

GEOTECHNICAL REPORT



741 Marine Drive
Bellingham, WA 98225
20611-6777 Avenue NE
Arlington, WA 98223

360 733 7318
888 251 5276

360 733 7318

July 6, 2018
Project No. 18-0458

Mr. Goutam Jain
4114 198th Street SW, Suite 2
Lynnwood, WA 98036

c/o: Mr. Kamal Singh

Re: Geotechnical Engineering Report
Proposed Mixed-Use Development
6804 204th Street NE
Arlington, WA 98223

Dear Mr. Singh:

As requested, GeoTest Services, Inc. (GTS) is pleased to submit this report summarizing the results of our geotechnical evaluation for the proposed mixed-use development to be constructed at the above referenced address in Arlington, Washington (see *Vicinity Map*, Figure 1). This report has been prepared in general accordance with the terms and conditions established in our services agreement dated June 7, 2018, and authorized by Mr. Kamal Singh.

PURPOSE AND SCOPE OF SERVICES

The purpose of this investigation is to evaluate general subsurface conditions beneath the site from which conclusions and recommendations pertaining to project design can be formulated. Specifically, our scope of services includes the following tasks:

- Explore soil and groundwater conditions underlying the site by advancing a total of 6 exploration test pits with a tracked excavator to evaluate subsurface conditions.
- Perform laboratory testing on representative samples in order to classify and evaluate the engineering characteristics of the soils encountered. In addition, estimate long-term infiltration rates and determine stormwater treatment potential.
- Provide a report containing a description of subsurface conditions, exploration logs, and our findings and recommendations pertaining to site preparation and earthwork. This report also contains information about fill and compaction, wet weather earthwork, seismic design, foundation recommendations, concrete slab-on-grade construction, foundation and site drainage, utilities, temporary and permanent slopes, pavement sections for driveway and parking areas, and construction monitoring.

PROJECT DESCRIPTION

At the time of this report, design concepts have not been finalized. The proposed development on the subject property will likely consist of a mix of retail space and residential units, similar to nearby mixed-use developments. A gas station and small convenience store may also be incorporated into the proposed development. GTS anticipates that new construction will consist

of single or multi-story structures utilizing wood-framing, shallow conventional footings, and slab-on-grade floors. Structural loads are anticipated to be relatively light. GTS anticipates the construction of new drive paths and parking areas throughout the entirety of the development. GTS also anticipates that stormwater infiltration facilities may be constructed throughout the development.

SITE CONDITIONS

This section presents a description of the general surface and subsurface conditions observed at the project site during the time of our field investigation. Interpretations of the site conditions are based on the results and review of available information, site reconnaissance, subsurface explorations, laboratory testing, and previous experience in the project vicinity.

Surface Conditions

The subject property is generally rectangular, level, and occupies approximately 4.4 acres. The site is bordered to the north by 204th Street NE, to the east by 67th Avenue NE, to the east by railroad tracks, and to the south by two industrial buildings. The property occupies approximately 550 feet of frontage along 204th Street NE and 200 feet along 67th Avenue Northeast. Vegetation on the property consists of grass, scattered deciduous and coniferous trees, and various bushes. No surface water was observed at the time of our visit.

A review of historical imagery from Google Earth shows that a single family residence previously occupied the central portion of the site.



Photo 1 – Western half of subject property, looking northwest. Taken on June 21, 2018.

Subsurface Soil Conditions

Subsurface conditions were explored by advancing 6 exploration test pits (TP-1 through TP-6) on June 21, 2018. The explorations were advanced using a subcontracted excavator to depths of 10 to 15 feet below ground surface (BGS). Grab soil samples were obtained from the side wall of the test pit excavations. All explorations were continuously observed by a staff geologist from our firm. See the attached *Site and Exploration Map* (Figure 2) for the approximate locations of our test pit explorations.



Photo 2 – Typical subsurface profile as seen in test pit TP-1. Top dark brown layer is topsoil. Light brown layer is weathered native gravel. Bottommost grey layer is unweathered native sandy gravel.

The subsurface profile generally consisted of 0.5 to 1 feet of loose, dark brown, gravelly, silty, sand (topsoil), overlying native, medium dense, light brown, oxidized, silty, sandy, gravel (weathered Arlington Gravel). At approximately 1.5 to 2.5 feet BGS, the weathered soils graded to a medium dense, gray, very sandy gravel (unweathered Arlington Gravel). The unweathered gravel was stratified, displaying alternating beds of very sandy gravel, and very gravelly sand. At depths of 6 to 9 feet BGS, a medium dense, poorly graded sand, with a minimal gravel content was encountered. This was interpreted to be more representative of native, Marysville Sand deposits.

TP-5 slightly deviated from the above sequence in that 2.5 feet of sandy gravel with scattered debris and organics overlaid the native Arlington Gravel instead of the topsoil and weathered

deposits. We interpret this to be fill soils that were used to fill in the footprint of the single family residence that previously occupied this portion of the site.

General Geologic Conditions

Geologic information for the project site was obtained from the *Geologic Map of the Arlington West 7.5-minute Quadrangle, Snohomish County, WA* (Minard, 1985) published by the United States Geological Survey. According to the Minard map, subsurface soils in the vicinity of the project consist of the Vashon Stade Recessional Outwash – Arlington Gravel Member (Qvra), and the Marysville Sand Member (Qvrm).

The Arlington Gravel and Marysville Sand each consist primarily of well drained stratified outwash sand and gravel deposited by meltwater from the stagnating and receding Vashon glacier. The members differ in their relative gravel and sand contents, by which they are differentiated in the Arlington area.

Soils encountered during our subsurface explorations were found to be generally consistent with the mapped Recessional Outwash – Arlington Gravel and Marysville Sand deposits. In this report, these native soils will be referred to as "Arlington Gravel" and "Marysville Sand".

Groundwater

At the time of the GTS visit on June 21, 2018, groundwater was not encountered in any of our explorations. A review of publicly available well data from the Washington Department of Ecology Well Log Viewer suggests that groundwater has historically been encountered at depths of about 17 to 20 feet BGS in the site vicinity.

The groundwater conditions described in this report are for the specific locations and dates indicated, and therefore may not necessarily be indicative of other locations and/or times. Please consider that groundwater table levels are generally higher (at shallower depths) during the wetter months (October through May).

Liquefaction Hazard Potential

Based on a review of information obtained from the Washington State Department of Natural Resource Geologic Information Portal, the subject site is classified as having a low to moderate liquefaction susceptibility. However, this map only provides an estimate of the likelihood that soil will liquefy as a result of an earthquake and is meant as a general guide to delineate areas prone to liquefaction.

Liquefaction is defined as a significant rise in pore water pressure within a soil mass caused by earthquake-induced cyclic shaking. The shear strength of liquefiable soil is reduced during large and/or long duration earthquakes as the soil consistency approaches that of semi-solid slurry. Liquefaction can result in significant and widespread structural damage if not properly mitigated. Deposits of loose, granular soil below the groundwater table are most susceptible to liquefaction. Damage caused by foundation rotation, lateral spreading, and other ground movements can result from soil liquefaction.

Based on our subsurface explorations, the site is underlain by native medium dense, very sandy, gravel soils. GTS did not encounter the groundwater table during our explorations and a review of local well log data suggests it is typically encountered at approximate depths of 17 to

20 feet BGS in the site vicinity. Based on the presence of medium-dense, sandy gravel and a low groundwater table, it is GTS's opinion that the potential for liquefaction underlying the subject property is low. Thus, it is also our opinion that no additional mitigations are required for the proposed development.

CONCLUSIONS AND RECOMMENDATIONS

It is GTS's opinion that subsurface conditions at the site are suitable for the construction of the proposed building provided that the recommendations contained in the geotechnical engineering report are incorporated into the project design.

The test pits generally exposed similar subsurface soil conditions. Native, medium dense, sandy, gravel (Arlington Gravel) was located within approximately 1.5 to 2.5 feet of existing site grades. GTS recommends that the topsoil and near-surface, loose fill soils (where encountered) be removed from the building footprints down to the native, medium-dense, gravelly sand. It is highly possible that loose fill soils may exist under the previous house footprint. Loose fill soils should be removed down to competent native soils.

Once competent native soils have been exposed, GTS recommends that the subgrade surface be compacted to a firm and unyielding condition with an appropriate piece of construction compaction equipment. The foundations can bear directly on the prepared native subgrade or on compacted structural fill placed atop these soils. Further recommendations regarding the placement and compaction of structural fill can be found in the *Fill and Compaction* section of this report.

Based on the native soils encountered in the test pits, it appears that the subject site is suitable for stormwater infiltration. We have presented preliminary design infiltration rates based on grain size analyses, per the *2012 Stormwater Management Manual for Western Washington* (amended December 2014), in the *Stormwater Infiltration Potential* section of this report.

Site Preparation and Earthwork

The portions of the site to be occupied by proposed foundations and floor slabs should be prepared by removing any existing topsoil, deleterious material and significant accumulations of organics from the area to be developed. Prior to the placement of any foundation elements or structural fill, the exposed subgrade under all areas to be occupied by soil-supported floor slabs and spread or continuous foundations should be recompacted to a firm and unyielding condition and proof rolled with a loaded dump truck, large self-propelled vibrating roller, or similar piece of equipment applicable to the size of the excavation. The purpose of this effort is to identify possible loose or soft soil deposits and, if feasible, recompact the soil disturbed during site excavation activities.

Proof rolling should be carefully observed by qualified geotechnical personnel. Areas exhibiting significant deflection, pumping, or oversaturation that cannot be readily compacted should be overexcavated to firm soil. Overexcavated areas should be backfilled with compacted granular material placed in accordance with subsequent recommendations for structural fill. During periods of wet weather, proof rolling could damage the exposed subgrade. Under wet conditions, qualified geotechnical personnel should observe the subgrade to determine if proof rolling is feasible.

Fill and Compaction

Structural fill used to obtain final elevations for footings and soil-supported floor slabs, must be properly placed and compacted. Suitable, non-organic, predominantly granular soil may be used for fill material provided the material is properly moisture conditioned prior to placement and compaction, and the specified degree of compaction is obtained. Material containing topsoil, wood, trash, organic material, or construction debris is unsuitable for reuse as structural fill and should be properly disposed of off site or placed in nonstructural areas.

Soils containing more than approximately 5 percent fines are considered moisture sensitive, and are difficult to compact to a firm and unyielding condition when over the optimum moisture content by more than approximately 2 percent. The optimum moisture content is that which allows the greatest dry density to be achieved at a given level of compactive effort.

Reuse of On-Site Soil

Native site soils in the upper 1 to 2 feet of the explorations have somewhat variable, but slightly elevated, 'fines' contents (percent passing the U.S. No. 200 sieve). The native, sandy gravel (Arlington Gravel) found below the near-surface topsoil and/or fill contained low to very low fines contents.

It is our opinion that the near-surface native soils are suitable for reuse as structural fill when placed at or near optimum moisture contents as determined by ASTM D1557 and if allowed for in the project plans and specifications. Materials with elevated levels of organics cannot be reused as structural fill and should be segregated from mineral soils.

The contractor and owner should be prepared to manage over optimum moisture content soils. Moisture content of the site soils may be difficult to control during periods of wet weather.

Imported Structural Fill

We recommend that imported structural fill consist of clean, well-graded sandy gravel, gravelly sand, or other approved naturally occurring granular material (pit run) with at least 30 percent retained on the No. 4 sieve, or a well-graded crushed rock. Structural fill for dry weather construction may contain on the order of 10 percent fines (that portion passing the U.S. No. 200 sieve) based on the portion passing the U.S. No. 4 sieve. Soil containing more than about 5 percent fines cannot consistently be compacted to a dense, non-yielding condition when the water content is greater than optimum.

Accordingly, we recommend that imported structural fill with less than 5 percent fines be used during wet weather conditions. Due to wet weather or wet site conditions, soil moisture contents could be high enough that it may be very difficult to compact even 'clean' imported select granular fill to a firm and unyielding condition. Soils with over-optimum moisture contents should be scarified and dried back to more suitable moisture contents during periods of dry weather or removed and replaced with fill soils at a more suitable range of moisture contents.

Backfill and Compaction

Structural fill should be placed in horizontal lifts 8 to 10 inches in loose thickness and be thoroughly compacted. All structural fill placed under load bearing areas should be compacted to at least 95 percent of the maximum dry density, as determined using test method ASTM D1557. The top of the compacted structural fill should extend outside all foundations and other structural improvements a minimum distance equal to the thickness of the fill placed beneath the footing. GTS recommends that compaction be tested periodically during placement of structural fill.

Wet Weather Earthwork

If construction is accomplished during wet weather, GTS recommends that structural fill consist of imported, clean, well-graded sand or sand and gravel. If fill is to be placed or earthwork is to be performed under wet conditions, the contractor may reduce soil disturbance by:

- Limiting the size of areas that are stripped of topsoil and left exposed
- Accomplishing earthwork in small sections
- Limiting construction traffic over unprotected soil
- Sloping excavated surfaces to promote runoff
- Limiting the size and type of construction equipment used
- Providing gravel 'working mats' over areas of prepared subgrade
- Removing wet surficial soil prior to commencing fill placement each day
- Sealing the exposed ground surface by rolling with a smooth drum compactor or rubber-tired roller at the end of each working day
- Providing up-gradient perimeter ditches or low earthen berms and using temporary sumps to collect runoff and prevent water from ponding and damaging exposed subgrades.

Seismic Design Considerations

The Pacific Northwest is seismically active and the site could be subject to ground shaking from a moderate to major earthquake. Consequently, moderate levels of shaking should be accounted for during the design life of the project, and the proposed structure should be designed to resist earthquake loading using appropriate design methodology.

For structures designed using the seismic design provisions of the 2015 International Building Code, the native soil underlying the site within the upper 100 feet are classified as Site Class D, according to 2010 ASCE -7 Standard – Table 20.3-1, Site Class Definitions. The corresponding values for calculating a design response spectrum for the assumed soil profile type is considered appropriate for the site.

Please reference the following values for seismic structural design purposes:

Conterminous 48 States – 2015 International Building Code
Zip Code 98223
Central Latitude = 48.18081, Central Longitude = -122.13873

Short Period (0.2 sec) Spectral Acceleration

Maximum Considered Earthquake (MCE) Value of $S_s = 1.061$ (g)

Site Response Coefficient, $F_a = 1.076$ (Site Class D)

Adjusted spectral response acceleration for Site Class D, $S_{MS} = S_s \times F_a = 1.141$ (g)

Design spectral response acceleration for Site Class D, $S_{DS} = 2/3 \times S_{MS} = 0.761$ (g)

One Second Period (1 sec) Spectral Acceleration

Maximum Considered Earthquake (MCE) Value of $S_1 = 0.413$ (g)

Site Response Coefficient, $F_v = 1.587$ (Site Class D)

Adjusted spectral response acceleration for Site Class D, $S_{M1} = S_1 \times F_v = 0.655$ (g)

Design spectral response acceleration for Site Class D, $S_{D1} = 2/3 \times S_{M1} = 0.437$ (g)

Foundation Support

Foundation support for the proposed improvements may be provided by continuous and individual spread footings founded directly on native, medium-dense, gravelly sand (Arlington Gravel) soils, or on compacted, structural fill placed over competent, native soils. GTS recommends that qualified geotechnical personnel confirm that suitable bearing conditions have been reached prior to placement of structural fill or foundation formwork.

To provide proper support, GTS recommends that existing topsoil (if present) and existing fill be removed from beneath all building foundation areas down to the native soils. The surface should be compacted to a firm and unyielding condition with a smooth-drum roller or a similar piece of construction equipment. Once suitable bearing conditions have been confirmed, then foundations can bear directly on native soils, or the footprints constructed with properly compacted structural fill.

Continuous and isolated spread footings should be founded a minimum of 18 inches below the lowest adjacent final grade for freeze/thaw protection. The footings should be sized in accordance with the structural engineer's prescribed design criteria and seismic considerations.

Allowable Bearing Capacity

Assuming the above foundation support criteria are satisfied, continuous and individual spread footings founded directly on compacted, medium-dense, native sandy gravel (Arlington Gravel) or on compacted structural fill placed atop these compacted, native soils, may be proportioned using a net allowable soil bearing pressure of 2,500 pounds per square foot (psf).

The "net allowable bearing pressure" refers to the pressure that can be imposed on the soil at foundation level resulting from the total of all dead plus live loads, exclusive of the weight of the footing or any backfill placed above the footing. The net allowable bearing pressure may be increased by one-third for transient wind or seismic loads.

Foundation Settlement

Settlement of shallow foundations depends on foundation size and bearing pressure, as well as the strength and compressibility characteristics of the underlying soil. Assuming construction is accomplished as previously recommended and for the maximum allowable soil bearing pressure recommended above, GTS estimates the total settlement of building foundations to be less than about one inch and differential settlement between two adjacent load-bearing components supported on competent soil to be less than about one half the total settlement.

Floor Support

Conventional slab-on-grade floor construction appears to be feasible for the planned site improvements. Floor slabs may be supported on properly placed and compacted structural fill placed over properly prepared native soil. Prior to placement of any new structural fill for slab subgrade preparation, the native soil subgrade should be proof-rolled as recommended in the *Site Preparation and Earthwork* section of this report and approved for continued construction.

We recommend that interior concrete slab-on-grade floors be underlain by a minimum of 6 inches of compacted, clean, free-draining gravel (crushed rock with no fines). The purpose of this layer is to provide uniform support for the slab, provide a capillary break, and act as a drainage layer. To help reduce the potential for water vapor migration through floor slabs, a continuous 10-mil minimum thick polyethylene sheet with tape-sealed joints should be installed below the slab to act as a vapor retarder. The vapor retarder should be installed and sealed in accordance with the manufacturer's instructions. 15-mil vapor barriers are also available and can be obtained from manufactures such as Stego and others if moisture control within the building is more critical.

The American Concrete Institute (ACI) guidelines suggest that the slab may either be poured directly on the vapor barrier or on a granular curing layer placed over the vapor barrier depending on conditions during the time of construction. GTS recommends that the architect or structural engineer specify if a curing layer should be used. If moisture control within the building is critical, we recommend that the vapor barrier be observed by a representative of GTS to confirm that openings have been properly sealed. Use of a curing layer is generally only recommended during drier months of the year and/or when limited rain is expected during the slab-on-grade construction process. If the slab is constructed during the wet season, we do not recommend the use of curing layer as excessive moisture emissions through the slab may occur.

Exterior concrete slabs-on-grade, such as sidewalks, may be supported directly on undisturbed native or on properly placed and compacted structural fill; however, long-term performance will be enhanced if exterior slabs are placed on a layer of clean, durable, well-draining granular material.

Resistance to Lateral Loads

The lateral earth pressures that develop against retaining walls will depend on the method of backfill placement, degree of compaction, slope of backfill, type of backfill material, provisions for drainage, magnitude and location of any adjacent surcharge loads, and the degree to which the wall can yield laterally during or after placement of backfill. If the wall is allowed to rotate or yield so the top of the wall moves an amount equal to or greater than about 0.001 to 0.002 times its height (a yielding wall), the soil pressure exerted comprises the active soil pressure. When a wall is restrained against lateral movement or tilting (a nonyielding wall), the soil pressure exerted comprises the at rest soil pressure. Wall restraint may develop if a rigid structural network is constructed prior to backfilling or if the wall is inherently stiff.

GTS recommends that yielding walls under drained conditions be designed for an equivalent fluid density of 35 pounds per cubic foot (pcf) for structural fill in active soil conditions. Nonyielding walls under drained conditions should be designed for an equivalent fluid density of 55 pcf for structural fill in at-rest conditions. Design of walls should include appropriate lateral

pressures caused by surcharge loads located within a horizontal distance equal to or less than the height of the wall. To account for uniform surcharge pressures, a uniformly distributed lateral pressure should be added to the lateral soil pressures. The uniform pressure should be equal to 35 percent of the vertical surcharge yielding walls and 50 percent for non-yielding walls. GTS also recommends that a seismic surcharge pressure of $8H$ be included where H is the wall height in feet. The seismic surcharge should be modeled as a rectangular distribution with the resultant applied at the midpoint of the wall.

Passive earth pressures developed against the sides of building foundations, in conjunction with friction developed between the base of the footings and the supporting subgrade will resist lateral loads transmitted from the structure to its foundation. For design purposes, the passive resistance of well-compacted fill placed against the sides of foundations may be considered equivalent to a fluid with a density of 300 pcf. The recommended value includes a safety factor of about 1.5. In order to calculate the passive resistance, GTS presumes that the ground surface adjacent to the structure is level in the direction of movement for a distance equal to or greater than twice the embedment depth and also assumes drained conditions that will prevent the buildup of hydrostatic pressure in the compacted fill. In design computations, the upper 12 inches of passive resistance should be neglected if the soil is not covered by floor slabs or pavement. If future plans call for the removal of the soil providing resistance, the passive resistance should not be considered. Retaining walls should include a drain system constructed in general accordance with the recommendations presented in the *Foundation and Site Drainage* section of this report.

An allowable coefficient of base friction of 0.35, applied to vertical dead loads only, may be used between the base of the footing and the underlying imported granular structural fill and/or suitable native deposits. If passive and frictional resistance are considered together during wall design, one half the recommended passive soil resistance value should be used since larger strains are required to mobilize the passive soil resistance as compared to frictional resistance. A safety factor of about 1.5 is included in the base friction design value. GTS does not recommend increasing the coefficient of friction to resist seismic or wind loads.

Foundation and Site Drainage

To reduce the potential for groundwater and surface water to seep into interior spaces, GTS recommends that an exterior footing drain system be constructed around the perimeter of new building foundations as shown in the *Typical Footing Drain Section* (Figure 3). The drain should consist of a perforated pipe measuring 4 inches in diameter at minimum, surrounded by at least 12 inches of filtering media with the pipe sloped to carry the discharge water to an approved collection system. The filtering media may consist of open-graded drain rock wrapped by a nonwoven geotextile fabric such as Mirafi 140N (or equivalent) or with a graded sand and gravel filter. For foundations supporting retaining walls, drainage backfill should be carried up the back of the wall and be at least 12 inches wide. The drainage backfill should extend from the foundation drain to within approximately 1 foot of the finished grade and consist of open-graded drain rock containing less than 3 percent by weight passing the U.S. Standard No. 200 sieve (based on a wet sieve analysis of that portion passing the U.S. Standard No. 4 sieve). The invert of the footing drain pipe should be placed slightly below the elevation of the footing or 12 inches below the adjacent floor slab grade, whichever is deeper, so that water will be contained. This prevents water from seeping through walls or floor slabs. The drain system should include cleanouts to allow for periodic maintenance and inspection.

Positive surface gradients should be provided adjacent to the proposed building to direct surface water away from the building and toward suitable drainage facilities. Roof drainage should not be introduced into the perimeter footing drains, but should be separately discharged directly to the stormwater collection system or similar municipality-approved outlet. If present, pavement and sidewalk areas should be sloped and drainage gradients should be maintained to carry surface water away from the building towards an approved stormwater collection system. Surface water should not be allowed to pond and soak into the ground surface near buildings or paved areas during or after construction. Construction excavations should be sloped to drain to sumps where water from seepage, rainfall, and runoff can be collected and pumped to a suitable discharge facility.

Temporary and Permanent Slopes

The contractor is responsible for the actual construction slope configurations and maintenance of safe working conditions, including temporary excavation stability, as this party is able to monitor the construction activities and has direct control over the means and methods of construction. All applicable local, state, and federal safety codes should be followed. All open cuts should be monitored during and after excavation for any evidence of instability. If instability is detected, the contractor should flatten the side slopes or install temporary shoring.

Temporary excavations in excess of 4 feet in depth should be shored or sloped in accordance with Safety Standards for Construction Work, WAC 296-155-66403.

According to WAC 296-155-66403, temporary unsupported excavations in the near surface fill and dry, native soils encountered at the project site are classified as Type B and may be sloped as steep as 1H: 1V. All soils encountered are classified as Type C in the presence of groundwater seepage. Flatter slopes or temporary shoring may be required in areas where groundwater flow is present and unstable conditions develop. Temporary slopes and excavations should be protected as soon as possible using appropriate methods to prevent erosion from occurring during periods of wet weather.

GTS recommends that permanent cut or fill slopes be designed for inclinations of 2H: 1V or flatter. If used for this project, slopes for detention ponds should be designed for inclinations of 3H: 1V or flatter. All permanent slopes should be vegetated or otherwise protected to limit the potential for erosion as soon as practical after construction. As the subject site is in close proximity to an airport, the project team should research what potential impacts (if any) that proposed detention ponds would have on airport operations, or be made aware of restrictions that would prevent the use of ponds in the area.

Utilities

It is important that utility trenches be properly backfilled and compacted to reduce cracking or localized loss of foundation, slab, or pavement support. GTS anticipates that excavations for new shallow underground utilities will expose native gravel and sand.

Trench backfill in improved areas (beneath structures, pavements, sidewalks, etc.) should consist of structural fill as presented in the *Fill and Compaction* section of this report. Outside of improved areas, trench backfill may consist of reused native deposits or clean fill, provided it can be compacted to the project specifications. Trench backfill should be placed and compacted in general accordance with the recommendations presented in the *Fill and Compaction* section of this report.

Surcharge loads on trench support systems due to construction equipment, stockpiled material, and vehicle traffic should be included in the design of any anticipated shoring system. The contractor should implement measures to prevent surface water runoff from entering trenches and excavations. In addition, vibration as a result of construction activities and traffic may cause caving of the trench walls.

The contractor is responsible for trench configurations. All applicable local, state, and federal safety codes should be followed. All open cuts should be monitored by the contractor during excavation for any evidence of instability. If instability is detected, the contractor should flatten the side slopes or install temporary shoring. If groundwater or groundwater seepage is present, and the trench is not properly dewatered, the soil within the trench zone may be prone to caving, channeling, and running. Trench widths may be substantially wider than under dewatered conditions.

Pavement Subgrade Preparation

Selection of a pavement section is typically a choice relative to higher initial cost and lower long-term maintenance or lower initial cost with more frequent maintenance. For this reason, we recommend that the owner participate in the selection of proposed pavement improvements planned for the site. Site grading plans should include provisions for sloping of the subgrade soils in proposed pavement areas, so that passive drainage of the pavement section(s) can proceed uninterrupted during the life of the project. The proposed pavement areas should be prepared as indicated in the *Site Preparation and Earthwork* section of this report.

We anticipate that asphalt pavement will be used for new access drive and parking areas. We recommend a standard, or "light duty", pavement section consist of 2.5 inches of ½-inch HMA asphalt above 8 inches of crushed surfacing base course (CSBC) meeting criteria set forth in the Washington State Department of Transportation (WSDOT) Standard Specification 9-03.9[3]. Areas that will be accessed by more heavily loaded vehicles, semi and garbage trucks, etc. will require a thicker asphalt section and should be designed using a paving section consisting 4 inches of Class ½-inch HMA asphalt surfacing above 8 inches of CSBC meeting criteria set forth in WSDOT Standard Specification 9-03.9[3].

We are available to further consult, review and/or modify our pavement section recommendations based on further discussion and/or analysis with the project team/owner. The above pavement sections should be considered initial recommendations and may be accepted and/or modified by the site civil engineer based on the actual finished site grading elevations and/or the owner's preferences.

Stormwater Infiltration Potential

To estimate the infiltration potential of the on-site subsurface soils, eight representative soil samples were selected and mechanically tested for grain size distribution and interpretation according to the 2012 *Stormwater Management Manual for Western Washington* (SMMWW) (amended December 2014) soil grain size analysis method, Section 3.3.6. A summary of these results are reproduced in Table 1 below.

| TABLE 1 Design Saturated Hydraulic Conductivity Based on the 2012 SMMWW (amended December 2014) | | | | | |
|---|-------------------|---------------------|----------------------------|---|--|
| Test Pit Number | Sample Depth (ft) | Classification USCS | Geologic Unit | K _{sat} Uncorrected Rate (Inches/Hour) | Calculated Infiltration Rate Per ASTM D422 Simplified Approach (Inches/Hour) |
| TP-1 | 2 | SP | Arlington Gravel | 107.0 | 30.8 |
| TP-1 | 9 | SP | Marysville Sand | 67.4 | 19.4 |
| TP-2 | 0.5 | SM | Topsoil | 9.7 | 2.8 |
| TP-2 | 0.75 | GP | Weathered Arlington Gravel | 47.9 | 13.8 |
| TP-2 | 5 | GP | Arlington Gravel | 73.1 | 21.0 |
| TP-4 | 3 | GW | Arlington Gravel | 179.0 | 51.6 |
| TP-6 | 1 | SM/SP | Weathered Arlington Gravel | 34.1 | 9.8 |
| TP-6 | 3 | SP | Arlington Gravel | 60.3 | 17.4 |

Notes:
 - K_{sat} = Initial Saturated Hydraulic Conductivity
 - Listed infiltration rates are estimated long-term (design) rates based on the soil grain size analysis method.
 - Correction Factors Used: CF_V = 0.8, CF_T = 0.4, CF_M = 0.9

In the simplified approach (Section 3.3.4) the infiltration rate is derived by applying appropriate correction factors to the measured saturated hydraulic conductivity (K_{sat}) from the ASTM 422 grain size analysis.

Saturated hydraulic conductivity is a quantitative measure of a saturated soil's ability to transmit water when subjected to a hydraulic gradient, the ease with which pores of a saturated soil permit water movement.

Saturated Hydraulic Conductivity is expressed as follows:

$$\text{Log}_{10}(K_{\text{sat}}) = -1.57 + 1.90D_{10} + 0.015D_{60} - 0.013D_{90} - 2.08f_{\text{fines}}$$

Where D₁₀, D₆₀, and D₉₀ are the grain sizes in mm for which 10 percent, 60 percent, and 90 percent is more fine and f_{fines} is the fraction of the soil (by weight) that passes the U.S. No. 200 sieve. K_{sat} is measured in cm/sec.

With this equation, we can determine the saturated hydraulic conductivity for our representative samples. See example below:

Test Pit TP-2 at 0.75 feet BGS: K_{sat} = 0.03378 cm/sec or approximately 47.87 inches/hour.

Applying correction factors for site variability (0.8), test method (0.4) and degree of influent control to prevent siltation and bio-buildup (0.9) gives a corrected long term design rate of 13.79 inches/hour for the example above.

Based on the simplified approach, the native, Arlington Gravel exhibited long-term design infiltration rates ranging from 13.8 to 51.6 inches per hour. Please note that the simplified approach often provides *overestimations* of infiltration rates for site soils. As such, GTS recommends using a preliminary design infiltration rate of 10 inches per hour for the native, gravelly sands encountered approximately 0.5 to 1 feet BGS.

Please note that the calculated rate given in this section is representative of a preliminary design infiltration rate. If a higher infiltration rate is required, the design rate would best be established by performing a Pilot Infiltration Test. This testing was outside of the scope of this report. However, GTS can provide a fee estimate for this testing upon request.

Stormwater Pollutant Treatment

Prior to offsite discharge, stormwater may require some form of pollutant pretreatment with an amended soil. The reuse of onsite topsoil is often the most sustainable and cost effective method for pollutant treatment purposes. Cation exchange capacities and organic contents of site topsoil and shallow subsurface soils were tested to determine their pollutant treatment suitability.

Cation Exchange Capacity and Organic Content Testing

Five composite samples were collected during subsurface explorations for pollutant treatment purposes. Cation exchange capacity (CEC) and organic content (LOI) tests were performed by Northwest Agricultural Consultants. Laboratory test results are presented in Table 2.

| TABLE 2 CEC & Organic Content Laboratory Test Results | | | | | |
|---|-------------------|----------------------------|--|---------------------|-----|
| Test Pit Number | Sample Depth (ft) | Geologic Unit | Cation Exchange Capacity (meq/100 grams) | Organic Content (%) | pH |
| TP-1 | 0.5 | Topsoil | 24.1 | 13.61 | 5.4 |
| TP-1 | 0.75 | Weathered Arlington Gravel | 7.5 | 3.11 | 5.5 |
| TP-4 | 0.5 | Topsoil | 18.8 | 8.40 | 5.9 |
| TP-4 | 1.5 | Weathered Arlington Gravel | 6.0 | 2.51 | 6.3 |
| TP-6 | 0.5 | Topsoil | 17.9 | 7.78 | 5.7 |
| Notes: - SSC-6 Criteria for on-site pollutant treatment: CEC ≥ 5.0 meq/100 grams and Organic content ≥ 1 % | | | | | |

Based on the results listed in Table 2, the near surface topsoil and weathered Arlington Gravel generally appear suitable for on-site pollutant treatment purposes based on the 2012 *Stormwater Management Manual for Western Washington (amended December 2014)*.

Geotechnical Consultation and Construction Monitoring

GeoTest Services recommends that we be involved in the project design review process. The purpose of the review is to verify that the recommendations presented in this report have been properly interpreted and incorporated in the design and specifications. Project concepts had not been developed at the time of this report. Thus, it should be expected that additional input may be needed to finalize the design.

We also recommend that geotechnical construction monitoring services be provided. These services should include observation by GeoTest personnel during structural fill placement, compaction activities and subgrade preparation operations to confirm that design subgrade conditions are obtained beneath the proposed buildings. Periodic field density testing should be performed to verify that the appropriate degree of compaction is obtained. The purpose of these services is to observe compliance with the design concepts, specifications, and recommendations contained within this report. In the event that subsurface conditions differ from those anticipated before the start of construction, GeoTest Services would be pleased to provide revised recommendations appropriate to the conditions revealed during construction.

GeoTest is also available to provide a full range of materials testing and special inspection during building construction as required by the local building department and the International Building Code. This may include specific construction inspections on materials such as reinforced concrete, reinforced masonry, wood framing and structural steel. These services are supported by our fully accredited materials testing laboratory.

USE OF THIS REPORT

GeoTest Services has prepared this report for the exclusive use of Mr. Goutam Jain, Mr. Kamal Singh and their representatives, for specific application to the design of the proposed Arlington Mixed-Use project to be constructed in Arlington, Washington. Use of this report by others is at the user's sole risk. This report is not applicable to other sites. Our services have been conducted in accordance with generally accepted practices of the geotechnical engineering profession; no other warranty, express or implied, is made as to the professional advice included in this report.

Our site explorations indicate subsurface conditions at the dates and locations indicated. It is not warranted that these conditions are representative of conditions at other locations and times. The analyses, conclusions, and recommendations contained in this report are based on site conditions to the limited depth of our explorations at the time of our exploration program, a geological reconnaissance of the area, and a review of previously published USGS geological information for the site. If variations in subsurface conditions are encountered during construction that differs from those in this report, GTS should be allowed to review the recommendations contained in this report, and revise if necessary. If there is a substantial lapse of time between submission of this report and the start of construction, or if conditions change due to construction operations at or adjacent to the project site, we recommend that we review this report to determine the applicability of the conclusions and recommendations contained herein.

The earthwork contractor is responsible to perform all work in conformance with all applicable WISHA/OSHA regulations. GeoTest Services, Inc. should not be assumed to be responsible for job site safety on this project, and this responsibility is specifically disclaimed.

GTS appreciates the opportunity to provide geotechnical services on this project and look forward to assisting you during the construction phase. If you have any questions regarding the information contained in this report or if we may be of further service, please contact the undersigned.

Respectfully Submitted,
GeoTest Services, Inc.



[Signature]
Gerry D. Baustista, Jr., P.E.
Project Geotechnical Engineer

Gerry D. Baustista, Jr., P.E.
Project Geotechnical Engineer

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|--------------|-------------------------|---|
| Attachments: | Figure 1 | vicinity map |
| | Figure 2 | Site and Exploration Plan |
| | Figure 3 | Typical Footing and Wall Drain Section |
| | Figure 4 | Soil Classification System and Key |
| | Figures 5-7 | Test Pit Logs |
| | Figures 8-9 (1 page) | Grain Size Test Data |
| | (3 pages) | Cation Exchange Capacity and Organic Content Results |
| | | GeoTest – Report Limitations and Guidelines for its Use |

References:

Minard, J.P., 1985, *Geologic map of the Arlington West 7.5-minute quadrangle, Snohomish County, Washington*. U.S. Geological Survey, Miscellaneous Field Studies Map MF-1740, scale 1:24,000.

Washington State Department of Ecology, *Stormwater Management Manual for Western Washington*, 2012 (amended December 2014).