GEOTECHNICAL ENGINEERING STUDY

For

PRESTIGE WORLDWIDE HOLDINGS, LLC. SITE

2003 136TH AVENUE EAST
SUMNER, PIERCE COUNTY, WA 98390

Prepared For

JOHANSEN EXCAVATING INC.

P.O. BOX 188, PUYALLUP, WA 98321

Prepared By

PGE Pacific Geo Engineering
Geotechnical Engineering, Consulting & Inspection

P.O. BOX 1419, ISSAQUAH, WASHINGTON 98027

PGE PROJECT NUMBER 16-495

June 06, 2016

PGE Pacific Geo Engineering
Geotechnical Engineering, Consulting & Inspection

June 06, 2016

Johansen Excavating Inc.
P.O. Box 188
Puyallup, Pierce County, WA 98371

Attn.: Mr. Jacob Cimmer
Vice President

Re: Geotechnical Engineering Study
Proposed Prestige Worldwide Holdings, LLC.
2003 136th Avenue East
Sumner, Pierce County, WA 98321
PGE Project No. 16-495

Dear Mr. Cimmer:

Pacific Geo Engineering, LLC (PGE) has completed the geotechnical engineering study for the subject site located at the above address in Spanaway, Pierce County, Washington. This report includes the results of our subsurface exploration and engineering evaluation, and provides recommendations for the geotechnical aspects of the design and development of the project.

Based on this study, there are no geotechnical considerations that would preclude the proposed development as planned, therefore, the subject site is considered suitable for the proposed development.

We trust the information presented in this report is sufficient for your current needs. We appreciate the opportunity to provide the geotechnical services at this phase of the project and look forward to continued participation during the design and construction phase of this project. Should you have any questions or concerns, which have not been addressed, or if we may be of additional assistance, please do not hesitate to call us at 425-218-9316 or 425-643-2616.

Santanu Mowar

Respectfully submitted,

Santanu Mowar, MSCE, P.E.

PGE Pacific Geo Engineering
Geotechnical Engineering, Consulting & Inspection

SANTANU MOWAR
STATE OF WASHINGTON
37403
REGISTERED
PROFESSIONAL ENGINEER
Santanu Mowar
EXPIRES 01-01-2017

TABLE OF CONTENTS

Page No.

1.0 **INTRODUCTION** 1
2.0 **PROPOSED DEVELOPMENT** 1
3.0 **SCOPE OF SERVICES** 2
3.1 Field Investigation 2
3.2 Laboratory Testing 3
3.3 Engineering Evaluation 3
4.0 **SURFACE AND SUBSURFACE FEATURES** 4
4.1 Site Location 4
4.2 Site Description 4
4.3 Regional Geology 4
4.4 Soil & Groundwater Conditions 5
4.5 Soil Conservation Survey Soil Description 6
5.0 **CONCLUSIONS AND RECOMMENDATIONS** 6
5.1 General 6
5.2 Site Preparation 8
5.2.1 Clearing and Grubbing 8
5.2.2 Subgrade Preparation 8
5.2.3 New Structural Fills 9
5.2.4 Fill Placement & Compaction Requirements 10
5.2.5 Settlement Monitoring 11
5.2.6 Fill Pad Permanent Slopes 11
5.2.7 Site Drainage 11
5.2.8 Utility Support and Backfill 12
5.2.9 Construction Monitoring 13
5.3 Foundation Recommendations 14
5.4 Floor Slabs 15
5.5 Pavement Thickness 15
5.6 Geologic Hazards 16
5.6.1 Erosion Hazard 17
5.6.2 Seismic Design Parameters 17
5.6.3 Seismically Induced Geotechnical Hazards 17
5.6.4 Landslide Hazard 18
6.0 **GEOTECHNICAL SPECIAL INSPECTIONS** 18
7.0 **ADDITIONAL SERVICES RECOMMENDED** 19
7.1 Design Phase Engineering Services 19
7.2 Construction-time Testing and Inspections 19
8.0 **REPORT LIMITATIONS** 20

LIST OF FIGURES

Figure 1 Vicinity Map & Site Exploration Plan
Figure 2 TESC Plan
Figure 3 TESC Plan

LIST OF TABLES

Table 1 Puyallup Sandy Loam (31A)
Table 2 Structural Fill

LIST OF APPENDICES

Appendix A Test Pit Log & Test Pit Photo
Appendix B Laboratory Test Results

1.0 INTRODUCTION

This report presents the findings of our subsurface explorations and geotechnical engineering evaluation for a proposed commercial yard, to be located east of Highway 167 and just north of 24th Street East on the 136th Avenue East in Sumner, Pierce County, Washington. The site location is shown on the Vicinity Map, Figure 1. This study was accomplished in general accordance with our proposal No. 16-04-489, dated April 7, 2016, and was granted to proceed by written authorization of Mr. Jacob Cimmer of Johansen Excavating, Inc. on May 9, 2016.

2.0 PROPOSED DEVELOPMENT

The proposed development plan is shown in the Site & Exploration Plan, Figure 1, prepared by Innova Architect, and in the TESC and Grading Plans, Figure 2 and 3, prepared by Larson & Associates. The proposed development plan calls for constructing a commercial yard with a 20,000 SF, two-storey building, and associated driveway and parking areas around the building. Also, a proposed 53,757 SF gravel area east of the proposed building area will be re-built for equipment and vehicle storage.

Based on the information provided by Innova Architects, the perimeter wall load will be 5.7 kips per lineal foot, the isolated column load will be 135 kips, and the slab-on-grade floor load will be 350 pounds per square foot (psf).

The existing native grades and the final design building grades were available from Larson and Associates. The TESC plans show that in general, the current native grades are 63 and 64 at the proposed building pad and re-built vehicle storage areas, respectively. The final grades in the building pad area will be 69, which demonstrates that the approximate fill thickness will be 6 feet in the building pad area. Approximately, 2 to 4 feet of fills will be required in the proposed storage vehicle area to achieve the final grade in this area.

The proposed development will include asphalt-paved driveway and parking areas around the building, and re-built gravel paved area in the existing gravel paved area raising the current gravel paved grade to 2 to 4 feet high. We anticipate vehicle traffic in the proposed building pad area will primarily consists of passenger vehicles with occasional waste management trucks, and in the proposed storage vehicle area the traffic will consists of large commercial trucks and track vehicles.

The conclusions and recommendations contained in this report are based upon our understanding of the above design features of the development. We recommend that PGE should be allowed to review the final grades and the actual features after the final construction plans are prepared so that the conclusions and recommendations contained in this report may be re-evaluated and modified, if necessary.

3.0 SCOPE OF SERVICES

The purpose of this study was to evaluate the geotechnical aspects of the proposed development, and to identify and address the geotechnical issues that may impact the proposed site development. The scope of this geotechnical study included field explorations, laboratory testing, geologic literature review, and engineering evaluation of the field and laboratory data. This study also included interpretation of this information to generate pertinent geotechnical recommendations and conclusions that may be used for the design and construction of the development.

The scope of our work did not include any wetland study, or any environmental analysis or evaluation to find the presence of any hazardous or toxic materials in the soil, surface water, groundwater, or air in or around this site.

3.1 Field Investigation

The subsurface conditions of the project site were explored on May 16, 2016, with a total of seven (7) test pits (TP 1 to 7) excavated to depths of about 7 feet below the existing grades. The general vicinity of the exploration areas with the individual test pit locations are shown on the Site & Exploration Plan, Figure 1.

The test pits were completed using a backhoe provided by the client. Test pits were backfilled with loosely compacted excavated soils. The specific number, location, and depth of the test pits were selected in relation to the existing and proposed site features and the purpose of evaluation. The locations of the test pits were selected by Mr. Santanu Mowar of PGE and were plotted on Figure 1. The test pit locations should be considered accurate only to the degree implied by the measuring methods.

A professional geotechnical engineer from our firm observed the excavation works, continually logged the subsurface conditions in the test pits, collected representative bulk samples from different soil layers of the test pits, visually-manually classified the soil samples in the field according to the methods presented in ASTM D-2488-93 (based on the soil samples' density/consistency, moisture condition, grain size, and plasticity estimations) and the 'Key to Exploration Logs' figure in Appendix A, and observed pertinent site features. Samples were designated according to the test pit number and depth, stored in watertight plastic containers, and later on transported to our laboratory for further visual examination and testing.

Results of the field investigation are presented on the Test Pit Log, which is presented in Appendix A. The final exploration log was prepared with our observation and interpretation of the excavation, visual examination of the samples in the field and later on in the laboratory, and the subsequent laboratory test results. The soils were classified according to the methods presented on the

Figure 'Key to Exploration Logs' in Appendix A. This figure also provides a legend explaining the symbols and abbreviations used in the soil exploration log. The soil log indicates the depth where the soils change. It should be noted that the indicated stratification lines on the log represent the approximate boundaries between soil types. The actual transitions of varying soil strata may be more gradual in the field.

3.2 Laboratory Testing

Laboratory tests were conducted on several selected representative soil samples collected from the soil test pits excavated during this study to evaluate the general physical properties and engineering characteristics of the soils encountered. The bulk samples were visually-manually classified in the laboratory following the procedure described in ASTM D-2488-93 (based on the soil samples' density/consistency, moisture condition, grain size, and plasticity estimations), and later on the soil samples' classifications were supplemented by laboratory tests data in accordance with the procedure described in ASTM D-2487-98. Moisture content tests were conducted on selected samples in accordance with ASTM D-2216 procedures. The results of the moisture content tests are presented on the test pit logs in Appendix A. Two (2) sieve analysis tests (grain size distribution analysis tests) were performed on selected samples in accordance with ASTM D-422 procedure. The results of the sieve test results with the USCS classifications of the soils are presented on the grain-size distribution graphs, Figure 1 and 2 enclosed in Appendix B.

3.3 Engineering Evaluation

The results from the field and laboratory tests were evaluated and engineering analyses were performed to develop the design information and the geotechnical engineering recommendations for the following items of the proposed development:

<u>General Site Development & Earthwork & Grading</u>

- Descriptions of the soil and groundwater conditions encountered.
- Grading and earthwork, including site preparation, and fill placement and its compaction.
- Structural fills requirement guidelines.
- Underground utility structure trench backfilling and pipe bedding.
- Site drainage including permanent subsurface drainage systems and temporary groundwater control measures, if necessary.
- Erosion control.
- Potential geologic hazards: landslide, erosion, and seismic.
- Geotechnical special inspection requirements.

Structures

- Foundation types and allowable bearing capacity value for supporting the proposed building structure.
- Estimated settlement for the recommended bearing capacity and observed soil conditions.
- Frictional and passive values for the resistance of lateral forces.
- Slab-on-grade for the proposed building structures.
- Subgrade preparation for slab-on-grade.
- Seismic design considerations, including the site coefficient per 2012 IBC.
- Pavement thickness recommendations for the asphalt pavement section for the proposed driveways and parking areas around the proposed building.

4.0 SURFACE AND SUBSURFACE FEATURES

4.1 Site Location

The proposed commercial development is to be located at 2003 136th Ave. E in Sumner, Pierce County, Washington. The project site is bounded by 136th Avenue East running north-south along the frontage on the west side of the property, an industrial yard along the north, vacant undeveloped parcel along the south, and a railway running north-south along the east side of the property. The site has an access from the 136th Avenue East via a gravel drive way. The general location of the site and the proposed development are shown on the Site & Exploration Plan, Figure 1.

4.2 Site Description

The project site is located within a region dominated by industrial yards with undeveloped parcels. The majority of the subject site is vacant covered with grasses and bushes, and few small scattered trees. There are three existing buildings, and existing concrete and gravel paved areas, which will be removed. The project area is currently vacant and relatively flat. The site has high point of 68 in the east and low point of around 64 in the west. The fluctuation in elevation is minimal and widespread.

4.3 Regional Geology

The site is in the Puget Sound Lowland, a north-south trending structural and topographic depression lying between Olympic Mountains on the west and Cascade Mountains on the east. The lowland depression experienced successive glaciation and nonglaciation activities over the time of Pleistocene period. During the most recent Fraser glaciation, which advanced from and retreated to British Columbia between 13,000 and 20,000 years ago, the lowland depression was buried under about 3,000 feet of continental glacial ice. During the successive glacial and nonglacial intervals, the lowland

depression, which is underlain by Tertiary volcanic and sedimentary bedrock, was filled up above the bedrocks to the present-day land surface with Quaternary sediments, which consisted of Pleistocene glacial and nonglacial sediments. The glacial deposits include concrete-like lodgement till, lacustrine silt, fine sand and clay, advance and recessional outwash composed of sand or sand and gravel, and some glaciomarine materials. The nonglacial deposits include largely fluvial sand and gravel, overback silt and clay deposits, and peat attesting to the sluggish stream environments that were apparently widespread during nonglacial times.

4.4 Soil & Groundwater Conditions

Visual Soil Descriptions

The average thickness of the topsoil in the test pits are about 6 inches, which is composed of slightly moist, loose, dark brown, SILT with roots and organics. The topsoil is then underlain by moist, medium dense, brown SAND with Silt (USCS: SP-SM), which extends upto the top of the black, medium dense, wet SAND (USCS: SP) encountered at approximately 5 feet depth below the existing grades. The SAND extended up to the bottom of the test pits, and may extends further down beyond the bottom of the test pits. Cave-in was noticed within the SAND deposit as soon as the seepage occurred. However, the upper, brown SAND with Silt deposit was remained in intact condition during the cave-in of the SAND, acting as a bridge above the caved-in SAND and the water. The test pits had to terminate at approximately 7 feet depth below the grades due to the on-going conditions of the cave-in of the SAND and the seepage in the test pits.

Groundwater Conditions

Groundwater was encountered in the test pits at approximately 5 feet below the existing grades. No signs of mottling were noticed within the upper, brown sand with silt layer above the black sand deposit. As mentioned above, cave-in of the black sand deposit immediately below the upper sand with silt deposit was noticed in almost each test pit. During the cave-in condition, the upper silt deposit acted like a bridge preventing the further cave-in of the entire test pits.

It is to be noted that seasonal fluctuations in the groundwater elevations may be expected in the amount of rainfall, surface runoff, and other factors not apparent at the time of our exploration. Typically, the groundwater levels rise higher and the seepage flow rates increase during the wet winter months in the Puget Sound area. The possibility of groundwater level fluctuations should be considered when designing and developing the proposed development.

The preceding discussion on the subsurface conditions of the site is intended as a general review to highlight the major subsurface stratification features and material characteristics. For more complete and specific information at individual test pit locations, please review the Test Pit Log included in

Appendix A. The test pit log includes soil descriptions, stratification, and location of the samples and laboratory test data. It should be noted that the stratification lines shown on the test pit log represent the approximate boundaries between various soil strata; actual transitions may be more gradual or more severe. The subsurface conditions depicted in the test pit log are for the test pit locations indicated only, and it should not necessarily be expected that these conditions are representative at other locations of the site.

4.5 Soil Conservation Survey Soil Descriptions

According to the United States Department of Agriculture (USDA) Soil Conservation Survey (SCS) for Pierce County, Washington, the proposed development areas are underlain by the soil unit 'Puyallup fine sandy loam'. Puyallup fine sandy loam is nearly level soil and well drained. It formed in sandy mixed alluvium under trees on the natural levees along the Nisqually and Puyallup Rivers.

A typical soil profile for this category is as follows:

Puyallup Sandy Loam (31)		
Depth, inch	**USDA Texture**	**USCS Soil Definition**
0 – 13	Fine sandy loam	SM
13 - 29	Loamy sand	SM
29 – 60	Fine sandy loam	SM

In general, the above mapped stratigraphy and its USCS classification as per the manual correlate well with the soil profile that was observed during our exploration, and also with the USCS soil descriptions determined from the subsequent laboratory grain size analyses performed on the representative samples. However, the mapped unit may contain inclusions of other soil types or may contain entirely different soil types in areas away from the test pits.

5.0 CONCLUSIONS AND RECOMMENDATIONS

The following sections of this report present detailed recommendations on the pertinent geotechnical issues that are anticipated for the design and construction of the proposed development. These recommendations should be incorporated into the project design, drawings, and specifications.

5.1 General

Based on this study, there are no geotechnical considerations that would preclude the proposed development as planned, therefore, the subject site is considered suitable for the proposed development.

According to the proposed site development plan designed by Larson and Associates, the final building pad grade will be achieved by placing almost 6 feet thick of new structural fill above the current native grade. As per the plan, the existing gravel paved area east of the proposed building pad area will be raised from its current grade to 2 to 4 feet high by placing new fills above the current gravel paved area.

We recommend that the building footings, floor slab, asphalt-paved driveways and the parking spaces, and any other load-bearing structures must be placed on the proposed fill pad to be consisted of new structural fills compacted adequately to firm and unyielding conditions. The fill pad to be placed on the native grades must be prepared as a firm grade showing no signs of pumping and yielding to support the new fill pad and the load-bearing structures above the new fill pad. It should be noted that the proofrolling of the final native subgrades should be achieved to their firm and unyielding conditions to develop a stable and firm final native subgrades to receive the new fills. The new fills to be placed on the final native subgrades must be compacted adequately to a minimum of 95 percent of the fills' laboratory maximum dry density value as determined by ASTM Test Designation D-1557 (Modified Proctor) method. The final native subgrade preparation and the building of the proposed fill pad must be monitored and approved by the on-site geotechnical special inspector during the construction phases of the project.

An allowable bearing capacity of 1500 psf for the new fill pad can be used to design the building footings, and, a modulus of subgrade reaction value of about 150 pounds per cubic inch (pci) can be used to design the slab-on-grade floor.

The existing gravel paved area east of the proposed building pad area will be rebuilt by placing new fills on the existing gravel paved area to achieve the final grades in this area. The existing gravel on the current grade can be remained left in its current state provided the existing gravel grade shows no signs of pumping and yielding under the proofrolling of the current grade. After the proofrolling, and approved by the geotechnical inspector, new fills can be placed above the approved grade. The fills must be compacted adequately as described above for the building fill pad compaction.

5.2 Site Preparation

Preparation of the site should involve clearing, stripping, subgrade proofrolling, and filling. The following paragraphs provide specific recommendations on these issues.

5.2.1 Clearing and Grubbing

Building Pad Area

Initial site preparation for construction of the proposed structures such as the building, slab-on-grade floor, asphalt-paved driveways and parking areas, any other load-bearing structure, and placing new fills on the native grades should include stripping of vegetation and topsoil from the construction areas. Based on the topsoil thickness encountered at our test pit locations, we anticipate topsoil stripping depths of about 6 inches, however, thicker layers of topsoil may be present in unexplored portions of the building site. It should be realized that if the stripping operation takes place during wet winter months, it is typical a greater stripping depth might be necessary to remove the near-surface moisture-sensitive silty soils disturbed during the stripping; therefore, stripping is best performed during dry weather period. Stripped vegetation debris should be removed from the site. Stripped organic topsoils will not be suitable for use as structural fill but may be used for future landscaping purposes.

5.2.2 Subgrade Preparation

Building Pad Area

After the site clearing and site stripping, fill operations can be initiated to establish desired final building pad grades. Any exposed subgrades that are intended to provide direct support for new fills should be adequately proofrolled to evaluate their conditions and to identify the presence of any isolated soft and yielding areas and to verify that stable subgrades are achieved to support the proposed structures, and any new fills. Proofrolling should be done with a loaded dump truck or a front-end loader or a big vibratory roller under the supervision of the on-site geotechnical engineer. If it is found by the on-site geotechnical engineer that the soil is too wet near the subgrade to be proofrolled or it not feasible to proofroll the subgrade, then an alternative method (i,e., visual evaluation and probing with a 1/2-inch diameter steel T-probe) can be used by the geotechnical engineer to identify the presence of any isolated soft and yielding areas and to verify that stable subgrades are achieved to support the proposed new fills.

If any subgrade area are found in soft and moist conditions, ruts and pumps excessively, and cannot be stabilized in place by compaction the affected soils should be over-excavated completely to firm and unyielding suitable bearing materials, and to be replaced with new structural fills to desired final subgrade levels. If the depth of overexcavation to remove unstable soils becomes excessive, a geotextile

fabric, such as Mirafi 500X or equivalent in conjunction with structural fills may be considered to achieve a firm bearing subgrades to support the proposed structures and any new fills.

If needed to stabilize the soft/wet base of an overexcavated area, we recommend to consider a 6 to 12-inch layer of ballast rock or quarry spalls should be placed to form a base on which the structural fill needs to be placed and compacted to achieve the final grade. Ballast rock should meet the requirements for Class B Foundation Material in Section 9-03.17 and quarry spalls should meet the requirements in Section 9-13.6 of the 2014 WSDOT Standard Specifications. The ballast rock or quarry spalls should be pushed into the subgrade with the back of a backhoe bucket or with the use of a large-vibratory steel drummed roller without the use of vibration. Such decision should be made the on-site geotechnical engineer during the actual construction of the project.

The loosely backfilled soils in the areas of exploratory test pits should be overexcavated completely to the firm native soils and backfilled with adequately compacted new structural fills to the final grades. Tree stumps and large root balls should be removed completely and backfilled with new structural fills to the desired subgrade levels.

Variations in the quality and strength of the potential bearing soils in the native grades to support the new fill pad can occur with depth and distance between the test pits. Therefore, careful evaluation of the native bearing materials is recommended at the time of final native subgrade preparation to verify their suitability to support the proposed new fill pad and the structures above the fill pad.

5.2.3 New Structural Fills

Structural fill is defined as non-organic soil, free of deleterious materials, and well-graded and free-draining granular material, with a maximum of 5 percent passing the No. 200 sieve by weight, and not exceeding 6 inches for any individual particle. A typical gradation for structural fill is presented in the following table.

Structural Fill	
U.S. Standard Sieve Size	**Percent Passing by Dry Weight**
3 inch	100
¾ inch	50 –100
No. 4	25 – 65
No. 10	10 – 50
No. 40	0 – 20
No. 200	5 Maximum*

* Based on the ¾ inch fraction.

Other materials may be suitable for use as structural fill provided they are approved by the project geotechnical engineer. Such materials typically used include clean, well-graded sand and gravel (pit-run); clean sand; various mixtures of gravel; crushed rock; controlled-density-fill (CDF, it should meet the requirements in Section 2-09.3(1)E of the 2014 WSDOT Standard Specifications); and lean-mix concrete (LMC). Recycled asphalt, concrete, and glass, which are derived from pulverizing the parent materials are also potentially useful as structural fill in certain applications. These materials must be thoroughly crushed to a size deemed appropriate by the geotechnical engineer (usually less than 2 inches). The structural fills should have a maximum 2 to 3-inch particle diameter.

PGE recommends that the following guidelines may be followed on using proper fill materials to achieve the compaction and the associated design strength for the backfilling areas below the structures. The specifications for each category of fills recommended below are as per the 2014 WSDOT Standard Specifications.

(i) For fills to be placed for constructing foundation subgardes, we recommend that a minimum, the fills should meet the criteria for common borrow (WSDOT 9-03.14(3)). It should be noted that common borrow will be suitable for use as structural fill during dry weather conditions only. If structural fill is placed during wet weather, the structural fill should consist of gravel borrow (WSDOT 9-03.14(1)).

(ii) For general site use, import fill material 'Select Borrow' as per (WSDOT 9-03.14(2) can be used.

5.2.4 Fill Placement and Compaction Requirements

Generally, quarry spalls, controlled density fills (CDF), lean mix concrete (LMC) do not require special placement and compaction procedures. In contrast, clean sand, crushed rock, soil mixtures and recycled materials should be placed under special placement and compaction procedures and specifications described here. Such structural fills under structural elements should be placed in uniform loose lifts not exceeding 12 inches in thickness for heavy compactors and 4 inches for hand held compaction equipment. Each lift should be compacted to a minimum of 95 percent of the soil's laboratory maximum dry density as determined by ASTM Test Designation D-1557 (Modified Proctor) method, or to the applicable minimum City or County standard, whichever is the more conservative. The fill should be moisture conditioned such that its final moisture content at the time of compaction should be at or near (typically within about 2 percent) of its optimum moisture content, as determined by the ASTM method. If the fill materials are on the wet side of optimum, they can be dried by periodic windrowing and aeration or by intermixing lime or cement powder to absorb excess moisture.

In-place density tests should be performed to verify compaction and moisture content of the fill and base material. Each lift of fill or base material should be tested and approved by the soils engineer

prior to placement of subsequent lifts. As a guideline, it is recommended that field density tests be performed at the following frequency to determine that the compacted fills achieved the required compaction. At least one (1) density test per 2000 square feet of surface area of the compacted and paved areas fill pad areas and paved areas for each one-foot lift of fill.

If field density tests indicate that the last lift of compacted fills has not been achieved the required percent of compaction or the surface is pumping and weaving under loading, then the fill should be scarified, moisture-conditioned to near optimum moisture content, re-compacted, and re-tested prior to placing additional lifts.

5.2.5 Settlement Monitoring

We recommend that a settlement monitoring program should be implemented in the building site during the proofrolling of the native subgrades and the construction of the new fill pad to observe if any excessive settlement is taking place during these activities. The settlement monitoring should be started prior to beginning of the native subgrade proofrolling and the new fill placement. The monitoring frequency should be determined based on the previous day monitoring result.

5.2.6 Permanent Fill Pad Slopes

For permanent newly constructed fill pad, the side slopes should be laid back at a minimum slope inclination of 3:1 or greater, depending on the soils to be encountered in any particular area of the site. The new fill pad should extend beyond the limits of the load bearing area of the fill pad for a minimum of 5 feet of horizontal distance.

Where the above slopes are not feasible, protective facings and/or retaining structures should be considered. Permanent slopes should be re-vegetated as soon as practical to reduce the surface erosion and sloughing. Temporary erosion protection described later on in Section 5.7.1, 'Erosion Hazard' of this report should be used until permanent protection is established.

5.2.7 Site Drainage

Surface Drainage

The final site grades must be such that surface runoff will flow by gravity away from the structures, and should be directed to suitable collection points. We recommend providing a minimum drainage gradient of about 3% for a minimum distance of about 10 feet from the building perimeter. A combination of using positive site surface drainage and capping of the building surroundings by concrete, asphalt, or low permeability silty soils will help minimize or preclude surface water infiltration around the

perimeter of the buildings and beneath the floor slabs. Paved areas should be graded to direct runoff to catch basins and or other collection facilities. Collected water should be directed to the on-site drainage facilities by means of properly sized smooth walled PVC pipe. Interceptor ditches or trenches or low earthen berms should be installed along the upgrade perimeters of the site to prevent surface water runoff from precipitation or other sources entering the site. Surface water collection facilities should be designed by a professional civil engineer.

Footing Excavation Drain

Water must not be allowed to pond in the foundation excavations or on prepared subgrades either during or after construction. If due to the seasonal fluctuations, groundwater seepage is encountered within footing depths, we recommend that the bottom of excavation should be sloped toward one corner to facilitate removal of any collected rainwater, groundwater, or surface runoff, and then direct the water to ditches, and to collect it in prepared sump pits from which the water can be pumped and discharged into an approved storm drainage system.

Footing Drain

Footing drains should be used where (1) crawl spaces or basements will be below a structure, (2) a slab below the outside grade, and (3) the outside grade does not slope downward from a building. The drains must be laid with a gradient sufficient to promote positive flow to a controlled point of approved discharge. The foundation drains should be tightlined separately from the roof drains to this discharge point. Footing drains should consist of at least 4-inch diameter perforated PVC pipe. The pipe should be placed in a free-draining sand and gravel backfill. Either the pipe or the pipe and free-draining backfill should be wrapped in a non-woven geotextile filter fabric to limit the ingress of fines. Cleanouts should be provided. The drains should be located along the outside perimeter of the spread footings.

Downspout or Roof Drain

These should be installed once the building roof in place. They should discharge in tightlines to a positive, permanent drain system. Under no circumstances connect these tightlines to the perimeter footing drains.

5.2.8 Utility Support and Backfill

Based on the soils encountered at the site within the exploration depths, the upper, brown, medium dense, silty soils appear to be adequate for supporting utility lines; provided the utility lines maintain a minimum of 3 feet of separation between the bottom of the utility lines and the cave-in depth and the seepage depth. The utility lines' final bottom grades must be consisted of a firm and unyielding grade that will provide adequate support for the utility lines. A major concern with utility lines is

generally related to the settlement of trench backfill along utility alignments and pavements. Therefore, it is important that each section of utility be adequately supported on proper bedding material, the utility trench be properly backfilled, and the backfilling must be adequately compacted to firm and unyielding conditions.

It is recommend that utility trenching, installation, and backfilling conform to all applicable Federal, State, and local regulations such as WISHA and OSHA for open excavations. Utility bedding should be placed in accordance with manufacturer's recommendations and local ordinances. Bedding material for rigid and flexible pipe should conform to Sections 9-03.15 and 9-03.16, respectively, of the 2014 WSDOT/APWA (American Public Works Association) Standard Specifications for Road, Bridge, and Municipal Construction. For site utilities located within the Pierce County right-of-ways, bedding and backfill should be completed in accordance with the Pierce County specifications. As a minimum, 5/8 inch pea gravel or clean sand may be used for bedding and backfill materials. The bedding materials should be hand tamped to ensure support is provided around the pipe haunches. Trench backfill should be carefully placed and hand tamped to about 12 inches above the crown of the pipe before any heavy compaction equipment is brought into use. The remainder of the trench backfill should be compacted to 90 percent of the maximum dry density per ASTM Test Designation D-1557 (Modified Proctor) except for the uppermost 18 inches of backfill which should be compacted to 95 percent of the maximum dry density per ASTM Test Designation D-1557 (Modified Proctor). The backfill should be placed in lifts not exceeding 4 inches if compacted with hand-operated equipment or 8 inches if compacted with heavy equipment. Catch basins, utility vaults, and other structures installed flush with the pavement should be designed and constructed to transfer wheel loads to the base of the structure.

The utility trenches should not be left open for extended periods to prevent water entry and softening of the subgrade. Should soft soils be encountered at the bottom of the trench, it should be overexcavated and replaced with select fills. As an alternative to undercutting, a Geotextile fabric or crushed rock may be used to stabilize the trench subgrade. Where water is encountered in the trench excavations, it should be removed prior to fill placement. Alternatively, quarry spalls or pea gravel could be used below the water level if allowed in the project specifications.

5.2.9 Construction Monitoring

Problems associated with earthwork and construction can be avoided or corrected during the progress of the construction if proper inspection and testing services are provided. It is recommended that site preparation activities including but not limited to stripping, cut and filling, final subgrade preparation for foundation, floor slab, and pavement be monitored by a geotechnical inspector from our firm.

5.3 Building Foundation Recommendations

Spread Footing

Based on the proposed development plan of achieving the final building pad grade by raising the native grades by almost 6 feet thick fill pad, it is our opinion that the foundations of the proposed building should be supported on conventional shallow spread footings. The footings should be supported on the new fills to be placed above the 'competent' native subgrade soils. The 'competent' native subgrade is described as the native soil unit that must be compacted and proofrolled adequately (as the procedures described earlier in Section 5.2.2, 'Subgrade Preparation' of this report) to firm and unyielding conditions prior to placing new fills above the native subgrade. For the design of shallow footing foundation supported on the properly compacted structural fills (as described earlier in Section 5.2.4, 'Fill Placement and Compaction Requirements' of this report), we recommend using a maximum net allowable bearing capacity of 1,500 pounds per square foot (psf). The purpose of using a lower bearing capacity value is to avoid the possibility of any excessive settlement of the caved-in black sand layer encountered at the groundwater seepage level. In our engineering opinion, if the allowable bearing capacity value can be used as recommended then the building settlement can be kept within the tolerable limit. The combination of the adequately compacted 6 feet thick of fill pad and approximately 5 feet thick of upper, medium dense, brown sand with silt deposit is expected to be able to provide the above recommended bearing capacity value. For short-term loads, such as wind and seismic, a 1/3 increase in this allowable capacity can be used. We recommend that continuous footings have a minimum width of 18 inches and individual column footings a minimum width of 24 inches. All exterior footings should bear at least 18 inches below the final adjacent finish grade to provide adequate confinement of the bearing materials and frost protection.

Given the soil and groundwater conditions encountered and based on the use of lower bearing capacity value, we anticipate that the properly designed and constructed foundations supported on the proposed fill pad should experience total and differential settlements of less than 1 inch and 1/2 inch, respectively. The majority of these settlements are expected to occur during construction. This estimation was done without the aid of any laboratory consolidation test data, but on the basis of our experience with similar types of structures and subsoil conditions.

Lateral foundation loads can be resisted by friction between the foundation base and the supporting soil, and by passive earth pressure acting on the face of the embedded portion of the foundation. For frictional resistance, a coefficient of 0.35 can be used. For passive earth pressure, the available resistance can be computed using an equivalent fluid pressure of 320 pcf, which includes a factor of safety of 1.5. This value assumes the foundation must be poured "neat" against the undisturbed native soils or structural fill placed and compacted as described earlier in Section 5.2.4, 'Fill Placement and Compaction Requirements' of this report.

Alternate Deep Foundation Option

We recommend that if the lower allowable bearing capacity value is not a feasible option to design the building footings, and if any excessive settlement is noticed during the final native subgrade proofrolling, and/or during or after the fill placement and compaction then alternatively, a deep foundation option such as drilled piers or auger-cast piles should be considered to support the building structure. Due to the presence of the water and the cave-in conditions, we expect that the piers or the piles may require casing. Further soil investigation including drilling some deeper test bore holes will be required to determine the soil conditions below the test pit depths. A contingency plan should be kept in-place by the owner if excessive settlement of the native subgrades and the new fill pad are noticed during their constructions.

5.4 Slab-on-grade Floor For Building Structure

The proposed slab-on-grade floor for the proposed building can bear on adequately compacted new structural fill pad to be placed above the native subgrades prepared as described earlier in Section 5.1 and 5.2 of this report. After the final fill subgrade preparation is completed, the slab should be provided with a capillary break to retard the upward wicking of ground moisture beneath the floor slab. The capillary break would consist of a minimum of 6-inch thick clean, free-draining sand or pea gravel. The structural fill requirements specified in Section 5.2.6, Structural Fills, could be used as capillary break materials except that there should be no more than 2 percent of fines passing the no. 200 sieve. Alternatively, 'Gravel Backfill for Drains' per 2014 WSDOT Standard Specifications 9-03.12(4) can be used as capillary break materials. Where moisture by vapor transmission is undesirable, we recommend the use of a vapor barrier such as a layer of durable plastic sheeting (such as Crossstuff, Moistop, or Visqueen) between the capillary break and the floor slab to prevent the upward migration of ground moisture vapors through the slab. During the casting of the slab, care should be taken to avoid puncturing the vapor barrier. At owner's or architecture's discretion, the membrane may be covered with 2 inches of clean, moist sand as a 'curing course' to guard against damage during construction and to facilitate uniform curing of the overlying concrete slab. The addition of 2 inches of sand over the vapor barrier is a non-structural recommendation. Based on the subgrade preparation as described in Section 5.1 and 5.2 of this report, a modulus of subgrade reaction value of about 150 pounds per cubic inch (pci) can be used to estimate slab deflections, which could arise due to elastic compression of the subgrades.

5.5 Pavement Thickness (Building Pad Area)

A properly prepared subgrade is very important for the life and performance of the driveway pavements. Therefore, we recommend that all driveway and pavement areas be prepared as described in Section 5.1 and 5.2 of this report. Subgrades should either be comprised of adequately proofrolled competent undisturbed native soils, or be comprised of a minimum of one foot of granular structural fill that is compacted adequately. The structural fill should be compacted to 95 percent of the maximum dry

density as determined by Modified Proctor (ASTM Test Designation D-1557). It is possible that some localized areas of yielding and weak subgrade may still exist after this process. If such conditions occur, crushed rock or other qualified materials as addressed in Section 5.2.6 may be used to stabilize these localized areas.

We assumed that the traffic would mostly consist of passenger cars and occasional waste management trucks in the building pad area. Two types of pavement sections may be considered for such traffic, the minimum thicknesses of which are as follows:

- 2 inches of Asphalt Concrete (AC) over 2 inches of Crushed Surface Top Course (CSTC) over a 6 inches of Granular Subbase (CRB), or
- 2 inches of Asphalt Concrete (AC) over 3 inches of Asphalt Treated Base (ATB) material.

A greater asphalt thickness will be required in the driveway areas where larger commercial trucks and vehicles are expected, which is as follow.

- 3 inches of Asphalt Concrete (AC) over 2 inches of Crushed Surface Top Course (CSTC) over a 6 inches of Granular Subbase (CRB), or
- 3 inches of Asphalt Concrete (AC) over 4.5 inches of Asphalt Treated Base (ATB) material.

The 2014 Standard Specifications for Washington State Department of Transportation (WSDOT) and American Public Works Association (APWA) should be applicable to our recommendations that aggregate for AC should meet the Class-B grading requirements as specified in 9-03.8(6). For the Crushed Surfacing Top Course (CSTC), we recommend using imported, clean, crushed rock per WSDOT Standard Specifications 9-03.9(3). For the sub base course, we recommend using imported, clean, well-graded sand and gravel, such as Ballast or Gravel Borrow per WSDOT Standard Specifications 9-03.9(1) and 9-03.14, respectively. For the asphalt treated base course (ATB) the aggregate should be consistent with WSDOT Standard Specifications 9-03.6 (2).

Long-term performance of the pavement will depend on its surface drainage. A poorly-drained pavement section will deteriorate faster due to the infiltration of surface water into the subgrade soils, thereby reducing their supporting capability. Therefore, we recommend using a minimum surfacing drainage gradient of about 1% to minimize this problem and to enhance the pavement performance. Also, regular maintenance of the pavement must be considered.

5.6 Geologic Hazards

5.6.1 Erosion Hazard

Uncontrolled surface water with runoff over unprotected site surfaces during construction activities is considered the single most important factor that impacts the erosion potential of a site. The

erosion process may be accelerated significantly when factors such as soils with high fines, sloped surface, and wet weather combines together. Taking into consideration of the combination of the factors like the high fines content in the near surface silty soils, the project site is likely to experience some impact due to the erosion during the wet winter months.

The erosion hazard can be mitigated if the mass grading activities and the earthwork can be completed within the dry summer period. Also, measurements such as the control of surface water must be maintained during construction, and a temporary erosion and sedimentary control (TESC) plan, as a part of the Best Management Practices (BMP) must be developed and implemented as well. The TESC plan should include the use of geotextile barriers (silt fences) along any down-slope, straw bales to de-energize downward flow, controlled surface grading, limited work areas, equipment washing, storm drain inlet protection, and sediment traps. Also, vegetation clearing must be kept very limited in this site to reduce the exposed surface areas. A permanent erosion control plan is to be implemented following the completion of the construction. Permanent erosion control measurements such as establishment of landscaping, control of downspouts and surface drains, control of sheet flow over the final slope grades, prevention of discharging water over the final slopes and at the toe of the slope are to be implemented following the completion of the construction.

5.6.2 Seismic Design Parameters

Structural design of the proposed building at the project site should follow 2012 International Building Code (IBC) standards. Based on our evaluations of the subsurface conditions, Site Class E from Table 1613.5.2 of IBC should be used for design. we interpret the underlying bearing soils to correspond to 'C', which refers to very dense soils.

5.6.3 Seismically Induced Geotechnical Hazards

As a part of the seismic evaluation of the site, the liquefaction potential of the site was evaluated. Liquefaction is a phenomenon, which takes place due to the reduction or complete loss of soil strength due to increased pore water pressure during a major earthquake event. Liquefaction primarily affects geologically recent deposits of fine-grained sands that are below the groundwater table.

Based on the existing soil conditions explored during this study, our regional experience, and our knowledge of local seismicity, the potentials for the seismic hazards such as the liquefaction potential in this site and the associated hazards to the proposed building structure is considered very low to moderate depending on the level of earthquake magnitude that can takes place during the design life of the development.

A major earthquake event (0.35g) is considered as one with a 10 percent probability of exceedance, which if occurs, the proposed building might be expected to show some structural damages, but not collapse. If the horizontal accelerations exceeds 0.35g during a very large earthquake event then the building can experience severe damages. A minor earthquake event (0.15g) is considered as one with a 50 percent probability of exceedance during a 50-year design life, which is similar to the 2001 Nisqually earthquake, for which the building can survive with little damages.

Our liquefaction potential evaluation indicates that the possibility of occurrence of liquefaction in this site is almost nil during any minor earthquake event (0.15g). The combination of the factors like the 6 feet thick of adequately compacted new structural fill pad and the presence of almost 5 feet of upper, medium dense, sand with silt deposit, and the presence of water table at approximately 11 feet below the final pad grade the liquefaction potential in the building site is estimated to be minimum during an minor earthquake event (0.15g).

5.6.4 Landslide Hazard

In absence of any slope within the proposed development area the subject site is not considered to be potential for any landslide hazard.

6.0 GEOTECHNICAL SPECIAL INSPECTIONS

Pacific Geo Engineering (PGE) recommends that the following geotechnical special inspection services to be performed during the construction of the proposed development. According to PGE, the following items should be considered as a minimum but not limited to.

- A professional geotechnical engineer should be retained to provide geotechnical consultation, material testing, and construction monitoring services during the construction of the project.
- A pre-construction meeting should be held on-site to discuss the geotechnical aspects of the development and the special inspection services to be performed during the construction.
- The site preparation activities including but not limited to stripping, cut and filling, final subgrade preparation for foundation, floor slab, and pavement be monitored by a geotechnical engineer or his representative under the engineer's supervision.
- A list of the possible items that require special geotechnical inspection and approval by the geotechnical engineer is as follows:

  (i) Stripping of topsoils.
  (ii) Removal of unsuitable soils.
  (iii) Compaction and proofrolling of any exposed subgrades that are intended to rovide direct support for new construction and/or require new fills.

(iv) Any structural fills to be used in this site, and structural fills placement and its compaction.
(v) The temporary or permanent excavation inclinations and excavation stability.
(vi) The footing bearing materials, bearing capacity value, and the embedment depth of the footings prior to placing forms and rebars.
(vii) Subgrade preparation for soil supported slab-on-grade floors.
(viii) Subgrade preparation for driveways and pavements.
(ix) The compaction of the CSBC, CSTC, and the asphalt layers in driveways and pavements.
(xi) The installation of drainage systems such as footing excavation drain and footing drain, and daylighting of such drains and downspout or roof drains.
(xii) Bedding and the backfilling materials, and backfilling of utility lines.
(xiii) Buffer distances from the vegetation clearing limit and the vegetation clearing limit.
(xiv) The installation and functioning of the temporary and permanent erosion and sedimentation control plan.
(xv) The development consideration and construction limitations mentioned in this report.
(xvii) Any other items specified in the approved project plans to be prepared by other consultants relevant to the geotechnical aspect of the project.

7.0 ADDITIONAL SERVICES

Additional services described below can be performed by PGE in the event the project requires such services. These services will be performed upon written authorization of the client or the civil engineer, and with additional cost to perform such services.

7.1 Design Phase Engineering Services

- Review of final plans.

The above scope of services can be provided by PGE under a separate contract with the owner.

7.2 Construction-time Testing and Inspection

As the geotechnical engineer of record for the proposed development, we recommend that PGE should be retained to perform a review of the project plans and specifications to verify that the geotechnical recommendations of this report have been properly interpreted and incorporated into the project design and specifications. PGE should also be retained to provide geotechnical consultation, material testing, and construction monitoring services during the construction of the project described earlier in Section 6.0 of this report. These services are important for the project to confirm that the earthwork and the general site development are in compliance with the general intent of design concepts, specifications, and the geotechnical recommendations presented in this report. Also, participation of PGE

during the construction will help PGE engineers to make on-site engineering decisions in the event that any variations in subsurface conditions are encountered or any revisions in design and plan are made.

PGE can assist the owner before construction begins to develop an appropriate monitoring and testing plan to aid in accomplishing a fast and cost-effective construction process.

8.0 REPORT LIMITATIONS

The evaluation and recommendations presented in this report are based upon the information available from our subsurface explorations, and the project details furnished by the client. The study was performed using a mutually agreed-upon scope of work, which is presented in this report.

It should be noted that PGE cannot take the responsibility regarding the accuracy of the information available from other consultant. If any of the information considered during this study is not correct or if there are any revisions to the plans for this project, PGE should be notified immediately of such information and the revisions so that necessary amendment of our geotechnical recommendations can be made. If such information and revisions are not notified to PGE, no responsibility should be implied on PGE for the impact of such information and the revisions on the project.

Variations in soil and groundwater conditions may exist between the locations of the explorations and the actual conditions underlying the site. The nature and the extent of variations in soil and groundwater conditions may not be evident until construction occurs. If any soil and groundwater conditions are encountered at the site that are different from those described in this report, we should be notified immediately to review the applicability of our recommendations if there are any changes in the project scope.

This report may be used only by the client and for the purposes stated, within a reasonable time from its issuance. Land use, site conditions (both off and on-site), or others factors including advances in our understanding of applied science, may change over time and could materially affect our findings. Therefore, this report should not be relied upon after 24 months from its issuance. PGE should be notified if the project is delayed by more than 24 months from the date of this report so that we may review to determine that the conclusions and recommendations of this report remain applicable to the changed conditions.

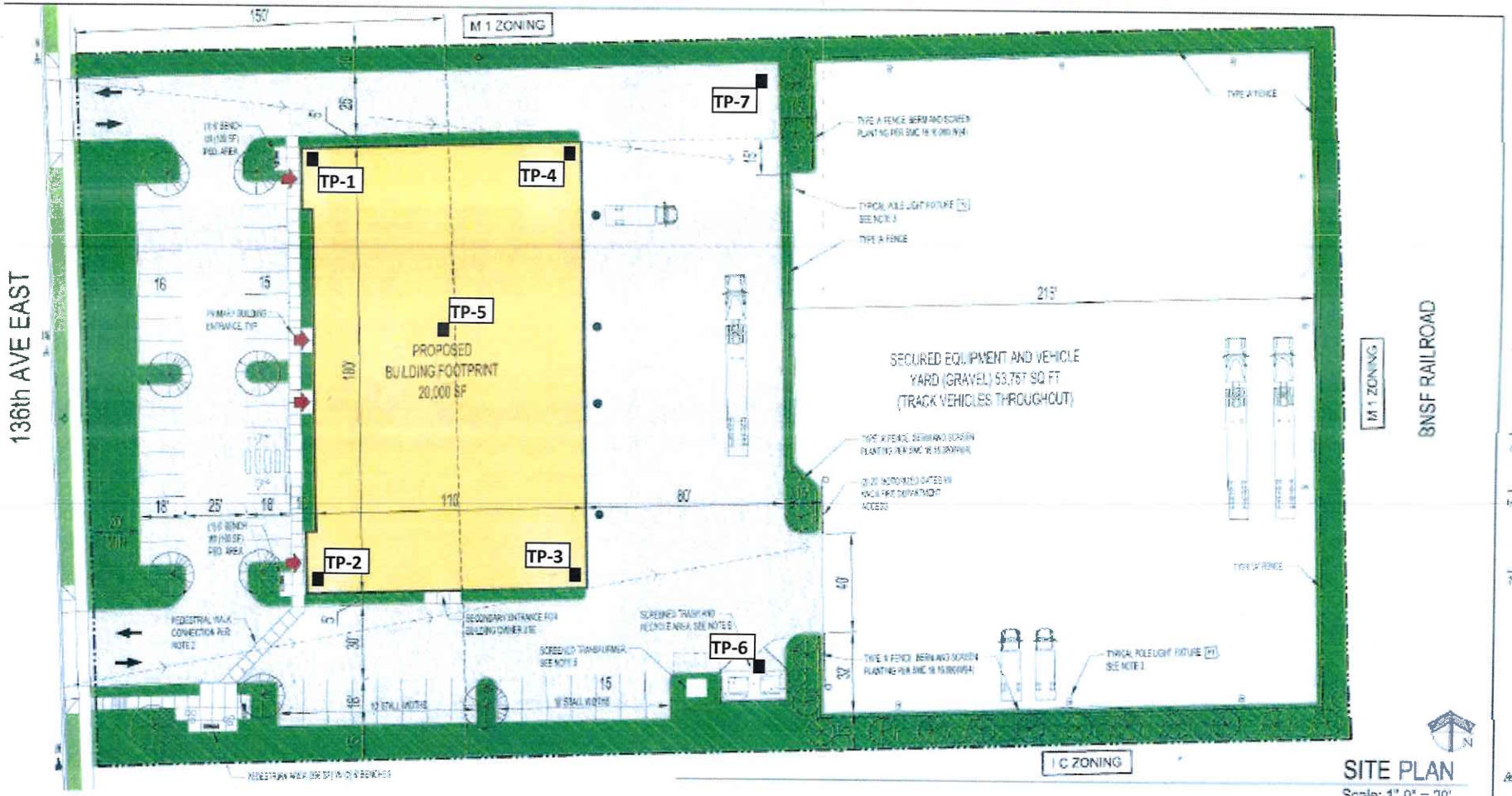
The scope of our work does not include services related to construction safety precautions. Our recommendations are not intended to direct the contractors' method, techniques, sequences or procedures, except as specifically described in our report for consideration in design. Additionally, the scope of our work specifically excludes the assessment of environmental characteristics, particularly those involving hazardous substances.

This report including its evaluation, conclusions, specifications, recommendations, or professional advice has been prepared for planning and design purposes for specific application to the proposed project in accordance with the generally accepted professional geotechnical engineering practices in the local areas at the time this report was written. No warranty, express or implied, is made.

This report is the property of our client, and has been prepared for the exclusive use of our client and its authorized representatives for the specific application to the proposed development at the subject site in Sumner, Washington.

It is the client's responsibility to see that all parties to this project, including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety. The use of information contained in this report for bidding purposes should be done at the contractor's option and risk. Any party other than the client who wishes to use this report shall notify PGE of such intended use and for permission to copy this report. Based on the intended use of the report, PGE may require that additional work be performed and that and updated report be reissued. Noncompliance with any of these requirements will release PGE from any liability resulting from the use of this report.

If there is a substantial lapse of time between the submission of this report and the start of the proposed construction work, or if the present conditions of the site changes during the lapsed time due to natural causes or construction activity at or adjacent to the site, it is recommended that this report be reviewed to determine that the conclusions and recommendations of this report remain applicable to the changed conditions.



SITE PLAN

Scale: 1"-0" = 20'

Project Information

Site Area =	3.34 Acres (146,308 SF)
Parcel(s) =	44064-79-001
Zoning =	IC: Interchange Commercial
Building Footprint Area =	20,000 SF
Building Mezzanine Floor Area =	4,600 SF
Max Outside Storage Yard, 45% of lot area =	65,823 SF
Type of Construction =	5B
Fire Protection =	Sprinklers Throughout

Parking Requirements:

Office Area (Main Floor) 2,700 SF at 2.5/1,000 SF =	20
Warehouse Area (Main Floor) =	6
Office Area (2nd Floor) 4,600 SF at 2.5/1,000 SF =	12
Minimum Stalls Required =	38
Stalls Provided =	60
Max Increase to allowable stalls is 25% =	48
Bike Stalls Required = 38 x 0.10 (2%)	4

Site Notes

1. All parking stalls depicted are min 9'x18', stalls where indicated are are 10'x20'. Minimum stall quantity requirements per SMC 18.40
2. 5' min width concrete pedestrian sidewalk connection placed flush in asphalt paving (contrasting surfaces).
3. All exterior lighting shall be fitted with shielding to confine lighting to project site.
4. Lighting shall be provided throughout the site to fulfill the site and pedestrian illumination requirements per Sumner DG2.2.6 and DG2.5.4.
5. All utilities and rooftop HVAC shall be screened from the right of way. This includes screening and placement coordination of utility purveyor transformers, meters, etc.
6. Provide screening at Trash and Recycle per Sumner DG2.1.8. Currently proposed as Type 'A' fence and landscape screening.
7. See Civil drawings for additional site dimensions and horizontal control.

Site Feature Legend

- Bike rack per SMC 18.42.065. Provide bike parking for 10% of required auto stalls, (4 bike required, 4 bike provided). Bike parking to be provided under future tenant improvements.
- Single fixture light on 25' pole. See note 3 and 6.
- Dual fixture light on 25' pole. See note 3 and 6.
- Primary Building Entrance, see exterior elevations
- (4) 8' Receptacles proposed in pedestrian oriented space.
- Air-Grade Overhead Door
- Berming at landscape areas as required to fulfill buffer requirements.

TYPE 'A' FENCE = 6ft tall chain link fence, black vinyl coated with plastic slat inserts. Color of slats shall be black.

TYPE 'B' FENCE = 4ft tall chain link fence, black vinyl coated.

Pedestrian Oriented Space Calculations

Lot area = 3.36 AC (146,308 sf - 20,000 sf bldg. footprint)

Required lot space area - 126,308 x 0.02 = 2,526 sf

Building area: 20,000 sf footprint

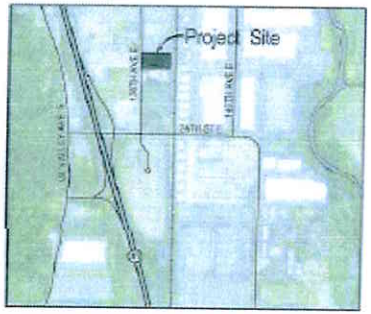
Required building area: 20,000 x 0.01 = 200 sf

Total required pedestrian oriented space = 2,726 sf

Note: The pedestrian space calculation is not appropriate for a site of this size. We are seeking a calculation of Building area at a rate of 0.02 (400 sf) which is reasonable for this type of use.

Total provided pedestrian oriented space = 456 sf

Required seating area per 60 sf of 456 = 7.6
(Seat area shall be 3 feet or one individual seat)
Provided as (4) 6' long benches



VICINITY MAP

N.T.S.

architects
950 Pacific Avenue, Suite 450
Tacoma, WA 98402
253-572-4803

Project Title
Johansen Excavating Inc.
Sumner Facility

Client
Prestige Worldwide Holdings, LLC.
P.O. Box 674
Buckley, WA 98321
360-500-4493

Contractor

Job Number
16-123

Issue Set and Date
Issued for Design Review
March 23, 2016

NOT FOR CONSTRUCTION

REGISTERED ARCHITECT
JONES C. TSANG
STATE OF WASHINGTON

Revisions

Sheet Title
Site Plan

Designed by: S Kubaly
Drawn by: S Miller
Checked by: J Tsang

Sheet Number
A0.1
1 of 1 Sheets

Figure 1

PRESTIGE WORLDWIDE HOLDINGS LLC.

PORTIONS OF NW 1/4, AND SW 1/4, SEC.12, TWN.20 N., RNG. 4 E., W.M.

TESC AND GRADING PLAN

COVER SHEET

VICINITY MAP

N.T.S.

SITE ADDRESS

2003 136TH AVE E.
SUMNER, WA 98390

PARCEL NUMBERS

4495401322, 4495401330, 4495401321

SHEET INDEX

1 OF 4	CVR	COVER AND STORM
2 OF 4	TP1	T.E.S.C. & GRADING PLAN
3 OF 4	TP2	T.E.S.C. DETAILS & SPECIFICATIONS
4 OF 4	TP3	T.E.S.C. DETAILS

SITE DATA

ACREAGE: 3.36

ZONING: INTERCHANGE COMMERCIAL "IC"

PROPOSED USE: STAGING AREA

ANTICIPATED CUT: 2,062 CU. YD.

ANTICIPATED FILL: 16,065 CU. YD.

LAND DESCRIPTION

PARCEL A:

LOT 5, BLOCK 69, C.D. HILLMAN'S PACIFIC CITY DIVISION NO. 4, ACCORDING TO THE PLAT THEREOF RECORDED IN VOLUME 8 OF PLATS, PAGE 36, RECORDS OF PIERCE COUNTY, WASHINGTON, EXCEPT THE NORTH 200 FEET OF THE WEST 100 FEET THEREOF,
AND EXCEPT THE WEST 110 FEET OF THE SOUTH 88 FEET THEREOF, IN PIERCE COUNTY, WASHINGTON.

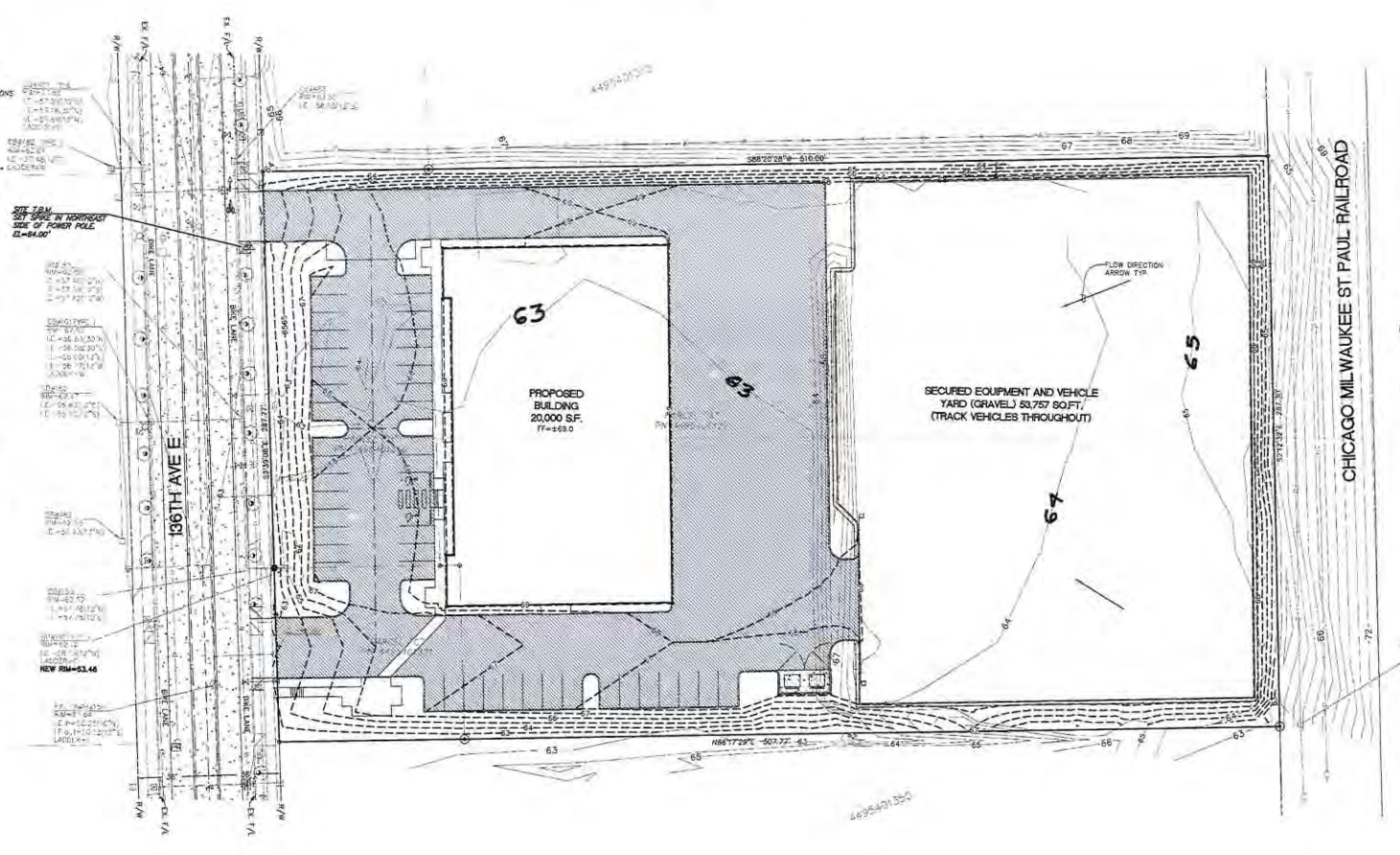
PARCEL B:

THE NORTH 200 FEET OF THE WEST 100 FEET OF LOT 5, BLOCK 69, C.D. HILLMAN'S PACIFIC CITY DIVISION NO. 4, ACCORDING TO THE PLAT THEREOF RECORDED IN VOLUME 8 OF PLATS, PAGE 36, RECORDS OF PIERCE COUNTY, WASHINGTON, EXCEPT THAT PORTION CONVEYED TO THE CITY OF SUMNER UNDER RECORDING NO. 201312180246 BEING A RE-RECORD OF RECORDING NO. 201306270451.

PARCEL C:

THE WEST 110 FEET OF THE SOUTH 88 FEET OF LOT 5, BLOCK 69, C.D. HILLMAN'S PACIFIC CITY DIVISION NO. 4, ACCORDING TO THE PLAT THEREOF RECORDED IN VOLUME 8 OF PLATS, PAGE 36, RECORDS OF PIERCE COUNTY, WASHINGTON, EXCEPT THAT PORTION CONVEYED TO THE CITY OF SUMNER UNDER RECORDING NO. 201312180246 BEING A RE-RECORD OF RECORDING NO. 201306270451.

PIERCE COUNTY, WASHINGTON



SCALE: 1"=30'

VERTICAL DATUM
PIERCE COUNTY

NAVD 88 ESTABLISHED USING GPS RTK ROVER CONSTRAINED TO PIERCE COUNTY CONTINUOUSLY OPERATING REFERENCE STATIONS (CORS)

CONTOUR INTERVAL=1'

TOPOGRAPHY PREPARED BY LARSON & ASSOCIATES

LEGEND

- PROPERTY LINE
- R/W LINE
- EASEMENT LINE
- ADDITIONAL 10' UTILITIES EASEMENT
- PROPOSED SANITARY SEWER MANHOLE
- PROPOSED SEWER MAIN
- PROPOSED SANITARY SEWER STUB
- PROPOSED STORM DRAIN MANHOLE
- PROPOSED STORM CATCH BASIN
- PROPOSED STORM MAIN
- PROPOSED STORM STUB
- PROPOSED SIDEWALK
- PROPOSED SHOULDER
- 456 PROPOSED CONTOUR
- 456 EXISTING CONTOUR
- EXISTING EDGE OF PAVEMENT
- EXISTING STORM MANHOLE
- EXISTING STORM MAIN
- EXISTING SANITARY SEWER MANHOLE
- EXISTING SANITARY SEWER MAIN
- EXISTING CATCH BASIN
- EXISTING POWER POLE
- EXISTING WATER GATE VALVE
- EXISTING FIRE HYDRANT
- EXISTING WATER MAIN
- EXISTING TELEPHONE RISER
- EXISTING POWER J-BOX
- EXISTING CABLE J-BOX
- EXISTING WATER VAULT
- SITE TEMPORARY BENCH MARK
- EXISTING STREET TREE

UTILITY CONFLICT NOTE:

CAUTION:

THE CONTRACTOR SHALL BE FULLY RESPONSIBLE FOR VERIFYING THE LOCATION, AND DEPTH OF ALL EXISTING UTILITIES WHETHER SHOWN ON THESE PLANS OR NOT BY POTHOLING THE UTILITIES AND SURVEYING THE HORIZONTAL AND VERTICAL LOCATION PRIOR TO CONSTRUCTION. THIS SHALL INCLUDE CALLING UTILITY LOCATE @ 811 AND THEN POTHOLING ALL OF THE EXISTING UTILITIES AT LOCATIONS OF NEW UTILITY CROSSINGS TO PHYSICALLY VERIFY WHETHER OR NOT CONFLICTS EXIST. LOCATION OF SAID UTILITIES AS SHOWN ON THESE PLANS ARE BASED UPON THE UNVERIFIED PUBLIC INFORMATION AND ARE SUBJECT TO VARIATION. IF CONFLICTS SHOULD OCCUR, THE CONTRACTOR SHALL CONSULT LARSON & ASSOCIATES, INC. TO RESOLVE ALL PROBLEMS PRIOR TO PROCEEDING WITH CONSTRUCTION.

CONTRACTOR / DEVELOPER NOTE:

THE CONTRACTOR AND/OR DEVELOPER SHALL BE RESPONSIBLE FOR COORDINATING ALL REQUIRED MATERIAL TESTING, COMPACTION TESTING, AND APPLICABLE INSPECTIONS AS REQUIRED BY PIERCE COUNTY AND PRIVATE ENGINEER. THE CONTRACTOR SHALL SUPPLY CERTIFYING ENGINEER WITH DOCUMENTATION SIGNED BY A PROFESSIONAL SOILS AND/OR MATERIALS ENGINEER SHOWING THAT THE ROAD SECTION WAS BUILT ACCORDING TO THE RECOMMENDATIONS OF THE GEOTECHNICAL REPORT. THE BACKFILL MATERIAL MEETS MINIMUM COUNTY/STATE REQUIREMENTS, COMPACTION WAS ACHIEVED IN ALL TRENCHES AND ROAD SECTION, AND ALL CONSTRUCTION MATERIALS AND CONSTRUCTION METHODS SHOWN ON THESE PLANS HAVE BEEN FOLLOWED.

REVISION BLOCK

NO.	DATE	DESCRIPTION	BY

JOB NUMBER 8433

SCALE HOR. 1"=30' VERT. N/A

DESIGNED CJM DRAWN FJM CHECKED CJM

PROPONENT: JACOB DIMMER, PRESTIGE EXCAVATING INC. P.O. BOX 674 BUCKLEY, WA. 98321 PH: (360)829-6493

LARSON and ASSOCIATES
Land Surveyors & Engineers, Inc.
4401 So. 66th St. TACOMA, WA. 98409 (253) 474-3404

TESC AND GRADING PLAN
COVER SHEET

DATE 6-3-16

DRAWING NO. 8433BASE

SHEET 1 OF 4

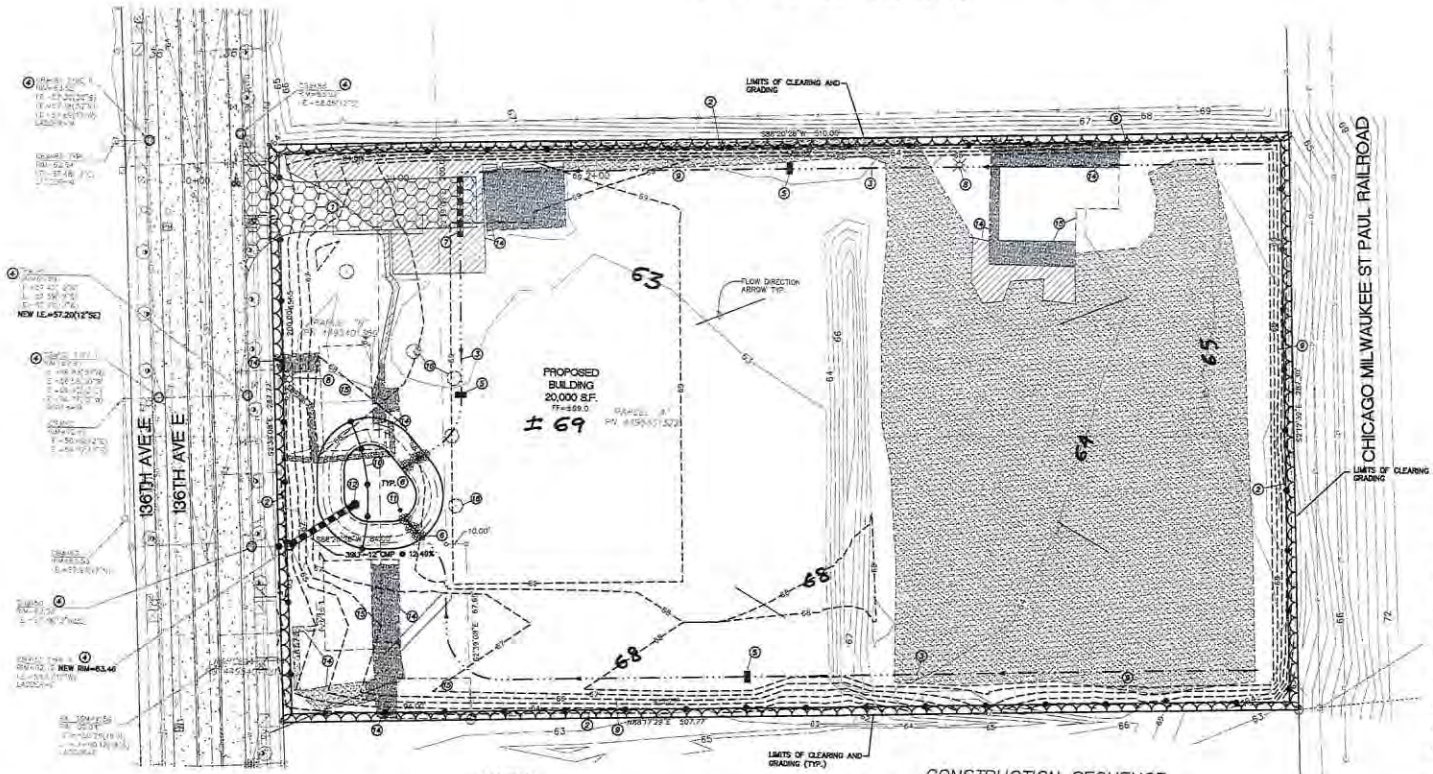
CVR

Figure 2

PRESTIGE WORLDWIDE HOLDINGS LLC.

PORTIONS OF NW 1/4, AND SW 1/4, SEC.12, TWN.20 N., RNG. 4 E., W.M.

T.E.S.C. AND GRADING PLAN



LEGEND

- CLEARING LIMITS
- EXISTING CONTOURS
- PROPOSED CONTOURS
- MIRAFI SILT FENCE (2)
- INTERCEPTOR SWALE (3)
- ROCK CHECK DAM (5)
- RIP RAP PAD (6)
- TEMPORARY 12" CMP CULVERT (7)
- CATCH BASIN INSERT (4)

CONSTRUCTION NOTES

1. CONSTRUCT TEMPORARY CONSTRUCTION ENTRANCE 24'W X 120'L PER SHEET 3.
2. INSTALL TEMPORARY MIRAFI SILT FENCE AS SHOWN & PER SHEETS 3&4.
3. INSTALL TEMPORARY INTERCEPTOR SWALE AS SHOWN & PER SHEET 4.
4. INSTALL INLET PROTECTION & TEMPORARY SEDIMENT CONTROL PER NOTES AND DETAILS ON SHEETS 3 & 4
5. INSTALL ROCK CHECK DAM AS NEEDED PER SHEET 4.
6. INSTALL RIP-RAP PADS 5'L X 4'W X 1'D WITH 6" QUARRY SPALLS.
7. INSTALL 64 LF 12"ø CMP CULVERT @ S=2.03%
8. EXISTING UTILITY TO BE REMOVED OR RELOCATED. COORDINATE WORK WITH COUNTY INSPECTOR & PROPER UTILITY COMPANY.
9. SEED AND/OR MULCH SIDE SLOPES AS REQUIRED. SEE SHEET 3. ~ APPLY 6" OF TOP SOIL ON GRAVELLY AREA PRIOR TO SEEDING.
10. INSTALL FILTER FABRIC FENCE POND DIVIDER AS SHOWN & PER DETAIL ON SHEET 3. KEY INTO BANK TO PREVENT FLOW AROUND SIDES.
11. INSTALL WHITE 2" X 4" STAFF GAUGE WITH PROMINENT 1.0' MARKED ABOVE BOTTOM OF POND.
12. INSTALL TEMPORARY RISER STRUCTURE AS SHOWN & PER DETAILS ON SHEET 3.
13. CONSTRUCT CONTRACTOR STORAGE, WASHDOWN, STAGING, FUELING & MAINTENANCE AREA. REPORT FUEL & LUBRICANT SPILLS TO DIVISION OF EMERGENCY MANAGEMENT (1-800-258-3990).
14. CONTRACTOR TO REMOVE EXISTING CONCRETE AND PAVING AND HAUL TO APPROPRIATE DISPOSAL SITE.
15. CONTRACTOR TO REMOVE EXISTING BUILDINGS.
16. CONTRACTOR TO REMOVE EXISTING TREES.

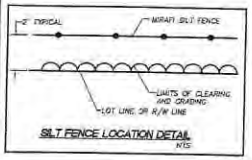
TEMP SEDIMENT POND

TOP EL.=69.00
WS EL.=64.50
BERM ELEV. =63.00
BOTTOM EL.=54.50
AREA REQUIRED @ 10 YEAR W.S. ELEV.- 1,000 SQ. FT.
AREA PROVIDED @ 10 YEAR W.S. ELEV.-2,713 SQ. FT.

ANTICIPATED PERMIT QUANTITIES

1. EARTHWORK:
   a) FILL: 23,996 CY
   b) CUT: 1,53 CY

REVISION BLOCK			
NO.	DATE	DESCRIPTION	BY



SILT FENCE LOCATION DETAIL
NTS

INSPECTION SCHEDULE

1. CONSTRUCTION ENTRANCE SHALL BE INSPECTED WEEKLY & CLEANED AS NEEDED (OR NEW SPALLS ADDED) AS NEEDED TO MAINTAIN A ROUGH SURFACE.
2. THE EXISTING ACCESS ROUTES INTO THE PROJECT SITE SHALL BE INSPECTED EVERY 2 DAYS & SWEPT OR CLEANED ON A WEEKLY BASIS OR MORE FREQUENTLY IF NEEDED.
3. ALL CUT & FILL SIDE SLOPES SHALL BE INSPECTED EVERY 2 DAYS AND/OR AFTER EVERY STORM EVENT TO REPAIR ANY EROSION OR SLOPE SCOURING.
4. SILT FENCES & DITCHES SHALL BE INSPECTED EVERY 4 DAYS.
5. STAKED HAY BALES SHALL BE INSPECTED ON A WEEKLY BASIS.
6. INSPECT INLET PROTECTION ON A WEEKLY BASIS AND AFTER EVERY MAJOR STORM EVENT.
7. INSPECT FILTER FABRIC DIRT BAG ON A WEEKLY BASIS AND AFTER EVERY MAJOR STORM EVENT.
8. TEMPORARY SEDIMENT PONDS & TRAPS SHALL BE INSPECTED ON A WEEKLY BASIS AND AFTER EVERY MAJOR STORM EVENT. REMOVE SEDIMENT TO MAINTAIN A MIN. OF 1.0' OF STORAGE.

CALL BEFORE YOU DIG

THE CONTRACTOR SHALL BE FULLY RESPONSIBLE FOR THE LOCATION AND PROTECTION OF ALL EXISTING UTILITIES. THE CONTRACTOR SHALL VERIFY ALL UTILITY LOCATIONS PRIOR TO CONSTRUCTION BY CALLING THE UNDERGROUND LOCATE LINE AT 811 A MINIMUM OF 48 HOURS PRIOR TO ANY EXCAVATION.

UTILITY CONFLICT NOTE:

CAUTION:

THE CONTRACTOR SHALL BE FULLY RESPONSIBLE FOR VERIFYING THE LOCATION, AND DEPTH OF ALL EXISTING UTILITIES WHETHER SHOWN ON THESE PLANS OR NOT BY POTHOLING THE UTILITIES AND SURVEYING THE HORIZONTAL AND VERTICAL LOCATION PRIOR TO CONSTRUCTION. THIS SHALL INCLUDE CALLING UTILITY LOCATE @ 811 AND THEN POTHOLING ALL OF THE EXISTING UTILITIES AT LOCATIONS OF NEW UTILITY CROSSINGS TO PHYSICALLY VERIFY WHETHER OR NOT CONFLICTS EXIST. LOCATION OF SAID UTILITIES AS SHOWN ON THESE PLANS ARE BASED UPON THE UNVERIFIED PUBLIC INFORMATION AND ARE SUBJECT TO VARIATION. IF CONFLICTS SHOULD OCCUR, THE CONTRACTOR SHALL CONSULT LARSON & ASSOCIATES, INC. TO RESOLVE ALL PROBLEMS PRIOR TO PROCEEDING WITH CONSTRUCTION.

NOTES:

1. ALL FILL SHALL BE CLEAN EARTHEN MATERIAL ONLY, WITH NO CONCRETE, GARBAGE, SOLID WASTE OR ANY OTHER UNACCEPTABLE MASS.
2. ALL ASBESTOS ENCOUNTERED MUST BE REMOVED AND DISPOSED OF IN ACCORDANCE WITH PUGET SOUND AIR POLLUTION CONTROL AGENCY (PSAPCA) AND TACOMA- PIERCE COUNTY HEALTH DEPARTMENT REGULATIONS.
3. ALL DEMOLITION MATERIAL MUST GO TO A LICENSED SOLID WASTE HANDLING OR DISPOSAL FACILITY.
4. ALL SEPTIC TANKS SHALL BE PUMPED BY A CERTIFIED SEPTIC HAULER, FILLED WITH SOIL AND A DECOMMISSIONING APPLICATION COMPLETED PER TACOMA-PIERCE COUNTY HEALTH DEPARTMENT REQUIREMENTS.
5. CONTRACTOR SHALL OBTAIN DEMOLITION PERMIT AS REQUIRED PRIOR TO ANY DEMOLITION OR STRUCTURE REMOVAL.
6. ALL SLOPES SHALL MAINTAIN MINIMUM SETBACKS IN ACCORDANCE WITH THE GRADING NOTES ON SHEET 3.
7. THE CONTRACTOR SHALL PLACE THE STRUCTURAL FILL MATERIAL IN EIGHT INCH (8") LOOSE LIFTS CONDITIONED TO THE PROPER MOISTURE CONTENT FOR COMPACTION AND COMPACTED TO AT LEAST NINETY-FIVE (95%) OF THE MAXIMUM DRY DENSITY, AS DETERMINED BY THE ASTM COMPACTION TEST PROCEDURE D-1557-70.
8. THE OWNER'S RETAINED GEOTECHNICAL ENGINEER SHALL TEST ALL FILL MATERIAL & OBTAIN SUFFICIENT COMPACTION TESTS TO VERIFY SOIL STABILITY.
9. EROSION CONTROL MEASURES OTHER THAN THOSE SPECIFIED MAY BE NEEDED TO PREVENT MIGRATION OF SEDIMENT. SEE SHEET 3 FOR ADDITIONAL MEASURES.
10. CONTRACTOR TO HAVE EQUIPMENT AND MATERIALS I.E.: STRAW, 12" & 18" CMP, QUARRY SPALLS, TEMP RISER STRUCTURES, ETC. ON-SITE DURING CONSTRUCTION FOR EMERGENCY SITUATIONS. CONTRACTOR TO DEVELOP PLAN OF ACTION FOR EROSION MEASURES PRIOR TO STARTING CONSTRUCTION.
11. PRIOR TO START OF CONSTRUCTION, A NPDES PERMIT SHALL BE OBTAINED FROM THE DEPARTMENT OF ECOLOGY IF NEEDED.
12. PRIOR TO START OF CONSTRUCTION, A FOREST PRACTICE PERMIT SHALL BE OBTAINED FROM THE DEPARTMENT OF NATURAL RESOURCES, IF NEEDED.
13. TO LIMIT IMPACTS TO NEIGHBORING PROPERTIES, DUST CONTROL WILL BE REQUIRED.

CONSTRUCTION SEQUENCE

1. CALL THE CITY OF SUMNER FOR PRE-CONSTRUCTION MEETING.
2. CLEARLY FLAG ALL LIMITS OF CLEARING AND GRADING PER APPROVED PLANS.
3. INSTALL TEMPORARY CONSTRUCTION ENTRANCE AS SHOWN AND PER SHEET 3.
4. INSTALL TEMPORARY MIRAFI FILTER FENCES AS SHOWN AND PER SHEET 3.
5. REMOVE EXISTING STRUCTURES AS INDICATED ON THE T.E.S.C. PLAN. ~ ACQUIRE APPROPRIATE DEMOLITION PERMITS.
6. CONSTRUCT TEMPORARY SEDIMENT/STORM PONDS AND PERMANENT DRAIN AND MAKE TIGHTLINE CONNECTION TO EL. 69@146 IN 136TH AVE E. PER DESIGN ON SHEET 4.
7. CONTRACTOR IS REQUIRED TO HAVE APPROVED CITY INSPECTION OF ALL INSTALLED T.E.S.C. MEASURES BEFORE SITE GRADING MAY BEGIN.
8. CLEAR AND GRADE SITE PER APPROVED PLANS.
9. ANY VEGETATION OR OTHER MATERIALS TAKEN OFF SITE SHALL BE HAULED TO MERIDIAN VALLEY LAND FILL AT 17925 MERIDIAN E., PUYALLUP, WASHINGTON 98372.
10. INSTALL UTILITIES (IE SANITARY, WATER, ETC.)
11. CONSTRUCT PAVED ROADWAY AND STORM DRAIN SYSTEM PER APPROVED PLANS.
12. PROVIDE C.B. PROTECTION UNTIL ALL CONSTRUCTION IS COMPLETED AND ALL EXPOSED SLOPES ARE SEEDED & STABILIZED FOR EROSION & SEDIMENTATION.
13. NOTE: THE BUILDING CONTRACTOR IS RESPONSIBLE FOR MAINTENANCE OF STORM SYSTEM DURING BUILDING AND LANDSCAPE CONSTRUCTION.
14. HYDRO SEED AND/OR MULCH SLOPES AND OTHER EXPOSED AREAS IMMEDIATELY AFTER GRADING IS COMPLETED AS OUTLINED IN "EROSION CONTROL NOTES".
15. CLEAN OUT AND TEST ALL STORM DRAIN FACILITIES.
16. INSPECT AND MAINTAIN ALL EROSION CONTROL FACILITIES AT REGULAR INTERVALS & COMPLETE REQUIRED REPORT. CLEAN AS REQUIRED UNTIL RISK OF SEDIMENTATION HAS PASSED.
17. UNTIL ALL PROJECTS WHICH PRODUCE SURFACE RUNOFF ARE COMPLETED AND ALL EXPOSED GROUND SURFACES ARE STABILIZED BY VEGETATION OR LANDSCAPING, THE PERMANENT PONDS MAY NOT BE OPERATED AND NO SURFACE RUNOFF MAY BE PERMITTED TO ENTER THE PERMANENT STORM SYSTEM.

JOB NUMBER: 8433
SCALE: HOR. 1"=30' VERT. N/A
DESIGNED: GJM DRAWN: FAM CHECKED: GJM

PROPONENT: JACOB CHMER, JOHANSEN EXCAVATING INC., P.O. BOX 674, BUCKLEY, WA. 98321, PH: (360)829-6493

LARSON and ASSOCIATES
Land Surveyors & Engineers, Inc.
4401 So. 86th ST. TACOMA, WA. 98409 (253) 474-3404

TESC AND GRADING PLAN

DATE: 6-3-16
DRAWING NO. 8433SASE
SHEET 2 OF 4
TP1

Figure 3

PGE Pacific Geo Engineering

Geotechnical Engineering, Consulting & Inspection

KEY TO EXPLORATION LOGS

Sample Descriptions:

Classification of soils in this report is based on visual field and laboratory observations, which include density/consistency, moisture condition, grain size, and plasticity estimates, and should not be construed to imply field or laboratory testing unless presented herein. Visual-manual classification methods in accordance with ASTM D-2488 were used as an identification guide. Where laboratory data available, soil classifications are in general accordance with ASTM D2487. Soil density/consistency in borings is related primarily to the Standard Penetration Resistance values. Soil density/consistency in test pits is estimated based on visual observations of excavations. Undrained shear strength = ½ unconfined compression strength.

RELATIVE DENSITY OR CONSITENCY VS. SPT N-VALUE

COARSE GRAINED SOILS: SAND OR GRAVEL			FINE GRAINED SOILS: SILT OR CLAY		
Density	N (Blows/ft.)	Approx. Relative Density (%)	Consistency	N (Blows/ft.)	Approx. Undrained Shear Strength (psf)
Very Loose	0 – 4	0- 15	Very Soft	0 – 2	<250
Loose	4 – 10	15 – 35	Soft	2 – 4	250 –500
Medium Dense	10 – 30	35 – 65	Medium Stiff	4 – 8	500 – 1000
Dense	30 – 50	65 – 85	Stiff	8 – 15	1000 – 2000
Very Dense	>50	85 – 100	Very Stiff Hard	15 – 30 > 50	2000 – 4000 > 4000

MOISTURE CONTENT DEFINITIONS

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, from below water table

DESCRIPTIONS FOR SOIL STRATA AND STRUCTURE

General Thickness or Spacing		Structure		General Attitude	
Parting	< 1/16 in	Pocket	Erratic, discontinuous deposit of limited extent	Near Horizontal	0 - 10 deg
Seam	1/16 - 1/2 in	Lens	Lenticular deposit	Low Angle	10 - 45 deg
Layer	½ - 12 in	Varved	Alternating seams of silt and clay	High Angle	45 - 80 deg
Stratum	> 12 in	Laminated	Alternating seams	Near Vertical	80 - 90 deg
Scattered	< 1 per ft	Interbedded	Alternating Layers		
Numerous	> 1 per ft	Fractured	Breaks easily along definite fractured planes		
		Slickensided	Polished, glossy, fractured planes		
		Blocky, Diced	Breaks easily into small angular lumps		
		Sheared	Disturbed texture, mix of strengths		
		Homogeneous	Same color and appearance throughout		

Appendix A

Soil Test Pit Log & Test Pit Photo

SOIL TEST PIT LOG

TEST PIT – 1, 2, 3, 4, 5, 6, & 7

Date of Excavation: 05/16/16

Approx. Depth, Ft.	USCS	Soil Descriptions	Test Pit No. Sample No./ Depth, Ft.	Moisture Content %	- #200 % (Fines content)
0 - 0.5	-	**Topsoil:** Approximately 6" thick Drk. Brn. Silt w/ roots & organics; Moist, Loose	-	-	-
0.5 - 5	SP-SM	Brn., SAND with Silt; Moist, Med. Dense	TP-1-S1 @ 2'	13.7	10.3 (Sieve Test Graph 1)
5 – 7	SP	Blk. SAND; Wet, Med. Dense	TP-1-S2 @ 5'	210	2.2 (Sieve Test Graph 2)

Note: Test pits were terminated at 7 ft below the existing grades.
No signs of mottling were noticed within the upper Brn. Sand with Silt layer.
Cave-in of the Blk. Sand layer was noticed below the upper Brn. Sand with Silt layer.
Groundwater table was encountered below the upper Brn. Sand with Silt layer at 5 feet below the current grades.
The test pits were left open till the end of the excavations of all the test pits, and it was noticed that the water level remained steady till the end of the backfilling of the test pits.

SITE PHOTO

Photo 1 - Site looking west (136th Avenue East) from existing trailer office corner

Photo 2 - Site looking SW corner from existing trailer office corner

Photo 3 - Site looking south from existing trailer office corner

Photo 4 - Site looking SE corner from existing trailer office corner

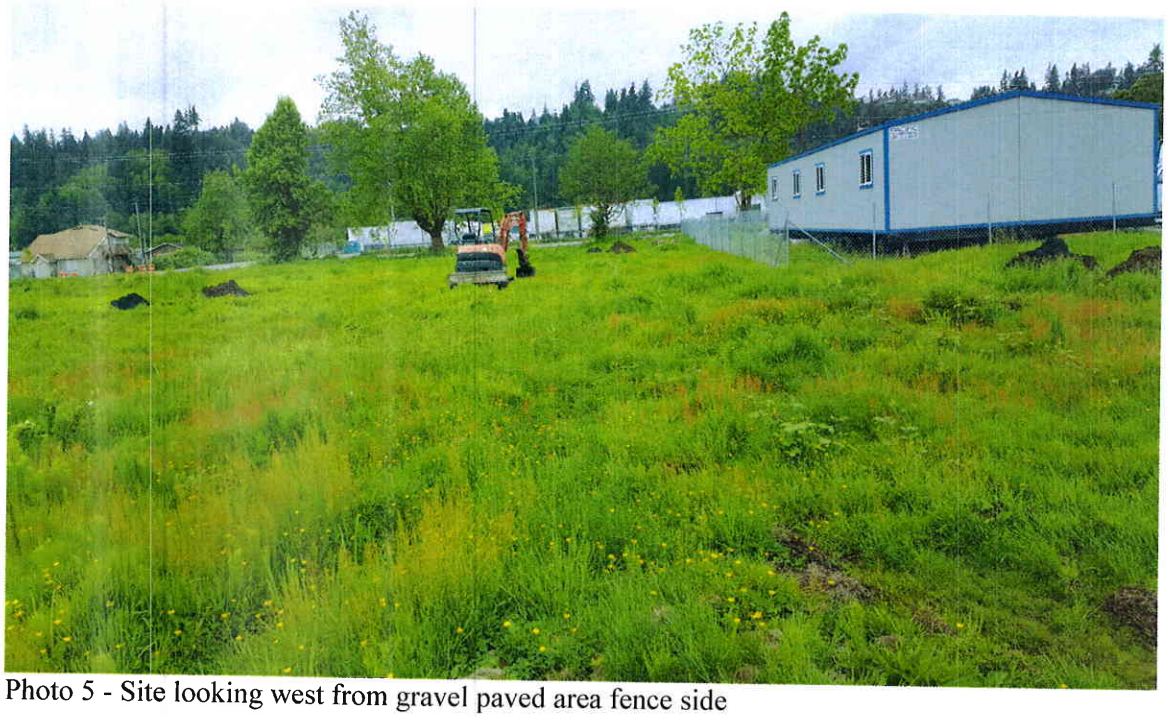


Photo 5 - Site looking west from gravel paved area fence side

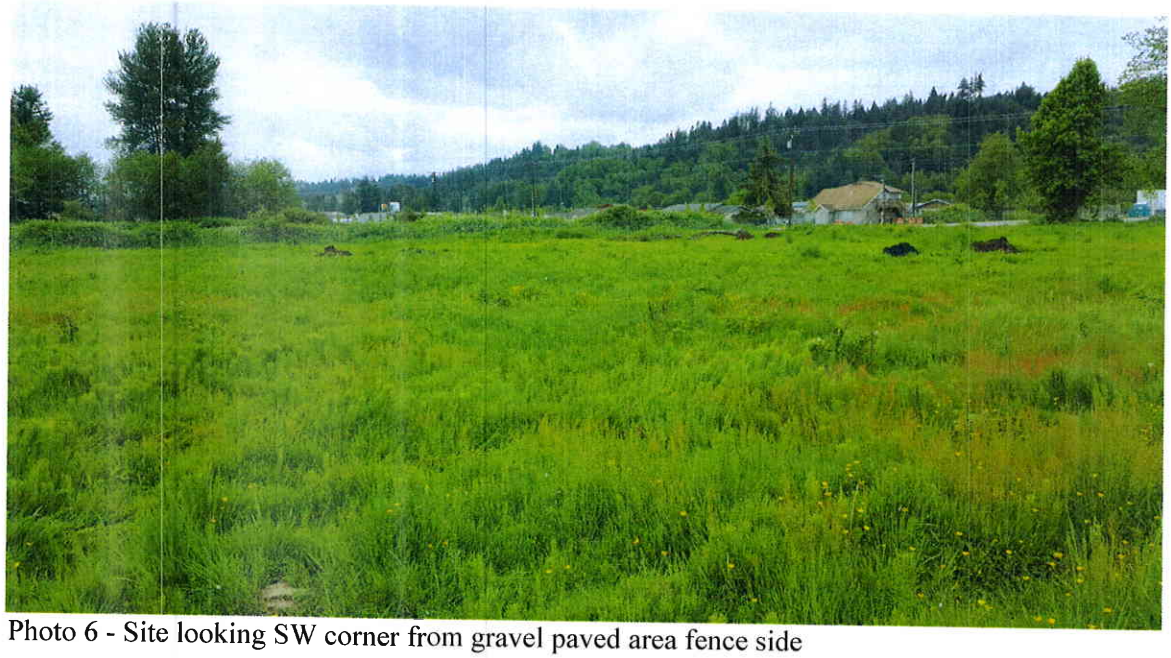


Photo 6 - Site looking SW corner from gravel paved area fence side

Photo 7 - Site looking SE corner from gravel paved area driveway

Photo 8 - Site looking south of gravel paved area from gravel paved driveway

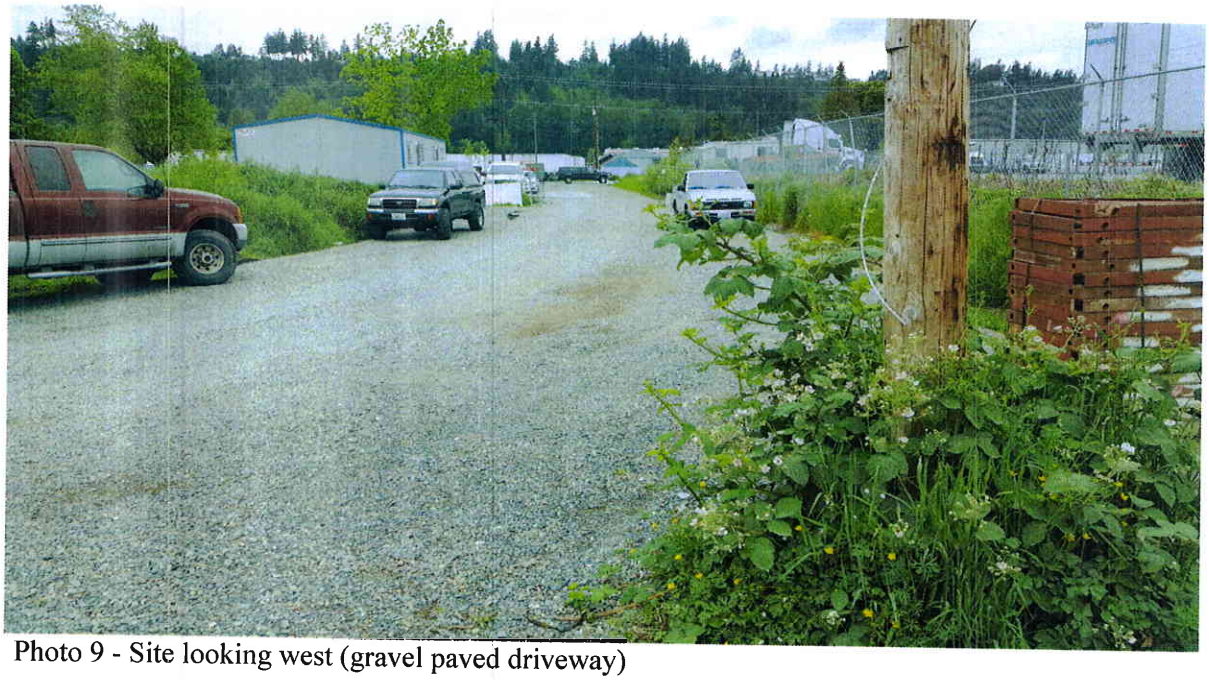


Photo 9 - Site looking west (gravel paved driveway)

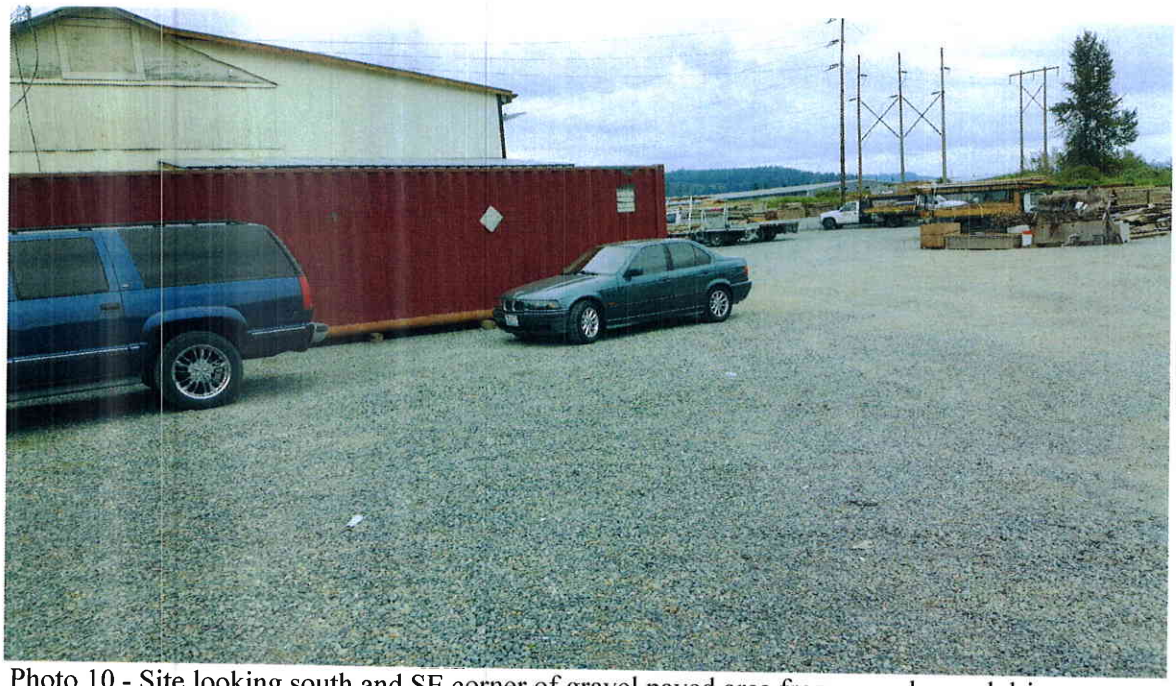


Photo 10 - Site looking south and SE corner of gravel paved area from gravel paved driveway

Photo 11 - Site looking NE corner and the adjacent north property from gravel paved driveway

TEST PIT SOIL LOG PHOTO

Photo 12 - Test Pit 1 Soil Log :-

Soil Layer 1 - 0.5 ft to ~ 5 ft - Brn. SAND with Silt (USCS: SP-SM); Moist, Med. Dense
Soli Layer 2 - ~ 5 ft to test pit bottom - Blk. SAND (USCS: SP); Wet, Med. Dense

Groundwater table caused caving of the black sand deposit. The upper, brown sand with silt layer acted like a bridge above the caving

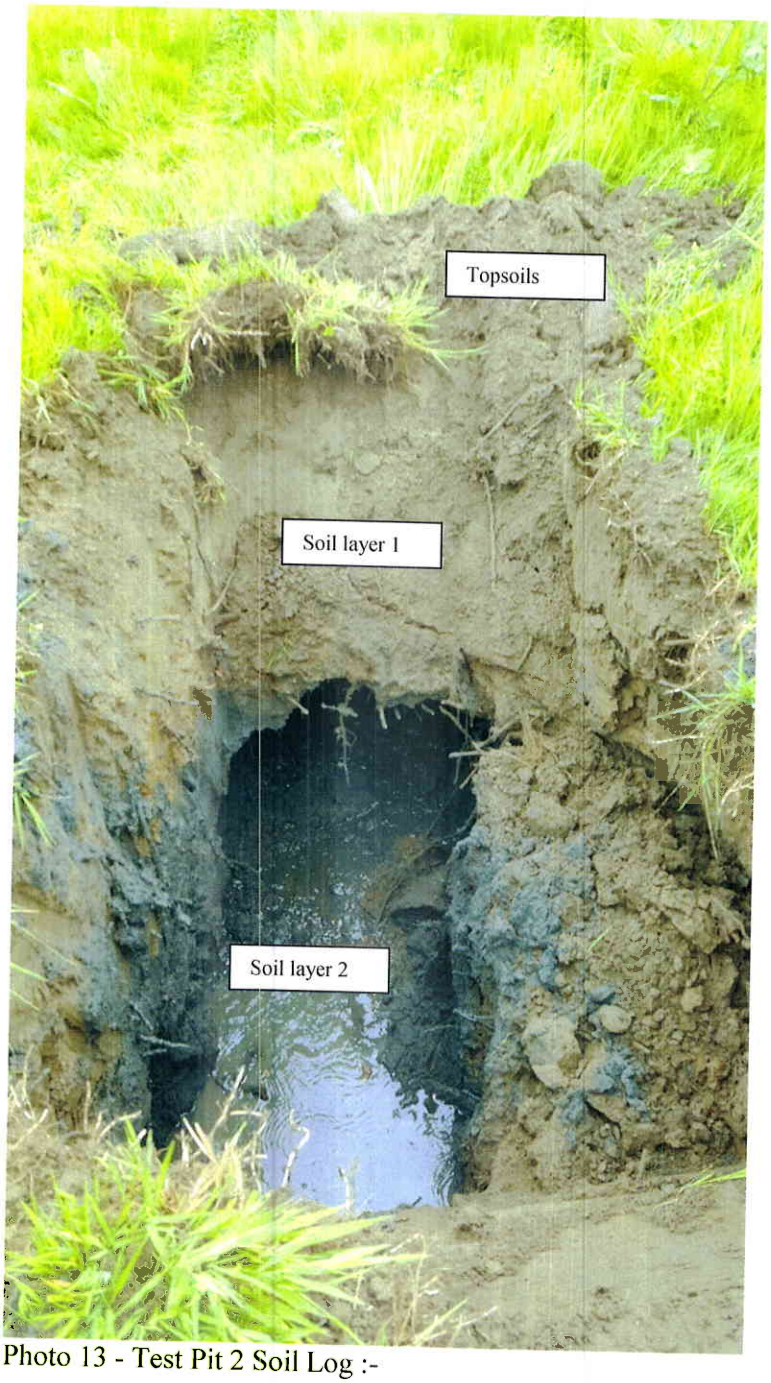


Photo 13 - Test Pit 2 Soil Log :-

Soil Layer 1 - 0.5 ft to ~ 5 ft - Brn. SAND with Silt (USCS: SP-SM); Moist, Med. Dense
Soli Layer 2 - ~ 5 ft to test pit bottom - Blk. SAND (USCS: SP); Wet, Med. Dense

Groundwater table caused caving of the black sand deposit. The upper, brown sand with silt layer acted like a bridge above the caving

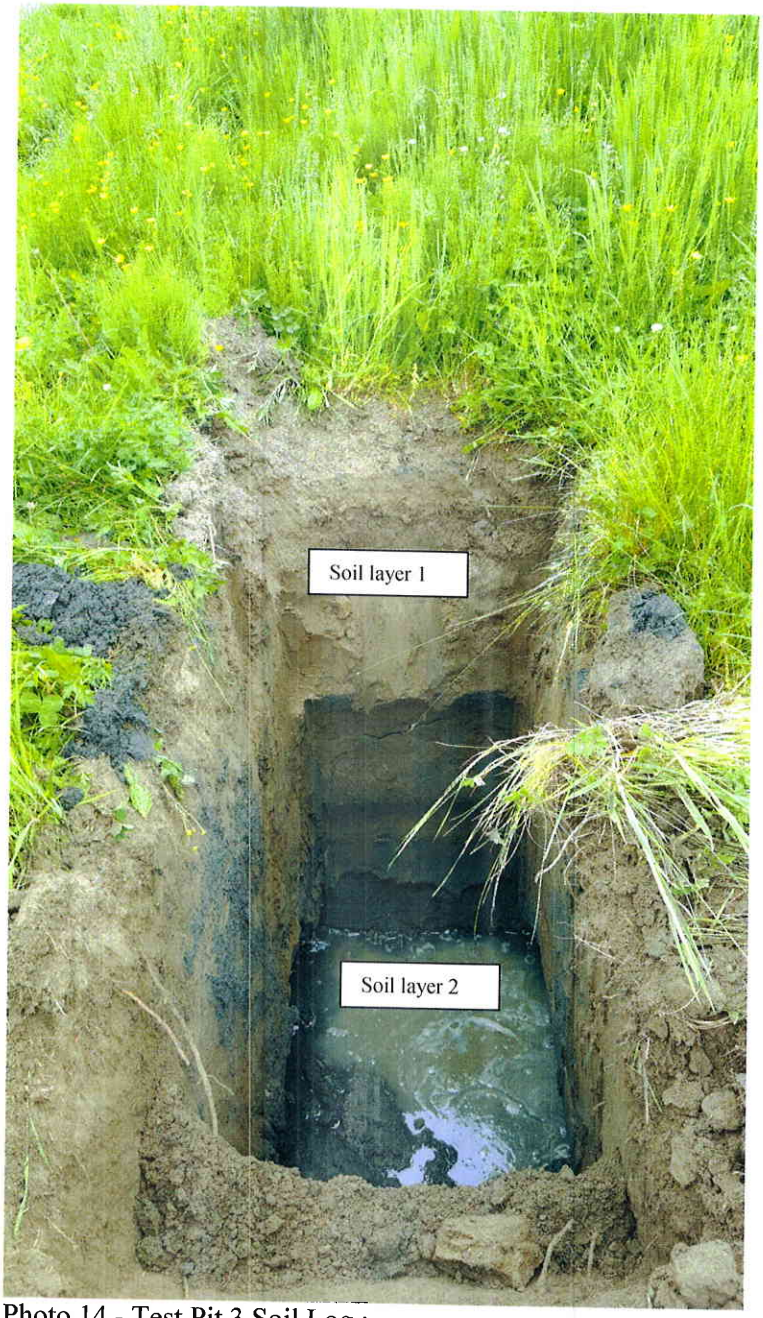


Photo 14 - Test Pit 3 Soil Log :-

Soil Layer 1 - 0.5 ft to ~ 5 ft - Brn. SAND with Silt (USCS: SP-SM); Moist, Med. Dense
Soli Layer 2 - ~ 5 ft to test pit bottom - Blk. SAND (USCS: SP); Wet, Med. Dense

Groundwater table caused caving of the black sand deposit. The upper, brown sand with silt layer acted like a bridge above the caving

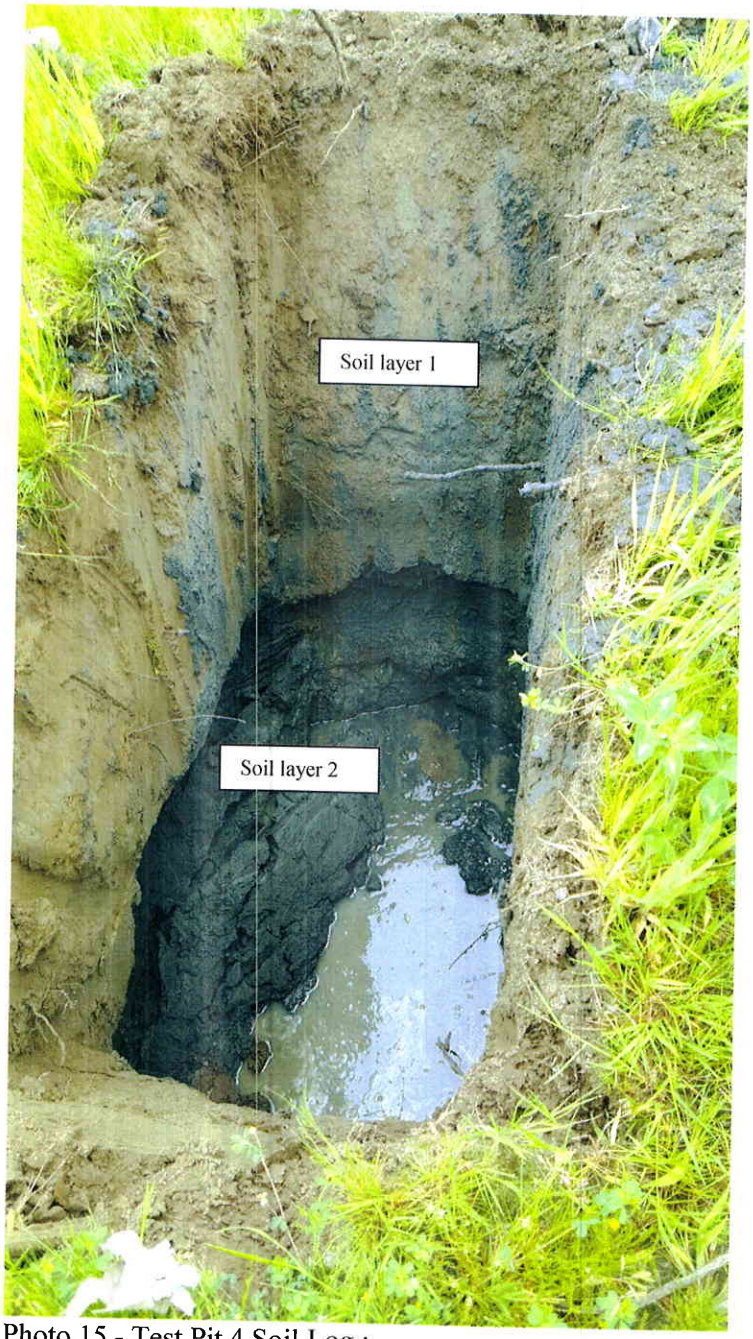


Photo 15 - Test Pit 4 Soil Log :-

Soil Layer 1 - 0.5 ft to ~ 5 ft - Brn. SAND with Silt (USCS: SP-SM); Moist, Med. Dense
Soli Layer 2 - ~ 5 ft to test pit bottom - Blk. SAND (USCS: SP); Wet, Med. Dense

Groundwater table caused caving of the black sand deposit. The upper, brown sand with silt layer acted like a bridge above the caving

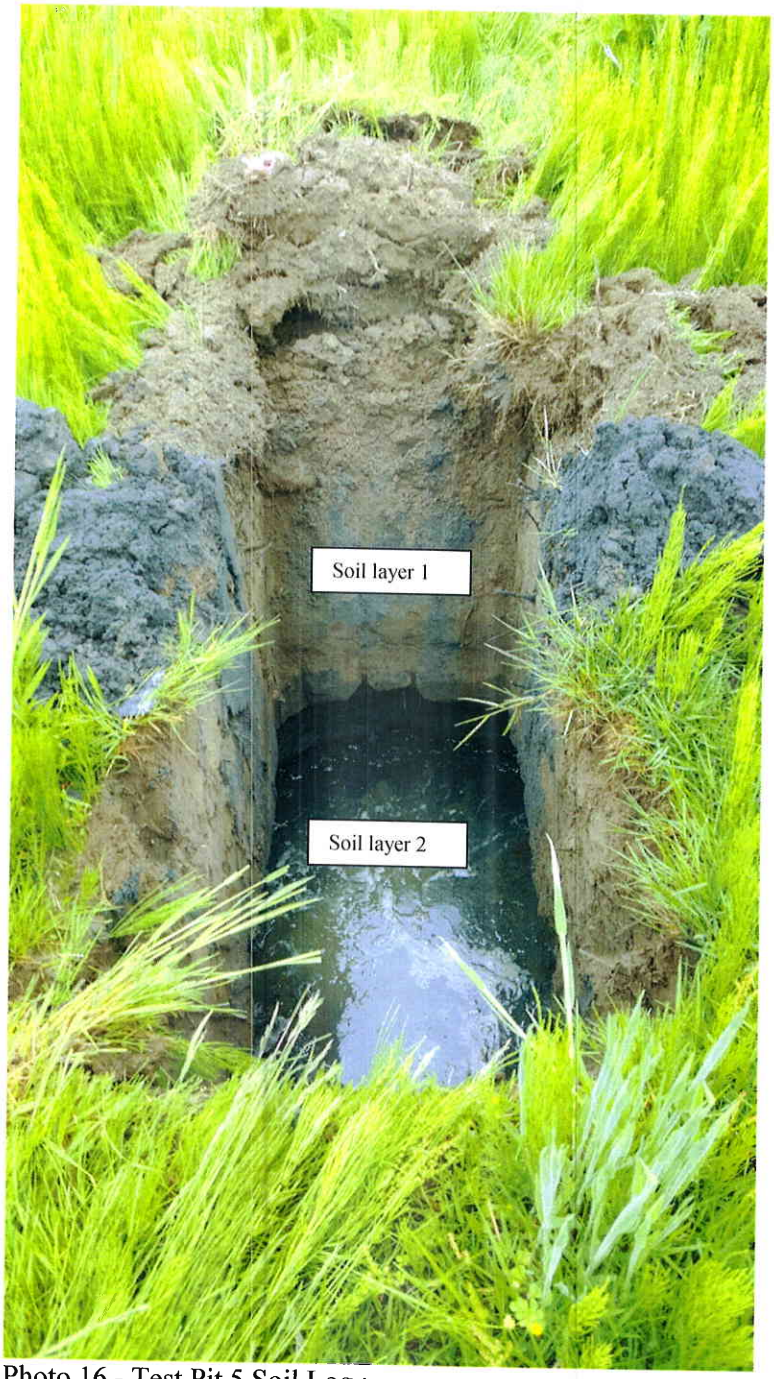


Photo 16 - Test Pit 5 Soil Log :-

Soil Layer 1 - 0.5 ft to ~ 5 ft - Brn. SAND with Silt (USCS: SP-SM); Moist, Med. Dense
Soli Layer 2 - ~ 5 ft to test pit bottom - Blk. SAND (USCS: SP); Wet, Med. Dense

Groundwater table caused caving of the black sand deposit. The upper, brown sand with silt layer acted like a bridge above the caving

Photo 17 - Test Pit 6 Soil Log :-

Soil Layer 1 - 0.5 ft to ~ 5 ft - Brn. SAND with Silt (USCS: SP-SM); Moist, Med. Dense
Soli Layer 2 - ~ 5 ft to test pit bottom - Blk. SAND (USCS: SP); Wet, Med. Dense

Groundwater table caused caving of the black sand deposit. The upper, brown sand with silt layer acted like a bridge above the caving

Photo 18 - Test Pit 7 Soil Log :-

Soil Layer 1 - 0.5 ft to ~ 5 ft - Brn. SAND with Silt (USCS: SP-SM); Moist, Med. Dense
Soli Layer 2 - ~ 5 ft to test pit bottom - Blk. SAND (USCS: SP); Wet, Med. Dense

Groundwater table caused caving of the black sand deposit. The upper, brown sand with silt layer acted like a bridge above the caving

Photo 19 - Typical excavated soils :-

Soil Layer 1 - 0.5 ft to ~ 5 ft - Brn. SAND with Silt (USCS: SP-SM); Moist, Med. Dense
Soli Layer 2 - ~ 5 ft to test pit bottom - Blk. SAND (USCS: SP); Wet, Med. Dense

Photo 20 - Typical excavated soil - Layer 2- Blk. SAND (USCS: SP); Wet, Med. Dense

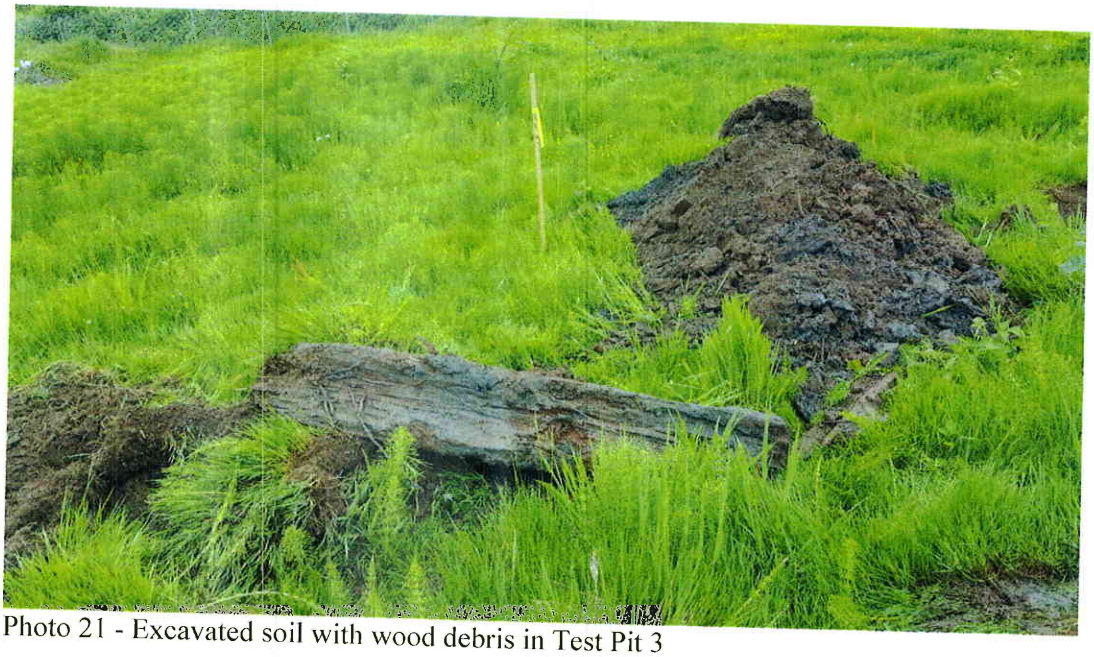


Photo 21 - Excavated soil with wood debris in Test Pit 3

Appendix B

Laboratory Test Results

PGE Pacific Geo Engineering LLC

Geotechnical Engineering, Consulting & Inspection

UNIFIED SOIL CLASSIFICATION SYSTEM

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests[A]				Soil Classification	
				Group Symbol	Group Name[B]
Coarse-Grained Soils More than 50% retained on No. 200 sieve	Gravels More than 50% of coarse fraction retained on No. 4 sieve	Clean Gravels Less than 5% fines[C]	Cu ≥ 4 and 1 ≤ Cc ≤ 3[E]	GW	Well-graded gravel[F]
			Cu < 4 and/or 1 > Cc > 3[E]	GP	Poorly graded gravel[F]
		Gravels with Fines More than 12% fines[C]	Fines classify as ML or MH	GM	Silty gravel[F, G, H]
			Fines classify as CL or CH	GC	Clayey gravel[F, G, H]
	Sands 50% or more of coarse fraction passes No. 4 sieve	Clean Sands Less than 5% fines[E]	Cu ≥ 6 and 1 ≤ Cc ≤ 3[E]	SW	Well-graded sand[I]
			Cu < 6 and/or 1> Cc > 3[E]	SP	Poorly graded sand[I]
		Sands with Fines More than 12% fines[D]	Fines classify as ML or MH	SM	Silty sand[G, H, I]
			Fines classify as CL or CH	SC	Clayey sand[G, H, I]
Fine-Grained Soils 50% or more passes the No. 200 sieve	Silts and Clays Liquid limit less than 50	inorganic	PI > 7 and plots on or above "A" line[J]	CL	Lean clay[K, L, M]
			PI < 4 or plots below "A" line[J]	ML	Silt[K, L, M]
		organic	Liquid limit — oven dried / Liquid limit — not dried < 0.75	OL	Organic clay[K, L, M, N] / Organic silt[K, L, M, O]
	Silts and Clays Liquid limit 50 or more	inorganic	PI plots on or above "A" line	CH	Fat clay[K, L, M]
			PI plots below "A" line	MH	Elastic silt[K, L, M]
		organic	Liquid limit — oven dried / Liquid limit — not dried < 0.75	OH	Organic clay[K, L, M, P] / Organic silt[K, L, M, Q]
Highly organic soils		Primarily organic matter, dark in color, and organic odor		PT	Peat

[A]Based on the material passing the 3-in. (75-mm) sieve.

[B]If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.

[C]Gravels with 5 to 12% fines require dual symbols:
GW-GM well-graded gravel with silt
GW-GC well-graded gravel with clay
GP-GM poorly graded gravel with silt
GP-GC poorly graded gravel with clay

[D]Sands with 5 to 12% fines require dual symbols:
SW-SM well-graded sand with silt
SW-SC well-graded sand with clay
SP-SM poorly graded sand with silt
SP-SC poorly graded sand with clay

[E]$Cu = D_{60}/D_{10}$ $\quad Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$

[F]If soil contains ≥ 15% sand, add "with sand" to group name.

[G]If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.

[H]If fines are organic, add "with organic fines" to group name.

[I]If soil contains ≥ 15% gravel, add "with gravel" to group name.

[J]If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.

[K]If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel", whichever is predominant.

[L]If soil contains ≥ 30% plus. No. 200 predominantly sand, add "sandy" to group name.

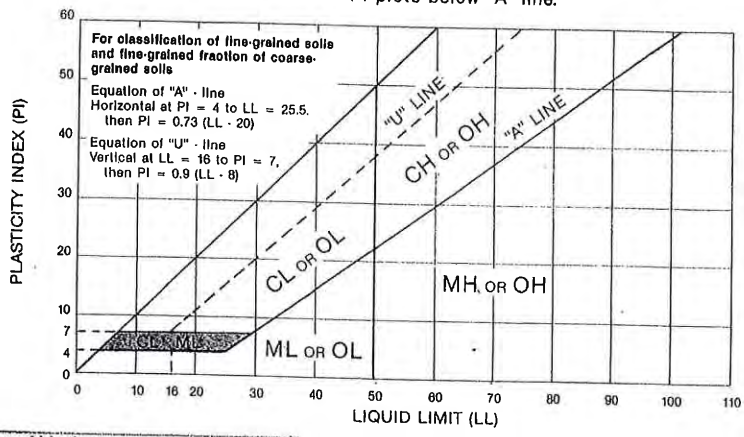
[M]If soil contains ≥ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.

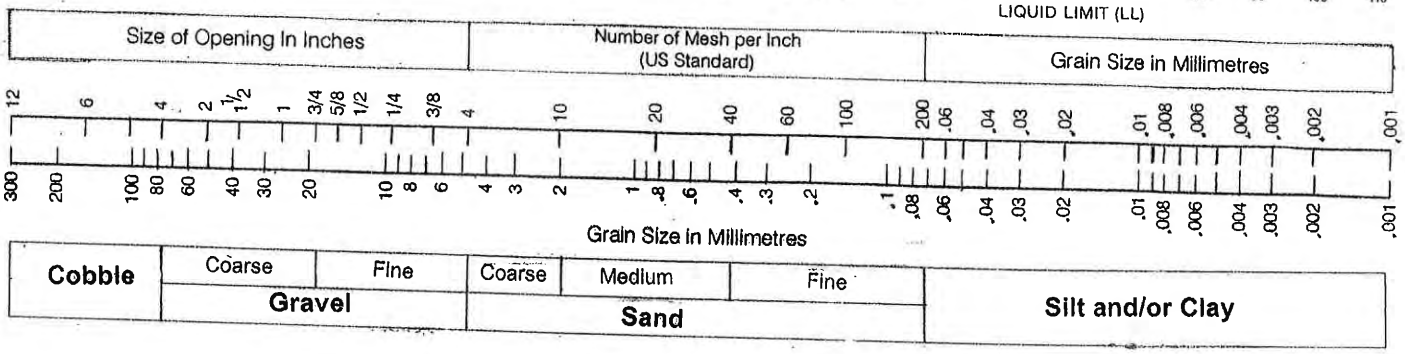
[N]PI ≥ 4 and plots on or above "A" line.

[O]PI < 4 or plots below "A" line.

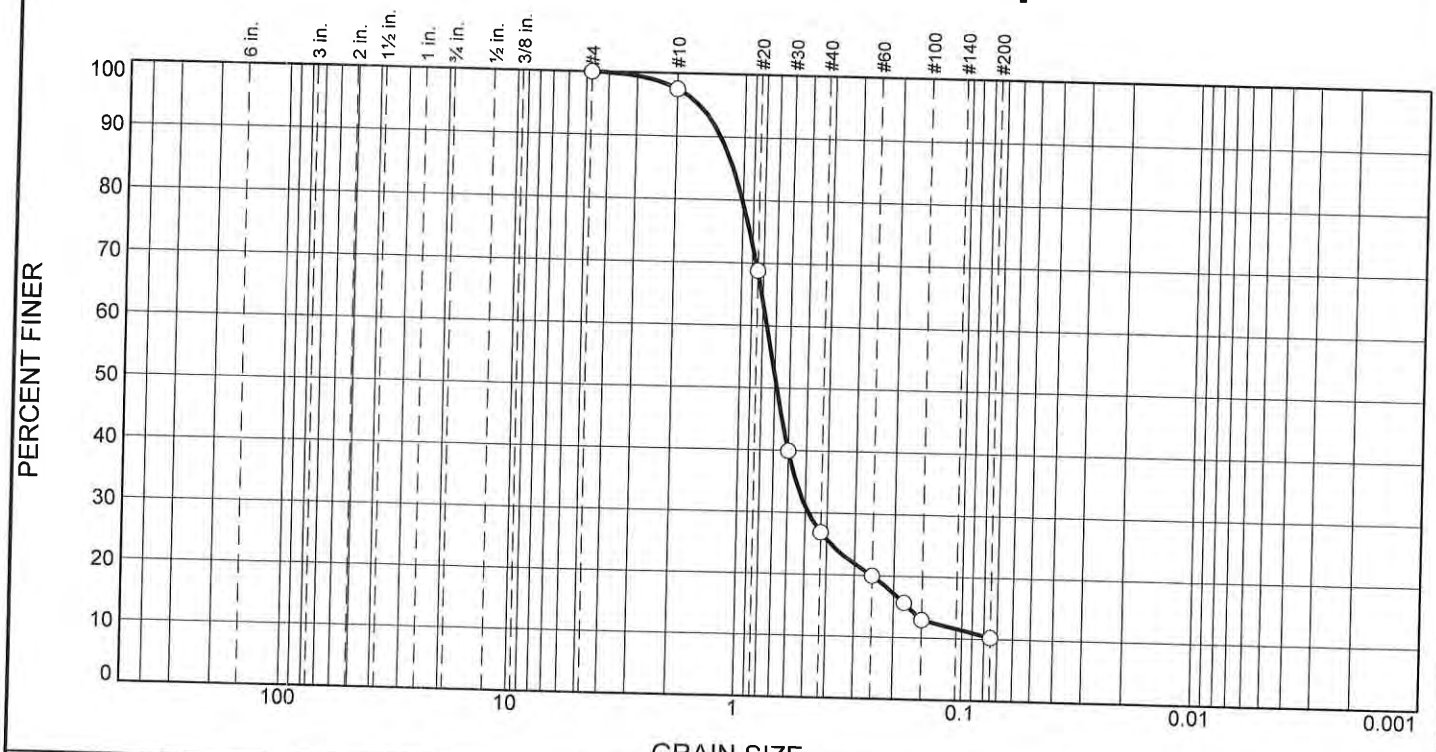
[P]PI plots on or above "A" line.

[Q]PI plots below "A" line.





Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	2.5	70.6	16.6	10.3	

TEST RESULTS

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	100.0		
#10	97.5		
#20	68.8		
#30	40.0		
#40	26.9		
#60	20.0		
#80	15.7		
#100	13.0		
#200	10.3		

* (no specification provided)

Material Description

Brn. SAND with Silt

Atterberg Limits (ASTM D 4318)

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

Classification

USCS (D 2487)= SP-SM **AASHTO (M 145)=** A-1-b

Coefficients

D_{90}= 1.2737 D_{85}= 1.1129 D_{60}= 0.7656
D_{50}= 0.6833 D_{30}= 0.4825 D_{15}= 0.1719
D_{10}= C_u= C_c=

Remarks

Date Received: 05-16-16 **Date Tested:** 05-17-16

Tested By: Chrstopher Mooreddrall

Checked By: Santanu Mowar

Title: Principal

Location: Test Pit 1

Sample Number: S-1 **Depth:** 2 ft **Date Sampled:** 05-16-16

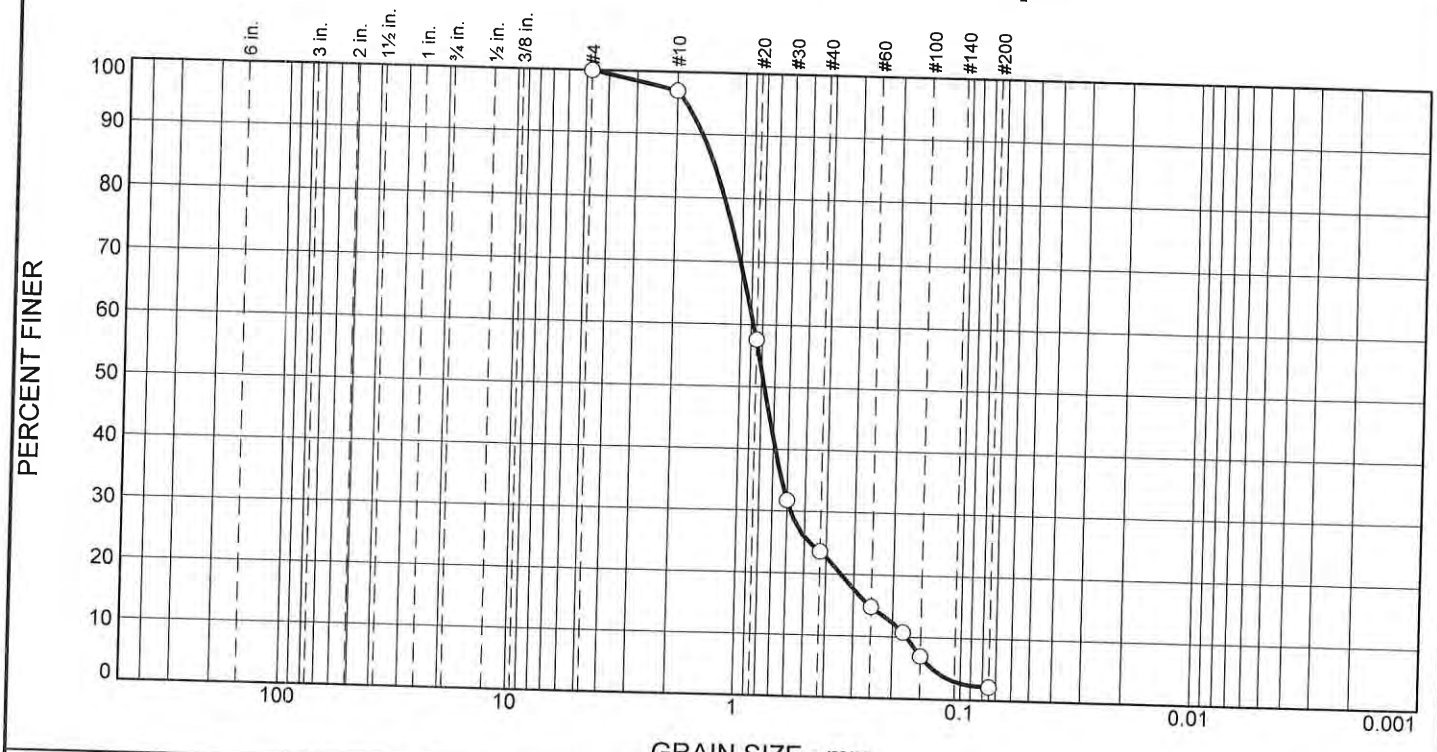
Pacific Geo Engineering, LLC
Geotechnical Engineering, Consultation, Testing, & Inspection

Client: Johansen Excavating, Inc.

Project: Prestige Worldwide Holdings, Site

Project No: 16-495 **Figure** 1

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	3.0	73.4	21.4	2.2	

TEST RESULTS

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	100.0		
#10	97.0		
#20	57.6		
#30	31.8		
#40	23.6		
#60	14.8		
#80	10.8		
#100	7.0		
#200	2.2		

* (no specification provided)

Material Description

Blk. SAND

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= SP AASHTO (M 145)= A-1-b

Coefficients

D_{90}= 1.5302 D_{85}= 1.3434 D_{60}= 0.8769
D_{50}= 0.7739 D_{30}= 0.5768 D_{15}= 0.2539
D_{10}= 0.1727 C_u= 5.08 C_c= 2.20

Remarks

Date Received: 05-16-16 **Date Tested:** 05-17-16

Tested By: Chris Mooreddrall

Checked By: Santanu Mowar

Title: Principal

Location: Test Pit 1
Sample Number: S-2 **Depth:** 6 ft **Date Sampled:** 05-16-16

Pacific Geo Engineering, LLC
Geotechnical Engineering, Consultation, Testing, & Inspection

Client: Johansen Excavating, Inc.
Project: Prestige Worldwide Holdings, Site

Project No: 16-495 **Figure** 2