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January 14, 2022

Maile Intrachat and Hoa Hoang
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hoahoang1@gmail.com

Geotechnical Engineering Evaluation
Intrachat-Hoang Residence Development
7929 East Mercer Way
Mercer Island, Washington
NGA File No. 1276521

Dear Maile and Hoa:

We are pleased to submit the attached report titled **"Geotechnical Engineering Evaluation – Intrachat-Hoang Residence Development – 7929 East Mercer Way – Mercer Island, Washington."** The tax parcel for this property is 302405-9176. Our services were completed in general accordance with the proposal signed by you on November 11, 2021.

The property consists of an irregular-shaped parcel covering approximately 0.7-acres. The site is currently occupied by a single-family residence along the southeastern portion of the property. Topographically, the site includes a steep southeast-facing slope which descends from Island Crest Way above and to the northwest of the property down to the existing residence. Near the residence and driveway in the southeastern portion of the property the ground surface slopes gently to moderately from northwest to southeast. We understand the planned development will consist of removing the existing single-family residence structure and constructing a new three-story single-family residence in the same approximate location. As part of the proposed residence construction, we also understand new shoring walls are proposed to the northwest and immediately along the southeastern portion of the new residence. Final grading and stormwater plans were not developed at the time this report was prepared; therefore, we recommend we be retained to review the finalized plans to ensure they are compatible with the existing site conditions.

We explored the soil and groundwater conditions in the vicinity of the proposed residence with two geotechnical borings extending to depths in the range of approximately 31.5- to 32.5-feet below the existing ground surface. Our explorations indicated that the site is underlain by surficial undocumented fill with variably loose to dense granular sand and gravel deposits at depth. Based on information available to us, it appears the site has been subjected to historical mass wasting events as a result of ancient large-scale landslide activity originating along the steep slopes within and above the subject parcel. The soils encountered in our borings were interpreted as colluvial deposits, which were likely displaced from native glacial deposits along the steep slopes above and to the northwest of the property during historical landslide events. Based on our site reconnaissance the slopes within and bordering the property appear to be currently relatively stable with respect to deep-seated landsliding, and it is our opinion that the planned development is feasible from a geotechnical standpoint, provided that our recommendations are incorporated into the design and strictly followed during construction.

Due to the significant depth of variably loose to medium dense sandy colluvial deposits encountered in our borings we recommend the new residence foundations are supported on deep foundation elements to transfer structure loads through the looser colluvial deposits and down to bearing native soils. In our opinion the deep foundations should consist of 4-inch diameter steel pipe piles driven to refusal.

Additionally, due to the proximity of the prominent southeast-facing steep slope descending towards the northern portion of the proposed residence, we recommend development plans include debris catchment measures to protect the residence from potential shallow slope failures. Given that the proposed plans include a new shoring wall along the toe of the steep southeast-facing slope to the north of the residence, we recommend this structure is dually utilized as a debris catchment measure by extending the wall a minimum of 4-feet above the finished ground surface. Other retaining walls proposed along the northern portion of the proposed driveway and below the residence may consist of pile supported, concrete cast-in-place walls or reinforced earth walls, as discussed in this report. No unshored cuts over three feet should be attempted at the toe of the steep slope. General recommendations for retaining walls, as well as recommendations for site grading, subgrade preparation, drainage, and erosion control are further discussed in the attached report.

We recommend that we be retained to provide a review of the finalized plans after they have been developed to verify that our recommendations have been incorporated into project plans. 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 the work differ from those anticipated, and to evaluate whether or not earthwork and foundation installation activities comply with contract plans and specifications.

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

Sincerely,

NELSON GEOTECHNICAL ASSOCIATES, INC.

Khaled M. Shawish, PE
Principal

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INTRODUCTION

This report presents the results of our preliminary geotechnical engineering investigation and evaluation of the proposed Intrachat-Hoang residence development on Mercer Island, Washington. The project site is located at 7929 East Mercer Way, as shown on the Vicinity Map in Figure 1. The Tax Parcel number for the property is listed as 3024059176. For our use in preparing this report, we have been provided with the following documents:

- Architectural set, sheets A1.01-A3.02, titled "Intrachat-Hoang Residence," dated July 30, 2021, prepared by Warm Modern Living.

- Boundary and Topographic Survey for Hoa Hoang, dated June 15, 2021, prepared by Axis Surveying & Mapping.

The site is currently occupied by a single-family residence within the lower southeastern portion of the property with driveway access entering the site along the northeastern corner of the parcel. The preliminary development plans indicate removal of the existing site structures and construction of a new single-family residence and associated retaining walls within the same approximate location as the existing house. The existing site conditions are 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. Reviewing available soil and geologic maps of the area.
2. Exploring the subsurface soil and groundwater conditions within the proposed development areas with up to two geotechnical boreholes using a limited-access drill rig. Drilling services were subcontracted by NGA.
3. Mapping the conditions on the site slopes using shallow, hand-tool explorations where necessary to construct geological cross sections and qualitatively evaluate slope stability.

4. Conducting numerical limit-equilibrium modeling on representative slopes affected by the development to provide a quantitative evaluation of slope stability, if necessary.
5. Performing laboratory grain-size sieve analysis on soil samples, as necessary.
6. Providing recommendations for structure setbacks from geologic hazards, as necessary.
7. Providing recommendations for earthwork and foundation support.
8. Providing recommendations for retaining walls.
9. Providing recommendations for temporary and permanent slopes.
10. Providing recommendations for subsurface utilities and pavement subgrade preparation.
11. Providing our general opinion on stormwater infiltration feasibility.
12. Providing our recommendations for erosion control during wet weather construction and review Temporary Erosion and Sediment Control Plans (TESC Plans) as provided.
13. Providing general recommendations for site drainage and erosion control.
14. Documenting the results of our findings, conclusions, and recommendations in a written geotechnical report.

SITE CONDITIONS

Surface Conditions

The site consists of an irregular-shaped parcel covering approximately 0.7-acres. The southeastern portion of the property is currently occupied by a single-family residence and associated driveway extending from the northeast corner of the site. The ground surface along the developed portion of the site is generally gently to moderately sloping from northwest to southeast, while the areas above the residence extending as far as Island Crest Way to the northwest of the property are occupied by a steep southeast-facing slope containing gradients in the range of approximately 25 to 43 degrees (46.6 to 93.3 percent) as shown on Cross-Section A-A’ in Figure 3.

An existing approximately 2.5-foot-tall timber retaining wall and elevated paver patio area currently occupies the space between the north side of the residence and the toe of the steep southeast-facing slope. Further northeast, along the north side of the driveway the toe of slope area is faced by rockeries, minor landscape retaining walls, and assorted landscaping plants. The lower southern portion of the driveway is partially supported by an existing approximately 4-foot-tall modular block wall, which terminates along the southeast corner of the residence. This wall appeared to be settling in several different locations.

Around the residence, driveway, and southern backyard area, vegetation consisted of scattered young to mature trees, grass yard areas, and landscaping. The steep southeast-facing slope above the residence was primarily vegetated with loosely spaced young to mature deciduous and coniferous trees with sparse underbrush including ivy, ferns, and blackberries. The ground surface along the steep slope was generally in a loose condition and several mature coniferous trees, particularly along the upper portion of the slope, near or just above the northwest property line displayed gentle "s" curvature along the trunks, although trees along the lower to mid portions of the slope were typically straight with normal growth pattern. The abnormal growth on the upper trees is likely attributed to shallow soil creep during the lifespan of the trees. We did not observe indications of recent erosional or shallow sloughing activity along the steep slopes within the site during our reconnaissance or indications of groundwater seepage emitting from the steep slopes.

Subsurface Conditions

Geology: The geologic units for this area are shown on the <u>Geologic Map of Mercer Island, Washington</u>, by Kathy G. Troost and Aaron P. Wisher (City of Mercer Island, ESS, University of Washington, 2006). The site is mapped within an advance outwash (Qva) deposit with till (Qvt) and Lawton clay (Qvlc) indicated as the overlying and underlying geologic units, respectively. Advance outwash is generally described as well-sorted sand and gravel deposited by streams issuing from advancing ice sheet, while Lawton Clay is described as massive silt, clayey silt, and silty clay deposited in lakes dammed by the continental glacier during the Vashon Stade. Till is described as a compact diamicton of sand, silt, gravel, and clay glacially consolidated by overlying ice sheet. The geologic mapping in this area also indicates the potential presence of mass wasting deposits mantling the underlying undisturbed native glacial deposits within the site and regionally along Mercer Way on the south tip of Mercer Island. The soils encountered in our explorations varied from fine to medium sand to gravelly fine to coarse sand with relatively minor silt content, which appeared consistent with the description of advance outwash. However, the soil density at depth ranged from loose to locally medium dense or better which is not typical of a glacially overridden deposit, therefore we interpreted the soils encountered at depth to consist of colluvial deposits with the sandy advance outwash material mapped within and above the site as the parent material.

Explorations: The subsurface conditions within the site were explored on November 22, 2021 by monitoring the drilling of two geotechnical borings to depths in the range of approximately 31.5- to 32.5-feet below the existing ground surface using a limited access drill rig. The approximate locations of our explorations are shown on the Site Plan in Figure 2.

A geologist from Nelson Geotechnical Associates, Inc. (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 explorations. A Standard Penetration Test (SPT) was performed on each of the samples during drilling to document soil density at depth. The SPT consists of driving a 2-inch outer-diameter, split-spoon sampler 18 inches using a 140-pound hammer falling 30 inches. The number of blows required to drive the sampler the final 12 inches is referred to as the "N" value and is presented on the boring logs. The N value is used to evaluate the strength and density of the deposit. The soils were visually classified in general accordance with the Unified Soil Classification System presented in Figure 4. The logs of our borings are attached to this report and presented as Figures 5 and 6. We present a brief summary of the subsurface conditions in the following paragraph. For a detailed description of the subsurface conditions, the boring logs should be reviewed.

At the surface of each boring, we encountered approximately 4- to 6.5-feet of dark brown to gray-brown, slightly silty fine to medium sand with gravel, minor roots and organics in a loose condition, which we interpreted as undocumented fill soils. Underlying the undocumented fill soils, we encountered yellowish brown to gray, fine to coarse sand with varying silt and gravel content. In the lower southern Boring 2 (B-2), below approximately 15-feet the soils became progressively coarser grained to a gravelly fine to coarse sand at approximately 25-feet below the existing ground surface. The relative soil density at depth, indicated by the Standard Penetration Test (SPT) values generally ranged from loose to medium dense with inconsistent and localized dense values between 20- to 25-feet in Boring 1 (B-1). Based on the soil characteristics observed in the borings, we interpret the deposits underlying the site consist of displaced advance outwash soils from ancient landslide activity. Borings 1 and 2 met refusal due to groundwater heave conditions at respective depths of approximately 32.5- and 31.5-feet below the existing round surface.

Hydrogeologic Conditions

Groundwater was encountered in Borings 1 and 2 at respective depths of 22- and 18-feet below the existing ground surface. We anticipate the groundwater observed is perched groundwater mantling the underlying mostly impermeable Lawton clay deposits. Perched water occurs when surface water infiltrates through less dense, more permeable soils and accumulates on top of relatively low permeability materials. The more permeable soils consist of the topsoil/weathered soils. The low permeability soil consists of relatively silty glacial soils. 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

The 2018 International Building Code (IBC) seismic design section provides a basis for seismic design of structures. 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	1.458	0.555	1.000	1.500	0.972	0.555

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.

Hazards associated with seismic activity include liquefaction potential and amplification of ground motion by soft deposits. Liquefaction is caused by a rise in pore pressures in a loose, fine sand deposit beneath the groundwater table. The sandy colluvial deposits interpreted to underlie the site have a moderate potential for liquefaction or amplification of ground motion. Recommendations for pile supported foundations, as documented in this report should greatly reduce the potential for liquefaction induced structure damage.

Although ancient landslide scarps are mapped in the vicinity of the site, the glacial soils interpreted to form the core of the site slopes are considered stable with respect to deep-seated slope failures within the site. However, the overlying loose surficial materials on the slopes have the potential for shallow sloughing failures during seismic events. Such events should not adversely impact the planned residence structure provided the foundations are designed with the recommended pile support and debris protection measures are incorporated into the overall design as described in this report.

Erosion Hazard

The criteria used for determination of erosion hazard 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 Soil Conservation Service (SCS), lists the soils within the site as Everett-Alderwood gravelly sandy loams, 6 to 15 percent slopes and Kitsap silt loam, 15 to 30 percent slopes, for the lower central to southern and steeply sloping northwestern portions of the site, respectively. The soil erosion hazard is not rated for this site; however, we anticipate the site would yield a slight to moderate erosion hazard where the vegetative cover is removed. The on-site soils should have a low hazard for erosion where the vegetation is not disturbed.

Landslide Hazard/Slope Stability

The criteria used for evaluation of landslide hazards include soil type, slope gradient, and groundwater conditions. We understand the proposed single-family residence will be situated within the southeastern portion of the site on a gently sloping bench area bound by gentle to moderate southeast-facing slopes to the southeast and a steep southeast-facing slope to the northwest. The upper steep slopes generally descend from above the northwestern property line near Island Crest Way at gradients in the range of 25 to 43 degrees (46.6 to 93.3 percent) while the lower slopes descend away from the existing residence at gradients in the range of 15 to 25 degrees (26.8 to 46.6 percent). The ground surface gradients throughout the site are depicted on Cross-Section A-A' in Figure 3 attached to this report. The overall vertical relief of the upper steep slope area is approximately 130 to 145 feet.

We did not observe evidence of recent significant slope instability within the property during our investigation, such as indications of recent deep-seated landsliding, erosional activity, shallow sloughing events, or groundwater seepage emitting from the face of the slopes. However, the abnormal growth of mature coniferous trees along the upper portion of the slope is indicative of on-going surficial soil creep impacting the upper loose soils present along the slope, although we did not observe any other surficial expression to suggest recent activity. An ancient landslide scarp has been identified and mapped above and to the north/northwest of the subject site. This scarp and overall steep slope area, when viewed on LiDAR imagery, depicts very steep/sharp features, generally indicating more geologically active conditions. We anticipate most of the chronic stability issues that may have occurred on this mapped scarp/slope area occurred as a reaction to post-glacial conditions some 10- to 14- thousand years ago and have since reached a more stable configuration with respect to deep-seated failures.

The core of the site slopes is inferred to consist primarily of medium dense or better native glacial soils. Inclinations of up to 43 degrees (93.3 percent) on the slopes within the property indicate high internal strength within the underlying soils. Relatively shallow sloughing failures as well as surficial erosion are natural processes and should be expected on these 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 steep slopes, there is not a significant potential for deep-seated slope failure under current site conditions. Proper site grading and drainage as well as adequate foundation placement as recommended in this report should help maintain current stability conditions. It is critical that no cuts over 3.0 feet are attempted into the toe of the slope unless such cuts are shored from the top down.

CONCLUSIONS AND RECOMMENDATIONS

General

It is our opinion that the planned development is feasible from a geotechnical standpoint. It is also our opinion that the soils that underlie the site and form the core of the site slopes should be stable with respect to deep-seated earth movements, due to their inherent strength and slope geometry. There is, however, a significant potential for shallow sloughing and erosional events to occur on the steeper portions of the slopes, especially during extreme weather conditions or seismic activity. Proper erosion and drainage control measures, as well as debris protection systems as recommended in this report, should reduce this potential. We recommend that we be retained to review the finalized plans after they have been developed. Our explorations indicated that the site is generally underlain by loose to medium dense colluvial deposits to the depths explored. For bearing capacity and settlement consideration, we recommend new foundations elements are supported on 4-inch diameter steel piles driven to refusal, as discussed in the **Deep Foundations** subsection of this report.

Based on the provided plans, we understand new retaining structures are planned to the north of the house and extending northeast along the proposed driveway, as well as immediately below the proposed residence. Immediately north of the residence, the most feasible option, in our opinion, would be the construction of a soldier pile retaining wall, which would limit site disturbance to the slope, as well as facilitate excavation for the proposed residence, and serve as a debris catchment measure. Depending on the overall exposed height of the wall, it may be designed as a cantilevered wall or incorporate tieback components to resist lateral forces. Due to the proximity of the proposed residence to the toe of the steep southeast-facing slope, we recommend the soldier pile shoring wall be also utilized as a debris protection measure in the event of a shallow slope failure above the residence.

Other retaining walls to the north of the driveway and below the residence may consist of traditional concrete walls or reinforced earth walls, as long as no cuts taller than 3.0 feet are attempted into the toe of the steep slope. These recommendations are further discussed in the **Solider Pile Retaining Walls and Other Retaining Walls** subsections of this report.

All grading operations and drainage improvements planned as part of this development should be planned and completed in a matter which enhances the stability of the steep slopes, not reduces it. Excavation spoils should not be stockpiled near the slopes or be allowed to encroach on the slopes. Also, runoff generated within the site should be collected and routed into a permanent discharge system and not be allowed to flow over the slopes. Future vegetation management on the slope should be the subject of a specific evaluation and a plan approved by the City of Mercer Island. The slopes should be monitored on an on-going basis, especially during the wet season, for any signs of instability, and corrective actions promptly taken should any signs of instability be observed. Lawn clipping and any other household trash or debris should never be allowed to reach the slopes.

The surficial soils encountered on this site are considered moisture-sensitive and will disturb easily when wet. To lessen the potential impacts of construction on the slopes and to reduce cost overruns and delays, we recommend that construction take place during the drier summer months. If construction takes place during the rainy months, additional expenses and delays should be expected. Additional expenses could include the need for placing erosion control and temporary drainage measures to protect the slopes, the need for placing a blanket of rock spalls on exposed subgrades, and construction traffic areas prior to placing structural fill, and the need for importing all-weather material for structural fill.

Under no circumstances, should water be allowed to flow over or concentrate on the site slopes, both during construction, and after construction has been completed. We recommend that stormwater runoff from the roof and yard drains be collected and tightlined to a suitable discharge point. The slopes should be protected from erosion. We recommend that all disturbed areas be replanted with vegetation to re-establish vegetation cover as soon as possible.

As part of the proposed driveway expansion we understand that two mature fir trees may need to be removed. These trees are generally situated immediately northwest of the existing driveway along gently sloping ground surface along the toe of the steep southeast-facing slope. In our opinion, removal of these trees could be performed in a manner that does not adversely impact slope stability conditions. Specific recommendations for erosion control and vegetation management are presented in the **Erosion Control and Slope Protection Measures** subsection below.

Erosion Control and Slope Protection Measures

The erosion hazard for the on-site soils is considered to be moderate, but the actual hazard 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 or flowing over the slopes. Stockpiles should be covered with plastic sheeting during wet weather and stockpiled material should be no closer than 25 feet from the top of the slopes. Disturbed areas should be planted as soon as practical, and the vegetation should be maintained until it is established. The erosion potential for areas not stripped of vegetation should be low.

Protection of the steep slope areas should be performed as required by the City of Mercer Island. Specifically, we recommend that the sloping areas outside the proposed development area not be disturbed or modified through placement of any fill or removal of the existing vegetation. No additional material of any kind should be placed on either slope or be allowed to reach the slopes, such as excavation spoils, lawn clippings, and other yard waste, trash, and soil stockpiles. Trees should not be cut down or removed from the slopes unless a mitigation plan is developed, such as the replacement of vegetation for erosion protection. Vegetation should not be removed from the slopes. Replacement of vegetation should be performed in accordance with the City of Mercer Island code. Any proposed development within the steep slope areas outside the proposed residence should be the subject of a specific geotechnical evaluation. Under no circumstances should water be allowed to concentrate on the slopes.

We understand two mature coniferous trees, situated on gently sloping ground surface at the toe of the steep southeast-facing slope and immediately north of the existing driveway are to be removed to facilitate proposed driveway widening as well as retaining wall installation. In our opinion, this will not adversely impact slope stability provided the recommendations in this report are closely followed. The subject trees should be cut, such that the stumps are left in place. This vegetation removal plan will leave the roots in place and will continue to maintain stability and erosion protection. All cut trees and resulting debris should be removed from the surface of the slope after cutting. Best Management Practices should be followed to minimize disturbance to the slopes during tree and debris removal.

If any areas are disturbed or exposed soils exist after tree and branch removal, these areas should be covered with erosion material and planted as soon as practical, and the vegetation should be maintained until it is established. Erosion control measures could consist of placement of netting, hydro-seeding and/or straw mulching as an effective means for minimizing erosion and allowing vegetation to begin rapidly. We should be retained to provide specific recommendations for erosion control measures during tree and branch removal activities, as needed. Also, surface water should be directed away from the affected areas. This could be accomplished through the placement of berms/straw wattles.

If tree trimming results in substantial areas of soil exposed to precipitation, we recommend that the exposed soil be covered with heavy duty jute netting. The jute netting should be staked at the top of the slope with 2- to 3-foot-long metal rebar that has a metal "T" welded to the end. The mat should be staked to the surface every five feet. After the matting is placed, we recommended that deep-rooted vegetation be planted on the slope and grass seed be placed to re-establish vegetation growth. The vegetation should be maintained until established. We recommend a mixture of 25% each of the following vegetation: Snowberry (Symphoricarpos albus), Nootka rose (Rosa nutkana), Ocean Spray (Holodiscus discolor), and Oregon-grape (Manhonia nervosa). We should be retained to review and comment on the slope vegetation plan and observe the slope repairs if they become necessary.

Site Preparation and Grading

After erosion control measures are implemented and the shoring wall north of the proposed residence is successfully installed, site preparation for the proposed residence should consist of stripping surficial topsoil and/or undocumented fill soils down to desired foundation subgrade elevations. The stripped materials should be removed from the site. Stockpiles should be kept a minimum of 25 feet away from the top of the steep slopes and should be covered with plastic. 4-inch diameter pipe piles, as discussed in this report could then be driven along proposed foundation, slab, and retaining wall elements, due to the significant depth of loose colluvial soils.

Any planned exterior hard surfaces, such as patios or pavements should be supported on a modified subgrade to minimize the potential for settlement and cracking as a result of loose subgrade conditions at depth. We recommend the subgrade for exterior hard surfaces is prepared by overexcavating the desired subgrade by a minimum of 1.0 feet and replacing with coarse-grained, angular crushed rock, such as 1.25-inch minus crushed rock. If the ground surface, after site stripping, should appear to be loose, it should be compacted to a non-yielding condition. Areas observed to pump or weave during compaction should be over-excavated and replaced with properly compacted structural fill or rock spalls.

If loose soils are encountered in any slab 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.

This site is underlain by moisture sensitive soils. Due to these conditions, special 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 promptly covering exposed subgrades 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 prepared subgrade. Shallow groundwater, if encountered, should be intercepted with cut-off drains and routed around the planned grading area, or the groundwater should be controlled with sump-pumps or dewatering systems. Failure to follow these recommendations could cause erosion and failures on the slopes, as well as result in inadequate subgrades.

Temporary and Permanent Slopes

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

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

For planning purposes, we recommend that temporary cuts in the on-site soils be no steeper than 2H:1V. If significant groundwater seepage or surface water flow were encountered, we would expect that flatter inclinations would be necessary. We recommend that cut slopes be protected from erosion. The slope protection measures may include covering cut slopes with plastic sheeting and diverting surface runoff away from the top of cut slopes.

We do not recommend vertical slopes for cuts deeper than 3.0 feet at the toe of the steep slope or where worker access is necessary. We recommend that cut slope heights and inclinations conform to appropriate OSHA/WISHA regulations. If the above inclinations cannot be met due to property line constraints and/or worker access issues, we recommend that shoring be considered for the planned cuts. We are available to provide specific recommendations for temporary shoring once grading plans have been finalized.

Permanent cut and fill slopes should be no steeper than 2H:1V, unless specifically approved by NGA. Also, flatter inclinations may be required in areas where loose soils are encountered. Permanent slopes should be vegetated, and the vegetative cover maintained until established. We should specifically review all plans for grading on steep slopes for this project.

Deep Foundation Support

To eliminate the risk of structure settlement as a result of the loose colluvial soils encountered at depth, we recommend the proposed structure, including building slab, and retaining wall elements be supported on 4-inch diameter pin piles to transfer foundation loads through the upper loose soils down to the underlying competent materials, interpreted to underlie the site at depth. Our explorations did not encounter significant debris within the upper fill soils; however, there is potential that debris within the fill has the possibility to impede some of the piles. There should be contingencies in the budget and design for additional/relocated piles to replace piles that may be obstructed by debris. We also recommend that excavation equipment be available on-site during pile installation so that shallow obstructions can be removed from the planned pile locations.

We recommend that the four-inch pipe piles be utilized and should 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 80 pipe for the 4-inch pin piles. Maintaining these recommendations for minimum hammer size and refusal criteria is essential for obtaining successful piles.

Final pile depths should be expected to vary and will depend on the depth to competent soils. We recommend a test element is driven for planning purposes to better constrain refusal depth. The piles should be spaced a minimum of three feet apart to avoid a grouping effect on the piles. Due to the relatively small slenderness ratio of pin piles, maintaining pin pile confinement and lateral support is essential in preventing pile buckling. Pin piles should be suitably embedded into the reinforced concrete. The structural engineer should design the connections of the piles to the foundations. 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.

Soldier Pile Shoring Wall

General: We recommend a soldier pile shoring wall be designed to support temporary cuts along the toe of the steep slope, and under final conditions serve as a debris catchment measure to the northwest of the proposed residence. We recommend the shoring wall be designed as a stand-alone cantilever wall if possible. However, tieback anchors may be needed depending on the overall wall design. We anticipate cuts up to approximately 10 feet tall may need to be supported by shoring the wall along the northern portion of the proposed residence. Extreme care should be taken during installation, such that disturbance to the toe of slope area is kept to a minimum. This should be accomplished by installing vertical H-beams in front or behind the existing patio timber wall prior to deconstructing this wall or disturbing this area. Once the vertical elements are successfully installed, excavation and lagging may occur. Exposed vertical cuts should be limited to a maximum of 3-feet during lagging operations. A conceptual soldier pile wall detail and pressure diagram is shown on Figure 7. 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 or piles 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, concrete panels or metal plates. 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 type, wall height, drainage conditions, and the final geometry. We recommend that the soldier pile walls be designed using the earth pressure diagrams presented in Figure 7.

The shoring walls 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 65 pcf for active conditions. We also recommend that a uniform surcharge of 8H (psf) should be applied to the wall design to account for seismic loading. H in this case is the exposed height of the wall. The recommended debris catchment component of the wall should include a segment of wall extending above the ground surface behind the wall a minimum of 4-feet. This portion of the wall should be designed to resist an active pressure of 100 pcf. 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, applied on two-pile diameters under the excavation line. This value of the passive pressure incorporates a factor of safety of 2.0. The upper two feet of pile embedment should be neglected when calculating the passive resistance. The below-grade portion of the wall should be no less than 1.5 times the wall stick-up height.

The above load should be applied on the full center-to-center pile spacing above the base of the exposed portion of the wall. A 50 percent reduction of 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 removed from the excavations. Adequate gaps should be maintained between the lagging elements to allow for water seepage 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 an approved discharge point.

The wall designer should calculate the predicted wall deflection, including deflection resulting from the below-grade movement of the piles. The predicted deflection values should be confirmed in the field through a monitoring program.

If the shoring wall cannot be designed as stand-alone cantilever retaining wall, additional restraint systems such as grouted tie-backs may be needed to support the wall loads, as discussed below.

The shoring wall should be installed by a shoring contractor experienced with these types of systems. 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 will likely be encountered. It might be prudent to perform one or more “test” holes to confirm installation conditions prior to finalizing budget and work plans. Any sloughing or water that may collect in the drilled holes should be removed prior to pumping grout. Grout should be readily available on site at the time the holes are drilled and cased.

If groundwater seepage is encountered, we recommend that water be pumped out of the holes and the concrete be tremied from the bottom of the excavations to displace the groundwater to the surface. Extra Portland Cement, 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.

The wall should be lagged using pressure-treated timber, metal sheets, or concrete panels. Adequate gaps, typically by placing lagging nails between the boards, should be maintained between the lagging elements to allow water flow through the face of the wall. Lagging should be installed promptly after excavation. Lagging should be installed from the top of the wall to the bottom with exposed excavations being not greater than four feet in height. This is important to maintain the stability of the excavation. Any backfill placed behind the wall should consist of clean 2-inch crushed rock. The crushed rock should be tamped in place using hand tools to ensure all gaps are entirely filled. We should be retained to monitor on site activities during the soldier pile wall installation on a full-time basis.

Tie-back Anchors General: Depending on the final plans, tiebacks may be needed to resits lateral forces. The tieback anchors should consist of steel tendons inserted into drilled holes and then grouted into place. We recommend that five percent of the anchors, but no less than two, be treated as performance anchors and be tested to a minimum of 200 percent of the design loads. The soil creep characteristics would be evaluated during these tests.

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 wall into the slope behind the wall a horizontal distance of six feet. The 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. All anchors should be installed at an approximate inclination of 15 degrees below horizontal. An acceptable form of bond breaker (such as plastic sheathing) should be applied to the tendon within the length of the no-load zone.

Soil Design Values: The tiebacks will likely terminate into the competent glacial soils. For use in design of the performance anchors, we estimate an allowable grout to soil adhesion of 2,000 pounds per square foot (psf). This value is presented for planning purposes only and should be confirmed or modified using the data obtained from the performance testing prior to production tieback installation.

Anchor Installation and Testing: The contractor should be responsible for using equipment suited for the site conditions. It is possible that the soils will be stable so that open-hole drilling may be feasible for installing the tie-back anchors but the contractor should be prepared to case the holes if the soils start to cave or flow. 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. Five percent of the anchors, but a minimum of two, should be performance tested to 200 percent of the anchor design capacity. The performance tests 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. Final soil adhesion design values will be based on these tests. The test location should be determined in the field, based on soil conditions observed during anchor installation. All production tiebacks should be proof-tested to at least 130 percent of design capacity. The tieback testing program should be reviewed and monitored by NGA.

Other Retaining Walls

Final grading and development plans were not available at the time this report was prepared but retaining walls may be implemented as part of the proposed development. The lateral pressure acting on subsurface retaining walls is dependent on the nature and density of the soil behind the wall, the amount of lateral wall movement which can occur as backfill is placed, wall drainage conditions, the inclination of the backfill, and other possible surcharge loads.

For walls that are free to yield at the top at least one thousandth of the height of the wall (active condition), soil pressures will be less than if movement is limited by such factors as wall stiffness or bracing (at-rest condition). We recommend that walls supporting horizontal backfill and not subjected to hydrostatic forces be designed using a triangular earth pressure distribution equivalent to that exerted by a fluid with a density of 40 pcf for yielding (active condition) walls, and 60 pcf for non-yielding (at-rest condition) walls. A seismic design loading of 8H in psf 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 maximum 2H:1V backfill inclinations and do not account for additional surcharge loads. Additional lateral earth pressures should be considered for surcharge loads acting adjacent to subsurface walls and within a distance equal to the subsurface height of the wall. This would include the effects of surcharges such as traffic loads, floor slab and foundation loads, or other surface loads. We are available to provide consultation regarding additional loads on retaining walls during final design, if needed.

Retaining wall foundations should be pin pile supported. As such lateral pressures on walls may be resisted by passive resistance acting on the below-grade portion of the foundation, as discussed in the pipe pile foundations section 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 thin loose lifts and compacting it with small, hand-operated compactors within a distance behind the wall equal to at least one-half the height of the wall. The thickness of the loose lifts should be reduced to accommodate the lower compactive energy of the hand-operated equipment. The recommended level of compaction should still be maintained.

Permanent drainage 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 drainage systems.

Mechanically stabilized earth retaining walls may also be a feasible option for the proposed retaining structures along the north side of the proposed driveway or to replace the existing block wall along the south side of the driveway. We should be retained to provide a specific design and recommendations, as plans are finalized.

Structural Fill

General: Fill placed beneath foundations, slabs, pavements, 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. Sloping areas to receive fill should be benched to key the fill into the sloping grade. The benches should be level and a minimum of six feet wide.

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

Fill Placement: Following subgrade preparation, placement of structural fill may proceed. All 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 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.

Slab-on-Grade

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

Site Drainage

Surface Drainage: Final site grades should allow for drainage away from the top of the slopes and away from the planned residence structures. We suggest that the finished ground be sloped at a minimum gradient of three percent for a distance of at least 10 feet away from the buildings and top of the slopes. Runoff generated on this site should be collected and routed into a permanent discharge system away from steep slopes. This should include all downspouts and runoff generated on all hard surfaces and yards areas. Due to the sensitivity of the site and overall configuration, onsite stormwater infiltration should be considered marginally feasible. Under no circumstances should water be allowed to flow uncontrolled over the slopes. Water should not be allowed to collect in any area where footings or slabs are to be constructed. Under no circumstances should any water generated on this site be allowed to flow uncontrollably over the site slopes either during construction or on a permanent basis after the improvements are complete.

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 of the excavation and routed into a suitable outlet. We recommend that the residence down spouts and footing drains be tightlined to an appropriate discharge location at the bottom of the steep slopes or into the drainage system on the road.

We recommend the use of footing drains around structures. Footing drains should be installed at least one foot below planned finished floor elevation. The drains should consist of a minimum four-inch-diameter, rigid, slotted or perforated, PVC pipe surrounded by free-draining material wrapped in a filter fabric. We recommend that the free-draining material consist of an 18-inch-wide zone of clean (less than three-percent fines), granular material placed along the back of walls.

Washed rock is an acceptable drain material, or a drainage composite may be used instead. The free-draining material should extend up the wall to one foot below the finished surface. The top foot of soil should consist of low permeability soil placed over plastic sheeting or building paper to minimize the migration of surface water or silt into the footing drain. Footing drains should discharge into tightlines leading to an appropriate collection and discharge point with convenient cleanouts to prolong the useful life of the drains. Roof drains should not be connected to wall or footing drains.

USE OF THIS REPORT

NGA has prepared this report for **Maile Intrachat, Hoa Hoang,** and their agents, for use in the planning and design of the development planned on this site only. The scope of our work does not include services related to construction safety precautions and our recommendations are not intended to direct the contractors' methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design. There are possible variations in subsurface conditions between the explorations and also with time. Our report, conclusions, and interpretations should not be construed as a warranty of subsurface conditions. A contingency for unanticipated conditions should be included in the budget and schedule. This report is preliminary, and therefore, we recommend that we be retained to review the project plans after they have been developed to determine that recommendations in the report were incorporated into project plans.

All people who own or occupy homes with hills should realize that landslide movements are always a possibility. The landowner should periodically inspect the site slopes, especially after a winter storm. If distress is evident, a geotechnical engineer should be contacted for advice on remedial/preventative measures. The probability that landsliding will occur is substantially reduced by the proper maintenance of drainage control measures at the site (the runoff from the roofs should be led to an approved discharge point). Therefore, the homeowner should take responsibility for performing such maintenance. Consequently, we recommend that a copy of our report be provided to any future homeowners of the property if the home is sold.

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 during the work differ from those anticipated, and to evaluate whether or not earthwork and foundation installation activities comply with contract plans and specifications. We should be contacted a minimum of one week prior to construction activities and could attend pre-construction meetings if requested.

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

o-o-o

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

Sincerely,

NELSON GEOTECHNICAL ASSOCIATES, INC.

Alex Rinaldi

20120239

Alex B. Rinaldi

Alex B. Rinaldi, LG
Project Geologist

KHALED M. SHAWISH

35215

1.14.2022

Khaled M. Shawish, PE
Principal

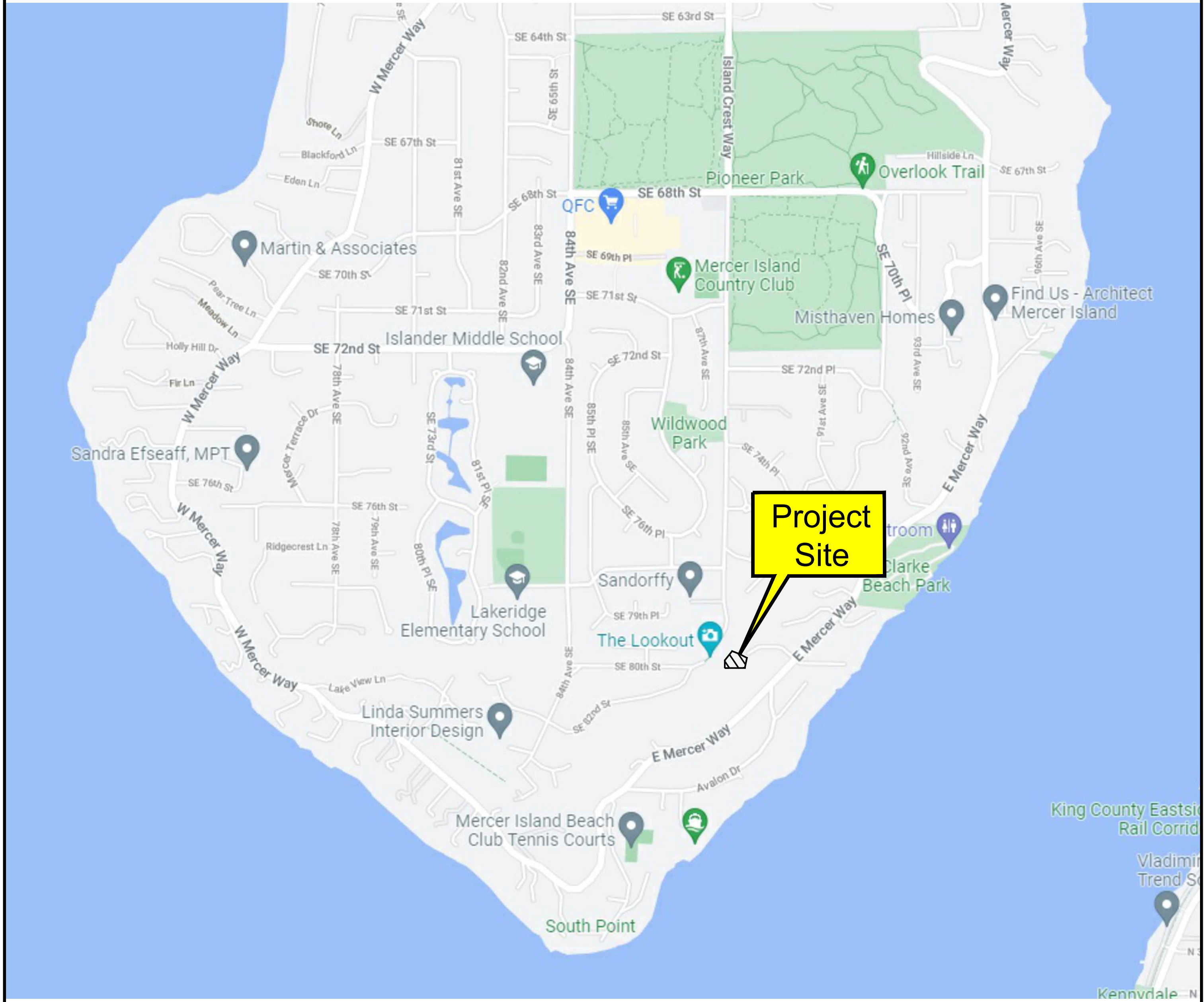
ABR:KMS:dy

Seven Figures Attached

VICINITY MAP

Not to Scale

N



Mercer Island, WA

Project Number 1276521	Intrachat-Hoang Residence Development Vicinity Map	NELSON GEOTECHNICAL ASSOCIATES, INC	No.	Date	Revision	By	CK
Figure 1		NGA www.nelsongeotech.com **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	1	12/6/21	Original	DPN	ABR

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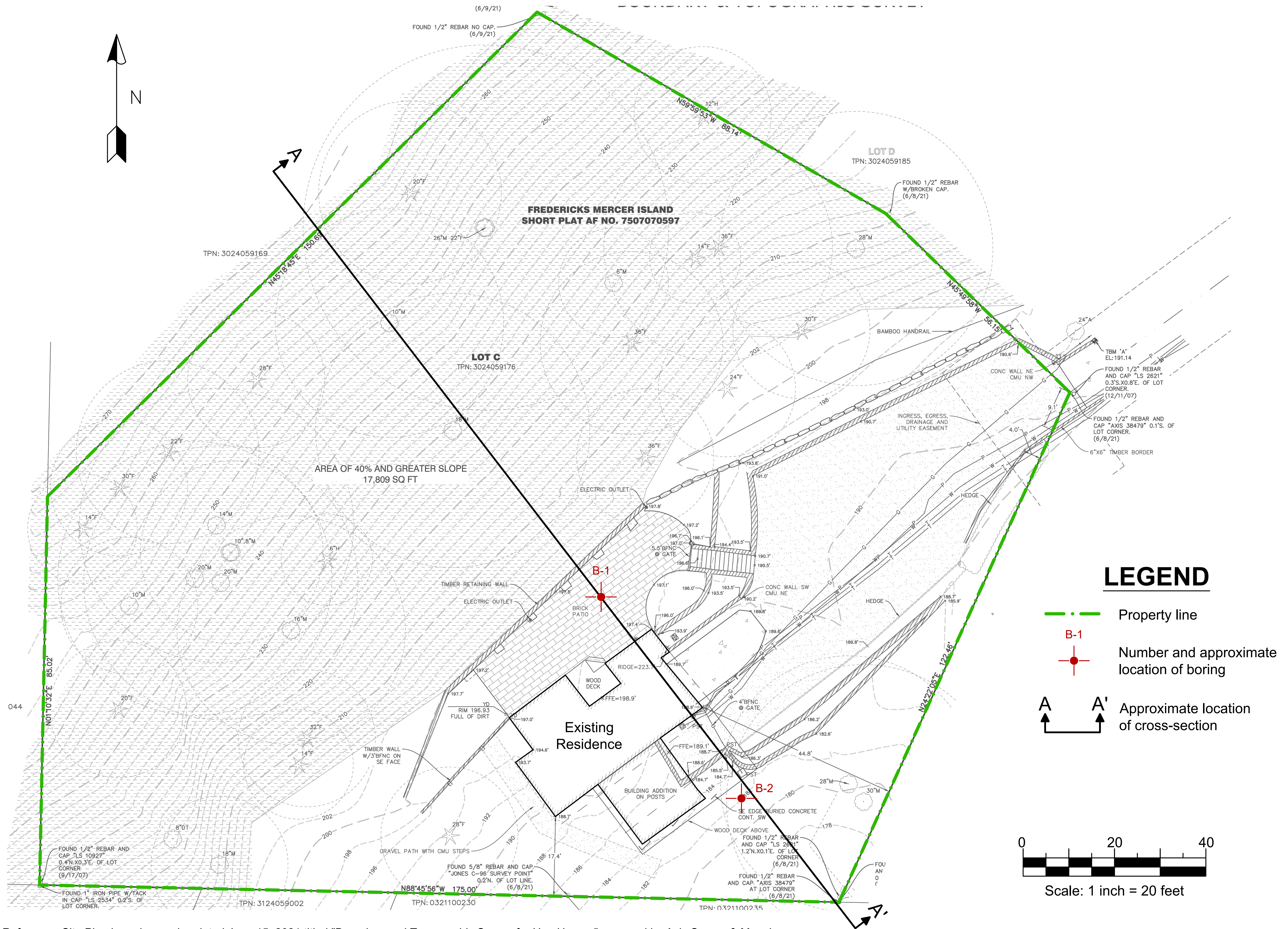

LEGEND

🟩 Property line

B-1 Number and approximate location of boring

A A' Approximate location of cross-section

0 20 40

Scale: 1 inch = 20 feet

Reference: Site Plan based on a plan dated June 15, 2021 titled "Boundary and Topographic Survey for Hoa Hoang," prepared by Axis Survey & Mapping.

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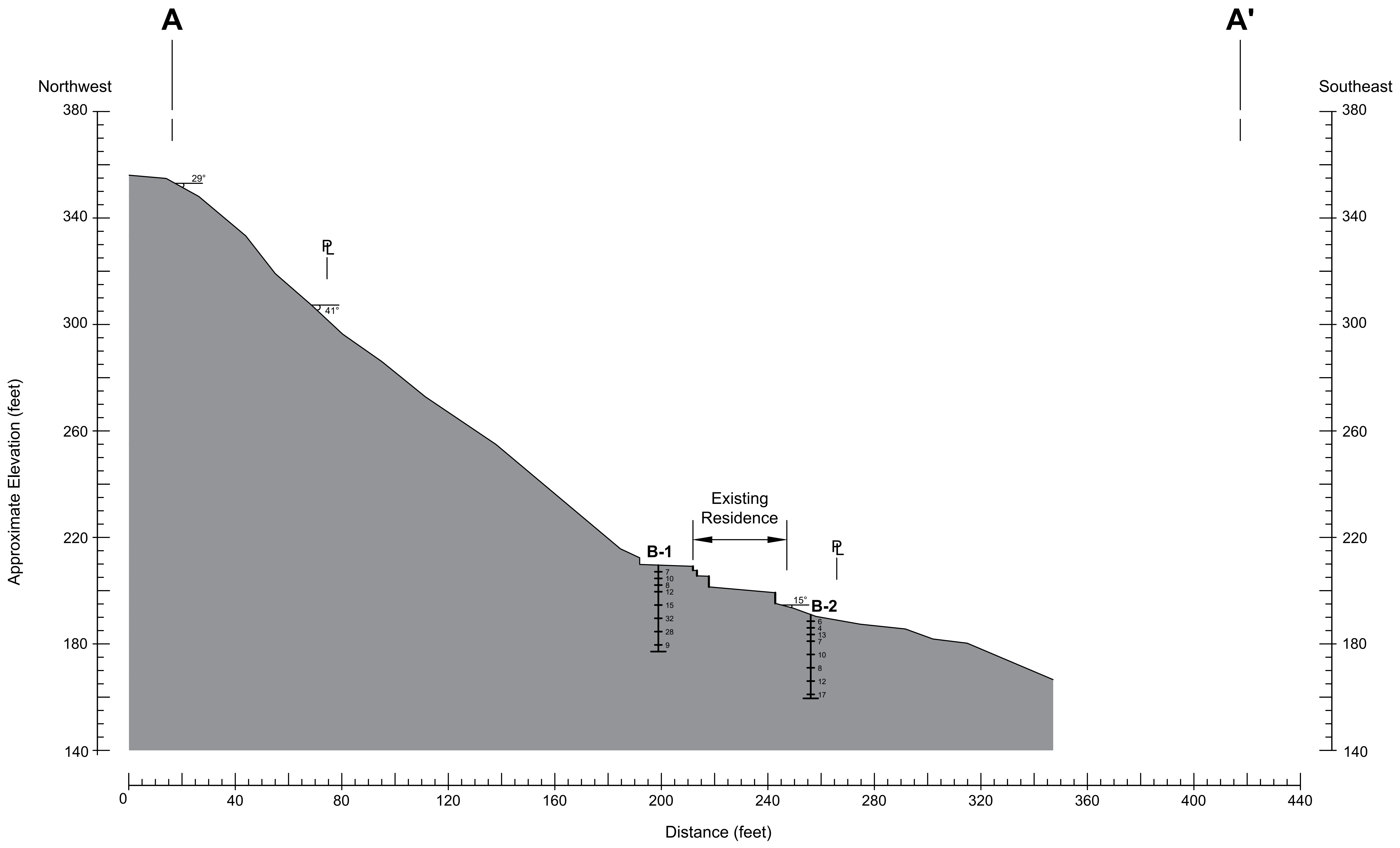
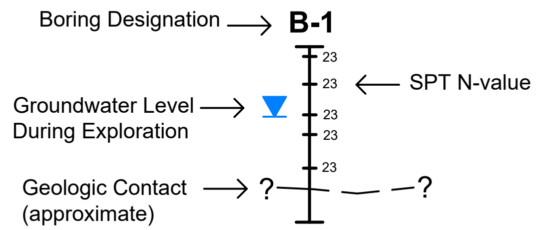
Intrachat-Hoang Residence
Development
Site Plan

Project Number
1276521

Figure 2



Exploration



NOTES:

1) Stratigraphic conditions are interpolated between the explorations. Actual conditions may vary.
2) Elevations are approximate.

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

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Intrachat-Hoang Residence
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Cross-Section A-A'

Project Number
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Figure 3

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

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Figure 4

Intrachat-Hoang Residence
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Soil Classification Chart

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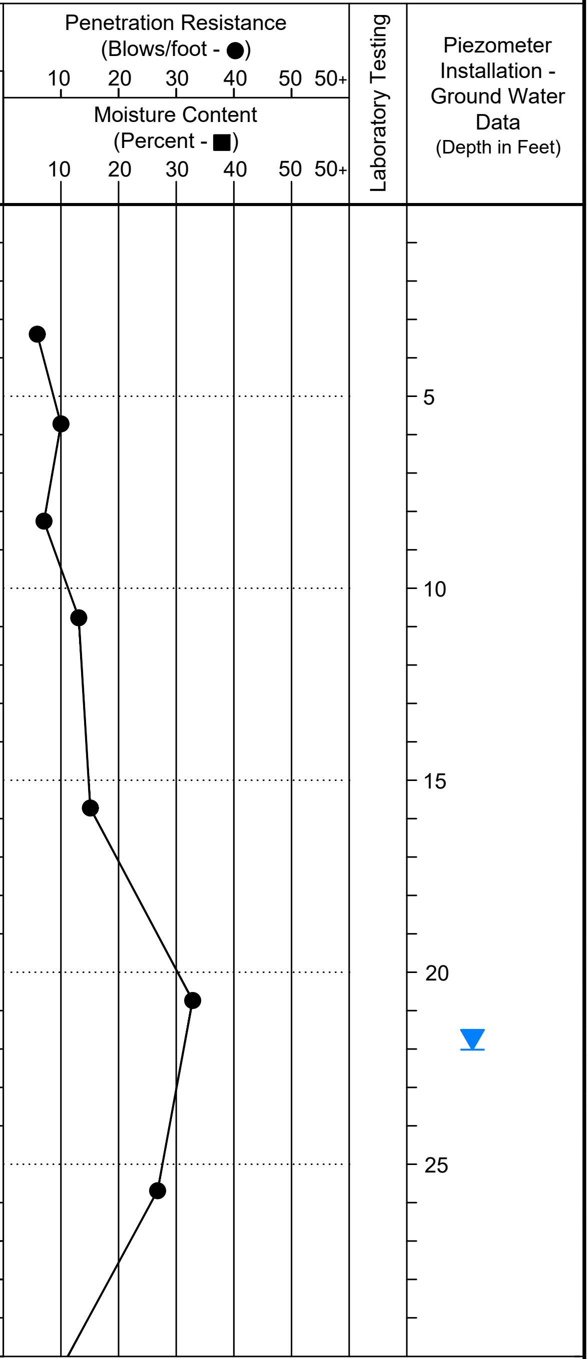
BORING LOG

B-1

Approximate Ground Surface Elevation: 197 ft

Soil Profile: Description	Graphic Log	Group Symbol	Sample Data: Blow Count	Sample Location (depth in feet)
Patio pavers and bedding sand Gray-brown to brown, silty fine to medium sand with gravel and trace roots (loose, moist) (**FILL**)				
			7	⬓
-becomes gray-brown, fine to medium sand with silt and trace gravel, loose to medium dense, moist to wet			10	⬓
-becomes loose, moist, no gravel			8	■
-becomes medium dense, moist to wet			12	■
			15	■
-becomes gray, dense, wet, with trace gravel			32	■
-becomes medium dense			28	■



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
1276521
Figure 5
Page 1 of 2

Intrachat-Hoang Residence
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Boring Log

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Logged by: ABR on 11/22/2021

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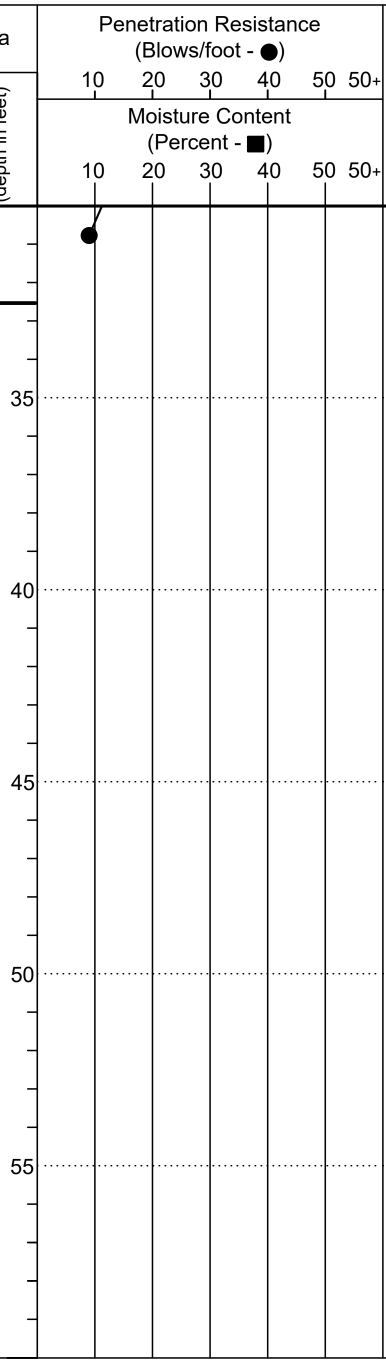
BORING LOG

B-1 (cont.)

Logged by: ABR on 11/22/2021

Soil Profile			Sample Data		Laboratory Testing	Piezometer Installation - Ground Water Data (Depth in Feet)
Description	Graphic Log	Group Symbol	Blow Count	Sample Location (depth in feet)		
-becomes loose			9	■		
Boring met refusal due to heave at 32.5 feet below existing grade on 11/22/2021. Groundwater seepage was encountered at 22.0 feet during drilling.						



Piezometer depths: 35, 40, 45, 50, 55

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
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Figure 5
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Intrachat-Hoang Residence Development
Boring Log

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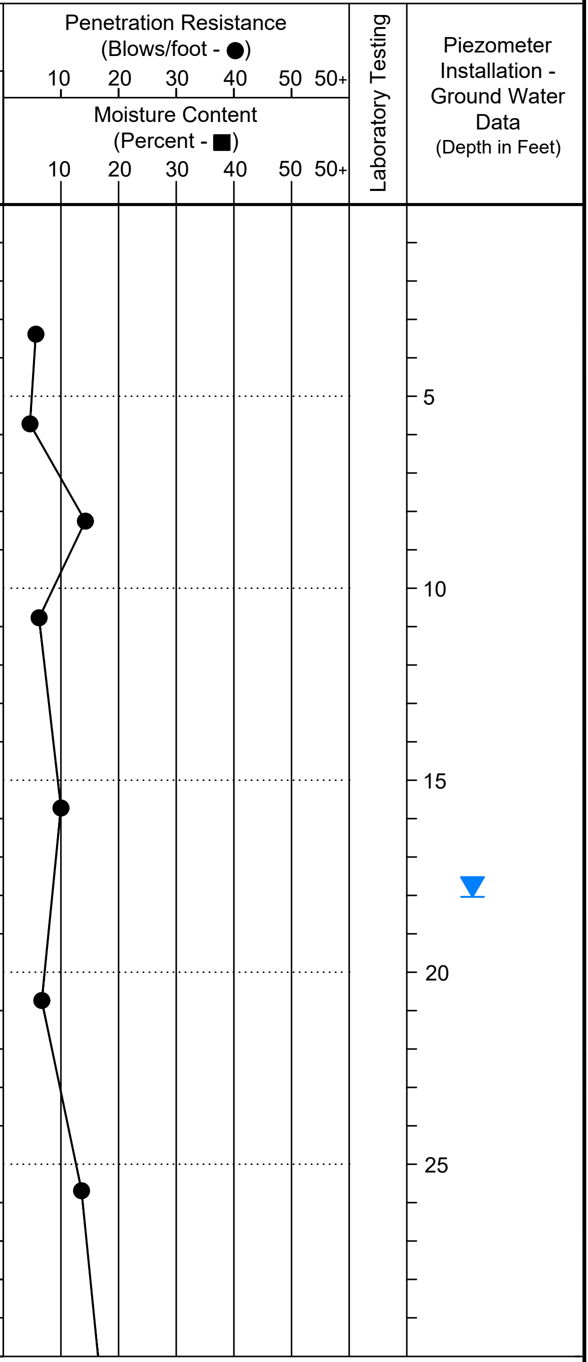
BORING LOG

B-2

Approximate Ground Surface Elevation: 197 ft

Soil Profile			Sample Data	
Description	Graphic Log	Group Symbol	Blow Count	Sample Location (depth in feet)
Grass Dark brown, silty fine to coarse sand with organics (loose, moist) (**topsoil** / **FILL**)			6	⬓
-becomes gray-brown, fine to medium sand with silt, very loose to loose			4	■
-becomes yellowish-brown to gray brown, medium dense, moist to wet, with trace gravel			13	■
-becomes loose, no gravel			7	■
-becomes orange-brown, fine to coarse sand with silt, gravel, and iron-oxide staining, loose to medium dense, wet			10	■
-becomes gray, fine to coarse sand with gravel and silt, loose			8	■
-becomes gravelly, fine to coarse sand with silt, medium dense			12	■



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
1276521
Figure 6
Page 1 of 2

Intrachat-Hoang Residence Development
Boring Log

NELSON GEOTECHNICAL ASSOCIATES, INC

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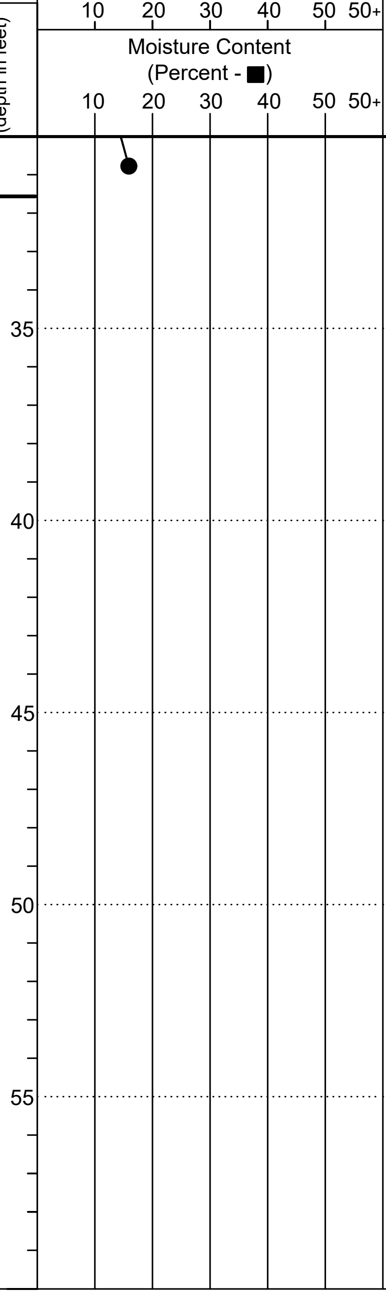
BORING LOG

B-2 (cont.)

Soil Profile			Sample Data		Penetration Resistance (Blows/foot - ●) / Moisture Content (Percent - ■)	Laboratory Testing	Piezometer Installation - Ground Water Data (Depth in Feet)
Description	Graphic Log	Group Symbol	Blow Count	Sample Location (depth in feet)			
-becomes fine to coarse sand with gravel and silt			17	■	●		
Boring met refusal due to heave at 31.5 feet below existing grade on 11/22/2021. Groundwater seepage was encountered at 18.0 feet during drilling.							



LEGEND

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

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

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

Logged by: ABR on 11/22/2021

Project Number
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Figure 6
Page 2 of 2

Intrachat-Hoang Residence Development
Boring Log

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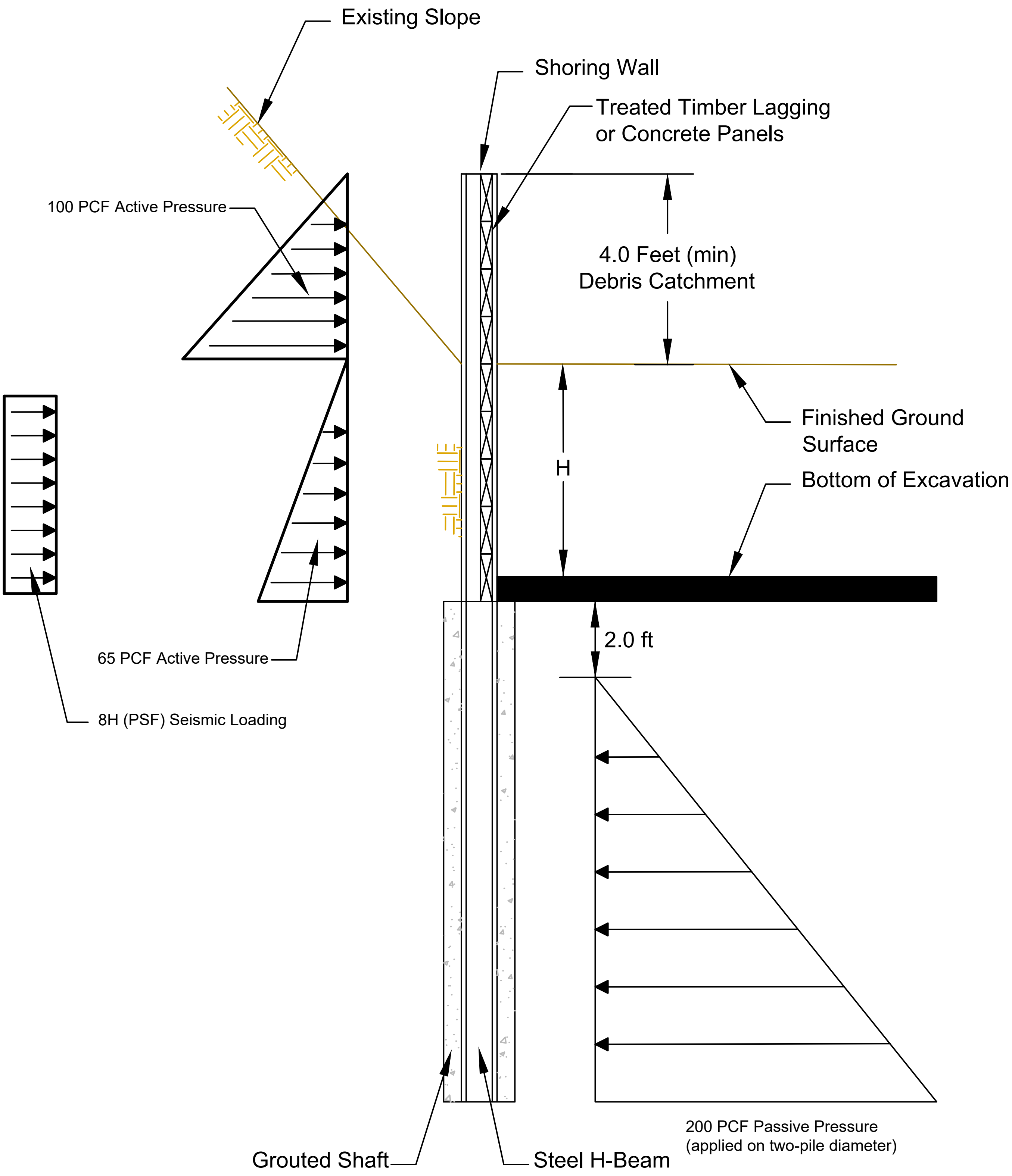
Conceptual Soldier Pile Wall Detail and Loading Design

NOT FOR CONSTRUCTION USE



NOT TO SCALE

Project Number
1276521

Figure 7

Intrachat-Hoang Residence Development
Conceptual Shoring Wall Detail and Loading Design

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