GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED MULTI-STORY BUILDING
4301 ALDERWOOD MALL BOULEVARD
LYNNWOOD, WASHINGTON 98005

Project No. 092-21009
NOVEMBER 1, 2021

Prepared for:

KOZ DEVELOPMENT
ATTN: JOSHUA SCOTT
1830 BICKFORD AVENUE, #201
SNOHOMISH, WA 98290

Prepared by:

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November 1, 2021 | Krazan File Number 092-21009

koz Development LLC
1830 Bickford Avenue, #201
Snohomish, WA 98290

Attn: **Joshua Scott** Email: Josh@kozdevelopment.com
VP, Design and Construction Phone: 206-755-1290

Reference: **Geotechnical Engineering Investigation**
Proposed Multi-Use Building
4301 Alderwood Mall Boulevard
Lynnwood, WA

Dear Mr. Scott,

In accordance with your request, we have completed a Geotechnical Engineering Investigation for the referenced site. The results of our investigation are presented in the attached report.

If you have any questions, or if we can be of further assistance, please do not hesitate to contact our office.

Respectfully submitted,
KRAZAN & ASSOCIATES, INC.

Michael D. Rundquist

Michael D. Rundquist, P.E.
Senior Project Manager

MDR

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INTRODUCTION

This report presents the results of our Geotechnical Engineering Investigation for the proposed building project located at 4301 Alderwood Mall Boulevard in Lynnwood, Washington as shown on the Vicinity Map in Figure 1. Discussions regarding site conditions are presented in this report, together with conclusions and recommendations pertaining to site preparation, excavations, foundations, structural fill, utility trench backfill, pavement design, drainage, and erosion control.

We have been provided with undated scaled conceptual drawings by the client for the project. We were also provided with the "ALTA/NSPS Land Title Survey" for the site, prepared by Terrane, dated July 29, 2021.

The approximate locations of the exploratory soil borings are presented in the Site Plan in Figure 2 following the text of this report. A description of the field investigation as well as the exploratory soil boring logs and laboratory test results are presented in Appendix A. Appendix B contains a guide to aid in the development of earthwork specifications for the project. Pavement design guidelines are presented in Appendix C. The recommendations in the main text of the report have precedence over the more general specifications in the appendices.

PURPOSE AND SCOPE

This geotechnical investigation was conducted to evaluate the subsurface soil and groundwater conditions at the site, to develop geotechnical engineering recommendations for use in project design and to provide criteria for site preparation and earthwork construction.

Our geotechnical engineering services were performed in general accordance with our proposal for this project, dated August 5, 2021 (Krazan proposal number G21654WAL). The geotechnical services performed for this project generally include the following:

- Exploration of the subsurface soil and groundwater conditions by conducting five (5) geotechnical borings using a subcontracted drill rig;

- Provide a site plan showing the geotechnical boring locations;
- Provide comprehensive boring logs including soil stratification and classification, and groundwater levels where applicable;
- Provide laboratory test results;
- Provide foundation recommendations for the proposed structures including foundation type, allowable bearing pressure, anticipated settlements (both total and differential), coefficient of horizontal friction, and frost penetration depth;
- Provide recommendations for seismic design considerations including site coefficient and ground acceleration based on the 2018 IBC;
- Provide recommendations for temporary excavations;
- Provide recommendations for construction and excavation considerations, topsoil/unsuitable soil stripping depth, identification of potentially problematic soils or groundwater conditions, and depth of over-excavation if required;
- Provide recommendations for structural fill materials, placement, and compaction;
- Provide recommendations regarding the suitability of on-site soils as structural fill;
- Provide recommendations for site drainage and erosion control;
- Provide our opinion regarding the feasibility/infeasibility of stormwater infiltration at this site;
- Provide recommendations for retaining wall design;
- Provide recommendations for temporary shoring design;
- Provide recommendations for pavement design;

Environmental services, such as chemical analysis of soil and groundwater for possible environmental contaminants, are not included in our scope of services for this project. In-situ infiltration testing was also not included in our scope of services for this project.

PROPOSED DEVELOPMENT

The conceptual plans indicate that the proposed development will include design and construction of a five-story wood frame structure over a ground level concrete podium building, with a portion of the site also being developed as a single-story below grade parking structure. The site development will also include design and construction of pavement, utilities, and landscape areas.

SITE CONDITIONS

The site consists of a single assessor parcel (00372600701905) and covers an area of about 1.1 acres. The site has previously been developed as a commercial property and is situated in a neighborhood of other commercial developments. The concrete foundations and slab of a demolished commercial building are located on the southern portion of the property, and the remainder of the site consists of paved parking and landscaped areas. The property is bordered by Alderwood Mall Boulevard to the south, and commercial properties to the north, east and west.

The property slopes gently down to the southwest toward Alderwood Mall Boulevard. Based on the ALTA/NSPS survey of the site, surface elevations range from about 396 feet in the northeast portion of the site, to about 378 feet near the southwest corner of the property, along the sidewalk and the driveway entrance from Alderwood Mall Boulevard. There are rockeries, up to about five feet in height, along the eastern and southern property lines. It appears that some grading, including some fill placement, may have occurred during previous development of this area.

GEOLOGIC SETTING

The site lies within the central Puget Lowland. The lowland is part of a regional north-south trending trough that extends from southwestern British Columbia to near Eugene, Oregon. North of Olympia, Washington, this lowland is glacially carved, with a depositional and erosional history including at least four separate glacial advances/retreats. The Puget Lowland is bounded to the west by the Olympic Mountains and to the east by the Cascade Range. The lowland is filled with glacial and nonglacial sediments.

We referred to the "Geologic Map of the Edmonds East and part of the Edmonds West Quadrangles, Washington", prepared by James P. Minard (USGS 1983), during our project research. The geologic map indicates that the site vicinity is underlain by Quaternary Vashon glacial till (Qvt). Glacial till consists of compact clay, silt, sand, gravel, cobbles and boulders deposited at the base of the continental glacier. Our explorations generally encountered surficial layers of pavement and undocumented fill overlying the competent native glacial soils.

FIELD INVESTIGATION

Four (4) exploratory soil borings were completed to evaluate the subsurface soil and groundwater conditions in the asphalt parking lot areas of the property. The soil explorations were completed on August 19, 2021 with a subcontracted drill rig. The soil borings extended to depths of approximately 11.5 to 28.5 feet below the existing ground surface (bgs). Krazan geotechnical representatives were present during the explorations, examined the soils and geologic conditions encountered, obtained samples of the subsurface soils using the Standard Penetration Test (SPT), and maintained logs of the explorations. The approximate locations of the soil borings are shown on the Site Plan in Figure 2.

Representative samples of the subsurface soils encountered in the geotechnical explorations were collected and sealed in plastic bags. The soils encountered in the borings were visually classified in general accordance with the Unified Soil Classification System (USCS). A more detailed description of the field investigation is presented in Appendix A.

SOIL PROFILE AND SUBSURFACE CONDITIONS

The information provided below includes a brief summary of the materials encountered in the soil explorations. Detailed soil logs are presented in Appendix A.

Soil boring B-1 was located in the parking lot north of the demolished building foundation, in the central portion of the property. Asphalt pavement was encountered to a depth of about 0.2 feet. Underlying the asphalt, the soil boring encountered moist, loose to medium dense, silty sand with a trace of gravel to a depth of approximately 3.0 feet bgs. We interpreted the silty sand with a trace of gravel to be undocumented fill. Below the fill material, the soil boring encountered moist, dense to very dense silty sand with gravel to the depth explored of approximately 28.5 feet bgs. We interpreted the dense to very dense silty sand with gravel to be native glacial till soil.

Soil boring B-2 was located in the parking lot west of the demolished building foundation, in the southwest portion of the property. Asphalt pavement was encountered to a depth of about 0.2 feet. Underlying the asphalt, the soil boring encountered moist to wet, medium dense, silty sand with gravel to a depth of approximately 5.5 feet bgs. We interpreted the silty sand with gravel to be undocumented fill. Below the fill material, the soil boring encountered moist to wet, medium dense silty sand with gravel to a depth of about 7.0 feet bgs. We interpreted the medium dense silty sand with gravel to be weathered native granular soils. Underlying the weathered horizon, the soil boring encountered moist, dense to very dense silty sand with gravel to the depth explored of approximately 11.5 feet bgs. We interpreted the dense to very dense silty sand with gravel to be native glacial till soil.

Soil boring B-3 was located in the parking lot east of the demolished building foundation, in the southeast portion of the property. Asphalt pavement was encountered to a depth of about 0.2 feet. Underlying the asphalt, the soil boring encountered moist to wet, medium dense, silty sand with gravel to a depth of approximately 5.5 feet bgs. We interpreted the silty sand with gravel to be undocumented fill. Below the fill material, the soil boring encountered moist to wet, medium dense silty sand with gravel to a depth of about 7.0 feet bgs. We interpreted the medium dense silty sand with gravel to be weathered native granular soils. Underlying the weathered horizon, the soil boring encountered moist, dense to very dense silty sand with gravel to the depth explored of approximately 11.5 feet bgs. We interpreted the dense to very dense silty sand with gravel to be native glacial till soil.

Soil boring B-4 was located in the parking lot in the northeast portion of the site. Asphalt pavement was encountered to a depth of about 0.2 feet. Underlying the asphalt, the soil boring encountered moist, loose to medium dense, silty sand with a trace of gravel to a depth of approximately 8.0 feet bgs. We interpreted the silty sand with a trace of gravel to be undocumented fill. Underlying the fill, the soil boring encountered moist, dense to very dense, silty sand with a trace of gravel to a depth of approximately 10.5 feet bgs, which was interpreted as native weathered soil. Below the silty sand, the boring encountered moist, hard, bedded gray sandy silt to a depth of 14.0 feet bgs, which was interpreted as glacially consolidated cohesive soil. Underling the sandy silt, the boring encountered moist, dense to very dense silty sand with gravel to the depth explored of approximately 11.5 feet bgs. We interpreted the dense to very dense silty sand with gravel to be native glacial till soil.

There is potential for cobbles and boulders to be encountered in the native glacial soils and there may be debris in the fill material. There is also potential for thicker layers of loose/soft soils, and/or undocumented fill in unexplored areas of the site.

GROUNDWATER

The soil borings were checked for the presence of groundwater during drilling operations. Groundwater seepage was only encountered in the B-3 at a depth of about 10.5 feet bgs. The groundwater seepage was interpreted as perched groundwater. It is our opinion that perched groundwater could occur in some areas of this property, especially during and after prolonged periods of wet weather.

Perched water occurs when surface water infiltrates through less dense, more permeable soils and accumulates on top of a relatively low permeability soil layer. 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 water to decrease during drier times of the year and increase during wetter periods.

It should be recognized that groundwater elevations may fluctuate with time. The groundwater level will be dependent upon seasonal precipitation, irrigation, land use, and climatic conditions, as well as other factors. Therefore, groundwater levels at the time of the field investigation may be different from those encountered during the construction phase of the project. The evaluation of such factors is beyond the scope of this report.

GEOLOGIC HAZARDS

Erosion Hazard

The Natural Resources Conservation Service (NRCS) map for Snohomish County Area, Washington indicates that the soils in the site vicinity consist of Alderwood Urban land (2 to 8 percent slopes) (Hydrologic Soil Group B). The NRCS indicates that the Alderwood Urban land complex soils with 2 to 8 percent slopes have a slight to moderate risk of soil erosion when exposed.

It has been our experience that soil erosion potential can be minimized through landscaping and surface water runoff control. Typically, erosion of exposed soils will be most noticeable during periods of rainfall and may be controlled by the use of normal temporary erosion control measures, such as silt fences, hay bales, mulching, control ditches or diversion trenching, and contour furrowing. Erosion control measures should be in place before the onset of wet weather.

Seismic Hazard

The 2018 International Building Code (IBC), Section 1613.2.2, refers to Chapter 20 of ASCE 7-16 for Site Class Definitions. It is our opinion that the overall soil profile corresponds to Site Class C as defined by Table 20.3-1 "Site Class Definitions," according to the ASCE 7-16 Standard. Site Class C applies to a "very dense soil and soft rock" profile. The seismic site class is based on a soil profile extending to a depth of 100 feet. The soil explorations on this site extended to a maximum depth of approximately 28.5 feet and this seismic site class designation is based on the assumption that dense to very dense conditions continue below the depth explored.

We referred to the Applied Technology Council (ATC) website and 2018 IBC to obtain values for S_S, S_{MS}, S_{DS}, S_1, S_{M1}, S_{D1}, F_a, F_v *and* T_s. The ATC website utilizes the most updated published data on seismic conditions from the United States Geological Survey. The seismic design parameters for this site are presented in the following table:

Seismic Design Parameters
(Reference: 2018 IBC Section 1613.2.2, ASCE7-16, and ATC)

Seismic Item	Value
Site Class	C
Site Coefficient F_a	1.200
S_s	1.303
S_{MS}	1.563
S_{DS}	1.042
Site Coefficient F_v	1.500
S_1	0.461
S_{M1}	0.6.91
S_{D1}	0.461
PGA*	0.558

**For an earthquake magnitude with a 2 percent probability of being exceeded in 50 years.*

Additional seismic considerations include liquefaction potential and amplification of ground motions by loose/soft soil deposits. The liquefaction potential is highest for loose sand with a high groundwater table. The medium dense to very dense native soils interpreted to underlie the site are considered to have a low potential for liquefaction and amplifications of ground motion.

The Liquefaction Susceptibility Map of Snohomish County, Washington, by Stephen Palmer, et al. (WADNR, September 2004) indicates that the property is located in an area of very low liquefaction susceptibility. Therefore, a site-specific liquefaction analysis was not performed.

CONCLUSIONS AND RECOMMENDATIONS

General

It is our opinion from a geotechnical standpoint that the site is compatible with the proposed multi-story building with underground parking, provided that our geotechnical recommendations are incorporated into project plans and are implemented during construction.

Our geotechnical explorations generally encountered loose to medium dense undocumented fill, underlain by medium dense to very dense native granular soils. Based on our explorations, the competent native soils were encountered at depths of about 2.0 to 8.0 feet bgs. The thickness of the fill materials may vary in unexplored areas of the site.

The medium dense to very dense native soils are interpreted to be glacially compacted deposits, and are considered suitable for foundation support. The undocumented fill is not considered suitable for foundation support in its current condition.

The dense to very dense glacial till soils interpreted to underlie this site have very low permeability and are not considered suitable for infiltration management of stormwater runoff.

Groundwater seepage was encountered in soil boring B-3 at approximately 10.5 feet in depth at the time of our explorations. Moist to wet soils were encountered in soil borings B-2, and B-3 directly below the asphalt pavement. In our opinion, there is the potential for perched groundwater seepage to develop on this site, particularly after periods of heavy precipitation.

Recommendations for foundation support and temporary shoring design, including over-excavation of existing fill materials, are provided in this report. Specific geotechnical recommendations for foundation design are presented in the Shallow **Foundations** section of this report.

The soils encountered in our explorations are considered to be moisture sensitive and will be difficult or impossible to compact in wet conditions. The moisture content of the silty soils is important in determining if the soils can be used as structural fill at the time of construction. Krazan and Associates is available on request to evaluate the suitability of the on-site soils for use as structural fill material during earthwork construction. The moisture sensitive subgrade soils can degrade rapidly in wet weather. The exposed subgrade should be protected from construction traffic with a layer of crushed rock or lean concrete during wet weather construction.

Site Preparation

Site preparation for this project is anticipated to included demolition of the existing slab and foundations and some removal of asphalt pavement. If loose undocumented fill is encountered, caving of may occur during excavation or if the excavations are left open. Caving could cause undermining of adjacent portions of the existing pavement. The contractor should be prepared to shore the trench walls if caving conditions are encountered.

If over-excavation below the planned bottom of footing elevation is necessary to expose the suitable native subgrade soil, it may be feasible to place structural fill to the planned footing elevation. However, if caving conditions are encountered, it may be prudent to consider using clean rock ballast, clean rock chips, or Controlled Density Fill (CDF) as structural fill materials to eliminate the need for vibratory compaction methods. Vibratory compaction may cause caving of the excavation walls. Alternatively, the over-excavation could be filled with extra concrete. The structural engineer should be consulted regarding rebar placement if extra concrete is to be used.

During wet weather conditions, which typically occur from October through May, subgrade stability problems and grading difficulties may develop due to excess moisture, disturbance of moisture sensitive soils and/or the presence of perched groundwater. Earthwork construction during extended periods of wet weather could create the need to remove wet disturbed soils if they cannot be suitably compacted due to elevated moisture contents. The on-site soils encountered in our borings have significant silt content, and are considered to be moisture sensitive. Silty soils typically disturb easily and are difficult to compact when wet. If over-excavation is necessary, it should be confirmed through continuous monitoring and testing by a qualified geotechnical engineer or geologist. Soils that have become unstable may require drying to near their optimal moisture content before compaction is feasible. Selective drying may be accomplished by scarifying or windrowing surficial material during extended periods of dry, warm weather (typically during the summer months). If the soils cannot be dried back to a workable moisture condition, remedial measures may be required. Preparation of the site for wet weather conditions may consist of the placement of a layer of aggregate base for the protection of exposed soils during construction.

It should be understood that even if Best Management Practices (BMPs) for soil protection are implemented for the wet season, there is a significant chance that additional soil mitigation work will be needed.

Any buried structures encountered during construction should be properly removed and backfilled. Excavations, depressions, or soft and pliant areas extending below the planned finish subgrade levels should be excavated to expose firm undisturbed native soil, and backfilled with structural fill. In general, any septic tanks, underground storage tanks, debris pits, cesspools, or similar structures should be completely removed. Concrete footings should be removed to an equivalent depth of at least 3 feet below proposed footing elevations or as recommended by the geotechnical engineer. The resulting excavations should be backfilled with structural fill.

A geotechnical representative of our firm should be present to observe and evaluate foundation excavations, native subgrade soil, and structural fill placement. This observation and evaluation are integral parts of our service, as acceptance of foundation construction is dependent upon the excavation and subgrade conditions and the stability of the material. The geotechnical engineer may reject any material that does not meet compaction and stability requirements. Further recommendations, contained in this report, are predicated upon the assumption that earthwork construction will conform to the recommendations set forth in this section and in the Structural Fill Section of this report.

Temporary Excavations

The on-site soils have variable cohesion strengths, therefore the safe angles to which these materials may be cut for temporary excavations is variable, as the soils may be prone to caving and slope failures in temporary excavations deeper than 4 feet. Very loose to medium dense undocumented fill was encountered in our borings. Temporary excavations in the undocumented fill soils should be sloped no steeper than 2H:1V (horizontal to vertical) where room permits. However, if medium dense to dense native soils area exposed during excavations, then the excavations maybe sloped to 1H:1V. Steeper inclinations may be feasible for temporary excavations in very dense glacial till.

It may be desirable to use near-vertical excavation walls to reduce disturbance to the existing pavement. If so, the contractor should be prepared to shore the excavation walls for soil stability and worker safety.

All temporary cuts should be in accordance with Washington Administrative Code (WAC) Part N, Excavation, Trenching, and Shoring. The temporary slope cuts should be visually inspected daily by a qualified person during construction work activities and the results of the inspections should be included in daily reports. The contractor is responsible for maintaining the stability of the temporary cut slopes and minimizing slope erosion during construction. The temporary cut slopes should be closely monitored until the foundations or utility installations are completed and backfilled. Materials should not be stored and equipment operated within 10 feet of the top of any temporary cut slope.

A Krazan & Associates geologist or geotechnical engineer should observe, at least periodically, the temporary cut slopes during the excavation work. The reason for this excavation observation is that all soil conditions may not be fully delineated by the limited sampling of the site from the geotechnical explorations. In the case of temporary slope cuts, the existing soil conditions may not be fully revealed until the excavation work exposes the soil. Typically, as excavation work progresses, the maximum inclination of the temporary slope will need to be evaluated by the geotechnical engineer so that supplemental recommendations can be made. Soil and groundwater conditions can be highly variable. Scheduling for soil work will need to be adjustable, to deal with unanticipated conditions, so that the project can proceed smoothly and required deadlines can be met. If any variations or undesirable conditions are encountered during construction, Krazan & Associates should be notified so that supplemental recommendations can be made.

Structural Fill

Fill placed beneath foundations 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. 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 structural fill should be suitably prepared as described in the **Site Preparation** section of this report, and the prepared subgrade should be evaluated by a Krazan representative, prior to beginning fill placement.

Best Management Practices (BMP's) should be followed when considering the suitability of the on-site soils for use as structural fill. The on-site soils may be suitable for reuse as structural fill, provided the soil is free of organic material and debris, and it is within ± 2 percent of the optimum moisture content. The on-site soils encountered in our borings have significant silt content, and are considered to be moisture sensitive and will be difficult to compact during the wet weather. *Any cobbles and boulders and organic debris* should be removed prior to use as structural fill. If the on-site soils are stockpiled for later use as structural fill, the stockpiles should be covered to protect the soil from wet weather conditions. We recommend that a representative of Krazan & Associates be on site during the excavation work to determine which soils are suitable for placement as structural fill.

Imported, all weather granular structural fill material should consist of well-graded gravel or a sand and gravel mixture with a maximum grain size of 3 inches and less than 5 percent fines (material passing the U.S. Standard No. 200 Sieve). Structural fill can also consist crushed rock, rock spalls and controlled density fill (CDF). All structural fill material should be submitted for approval to the geotechnical engineer at least 48 hours prior to delivery to the site.

Granular fill soils should be placed in horizontal lifts not exceeding 8 inches in thickness prior to compaction, moisture-conditioned as necessary, (moisture content of soil shall not vary by more than ±2 percent of optimum moisture) and the material should be compacted to at least 95 percent of the maximum dry density based on ASTM D1557 Test Method. In-place density tests should be performed on all structural fill to document proper moisture content and adequate compaction. Additional lifts should not be placed if the previous lift did not meet the compaction requirements or if soil conditions are not considered stable.

Foundations

General: The proposed structure may be supported on a conventional spread foundation system bearing on the medium dense or firmer native soils or on structural fill including granular soils, rock spalls or CDF extending to the medium dense or firmer native soils. Based on our soil borings, we interpreted the medium dense or firmer native load bearing soils at this site to range from about 2.0 to 8.0 feet bgs. The depth to bearing soils may vary in unexplored portions of the site. A representative of Krazan and Associates should evaluate the foundation bearing soil and evaluate all structural fill subgrade and monitor all structural fill placement, where utilized.

We have assumed column loads of 50,000 to 60,000 pounds and wall loads of 4,000 plf for the soil bearing capacity and settlement analyses. *We should be contacted to re-evaluate the potential settlement and the allowable bearing pressure, if the design loads vary significantly from these assumed values*

Soil Bearing: Conventional shallow spread footings supported on medium dense of firmer native soils, or on structural fill extending to the dense or firmer native soils, may be designed using an allowable soil bearing pressure of **4,000 pounds per square foot (psf)** for dead plus live loads. This value may be increased by 1/3 for short duration loads such as wind or seismic loading. A representative of Krazan and Associates should evaluate the foundation bearing soil and observe structural fill placement.

Footings should have a minimum embedment depth of 18 inches below pad subgrade (soil grade) or adjacent exterior grade, whichever is lower. Footing widths should be based on the anticipated loads and allowable soil bearing pressure. Footings should have a minimum width of at least 12 inches regardless of load. Water should not be allowed to accumulate in footing trenches. All undocumented fill and loose or disturbed soils should be removed from the foundation excavations prior to placing concrete. The exposed subgrade soil should be protected from construction traffic with a layer of crushed rock or lean concrete during wet weather construction.

Structural Fill in Footing Areas: If structural fill consisting of granular soils and rock spalls are used, then the foundation excavations would need to be widened on both sides of the footing a distance equal to one-half of the depth of the over-excavation below the bottom of the footing. Structural fill consisting granular soils should be compacted to at least 95 percent of the maximum dry density based on ASTM Test Method D1557. To reduce the volume of extra excavation needed for the footing trenches and to simplify structural fill placement, it may be practical to place CDF to fill the deeper footing trenches to the planned footing subgrade elevations. If CDF is used, the trench may be excavated only slightly wider (6 inches wider on each side) than the footing.

Potential Foundation Settlement: For foundations constructed as recommended, the total settlement is not expected to exceed one-inch. Differential settlement should be less than ½-inch. Most settlement is expected to occur during construction, as the loads are applied. However, additional post-construction settlement may occur if the foundation soils are flooded or saturated. It should be noted that the risk of liquefaction and seismically induced settlement is considered low, given the composition and density of the native, on-site soils.

Design Parameters – Lateral Resistance: Resistance to lateral displacement can be computed using an allowable friction factor of 0.4 acting between the bases of foundations and the supporting subgrade soil. Lateral resistance for footings can also be developed using an allowable equivalent fluid passive pressure of 350 pounds per cubic foot (pcf) acting against the appropriate vertical footing faces (neglecting the upper 12 inches). The allowable friction factor and allowable equivalent fluid passive pressure values include a factor of safety of 1.5. The frictional and passive resistance of the soil may be combined without reduction in determining the total lateral resistance. These values are based on footings founded on competent native soils or on structural fill extending to the competent native soils, and structural fill placement in all foundation excavations

Foundation Drainage: Seasonal rainfall, water run-off, and the normal practice of watering trees and landscaping areas around the proposed structures, should not be permitted to flood and/or saturate foundation subgrade soils. To prevent the buildup of water within the footing areas, continuous footing drains (with cleanouts) should be provided at the bases of the footings. The footing drains should consist of a minimum 4-inch diameter rigid perforated PVC pipe, sloped to drain, with perforations placed near the bottom and enveloped in all directions by washed rock and wrapped with filter fabric to limit the migration of silt and clay into the drain. Footing drains should be directed to a suitable outlet, and should be separated from roof and shoring drainage systems.

Temporary Shoring Walls

General: We understand that the excavations for the building foundations may be on the order of 5 to 32 feet below the existing grade. In our opinion, a soil nail wall or soldier pile wall may be the most practical types of shoring systems for this project.

A shoring wall can be installed to provide top down support for the excavation. The shoring wall can be designed as a temporary 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 shoring system, adjacent ground, and nearby structures, should be monitored for movement during construction. A system of survey points should be established prior to commencing with the excavation activities. Survey readings should be taken periodically and be compared to the original baseline measurements. The survey monitoring should continue until the permanent walls and floors are in place at least to level of the surrounding grade, and survey cessation has been recommended by the geotechnical engineer, and cessation is approved by the jurisdiction.

It should be understood that the risk of movement and settlement in adjacent structures can be reduced with proper construction procedures and with a well-designed, sufficiently stiff shoring system, however a certain level of risk of movement in the nearby features will always be present.

Soldier Pile Walls

A soldier pile wall typically consists of a series of steel H-beams installed prior to the building excavation. The soldier piles are installed 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 Control Density Fill (CDF). The concrete shafts are typically embedded below the bottom of the planned excavation by a depth of one to two times the height of the cut to be shored, if tiebacks are not used. The steel beams are extended above the finished ground surface to provide shoring capabilities for the cut. The beams are typically spanned by pressure treated timber lagging installed from the top down as the excavation is being made. The H-beam type, shaft diameter, shaft embedment, and pile spacing are dependent on the nature of the soils anticipated in the cut and at depth, wall height, drainage conditions, the need for tiebacks, and the final geometry.

Tiebacks extending into the native soils behind the shoring wall may be used to provide additional resistance for the shoring wall. The feasibility of the tiebacks, however, will depend on boundary line issues and overall cost. It might be possible to brace the shoring wall internally to avoid installing the tiebacks, or the building configuration may be modified to allow more room between the building and property boundaries. These options should be evaluated during the design phase of the project.

Solider Pile 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. The wall designer should be provided a copy of our report, and we should be retained to review the geotechnical aspects of the shoring wall design prior to construction.

If the shoring wall is allowed to yield at the top at least one thousandth of the height of the above ground portion of the wall, the wall should be designed for an active loading condition. If the wall is restrained from yielding by external bracing, tiebacks, or wall stiffness, the wall should be designed for an at-rest loading condition. For an active loading condition, the design of the soldier piles and lagging should be calculated based on a triangular pressure distribution equivalent to that exerted by a fluid with a density of 35 pounds per cubic foot (pcf). If at-rest conditions prevail, the wall should be designed to resist loads resulting from a triangular pressure distribution equivalent to that exerted by a fluid with a density of 55 pcf. These values are based on the assumption of well-drained retained soil and a level backfill behind the wall.

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 tiebacks extending into the native soils behind the shoring wall. The passive resistance for soldier piles embedded in the dense to very dense glacial till could be calculated based on a 350 pcf equivalent fluid density acting on two effective pile diameters below the base of the cut. This passive resistance value incorporates a factor of safety of 1.5. The below-grade portion of the wall should not be shorter than wall stick-up height, if tiebacks are not used.

Soldier Pile Wall Installation: The shoring wall should be installed by a shoring contractor experienced with this type of system. Although we anticipate that an open-hole drilling method will be adequate for installing the soldier piles, the shoring contractor should be capable of casing the holes if sloughing and/or water seepage is encountered. It might be prudent to perform a few "test" holes to confirm installation conditions prior to finalizing work plans. Any sloughing or water that may collect in the drilled holes should be removed prior to pouring grout. Grout should be readily available on site at the time the holes are drilled. The holes should not be left open for any length of time, as that may increase the potential for caving and water seepage to impact wall installation.

There may be obstructions in unexplored areas of the site. The contractor should be prepared to penetrate or remove obstructions if they are encountered.

If groundwater seepage is encountered, we recommend that the concrete be tremied from the bottom of the soldier pile borings to displace the groundwater to the surface. The shoring contractor might elect to introduce polymers into the soldier pile borings to reduce the effects of seepage. We should be retained to observe shoring wall installation.

Tiebacks

General: Tieback soil anchors may be used to provide additional resistance for the shoring wall, if needed. The tiebacks can be permanent and form a part of the structure, or temporary until the excavation is braced by the building. If the tiebacks are made permanent, they will have to meet double corrosion standards, and soil creep needs to be considered, when determining their capacity. The contractor or the wall designer should submit working drawings for review showing their proposed method of tieback 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 tieback strands/bars shall be detailed on the drawings. We recommend that five percent of the tiebacks, but not less than two, be treated as verification tiebacks 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 portion of all tiebacks where anchorage is achieved by soil/grout adhesion 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 by a line extending horizontally from the base of the excavation into the soil a distance of one-third the height of the shoring wall (H/3). The line should extend up from the base elevation at an angle of 60 degrees above the horizontal. We expect that medium dense sand and gravel will exist beyond the no-load zone. However, we do not have subsurface data at the tieback locations. We recommend that we monitor soil conditions during tieback installation in order to evaluate adequate penetration into these soils.

The portion of the tieback 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 tieback rod within the length of the no-load zone.

Soil Design Values: We have limited explorations in the areas where the tiebacks might be planned. Therefore, the design strengths for tieback adhesion are difficult to estimate. For use in design of the verification tiebacks installed in the dense to very dense glacial till soils, we estimate an allowable grout to soil bond strength of 22 pounds per square inch (pci). We expect that this value is high if silts are encountered. If silts are encountered, creep will become an important factor in the design loads. This value is presented for planning purposes only and should be confirmed or modified using the data obtained from the verification testing prior to production tieback installation.

Tieback 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 tiebacks. Secondary grouting to increase soil adhesion may be used; however, if secondary grouting is used, the tiebacks should be tested using the methods outlined for the verification testing.

Five percent of the tiebacks, but a minimum of two, should be verification tested to 200 percent of the tieback design capacity. The verification 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 tiebacks should be proof tested to at least 130 percent of design capacity. The tieback testing program should be reviewed and monitored by a geotechnical representative.

Soil Nail Wall Design Recommendations

General: Soil nailing is a technique for reinforcing, stabilizing, and retaining steep slopes through the introduction of relatively small, closely spaced anchors (usually steel bars) grouted into a soil mass, the face of which is then locally stabilized. A zone of reinforced ground is the result, which functions as a temporary soil retention system. Soil nail walls are retaining walls that are built from the top downwards in situations where the soil has enough apparent cohesion that it can stand up on its own during construction. The walls consist of soil nails, typically spaced three to six feet on center with a reinforced shotcrete facing.

The soil nail wall proposed for protection of the excavation should consist of a grid of soil nail anchors drilled and grouted into the slope, and attached to a reinforced shotcrete facing. This wall system is suitable for the intended purpose of supporting and protecting the excavation from erosion and sloughing, but should not be considered a permanent slope stabilization method. The final extent and heights of the soil nail wall should be determined after an accurate site survey is completed.

The process begins by making a 4-foot vertical cut along the top of the shoring area, followed immediately by the drilling, placement and grouting of the soil nails. A geotextile drainage composite is placed over the face of the excavation between the soil nails to avoid buildup of hydrostatic pressure behind the shotcrete facing. The cut is then covered with reinforcing steel and wire mesh. Shotcrete is placed over the face of the excavation, incorporating the reinforcing steel and wire mesh into the soil nail wall. As the excavation progresses downward, the drainage composite is extended until reaching the base of the excavation where weep holes are placed through the shotcrete to allow seepage to be collected in the drainage system for the building retaining walls.

The soil nail spacing, lengths, diameters, angle of declination, and bar type should be determined by the shoring designer. The grouted soil nails should be sized to provide resistance for an active pressure resulting from a fluid with an equivalent density of 35 pounds per cubic foot (pcf) acting along the back of the wall. This design pressure does not include surcharge loading due to the slope or other factors. We are available to consult with the design team as grading plans are developed to discuss surcharge loads that may be necessary for the wall design.

Soil nails are not pre-stressed like tiebacks since they are designed as passive elements. Accordingly, soil nail walls will typically deflect more than a soldier pile wall. The shoring designer should provide an estimate of the lateral deflection that is anticipated for the soil nail wall. Caving of loose or granular soil, or from zones of seepage, can require that the shoring contractor modify their installation techniques. This can increase the cost and time necessary to install the wall.

Recommended Soil Design Values: Our explorations generally encountered competent native glacial soils consisting of silty sand with gravel and cobbles. Compressible soils were not encountered in our explorations. For use in design of the soil nails, we present estimated soil parameters in the following table.

These values are presented for planning purposes only and should be confirmed or modified using the data obtained from the verification testing of sacrificial nails prior to the installation of the production soil nails.

Soil Nail Installation and Testing Recommendations: The contractor should be responsible for using equipment suited for the site conditions. A minimum of two nails in each soil type should be verification tested to 200 percent of the design capacity. The verification tests should consist of cyclic loading in increments of 25 percent of the design load. Final soil adhesion design values will be based on the verification test results. The test locations should be determined in the field based on soil conditions observed during construction. At least five percent of the production soil nails should be proof tested to at least 130 percent of design capacity. The shoring designer should provide the soil nail testing program. The soil nail testing program should be reviewed and monitored by Krazan. We can discuss the soil nail test procedures with the contractor at the time of construction.

Design Parameters for Soil Nail

Depth below ground surface (feet)	**Soil Description**	**Estimated *effective* angle of internal friction (degrees)**	**Estimated *effective* cohesion (psf)**	**Allowable Bond Strength (psi)**	**Estimated soil unit weight (moist) (pcf)**
0.5-8.0	Medium dense sand with gravel (undocumented fill)	28	0	6	120
8.5-28.0+	Very dense silty sand with gravel (native glacial soils)	38	100	22	135

Waterproofing

If a damp-proof wall is desired, a building envelope consultant should be retained to provide recommendations. Generally, waterproof barriers, such as bentonite panels, should be used between buried walls and the earth or between walls cast directly against the shoring. We recommend that at a minimum, waterproofing is considered for any below grade storage areas and any other areas that are desired to be moisture proof. For areas that are shored, the waterproofing can be placed over a geo-composite drain prior to pouring or shooting the concrete wall.

Lateral Earth Pressures and Retaining Walls

We have developed criteria for the design of retaining or below grade walls. Our design parameters are based on retention of the granular native soils or structural fill. The wall design criteria are based on the total unit weight of backfill of 135 pounds per cubic foot (pcf) and friction angle of 36 degrees. The parameters are also based on level, well-drained wall backfill conditions. Walls may be designed as "restrained" retaining walls based on "at-rest" earth pressures, plus any surcharge on top of the walls as described below, if the walls are braced to restrain movement and/or movement is not acceptable. Unrestrained walls may be designed based on "active" earth pressure, if the walls are not part of the buildings and some movement of the retaining walls is acceptable. Acceptable lateral movement equal to at least 0.2 percent of the wall height would warrant the use of "active" earth pressure values for design. 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 35 pcf for yielding (active condition) walls, and 55 pcf for non-yielding (at-rest condition) walls.

The following table, titled **Wall Design Criteria**, presents the recommended soil related design parameters for retaining walls with well-drained level backfill.

Wall Design Criteria	
"At-rest" Conditions (Lateral Earth Pressure)	55 pcf Equivalent Fluid Density (EFD) (Triangular Distribution)
"Active" Conditions (Lateral Earth Pressure)	35 pcf (EFD) (Triangular Distribution)
Seismic Increase for "Active" Conditions (Lateral Earth Pressure)	7 psf *H (Uniform Distribution) Where H is the height of the wall in feet
Passive Earth Pressure on Low Side of Wall (includes factor of safety of 1.5)	Neglect upper 1-foot, then 350 pcf (EFD) in glacial till or structural fill
Soil-Footing Coefficient of Sliding Friction (includes factor of safety of 1.5)	0.4 in glacial till or structural fill

If vehicular loads are expected to act on the surface of the wall backfill within a horizontal distance of less than or equal to one-half of the wall height behind the back face of the wall, a live load surcharge should be applied for the design. In this case, we recommend the addition of vehicle surcharges of 70 psf and 100 psf to the active and at-rest earth pressures, respectively.

The stated lateral earth pressures do not include the effects of hydrostatic pressure generated by water accumulation behind the retaining walls or loads imposed by construction equipment, foundations or roadways adjacent to the wall (surcharge loads). To minimize the lateral earth pressure and prevent the buildup of water pressure against the walls, continuous footing drains (with cleanouts) should be provided at the bases of the walls. The footing drains should consist of a minimum 4-inch diameter rigid PVC perforated pipe, sloped to drain, with perforations placed near the bottom. The drainpipe should be enveloped by 6 inches of washed gravel in all directions wrapped in filter fabric to prevent the migration of silt and clay into the drain. Wall drains should be directed to a suitable outlet.

The wall fill material adjacent to and extending a lateral distance of at least 2 feet behind the walls should consist of free-draining granular material. All free-draining backfill should contain less than 3 percent fines (passing the U.S. Standard No. 200 Sieve) based upon the fraction passing the U.S. Standard No. 4 Sieve with at least 30 percent of the material being retained on the U.S. Standard No. 4 Sieve. Alternatively, a drainage composite may be used. This drainage layer should be connected to a tightline. It should be realized that the primary purpose of the free-draining material is the reduction of hydrostatic pressure. Some potential for the moisture to contact the back face of the wall may exist, even with treatment, which may require that more extensive waterproofing be specified for walls, which require interior moisture sensitive finishes.

We recommend that the wall fill be compacted to at least 95 percent of the maximum dry density based on ASTM D1557 Test Method. In-place density tests should be performed to verify adequate compaction. Soil compactors place transient surcharges on the backfill. Consequently, only light hand operated equipment is recommended for fill compaction within 3 feet of walls so that excessive stress is not imposed on the walls.

Floor Slabs and Exterior Flatwork

Before the placement of concrete floors or sidewalks on the site, or before any floor supporting fill is placed, the organic topsoil should be removed. Our soil explorations generally exposed up to 8 feet of undocumented fill overlying competent native soil. If undocumented fill is encountered at the planned slab subgrade elevations, then we recommended that the undocumented fill soils should be excavated to *at least 1 foot* below the planned slab subgrade and then backfilled with structural fill. Deeper excavation may be required, if soft/loose and yielding soil conditions are exposed during over-excavation. We should evaluate the subgrade soil conditions, and observe the over-excavation and structural fill placement during construction.

Floor slabs may be designed using a modulus of subgrade reaction value of k = 150 pounds per cubic inch (pci) for slabs supported on medium dense or firmer native soils or on at least one-foot of structural fill as recommended.

In areas where it is desired to reduce floor dampness, such as areas covered with moisture sensitive floor coverings, we recommend that concrete slab-on-grade floors be underlain by a water vapor retarder system. The water vapor retarder should consist of a vapor retarder sheeting underlain by a minimum of 4-inches of compacted clean (less than 5 percent passing the U.S. Standard No. 200 Sieve), open-graded coarse rock chips of ¾-inch maximum size. The vapor retarder sheeting should be protected from puncture damage. In addition, ventilation of the structure may be prudent to reduce the accumulation of interior moisture.

Erosion and Sediment Control

Erosion and sediment control (ESC) is used to minimize the transportation of sediment to wetlands, streams, lakes, drainage systems, and adjacent properties. Erosion and sediment control measures should be implemented and these measures should be in general accordance with local regulations. As a minimum, the following basic recommendations should be incorporated into the design of the erosion and sediment control features of the site:

1) Phase the soil, foundation, utility and other work, requiring excavation or the disturbance of the site soils, to take place during the dry season (generally May through September). However, provided precautions are taken using Best Management Practices (BMP's), grading activities can be undertaken during the wet season (generally October through April), but it should also be known that this may increase the overall cost of the project.

2) All site work should be completed and stabilized as quickly as possible.

3) Additional perimeter erosion and sediment control features may be required to reduce the possibility of sediment entering the surface water. This may include additional silt fences, silt fences with a higher Apparent Opening Size (AOS), construction of a berm, or other filtration systems.

4) Any runoff generated by dewatering discharge should be treated through construction of a sediment trap if there is sufficient space. If space is limited, other filtration methods will need to be incorporated.

Groundwater Influence on Construction

Groundwater seepage was encountered in soil boring B-3 at approximately 13.5 feet below the surface at the time of our explorations, which was interpreted as perched groundwater. Additionally, moist to wet soils were encountered in soil borings B-1, B-2, and B-4 directly below the asphalt pavement. In our opinion, there is the potential for perched groundwater seepage to develop on this site, particularly after periods of heavy precipitation.

It should be recognized that groundwater elevations may fluctuate with time. The groundwater level will be dependent upon seasonal precipitation, irrigation, land use, and climatic conditions, as well as other factors. Therefore, groundwater levels at the time of the field investigation may be different from those encountered during the construction phase of the project. The evaluation of such factors is beyond the scope of this report.

If groundwater is encountered during construction, we should observe the conditions to determine if dewatering will be needed. Design of temporary dewatering systems to remove groundwater should be the responsibility of the contractor. If earthwork is performed during or soon after periods of precipitation, the subgrade soils may become saturated. These soils may “pump,” and the materials may not respond to densification techniques. Typical remedial measures include: disking and aerating the soil during dry weather; mixing the soil with drier materials; removing and replacing the soil with an approved fill material. A qualified geotechnical engineering firm should be consulted prior to implementing remedial measures to observe the unstable subgrade conditions and provide appropriate recommendations.

Drainage and Landscaping

Special attention to the drainage and irrigation adjacent to the buildings is recommended. Grading should establish drainage away from the structures and this drainage pattern should be maintained. Water should not be allowed to collect adjacent to the structures. Excessive irrigation within landscaped areas adjacent to the structure should not be allowed to occur.

The ground surface should slope away from building pads and pavement areas, toward appropriate drop inlets or other surface drainage devices. It is recommended that adjacent exterior grades be sloped a minimum of 2 percent for a minimum distance of 5 feet away from structures. Roof drains should be tightlined away from foundations to a suitable outlet. Roof drains should not be connected to the footing drains.

Subgrade soils in pavement areas should be inclined at a minimum of 1 percent and drainage gradients should be maintained to carry all surface water to collection facilities, and suitable outlets. These grades should be maintained for the life of the development.

Specific recommendations for and design of storm water disposal systems or septic disposal systems are beyond the scope of our services and should be prepared by other consultants that are familiar with design and discharge requirements.

Utility Trench Backfill

Utility trenches should be excavated according to accepted engineering practices following Occupational Safety and Health Administration (OSHA) standards, by a contractor experienced in such work. The responsibility for the safety of open trenches should be borne by the contractor. Traffic and vibration adjacent to trench walls should be minimized; cyclic wetting and drying of excavation side slopes should be avoided. Depending upon the location and depth of some utility trenches, groundwater flow into open excavations could be experienced, especially during or shortly following periods of precipitation.

All utility trench backfill should consist of structural fill. Utility trench backfill placed in or adjacent to buildings and exterior slabs should be compacted to at least 95 percent of the maximum dry density based on ASTM Test Method D1557. The upper 5 feet of utility trench backfill placed in pavement areas should be compacted to at least 95 percent of the maximum dry density based on ASTM Test Method D1557. Below 5 feet, utility trench backfill in pavement areas should be compacted to at least 90 percent of the maximum dry density based on ASTM Test Method D1557. Pipe bedding should be in accordance with the pipe manufacturer's recommendations.

It is recommended that the utility trenches within the building pads be compacted, as specified in this report, to minimize the transmission of moisture through the utility trench backfill.

The contractor is responsible for removing all water-sensitive soils from the trenches regardless of the backfill location and compaction requirements. The contractor should use appropriate equipment and methods to avoid damage to the utilities and/or structures during fill placement and compaction.

Pavement Design

Our soil explorations generally exposed up to 4.5 feet of undocumented fill overlying competent native soil. If soft/loose undocumented fill soils are encountered in the pavement subgrade, we recommend that subgrade modification techniques be considered. Subgrade modification typically includes the overexcavation of unsuitable materials, the placement of a high-strength geotextile fabric at the bottom of the over-excavated area, and then the placement of structural fill. Subgrade modification such as this is intended to disperse surcharge loads and therefore aid in pavement performance.

Where loose soils are encountered in the pavement subgrade, we recommend overexcavation of the soft/loose or undocumented fill soils to at least 12 inches below the planned pavement subgrade elevation. We recommend that a high-strength geotextile separation fabric, such as Mirafi 600X or equivalent then be placed over the compacted soil. After the fabric is placed, the area should be filled to the planned pavement subgrade elevation with structural fill. It should be noted that subgrade soils that have relatively high silt contents may be highly sensitive to moisture conditions. The subgrade strength and performance characteristics of a silty subgrade material may be dramatically reduced if it becomes wet. Therefore, we recommend that the pavement subgrade not be exposed for long periods, especially during wet weather.

Deeper excavation may be required, if soft/loose and yielding soil conditions are exposed during over-excavation. We should evaluate the subgrade soil conditions, and observe the over-excavation and structural fill placement during construction.

Traffic loads were not provided, however, based on our knowledge of the proposed project, we expect the traffic to range from light duty (passenger automobiles) to heavy duty (firetrucks). The following tables show the minimum recommended pavement sections for both light duty and heavy-duty traffic loads.

ASPHALTIC CONCRETE (FLEXIBLE) PAVEMENT
LIGHT DUTY

Asphaltic Concrete	Aggregate Base*
3.0 in.	6.0 in.

ASPHALTIC CONCRETE (FLEXIBLE) PAVEMENT
HEAVY-DUTY

Asphaltic Concrete	Aggregate Base*
4.0 in.	6.0 in.

PORTLAND CEMENT CONCRETE (RIGID) PAVEMENT

Min. PCC Depth	Aggregate Base*
6.0 in.	6.0 in.

* *95% compaction based on ASTM Test Method D1557*

The pavement specification in Appendix C provides additional recommendations. The asphaltic concrete depth in the flexible pavement tables should be a surface course type asphalt, such as Washington Department of Transportation (WSDOT) ½-inch HMA. The rigid pavement design is based on a Portland Cement Concrete (PCC) mix that has a 28-day compressive strength of 4,000 pounds per square inch (psi) with a fiber mesh. The design is also based on a concrete flexural strength or modulus of rupture of 575 psi.

Testing and Inspection

A representative of Krazan & Associates, Inc. should be present at the site during the earthwork activities to confirm that actual subsurface conditions are consistent with the exploratory fieldwork. This activity is an integral part of our services as acceptance of earthwork construction is dependent upon compaction testing and stability of the material. This representative can also verify that the intent of these recommendations is incorporated into the project design and construction. We should also be present during the construction of stormwater management system to evaluate the soils. Krazan & Associates, Inc. will not be responsible for grades or staking, since this is the responsibility of the Prime Contractor. Furthermore, Krazan & Associates is not responsible for the contractor’s procedures, methods, scheduling or management of the work site.

LIMITATIONS

Geotechnical engineering is one of the newest divisions of Civil Engineering. This branch of Civil Engineering is constantly improving as new technologies and understanding of earth sciences improves. Although your site was analyzed using the most appropriate current techniques and methods, undoubtedly there will be substantial future improvements in this branch of engineering. In addition to improvements in the field of geotechnical engineering, physical changes in the site either due to excavation or fill placement, new agency regulations or possible changes in the proposed structure after the time of completion of the soils report may require the soils report to be professionally reviewed. In light of this, the owner should be aware that there is a practical limit to the usefulness of this report without critical review. Although the time limit for this review is strictly arbitrary, it is suggested that two years be considered a reasonable time for the usefulness of this report.

This report has been prepared for the exclusive use of koz Development LLC and their assigns. Foundation and earthwork construction are characterized by the presence of a calculated risk that soil and groundwater conditions have been fully revealed by the original foundation investigation. This risk is derived from the practical necessity of basing interpretations and design conclusions on limited sampling of the earth. Our report, design conclusions and interpretations should not be construed as a warranty of the subsurface conditions. Actual subsurface conditions may differ, sometimes significantly, from those indicated in this report. The recommendations made in this report are based on the assumption that soil conditions do not vary significantly from those disclosed during our field investigation. The findings and conclusions of this report can be affected by the passage of time, such as seasonal weather conditions, manmade influences, such as construction on or adjacent to the site, natural events such as earthquakes, slope instability, flooding, or groundwater fluctuations. If any variations or undesirable conditions are encountered during construction, the geotechnical engineer should be notified so that supplemental recommendations can be made.

The conclusions of this report are based on the information provided regarding the proposed construction. If the proposed construction is relocated or redesigned, the conclusions in this report may not be valid. The geotechnical engineer should be notified of any changes so that the recommendations can be reviewed and reevaluated.

Misinterpretations of this report by other design team members can result in project delays and cost overruns. These risks can be reduced by having Krazan & Associates, Inc. involved with the design teams' meetings and discussions after submitting the report. Krazan & Associates, Inc. should also be retained for reviewing pertinent elements of the design team's plans and specifications. Contractors can also misinterpret this report. To reduce this, risk Krazan & Associates. Inc. should participate in pre-bid and preconstruction meetings, and provide construction observations during the site work.

This report is a geotechnical engineering investigation with the purpose of evaluating the soil conditions in terms of foundation design. The scope of our services did not include any environmental site assessment for the presence or absence of hazardous and/or toxic materials in the soil, groundwater or atmosphere, or the presence of wetlands. Any statements or absence of statements, in this report or on any soils log regarding odors, unusual or suspicious items, or conditions observed are strictly for descriptive purposes and are not intended to convey engineering judgment regarding potential hazardous and/or toxic assessments.

The geotechnical information presented herein is based upon professional interpretation utilizing standard engineering practices and a degree of conservatism deemed proper for this project. It is not warranted that such information and interpretation cannot be superseded by future geotechnical developments. We emphasize that this report is valid for this project as outlined above, and should not be used for any other site. Our report is prepared for the exclusive use of our client. No other party may rely on the product of our services unless we agree in advance to such reliance in writing.

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If you have any questions, or if we may be of further assistance, please do not hesitate to contact our office at (425) 485-5519.

Respectfully submitted,
KRAZAN & ASSOCIATES, INC.

11/01/2021

Michael D. Rundquist, P.E.
Senior Project Manager

MDR:VC

Vicinity Map

(Not to Scale)

4301 Alderwood Mall Blvd, Lynnwood, WA

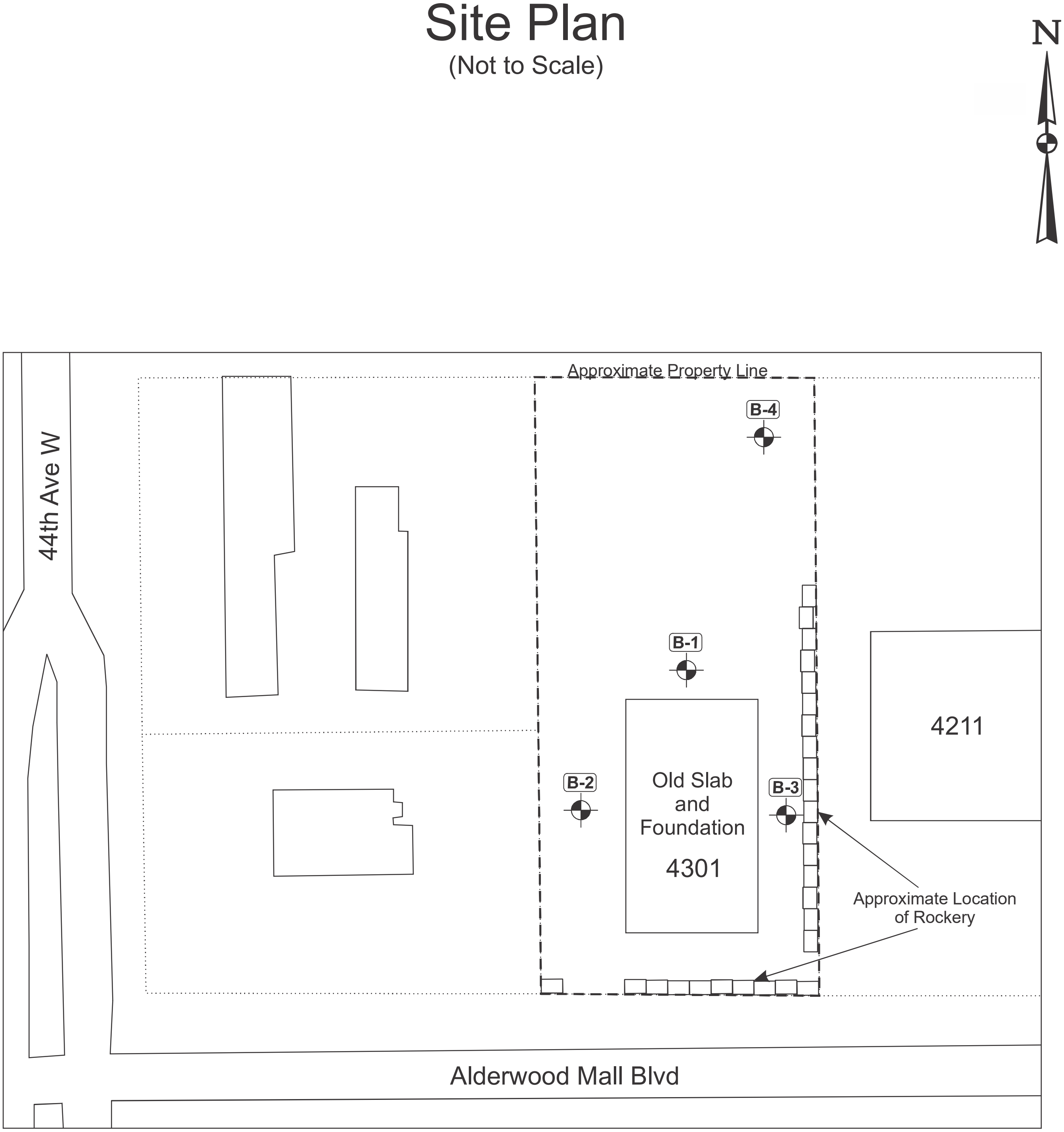
Reference: The Vicinity Map is based on the USGS Topographic Map titled, "East Edmonds, WA, US Topo Map" dated 2020

Krazan & ASSOCIATES, INC.

Proposed Multi-Story Building - Lynnwood, WA		
Date: Aug 2021	Project Number: 092-21009	
Drawn By: EC	Figure 1	Not to scale

Site Plan

(Not to Scale)



Reference: The site plan is based on the USGS Topographic Map titled, "East Edmonds, WA, US Topo Map" dated 2020

LEGEND

Number and Approximate Location of Soil Boring

Krazan & ASSOCIATES, INC.

Proposed Multi-Story Building - Lynnwood, WA		
Date: August 2021	Project Number: 092-21009	
Drawn By: EC	Figure 2	Not to scale

APPENDIX A

FIELD INVESTIGATION AND LABORATORY TESTING

Field Investigation

The field investigation consisted of a surface reconnaissance and a subsurface exploration program. Four (4) geotechnical borings were completed for sampling the subsurface soils at this site. The soil borings were completed on August 17, 2021 by a Krazan subcontractor utilizing a mobile drill rig. The soil borings were advanced to depths of about 11.5 to 28.5 feet below the existing ground surface. The approximate exploration locations are shown on the Site Plan in Figure 2. The depths shown on the attached soil logs are below the existing ground surface at the time of our exploration.

Soil samples were obtained by using the Standard Penetration Test (SPT) as described in ASTM Test Method D1586. The Standard Penetration Test and sampling method consists of driving a standard 2-inch outside-diameter, split barrel sampler into the subsoil with a 140-pound hammer free falling a vertical distance of 30 inches. The summation of hammer-blows required to drive the sampler the final 12-inches of an 18-inch sample interval is defined as the Standard Penetration Resistance, or N-value. The blow count is presented graphically on the boring logs in this appendix. The resistance, or "N" value, provides a measure of the relative density of granular soils or of the relative consistency of cohesive soils.

The soils encountered were logged in the field during the subsurface exploration and, with supplementary laboratory test data, are described in accordance with the Unified Soil Classification System (USCS). All samples from the explorations were returned to our laboratory for evaluation. The logs of the soil explorations are presented in this appendix.

Laboratory Testing

The laboratory testing program was developed primarily to determine the index properties of the soils. Test results were used for soil classification and as criteria for determining the engineering suitability of the surface and subsurface materials encountered. The laboratory test results are included in this appendix.

Soil Classification

USCS Soil Classification				
Major Division			Group Description	
Coarse-Grained Soils < 50% passes #200 sieve	Gravel and Gravelly Soils < 50% coarse fraction passes #4 sieve	Gravel (with little or no fines)	GW	Well-Graded Gravel
			GP	Poorly Graded Gravel
		Gravel (with > 12% fines)	GM	Silty Gravel
			GC	Clayey Gravel
	Sand and Sandy Soils > 50% coarse fraction passes #4 sieve	Sand (with little or no fines)	SW	Well-Graded Sand
			SP	Poorly Graded Sand
		Sand (with > 12% fines)	SM	Silty Sand
			SC	Clayey Sand
Fine-Grained Soils > 50% passes #200 sieve	Silt and Clay Liquid Limit < 50		ML	Silt
			CL	Lean Clay
			OL	Organic Silt and Clay (Low Plasticity)
	Silt and Clay Liquid Limit > 50		MH	Inorganic Silt
			CH	Inorganic Clay
			OH	Organic Clay and Silt (Med. to High Plasticity)
Highly Organic Soils			PT	Peat

Relative Density with Respect to SPT N-Value			
Coarse-Grained Soils		Fine-Grained Soils	
Density	N-Value (Blows/Ft)	Density	N-Value (Blows/Ft)
Very Loose	0 - 4	Very Soft	0 - 1
Loose	5 -10	Soft	2 - 4
Medium Dense	11 - 30	Medium Stiff	5 - 8
Dense	31 - 50	Stiff	9 - 15
Very Dense	> 50	Very Stiff	16 - 30
		Hard	> 30

Krazan & ASSOCIATES, INC.

Proposed Multi-Story Building - Lynnwood, WA		
Date: August 2021	References: USCS	
Drawn By: EC		Project Number: 092-21009

KRAZAN AND ASSOCIATES, INC.

LOG OF EXPLORATORY BORING B-1

PROJECT: Proposed Multi-Story Building
DATE: 8/19/21
PROJECT NO.: 092-21009
PAGE: 1 of 2
LOGGED BY: Eddie Canfield
SURFACE ELEVATION: --
CONTRACTOR: EDI
BORING TYPE: Hollow Stem Auger
SAMPLE METHOD: Split Spoon, SPT
LOCATION: Lynnwood, WA

DEPTH (ft)	USC	WATER LEVEL	MATERIAL DESCRIPTION	BLOW COUNTS (per 6")	N-VALUE (Last 12" of SPT)	SAMPLES	N-VALUE (GRAPH) 10 20 30 40	Natural Moisture Content (Percent) 10 20 30 40
			2" Thick Asphalt					
			Brown Silty Sand with Gravel (Moist)(Medium Dense)(Fill?)	9				
			Light Brown to Tan Sand with Trace Gravel (SM) (Moist)(Very Dense)(Glacial Till)	38 50(6)	50+			7.6
5			Gray Silty Sand with Gravel (SM) (Moist)(Very Dense)(Glacial Till)	41 50(2)	50+			6.8
				50(6)	50+			
10				50(6)	50+			
15				50(0)				

Water Level Initial: ▽ Final: ▼

Water Observations: Groundwater seepage was not observed.

Notes: --

KRAZAN AND ASSOCIATES, INC.

LOG OF EXPLORATORY BORING B-1

PROJECT: Proposed Multi-Story Building **DATE: 8/19/21**
PROJECT NO.: 092-21009 **PAGE: 2 of 2**
LOGGED BY: Eddie Canfield **SURFACE ELEVATION: --**
CONTRACTOR: EDI **BORING TYPE: Hollow Stem Auger**
SAMPLE METHOD: Split Spoon, SPT **LOCATION: Lynnwood, WA**

DEPTH (ft)	USC	WATER LEVEL	MATERIAL DESCRIPTION	BLOW COUNTS (per 6")	N-VALUE (Last 12" of SPT)	SAMPLES	N-VALUE (GRAPH) 10 20 30 40	Natural Moisture Content (Percent) 10 20 30 40
			Gray Silty Sand with Gravel (SM) (Moist)(Very Dense)(Glacial Till)	50(0)	50+			
20				50(6)	50+			
25				50(0)	50+			
				50(0)	50+			
			End of Exploratory Boring					
30								

Water Level Initial: ▽ Final: ▼

Water Observations: Groundwater seepage was not observed.

Notes: --

LOG OF EXPLORATORY BORING B-2

KRAZAN AND ASSOCIATES, INC.

PROJECT: Proposed Multi-Story Building
DATE: 8/19/2021
PROJECT NO.: 092-21009
PAGE: 1 of 1
LOGGED BY: Eddie Canfield
SURFACE ELEVATION: --
CONTRACTOR: EDI
BORING TYPE: Hollow Stem Auger
SAMPLE METHOD: Split Spoon, SPT
LOCATION: Lynnwood, WA

DEPTH (ft)	MATERIAL DESCRIPTION	BLOW COUNTS (per 6")	N-VALUE (Last 12" of SPT)
0	3" Thick Asphalt		
	Brown Silty Sand with Gravel (Moist to Wet)(Medium Dense)(Fill?)	7, 7, 5	12
5		9	26
	Brown Silty Sand with Gravel (SM) (Moist to Wet)(Medium Dense)(Weathered Soils)	11, 15	
	Gray Silty Sand with Gravel (SM) (Moist)(Very Dense)(Glacial Till)	33, 50(4)	50+
10		50(4)	50+
	End of Exploratory Boring		
15			

N-VALUE (GRAPH)

Natural Moisture Content (Percent)

10 20 30 40

Water Level Initial: ⊻ Final: ▼

Water Observations: Groundwater seepage was not observed.

Notes: --

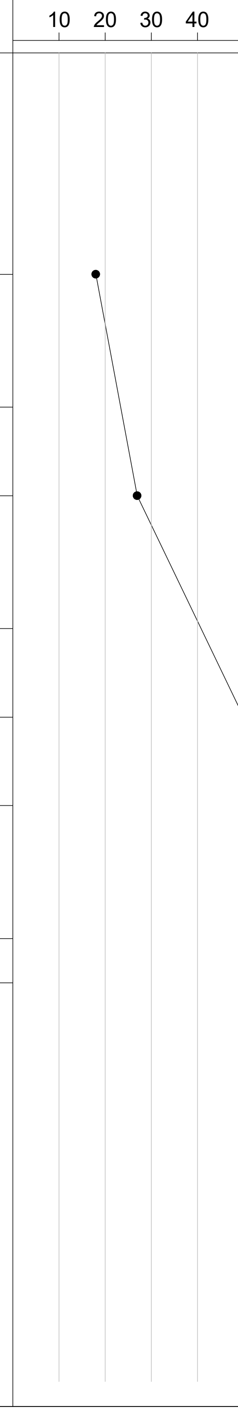
KRAZAN AND ASSOCIATES, INC.

LOG OF EXPLORATORY BORING B-3

PROJECT: Proposed Multi-Story Building
DATE: 8/19/2021
PROJECT NO.: 092-21009
PAGE: 1 of 1
LOGGED BY: Eddie Canfield
SURFACE ELEVATION: --
CONTRACTOR: EDI
BORING TYPE: Hollow Stem Auger
SAMPLE METHOD: Split Spoon, SPT
LOCATION: Lynnwood, WA

DEPTH (ft)	MATERIAL DESCRIPTION	BLOW COUNTS (per 6")	N-VALUE (Last 12" of SPT)
0	3" Thick Asphalt		
	Brown Silty Sand with Gravel (Moist to Wet)(Medium Dense)(Fill?)		
2.5		9, 9, 9	18
5		8, 10, 17	27
5.5	Brown Silty Sand with Gravel (SM) (Moist to Wet)(Medium Dense)(Weathered Soils)		
7	Gray Silty Sand with Gravel (SM) (Moist)(Very Dense)(Glacial Till)		
7.5		31, 50(6)	50+
10		50(6)	50+
11.5	End of Exploratory Boring		

N-VALUE (GRAPH)



Natural Moisture Content (Percent)

Water Level Initial: ⊽ Final: ▼

Water Observations: Groundwater seepage was observed at a depth of about 10.5 feet.

Notes:

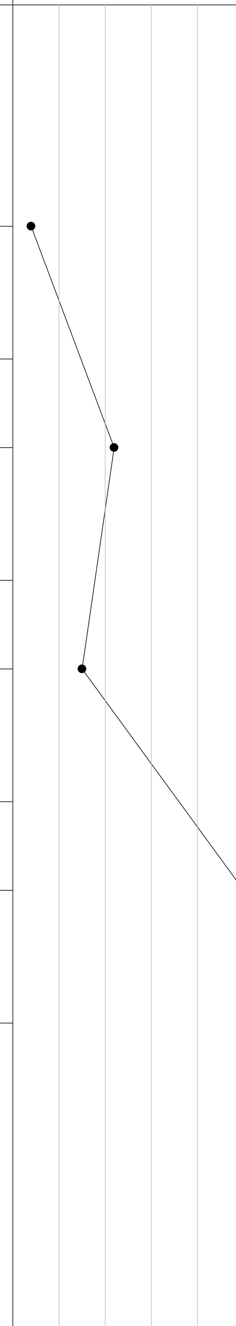
KRAZAN AND ASSOCIATES, INC.

LOG OF EXPLORATORY BORING B-4

PROJECT: Proposed Multi-Story Building
PROJECT NO.: 092-21009
LOGGED BY: Eddie Canfield
CONTRACTOR: EDI
SAMPLE METHOD: Split Spoon, SPT

DATE: 8/19/2021
PAGE: 1 of 1
SURFACE ELEVATION: --
BORING TYPE: Hollow Stem Auger
LOCATION: Lynnwood, WA

DEPTH (ft)	USC	WATER LEVEL	MATERIAL DESCRIPTION	BLOW COUNTS (per 6")	N-VALUE (Last 12" of SPT)	SAMPLES	N-VALUE (GRAPH) 10 20 30 40	Natural Moisture Content (Percent) 10 20 30 40
0			2" Thick Asphalt					
			Brown Silty Sand with Gravel (Moist)(Very Loose to Medium Dense)(Fill?)					
2.5				5, 2, 2	4			
5				5, 12, 10	22			
7.5				5	15			13.3
8			Gray to Brown Silty Sand with Trace Gravel (SM) (Moist)(Medium Dense)(Weathered Soils?)	5, 10				
10				18	50+			10.5
10.5			Gray Silty Sand with Gravel (SM) (Moist)(Very Dense)(Glacial Till)	32, 50(5)				
11.5			End of Exploratory Boring					
15								

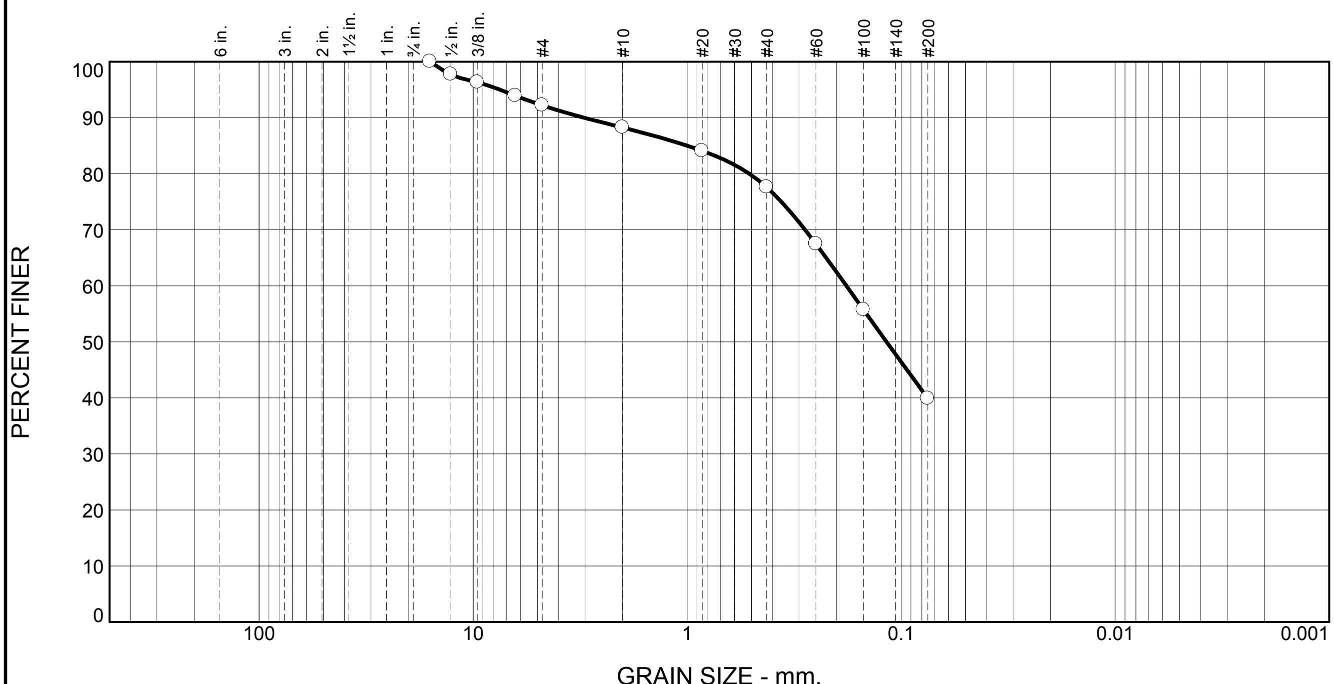


Water Level Initial: ▽ Final: ▼

Water Observations: Groundwater seepage was not observed.

Notes:

Krazan & Associates Sieve Analysis



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	7.8	4.0	10.6	37.7	39.9	

Test Results (ASTM C-136 & ASTM C-117)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
.625	100.0		
.5	97.7		
.375	96.3		
.25	93.9		
#4	92.2		
#10	88.2		
#20	84.1		
#40	77.6		
#60	67.4		
#100	55.7		
#200	39.9		

* (no specification provided)

Material Description

Light brown to tan silty sand with trace of gravel.

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= SM **AASHTO (M 145)=** A-4(0)

Coefficients

D_{90}= 3.0229 D_{85}= 0.9994 D_{60}= 0.1803
D_{50}= 0.1169 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Natural Moisture Content (ASTM D-2216): 7.6%

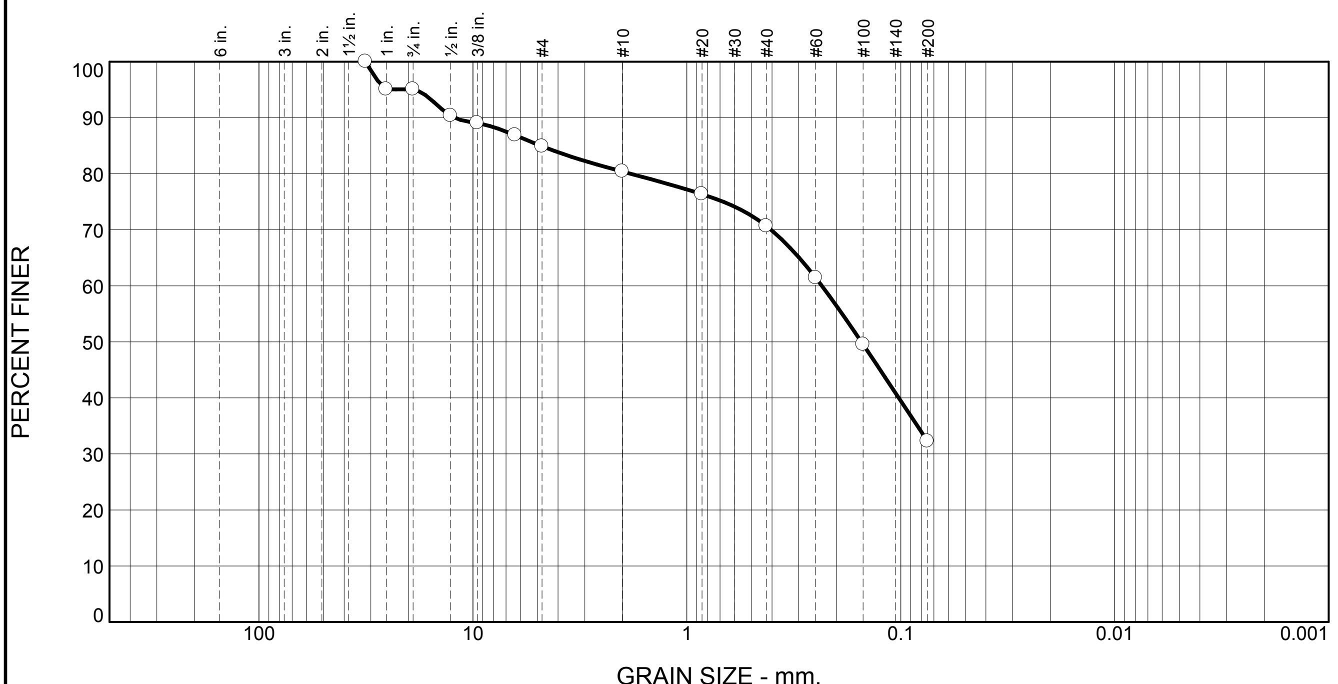
Date Received: 9/30/2021 **Date Tested:** 10/4/2021
Tested By: Cole Demas
Checked By: Corbett Mercer
Title: Lab Manager

Location: Boring 1
Sample Number: 76424-A **Depth:** 2.5'
Date Sampled: 9/30/2021

Krazan

Client: KOZ Development LLC
Project: Proposed Apartment Building
Project No: 09221009 **Figure**

Krazan & Associates Sieve Analysis



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	4.9	10.2	4.5	9.7	38.4	32.3	

Test Results (ASTM C-136 & ASTM C-117)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1.25	100.0		
1	95.1		
.75	95.1		
.5	90.4		
.375	89.0		
.25	86.9		
#4	84.9		
#10	80.4		
#20	76.4		
#40	70.7		
#60	61.4		
#100	49.5		
#200	32.3		

* (no specification provided)

Material Description

Gray silty sand with gravel.

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= SM **AASHTO (M 145)=** A-2-4(0)

Coefficients

D_{90}= 12.1963 D_{85}= 4.8252 D_{60}= 0.2339
D_{50}= 0.1530 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Natural Moisture Content (ASTM D-2216): 6.8%

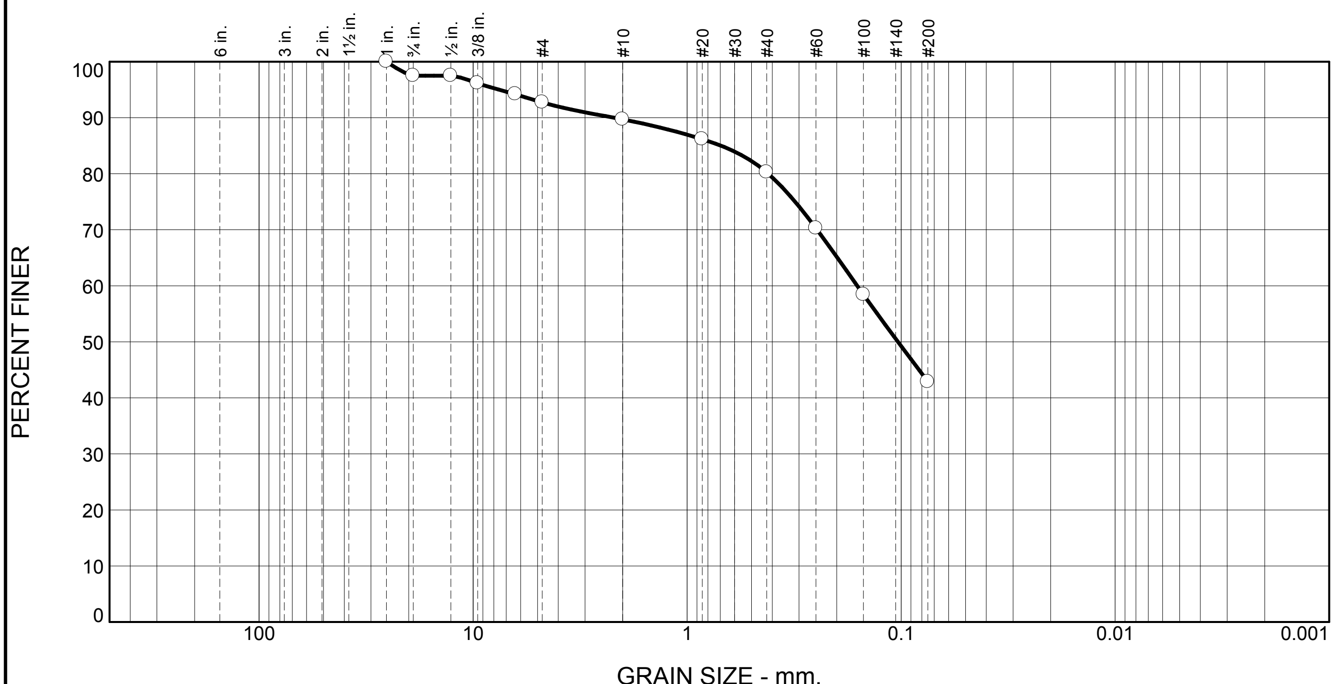
Date Received: 9/30/2021 **Date Tested:** 10/4/2021
Tested By: Cole Demas
Checked By: Corbett Mercer
Title: Lab Manager

Location: Boring 1
Sample Number: 76424-B **Depth:** 5'
Date Sampled: 9/30/2021

Krazan

Client: KOZ Development LLC
Project: Proposed Apartment Building
Project No: 09221009 **Figure**

Krazan & Associates Sieve Analysis



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	2.5	4.8	3.0	9.5	37.3	42.9	

Test Results (ASTM C-136 & ASTM C-117)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1	100.0		
.75	97.5		
.5	97.5		
.375	96.1		
.25	94.2		
#4	92.7		
#10	89.7		
#20	86.2		
#40	80.2		
#60	70.3		
#100	58.4		
#200	42.9		

* (no specification provided)

Material Description

Gray to brown silty sand with trace of gravel.

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= SM **AASHTO (M 145)=** A-4(0)

Coefficients

D_{90}= 2.1972 D_{85}= 0.6936 D_{60}= 0.1604
D_{50}= 0.1035 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Natural Moisture Content (ASTM D-2216): 13.3%

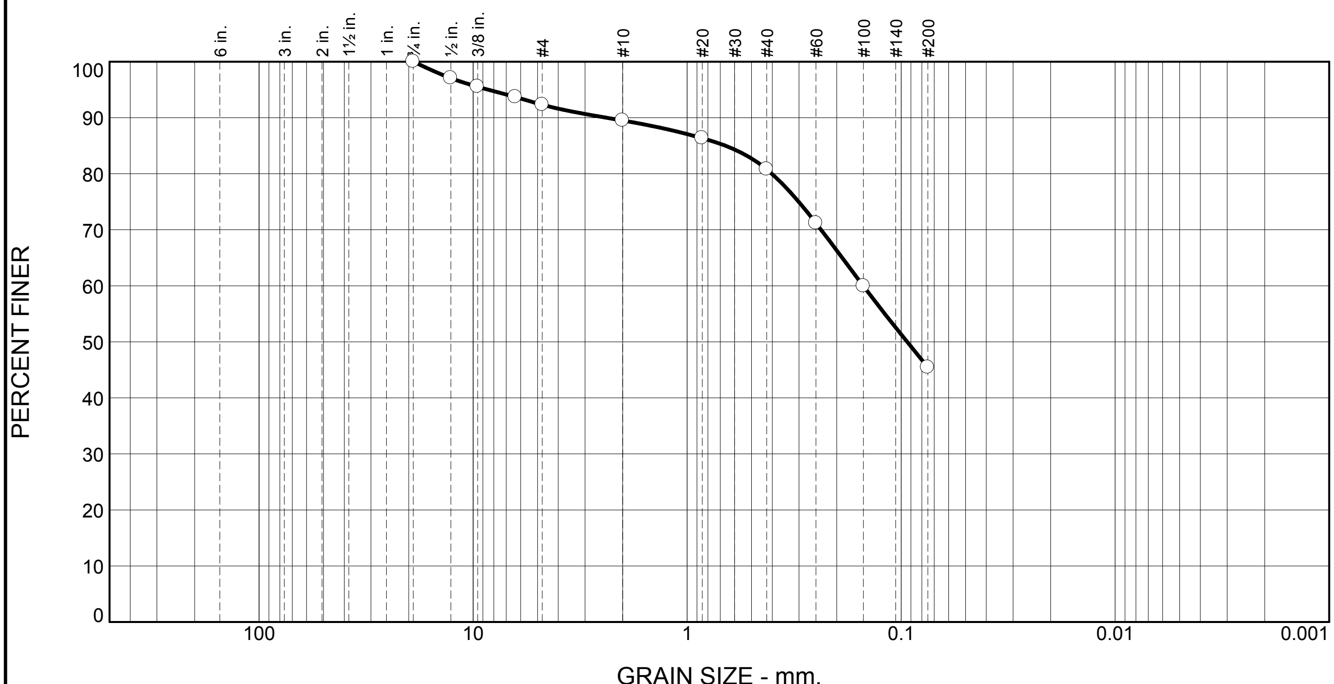
Date Received: 9/30/2021 **Date Tested:** 10/4/2021
Tested By: Cole Demas
Checked By: Corbett Mercer
Title: Lab Manager

Location: Boring 4
Sample Number: 76424-C **Depth:** 7.5' **Date Sampled:** 9/30/2021

Krazan

Client: KOZ Development LLC
Project: Proposed Apartment Building
Project No: 09221009 **Figure**

Krazan & Associates Sieve Analysis



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	7.7	2.8	8.7	35.4	45.4	

Test Results (ASTM C-136 & ASTM C-117)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
.75	100.0		
.5	97.1		
.375	95.5		
.25	93.7		
#4	92.3		
#10	89.5		
#20	86.4		
#40	80.8		
#60	71.2		
#100	59.9		
#200	45.4		

* (no specification provided)

Material Description

Brown silty sand with trace of gravel.

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= SM **AASHTO (M 145)=** A-4(0)

Coefficients

D_{90}= 2.3765 D_{85}= 0.6613 D_{60}= 0.1505
D_{50}= 0.0937 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Natural Moisture Content (ASTM D-2216): 10.5%

Date Received: 9/30/2021 **Date Tested:** 10/4/2021
Tested By: Cole Demas
Checked By: Corbett Mercer
Title: Lab Manager

Location: Boring 4
Sample Number: 76424-D **Depth:** 10' **Date Sampled:** 9/30/2021

Krazan

Client: KOZ Development LLC
Project: Proposed Apartment Building
Project No: 09221009 **Figure**

APPENDIX B

EARTHWORK SPECIFICATIONS

GENERAL

When the text of the report conflicts with the general specifications in this appendix, the recommendations in the report have precedence.

SCOPE OF WORK: These specifications and applicable plans pertain to and include all earthwork associated with the site rough grading, including but not limited to the furnishing of all labor, tools, and equipment necessary for site clearing and grubbing, stripping, preparation of foundation materials for receiving fill, excavation, processing, placement and compaction of fill and backfill materials to the lines and grades shown on the project grading plans, and disposal of excess materials.

PERFORMANCE: The Contractor shall be responsible for the satisfactory completion of all earthwork in accordance with the project plans and specifications. This work shall be inspected and tested by a representative of Krazan and Associates, Inc., hereinafter known as the Geotechnical Engineer and/or Testing Agency. Attainment of design grades when achieved shall be certified to by the project Civil Engineer. Both the Geotechnical Engineer and Civil Engineer are the Owner's representatives. If the contractor should fail to meet the technical or design requirements embodied in this document and on the applicable plans, he shall make the necessary readjustments until all work is deemed satisfactory as determined by both the Geotechnical Engineer and Civil Engineer. No deviation from these specifications shall be made except upon written approval of the Geotechnical Engineer, Civil Engineer or project Architect.

No earthwork shall be performed without the physical presence or approval of the Geotechnical Engineer. The Contractor shall notify the Geotechnical Engineer at least 2 working days prior to the commencement of any aspect of the site earthwork.

The Contractor agrees that he shall assume sole and complete responsibility for job site conditions during the course of construction of this project, including safety of all persons and property; that this requirement shall apply continuously and not be limited to normal working hours; and that the Contractor shall defend, indemnify and hold the Owner and the Engineers harmless from any and all liability, real or alleged, in connection with the performance of work on this project, except for liability arising from the sole negligence of the Owner of the Engineers.

TECHNICAL REQUIREMENTS: All compacted materials shall be densified to a density not less than 95 percent of maximum dry density as determined by ASTM Test Method D1557 as specified in the technical portion of the Geotechnical Engineering Report. The results of these tests and compliance with these specifications shall be the basis upon which satisfactory completion of work will be judged by the Geotechnical Engineer.

SOIL AND FOUNDATION CONDITIONS: The Contractor is presumed to have visited the site and to have familiarized himself with existing site conditions and the contents of the data presented in the soil report.

The Contractor shall make his own interpretation of the data contained in said report, and the Contractor shall not be relieved of liability under the contractor for any loss sustained as a result of any variance between conditions indicated by or deduced from said report and the actual conditions encountered during the progress of the work.

DUST CONTROL: The work includes dust control as required for the alleviation or prevention of any dust nuisance on or about the site or the borrow area, or off-site if caused by the Contractor's operation either during the performance of the earthwork or resulting from the conditions in which the Contractor leaves the site. The Contractor shall assume all liability, including Court costs of codefendants, for all claims related to dust or windblown materials attributable to his work.

SITE PREPARATION

Site preparation shall consist of site clearing and grabbing and preparations of foundation materials for receiving fill.

CLEARING AND GRUBBING: The Contractor shall accept the site in this present condition and shall demolish and/or remove from the area of designated project, earthwork all structures, both surface and subsurface, trees, brush, roots, debris, organic matter, and all other matter determined by the Geotechnical Engineer to be deleterious. Such materials shall become the property of the Contractor and shall be removed from the site.

Tree root systems in proposed building areas should be removed to a minimum depth of 3 feet and to such an extent which would permit removal of all roots larger than 1 inch. Tree root removed in parking areas may be limited to the upper 1½ feet of the ground surface. Backfill or tree root excavation should not be permitted until all exposed surfaces have been inspected and the Geotechnical Engineer is present for the proper control of backfill placement and compaction. Burning in areas, which are to receive fill materials, shall not be permitted.

SUBGRADE PREPARATION: Surfaces to receive Structural fill shall be prepared as outlined above, excavated/scarified to a depth of 12 inches, moisture-conditioned as necessary, and compacted to 95 percent compaction.

Loose and/or areas of disturbed soils shall be moisture conditioned and compacted to 95 percent compaction. All ruts, hummocks, or other uneven surface features shall be removed by surface grading prior to placement of any fill material. All areas which are to receive fill materials shall be approved by the Geotechnical Engineer prior to the placement of any of the fill material.

EXCAVATION: All excavation shall be accomplished to the tolerance normally defined by the Civil Engineer as shown on the project grading plans. All over excavation below the grades specified shall be backfilled at the Contractor's expense and shall be compacted in accordance with the applicable technical requirements.

FILL AND BACKFILL MATERIAL: No material shall be moved or compacted without the presence of the Geotechnical Engineer. Material from the required site excavation may be utilized for construction site fills provided prior approval is given by the Geotechnical Engineer. All materials utilized for constructing site fills shall be free from vegetable or other deleterious matter as determined by the Geotechnical Engineer.

PLACEMENT, SPREADING AND COMPACTION: The placement and spreading of approved fill materials and the processing and compaction of approved fill and native materials shall be the responsibility of the Contractor. However, compaction of fill materials by flooding, ponding, or jetting shall not be permitted unless specifically approved by local code, as well as the Geotechnical Engineer.

Both cut and fill shall be surface compacted to the satisfaction of the Geotechnical Engineer prior to final acceptance.

SEASONAL LIMITS: No fill material shall be placed, spread, or rolled while it is frozen or thawing or during unfavorable wet weather conditions. When the work is interrupted by heavy rains, fill operations shall not be resumed until the Geotechnical Engineer indicates that the moisture content and density of previously placed fill are as specified.

APPENDIX C

PAVEMENT SPECIFICATIONS

1. DEFINITIONS – The term “pavement” shall include asphalt concrete surfacing, untreated aggregate base, and aggregate subbase. The term “subgrade” is that portion of the area on which surfacing, base, or subbase is to be placed.

2. SCOPE OF WORK – This portion of the work shall include all labor, materials, tools and equipment necessary for and reasonable incidental to the completion of the pavement shown on the plans and as herein specified, except work specifically noted as “Work Not Included.”

3. PREPARATION OF THE SUBGRADE – The Contractor shall prepare the surface of the various subgrades receiving subsequent pavement courses to the lines, grades, and dimensions given on the plans and as per the pavement design section of this report. The upper 12 inches of the soil subgrade beneath the pavement section shall be compacted to a minimum compaction of 95% of maximum dry density as determined by test method ASTM D1557. The finished subgrades shall be tested and approved by the Geotechnical Engineer prior to the placement of additional pavement of additional pavement courses.

4. AGGREGATE BASE – The aggregate base shall be spread and compacted on the prepared subgrade in conformity with the lines, grades, and dimensions shown on the plans. The aggregate base should conform to WSDOT Standard Specification for Crushed Surfacing Base Course or Top Course (Item 9-03.9(3)). The base material shall be compacted to a minimum compaction of 95% as determined by ASTM D1557. Each layer of subbase shall be tested and approved by the Geotechnical Engineer prior to the placement of successive layers.

5. ASPHALTIC CONCRETE SURFACING – Asphaltic concrete surfacing shall consist of a mixture of mineral aggregate and paving grade asphalt, mixed at central mixing plant and spread and compacted on a prepared base in conformity with the lines, grades, and dimensions shown on the plans. The drying, proportioning, and mixing of the materials shall conform to WSDOT Specifications.

The prime coat, spreading and compacting equipment, and spreading and compacting the mixture shall conform to WSDOT Specifications, with the exception that no surface course shall be placed when the atmospheric temperature is below 50 degrees F. The surfacing shall be rolled with combination steel-wheel and pneumatic rollers, as described in WSDOT Specifications. The surface course shall be placed with an approved self-propelled mechanical spreading and finishing machine.

6. TACK COAT – The tack (mixing type asphaltic emulsion) shall conform to and be applied in accordance with the requirements of WSDOT Specifications.