., Wenatchee, WA 98801, (509) 665-7696 / Fax: 665-7692; www.nelsongeotech.com					

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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 1518224	Cade Hill Homes 86th Avenue SE Residential Development Soil Classification Chart	NELSON GEOTECHNICAL ASSOCIATES, INC	No.	Date	Revision	By	CK
Figure 3			1	6/7/24	Original	ABT	LSB

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LOG OF EXPLORATION

DEPTH (FEET)	USCS	SOIL DESCRIPTION
TEST PIT ONE		
0.0 – 1.0		GRASS / **TOPSOIL**
1.0 – 3.5	SM	BROWN-GRAY, SILTY, FINE TO MEDIUM SAND WITH GRAVEL (MEDIUM DENSE TO DENSE, MOIST)
3.5 – 8.0	SM	GRAY, SILTY, FINE TO MEDIUM SAND WITH GRAVEL (DENSE TO VERY DENSE, MOIST)
		SAMPLE WAS COLLECTED AT 4.0 FEET GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 8.0 FEET ON 5/31/24
TEST PIT TWO		
0.0 – 1.0		GRASS / **TOPSOIL**
1.0 – 2.5	SM	BROWN-GRAY, SILTY, FINE TO MEDIUM SAND WITH GRAVEL (MEDIUM DENSE TO DENSE, MOIST)
2.5 – 7.0	SM	GRAY, SILTY, FINE TO MEDIUM SAND WITH GRAVEL (DENSE TO VERY DENSE, MOIST)
		SAMPLE WAS COLLECTED 3.5 FEET GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 7.0 FEET ON 5/31/24
TEST PIT THREE		
0.0 – 1.0		GRASS / **TOPSOIL**
1.0 – 2.5	SM	BROWN-GRAY, SILTY, FINE TO MEDIUM SAND WITH GRAVEL (MEDIUM DENSE, MOIST)
2.5 – 7.0	SM	GRAY, SILTY, FINE TO MEDIUM SAND WITH GRAVEL (DENSE TO VERY 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 5/31/24
TEST PIT FOUR		
0.0 – 1.0		GRASS / **TOPSOIL**
1.0 – 2.5	SM	BROWN-GRAY, SILTY, FINE TO MEDIUM SAND WITH GRAVEL (MEDIUM DENSE, MOIST)
2.5 – 6.0	SM	GRAY, SILTY, FINE TO MEDIUM SAND WITH GRAVEL (DENSE TO VERY DENSE, MOIST)
		SAMPLES WAS COLLECTED AT 3.5 FEET GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 6.0 FEET ON 5/31/24
TEST PIT FIVE		
0.0 – 3.0		DARK BROWN, SILTY, FINE TO MEDIUM SAND WITH GRAVEL AND ORGANICS (LOOSE, MOIST) **FILL**
3.0 – 5.0	SM	RED-BROWN, SILTY, FINE TO MEDIUM SAND WITH GRAVEL (MEDIUM DENSE, MOIST)
5.0 – 8.0	SM	GRAY, SILTY, FINE TO MEDIUM SAND WITH GRAVEL (DENSE TO VERY DENSE, MOIST)
		SAMPLES WAS COLLECTED AT 5.5 FEET GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 8.0 FEET ON 5/31/24

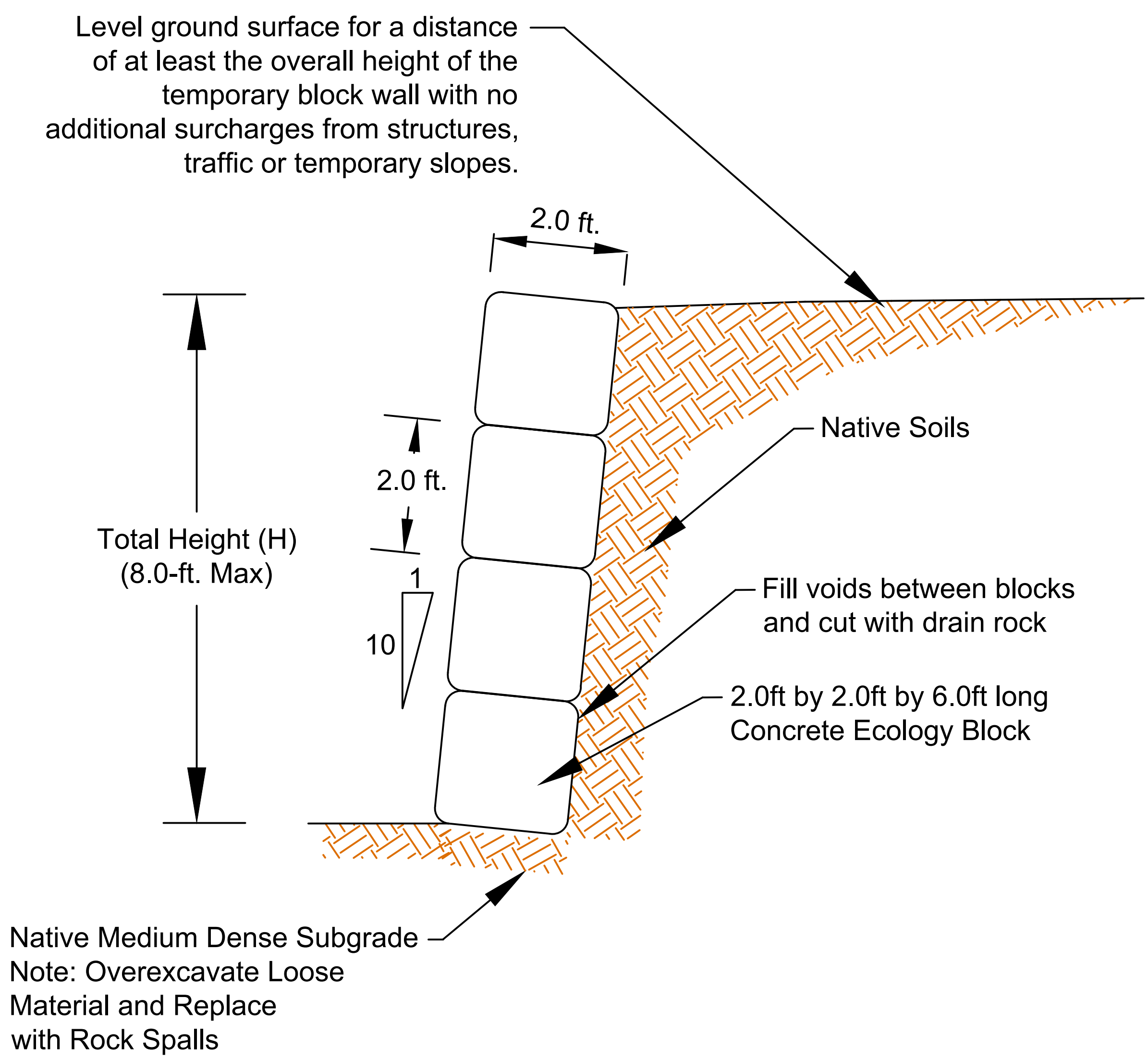
LOG OF EXPLORATION

DEPTH (FEET)	USCS	SOIL DESCRIPTION
INFILTRATION TEST PIT ONE		
0.0 – 2.0		**TOPSOIL / <u>FILL</u>**
2.0 – 3.5	SM	RED-BROWN, SILTY, FINE TO MEDIUM SAND WITH GRAVEL (MEDIUM DENSE, MOIST)
3.5 – 4.5	SM	GRAY, SILTY, FINE TO MEDIUM SAND WITH GRAVEL (DENSE TO VERY DENSE, MOIST)
		SAMPLES WERE NOT COLLECTED GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED INFILTRATION TEST PIT WAS COMPLETED AT 4.5 FEET ON 5/31/24
INFILTRATION TEST PIT TWO		
0.0 – 2.5		**TOPSOIL / <u>FILL</u>**
2.5 – 3.5	SM	RED-BROWN, SILTY, FINE TO MEDIUM SAND WITH GRAVEL (MEDIUM DENSE, MOIST)
3.5 – 4.5	SM	GRAY, SILTY, FINE TO MEDIUM SAND WITH GRAVEL (DENSE TO VERY DENSE, MOIST)
		SAMPLES WERE NOT COLLECTED GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED INFILTRATION TEST PIT WAS COMPLETED AT 4.5 FEET ON 5/31/24

Temporary Ecology Block Shoring Wall Detail

(Not to Scale)

N



Project Number	Cade Hill Homes 86th Avenue SE Residential Development Temporary Ecology Block Shoring Wall Detail	NELSON GEOTECHNICAL ASSOCIATES, INC
1518224		
Figure 6		

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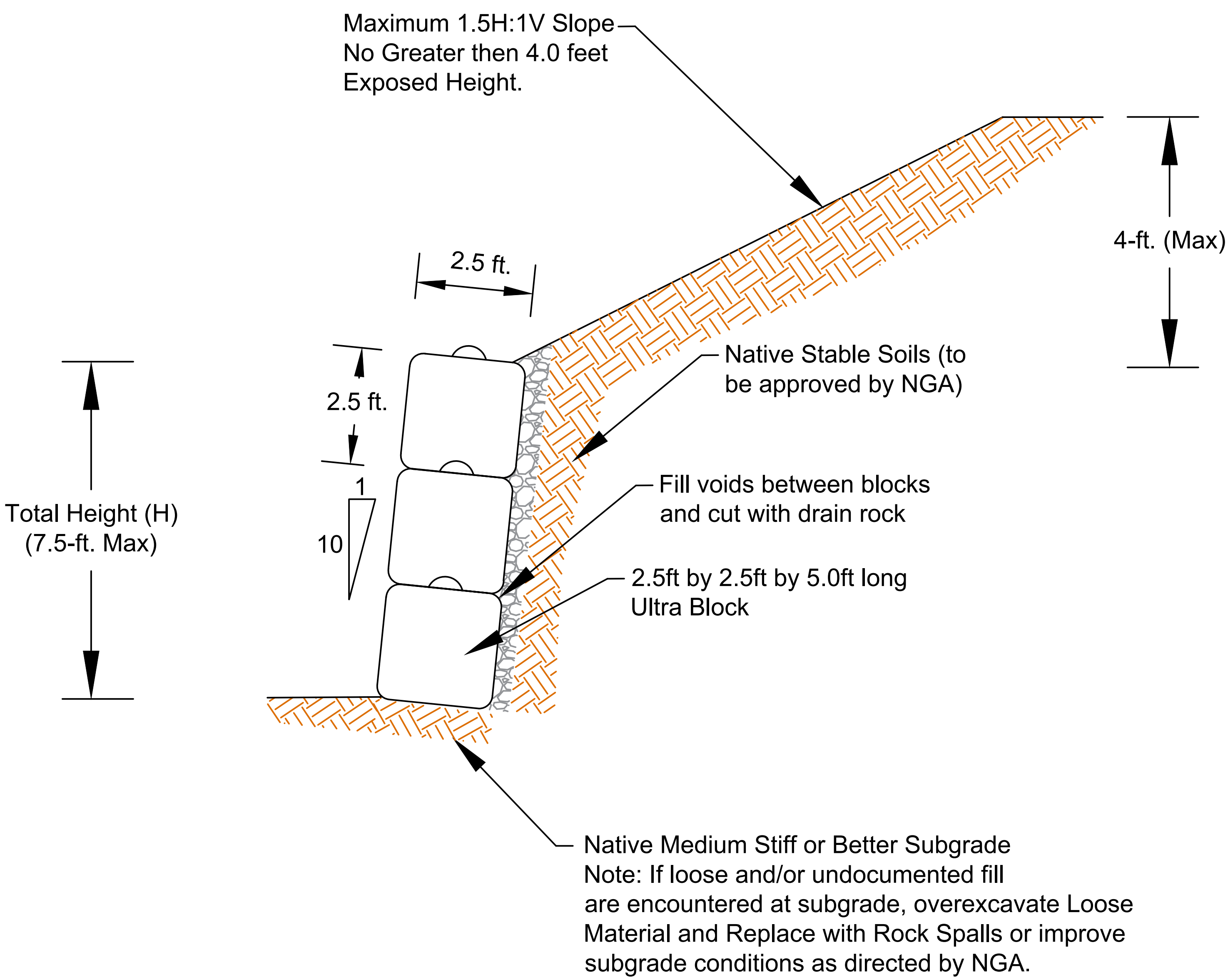
Temporary Ultra Block Shoring Wall Detail

(Not to Scale)



Project Number 1518224	Cade Hill Homes 86th Avenue SE Residential Development Temporary Ultra Block Shoring Wall Detail	NELSON GEOTECHNICAL ASSOCIATES, INC 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
Figure 7		

No.	Date	Revision	By	CK
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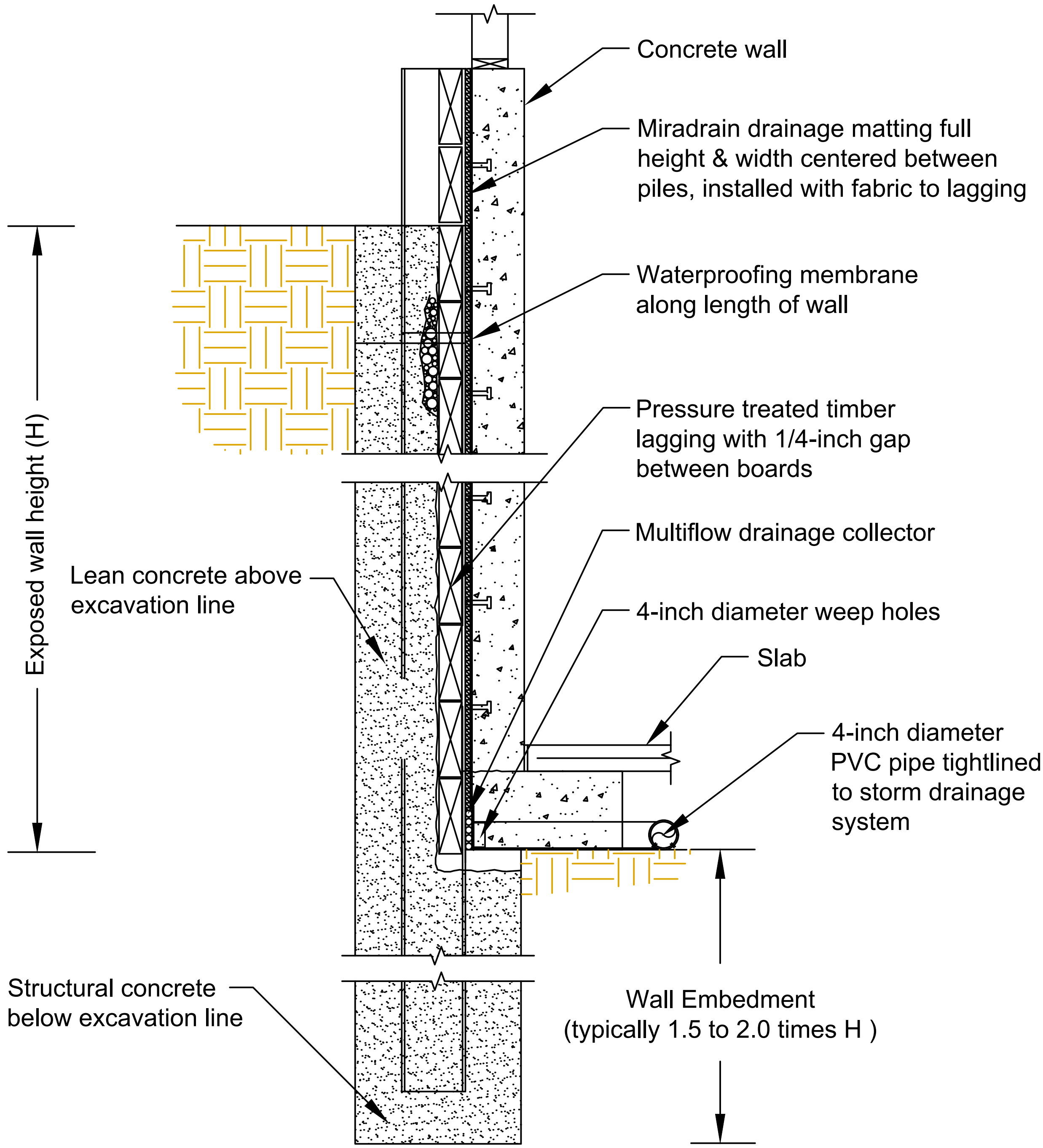
Conceptual Soldier Pile Wall Detail

NOT FOR CONSTRUCTION USE



NOT TO SCALE

Project Number 1518224	Cade Hill Homes 86th Avenue SE Residential Development Soldier Pile Wall Detail	NGA NELSON GEOTECHNICAL ASSOCIATES, INC 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
Figure 8		

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