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Mr. Daniel Bretzke
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Geotechnical Engineering Evaluation
Harbor Avenue SW Site
27XX Harbor Avenue SW
Seattle, Washington
NGA File No. 986517

Dear Mr. Bretzke:

We are pleased to submit this report titled "Geotechnical Engineering Evaluation – Harbor Avenue SW Site – 27XX Harbor Avenue SW – Seattle, Washington." This report documents our surface and subsurface explorations within the site, and provides general recommendations for future site development. Our services were completed in general accordance with the agreement which was authorized by the City of Seattle on March 1, 2017.

The trapezoidal property is situated on terraced topography, which descends moderately to steeply from a gently sloping upper bench area within the western portion of the property to a relatively level lower area within the eastern portion of the site along Harbor Avenue SW. The property is currently undeveloped, and is bordered to the south by an existing commercial building. We understand that this property is currently owned by the City of Seattle. We understand the City may sell this property and therefore retain us to evaluate the site and provide this report to be used as part of the sale document package. Specific future site grading, utility, or stormwater management plans are not known at this time. We therefore should be retained to review final development plans prior to applying for a permit.

We performed two geotechnical borings using a track-mounted drill rig along within one backhoe-excavated test pit in the areas of potential development and within the steeply sloping areas. Our explorations generally encountered clay soils which we interpreted as native glaciolacustrine deposits in the upper areas of the site, and interbedded silty fine to medium sand, clay, and fine to coarse sand with silt and fine gravel within the exploration in the lower area of the site to the east, which we interpret as native deposits of the Olympia interglaciation. Although the surface has been mapped as landslide deposits originating from scarps along SW Admiral Way to the west of the site, the core of the steep slope is inferred to consist primarily of native glacial and interglacial cohesive soils at depth.

The site is located within an area designated as a potential landslide area, as defined by the Seattle Department of Design Construction and Land Use. Previously documented landslides within the vicinity of the site include several small shallow colluvial landslides generally associated with hydrologic factors relating to the soils at the contact of the upper relatively permeable sand soils and the lower low permeability fine-grained soils. The site appears to be relatively stable under current conditions.

However, there is a potential for shallow to intermediate landslides to impact the site and neighboring properties, even in its current state. This is especially true during severe storms or seismic events. Total elimination of all risks and hazards associated with deep-seated types of landslides are generally not feasible and typically not economical. The recommendations presented in this report should aid in maintaining and/or improving the current stability conditions observed at the site and provide mitigation measures for the proposed development if such a shallow or intermediate landsliding event were to occur within the site. It should be known and accepted that these types of slides could also present a risk to portions of the proposed development even after incorporating the recommendations provided in this report.

Based on our site reconnaissance and explorations, we have concluded that development on this site should be feasible from a geotechnical standpoint, provided acceptance and understanding of risk associated with potential future earth movement being present within the site. Due to the relatively loose/soft nature of the upper soils within the lower eastern portion of the property, we recommend supporting any proposed structures on deep foundation systems consisting of drilled augercast piles in order to advance the structure loads through the upper, loose to medium dense, fine-grained and granular soils down to the more competent, native deposits at depth. Supporting structures on deep foundations will also provide mitigation measures to reduce the overall risk associated with shallow to intermediate slides that could potentially occur within the site.

To protect any proposed structures within the lower portion of the site from potential shallow to intermediate landslides that may originate from the moderate to steep slope area above, we have recommended that a soldier pile shoring/debris catchment wall be constructed along the toe of the steep slope area. Specific development plans were not available at the time this report was prepared, but we would anticipate retaining wall heights of up to 15 feet may be needed to support the upper western slope similar to the neighboring property to the south.

In the attached report, we have also provided general recommendations for site grading, slabs-on-grade, structural fill placement, retaining walls, erosion control, and drainage. These recommendations are preliminary in nature. We should be retained to review and comment on final development plans and observe the earthwork phase of construction.

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

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INTRODUCTION

This report presents the results of our geotechnical engineering evaluation of the Harbor Avenue SW Site project located at 27XX Harbor Avenue SW in Seattle, Washington, as shown on the Vicinity Map in Figure 1. The current parcel number on file for the property is 6911200225. The purpose of this study is to explore and characterize the surface and subsurface conditions within the site and to provide general opinions and recommendations for potential future site development.

The trapezoidal property is situated on terraced topography, which descends moderately to steeply from a gently sloping upper bench area within the western portion of the property to a relatively level lower area within the eastern portion of the site along Harbor Avenue SW. The property is currently undeveloped and moderately vegetated with mature trees, and is bordered to the south by an existing commercial building. Slope stability, site grading, foundation support, and drainage are the major geotechnical issues associated with this development. We understand that this property is currently owned by the City of Seattle and may be sold in the foreseeable future. Specific grading, utility, and stormwater management plans were not available at the time this report was prepared, but this report will be part of the sale documents package to aid potential future site owners in understanding the risks associated with development within this site.

To aid in our preparation of this report, we have been provided with a topographic and boundary survey titled "Topographic and Boundary Survey for City of Seattle – 2765 Harbor Avenue SW – PMA 4217 – Seattle, Washington," dated December 23, 2016 and prepared by Signature Surveying & Mapping, PLC.

SCOPE

The purpose of this study is to explore and characterize the site surface and subsurface conditions, to provide an assessment of the site's geologic hazards, and to provide our geotechnical opinions and preliminary recommendations regarding potential site development. Specifically, our scope of services includes the following:

1. A review of available soil and geologic maps of the area.
2. Exploring the subsurface soil and groundwater conditions with two drilled borings using a track-mounted drill rig. Drill rig was subcontracted by NGA.
3. Exploring the subsurface soil and groundwater conditions within the property with track-mounted, backhoe-excavated test pits. Equipment was also utilized to provide access and mobilization for the drill rig to the upper boring location, and was subcontracted by NGA.
4. Performing laboratory analyses on selected soil samples.
5. Mapping the conditions on the slope and evaluating current slope stability conditions.
6. Performing shallow hand explorations within the steep slope areas.
7. Performing numerical slope stability analysis, as needed.
8. Providing preliminary recommendations for building setback from the steep slope.
9. Providing preliminary recommendations for earthwork, including cuts and fills.
10. Providing preliminary recommendations for temporary and permanent slopes.
11. Providing preliminary recommendations for foundation support alternatives, slab on grade, and pavement subgrades.
12. Providing preliminary recommendations for deep foundation support, as needed.
13. Providing preliminary recommendations for permanent retaining and shoring walls.
14. Providing preliminary recommendations for site drainage and erosion control.
15. Providing preliminary estimates for the earthwork and foundation phases of a proposed development.
16. Documenting the results of our findings, conclusions, and recommendations in a written preliminary geotechnical report.

SITE CONDITIONS

Surface Conditions

The 0.46-acre site is located along the western side of Harbor Avenue SW, and is trapezoidal in shape, approximately 77 feet wide and up to 270 feet long on its east-west axis. The property is currently undeveloped. The site is bordered to the south by an existing commercial building. The eastern and western property lines are adjacent to an undeveloped easement extension of Fauntleroy Avenue SW, and Harbor Avenue SW, respectively. Topography within the site slopes gently up to the west for a distance of approximately 50 feet from Harbor Avenue SW, then slopes up at inclination of 25 degrees (47 percent grade) before reaching a gentle to moderately east-sloping terrace, about 50 feet in width, in the central section of the property. To the west of the central terrace, a 20-foot-long east-facing slope travels upward at an inclination of approximately 23 degrees (42 percent) before reaching the uppermost terrace which is composed of a gently east-facing slope and the western property line. The existing site layout is shown on the Site Plan in Figure 2. The existing site conditions, site topography, and interpreted subsurface conditions are presented as Cross Section A-A’ in Figure 3.

The property is generally covered with young and mature trees, grass, and underbrush. We did not observe significant signs of recent slope movement within the site. However, we did observe some indications of past surficial sloughing and erosion that appears to have occurred on the site slopes in the past. A narrow drainage channel with flowing water was observed along the southern property boundary. During our site visit on April 27, 2017, we observed minor to moderate groundwater flowing through this channel and continuing to flow through the lower eastern portion of the site onto Harbor Avenue SW.

Subsurface Conditions

Geology: The geologic units for this area are shown on The Geologic Map of Seattle -- A Progress Report, by Troost, K.G., Booth, D.B., Wisher, A.P., and Shimel, S.A. (USGS, 2005). The site is mapped as glaciolacustrine clay (Qvlc) (Lawton Clay) within the upper portion of the site with Olympia Interglaciation Beds (Qob) mapped within the central, sloping portion of the site. The lower section of the site, adjacent to Harbor Avenue SW is mapped as tideflat deposits (Qtf). The Lawton Clay is described as laminated to massive silt, clayey silt, and silty clay with scattered dropstones. Deposits of the Olympia Interglaciation are described as thinly interbedded sand, silt, gravel, and peat. Tideflat deposits are described as silt, sand, organic sediment and detritus, exposed along coastlines and now covered with anthropogenic fill. Our explorations generally encountered silt and clay soils in upper areas of the site, and interbedded silty fine to medium sand, clay, and fine to coarse sand with silt and fine gravel within the exploration in the lower area of the site to the east, consistent with the description of mapped deposits at depth.

Explorations

We visited the site on April 27, 2017 to explore the subsurface soil and groundwater conditions by drilling two exploratory borings and a test pit using a track-mounted drill rig and excavator, respectively. The approximate locations of our explorations are indicated 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 5. The logs of our borings are presented as Figures 5 and 6, and the log of the test pit is presented as Figure 7. The following paragraphs contain a general description of the subsurface conditions encountered in the explorations. For a detailed description of the subsurface conditions, the boring and test pit logs should be reviewed.

Boring B-1 within the steep slope area and Boring B-2 within the lower portion of the site of the site encountered approximately 15 to 20 feet of disturbed soils consisting of very loose/soft fine to medium sands with varying amounts of silt and gravel, and blue-gray, silt with clay and fine sand that we interpreted as old landslide/colluvium deposits. Below the landslide/colluvium soils in Borings B-1 and B-2, we encountered very stiff, blue-gray silt with varying amounts of clay and fine sand that we interpreted this unit as native Lawton Clay deposits mapped for the site. Underlying the native Lawton Clay soils in Boring B-2, we encountered medium dense to very dense, gray-brown fine to coarse sand with varying amounts of gravel and silt that we interpreted as native alluvium deposits. Boring B-1 was terminated within the native Lawton Clay deposits at a depth of 51.5 feet below the existing ground surface, while Boring B-2 was terminated within the native alluvium deposits at a depth of 46.5 feet below the existing ground surface.

Test Pit 1 within the upper terrace of the site in the west encountered 1.5 feet of surficial topsoil before encountering gray silt with clay, sand and organic debris in a soft to medium stiff condition that we interpreted as older landslide deposits/colluvium. Test Pit 1 was completed within the older landslide/colluvium deposits at a depth of 7.5 feet below the ground surface.

Hydrogeologic Conditions

Groundwater seepage was encountered at a depth of 4.0 feet in the upper, westernmost explorations within the site that we interpreted as perched groundwater. Boring B-1 in the central portion of the site encountered wet conditions at 25.0 feet below the ground surface, although water level was not directly measured. Boring B-2 encountered groundwater under artesian conditions within a gravel layer at a depth of 37.0 feet below the existing ground surface, pushing up to a depth of 15.0 feet below the existing ground surface. It is our opinion that the groundwater observed within Boring B-2 is part of the regional groundwater table. We do not anticipate that groundwater levels would fluctuate significantly through the year.

We observed water flowing through a drainage channel that trends along the southern property line from the upper western portion of the property down to the lower eastern portion of the property. Water within this drainage channel appears to originate from the upper western portion of the site and likely consists of surficial overland and seepage flow within the upper western area. This water ultimately flows through the lower eastern portion of the property to Harbor Avenue SW.

SENSITIVE AREA EVALUATION

Seismic Hazard

The 2015 International Building Code (IBC) seismic design section provides a basis for seismic design of structures. Since medium stiff to hard soils were generally encountered underlying 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 2015 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 – 2015 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	1.461	0.566	1.000	1.500	0.974	0.566

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, and greater West Seattle is contained within the Seattle Fault Zone (SFZ): an active, shallow region of seismicity within central Puget Sound. The latest recorded rupture within the SFZ has been dated to approximately 1,100 years before the present. The SFZ can produce a M6—7.5 earthquake on a recurrence interval of several hundred years. In our opinion, the risk of a surface fault rupture within this site is low.

An earthquake within the SFZ poses considerable risk for tsunami along shoreline and low-lying areas along the coastal areas of the city, especially in Elliott Bay. The Tsunami Hazard Map of the Elliott Bay Area, Seattle, Washington, by Timothy J. Walsh, et al. (WADNR, 2003) shows tsunami hazard associated with a modelled M7.3 event on the Seattle Fault. The site does not have an associated hazard; however, neighboring property to the east of Harbor Avenue shows a minor hazard with 0 – 1.6 feet of inundation. Adjacent low-lying areas within the Industrial Districts on the Duwamish Waterway are of highest risk for larger inundation.

Hazards associated with seismic activity also 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. It is our opinion that the very stiff/hard fine-grained soils and dense to very dense granular soils interpreted to underlie the site at depth have a low potential for liquefaction or amplification of ground motion. The recommendations presented in this report for supporting the new residences on deep foundations, along with debris protection, erosion control, and drainage recommendations, should reduce this potential.

The competent glacial and interglacial soils interpreted to form the core of the site slopes are considered stable with respect to deep-seated slope failures in a static condition. However, as discussed in the **Slope Stability** subsection of this report, potential deep-seated landslide movement could occur on the steep slope areas during a large seismic event.

Erosion Hazard

The criteria used for determination of the erosion hazard for affected areas include soil type, slope gradient, vegetation cover, and groundwater conditions. The erosion sensitivity is related to vegetative cover and the specific surface soil types, which are related to the underlying geologic soil units. The Soil Survey of King County Area, Washington, by the Soil Conservation Service (SCS), does not list the erosion hazard for the Seattle area. Based on our experience in the area and our observations in the field, it is our opinion that the site would have a moderate to high erosion hazard for areas where the soils are exposed.

Landslide Hazard/Slope Stability

The criteria used for the evaluation of landslide hazards include soil type, slope gradient, and groundwater conditions. The site is situated on terraced topography that descends steeply from two upper, gently easterly-sloping terraces in the west, to a lower section of the property in the east, adjacent to Harbor Avenue SW at gradients in the range of 23 to 25 degrees (42 to 47 percent).

Landslide and mass wasting related deposits have been recorded within the site on <u>The Geologic Map of Seattle -- A Progress Report</u>, by Troost, K.G., Booth, D.B., Wisher, A.P., and Shimel, S.A. (USGS, 2005). The map also records two extended scarps along both sides of SW Admiral Way, located above and to the west of the site.

Review of the previous geotechnical report prepared for the property to the south by Shannon & Wilson, Inc. and the regional study titled "Seattle Landslide Study," dated February 7, 2000 that was also prepared by Shannon & Wilson, indicate that at least five historically documented landslides have occurred on or within the vicinity of the subject property. The first occurred in February of 1948 at 3014 Harbor Lane

SW, and was a small, shallow colluvial-type landslide, reported to have been triggered by groundwater and surface water conditions. Two large, deep-seated landslides occurred at 3012 34th Avenue SW in January and February of 1956 and 1961, respectively. Soil creep and minor surface ruptures were documented at 3520 SW Admiral Way in January of 1986. A shallow colluvial-type slide occurred at 3012 Harbor Lane SW in April of 1991, and caused minor damage.

Landslides within this portion of West Seattle are generally associated with the presence of saturated, heterogeneous soils on moderate to steep slopes, which result in shallow colluvial modes of mass wasting. Saturation may be attained by groundwater conditions, precipitation, or a combination of these factors. Our borings encountered approximately 5.0 to 20.0 feet of soft to stiff fine-grained soils underlain by stiff to hard fine-grained soils and very dense granular soils at depth. Within our explorations, we observed some samples in the upper fine-grained soils appeared to be disturbed, indicating that some slope movement may have occurred within this site at some time. We did not observe signs of recent deep-seated slope failures on the property during our visit. However, localized areas of surface instability, erosion, and surface sliding appear to have occurred on the site slopes in the recent past.

Backwasting (movement of near-surface soil) through soil erosion processes or local surface slides is common to slopes, particularly where the soils are exposed to weathering. Normal surface erosion and shallow sloughing failures should be expected to continue on the steeply sloping portions of the site on a regular basis. Larger scale slides could occur within the site slope especially during a sizeable seismic event or as a result of heavy rainfall. Measures that would fully stabilize the steep slope area against deep-seated landslides would likely include significant shoring walls along the top and bottom of the steep slopes that would likely not be economical. However, the geotechnical recommendations provided in this report for deep-foundation support, retaining and debris wall support along the lower portion of the slope, erosion control, and other development considerations should reduce the potential impact of site development on the site slopes.

Slope Stability Analysis

The site slope within the proposed development area was analyzed for stability along Cross Sections A-A' for the existing conditions and the recommended shoring retaining wall support along the lower portion of the slope using the computer program Slope/W, by Geo-Slope International. Slope/W is a two-dimensional, limit equilibrium slope stability program that generates random potential failure surfaces or specific failure surfaces and determines their corresponding factors of safety with respect to failure. By generating a large number of random surfaces, a critical failure surface with the minimum factor of safety can be identified.

The slope stability analyses were performed using information gathered from the field explorations and soil properties were assigned to the soil layers to reasonably reflect their engineering characteristics. Stability analyses were performed localized to the areas along the cross section. Stability analyses were performed for non-seismic and seismic conditions for the existing conditions. A peak ground acceleration of 0.20g was used in the seismic analyses. The soil parameters used in our analyses, along with the results of the analyses, are presented in Figures 13 through 16.

Our slope stability analyses indicated the site is moderately stable with regards to shallow to intermediate landsliding events originating within the upper moderate to steep slope and extending down to the proposed development area under static conditions with a critical slip surface resulting in a factor of safety of 1.4. However, the critical slip surface for the anticipated seismic condition results in a factor of safety of 0.8 indicating a slope failure extending from the upper moderate to steep slope through the lower bench area. We also modeled the placement of an approximately 15-foot tall soldier pile shoring wall resisting approximate uniform surcharge loads of 1,600 PSF to protect the lower development area from such shallow to intermediate landslide events and support the lower portion of the steep slope area. The critical slip surfaces for this case achieved factors of safety greater than or equal to 1.5 and 1.2 for the static and seismic cases, respectively indicating relatively stable conditions.

LABORATORY ANALYSIS

We performed one grain size sieve analysis and four Atterberg Limit analyses on soil samples obtained from the explorations. The sieve analysis indicated that the soil sample in Boring B-2 at approximately 35 feet below the ground surface consisted of gravelly fine to coarse sand with silt. The sieve test result is presented in Figure 12. The Atterberg Limit test indicated that the soil in Boring B-1 at 10.0 feet is classified as a plastic silt with a Liquid Limit (LL) of 32 and a Plasticity Index (PI) of 65 while the samples tested from Borings B-1 at depths of 20.0 and 40.0, and Boring B-2 at a depth of 25.0 feet were classified as silt with LL in the range of 26 to 28 and PI in the range of 33 to 42. The Atterberg Limit results are presented in Figures 8 through 11.

CONCLUSIONS AND RECOMMENDATIONS

General

Based on our site reconnaissance and explorations, engineering analysis, and review of previous geotechnical work, we have concluded that development on this site should be feasible from a geotechnical standpoint, provided acceptance and understanding that risk associated with potential future earth movement is present within the site. Due to the variable and inconsistent density of the upper surficial soils within the lower proposed development area, we recommend supporting new structures on a deep foundation system consisting of drilled augercast piles in order to advance the structure loads through the upper loose to medium dense surficial soils down to the more competent native deposits at depth. This is further described in the **Deep Foundations** subsection of this report. The recommended debris wall will also provide mitigation measures to reduce the overall risk associated with shallow to intermediate slides that could potentially occur within the site. We should be retained to review the project plans prior to applying for a permit, and to monitor earthwork and foundation system installation during construction.

We did not observe signs of recent deep-seated slope failures on the property during our visit. However, we did observe indications of past sloughing and erosion within the steep slope area within the site. The proposed development area appears to be relatively stable under static conditions. The site is located within an area of numerous previous known landslides generally associated with saturation of the upper surficial soils. There is potential for shallow to deep-seated landslides to impact the site and neighboring properties within the vicinity of the site. This is especially true during heavy rain events or a large seismic event, as indicated by the slope stability analysis. To mitigate the risks associated with the shallow to intermediate rotational landslides on the proposed development originating from the upper western portion of the site, we recommend constructing a soldier pile shoring/debris wall with tiebacks to buttress the toe of the steep east-facing slope to the west of any proposed site development. This is further described in the **Shoring Wall** subsection of this report.

Total elimination of all risks and hazards associated with deep-seated types of landslides is not feasible and is not economical. Such measures would likely include extensive shoring and retaining walls both along the upper and lower portions of the slope both within and outside of the subject site. The recommendations presented in this report should aid in maintaining and/or improving the current stability conditions observed at the site and provide mitigation measures for the proposed structures if shallow or intermediate landsliding events were to occur within the site. It should be known and accepted that these types of slides could also present a risk to the proposed development even while incorporating the recommendations provided in this report.

We recommend that if slabs-on-grade are utilized in the lower portions of the proposed structures that the slabs be designed as structural slabs and be supported on the deep foundation system. All foundation elements should be tied together to form a rigid unit. Other hard surfaces, such as paved areas or walkways that are supported on the existing soil have some risk of future settlement, cracking, and the need for maintenance. To reduce this risk, we recommend over-excavating a minimum of 1.5 feet of the upper soil from slab and pavement areas and replacing this material with compacted pit run or crushed rock structural fill underlain by a fabric and geogrid. This is further discussed later in this report. This recommendation is only for hard surfaces to be supported on grade and does not apply for the lower floor structural slab. Even with the recommended treatment, some settlement of the underlying loose material should be anticipated.

The control of surface and near-surface water is very important for the long-term stability of the site and steep slopes. The site slopes should be protected from erosion. Stormwater runoff should be collected into permanent catch basins or yard drains and routed away from the structures and the site slopes. This is further described in the **Site Drainage** subsection of this report. Specific recommendations for erosion control are presented in the **Erosion Control** subsection of this report. 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 temporary and final site grading be designed to direct surface water away from the structures and away from the steep slopes.

All grading operations and drainage improvements planned as part of this development should be designed and implemented in a matter that enhances the stability of the steep slope, not reduces it. Excavation spoils should not be stockpiled near the slope or be allowed to encroach on the slopes. Future vegetation management on the slope should be the subject of a specific evaluation and a plan approved by the City of Seattle. 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 near-surface soils encountered within our explorations are considered moisture sensitive and will disturb easily when wet. We recommend that construction take place during extended periods of dry weather if possible. If construction takes place during wet weather, additional expenses and delays should be expected due to the wet conditions. Additional expenses could include the need to export on-site soil, the import of clean, granular soil for fill, and the need to place a blanket of rock spalls or crushed rock in the construction traffic areas and on exposed subgrades prior to placing structural fill or structural elements. Also, wet weather grading carries a significant risk of causing slides on the property.

We recommend that NGA be retained to review final project plans and provide consultation regarding structure placement, setback distances, site grading, drainage plans, and foundation support. 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.

Erosion Control and Slope Protection

The erosion hazard for the on-site soils is considered moderate to severe, 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 flowing over the site slopes and off site. Disturbed areas should be replanted with vegetation at the end of construction. The vegetation should be maintained until established. Final grading should incorporate appropriate erosion control measures to route stormwater runoff away from the top of slopes and to appropriate discharge locations. The erosion potential for areas not stripped of vegetation should be low to moderate.

All runoff generated within the site, including roof downspouts, yard areas, and hard surfaces should be collected into catch basins and yard drains and tightlined into an approved stormwater management system. Under no circumstances should runoff be allowed to flow over the slope either during construction or after construction has been completed.

Protection of the slopes should be performed as required by the City of Seattle. Specifically, we recommend that the slopes 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 the steep slope, such as excavation spoils 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 be preserved on the slopes. Replacement of vegetation should be performed in accordance with the City of Seattle code. Under no circumstances should water be allowed to concentrate on the slopes. Any sloping areas disturbed during construction should be planted with vegetation as soon as practical to reduce the potential for erosion. Future yard waste, grass clippings, or any waste material should not be cast over the slope or piled above the slope. Stockpiled materials should not be placed on or near the site slopes.

Temporary and Permanent Slopes

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

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 2.0 Horizontal to 1.0 Vertical (2H:1V). If perched groundwater or loose soils are encountered, we would expect that flatter inclinations would be necessary. We recommend that cut slopes be protected from erosion. Measures taken may include covering cut slopes with plastic sheeting and diverting surface runoff away from the top of cut slopes. We do not recommend vertical slopes for cuts deeper than four feet, if worker access is necessary. We recommend that cut slope heights and inclinations conform to appropriate OSHA/WISHA regulations.

We recommend that the final slope inclinations for structural fill and the native soils be no steeper than 3H:1V. However, flatter inclinations may be necessary if silt or clay are exposed in permanent cut slopes. Final slopes should be vegetated and covered with jute netting. The vegetation should be maintained until it is established.

Site Preparation and Grading

Site preparation should consist of excavating the proposed structure footprints down to planned elevations. We recommend that the excavations associated with the proposed structure construction be kept to a minimum to limit disturbance to the site. Site preparation should also consist of stripping any organic topsoil and loose/soft soils in areas that will support foundations, slabs, pavement, or structural fill. The stripped material should be promptly hauled off-site. If the exposed soils are deemed loose/soft, they should be compacted to a non-yielding condition. Areas observed to pump or weave during compaction should be over-excavated and replaced with rock spalls. If significant surface water flow is encountered during construction, this flow should be diverted around areas to be developed and the exposed subgrade maintained in a semi-dry condition. In wet conditions, the exposed subgrade should not be compacted, as compaction of a wet subgrade may result in further disturbance of the soils. A layer of crushed rock may be placed over the prepared areas to protect them from further disturbance.

As mentioned earlier, the site soils are considered moisture sensitive and will disturb easily when wet. We recommend that earthwork construction take place during periods of extended dry weather, and suspended during periods of precipitation if possible. If work is to take place during periods of wet weather, care should be taken during site preparation not to disturb the site soils. This can be accomplished by utilizing large excavators equipped with smooth buckets and wide tracks to complete earthwork, and diverting surface and groundwater flow away from the prepared subgrades. Also, construction traffic should not be allowed on the exposed subgrade. A blanket of rock spalls should be used in construction access areas if wet conditions are prevalent. The thickness of this rock spall layer should be based on subgrade performance at the time of construction. For planning purposes, we recommend a minimum 1-foot thick layer of rock spalls.

Deep Foundations

Due to the presence of a thick layer of variable density upper surficial soils and groundwater within the proposed development areas, we recommend that any future structure be entirely supported on 16- to 24-inch diameter augercast piles extending a minimum of ten feet into the competent native granular soils found at depth. Based on our explorations, the piles need to be a minimum of 40 to 50 feet deep below current grades to satisfy this requirement. Augercast piles are installed with a hollow-stem auger advanced to the desired pile depth. After reaching a minimum recommended penetration into bearing soils, a pressure head is created when grout is pumped into the hollow stem of the auger before starting auger withdrawal. After the grout head is developed, withdrawal of the auger is timed to maintain the grout pressure head and avoid intrusion of loose soil into the sides of the pile excavation or discontinuity or “necking” of the pile. The actual volume of the concrete pumped into each pile is recorded and compared with the theoretical volume of the pile. Piles with a ratio of actual to theoretical great volume less than 1.1 should be re-drilled.

The augercast piles should provide the necessary vertical support for the structure as well as some lateral resistance. The success of this method will depend, in part, on site access for the drill rig and other equipment needed for pile installation. Obstructed piles should be relocated and/or additional piles installed. Some discussion on relocation of piles should be made with your structural engineer prior to start of drilling. It is usually best to make any changes while the drill rig is on site.

For preliminary design, we recommend that the piles penetrate a minimum of 10 feet into the underlying competent native granular soils to provide adequate end bearing and friction capacities. The competent native granular material was encountered at approximately 30 feet below the existing ground surface within the lower proposed development area. We present design friction, end bearing, and total axial compression capacities for 16-, 18-, and 24-inch augercast piles, installed to depths of 40- and 50-feet below the existing ground surface as recommended above, in the following table. The friction component should be used to resist uplift forces. The provided values do not include the weight of the piles. If the piles weight will be utilized to resist uplift forces, the buoyant unit weight and adequate safety factors should be used. The following capacities for design end bearing, design friction include factors of safety of 3.0 and 2.0, respectively. Piles should be spaced a minimum of four pile diameters from center of pile to center of pile to avoid grouping.

Pile Diameter (Inches)	Total Pile Depth (Feet)	Design Friction (Tons)	Design End Bearing (Tons)	Total Design Capacity (Tons)
16	40	10	20	30
16	50	15	25	40
18	40	13	25	38
18	50	16	30	46
24	40	20	50	70
24	50	25	60	85

Lateral resistance of the piles could be calculated based on equivalent fluid densities of 100 pcf for the upper surficial soils, and 200 pcf for the lower portion of the pile that is embedded into the native medium dense or better granular soils. The equivalent fluid density could be applied to two pile diameters. We should be retained to review pier design and observe augercast pile installation.

Shoring Wall

General: A shoring wall up to 15 feet may be needed to allow for excavation at the toe of the steep to moderate slope in that area to the west of the proposed development area. This wall will also be designed to support the toe of the slope from shallow to moderate landslides and act as a debris wall to protect the building against potential shallow failures on the upper slopes. As mentioned earlier, the shoring wall can be designed as a separate system and the building designed and constructed independent of the wall, or the shoring wall could be integrated in the building design. In the latter case, the shoring wall would likely be designed to resist vertical loads as well as lateral loads.

The most likely and feasible shoring system is a solider pile wall. A solider pile wall typically consists of a series of steel H-beams placed vertically at a certain distance from one another (typically six to ten feet). The beams are usually placed in drilled shafts that are filled with concrete or grout. The concrete shafts are typically embedded below the bottom of the planned excavation a distance equals one to two times the height of the cut to be shored, if tie-backs are not used. The steel beams are extended above finished ground surface to provide shoring capabilities for the cut. The beams are typically spanned by pressure treated timber lagging. The H-beam size, shaft diameter, shaft embedment, and pile spacing are dependent on the nature of the soils anticipated in the cut and at depth, cut height, drainage conditions, the need for tie-backs, and the final geometry.

Tie-backs extending into the native soils behind the shoring wall may be used to provide additional resistance for the taller portions of the shoring wall due to the planned wall loading (typically shoring walls over 12 feet in height require tie-backs).

Soldier Wall Design: The shoring wall should be designed by an experienced structural engineer licensed in the State of Washington. In many cases, shoring contractors have qualified structural engineers on board, or have a working relationship with qualified wall designers who can complete that wall design. In any case, the wall designer should be provided a copy of our report, and we should be retained to review the shoring wall design prior to construction.

We recommend that the shoring wall be designed using a uniform earth pressure of 1,600 psf for the exposed height of the retaining wall to support the toe of the steep slope against shallow to intermediate landsliding above the wall.

The shoring wall should extend above the top of the cut a minimum distance of five feet to allow for debris catchment should any sloughing or failures on the adjacent slope take place. We recommend that access to the back of the wall be provided so that debris can be periodically cleared from behind the wall. We recommend that the above ground portion of the wall be designed to resist an equivalent fluid density of 100 pcf.

The above loads should be applied on the full center-to-center pile spacing above the base of the cut. These loads could be resisted by passive resistance acting on the below-grade portion of the piles, and/or by tie-backs extending into the native soils behind the shoring wall. Lateral resistance of the subsurface portion of the piles could be calculated based on equivalent fluid densities of 100 pcf for the upper surficial soils, and 200 pcf for the lower portion of the pile that is embedded into the native medium dense or better granular soils. The equivalent fluid density could be applied to two pile diameters. This value incorporates a factor of safety of 2. We recommend that the soldier piles be embedded a minimum of five feet into the dense to very dense native granular soils to develop suitable kickout resistance from the passive soil pressure. We anticipate that the soldier piles would need to be extended down to approximately 35 feet below the existing ground surface to achieve this minimum embedment.

Shoring Wall Installation: The shoring wall should be installed by a shoring contractor experienced with this type of system. Due to potential caving conditions, we recommend that the soldier piles be installed similarly to the augercast piles as described in the **Deep Foundations** subsection of this report. We should be retained to observe shoring wall installation.

Tie-Backs

General: Based on the planned heights and overall loads on the shoring wall, we anticipate the need for tie-backs. The tie-backs should be designed as permanent, and meet double corrosion standards, and soil creep needs to be considered when determining their capacity. The contractor or the wall designer should submit for review working drawings showing their proposed method of tie-back support. The drawings shall include lists of materials to be used, sequence of operations and sufficient number details and notes to clearly illustrate the scope of work. Double corrosion protection for the anchor strands/bars shall be detailed on the drawings. We recommend that five percent of the anchors, but not 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 in these tests.

No-Load Zone: The anchor portion of all tie-backs must be located a sufficient distance behind the retained excavation 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 shall be defined as the observed landslide/colluvium soils that underlie the proposed development area and located within the upper portion of the moderate to steep slope areas. We encountered competent fine-grained soils exist below and beyond the proposed no-load zone. We recommend that the grouted portion of the anchor extend a minimum of ten feet beyond the no-load zone and founded in the competent native fine-grained soils but the overall measurement should be dictated by the proposed loads on the anchors. The exact location of the proposed shoring wall is unknown at the time this report was written, so we should be retained to evaluate the proposed wall location and estimate anticipated no-load zone dimensions and overall tieback anchor lengths. We also recommend that we monitor soil conditions during anchor installation in order to evaluate adequate penetration into these soils.

The anchor portion of the tie-back within the no-load zone should be immediately backfilled. The sole purpose of the backfill is to prevent possible collapse of the holes, loss of ground and surface subsidence. We recommend that the backfill consist of sand, gravel or a non-cohesive mixture. A sand cement grout should be utilized only if an acceptable form of bond breaker (such as plastic sheathing) is applied to the tie-back rod within the length of the no-load zone. A sand cement grout should not be allowed in the performance test anchors, as it will impact the test results.

Soil Design Values: For use in design of the performance anchors, we estimate an allowable grout to soil adhesion of 1,500 pounds per square foot. We expect that this value is low for sands and high for silts. 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 tie-back installation.

Tie-back Installation and Testing: The contractor should be responsible for using equipment suited for the site conditions. We do not recommend the use of open hole methods for the purpose of installing the tie-backs. Secondary grouting to increase soil adhesion may be used, however, 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 excavation. All production tie-backs should be proof-tested to at least 130 percent of design capacity. The tie-back testing program should be reviewed and monitored by NGA.

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 distribution equivalent to that exerted by a fluid with a density of 45 pounds per cubic foot (pcf) for yielding (active condition) walls, and 65 pcf for non-yielding (at-rest condition) walls. 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 backfill and are based on the assumption of a horizontal ground surface behind the wall for a distance of at least the subsurface height of the wall, and do not account for surcharge loads. Additional lateral earth pressures should be considered for surcharge loads acting adjacent to subsurface walls and within a distance equal to the subsurface height of the wall. This would include the effects of surcharges such as traffic loads, floor slab loads, slopes, or other surface loads. We could consult with you and your structural engineer regarding additional loads on retaining walls during final design, if needed.

The lateral pressures on walls may be resisted by passive resistance acting on the below-grade portion of the foundation such as the augercast piles. Recommendations for passive resistance to lateral loads are presented in the **Deep 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 eight-inch 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 lessened 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.

Structural Slabs

As mentioned earlier, we recommend that if a lower floor slab is utilized, that this slab be designed as a structural slab fully supported on the deep foundation system. We recommend that slabs be underlain by at least six inches of free-draining gravel with less than three percent by weight passing the 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 may be used to protect the vapor barrier membrane and to aid in curing the concrete; however, this sand layer is optional and is intended to protect the vapor barrier membrane during construction. Other slabs and hard surfaces that may be supported on the existing soils should be underlain by a minimum of two feet of railroad ballast in addition to the capillary break and vapor barrier.

Pavements

Pavement and other hard surface subgrade preparation, and structural fill placement where required, should be completed as recommended in the **Site Preparation and Grading** and **Structural Fill** subsections of this report. We recommend that the proposed pavement areas be over-excavated by a minimum of 18-inches and the entire resulting subgrade should be covered with a non-woven filter fabric overlain by a Tensar TX160 geogrid and backfilled with crushed rock. The pavement subgrade should be heavily compacted and proof-rolled with a heavy, rubber-tired piece of equipment, to identify soft or yielding areas that require repair prior to placing the pavement base course. We should be retained to observe the proof-rolling and recommend subgrade repairs prior to placement of the asphalt or hard surfaces.

Structural Fill

General: Fill placed beneath foundations, slabs, 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 excavated to expose native medium dense or better soil, and be free of any organics and standing water. Sloping ground to receive fill should be benched for stability prior to placing fill. The benches should be level and be a minimum of four six 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 fill should contain no more than five-percent fines (soil finer than U.S. No. 200 sieve, based on that fraction passing the U.S. 3/4-inch sieve). The use of the on-site soils as structural fill is not recommended. We should be retained to evaluate all material proposed for use as structural fill prior to construction.

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

Site Drainage

Surface Drainage: 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 structure and site slopes. All surface water should be collected by permanent catch basins and drain lines and be discharged to an approved stormwater management system. Collected water should be either directed into a permanent storm drain in the street or routed to the bottom of the slope via tightline pipes. In any case, stormwater from the site should not be infiltrated. Surface drains should be maintained separately and not be interconnected with foundation or wall drains.

Subsurface Drainage: If groundwater seepage is encountered or if excessive rainfall occurs during construction, we recommend that the contractor slope the bottom of the excavation and direct the water to ditches and small sump pits. The collected water can then be pumped to a suitable discharge point away from the slope or the discharge point should be located at the bottom of the slope. If located at the bottom of the slope, easements from the neighboring property owners would need to be received due to the toe of the steep slopes being located on neighboring properties.

We recommend using footing drains around the residence and behind all retaining walls. 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 3 percent fines), granular material placed along the back of the wall. Pea gravel is an acceptable drain material or drainage composite may also 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 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 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.

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.

SLOPE MONITORING

We also recommend that we be retained to periodically observe the steep slopes and evaluate the existing stability conditions, especially after a significant storm event. If any distress is observed, we can then provide recommendations for mitigation measures at that time.

USE OF THIS RERORT

This preliminary report has been prepared for Mr. Daniel Bretzke with the City of Seattle, and his agents, for use in the planning and design of the proposed 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 letter. 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 during the work differ from those anticipated, and to evaluate whether or not retaining wall and foundation support installation complies with our recommendations. We should be contacted a minimum of one week prior to construction activities.

All people who own or occupy homes on or near hillsides should realize that landslide movements are always a possibility. The landowner should periodically inspect the slope, 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 and all other hard surfaces 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.

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.

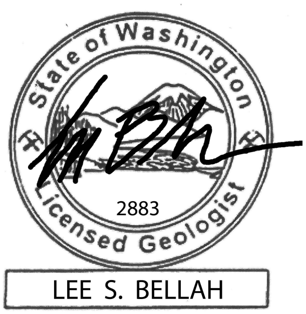
o-o-o

We appreciate the opportunity 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 T. Curd, GIT
Staff Geologist



Lee S. Bellah, LG
Project Geologist

Khaled Shawish, PE
Principal

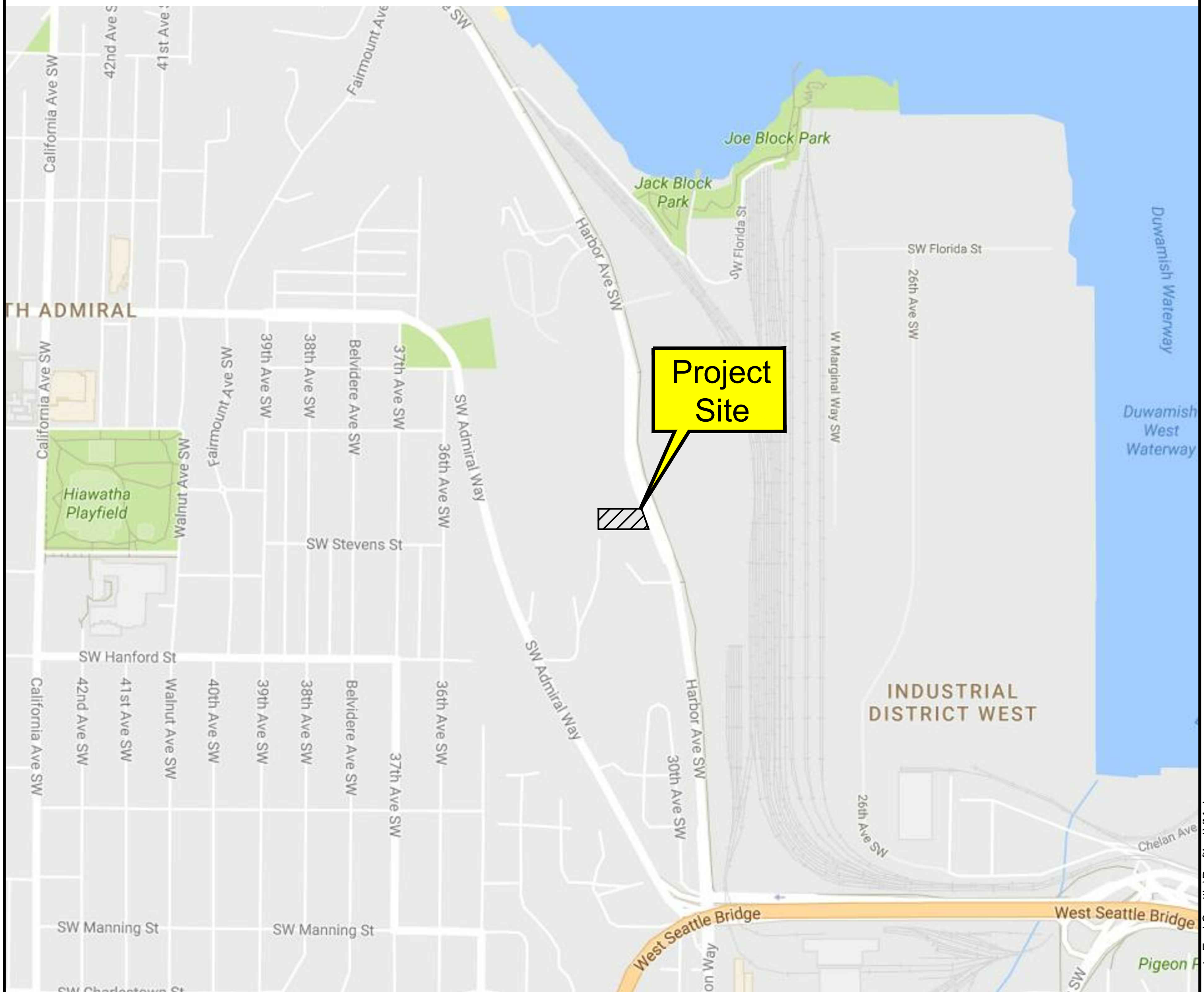
CTC:LSB:KMS:dy

Sixteen Figures Attached

VICINITY MAP

Not to Scale

N



Seattle, WA

Project Number 986517	Harbor Avenue SW Property Evaluation Vicinity Map	Nelson Geotechnical Associates, Inc. NGA Geotechnical Engineers & Geologists	No.	Date	Revision	By	CK
Figure 1		17311-135th Ave. NE, A-500 Woodinville, WA 98072 (425) 486-1669 / Fax 481-2510 Snohomish County (425) 339-1669 Wenatchee/Chelan (509) 665-7696 www.nelsongeotech.com	1	5/4/17	Original	DPN	CTC

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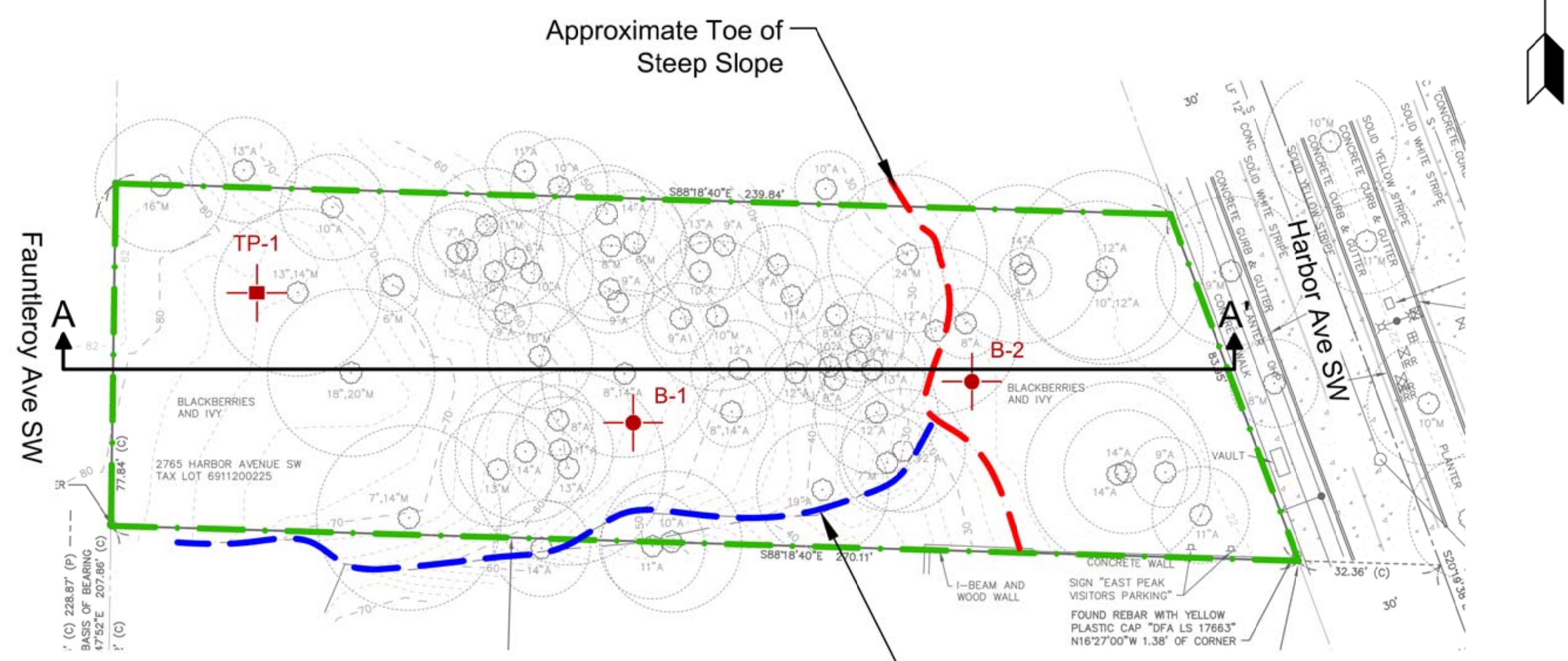
Site Plan



LEGEND

- Property line
- TP-1 — Number and approximate location of test pit
- B-1 — Number and approximate location of boring
- A — A' — Approximate location of cross-section

0 — 40 — 80

Scale: 1 inch = 40 feet

Reference: Site plan based on a plan dated December 23, 2016 titled "City of Seattle - 2765 Harvor Avenue SW - PMA 4217," prepared by Signature Surveying & Mapping.

Project Number	Harbor Avenue SW Property Evaluation Site Plan	Nelson Geotechnical Associates, Inc. Geotechnical Engineers & Geologists
986517		17311-135th Ave. NE, A-500, Woodinville, WA 98072, (425) 486-1669 / Fax 481-2510; Snohomish County (425) 337-1669, Wenatchee/Chelan (509) 665-7696, www.nelsongeotech.com
Figure 2		

No.	Date	Revision	By	CK
1	5/4/17	Original	DPN	CTC

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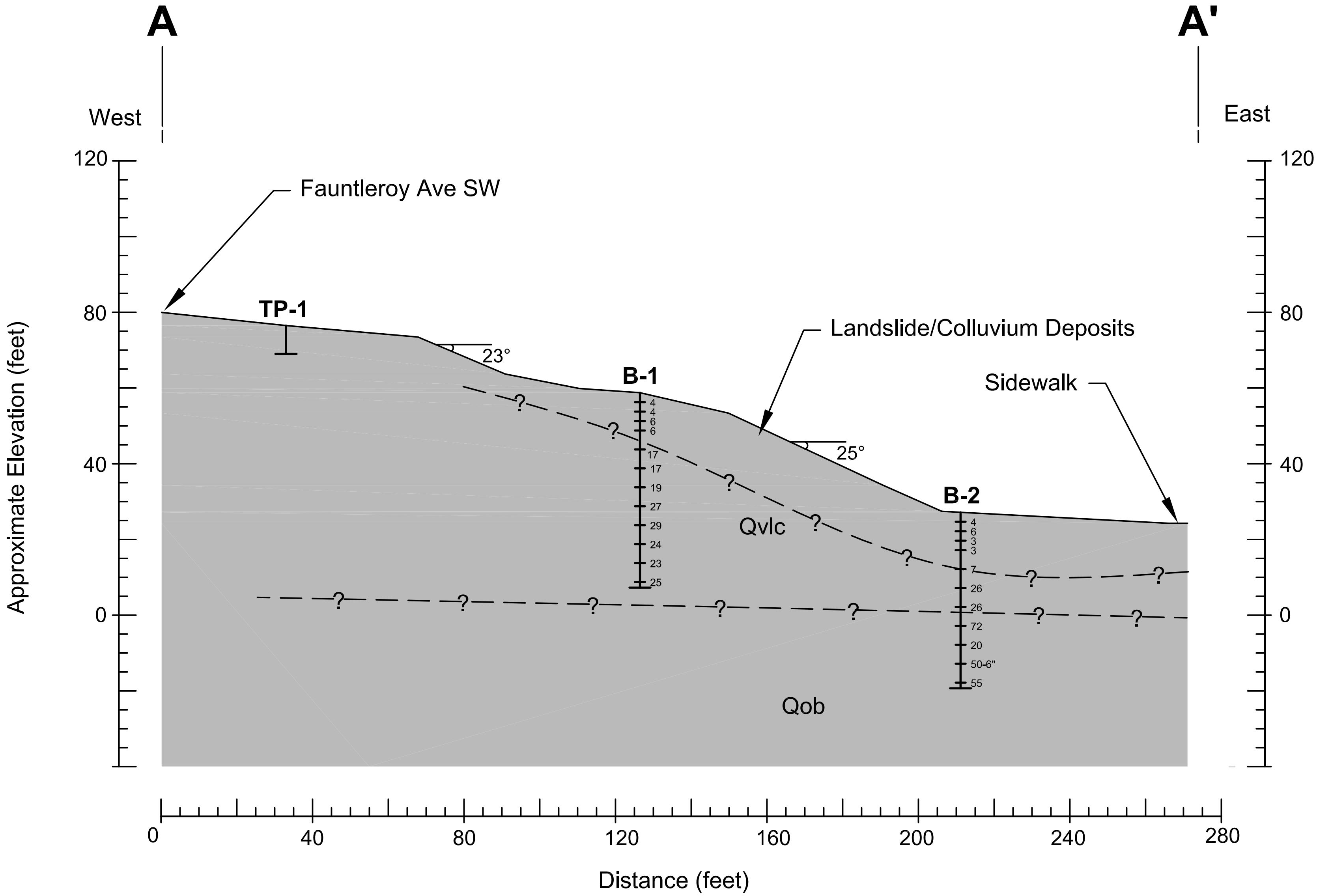

Exploration

Boring / Test Pit Designation ⟶ **B-1/TP-1**

SPT N-value

Groundwater Level During Exploration

Geologic Contact (approximate)

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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Project Number 986517	Harbor Avenue SW Property Evaluation Cross-Section A-A'
Figure 3	

NELSON GEOTECHNICAL ASSOCIATES, INC.
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(509) 665-7696

No.	Date	Revision	By	CK
1	5/4/17	Original	DPN	CTC

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, FLAT 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 986517	Harbor Avenue SW Property Evaluation Soil Classification Chart	NELSON GEOTECHNICAL ASSOCIATES, INC. GEOTECHNICAL ENGINEERS & GEOLOGISTS	No.	Date	Revision	By	CK
Figure 4		17311-135th Ave. NE, A-500 Woodinville, WA 98072 (425) 486-1669 / Fax 481-2510 Snohomish County (425) 337-1669 Wenatchee/Chelan (509) 665-7696 www.nelsongeotech.com	1	5/4/17	Original	DPN	CTC

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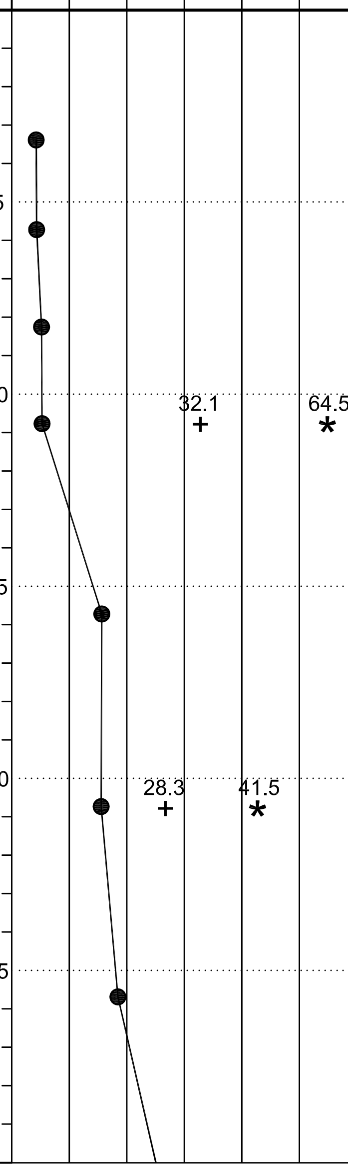
BORING LOG

B-1

Logged by: CTC on 4/27/17

Approximate Ground Surface Elevation: 64 ft.

Soil Profile: Description	Graphic Log	Group Symbol	Sample Data: Blow Count	Sample Location (depth in feet)	Penetration Resistance (Blows/foot - ●) / Moisture Content (Percent - ■)	Laboratory Testing	Piezometer Installation - Ground Water Data (Depth in Feet)
Dark brown, silty fine to medium sand with trace gravel (very loose to loose, moist to wet)		SM	4				
Blue-gray, Silt with clay and trace fine sand (soft to medium stiff, moist)**(Landslide Debris/Colluvium)**		MH	4	5			5
-becomes medium stiff			6				
-becomes blue and brown-gray, disturbed appearance with blocky inclusions and slickensides			6	10	32.1 + 64.5 *	A	10
-Becomes blue-gray silt with fine sand and clay (very stiff,moist)		ML	17	15			15
			17	20	28.3 + 41.5 *	A	20
-becomes wet			19	25			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 986517	Harbor Avenue SW Property Evaluation Boring Log
Figure 5	
Page 1 of 2	

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No.	Date	Revision	By	CK
1	5/4/17	Original	DPN	CTC

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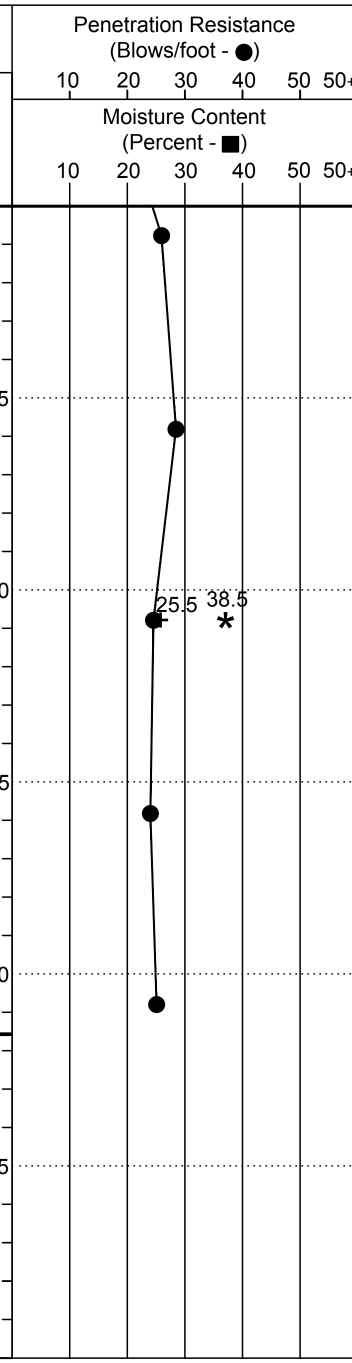
BORING LOG

B-1 (cont.)

Logged by: CTC on 4/27/17

Soil Profile: Description	Graphic Log	Group Symbol	Sample Data: Blow Count	Sample Location (depth in feet)	Penetration Resistance (Blows/foot - ●) / Moisture Content (Percent - ■)	Laboratory Testing	Piezometer Installation - Ground Water Data (Depth in Feet)
Blue and brown-gray, silt with trace fine sand and clay (very stiff, dry to moist)		ML	27				
			29	35			35
-with trace coarse sand			24	40	25.5 38.5	A	40
-becomes dark blue-gray			23	45			45
			25	50			50
Boring terminated below existing grade at 51.5 feet on 4/27/17. Groundwater seepage was not encountered during drilling				55			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 986517

Figure 5

Harbor Avenue SW Property Evaluation Boring Log

NELSON GEOTECHNICAL ASSOCIATES, INC.
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Snohomish County (425) 339-1669
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BORING LOG

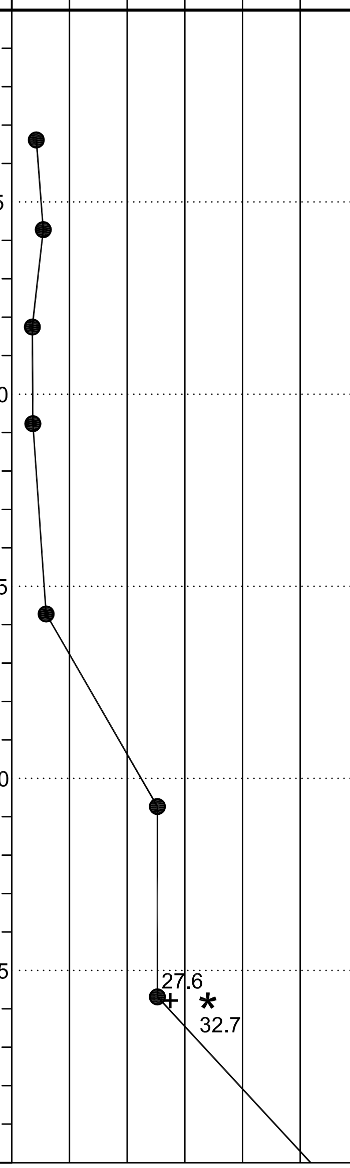
B-2

Approximate Ground Surface Elevation: 64 ft.

Description	Graphic Log	Group Symbol	Blow Count	Sample Location (depth in feet)	Laboratory Testing	Piezometer Installation - Ground Water Data (Depth in Feet)
Brown, silty fine to medium sand with roots (very loose to loose, wet) -becomes gray-brown, with 6" layer of sandy silt		SM	4			
Gray-brown, silty clay with trace fine sand and iron-oxide staining (medium stiff to stiff, moist)		CL	6	5		5
-peat lense Gray brown, clayey fine to medium sand with silt (very loose, wet)		SM	3			
Gray-brown, silty clay with trace iron-oxide staining (soft, wet) -becomes gray with 1" peat lenses		CL	3	10		10
Blue-gray, silty fine to medium sand with <1" peat lenses (loose, dry to moist) -no peat lenses		SM	7	15		15
Gray, silt with clay and trace fine to coarse sand (very stiff, wet)		ML	26 26	20 25	A	20 25

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

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



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	Harbor Avenue SW Property Evaluation Boring Log	NELSON GEOTECHNICAL ASSOCIATES, INC. GEOTECHNICAL ENGINEERS & GEOLOGISTS	No.	Date	Revision	By	CK
986517		17311-135th Ave. NE, A-500 Woodinville, WA 98072 (425) 486-1669 / Fax 481-2510	1	5/4/17	Original	DPN	CTC
Figure 6		Snohomish County (425) 339-1669 Wenatchee/Chelan (509) 665-7696 www.nelsongeotech.com					
Page 1 of 2							

Logged by: CTC on 4/27/17

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BORING LOG

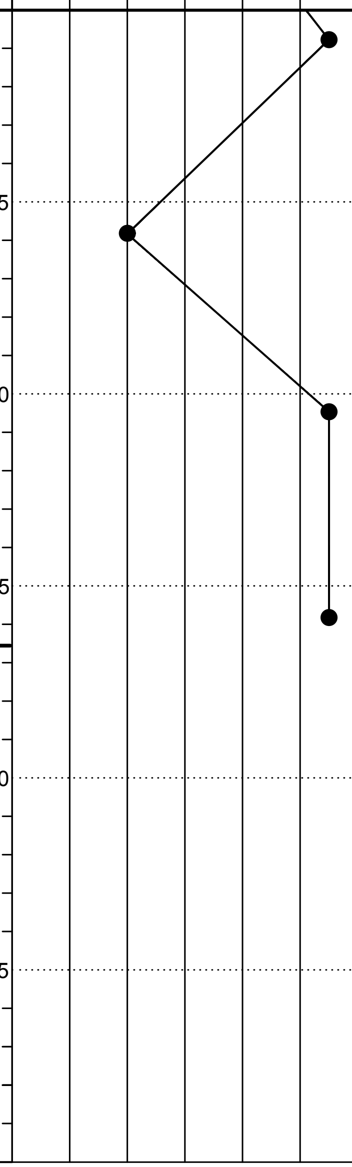
B-2 (cont.)

Soil Profile: Description	Graphic Log	Group Symbol	Sample Data: Blow Count	Sample Location (depth in feet)	Laboratory Testing
Gray-brown, fine to coarse sand with silt and fine gravel (very dense, wet)		SP-SM	72		
-becomes medium dense, gravelly			20	35	G
-becomes brown, no gravel, very dense			50-6"	40	
-becomes dark blue-gray			55	45	
Boring terminated below existing grade at 46.5 feet on 4/27/17. Groundwater seepage was not encountered during drilling				50, 55	

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

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

Piezometer Installation - Ground Water Data (Depth in Feet) 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
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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 986517

Figure 6

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Harbor Avenue SW Property Evaluation Boring Log

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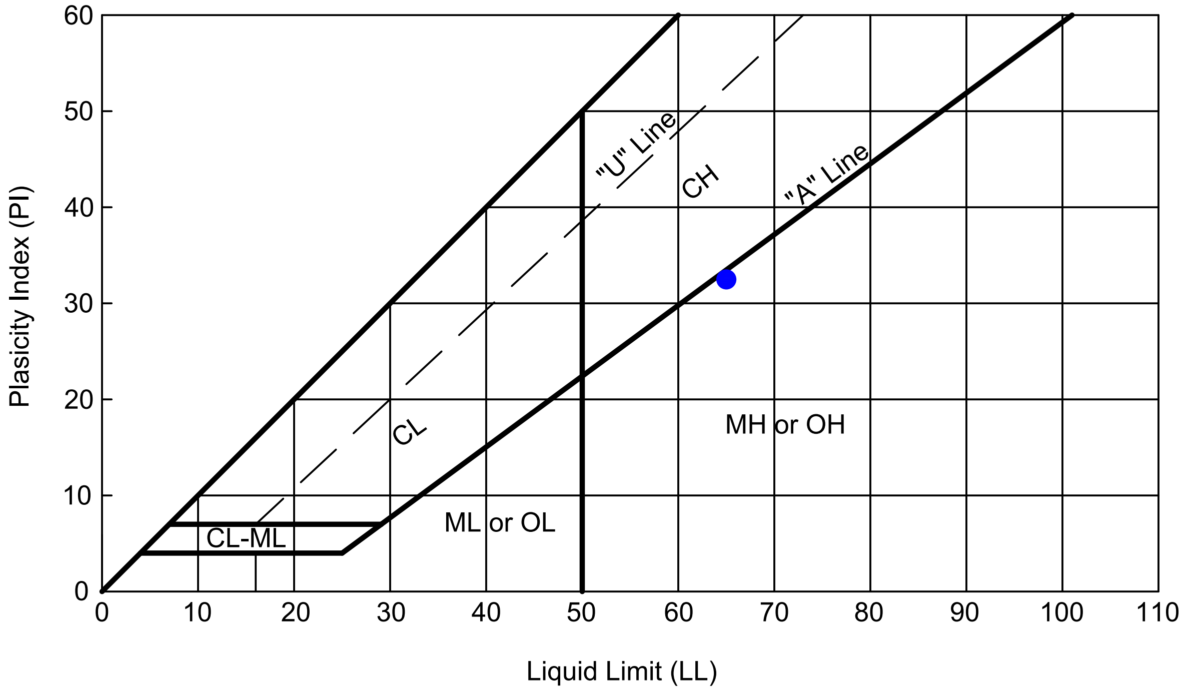
LOG OF EXPLORATION

DEPTH (FEET)	USC	SOIL DESCRIPTION
TEST PIT ONE		
0.0 – 1.5		(**TOPSOIL**)
1.5 – 3.0	ML	GRAY SILT WITH CLAY AND SAND AND TRACE ORGANIC DEBRIS (SOFT TO MEDIUM STIFF, MOIST TO WET)
3.0 – 3.5		(**BURIED TOPSOIL)**
3.5 – 7.5	ML	LIGHT BROWN-GRAY SILT WITH CLAY AND TRACE FINE SAND WITH IRON OXIDATION STAINING (MEDIUM STIFF-STIFF, WET) (**DISTURBED**)
		SAMPLES WERE COLLECTED AT 2.0, 4.0, AND 7.0 FEET GROUNDWATER SEEPAGE WAS ENCOUNTERED AT 4.0 FEET NO TEST PIT CAVING WAS ENCOUNTERED TEST PIT WAS COMPLETED AT 7.5 FEET ON 4/27/2017

Atterberg Limits



Symbol	Sample	Plastic Limit	Liquid Limit	Plasticity Index	USC Symbol
●	B-1 @ 10.0-11.5ft	32.1	64.5	32.4	MH

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Figure 8		17311-135th Ave. NE, A-500 Woodinville, WA 98072 (425) 486-1669 / Fax 481-2510 Snohomish County (425) 337-1669 Wenatchee/Chelan (509) 665-7696 www.nelsongeotech.com	1	5/26/17	Original	DPN	LSB

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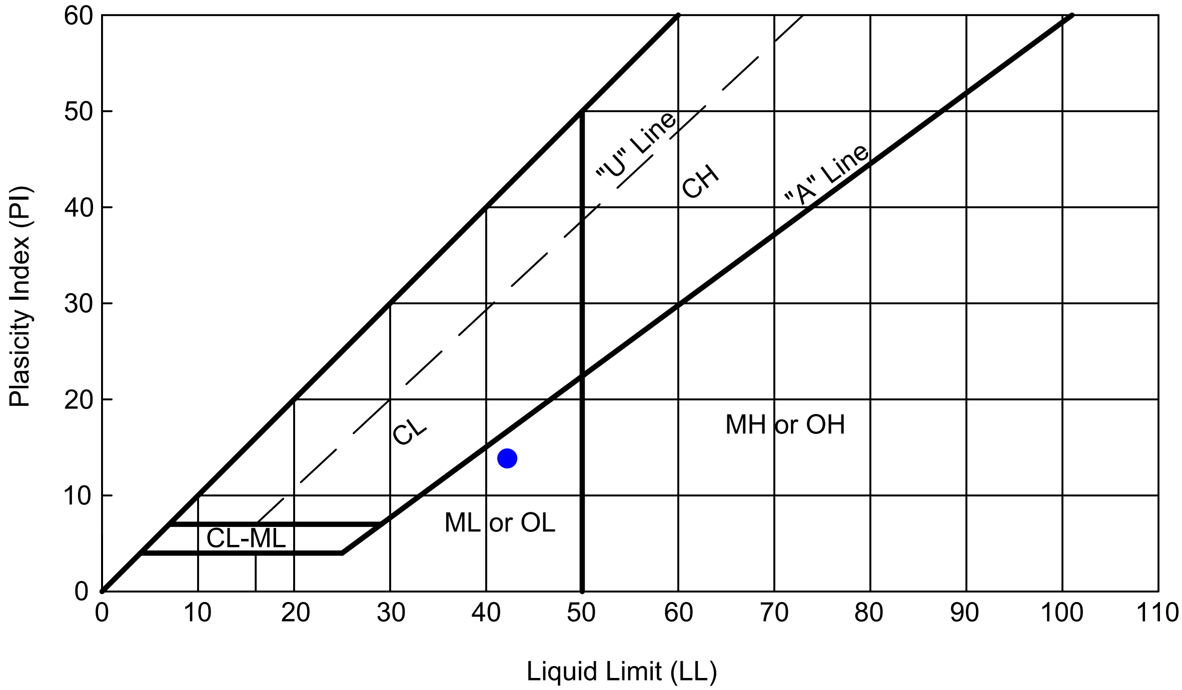
Atterberg Limits



Symbol	Sample	Plastic Limit	Liquid Limit	Plasticity Index	USC Symbol
●	B-1 @ 20.0-21.5ft	28.3	41.5	13.2	ML

Project Number 986517	Harbor Avenue SW Property Evaluation Atterberg Limits	Nelson Geotechnical Associates, Inc. Geotechnical Engineers & Geologists	No.	Date	Revision	By	CK
Figure 9			1	5/26/17	Original	DPN	LSB

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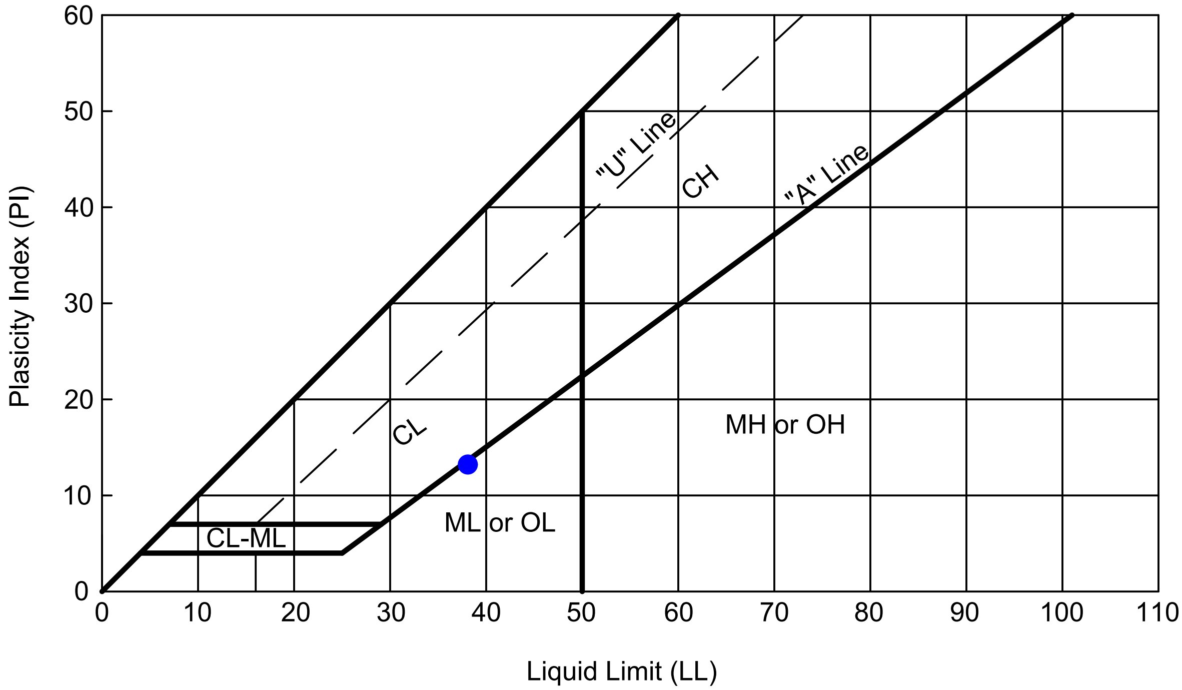
Atterberg Limits



Symbol	Sample	Plastic Limit	Liquid Limit	Plasticity Index	USC Symbol
●	B-1 @ 40.0-41.5ft	25.5	38.5	13.0	ML

Project Number 986517	Harbor Avenue SW Property Evaluation Atterberg Limits	Nelson Geotechnical Associates, Inc. Geotechnical Engineers & Geologists	No.	Date	Revision	By	CK
Figure 10		17311-135th Ave. NE, A-500 Woodinville, WA 98072 (425) 486-1669 / Fax 481-2510 — Snohomish County (425) 337-1669 Wenatchee/Chelan (509) 665-7696 www.nelsongeotech.com	1	5/26/17	Original	DPN	LSB

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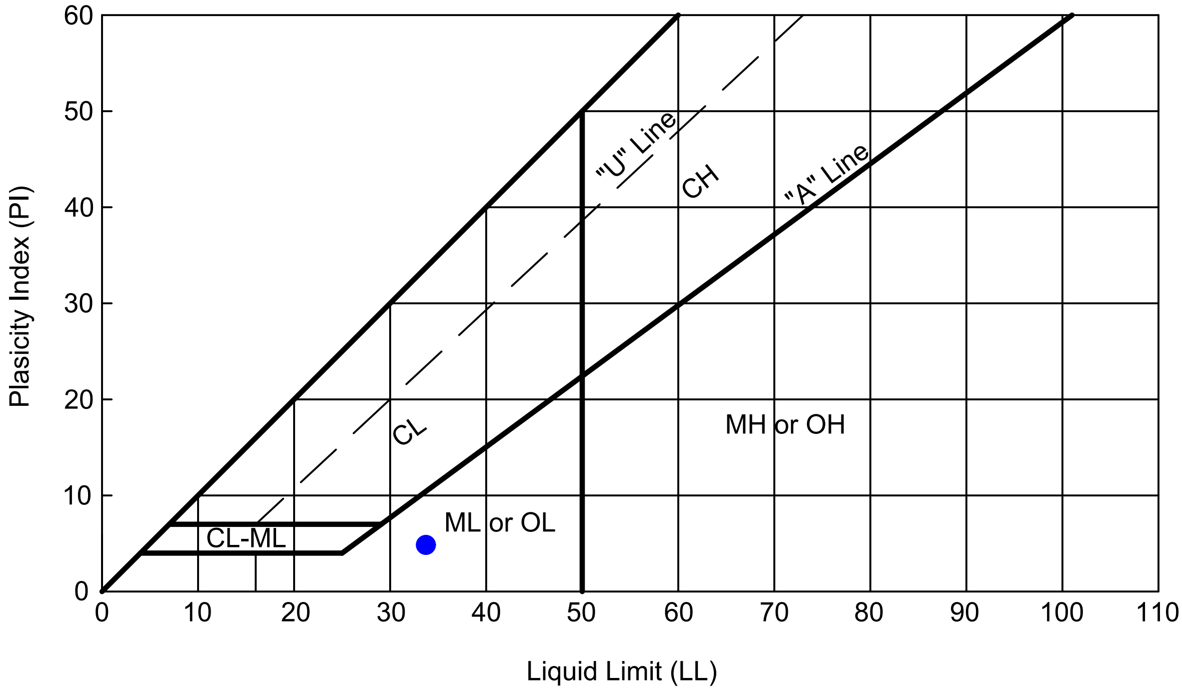
Atterberg Limits



Symbol	Sample	Plastic Limit	Liquid Limit	Plasticity Index	USC Symbol
●	B-2 @ 25.0-26.5ft	27.6	32.7	5.1	ML

Project Number 986517	Harbor Avenue SW Property Evaluation Atterberg Limits	NELSON GEOTECHNICAL ASSOCIATES, INC. NGA GEOTECHNICAL ENGINEERS & GEOLOGISTS	No.	Date	Revision	By	CK
Figure 11		17311-135th Ave. NE, A-500 Woodinville, WA 98072 (425) 486-1669 / Fax 481-2510 Snohomish County (425) 337-1669 Wenatchee/Chelan (509) 665-7696 www.nelsongeotech.com	1	5/26/17	Original	DPN	LSB

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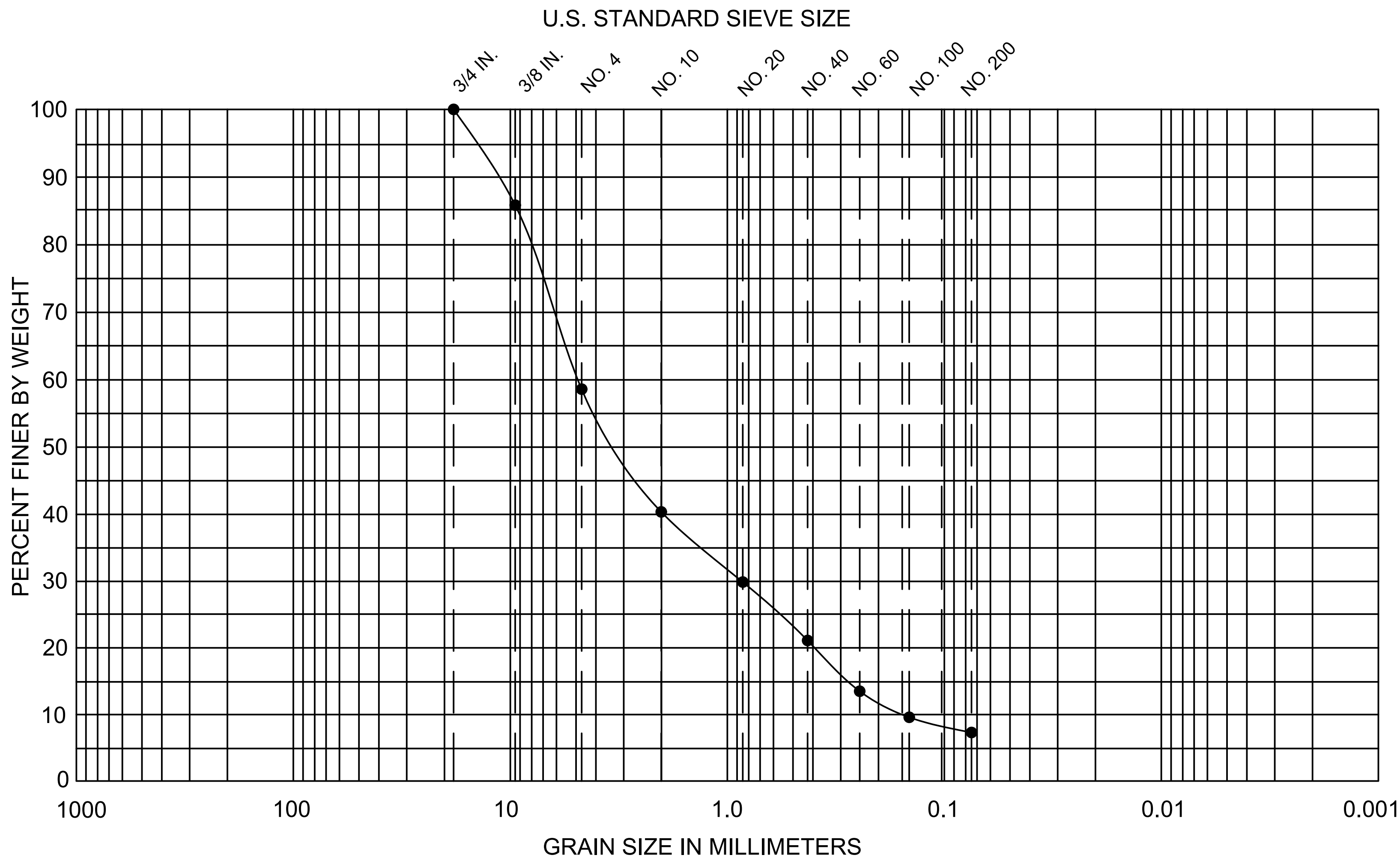

COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

U.S.C. SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION	SOIL DISTRIBUTION
●SP-SM	B-2	35.0 - 36.5 feet	Gray-brown, gravelly fine to coarse sand with silt	Gravel = 41% Sand = 52% Silt/Clay = 7%

Project Number
986517

Figure 12

Harbor Avenue SW
Property Evaluation
Sieve Analysis

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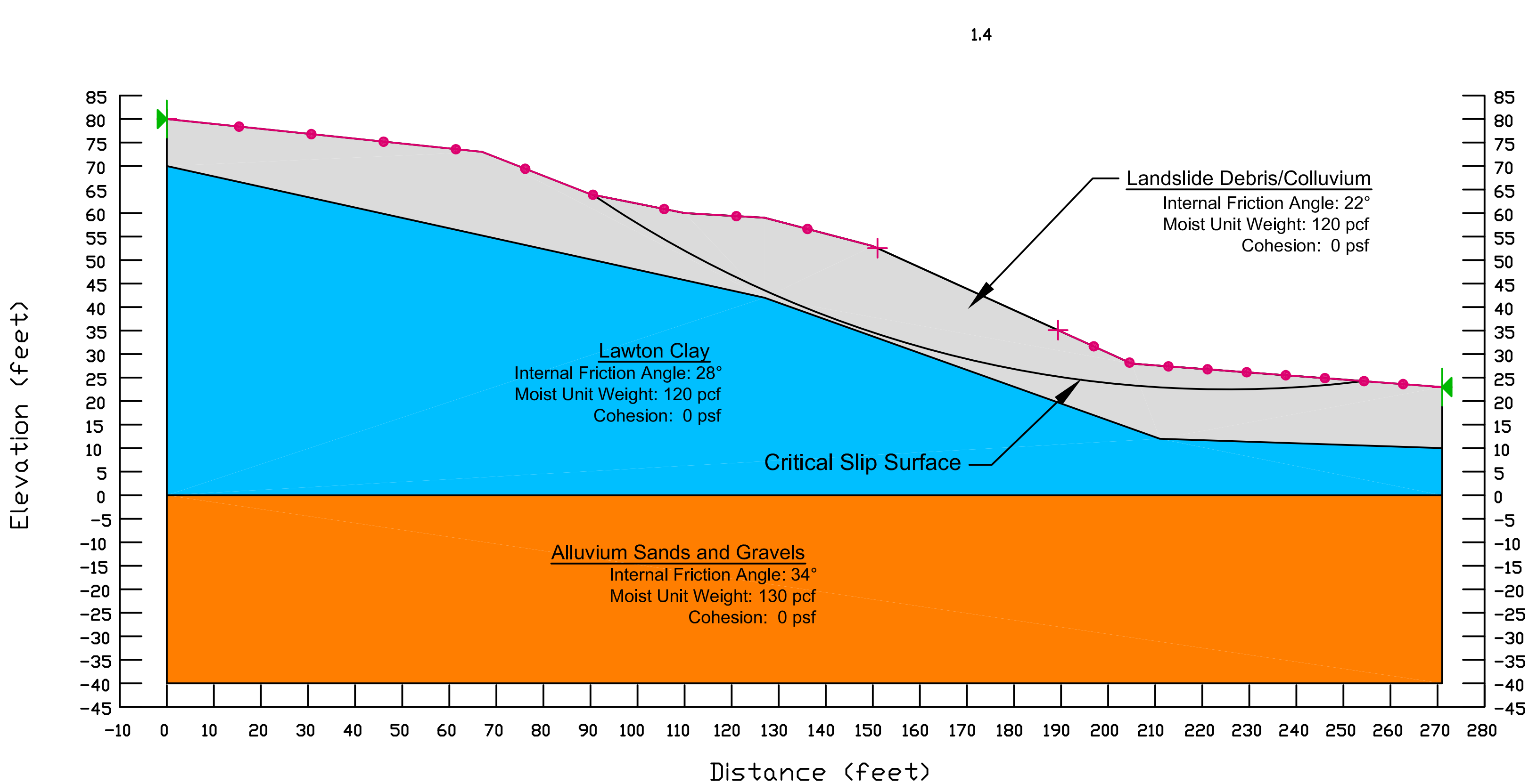
Slope Stability Analysis- Cross Section A-A' Existing Static Conditions

Morgenstern-Price most critical surface with minimum FOS = 1.4



NOTES: Not to scale
Elevations are Approximate

Project Number	Harbor Avenue SW Property Evaluation Cross Section A-A' Static Slope Stability Analysis	Nelson Geotechnical Associates, Inc.	No.	Date	Revision	By	CK
986517		Geotechnical Engineers & Geologists	1	6/22/17	Original	LSB	KMS
Figure 13		17311-135th Ave. NE, A-500 Woodinville, WA 98072 (425) 486-1669 / Fax 481-2510; Snohomish County (425) 337-1669 Wenatchee/Chelan (509) 784-2756 www.nelsongeotech.com					

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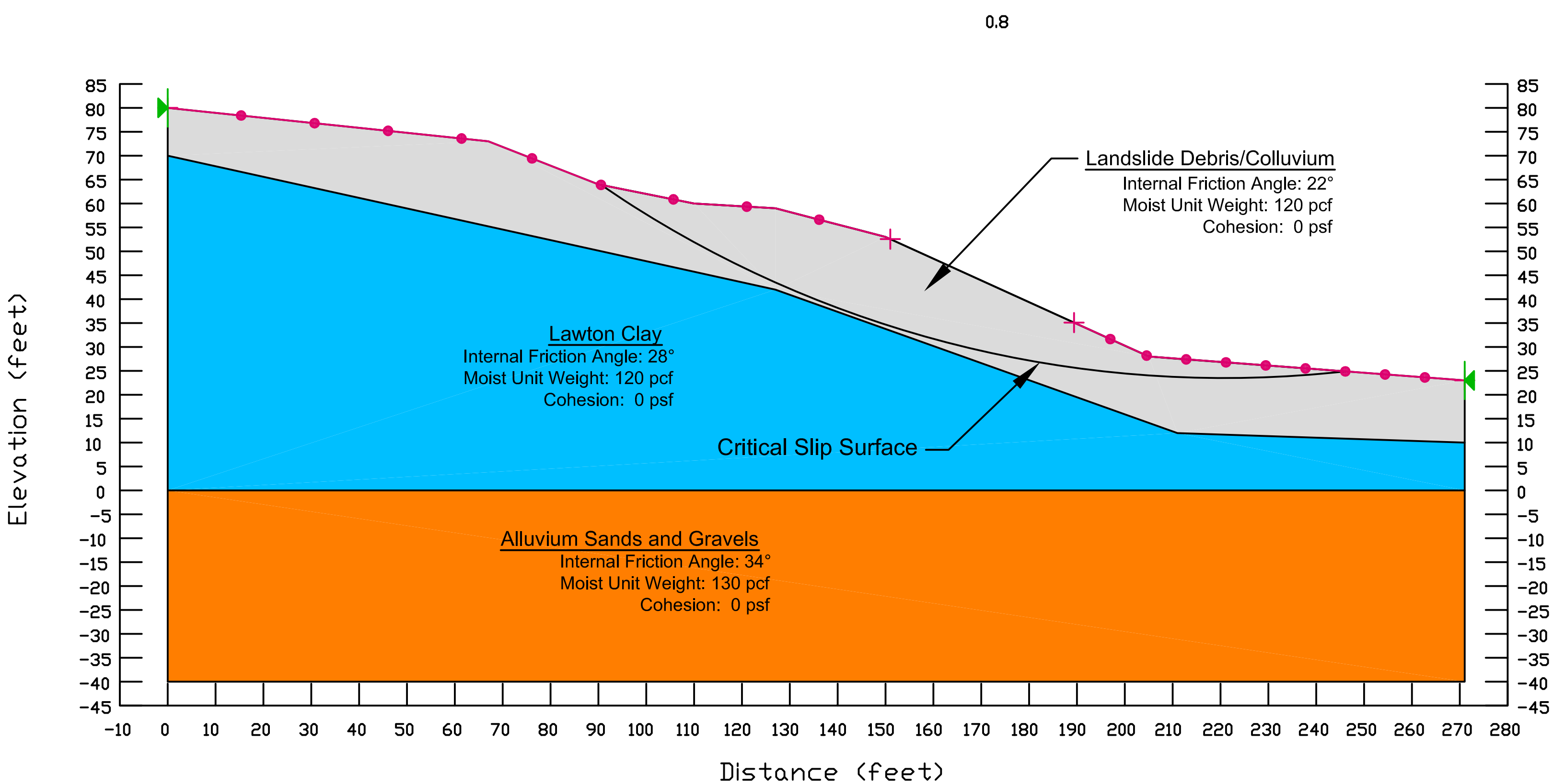
Slope Stability Analysis- Cross Section A-A' Exisitng Seismic Conditions

Morgenstern-Price most critical surface with minimum FOS = 0.8
with a seismic coefficient of ground acceleration = 0.20g



NOTES: Not to scale
Elevations are Approximate

Project Number	Harbor Avenue SW Property Evaluation	NELSON GEOTECHNICAL ASSOCIATES, INC.	No.	Date	Revision	By	CK
986517	Cross Section A-A' Existing Seismic Slope Stability Analysis	GEOTECHNICAL ENGINEERS & GEOLOGISTS	1	6/22/17	Original	LSB	KMS
Figure 14		17311-135th Ave. NE, A-500 Woodinville, WA 98072 (425) 486-1669 / Fax 481-2510 — Snohomish County (425) 337-1669 Wenatchee/Chelan (509) 784-2756 www.nelsongeotech.com					

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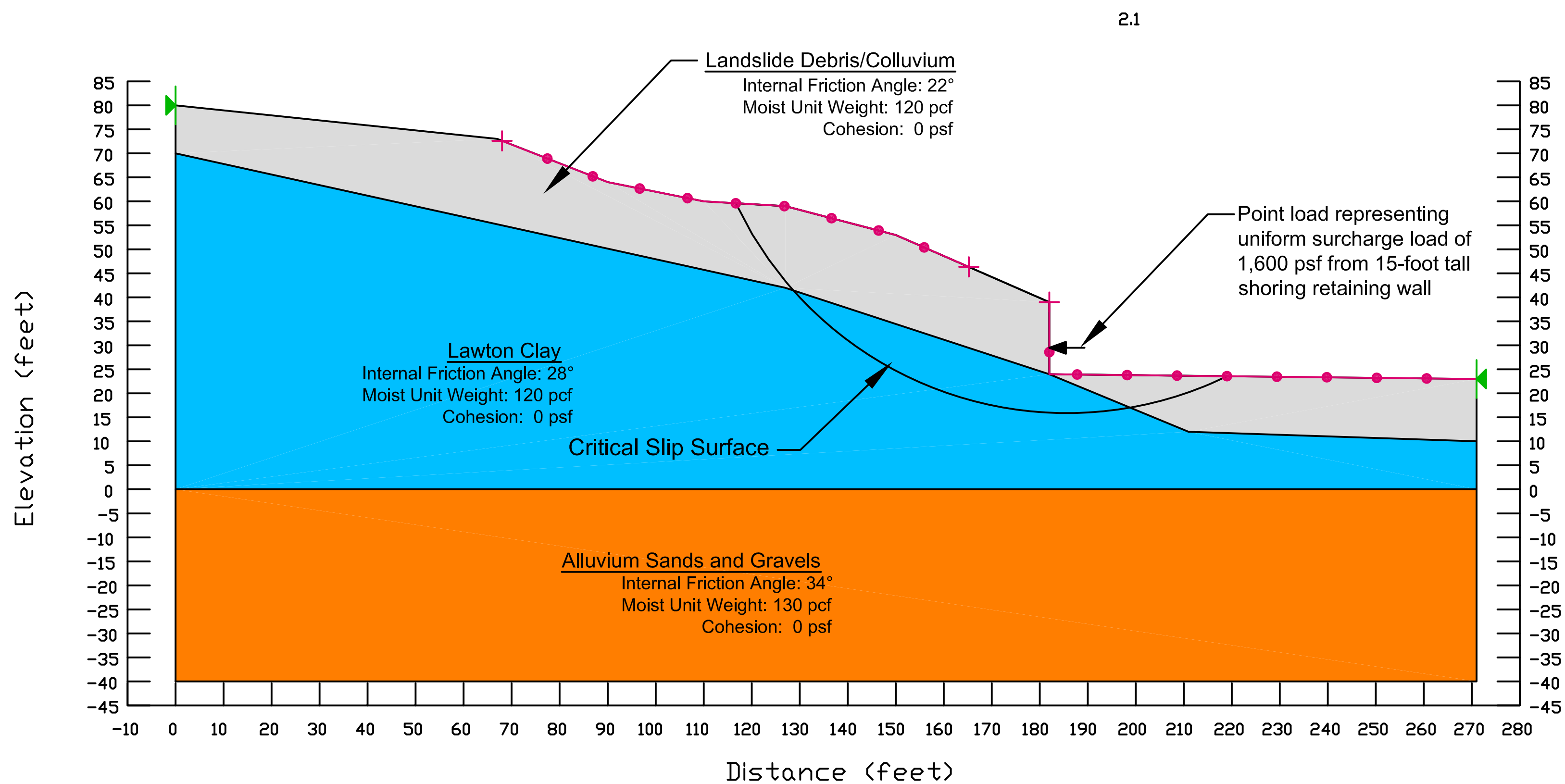
Slope Stability Analysis- Cross Section A-A' Proposed Static Conditions

Morgenstern-Price most critical surface with minimum FOS =2.1



NOTES: Not to scale
Elevations are Approximate

Project Number	Harbor Avenue SW Property Evaluation Cross Section A-A' Proposed Static Slope Stability Analysis	Nelson Geotechnical Associates, Inc.	No.	Date	Revision	By	CK
986517		Geotechnical Engineers & Geologists	1	6/22/17	Original	LSB	KMS
Figure 15							

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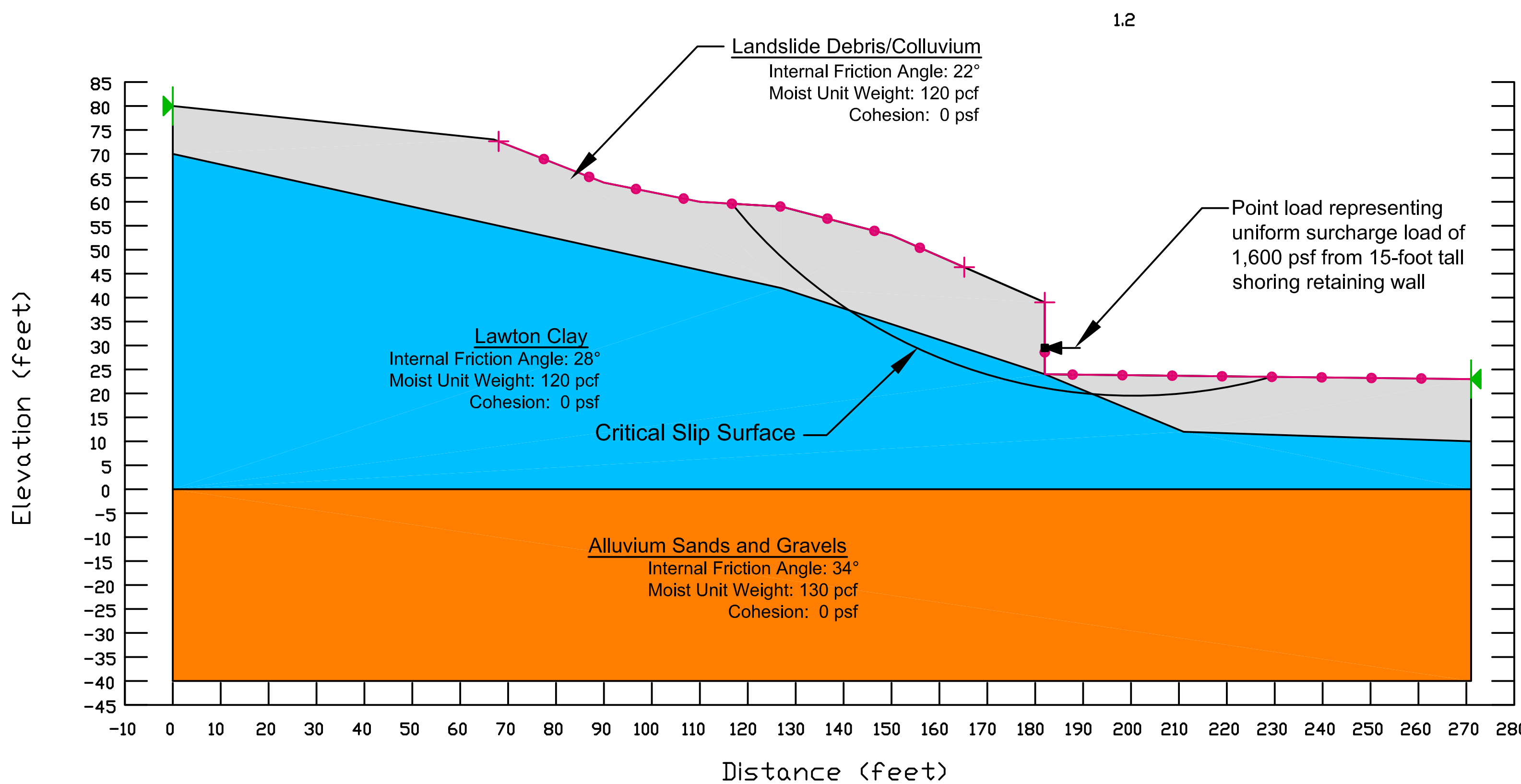
Slope Stability Analysis- Cross Section A-A' Proposed Seismic Conditions

Morgenstern-Price most critical surface with minimum FOS =1.2 with a seismic coefficient of ground acceleration = 0.20g



NOTES: Not to scale
Elevations are Approximate

Project Number	Harbor Avenue SW Property Evaluation	NELSON GEOTECHNICAL ASSOCIATES, INC.	No.	Date	Revision	By	CK
986517	Cross Section A-A' Proposed Seismic Slope Stability Analysis	GEOTECHNICAL ENGINEERS & GEOLOGISTS	1	6/22/17	Original	LSB	KMS
Figure 16							

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