PanGEO
INCORPORATED
Geotechnical & Earthquake Engineering Consultants

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File No. 12-163

Ms. Stacey Clear, P.E.
Gray & Osborne, Inc.
3710 168th St. NE, Suite B210
Arlington, Washington 98223

Subject: **GEOTECHNICAL REPORT**
Marysville Decant Facility
80 Columbia Avenue, Marysville, WA

Dear Ms. Clear,

As requested, PanGEO, Inc. completed a geotechnical engineering study to assist the project team with the design and construction of the proposed decant facility in Marysville, Washington. The results of our study are summarized in the attached report. In summary, a thick layer of compressible soil underlies the site. As such, we recommend that the decant structure be supported on piles or improved ground to prevent undesired settlements. In our opinion, other structures at the facility that are more tolerant to settlement maybe supported by a shallow foundation system after a preload has been applied to the area. In addition, groundwater is relatively shallow at the site, and the need for construction dewatering should be anticipated.

We appreciate the opportunity to work with you on this project. Please call if there are any questions regarding this report.

Sincerely,

Siew L. Tan, P.E.
Principal Geotechnical Engineer

Encl.: Geotechnical Report

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1.0 INTRODUCTION

This report presents the results of our geotechnical study for the proposed decant facility for the City of Marysville, Washington. Our scope of work included reviewing existing subsurface data, conducting a site reconnaissance, drilling five test borings, advancing two cone penetration test soundings, performing laboratory testing, and performing engineering analyses to develop the geotechnical recommendations outlined in this report.

2.0 SITE AND PROJECT DESCRIPTION

The proposed project is located on the grounds of the existing City of Marysville Public Works yard at 80 Columbia Avenue in Marysville, Washington. The approximate location of the property is shown on the attached Figure 1. The property as a whole is bounded on the west by Columbia Avenue, on the north by several single-family residences, on the east by vacant land, and on the south by a sewage lagoon. The specific area of focus for this study is the approximately south half of the property, immediately north of the existing sewage lagoon. This area is roughly 300 feet by 300 feet in size, generally flat, partially paved, and is currently being used as a storage yard. Plate 1 on the following page depicts current site conditions in the area of the proposed decant facility.

As currently planned, we understand the proposed improvements will include constructing a decant station, retrofitting an existing material storage area, and adding new storage areas. New storm drain pipes and a new storm drain system may also be installed as part of the proposed improvements. The layout of the proposed improvements is not available at this time.

Plate 1. *An aerial view of the project site (image source: Google maps).*

3.0 SUBSURFACE EXPLORATIONS

Because the final layout of the proposed improvements had not been determined at the time of our subsurface investigation (October 25 & 29, 2012), two cone penetration tests (CPTs) and five test borings were conducted at a uniform spacing within the general area of where the proposed structures may be located. Locations of the CPTs and test borings were located in the field by taping from existing features. The approximate locations of the seven tests are indicated on the attached Site and Exploration Plan (Figure 2).

3.1 Test Boring

Five test borings (BH-1 through BH-5) were drilled on at the site on October 29, 2012. The approximate locations of the borings are shown on the attached Site and Exploration Plan (Figure 2). The borings were drilled to depths of about 20 feet below surface grades using a track mounted drill rig owned and operated by Boretec Inc. The drill rig was equipped with 6-inch and 8-inch outside diameter hollow stem augers, and soil samples were obtained from the borings at 2½- and 5-foot depth intervals in general accordance with Standard Penetration Test (SPT) sampling methods (ASTM test method D-1586) in which the samples are obtained using a 2-inch outside diameter split-spoon sampler. The sampler was driven into the soil a distance of 18 inches using a 140-pound weight falling a distance of 30 inches. The number of blows required for each 6-inch increment of sampler penetration was recorded. The number of blows required to achieve the last 12 inches of sample penetration is defined as the SPT N-value. The N-value provides an empirical measure of the relative density of cohesionless soil, or the relative consistency of fine-grained soils. When soft soil deposits were encountered, samples were obtained through the use of a thin walled sampler (Shelby Tube), in an attempt to minimize sample disturbance. After completion of borings BH-1 and BH-5, 2-inch diameter standpipe piezometers were installed in the borings for future monitoring of groundwater levels.

An engineer from PanGEO was present throughout the field exploration program to observe the drilling, assist in sampling, to document the soil samples obtained from the borings, and to verify the proper installation of piezometers. Detailed information from the field exploration program is presented in Appendix A. The soil samples retrieved from the borings were described using the system outlined on Figure A-1 of Appendix A and the summary boring logs are included as Figures A-2 through A-6.

3.2 Cone Penetration Test

The Cone Penetration Tests (CPTs) were conducted on October 25, 2012. A CPT consists of pushing an instrumented cone, approximately 1-inch in diameter, into a soil deposit from a truck mounted reaction frame, and measuring the resistance and pore water pressure on the tip and side of the cone. Higher tip resistance measurements indicate the soil deposit has a higher strength or density than lower tip resistance measurements. The resistances to continuous penetration encountered by the cone tip and adjacent friction sleeve also exhibit high sensitivity to changes in soil type, which may be correlated to differing soil types and strength parameters.

The CPTs were performed by In Situ Engineering of Snohomish, Washington, under a subcontract to PanGEO. Both CPT-01 and CPT-02 were advanced to a depth of 80 feet below the ground surface. The subcontractor backfilled the sounding holes using sand and bentonite. Summary CPT logs are included in Appendix B of this report for reference.

3.3 LABORATORY TEST

Selected soil samples were tested in general accordance with test methods of the American Society for Testing and Materials (ASTM). The tests include samples for in-situ moisture content, Atterberg limits testing, organic matter and consolidation characteristics.

Moisture Content Testing – Moisture content tests were performed in general accordance with ASTM D 2216. The test results are included on the appropriate summary boring logs in Appendix A.

Atterberg Limits Testing – The Liquid Limit and Plastic Limit were determined for selected soil samples, in accordance with ASTM D 4318. The test results are indicated on the appropriate summary boring logs in Appendix A, as well as with the laboratory test results in Appendix C.

Ash and Organic Matter – The organic content of one sample was determined in general accordance with ASTM D2974, using moisture method ‘A’ and ash content method ‘C’. The test results are indicated on the appropriate summary boring log in Appendix A, as well as with the laboratory test results in Appendix C.

Consolidation Testing – One-dimensional consolidation tests were conducted on relatively undisturbed soil samples extruded from Shelby tubes. The tests were performed in general accordance with ASTM D 2435 Method B, using a fixed-ring consolidometer. The primary purpose of the consolidation test is to aid in the estimation of potential consolidation upon placement of additional loads. The results of the consolidation testing are included in Appendix C.

4.0 SUBSURFACE CONDITIONS

4.1 SOIL

Based on our review of a geologic map of the Marysville area (Minard, 1985), the site is underlain by younger alluvium and estuarine deposits. This deposit consists of stream-laid stratified sediments primarily consisting of sand, silt and clay. Organics are also common in this deposit. Tidal flat mud and sand, and localized peat deposits, are included in this mapping unit as well. Our subsurface investigations performed at the site confirmed the mapped geology, and generally encountered about 5 feet of fill soil, over what we interpreted to be tideflat deposits and alluvium. A summary of the soil units encountered in the explorations is provided below (approximate depths are measured from the existing ground surface). Detailed subsurface information is provided on the summary boring and CPT logs included in Appendix A and Appendix B. In general, three distinct soil units were encountered at the site, as summarized below:

Unit 1 Fill – Approximately 4 to 6 feet of loose to medium dense silty sand with some to trace gravel was encountered in our explorations. We interpret this soil unit as fill that was placed for the development of the existing facility.

Unit 2 Tideflat Deposits – A layer of very soft to medium stiff silt and clay was encountered directly below the fill. Between depths of about 5 and 13 feet, layers of organic silt and peat were encountered within this soil unit. This soil unit extended to about 28 and 35 feet in CPT-01 and CPT-02, respectively. The test borings were terminated within this soil unit. We interpret this unit as tideflat deposits, likely deposited in a former intertidal zone of Possession Sound to the west. This unit is considered compressible and could settle significantly if additional loads are added.

Unit 3 Alluvium – In both CPTs, a sequence of medium dense to dense sand and silty sand with silt and clay interbeds was encountered directly below the tideflat deposits and extended to the maximum depth of explorations at 80 feet. This soil unit appears to be river alluvium deposited by the nearby Snohomish River. A clean, medium to coarse sand, which also appeared to be alluvium, was encountered in test boring BH-4, between a depth of about 8 and 16 feet below the ground surface.

4.2 GROUNDWATER

Groundwater was observed in our test borings at the time of drilling. It appears that there is shallow, perched groundwater located within the fill, perched on the underlying silt and clay. This perched groundwater was observed within the fill soils encountered in borings BH-3, BH-4 and BH-5 about 3 feet below the ground surface. Based on measurements made in the piezometers (BH-1 and BH-5) on November 7, 2012, the perched groundwater is about 3 to 4 feet below surface grades.

Based on the wetness of soil samples recovered from the test borings, we estimate the static groundwater level to be about 10 to 15 feet below the existing grade at the time of drilling. One exception was at the location of BH-4, which encountered a saturated sand layer between about 4 and 16 feet below the ground surface, overlying saturated silt and clay.

To determine if there is a tidal influence on the groundwater level at the site, we installed a data logger in the BH-5 piezometer for a 24-hour period. Based on the results of the 24-hour water level measurement, as noted in Figure 3, the groundwater level at the site does not appear to be influenced significantly by the tides.

5.0 DESIGN RECOMMENDATIONS

5.1 SEISMIC DESIGN PARAMETERS

5.1.1 Site Class

We anticipate that the seismic design of the building will be accomplished in accordance with the 2009 International Building Code (IBC). The following provides seismic design parameters for the site that are in conformance with the 2009 edition of the International Building Code (IBC), which specifies a design earthquake having a 2% probability of occurrence in 50 years (return interval of 2,475 years), and the 2002 USGS seismic hazard maps:

Site Class	Spectral Acceleration at 0.2 sec. (g) S_S	Spectral Acceleration at 1.0 sec. (g) S_1	Site Coefficients		Design Spectral Response Parameters		Control Periods (sec.)		Design PGA (S_{DS}/2.5)
			F_a	F_v	S_{DS}	S_{D1}	T_O	T_S	
D	1.10	0.38	1.1	1.6	0.78	0.42	0.11	0.54	0.31

The spectral response accelerations were obtained from the USGS website (2002 data) for the project latitude and longitude.

5.1.2 Soil Liquefaction

Liquefaction occurs when saturated sands are subjected to cyclic loading, and causes the pore water pressure to increase in the sand thereby reducing the inter-granular stresses. As the inter-granular stresses are reduced, the shearing resistance of the sand decreases. If pore pressures develop to the point where the effective stresses acting between the grains become zero, the soil particles will be in suspension and behave like a viscous fluid. Typically loose, saturated, clean granular soils, that have a low enough permeability to prevent drainage during cyclic loading, have the greatest potential for liquefaction, while more dense soil deposits with higher silt or clay contents have a lesser potential. Potential effects of soil liquefaction include temporary loss/reduction of foundation capacity and settlement.

Because the upper approximately 30 feet of soils primarily consist of silt and clay, in our opinion the potential for earthquake-induced soil liquefaction in this soil unit is considered to be low. Below the silt and clay the medium dense to dense sand and silty sand with silt and clay interbeds is also expected to have a low potential to liquefy during a seismic event, due to the relatively high density, and silt/clay content. During the IBC code-level earthquake, we anticipate that any liquefaction that does occur in the lower sand layer will have a minimal effect on the ground surface. In addition, the proposed deep foundation system for the decant structure will mitigate any potential effects of soil liquefaction on the proposed structure.

5.2 FOUNDATION

5.2.1 Foundation Alternatives

Multiple foundation systems were considered for the proposed decant facility. Due to the thick layer of compressible soils, which also contain layers of organic silt and peat which can continue to settle even after being pre-loaded, a decant facility supported by a shallow foundation system will likely experience total and differential settlements that are not tolerable, and could impact the long-term functionality of the facility. As such, we recommend that the decant structure be supported by either a deep foundation system, or by a shallow foundation system bearing on improved ground. If the deep foundation option is selected, in our experience small diameter (6 to 8-inch) driven pipe piles or augercast piles represent the two most cost effective deep foundation systems for this project. If ground improvement methods are utilized, we anticipate that aggregate piers would be a feasible and cost effective improvement method.

Because we understand that the storage bins and other site structures are not as sensitive to settlement as the decant structure, we recommend that a conventional shallow foundation system be utilized to support the other structures. In our opinion a cost effective method to reduce the magnitude of settlements for structures supported by shallow foundations is to preload the existing ground surface over the area of the proposed development prior to construction.

5.2.2 Driven Pipe Pile Foundations – Decant Structure

In our opinion, small diameter driven pipe piles represent a feasible and cost effective foundation option for the proposed decant structure. Small diameter pipe piles may be utilized to transfer the structure loads through the upper loose/soft soil (Unit 1 and Unit 2) to the underlying medium dense to dense sand and silty sand.

Pile Sizes **–** In our opinion 6- or 8-inch diameter piles represent an appropriate size pile to support the proposed structure. 6-inch and 8-inch piles are typically installed using medium-sized hydraulic hammers (2,000 to 3,000 pound) mounted on an excavator.

***Pile Capacity* -** An allowable axial compression capacity of 30 and 50 kips may be used for 6- and 8-inch diameter piles, respectively, with an approximate factor of safety of 2.0. Penetration resistance required to achieve the capacities will be determined based on the hammer used to install the pile. Tensile capacity of pin piles should be ignored in design calculations.

It is our experience that the driven pipe pile foundations should provide adequate support with total settlements on the order of ½-inch or less.

***Pile Specifications* -** We recommend that the following specifications be included on the foundation plan:

1. 6-inch or 8-inch diameter piles should consist of galvanized Schedule-40, ASTM A-53 Grade “A” pipe.

2. 6- and 8-inch piles shall be driven to refusal with a minimum 2,000-lb hydraulic hammer. The driving criteria will be determined based on the actual hammer size selected by the contractor, and a static load test program (see discussion in Item 4).

3. Piles shall be driven in nominal sections and connected with compression fitted sleeve couplers. We discourage welding of pipe joints, particularly when galvanized pipe is used, as we have frequently observed welds broken during driving.

4. At least one of the piles should be load tested. All load tests shall be performed in general accordance with the procedure outlined in ASTM D1143. The maximum test load shall be 2 times the design load. The objective of the testing program is to verify

the adequacy of the driving criteria, and the efficiency of the hammer used for the project.

5. The geotechnical engineer of record or his/her representative shall provide full time observation of pile installation and testing.

The quality of a pin pile foundation is dependent, in part, on the experience and professionalism of the installation company. We recommend that a company with experienced personnel be selected to install the piles.

***Lateral Forces* -** Lateral capacity of vertical pin piles should be ignored in design calculations. Lateral forces from wind or seismic loading may be resisted by the passive earth pressures acting against the pile caps and grade beams. Passive resistance values may be determined using an equivalent fluid weight of 300 pounds per cubic foot (pcf). This value includes a safety factor of at least 1.5 and assumes that properly compacted granular fill will be placed adjacent to and surrounding the pile caps and grade beams, and will extend a horizontal distance 2 times the height of the pile cap.

***Estimated Pile Length* -** The required pile length in order to develop the recommended pile capacity will depend on the actual driving conditions encountered, which are expected to vary across the site. For planning and cost estimating purposes, however, we estimate that the pile will need to be embedded in the underlying dense sand about 15 feet. Therefore, we estimate that an average pile length of about 45 to 55 feet will be needed below the existing ground surface. It should be noted that the pile capacity may not be achieved at the end of initial driving, and that the load test may need to occur after the pile set-up has occurred.

***Obstructions* –** Obstructions may be encountered within the fill soil at the site. Where possible, the obstructions should be removed to facilitate the pile driving. If obstructions cannot be removed, the structural engineer of record should be notified to revise the pile layout to accommodate moving the piles.

5.2.3 Augercast Pile Foundations – Decant Structure

Augercast piles are installed by drilling with a continuous flight hollow stem auger to the required depth, and pumping grout through the hollow stem of the auger as the auger is slowly withdrawn from the hole. After the auger is completely removed, steel reinforcement is placed in the grout-filled hole. The rate at which the auger is withdrawn must be consistent with the grout supply. If the auger is withdrawn too quickly, the pile will be under-grouted, resulting in "necking" of the pile, or contamination of grout materials from caving soil.

Augercast piles may be designed to withstand axial compression, axial uplift and lateral loads.

Axial Capacity - The piles should extend at least 50 feet below the existing ground surface. This would provide at least 10 feet of embedment into the medium dense to dense sand. Table 1 below summarizes our recommended ultimate axial pile capacities for static loading conditions for two different augercast pile diameters. A factor of safety of at least 2.5 and 1.2 shall be used to calculate the allowable pile capacities for static and transient loading cases, respectively. All capacities listed in Table 1 assume a pile embedment of at least 10 feet into the sand bearing stratum.

Table 1
Ultimate Axial Pile Capacities

Pile Diameter	Compression	Uplift
18-inch	235 kips	130 kips
24-inch	365 kips	180 kips

A Factor of Safety of at least **2.5** shall be used to compute the Allowable pile capacity for static loads.
A Factor of Safety of at least **1.2** shall be used to compute the Allowable pile capacity for transient loads.

Where pile groups will be needed due to heavy column loads, the auger-cast piles should be spaced a minimum distance of 3 times the pile diameter on center. We expect settlements of less than ½-inch for piles designed according to the above recommendations.

Lateral Capacity - Lateral loads on the building may be resisted by a combination of passive earth pressure against the buried portion of the foundation elements, and the lateral load capacity of the auger-cast piles. We recommend that an allowable passive pressure of 300 pcf be used in the design calculation. The recommended value includes a factor of safety of at least 1.5.

We also recommend that the following lateral pile capacity for design purposes, assuming a maximum allowable lateral deflection of ½ inch at the pile top and fixed head condition:

Table 2
Recommended Lateral Augercast Pile Capacities

Pile Diameter	Recommended Lateral Capacities
18-inch	15 kips
24-inch	20 kips

Drill Spoil Disposal – Installation of augercast piles will generate a considerable amount of spoil. In areas where soil contamination may be present, the disposal cost can be very significant. As a result, we recommend that an environmental assessment of the site be considered before the project design is finalized.

***Obstructions* -** Due to known fill at the project site, obstructions could be encountered during pile installations. Depending on the depth, size and orientation of the obstruction, augercast piles that encounter obstructions may have to be relocated if the obstruction cannot be penetrated or exhumed with conventional earthwork equipment.

5.2.4 Ground Improvement with Aggregate Piers – Decant Structure

A shallow foundation system consisting of a mat slab or a structural slab with thickened edges supported on improved ground is considered feasible to support the proposed decant station. In our opinion, a feasible soil improvement technique consists of improving the loose/soft to medium stiff silt and clay (Unit 1 and Unit 2) below the proposed structure with aggregate piers. Aggregate piers consist of compacting columns of well-graded crushed rock to increase the bearing capacity of poor soils, and to reduce settlements. Because the aggregate piers increase the stiffness of the subsurface soils, and provide additional drainage pathways for excess pore water pressure during a seismic event, the potential for earthquake induced liquefaction in the improved soils is also reduced. After the aggregate piers are installed, a mat slab or structural slab with thickened edges can be constructed directly on the improved soil. Because specialty contractors install aggregate piers using a proprietary system, the contractor determines the lengths and spacing of piers, the allowable soil bearing pressure of the improved soil, improved soil characteristics and anticipated settlements. Specifically, the specialty contractor is responsible for the foundation design, and will provide design drawings and calculations stamped by a registered professional engineer.

***Lateral Resistance* -** Lateral forces from wind or seismic loading may be resisted by a combination of passive earth pressures acting against the embedded portions of the mat slab foundation, and by friction acting on the base of the slab. Passive resistance values may be determined using an equivalent fluid weight of 300 pounds per cubic foot (pcf). This value includes a factor of safety of at least 1.5 assuming that properly compacted structural fill will be placed adjacent to the sides of the foundation. A friction coefficient of 0.3 may be used to determine the frictional resistance at the base of the slab. This coefficient includes a factor of safety of approximate 1.5.

5.2.5 Storage Bins

We understand that the new storage bins (covered or uncovered) may contain soil stockpiles up to about 10 feet high. As such, we anticipate ground pressures of up to 1,200 psf in the storage bin areas. Based on the results of our consolidation analyses of the underlying soft soils, we estimate that as much as 14 inches of settlement could occur under the anticipated bearing pressure without improvement to the soils. This magnitude of settlement likely will be excessive for the storage bin areas. The magnitude of the estimated settlement can be reduced by means of pre-loading. Assuming a 10-foot preload, we estimate that the foundation settlement likely will be reduced to about 1 to 3 inches of post-construction settlement. We estimate that a pre-load period of about 8 to 12 weeks will be needed. The actual pre-load period should be based on the actual settlement monitoring results.

Preloading of a site has long been used as a cost-effective way to reduce long-term settlements of structures located over soft, compressible soil deposits. Preloading consists of placing thick layers of fill over existing soils for a specified period, and then removing all or part of the fill. Properly designed and executed, preloading accelerates settlements, and reduces post-construction settlements to tolerable levels.

In addition to the preload, we also recommend that the storage bin areas be underlain by at least 12 inches of properly compacted structural fill, such as crushed surfacing base course. To improve the performance of the pavement surface in the storage bin area, we recommend that a layer of geogrid reinforcement (Tensar TX-160) be placed below the layer of crushed rock.

Because the actual magnitude of ground settlement during the preload could vary from our estimates, we recommend that the geogrid and crushed rock be placed after the preload fill has been removed. Details of preload construction and monitoring are outlined in Section 5.4 of this report.

5.3 CONCRETE SLAB FOR DECANT STRUCTURE

We recommend that a structural slab, in lieu of a conventional slab on grade, be utilized for the new decant structure to mitigate the risk of post-construction settlement and distress. We do not anticipate that the floor slabs of the decant station will be sensitive to moisture. However, if portions of the building will have floor treatments that are sensitive to moisture, the slabs may be constructed on a 4-inch thick capillary break material consisting of free-draining, crushed rock or well-graded gravel compacted to a firm and unyielding condition. The capillary break material should have no more than 10 percent passing the No. 4 sieve and less than 5 percent by weight of the material passing the U.S. Standard No. 100 sieve. A 10-mil polyethylene vapor barrier may also be placed below the slab for areas with moisture sensitive floor finishes.

5.4 PRELOAD

5.4.1 Preload Design and Construction

Prior to the placement of the preload fill, the existing asphalt pavement should be removed in the area of the preload. Preload fill may consist of structural fill, on-site native soils, or materials that are readily available locally. Density testing is not required for the preload fill, however, a reasonable compaction effort should be made by track-walking or wheel rolling with equipment. A total unit weight of 115 pounds per cubic foot (pcf) was assumed for the settlement analysis; preload fill material should not be less than 115 pcf. The preload surface should be crowned slightly to promote surface water drainage. Preload fill slopes should be constructed no steeper than 1H:1V.

Upon removal of the preload fills, the underlying subgrade should be proof rolled. Any soft or disturbed areas identified during proof rolling should be removed and replaced with properly compacted structural fill. Proof rolling and any remedial grading should be performed under the observation of the geotechnical engineer.

5.4.2 Preload Monitoring

Settlement of the preload fills should be monitored during and after fill placement. Monitoring of settlements should be performed using settlement plates, as shown on the Settlement Plate Schematic, Figure 4. The location, type, and frequency of reading the settlement plates should be determined prior to grading. The settlement monitoring program should be performed by a licensed surveyor and evaluated by PanGEO, to determine the timing of preload fill removal.

The settlement plates should be installed on firm ground or on sand pads if needed for stability. Plates should be installed at elevations shown on the project plans, prior to placing any fill above the plate level. Locations of settlement plates should be clearly marked and readily visible (red flagged) to equipment operators.

The contractor should maintain at least a 5-foot horizontal clearance with heavy equipment. Fill within the clearance area should be hand compacted to project specifications. In the event of damage to a settlement plate or measurement rod resulting from equipment operating within the prescribed clearance area, the contractor should immediately notify the geotechnical engineer and should be responsible for restoring the settlement plate to working order.

A licensed land surveyor should be retained to monitor the settlement plates. Initial readings should be taken on top of the measurement rod and at the adjacent ground level prior to fill placement. For ease in handling, the measurement rod and casing may be installed in 3- to 5-foot sections. As fill progresses, couplings should be used to install additional lengths. Continuity should be maintained when adding extensions, by reading the top of the measurement rod, then immediately adding the new section and reading the top of the added rod. Both readings should be recorded.

After initial placement, readings should be taken twice weekly during fill placement, and once weekly thereafter. At the recommended time intervals, the licensed land surveyor should record the elevation of the top of the measurement rod, and note the elevation of the adjacent fill surface. The measurement rod readings should be to the nearest 0.01 foot (or the nearest 0.005 foot if possible); fill elevations should be measured to the nearest 0.1 foot. All elevations should be referenced to a benchmark located on stable ground at least 100 feet from the preload embankment.

Readings should be submitted in a timely fashion to the geotechnical engineer, who should analyze the settlement data, to provide a basis for determining when the desired effect of preloading has been achieved. Preload soils should not be removed until authorized by the geotechnical engineer.

5.5 PAVEMENT

5.5.1 Storage Bin Pavement

As described above, we recommend that the area of the new storage bins be preloaded to reduce the magnitude of long-term settlement. After the preload has been removed, we recommend that a pavement section consisting of a minimum of 6 inches of hot mix asphalt (HMA) over 12 inches of crushed surfacing base course (CSBC) be utilized in the storage bin areas. To improve the performance of the pavement, we recommend, that a layer of geogrid reinforcement (Tensar TX-160) be placed below the layer of crushed rock. Prior to placing the geogrid, the area of the storage bins should be proof-rolled with a heavy roller (7 to 10 tons) to a firm condition. The crushed rock base should be compacted to a minimum of 95% of the materials maximum dry density (Modified Proctor ASTM D-1557).

5.5.2 Site Pavements

We understand that new pavement will be constructed at the decant facility. Assuming the pavement will be used by the vactor trucks and other heavy trucks, as a minimum, the new pavement section should consist of 4-inches HMA, overlying an 8-inch thick layer of crushed surfacing base course (CSBC), overlying a properly compacted subgrade. Both the subgrade and crushed rock base should be compacted to a minimum of 95% of the materials maximum dry density (Modified Proctor ASTM D-1557).

Where Portland cement concrete pavement will be used to support heavy trucks, we recommend a minimum pavement section of 8 inches of concrete (4,000 psi minimum) on 6 inches crushed surfacing base/top course compacted to at least 95% of its maximum dry density (ASTM D1557).

5.6 BELOW-GRADE WALLS

Below-grade structures, such as the walls of the basins or storage tanks, should be designed to resist at-rest lateral earth pressures. The buried walls should be designed for an equivalent fluid pressure of 55 pounds per cubic foot (pcf). This recommendation assumes that the backfill behind the subsurface walls will consist of properly compacted structural fill. If the below-grade walls will be subjected to the influence of truck surcharge loading within a horizontal distance equal to or less than the height of the walls, the surcharge pressure may be calculated based on an additional 2 feet of retained soil.

5.7 UNDERGROUND UTILITIES

5.7.1 Pipe Support and Bedding

Based on our field explorations, we anticipate the exposure of variable, but generally adequate subsoil conditions at pipe invert elevations less than about 5 feet below the ground surface. Generally, loose to medium dense silty fine to medium sand with some gravel was encountered within about 5 feet of the ground surface. Below 5 feet, soft organic silt and peat was encountered. In our opinion the relatively undisturbed silty sand and sands should provide suitable support for the proposed pipelines; however, for utilities deeper than about 5 feet, if soft peat, clay, or organic-rich soil is exposed along the bottom of any trench, we recommend about 6 to 12 inches of the soft soils be removed and replaced with additional bedding material.

Underground utilities associated with the project should be placed, bedded, and backfilled in accordance with WSDOT Standard Specification 7-04 (storm sewers), 7-10 (water mains) and 7-17 (sanitary sewers), or other applicable specifications. In general, pipe bedding materials should be placed in loose lifts not exceeding 6 inches in thickness, and compacted to a minimum relative compaction of 95 percent maximum dry density, per ASTM D1557. Bedding materials and thicknesses provided should be suitable for the utility system and materials installed, and in accordance with any applicable manufacturers' recommendations.

Pipe bedding materials should be placed on relatively undisturbed native soil, or compacted structural fill soils. If the native subgrade soils are disturbed, the disturbed material should be removed and replaced with compacted structural fill or bedding material.

5.7.2 Trench Backfill

Beneath structural or paved areas, we recommend that trench backfill be select granular material, meeting the requirements for structural fill. During placement of the initial lifts, the trench backfill material should not be bulldozed into the trench or dropped directly on the pipe. Furthermore, heavy vibratory equipment should not be permitted to operate directly over the pipe until a minimum of 3 feet of backfill has been placed.

In order to minimize subsequent settlement of the trench backfill, it is recommended that the trench backfill be placed in 8- to 12-inch, loose lifts and compacted using mechanical equipment to about 90 percent maximum dry density, as determined by Standard Proctor (per ASTM D698). In structural or paved areas, the upper 2 feet of the backfill should be compacted to at least 95 percent maximum dry density, per ASTM D1557.

It is anticipated that selected excavation spoils may be used as trench backfill if they are placed at or near optimum content and proper compaction control is utilized. In our opinion the top approximately 4 to 5 feet of soil at the site (sand and silty sand) may be potentially re-used as trench backfill. However, some of the soils may be too wet to achieve the recommended compaction requirements. If the material is not compacted as recommended, the potential for backfill settlement will be increased. Below a depth of about 5 feet, the organic silt and clay will not be suitable for re-use as trench backfill.

Underground utilities should be designed to accommodate differential and total settlements on the order of several inches over the design life of the project. Utilities constructed within or adjacent to preload areas should be installed after preloading.

6.0 CONSTRUCTION CONSIDERATIONS

6.1 Pavement Subgrade Preparation

All existing asphalt and debris should be removed from the area prior to subgrade preparation. Prior to crushed rock placement, the exposed subgrade should be compacted with a heavy roller to compact the underlying subgrade to a dense and unyielding condition. Any areas of loose or soft soils that cannot be adequately compacted should be removed and replaced with properly compacted structural fill.

6.2 Structural Fill and Compaction

If structural fill is needed, we recommend using a granular fill material such as Gravel Borrow (WSDOT 9-03.14(1)) or other approved equivalent. The structural fill should be moisture conditioned to within about 3 percent of optimum moisture content, placed in loose, horizontal lifts less than 8 inches in thickness, and systematically compacted to a dense and unyielding condition, and to at least 95 percent of the maximum dry density, as determined using test method ASTM D 1557.

6.3 TEMPORARY EXCAVATION AND DEWATERING

Maximum temporary excavation depths are expected to be about 4 to 5 feet deep. Temporary excavations greater than 4 feet deep should be properly sloped or shored. All temporary excavations should be performed in accordance with Part N of WAC (Washington Administrative Code) 296-155. The contractor is responsible for maintaining safe excavation slopes and/or shoring. For planning purposes, the temporary excavations in fill may be sloped to as steep as 1.5H:1V (Horizontal:Vertical). The temporary cut slopes should be re-evaluated by a representative of PanGEO during construction based on actual observed soil conditions.

We expect excavations deeper than about 3 feet below grade to encounter seepage from perched groundwater in the upper fill material. In addition, the static groundwater level should be assumed to be within about 10 feet of the ground surface. Due to the fine grain nature of the soils at the site, we anticipate that groundwater seepage in the temporary excavations can be controlled with sumps and pumps.

6.4 DRAINAGE CONSIDERATIONS

Surface runoff can be controlled during construction by careful grading practices. This may include the construction of shallow, upgrade perimeter ditches or low earthen berms to collect runoff and prevent water from entering the excavation. All collected water should be directed to a positive and permanent discharge system such as a storm sewer. It should be noted that some of the site soils are prone to surficial erosion. Special care should be taken to avoid surface water on open cut excavations, and exposed slopes should be protected with plastic sheeting.

Permanent control of surface water and roof runoff should be incorporated in the final grading design. In addition to these sources, irrigation and rain water infiltrating into any landscape and/or planter areas adjacent to paved areas or building foundations should also be controlled. Water should not be allowed to pond immediately adjacent to buildings or paved areas. All collected runoff should be directed into conduits that carry the water away from pavements or the structure and into storm drain systems or other appropriate outlets. Adequate surface gradients should be incorporated into the grading design such that surface runoff is directed away from structures.

6.5 Wet Weather Earthwork and Erosion Considerations

The site soils contain a moderate to high amount of fines, and are therefore considered moisture sensitive. As a result, it may be more economical to perform earthwork in the drier summer months to reduce the potential of site soils becoming soft due to excessive moisture. Any softened soils should be removed and replaced with structural fill.

General recommendations relative to earthwork performed in wet weather or in wet conditions are presented below:

- Because site soils are considered moisture sensitive, all subgrade surfaces should be protected against inclement weather.
- Earthwork should be performed in small areas to minimize subgrade exposure to wet weather. Excavation or the removal of unsuitable soil should be followed promptly by the placement and compaction of structural fill. The size and type of construction equipment used may have to be limited to reduce soil disturbance.
- During wet weather, the allowable fines content of the structural fill should be reduced to no more than 5 percent by weight based on the portion passing ¾-inch sieve. The fines should be non-plastic.
- The ground surface within the construction area should be graded to promote run-off of surface water and to prevent the ponding of water, and to prevent surface water from entering the excavations.
- Bales of straw and/or geotextile silt fences should be strategically located to control erosion and the movement of sediment. Erosion control measures should be installed along all the property boundaries.
- Excavation slopes and soils stockpiled on site should be covered with plastic sheeting.
- Under no circumstances should soil be left uncompacted and exposed to moisture.

7.0 LIMITATIONS

We have prepared this report for use by Gray and Osborne Inc., the City of Marysville, and the project team. Recommendations contained in this report are based on a site reconnaissance, a subsurface exploration program, review of pertinent subsurface information, and our understanding of the project. The study was performed using a mutually agreed-upon scope of work.

Variations in soil conditions may exist between the explorations and the actual conditions underlying the site. The nature and extent of soil variations may not be evident until construction occurs. If any soil 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. Additionally, we should also be notified to review the applicability of our recommendations if there are any changes in the project scope.

The scope of our work does not include services related to construction safety precautions. Our recommendations are not intended to direct the contractors' methods, 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. We are not mold consultants nor are our recommendations to be interpreted as being preventative of mold development. A mold specialist should be consulted for all mold-related issues.

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 other 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. PanGEO should be notified if the project is delayed by more than 24 months from the date of this report so that we may review the applicability of our conclusions considering the time lapse.

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 PanGEO of such intended use and for permission to copy this report. Based on the intended use of the report, PanGEO may require that additional work be performed and that an updated report be reissued. Noncompliance with any of these requirements will release PanGEO from any liability resulting from the use this report.

Within the limitation of scope, schedule and budget, PanGEO engages in the practice of geotechnical engineering and endeavors to perform its services in accordance with generally accepted professional principles and practices at the time the Report or its contents were prepared. No warranty, express or implied, is made.

We appreciate the opportunity to be of service to you on this project. Please feel free to contact our office with any questions you have regarding our study, this report, or any geotechnical engineering related project issues.

Sincerely,

PanGEO, Inc.

JON C. REHKOPF
STATE OF WASHINGTON
40412
REGISTERED
PROFESSIONAL ENGINEER
1/16/13
EXPIRES 7/4/2014

Jon C. Rehkopf, P.E.
Senior Project Geotechnical Engineer

Siew L. Tan, P.E.
Principal Geotechnical Engineer

8.0 REFERENCES

International Building Code (IBC), 2009, International Code Council.

Minard, J.P. (1985), *Geologic map of the Marysville quadrangle, Snohomish County, Washington.* USGS, Map Scale: 1:24,000.

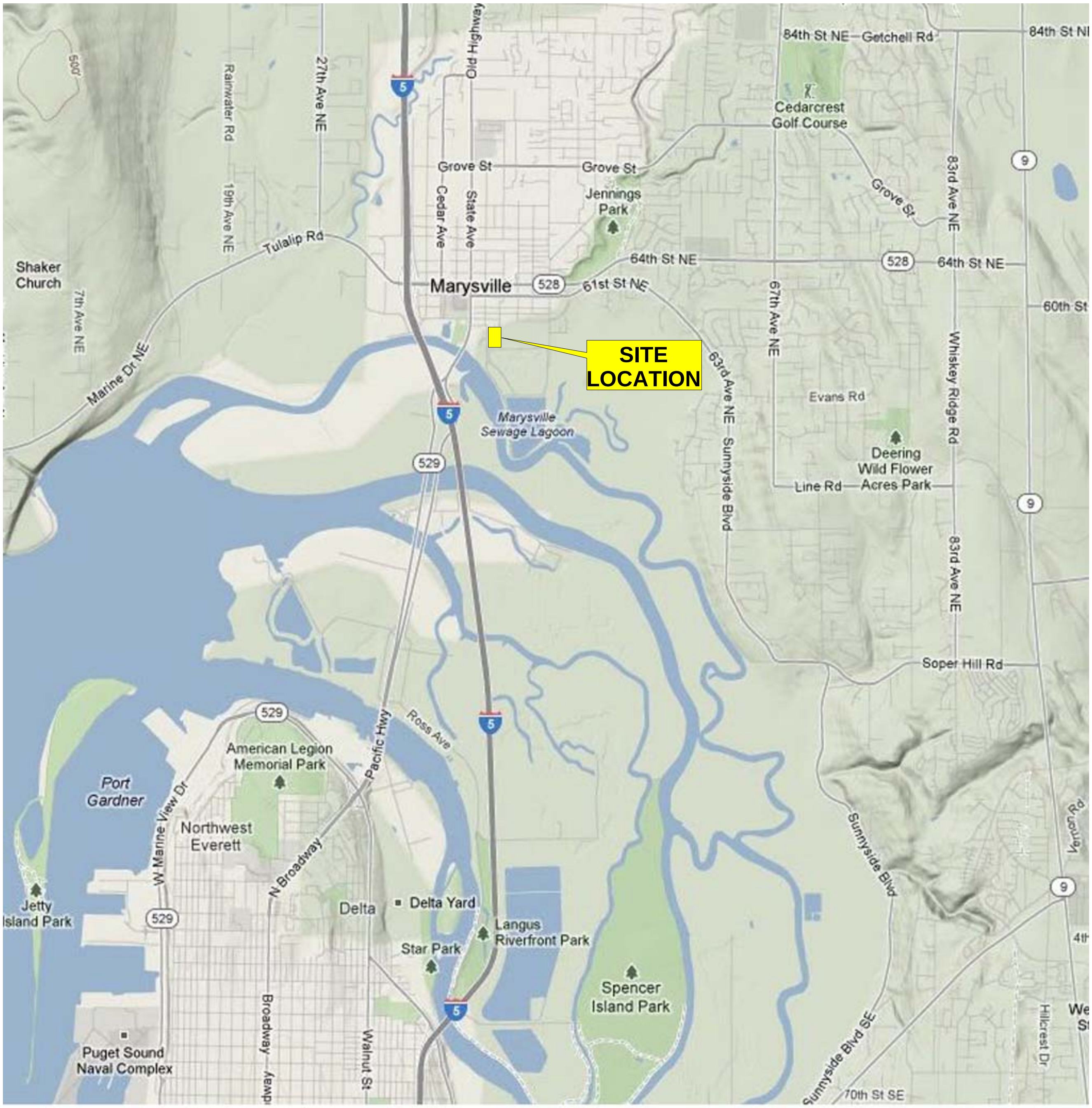


Image Credit: Google Maps

Not To Scale

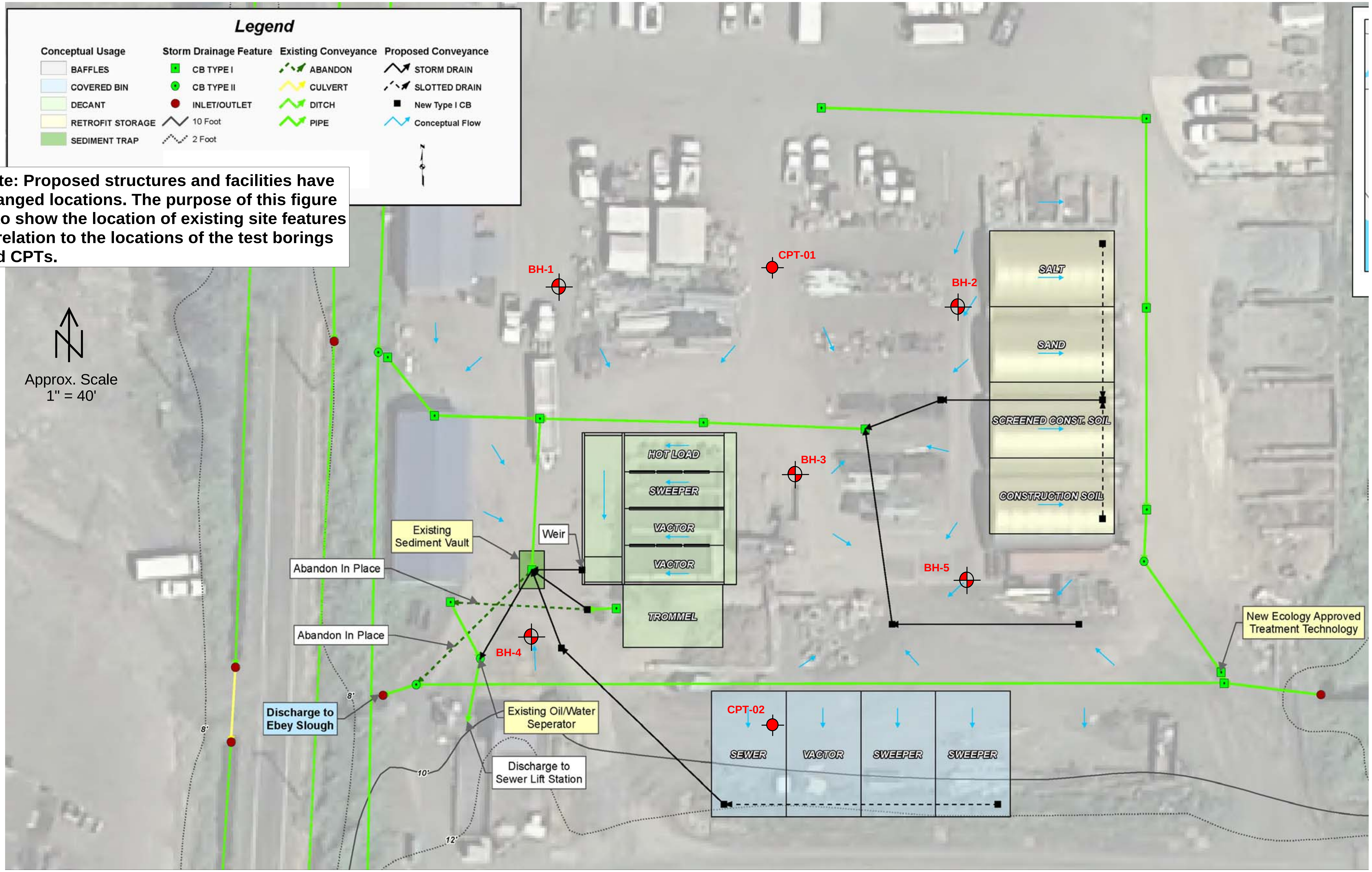
PanGEO INCORPORATED

Marysville Decant Facility
80 Columbia Avenue
Marysville, Washington

VICINITY MAP

Project No.	Figure No.
12-163	1

12-009 Fig 1 Vicinity.grf 1/14/13 STS



Legend:

CPT-01 Approximate CPT Location

BH-1 Approximate Test Boring Location

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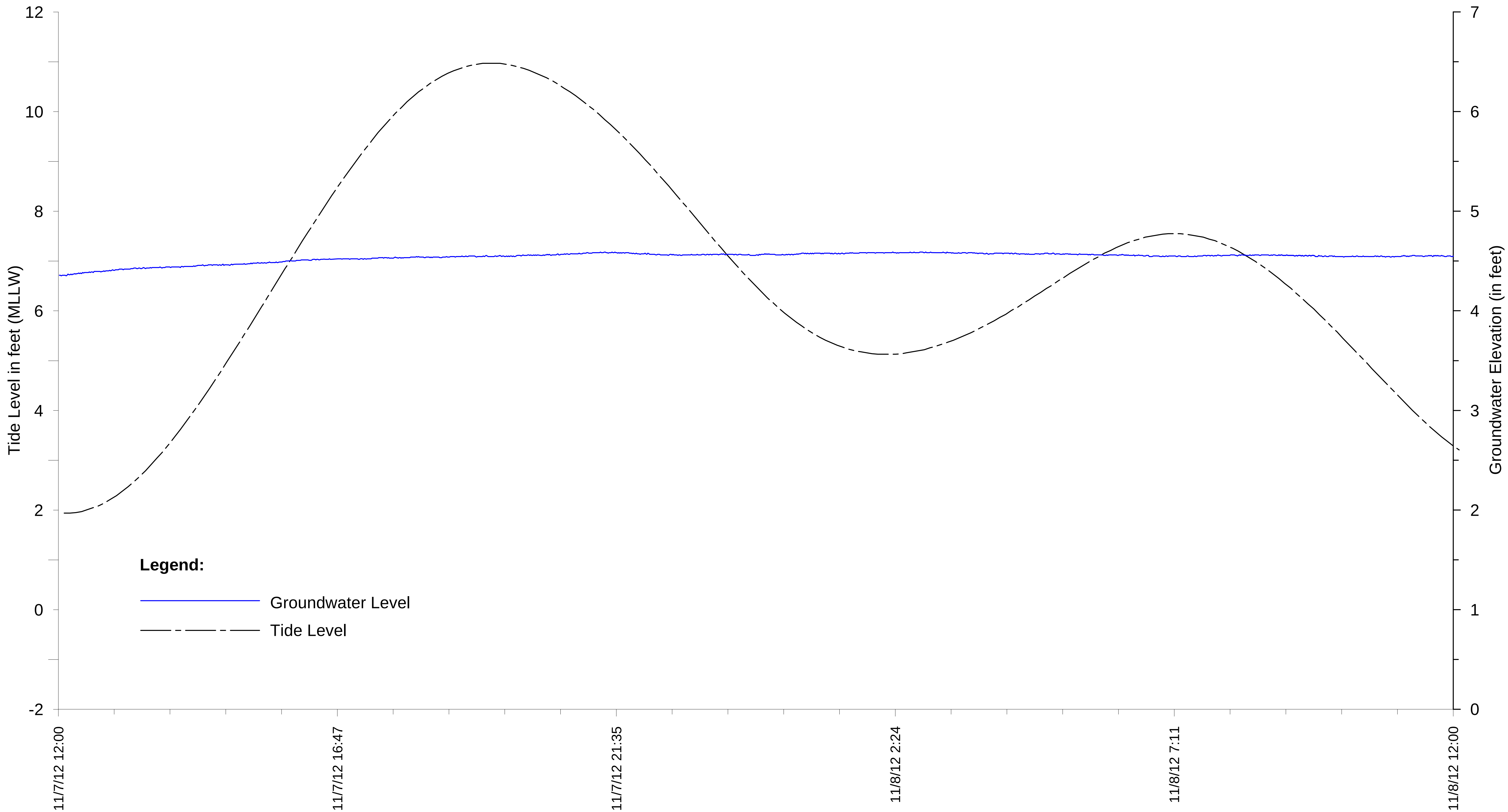
Marysville Decant Facility
80 Columbia Avenue
Marysville, Washington

SITE AND EXPLORATION PLAN

Project No.	Figure No.
12-163	2

Proposed Boring Plan.grf 1/15/13 (17:13) JCR



Notes: 1. Interpreted reference elevation at top of monitoring well is 8.5 feet.

2. Datum for tidal elevation is MLLW.

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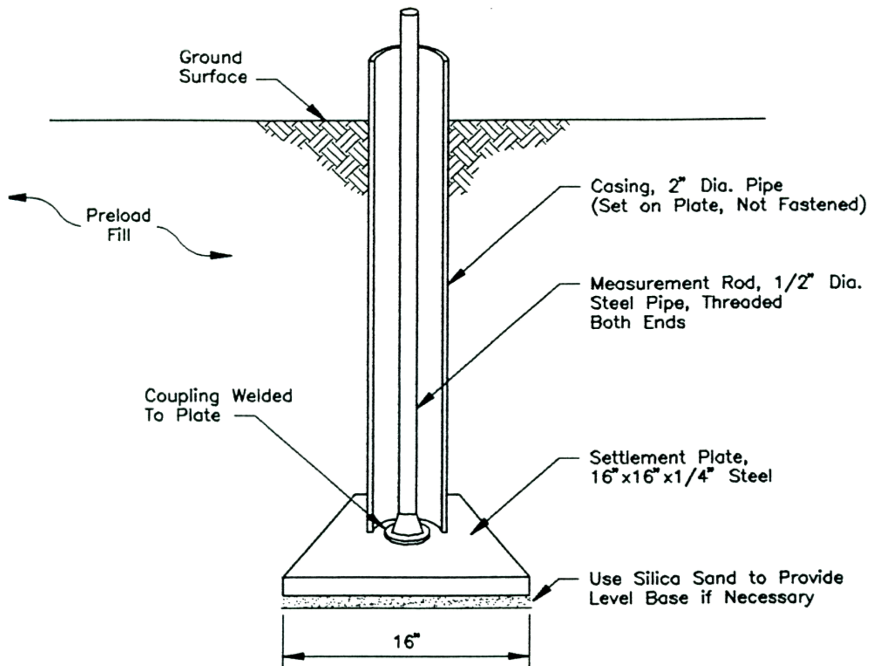
Marysville Decant Facility
80 Columbia Avenue
Marysville, Washington

GROUNDWATER AND TIDAL LEVELS
November 7 - 8, 2012

Project No. 12-163 | Figure No. 3

Groundwater - Tidal Interactions.grf 1/14/13 (13:25) SHE



NOTES:

1. Locations of settlement plates shall be clearly marked and readily visible to equipment operators.
2. Contractor shall maintain at least 5 feet of horizontal clearance for heavy equipment from the base of settlement plates. Fill within 5 feet of settlement plates shall be compacted using hand operated equipment.
3. In the event of damage to settlement plates, contractor shall immediately notify PanGEO and shall be responsible for restoring the settlement plates to working order.
4. Install plates on firm ground or on sand pads if needed for stability. Take initial readings on top of the rod and at adjacent ground level prior to placement of fill.
5. For ease in handling, rod and casing are usually installed in short sections. As fill progresses, couplings are used to installed additional lengths. Continuity is maintained by reading the top of the measurement rod, then immediately adding the new section and reading the top of the added rod. Both readings should be recorded.
6. Record the elevations of the top of the measurement rod at pre-determined time intervals, but no less than once a week. Each time, note the elevation of the adjacent ground surface.
7. Read the measurement rods to the nearest 0.01 ft, or 0.005 ft if possible. Note the fill elevation to the nearest 0.1 ft.
8. The elevations should be referenced to a temporary benchmark located on stable ground at least 100 feet from the preload fill.

Fig4_settlement plate.ppt 1/14/2013(10:12 AM) JCR

PanGEO INCORPORATED

Marysville Decant Facility
80 Columbia Avenue
Marysville, Washington

SETTLEMENT PLATE SCHEMATIC

Project No.	Figure No.
12-163	4

APPENDIX A

SUMMARY BORING LOGS

RELATIVE DENSITY / CONSISTENCY

SAND / GRAVEL			SILT / CLAY		
Density	SPT N-values	Approx. Relative Density (%)	Consistency	SPT N-values	Approx. Undrained Shear Strength (psf)
Very Loose	<4	<15	Very Soft	<2	<250
Loose	4 to 10	15 - 35	Soft	2 to 4	250 - 500
Med. Dense	10 to 30	35 - 65	Med. Stiff	4 to 8	500 - 1000
Dense	30 to 50	65 - 85	Stiff	8 to 15	1000 - 2000
Very Dense	>50	85 - 100	Very Stiff	15 to 30	2000 - 4000
			Hard	>30	>4000

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS		GROUP DESCRIPTIONS	
Gravel 50% or more of the coarse fraction retained on the #4 sieve. Use dual symbols (eg. GP-GM) for 5% to 12% fines.	GRAVEL (<5% fines)	GW	Well-graded GRAVEL
		GP	Poorly-graded GRAVEL
	GRAVEL (>12% fines)	GM	Silty GRAVEL
		GC	Clayey GRAVEL
Sand 50% or more of the coarse fraction passing the #4 sieve. Use dual symbols (eg. SP-SM) for 5% to 12% fines.	SAND (<5% fines)	SW	Well-graded SAND
		SP	Poorly-graded SAND
	SAND (>12% fines)	SM	Silty SAND
		SC	Clayey SAND
Silt and Clay 50%or more passing #200 sieve	Liquid Limit < 50	ML	SILT
		CL	Lean CLAY
		OL	Organic SILT or CLAY
	Liquid Limit > 50	MH	Elastic SILT
		CH	Fat CLAY
		OH	Organic SILT or CLAY
Highly Organic Soils		PT	PEAT

Notes:

1. Soil exploration logs contain material descriptions based on visual observation and field tests using a system modified from the Uniform Soil Classification System (USCS). Where necessary laboratory tests have been conducted (as noted in the "Other Tests" column), unit descriptions may include a classification. Please refer to the discussions in the report text for a more complete description of the subsurface conditions.

2. The graphic symbols given above are not inclusive of all symbols that may appear on the borehole logs. Other symbols may be used where field observations indicated mixed soil constituents or dual constituent materials.

DESCRIPTIONS OF SOIL STRUCTURES

Layered: Units of material distinguished by color and/or composition from material units above and below

Laminated: Layers of soil typically 0.05 to 1mm thick, max. 1 cm

Lens: Layer of soil that pinches out laterally

Interlayered: Alternating layers of differing soil material

Pocket: Erratic, discontinuous deposit of limited extent

Homogeneous: Soil with uniform color and composition throughout

Fissured: Breaks along defined planes

Slickensided: Fracture planes that are polished or glossy

Blocky: Angular soil lumps that resist breakdown

Disrupted: Soil that is broken and mixed

Scattered: Less than one per foot

Numerous: More than one per foot

BCN: Angle between bedding plane and a plane normal to core axis

COMPONENT DEFINITIONS

COMPONENT	SIZE / SIEVE RANGE	COMPONENT	SIZE / SIEVE RANGE
Boulder:	> 12 inches	Sand	
Cobbles:	3 to 12 inches	Coarse Sand:	#4 to #10 sieve (4.5 to 2.0 mm)
Gravel		Medium Sand:	#10 to #40 sieve (2.0 to 0.42 mm)
Coarse Gravel:	3 to 3/4 inches	Fine Sand:	#40 to #200 sieve (0.42 to 0.074 mm)
Fine Gravel:	3/4 inches to #4 sieve	Silt	0.074 to 0.002 mm
		Clay	<0.002 mm

TEST SYMBOLS

for In Situ and Laboratory Tests listed in "Other Tests" column.

ATT	Atterberg Limit Test
Comp	Compaction Tests
Con	Consolidation
DD	Dry Density
DS	Direct Shear
%F	Fines Content
GS	Grain Size
Perm	Permeability
PP	Pocket Penetrometer
R	R-value
SG	Specific Gravity
TV	Torvane
TXC	Triaxial Compression
UCC	Unconfined Compression

SYMBOLS

Sample/In Situ test types and intervals

- 2-inch OD Split Spoon, SPT (140-lb. hammer, 30" drop)
- 3.25-inch OD Spilt Spoon (300-lb hammer, 30" drop)
- Non-standard penetration test (see boring log for details)
- Thin wall (Shelby) tube
- Grab
- Rock core
- Vane Shear

MONITORING WELL

- Groundwater Level at time of drilling (ATD)
- Static Groundwater Level
- Cement / Concrete Seal
- Bentonite grout / seal
- Silica sand backfill
- Slotted tip
- Slough
- Bottom of Boring

MOISTURE CONTENT

Dry	Dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water

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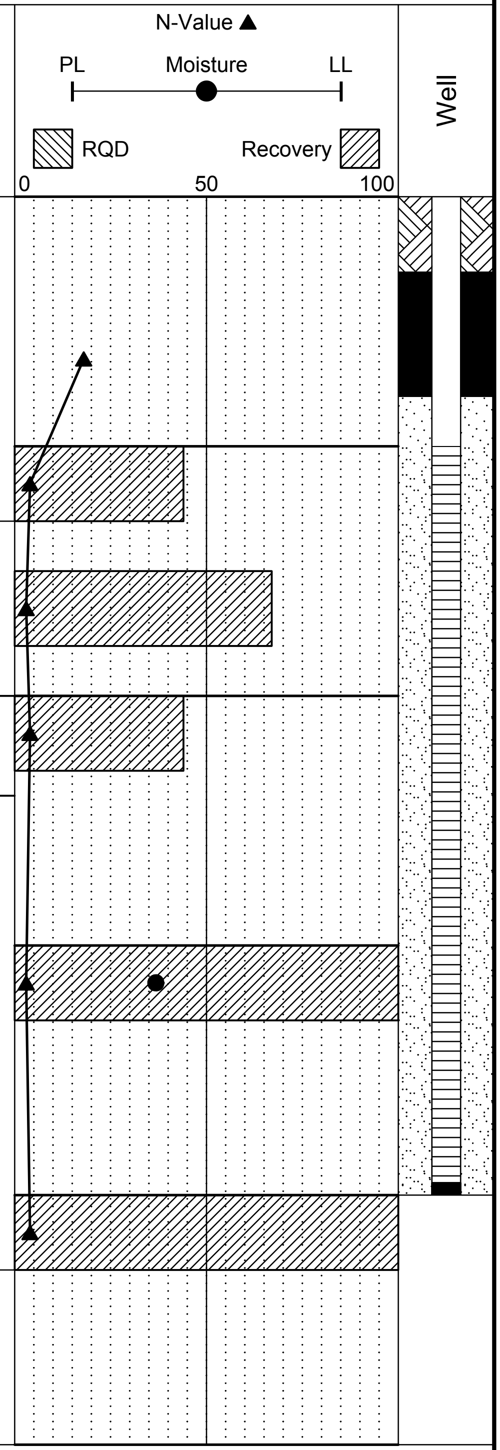
Terms and Symbols for Boring and Test Pit Logs

Figure A-1

Project:	Decant Facility	Surface Elevation:	~9 ft
Job Number:	12-163	Top of Casing Elev.:	~8 1/2 ft
Location:	Marysville, Washington	Drilling Method:	Track-mounted HSA
Coordinates:	Northing: , Easting:	Sampling Method:	SPT w/ cathead

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0						3 1/2 inches of HMA over 3 to 4 inches of crushed rock over loose to medium dense, moist to wet, brown/gray, silty fine to medium SAND with trace gravel and scattered organics (Fill).
	S-1		13 11 7			No recovery (driving on rock?).
5	S-2		3 2 2			
						Soft, moist to wet, dark brown, layered organic SILT with trace fine sand and PEAT.
	S-3		2 2 1			
10	S-4		1 3 1			Loose, moist to wet, gray-brown, silty fine to medium SAND; 3 inches of wood in tip.
						Soft, moist to wet, gray, silty CLAY with trace fine sand and scattered organic bits.
15	S-5		1 1 2			Sample is wet to saturated.
20	S-6		2 2 2			
						Bottom of boring.
25						



Completion Depth: 21.5ft
Date Borehole Started: 10/29/12
Date Borehole Completed: 10/29/12
Logged By: JCR
Drilling Company: Boretec

Remarks: No water in hole at time of drilling. Based on the moisture content of the samples, groundwater is anticipated to be between about 10 and 15 feet below the ground surface.

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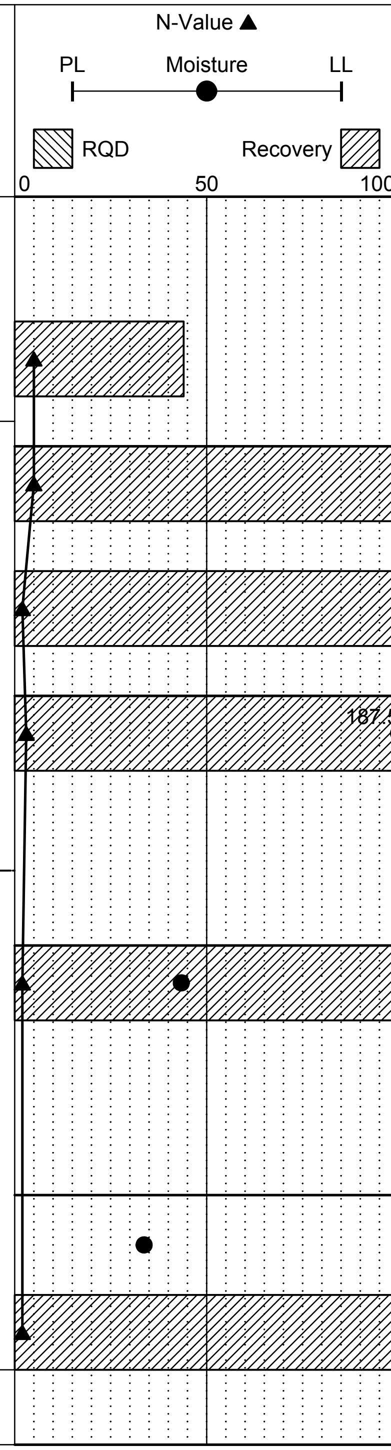
LOG OF TEST BORING BH-1

Figure A-2

The stratification lines represent approximate boundaries. The transition may be gradual.

Project:	Decant Facility	Surface Elevation:	~9 ft
Job Number:	12-163	Top of Casing Elev.:	n/a
Location:	Marysville, Washington	Drilling Method:	Track-mounted HSA
Coordinates:	Northing: , Easting:	Sampling Method:	SPT w/ cathead

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0						3 3/4 inches of HMA over 3 to 4 inches of crushed rock over loose, moist, brown/gray, silty fine to medium SAND with some gravel (Fill).
	S-1		3 3 2			
5						Soft to medium stiff, moist to wet, brown and gray, layered clayey organic SILT and PEAT.
	S-2		2 2 3			
	S-3		1 1 1			6 inch layer of soft, moist to wet, gray, CLAY with some fiberous organics.
10	S-4		1 1 2			Becomes layered gray CLAY with prevalent fiberous organics and red-brown organic silt and PEAT.
						Soft, wet to saturated, gray, silty CLAY with trace fine sand and some organic bits.
15	S-5		2 1 1			
20	S-6					No recovery from Shelby Tube.
	S-7		2 1 1			
						Bottom of boring.
25						



Completion Depth:	23.5ft
Date Borehole Started:	10/29/12
Date Borehole Completed:	10/29/12
Logged By:	JCR
Drilling Company:	Boretec

Remarks: No water in hole at time of drilling. Based on the moisture content of the samples, groundwater is anticipated to be between about 10 and 15 feet below the ground surface.

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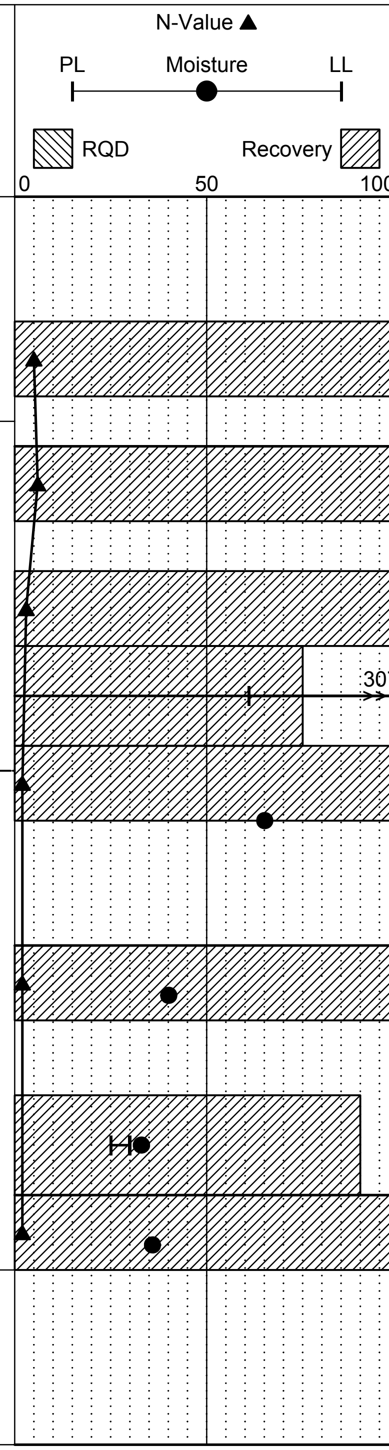
LOG OF TEST BORING BH-2

Figure A-3

The stratification lines represent approximate boundaries. The transition may be gradual.

Project:	Decant Facility	Surface Elevation:	~9 ft
Job Number:	12-163	Top of Casing Elev.:	n/a
Location:	Marysville, Washington	Drilling Method:	Track-mounted HSA
Coordinates:	Northing: , Easting:	Sampling Method:	SPT w/ cathead

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0						3 1/2 inches of HMA over 3 to 4 inches of crushed rock over loose, wet to saturated, brown/gray, silty fine to medium SAND with trace gravel and scattered organics (Fill).
	S-1		4 3 2			
5						Soft to medium stiff, moist to wet, brown and gray, layered clayey organic SILT with some sand and PEAT.
	S-2		3 3 3			
	S-3		2 2 1			Becomes dark brown to red-brown organic silt and PEAT (very fiberous).
10	S-4			Con		Organic Content: 11.1%.
	S-5		1 1 1			Soft, wet, gray, silty CLAY with trace fine sand and some organic bits.
15	S-6		1 1 1			Becomes wet to saturated.
	S-7			Con		
20	S-8		1 1 1			No organics.
						Bottom of boring.
25						



Completion Depth:	21.5ft
Date Borehole Started:	10/29/12
Date Borehole Completed:	10/29/12
Logged By:	JCR
Drilling Company:	Boretec

Remarks: No water in hole at time of drilling. Perched water between about 1 and 4 1/2 feet deep. Based on the moisture content of the samples, groundwater is anticipated to be between about 10 and 15 feet below the ground surface.

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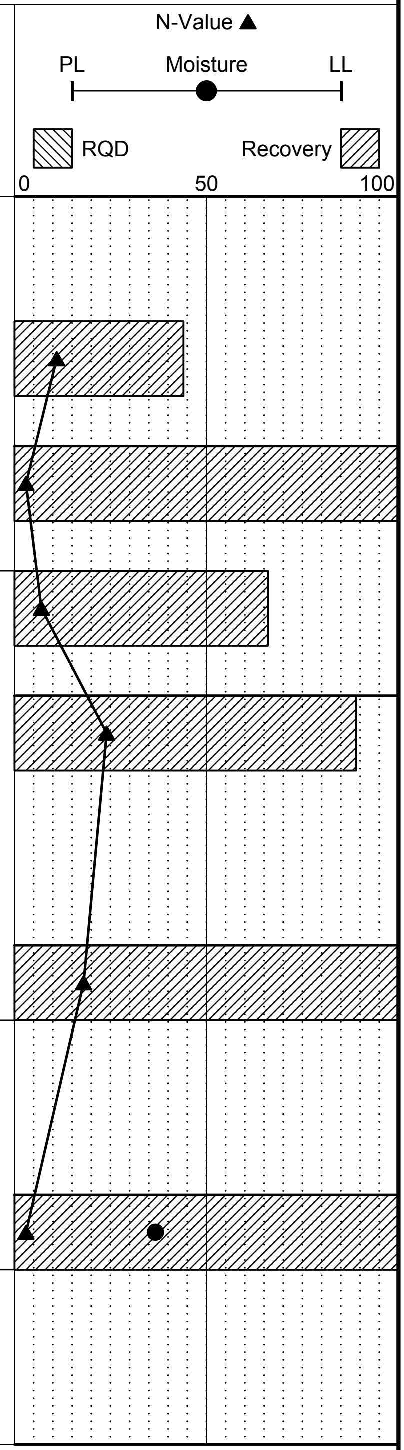
LOG OF TEST BORING BH-3

Figure A-4

The stratification lines represent approximate boundaries. The transition may be gradual.

Project:	Decant Facility	Surface Elevation:	~9 ft
Job Number:	12-163	Top of Casing Elev.:	n/a
Location:	Marysville, Washington	Drilling Method:	Track-mounted HSA
Coordinates:	Northing: , Easting:	Sampling Method:	SPT w/ cathead

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0						Sand/gravel surfacing over medium dense, moist, gray/brown, silty SAND with some to trace gravel (Fill).
	S-1		8 6 5			
5	S-2		2 1 2			Becomes loose, saturated, silty medium SAND with trace organics.
	S-3		8 5 2			Loose to medium dense, saturated, speckled gray/brown/white, medium to coarse SAND with some gravel, trace to no silt.
10	S-4		7 12 12			Gravelly drilling between about 11 1/2 feet and 16 1/2 feet.
15	S-5		14 13 5			
						Soft, wet to saturated, gray, silty CLAY with some fine to medium sand and trace organic bits.
20	S-6		2 1 2			
						Bottom of boring.
25						



Completion Depth:	21.5ft
Date Borehole Started:	10/29/12
Date Borehole Completed:	10/29/12
Logged By:	JCR
Drilling Company:	Boretec

Remarks: Groundwater about 5 feet below surface grade at time of drilling.

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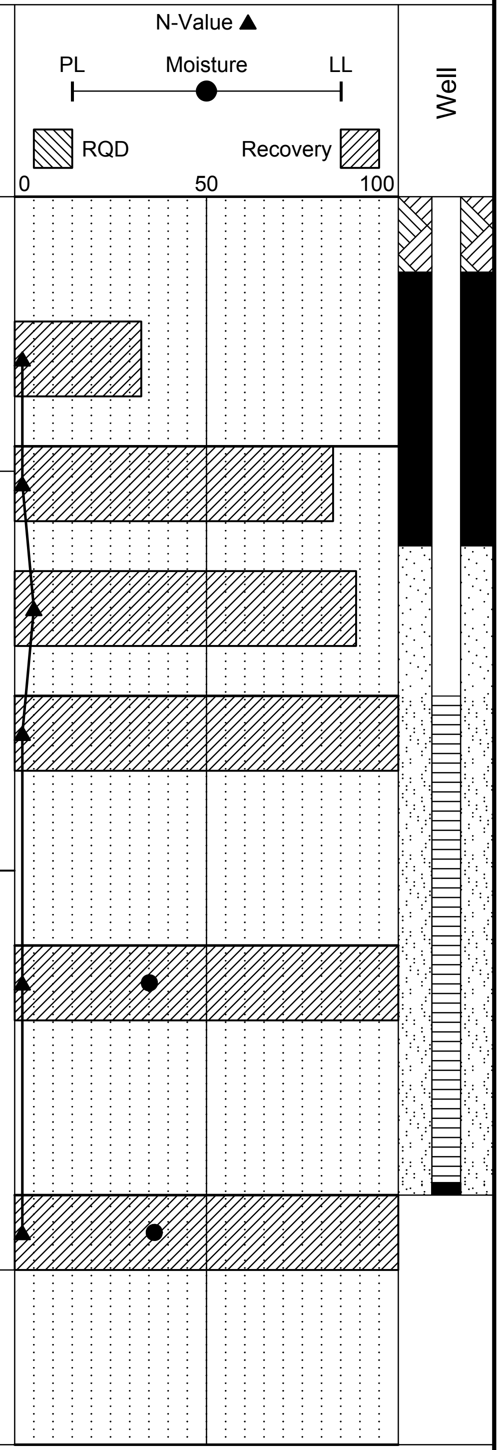
LOG OF TEST BORING BH-4

Figure A-5

The stratification lines represent approximate boundaries. The transition may be gradual.

Project:	Decant Facility	Surface Elevation:	~9 ft
Job Number:	12-163	Top of Casing Elev.:	~8 1/2 ft
Location:	Marysville, Washington	Drilling Method:	Track-mounted HSA
Coordinates:	Northing: , Easting:	Sampling Method:	SPT w/ cathead

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0						3 1/2 inches of HMA over 3 to 4 inches of crushed rock over very loose, wet to saturated, brown/gray, silty medium SAND with trace gravel (Fill).
	S-1		2 1 1			
5	S-2		1 1 1			Soft to medium stiff, moist to wet, brown and gray, layered clayey organic SILT and PEAT.
	S-3		3 3 2			
10	S-4		2 1 1			Becomes red-brown layers of CLAY with prevalent fiberous organics and PEAT.
						Soft, wet to saturated, gray, silty CLAY with trace fine sand and scattered organic bits.
15	S-5		2 1 1			
20	S-6		1 1 1			
						Bottom of boring.
25						



Completion Depth: 21.5ft
Date Borehole Started: 10/29/12
Date Borehole Completed: 10/29/12
Logged By: JCR
Drilling Company: Boretec

Remarks: Approximately 1 1/2 feet of water in bottom of hole at end of drilling. Perched water between about 1 and 5 1/2 feet deep. Based on the moisture content of the samples, groundwater is anticipated to be between about 10 and 15 feet below the ground surface.

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LOG OF TEST BORING BH-5

Figure A-6

The stratification lines represent approximate boundaries. The transition may be gradual.

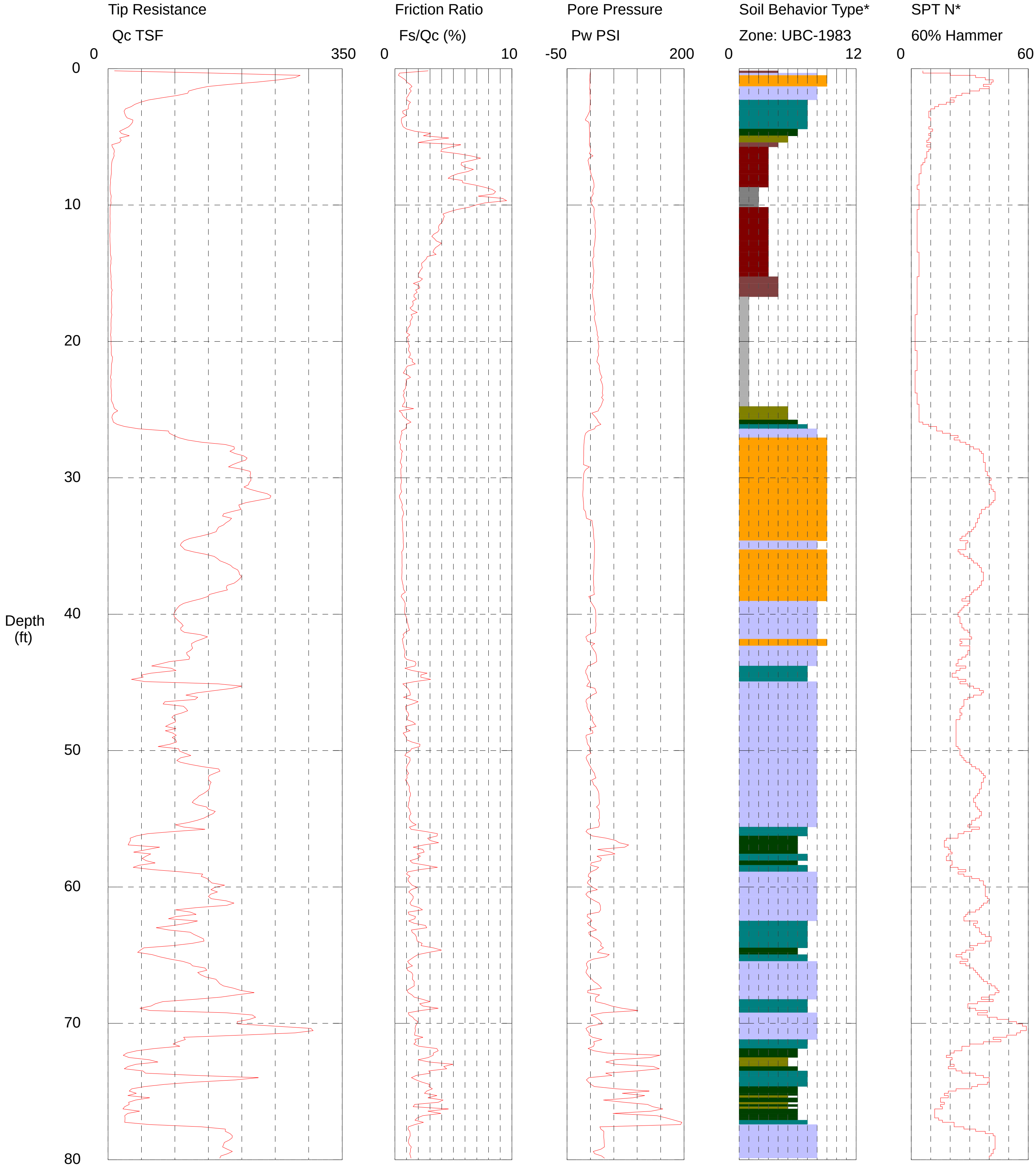
APPENDIX B

CPT SUMMARY LOGS

PanGeo

Operator: Gerdes
Sounding: CPT-01
Cone Used: DPG1186

CPT Date/Time: 10/25/2012 9:50:15 AM
Location: Marysville WA
Job Number: 12-163



Maximum Depth = 80.05 feet

Depth Increment = 0.164 feet

1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

InSitu Engineering

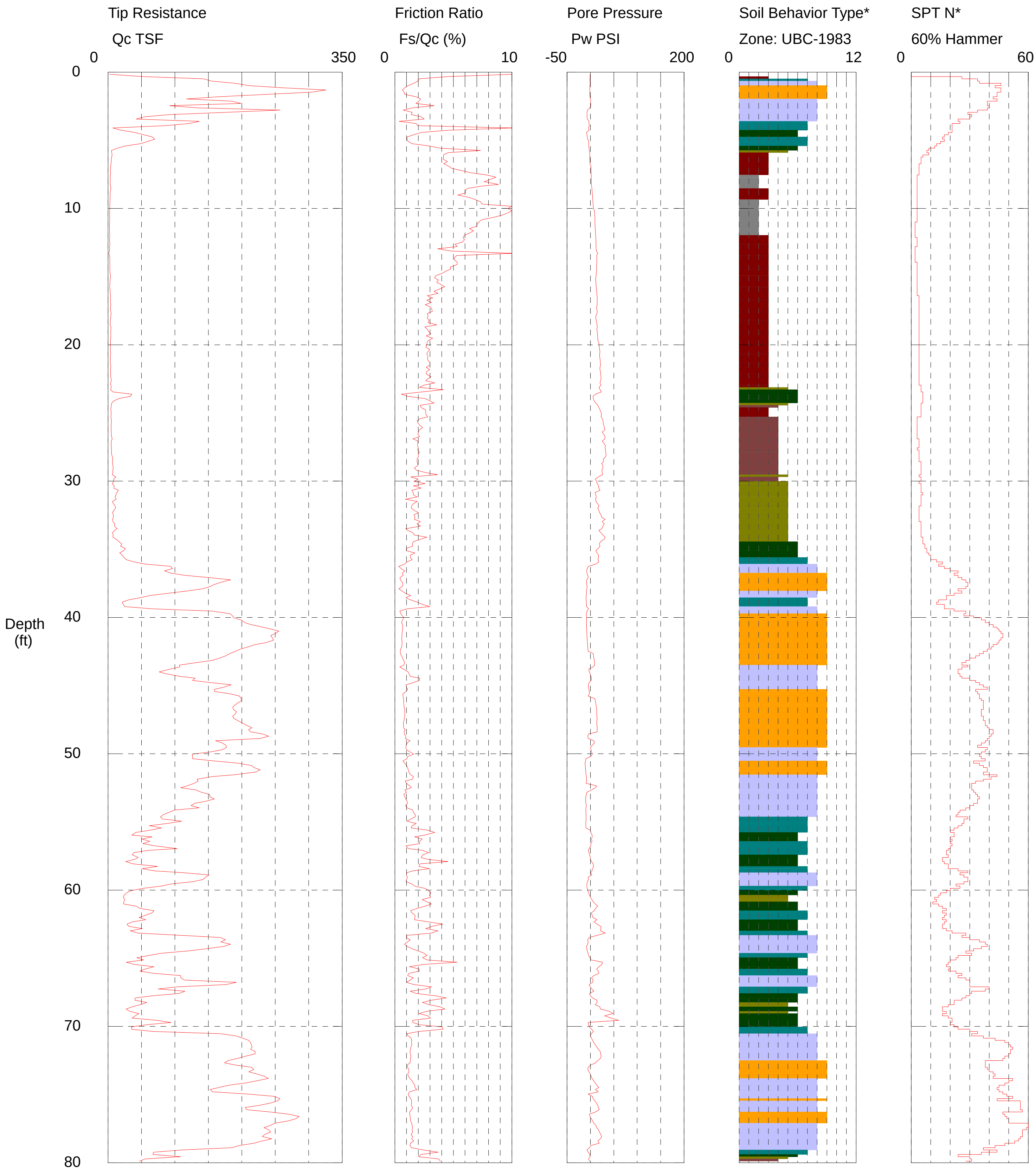
Pre-Drilled 1foot

*Soil behavior type and SPT based on data from UBC-1983

PanGeo

Operator: Gerdes
Sounding: CPT-02
Cone Used: DPG1186

CPT Date/Time: 10/25/2012 11:33:11 AM
Location: Marysville WA
Job Number: 12-163



Maximum Depth = 80.05 feet

Depth Increment = 0.164 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

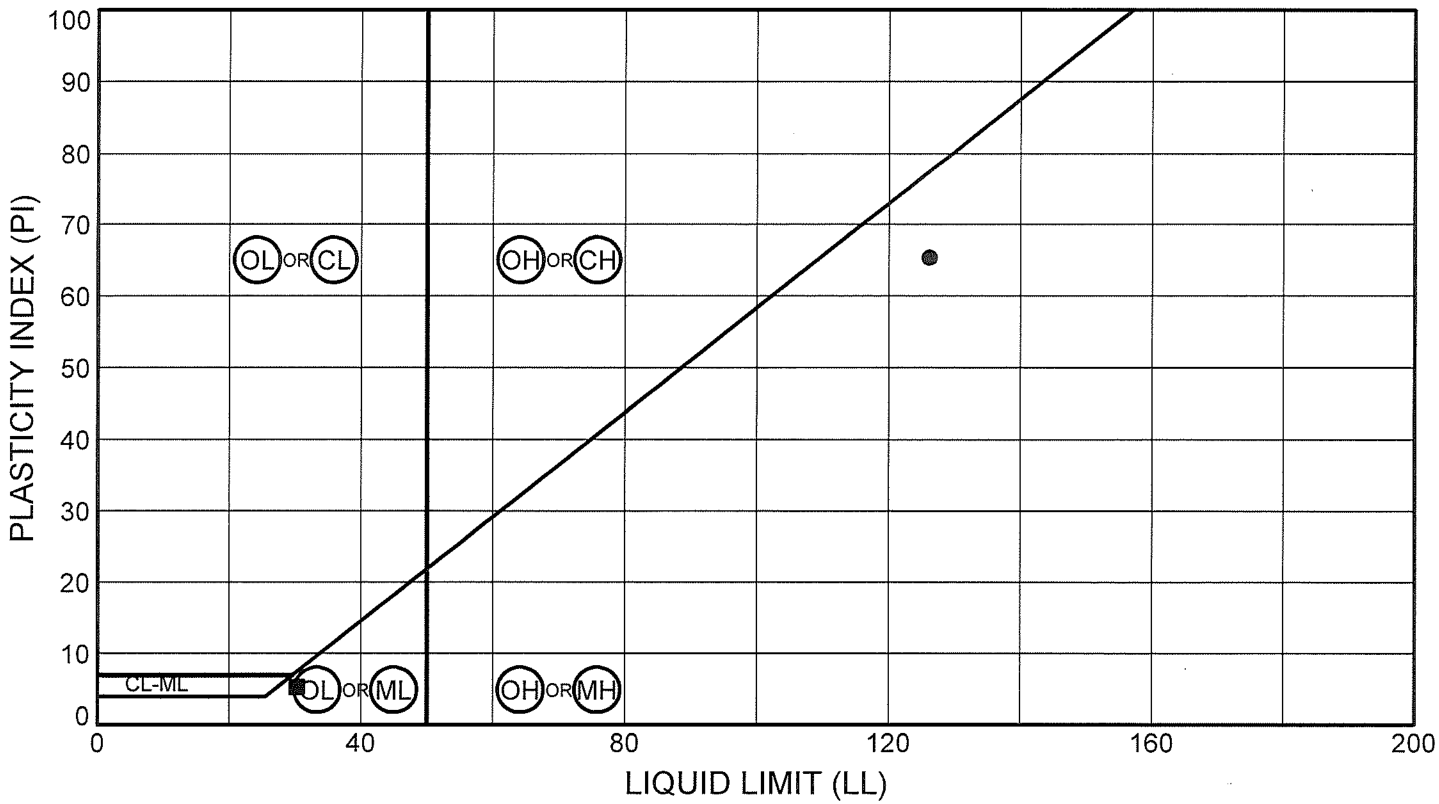
InSitu Engineering

Pre-Drilled 1.5 feet

*Soil behavior type and SPT based on data from UBC-1983

APPENDIX C

LABORATORY TEST RESULTS



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	OD*	% MC	LL	PL	PI	% Fines
●	BH-3	SH-4	9.0 - 11.0	(OH) Dark brown, organic SILT (organic content 11.1%)	0.51	307	126	61	65	
■	BH-3	SH-7	18.0 - 20.0	(ML) Dark olive brown, SILT		33	30	25	5	

*OD: Organic Determination

HWA GEOSCIENCES INC.

Laboratory Testing for PanGeo
Marysville Decant

LIQUID LIMIT, PLASTIC LIMIT AND
PLASTICITY INDEX OF SOILS
METHOD ASTM D4318

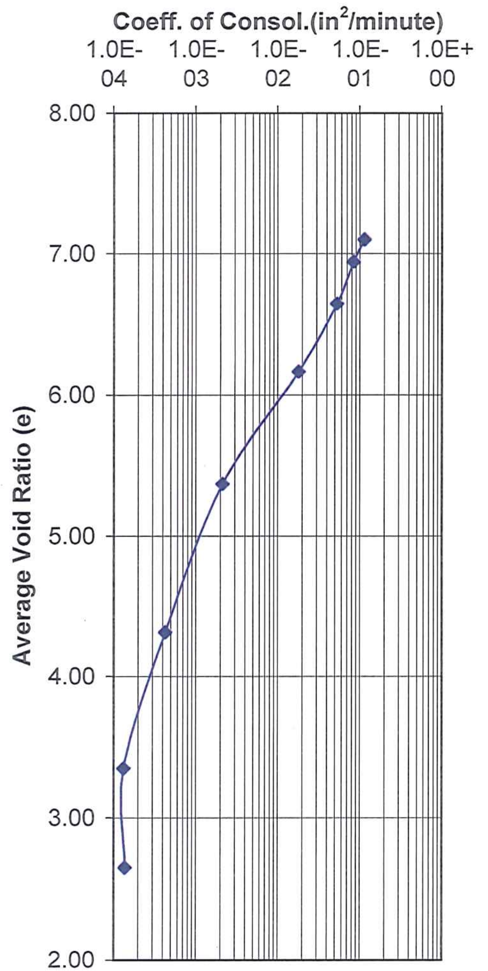
PROJECT NO.: 2012-022 T600 FIGURE: 1

HWA GEOSCIENCES INC.

ONE DIMENSIONAL CONSOLIDATION
ASTM D 2435

Project Name: Marysville Decant
Project Number: 2012-022 T600
Borehole Number: BH-3
Sample Number: SH-4
Sample Depth: 9-11 ft
Soil Description: Dark brown OH

	Start	Finish	
Moisture Content	228.7	114.1	%
Saturation	85.1	102.0	%
Dry Density	20.4	41.9	pcf



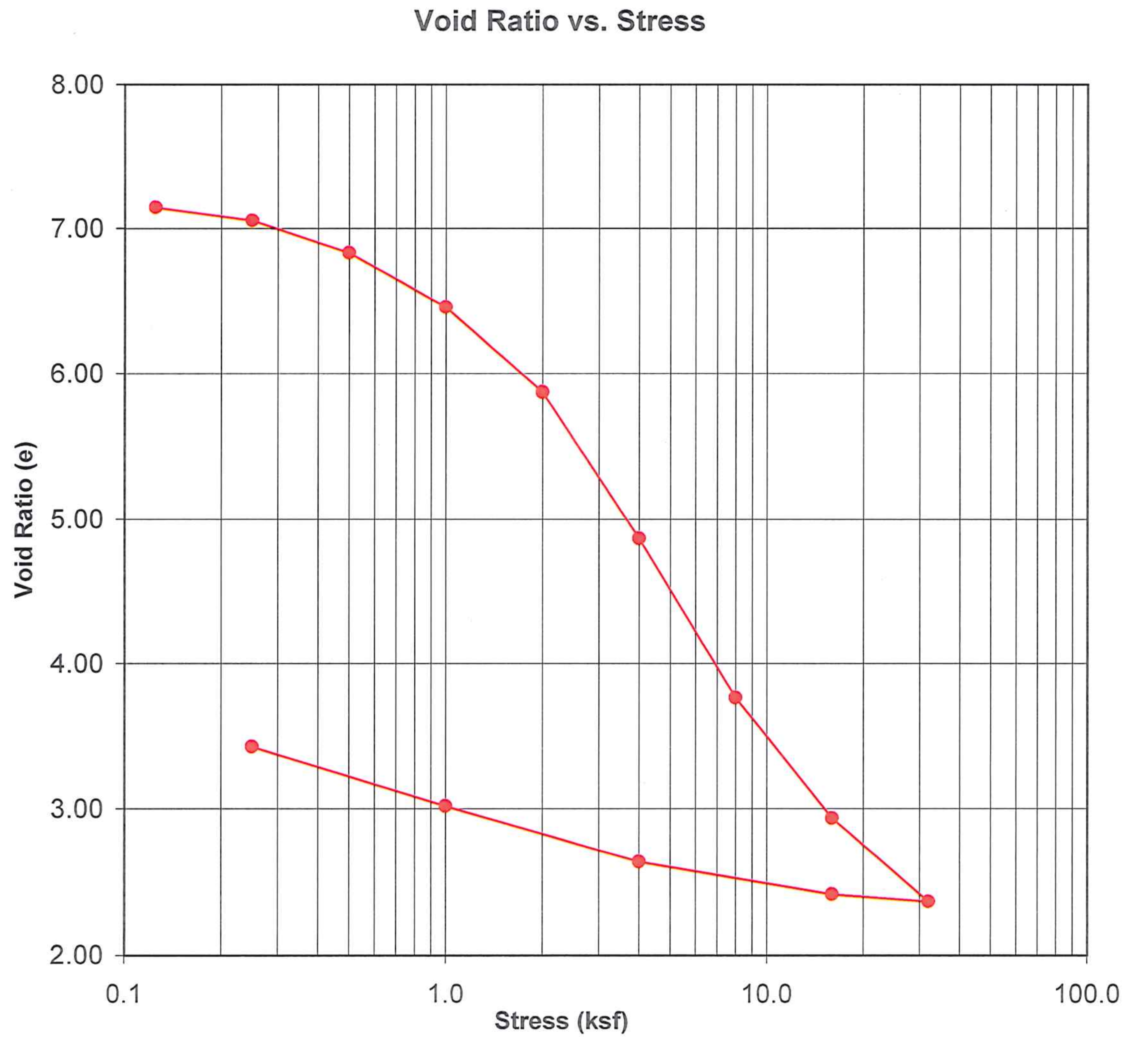


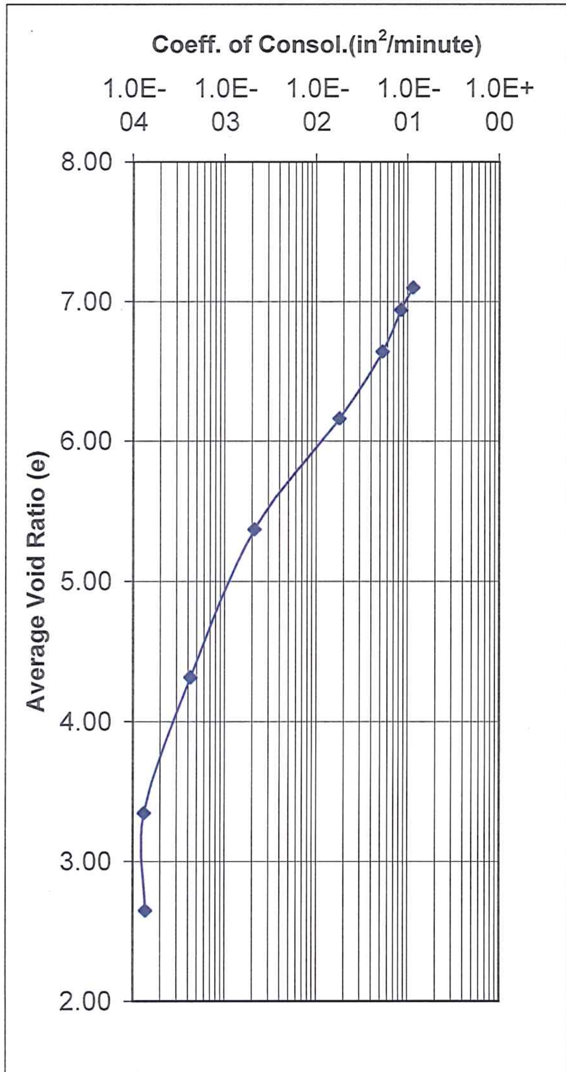
FIGURE 2

HWA GEOSCIENCES INC.

ONE DIMENSIONAL CONSOLIDATION
ASTM D 2435

Project Name:	Marysville Decant
Project Number:	2012-022 T600
Borehole Number:	BH-3
Sample Number:	SH-4
Sample Depth:	9-11 ft
Soil Description:	Dark brown OH

	Start	Finish	
Moisture Content	228.7	114.1	%
Saturation	85.1	102.0	%
Dry Density	20.4	41.9	pcf



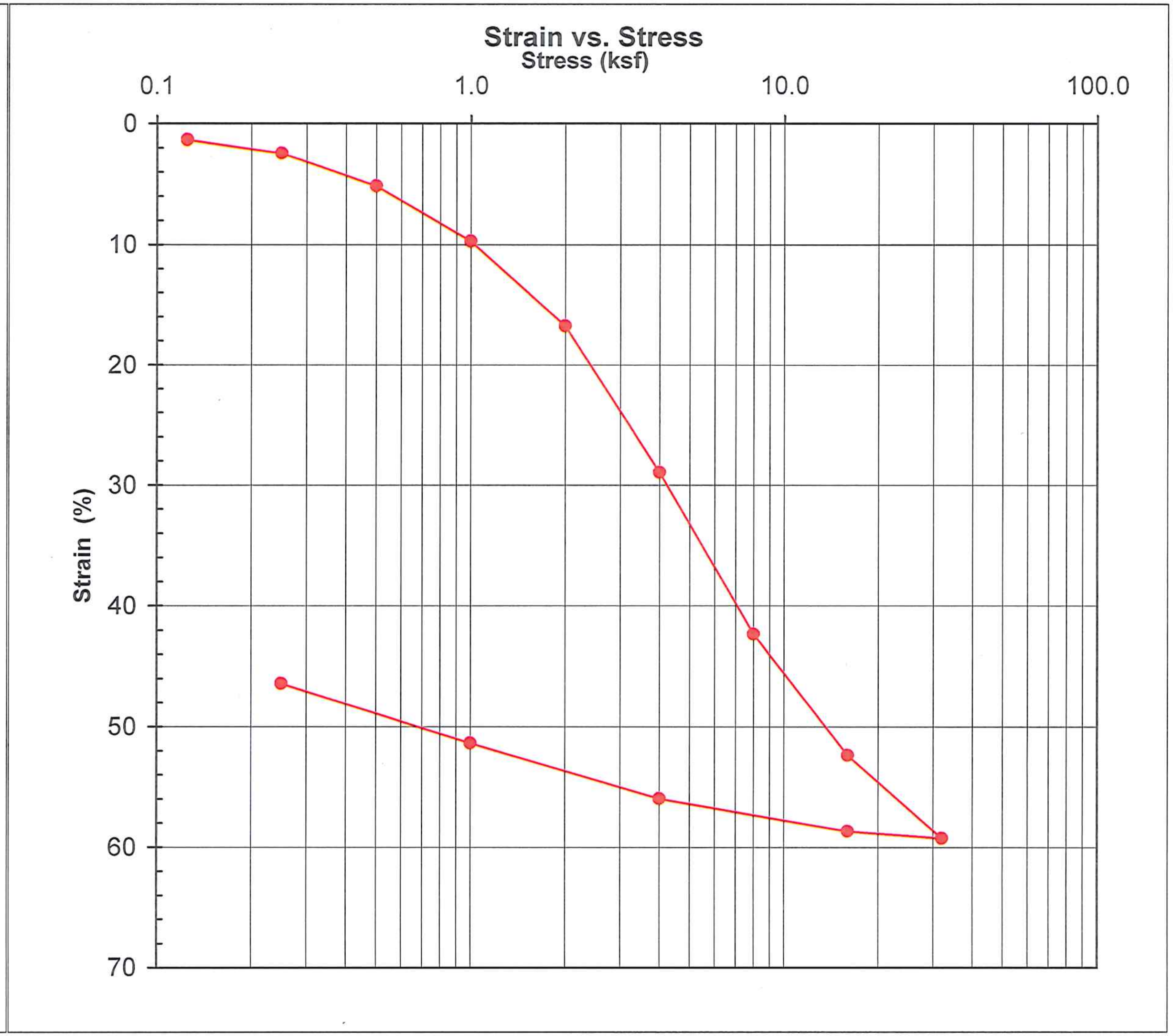


FIGURE 3

HWA GEOSCIENCES INC.

ONE DIMENSIONAL CONSOLIDATION
ASTM D 2435

Project Name: Marysville Decant
Project Number: 2012-022 T600
Borehole Number: BH-3
Sample Number: SH-7
Sample Depth: 18-20ft
Soil Description: Dark olive brown SILT with sand

	Start	Finish	
Moisture Content	31.2	24.7	%
Saturation	96.0	99.8	%
Dry Density	88.8	99.9	pcf

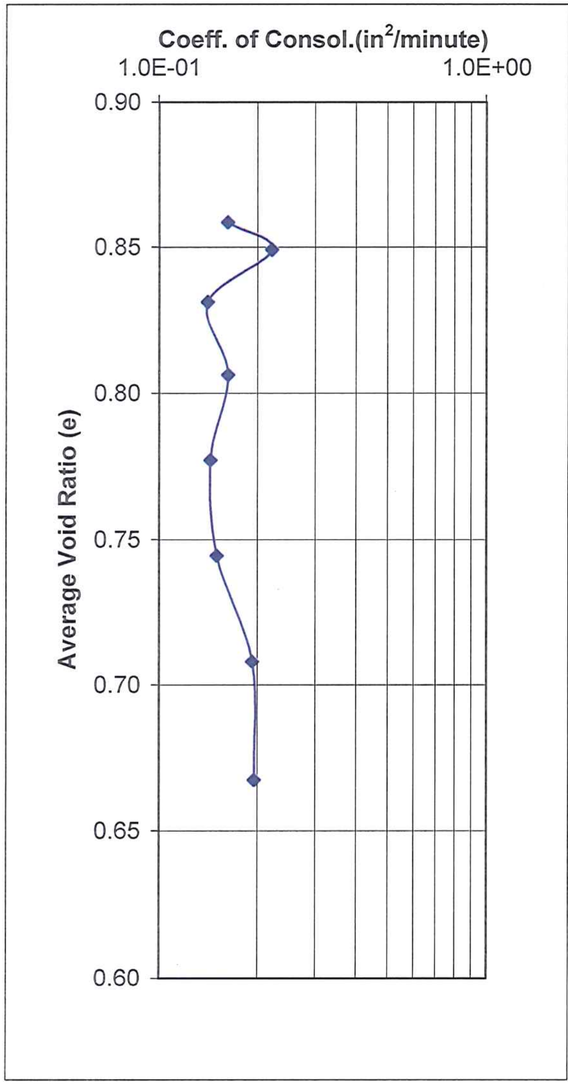


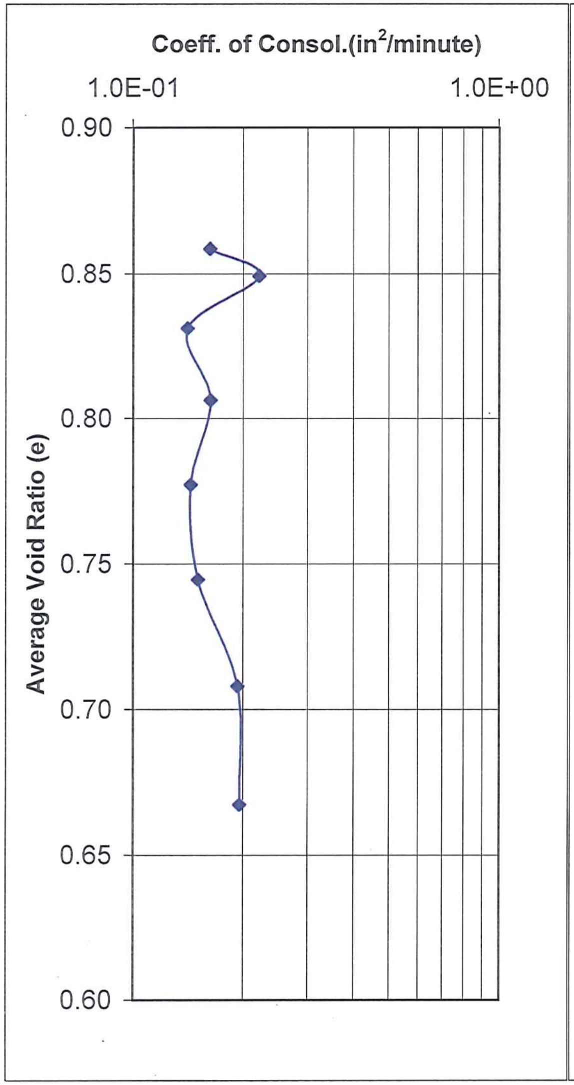
FIGURE 4

HWA GEOSCIENCES INC.

ONE DIMENSIONAL CONSOLIDATION
ASTM D 2435

Project Name:	Marysville Decant
Project Number:	2012-022 T600
Borehole Number:	BH-3
Sample Number:	SH-7
Sample Depth:	18-20ft
Soil Description:	Dark olive brown SILT with sand

	Start	Finish	
Moisture Content	31.2	24.7	%
Saturation	96.0	99.8	%
Dry Density	88.8	99.9	pcf



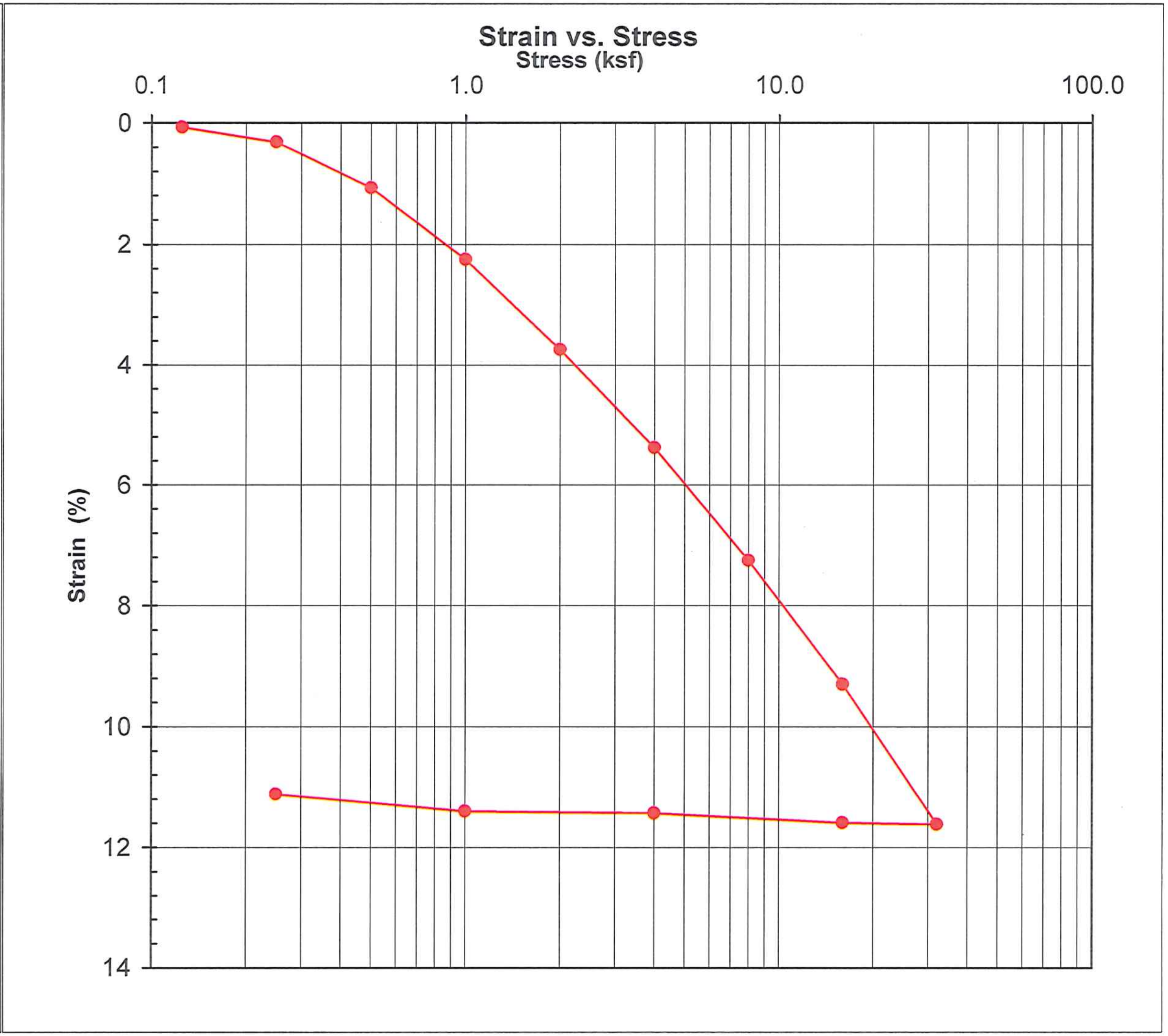


FIGURE 5