QWULOOLT SECTION 544 ECOSYSTEM RESTORATION PROJECT
Marysville, Washington

Geotechnical Report

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Page

1.0 INTRODUCTION 1

2.0 SITE BACKGROUND AND PROJECT DESCRIPTION 1
2.1 Authorization 1
2.2 History 1
2.3 Purpose 2
2.4 Goal 3
2.5 Federal Interest 3

3.0 SUBSURFACE EXPLORATIONS AND LABORATORY TESTING 3
3.1 Subsurface Explorations 3
3.2 Laboratory Testing 4

4.0 GEOLOGY AND SUBSURFACE CONDITIONS 4
4.1 Regional Geologic Setting 4
4.2 Local Geology 4
4.3 Project Site Subsurface Conditions 4
4.3.1 Soil Conditions 5

5.0 DESIGN CRITERIA AND ENGINEERING ANALYSIS 5
5.1 Seismic Design Criteria 5
5.1.1 Ground Motions 5
5.1.2 Earthquake-induced Geologic Hazards 6
5.1.2.1 Liquefaction 6
5.1.2.2 Lateral Spreading 7
5.1.2.3 Ground Settlement 7
5.1.2.4 Ground Fault Rupture 8
5.1.2.5 Slope Instability 8
5.2 Slope Stability 8
5.2.1 Soil Strength Parameters 10
5.2.2 Slope Stability Analysis 11
5.3 Settlement 12
5.3.1 Soil Parameters for Settlement Analysis 13
5.3.2 Software 13
5.3.3 Settlement Mitigation 13
5.3.4 Settlement Analysis 14
5.4 Erosion Control 14
5.5 Seepage Analysis 15

6.0 CONSTRUCTION CONSIDERATIONS 16
6.1 Year 1 Construction Efforts 16

6.1.1 Temporary Storm Water Pond and Construction Drainage 16
6.1.2 New Allen Creek Channel 16
6.1.3 New Channels and Habitat Berms 17
6.1.4 Clearing and Grubbing 17
6.1.5 Levee Material 17
6.1.6 Haul Roads 17
6.1.7 Staging areas 17
6.1.8 Levee Excavation 17
6.1.9 Instrumentation 18
6.1.10 Geotextile 19
6.1.11 Geogrid Reinforcement 19
6.1.12 Excavation Roadway 19
6.1.13 Levee Excavation Select Fill 19
6.1.14 New Levee Embankment 19
6.1.15 Ebey Slough Levee Back Slope Reinforcement 20
6.1.16 Erosion Protection 20
6.1.17 Access Restriction 20
6.2 Year 2 Construction Efforts 20
6.2.1 Surcharge removal 20
6.2.2 New Storm Water Pond 21
6.2.3 Outlet Culvert 21
6.2.4 Slope Protection 22
6.2.5 New Levee Roadway 22
6.2.6 Hydroseeding 22
6.2.7 Fencing and Gates 22
6.2.8 Signage 22
6.2.9 Tide gates 22
6.2.10 Ebey Slough Breach Excavation 23
6.2.11 Disposal of Excavated Material 23

7.0 CONSTRUCTION CHALLENGES 23
7.1 North Allen Creek Channel 23
7.2 Trees at South end of New Levee Embankment 24
7.3 Ebey Slough Breach 24
7.4 Marysville Sewage Lagoon Levee 25
7.5 Residential Areas 25
7.6 Wet Weather and Wet Condition Considerations 25

8.0 PLANS REVIEW AND CONSTRUCTION OBSERVATION 25

9.0 LIMITATIONS OF THE GEOTECHNICAL REPORT 26

10.0 REFERENCES 26

TABLES

1 Summary of Soil Properties
1.1 Estimated In-Situ Foundation Soil Properties for Settlement and Slope Stability
1.2 Estimated Effective Strength and Hydraulic Properties
1.3 Estimated Liquefied Residual Strength of Foundation Soils
2 Summary of Slope Stability Results

APPENDICES

A Geotechnical Reports
A-1 Kleinfelder Subsurface Exploration Phase I, 2008
A-2 Kleinfelder Subsurface Exploration Phase II, 2010
A-3 Kleinfelder Subsurface Exploration Phase III, 2011.
A-4 Shannon & Wilson Marysville Wastewater Treatment Plant Stability Analysis, 2004
A-5 Tulalip Tribe Stockpile Sampling Report, 2011
A-6 Christofferson Site Geotechnical Report, 2009
B Results of Slope Stability Analyses
C Results of Seepage Analyses
D Results of Settlement Analyses
E Results of Liquefaction Analyses

GEOTECHNICAL ENGINEERING REPORT

QWULOOLT SECTION 544 ECOSYSTEM RESTORATION PROJECT
MARYSVILLE, WASHINGTON

1.0 INTRODUCTION

This report presents the United States Army Corps of Engineers (USACE) Seattle District (USACE) studies for the design and construction of the proposed new levee for the Qwuloolt Section 544 Ecosystem Restoration Project (Qwuloolt Project), Marysville, Washington.

The main features of design for the project are a 3,960 feet long new levee, a storm water retention pond, an outlet culvert, and a breach of the Ebey Slough levee..

2.0 SITE BACKGROUND AND PROJECT DESCRIPTION

2.1 Authorization

The proposed project is authorized by Section 544 of the Water Resources Development Act of 2000 which authorizes implementation of critical restoration projects in the watersheds that drain directly into Puget Sound. The intent of the Section 544 Qwuloolt Restoration Project is to restore important hydrologic processes (both tidal and stream) which will lead to the establishment of intertidal habitat suitable for rearing and foraging by juvenile salmonids. This would be accomplished by lowering and breaching the existing dike at Ebey slough to reestablish tidal inundation to the site, and reconnecting the site to the slough. Additional restoration work will provide an unobstructed fish access to Allen Creek.

2.2 History

The Qwuloolt restoration project is an area of former agricultural land (Portinga property) located along Ebey Slough in the southern portion of Marysville in the Snohomish river delta which drains directly into Puget Sound. The area around the Qwuloolt project site was historically a tidal marsh. At what was known as the Portinga farm, a dike was constructed on the north bank of Ebey Slough and tidegates were installed at the mouth of Allen and Jones Creeks, and a series of ditches were dug to convert the land to pasture. As a

consequence, the dike and tidegates prevented tidal access to the historic floodplain which destroyed the estuary marsh habitat and restricted salmon and other estuarine-dependent species from utilizing this highly productive environment. This project specifically addresses improving the marine health of Puget Sound and provides critical habitat for several listed species under the Endangered Species Act.

The Qwuloolt Habitat Restoration Project (QHRP) offers a rare opportunity to restore a portion of the important estuarine habitat and associated functions. Several of the agencies involved (USFWS, NMFS, WDOE and WDFW) have, as part of their mission, goals to restore important habitat and associated flora and fauna. The Corps 544 program also allows federal participation in projects and activities, which have "immediate and substantial ecosystem restoration, preservation and protection benefits" (Dawson 2003). The overall restoration effort was originally started as a result of a Natural Resource Damage Assessment (NRDA) for the Tulalip Landfill remediation. The Natural Resource Trustees (Tulalip Tribe, NMFS, USFWS, and WDOE) were looking for a project that would provide some of the lost trust resources as a result of release of hazardous chemicals from the former landfill. But the project soon changed into something larger as additional partners and grants became available (such as Wetland Reserve Program, Coastal Grants, Washington State Salmon Restoration Fund, and NOAA stimulus funds).

The non-federal sponsor is The Tulalip Tribes of Washington, and per federal regulations, will be responsible for funding 100 percent of the costs associated with operation, maintenance, repair, replacement, and rehabilitation (OMRR&R) of the cost shared project.

2.3 Purpose

The overall purpose of the Qwuloolt project is to restore the natural resources of the former estuarine marsh to as close to pre-settlement conditions as possible. To achieve this purpose requires restoration of historic hydrologic processes and functions such as tidal exchange, tidal channel formation and migration, deposition and marsh plain development and other associated functions. By restoring the historic tidal regime to the site, the following project objectives will also be accomplished.

2.4 Goal

The goal of the QHRP and the overall restoration efforts is to restore tidal processes to 400 acres of currently fallow pasturelands. This will improve local streams and wetlands for fish such as threatened Chinook salmon, bull trout, and steelhead. Fish will also be able to access the project area for refuge, feeding, and spawning.

2.5 Federal Interest

All of the project partners involved in the overall restoration effort understand that the loss of historic estuarine habitat has had a profound effect on the ecological health of the Snohomish River basin and Puget Sound, as a whole. With the loss of habitat, there has been a marked decline in many of the species that are dependent on estuaries for a portion of their life. This decline has been so precipitous that several species have been listed under the Endangered Species Act (species such as steelhead, Chinook and bull trout) over the last several years. There is also recognition that estuaries provide a wide variety of resources and functions to the fauna and flora of the Northwest, beyond endangered species support, which have inherent cultural benefits, as well. This project looks to restore both hydrologic and geomorphic processes to the site. The project is part of the overall Puget Sound Initiative, and is in the Federal Interest.

3.0 SUBSURFACE EXPLORATIONS AND LABORATORY TESTING

3.1 Subsurface Explorations

The Subsurface exploration program for the project consisted of borings, cone penetration tests (CPTs) with dissipation tests, and test pits along and near the new levee alignment, and near the proposed Ebey Slough levee breach and the southeast portion of the Marysville sewage lagoon levee. The subsurface explorations and testing at the Qwuloot Project site were performed between 2004 and 2011 by Shannon and Wilson, Inc., Seattle, Washington, and Kleinfelder, Bellevue, Washington, under contract to the Tulalip Tribe. Investigation of the borrow source for the new levee embankment fill was conducted in 2011 by Kleinfelder under contract to the Tulalip Tribe. Copies of these reports are in Appendix A. The USACE utilized the soil and geotechnical analytical laboratory analyses in these reports for the development of the new levee design.

3.2 Laboratory Testing

Geotechnical analytical laboratory testing performed on soil samples collected during the subsurface exploration program included visual classification, moisture content determinations, grain size analyses, hydrometer analyses, fines content determinations, Atterberg limits tests, organic content, Unconsolidated Undrained (UU) and Consolidated Undrained (CU) Triaxial tests, and consolidation tests. The laboratory tests were conducted to evaluate index and engineering properties of the soils.

4.0 GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geologic Setting

The Qwuloolt Project is in the Puget Sound Lowland. The Lowlands was modified by Pleistocene glacial activity that carved deep troughs and left numerous elongated lakes and stream channel. The glaciers deposit this and extensive deposits of unconsolidated to consolidated sand, silt, gravel, and clay. Coastal and stream erosion, landslides, earthquakes, and human development activities have modified the local geology.

4.2 Local Geology

The site is boarded by high bank areas underlain by glacial till on the north and east sides of the Project. The Marysville sewage lagoon levee is on the west side of the Project. The southern end of the project is separated from Ebey Slough by the Ebey Slough levee. The Qwuloot Project is underlain by recent alluvium composed of clay, silt, peat, sand, and gravel. This alluvium is at least 100 feet thick. The depth to groundwater varies from five feet below the ground surface in the summer to the surface in the winter months. The site ranges in elevation from 3 to 9 feet above mean seal level and is generally flat.

4.3 Project Site Subsurface Conditions

Our understanding of the project site subsurface conditions is based on soil units encountered in the recent subsurface explorations and described in logs of previous subsurface explorations that we reviewed within the project area.

4.3.1 Soil Conditions

In general, subsurface conditions within the project area are consistent with the geologic history of the area. Based on site reconnaissance, review of information from previous geotechnical studies, and recent exploration and laboratory test programs, we conclude that to the depth of interest for the proposed work, the project site is underlain by very soft to soft organic-rich clayey silt and pockets of peat, which overlies thick alluvial deposits composed of loose to medium dense silty sand. Sheets B-201 and B-202 of the plan set represent our interpretation of the geologic subsurface conditions along the proposed levee alignment.

5.0 DESIGN CRITERIA AND ENGINEERING ANALYSIS

The following sections present our analyses criteria, results of our geotechnical engineering studies, and our geotechnical recommendations for the design and construction of the proposed new levee embankment .

5.1 Seismic Design Criteria

The guidelines for seismic stability evaluation of USACE levees, USACE 2011, indicate that seismic degradation of levees subject to continuous water retention located in a seismically active area and founded on potentially liquefiable soils, consists primarily of cracking and lateral spreading, which can be repaired before becoming a threat to the overall stability of the levee, if the coincident water level was relatively low, so neither overtopping nor internal erosion occurred. If coincident water was relatively high during the design earthquake, major slumping and breach of the levee could occur.

5.1.1 Ground Motions

We evaluated the design ground motion at the project site using the Probabilistic Seismic Hazard Analysis (PSHA) which incorporates the frequency of occurrence of earthquakes of different magnitudes on various seismic sources, the uncertainty of the earthquake locations on the sources, and the ground motion prediction, including its uncertainty. We used the most recent ground motion prediction equations published in the USGS' Interactive Deaggregation (2008) online tool.

According to the guidelines for seismic stability evaluation of USACE levees, USACE 2011, we performed the evaluation based on a 108-year return period, which corresponds to a probability of exceedance of 50% in 75 years according to USACE ER 1110-2-1806 (USACE, 1995). The resultant Peak Ground Acceleration (PGA) and earthquake magnitude are 0.18g and 6.6, respectively. Based on standard penetration test (SPT) N values and limited shear wave velocity testing at the site (evaluated at <600 ft/sec for the upper 100 feet), the site is class is E. The output from the USGS deaggregation tool is presented in Appendix E.

5.1.2 Earthquake-induced Geologic Hazards

Earthquake-induced geologic hazards include liquefaction, lateral spreading, settlement, ground surface fault rupture, and slope instability. The following sections present our evaluation of potentially significant geologic hazards at this site.

5.1.2.1 Liquefaction

Liquefaction is an earthquake hazard associated with sites where the subsurface typically consists of loose, saturated, cohesionless (granular) soils. The liquefaction potential of a soil primarily depends on its gradation and density, and the intensity and duration of ground shaking. Relatively clean, sandy soil with low fines content is known to be potentially liquefiable depending upon its density (Seed and Idriss, 1971). Silts with low plastic index (PI) values, like those encountered in the project area, are subject to liquefaction.

An important factor in liquefaction analyses is the percent fines (percent of soil by weight smaller than 0.075 millimeter or a No. 200 sieve) of the soil deposit, and the PI value, when applicable. Grain size analyses and Atterberg limits were used to estimate these properties for the subsurface soils at the site. Where laboratory tests were not performed, soil samples were compared with samples that were tested, and the fines content and PI were estimated.

For the liquefaction analysis, the most widely used methods are based on correlations between SPT N-value. Three methods were used to evaluate liquefaction potential at the levee site; Youd and others (2001), Seed and others (2003), and Idriss and Boulanger (2006). Liquefaction analyses were performed using the above PGA and earthquake magnitude

A predominantly very soft to soft silt layer is present beneath the proposed levee alignment. The silt layer is underlain by a loose to medium dense silty sand layer. Liquefaction susceptibility was evaluated using SPT N- values and laboratory-measured soil fines contents using an in-house developed spreadsheet Liquefaction analyses were performed on thirteen (13) borings drilled at the project site along the proposed levee alignment. The analyses showed that most of the 30 –foot thick soft silt layer underlying the project site is potentially liquefiable with a factor of safety (FOS) of less than 1.0 against liquefaction under the design ground motion, suggesting liquefaction or elevated pore pressure could develop in these layers. Post-liquefaction settlement estimates indicate possible settlements of up to 10 inches. These values should be considered estimates of post-earthquake ground behavior. The results showing the FOS against liquefaction for the 13 analyzed borings are presented in Appendix E.

5.1.2.2 Lateral Spreading

The reduction of soil shear strength due to liquefaction combined with relatively low static shear stresses in a soil may cause significant permanent lateral ground deformations on gently sloping ground or on level ground adjacent to a “free face” (e.g., river, channel, or waterway). The levee site is not on or near gently sloping ground or next to a free face; therefore, spreading is not deemed a significant seismic hazard at this site.

5.1.2.3 Ground Settlement

Loose, granular soils that are susceptible to liquefaction are also susceptible to earthquake-induced densification. The resulting permanent ground surface settlement may not occur uniformly over an area, and the differential settlement can damage existing or future structures supported on the loose

soil. Without considering ground improvement, liquefaction-induced settlement of the site soil could range between 10 and 20 inches.

5.1.2.4 Ground Fault Rupture

Although faults are often shown as lines on maps, surface fault rupture typically occurs in a zone near a mapped fault. For the distance of the project site from the mapped faults and the postulated return period of ground-rupturing earthquakes on the order of at least a few thousand years, the potential for surface fault rupture within the area of the proposed new levee during the design life of the bridge is relatively low.

5.1.2.5 Slope Instability

Ground acceleration, ground shaking, and liquefaction- induced reduction in soil strength , can all contribute to slope instability. The results of the slope stability analyses conducted for this project are discussed in Section 6.1. The results showed that embankment instability during and post a seismic event is a potential hazard. The new levee embankment does not meet the minimum required FOS for seismic and post-seismic loading without a mitigation measure.

5.2 Slope Stability

Global stability analyses were performed for short-term (end of construction), static, steady state seepage, rapid drawdown, pseudo-static, and post-seismic (liquefied) loading conditions. Slope stability of the proposed levee embankment was evaluated at three representative sections along the levee alignment with different heights; 8, 10 ,and 15 feet and different long term configurations as explained in Section 52.2. in accordance with USACE Design and Construction of Levees, Section 6-5 (USACE, 2000):

(1) **End of construction.** Undrained shear strength parameters were used for the soft silt layers and effective strength parameter for the underlying sand layer. This condition requires a minimum FS of 1.3. It should be noted that due to surcharge configuration, constructed slopes for all three sections ar approximately 2H:1V. The northern 8-foot high section is not analyzed for this case.

(2) **Steady-state**. This condition represents the long-term stability where the water level on the levee embankment is at the design high water level (10-year event). Effective soil strength parameters were used for the foundation soils. A second long term static condition was evaluated at the MHHW level for comparison with the seismic case. This condition is applicable to the landside slope, the minimum required FS is 1.4 according to EM 1110-2-1913: USACE Design and Construction of Levees, Table 6-1b (USACE, 2000).

(3) **Rapid drawdown.** The water level was assumed to drop instantaneously from the 10-year, 0.1 annual chance exceedance (ACE) design water surface elevation to the natural ground surface. This condition is applicable to the waterside slope. Effective soil strength parameters were used in the analysis. For this condition, the minimum required FOS is 1.0 or 1.2, according to USACE (2000), for the cases where water levels are expected to persist for short periods or long periods, respectively. It is assumed that the retained water affected by the tidal fluctuations would prevent high water from loading the embankment for extended periods of time. Therefore, a FOS of 1.0 was considered.

(4) **Seismic**. Two seismic-related loading conditions were considered. The first is a pseudo-static case applying a horizontal coefficient, k_h, of 0.09 and using the long term soil strength properties. The Pseudo-static slope stability analysis was performed on the predefined static critical failure surface. The second case is a post-liquefied case using residual shear strength of the foundation soils. Internal forces were not included when considering liquefaction or seismic-induced soil strength degradation.

Slope stability analyses were performed using Slope/W 2007 V.7.13 (Geo-Slope International, Ltd., 2007). The FOSs reported are calculated using the Spencer method which satisfies both moment and force equilibrium, and considers both shear and normal interslice forces.

5.2.1 Soil Strength Parameters

The soil strength parameters were evaluated based on exploration data, laboratory data, empirical relationships, and experience with similar soil types of the region. Engineering judgment was used in evaluation of the design undrained shear strength of foundation soils as obtained from laboratory testing as the confining pressures used in CU tests were in some cases larger than actual effective stresses on the in-situ soil samples. When there was a need for an increased shear strength during construction to maintain the required stability, a Stress History and Normalized Soil Engineering Properties (SHANSEP) concept was used to estimate this increase. This often used to estimate the increase in undrained shear strength after a construction lift is placed and the layer is left to fully or partially consolidate. The SHANSEP equation uses a ratio of undrained strength and effective vertical stress with adjustment factors for over-consolidation ratio (OCR) to define a relationship between undrained strength and effective vertical stresses. Consolidation tests revealed the soil to be classified as normally consolidated. Therefore, the OCR was taken as 1.0. From the available CU tests, a literature review, and engineering judgment a SHANSEP ratio value of 0.3 was used to normalize the undrained strength for in-situ effective vertical stresses. These in-situ, undrained shear strengths were used in the during-construction analysis of the levee and presented in Table 1.1.

Analysis of the levee at static cases incorporated effective stresses of the foundation soils. Soil strengths for the drained effective strengths were assigned using the effective stress shear envelope from the consolidated undrained (CU) tests performed on selected samples and using values published in literature for similar soils. The effective strength soil parameters are referenced in Table 1.2.

Based on the liquefaction analysis, the potentially liquefiable soils were determined to either perform clay-like or sand-like. Clay-like liquefiable soils were assigned a residual undrained strength while sand-like liquefiable soils were assigned a residual friction angle. The post-seismic residual strength for liquefied soils are presented in Table 1.3.

5.2.2 Slope Stability Analysis

The global stability of the proposed slopes was evaluated for the levee embankment at three representative cross sections. The three sections are at stations 3+50, 23+50 and 37+50 representing northern, typical and southern embankment sections. Table 2 summarizes these stability analyses results. To mitigate for instability due to liquefaction-induced strength reduction, soil reinforcement is recommended.

The presence of a thick organic and clayey silt layer in the upper 30 feet below ground surface requires staged construction to allow the silt layer to consolidate and gain strength. Soft silt in its in-situ state has low undrained shear strength (assumed 250 psf in the upper 15 feet and 900 psf in the lower 15 feet). It is recommended that a maximum of 5 feet of fill be placed at a time. To make use of the gain in strength in the consolidating soils, each fill placement should be followed by at least six-week waiting/settlement period before placing additional fill. As an alternative to the waiting/ settlement periods after placement of fill, a base reinforcement is added to enable placement of embankment lifts with a satisfactory FOS, without relying on strength gain from soft silt consolidation. The reinforcement will be uniaxial geogrids with a minimum allowable strength of 7,500 lb/foot placed at selected number of layers spaced 1 foot.

A settlement monitoring program should be implemented, and if the results of the monitoring program indicate that about 90 percent of the settlement projected to occur under the applied load has occurred, the next construction stage may begin before the full six weeks have elapsed. Fills should be placed to a uniform thickness over the full alignment length. Uniform fill placement will reduce the potential for shear or horizontal movement of the underlying soft silt.

The southern section approximately 350 linear feet of levee is typically 15 feet in height and will require four (4) layers of base reinforcement. Also necessary for stability are at least 22.5 foot wide, four foot tall (above ground surface), stability berms created from the excess surcharge material removed after foundation consolidation.

The typical 10 foot levee section requires two (2) geogrid layers of base reinforcement and 10- foot wide, four foot tall (above ground surface) stability berms. The northern section approximately 900 linear feet of levee is typically 8 feet in height with a 2H:1V waterward slope and a 2.5H:1V landward slope. Because of easement restriction, no stability berms

could be constructed for this section, instead three (3) geogrid layers were required as base reinforcement for stability. The northern section does not allow for placement of stability berms due to unavailability of easement. Typical sections of the levee are provided in the project's plans.

5.3 Settlement

Elastic, consolidation, and secondary compression settlement analyses were conducted at representative sections based on available subsurface conditions interpreted from geotechnical reports and proposed levee geometry. For the consolidating materials along the project's alignment, which consist primarily of very soft to soft organic and clayey silt, vertical settlement analyses were performed using one-dimensional vertical consolidation theory. Consolidation test results, CPT dissipation tests, published data, and in-situ test correlations were reviewed and used in selecting settlement properties. Consolidation test results on various soil samples are presented in Appendix A. Elastic settlement analyses for non-plastic silt, and sand and gravel, were performed, assuming:

1. Post-construction settlement within levee materials would be negligible.
2. Elastic settlement will occur relatively quickly (i.e. settlement should occur essentially as the load is applied).
3. Secondary compression (creep) will start after 95 percent of primary consolidation has occurred.

Because of analysis uncertainties, settlement estimates indicate likely approximate settlement magnitude across the alignment. Variations in the assumed subsurface stratigraphy, estimated consolidation soil properties, and/or assumed construction sequencing may result in greater or less than actual settlements.

The settlement analyses considered multiple time stages to differentiate between the settlement that will occur during the construction phase and long-term post-construction settlement. These stages were selected to satisfy construction requirement and time constraint during Year 1 of construction. This constraint does not ensure adequate time for excess pore water pressure dissipation and shear strength gains in the consolidating soil layers to support the new levee embankment. As discussed in Section 5.2.2 inclusion of uniaxial geogrids allows construction to be completed within two construction seasons.

5.3.1 Soil Parameters for Settlement Analysis

Elastic, vertical and horizontal consolidation, and secondary compression soil parameters used in the analyses are evaluated on site-specific subsurface conditions (based on available borings, CPTs, laboratory consolidation test results, field dissipation test results, published data, and appropriate in situ test correlations).

Modulus of elasticity values were estimated based on field Standard Penetration Test (SPT) N-values and CPT tip resistance values. Consolidation settlement parameters are based on laboratory one-dimensional consolidation testing data, correlations with Atterberg limits, in-situ CPT testing, and published data. Overconsolidation ratios, strain compression indices, strain recompression indices, secondary strain compression indices, and coefficients of consolidation for individual stratum layers were all evaluated. Table 1.1 summarizes the soil parameters used in the settlement analyses.

5.3.2 Software

The commercial software Settle^{3D} developed by RocScience (2008) was used to estimate embankment settlement. Specifically, Settle^{3D} was used to model Boussinesq stress distributions for complex embankment configurations and to estimate the elastic and time-rate primary consolidation settlement estimates for the construction sequencing. Settle^{3D} uses traditional one-dimensional consolidation theory to calculate settlement along user defined horizontal lines or discrete points at various depths. Long-term settlement was analyzed at the location of maximum primary consolidation settlement.

5.3.3 Settlement Mitigation

Settlement along the alignment was calculated based on conditions encountered in representative subsurface explorations. The secondary compression rate was evaluated using relationships presented in Mesri and Fang (1991), and estimated using published values and lab data as presented in Appendix A.

To mitigate for consolidation settlement and reduce post-construction settlement, we are recommending overbuilding the embankment above the finish grade by the amount of settlement that is expected to occur in addition to preloading with

approximately 3-foot surcharge that will be removed in Year 2 of construction. Placing a surcharge fill is recommended to overconsolidate underlying compressible soil. The surcharge fill should remain in place until 95 percent primary consolidation has occurred in underlying compressible soil. The surcharge fill should be then removed down to the final top elevation.

Transitions from thicker to thinner surcharge plus overbuild fills should be at a 20H:1V slope. The full combined surcharge and overbuild fill should be constructed between the stations where the transition takes place as identified at later design stage. The 20H:1V transition should occur over the adjacent stations that have thinner combined surcharge and overbuild fills. Where no adjacent surcharge fills are recommended, the combined surcharge and overbuild fill thickness should taper down at 20H:1V to the top elevation.

5.3.4 Settlement Analysis

Settlement analyses estimated about18 to 24 inches of total settlement under the centerline of the levee embankment of 15-foot height, and 10 to 16 inches under a 10-foot high embankment. Ninety-five percent of this settlement is estimated to take place within two years of reaching the total height of the levee embankment Surcharging the embankments with overbuild fill placed to compensate for the expected settlement is recommended. The total fill height is about 20 feet for the 15-foot high embankment, and 14 feet for the 10-foot high embankment. The analyses results indicate that the application of the surcharge would reduce the secondary compression to less than 1.5 inches in five years. Settlement is expected during and after fill placement; therefore, the total thickness of the placed fill, and not simply the top of fill elevation, should be monitored. Total settlement profiles are presented in Appendix D for two heights of the levee embankment; 10-ft, and 15-ft

5.4 Erosion Control

Erosion protection measures should be applied to landside and waterside slopes to reduce erosion and shallow sloughing, and improve long-term stability of the slopes. Riprap, aggregate surfacing, or appropriate vegetation could be used as erosion control measures.

5.5 Seepage Analysis

Underseepage and seepage through the three representative levee cross sections were estimated using methods presented in USACE manuals (EM 1110-2-1913, April 30, 2000; EM 1110-2-1901, February, 2005). This analysis was accomplished using the finite element modeling program, Seep/W.

The levee is designed to detain water twice a day. If the design water level persists for a prolonged period, seepage may occur through and underneath the levee. The foundation is a lower permeability clayey silt overlain by a more permeable silty sand levee embankment, thus, making through seepage more probable. The most significant water head on the levee will be experienced during a flood event coinciding with a high tide, which is evaluated at elevation 11.7 feet. For reference, the levee crown elevation is designed at elevation 14.5 feet.

The horizontal coefficient of hydraulic conductivity, K_h, for the foundation soils is assumed to be $1x10^{-6}$ cm/s, and was selected from permeability testing of the in-situ soil. The hydraulic conductivity of the levee fill material is assumed to be $5x10^{-4}$ cm/s and was attained through grain-size analyses using correlations and empirical equations, as well as typical values based on soil classification. Prolonged seepage conditions could cause piping of the silty sand levee embankment material. However, seepage analysis illustrates that the exit gradients experienced at the landward toe of the levee under design flood steady state seepage conditions are not conducive to seepage or piping. Maximum average exit gradients experienced at the landward toe of the levee were less than 0.2. Therefore, no seepage mitigation is recommended for the design. Appendix C presents the results of the seepage analyses.

No substantial quantity of seepage flux is expected through the typical levee embankment section. Flux quantities were calculated to be in the range of 0.5 to 5 cubic feet per day per linear foot of the levee embankment. This range was established under steady state seepage conditions for the 0.1 annual chance exceedance (ACE) design flood elevation. Sensitivity analyses were incorporated in this estimate for variation in hydraulic conductivity of the borrow material since permeability testing data is unavailable. The seepage quantity is heavily dependent on the hydraulic conductivity of the compacted borrow source fill.

The analysis performed accounts for the immediate seepage through the levee. This analysis does not include large scale water balance and ground water modeling. The quantity of seepage under flood loadings is calculated, but no analysis on the quality of the water has been performed. The increase in the salinity of the groundwater, due to the filling of the project with brackish water, is beyond the scope of this analysis.

6.0 CONSTRUCTION CONSIDERATIONS

General construction considerations for the construction of the Ebey Slough levee breach, outlet culvert, and storm water pond are presented below. The project is designed to be completed in two construction seasons due to the weak nature of the native soil. Work at the site is only possible when the site is free of standing water, typically from June to late October . The new levee with surcharge will be constructed in Year 1. The surcharge will be removed from the new levee in Year 2 and used to construct the levee stability berms. The Ebey Slough levee breach, storm water pond, and outlet culvert will be excavated in Year 2.

6.1 Year 1 Construction Efforts

6.1.1 Temporary Storm Water Pond and Construction Drainage

A temporary storm water pond and construction drainage ditches will be constructed to retain runoff from the Industrial area during Year 1. Temporary backup pumps are recommended to be placed adjacent to the pond to pump runoff water over the new levee embankment to the restoration site in the event the runoff is greater than the pond capacity.

6.1.2 New Allen Creek Channel

The north portion of Allen Creek will be moved to a new channel east of the new levee footprint. This effort will divert the creek to the restoration site and away from the levee construction area. The water from Allen Creek will exit the restoration site through two existing tide gates in the Ebey Slough levee to the east of the south end of the new levee embankment.

6.1.3 New Channels and Habitat Berms

New channels and habitat berms will be constructed to the east of the new levee embankment. This work will be performed by the Tulalip Tribe and is not part of this design.

6.1.4 Clearing and Grubbing

Clearing and grubbing will be performed before the start of new levee construction. Tree, brush, and debris will be cleared from the levee alignment to a distance of 15 feet water ward and landward of the new levee toes. Tree stumps and roots will be grubbed.

6.1.5 Levee Material

Fill for the new levee will be provided by the Tulalip Tribe at no cost to the Government. The material is mostly silt with sand. This soil is sensitive to moisture content for proper compaction. The Tulalip Tribe will process and transport the soil to the soil stockpile at the Qwuloolt Project.

6.1.6 Haul Roads

Haul roads will be constructed at the site to access the new levee and breach area. Temporary haul roads should be constructed of ballast underlain by a geotextile to prevent loss of the ballast into the soft soils at the site. These roads will allow greater ease in access to the construction areas. The roads will be constructed in Year 1 and removed in Year 2.

6.1.7 Staging areas

Four staging areas will be constructed at the Project (see plans for locations). Access from the southeast end of the site is required to allow equipment to access the east side of the Year 2 breach excavation.

6.1.8 Levee Excavation

The surface of the soil is covered with one to two feet of dense grass and root vegetation. The material will be stripped to a depth of two feet below ground surface along the levee alignment. Excavated material will be disposed of to the east

of the new levee and used for construction of other restoration berms. This material should not be placed on the west of the new levee since this would interfere with site access and construction of the storm water pond. The excavation must be dewatered before the placement of levee foundation material.

6.1.9 Instrumentation

Settlement gages, inclinometers, and vibrating wire piezometers will be installed in the new levee excavation. These instruments will be monitored to determine total settlement, lateral soil deformation, and pore water pressure for timing of levee fill placement and for estimation of total surcharge removal after Year 1 consolidation.

Fill settlement should be monitored to evaluate when primary consolidation is 95 percent complete, after which, surcharge fills can be removed and final grading can be performed. By monitoring the fill settlement, the surcharge duration can be adjusted from the estimated times presented in this report. This degree of consolidation was assumed in our stability analyses, for short-term, staged, and long-term embankment stability and in providing estimates of construction stage and post-construction settlement.

Where proposed total embankment fill thickness, including overbuild fill and surcharge fill, will exceed 8 feet in height from foundation, we recommend monitoring proposed embankment or berm construction where slope stability analyses show low factors of safety. We recommend monitoring using inclinometers, settlement measurement systems, and vibrating wire piezometers installed in foundation soil. By monitoring, the possibility of slope failures during construction can be reduced. In other projects, a displacement ratio (horizontal displacement measured with an inclinometer/vertical settlement measured using settlement plates) has been used to assess slope stability. Typically, tangent displacement ratios less than 0.4 indicate the embankment is stable. Therefore, a series of threshold displacement ratio values are adopted for a project, e.g.:

- < 0.2 – No construction limitations
- 0.2 to 0.3 – Increase monitoring frequency
- 0.3 to 0.4 – Stop fill placement, prepare specific action plan
- > 0.4 – Implement action plan, e.g., buttress embankments, remove fill

6.1.10 Geotextile

A geotextile will be placed on the bottom of the new levee excavation to serve as a separation layer between the levee foundation fill and the native soils. The geotextile will reduce loss of these materials into the soft soil at the site during construction.

6.1.11 Geogrid Reinforcement

For this 95 percent design, the geogrid considered is a uniaxial grid with long-term design strength of 7,500 pounds per foot. The geogrid will aid in construction of the foundation and embankment stability. Two lower layers of geogrid will be placed on the geotextile in the new levee excavation before the placement of foundation fill. The geogrid layers are recommended to be placed in 1-foot lifts. Slope stability analyses showed that 4, 3 and 2 layers of geogrid are required for the southern , northern, and typical section, respectively. The Slope Stability section 6.1 describes geogrid requirements along the levee embankment in more details. The geogrid allows the construction of the new levee embankment and surcharge in Year 1 and also provides satisfactory performance for post seismic conditions.

6.1.12 Excavation Roadway

The excavation roadway will be placed on the lower geogrid layers along the centerline of the new levee. This roadway is required to improve access to the levee excavation for the placement of the levee foundation fill and for equipment. The roadway will be 2 feet thick and 20 feet. The roadway will be left in place.

6.1.13 Levee Excavation Select Fill

Select fill will be placed in the new levee excavation on both sides of the excavation roadway. The select fill will isolate the roadway and reduce possible underseepage of water through the foundation.

6.1.14 New Levee Embankment

The new levee embankment will be constructed in lifts to an elevation of 18.5 and 19.5 feet (without considering of settlement). This effort will include the placement of the surcharge soil load and result in quality compaction of the embankment soils

at the edges of the lifts during placement. The side slopes of the middle and south new levee embankments will be 1V:2H, due to overbuilding with the surcharge soil. The side slopes of the north new levee will be 1V:2H, which will result in the extension of the surcharge fill beyond the foot print of new levee excavation. These slopes are considered the maximum slopes possible for construction, for the minimal desired embankment stability.

6.1.15 Ebey Slough Levee Back Slope Reinforcement

The Ebey Slough levee between the Marysville Sewage Lagoon levee and the south end of the new levee embankment will be reinforced with light loose riprap to protect the levee in the event of overtopping by flood waters. The riprap will be placed to a slope of 1V:2.5H. The existing tide gate inlet will be extended before the riprap is placed.

6.1.16 Erosion Protection

The new levee embankment will be covered with at least 1 inch of fibrous hydromulch, straw, or other appropriate temporary erosion control at the end of Year 1 construction. These controls will protect the embankment from excessive erosion by precipitation and runoff during the winter months.

6.1.17 Access Restriction

Access restriction barriers and signage will be required to reduce public access to the top and slopes of the mulch covered embankment. Disturbance of the mulch and embankment by humans or recreational vehicles could result in local areas of erosion. Monitoring and maintenance of the mulch will be required until the start of Year 2 construction.

6.2 Year 2 Construction Efforts

6.2.1 Surcharge removal

The surcharge will be removed in Year 2. The surcharge will be removed to the design elevation of the new levee. The upper side slopes of the south and middle new levee will be cut to 1V:3H. The surcharge material will be used to construct the levee berms. The upper one-foot of native soil will be stripped from the berm

footprint before the placement of the surcharge material. The berms are required for long term stability and are an economical disposal location for the surcharge material. All surcharge material will be removed from the north new levee and used as fill for the middle new levee berms. The north new levee side slopes will be cut to 1V:2H on the waterward side and 1V:2.5H on the landward side. This is the minimum requirement for levee slopes in Snohomish County, Washington.

6.2.2 New Storm Water Pond

The new storm water pond will be installed during or after the removal of the surcharge from the new levee embankment.

6.2.3 Outlet Culvert

The outlet culvert will be installed in Year 2, after the soft soils under the new levee embankment have consolidated. The outlet culvert will extend from the new stormwater pond to the new channel of Allen Creek and the culvert will pass under the new levee embankment. This sequencing of construction is desired to reduce possible deformation of the culvert, if installed in Year 1, due to the load of the new levee embankment. In addition, the consolidation of the soil under the new levee embankment will provide a stronger foundation for the outlet culvert. The recommended backfill around the culvert is flowable earth fill. This material will reduce seepage and will not require as much compaction effort as select fill.

The new levee embankment would need to be removed to the foundation depth of the culvert along the culvert alignment. The culvert would be constructed on existing soils and the new levee embankment would be reconstructed to a height of elevation 15 feet. Minor overbuild of the reconstructed section of the new levee may be required to account of additional settlement. A geomembrane may be required on the water ward side of the reconstructed section of levee to reduce possible seepage through the newly placed fill in Year 2 when the Ebey Slough levee is breached and the restoration site will be inundated with water.

An additional analysis of the culvert foundation is required for the 95 percent design effort. It may be possible to install the outlet culvert in Year 1 if a concrete mat foundation and piles are used to prevent deformation of the culvert.

6.2.4 Slope Protection

Portions of the waterward face of the new levee will be protected with a combination of light loose and a custom-graded riprap underlain by aggregate filter layer (filter criteria and calculations are included in Appendix C). A buried toe will be installed along the lower slope of the North levee section adjacent to Allen Creek. Specific locations to receive riprap are indicated on the plans.

6.2.5 New Levee Roadway

The new levee roadway will be constructed on the top of the new levee after the placement of the slope protection. The road section will consist of 6 inches of crushed aggregate surface course. The top of the new levee will be sloped at 2 percent before the placement and compaction of the aggregate products.

6.2.6 Hydroseeding

A mixture of seed, fertilizer, and mulch will be spread on the new levee slopes and disturbed areas to be reclaimed. The desired seed mix is native species grasses. The seed mix is a standard Washington State Department of Transportation specification, and is reproduced in the project SEEDING specification section.

6.2.7 Fencing and Gates

Fencing may be required to restrict access to the new levee from vehicles and recreational vehicles.

6.2.8 Signage

Signage for safety and interpretive purposes may be required.

6.2.9 Tide gates

The existing tide gates at the south end of the new levee embankment will be closed. The tide gate to the west of the south end of the new levee will remain in place to provide site drainage in the event of the flooding of the protected area to the west of the new levee embankment.

6.2.10 Ebey Slough Breach Excavation

A large breach will be installed in the existing Ebey Slough levee at the end of Year 2 construction. The excavation will be advanced to the south towards the existing levee. The remaining section of the breach between the restoration site and Ebey Slough would be removed during a low tide cycle.

6.2.11 Disposal of Excavated Material

Excavated material from the levee excavation and the new storm water pond will be placed to the east of the new levee embankment. The material may be used to construct additional habitat berms and/or used to extend the levee berms into the restoration site to reduce wave run up on the new levee embankment. Excavated material from the breach will be placed on the landward side of the Ebey Slough levee in the breach area. Aggregate materials from the temporary haul roads may be used to provide a working surface and erosion protection to the top of the lowered section of the Ebey Slough levee at the breach opening and/or as erosion protection to the material excavated from the breach.

7.0 CONSTRUCTION CHALLENGES

7.1 North Allen Creek Channel

The north new levee is restricted in alignment due to property easement. Due to the limited space for construction, the levee section was changed from having side slopes of 1V:3H to a waterward slope of 1V:2H and a landward slope of 1V:2.5H. Levee berms will not be constructed along this section of the new levee. The alignment was shifted to the east due to the easement limitations. The east toe of the new levee currently extends over sections of the Allen Creek channel. In addition, the adjacent property owner to the east of the new levee intends to raise a section of his property with fill. The estimated distance between the east toe of the new levee embankment and the west toe of the fill is less than 100 feet. Therefore, the area of the channel has been decreased. Light loose riprap was selected as slope protection for the east slope of the north new levee to reduce the risk of scour of the embankment. Riprap slope protection may also be required on the west slope of the new fill.

7.2 Trees at South end of New Levee Embankment

A stand of cottonwood trees exist at the south end of the new levee embankment. Red-tailed Hawk nests are present in the branches of some of the trees. The trees may be removed after the fledglings have left the nest, which may be in late August. The south end of the new levee would be constructed in September. However, there are on-going discussions about keeping the trees in place. In this case, the new levee embankment would need to be shifted to the east. This option would reduce the area of the restoration site. However, the new levee alignment should not be shifted to the west since it will extend over the former environmental cleanup site. Also, the City of Marysville has expressed concern about the new levee connecting with the sewage lagoon levee due to possible damage caused by differential settlement between the sewage lagoon levee and the new levee. A final decision on the alignment of the south end of the levee must be made before the 95 percent design.

7.3 Ebey Slough Breach

The Ebey Slough breach and channel present challenges for excavation, safety, and release of turbid water from the site. The soil at the breach location is expected to be very soft silt. The excavation is expected to be filled with water on the landward side of the levee due to groundwater. The removal of soil in-the-wet may result in excessive disturbance of the soil and generation of loose sediment in the excavation. Excavated material may need to drain before it may be placed and shaped to grade at the disposal sites.

The excavation may need to be dewatered to reduce sediment generation. In this case, the side slope stability of the excavation may decrease due to the removal of the hydrostatic load of water in the excavation and slow drainage of groundwater from the soils into the excavation. A minimum safe distance of 75 feet from the face of the excavation may be advanced towards to the levee has been indicated on the plans.

The current desired approach to the levee breach is to complete the removal during a single low tide event. The amount of material that will need to be removed in this effort depends on the safe distance the channel excavation may be advanced towards the landward side of the levee.

Environmental requirements restrict the release of turbid water from the site. It is likely that the breach will produce significant amounts of suspended sediment; however, inflow velocities have been calculated to be graeter than outflow velocities. Limiting excavation to periods of low slack

tide and rising tides should allow sediment to flow into the project site where it will settle, thus reducing turbidity into Ebey Slough.

7.4 Marysville Sewage Lagoon Levee

The existing Marysville sewage lagoon levee was constructed in the early 1960's. The levee and foundation soils are composed of silts and organic-rich soils. Available geotechnical reports (Shannon& Wilson 2004) presented slope stability analyses for nearby sections of the levee. It is our professional opinion that distance between the sewage lagoon levee and this project is adequate to negate any potential adverse impacts on the static stability of the sewage lagoon levee.

7.5 Residential Areas

Residential areas exist to the east and north of the restoration site. The Tulalip Tribe's consultant has concluded that some infrastructure should be relocated, and the sponsor is taking steps to do so.

7.6 Wet Weather and Wet Condition Considerations

In the Puget Sound region, wet weather generally begins about mid-October and continues through about May, although rainy periods could occur at any time of the year. Thus, it would be advisable to schedule earthwork during the drier weather months of June through September. Most of the soil at the site likely contains sufficient fines to produce an unstable mixture when wet. Soil specified for construction of detention pond berms has high fines content. Soil with fines contents higher than 5 to 8 percent is highly susceptible to changes in water content and tends to become unstable and difficult or impossible to compact if the moisture content significantly exceeds the optimum. During wet weather months, the groundwater levels could increase, resulting in seepage into site excavations. Performing earthwork during dry weather would reduce these problems and costs associated with rainwater, trafficability, and handling of wet soil. Placing and compacting fill for the new levee may not be practicable during wet weather.

8.0 PLANS REVIEW AND CONSTRUCTION OBSERVATION

We recommend a Professional Geotechnical Engineer be retained to review those portions of the plans and specifications pertaining to foundations and earthwork to evaluate whether they are consistent with our recommendations contained in this report. We also recommend that a

licensed engineer be retained to observe the geotechnical aspects of construction. This observation will allow us to evaluate the subsurface conditions as they are exposed during construction, and to evaluate that the work is accomplished in accordance with to the recommendations included in the report. Scope of services should include observing earthwork construction, placement and compaction of embankment fill, and accomplishing other geotechnical related earthwork activities.

9.0 LIMITATIONS OF THE GEOTECHNICAL REPORT

The recommendations in this report are based on the best information available at the time preparation. Site soils may vary between subsurface explorations. The Contractor should immediately contact the USACE geotechnical designers in the event subsurface conditions are different from those assumed in this report.

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U.S. Army Corps
of Engineers
Seattle District

Qwuloolt Ecosystem Restoration Project Geotechnical Report

Marysville, WA

TABLES

TABLE 1.1
ESTIMATED IN-SITU FOUNDATION SOIL PROPERTIES FOR SETTLEMENT AND SLOPE STABILITY

Soil Type[1]	Top Depth (ft)	Bottom Depth (ft)	Thickness (ft)	Unit Weight (pcf)	Undrained Shear Strength (psf)	Internal Friction Angle (degrees)	Elastic Modulus (ksf)		OCR	$C_{c\varepsilon}$		C_v (ft^2/day)		$C_{\alpha\varepsilon}$
							Loading	Unloading/ Reloading		Loading	Unloading/ Reloading	Loading	Unloading/ Reloading	
Upper Clayey Silt	0	15	15	95	250	-	-	-	1	0.4	0.04	0.25	0	0.012
Lower Clayey Silt	15	30	15	105	900	-	-	-	1	0.25	0.025	0.1	0	0.008
Silty Sand	30	50	20	115	-	28	150	300	-	-	-	-	-	-

TABLE 1.2
ESTIMATED EFFECTIVE STRENGTH AND HYDRAULIC PROPERTIES

Soil Type[1]	Top Depth (ft)	Bottom Depth (ft)	Thickness (ft)	Unit Weight (pcf)	Internal Friction Angle (degrees)	Hydraulic Conductivity (cm/s)	Kh/Kv Ratio
Upper Clayey Silt	0	15	15	95	28	1.E-06	1
Lower Clayey Silt	15	30	15	105	25	1.E-06	1
Silty Sand	30	50	20	115	28	1.E-03	1
Levee Embankment (SM)	-	-	-	115	32	5.E-04	1
2" Minus Gravel	-	-	-	130	38	1.E-02	1
Quarry Spalls	-	-	-	130	38	1.E-01	1
Class II Riprap	-	-	-	135	45	1.E-01	1

TABLE 1.3
ESTIMATED LIQUEFIED RESIDUAL STRENGTH OF FOUNDATION SOILS

Soil Type[1]	Top Depth (ft)	Bottom Depth (ft)	Thickness (ft)	Unit Weight (pcf)	Undrained Shear Strength (psf)	Internal Friction Angle (degrees)
Liquefiable Clay-like Behavior Silt	0	10	15	95	50	-
Liquefiable Sand-like Behavior Silt	10	25	15	105	-	8

ft = feet
ksf = kips per square foot
OCR = overconsolidation ratio
pcf = pounds per cubic foot
cm = centimeters
s = second
Note: Soil type names in parentheses are the names of the soil unit layers in the slope stability and seepage analyses

TABLE 2
SUMMARY OF SLOPE STABILITY RESULTS

			During Construction				Long Term			Seismic	
			1st Lift[1]	2nd Lift[1]	3rd Lift[1]	Surcharge Load[2]	Static Slope Stability[3]	Long Term (Steady Seepage)[4]	Rapid Drawdown	Pseudo-Static[5]	Post-Seismic[6]
Minimum Factor of Safety* Min FOS			*1.3*	*1.3*	*1.3*	*1.3*	*1.3*	*1.4*	*1.0 - 1.2*	*1.1*	*1.1*
Levee Height (foot)	Station	Geogrids									
8	3+50	Yes	N/P	N/P	N/A	N/P	1.6 [7,8]	1.2 [7,8]	1.6 [7,8]	1.4 [7,8]	1.1 [7]
		No	N/P	N/P	N/A	N/P					0.3 [7]
10	23+50	Yes	2.1	1.5	N/A	1.3	1.7 [8]	1.4 [8]	1.2 [8]	1.4	1.1
		No	1.5	1.0	N/A	0.9				1.1	0.7
15	37+50	Yes	2.5	1.8	1.5	1.4	2.1 [8]	1.5 [8]	1.4 [8]	1.2	1.2
		No	1.6	1.0	0.8	0.8				1.0	0.7

* Minimum failure surface depth considered is 3 feet

N/P Not performed as results are expected to meet min FOS

N/A Not Applicable

1 The analysis was performed based on a pre-consoldiation strength properties of the foundation soils asuming the presence of a geogrid reinforcement layer(s)at the bottom of the levee embankment. More details are presented in the text.

2 Surcharge load applied to the constructed levee assumes a degree of consolidation of the foundation achievable from the construction sequence plan. The

3 Static slope stability analysis incorporates MHHW at about elevation 9-feet

4 Long term steady seepage analysis incorporates 10-year flood water elevation of 11.7 feet

5 Pseudo-static analysis incorporates a 0.09g (1/2 PGA)horizontal seismic coefficient and using the predetermined failure surface from the static stability analysis.

6 Post-seismic case incorporates resiudal liquefied strengths of the foundation soils based on the liquefaction analysis.

7 The minimum of either landward slope (1V:2.5H) or riverward slope (1V:2H)

8 Failure surface with min FOS passes through embankment

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Qwuloolt Ecosystem Restoration Project Geotechnical Report

Marysville, WA

APPENDIX A

Geotechnical Investigations

KLEINFELDER
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September 15, 2008
Kleinfelder Project No. 96156

ESA Adolfson
5309 Shilshole Avenue NW
Suite 200
Seattle, Washington. 98107

Attention: Mr. Dave Carlton

Subject: Subsurface Explorations and Laboratory Testing Data Report
Qwuloolt Levees, Bridge Abutments and Culvert – 35 Percent Design
155th Avenue SE and 156th Avenue SE
Marysville, Washington

Dear Mr. Carlton:

This letter transmits one electronic copy of our draft subsurface data report summarizing exploration and laboratory test results for the Qwuloolt Levees project in Marysville, Washington. This report was prepared in accordance with our *Proposal for Subsurface Explorations and Laboratory Testing Services*, dated July 22, 2008.

We appreciate the opportunity to provide geotechnical services to you on this project. We are prepared to finalize this report within one week of receiving review comments. Please contact the undersigned at (425) 562-4200 if you have any questions regarding this report or if we can provide assistance with other aspects of the project.

Sincerely,

KLEINFELDER WEST, INC.

DRAFT

Jason L. Washburn, P.E.
Project Geotechnical Engineer

Attachments: Subsurface Explorations and Laboratory Testing Data Report dated September 15, 2008

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Prepared for:

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Subsurface Explorations and Laboratory Testing Data Report Qwuloolt Levees, Bridge Abutments and Culvert – 35 Percent Design Marysville, Washington

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September 15, 2008
Kleinfelder Job No. 96156

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TABLE OF CONTENTS

1.0 INTRODUCTION 1
1.1 GENERAL 1
1.2 PROJECT DESCRIPTION 1
1.3 SCOPE OF SERVICES AND AUTHORIZATION 1

2.0 FIELD AND LABORATORY PROGRAMS 2
2.1 GENERAL 2
2.2 BORINGS 2
2.4 LABORATORY TESTING 3

3.0 SITE CONDITIONS 4
3.1 SURFACE CONDITIONS 4
3.2 GENERAL GEOLOGIC CONDITIONS 4
3.3 SOIL CONDITIONS 4
3.4 GROUNDWATER CONDITIONS 5

5.0 LIMITATIONS 6

FIGURES

Figure 1: Site Vicinity Map
Figure 2: Site & Exploration Plan – West Levee, Bridge Abutments, and Culvert
Figure 3: Site & Exploration Plan – Archie Levee
Figure 4: Site & Exploration Plan – Southeast Levee

APPENDICES

Appendix A: Exploration Logs
Appendix B: Laboratory Test Results
Appendix C: Important Information About Your Geotechnical Engineering Report

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1.0 INTRODUCTION

1.1 GENERAL

This report presents the results of Kleinfelder's subsurface exploration and laboratory testing performed in support of the Qwuloolt Levee Project located in Marysville, Washington. The project location and exploration locations are shown on the enclosed *Vicinity Map*, Figure 1, and *Site and Exploration Plan*, Figures 2, 3, and 4. The purpose of our study was to perform limited drilling explorations along the proposed levee alignments and at potential structure locations to provide data for use by others in preparing 35 percent design.

1.2 PROJECT DESCRIPTION

We understand that the Tulalip Tribe is partnering with the US Army Corps of Engineers (USACE) to restore the roughly 450 acre Qwuloolt Wetland. The plan is to breach an existing levee and allow tidal flows to inundate the wetlands. As currently envisioned, the project will include a 3,650-foot long levee located roughly parallel with 47th Avenue NE along the west boundary of the wetland, a 1,165-foot long levee located near the northeast corner of the site, a 575-foot long levee near the southeast corner of the wetland, a pedestrian bridge near the southwest corner of the site and a culvert replacement for Allen Creek at 3rd Street.

1.3 SCOPE OF SERVICES AND AUTHORIZATION

Our work was performed in accordance with the scope of work described in our *Proposal for Subsurface Explorations and Laboratory Testing Services*, dated July 22, 2008. Written authorization and a signed consultant Agreement for the work was provided on August 4, 2008.

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2.0 FIELD AND LABORATORY PROGRAMS

2.1 GENERAL

Surface and subsurface conditions at the project site were explored from August 5 through 13, 2008. The exploration and testing program included the following elements:

- A visual surface reconnaissance of the site;
- Eleven hollow-stem auger or mud rotary borings with Standard Penetration Tests, advanced at locations selected by USACE;
- 31 Hydrometer Analysis and Grain Size Distribution tests, performed on representative soil samples obtained from the borings;
- 30 Atterberg Limit Analysis, performed on representative soil samples obtained from the borings;
- 4 Wash Fines Analyses (percent passing the US Standard No. 200 sieve), performed on representative soil samples obtained from the borings; and
- 75 Moisture Content determinations, performed on representative soil samples obtained from the borings.

2.2 BORINGS

Eleven borings were advanced at the approximate locations shown on Figures 2 through 4; summary logs are presented in Appendix A. The borings were located in the field using a hand-held GPS and coordinates provided to us by the USACE. The locations should be considered approximate based on the methods used.

The borings were advanced using mud rotary and hollow-stem auger drilling techniques and extended to depths of 21½ to 51½ feet below the existing ground surface (bgs). Drilling was performed under the full-time observation of a Kleinfelder geotechnical engineer between August 5 and 13, 2008. A track-mounted drill rig, operated by an independent drilling firm working under subcontract to Kleinfelder, was utilized to provide off-road access. During drilling, the Kleinfelder geotechnical engineer logged the subsurface conditions and collected representative soil samples. All samples were stored in plastic bags and later transported to our laboratory for further visual examination and testing. After each boring was completed, the borehole was backfilled with bentonite chips.

Soil samples were obtained at 2½-foot intervals within the upper 10 feet and at 5-foot intervals below 10 feet by means of the Standard Penetration Test (SPT) per ASTM D-

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1586. This testing and sampling procedure consists of driving a standard 2-inch outside diameter steel split-spoon sampler 18 inches into the soil with a 140-pound hammer free-falling 30 inches. The number of blows required to drive the sampler through each 6-inch interval is counted, and the total number of blows struck during the final 12 inches is recorded as the Standard Penetration Resistance, or "SPT blow count."

Groundwater depth shown on the logs for the time of drilling were based on the moisture content of soil samples, the wetted height on the drilling rods, and the water level measured in the borehole after the auger was extracted. It should be noted that these initial groundwater depth readings can be unreliable. Mud rotary drilling uses fluids which make it difficult to approximate the groundwater elevations. Groundwater depths shown on the boring logs when this drilling method was used were approximated given the observed moisture content of the sampled soil.

2.3 LABORATORY TESTING

Laboratory tests are summarized and results presented in the Appendix B. Graphical results of certain laboratory tests are also depicted on the exploration logs in Appendix A.

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3.0 SITE CONDITIONS

3.1 SURFACE CONDITIONS

The project site is bounded by 47th Avenue NE to the west, 3rd Street/61st Street NE to the north, Sunnyside Boulevard to the east, and Snohomish River to the south. The wetland contains ditches that are either dry or water filled and Allen and Jones Creeks. Grasses cover a majority of the site with occasional scattered trees and blackberry bushes. Several un-maintained roads provide limited access to portions of the site. At the south end of the site, a levee prevents river and tidal flows from entering the site. Allen and Jones Creeks drain through the levee via backflow restrictors. Topography is in generally level and slopes upward at the edges of the site.

3.2 GENERAL GEOLOGIC CONDITIONS

General geological information for the project area was obtained from the *Geologic Map of The Marysville Quadrangle, Snohomish County, Washington, 1985,* which indicates the presence of recessional outwash and glacial till on the slopes surrounding the floodplain and recent alluvium in the floodplain.

3.3 SOIL CONDITIONS

Based on the borings advanced at the site, the soil conditions at the various locations generally consist of alluvium with minor amounts of fill and weathered till and advanced outwash.

Specific descriptions of the soils units include

- FILL: This unit was encountered in the upper 7 feet of boring B-10 located near the culvert. A thin layer of fill may have been present in the upper 1 or 2 feet of boring B-1. The fill encountered in boring B-10 generally consisted of medium dense, silty sand and sandy silt with pieces of concrete.
- ALLUVIUM: This unit was encountered in all borings and extended to the completed depths with the exception of boring B-7. Thicknesses encountered ranged from roughly 18½ to 51½ feet. The alluvium typically consisted of very loose/very soft to dense/very stiff, sandy silt, silty sand, clay, organic silt, peat, sand with variable silt, and silt with variable sand.
- WEATHERED GLACIAL TILL and ADVANCED OUTWASH: Soils underlying the alluvium at a depth of 18½ feet bgs in boring B-7 appeared to be medium dense to very dense, sand with trace silt and sand with silt.

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3.4 GROUNDWATER CONDITIONS

The groundwater conditions at the site, based on drilling observations ranged from 2 to 13 feet below the ground surface. Groundwater levels will likely fluctuate throughout the year in response to changing precipitation patterns, off-site construction activities, changes in site utilization, and other factors. Water levels are expected to be closely related to river, creek and tidal levels; higher levels will generally occur during the late fall, winter and early spring months, while lower levels may occur during the drier summer and early fall months.

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4.0 LIMITATIONS

The data contained in this report is based on the field explorations and our understanding of the proposed project. The investigation was performed using a mutually agreed upon scope of services, based on common geotechnical standard of practice. It is our opinion that this study was a cost-effective method to explore the subject site in support of 35 percent design.

It is possible that variations in soil and groundwater conditions exist between the points explored. The nature and extent of these variations may not be evident until construction occurs. If soil or groundwater conditions are encountered at this site that are different from those described in this report, our firm, and the design team, should be immediately notified so that any necessary revisions can be made by the designers. In addition, if the scope of the proposed project, locations of facilities, or design loads change from the descriptions given in this report, our firm, and the design team, should be notified.

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

This report has been prepared for use in the 35 percent design of the levees, pedestrian bridge and culvert replacement by ESA Adolfson, and their consultants, in accordance with the generally accepted geotechnical standards of practice at the time the data report was written. No warranty, express or implied, is made.

This report may be used only by ESA Adolfson, and their consultants, and only for the purposes stated within a reasonable time from its issuance, but in no event later than three years from the date of the report. Land or facility use, on and off-site conditions, regulations, or other factors may change over time, and additional work may be required with the passage of time. Any party other than ESA Adolfson who wishes to use this report shall notify Kleinfelder of such intended use. Based on the intended use of the report, Kleinfelder may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client, or anyone else, will release Kleinfelder from any liability resulting from the use of this

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report by any unauthorized party. In addition, the client agrees to defend, indemnify, and hold Kleinfelder harmless, from any claim or liability associated with such unauthorized use or non-compliance.

It is the responsibility of ESA Adolfson to see that all parties to the 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. Further guidelines and information on this geotechnical report can be found in the ASFE publication entitled: *Important Information About Your Geotechnical Engineering Report*, enclosed in Appendix C of this report.

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PLOTTED: 15 Sep 2008, 4:07pm, jstewart
CAD FILE: G:\96156\ LAYOUT: Vicinity
Images: Archie Levee.tif Images: Qwuloolt_Levee_Align.tif Images: SE Levee.tif Images: Vicinity.tif
ATTACHED IMAGES:
ATTACHED XREFS:
SEATTLE, WA

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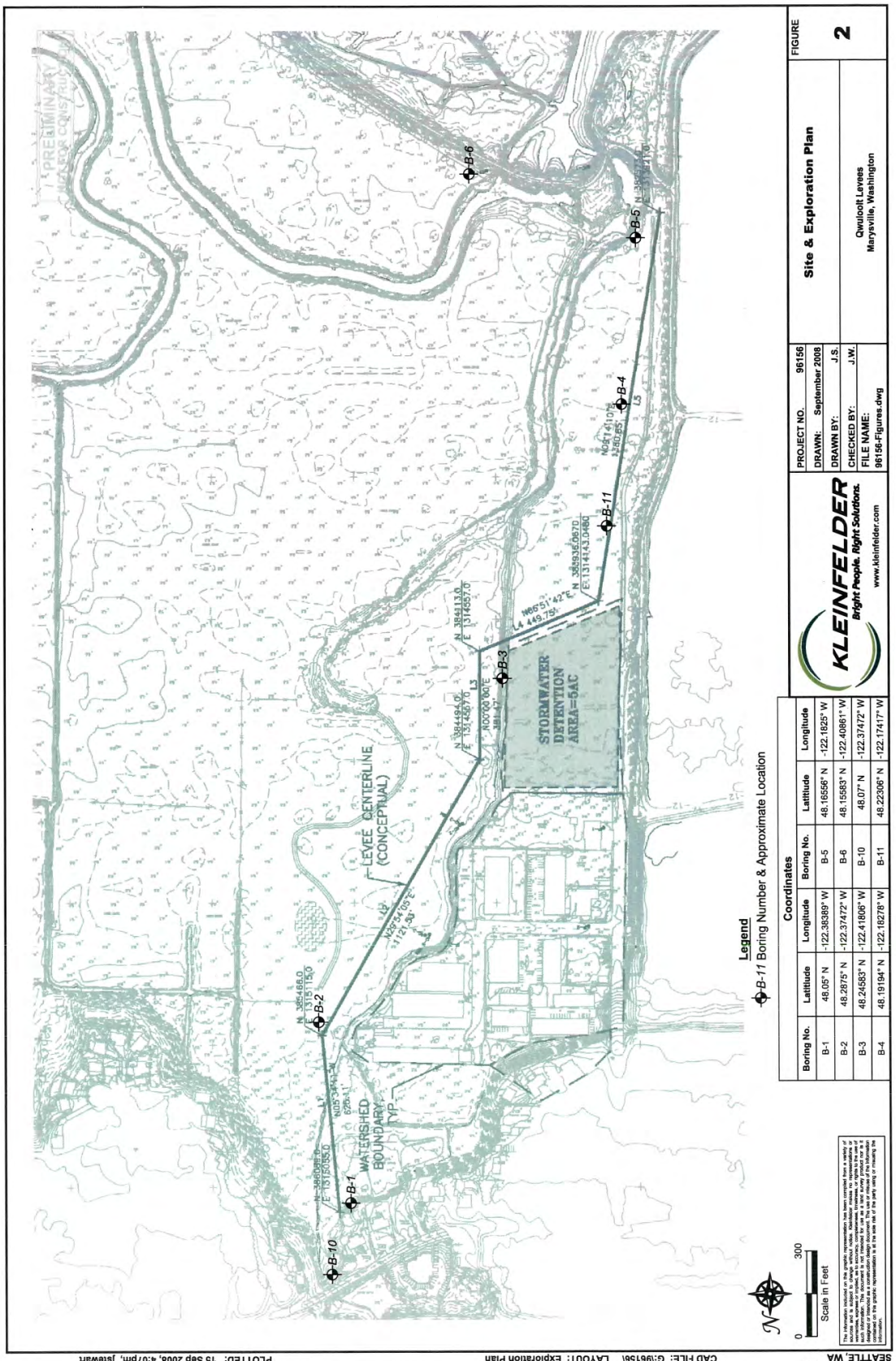
PROJECT NO.	96156
DRAWN:	September 2008
DRAWN BY:	J.S.
CHECKED BY:	J.W.
FILE NAME: 96156-Figures.dwg	

Site Vicinity

Qwuloolt Levees
Marysville, Washington

FIGURE
1



ATTACHED IMAGES: Images: Archie Levee.tif Images: Qwuloolt_Levee_Align.tif Images: SE Levee.tif Images: Vicinity.tif
ATTACHED XREFS:
SEATTLE, WA
CAD FILE: G:\96156\ LAYOUT: Exploration Plan
PLOTTED: 15 Sep 2008, 4:07pm, jstewart

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Legend

B-8 Boring Number & Approximate Location

Coordinates		
Boring No.	Latitiude	Longitude
B-7	48.04607° N	-122.15406° W
B-8	48.04602° N	-122.15213° W

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FILE NAME:	96156-Figures.dwg

Archie Levee

Qwuloolt Levees
Marysville, Washington

FIGURE
3

PLOTTED: 15 Sep 2008, 4:07pm, jstewart
CAD FILE: G:\96156\ LAYOUT: Archie Levee
ATTACHED IMAGES: Images: Archie Levee.tif Images: Qwuloolt_Levee_Align.tif Images: SE Levee.tif Images: Vicinity.tif
ATTACHED XREFS:
SEATTLE, WA

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Legend

B-9 Boring Number & Approximate Location

Coordinates

Boring No.	Latitiude	Longitude
B-9	48.03556° N	-122.15260° W

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SE Levee

Qwuloolt Levees
Marysville, Washington

FIGURE
4

PLOTTED: 15 Sep 2008, 4:08pm, jstewart
CAD FILE: G:\96156\ LAYOUT: SE Levee
ATTACHED IMAGES: Images: Archie Levee.tif Images: Qwuloolt_Levee_Align.tif Images: SE Levee.tif Images: Vicinity.tif
ATTACHED XREFS:
SEATTLE, WA

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APPENDIX A
EXPLORATION LOGS

DRAFT

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOIL MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, 0% TO 15% FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, 0% TO 15% FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, SILTY GRAVEL-SAND MIXTURES
				GC	CLAYEY GRAVELS, CLAYEY GRAVEL-SAND MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, 0% TO 15% FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, 0% TO 15% FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SILTY SAND-GRAVEL MIXTURES
				SC	CLAYEY SANDS, CLAYEY SAND-GRAVEL MIXTURES
FINE GRAINED SOIL MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

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NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

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CAD FILE: G:\96156\ LAYOUT: Layout1
ATTACHED IMAGES:
ATTACHED XREFS:
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CHECKED BY:	J.W.
FILE NAME:	Soil Classification.dwg

Soil Classification Legend

Qwuloolt Levees
Marysville, Washington

APPENDIX
A-1

DEPTH (feet)	WELL/PIEZO CONSTRUCTION	WATER LEVEL	TESTING PROGRAM: LAB ANALYSIS: MOISTURE CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	% PASSING No. 200 SIEVE	FIELD: Unit Weight (pcf)	RECOVERY	BLOWS/6 in** (uncorrected)	SAMPLER *	SAMPLE NUMBER	U.S.C.S. NAME	SYMBOL	SOIL DESCRIPTION
0												ML		Medium stiff, dark brown, moist to wet, SILT (ML): some sand.
		▽										SP		Loose, gray, wet, SAND (SP): fine sand, trace silt, some roots.
								10	1 2 3		S-1			
5			69.7					80	1 0 0		S-2	ML		Very soft, gray-brown, wet, SILT (ML): some grass and roots, trace to some fine sand, non-plastic.
			66.9					80	0 0 2		S-3			
10			63.8			90.7		100	0 1 1		S-4			
15			50.9					100	0 0 0		S-5			
20														

DATE DRILLED: 8-12-08
LOGGED BY: Jason Washburn
REVIEWED BY:

SURFACE ELEVATION (feet):
TOTAL DEPTH (feet): 31.5
DIAMETER OF BORING (in):

DRILLING METHOD: Hollow Stem Auger
DRILLING COMPANY: Holocene
CASING SIZE:

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SOILS AND MATERIALS TESTING
PROJECT NUMBER: 96156

Qwuloolt Levees
Marysville, Washington
BORING LOG
B-1

A-2a
PAGE 1 of 2

THIS SUMMARY APPLIES ONLY AT THIS LOCATION AND AT THE TIME OF LOGGING. CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH TIME. DATA PRESENTED IS A SIMPLIFICATION.

APPROV: _____ BY: _____

2007 STANDARD IN/OUT 96156-QWULLOOT.GPJ 2000REV.GDT 9/15/08

DEPTH (feet)	WELL/PIEZO CONSTRUCTION	WATER LEVEL	TESTING PROGRAM – LAB ANALYSIS: MOISTURE CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	% PASSING No. 200 SIEVE	Unit Weight (pcf)	FIELD: RECOVERY	BLOWS/6 in** (uncorrected)	SAMPLER *	SAMPLE NUMBER	U.S.C.S. NAME	U.S.C.S. SYMBOL	SOIL DESCRIPTION
20			30.1			49.3		100	4 7 6		S-6	SM		Medium dense, wet, gray-brown, Silty SAND (SM).
25			32.6	19.0	23.0	87.0		100	2 2 5		S-7	CL-ML		Some wood in sample S-7. Medium stiff, gray, wet, Silty CLAY (CL-ML); some wood/organics, trace sand.
30			31.3					90	1 3 7		S-8			Gray with olive mottling in sample S-8.
31.5														Boring completed at 31.5 feet bgs on 8/12/2008. Groundwater encountered at ~2 feet bgs at time of drilling. Boring backfilled with bentonite chips.

* SAMPLER TYPE: Cal. (3"OD) Split Spoon; SPT (2" OD) Split Spoon; Core Sample; Shelby Tube; Grab; No Recovery

**HAMMER WEIGHT: 300 lbs (30" Drop); 140 lbs (30" Drop)

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BY: APPROV:

2007 STANDARD IN/OUT 96156-QWULLOOT.GPJ 2000REV.GDT 9/15/08

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SOILS AND MATERIALS TESTING
PROJECT NUMBER: 96156

Qwuloolt Levees
Marysville, Washington
BORING LOG
B-1

A-2b
PAGE 2 of 2

DEPTH (feet)	WELL/PIEZO CONSTRUCTION	WATER LEVEL	TESTING PROGRAM: LAB ANALYSIS: MOISTURE CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	% PASSING No. 200 SIEVE	FIELD: Unit Weight (pcf)	RECOVERY	BLOWS/6 in** (uncorrected)	SAMPLER *	SAMPLE NUMBER	U.S.C.S. NAME	U.S.C.S. SYMBOL	SOIL DESCRIPTION
0												ML		Very soft, mottled gray and brown, moist to wet, SILT (ML): some organics and grass roots.
		▽	67.5					90	0 0 1	X	S-1			
5			61.3					90	0 0 1	X	S-2			
			109.3	54.0	68.0	89.1		100	0 0 1	X	S-3	MH		Very soft, gray to gray-brown, wet, Elastic SILT (MH): some organics.
10			43.4					100	0 0 0	X	S-4	ML		Very soft, gray, wet, SILT (ML): trace organics.
												ML		Soft, gray, wet, sandy SILT (ML): fine sand, trace organics, non-plastic.
15			22.9			52.0		90	1 2 1	X	S-5			
												SM		Very loose to medium dense, gray, wet, Silty SAND (SM).
20														

DATE DRILLED: 8-8-08
LOGGED BY: Jason Washburn
REVIEWED BY:

SURFACE ELEVATION (feet):
TOTAL DEPTH (feet): 31.5
DIAMETER OF BORING (in):

DRILLING METHOD: Mud Rotary
DRILLING COMPANY: Holocene
CASING SIZE:

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SOILS AND MATERIALS TESTING
PROJECT NUMBER: 96156

Qwuloolt Levees
Marysville, Washington
BORING LOG
B-2

A-3a
PAGE 1 of 2

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APPROV: ____ BY: ____

2007 STANDARD IN/OUT 96156-QWULLOOT.GPJ 2000REV.GDT 9/15/08

DEPTH (feet)	WELL/PIEZO CONSTRUCTION	WATER LEVEL	TESTING PROGRAM: LAB ANALYSIS: MOISTURE CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	% PASSING No. 200 SIEVE	FIELD: Unit Weight (pcf)	RECOVERY	BLOWS/6 in** (uncorrected)	SAMPLER *	SAMPLE NUMBER	U.S.C.S. NAME	U.S.C.S. SYMBOL	SOIL DESCRIPTION
20			25.0					80	3 1 2		S-6			
25			24.4			39.0		80	6 5 7		S-7			1.0" gray, wet, SILT layer at 25.6'.
30			26.2					80	8 10 13		S-8			Gray-brown in sample S-8.
31.5														Boring completed at 31.5 feet bgs on 8/8/2008. Groundwater encountered at approximately 3 feet bgs at time of drilling. Boring backfilled with bentonite chips.

* SAMPLER TYPE: Cal. (3"OD) Split Spoon; SPT (2" OD) Split Spoon; Core Sample; Shelby Tube; Grab; No Recovery

**HAMMER WEIGHT: 300 lbs (30" Drop); 140 lbs (30" Drop)

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BY: ____ APPROV: ____

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SOILS AND MATERIALS TESTING

PROJECT NUMBER: 96156

Qwuloolt Levees
Marysville, Washington
BORING LOG
B-2

A-3b
PAGE 2 of 2

2007 STANDARD IN/OUT 96156-QWULLOOT.GPJ 2000REV.GDT 9/15/08

DEPTH (feet)	WELL/PIEZO CONSTRUCTION	WATER LEVEL	TESTING PROGRAM: LAB ANALYSIS: MOISTURE CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	% PASSING No. 200 SIEVE	FIELD: Unit Weight (pcf)	RECOVERY	BLOWS/6 in** (uncorrected)	SAMPLER *	SAMPLE NUMBER	U.S.C.S. NAME	U.S.C.S. SYMBOL	SOIL DESCRIPTION
														SE Detention Pond - SE Levee
0												ML		Very soft, brown to dark brown-gray, moist, SILT (ML): peat and rootlets.
			164.7					90	1 0 1		S-1			
5			336.6					80	0 2 1		S-2	ML/OL		Very soft, dark brown with some gray, moist, SILT (ML/OL): peat, some wood in sampler tip.
		▽												
								0	0 0 0		S-3			
												ML		Very soft, gray, wet, SILT (ML): trace fine organics and sand, non-plastic.
10			30.9			96.2		100	1 0 0		S-4			
15			37.2					80	0 0 0		S-5			
												ML		Very soft, gray, wet, Sandy SILT (ML): fine sand, trace organics, non-plastic.
20														

DATE DRILLED: 8-5-08
LOGGED BY: Jason Washburn
REVIEWED BY:

SURFACE ELEVATION (feet):
TOTAL DEPTH (feet): 31.5
DIAMETER OF BORING (in):

DRILLING METHOD: Mud Rotary
DRILLING COMPANY: Holocene
CASING SIZE:

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SOILS AND MATERIALS TESTING
PROJECT NUMBER: 96156

Qwuloolt Levees
Marysville, Washington
BORING LOG
B-3

A-4a
PAGE 1 of 2

THIS SUMMARY APPLIES ONLY AT THIS LOCATION AND AT THE TIME OF LOGGING. CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH TIME. DATA PRESENTED IS A SIMPLIFICATION.

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2007 STANDARD IN/OUT 96156-QWULLOOT.GPJ 2000REV.GDT 9/15/08

DEPTH (feet)	WELL/PIEZO CONSTRUCTION	WATER LEVEL	TESTING PROGRAM – LAB ANALYSIS: MOISTURE CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	% PASSING No. 200 SIEVE	FIELD: Unit Weight (pcf)	RECOVERY	BLOWS/6 in** (uncorrected)	SAMPLER *	SAMPLE NUMBER	U.S.C.S. NAME	U.S.C.S. SYMBOL	SOIL DESCRIPTION
20			33.0					80	2, 1, 1		S-6			
25			34.6			65.2		100	1, 0, 1		S-7			
30			31.6			56.4		90	1, 1, 2		S-8			
31.5														

Boring completed at 31.5 feet bgs on 8/5/2008. Groundwater encountered at approximately 6 feet at time of drilling. Boring backfilled with bentonite chips.

THIS SUMMARY APPLIES ONLY AT THIS LOCATION AND AT THE TIME OF LOGGING. CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH TIME. DATA PRESENTED IS A SIMPLIFICATION.

BY: _____ APPROV: _____

* SAMPLER TYPE: Cal. (3"OD) Split Spoon; SPT (2" OD) Split Spoon; Core Sample; Shelby Tube; Grab; No Recovery

**HAMMER WEIGHT: 300 lbs (30" Drop); 140 lbs (30" Drop)

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SOILS AND MATERIALS TESTING

PROJECT NUMBER: 96156

Qwuloolt Levees
Marysville, Washington
BORING LOG
B-3

A-4b
PAGE 2 of 2

2007 STANDARD IN/OUT 96156-QWULLOOT.GPJ 2000REV.GDT 9/15/08

DEPTH (feet)	WELL/PIEZO CONSTRUCTION	WATER LEVEL	TESTING PROGRAM: LAB ANALYSIS: MOISTURE CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	% PASSING No. 200 SIEVE	FIELD: Unit Weight (pcf)	RECOVERY	BLOWS/6 in** (uncorrected)	SAMPLER *	SAMPLE NUMBER	U.S.C.S. NAME	U.S.C.S. SYMBOL	SOIL DESCRIPTION
0												ML		Soft to medium stiff, mottled gray and brown, moist, SILT (ML): some small roots.
			70.6					80	6 4 3		S-1			
5		▽	185.1					100	0 0 1		S-2	OL/ML		Very soft, dark brown to brown and gray, moist to wet, SILT (OL/ML): heavy organics and peat.
			87.1	52.0	70.0	85.9		100	0 0 1		S-3	MH		Very soft, dark brown to brown, wet, Elastic SILT (MH): some sand and organics.
10			48.8	33.0	45.0	97.7		100	0 0 0		S-4	ML		Very soft to soft, gray, wet, SILT (ML): trace fine sand and some organics and grass.
15			31.8					80	0 1 2		S-5			
												ML		Soft, gray, wet, SILT (ML): some fine sand, trace organics, non-plastic.
20														

DATE DRILLED: 8-6-08
LOGGED BY: Jason Washburn
REVIEWED BY:

SURFACE ELEVATION (feet):
TOTAL DEPTH (feet): 31.5
DIAMETER OF BORING (in):

DRILLING METHOD: Mud Rotary
DRILLING COMPANY: Holocene
CASING SIZE:

KLEINFELDER
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SOILS AND MATERIALS TESTING
PROJECT NUMBER: 96156

Qwuloolt Levees
Marysville, Washington
BORING LOG
B-4

A-5a
PAGE 1 of 2

THIS SUMMARY APPLIES ONLY AT THIS LOCATION AND AT THE TIME OF LOGGING. CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH TIME. DATA PRESENTED IS A SIMPLIFICATION.

BY: APPROV:

2007 STANDARD IN/OUT 96156-QWULLOOT.GPJ 2000REV.GDT 9/15/08

DEPTH (feet)	WELL/PIEZO CONSTRUCTION	WATER LEVEL	TESTING PROGRAM: LAB ANALYSIS: MOISTURE CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	% PASSING No. 200 SIEVE	FIELD: Unit Weight (pcf)	RECOVERY	BLOWS/6 in** (uncorrected)	SAMPLER *	SAMPLE NUMBER	U.S.C.S. NAME	U.S.C.S. SYMBOL	SOIL DESCRIPTION
20								100	0 1 2		S-6			
25			39.5			89.4		90	0 1 2		S-7			
30								100	1 1 1		S-8			
31.5														Boring completed at 31.5 feet bgs on 8/6/2008. Groundwater encountered at approximately 5 feet bgs at time of drilling. Boring backfilled with bentonite chips.

* SAMPLER TYPE: Cal. (3"OD) Split Spoon; SPT (2" OD) Split Spoon; Core Sample; Shelby Tube; Grab; No Recovery

**HAMMER WEIGHT: 300 lbs (30" Drop); 140 lbs (30" Drop)

THIS SUMMARY APPLIES ONLY AT THIS LOCATION AND AT THE TIME OF LOGGING. CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH TIME. DATA PRESENTED IS A SIMPLIFICATION.

APPROV: _____ BY: _____

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SOILS AND MATERIALS TESTING
PROJECT NUMBER: 96156

Qwuloolt Levees
Marysville, Washington
BORING LOG
B-4

A-5b
PAGE 2 of 2

2007 STANDARD IN/OUT 96156-QWULLOOT.GPJ 2000REV.GDT 9/15/08

DEPTH (feet)	WELL/PIEZO CONSTRUCTION	WATER LEVEL	TESTING PROGRAM: LAB ANALYSIS: MOISTURE CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	% PASSING No. 200 SIEVE	FIELD: Unit Weight (pcf)	RECOVERY	BLOWS/6 in** (uncorrected)	SAMPLER *	SAMPLE NUMBER	U.S.C.S. NAME	U.S.C.S. SYMBOL	SOIL DESCRIPTION
0												ML		Very soft, gray with some iron oxide staining, moist, SILT (ML): some small roots and organics.
			44.2					80	1 1 2	X	S-1			
5			28.0	33.0	36.0	78.2		100	0 0 2	X	S-2	ML		Very soft, gray, moist, SILT with Sand (ML): trace organics.
		∇	80.1					100	0 0 2	X	S-3			
10			118.8					100	0 0 0	X	S-4	ML		Very soft, gray, wet, SILT (ML): trace organics.
15			36.9					100	0 0 0	X	S-5			
20												ML		Very soft to soft, gray, wet, SILT with Sand (ML): trace organics, non-plastic.

DATE DRILLED: 8-6-08
LOGGED BY: Jason Washburn
REVIEWED BY:

SURFACE ELEVATION (feet):
TOTAL DEPTH (feet): 51.5
DIAMETER OF BORING (in):

DRILLING METHOD: Mud Rotary
DRILLING COMPANY: Holocene
CASING SIZE:

THIS SUMMARY APPLIES ONLY AT THIS LOCATION AND AT THE TIME OF LOGGING. CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH TIME. DATA PRESENTED IS A SIMPLIFICATION.

APPROV: _____ BY: _____

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SOILS AND MATERIALS TESTING
PROJECT NUMBER: 96156

Qwuloolt Levees
Marysville, Washington
BORING LOG
B-5

A-6a
PAGE 1 of 3

2007 STANDARD IN/OUT 96156-QWULLOOT.GPJ 2000REV.GDT 9/15/08

DEPTH (feet)	WELL/PIEZO CONSTRUCTION	WATER LEVEL	TESTING PROGRAM: LAB ANALYSIS: MOISTURE CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	% PASSING No. 200 SIEVE	FIELD: Unit Weight (pcf)	RECOVERY	BLOWS/6 in** (uncorrected)	SAMPLER *	SAMPLE NUMBER	U.S.C.S. NAME	U.S.C.S. SYMBOL	SOIL DESCRIPTION
20			23.1			72.1		100	0 0 0		S-6			
25			38.6					100	0 0 1		S-7			
												ML		Soft, gray, wet, Sandy SILT (ML); fine sand, non-plastic.
30			24.1			55.0		100	1 2 2		S-8			
												SM		Medium dense, gray, wet, Silty SAND (SM); trace to some organics, fine sand.
35			28.5					80	5 4 8		S-9			
40														

* SAMPLER TYPE: Cal. (3"OD) Split Spoon; SPT (2" OD) Split Spoon; Core Sample; Shelby Tube; Grab; No Recovery

**HAMMER WEIGHT: 300 lbs (30" Drop); 140 lbs (30" Drop)

THIS SUMMARY APPLIES ONLY AT THIS LOCATION AND AT THE TIME OF LOGGING. CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH TIME. DATA PRESENTED IS A SIMPLIFICATION.

BY: _____ APPROV: _____

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SOILS AND MATERIALS TESTING

PROJECT NUMBER: 96156

Qwuloolt Levees
Marysville, Washington
BORING LOG
B-5

A-6b

PAGE 2 of 3

2007 STANDARD IN/OUT 96156-QWULLOOT.GPJ 2000REV.GDT 9/15/08

DEPTH (feet)	WELL/PIEZO CONSTRUCTION	WATER LEVEL	TESTING PROGRAM: LAB ANALYSIS: MOISTURE CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	% PASSING No. 200 SIEVE	FIELD: Unit Weight (pcf)	RECOVERY	BLOWS/6 in** (uncorrected)	SAMPLER *	SAMPLE NUMBER	U.S.C.S. NAME	U.S.C.S. SYMBOL	SOIL DESCRIPTION
40								70	11 10 11		S-10			
45			41.8			39.2		90	2 5 10		S-11			
												SP-SM		Medium dense, gray, wet, SAND with Silt (SP-SM); trace organics and shells.
50			30.5					80	7 7 14		S-12			
51.5														

Boring completed at 51.5 feet bgs on 8/7/2008. Groundwater encountered at approximately 7 feet bgs at time of drilling. Boring backfilled with bentonite chips.

THIS SUMMARY APPLIES ONLY AT THIS LOCATION AND AT THE TIME OF LOGGING. CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH TIME. DATA PRESENTED IS A SIMPLIFICATION.

APPROV: _____

BY: _____

* SAMPLER TYPE: Cal. (3"OD) Split Spoon; SPT (2" OD) Split Spoon; Core Sample; Shelby Tube; Grab; No Recovery

**HAMMER WEIGHT: 300 lbs (30" Drop); 140 lbs (30" Drop)

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS
SOILS AND MATERIALS TESTING

PROJECT NUMBER: 96156

Qwuloolt Levees
Marysville, Washington
BORING LOG
B-5

A-6c

PAGE 3 of 3

2007 STANDARD IN/OUT 96156-QWULLOOT.GPJ 2000REV.GDT 9/15/08

DEPTH (feet)	WELL/PIEZO CONSTRUCTION	WATER LEVEL	MOISTURE CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	% PASSING No. 200 SIEVE	Unit Weight (pcf)	RECOVERY	BLOWS/6 in** (uncorrected)	SAMPLER *	SAMPLE NUMBER	U.S.C.S. NAME	U.S.C.S. SYMBOL	SOIL DESCRIPTION
0												ML		Soft, mottled brown, gray, and black, moist, SILT (ML): some pieces of ash or charcoal.
			79.9					80	1 2 1		S-1			
5		▽	129.5					100	0 0 0		S-2	ML		Very soft, gray, wet, SILT (ML): some organics, grass.
			172.3					100	0 0 0		S-3	OL		Very soft, brown-gray, wet, Organic SILT (OL): heavy organics.
10			57.9					100	0 0 2		S-4	ML		Very soft, gray, wet, SILT (ML): some to heavy organics.
												SM		Medium dense, gray, wet, Silty SAND (SM): fine sand.
15			26.8			26.7		80	10 12 6		S-5			
												ML		Medium stiff, gray, wet, Sandy SILT (ML): fine sand, non-plastic.
20														

TESTING PROGRAM: LAB ANALYSIS / FIELD

DATE DRILLED: 8-8-08
LOGGED BY: Jason Washburn
REVIEWED BY:

SURFACE ELEVATION (feet):
TOTAL DEPTH (feet): 51.5
DIAMETER OF BORING (in):

DRILLING METHOD: Mud Rotary
DRILLING COMPANY: Holocene
CASING SIZE:

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SOILS AND MATERIALS TESTING
PROJECT NUMBER: 96156

Qwuloolt Levees
Marysville, Washington
BORING LOG
B-6

A-7a
PAGE 1 of 3

THIS SUMMARY APPLIES ONLY AT THIS LOCATION AND AT THE TIME OF LOGGING. CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH TIME. DATA PRESENTED IS A SIMPLIFICATION.

APPROV: ____ BY: ____

2007 STANDARD IN/OUT 96156-QWULLOOT.GPJ 2000REV.GDT 9/15/08

DEPTH (feet)	WELL/PIEZO CONSTRUCTION	WATER LEVEL	TESTING PROGRAM – LAB ANALYSIS: MOISTURE CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	% PASSING No. 200 SIEVE	FIELD: Unit Weight (pcf)	RECOVERY	BLOWS/6 in** (uncorrected)	SAMPLER *	SAMPLE NUMBER	U.S.C.S. NAME	U.S.C.S. SYMBOL	SOIL DESCRIPTION
20			34.8			59.0		100	3 3 2		S-6			
												SP		Medium dense, gray, wet, SAND (SP): some to trace silt, fine to medium sand, trace organics.
25			23.5			11.6		90	9 9 16		S-7			
30			22.5					80	8 11 11		S-8			
												ML		Medium stiff, gray, wet, Sandy SILT (ML): fine sand and trace organics, non-plastic.
35			32.3			68.8		80	0 2 3		S-9			
												SP		Medium dense, gray, wet, SAND (SP): trace silt and organics, a shell.
40														

* SAMPLER TYPE: Cal. (3"OD) Split Spoon; SPT (2" OD) Split Spoon; Core Sample; Shelby Tube; Grab; No Recovery

**HAMMER WEIGHT: 300 lbs (30" Drop); 140 lbs (30" Drop)

THIS SUMMARY APPLIES ONLY AT THIS LOCATION AND AT THE TIME OF LOGGING. CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH TIME. DATA PRESENTED IS A SIMPLIFICATION.

BY: _____ APPROV: _____

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SOILS AND MATERIALS TESTING

PROJECT NUMBER: 96156

Qwuloolt Levees
Marysville, Washington
BORING LOG
B-6

A-7b
PAGE 2 of 3

2007 STANDARD IN/OUT 96156-QWULLOOT.GPJ 2000REV.GDT 9/15/08

DEPTH (feet)	WELL/PIEZO CONSTRUCTION	WATER LEVEL	TESTING PROGRAM: LAB ANALYSIS: MOISTURE CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	% PASSING No. 200 SIEVE	FIELD: Unit Weight (pcf)	RECOVERY	BLOWS/6 in** (uncorrected)	SAMPLER *	SAMPLE NUMBER	U.S.C.S. NAME	U.S.C.S. SYMBOL	SOIL DESCRIPTION
40			26.8					90	9 10 11		S-10			
												SP-SM		Medium dense, gray, wet, SAND with Silt (SP-SM): trace organics.
45			39.0					100	1 11 7		S-11			
												SP		Dense, gray, wet, SAND (SP): trace silt, fine to medium sand.
50			23.6			7.8		80	13 16 16		S-12			
51.5														

Boring completed at 51.5 feet bgs on 8/8/2008. Groundwater encountered at approximately 5 feet bgs at time of drilling. Boring backfilled with bentonite chips.

THIS SUMMARY APPLIES ONLY AT THIS LOCATION AND AT THE TIME OF LOGGING. CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH TIME. DATA PRESENTED IS A SIMPLIFICATION.

* SAMPLER TYPE: Cal. (3"OD) Split Spoon; SPT (2" OD) Split Spoon; Core Sample; Shelby Tube; Grab; No Recovery

**HAMMER WEIGHT: 300 lbs (30" Drop); 140 lbs (30" Drop)

APPROV: _____ BY: _____

2007 STANDARD IN/OUT 96156-QWULLOOT.GPJ 2000REV.GDT 9/15/08

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SOILS AND MATERIALS TESTING

PROJECT NUMBER: 96156

Qwuloolt Levees
Marysville, Washington
BORING LOG
B-6

A-7c

PAGE 3 of 3

DEPTH (feet)	WELL/PIEZO CONSTRUCTION	WATER LEVEL	TESTING PROGRAM: LAB ANALYSIS: MOISTURE CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	% PASSING No. 200 SIEVE	FIELD: Unit Weight (pcf)	RECOVERY	BLOWS/6 in** (uncorrected)	SAMPLER *	SAMPLE NUMBER	U.S.C.S. NAME	SYMBOL	SOIL DESCRIPTION
0												ML		Very soft, brown, dry, SILT (ML): heavy organics.
								10	2 0 1		S-1			
5								10	1 1 1		S-2	ML		Very soft, brown-gray, dry to moist, SILT (ML): trace organics, charred debris.
		▽										ML		Very soft, gray, wet, SILT (ML): trace sand and organics, non-plastic.
			33.0			96.3		100	0 0 0		S-3			
10								35	1 1 1		S-4			
15			22.2	23.0	27.0	96.7		100	0 3 13		S-5			
												ML		Stiff, brown-gray, wet, Sandy SILT (ML): fine to coarse sand, trace fine subround to subangular gravel.
												SP		Medium dense, gray, wet, SAND (SP): fine to medium sand, trace silt and subangular to subrounded gravel, non stratified.
20														

DATE DRILLED: 8-11-08
LOGGED BY: Chris Allen
REVIEWED BY:

SURFACE ELEVATION (feet):
TOTAL DEPTH (feet): 30.1
DIAMETER OF BORING (in):

DRILLING METHOD: Hollow Stem Auger
DRILLING COMPANY: Holocene
CASING SIZE:

THIS SUMMARY APPLIES ONLY AT THIS LOCATION AND AT THE TIME OF LOGGING. CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH TIME. DATA PRESENTED IS A SIMPLIFICATION.

APPROV: ____ BY: ____

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SOILS AND MATERIALS TESTING
PROJECT NUMBER: 96156

Qwuloolt Levees
Marysville, Washington
BORING LOG
B-7

A-8a
PAGE 1 of 2

2007 STANDARD IN/OUT 96156-QWULLOOT.GPJ 2000REV.GDT 9/15/08

DEPTH (feet)	WELL/PIEZO CONSTRUCTION	WATER LEVEL	TESTING PROGRAM – LAB ANALYSIS: MOISTURE CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	% PASSING No. 200 SIEVE	FIELD: Unit Weight (pcf)	RECOVERY	BLOWS/6 in** (uncorrected)	SAMPLER *	SAMPLE NUMBER	U.S.C.S. NAME	U.S.C.S. SYMBOL	SOIL DESCRIPTION
20			17.1			10.7		50	15 15 10		S-6			
												SP-SM		Very dense, gray, wet, SAND with Silt (SP-SM): fine to medium sand, trace subangular gravel, non stratified.
25								27	1 16 37		S-7			
30.1								0	50/1"		S-8			No recovery, possible cobble encountered at sample S-8.

Boring completed at 30.1 feet bgs on 8/11/2008. Groundwater encountered at approximately 6.5 feet bgs at time of drilling. Boring backfilled with bentonite chips.

THIS SUMMARY APPLIES ONLY AT THIS LOCATION AND AT THE TIME OF LOGGING. CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH TIME. DATA PRESENTED IS A SIMPLIFICATION.

APPROV: _____ BY: _____

* SAMPLER TYPE: Cal. (3"OD) Split Spoon; SPT (2" OD) Split Spoon; Core Sample; Shelby Tube; Grab; No Recovery

**HAMMER WEIGHT: 300 lbs (30" Drop); 140 lbs (30" Drop)

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SOILS AND MATERIALS TESTING

PROJECT NUMBER: 96156

Qwuloolt Levees
Marysville, Washington
BORING LOG
B-7

A-8b
PAGE 2 of 2

2007 STANDARD IN/OUT 96156-QWULLOOT.GPJ 2000REV.GDT 9/15/08

DEPTH (feet)	WELL/PIEZO CONSTRUCTION	WATER LEVEL	MOISTURE CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	% PASSING No. 200 SIEVE	Unit Weight (pcf)	RECOVERY	BLOWS/6 in** (uncorrected)	SAMPLER *	SAMPLE NUMBER	U.S.C.S. NAME	U.S.C.S. SYMBOL	SOIL DESCRIPTION
0												ML		Stiff, mottled gray and brown with some iron oxide staining, moist, SILT (ML): trace organics.
			24.4					70	1 5 8		S-1			
5								10	2 5 7		S-2			
			24.6	22.0	30.0	98.7		100	3 4 7		S-3	CL		Stiff, mottled gray and brown with some iron oxide staining, moist, CLAY (CL): with silt, trace sand and organics.
10			32.4					100	1 3 4		S-4	ML		Medium stiff, gray-brown with some iron oxide staining, moist to wet, SILT (ML); laminated, trace fine sand.
		∇										CL		Soft, gray, wet, CLAY (CL).
15			38.8					100	1 2 1		S-5			
20														

TESTING PROGRAM: LAB ANALYSIS / FIELD

DATE DRILLED: 8-11-08
LOGGED BY: Chris Allen
REVIEWED BY:

SURFACE ELEVATION (feet):
TOTAL DEPTH (feet): 31.5
DIAMETER OF BORING (in):

DRILLING METHOD: Hollow Stem Auger
DRILLING COMPANY: Holocene
CASING SIZE:

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SOILS AND MATERIALS TESTING
PROJECT NUMBER: 96156

Qwuloolt Levees
Marysville, Washington
BORING LOG
B-8

A-9a
PAGE 1 of 2

THIS SUMMARY APPLIES ONLY AT THIS LOCATION AND AT THE TIME OF LOGGING. CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH TIME. DATA PRESENTED IS A SIMPLIFICATION.

BY: APPROV:

2007 STANDARD IN/OUT 96156-QWULLOOT.GPJ 2000REV.GDT 9/15/08

DEPTH (feet)	WELL/PIEZO CONSTRUCTION	WATER LEVEL	TESTING PROGRAM – LAB ANALYSIS: MOISTURE CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	% PASSING No. 200 SIEVE	FIELD: Unit Weight (pcf)	RECOVERY	BLOWS/6 in** (uncorrected)	SAMPLER *	SAMPLE NUMBER	U.S.C.S. NAME	U.S.C.S. SYMBOL	SOIL DESCRIPTION
20			37.1	21.0	30.0	99.9		100	1 0 1		S-6			
25			38.7					100	1 2 1		S-7			
												ML		Very soft, gray, wet, SILT (ML): trace sand.
30			19.3	23.0	30.0	98.0		100	0 0 1		S-8			
31.5														

Boring completed at 31.5 feet bgs on 8/11/2008. Groundwater encountered at approximately 13 feet bgs at time of drilling. Boring backfilled with bentonite chips.

THIS SUMMARY APPLIES ONLY AT THIS LOCATION AND AT THE TIME OF LOGGING. CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH TIME. DATA PRESENTED IS A SIMPLIFICATION.

BY: _____ APPROV: _____

2007 STANDARD IN/OUT 96156-QWULLOOT.GPJ 2000REV.GDT 9/15/08

* SAMPLER TYPE: Cal. (3"OD) Split Spoon; SPT (2" OD) Split Spoon; Core Sample; Shelby Tube; Grab; No Recovery

**HAMMER WEIGHT: 300 lbs (30" Drop); 140 lbs (30" Drop)

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SOILS AND MATERIALS TESTING

PROJECT NUMBER: 96156

Qwuloolt Levees
Marysville, Washington
BORING LOG
B-8

A-9b

PAGE 2 of 2

DEPTH (feet)	WELL/PIEZO CONSTRUCTION	WATER LEVEL	TESTING PROGRAM: LAB ANALYSIS: MOISTURE CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	% PASSING No. 200 SIEVE	FIELD: Unit Weight (pcf)	RECOVERY	BLOWS/6 in** (uncorrected)	SAMPLER *	SAMPLE NUMBER	U.S.C.S. NAME	U.S.C.S. SYMBOL	SOIL DESCRIPTION
0												ML		Very soft, brown, moist, SILT (ML): trace sand and organics.
								0	1 0 1		S-1			
5		▽						10	1 0 1		S-2	PT		Very soft, brown, moist to wet, PEAT (PT): wood and organics.
								10	0 0 0		S-3	OL		Very soft, brown, wet, Organic SILT (OL): wood and organics.
10			75.3					30	11 1 1		S-4	ML		Very soft, gray and brown, wet, SILT (ML): trace organics/wood, some plasticity.
15			20.0			40.9		66	12 15 14		S-5	SM		Medium dense, gray, moist to wet, Silty SAND (SM): fine sand with zone of fine to medium sand, trace subround to subangular gravel.
20														

DATE DRILLED: 8-12-08
LOGGED BY: Chris Allen
REVIEWED BY:

SURFACE ELEVATION (feet):
TOTAL DEPTH (feet): 31.5
DIAMETER OF BORING (in):

DRILLING METHOD: Hollow Stem Auger
DRILLING COMPANY: Holocene
CASING SIZE:

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SOILS AND MATERIALS TESTING
PROJECT NUMBER: 96156

Qwuloolt Levees
Marysville, Washington
BORING LOG
B-9

A-10a
PAGE 1 of 2

THIS SUMMARY APPLIES ONLY AT THIS LOCATION AND AT THE TIME OF LOGGING. CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH TIME. DATA PRESENTED IS A SIMPLIFICATION.

BY: _____ APPROV: _____

2007 STANDARD IN/OUT 96156-QWULLOOT.GPJ 2000REV.GDT 9/15/08

DEPTH (feet)	WELL/PIEZO CONSTRUCTION	WATER LEVEL	TESTING PROGRAM: LAB ANALYSIS: MOISTURE CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	% PASSING No. 200 SIEVE	FIELD: Unit Weight (pcf)	RECOVERY	BLOWS/6 in** (uncorrected)	SAMPLER *	SAMPLE NUMBER	U.S.C.S. NAME	U.S.C.S. SYMBOL	SOIL DESCRIPTION
20			16.5			41.3		100	9 13 17		S-6			Interbedded SILT layers.
												ML		Very soft, gray, wet, SILT (ML): interbedded trace sandy silt and silt, trace organics.
25			25.3					30	1 1 1		S-7			
												ML		Hard, gray, wet, Sandy SILT (ML): silt is interbedded/chaotically stratified with fine sand with silt.
30			21.9			54.2		100	4 14 20		S-8			
31.5														Boring completed at 31.5 feet bgs on 8/13/2008. Groundwater encountered at approximately 5.5 feet bgs at time of drilling. Boring backfilled with bentonite chips.

* SAMPLER TYPE: Cal. (3"OD) Split Spoon; SPT (2" OD) Split Spoon; Core Sample; Shelby Tube; Grab; No Recovery

**HAMMER WEIGHT: 300 lbs (30" Drop); 140 lbs (30" Drop)

THIS SUMMARY APPLIES ONLY AT THIS LOCATION AND AT THE TIME OF LOGGING. CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH TIME. DATA PRESENTED IS A SIMPLIFICATION.

BY: APPROV:

KLEINFELDER Bright People. Right Solutions.

GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS
SOILS AND MATERIALS TESTING

PROJECT NUMBER: 96156

Qwuloolt Levees
Marysville, Washington
BORING LOG
B-9

A-10b
PAGE 2 of 2

2007 STANDARD IN/OUT 96156-QWULLOOT.GPJ 2000REV.GDT 9/15/08

DEPTH (feet)	WELL/PIEZO CONSTRUCTION	WATER LEVEL	TESTING PROGRAM: LAB ANALYSIS: MOISTURE CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	% PASSING No. 200 SIEVE	FIELD: Unit Weight (pcf)	RECOVERY	BLOWS/6 in** (uncorrected)	SAMPLER *	SAMPLE NUMBER	U.S.C.S. NAME	U.S.C.S. SYMBOL	SOIL DESCRIPTION
0												SM		Medium dense, brown, moist, Silty SAND (SM): some gravel, piece of concrete. (FILL)
								20	44 6 8		S-1			
5								40	3 3 3		S-2	ML		Medium stiff, brown, moist, Sandy SILT (ML): some gravel. (FILL)
		▽	21.9			62.4		40	2 1 1		S-3	ML		Soft to medium stiff, gray-brown, wet, SILT with Sand (ML); trace organics, non-plastic.
10			19.2					40	2 4 3		S-4			Some gravel in sample S-4.
												ML		Soft, gray-brown, wet, SILT (ML): some fine sand, heavy organics, non-plastic.
15			32.0			91.8		80	0 1 2		S-5			
												ML		Very soft, gray-brown, wet, SILT (ML): some fine sand and organics.
20														

DATE DRILLED: 8-13-08
LOGGED BY: Jason Washburn
REVIEWED BY:

SURFACE ELEVATION (feet):
TOTAL DEPTH (feet): 21.5
DIAMETER OF BORING (in):

DRILLING METHOD: Hollow Stem Auger
DRILLING COMPANY: Holocene
CASING SIZE:

THIS SUMMARY APPLIES ONLY AT THIS LOCATION AND AT THE TIME OF LOGGING. CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH TIME. DATA PRESENTED IS A SIMPLIFICATION.

APPROV: _____ BY: _____

KLEINFELDER Bright People. Right Solutions.
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS
SOILS AND MATERIALS TESTING
PROJECT NUMBER: 96156

Qwuloolt Levees
Marysville, Washington
BORING LOG
B-10

A-11a
PAGE 1 of 2

2007 STANDARD IN/OUT 96156-QWULLOOT.GPJ 2000REV.GDT 9/15/08

DEPTH (feet)	WELL/PIEZO CONSTRUCTION	WATER LEVEL	TESTING PROGRAM: LAB ANALYSIS: MOISTURE CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	% PASSING No. 200 SIEVE	FIELD: Unit Weight (pcf)	RECOVERY	BLOWS/6 in** (uncorrected)	SAMPLER *	SAMPLE NUMBER	U.S.C.S.: NAME	SYMBOL	SOIL DESCRIPTION
20			68.7	37.0	47.0	91.7		80	0 1 1		S-6			
21.5														Boring completed at 21.5 feet bgs on 8/13/2008. Groundwater encountered at approximately 7 feet bgs at time of drilling. Boring backfilled with bentonite chips.

* SAMPLER TYPE: Cal. (3"OD) Split Spoon; SPT (2" OD) Split Spoon; Core Sample; Shelby Tube; Grab; No Recovery

**HAMMER WEIGHT: 300 lbs (30" Drop); 140 lbs (30" Drop)

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS
SOILS AND MATERIALS TESTING

PROJECT NUMBER: 96156

Qwuloolt Levees
Marysville, Washington
BORING LOG
B-10

A-11b
PAGE 2 of 2

THIS SUMMARY APPLIES ONLY AT THIS LOCATION AND AT THE TIME OF LOGGING. CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH TIME. DATA PRESENTED IS A SIMPLIFICATION.

BY: _____ APPROV: _____

2007 STANDARD IN/OUT 96156-QWULLOOT.GPJ 2000REV.GDT 9/15/08

DEPTH (feet)	WELL/PIEZO CONSTRUCTION	WATER LEVEL	TESTING PROGRAM: LAB ANALYSIS: MOISTURE CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	% PASSING No. 200 SIEVE	FIELD: Unit Weight (pcf)	RECOVERY	BLOWS/6 in** (uncorrected)	SAMPLER *	SAMPLE NUMBER	U.S.C.S. NAME	U.S.C.S. SYMBOL	SOIL DESCRIPTION
0												ML		Very soft, gray-brown, wet, SILT (ML): trace fine sand and trace organics, non-plastic.
								5	2 0 0		S-1			
5		▽						0	1 0 0		S-2			
			84.1			96.5		100	0 0 0		S-3			Heavy organics, grass in sample S-3.
10			32.0					100	0 0 0		S-4			Trace organics in sample S-4.
												ML		Soft, gray-brown, wet, SILT with Sand (ML): fine sand, trace organics, non-plastic.
15			31.5			79.9		100	1 1 2		S-5			
20														

DATE DRILLED: 8-13-08
LOGGED BY: Jason Washburn
REVIEWED BY:

SURFACE ELEVATION (feet):
TOTAL DEPTH (feet): 31.5
DIAMETER OF BORING (in):

DRILLING METHOD: Hollow Stem Auger
DRILLING COMPANY: Holocene
CASING SIZE:

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS
SOILS AND MATERIALS TESTING
PROJECT NUMBER: 96156

Qwuloolt Levees
Marysville, Washington
BORING LOG
B-11

A-12a
PAGE 1 of 2

THIS SUMMARY APPLIES ONLY AT THIS LOCATION AND AT THE TIME OF LOGGING. CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH TIME. DATA PRESENTED IS A SIMPLIFICATION.

BY: APPROV:

2007 STANDARD IN/OUT 96156-QWULLOOT.GPJ 2000REV.GDT 9/15/08

DEPTH (feet)	WELL/PIEZO CONSTRUCTION	WATER LEVEL	TESTING PROGRAM: LAB ANALYSIS: MOISTURE CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	% PASSING No. 200 SIEVE	FIELD: Unit Weight (pcf)	RECOVERY	BLOWS/6 in** (uncorrected)	SAMPLER *	SAMPLE NUMBER	U.S.C.S. NAME	U.S.C.S. SYMBOL	SOIL DESCRIPTION
20									0 1 2		S-6			
25			31.2					100	1 3 6		S-7	SM		Loose to medium dense, gray-brown, wet, Silty SAND (SM); some organics.
30			27.3			20.1		100	1 4 8		S-8			
31.5														Boring completed at 31.5 feet bgs on 8/13/2008. Groundwater encountered at approximately 5 feet bgs at time of drilling. Boring backfilled with bentonite chips.

* SAMPLER TYPE: Cal. (3"OD) Split Spoon; SPT (2" OD) Split Spoon; Core Sample; Shelby Tube; Grab; No Recovery

**HAMMER WEIGHT: 300 lbs (30" Drop); 140 lbs (30" Drop)

THIS SUMMARY APPLIES ONLY AT THIS LOCATION AND AT THE TIME OF LOGGING. CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH TIME. DATA PRESENTED IS A SIMPLIFICATION.

APPROV: ____ BY: ____

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS
SOILS AND MATERIALS TESTING
PROJECT NUMBER: 96156

Qwuloolt Levees
Marysville, Washington
BORING LOG
B-11

A-12b
PAGE 2 of 2

2007 STANDARD IN/OUT 96156-QWULLOOT.GPJ 2000REV.GDT 9/15/08

DRAFT-FOR REVIEW PURPOSES ONLY

APPENDIX B
GEOTECHNICAL LABORATORY TESTING

B.1 GENERAL

Laboratory tests were conducted on representative soil samples to better identify the soil classification of the units encountered and to evaluate the material's general physical properties and engineering characteristics. A brief description of the tests performed for this study is provided below. The results of laboratory tests performed on specific samples are provided at the appropriate sample depths on the individual boring logs. However, it is important to note that these test results may not accurately represent in situ soil conditions.

B.2 SOIL CLASSIFICATION

Soil samples were visually examined in the field by our representative at the time they were obtained. They were subsequently packaged and returned to our laboratory where they were reexamined and the original description checked and verified or modified. With the help of information obtained from the other classification tests, described below, the samples were described in general accordance with the Unified Classification System, ASTM Standard D2487. The resulting descriptions are provided at the appropriate locations on the individual boring logs, located in Appendix A, and are qualitative only.

B.3 GRAIN-SIZE DISTRIBUTION AND HYDROMETER

Detailed grain-size distribution analyses with hydrometers were conducted on 31 samples in general accordance with ASTM Standard D422 to determine the grain-size distribution of the on-site soil.

B.4 MOISTURE CONTENT

Moisture content tests were performed on 75 samples obtained from the borings. The purpose of these tests is to approximately ascertain the in-place moisture content of the soil sample at the time it was collected. The moisture content is determined in general accordance with ASTM Standard D2216. The results of these tests are presented at the appropriate sample depths on the exploration logs.

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B.5 ATTERBERG LIMITS

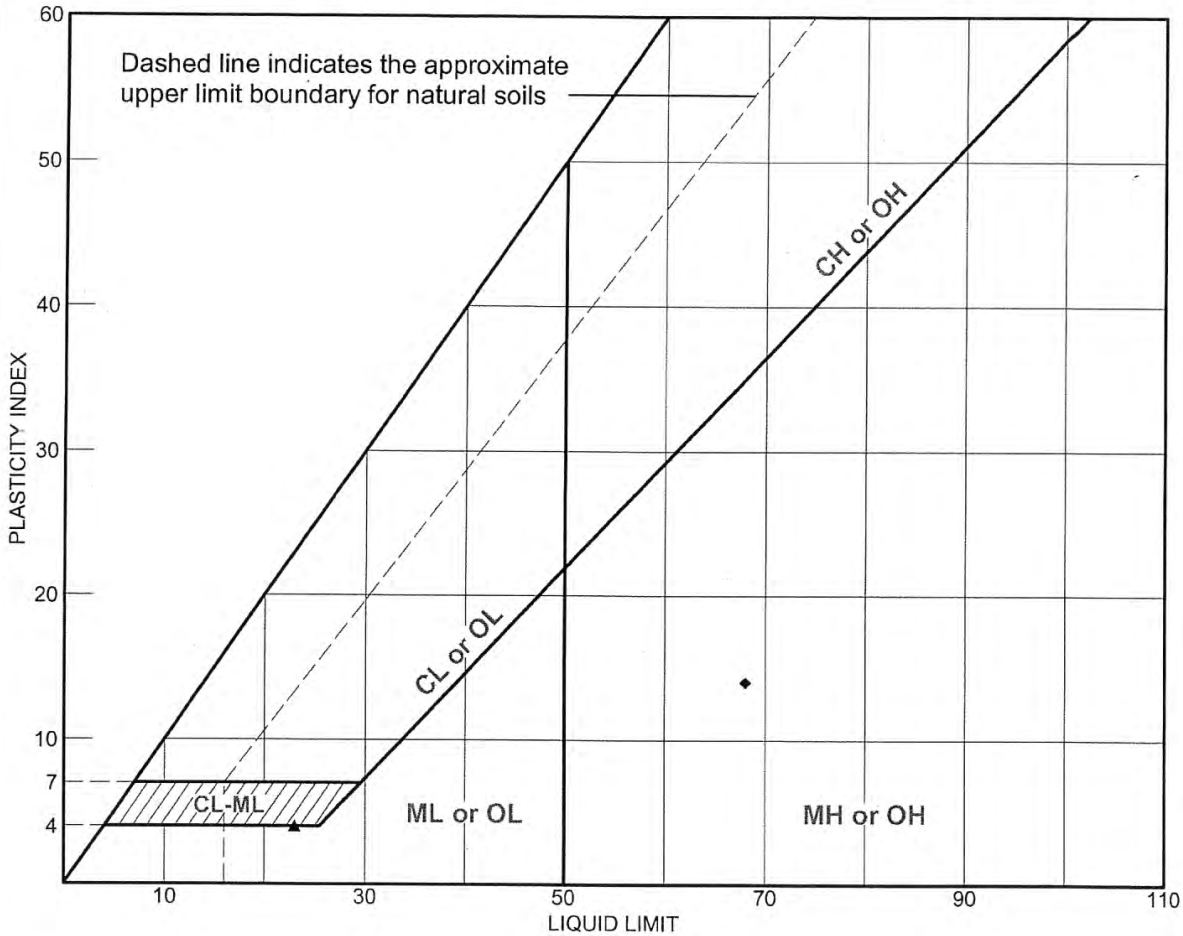
The plastic and liquid limits were determined for 30 samples in general accordance with ASTM D4318. The results, when not non-plastic (NP), are presented at the appropriate sample depths on the exploration logs.

B.6 PERCENT FINES (FINES WASH)

Percent fines tests were performed on four samples obtained from the borings. These tests indicate the portion of the sample that is considered to be fine grained (passing the US Standard No. 200 size sieve). The tests were performed in general accordance with ASTM D422.

DRAFT

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA

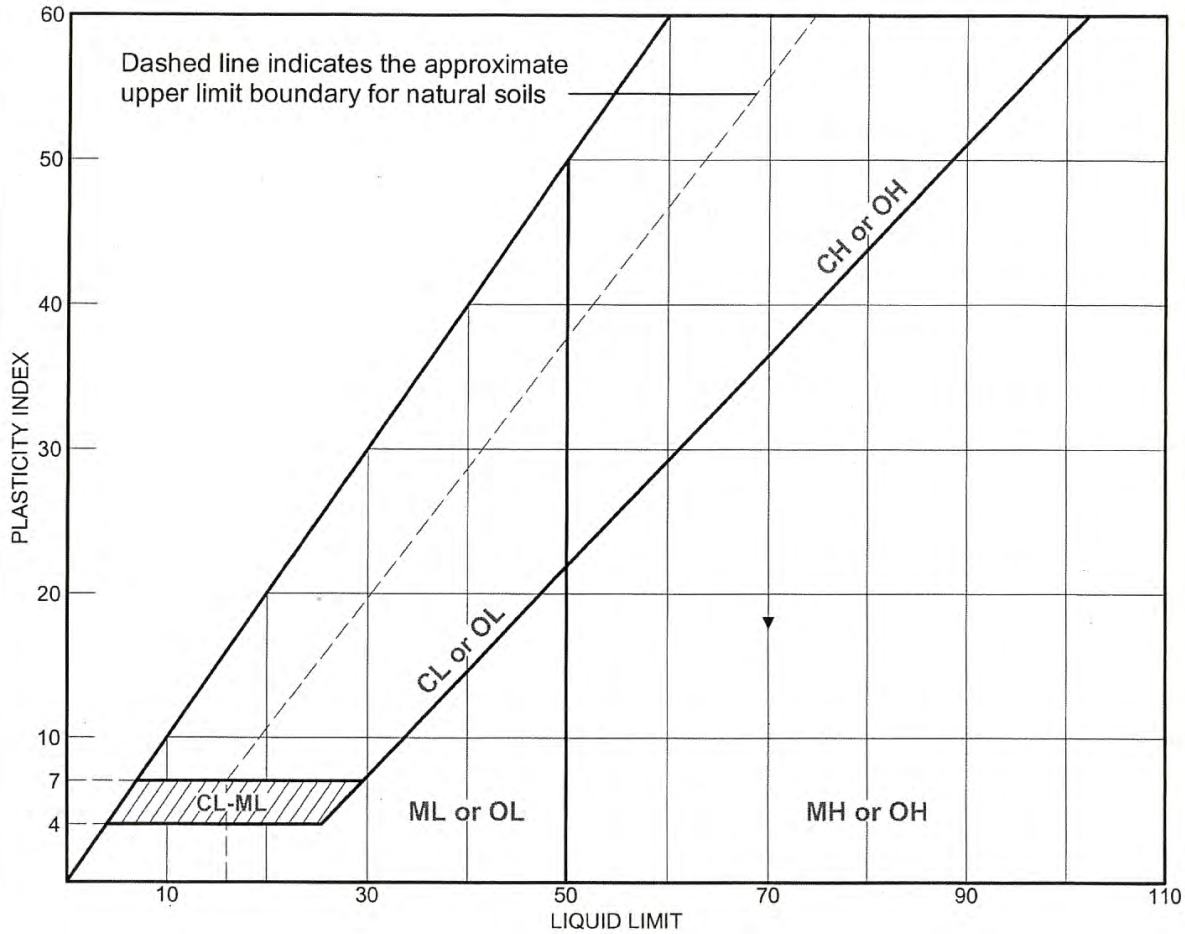
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	B-1	8200	10'	63.8	NP	NV	NP	ML
■	B-1	8200	20'	30.1	NP	NV	NP	SM
▲	B-1	8200	30'	31.3	19	23	4	CL-ML
◆	B-2	8200	7.5'	109.3	54	68	14	MH
▼	B-2	8200	15'	22.9	NP	NV	NP	ML

LIQUID AND PLASTIC LIMITS TEST REPORT

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No.: 96156 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA

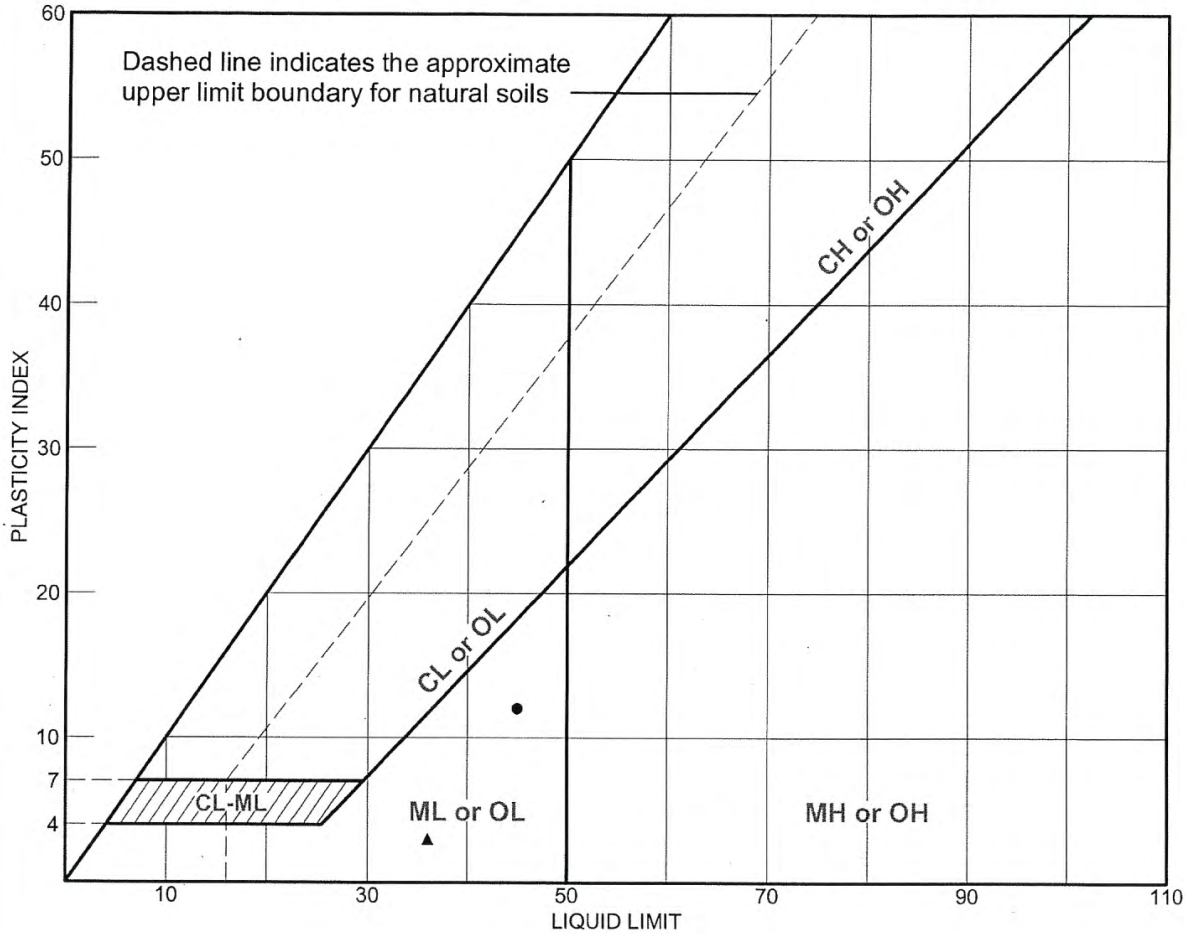
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	B-2	8200	25'	24.4	NP	NV	NP	SM
■	B-3	8200	10'	30.9	NP	NV	NP	ML
▲	B-3	8200	25'	34.6	NP	NV	NP	ML
◆	B-3	8200	30'	31.6	NP	NV	NP	ML
▼	B-4	8200	7.5'	87.1	52	70	18	MH

LIQUID AND PLASTIC LIMITS TEST REPORT

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No.: 96156 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA

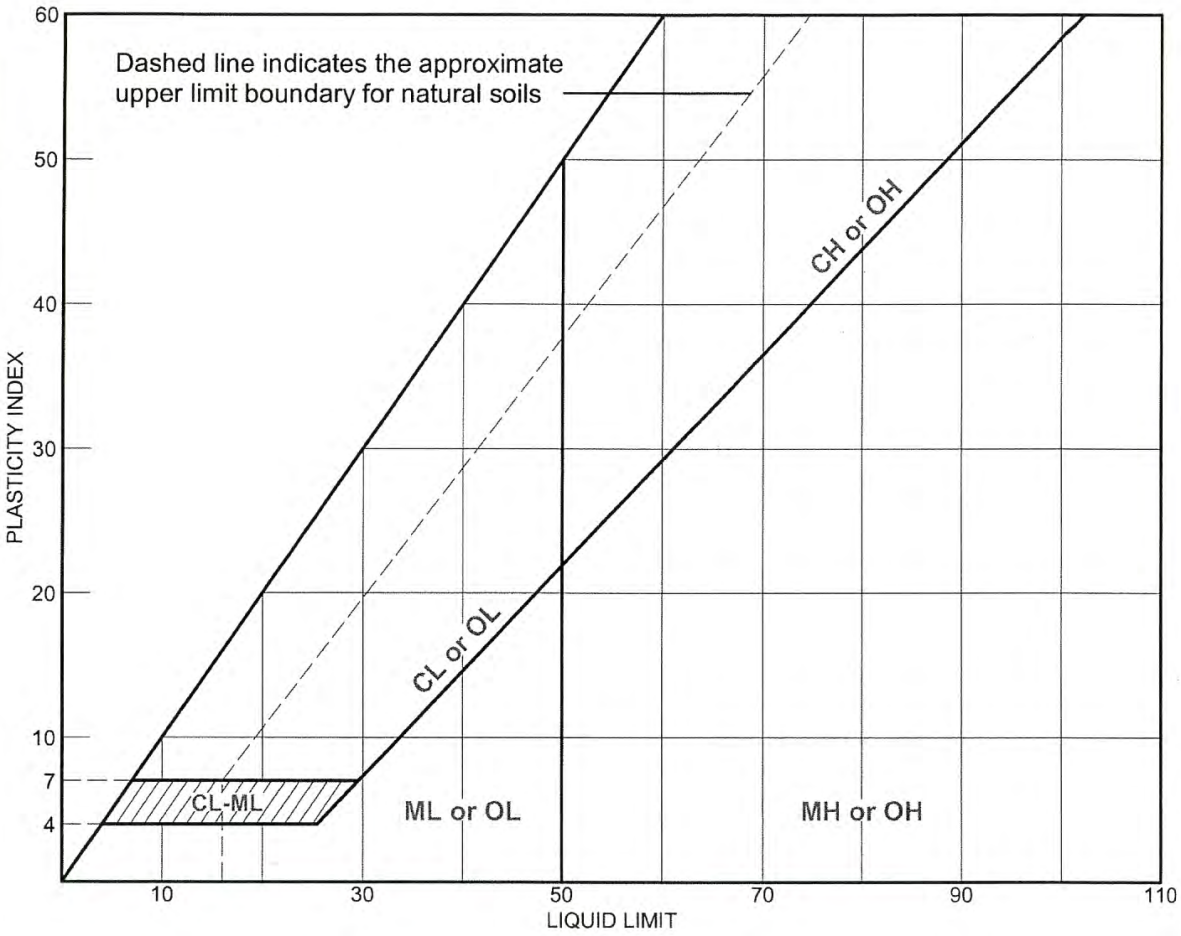
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	B-4	8200	10'	48.8	33	45	12	ML
■	B-4	8200	25'	39.5	NP	NV	NP	ML
▲	B-5	8200	5'	28.0	33	36	3	ML
◆	B-5	8200	20'	23.1	NP	NV	NP	ML
▼	B-5	8200	30'	24.1	NP	NV	NP	ML

LIQUID AND PLASTIC LIMITS TEST REPORT

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No.: 96156 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA

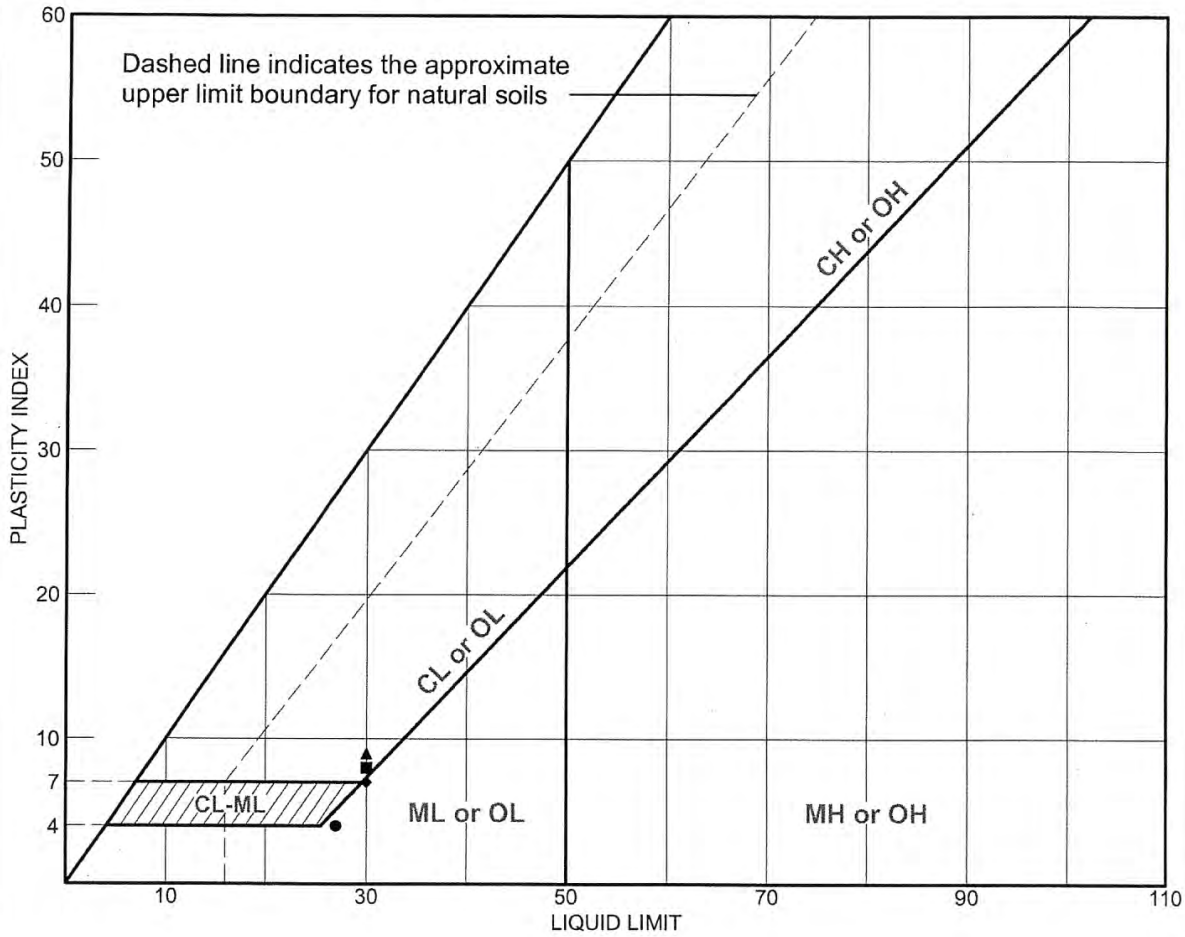
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	B-5	8200	45'	41.8	NP	NV	NP	SM
■	B-6	8200	20'	34.8	NP	NV	NP	ML
▲	B-6	8200	35'	32.3	NP	NV	NP	ML
◆	B-6	8200	15'	34.8	NP	NV	NP	SM
▼	B-7	8200	7.5'	33.0	NP	NV	NP	ML

LIQUID AND PLASTIC LIMITS TEST REPORT

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No.: 96156 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA

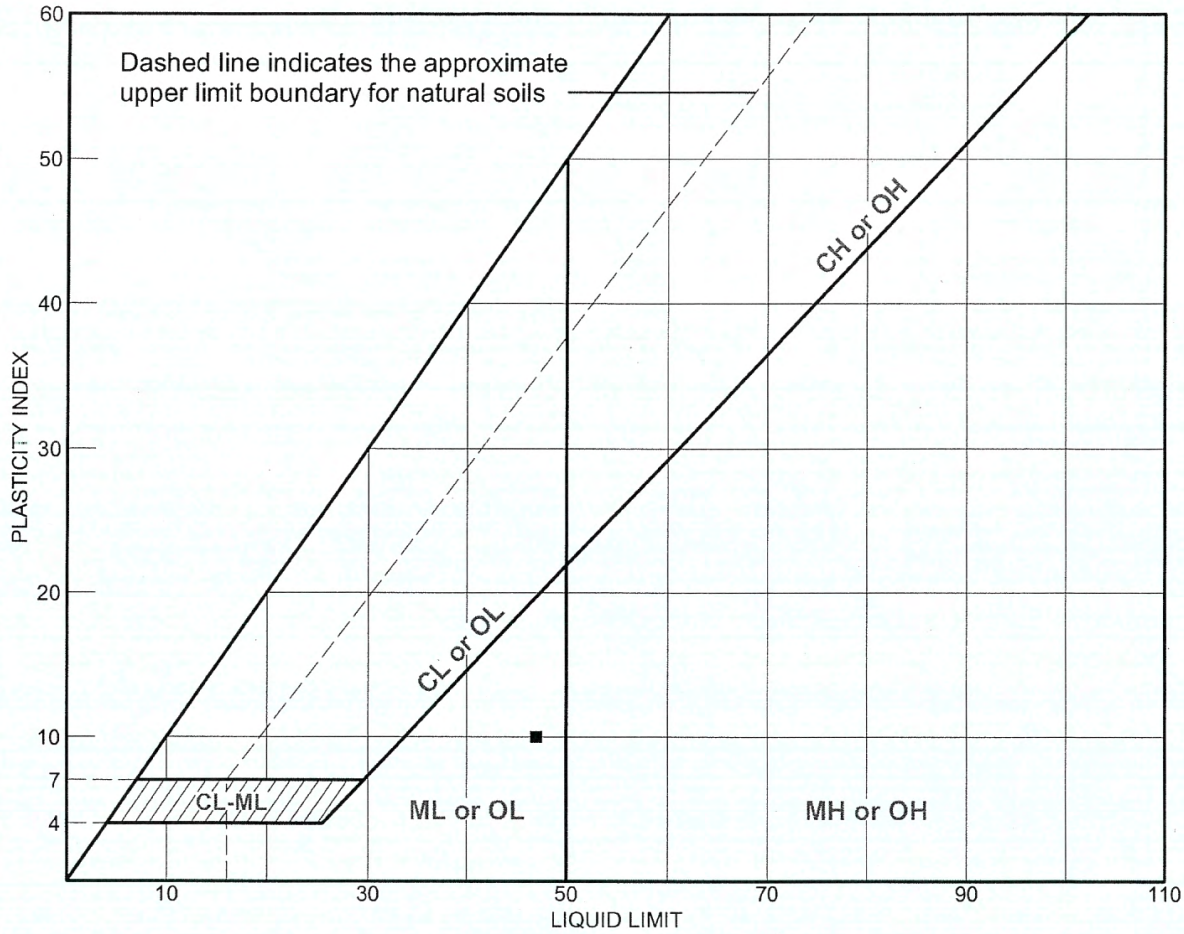
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	B-7	8200	15'	22.2	23	27	4	ML
■	B-8	8200	7.5'	24.6	22	30	8	CL
▲	B-8	8200	20'	37.1	21	30	9	CL
◆	B-8	8200	30'	19.3	23	30	7	ML
▼	B-9	8200	20'	16.5	NP	NV	NP	SM

LIQUID AND PLASTIC LIMITS TEST REPORT

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No.: 96156 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA

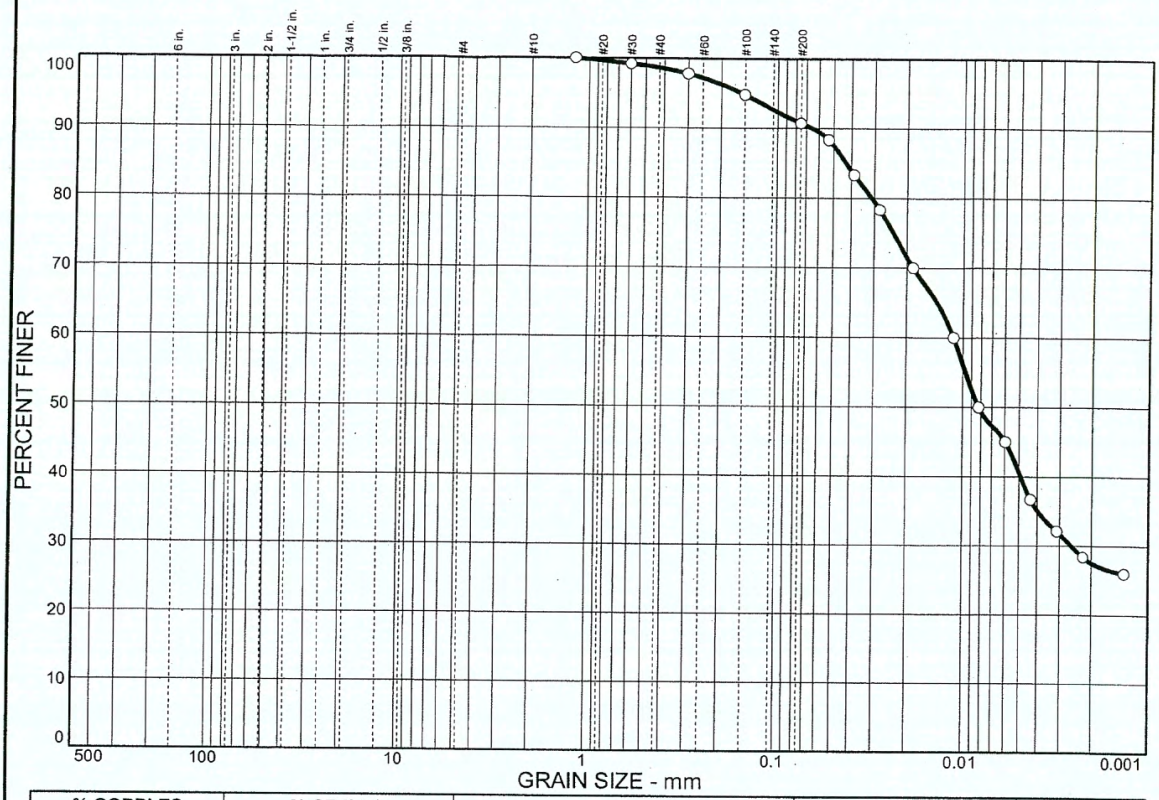
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	B-10	8200	15'	32.0	NP	NV	NP	ML
■	B-10	8200	20'	68.7	37	47	10	ML
▲	B-11	8200	10'	32	NP	NV	NP	ML
◆	B-11	8200	15'	31.5	NP	NV	NP	ML
▼	B-11	8200	30'	27.3	NP	NV	NP	SM

LIQUID AND PLASTIC LIMITS TEST REPORT

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No.: 96156 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	9.3	50.2	40.5

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#16	100.0		
#30	99.2		
#50	97.8		
#100	94.8		
#200	90.7		

* (no specification provided)

Soil Description

Silt
Lab Sample 8200C

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 0.0427 D_{60}= 0.0112 D_{50}= 0.0082
D_{30}= 0.0026 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

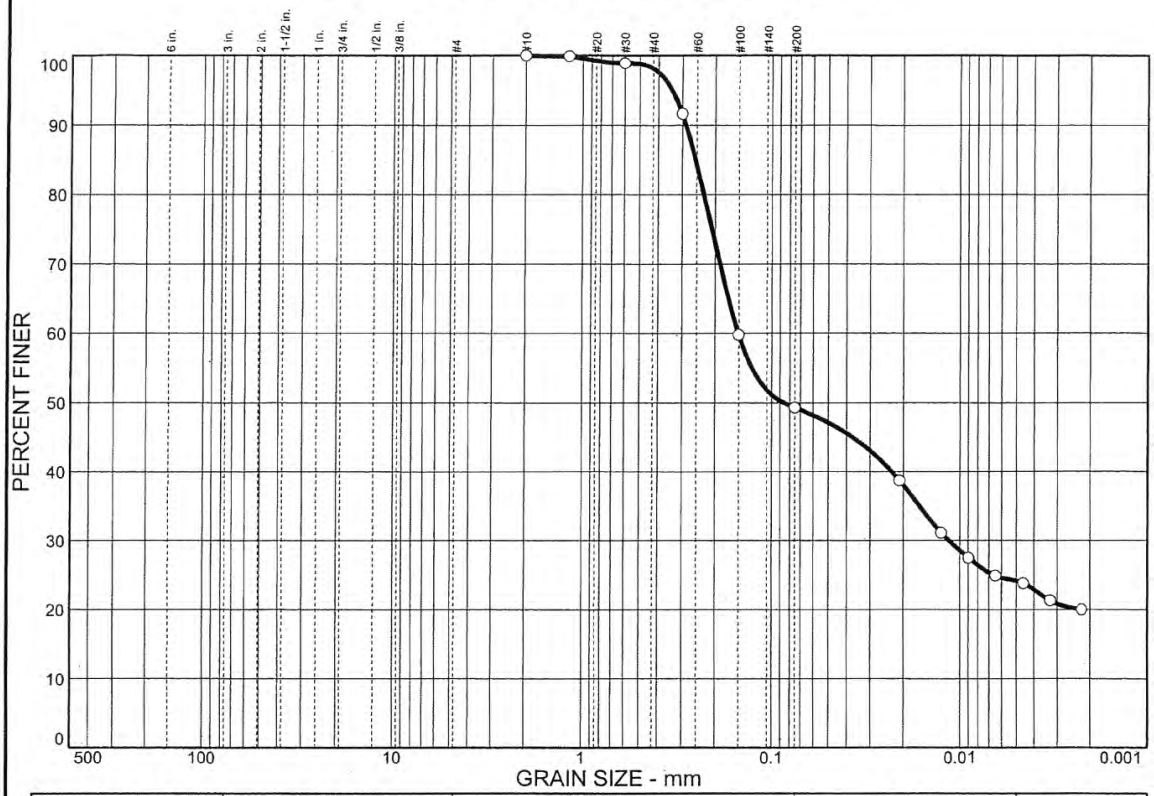
Sampled By: Jason Washburn
Tested By: Ryan Hogg
Checked By: John Schwartz

Sample No.: 8200 **Source of Sample:** B-1 **Date:** 9/8/08
Location: S-4 **Elev./Depth:** 10'

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No: 96156 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	50.7	25.1	24.2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	99.9		
#30	98.9		
#50	91.6		
#100	59.8		
#200	49.3		

* (no specification provided)

Soil Description

Lab Sample 8200E

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 0.255 D_{60}= 0.151 D_{50}= 0.0859
D_{30}= 0.0114 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sampled By Jason Washburn
Tested By Ryan Hogg
Checked By John Schwartz

Sample No.: 8200 **Source of Sample:** B-1 **Date:** 09/09/08
Location: S-6 **Elev./Depth:** 20'

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No: 96156 **Figure**

Particle Size Distribution Report

% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	10.9	43.6	45.5

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	99.6		
#30	96.9		
#50	93.7		
#100	91.0		
#200	89.1		

* (no specification provided)

Soil Description

Elastic silt
Lab Sample 8200J

Atterberg Limits

PL= 54 LL= 68 PI= 14

Coefficients

D_{85}= 0.0562 D_{60}= 0.0074 D_{50}= 0.0056
D_{30}= 0.0023 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= MH AASHTO=

Remarks

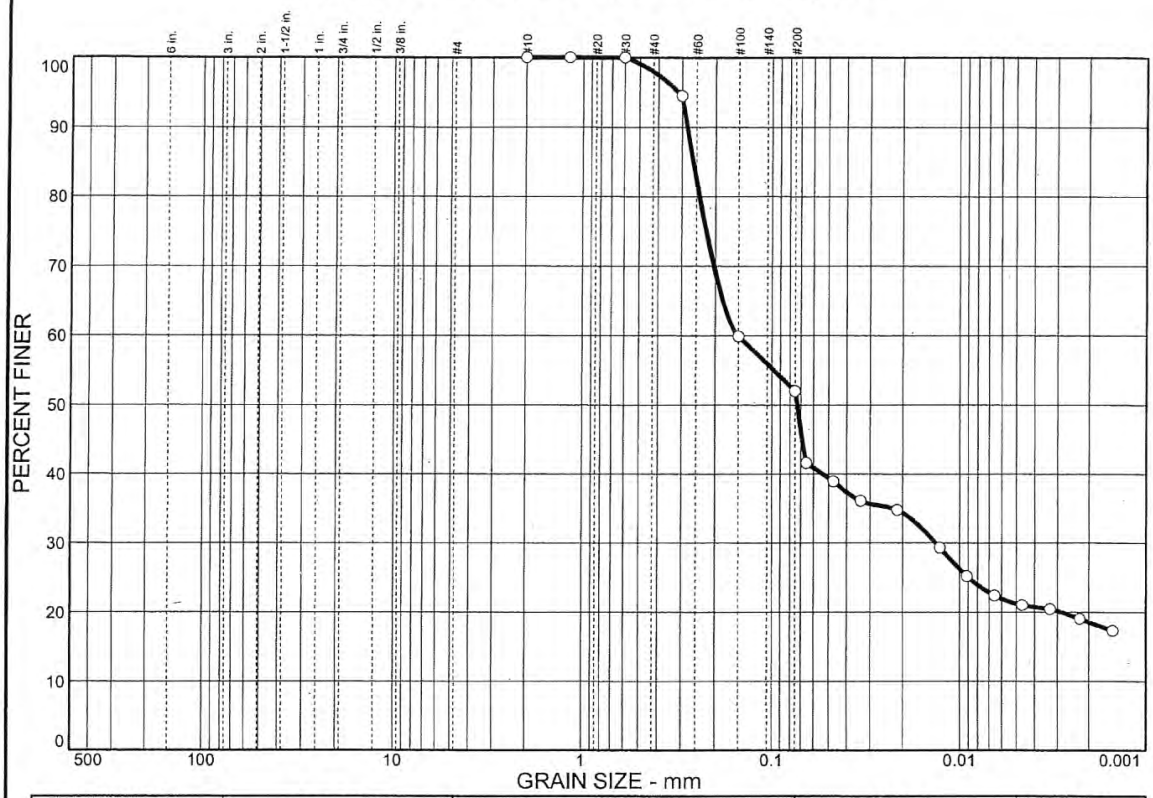
Sampled By Jason Washburn
Tested By Ryan Hogg
Checked By John Schwartz

Sample No.: 8200 **Source of Sample:** B-2 **Date:** 09/10/08
Location: S-3 **Elev./Depth:** 7.5'

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No: 96156 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	48.0	30.8	21.2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	100.0		
#30	100.0		
#50	94.5		
#100	59.9		
#200	52.0		

* (no specification provided)

Soil Description

Lab Sample 8200L Sandy silt

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 0.261 D_{60}= 0.151 D_{50}= 0.0733
D_{30}= 0.0134 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

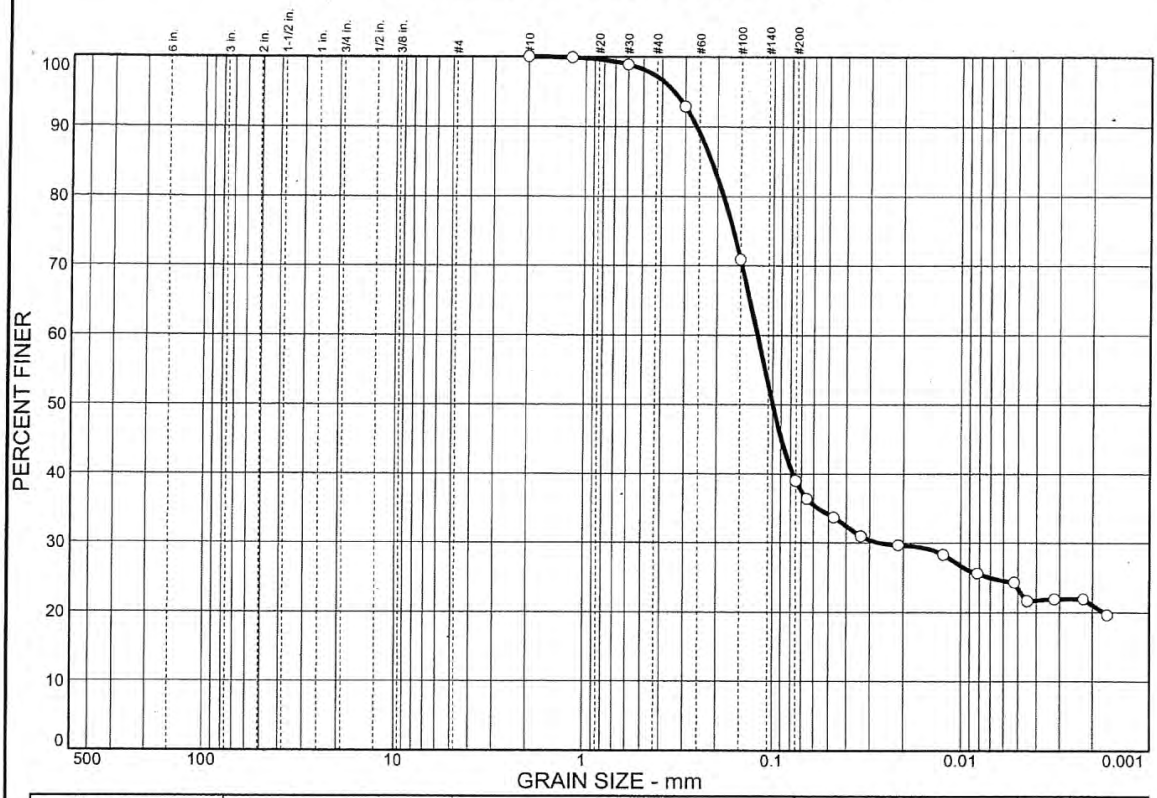
Sampled By: Jason Washburn
Tested By: Ryan Hogg
Checked By: John Schwartz

Sample No.: 8200 **Source of Sample:** B-2 **Date:** 9/8/08
Location: S-5 **Elev./Depth:** 15'

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No: 96156 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	61.0	15.6	23.4

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	99.9		
#30	98.9		
#50	92.9		
#100	70.9		
#200	39.0		

* (no specification provided)

Soil Description

Silty sand
Lab Sample 8200N

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 0.216 D_{60}= 0.121 D_{50}= 0.0998
D_{30}= 0.0263 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

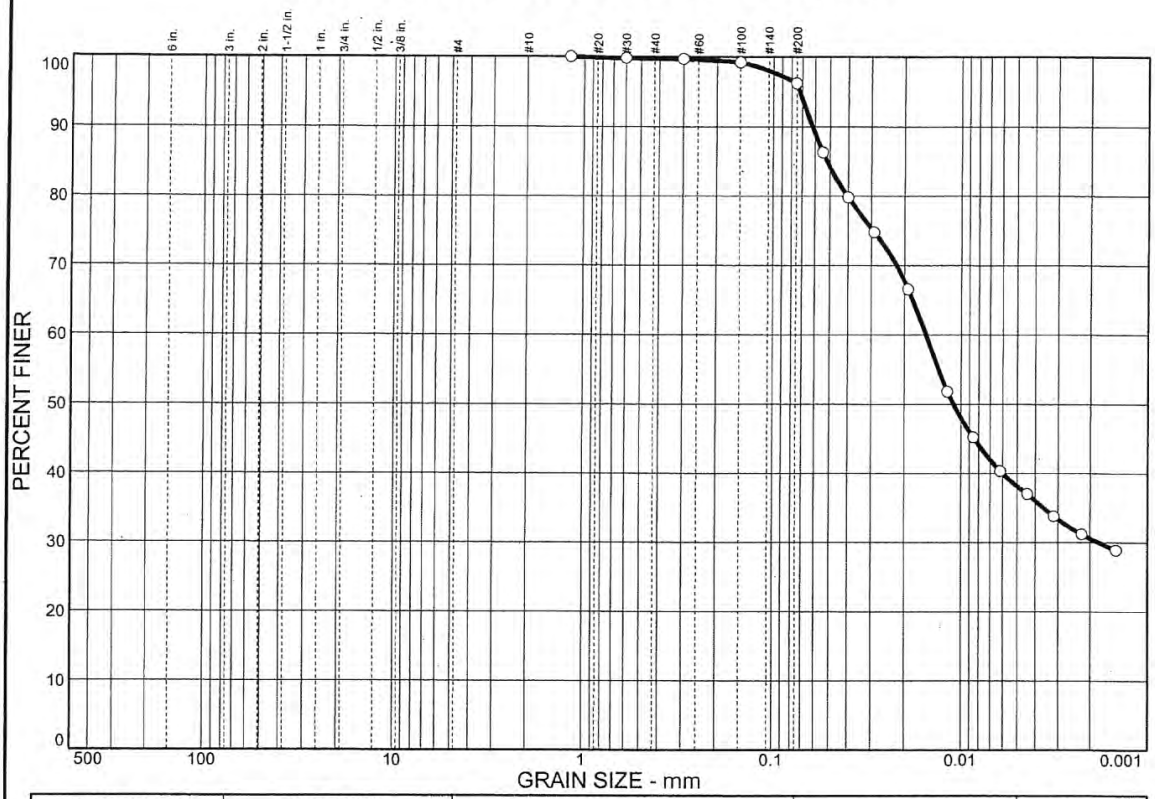
Sampled By: Jason Washburn
Tested By: Ryan Hogg
Checked By: John Schwartz

Sample No.: 8200 **Source of Sample:** B-2 **Date:** 9/8/08
Location: S-7 **Elev./Depth:** 25'

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No: 96156 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	3.8	58.1	38.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#16	100.0		
#30	99.8		
#50	99.7		
#100	99.2		
#200	96.2		

* (no specification provided)

Soil Description

Silt
Lab Sample 8200R

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 0.0517 D_{60}= 0.0154 D_{50}= 0.0110
D_{30}= 0.0018 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

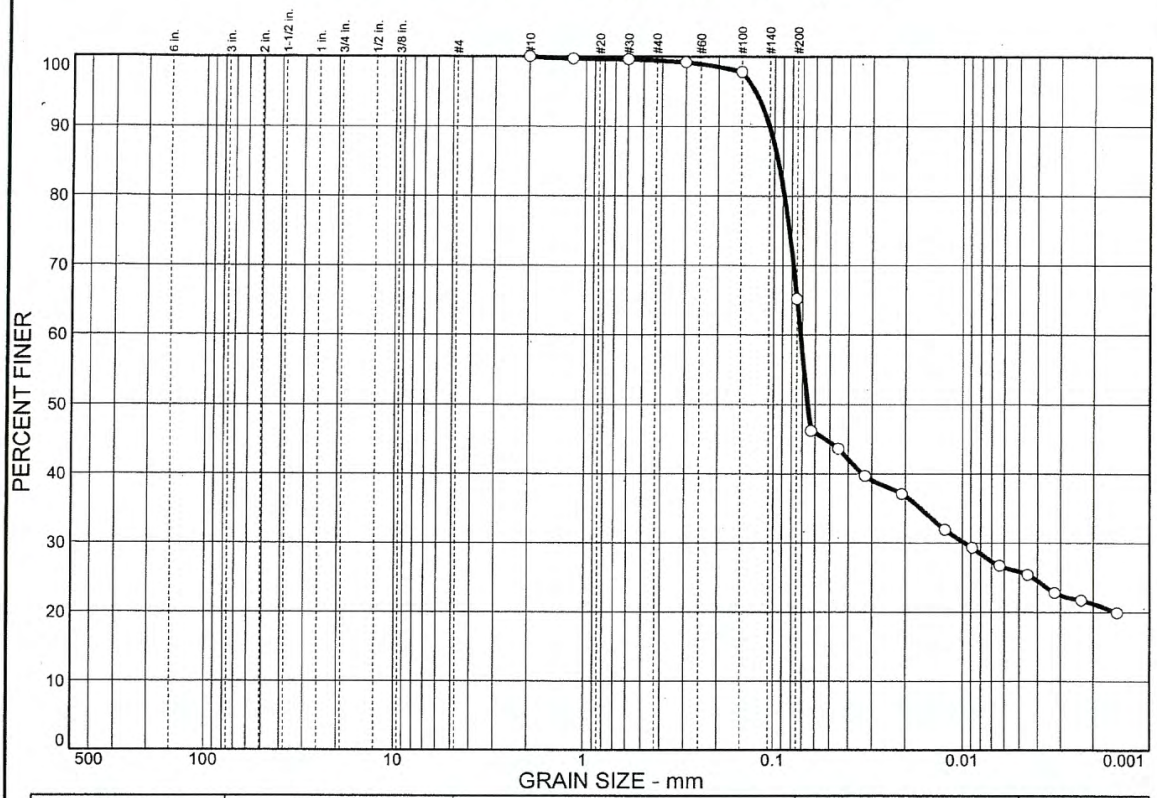
Sampled By: Jason Washburn
Tested By: Ryan Hogg
Checked By: John Schwartz

Sample No.: 8200 **Source of Sample:** B-3 **Date:** 9/8/08
Location: S-4 **Elev./Depth:** 10'

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No: 96156 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	34.8	39.3	25.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	99.7		
#30	99.6		
#50	99.2		
#100	97.8		
#200	65.2		

* (no specification provided)

Soil Description

Sandy silt
Lab Sample 8200U

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 0.0955 D_{60}= 0.0717 D_{50}= 0.0656
D_{30}= 0.0097 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

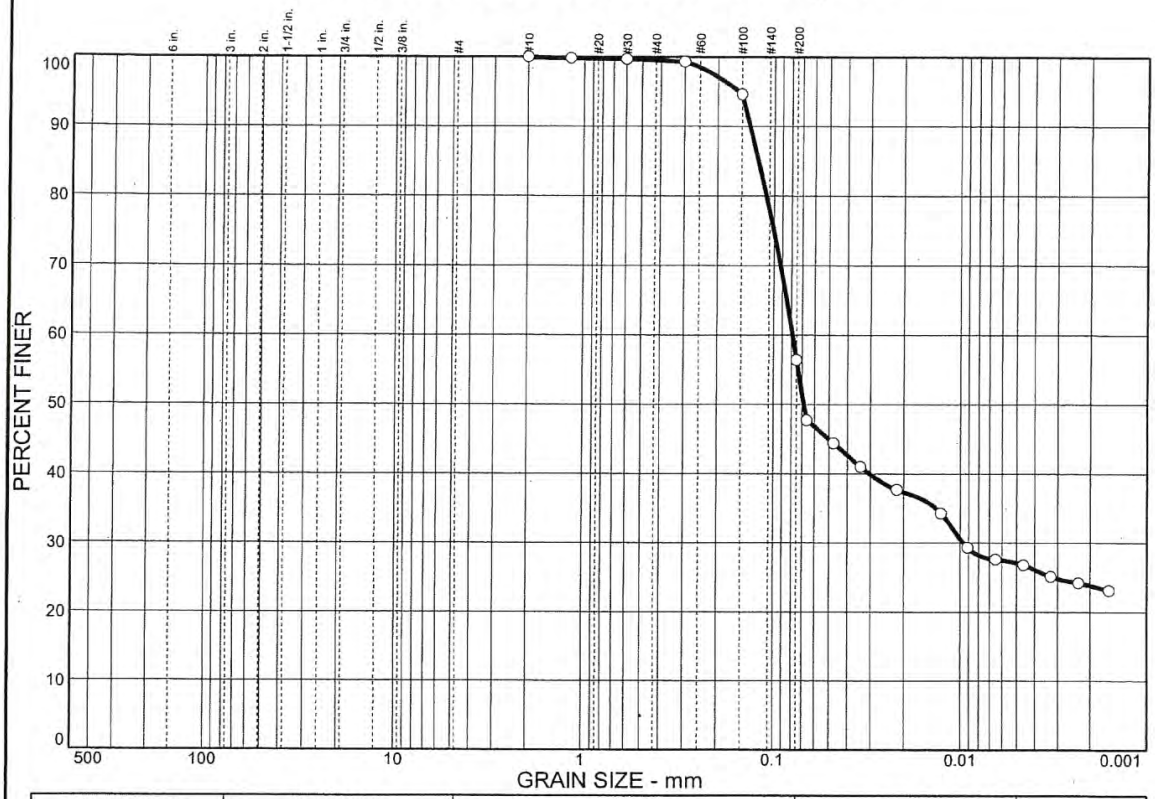
Sampled By: Jason Washburn
Tested By: Ryan Hogg
Cecked By: John Schwartz

Sample No.: 8200 **Source of Sample:** B-3 **Date:** 9/8/08
Location: S-7 **Elev./Depth:** 25'

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No: 96156 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	43.6	29.3	27.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	99.8		
#30	99.7		
#50	99.3		
#100	94.6		
#200	56.4		

* (no specification provided)

Soil Description

Sandy silt
Lab Sample 8200V

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 0.122 D_{60}= 0.0788 D_{50}= 0.0686
D_{30}= 0.0096 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

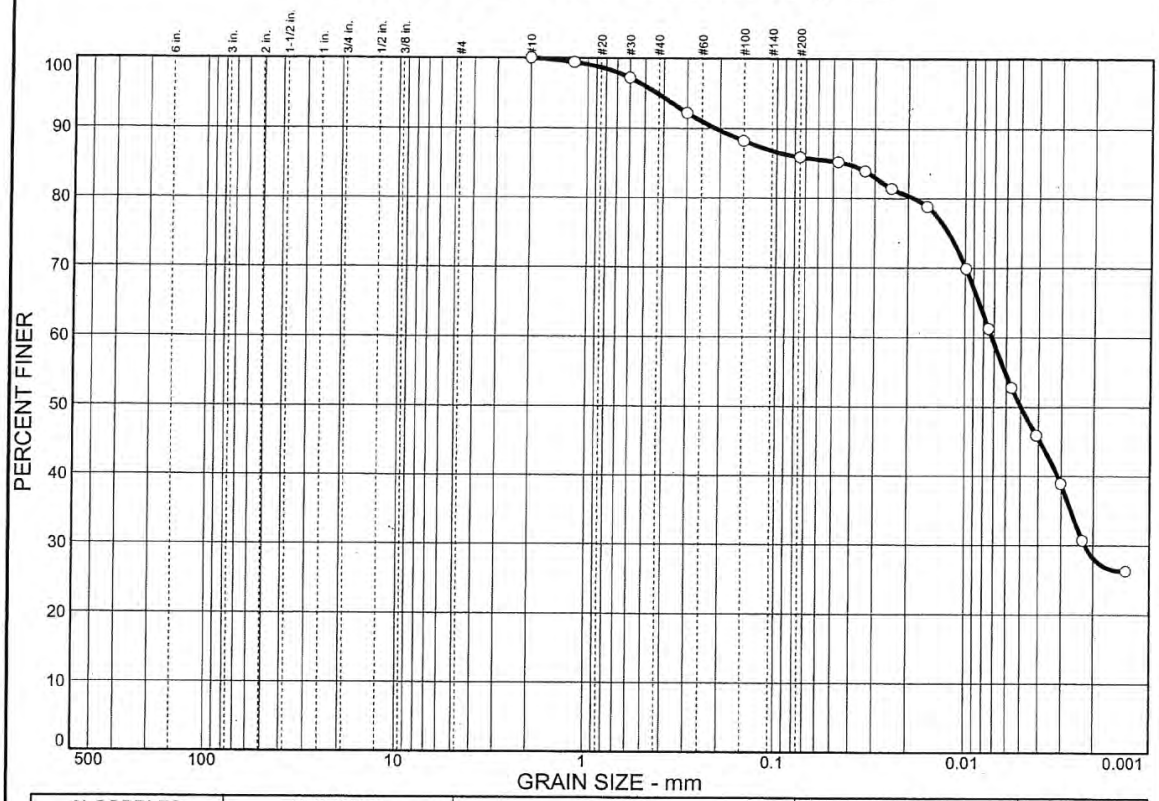
Sampled By: Jason Washburn
Tested By: Ryan Hogg
Checked By: John Schwartz

Sample No.: 8200 **Source of Sample:** B-3 **Date:** 9/8/08
Location: S-8 **Elev./Depth:** 30'

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No: 96156 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	14.1	35.9	50.0

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	99.4		
#30	97.2		
#50	92.2		
#100	88.2		
#200	85.9		

* (no specification provided)

Soil Description

Elastic silt
Lab Sample 8200Z

Atterberg Limits

PL= 52 LL= 70 PI= 18

Coefficients

D_{85}= 0.0432 D_{60}= 0.0072 D_{50}= 0.0050
D_{30}= 0.0022 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= MH AASHTO=

Remarks

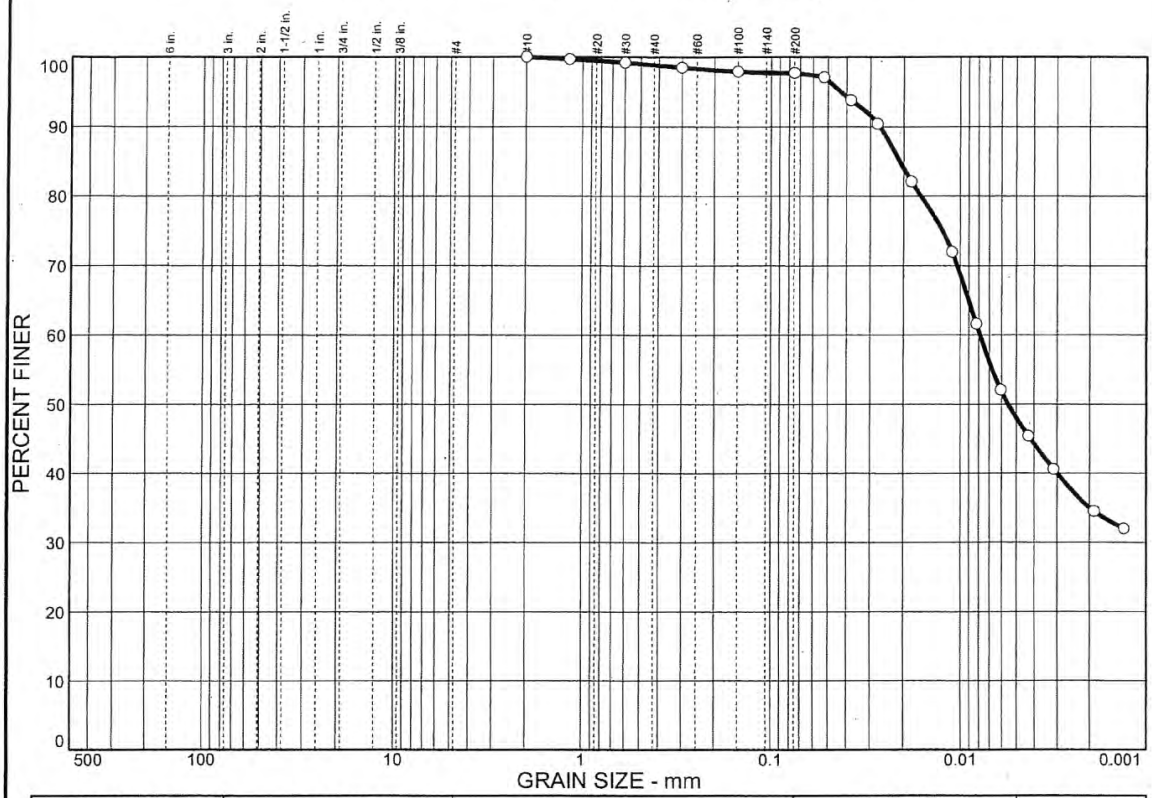
Sampled By: Jason Washburn
Tested By: Ryan Hogg
Checked By: John Schwartz

Sample No.: 8200 **Source of Sample:** B-4 **Date:** 9/8/08
Location: S-3 **Elev./Depth:** 7.5'

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No: 96156 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	2.3	49.7	48.0

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	99.7		
#30	99.2		
#50	98.5		
#100	97.9		
#200	97.7		

* (no specification provided)

Soil Description

Silt
Lab Sample 8200AA

Atterberg Limits

PL= 33 LL= 45 PI= 12

Coefficients

D_{85}= 0.0209 D_{60}= 0.0079 D_{50}= 0.0055
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

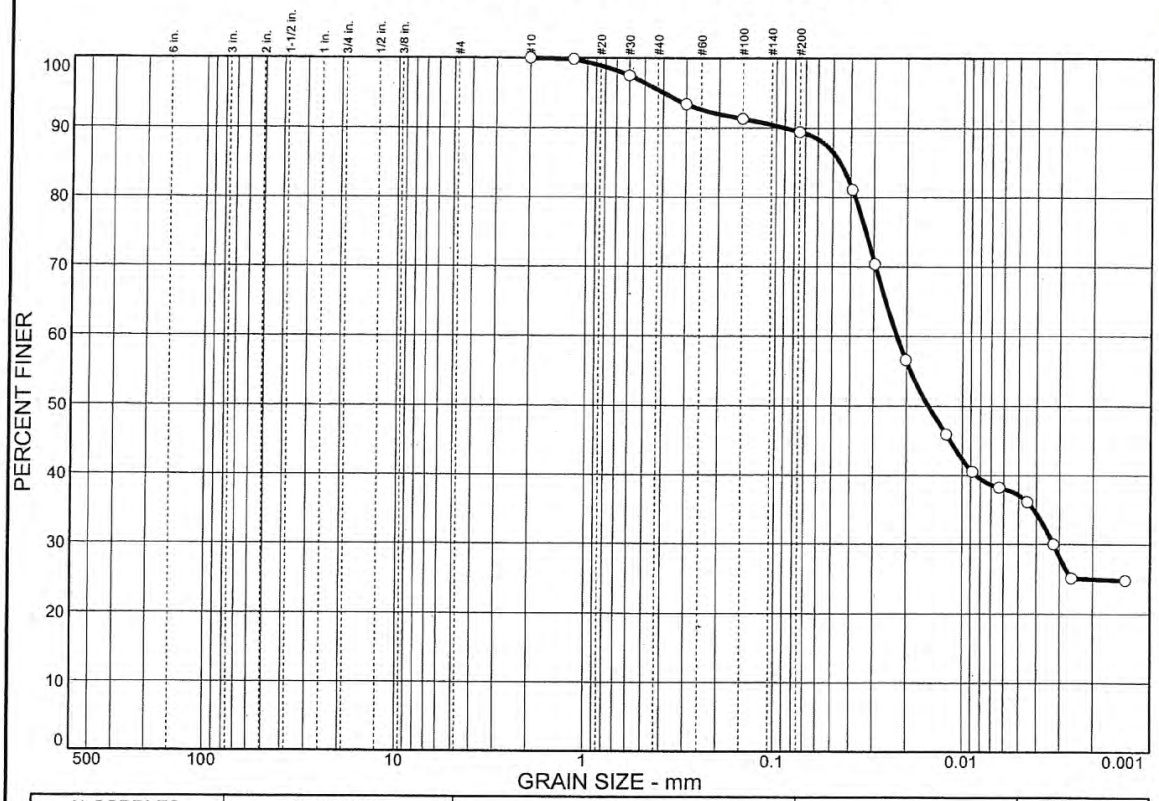
Sampled By Jason Washburn
Tested By Ryan Hogg
Checked By John Schwartz

Sample No.: 8200 **Source of Sample:** B-4 **Date:** 09/10/08
Location: S-4 **Elev./Depth:** 10'

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No: 96156 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	10.6	52.3	37.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	99.8		
#30	97.5		
#50	93.4		
#100	91.3		
#200	89.4		

* (no specification provided)

Soil Description

Silt
Lab Sample 8200AC

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 0.0458 D_{60}= 0.0224 D_{50}= 0.0152
D_{30}= 0.0032 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

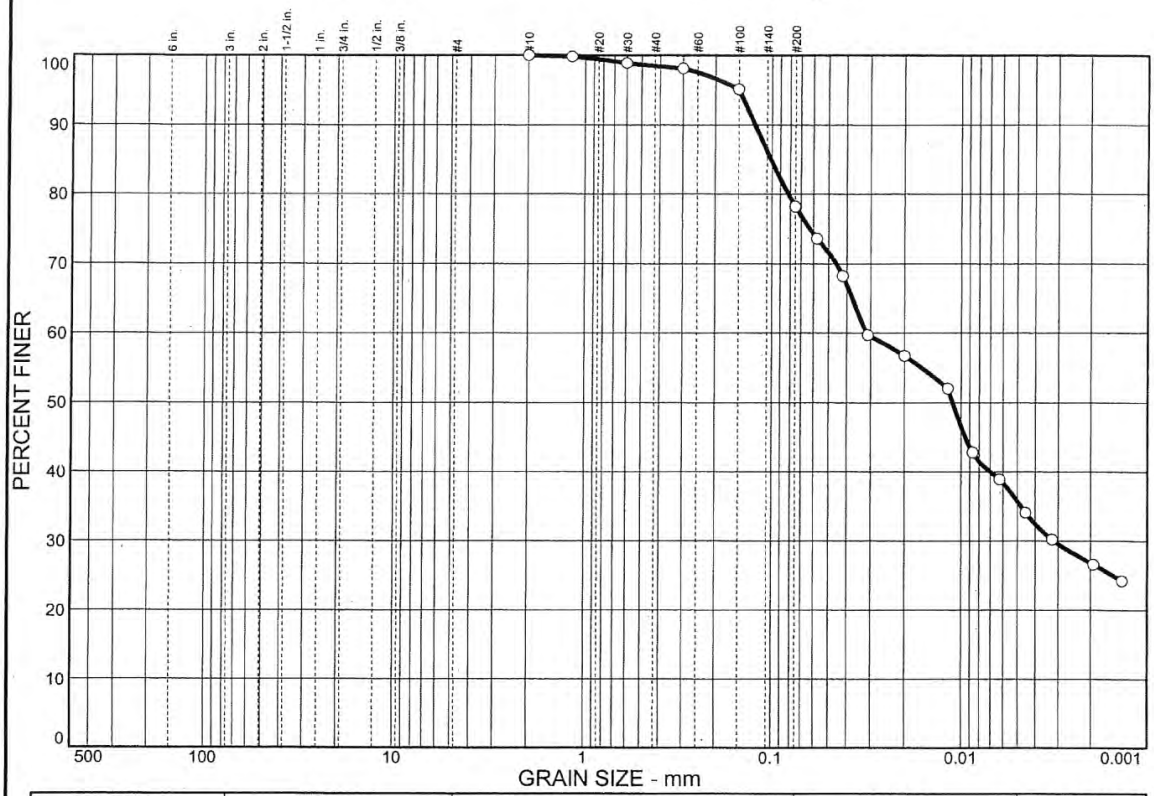
Sampled By: Jason Washburn
Tested By: Ryan Hogg
Checked By: John Schwartz

Sample No.: 8200 **Source of Sample:** B-4 **Date:** 9/8/08
Location: S-7 **Elev./Depth:** 25'

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No: 96156 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	21.8	42.5	35.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	99.8		
#30	98.9		
#50	98.1		
#100	95.1		
#200	78.2		

* (no specification provided)

Soil Description

Silt with sand
Lab Sample 8200AE

Atterberg Limits

PL= 33 LL= 36 PI= 3

Coefficients

D_{85}= 0.101 D_{60}= 0.0314 D_{50}= 0.0111
D_{30}= 0.0032 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

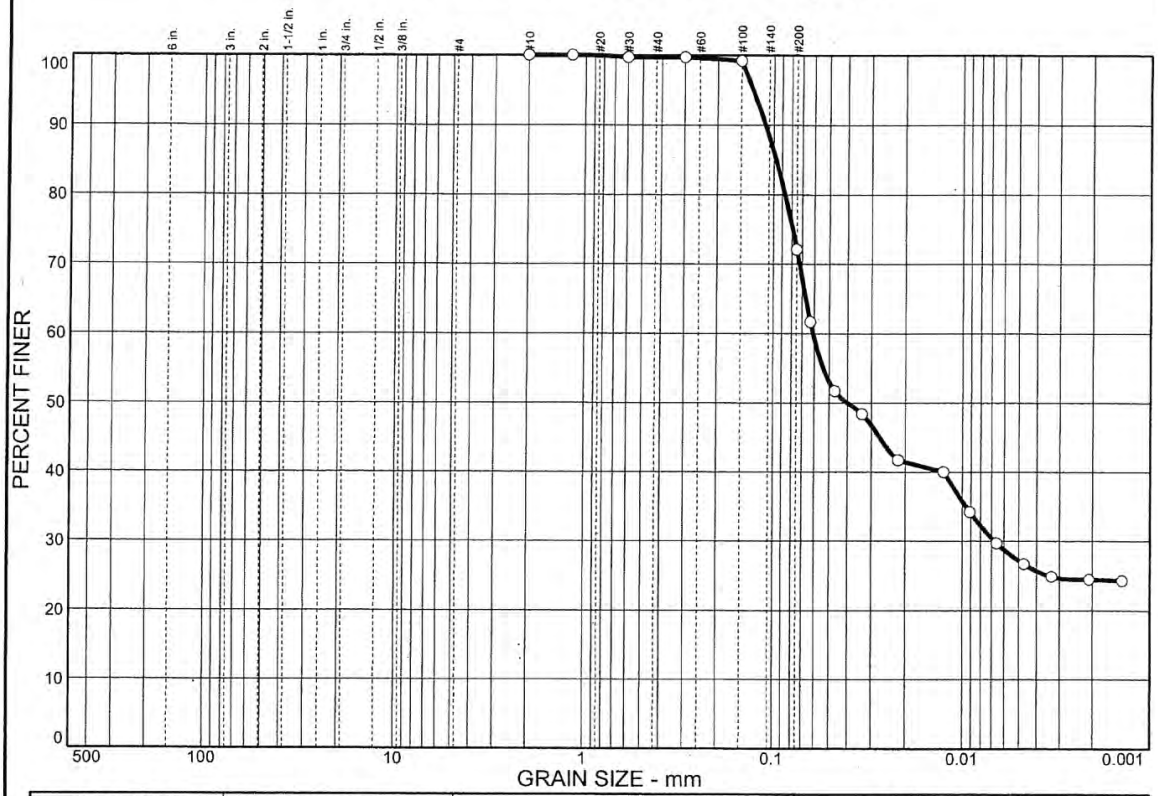
Sampled By: Jason Washburn
Tested By: Ryan Hogg
Checked By:John Schwartz

Sample No.: 8200 **Source of Sample:** B-5 **Date:** 9/8/08
Location: S-2 **Elev./Depth:** 5'

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No: 96156 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	27.9	44.9	27.2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	100.0		
#30	99.7		
#50	99.7		
#100	99.2		
#200	72.1		

* (no specification provided)

Soil Description

Silt with sand
Lab Sample 8200AI

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 0.0972 D_{60}= 0.0611 D_{50}= 0.0401
D_{30}= 0.0067 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

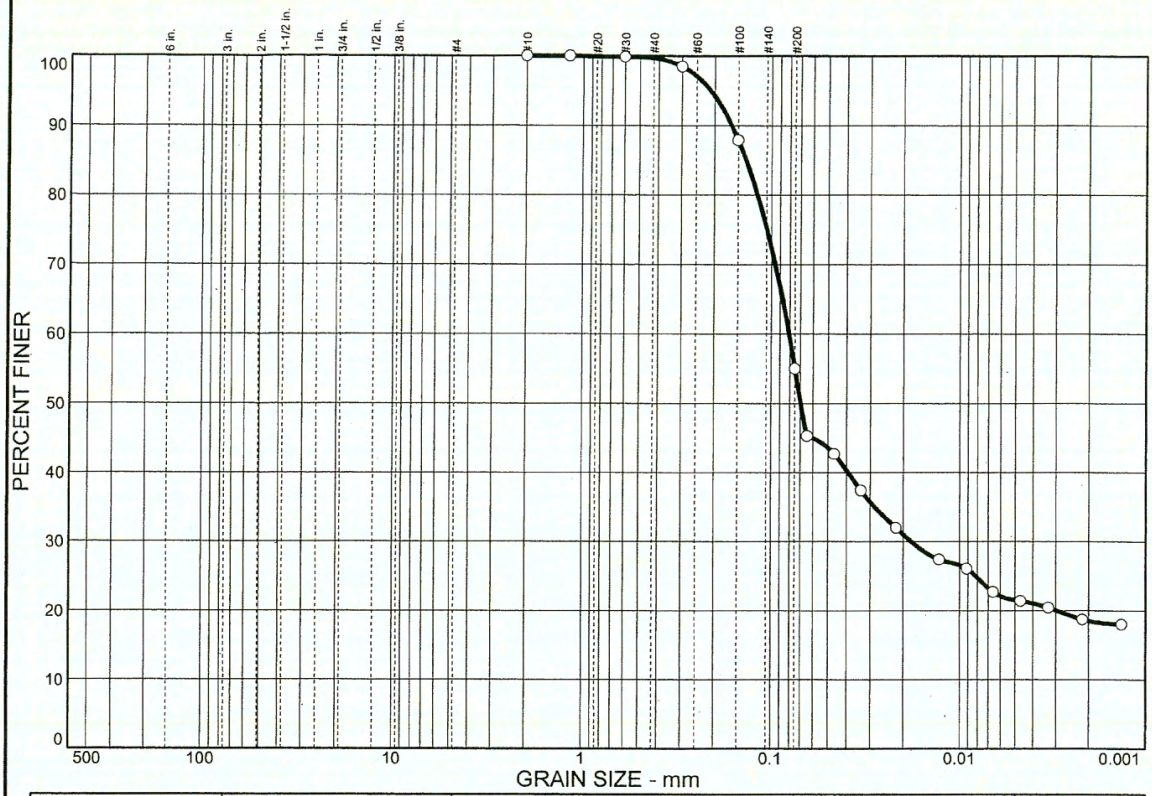
Sampled By: Jason Washburn
Tested By: Ryan Hogg
Checked By: John Schwartz

Sample No.: 8200 **Source of Sample:** B-5 **Date:** 9/8/08
Location: S-6 **Elev./Depth:** 20'

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No: 96156 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	45.0	33.5	21.5

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	100.0		
#30	99.9		
#50	98.5		
#100	87.9		
#200	55.0		

* (no specification provided)

Soil Description

Sandy silt
Lab Sample 8200AK

Atterberg Limits

PL= N/A LL= N/A PI= N/A

Coefficients

D_{85}= 0.136 D_{60}= 0.0807 D_{50}= 0.0699
D_{30}= 0.0181 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

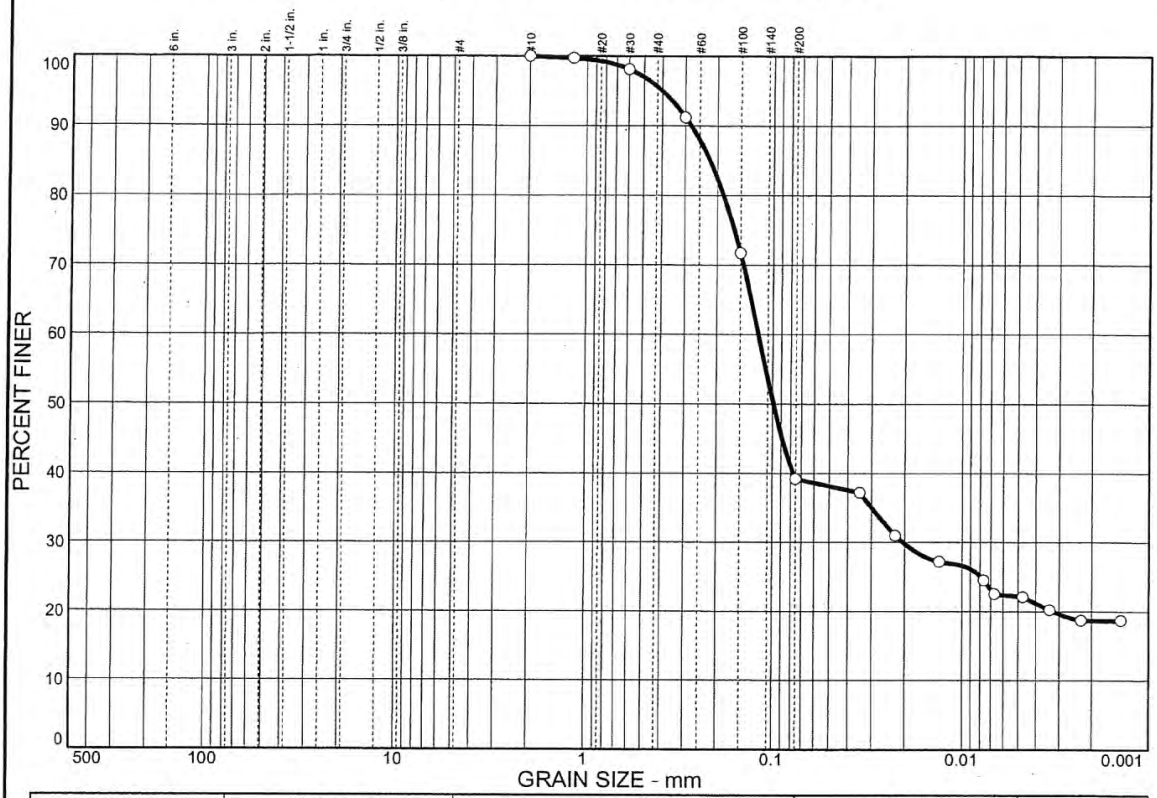
Sampled By: Jason Washburn
Tested By: Ryan Hogg
Checked By: John Schwartz

Sample No.: 8200 **Source of Sample:** B-5 **Date:** 9/8/08
Location: S-8 **Elev./Depth:** 30'

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No: 96156 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	60.8	17.0	22.2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	99.7		
#30	98.1		
#50	91.2		
#100	71.7		
#200	39.2		

* (no specification provided)

Soil Description

Silty sand
Lab Sample 8200AM

Atterberg Limits

PL= N/A LL= N/A PI= N/A

Coefficients

D_{85}= 0.220 D_{60}= 0.119 D_{50}= 0.0987
D_{30}= 0.0204 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

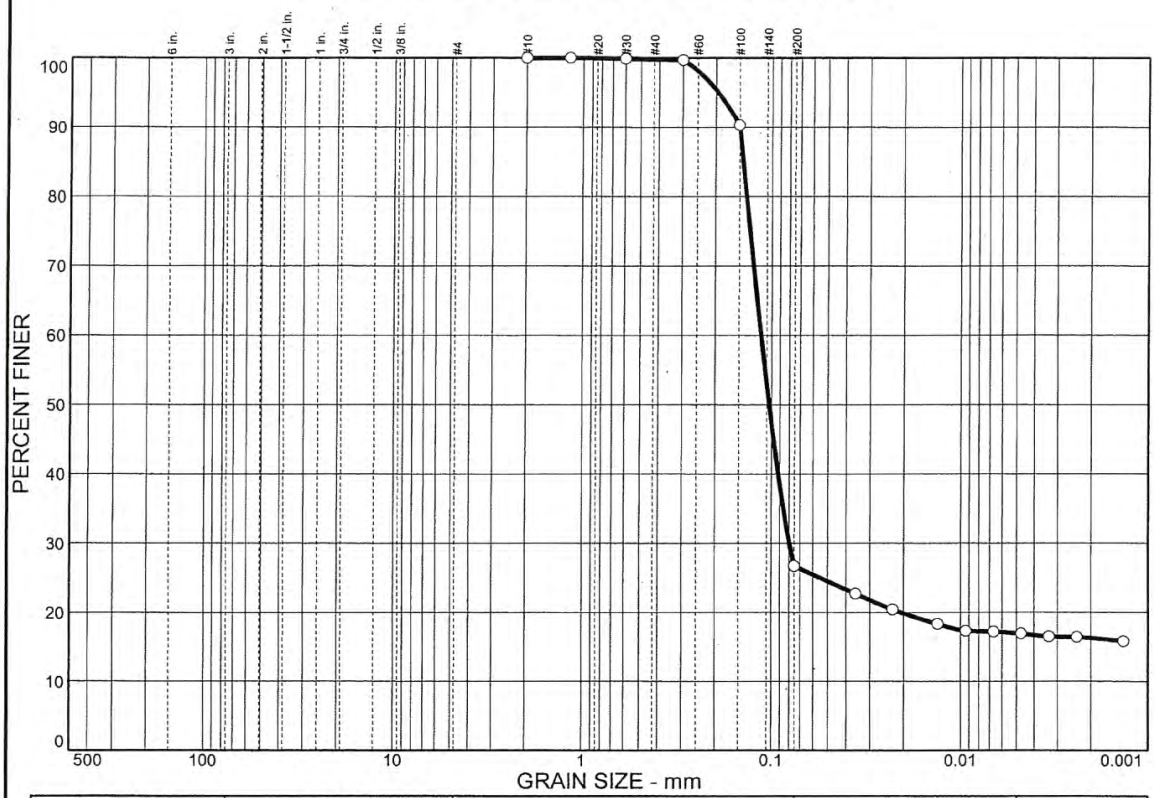
Sampled By: Jason Washburn
Tested By: Ryan Hogg
Checked By: John Schwartz

Sample No.: 8200 **Source of Sample:** B-5 **Date:** 9/8/08
Location: S-11 **Elev./Depth:** 45'

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No: 96156 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	73.3	9.7	17.0

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	100.0		
#30	99.9		
#50	99.7		
#100	90.3		
#200	26.7		

* (no specification provided)

Soil Description

Silty sand
Lab Sample 8200AS

Atterberg Limits

PL= N/A LL= N/A PI= N/A

Coefficients

D_{85}= 0.143 D_{60}= 0.114 D_{50}= 0.103
D_{30}= 0.0798 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

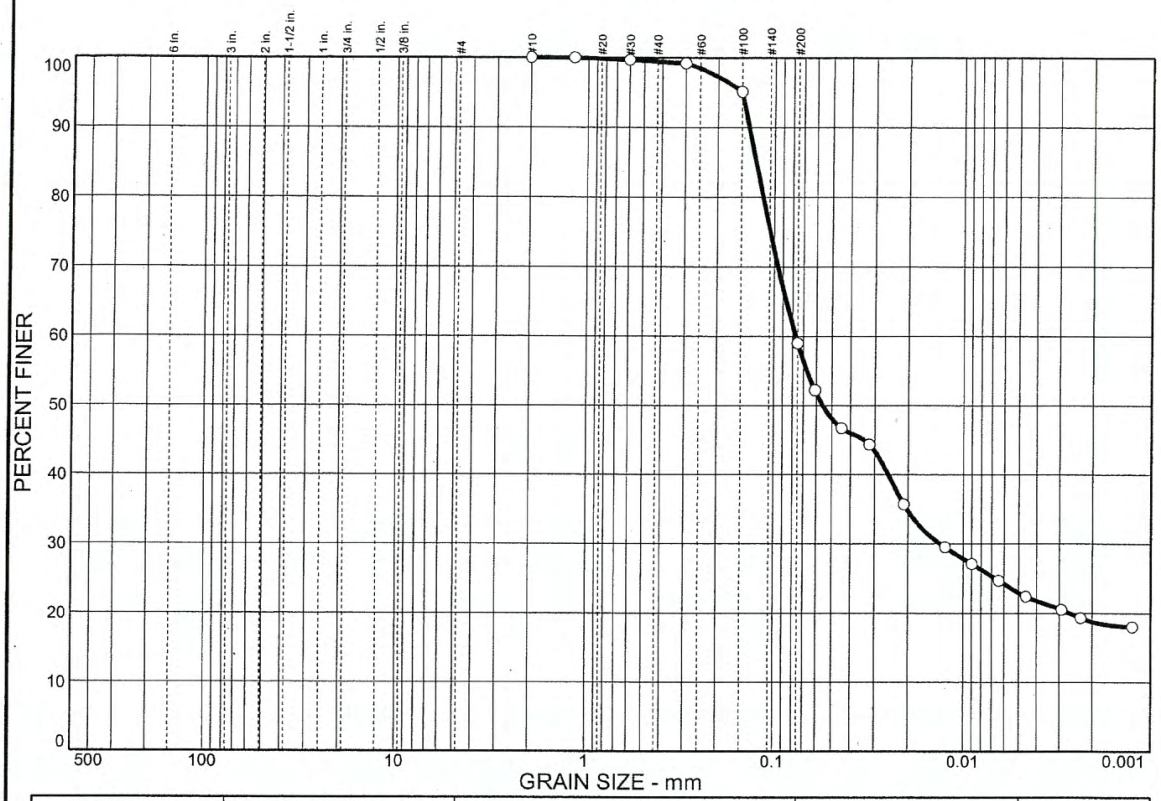
Sampled By: Jason Washburn
Tested By: Ryan Hogg
Checked By: John Schwartz

Sample No.: 8200 **Source of Sample:** B-6 **Date:** 9/8/08
Location: S-5 **Elev./Depth:** 15'

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No: 96156 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	41.0	36.1	22.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	100.0		
#30	99.7		
#50	99.2		
#100	95.1		
#200	59.0		

* (no specification provided)

Soil Description

Sandy silt
Lab Sample 8200AT

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 0.126 D_{60}= 0.0769 D_{50}= 0.0556
D_{30}= 0.0134 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

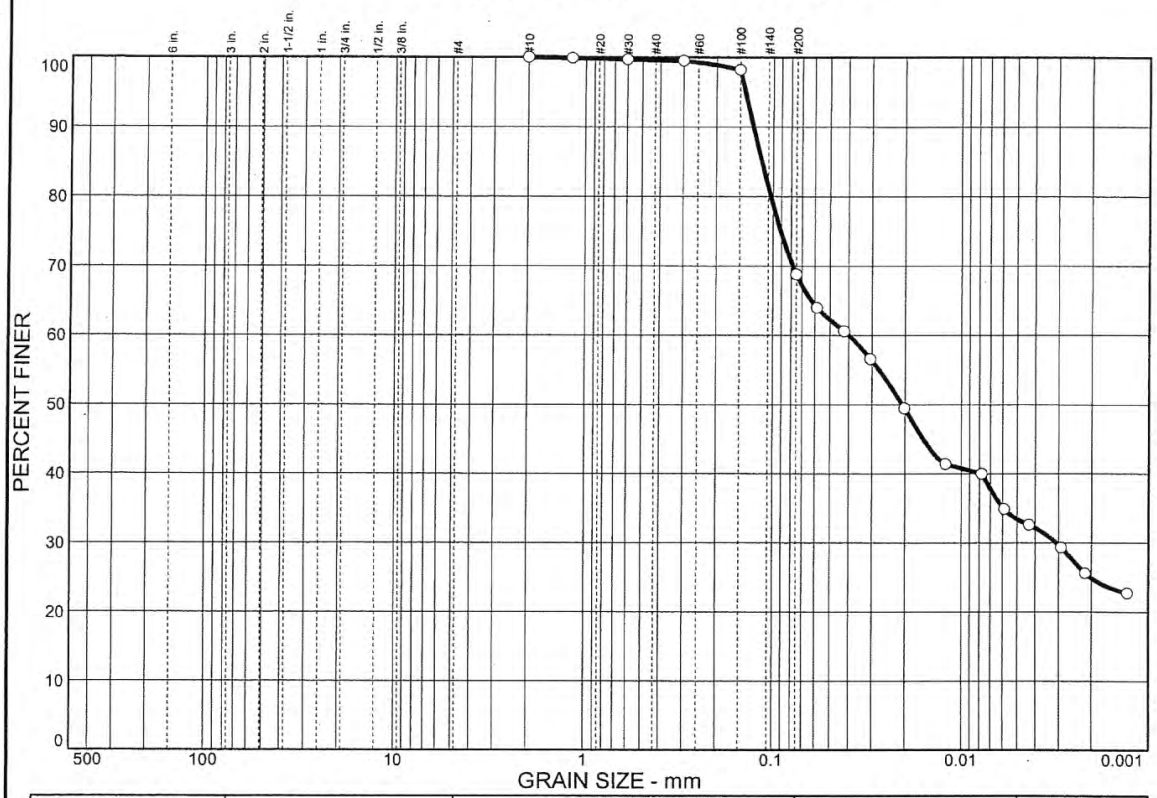
Sampled By: Jason Washburn
Tested By: Ryan Hogg
Checked By: John Schwartz

Sample No.: 8200 **Source of Sample:** B-6 **Date:** 9/8/08
Location: S-6 **Elev./Depth:** 20'

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No: 96156 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	31.2	35.4	33.4

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	99.9		
#30	99.7		
#50	99.5		
#100	98.2		
#200	68.8		

* (no specification provided)

Soil Description

Sandy silt
Lab Sample 8200AX

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 0.115 D_{60}= 0.0399 D_{50}= 0.0207
D_{30}= 0.0031 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

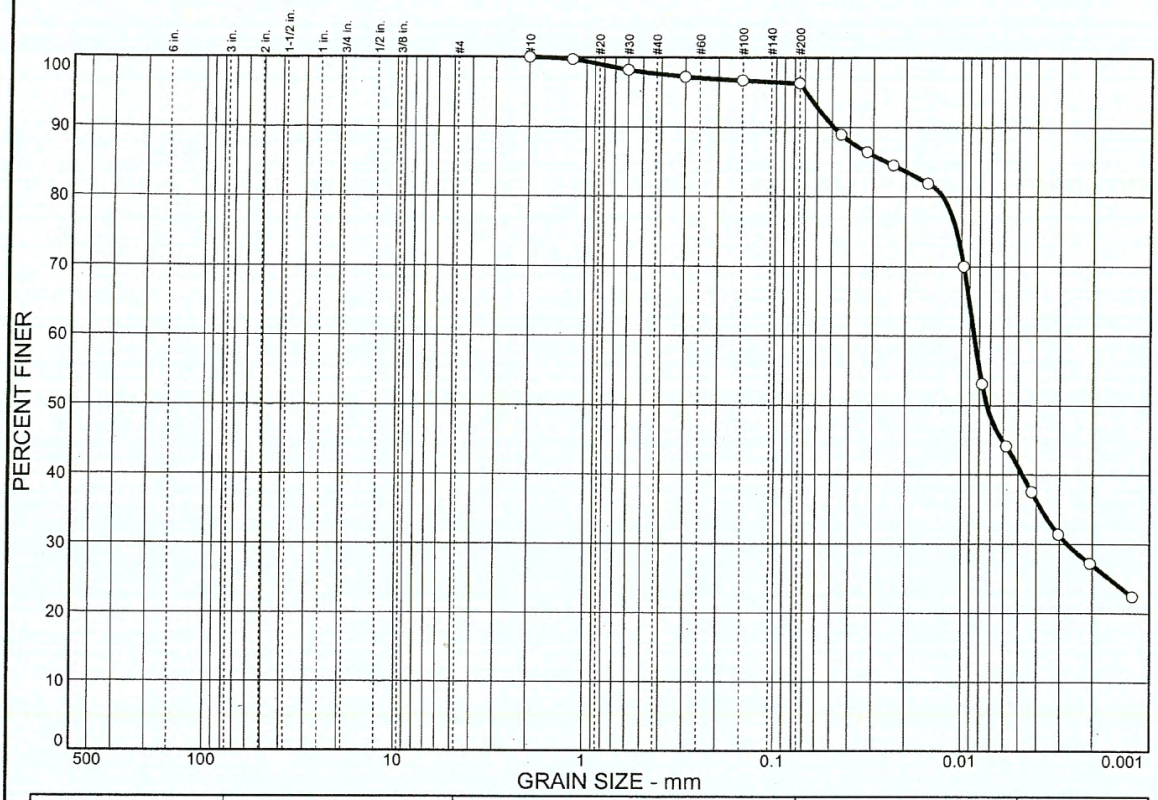
Sampled By Jason Washburn
Tested By Ryan Hogg
Checked By John Schwartz

Sample No.: 8200 **Source of Sample:** B-6 **Date:** 09/10/08
Location: S-9 **Elev./Depth:** 35'

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No: 96156 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	3.7	55.2	41.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	99.7		
#30	98.2		
#50	97.2		
#100	96.7		
#200	96.3		

* (no specification provided)

Soil Description

Silt
Lab Sample 8200BB

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 0.0255 D_{60}= 0.0085 D_{50}= 0.0073
D_{30}= 0.0027 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

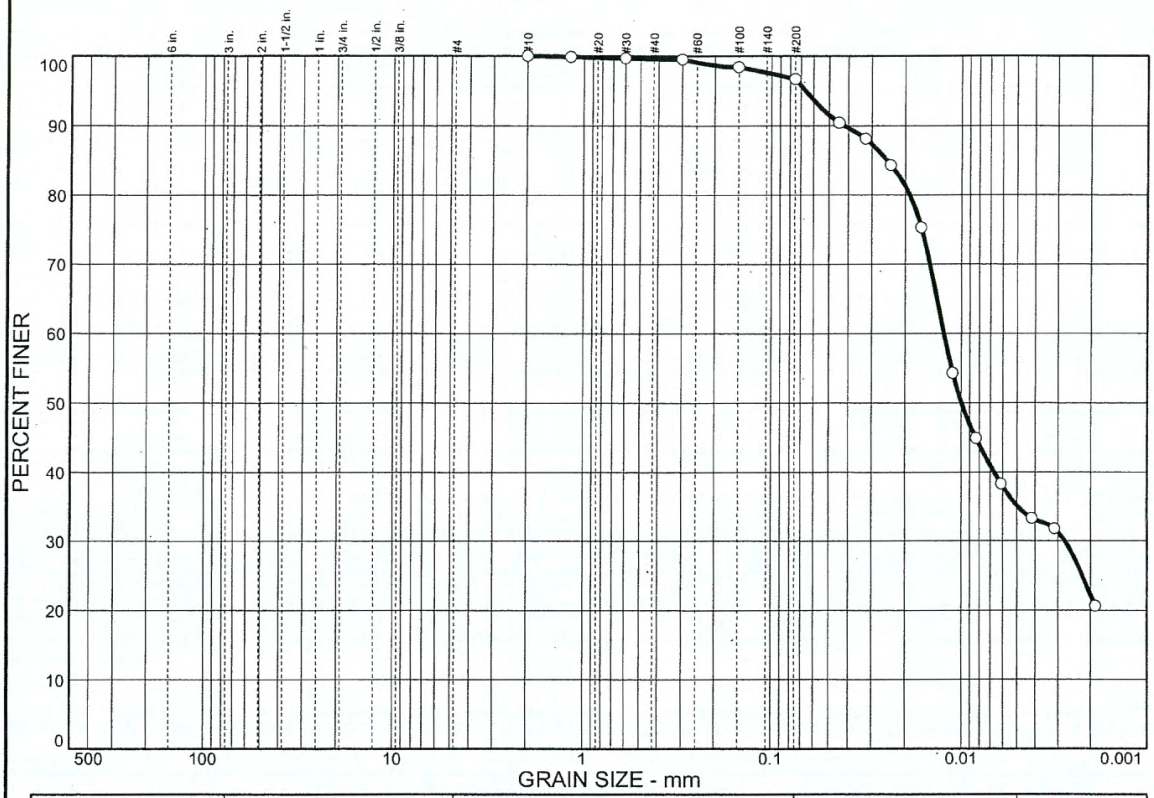
Sampled By: Jason Washburn
Tested By: Ryan Hogg
Checked By: John Schwartz

Sample No.: 8200 **Source of Sample:** B-7 **Date:** 9/8/08
Location: S-3 **Elev./Depth:** 7.5'

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No: 96156 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	3.3	61.6	35.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	99.9		
#30	99.7		
#50	99.5		
#100	98.4		
#200	96.7		

* (no specification provided)

Soil Description

Silt
Lab Sample 8200BC

Atterberg Limits

PL= 23 LL= 27 PI= 4

Coefficients

D_{85}= 0.0250 D_{60}= 0.0124 D_{50}= 0.0100
D_{30}= 0.0028 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

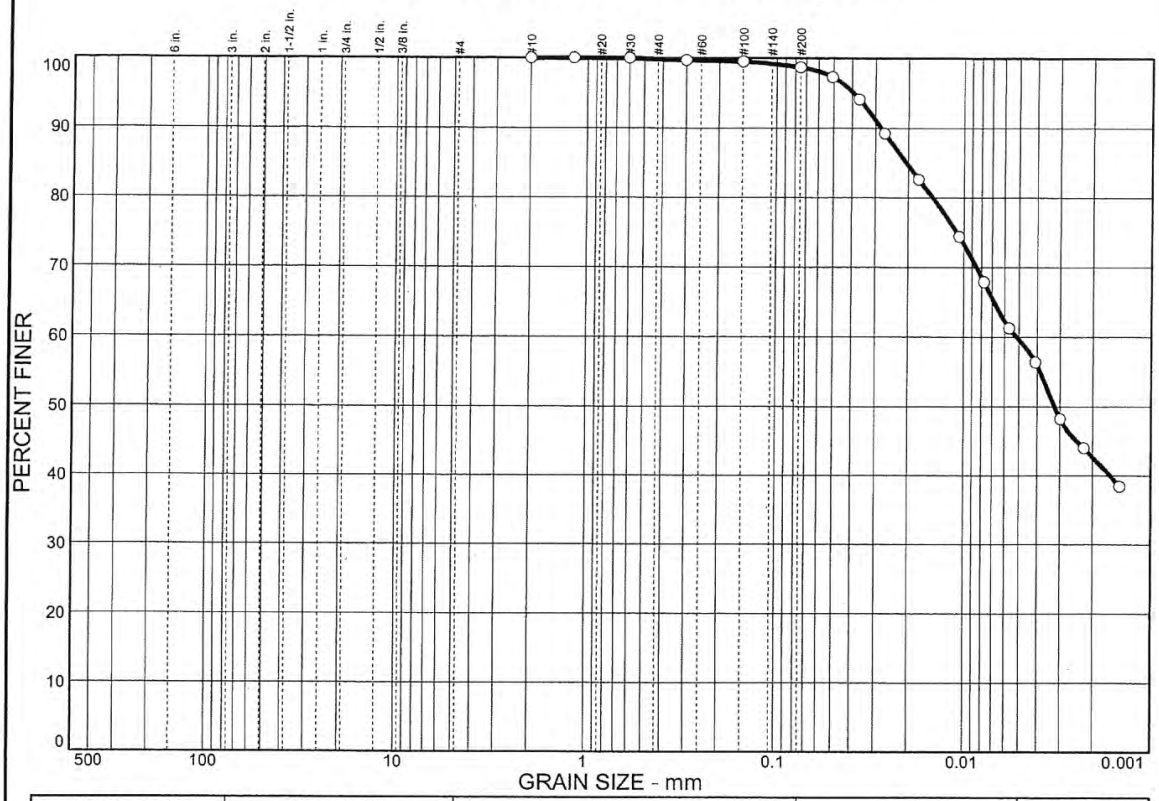
Sampled By Jason Washburn
Tested By Ryan Hogg
Checked By John Schwartz

Sample No.: 8200 **Source of Sample:** B-7 **Date:** 09/10/08
Location: S-5 **Elev./Depth:** 15'

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No: 96156 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	1.3	39.1	59.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	100.0		
#30	100.0		
#50	99.7		
#100	99.5		
#200	98.7		

* (no specification provided)

Soil Description

Lean clay
Lab Sample 8200BF

Atterberg Limits

PL= 22 LL= 30 PI= 8

Coefficients

D_{85}= 0.0205 D_{60}= 0.0052 D_{50}= 0.0032
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

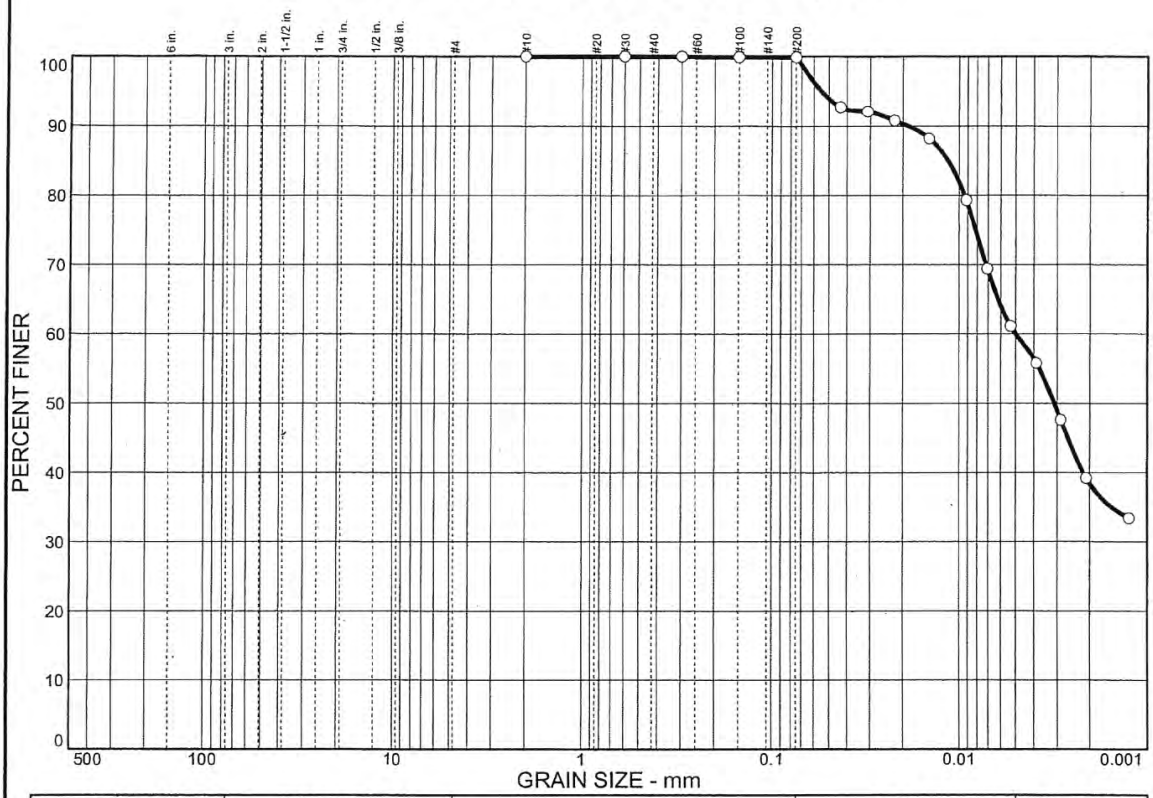
Sampled By: Jason Washburn
Tested By: Ryan Hogg
Checked By: John Schwartz

Sample No.: 8200 **Source of Sample:** B-8 **Date:** 9/8/08
Location: S-3 **Elev./Depth:** 7.5'

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No: 96156 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	0.1	39.9	60.0

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#30	100.0		
#50	100.0		
#100	99.9		
#200	99.9		

* (no specification provided)

Soil Description

Lean clay
Lab Sample 8200BI

Atterberg Limits

PL= 21 LL= 30 PI= 9

Coefficients

D_{85}= 0.0115 D_{60}= 0.0050 D_{50}= 0.0031
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

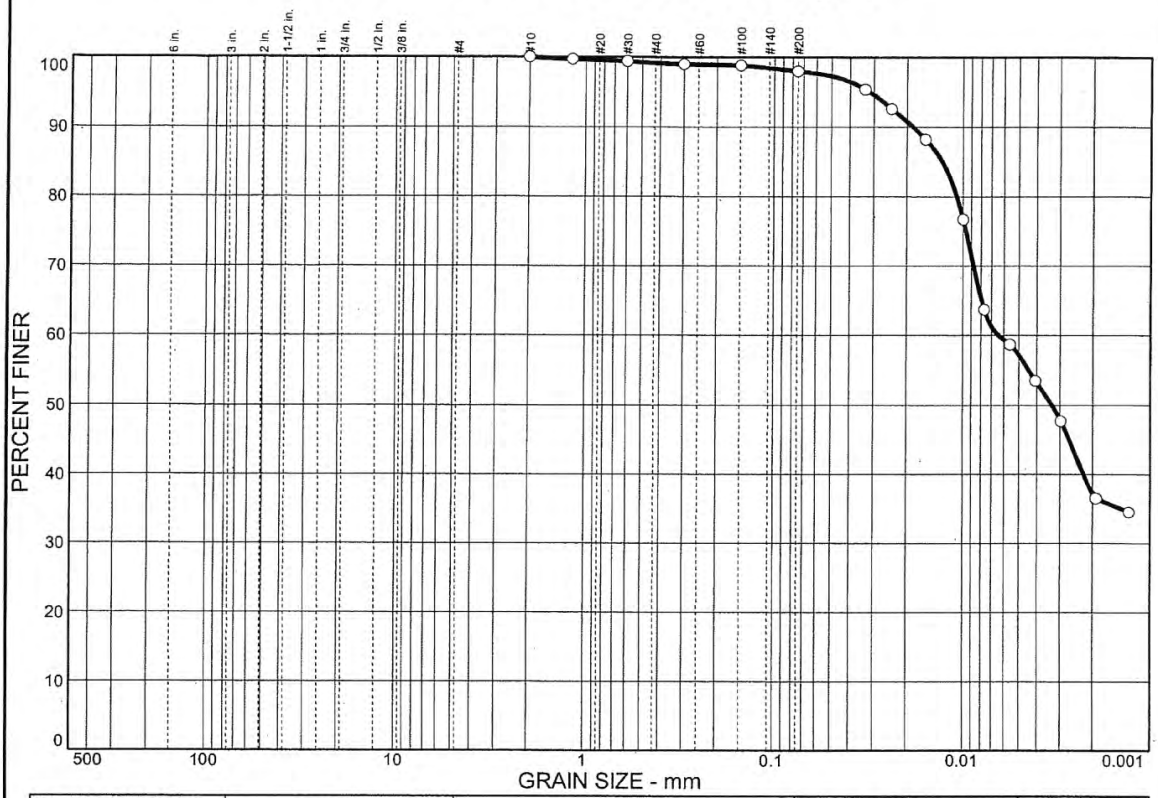
Sampled By: Jason Washburn
Tested By: Ryan Hogg
Checked By: John Schwartz

Sample No.: 8200 **Source of Sample:** B-8 **Date:** 9/8/08
Location: S-6 **Elev./Depth:** 20'

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No: 96156 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	2.0	40.8	57.2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	99.7		
#30	99.4		
#50	99.0		
#100	98.8		
#200	98.0		

* (no specification provided)

Soil Description

Silt
Lab Sample 8200BK

Atterberg Limits

PL= 23 LL= 30 PI= 7

Coefficients

D_{85}= 0.0129 D_{60}= 0.0064 D_{50}= 0.0033
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

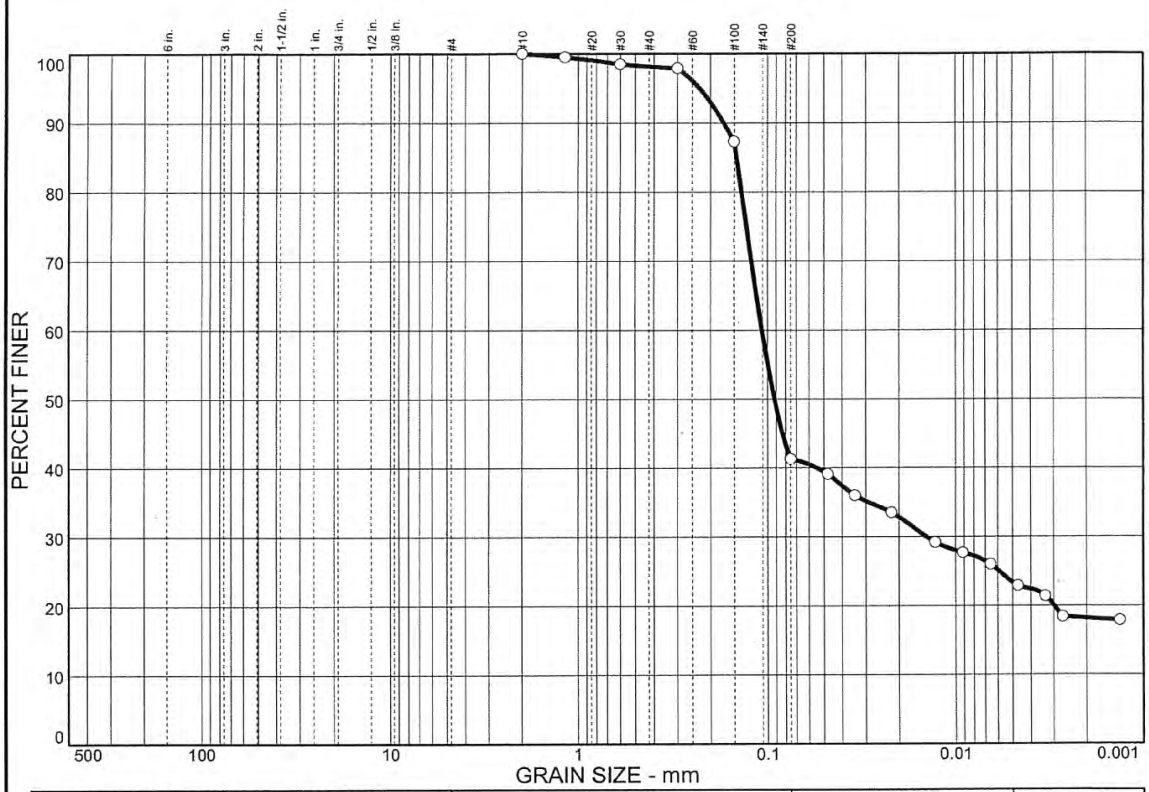
Sampled By: Jason Washburn
Tested By: Ryan Hogg
Checked By: John Schwartz

Sample No.: 8200 **Source of Sample:** B-8 **Date:** 9/8/08
Location: S-8 **Elev./Depth:** 30'

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No: 96156 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	58.7	17.9	23.4

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	99.5		
#30	98.5		
#50	97.9		
#100	87.3		
#200	41.3		

* (no specification provided)

Soil Description

Silty sand
Lab Sample 8200BN

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 0.146 D_{60}= 0.107 D_{50}= 0.0918
D_{30}= 0.0143 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

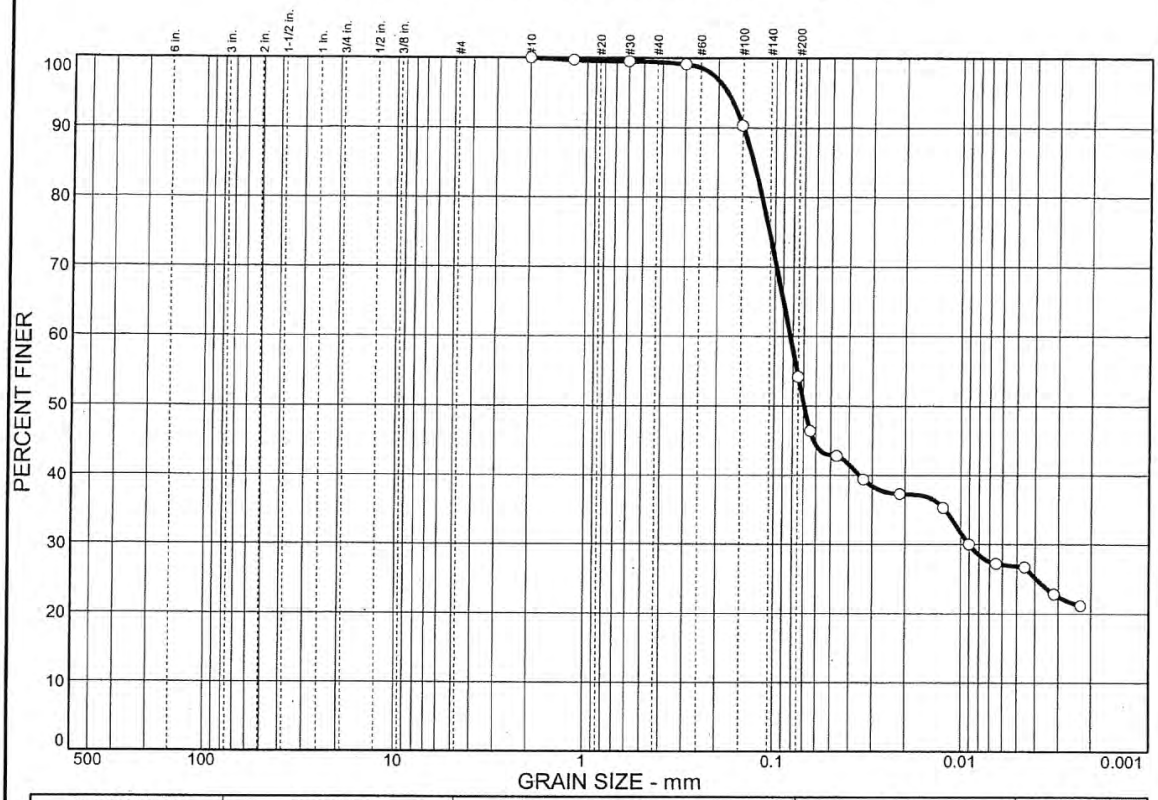
Sampled By: Jason Washburn
Tested By: Ryan Hogg
Checked By: John Schwartz

Sample No.: 8200 **Source of Sample:** B-9 **Date:** 9/8/08
Location: S-6 **Elev./Depth:** 20'

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No: 96156 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	45.8	27.4	26.8

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	99.7		
#30	99.5		
#50	99.1		
#100	90.3		
#200	54.2		

* (no specification provided)

Soil Description

Sandy silt
Laboratory Number 8200BO

Atterberg Limits

PL= N/A LL= N/A PI= N/A

Coefficients

D_{85}= 0.131 D_{60}= 0.0827 D_{50}= 0.0697
D_{30}= 0.0090 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

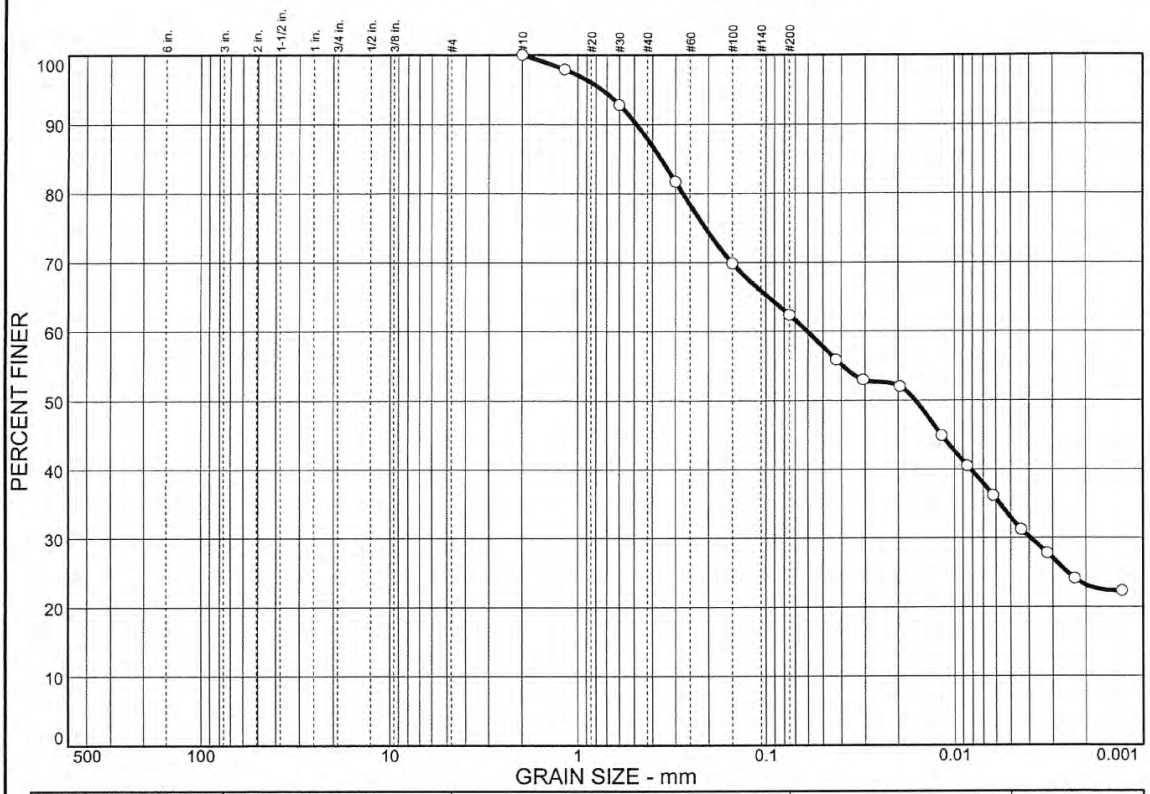
Sampled By Jason Washburn
Tested By Ryan Hogg
Checked By John Schwartz

Sample No.: 8200 **Source of Sample:** B-9 **Date:** 09/10/08
Location: S-8 **Elev./Depth:** 30'

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No: 96156 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	37.6	29.5	32.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	97.9		
#30	92.8		
#50	81.7		
#100	69.9		
#200	62.4		

* (no specification provided)

Soil Description

Sandy silt
Laboratory Mumber 8200BP

Atterberg Limits

PL= N/A LL= N/A PI= N/A

Coefficients

D_{85}= 0.360 D_{60}= 0.0605 D_{50}= 0.0162
D_{30}= 0.0040 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

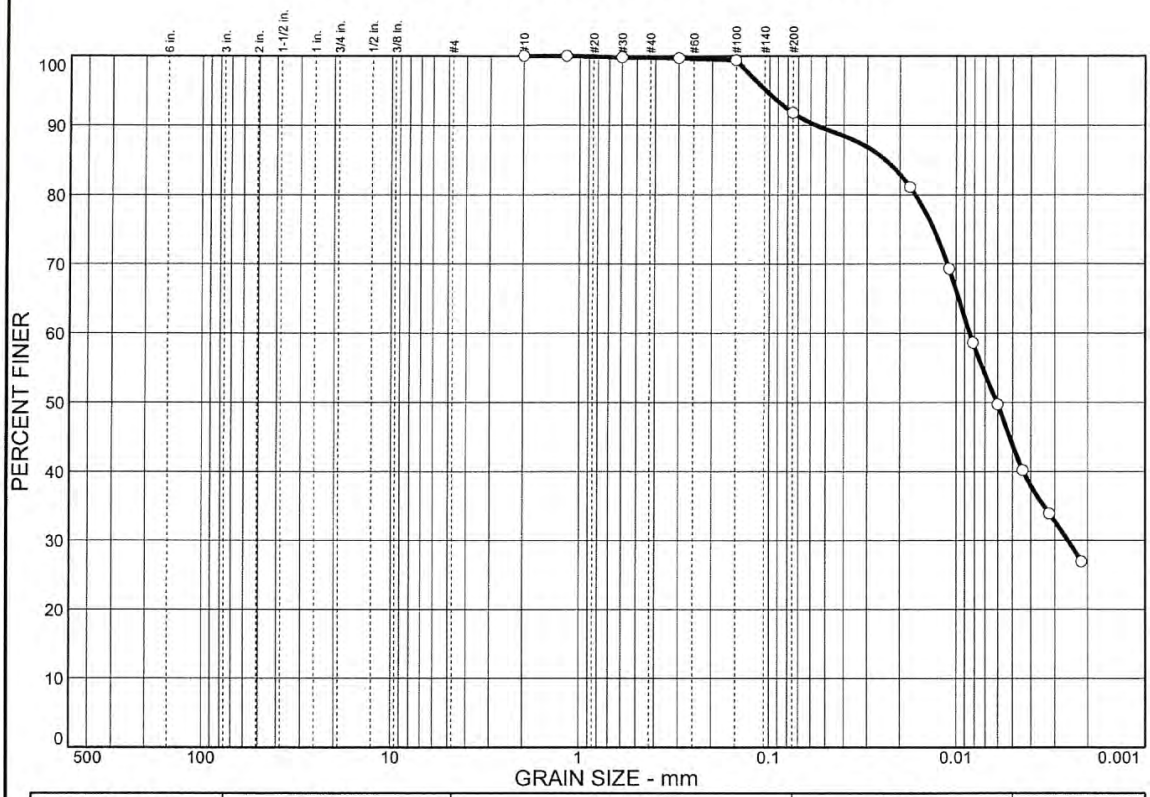
Sampled By Jason Washburn
Tested By Ryan Hogg
Checked By John Schwartz

Sample No.: 8200 **Source of Sample:** B-10 **Date:** 09/15/08
Location: S-3 **Elev./Depth:** 7.5

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No: 96156 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	8.2	47.9	43.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	100.0		
#30	99.8		
#50	99.7		
#100	99.4		
#200	91.8		

* (no specification provided)

Soil Description

Silt
Lab Sample 8200BR

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 0.0239 D_{60}= 0.0085 D_{50}= 0.0061
D_{30}= 0.0026 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

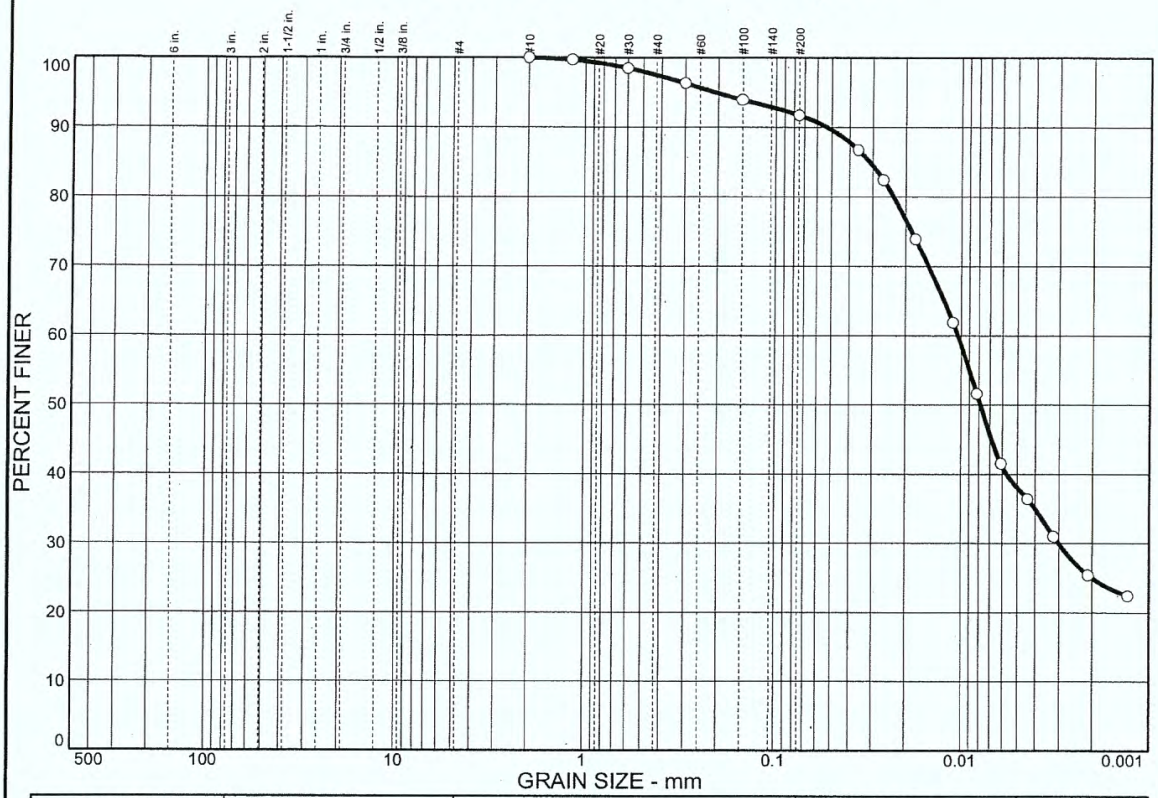
Sampled By: Jason Washburn
Tested By: Ryan Hogg
Checked By: John Schwartz

Sample No.: 8200 **Source of Sample:** B-10 **Date:** 9/8/08
Location: S-5 **Elev./Depth:** 15'

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No: 96156 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	8.3	53.7	38.0

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	99.7		
#30	98.5		
#50	96.3		
#100	93.9		
#200	91.7		

* (no specification provided)

Soil Description

Silt
Lab Sample 8200BS

Atterberg Limits

PL= 37 LL= 47 PI= 10

Coefficients

D_{85}= 0.0312 D_{60}= 0.0104 D_{50}= 0.0079
D_{30}= 0.0030 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

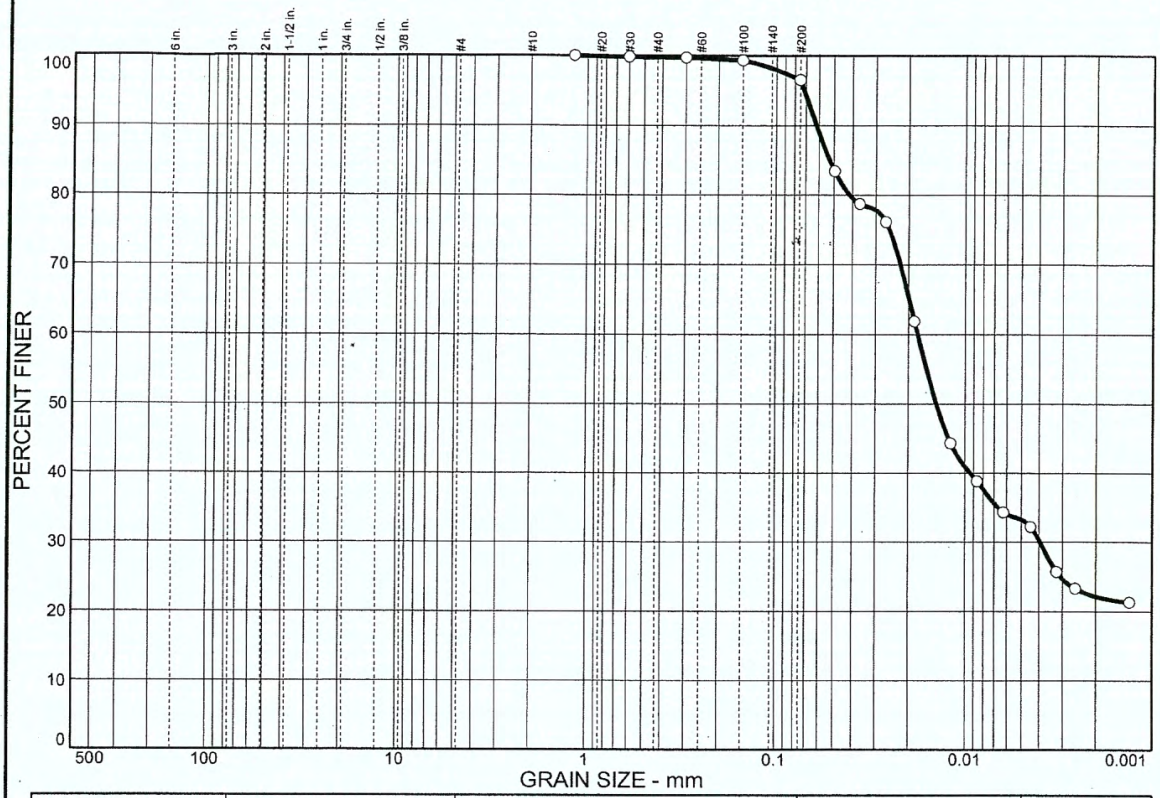
Sampled By Jason Washburn
Tested By Ryan Hogg
Checked By John Schwartz

Sample No.: 8200 **Source of Sample:** B-10 **Date:** 09/10/08
Location: S-6 **Elev./Depth:** 20'

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No: 96156 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	3.5	63.3	33.2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#16	100.0		
#30	99.8		
#50	99.7		
#100	99.3		
#200	96.5		

* (no specification provided)

Soil Description

Silt
Lab Sample 8200BU

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 0.0527 D_{60}= 0.0178 D_{50}= 0.0142
D_{30}= 0.0039 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

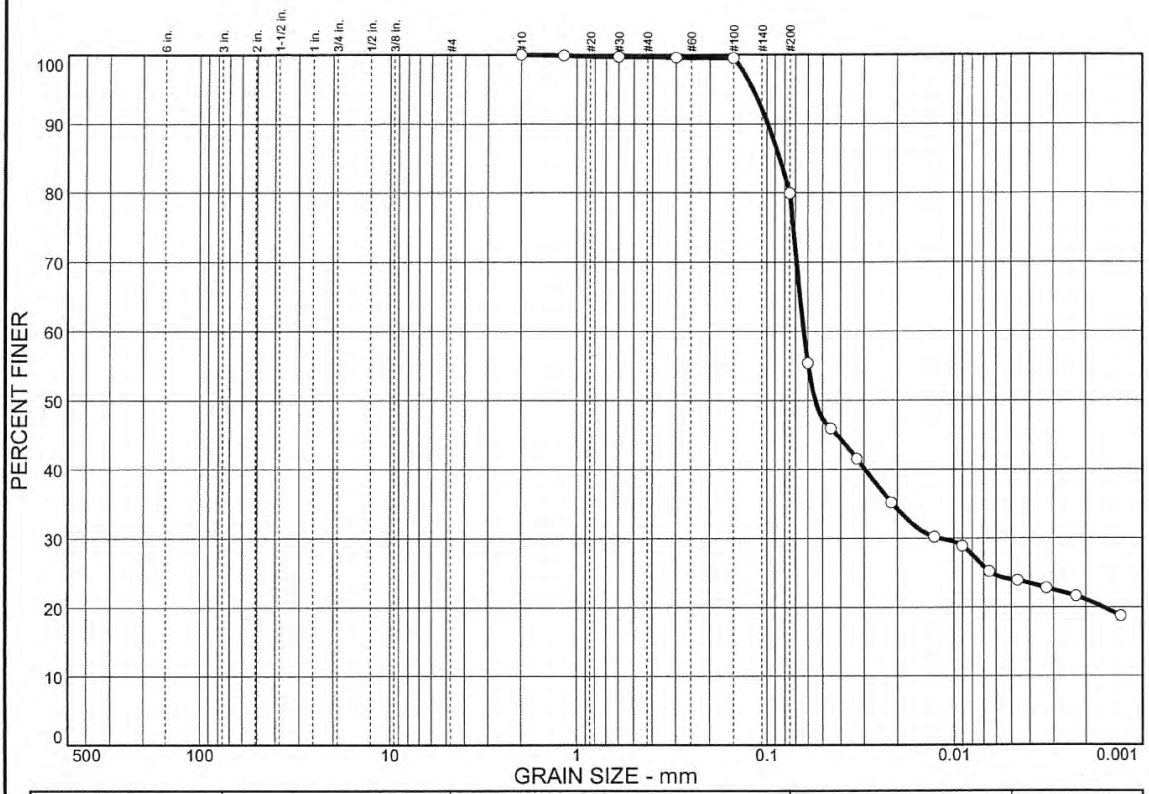
Sampled By: Jason Washburn
Tested By: Ryan Hogg
Checked By: John Schwartz

Sample No.: 8200 **Source of Sample:** B-11 **Date:** 9/8/08
Location: S-4 **Elev./Depth:** 10'

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No: 96156 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	20.1	55.8	24.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	99.9		
#30	99.7		
#50	99.6		
#100	99.5		
#200	79.9		

* (no specification provided)

Soil Description

Silt with sand
Lab Sample 8200BV

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 0.0853 D_{60}= 0.0634 D_{50}= 0.0548
D_{30}= 0.0119 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

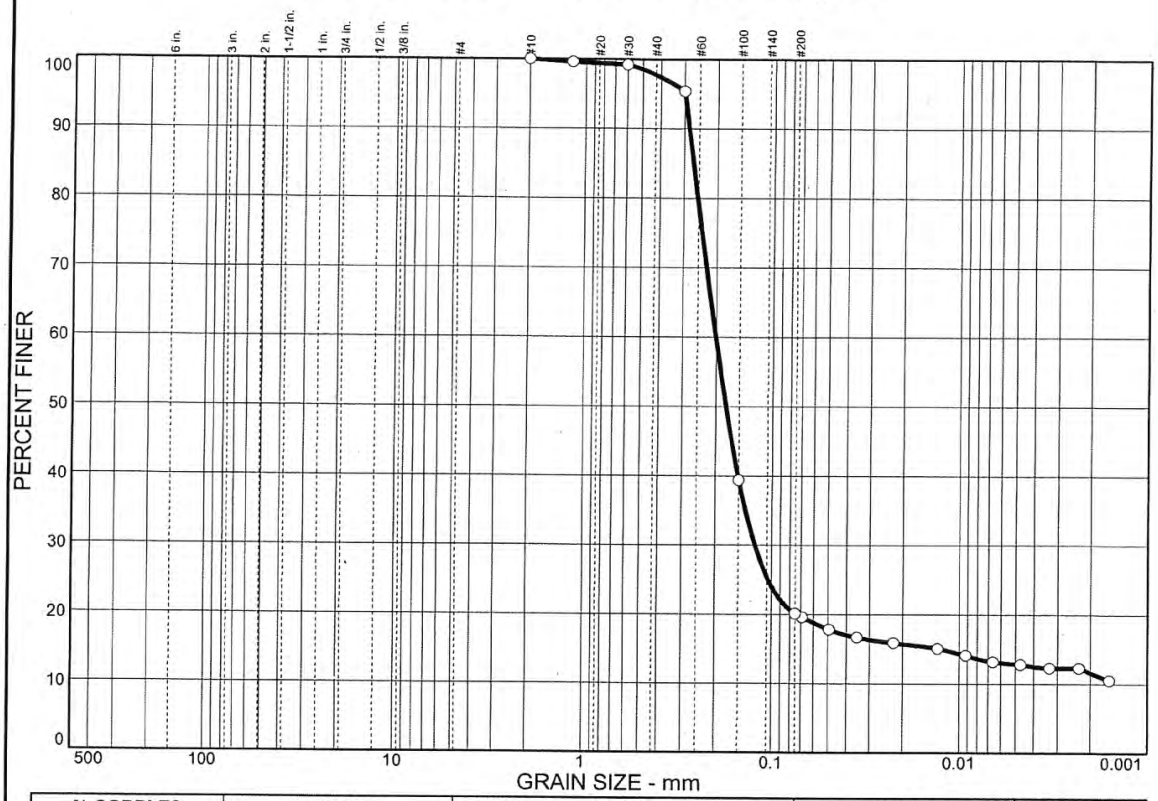
Sampled By Jason Washburn
Tested By Ryan Hogg
Checked By John Schwartz

Sample No.: 8200 **Source of Sample:** B-11 **Date:** 09/14/08
Location: S-5 **Elev./Depth:** 15'

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No: 96156 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	79.9	7.2	12.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#16	99.6		
#30	99.2		
#50	95.4		
#100	39.4		
#200	20.1		

* (no specification provided)

Soil Description

Silty sand
Lab Sample 8200BX

Atterberg Limits

PL= N/A LL= N/A PI= N/A

Coefficients

D_{85}= 0.263 D_{60}= 0.193 D_{50}= 0.171
D_{30}= 0.123 D_{15}= 0.0017 D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sampled By: Jason Washburn
Tested By: Ryan Hogg
Checked By: John Schwartz

Sample No.: 8200 **Source of Sample:** B-11 **Date:** 9/8/08
Location: S-8 **Elev./Depth:** 30'

KLEINFELDER, INC.

Client: Tulalip Qwuloolt
Project: Tulalip Qwuloolt Levees
Project No: 96156 **Figure**

DRAFT-FOR REVIEW PURPOSES ONLY

APPENDIX C
IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL ENGINEERING REPORT

DRAFT

Important Information About Your Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual

subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.

ASFE
The Best People on Earth

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
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e-mail: info@asfe.org www.asfe.org

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IIGER06045.0M

U.S. Army Corps
of Engineers
Seattle District

Qwuloolt Ecosystem Restoration Project Geotechnical Report

Marysville, WA

APPENDIX

TABLE OF CONTENTS

APPENDIX A. GEOTECHNICAL INVESTIGATIONS
- A-1 Kleinfelder Subsurface Exploration Phase I, 2008.
- A-2 Kleinfelder Subsurface Exploration Phase II, 2010.
- A-3 Kleinfelder Subsurface Exploration Phase III, 2011.
- A-4 Shannon & Wilson Marysville Wastewater Treatment Plant Stability Analysis, 2004.
- A-5 Tulalip Tribe Stockpile Sampling Report, 2011.
- A-6 Christofferson Site Geotechnical Report, 2009

APPENDIX B. RESULTS OF SLOPE STABILITY ANALYSES

APPENDIX C. RESULTS OF SEEPAGE ANALYSES

APPENDIX D. RESULTS OF SETTLEMENT ANALYSES

APPENDIX E. RESULTS OF SEISMIC ANALYSES