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June 20, 2018

Exhibit 8

Ms. Julie Nealey
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Edmonds, WA 98020
stellanfloyds@gmail.com

Geotechnical Engineering Evaluation
Stella and Floyds Commercial Development
13209 Bothell-Everett Highway
Mill Creek, Washington
NGA Job No. 10362B18

Dear Ms. Nealey:

We are pleased to submit the attached report titled "Geotechnical Engineering Evaluation – Stella and Floyds Commercial Development – 13209 Bothell-Everett Highway – Bothell, Washington." This report summarizes our observations of the existing surface and subsurface conditions within the site, and provides general recommendations for the proposed site development. Our services were completed in general accordance with the proposals signed by you on April 13, 2018 and May 31, 2018.

The property is currently undeveloped and heavily vegetated with underbrush and a dense canopy of young to mature trees. The ground surface is generally level to gently sloping. A large wetlands area occupies the majority of the site within the central, eastern, and southeastern portions of the property. Specific grading plans were not available at the time this report was prepared, however, we understand that the proposed development plan will likely include the construction of an office building, five dog house structures, and a parking area, along with associated access roadways and underground utilities.

We monitored the excavation of six test pit explorations throughout the property. Within one of our test pits we conducted a small-scale pilot infiltration test (PIT). Our explorations indicated that the site was underlain by surficial undocumented fill with competent, native glacial soils at depth.

It is our opinion that the proposed site development is feasible from a geotechnical engineering standpoint, provided that our recommendations for site development are incorporated into project plans. In general, the native soils underlying the site should adequately support the planned structures. Foundations should be advanced through any loose soils down to the competent glacial material interpreted to underlie the site, for bearing capacity and settlement considerations. These soils should generally be encountered approximately one to three feet below the existing ground surface, based on our explorations. If loose soils or undocumented fill are encountered in unexplored areas of the site, they should be removed and replaced with structural fill for foundation and pavement support. Final stormwater plans have also not been developed, but we understand that on-site infiltration is being considered for this site. Based on our onsite testing it our opinion that stormwater infiltration is marginally feasible within the site. The subsurface soils generally consisted of surficial undocumented fill soils underlain by dense silty fine to medium sand with varying amounts of gravel and iron-oxide weathering that we interpreted as native glacial soils at relatively shallow depths. We did not encounter groundwater within our explorations throughout the site. We recommend that any stormwater infiltration systems within the site be designed with an incorporated overflow system and maintain the minimum groundwater separation as specified in the 2014 Department of Ecology Stormwater Management Manual for Western Washington.

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. We should be retained to review and comment on final development plans and observe the earthwork phase of construction. We also recommend that NGA be retained to provide monitoring and consultation services during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during construction differ from those anticipated, and to evaluate whether or not earthwork and foundation installation activities comply with contract plans and specifications.

It has been a pleasure to provide service to you on this project. Please contact us if you have any questions regarding this report or require further information.

Sincerely,

NELSON GEOTECHNICAL ASSOCIATES, INC.

Khaled M. Shawish, PE
Principal Engineer

TABLE OF CONTENTS

LIST OF FIGURES

Geotechnical Engineering Evaluation
Stella and Floyds Commercial Development
13209 Bothell-Everett Highway
Mill Creek, Washington

INTRODUCTION

This report presents the results of our geotechnical engineering investigation and evaluation of the planned Stella and Floyds Commercial Development project in the Mill Creek area of Snohomish County, Washington. The project site is located at 13209 Bothell-Everett Highway, as shown on the Vicinity Map in Figure 1. The purpose of this study is to explore and characterize the site's surface and subsurface conditions and to provide geotechnical recommendations for the planned site development. For our use in preparing this report, we have been provided with a preliminary site plan showing the proposed development, titled "Stella and Floyds," dated May 1, 2017, prepared by Capitol Architects Group.

The property is currently undeveloped and heavily forested with dense underbrush and young to mature trees. A wetlands area occupies the majority of the central, eastern, and southeastern portions of the site. We understand the proposed developments will consist of constructing several dog houses, a parking lot, and office building along the western and northern portions of the site. Final development and grading plans have not been prepared at the time this report was issued. Final stormwater plans have also not been developed, however, we understand that stormwater may be directed to on-site infiltration systems, if feasible. The existing and proposed site layout is shown on the Site Plan in Figure 2.

SCOPE

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

1. Review available soil and geologic maps of the area.
2. Explore the subsurface soil and groundwater conditions within the site with trackhoe excavated test pits. Trackhoe to be provided/subcontracted by NGA.
3. Provide long-term design infiltration rates based on on-site Pilot Infiltration Testing (PIT) per the 2014 DOE SWMMWW.
4. Perform laboratory grain-size sieve analysis on soil samples, as necessary.
5. Provide recommendations for earthwork, foundation support, and slabs-on-grade.
6. Provide recommendations for temporary and permanent slopes.
7. Provide recommendations for pavement subgrade.
8. Provide recommendations for infiltration system installation.
9. Provide recommendations for site drainage and erosion control.
10. Document the results of our findings, conclusions, and recommendations in a written geotechnical report.

SITE CONDITIONS

Surface Conditions

The site consists of a roughly rectangular-shaped parcel covering approximately 2.68 acres. The site is undeveloped and heavily forested with young to mature trees and dense underbrush. A wetlands area and associated buffer encompass the central, eastern, and southeastern portions of the site. The ground surface within the site is relatively level to gently sloping. The site is bounded to the north by Bothell-Everett Highway, to the east by Lowe's, to the south by Lowe's detention pond, and to the west by Les Schwab Tire. We did not observe surface water throughout the site during our site visits on April 26 and June 6, 2018.

Subsurface Conditions

Geology: The site is mapped on the Geologic map of the Everett 7.5 minute quadrangle, Snohomish County, Washington, by James P. Minard (US Geological Survey, 1985). The site is mapped as glacial till (Qvt). Till is generally described as a nonsorted mixture or mud, sand, pebbles, cobbles, and diamicton boulders. Our explorations typically encountered undocumented fill underlain by compact silty fine to medium sand with gravel consistent with the description of native glacial till deposits at depth.

Explorations: The subsurface conditions within the site were explored on April 26 and June 6, 2018 by monitoring the excavation of six total track hoe excavated test pits that ranged in depth from 3.0 to 7.0 feet below the existing ground surface. The approximate locations of our explorations are shown on the Site Plan in Figure 2. A geologist from NGA was present during the explorations, examined the soils and geologic conditions encountered, obtained samples of the different soil types, and maintained logs of the test pits.

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

At the surface of each exploration we generally encountered 1.5 to 2.0 feet of dark brown to reddish brown, organic-rich silty sand with varying amounts of gravel, and roots, which we interpreted as topsoil and/or undocumented fill soils. Underlying the topsoil and undocumented fill we encountered medium dense or better orange-brown to gray, silty fine to medium sand with gravel, iron-oxide staining, and trace roots, which we interpreted as weathered and unweathered glacial till soils. Test Pit 1 through 5 and Infiltration Pit 1 terminated at respective depths of 7.0, 7.0, 4.5, 7.0, 3.0, and 4.5 feet below the existing ground surface, respectively.

Hydrogeologic Conditions

We did not encounter groundwater within our explorations throughout the site. If groundwater is encountered during construction we would interpret this as perched groundwater. 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 and undocumented fill. The low permeability soil consists of relatively silty native glacial deposits. Perched water does not represent a regional groundwater "table" within the upper soil horizons. Perched water tends to vary spatially and is dependent upon the amount of rainfall. We would expect the amount of perched groundwater to decrease during drier times of the year and increase during wetter periods.

SENSITIVE AREA EVALUATION

Seismic Hazard

We reviewed the 2018 International Building Code (IBC) for seismic site classification for this project. Since competent glacial till soils are inferred to underlie the site at depth, the site conditions best fit the IBC description for Site Class D.

Table 1 below provides seismic design parameters for the site that are in conformance with the 2018 IBC, which specifies a design earthquake having a 2% probability of occurrence in 50 years (return interval of 2,475 years), and the 2008 USGS seismic hazard maps.

Table 1 – 2018 IBC Seismic Design Parameters

Site Class	Spectral Acceleration at 0.2 sec. (g) S_s	Spectral Acceleration at 1.0 sec. (g) S_1	Site Coefficients		Design Spectral Response Parameters	
			F_a	F_v	S_{DS}	S_{D1}
D	1.36	0.531	1.000	1.500	0.907	0.531

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

The site is located within the South Whidbey Island Fault Zone (SWIFZ): an active, shallow region of seismicity within central Puget Sound stretching from the Strait of Juan de Fuca to North Bend. Information published in 2013 by the Washington State Department of Natural Resources suggests the SWIFZ last ruptured less than 2,700 years ago, and that the fault zone can produce a M7.5 earthquake. In our opinion, the possibility of faulting ground rupture caused by this fault zone is considered low.

Hazards associated with seismic activity include liquefaction potential and amplification of ground motion. Liquefaction is caused by a rise in pore pressures in a loose, fine sand deposit beneath the groundwater table. It is our opinion that the medium dense or better glacial deposits interpreted to underlie the site have a low potential for liquefaction or amplification of ground motion.

Erosion Hazard

The criteria used for determination of the erosion hazard for affected areas include soil type, slope gradient, vegetation cover, and groundwater conditions. The erosion sensitivity is related to vegetative cover and the specific surface soil types, which are related to the underlying geologic soil units. The Soil Survey of King County Area, Washington, by the Soil Conservation Service (SCS) was reviewed to determine the erosion hazard of the on-site soils. The surface soils for this site were mapped as Alderwood-Urban land complex, 2 to 8 percent slopes. The erosion hazard for this material is listed as slight. This site is relatively level to gently sloping and there are no steep slopes on the property. It is our opinion that the erosion hazard for site soils should be low in areas where the site is not disturbed.

CONCLUSIONS AND RECOMMENDATIONS

General

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

Based on the results of our infiltration testing and soil explorations throughout the site, it is our opinion that traditional stormwater infiltration systems within this site are not feasible, however low-impact design infiltration systems, such as pervious pavements, rain gardens, and bio-swales may be feasible. We recommend any low-impact systems within the site be designed with an incorporated overflow system directed towards an approved point of discharge. This is further discussed in the **Site Drainage** section of this report.

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

Erosion Control

The erosion hazard for the on-site soils is interpreted to slight for exposed soils, but actual erosion potential will be dependent on how the site is graded and how water is allowed to concentrate. Best Management Practices (BMPs) should be used to control erosion. Areas disturbed during construction should be protected from erosion. Erosion control measures may include diverting surface water away from the stripped or disturbed areas. Silt fences and/or straw bales should be erected to prevent muddy water from leaving the site. Disturbed areas should be planted as soon as practical and the vegetation should be maintained until it is established. The erosion potential of areas not stripped of vegetation should be low.

Site Preparation and Grading

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

After site stripping, if the exposed subgrade is deemed loose, it should be compacted to a non-yielding condition and then proof-rolled with a heavy rubber-tired piece of equipment. Areas observed to pump or weave during the proof-roll test should be reworked to structural fill specifications or over-excavated and replaced with properly compacted structural fill or rock spalls. If loose soils are encountered in the pavement areas, the loose soils should be removed and replaced with rock spalls or granular structural fill. If significant surface water flow is encountered during construction, this flow should be diverted around areas to be developed, and the exposed subgrades should be maintained in a semi-dry condition.

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

The site soils are considered to be moisture-sensitive and will disturb easily when wet. We recommend that construction take place during the drier summer months if possible. However, if construction takes place during the wet season, additional expenses and delays should be expected due to the wet conditions. Additional expenses could include the need for placing a blanket of rock spalls on exposed subgrades, construction traffic areas, and paved areas prior to placing structural fill. Wet weather grading will also require additional erosion control and site drainage measures. Some of the on-site soils may be suitable for use as structural fill, depending on the moisture content of the soil at the time of construction. NGA should be retained to evaluate the suitability of all on-site and imported structural fill material during construction.

Temporary and Permanent Slopes

Temporary cut slope stability is a function of many factors, including the type and consistency of soils, depth of the cut, surcharge loads adjacent to the excavation, length of time a cut remains open, and the presence of surface or groundwater. It is exceedingly difficult under these variable conditions to estimate a stable, temporary, cut slope angle. Therefore, it should be the responsibility of the contractor to maintain safe slope configurations at all times as indicated in OSHA guidelines for cut slopes.

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

For planning purposes, we recommend that temporary cuts in the upper undocumented fill soils be no steeper than 2 Horizontal to 1 Vertical (2H:1V). Temporary cuts in the competent native glacial soils at depth should be no steeper than 1.5H:1V. If significant groundwater seepage or surface water flow were encountered, we would expect that flatter inclinations would be necessary. We recommend that cut slopes be protected from erosion. The slope protection measures may include covering cut slopes with plastic sheeting and diverting surface runoff away from the top of cut slopes. We do not recommend vertical slopes for cuts deeper than four feet, if worker access is necessary. We recommend that cut slope heights and inclinations conform to appropriate OSHA/WISHA regulations.

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

Foundations

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

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

For foundations constructed as outlined above, we recommend an allowable design bearing pressure of not more than 2,500 pounds per square foot (psf) be used for the design of footings founded on the medium dense or better native soils or structural fill extending to the competent native material. The foundation bearing soil should be evaluated by a representative of NGA. We should be consulted if higher bearing pressures are needed. Current IBC guidelines should be used when considering increased allowable bearing pressure for short-term transitory wind or seismic loads. Potential foundation settlement using the recommended allowable bearing pressure is estimated to be less than 1-inch total and ½-inch differential between adjacent footings or across a distance of about 20 feet, based on our experience with similar projects.

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

Retaining Walls

Specific grading plans for this project were not available at the time this report was prepared, but retaining walls may be incorporated into project plans. In general, the lateral pressure acting on 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 40 pcf for yielding (active condition) walls, and 60 pcf for non-yielding (at-rest condition) walls. A seismic design loading of 8H should also be included in the wall design. It represents the total height of the wall.

These recommended lateral earth pressures are for a drained granular backfill and are based on the assumption of a horizontal ground surface behind the wall for a distance of at least the 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 the structural engineer regarding additional loads on retaining walls during final design, if needed.

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

All wall backfill should be well compacted as outlined in the **Structural Fill** subsection of this report. Care should be taken to prevent the buildup of excess lateral soil pressures due to over-compaction of the wall backfill. This can be accomplished by placing wall backfill in 8-inch loose lifts and compacting the backfill with small, hand-operated compactors within a distance behind the wall equal to at least one-half the height of the wall. The thickness of the loose lifts should be reduced to accommodate the lower compactive energy of the hand-operated equipment. The recommended level of compaction should still be maintained.

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

Structural Fill

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

Materials: Structural fill should consist of a good quality, granular soil, free of organics and other deleterious material, and be well graded to a maximum size of about three inches. All-weather fill should contain no more than five-percent fines (soil finer than U.S. No. 200 sieve, based on that fraction passing the U.S. 3/4-inch sieve). Some of the more granular on-site soils may be suitable for use as structural fill, but this will be highly dependent on the moisture content of these soils at the time of construction. We should be retained to evaluate all proposed structural fill material prior to placement.

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

Slab-on-Grade

Slabs-on-grade should be supported on subgrade soils prepared as described in the **Site Preparation and Grading** subsection of this report. We recommend that all floor slabs be underlain by at least six inches of free-draining gravel with less than three percent by weight of the material passing Sieve #200 for use as a capillary break. 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.

Pavements

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

Utilities

We recommend that underground utilities be bedded with a minimum six inches of pea gravel prior to backfilling the trench with on-site or imported material. Trenches within settlement sensitive areas should be compacted to 95% of the modified proctor as described in the **Structural Fill** subsection of this report. Trenches located in non-structural areas should be compacted to a minimum 90% of the maximum dry density.

Site Drainage

Infiltration: We conducted a Small PIT within Infiltration Pit 1, located as shown on the attached Schematic Site Plan in Figure 2. The test was conducted within a pit that measured 4.5-feet long by 3.0-feet wide by 4.5-feet deep. The pit was filled with 12-inches of water at the beginning of the day and we began the soaking period of the PIT for approximately 6 hours. At this time, the water flow rate into the hole was monitored with a Great Plains Industries (GPI) TM 075 water flow meter for the pre-soak period.

After the 6-hour soaking period was completed, the water level was maintained at approximately 12-inches for one hour for the steady-state period. The flow rate for Infiltration Pit 1 stabilized at 0.0235 gallons per minute (1.41 gallons per hour). This equated to an approximate infiltration rate of 0.168 inches per hour. The water was shut off after the steady-state period and monitored at least every 15 minutes for one hour. After 60 minutes, the water level within the pit dropped approximately 0.125 inches, resulting in a measured infiltration rate of 0.125 inch per hour.

In accordance with the Table 3.5 of the Department of Ecology 2014 SWMMWW, correction factors of 1.0, 0.5, and 0.9 for CFv, CFt, CFm, respectively were applied to the field measured infiltration rate of 0.125 inches per hour, obtained from the falling-head portion of the testing in Infiltration Pit 1. A total correction factor of 0.45 was applied to the measured field infiltration rate obtained from the falling head portion of the test to determine the long-term design infiltration rate.

Using the above correction factor, we calculated a long-term design infiltration rate of approximately 0.056 inches per hour. In our opinion, a long-term design infiltration rate of 0.056 inches per hour could be utilized to design the on-site low-impact infiltration systems within the native, silty fine to medium sand with gravel found on this site at depth.

It is our opinion that the subsurface soils within the site are not suitable for traditional stormwater infiltration systems, however low-impact design systems may be feasible within the site. The subsurface soils generally consisted of surficial undocumented fill soils underlain by silty fine to medium sand with gravel that we interpreted as native glacial till deposits. We did not encounter groundwater within our explorations to a maximum depth of 7.0 feet below the ground surface. We recommend that low-impact infiltration facilities, such as permeable pavements have an incorporated overflow component directed towards an approved point of discharge. We recommend these systems be sized and designed in accordance with the 2014 Department of Ecology Stormwater Management Manual for Western Washington in conjunction with the provided long-term design infiltration rate of 0.056 inches per hour.

We recommend that any proposed infiltration systems be placed as to not negatively impact any proposed or existing nearby structures and also meet all required setbacks from existing property lines, structures, and sensitive areas as discussed in the drainage manual. In general, infiltration systems should not be located within proposed fill areas within the site associated with site grading or retaining wall backfill as such condition could lead to failures of the placed fills and/or retaining structures. We should be retained to evaluate the infiltration system design and installation during construction.

Surface Drainage: The finished ground surface should be graded such that stormwater is directed to an appropriate stormwater collection system. Water should not be allowed to stand in any areas where footings, slabs, or pavements are to be constructed. Final site grades should allow for drainage away from the residences. 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 residences. Surface water should be collected by permanent catch basins and drain lines, and be discharged into an appropriate discharge system.

Subsurface Drainage: If groundwater is encountered during construction, we recommend that the contractor slope the bottom of the excavation and collect the water into ditches and small sump pits where the water can be pumped out and routed into a permanent storm drain.

We recommend the use of footing drains around the structures. Footing drains should be installed at least one foot below planned finished floor elevation. The drains should consist of a minimum 4-inch-diameter, rigid, slotted or perforated, PVC pipe surrounded by free-draining material wrapped in a filter fabric. We recommend that the free-draining material consist of an 18-inch-wide zone of clean (less than three-percent fines), granular material placed along the back of walls. Pea gravel is an acceptable drain material. The free-draining material should extend up the wall to one foot below the finished surface. The top foot of backfill should consist of impermeable soil placed over plastic sheeting or building paper to minimize surface water or fines migration into the footing drain. Footing drains should discharge into tightlines leading to an 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 should be retained to provide construction monitoring services during the earthwork phase of the project to evaluate subgrade conditions, temporary cut conditions, fill compaction, and drainage system installation.

USE OF THIS REPORT

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

We recommend that NGA be retained to provide monitoring and consultation services during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed 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.

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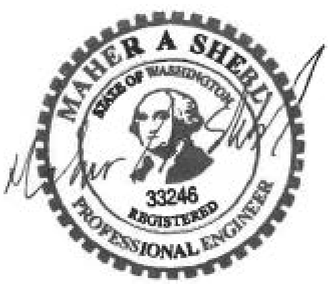
It has been a pleasure to provide service to you on this project. If you have any questions or require further information, please call.

Sincerely,

NELSON GEOTECHNICAL ASSOCIATES, INC.

Alex B. Rinaldi, GIT
Staff Geologist II



Maher A. Shebl, PE
Senior Engineer

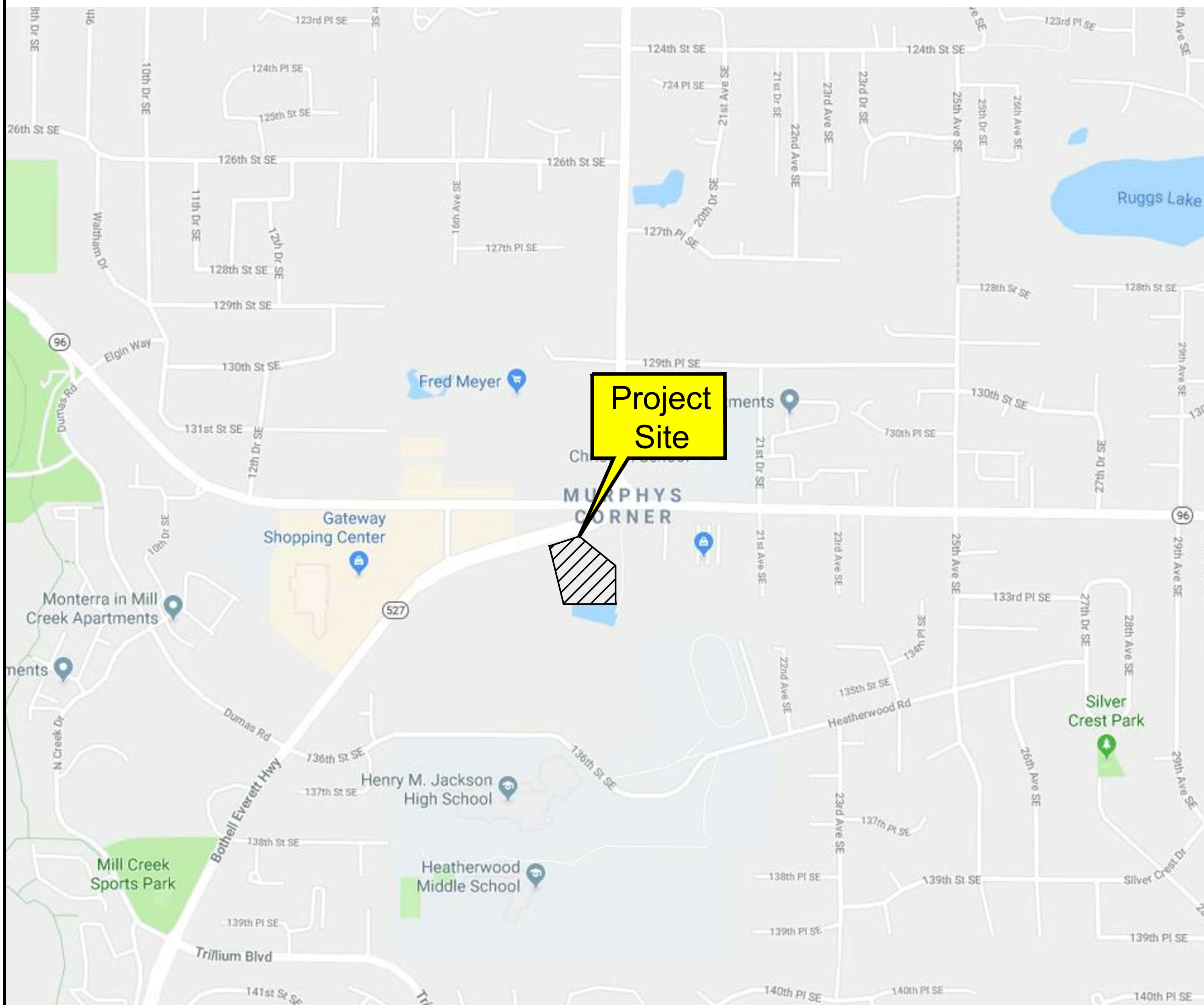
ABR:MAS:dy

Five Figures Attached

VICINITY MAP

Not to Scale

N



Mill Creek, WA

Project Number	Stella and Floyds Supplemental Infiltration Vicinity Map	NELSON GEOTECHNICAL ASSOCIATES, INC.	No.	Date	Revision	By	CK
10362B18		GEOTECHNICAL ENGINEERS & GEOLOGISTS	1	5/4/18	Original	DPN	ABR
Figure 1		Woodinville Office 17311-135th Ave. NE, A-500 Woodinville, WA 98072 (425) 486-1669 / Fax: 481-2510 www.nelsongeotech.com East Wenatchee Office 5526 Industry Lane, #2 East Wenatchee, WA 98802 (509) 665-7696 / Fax: 665-7692	2	6/12/18	Revision	DPN	ABR

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LEGEND

Property line

INF-1

Number and approximate location of infiltration test pit

TP-1

Number and approximate location of test pit

0 60 120

Scale: 1 inch = 60 feet

Reference: Site Plan based on a plan dated May 1, 2017 titled "Stella & Floyds," prepared by Capital Group.

Project Number	Stella and Floyds Supplemental Infiltration Site Plan
10362B18	
Figure 2	

NELSON GEOTECHNICAL ASSOCIATES, INC.

GEOTECHNICAL ENGINEERS & GEOLOGISTS

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UNIFIED SOIL CLASSIFICATION SYSTEM

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

NOTES:

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

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

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

SOIL MOISTURE MODIFIERS:

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

Moist - Damp, but no visible water.

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

Project Number	Stella and Floyds Supplemental Infiltration Soil Classification Chart
10362B18	
Figure 3	

NGA
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LOG OF EXPLORATION

DEPTH (FEET)	USC	SOIL DESCRIPTION
TEST PIT ONE		
0.0 – 1.5		DARK BROWN, ORGANIC-RICH SILTY FINE TO MEDIUM SAND WITH ROOTS (LOOSE TO MEDIUM DENSE, MOIST) (**TOPSOIL**)
1.5 – 3.6	SM	ORANGE-BROWN, SILTY FINE TO MEDIUM SAND WITH GRAVEL, IRON-OXIDE STAINING, AND TRACE ROOTS (MEDIUM DENSE TO DENSE, MOIST)
3.6 – 7.0	SM	GRAY, SILTY FINE TO MEDIUM SAND WITH GRAVEL AND TRACE IRON-OXIDE STAINING (MEDIUM DENSE TO DENSE, MOIST)
		SAMPLES WERE COLLECTED AT 2.3 AND 4.0 FEET GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 7.0 FEET ON 4/26/2018
TEST PIT TWO		
0.0 – 2.0		DARK BROWN, ORGANIC-RICH SILTY FINE TO MEDIUM SAND WITH ROOTS AND TRACE GARBAGE (LOOSE TO MEDIUM DENSE, MOIST) (**UNDOCUMENTED FILL**)
2.0 – 3.5	SM	ORANGE-BROWN, SILTY FINE TO MEDIUM SAND WITH GRAVEL, IRON-OXIDE STAINING, AND TRACE ROOTS (MEDIUM DENSE, MOIST)
3.5 – 7.0	SM	GRAY, SILTY FINE TO MEDIUM SAND WITH GRAVEL AND IRON-OXIDE STAINING (MEDIUM DENSE TO DENSE, MOIST)
		SAMPLES WERE COLLECTED AT 3.0 AND 7.0 FEET GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 7.0 FEET ON 4/26/2018
TEST PIT THREE		
0.0 – 2.0		DARK BROWN, ORGANIC-RICH SILTY FINE TO MEDIUM SAND WITH ROOTS AND TRACE GARBAGE (LOOSE TO MEDIUM DENSE, MOIST) (**UNDOCUMENTED FILL**)
2.0 – 3.3	SM	ORANGE-BROWN, SILTY FINE TO MEDIUM SAND WITH GRAVEL, IRON-OXIDE STAINING, AND SCATTERED ROOTS (MEDIUM DENSE, MOIST)
3.3 – 4.5	SM	GRAY, SILTY FINE TO MEDIUM SAND WITH GRAVEL AND IRON-OXIDE STAINING (MEDIUM DENSE TO DENSE, MOIST)
		SAMPLE WAS COLLECTED AT 4.0 FEET GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 4.5 FEET ON 4/26/2018
TEST PIT FOUR		
0.0 – 2.0		DARK BROWN, ORGANIC-RICH SILTY FINE TO MEDIUM SAND WITH ROOTS GARBAGE (LOOSE TO MEDIUM DENSE, MOIST) (**TOPSOIL**)
2.0 – 4.0	SM	ORANGE-BROWN, SILTY FINE TO MEDIUM SAND WITH GRAVEL, IRON-OXIDE STAINING, AND TRACE ROOTS (MEDIUM DENSE, MOIST)
4.0 – 7.0	SM	GRAY, SILTY FINE TO MEDIUM SAND WITH GRAVEL (MEDIUM DENSE TO DENSE, MOIST)
		SAMPLE WAS COLLECTED AT 7.0 FEET GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 7.0 FEET ON 4/26/2018

LOG OF EXPLORATION

DEPTH (FEET)	USC	SOIL DESCRIPTION
TEST PIT FIVE		
0.0 – 1.5		DARK BROWN TO REDDISH, ORGANIC-RICH SILTY FINE TO MEDIUM SAND WITH ROOTS AND WOOD DEBRIS (LOOSE TO MEDIUM DENSE, MOIST) (**TOPSOIL**)
1.5 – 2.5	SM	ORANGE-BROWN, SILTY FINE TO MEDIUM SAND WITH GRAVEL, IRON-OXIDE STAINING, AND TRACE ROOTS (MEDIUM DENSE, MOIST)
2.5 – 3.0	SM	GRAY, SILTY FINE TO MEDIUM SAND WITH GRAVEL (MEDIUM DENSE TO DENSE, MOIST)
		SAMPLE WAS NOT COLLECTED GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 3.0 FEET ON 4/26/2018
INFILTRATION PIT ONE		
0.0 – 2.8		UNDERBRUSH UNDERLAIN BY BROWN, SILTY FINE TO MEDIUM SAND WITH GRAVEL, ROOTS, ORGANICS, AND WOOD DEBRIS (LOOSE TO MEDIUM DENSE, MOIST) (**TOPSOIL/FILL**)
2.8 – 4.5	SM	GRAY, WELL-CEMENTED SILTY FINE TO MEDIUM SAND WITH GRAVEL AND IRON-OXIDE STAINING (MEDIUM DENSE TO DENSE, MOIST)
		SAMPLE WAS NOT COLLECTED GROUNDWATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 4.5 FEET ON 4/26/2018