May 15, 2020

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Attention: Richard Hutchinson, P.E., PTOE

Report of Geotechnical Engineering Services
Federal Way
47th and Dash Point Road Roundabout Project
47th Avenue SW and SW Dash Point Road
Federal Way, Washington
GeoDesign Project: DKS-13-01

INTRODUCTION

GeoDesign, Inc. is pleased to present our report of geotechnical engineering services to support the design of a roundabout located at the intersection of 47th Avenue SW and SW Dash Point Road in Federal Way, Washington. The project includes construction of a roundabout to reconfigure the existing three-way intersection (47th Avenue SW ends at SW Dash Point Road). The site is shown relative to surrounding features on Figure 1.

SCOPE OF SERVICES

The purpose of our geotechnical engineering evaluation was to explore subsurface conditions at the site and provide geotechnical engineering recommendations for design and construction. Our specific scope of services is summarized as follows:

- Reviewed preliminary design plans and available geotechnical and geologic information for the site or adjacent areas.
- Coordinated and managed the field exploration, including obtaining street-use right-of-way (ROW) and lane closure permits from the City of Federal Way and scheduling contractors and GeoDesign staff.
- Completed an on-site subsurface exploration program consisting of three borings to a depth of 16.5 feet below ground surface (BGS).

- Prepared this geotechnical engineering report that presents our findings, conclusions, and recommendations, including the following:
  - Subsurface soil and groundwater conditions
  - Structural foundation earth wall retaining wall design and appropriate soil parameters

SITE CONDITIONS

GEOLOGIC SETTING

Geologic maps of the area indicate that the site is underlain by glacial drift material consisting of outwash composed of fine to medium sand near the project location. Past grading activities may have impacted the surficial soil at the intersection. Based on the site reconnaissance and the subsurface explorations, we believe the subsurface conditions are generally consistent with the mapped geology.

SURFACE CONDITIONS

Surface conditions were evaluated during a field reconnaissance, which involved visiting the site to observe the existing conditions, observe the soil exposures in cut slopes, and through probing of the ground surface to depths up to approximately 18 inches.

The intersection is located in an upland area that slopes down to the north towards Dumas Bay. Both streets have a single travel lane in each direction with asphalt concrete (AC) shoulder areas that slope to shallow ditches or swales, and 47th Avenue SW ends at the intersection. Sidewalks and bike lanes are not present. 47th Avenue SW is generally level and SW Dash Point Road slopes downward from west to east.

We observed the cut slope that is present adjacent to the southwest corner of the intersection. The slope is sparsely vegetated and silty sand is exposed on the ground surface. Probing indicated dense soil conditions approximately 18 inches behind the face of the cut.

A shallow stormwater detention swale/pond is present adjacent to the southwest corner of the intersection. The pond is set back from the ROW several feet and vegetated with grass.

Shallow ditches or swales are present along the north and east sides of the intersection and vary in depth relative to the paved roadway surface up to approximately 4 feet. Geotechnical probing using a ½-inch-diameter steel probe rod indicated very dense soil conditions present at depths of 12 to 18 inches BGS.

SUBSURFACE CONDITIONS

The site was explored by drilling three borings to a depth of 16.5 feet BGS. The locations of the explorations are shown on Figure 2. Descriptions of our field explorations and laboratory testing programs and the exploration logs are presented in the Attachment.

Boring B-1 was performed on the shoulder of SW Dash Point Road, where 6 inches of AC overlying 16 inches of aggregate base was encountered. The remaining two explorations were performed outside of the road where aggregate base and native soil was encountered at the surface. A 3-inch-thick root zone was observed in boring B-2. Beneath the aggregate base in borings B-1 and B-3 and from the surface of B-2 native recessional outwash was encountered and extends to the maximum depths explored. The recessional outwash is composed of glacially consolidated fine to medium sand with varying amounts of gravel and silt. Based on SPT blow counts, the native soil varies in density between medium dense and dense and generally increase in density with increasing depth.

GROUNDWATER

Groundwater was not encountered in our explorations and was not observed in nearby wells installed at depths up to 100 feet BGS. Perched groundwater should be expected during periods of extended wet weather. Groundwater conditions at the site are expected to vary seasonally due to rainfall events and other factors not observed in our explorations.

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

Based on our review of available information, the development history of the site, and the results of our explorations and analyses, it is our opinion that the site is suitable for widening of the roadway.

- Widening on the north side of SW Dash Point Road and the east side of 47th Avenue SW will require constructing a fill embankment to support the road in areas where existing drainage ditches are present. Depending on the ROW space constraints, options for the edge of the fill are as follows:
    - Sloping at permanent slopes of 2H:1V
    - Retaining with a reinforced soil slope at inclinations up to 1H:1V
    - Retaining with a mechanically stabilized earth (MSE) wall using either flexible facing such as a vegetated flexbag system or using rigids elements such as large or small concrete masonry unit (CMU) blocks
    - Retaining with a gravity wall using large CMU blocks such as Redi-Rock or Ultrablock or, alternatively, gabion baskets
- Widening on the west side of 47th Avenue SW will include re-grading of the existing cut slope on the southwest side of the road to accommodate the new alignment. Potential options to accommodate the reconfigured cut slope are as follows:
    - Sloping at inclinations up to 1.5H:1V
    - Large-block CMU gravity walls such as Redi-Rock or Ultrablock wall types. These types of walls are applicable when sufficient space behind the face of the wall is available. Small-block CMUs are not appropriate for gravity wall situations in excess of 3.5 feet. The use of small CMU blocks, such as Allan Block or Keystone, would require construction of an MSE embankment that would require additional excavation behind the wall to accommodate the geogrid reinforcing.
    - Cantilever soldier pile wall if space behind the wall is limited and not enough space is available for constructing a gravity wall.
    - Once the proposed cuts along the west side of 47th Avenue SW are finalized, they should be reviewed to confirm that they will not impact the stormwater detention swale/pond located west of the intersection.

EMBANKMENT FILL CONSTRUCTION

Sidehill fill embankments will be used to widen the north side of SW Dash Point Road and the east side of 47th Avenue SW to support realignment of the road around the roundabout. Sidehill embankment fill construction should begin with stripping and grubbing to remove surficial organic material and should be completed in accordance with Washington State Department of Transportation Standard Specifications for Road, Bridge, and Municipal Construction – 2020 (WSS) 2-03.3(14)B – Earth Embankment Construction. The existing subgrade should be recompacted and proof rolled. If any soft or organic-rich areas are present, they should be over-excavated to firm bearing material and replaced with structural fill. Fill should be placed in horizontal lifts and keyed into the exposed soil in a stair step-like fashion. Compaction of the embankment fill should conform to WSS 2.03.3(14)C - Compacting Earth Embankments, Method C (95 percent of the maximum dry density as determined by ASTM D1557).

Embankment fill should consist of imported granular fill in accordance with WSS 9-03.14(1) – Gravel Borrow, with the exception that the percentage passing the U.S. Standard No. 200 sieve does not exceed 5 percent by dry weight.

Subgrade preparation prior to fill placement will likely require stabilization of loose material within the existing ditches. This soil should be stabilized through over-excavation to a depth of 1.5 feet and placement of an initial layer of stabilization material, which should be pushed and kneaded into the subgrade to establish a firm surface. A subgrade reinforcement geotextile should then be placed over the stabilization material and then backfilling with structural fill may begin.

EXCAVATION

Permanent Slopes

The existing slope on the west side of 47th Avenue SW will be re-graded to facilitate widening and realignment of the road. Soil exposed in the cut generally consists of weathered glacial drift material composed of sand and silty sand. We recommend permanent slope cuts in this area be graded to a maximum inclination of 2H:1V. Steeper inclinations up 1.25H:1V are possible; however, they will be susceptible to raveling and will not support establishment of vegetation. If seepage is encountered, it may be necessary to flatten the slopes and install mitigation measures to control runoff.

If the cut cannot be sloped within the available ROW, a retaining wall may be required to support the cut. We anticipate the cut slope will be greater than 6 feet in height; therefore, a rockery is not recommended.

Fill slopes may be created in other areas of the project alignment. We recommend the maximum slope of embankment fill slopes be constructed at 2H:1V. Newly constructed fill slopes should be over-built by at least 12 inches and then trimmed back to the required slope to maintain a firm face.

Based on the preliminary project plans, we do not anticipate any other areas where permanent slope cuts will be required.

RETAINING WALLS

General

The following recommendations should be used for design of gravity and cantilever soldier pile retaining structures that are used to achieve grade changes, including temporary shoring or shielding.

Our retaining wall design recommendations are based on the following assumptions: (1) the walls consist of conventional, cantilevered or embedded walls, (2) the walls are less than 10 feet in height, (3) the backfill is drained and consists of structural fill or retaining wall select backfill, as defined in this report, and (4) the backfill has a slope flatter than 2H:1V. Re-evaluation of our recommendations will be required if the retaining wall design criteria for the project varies from these assumptions.

Gravity walls can consist of cast-in-place concrete walls, large premanufactured CMU block units (such as Redi-Rock or Ultrablock), or gabions. The width of base block units for CMU block gravity walls varies from approximately 42 inches for walls that are 4 to approximately 7 feet in height and to approximately 60 inches for walls that are 7 to 12 feet in height.

Where gravity walls are less than 3.5 feet in height, small CMU blocks such as Allan Block or Keystone are also feasible. Small-block CMU walls over 4 feet in height are typically MSE walls where geogrid reinforcement is required in the backfill material placed behind the wall.

We recommend using gravity or cantilever soldier pile walls along the southwest corner of the intersection to reduce the potential for impacting the existing stormwater swale/pond at the southwest corner of the intersection. These types of walls will reduce the excavation required to complete the cut over a small-block MSE wall.

Retaining Wall Foundation

Adequate support for retaining walls will be provided by the existing subgrade, assuming it is prepared as recommended below. Based on the explorations, the on-site native soil to depths of approximately 1.5 feet BGS may be loose or become disturbed during stripping and grading activities and may require removal and replacement with stabilization material below wall foundations. The exposed foundation subgrade should be observed by the geotechnical engineer to verify conditions are consistent with the conditions described in this report and will provide adequate foundation support. After excavation the subgrade should be compacted to a firm, unyielding condition. If loose areas remain, they should be over-excavated to firm bearing material. The over-excavation should be backfilled with stabilization material.

Foundations supported on the improved subgrade should be designed for an allowable bearing pressure of 3,000 pounds per square foot (psf). This is a net bearing pressure; the weight of the footing and overlying backfill can be ignored in calculating footing sizes. The recommended allowable bearing pressure applies to the total of dead plus long-term live loads and may be increased by one-third to account for short-term loads, such as those resulting from wind or seismic forces.

Foundations for cast-in-place walls located in level ground areas should be founded at a depth of 18 inches below the adjacent grade. An exception to this is for walls sited near descending ground. If the ground descends at a slope of 2H:1V below a wall, a minimum embedment depth of 2 feet is required.

Based on our analysis, total post-construction static (consolidation-induced) settlement for conventional and semi-rigid foundation systems should be less than 1 inch, with differential settlement of up to ½ inch.

Retaining Wall Design Parameters

The magnitude of lateral earth pressures that develop against retaining walls depends on the degree to which the wall can yield laterally and other factors that include surcharge loads, groundwater and drainage conditions, slope of backfill in front of and behind the wall, method of backfill placement, degree of backfill compaction, and the type of backfill material.

If the wall can rotate or yield so the top of the wall moves an amount equal to or greater than approximately 0.001 times its height (a yielding wall), an “active” soil pressure condition will develop. If the wall is restrained against lateral movement or tilting (a non-yielding wall), an “at-rest” soil pressure condition will develop.

The retained soil will consist of dense, glacially consolidated material at cut slope locations and dense structural fill at fill embankments. We recommend yielding walls with level backfill under drained conditions be designed for an equivalent fluid density of 35 pounds per cubic foot (pcf) for active soil conditions. On the west side of 47th Avenue SW, where slopes in excess of 3H:1V extend above the wall, we recommend designing the wall using an equivalent fluid density of 55 pcf.

Design should include appropriate lateral pressures caused by surcharge loads located within a horizontal distance equal to the height of the wall (zone of influence). Traffic loads within the zone of influence should be designed with a uniformly distributed load equal to an additional 2 feet of fill, approximately 250 psf. For additional uniform surcharge pressures, a uniformly distributed lateral pressure equal to 35 percent of the vertical surcharge pressure should be added to the lateral soil pressures for yielding walls.

Resistance to lateral loads may be developed through base friction and through passive resistance on the embedded portion of the wall and foundation. Base friction resistance may be computed using a coefficient of friction of 0.4 applied to the dead load forces. Passive pressure may be computed using an equivalent fluid density of 300 pcf for a level ground surface and properly compacted backfill. If the ground surface in front of the wall is sloped, we should be contacted to provide a revised recommendation for passive resistance to account for the planned slope. The friction and equivalent fluid density values include a factor of safety of approximately 1.5. Adjacent sidewalks or the upper 12-inch depth of adjacent, unpaved areas should not be considered when calculating passive resistance.

Static lateral earth pressures acting on walls should also be increased to account for seismic loading conditions. We recommend a seismic load increment of 6 times the height of the wall

(6H in psf). This is based on a pseudo-static analysis using the Mononobe-Okabe equation for lateral earth pressure and one-half of the peak ground acceleration (PGA) value. A reduced PGA value is warranted if the PGA is only experienced for a few short durations during an earthquake and the ground motion is cyclical.

The height of the wall used in the above equation should be measured from the finished ground surface in front of the wall to the top of the wall. The seismic pressure should be applied as a uniform rectangular pressure from the top of the wall to the elevation of the finished ground surface in front of the wall, and the resultant should be applied at 0.6H of the exposed wall height.

These recommendations assume that adequate drainage will be provided behind below-grade walls and retaining structures, as discussed below.

Retaining Wall Drainage

We recommend the walls be provided with drainage to reduce the potential for hydrostatic water pressure buildup. Drainage can be achieved by using free-draining backfill material along the back side of the wall. Weep holes or perforated pipes can be used to collect and discharge groundwater.

Positive drainage should be provided behind retaining walls by placing a minimum 1-foot-wide zone of drain rock directly behind the wall. The free-draining backfill should meet the criteria for WSS 9-03.12(4) - Gravel Backfill for Drains. The free-draining backfill zone should extend from the base of the wall to within 1 foot of the finished ground surface. The top 1 foot of fill should consist of relatively impermeable soil to prevent infiltration of surface water into the wall drainage zone.

If weep holes are not preferred, a minimum 4-inch-diameter, perforated or slotted drainpipe should be installed within the free-draining material at the base of each wall. Drainpipe should consist of smooth-walled, perforated/slotted PVC pipe. The pipes should be placed with minimum slopes of 0.5 percent and routed to a suitable discharge location. The pipe installations should include a cleanout riser with cover located at the upper end of each pipe run. The cleanouts could be placed in flush-mount access boxes.

For walls where seepage at the face of a wall is not objectionable, the walls can be provided with weep holes to discharge water from the free-draining wall backfill material. The weep holes should be 3 inches in diameter and spaced approximately every 8 feet center-to-center along the base of the walls. The weep holes should be backed with galvanized, heavy-wire mesh to help prevent loss of the backfill material.

Retaining Wall Backfill

Backfill should be placed and compacted as recommended for embankment construction, with the exception of backfill placed immediately adjacent to walls. Backfill adjacent to walls should be compacted to a lesser standard to reduce the potential for generation of excessive pressure on the walls. Backfill located within a horizontal distance of 3 feet from the retaining walls should be compacted to approximately 90 percent of the maximum dry density, as determined by ASTM D1557. Backfill placed within 3 feet of the walls should be compacted in lifts less than 6 inches thick using hand-operated tamping equipment (such as a jumping jack or vibratory plate compactor). If flatwork (slabs, sidewalk, or pavement) will be placed adjacent to the walls, we recommend the upper 2 feet of fill be compacted to 95 percent of the maximum dry density, as determined by ASTM D1557.

Settlement of up to 1 percent of the wall height commonly occurs immediately adjacent to the wall as the wall rotates and develops active lateral earth pressures. Consequently, we recommend construction of flatwork adjacent to retaining walls be postponed at least four weeks after construction, unless survey data indicates settlement is complete prior to that time. A settlement monitoring program is not required or recommended.

RSS AND MSE WALLS

Reinforced soil slopes (RSS) and MSE retaining walls are often cost effective for supporting fill embankments where insufficient ROW is available for sloping, such as may be the case for embankments on the north side of SW Dash Point Road and the east side of 47th Avenue SW.

RSS and MSE walls consist of alternating layers of backfill soil and reinforcing material with interlocking facing elements. Commonly used reinforcing elements include steel grids and various geosynthetic products, such as geogrids and geotextiles. The vertical spacing of the reinforcing elements is typically on the order of 1 foot to 3 feet, depending on the height of the wall, reinforcing material, and other parameters. The length of the geogrid behind the wall is typically 70 to 100 percent of the wall height, depending on loading conditions and slope configurations above and below the wall. If geosynthetic products are selected, long-term creep characteristics should be taken into consideration in product selection.

Principal advantages of RSS and MSE walls include relatively low unit cost, tolerance of relatively large differential settlements, and a variety of facing options. The difference between the two systems is that MSE walls have a facing system that is structurally connected to the geogrid-reinforcing layers and can be constructed to near-vertical inclinations. RSSs without structural facing or a geogrid wrap are limited to approximately 1H:1V. Geogrid and turf reinforcement mats can be used to wrap the face of the fill and allows steeper slope inclinations.

MSE wall facing elements can consist of soft facing such as the FlexMSE system that uses geosynthetic bags that support vegetation growth or hard facing such as small-block (Allan Block or Keystone) or large-block (Redi Rock or Ultrablock) CMU.

Many MSE proprietary wall systems are available. Design of a reinforced slope or MSE wall system must be based on site-specific conditions and geotechnical parameters. The design procedures and wall details of several proprietary wall systems have been evaluated by the Washington State Department of Transportation (WSDOT), which has resulted in a pre-approved status for certain walls. An agreement between WSDOT and the proprietary wall manufacturer exists for pre-approved systems, which allows the proprietary wall manufacturer to competitively bid a project without having to provide a detailed wall design in the contract plans. Pre-approved proprietary wall systems with specific requirements and details are available in the Appendix of Chapter 15 of the WSDOT Geotechnical Design Manual (2015). WSDOT should be contacted for a current list of the pre-approved proprietary systems prior to choosing the system. If a non-pre-approved wall system is chosen, it will be necessary for the contractor's design engineer to completely design the wall.

We recommend proprietary wall system designs be reviewed by the geotechnical engineer to confirm that valid assumptions were made relative to material properties, site conditions, and other factors.

RSS and MSE Wall Design Parameters

We recommend the design parameters in Table 1 for use in design of RSS and MSE walls. The values shown below assume the backfill soil is compacted as recommended in the "Retaining Wall Backfill" section.

Table 1. Recommended Design Parameters for RSS and MSE Walls

Soil Properties	Reinforced Zone Soil	Retained Soil	Foundation Bearing Soil
	Gravel Borrow WSS 9-03.14(1)	Dense Structural Fill or Glacially Consolidated, Silty Sand	Dense, Silty Sand
Unit Weight (pcf)	135	135	135
Friction Angle (degrees)	36	36	36
Cohesion (psf)	0	0	0
Allowable Bearing Pressure (psf)	Not applicable	Not applicable	3,000

The RSS and MSE walls should be designed for seismic loading as discussed in the "Retaining Walls" section. MSE walls that are free to translate or move during a seismic event should be designed with a reduced coefficient of horizontal acceleration (kh) of approximately one-half of the PGA for the site. The vertical coefficient of acceleration (kv) shall be set to 0 for the analysis.

The minimum embedment depth of the MSE retaining walls will be a function of the height of the wall and the slope in front of the wall. We recommend the permanent cut slopes in front of and above the MSE wall be inclined no steeper than 2H:1V. Temporary cut slopes to install the MSE walls should be inclined no steeper than 1H:1V.

The minimum embedment depth for MSE walls founded on sloping ground should be provided as described in Table 2 but should not be less than 1 foot. In addition, the minimum embedment depth should be provided below a theoretical 4-foot-wide, horizontal bench that extends from the face of the wall and intersects the sloping ground in front of the wall.

Table 2. Minimum Embedment Depths for MSE Walls

Slope in Front of Wall	Minimum Embedment Depth (feet)
Horizontal	H/20 (1 foot minimum)
3H:1V	H/10
2H:1V	H/7

If the RSS and MSE walls will be subjected to the influence of surcharge loading (e.g., traffic loading) within a horizontal distance equal to the height of the wall, the walls should be designed for the additional horizontal pressure using an appropriate design method. A common practice is to assume a surcharge loading equivalent to 2 feet of additional fill to simulate traffic loading; we consider this method appropriate for typical situations. Where large surcharge loads, such as from heavy trucks, cranes, or other construction equipment, are anticipated near the retaining walls, the walls should also be designed to accommodate the additional lateral pressures resulting from these concentrated loads.

The foundation subgrade for the RSS and MSE walls should be prepared in accordance with the recommendations provided in the “Retaining Walls” section. We recommend the condition of all MSE wall foundation excavations be observed by GeoDesign to evaluate if the work is completed in accordance with our recommendations and the subsurface conditions are as expected. Recommendations for wall drainage are provided in the “MSE Wall Drainage” section.

If the foundation subgrade for the MSE walls is adequately prepared, we anticipate differential settlement along 100 linear feet of the MSE wall will be less than approximately 1 inch.

RSS and MSE walls should be designed with a factor of safety of 1.5 for sliding and pullout of reinforcing elements and a factor of safety of 2 for overturning. If proprietary wall systems are used, the wall supplier is responsible for evaluating these items. However, we recommend proprietary wall system designs be reviewed by a qualified geotechnical engineer to verify that valid assumptions were made relative to material properties and other factors.

MSE Wall Drainage

Positive drainage should be provided behind MSE retaining walls by placing a minimum 1-foot-wide zone of drain rock directly behind the reinforced fill zone to create a “chimney” drain. The free-draining backfill should meet the criteria for WSS 9-03.12(4) - Gravel Backfill for Drains. The free-draining backfill zone should extend from the base of the wall to within 1 foot of the finished ground surface. The top 1 foot of fill should consist of relatively impermeable soil to prevent infiltration of surface water into the wall drainage zone.

We recommend using either heavy-wall, solid pipe (SDR-35) or rigid, corrugated polyethylene pipe (ADS N-12 or equivalent) for the collector pipe. We recommend against using flexible tubing for wall drainpipe.

The pipes should be laid with a minimum slope of 0.5 percent and discharge into the stormwater collection system to convey the water to a suitable disposal location. The pipe installations should include cleanouts to allow for future maintenance.

CONSTRUCTION CONSIDERATIONS

FILL MATERIALS

Fill material may be required for site grading, backfilling over-excavations, pavement support, installation of utilities, and drainage. The Aggregate Source Approval certificates should not be used as acceptance that the material coming from a WSDOT-approved borrow pit will meet gradation or performance requirements. Confirmation sampling and testing should be performed on all proposed aggregate. The recommended fill materials are discussed below.

On-Site Soil

We anticipate on-site soil may be usable as structural fill during the dry season when moisture conditioning can be completed. On-site soil with deleterious materials such as organics should be disposed of off site.

Structural Fill

Imported granular material used for structural fill should be naturally occurring pit- or quarry-run rock, crushed rock, or crushed gravel and sand and should meet the specifications provided in WSS 9-03.14(1) – Gravel Borrow, with the exception that the percent passing the U.S. Standard No. 200 sieve does not exceed 5 percent by dry weight. The reduced percentage passing the No. 200 sieve results in a material less susceptible to deteriorating under wet weather conditions.

Hardscape/Pavement Base Course

Imported granular material used as aggregate base beneath hardscape areas should consist of 1¼-inch-minus material meeting the specifications provided in WSS 9-03.9(3) – Crushed Surfacing Base Course or Top Course, with the exception that the aggregate should have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve and at least two mechanically fractured faces. The imported granular material should be placed in lifts with a maximum uncompacted thickness of 12 inches and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

Trench Backfill

Backfill for utility trenches beneath improved areas should consist of structural fill, as defined above, and be compacted in accordance with the specifications for structural fill. Utility trenches beneath unimproved areas, such as landscaped areas, or areas where structural support is not necessary for surface improvements may be backfilled with common fill compacted to a minimum 90 percent of the maximum dry density, as determined by ASTM D1557.

Stabilization Material

Stabilization material to backfill over-excavations or to stabilize soft subgrade areas may consist of either:

- WSS 9-03.9(2) – Permeable Ballast, or
- WSS 9-13.7(2) - Backfill for Rock Wall

The initial lift of stabilization material used to fill over-excavations should be 18 inches thick and compacted to a firm condition. Successive lifts should be 12 inches thick and compacted to a dense, unyielding condition.

To prevent migration of the fine-grained subgrade soil upwards or structural fill, stabilization fabric should be placed between the stabilization material prior to placing structural fill. The geotextile should conform to the specifications for woven stabilization geotextile as defined in the "Geosynthetics" section.

GEOSYNTHETICS

If geotextiles are used on this project, the geotextiles should be installed in conformance with the specifications provided in WSS 2-12 – Construction Geosynthetic.

Stabilization Geotextile

To provide subgrade stabilization, reinforcement, and drainage, a geosynthetic is recommended in areas where soft subgrade conditions are encountered. This can be accomplished using a two-layer system composed of biaxial or triaxial geogrid and non-woven geotextile filter fabric or with the use of a single layer of heavy-duty geotextile with high permittivity characteristics such as Mirafi RS380i. The geotextile should conform to the specifications for woven soil stabilization material provided in WSS 9-33.2(1) – Geotextile Properties, Table 3 Geotextile for Separation or Soil Stabilization and meet the apparent opening size and water permittivity requirements in WSS 9-33.2(1) – Geotextile Properties, Table 5, Class A.

WET WEATHER CONSIDERATIONS

This section describes additional recommendations with potential budget and schedule impacts that may affect the owner and site contractor if earthwork occurs during the wet season. These recommendations are based on the site conditions and our experience on previous construction projects completed in the area.

- Soil encountered in the explorations is typically silty sand and sand with variable silt and gravel content. The material may be susceptible to deterioration during wet weather. If construction is completed or extends into the wet season, we recommend stabilizing the areas of the site where construction traffic is anticipated using a gravel working pad.
- Earthwork should be accomplished in small sections to minimize exposure to wet weather.
- Excavation or the removal of unsuitable soil should be followed promptly by the placement of storage aggregate.
- The size of construction equipment and access to the area should be limited to prevent soil disturbance.
- Increased handling, excavation, and disposal of wet and disturbed surface material should be expected.
- Protection of exposed soil subgrades and stockpiles will be required.
- Heavy rainfall can occur during winter months and can compromise earthwork schedules in this region.

- In general, snowfall is not dramatically high; however, frozen ground should not be proof rolled or compacted, and fill should not be placed over frozen ground.

OBSERVATION OF CONSTRUCTION

Recommendations provided in this report are based on subsurface information obtained from our explorations. We recommend retaining GeoDesign to review the geotechnical aspects of the plans and specifications for conformance with our recommendations and to observe earthwork geotechnical elements during construction, such as subgrades for foundations and hardscape areas, subsurface drainage elements, and embankment construction. Our services during construction complete the observational method by allowing us to confirm the conditions encountered during construction are consistent with those encountered in our subsurface explorations.

Satisfactory earthwork performance depends to a large degree on the quality of construction. Subsurface conditions and exposed subgrades should be observed during construction and compared with those encountered during the subsurface explorations. Recognition of changed conditions often requires and understanding of the design basis and experience; therefore, GeoDesign personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated and to verify that the work is completed in accordance with the construction drawings and specifications.

Observation and laboratory testing of the proposed fill material should be completed to verify that it is in conformance with our recommendations. Observation of the placement and compaction of the fill should be performed to verify it meets the required compaction and will be capable of providing structural support for the proposed infrastructure. A sufficient number of in-place density tests should be performed as the fill is placed to verify the required relative compaction is being achieved.

LIMITATIONS

We have prepared this report for use by DKS Associates and its consultants in design of this project. The data and report can be used for bidding or estimating purposes, but our report, conclusions, and interpretations should not be construed as warranty of the subsurface conditions and are not applicable to other nearby building sites.

Exploration observations indicate soil conditions only at specific locations and only to the depths penetrated. They do not necessarily reflect soil strata or water level variations that may exist between exploration locations. If subsurface conditions differing from those described are noted during the course of excavation and construction, re-evaluation will be necessary.

The site development plans and design details were preliminary at the time this report was prepared. If design changes are made, we request that we be retained to review our conclusions and recommendations and to provide a written modification or verification.

The scope of our services does not include services related to construction safety precautions and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in this report for consideration in design.

Within the limitations of scope, schedule, and budget, our services have been executed in accordance with generally accepted practices in this area at the time this report was prepared. No warranty, express or implied, should be understood.

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We appreciate the opportunity to be of continued service to you. Please call if you have questions concerning this report or if we can provide additional services.

Sincerely,

GeoDesign, Inc.

Kevin J. Lamb

Kevin J. Lamb, P.E.
Principal Engineer

KEVIN J. LAMB
STATE OF WASHINGTON
29441
REGISTERED PROFESSIONAL ENGINEER

Signed 05/15/2020

cc: Jerry Liu, DKS Associates (via email only)

TAP:KJL:kt
Attachments
One copy submitted (via email only)
Document ID: DKS-13-01-051520-geolr.docx

FIGURES

VICINITY MAP BASED ON AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH PRO®

GEODESIGN INC AN NV5 COMPANY	DKS-13-01	**VICINITY MAP**	
	MAY 2020	47TH AND DASH POINT ROAD ROUNDABOUT FEDERAL WAY, WA	**FIGURE 1**

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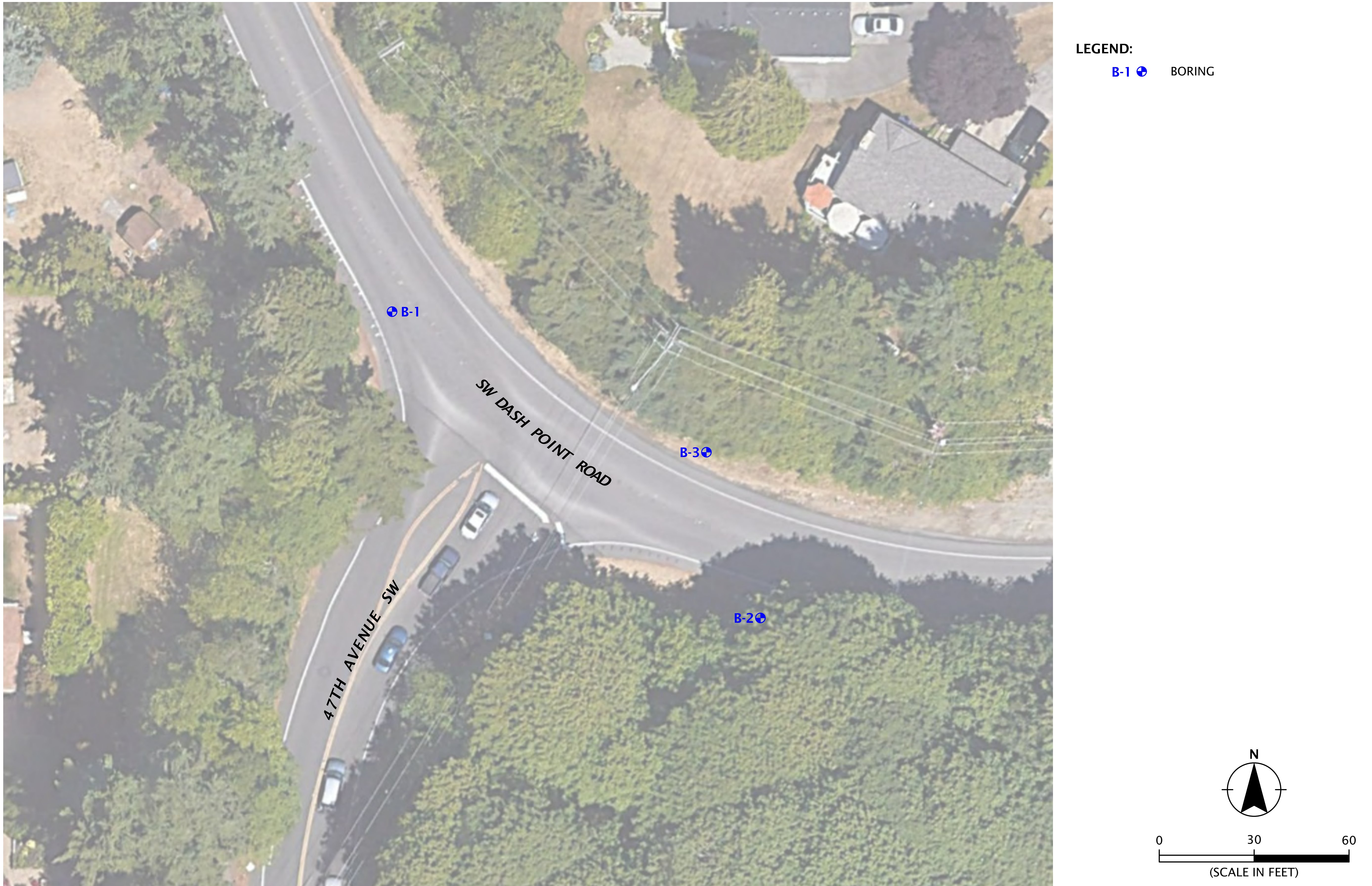
File Name: J:\A-D\DKS\DKS-13\DKS-13-01\Figures\CAD\DKS-13-01-VM01.dwg | Layout: FIGURE 1



SITE PLAN BASED ON AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH PRO®, MAY 4, 2020

FIGURE 2

SITE PLAN

47TH AND DASH POINT ROAD ROUNDABOUT
FEDERAL WAY, WA

DKS-13-01

MAY 2020

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ATTACHMENT

ATTACHMENT

FIELD EXPLORATIONS

GENERAL

We explored subsurface conditions by drilling three borings (B-1 through B-3) to a depth of 16.5 feet BGS on April 28, 2020. Drilling services were provided by Boretec 1, Inc. of Bellevue, Washington, using a trailer-mounted drill rig with hollow-stem auger techniques. The locations of the explorations are shown on Figure 2. The locations of the explorations were based on pacing from stationary objects and should be considered accurate to the degree in which they were measured. The exploration logs are presented in this attachment.

SOIL SAMPLING

We collected representative samples of the various soils encountered during drilling. Samples were collected from the borings using 1½-inch-inside diameter split-spoon sampler (SPT) in general accordance with ASTM D1586. The sampler was driven into the soil with a 140-pound hammer free-falling 30 inches. The sampler was driven a total distance of 18 inches. The number of blows required to drive the sampler the final 12 inches is recorded on the exploration logs, unless otherwise noted. We collected representative grab samples of the soil from the auger cuttings. Sampling methods and intervals are shown on the exploration logs.

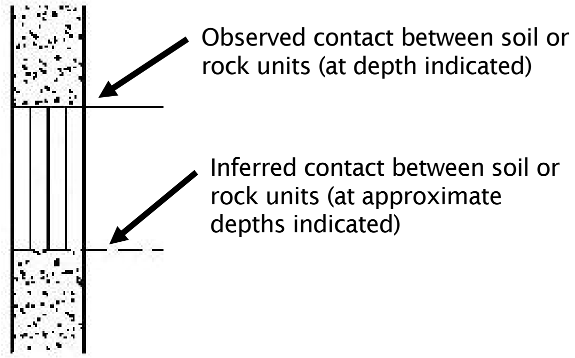
The SPT blow counts were conducted using two wraps around the cathead.

SOIL CLASSIFICATION

The soil samples were classified in accordance with the "Exploration Key" (Table A-1), "Soil Classification System" (Table A-2), which are presented in this appendix. The exploration logs indicate the depths at which the soil or their characteristics change, although the change could be gradual. If the change occurred between sample locations, the depth was interpreted. Classifications are shown on the exploration logs.

SYMBOL	SAMPLING DESCRIPTION
▯	Location of sample collected in general accordance with ASTM D1586 using Standard Penetration Test with recovery
▮	Location of sample collected using thin-wall Shelby tube or Geoprobe® sampler in general accordance with ASTM D1587 with recovery
▤	Location of sample collected using Dames & Moore sampler and 300-pound hammer or pushed with recovery
▤	Location of sample collected using Dames & Moore sampler and 140-pound hammer or pushed with recovery
⊠	Location of sample collected using 3-inch-O.D. California split-spoon sampler and 140-pound hammer with recovery
⊠	Location of grab sample
▒	Rock coring interval
▽	Water level during drilling
▼	Water level taken on date shown

Graphic Log of Soil and Rock Types



GEOTECHNICAL TESTING EXPLANATIONS

ATT	Atterberg Limits	P	Pushed Sample
CBR	California Bearing Ratio	PP	Pocket Penetrometer
CON	Consolidation	P200	Percent Passing U.S. Standard No. 200 Sieve
DD	Dry Density		
DS	Direct Shear	RES	Resilient Modulus
HYD	Hydrometer Gradation	SIEV	Sieve Gradation
MC	Moisture Content	TOR	Torvane
MD	Moisture-Density Relationship	UC	Unconfined Compressive Strength
NP	Non-Plastic	VS	Vane Shear
OC	Organic Content	kPa	Kilopascal

ENVIRONMENTAL TESTING EXPLANATIONS

CA	Sample Submitted for Chemical Analysis	ND	Not Detected
P	Pushed Sample	NS	No Visible Sheen
PID	Photoionization Detector Headspace Analysis	SS	Slight Sheen
		MS	Moderate Sheen
ppm	Parts per Million	HS	Heavy Sheen

GEODESIGN INC AN NV5 COMPANY	EXPLORATION KEY	TABLE A-1

RELATIVE DENSITY - COARSE-GRAINED SOIL

Relative Density	Standard Penetration Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)
Very Loose	0 – 4	0 – 11	0 – 4
Loose	4 – 10	11 – 26	4 – 10
Medium Dense	10 – 30	26 – 74	10 – 30
Dense	30 – 50	74 – 120	30 – 47
Very Dense	More than 50	More than 120	More than 47

CONSISTENCY - FINE-GRAINED SOIL

Consistency	Standard Penetration Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)	Unconfined Compressive Strength (tsf)
Very Soft	Less than 2	Less than 3	Less than 2	Less than 0.25
Soft	2 – 4	3 – 6	2 – 5	0.25 – 0.50
Medium Stiff	4 – 8	6 – 12	5 – 9	0.50 – 1.0
Stiff	8 – 15	12 – 25	9 – 19	1.0 – 2.0
Very Stiff	15 – 30	25 – 65	19 – 31	2.0 – 4.0
Hard	More than 30	More than 65	More than 31	More than 4.0

PRIMARY SOIL DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE-GRAINED SOIL (more than 50% retained on No. 200 sieve)	GRAVEL (more than 50% of coarse fraction retained on No. 4 sieve)	CLEAN GRAVEL (< 5% fines)	GW or GP	GRAVEL
		GRAVEL WITH FINES (≥ 5% and ≤ 12% fines)	GW-GM or GP-GM	GRAVEL with silt
			GW-GC or GP-GC	GRAVEL with clay
		GRAVEL WITH FINES (> 12% fines)	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND (50% or more of coarse fraction passing No. 4 sieve)	CLEAN SAND (<5% fines)	SW or SP	SAND
		SAND WITH FINES (≥ 5% and ≤ 12% fines)	SW-SM or SP-SM	SAND with silt
			SW-SC or SP-SC	SAND with clay
		SAND WITH FINES (> 12% fines)	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOIL (50% or more passing No. 200 sieve)	SILT AND CLAY	Liquid limit less than 50	ML	SILT
			CL	CLAY
			CL-ML	silty CLAY
			OL	ORGANIC SILT or ORGANIC CLAY
		Liquid limit 50 or greater	MH	SILT
			CH	CLAY
			OH	ORGANIC SILT or ORGANIC CLAY
HIGHLY ORGANIC SOIL			PT	PEAT

MOISTURE CLASSIFICATION

Term	Field Test
dry	very low moisture, dry to touch
moist	damp, without visible moisture
wet	visible free water, usually saturated

ADDITIONAL CONSTITUENTS

Secondary granular components or other materials such as organics, man-made debris, etc.

Percent	Silt and Clay In:		Percent	Sand and Gravel In:	
	Fine-Grained Soil	Coarse-Grained Soil		Fine-Grained Soil	Coarse-Grained Soil
< 5	trace	trace	< 5	trace	trace
5 – 12	minor	with	5 – 15	minor	minor
> 12	some	silty/clayey	15 – 30	with	with
			> 30	sandy/gravelly	Indicate %

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SOIL CLASSIFICATION SYSTEM

TABLE A-2

▲ BLOW COUNT
● MOISTURE CONTENT %
▥ RQD% ▨ CORE REC%

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	SAMPLE	BLOW COUNT (0–50–100)	INSTALLATION AND COMMENTS
0.0	ASPHALT CONCRETE (6.0 inches).				
	AGGREGATE BASE (1-1/4-inch minus crushed rock; 16.0 inches).	0.5	☒		
2.5	Medium dense, gray-brown SAND with gravel (SP); dry - **RECESSIONAL OUTWASH**.	2.0	■	11	
5.0 7.5	Medium dense, brown, silty SAND (SM), trace gravel; moist, sand is fine - **RECESSIONAL OUTWASH**.	4.0	■ ■	20 24	
10.0 12.5 15.0	Dense, gray-brown SAND (SP), trace silt and gravel; moist, sand is fine to medium - **RECESSIONAL OUTWASH**.	9.5	■ ■	35 33	Smooth drilling at 15.0 feet.
17.5 20.0	Exploration completed at a depth of 16.5 feet.	16.5			Surface elevation was not measured at the time of exploration.

DRILLED BY: Boretec1 LOGGED BY: A. Hodgins COMPLETED: 04/28/20

BORING METHOD: hollow-stem auger (see document text) BORING BIT DIAMETER: 4 1/2 inches

GEODESIGN INC
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DKS-13-01

MAY 2020

BORING B-1

47TH AND DASH POINT ROAD ROUNDABOUT
FEDERAL WAY, WA

FIGURE A-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % ▥ RQD% ▨ CORE REC% (0 – 50 – 100)	INSTALLATION AND COMMENTS
0.0	Medium dense, brown, silty SAND with gravel (SM), trace organics; moist, sand is fine to coarse (3-inch-thick root zone) - **RECESSIONAL OUTWASH**.			☒		
2.5					19	
5.0	trace gravel, without organics; sand is fine to medium at 5.0 feet				15	
7.5	Medium dense, brown SAND with silt (SP-SM), trace gravel; moist, sand is fine to medium - **RECESSIONAL OUTWASH**.	7.0			29	
10.0	Dense, brown SAND (SP), trace silt; moist, sand is fine to medium - **RECESSIONAL OUTWASH**.	9.5			35	
12.5						
15.0	medium dense at 15.0 feet				27	Smooth drilling at 15.0 feet.
17.5	Exploration completed at a depth of 16.5 feet.	16.5				Surface elevation was not measured at the time of exploration.
20.0						

DRILLED BY: Boretec1

LOGGED BY: A. Hodgins

COMPLETED: 04/28/20

BORING METHOD: hollow-stem auger (see document text)

BORING BIT DIAMETER: 4 1/2 inches

GEODESIGN INC
AN NV5 COMPANY

DKS-13-01

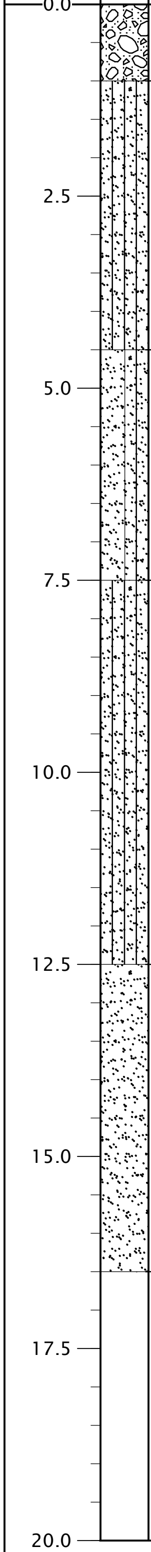
MAY 2020

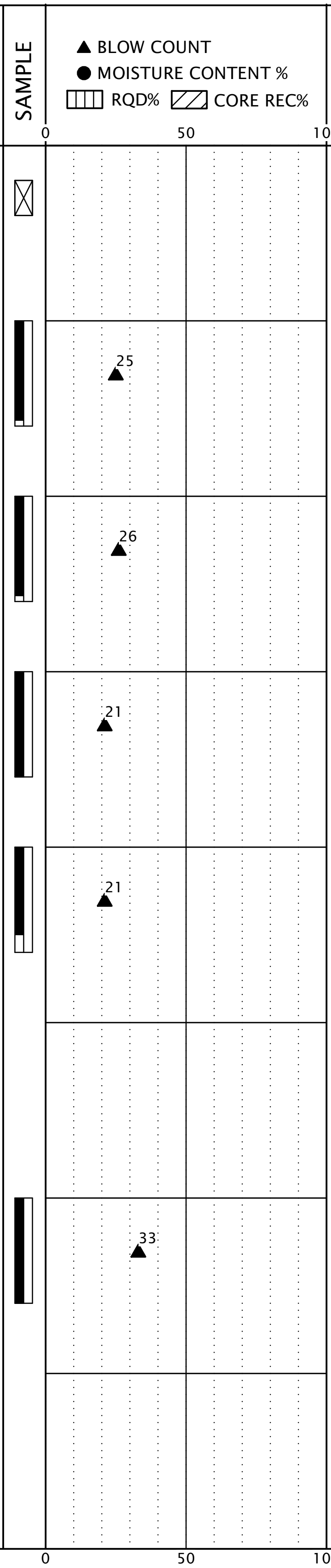
BORING B-2

47TH AND DASH POINT ROAD ROUNDABOUT
FEDERAL WAY, WA

FIGURE A-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	AGGREGATE BASE (1-1/4-inch minus crushed rock; 12.0 inches).			
	Medium dense, brown, silty SAND with gravel (SM), trace organics; moist, sand is fine to medium - **RECESSIONAL OUTWASH**.	1.0		
	Medium dense, brown SAND with silt (SP-SM), trace gravel; moist, sand is fine to medium - **RECESSIONAL OUTWASH**.	4.5		
	Medium dense, brown, silty SAND (SM); moist, sand is fine - **RECESSIONAL OUTWASH**.	7.5		
	Dense, brown with orange mottled SAND (SP), trace silt and gravel; moist, sand is fine to medium - **RECESSIONAL OUTWASH**.	12.5		
	Exploration completed at a depth of 16.5 feet.	16.5		Surface elevation was not measured at the time of exploration.
20.0				





DRILLED BY: Boretec1

LOGGED BY: A. Hodgins

COMPLETED: 04/28/20

BORING METHOD: hollow-stem auger (see document text)

BORING BIT DIAMETER: 4 1/2 inches

GEODESIGN INC
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DKS-13-01

MAY 2020

BORING B-3

47TH AND DASH POINT ROAD ROUNDABOUT
FEDERAL WAY, WA

FIGURE A-3