NELSON GEOTECHNICAL ASSOCIATES, INC.

17311-135th Ave. N.E. Suite A-500
Woodinville, WA 98072
(425) 486-1669
www.nelsongeotech.com

CITY OF MUKILTEO

Received by Email
2/18/21

December 22, 2020

Mr. Bob Ford
Mr. Jeff Sherwood
Via Email: bob@3sqft.biz
jeff@sherwoodappraisal.com

Geotechnical Engineering Evaluation
2nd Street Mixed-Use Building Development
823 – 2nd Street
Mukilteo, Washington
NGA File No. 1165520

Dear Mr. Ford and Sherwood:

We are pleased to submit the attached report titled **"Geotechnical Engineering Evaluation – 2nd Street Mixed-Use Building Development – 823 – 2nd 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 the proposal signed by you on October 29, 2020.

The subject site is currently occupied by an existing commercial structure within the southern portion of the site. Topography within the property is generally relatively level with moderate to steep slopes that wrap around the northern and western edge of the property. An approximately 10-foot tall, 3-tiered block wall is located on the northern edge of the property. The ground surface within the site is mostly grass yard areas, bark covered areas, landscaping areas and scattered young to mature trees. The subject property is bordered to the east and south by existing residences, to the west by a commercial property and to the north by 2nd Street. We understand that the proposed development plan will include removal of the existing site structures and construction of a new four-story mixed-use building within the site, along with a below-grade level. As a result of the below-grade level of the building, we anticipate that temporary shoring walls will likely be required to support temporary excavations.

We monitored the drilling of two geotechnical soil borings at the site on November 20, 2020. Our explorations indicated that portions of the site were underlain by loose, silty, fine sand with saturated sandy interbeds, but competent, native, fine-grained glacial soils at depth within the site.

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 loose soils underlying the site will need to be improved or mitigated to adequately support the planned structure, depending on final design. We have made geotechnical recommendations assuming a ten-foot below-grade structure is proposed. In our opinion, the soils that underlie the proposed building location at ten feet below existing grade are not suitable to provide adequate support for foundation and slab loads utilizing conventional shallow foundations, without experiencing significant settlement and distress to the new structure. We recommend that the foundation elements of the new structure be founded on a "floating" rigid foundation system to minimize potential differential settlement, but not fully eliminate it.

Specific grading plans were not available at the time this report was prepared. However, if the proposed development is to include a below grade basement level, we anticipate that tall cuts and retaining walls will likely be needed for the planned structure. Due to the proposed depth of the anticipated cuts and tight site constraints from existing properties and roadways surrounding the proposed development area, we anticipate that temporary/permanent shoring walls will be needed to support cut excavations for structure construction. These walls can ultimately be incorporated into the building as permanent retaining walls, if feasible.

In the attached report, we have also provided general recommendations for site grading, slabs-on-grade, structural fill placement, foundations, retaining walls, soldier pile wall installation, 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 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 proposed 2nd Street Mixed-Use Building Development project in Mukilteo, Washington. The project site is located at 823 – 2nd Street, 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 site is approximately square in shape and covers an area of approximately 0.29 acres. It is currently occupied by an existing single-story institutional structure. Topography within the property is generally relatively level with moderate to steep slopes that wrap around the northern and western edge of the property. An approximately 10-foot tall, 3-tiered block wall is constructed on the northern edge of the property. We understand that the proposed development plan will include removal of the existing site structures and construction of a new four-story mixed-use building within the site, with one below-grade basement level. As a result of the below-grade level of the building, we anticipate that temporary shoring walls will likely be required to support temporary excavations. Final development and grading plans have not been prepared at the time this report was issued. The existing site layout is shown on the 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. A review of available soil and geologic maps of the area.
2. Exploring the subsurface soil and groundwater conditions within the vicinity of the proposed structure with two, 25-foot deep borings using a limited-access track-mounted drill rig. Drill rig was subcontracted by NGA.
3. Performing laboratory analysis on soil samples, as necessary.
4. Providing recommendations for foundation support and embedment, including seismic consideration.
5. Providing recommendations for earthwork and foundation support.

6. Providing recommendations for temporary and permanent slopes.
7. Providing recommendations for temporary and permanent shoring walls.
8. Providing recommendations for retaining walls.
9. Providing recommendations for slab and pavement subgrade preparation.
10. Providing recommendations for utility installation.
11. Providing recommendations for site drainage and erosion control.
12. Documenting the results of our findings, conclusions, and recommendations in a written geotechnical report.

SITE CONDITIONS

Surface Conditions

The site is approximately square in shape and covers an area of approximately 0.29 acres. It is bordered to the east and south by existing residences, to the west by a commercial property and to the north by 2nd Street It is currently occupied by an existing single-story institutional structure. An approximately 10-foot tall, 3-tiered block wall is constructed on the northern portion of the property, as shown on Cross Section A-A' in Figure 3. Topography within the property is generally relatively level with moderate to steep slopes that wrap around the northern and western edge of the property with gradients up to 40 degrees (84 percent grade), as shown on Cross Section B-B' in Figure 4. The ground surface within the site is mostly grass yard areas, bark covered areas, landscaping areas and scattered young to mature trees. We observed surface water seepage near the top of the steep slope on the southwestern property line during our visit on November 20, 2020.

Subsurface Conditions

Geology: The geologic units for this site are shown on The Distribution and Description of Geologic Units in the Mukilteo Quadrangle, by James P. Minard (USGS, 1982). The site is mapped as Pre-Fraser deposits of the Whidbey Formation (Qw). The Whidbey Formation is described as cross-bedded sand and including clay beds in the lower portion of the unit. Our explorations generally encountered interbedded silty sand and fine-to medium sand underlain by fine-grained soils at depth that we interpreted as Whidbey Formation.

Explorations: The subsurface conditions within the site were explored on November 20, 2020 by monitoring the drilling of two geotechnical borings to depths of 26.5 feet below the existing ground surface. 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.

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

At the surface of the boring on the eastern portion of the property, we encountered up to 16.5 feet of loose to medium dense, silty fine sand interbedded with saturated fine to medium sand. On the western portion of the site, we encountered up to 11.5 feet of silty fine to medium sand in a medium dense condition. Underlying these deposits in both explorations, we encountered medium dense or better fine to medium sand with a minor component of silt. Borehole One on the eastern portion of the site terminated within this distinctive bed, but it was underlain by blue-gray silt in Borehole Two on the western portion of the site, composing a total thickness up to 9.5 feet. Both boreholes were terminated within the native Whidbey Formation deposits at depths of 26.5 feet below the existing ground surface.

Hydrogeologic Conditions

Groundwater seepage was observed from near-surface soils and in saturated interbeds within Borehole One on the eastern portion of the site. Zones of wet soils within our explorations here indicate the potential for perched groundwater seepage. Perched water occurs when surface water infiltrates through less dense, more permeable soils and accumulates on top of relatively low permeability materials. 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 competent interglacial soils are inferred to underlie the site at depth, 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 2018 IBC, which specifies a design earthquake having a 2% 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	1.474	0.569	1.000	1.500	0.983	0.569

The spectral response accelerations were obtained from the USGS Earthquake Hazards Program Interpolated Probabilistic Ground Motion website (2008 data) for the project latitude and longitude.

The site is located approximately two miles to the northwest of a main strand of the Southern Whidbey Island Fault. The Southern Whidbey Island Fault Zone (SWIFZ) is an active, shallow region of seismicity within central Puget Sound stretching from the Strait of Juan de Fuca to North Bend. Information published in 2013 by the Washington State Department of Natural Resources suggests the SWIFZ last ruptured less than 2,700 years ago, and that the fault zone can produce a M7.5 earthquake. In our opinion, the risk of a surface fault rupture within this specific site is low, given available data.

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/stiff or better glacial deposits interpreted to underlie the site at depth have a 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 Natural Resources Conservation Service (NRCS) for the Snohomish County Area lists this area as Kitsap silt loam, 0 to 8 percent slopes. This soil is listed as having a slight erosion hazard where exposed. We would interpret this site as having a low to moderate erosion hazard where the surficial soils are exposed on slopes. It is our opinion that the erosion hazard for site soils should be low in areas where the site is not disturbed.

Landslide Hazard/Slope Stability

The criteria used for evaluation of landslide hazards include soil type, slope gradient, and groundwater conditions. The slope on the adjacent property to the west in proximity to the site is a geologically sensitive area as defined in the Mukilteo Municipal Code (MMC) Section 17.52A.020 (H).

As shown on Cross Section B-B’ in Figure 4, the steep slope on the property line steps down westward at gradients up to 40 degrees (84 percent grade) and has total vertical relief of approximately 12 feet. Additionally, groundwater seepage was observed emanating from the upper portion of the slope on our site visit on November 20, 2020. Minor rill erosion was noted below the seepage zone on the adjacent property. We did not observe evidence of significant slope instability within or within the immediate vicinity of the subject property during our investigation, such as deep-seated landsliding.

The core of the slope is inferred to consist primarily of medium dense or better native glacial soils. Relatively shallow sloughing failures as well as surficial erosion are natural processes and should be expected on slopes during extreme weather conditions. It is our opinion that while there is potential for erosion, soil creep, and shallow failures within the loose surficial soils on the steeper slopes, there is not a significant potential for deep-seated slope failures under current site conditions. Proper site grading and drainage as well as foundation placement as recommended in this report should help maintain current stability conditions.

CONCLUSIONS AND RECOMMENDATIONS

General

It is our opinion that the planned site development is feasible from a geotechnical standpoint, provided the loose soils found at intermediate depth below the site are considered in the design and construction of the structure. Due to the presence of loose soils at expected subgrade depths for the new structure, we recommend that the foundation elements of the structure be designed as a “floating” rigid foundation system to minimize potential differential settlement but not fully eliminate it. Alternatively, the structure could be founded on pin piles to reach past the loose soils and transfer structure loads to competent soils at depth. These two options are further discussed in the **Foundation Support** subsection of this report.

Due to the proposed below grade basement level of the proposed structure, we anticipate that tall cuts will be needed to allow construction of the below grade portions of the structure. These cuts may not be able to be safely sloped back due to site constraints such as neighboring roadways, structures, property lines and utilities. We recommend that the below grade cuts be shored with a soldier pile retaining wall. The soldier pile wall could be designed as a permanent wall and incorporated into the building, if feasible. We provided 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 shoring walls in the **Soldier Pile Shoring Wall** subsections 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 easily, and 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 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. This will depend on the moisture content of the soils at the time of 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 as slight to moderate 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 wattles should be erected to prevent muddy water from leaving the site. Disturbed areas should be covered or 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 removing loose soils, topsoil from foundation, slab, and pavement areas down to subgrade elevations, as recommended in the **Foundation Support** section. The stripped soil should be removed from the site or stockpiled for later use as a landscaping fill. If significant surface water flow is encountered during construction, this flow should be diverted around subgrade areas and the exposed subgrade maintained in a semi-dry condition.

Pavement areas at depths expected to expose loose conditions should be prepared by over-excavating a minimum of 12 inches of material below finished subgrade and covering the exposed surface with a rigid geogrid such as the Tensar TX160 or equivalent. The geogrid should then be covered with 12 inches of clean 1¼-inch crushed rock, upon which pavement surfacing could be placed. Even with this treatment, some settlement of hard surfaces should be expected due to the underlying fill soils. Once placed, the geogrid should never be cut or disturbed. Utilities within slab and pavement areas should be placed and installed as to not disturb the geogrid.

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 extremely 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 these areas.

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 is always the responsibility of the contractor to maintain safe slope configurations 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 soils be no steeper than 2 Horizontal to 1 Vertical (2H:1V). If significant groundwater seepage or surface water flow were encountered, we would expect that flatter inclinations would be necessary. If temporary cut excavations are not able to achieve the recommended inclinations, we recommend that the cuts be shored with a soldier pile shoring wall as discussed in the **Soldier Pile Shoring Wall** subsection of this report.

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.

Shoring Wall

General: We understand that below grade basement level will be included as a part of the overall development and will likely require shoring systems to complete the earthwork. We would recommend that the proposed shoring system consist of a soldier pile shoring 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 eight 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 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. A uniform surcharge of 8H (in psf) 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 200 pcf on the below grade 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 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. If a concrete wall is proposed to be cast on the face of the shoring wall, we recommend that a drainage composite such as a Miradrain mat be placed between the face of the shoring wall and the concrete wall. The drainage composite should be directed to flow into a drainage collector at the base of the shoring wall and ultimately to an approved discharge point.

If the proposed soldier piles are to support permanent vertical loads for the proposed structure, we could consult with the structural engineer to provide vertical load carrying capacities for the piles during the final design, if needed.

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 prove difficult to achieve for installing the soldier piles in the on-site soils, and therefore we recommend that the shoring contractor should have the capability of casing the holes as sloughing and/or water seepage if 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.

Tiebacks

General: If tiebacks are needed to support lateral loads, we recommend that these systems consist of drilled, grouted tieback anchors. If tiebacks are utilized to support lateral loads for the shoring wall, we anticipate these systems will likely extend into neighboring properties and easements. Permission to extend these systems onto the neighboring properties and/or easements should be obtained prior to finalizing plans utilizing tieback anchors. All nearby existing utilities and structures should also be fully understood prior to finalizing the tieback design.

We recommend that at least two of the anchors be performance tested to a minimum of 200 percent of the design loads to confirm design values. We recommend that measurements be made by the contractor in the field at the time of tieback installation to verify that tiebacks do not encounter any existing structures or underground utilities.

No-Load Zone: The anchor portion of all tiebacks must be located a sufficient distance behind the wall face to develop resistance within a stable soil mass. We recommend the anchorage be obtained behind an assumed no-load zone. The no-load zone is defined by a line extending horizontally from the base of the shoring wall back towards the cut a distance of six feet. This line should then extend up from the base elevation at an angle from the horizontal of 60 degrees. We recommend that we monitor soil conditions during anchor installation in order to evaluate adequate penetration into competent soils.

Soil Design Values: The tiebacks must terminate in native, competent soil interpreted to exist below the fill. For use in design of the anchors, we estimate an allowable grout to soil adhesion of 1,500 pounds per square foot (psf) be utilized for anchors terminated within the competent native glacial soils. This value should be verified through two performance tests prior to ordering the production anchors.

Tieback Installation and Testing: The contractor should be responsible for using equipment suited for the site conditions. We do not recommend the use of an open-hole method for the purpose of installing the tiebacks due to the potential for soil caving. Secondary grouting to increase soil adhesion may be used; however, if secondary grouting is used, the anchors should be tested using the methods outlined for the performance testing. All anchors should be installed at an approximate inclination of 15 to 20 degrees below horizontal.

Two anchors should be performance-tested to 200 percent of the anchor design capacity. The performance test should consist of cyclic loading in increments of 25 percent of the design load, as outlined in the Federal Highways Administration (FHA) report No. FHWA/RD-82/047. The test locations should be determined in the field by NGA, based on soil conditions observed during anchor installation. All other tiebacks should be proof tested to at least 130 percent of design capacity.

Foundation Support

General: At the time this report was prepared, final grading and development plans were not available. Medium dense soils were found within portions of the proposed structure footprints at varying depths. We recommend that the foundation elements of the structure either be founded on a floating rigid foundation or deep foundation system consisting of driven piles, depending on tolerance for settlement.

Floating Foundations: This system includes a reinforced concrete mat or raft foundation that rests on a stabilized subgrade. Due to the presence of the thick deposit of loose soils on the eastern portion of the site, we recommend that the foundations be supported on and underlain by at least 12 inches of 2- to 4-inch rock spalls topped with six inches of 1¼-inch crushed rock. The exposed subgrade should not experience compaction prior to placement of the rock spalls to avoid disturbing the soils. The spalls should be placed in two equal lifts and each lift compacted using a hoe-pack until no movement is experienced. The crushed rock lift should then be placed over the spalls and also compacted to a firm condition. The rock spalls and crushed rock pads should extend a minimum of 12 inches on all sides of the foundation supports. Isolated footings should be avoided.

Where excessively soft soils or debris are encountered in the excavations, these materials should be further excavated as directed by NGA and replaced with rock spalls.

The mat 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 excavations. For foundations constructed as outlined above, we recommend an allowable design bearing pressure of not more than 1,000 pounds per square foot (psf) be used for the design of footings founded on the prepared crushed rock base. The mat foundations should be analyzed for a subgrade modulus of 100 pci (pounds per cubic inch). 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.

Lateral loads may be resisted by friction on the base of the footing and passive pressure against the below-grade portion of the foundation. A coefficient of friction of 0.3 may be used to calculate the base friction and should be applied to the vertical dead load only. Passive resistance may be calculated based on a passive pressure of 100 pounds per cubic foot (PCF).

When utilizing our recommendations for foundation preparation, we note that long-term settlement of up to 2 to 4 inches should be anticipated, including up to 1.0 inch of differential settlement.

Pin Pile Foundation Support: If the aforementioned settlement values cannot be tolerated, the most feasible deep foundation support systems would consist of 4-inch diameter pin piles driven to refusal. A structural engineer should design the new foundation supports and determine the location of the supports based on the recommendations provided in this report. If the anticipated loads cannot be supported using shallow foundations or pin piles, drilled concrete piers could be used to support the structure. NGA is available to work with your design team to explore this option, if needed.

For 4-inch driven steel pipe piles, we recommend that they be driven using a tractor-mounted hydraulic hammer, with an energy rating of at least 1,100 foot-lb. For this pile and hammer size, we recommend a design capacity of eight tons for each pile driven to refusal. The refusal criterion for this pile and hammer size is defined as less than one inch of movement during 15 seconds of continuous driving at a rate of 550 blows per minute or higher. We recommend using galvanized schedule 40 pipe for the 4-inch pin piles. Maintaining these recommendations for minimum hammer size and refusal criteria is essential for obtaining a successful outcome.

Our explorations encountered undocumented fills within the planned development area. If large objects or debris are present within the fill, there is a possibility that this material may obstruct some piles at shallow depths. There should be contingencies in the budget and design for additional/relocated piles that may be obstructed by possible debris in the fill. It would be best to conduct a test pile to confirm driving conditions.

Final pile depths should be expected to vary somewhat and will depend on the nature of the underlying soils. The pin piles should be driven to the above refusal criterion in order to provide the recommended design capacity. This should be determined in the field by the contractor under the supervision of NGA. Piles that do not meet this minimum embedment criterion should be rejected, and replacement piles should be driven after consulting with the structural engineer on the new pile locations. Due to the relatively small slenderness ratio of pin piles, maintaining pin pile confinement and lateral support is essential to preventing pile buckling.

Vertically driven pin piles do not provide meaningful lateral capacity. Due to the rigid pile support, friction between the foundation and subgrade soil should not be considered as resisting lateral pressures on this structure. We recommend that all lateral loads be resisted on battered pin piles and/or passive resistance on the below-grade portions of the foundations. The upper foot of soil should be neglected when calculating the passive resistance. We recommend using an equivalent fluid density of 150 pcf for calculating the passive resistance.

If new slabs-on-grade are designed as structural slabs and supported with pin piles, we recommend that they be designed to span the distance between the foundations and grade beams. Distance between piles should be a minimufm of 3 feet in order to avoid pile group action. We should be retained to review final plans and to monitor installation of the pin piles during construction.

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, 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 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.

A uniform surcharge of 8H (in psf) 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. To account for seismic loading, a uniform surcharge of 8H should also be included in the wall design where “H” is 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 except as provided above. 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 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 and should be tested.

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 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

Slabs-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. Due to wet soil conditions, we recommend that the slab be underlain by a drainage system to remove accumulated water beneath the slab.

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 minimum of six inches of clean granular pit run or crushed rock. 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% of the modified proctor as described in the **Structural Fill** subsection of this report. Trenches located in non-structural areas should be compacted to a minimum 90% of the maximum dry density. The trench backfill compaction should be tested.

Site Drainage

Surface Drainage: The finished ground surface should be graded such that stormwater is directed to an appropriate 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 proposed structure. We suggest that the finished ground be sloped at a minimum downward gradient of three percent, for a distance of at least 10 feet away from the proposed structure where possible. 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 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 Mr. Bob Ford and his 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

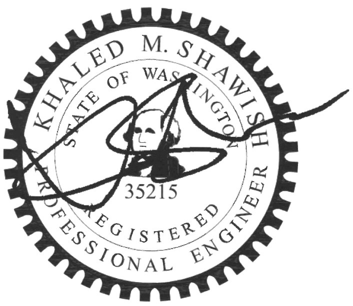
It has been a pleasure to provide service to you on this project. If you have any questions or require further information, please call.

Sincerely,

NELSON GEOTECHNICAL ASSOCIATES, INC.

Carston Curd

Carston T. Curd, GIT
Project Geologist



Khaled M. Shawish, PE
Principal

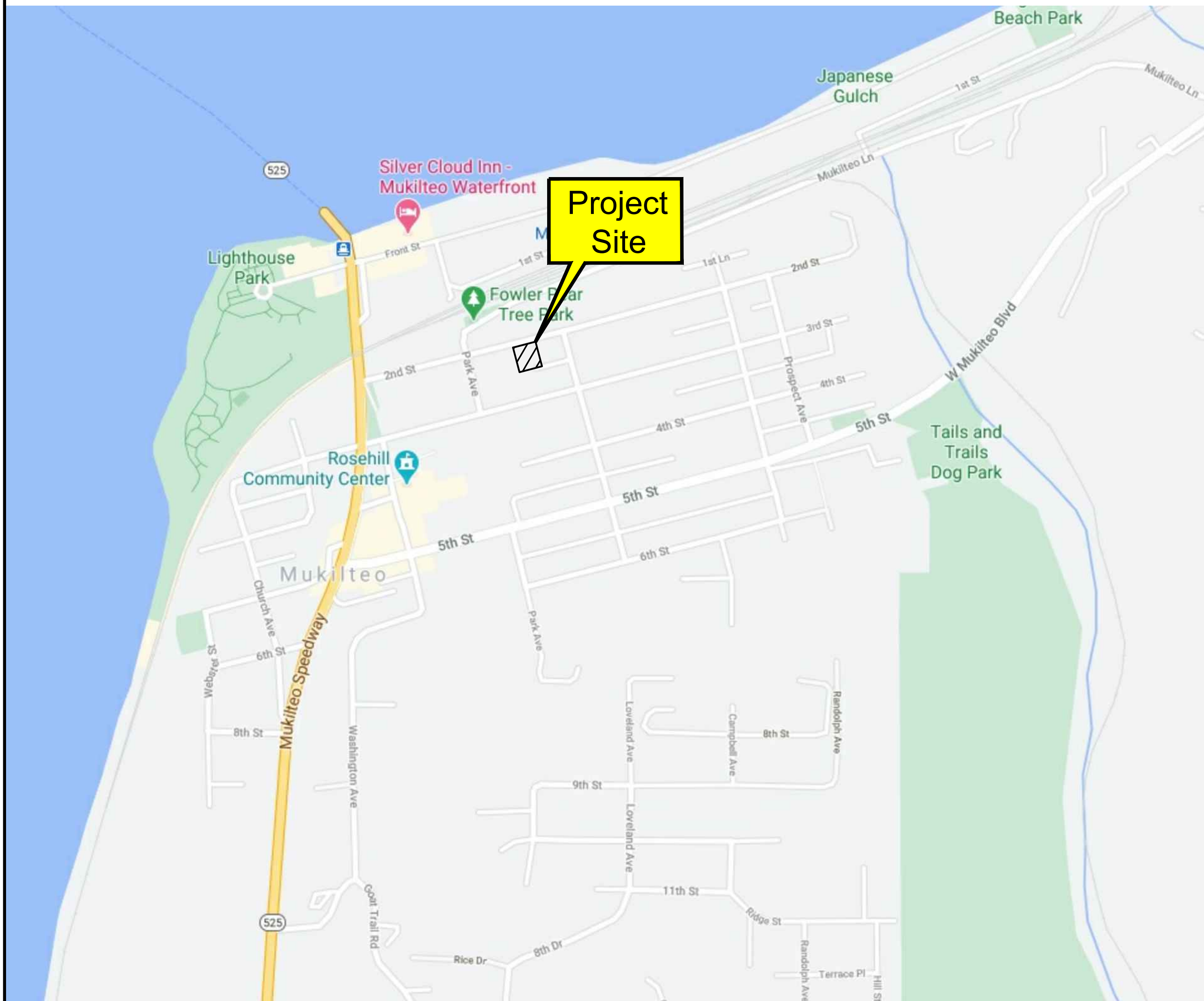
CTC:KMS:sg

Eight Figures Attached

VICINITY MAP

Not to Scale

N



Mukilteo, WA

Project Number	Sherwood Apartment Complex Development Vicinity Map	Nelson Geotechnical Associates, Inc.	No.	Date	Revision	By	CK
1165520			1	12/1/20	Original	DPN	CTC
Figure 1							

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Nelson Geotechnical Associates, Inc.
Geotechnical Engineers & Geologists
Woodinville Office
17311-135th Ave. NE, A-500
Woodinville, WA 98072
(425) 486-1669 / Fax: 481-2510
www.nelsongeotech.com
Wenatchee Office
105 Palouse St.
Wenatchee, WA 98801
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Schematic Site Plan

N

LEGEND

Property line

B-1

Number and approximate location of boring

A A'

Approximate location of cross-section

0 30 60

Approximate Scale: 1 inch = 30 feet

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

Project Number 1165520

Figure 2

Sherwood Apartment Complex Development Schematic Site Plan

NGA

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GEOTECHNICAL ENGINEERS & GEOLOGISTS

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
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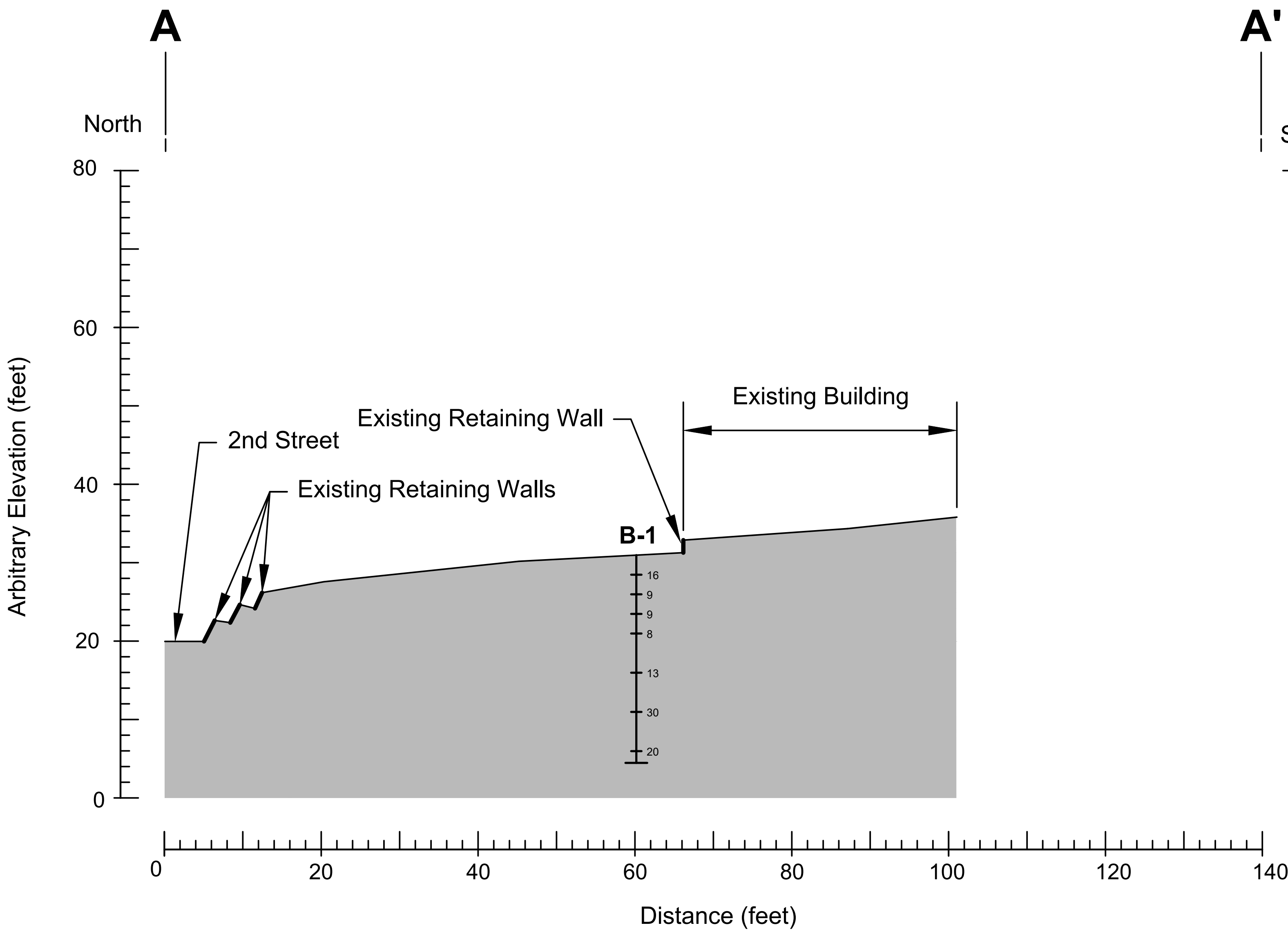
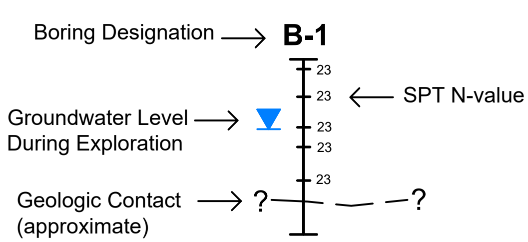


Exploration



NOTES:
1) Stratigraphic conditions are interpolated between the explorations. Actual conditions may vary.
2) Elevations are arbitrary.

Reference: Cross Section is based on field measurements using a hand-held clinometer and 100-ft tape measure.

Project Number 1165520

Figure 3

Sherwood Apartment Complex Development Cross-Section A-A'

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NELSON GEOTECHNICAL ASSOCIATES, INC.
GEOTECHNICAL ENGINEERS & GEOLOGISTS
Woodinville Office
17311-135th Ave NE, A-500
Woodinville, WA 98072
(425) 486-1669 / Fax: 481-2510
www.nelsongeotech.com
Wenatchee Office
105 Palouse St.
Wenatchee, WA 98801
(509) 665-7696 / Fax: 665-7692

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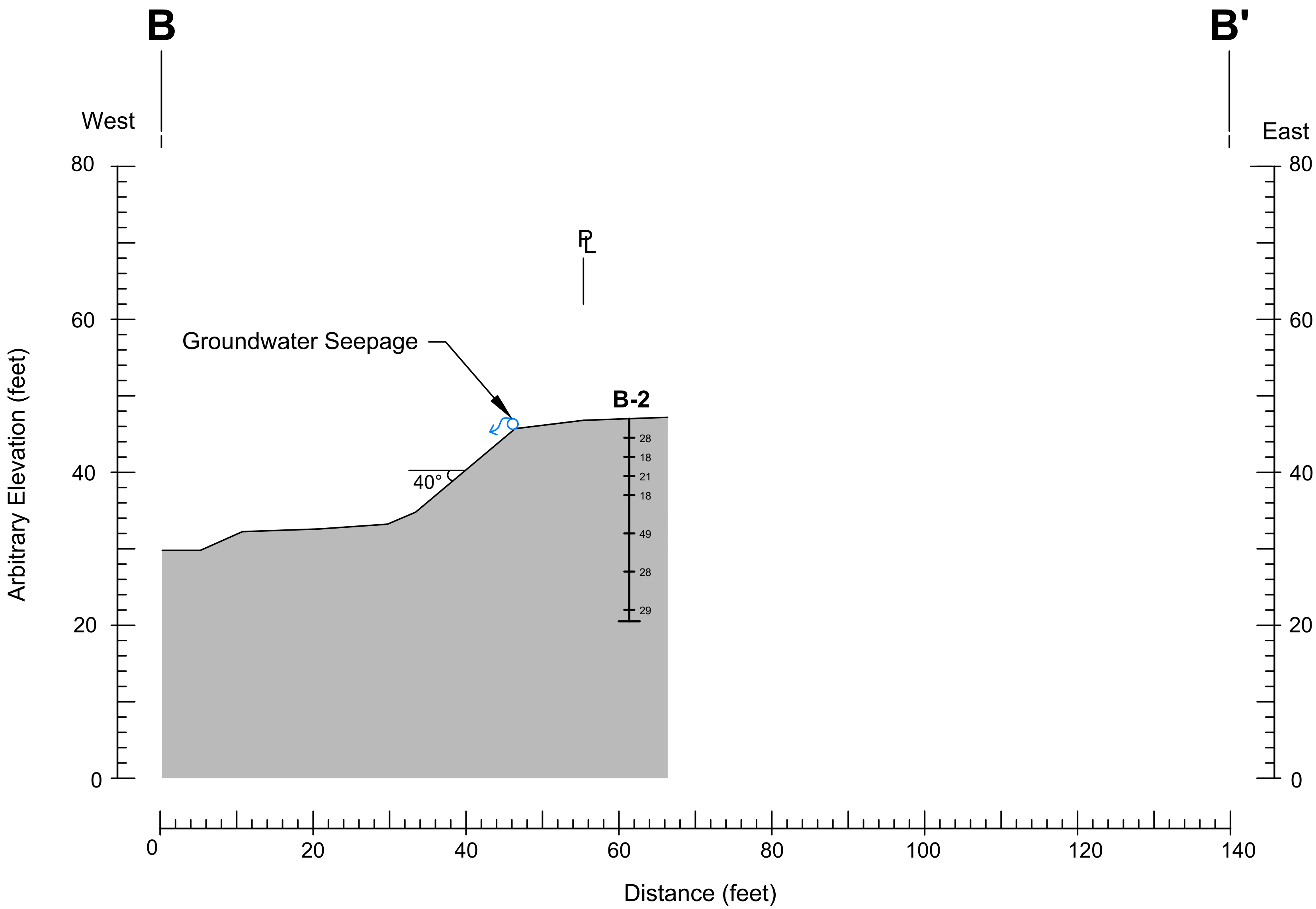
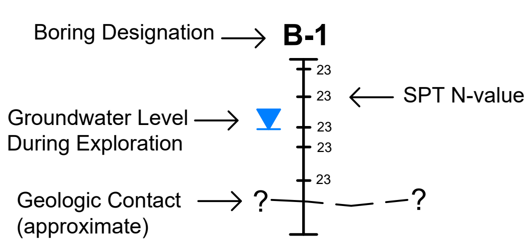


Exploration



NOTES:
1) Stratigraphic conditions are interpolated between the explorations. Actual conditions may vary.
2) Elevations are arbitrary.

Reference: Cross Section is based on field measurements using a hand-held clinometer and 100-ft tape measure.

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Figure 4

Sherwood Apartment Complex Development
Cross-Section A-A'

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GEOTECHNICAL ENGINEERS & GEOLOGISTS
Woodinville Office
17311-135th Ave. NE, A-500
Woodinville, WA 98072
(425) 486-1669 / Fax: 481-2510
www.nelsongeotech.com
Wenatchee Office
105 Palouse St.
Wenatchee, WA 98801
(509) 665-7696 / Fax: 665-7692

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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 1165520	Sherwood Apartment Complex Development Soil Classification Chart	NELSON GEOTECHNICAL ASSOCIATES, INC. GEOTECHNICAL ENGINEERS & GEOLOGISTS	No.	Date	Revision	By	CK
Figure 5			1	12/1/20	Original	DPN	CTC

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17311-135th Ave. NE, A-500
Woodinville, WA 98072
(425) 486-1669 / Fax: 481-2510

www.nelsongeotech.com

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105 Palouse St.
Wenatchee, WA 98801
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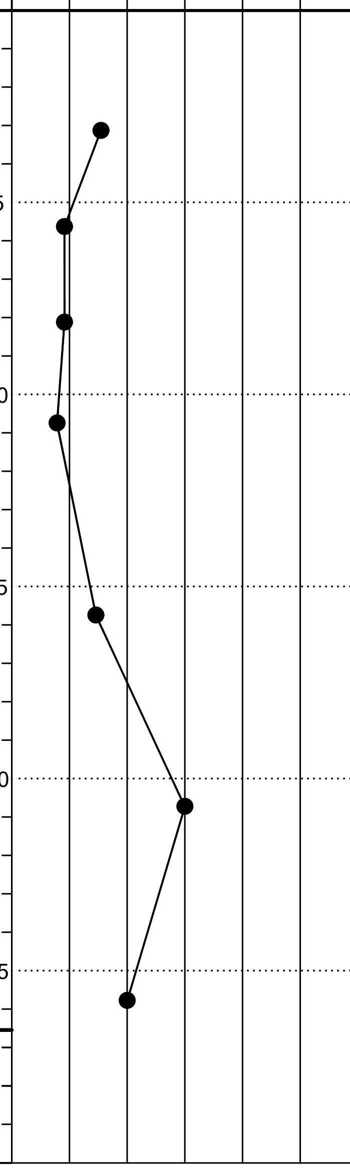
BORING LOG

B-1

Approximate Ground Surface Elevation: ??

Soil Profile: Description	Graphic Log	Group Symbol	Sample Data: Blow Count	Sample Data: Sample Location (Depth in feet)
Grass sod Light gray to light brown, silty fine sand with trace clay and iron-oxide staining (medium dense, dry to moist)		SM	16	
-becomes blue-gray to brown, loose, wet, with trace organics			9	5
-with saturated interbeds of brown, fine to medium sand with silt			9	
			8	10
-becomes blue-gray, silty fine sand, medium dense, dry to moist			13	15
Blue-gray, fine to medium sand with silt and silty interbeds (medium dense to dense, dry to moist)		SP-SM	30	20
Blue-gray, silty fine sand with silty fine to medium sand interbeds (medium dense, moist)		SM	20	25
Boring terminated at 26.5 feet below existing grade on 11/20/2020. Groundwater seepage was encountered at 6.3 feet during drilling.				

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	Sherwood Apartment Complex Development Boring Log	Nelson Geotechnical Associates, Inc. — Geotechnical Engineers & Geologists	No.	Date	Revision	By	CK
1165520			1	12/1/20	Original	DPN	CTC
Figure 6							
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17311-135th Ave. NE, A-500
Woodinville, WA 98072
(425) 486-1669 / Fax: 481-2510

www.nelsongeotech.com

Wenatchee Office
105 Palouse St.
Wenatchee, WA 98801
(509) 665-7696 / Fax: 665-7692

BORING LOG

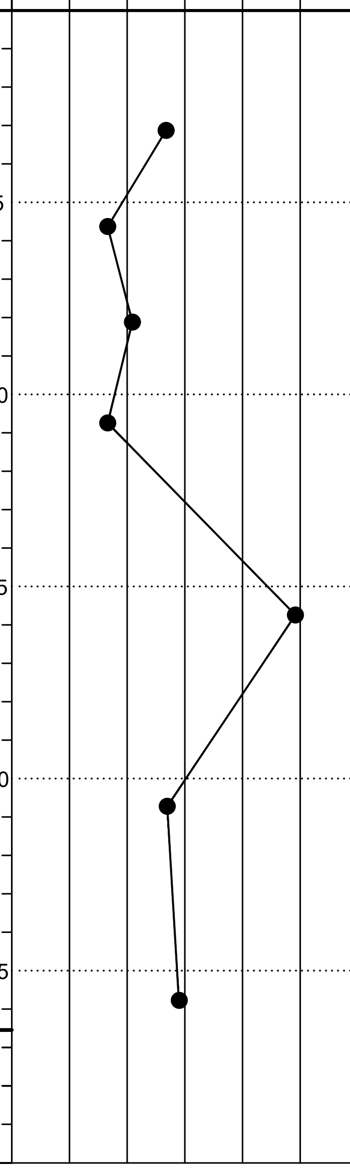
B-2

Approximate Ground Surface Elevation: ??

Soil Profile: Description	Group Symbol	Sample Data: Blow Count	Laboratory Testing	Piezometer Installation - Ground Water Data (Depth in Feet)
Weeds Tan, silty fine to medium sand with iron-oxide staining (medium dense, dry)		28		
-becomes gray, silty fine sand, dry to moist	SM	18		5
-brown, silty fine to medium sand		21		
-with trace coarse sand and organic particulate -becomes blue-gray, silty fine sand		18		10
Gray, fine to medium sand with silt and trace fine gravel (dense, moist)	SP-SM	49		15
Blue-gray, silty fine to medium sand with trace gravel (medium dense, moist)	SM	28		20
Blue-gray silt with fine sand (very stiff, moist)	ML			
-with interbeds of gray, silty fine to medium sand		29		25
Boring terminated at 26.5 feet below existing grade on 11/20/2020. Groundwater seepage was not encountered during drilling.				

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

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



Sample Location (Depth in feet)

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 1165520	Sherwood Apartment Complex Development Boring Log	NELSON GEOTECHNICAL ASSOCIATES, INC. GEOTECHNICAL ENGINEERS & GEOLOGISTS
Figure 7		
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Woodinville, WA 98072
(425) 486-1669 / Fax: 481-2510
www.nelsongeotech.com
Wenatchee Office
105 Palouse St.
Wenatchee, WA 98801
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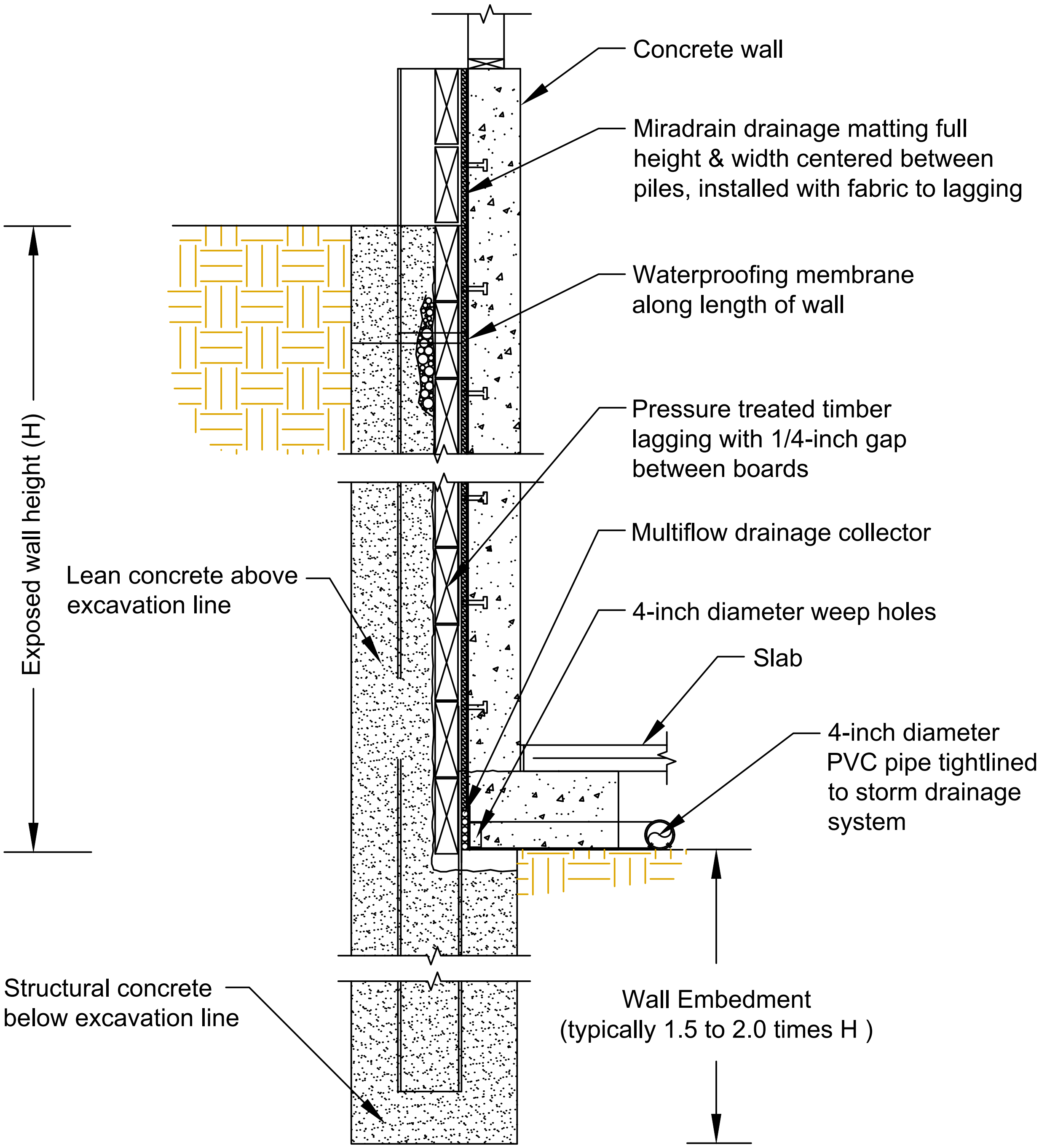
Conceptual Soldier Pile Wall Detail

NOT FOR CONSTRUCTION USE



NOT TO SCALE

Project Number 1165520	Sherwood Apartment Complex Development Soldier Pile Wall Detail	NELSON GEOTECHNICAL ASSOCIATES, INC.	No.	Date	Revision	By	CK
Figure 8		GEOTECHNICAL ENGINEERS & GEOLOGISTS Woodinville Office, 17311-135th Ave. NE, A-500, Woodinville, WA 98072, (425) 486-1669 / Fax: 481-2510 www.nelsongeotech.com East Wenatchee Office, 5526 Industry Lane, #2, East Wenatchee, WA 98802, (509) 665-7696 / Fax: 665-7692	1	12/22/20	Original	DPN	CTC

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