Geotechnical Report
Fife South
37th Street NE
Fife, Washington

1.0 PROJECT DESCRIPTION

The Fife South project will be a light industrial development and will include warehouse-style structures with office space and associated parking and utilities. Grading plans are not yet developed; however, we expect site grades will be raised in building areas to facilitate dock-high access and stormwater drainage. Development stormwater will likely be pumped to open-cell detention ponds for treatment and controlled discharge to the receiving drainage basins.

The buildings are expected to be warehouse structures with adjoining office areas. The warehouse-style buildings will have multiple dock-high doors providing truck access to finished floor elevations. Construction of the buildings will consist of reinforced concrete tilt-up walls, framed roofs, and concrete floors. We expect structural loads to be about 50 to 100 kips per foot for isolated columns, and 3 to 4 kips per foot for continuous perimeter bearing walls. We expect the dock-high walls and depressed loading dock walls will be designed as retaining walls approaching heights of four feet. Maximum product loading on the floors will be in the range of 250 to 300 pounds per square foot (psf).

The recommendations contained in the following sections of this report are based on our understanding of the above design features. If actual features vary or changes are made, we should review them in order to modify our recommendations as required. We should review the final design drawings and specifications to verify that our recommendations have been properly interpreted and incorporated into project design and construction.

2.0 SCOPE OF WORK

Our work was completed in accordance with our proposal dated November 18, 2000, which was authorized on December 4, 2000. Accordingly, on December 20 through 22, 2000, we observed the excavation of 42 test pits to depths of 10 feet below existing surface grades. We also subcontracted with Northwest Cone Exploration to perform 13 electric piezo-cone explorations that were advanced to maximum depths of 40 feet below existing site grades. Using the results of our field studies and laboratory testing, we performed analyses to develop geotechnical engineering recommendations for project design and construction. Specifically, this report addresses the following:

- Soil and groundwater conditions
- Seismic
- Site preparation and grading
- Excavations
- Preload
- Dewatering

- Foundation support
- Slab-on-grade floors
- Retaining walls
- Detention ponds
- Drainage
- Utilities
- Pavements

3.0 SITE CONDITIONS

3.1 Surface

The project site is an irregular-shaped parcel of land, encompassing approximately 125 acres. The approximate location of the site is shown on Figure 1. The site is bounded by 70th Avenue East to the west, an existing railroad right-of-way to the north, Freeman Road to the east, and 45th Street Court to the south. This site is currently undeveloped and is used for agricultural purposes. Wapato Creek flows through the northeast corner of the site, with the Puyallup River flowing westward to Puget Sound, approximately one-half mile south of the site. Site topography is relatively flat, ranging from Elev. 22 to the west to Elev. 33 to the east. At the time of our field exploration, the surface of the site had been cultivated and was relatively free of vegetation. An unharvested crop of cabbage was still present in the site's northwestern corner.

3.2 Subsurface

Soil conditions we observed at our test pit and cone exploration locations are alluvial in origin, consisting of layers of silt, silty sand, and sand. Our exploration revealed no discernable stratification of the soil layers, with conditions somewhat variable at each of the exploration locations. In general, the surface of the site is mantled by a dark brown sandy silt to silty sand containing some organic debris. The thickness of this upper layer is on the order of 12 to 24 inches. Underlying the surface material, we observed layers of silty sand, sand, and silt to the ten-foot termination depths of our test pits. The sand layers encountered were typically of a fine- to medium-grained texture and included laminations and seams of non-plastic silt. We observed the relative density of the soil layers to vary from very loose to medium dense, and that the soils were in a wet to water-bearing condition. In an alluvial environment, the presence of large organic debris buried beneath the surface is not uncommon. Such conditions were observed at Test Pit TP-6, where we observed a tree trunk at about eight feet. Similar buried organic debris may also be present throughout the site.

The deeper cone penetration exploration locations revealed similar soil deposits, extending to the maximum exploration depths of 40 feet below existing site grades. Typically, below depths of 20 to 25 feet, the relative density of the soil deposits is shown to increase with medium dense to dense conditions generally indicated. Exceptions to this were observed at CPT-1, CPT-4, CPT-6, CPT-10 and CPT-13, where very loose to loose soil strata are still indicated below depths of 30 feet. Generally, the very loose to loose soil layers are indicated to be composed of sandy silt to silt. Where medium dense to dense conditions are observed, the soil layers are indicated to consist of relatively clean sand.

The preceding discussion is intended as a general review of the soil conditions encountered. For a more complete description, please refer to the test pit logs and the cone penetration test data attached in Appendix A.

3.3 Groundwater

We observed the groundwater table at depths of three to eight feet below existing surface grades at the time of our exploration. Slotted standpipes were installed in about one-half of the test pits to allow for future monitoring of groundwater fluctuations. The depths to groundwater observed at these locations during excavation of the test pits, and approximately two weeks following, is shown in the following table:

Table A
Groundwater Levels in Test Pits

Test Pit Number	Depth to Groundwater (feet) at time of excavation	Depth to Groundwater (feet) 1/15/01
TP-1	4	At surface
TP-3	8	1.3
TP-5	8	1.3
TP-7	5	.3
TP-9	5	1.6
TP-11	7	.9
TP-13	6	1
TP-15	5	.4
TP-17	5	.4
TP-19	5	2.7
TP-21	5	2.8
TP-23	5	.5
TP-25	5	1
TP-27	3	.3
TP-29	5	2.3
TP-32	5	2.4
TP-33	6	1
TP-35	6	2.8
TP-39	5	4.7
TP-42	5	3.6

Fluctuations in the static groundwater level will occur seasonally and will, to some extent, be controlled by the elevation of Wapato Creek and, to a lesser extend, the Puyallup River. Typically, groundwater will reach maximum levels during the wet winter months, which is the time of year we completed our field studies. However, given the below-normal rainfall the area has experienced, we do not believe the recorded water levels at this time represent seasonal high levels. It is likely that groundwater is at or within one foot of the surface throughout the site during a normal winter season.

4.0 GEOLOGIC HAZARDS

4.1 Seismic

The Puget Sound area falls within Seismic Zone 3, as classified by the Uniform Building Code (UBC). Based on the soil conditions encountered and the local geology, from Table 16-J of the UBC, soil profile type S_E should be used in design.

Liquefaction is a phenomenon where there is a reduction or complete loss of soil strength due to an increase in water pressure induced by vibrations. Liquefaction mainly affects geologically recent deposits of fine-grained sands that are below the groundwater table. Soils of this nature derive their strength from intergranular friction. The generated water pressure, or pore pressure, essentially separates the soil grains and eliminates this intergranular friction, thus eliminating the soils' strength.

Soil conditions observed at the site will be subject to liquefaction during an earthquake. Our analysis of these conditions, under stresses induced by a Richter magnitude earthquake of 7.5 generating a sustained ground acceleration of .2g, indicates that varying soil layers in the overall soil profile would liquefy throughout the project site. Depending on the groundwater elevation at the time the earthquake would occur, soil liquefaction could develop to the current surface elevation.

Soil liquefaction occurring at this site could impact building development in the form of bearing capacity failure of standard spread footing foundations supported on liquefied soil, settlement and heave, or uplift of buried structures. An effective means to mitigate a portion of these potential impacts will be to raise site and building elevations. Because of the high groundwater table that exists at the site, raising site and building elevations will also be beneficial from a construction perspective and long-term performance of site pavements.

With these factors considered, as a minimum, we recommend raising overall site grades one foot and building floor elevations five feet above existing surface grades. Raising building elevations in this manner should eliminate the concern with respect to bearing capacity failures occurring beneath foundations due to liquefaction. The remaining potential impact would be in the form of surface subsidence or settlement. Based on our analysis of these conditions, it is our opinion that surface subsidence or settlement in the range of two to four inches could potentially occur during an earthquake of the referenced magnitude. We do not believe settlement of this magnitude would structurally impair the buildings; however, significant cracking of building walls and floor slabs could result. The structural engineer should review this condition and determine if differential movement of this magnitude would structurally impair building support.

Excess pore water pressures generated during an earthquake will also subject buried structures to additional uplift stress above that normally experienced due to static groundwater conditions. This factor should be taken into consideration for design of deep buried structures, such as lift stations. We should review lift station design in order to provide recommendations for additional uplift stress due to this condition.

If the owner is not willing to accept the risk associated with soil liquefaction occurring at the site, and potential impacts to buildings and site infrastructure, alternative means of building support and site preparation should be considered. These alternatives would range from improving existing soil conditions at the site by densification to make them more resistant to the liquefaction phenomenon to supporting buildings and other important site infrastructure on pile foundations. We can assist in evaluating these alternatives and develop design recommendations for implementation if desired.

5.0 DISCUSSION AND RECOMMENDATIONS

5.1 General

In our opinion, subject to the owner accepting potential impacts to building and site infrastructure due to liquefaction, there are no geotechnical considerations that would preclude development of the site as planned. The soft, fine-grained native soil layers observed at the site will consolidate under static dead loads imposed by the structures and by product loading on structure floor slabs. To mitigate the potential for post-construction settlement due to this consolidation, we recommend preloading the building locations. Preloading will involve placing the structural fill required to achieve the finish floor elevations and allowing settlements to occur under this load before building construction is initiated. As discussed in the preceding section, we recommend raising building site grades a minimum of five feet above existing grades to mitigate a portion of the potential building impacts due to soil liquefaction. This depth of fill will also be sufficient for preloading considerations. Analysis indicates that settlements with a minimum of five feet of fill above existing grades will range from three to four inches. These settlements are expected to occur in about two to four weeks following full application of the building fill.

After completing the preload, building construction can begin. The buildings can be supported on conventional spread footings bearing on a minimum of two feet of compacted structural fill. With expected building grades, overexcavation of native soils and replacement with structural fill will likely be required below the perimeter continuous footing adjacent loading dock areas.

The native soils encountered at the site contain a significant amount of fines and will be difficult to compact as structural fill when too wet. The ability to use native soil from site excavations as structural fill will depend on its moisture content and the prevailing weather conditions at the time of construction. With the elevated groundwater table and soil moisture contents, as indicated by laboratory testing at the time of our study, the contractor should be prepared to dry the native soils by aeration during the normally dry summer season to facilitate compaction as structural fill. Alternatively, stabilizing the moisture in the native soil with cement kiln dust (CKD), cement, or lime can be considered. If grading activities will take place during the winter season, the contractor should be prepared to import clean granular material for use as structural fill and backfill.

The following sections provide detailed recommendations regarding the above issues and other geotechnical design considerations. These recommendations should be incorporated into the final design drawings and construction specifications.

5.2 Site Preparation and Grading

At the time of our study, surface conditions at the site were wet and soft and we were unable to access the test pit locations with a rubber-tired backhoe. A track-mounted excavator was necessary to move around the area of the site. The grading contractor should expect similar conditions, especially if grading is initiated in early spring and should plan site grading activities accordingly.

At the time of our study, the majority of the site was cultivated and there was little surface vegetation. In general, it will not be necessary to strip the organic surface layer where structural fill depths above existing grade are a minimum of three feet, and two feet in building and pavement areas, respectively. Where surface vegetation exists, it should be mowed closed to the ground with cutting debris removed from the site. Where structural fill depths fall below these minimums, both the organic topsoil and vegetation should be stripped from below building and pavement areas. Topsoil will not be suitable for use as structural fill but can be used in landscaped areas. Unharvested crops, such as the cabbage in the northwest portion of the site, should be stripped and removed from below new construction regardless of the overlying fill depth.

Once clearing and grubbing operations are complete, grading to establish desired building grades can be initiated. Prior to placing fill, the native subgrade should be stabilized by aerating and recompacting the upper 12 inches of exposed soils. Alternatively, the soils' moisture can be stabilized by use of an additive, such as cement, CKD, or lime.

The near-surface dark brown sandy silt immediately underlying the topsoil possesses relatively low strength characteristics. Even when amended with cement or lime at a rate of 9 to 11 percent by dry compacted soil weight, compressive strengths of 100 pounds per square inch (psi) and less were obtained in the laboratory. In our opinion, this soil, when placed and compacted to a stable non-yielding condition, treated or untreated, would be suitable for support of normally loaded slab-on-grade floors. However, because of its limited strength, we do not recommend it be relied upon to provide immediate support for asphalt pavement.

To construct suitable support for pavements, we recommend raising pavement grades a minimum of 12 inches above existing grade using a suitable granular imported structural fill. To construct a soil cement base for immediate support of the pavement, the import fill should be blended with Type I Portland cement at a rate of six to seven pounds per cubic foot of soil, moisture conditioned as necessary, and then compacted as structural fill. The soil cement should be tested to determine its compressive strength. A minimum 7-day compressive strength of 300 psi is recommended.

If grading activities are planned during the wet winter months, and the on-site soils become too wet to achieve adequate compaction, the owner or contractor should be prepared to treat soils with CKD, lime, or cement, or import wet weather structural fill. For this purpose, we recommend importing a granular soil that meets the following grading requirements:

U.S. Sieve Size	Percent Passing
6 inches	100
No. 4	75 maximum
No. 200	5 maximum*

*Based on the 3/4-inch fraction

Prior to use, Terra Associates, Inc. should examine and test all materials to be imported to the site for use as structural fill. If building subgrades constructed using native soils will be exposed during wet weather, it would be advisable to place 12 inches of this granular structural fill on the building pad to prevent deterioration of the floor subgrade.

Structural fill should be placed in uniform loose layers not exceeding 12 inches and compacted to a minimum of 95 percent of the soils' maximum dry density, as determined by ASTM Test Designation D-698 (Standard Proctor). The moisture content of the soil at the time of compaction should be within two percent of its optimum, as determined by this same ASTM standard. In non-structural areas, or for backfill in utility trenches in pavement areas below a depth of 4 feet, the degree of compaction can be reduced to 90 percent.

5.3 Preload

We recommend preloading the building areas to limit building and floor slab settlements to tolerable levels. For this procedure, we recommend placing structural fill in the building areas to the design subgrade elevations and delaying building construction until settlements under this fill load have occurred. A minimum of five feet of fill, above existing grades, should be placed. The preload fill should extend a minimum of two feet beyond the building perimeter.

Total settlements under the building fill is estimated in the range of three to four inches. These settlements are expected to occur in about two to four weeks following full application of the building fill.

To verify the amount of settlement and the time rate of movement, the preload program should be monitored by installing settlement markers. The settlement markers should be installed on the existing grade prior to placing any building or preload fills. Once installed, elevations of both the fill height and marker should be taken daily until the full height of the preload is in place. Once fully preloaded, readings should continue weekly until the anticipated settlements have occurred.

It is critical that the grading contractor recognize the importance of the settlement marker installations. All efforts must be made to protect the markers from damage during fill placement. It is difficult, if not impossible, to evaluate the progress of the preload program if the markers are damaged or destroyed by construction equipment. As a result, it may be necessary to install new markers and extend the surcharging time period in order to ensure that settlements have ceased and building construction can begin.

Following the successful completion of the preload program, with foundations designed as recommended in Section 5.5 of this report, you should expect maximum total and differential post-construction settlements six months following full load application of one-half inch for perimeter foundations and one inch for interior column supports.

5.4 Excavations

All excavations at the site associated with confined spaces, such as utility trenches and lower building levels, must be completed in accordance with local, state, or federal requirements. Based on current Occupational Safety and Health Administration (OSHA) regulations, soils found on the project site would be classified as Group C soils.

For properly dewatered excavations more than four feet but less than 20 feet in depth, the side slopes should be laid back at a minimum slope inclination of 1.5:1 (Horizontal:Vertical). If there is insufficient room to complete the excavations in this manner, or if excavations greater than 20 feet in depth are planned, using temporary shoring to support the excavations may need to be considered.

Groundwater should be anticipated within excavations extending to depths of one to four feet and deeper below existing surface grades. Based on our study, the volume of water and rate of flow into the excavation may be significant and dewatering of the excavations may be necessary. Shallow excavations that do not extend more than two to three feet below the groundwater table can likely be dewatered by conventional sump pumping procedures along with a system of collection trenches. Deeper excavation may require dewatering by well points or isolated deep-pump wells. The utility subcontractor should be prepared to implement excavation dewatering by well point or deep pump wells, as needed. This will be an especially critical consideration for any deep excavations, such as that which may be required for lift station construction.

This information is provided solely for the benefit of the owner and other design consultants, and should not be construed to imply that Terra Associates, Inc. assumes responsibility for job site safety. It is understood that job site safety is the sole responsibility of the project contractor.

5.5 Foundations

In our opinion, the building may be supported on conventional spread footing foundations bearing on a minimum of two feet of structural fill. The structural fill should extend beyond the edge of the footing a minimum distance of one foot. Perimeter bearing walls adjacent to loading docks will require overexcavation and replacement of native soils. Structural fill used in these areas should consist of a granular import material meeting the grading recommendations for a wet weather structural fill. Foundations exposed to the weather should bear at a minimum depth of 1.5 feet below adjacent grades for frost protection.

We recommend designing foundations for a net allowable bearing capacity of 2,000 psf. For short-term loads, such as wind and seismic, a one-third increase in this allowable capacity can be used. With the expected building loads and this bearing stress applied, in general, total and differential settlements should not exceed one-half inch for perimeter foundations, and one inch for interior column supports.

For designing foundations to resist lateral loads, a base friction coefficient of 0.35 can be used. Passive earth pressures acting on the sides of the footings can also be considered. We recommend calculating this lateral resistance using an equivalent fluid weight of 300 pcf. We do not recommend including the upper 12 inches of soil in this computation because it can be affected by weather or disturbed by future grading activity. This value assumes the foundation will be constructed neat against competent native soil or backfilled with structural fill, as described in Section 5.2 of this report. The values recommended include a safety factor of 1.5.

5.6 Retaining Walls

The magnitude of earth pressure development on retaining walls will partly depend on the quality of backfill. We recommend placing and compacting wall backfill as structural fill. To guard against the buildup of hydrostatic pressure, wall drainage must also be installed. A typical wall drainage detail is attached at Figure 3.

With granular backfill placed and compacted as recommended and drainage properly installed, we recommend designing retaining walls for an earth pressure equivalent to a fluid weighing 35 pcf. When necessary, to account for traffic surcharge, the wall should be designed for an additional height of two feet. Friction at the base of foundations and passive earth pressure will provide resistance to these lateral loads. Values for these parameters are provided in the Section 5.5 of this report.

5.7 Slab-on-Grade Floors

Slabs-on-grade may be supported on the subgrade prepared as recommended in Section 5.2 of this report. Immediately below the floor slab, we recommend placing a four-inch thick capillary break layer of clean, free-draining sand or gravel that has less than three percent passing the No. 200 sieve. This material will reduce the potential for upward capillary movement of water through the underlying soil and subsequent wetting of the floor slab. Where moisture by vapor transmission is undesirable, such as covered floor areas, a durable plastic membrane should be placed on the capillary break layer. At the owner's discretion, the membrane can be covered with two inches of clean, moist sand to guard against damage during construction and to aid in the curing of the concrete.

For design of the floor slabs-on-grade, a subgrade modulus (k_s) of 200 pounds per cubic inch (pci) can be used.

5.8 Stormwater Detention Pond

Site development will include construction of detention ponds for collection, treatment, and controlled release of development stormwater. We would expect that these ponds will be constructed by a combination of excavation below existing surface grades and placement of a perimeter berm. Because of the elevation of the groundwater table, the ability to increase the live storage volume by excavation below existing surface grades will be limited. For design considerations, we recommend considering the groundwater table at a depth of one foot below existing surface grades. Excavation below the groundwater table can be considered for establishing the desired dead storage water volume for water quality treatment considerations.

Based on soil conditions observed at the site and the groundwater elevation, interior pond slopes should be graded at an inclination no steeper than 3:1. The interior pond slope surfaces above the dead storage elevation should also be appropriately vegetated to reduce erosion due to fluctuating water levels and direct surface precipitation.

Fill material used to construct perimeter berms should be placed and compacted structurally, as recommended in Section 5.2 of this report. Perimeter berms should have a minimum crest width of five feet and their exterior slopes graded at an inclination no steeper than 2:1.

5.9 Drainage

Surface

Final exterior grades should promote free and positive drainage away from the site at all times. Water must not be allowed to pond or collect adjacent to foundations or within the immediate building areas. We recommend providing a gradient of at least two percent for a minimum distance of ten feet from the building perimeters, except in paved locations. In paved locations, a minimum gradient of one percent should be provided, unless provisions are included for collection and disposal of surface water adjacent to the structure.

Subsurface

In our opinion, except where landscaping will occur directly adjacent to the structure, perimeter foundation drains would not be required. Where installed, the foundation drains should be tightlined separately from the roof drains with a gradient sufficient to promote positive flow to a controlled point of approved discharge. All drains should be provided with cleanouts at easily accessible locations.

As noted, groundwater was observed at a relatively shallow depth. To prevent impacts to the supporting capability of the pavement subgrade, particularly in loading dock areas, we recommend installing a system of shallow intercepting drains. The drains should be installed with a maximum pipe invert equivalent to a depth of three feet below the lowest pavement grade adjacent to the buildings. The drains should be installed with a positive drainage gradient to a controlled point of discharge.

The interceptor drains can and should be installed in the same trench as the storm sewer utility. For estimating the volume of groundwater intercepted by the drains, a flow rate of .01 gallons per minute (GPM) per foot of drain installed can be used.

5.10 Utilities

Utility pipes should be bedded and backfilled in accordance with American Public Works Associates (APWA) or the City of Fife specifications. As a minimum, trench backfill should be placed and compacted as structural fill as described in Section 5.2 of this report. At the time of our study, the soils' moisture content was above optimum; therefore, drying back or other means to condition the material will probably be necessary to facilitate proper compaction. If utility construction takes place during the winter, it may be necessary to import suitable wet weather fill for utility trench backfilling.

For any structure installed below a depth of three feet, buoyancy effects must be considered. Buoyancy or uplift will be resisted by the weight of the structure and the weight of the soil overlying its foundation or cover. For backfill placed as structural fill, a soil unit weight of 110 pcf can be used.

Buoyancy, or an unbalanced hydrostatic head, will also impact the trench bottom stability. Where an unbalanced hydrostatic head exists in the trench excavation, the trench bottom can heave and, subsequently, become unstable causing installed utility pipes to settle when overburdened stresses from utility trench backfill are replaced.

Two methods for stabilizing the trench bottoms can be considered. The first involves using well-point dewatering systems to lower the groundwater table adjacent to utility excavation and prevent development of an unbalanced hydrostatic head. Single-stage well-point dewatering systems are typically effective for utility excavations occurring to depths of 15 to 20 feet. The second method that can be used to mitigate heave or unstable soil conditions at the trench bottom involves overexcavation of the affected soils and replacement with additional free-draining bedding material. As a general rule, the depth of overexcavation below the pipe invert and replacement with free-draining bedding material would be equivalent to one foot for every two feet of unbalanced hydrostatic head.

5.11 Pavements

In order to prepare a stable subgrade and pavement base, we recommend using a soil cement application as discussed in Section 5.2 of this report. The cement should be blended uniformly with the import soil, with the mixture also moisture conditioned as necessary. The soil cement moisture should be within −1 to +3 percent of optimum, as determined by ASTM Test Designation D-698 (Standard Proctor) prior to compaction. Once blended and conditioned, the soil cement should be compacted to a minimum of 95 percent of its maximum dry density, as determined by this ASTM standard. The soil cement should achieve a minimum 7-day compressive strength of 300 psi.

Initial compaction of the soil cement should be accomplished with a sheep's foot compactor. Once compacted, rough grading can be completed with final compaction achieved using a steel drum roller. Compaction and rough grading should be completed within a two-hour time period following application and blending of the cement with the soil.

After grading and compaction, traffic should stay off the soil cement base for a minimum of three days to allow the base to cure and gain its initial compressive strength. Pavement construction should then be completed shortly following this initial curing period. If the soil cement base will not be paved over following initial curing and traffic will traverse the base, we recommend placing a two-inch thick layer of crushed rock over the soil cement to reduce surface degradation.

Quality control during construction of the soil cement base should include verifications of the following:

- Cement application rate
- Moisture and compaction
- Compressive strength

A minimum of three test specimens from the same soil cement sample should be prepared for compressive strength testing for each day's construction.

Traffic at the facility will mainly consist of cars and light trucks, with moderately heavy traffic in the form of tractor-trailer rigs. We recommend that the pavement section constructed over the 12 inches of soil cement base consist of 2 inches of asphalt concrete (AC). The AC should meet the requirements of a Class B mix, as outlined in Washington State Department of Transportation's standard specifications.

6.0 ADDITIONAL SERVICES

Terra Associates, Inc. should review the final design and specifications in order to verify that earthwork recommendations have been properly interpreted and incorporated into project design and construction. We should also provide geotechnical services during construction in order to observe compliance with the design concepts, specifications, and recommendations. This will allow for design changes if subsurface conditions differ from those anticipated prior to the start of construction.

7.0 LIMITATIONS

We prepared this report in accordance with generally accepted geotechnical engineering practices. This report is the property of Terra Associates, Inc. and is intended for specific application to the Fife South project. This report is for the exclusive use of Opus Northwest, LLC and their authorized representatives.

The analyses and recommendations presented in this report are based upon data obtained from the borings advanced on-site. Variations in soil conditions can occur, the nature and extent of which may not become evident until construction. If variations appear evident, Terra Associates, Inc. should be requested to re-evaluate the recommendations in this report prior to proceeding with construction.