SHANNON & WILSON, INC.

PROJECT MEMORANDUM

To:	Jude Grounds, PE,
	MWH Americas, Inc.
From:	Yuxin (Wolfe) Lang, PE, GE
	Jerry Jacksha, PE, GE
Date:	August 6th, 2012
Subject:	Lake Oswego-Tigard Water Partnership
	Lake Oswego & Tigard Water Treatment Plant Expansion Project
	Seismic Geologic Hazards and Mitigations

Introduction

This memorandum supplements the Draft Geotechnical Engineering Report for the Lake Oswego & Tigard Water Treatment Plant (WTP) Expansion Project (Shannon & Wilson, January, 2012), located in West Linn, Oregon. The Draft Report discussed the potential seismic geologic hazards and summarized the mitigation strategies for some of the new deep and intermediate-depth structures proposed at the site. However, the Draft Report only presented preliminary recommendations for new shallow on-grade structures, new pipelines and other buried utilities, and seismic retrofits of existing structures. This memorandum summarizes the final recommended mitigation strategies that will be used in the design phase. NOTE: some information from the Draft Report, such as the potential seismic hazards at the WTP site, is repeated in this memorandum for clarity and background information. The Final Geotechnical Engineering Report will include the final detailed mitigation and foundation recommendations presented in this memorandum and the Draft Report. The Final Report is expected to be completed in August 2012.

Geologic Overview

To assess the subsurface conditions for the evaluations of site-specific seismic geologic hazards and mitigation approaches, and structural foundation design, Shannon & Wilson (S&W) conducted nine soil borings, four test pits and seven cone penetration tests (CPTs), and installed two groundwater observation wells. The explorations revealed that the site is underlain by approximately 25 feet of soft to stiff silt and sandy silt (Fine-Grained Flood Deposits) above the water table, and approximately 25 to 30 feet of very loose to medium-dense silty sand below the water table overlying dense to very dense gravel (Dense-Grained Flood Deposits). The exploration locations are shown in **Figure 1,** and soil deposits encountered from the explorations are shown in a generalized subsurface profile at Cross-Section A-A' in **Figure 2.** Detailed subsurface explorations, laboratory testing, and characterizations are contained in the Geotechnical Data Report (GDR) for this project dated August 2012.

Summary of Site-Specific Seismic Hazards Evaluation

As shown on the City of West Linn Natural Hazards Mitigation Plan (published in 2007, and a copy is shown in **Figure 3**), the WTP site is located in a high earthquake hazard area. Considering that certain elements of the water treatment plant function to provide fire flows to Lake Oswego, Tigard, and West Linn, those elements are considered essential facilities, and so S&W completed a Site-Specific Seismic Hazards Evaluation in accordance with requirements of the Oregon Structure Specialty Code, 2010 Edition (OSSC 2010). Detailed procedures and results of the Site-Specific Seismic Hazards Evaluation are presented in our "Draft Geotechnical Engineering Report, Lake Oswego & Tigard Water Treatment Plant Expansion Project" dated January 2012". A summary of the results and individual geologic hazard discussion are presented in the following section.

Seismic Setting and Maximum Considered Earthquake

The Lake Oswego – Tigard WTP site is subject to seismic events from three major sources: 1) Cascadia Subduction Zone (CSZ) Megathrust earthquakes at the interface of Juan de Fuca and North American Plates; 2) Deep-focus, CSZ intraplate earthquakes (within Juan de Fuca and North American Plates); and 3) Shallow-focus earthquakes in local and regional continental crustal faults.

Maximum magnitude for CSZ interface earthquakes are expected to be in the range of Moment Magnitude (M_W) 8 to 9 with a possible reoccurrence interval of 500 to 600 years (Barnett and others, 2004). Intraplate earthquakes have occurred on a frequent basis in the Puget Sound, but there is no strong historical evidence for such events in Oregon. Crustal fault earthquakes are typically generated within the upper portion of the continental crust (North American Plate). The largest known crustal earthquake in the Pacific Northwest is the 1872 North Cascades quake at Magnitude 7.4. Other examples include the 1993 Magnitude 5.6 Scotts Mill earthquake and 1993 Magnitude 6 Klamath Falls earthquake.

The contribution of earthquake hazards from these three sources at the WTP location was analyzed using the Probabilistic Seismic Hazard Deaggregation results taken from the USGS website. In the analysis, the Maximum Considered Earthquake (MCE) is taken as an event with a 2-percent probability of exceedance in 50 years (return period of 2,475 years) as inferred in the 2010 OSSC. **Table 1** shows the relative hazard contributions from the CSZ and shallow crustal earthquake sources, which constitute the primary earthquake hazards at the site.

TABLE 1: EARTHQUAKE HAZARD CONTRIBUTION

Return period (years)	Exceedance Probability	Spectral Acceleration Period	CSZ Megathrust EQ	Shallow Crustal EQ
2475	2 %	0 sec (PGA)	14 %	86 %
		0.2 sec	16 %	84 %
		1 sec	47 %	53 %

Note: PGA stands for Peak Ground Acceleration which corresponds to spectral acceleration at zero second.

SHANNON & WILSON, INC.

Seismic Ground Motion Amplification Effect and Site Classification

To assess the dynamic response of the soil column to the seismic waves, S&W conducted a non-linear, site-specific response analysis using computer program of D-MOD2000 that computes the dynamic response of a layered soil profile to vertically propagating shear waves using a non-linear stress-strain model. The analysis indicated that the ground motion (seismic waves) will likely be amplified moderately through the soil deposits, and values of estimated ground motion accelerations in terms of response spectrum were included in the Draft Geotechnical Engineering Report. Also in the analysis, the site seismic classification was assigned as Site Class E due to the potential liquefaction of loose to medium-dense silty sand and very soft to medium-stiff silt below the water table during the earthquakes.

Fault Rupture Hazard

From a review of the available literature, there are no currently mapped faults within 3 kilometers from the WTP. Based upon all of this information, the risk of fault surface rupture is considered to be low.

Seismic Soil Liquefaction & Lateral Spreading Hazards

As required by the current building code (OSSC 2010), the three earthquake scenarios discussed above were evaluated for the liquefaction potential. S&W's liquefaction analysis determined that the saturated loose to medium-dense silty sand and very soft to medium-stiff silt located below the groundwater levels are susceptible to soil liquefaction under these earthquake scenarios. Under the smaller magnitude crustal earthquake event (such as Mw=6.0), only a minor portion of the saturated silty sand deposit will liquefy (liquefaction zone less than 5 feet thick) and the estimated total liquefaction settlement is generally less than 1 inch. Under the Mw 7.0 earthquake, the liquefaction zone thickness increases to about 15 feet and the total settlement becomes 2 to 3 inches. Under the mega thrust Mw 9.0 event, the whole layer of the saturated silty sand (about 25 to 30 feet thick) will liquefy, with the estimated total liquefaction settlements throughout the site ranging from 5 to 9 inches and differential settlements ranging from 1 to 5 inches, depending on the loading and embedment depths of the structures.

Because the existing, relatively flat WTP site is located approximately 500 feet from the Willamette River and groundwater is relatively deep (and potentially even deeper toward the river bank), lateral spreading hazards are considered low.

Other Effects of Liquefaction on Structural Foundations

In addition to the total and differential settlement issue, liquefaction will also reduce the soil foundation bearing capacity, especially for deep structures founded near and into the liquefiable zone. Moreover, for, the dramatic increase in pore water pressure associated with soil liquefaction will subject deep structures extending into the soil liquefaction zone to high lateral earth pressures and buoyancy forces..

SHANNON & WILSON, INC.

Other Hazards

Due to the relatively flat site, large distance and high elevation relative to the Willamette River, other seismic hazards, such as seismic slope failure and tsunamis or seiche, are not potential seismic hazards at this site.

Seismic Hazards Conclusion

Based on our site-specific seismic hazards evaluation, S&W concluded that the major geologic hazard triggered by the design seismic event is soil liquefaction and that excessive total and differential settlement, and reduction in foundation bearing capacity can be expected as a result of soil liquefaction.

Seismic Hazard Mitigation

Categories of the Proposed Improvements

Based on the current design concepts, S&W understands that the major improvements for this WTP expansion project will consist of 12 new structures, 1 existing structure foundation retrofit, and new yard piping and buried utilities. For clarity, S&W's proposed mitigation strategies for the seismic soil liquefaction hazards are grouped into five categories. **Table 2** presents these categories and individual structures and the foundation depth in each category. Among these structures, Clearwell, Finished Water Pump Station (FWPS), and Filtration are considered essential facilities due to their function to provide fire flow under seismic event.

TABLE 2: PROPOSED MAJOR WTP IMPROVEMENTS

WTP Improvement Categories	Structures	Depth (ft)
Deep Structures	Clearwell	28 – 35
	Washwater Equalization	27 – 33
	Thickened Solids Tank	20
Intermediate Depth Structures	FWPS	10
	Ballasted Flocculation	9
	Filtration	10
	Gravity Thickeners	6.5 – 13
Shallow/On-Grade Structures	Electrical Bldg	On-Grade
	Mech. Dewatering	On-Grade
	Ozone Contactor	4
	Chemical Bldg.	On-Grade
	LOX Storage/Generator Pad	On-Grade
	Surge Tank	On-Grade
	Admin Bldg	On-Grade
Existing Structure Retrofit	Operations Bldg/Clearwell	15
Pipelines and Other Buried Utilities	On-site Water Conduits	3 – 16
	Yard Piping & Utility Duct Banks	Varies

NOTE: Following expansion of the WTP, the remaining two Lagoons and Decant Pump Station will no longer be active elements of the water treatment process, and will only be used infrequently for backup, non-critical services. Therefore, they are deemed not warranted for seismic retrofit.

Soil liquefaction Seismic Hazard Mitigation Strategies

For the proposed new structures and the retrofit of the existing structure, three general design philosophies can be applied to mitigate the liquefaction hazard as defined in OSSC 2010, which include: 1) ground improvement (e.g. soil mixing to strengthen the problematic soil), 2) foundation selection (e.g. use of deep foundation types to transfer the load below the problematic soil), and 3) structural system selection (e.g. structural framing systems) to accommodate the liquefaction settlement impacts. Considering the structural allowable differential settlement criteria of either 3/8-inch per 30 feet or 1/4-inch per 40 feet, deep foundations will be used as the preferred liquefaction mitigation and foundation-support elements for these structures. The preferred deep foundation type is auger-cast pile.

- **Deep Structures Mitigation Strategy**

To optimize the pile capacities and lengths, relatively large diameter auger-cast piles (18-inch and/or 24-inch-diameters) will be used. Because of the soil liquefaction hazard, the seismic loading requires that axial compressive, uplift, and lateral load resistances for the piles will have to be derived from the dense, non-liquefiable gravel deposit located about 55 feet below the existing ground surface. The pile embedment depths into gravel will be 15 feet for 18-inch diameter piles and 10 feet for 24-inch-diameter piles. These correspond to pile lengths of 35 to 50 feet for 18-inch-diameter piles or 30 to 45 feet for 24-inch-diameter piles. For the recommended pile diameters and depths, the estimated seismic allowable axial compressive capacities are on the order of 190 kips for the 18-inch-diameter piles, and 200 kips for the 24-inch-diameter piles. ***Because of the support from auger-cast pile foundations, these deep structures will not be adversely affected by seismic soil liquefaction and should have seismic total and differential settlement less than the design criteria.***

- **Intermediate Depth Structures Mitigation Strategy**

For the intermediate depth structures, auger-cast piles will also be used as the foundation support elements and liquefaction hazard mitigation method. With similar pile diameters and embedment depths into the competent gravel deposits as stated above for the deep structures, the approximate pile lengths will be 57 to 63 feet for the 18-inch-diameter piles and 52 to 58 feet for 24-inch-diameter piles. The seismic allowable axial compressive capacities are estimated to be on the order of 180 kips for the 18-inch-diameter piles, and 190kips for the 24-inch-diameter piles. ***Because of the support from auger-cast pile foundation, the intermediate depth structures will not be adversely affected by seismic soil liquefaction and should have seismic total and differential settlement less than the design criteria.***

- **Shallow/On-Grade Structures Mitigation Strategy**

Auger-cast piles will be used as foundation support elements of the on-grade structures and liquefaction hazard mitigation method. For 24-inch-diameter piles, the recommended length is 74 feet with an allowable capacity of 160 kips. For 18-inch-diameter piles, the recommend length is 64 feet with an allowable capacity of 150 kips. ***For these pile supported structures, no adverse impacts are anticipated due to liquefaction and seismic total and differential settlements should be less than the design criteria.***

- **Existing Structure Retrofit Mitigation Strategy**

S&W understands that the seismic retrofit strategy for the existing Operations/Clearwell building involves perimeter auger-cast piles as the foundation-bearing elements with internal structural modifications and tie-ins to the underground Clearwell perimeter walls to connect the building with the piles. With this approach, the pile lengths and capacities should be similar to the intermediate depth structures, and the seismic total and differential settlements between piles will be within acceptable limits.

- **Pipelines and Other Buried Utility Mitigation Strategy**

For short water conduits inside the WTP connecting water treatment facilities, auger-cast piles will be used to support the conduits to mitigate the liquefaction total and differential settlements. For the relatively long Raw Water and Finish Water conduits, S&W understands that articulating joints will be used at the connections to the structures to mitigate the estimated liquefaction settlements. For the minor yard piping and utility duct banks, S&W understands the project team is considering using flexible connections and joints, extendable sleeves, and additional reinforcement to increase the strength and ductility for liquefaction settlement mitigation. Combined, these mitigation techniques can successfully accommodate the seismic total and differential settlements.

REFERENCES

American Society of Civil Engineers, 2006, Minimum design loads for buildings and other structures: Reston, Va. American Society of Civil Engineers, ASCE Standard ASCE/SEI 7-05.

City of West Linn, Natural Hazards Mitigation Plan, Prepared for City of West Linn, in cooperation with Clackamas County.

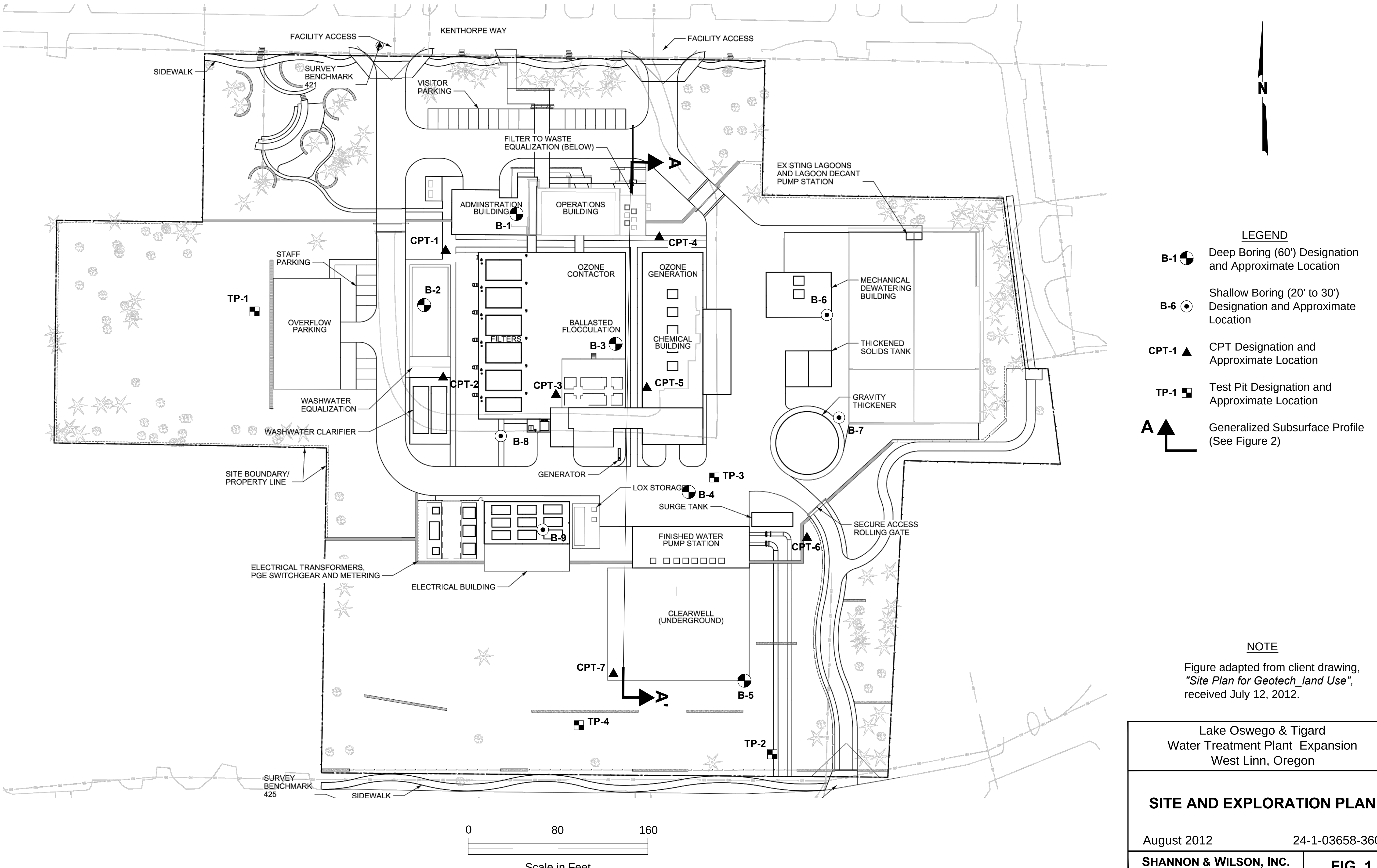
Oregon Department of Consumer and Business Services & The International Code Council, February 2010, 2010 Oregon Structural Specialty Code.

Shannon & Wilson, Inc., Draft Geotechnical Engineering Report, Lake Oswego & Tigard Water Treatment Plant Expansion Project, West Linn, Oregon, January 2012.

United States Geological Survey, 2002 Interactive Deaggregations: https://geohazards.usgs.gov /deaggint/2002/

Attachments:

Figure 1 – Site Exploration Plan
Figure 2 – Generalized Subsurface Profile A-A'
Figure 3 – City of West Linn Earthquake Hazard Map



NOTE

Figure adapted from client drawing, *"Site Plan for Geotech_land Use"*, received July 12, 2012.

Lake Oswego & Tigard
Water Treatment Plant Expansion
West Linn, Oregon

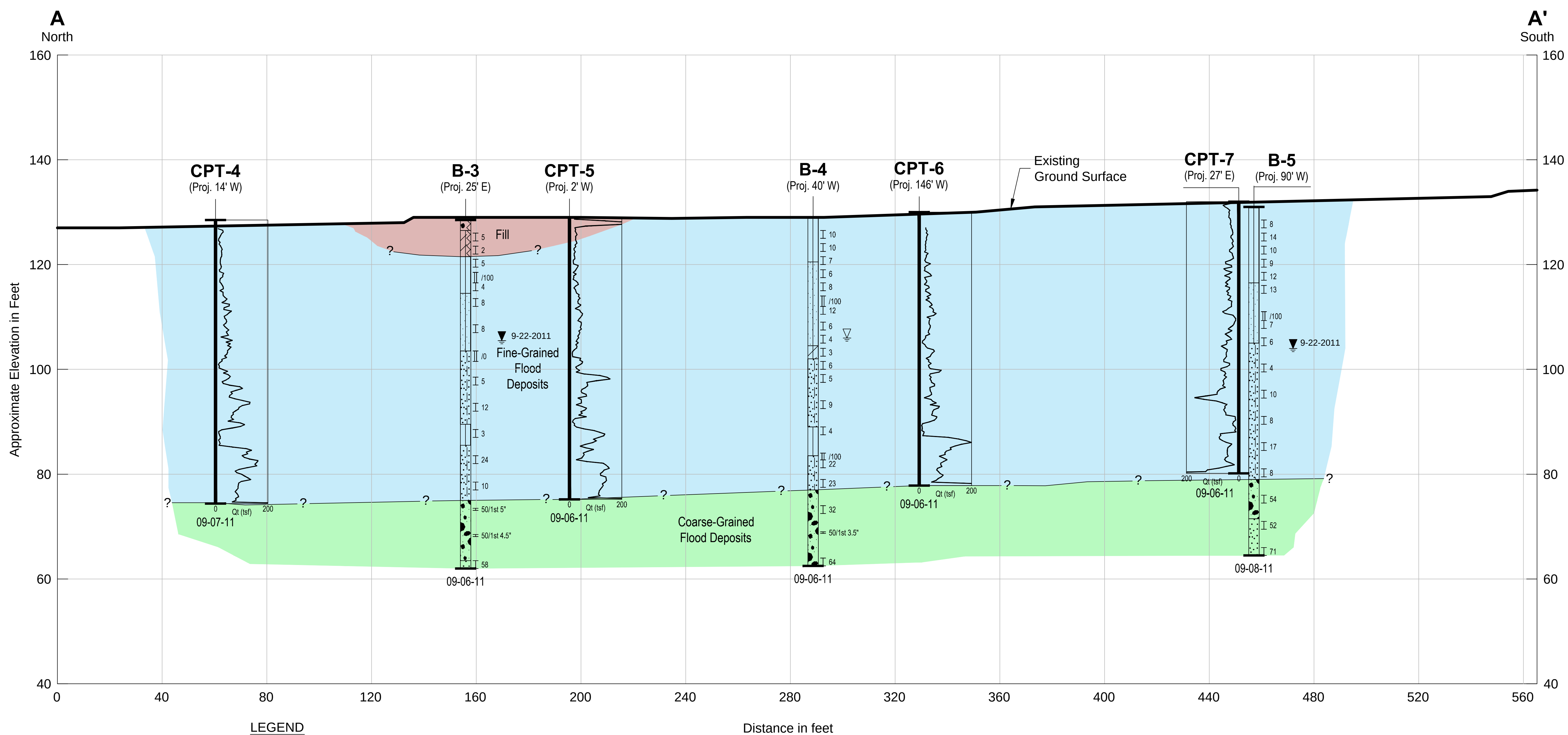
SITE AND EXPLORATION PLAN

August 2012 24-1-03658-360

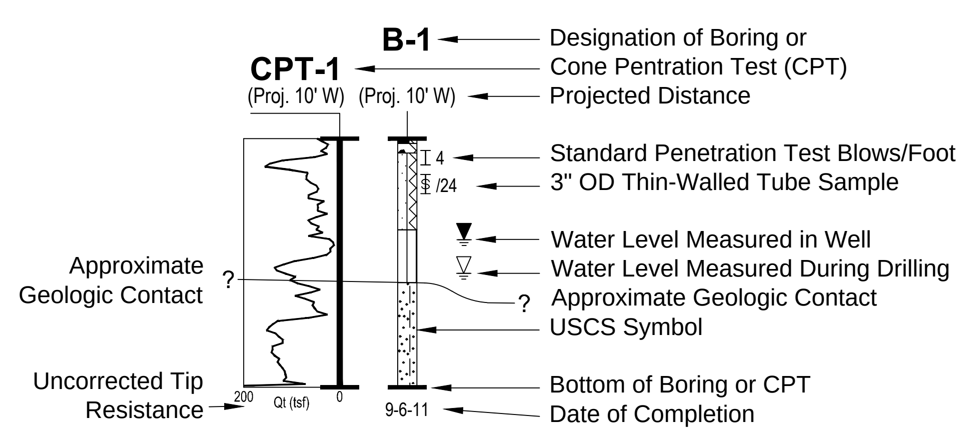
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 1

Filename: I:\WP\Projects\24-1 Portland\3600\3658 Lake Oswego Water Treatment Plant\Graphics\CAD\From Seattle Drafting\24-1-03658-245 Site Plan.dwg Date: 08-03-2012 Login: rrb



LEGEND



0 20 40

Vertical Scale in Feet

0 40 80

Horizontal Scale in Feet

SUBSURFACE MATERIAL LEGEND

- Fill
- Fine-Grained Flood Deposits
- Coarse-Grained Flood Deposits

NOTE

This subsurface proifle is generalized from materials observed in soil borings. Variations may exist between profile and actual conditions.

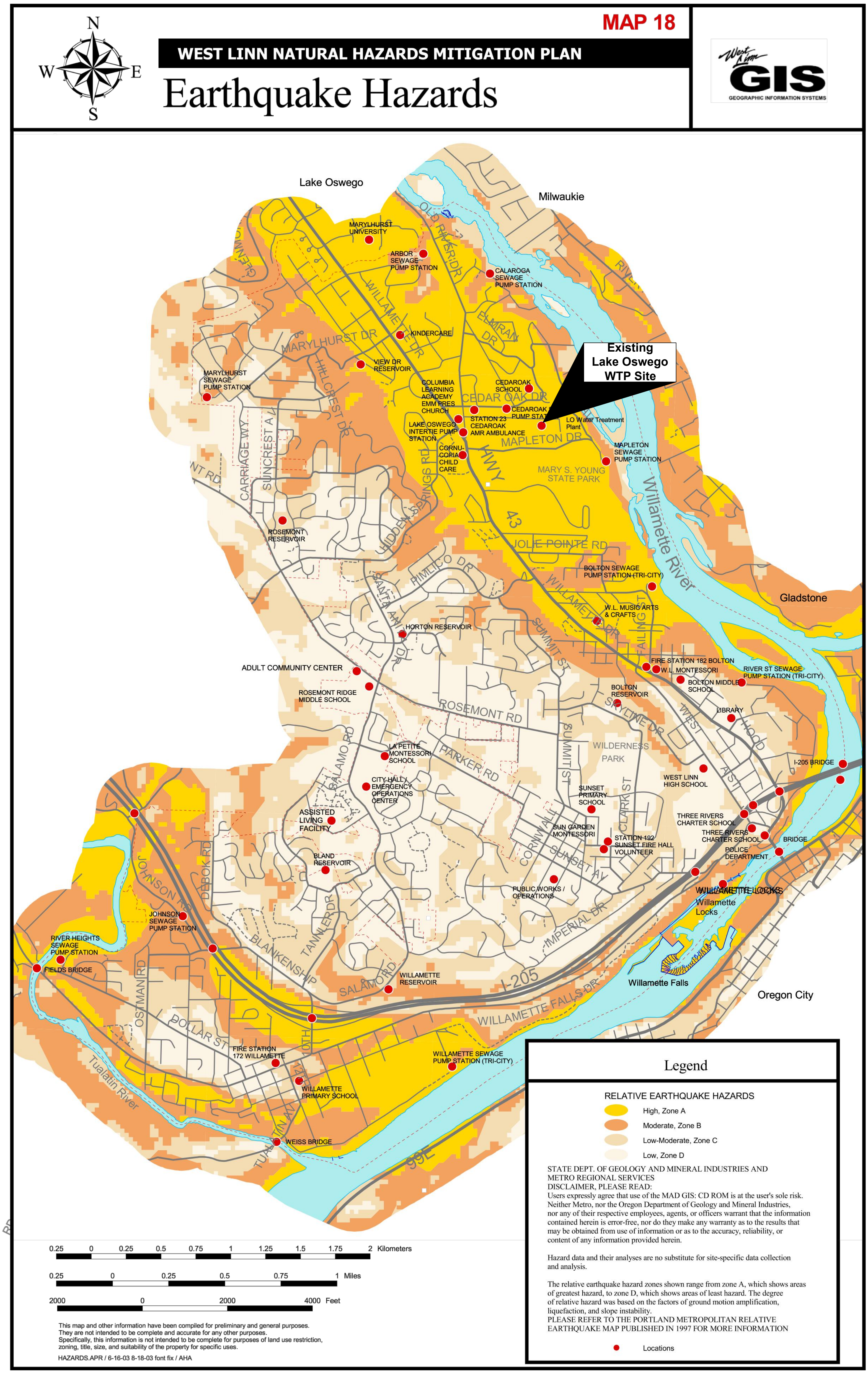
Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

GENERALIZED SUBSURFACE PROIFLE A-A'

August 2012 24-1-03658-360

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 2



Note:
Map is from City of West Linn Natural Hazards Mitigation Plan.

Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

CITY OF WEST LINN
EARTHQUAKE HAZARD MAP

August 2012 24-1-03658-360

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 3

GEODESIGN INC

August 8, 2012

Brown and Caldwell
6500 SW Macadam Avenue, Suite 200
Portland, OR 97239

Attention: Mr. Brett Teel

Geotechnical Peer Review
Lake Oswego-Tigard Water Partnership
Lake Oswego and Tigard Water Treatment Plant Expansion Project
Seismic Hazard Mitigation
Lake Oswego, Oregon
GeoDesign Project: BrownCald-49-21

INTRODUCTION

GeoDesign, Inc. has reviewed a project memorandum prepared by Shannon & Wilson, Inc. (S&W) dated August 6, 2012. The memorandum addresses mitigation of seismic hazards for the proposed Lake Oswego and Tigard Water Treatment Plant Expansion Project. The seismic hazards were identified during a geotechnical engineering study that is documented in a draft geotechnical engineering report that was issued in January 2012. Part of that study included conducting a site-specific seismic hazards evaluation that meets the requirements of the 2010 State of Oregon Structural Specialty Code (SOSSC).

SEISMIC HAZARDS

As required by SOSSC, S&W considered following potential seismic hazards:

- Seismic ground motion amplification
- Fault rupture
- Liquefaction and lateral spreading
- Seismic slope stability
- Tsunami
- Seiche

S&W concludes that the major seismic hazard is excessive total and differential settlement and a reduction in bearing capacity as a result of liquefaction. The other hazards are dismissed for the following reasons:

- *Seismic ground motion amplification:* A site-specific site response spectrum was computed and the results are to be incorporated in the structural design of structures.
- *Fault rupture:* The nearest mapped fault is not beneath the site.
- *Lateral spreading:* The water treatment plant site is flat and is at least 500 feet from the closest open face at the Willamette River. S&W also notes the relatively deep groundwater table.
- *Seismic Slope Stability, tsunami, and seiche:* S&W notes that the site is flat, is at a large distance from the Willamette River, and at a much higher elevation than the river.

MITIGATION

S&W proposes to mitigate the liquefaction hazard by supporting structures on auger-cast piles. They have grouped mitigation methods into five categories. Each method includes supporting the structure on 18- or 24-inch-diameter piles embedded 15 and 10 feet into the underlying gravel unit, respectively. However, the pile length and capacity varies based on the depth of the structure relative to the ground surface. Table 1 summarizes the pile capacity for each category.

Table 1. Pile Capacity

Category	Pile Capacity
Deep Structures	18-inch diameter - 190 kips 24-inch diameter - 200 kips
Intermediate Depth Structures	18-inch diameter - 180 kips 24-inch diameter - 190 kips
Shallow/On-Grade Structures	18-inch diameter - 150 kips 24-inch diameter - 160 kips
Existing Structure Retrofit	18-inch diameter - 180 kips 24-inch diameter - 190 kips
Pipelines and Other Buried Utilities[1]	Not provided

1. See below for additional details

Short water conduits inside the water treatment plant connecting water treatment facilities will be supported on auger-cast piles. Longer Raw Water and Finish Water conduits will have articulating joints at the connections to the structures. Yard piping and utility duct banks will have flexible connections and joints, extendable sleeves, and additional reinforcement.

CONCLUSIONS

Based on our review of the July 23, 2012 project memorandum and the January 2012 draft geotechnical engineering report, it is our opinion that potential seismic hazards have adequately been addressed and identified. It is our opinion that the planned mitigation is adequate under design levels of ground shaking. Furthermore, GeoDesign is confident that the pile capacities provided by S&W will easily be achieved for the recommended embedment depth in the gravel unit.

♦ ♦ ♦

We trust that this information meets your requirements. Please contact us if you have any questions or require additional information.

Sincerely,

GeoDesign, Inc.

Brett A. Shipton, P.E., G.E.
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
56207
OREGON
JULY 11, 2000
BRETT A SHIPTON
EXPIRES: 6/30/2014

BAS:kt
One copy submitted (via email only)
Document ID: BrownCald-49-21-080812-geol-sw_seismic_peer_review.doc

Draft Geotechnical Engineering Report
Lake Oswego & Tigard
Water Treatment Plant Expansion Project

West Linn, Oregon

January 2012

DRAFT

SHANNON & WILSON, INC.

GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

Draft Geotechnical Engineering Report
Lake Oswego & Tigard
Water Treatment Plant Expansion Project

West Linn, Oregon

January 2012

DRAFT

SHANNON & WILSON, INC.

GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

Excellence. Innovation. Service. Value
Since 1954

Submitted To:
Peter Kreft
MWH Americas, Inc.
806 SW Broadway, Suite 200
Portland, Oregon 97205

By:
Shannon & Wilson, Inc.
3990 Collins Way, Suite 100
Lake Oswego, Oregon 97035

24-1-03658-245

EXECUTIVE SUMMARY

The purpose of this report is to present Shannon & Wilson's geotechnical findings at the Lake Oswego & Tigard Water Treatment Plant for the proposed new treatment facilities. This report summarizes preliminary findings, alternatives analysis, and, where possible at this early stage of design, preferred alternatives to mitigate the potential seismic issues at the site. This report was developed in support of the Land Use Application process; final refinements to the analysis and recommendations herein will occur during detailed design and will be presented in the Final Geotechnical Engineering Report to be included as part of the Building Permit process.

Currently, the project is approaching the final stage of predesign, coinciding with the early stages of the detailed design phase. The exploration program and data collection have been completed, which consisted of nine soil borings, four test pits, seven cone penetration tests, two groundwater observation wells, and a laboratory testing program on selected soil samples. The explorations revealed that the site is underlain by approximately 25 feet of soft to stiff silt and sandy silt above the water table, and approximately 25 to 30 feet of very loose to medium-dense silty sand below the water table overlying dense to very dense gravel.

We also conducted a ***site-specific seismic hazards evaluation*** which yielded seismic spectral response accelerations which generally match, but are lower than the code-specified design parameters, and identified the key geotechnical issue: seismic liquefaction of the saturated silty sand deposit during the design earthquake events. This hazard condition is consistent with other 'High Zone A' sites in the region, the highest relative ranking on the City of West Linn's Earthquake Hazard Map (City of West Linn, 2007). As required by the current building code (OSSC 2010), three design earthquake scenarios were evaluated for the liquefaction analysis, which included a magnitude 6.0 crustal earthquake, a magnitude 7.0 Intraplate Subduction Zone earthquake, and a magnitude 8.5 to 9.0 Interface Subduction Zone earthquake. Our analysis indicated that only a minor portion of the saturated silty sand deposit will liquefy (liquefaction zone less than 5 feet thick with total settlement on the order of 0.25 to 0.5 inch) under the smaller magnitude 6.0 event. Under the medium magnitude 7.0 earthquake event, the liquefaction zone thickness increased to about 15 feet and the total settlement will be 2 to 3 inches. Under the mega-magnitude 9.0 event, the whole layer of the saturated silty sand (about 25 to 30 feet thick) will liquefy, with estimated total liquefaction settlements throughout the site of about 5 to 9 inches and differential settlements ranging from 1 to 5 inches, depending on the loading and embedment depths of the structures. The other key geotechnical issues are increased lateral earth pressure and uplift pressure (flotation effect) due to seismic liquefaction, and the complex shoring requirements due to phased construction of new structures and demolition of some existing structures while the plant must remain in operation.

At this early design phase, we understand that these differential liquefaction settlements exceed the design differential settlement criteria for the proposed new deep and intermediate/shallow depth water-holding main treatment structures. Therefore, after conceptual evaluations of typical mitigation techniques, preliminary assessments of mitigation methods for these two categories of structures (including various ground improvements and deep foundation options) were made. The most feasible mitigation alternatives with the current design criteria are ranked as follows:

- No. 1 Alternative: Auger-cast piles (drilled-in method)
- No. 2 Alternative: Soil Mixing Columns (ground improvement technique)

Also, if the differential settlement criteria can be relaxed somewhat, a third mitigation alternative can be considered, which is stone columns combined with vertical earthquake drains.

These two preferred alternatives, the third alternative, and other common mitigation techniques are discussed in the report. For this early design phase for the deep and intermediate/shallow water-holding main treatment structures, we recommended auger-cast piles as the preferred liquefaction mitigation and foundation-supporting elements for these structures.

The other categories of proposed structures consist of: (a) on-grade buildings and ancillary treatment structures, (b) remaining existing buildings and (c) new pipeline/utilities. For these categories of structures, the performance requirements, design differential settlement criteria and mitigation strategies, if needed, have not yet been established. Therefore, the design and any foundation mitigation recommendations will be addressed later in the design phase.

Details related to the site-specific seismic hazard evaluation, differential liquefaction settlements and mitigation, foundation design recommendations, and construction considerations are included in subsequent sections of this report.

For the No. 1 alternative, preliminary recommendations are that auger-cast piles be embedded sufficiently into the non-liquefiable dense gravel deposit underlying the liquefiable silty sand for axial compressive, uplift, and lateral bearing resistances. Also, the preliminary design evaluated 18-inch and 24-inch diameter piles, typical of the size used in the metro Portland and surrounding areas. For the deep water-holding structure, we estimate that the 18-inch auger-cast piles with 15 feet embedment into dense gravel deposit would be about 40 feet long, and 24-inch piles with 10 feet embedment into gravel would be about 35 feet long. For the water-holding intermediate depth structures, the 18-inch auger-cast piles would range in length from 45 to 60 feet, and 24-inch piles would range in length from 40 to 55 feet. Selection of final pile diameter and spacing and other details would occur in the upcoming design phase.

TABLE OF CONTENTS

TABLES

FIGURES

APPENDICES

DRAFT GEOTECHNICAL ENGINEERING REPORT
LAKE OSWEGO AND TIGARD
WATER TREATMENT PLANT EXPANSION PROJECT
WEST LINN, OREGON

1.0 INTRODUCTION

1.1 General

This report presents the results of our geotechnical site evaluations, engineering analysis, and recommendations to support design and construction of the Lake Oswego and Tigard Water Treatment Plant (WTP) Expansion Project in West Linn, Oregon. The Vicinity Map, Figure 1, shows the location of the existing Lake Oswego WTP. The cities of Lake Oswego and Tigard are the project owners, and MWH, Inc., (MWH) is leading the project design. Shannon & Wilson, Inc., (S&W) is providing geotechnical engineering services for the project under a subcontract to MWH.

1.2 Project Understanding

The existing WTP is located in West Linn, Oregon, on South Kenthorpe Way, approximately a quarter of a mile southwest from the Willamette River. It was originally built in 1968 but has undergone numerous upgrades since then to improve its performance. The current project will upgrade the capacity of the plant from 16 to 38 million gallons daily (mgd). To achieve this capacity, the plan is to reconfigure the plant from direct filtration to conventional filtration with intermediate ozonation followed by biologically active granular media filtration. Other modifications include a new, larger clearwell and finished water pumping station (FWPS), electrical system improvements, mechanical processes to treat process waste streams and residual solids, upgrades to chemical feed systems, and miscellaneous improvements to existing buildings and site landscaping. To achieve this goal, the plant will expand to the south onto property accessed by Mapleton Drive, referred to as the Mapleton Property. We also understand that during the expansion project, the existing WTP needs to stay in operation; therefore, the expansion project will be completed in stages to ensure no disruptions to the existing WTP operations.

The main facilities at the existing WTP include an Operations Building, three Sedimentation Basins, Filter Gallery with six filters, four concrete Backwash Lagoons (#1 through #4), a Clearwell and Finish Water Pump Station (FWPS) below the Operations Building, a Lime Building, and some chemical storage tanks. Except for the Operations Building with the underlying Clearwell and two Backwash Lagoons (#3 and #4), the rest of these main facilities will be demolished for the construction of new facilities for the expanded plant.

The proposed locations of the new facilities are shown in Figure 2, Site and Exploration Plan. Most of the new facilities (Ballasted Flocculation, Ozone Contactors, new Filters, WW Equalization Basin and WW Clarifier, Chemical Building, Dewatering Building, Gravity Thickeners and Pump Station, and new Administration/Operations building) will be constructed within the footprint of the existing WTP, with the exceptions of the Clearwell/Finished Water Pump Station (FWPS) and the Electrical/Maintenance Building (Electrical Building), which will be constructed on the Mapleton Property south of the WTP.

The Ballasted Flocculation, Ozone Contactors, new Filters, WW Equalization Basin and WW Clarifier, and the Clearwell/FWPS are the primary treatment and water-holding facilities on the site. These structures will generally be partially buried or fully buried structures. The embedment depths range from 5 feet (Ozone Contactors) to 35 feet (Clearwell/FWPS).

Other new structures are the operation and ancillary shallow/intermediate depth treatment/supporting facilities and are generally on-grade, one-story structures, except for the Administration/Operations building, (Admin Building) which will be two-story in height and the Gravity Thickeners and Pump Station which will be about 13 feet deep.

Detailed structural loading information for each structure is not available at this stage; however, for the purpose of this report, we have assumed that the primary water-treatment and water-holding structures will have contact pressures of on the order of 2,000 to 3,000 pounds per square foot (psf) as the dead load (including water). For the on-grade structures, we assume floor slab loads on the order of 150 to 300 psf and that the maximum column and wall loads will be on the order of 50 to 150 kips and 3 to 4 kips per linear foot, respectively. We also assume that the facility will be designed and constructed in accordance with provisions of the Oregon Structural Specialty Code (2010 OSSC) and ASCE 7-05.

1.3 Scope of Work

Shannon & Wilson's scope of work included both a geotechnical data collection phase and a site evaluation, engineering analysis, and recommendation phase. These phases and tasks are described below.

1.3.1 Data Collection Phase

With respect to the geotechnical data collection for the proposed WPT expansion, S&W's scope of work has included the following tasks:

- Developed and managed the field geotechnical exploration program (including nine soil borings, four test pits, seven cone penetration tests, and installation of two groundwater observation wells), and a laboratory testing program.
- Summarized the geotechnical exploration program and laboratory testing results in a draft Geotechnical Data Report (GDR) that was issued in December 2011.

A summary of the subsurface conditions from the GDR are described for reference in Section 3.0 and in Appendices A and B of this report.

1.3.2 Site Evaluation, Analysis, and Recommendation Phase

S&W's scope of work for this phase of the project has included the following tasks:

- Conducted a Site-Specific Seismic Hazards Evaluation including site-specific ground motion analysis.
- Conducted geotechnical engineering analysis for various foundation types and performed geotechnical constructability assessments for the foundation construction.
- Performed geotechnical engineering analysis for bearing capacities, settlements, foundation lateral load resistance, and lateral earth pressures.
- Conducted a conceptual assessment to evaluate the feasibility of open excavation, temporary shoring, and groundwater control systems.
- Provided recommendations for site preparation, structural fill, and compaction criteria.
- Summarized the site evaluations, analysis, conclusions, and recommendations in this draft Geotechnical Engineering Report.

2.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

2.1 General Site Geology

The Lake Oswego-Tigard WTP area is covered in geologic mapping by Beeson and others (1989). The mapping identifies the Columbia River Basalt Group bedrock (CRBG), the older sediments deposited by Willamette and Clackamas Rivers, and the surficial sediments associated with the Missoula Flood episodes.

The WTP site is situated along the east flank of the Tualatin Mountains, which are composed primarily of lava flows belonging to the Miocene Age CRBG. The CRBG, which originated from volcanic rifts in northeastern Oregon, repeatedly inundated this area between 17 and 6 million years ago. Tectonic stresses in the earth's crust began producing folds and faults in the CRBG flows even as the earliest of the lava flows were being emplaced, and over time portions of the CRBG have been uplifted to form the Tualatin Mountains. Most of the faultings associated with the Tualatin Mountains uplift are very old and are no longer considered active.

In most low-lying areas along the river, the CRBG has been buried beneath sequences of alluvial sediments. Commonly, a sequence of older sand and gravel deposited primarily by the Willamette and Clackamas Rivers directly overlies the basalt. Near the project site, this older sediments layer is exposed along Nixon Avenue east of the site. Beeson and others (1989) described the older sediments, which they termed "Unnamed conglomerate," as follows:

> "*Well-rounded pebbles and cobbles of mainly andesite to dacite, with minor amounts of Columbia River Basalt, in a poorly to moderately indurated lithic sandstone to sandy siltstone matrix. Andesite and dacite clasts often have weathering rinds, while Columbia River Basalt clasts display little evidence of decomposition. Unit varies in thickness from less than 30 to more than 200 feet. Conglomerate of the same composition is exposed within the adjacent Gladstone quadrangle and represents part of a thick (more than 400 feet) channel fill. Clast and matrix lithologies of this unit … probably represent deposits of Cascadian streams or an ancestral Clackamas River during [middle to late Pliocene]… time.*"

This older unit is then overlain by a sequence of younger sediment, largely sand and silt. The younger sediment consists predominantly of materials deposited by the catastrophic Missoula Floods that occurred during the late stages of the glacial epoch of the Pleistocene, some 15,500 to 13,000 years ago. The Missoula Floods consisted of many individual episodes of glacial outburst flooding, which overwhelmed the Columbia River and back-flooded up the Willamette Valley. Three facies of flood deposits are recognized in the greater Portland-Vancouver metropolitan area: coarse-grained facies, fine-grained facies, and channel facies. In the project area, only the fine-grained facies is present.

2.2 Summary of Subsurface Conditions

2.2.1 Field Exploration and Laboratory Testing Program

The subsurface conditions of the site were explored with nine soil borings (B-1 through B-9), four test pits (TP-1 through TP-4), and seven Cone Penetration tests (CPT-1 through CPT-7). The plan locations of the borings are shown in Figure 2, Site and Exploration Plan. The borings and CPTs were at or near the locations of the proposed new structures for the Expansion Project. The test pits were for the proposed pipelines and possible stormwater infiltration locations. The depths of borings ranged from approximately 30 to 65 feet, the depths of the CPTs were between 52 feet and 58 feet, and the test pits were approximately 15 feet deep. Upon completion of the borings, two groundwater observation wells were installed in borings B-3 and B-5. Details of the drilling, sampling, and CPT procedures are presented in the project GDR. A copy of the borings, test pits, and CPT logs are presented in Appendix A of the report for reference.

Upon the completion of the field explorations, a laboratory testing program consisting of visual-manual classification, moisture contents, Atterberg limits, sieve analysis, and standard proctor compaction tests was conducted on selected representative soil samples from borings and on bulk samples from the test pits. Details of the testing procedures and results are presented in the project GDR. A copy of the testing results is presented in Appendix B of this report for reference.

2.2.2 Previous Studies

We reviewed the available previous geotechnical information from the CH2M Hill 1975 study for the early developments of the WTP. A copy of this information is presented in Appendix C. There was also a geotechnical investigation conducted by Dames and Moore in the late 1990s when the lime building and concrete lagoons were designed, but appendices which include the boring logs were lost and were therefore unavailable for review.

2.2.3 Current On-Site Infiltration Testing

In addition to the current geotechnical explorations and testing performed by Shannon & Wilson, GreenWorks, PC, performed two infiltration tests for the potential on-site stormwater management facilities. Their findings are included in Appendix D of the “LO and Tigard Water

Treatment Plant Expansion Project – Preliminary Stormwater Management Report," dated January 2012.

2.2.4 Summary of Subsurface Conditions

The field exploration program has disclosed a relatively uniform stratigraphy consisting of three soil engineering units that are present across the entire project area: Artificial Fill, Fine-Grained Flood Deposits, and Older Sand and Gravel Alluvium. Our interpretations of the relationships between these soil units are illustrated in the generalized subsurface profiles A-A' and B-B' shown in Figures 3 and 4. The profile lines are shown on the Site and Exploration Plan, Figure 2.

In general, the Fill unit was about 1.5 to 5 feet thick and was encountered in the existing WTP either near existing buildings or structures, or underneath pavement. The unit consisted of pavement sections, base course materials, medium-dense sandy silty gravel, and very soft to medium-stiff clayey silt to sandy silt. The Fine-Grained Flood Deposits unit was either encountered underlying the Fill within the existing WTP or at the ground surface within the Mapleton property, and extended to about 52 to 57 feet in depth. The Fine-Grained soil unit consists of soft to stiff clayey silt and sandy silt, and very loose to medium-dense silty sand to sand. Underlying the Fine-Grained soil unit was the Older Sand and Gravel Alluvium unit. This unit consists of very dense sandy gravel to gravelly sand. More detailed descriptions of the soil units are presented in the project GDR.

2.2.5 Groundwater Measurements

Groundwater levels were measured in the installed observation wells. Table 1 presents the groundwater level measurements in the observation wells as follows:

TABLE 1: GROUNDWATER LEVEL MEASUREMENTS

Observation Wells	Date of Reading	Groundwater Depth[1] (in feet)	Groundwater Surface Elevation[2]
B-3	9/13/2011	23.1	105.4
	9/22/2011	23.3	105.2
	11/15/2011	23.5	105.0
	12/12/2011	24.4	104.1
B-5	9/13/2011	26.8	104.2
	9/22/2011	27.1	103.9
	11/15/2011	28.0	103.0
	12/12/2011	28.3	102.7

1 Groundwater depth is given in feet below the ground surface.

2 Surface elevations are approximate elevations from the base map provided to S&W by MWH Americas, Inc.

3.0 SITE SPECIFIC SEISMIC HAZARDS EVALUATION

3.1 General

As shown on the City of West Linn Natural Hazards Mitigation Plan (published in 2007), the WTP site is located in a high earthquake hazard area (see Figure 5). Considering that the water treatment facilities are typically classified as important facilities or even essential facilities, we completed a Site-Specific Seismic Hazards Evaluation in accordance with requirements in Oregon Structure Specialty Code, 2010 Edition (OSSC 2010). The OSSC 2010 allows for the development of a site-specific response spectrum, based on the analysis procedures specified in Chapter 21 of ASCE 7-05 (Minimum Design Loads for Buildings and Other Structures), using recorded or simulated horizontal ground motion acceleration time histories.

OSSC allows two approaches to determine site response used in design. The generalized "code-based" approach represents ground motions using an Maximum Credible Earthquake (MCE) acceleration response spectrum having a 2-percent probability of exceedance (of a specific spectral acceleration) in a 50-year period, also known as having a 2,475-year return period. The MCE ground response is scaled to a design spectrum by applying appropriate code-based factors to account for local subsurface conditions and a scale factor (2/3 of MCE) to establish a response corresponding to an approximate 1,000-year return period. This approach is generally considered conservative.

For the second approach, the current OSSC allows for a site-specific determination of site response and references use of ASCE 7-05. We performed a site-specific site response evaluation using the non-linear effective stress computer program D-MOD2000 by Geomotions, LLC. Details of the analyses are discussed in the following sections.

3.2 Seismic Setting

The Portland area is subject to seismic events from three major sources: 1) the Cascadia Subduction Zone (CSZ), 2) at the interface between the Juan de Fuca plate and the North American plate; intraslab faults within the Juan de Fuca plate; and 3) crustal faults in the North American plate. Maximum magnitude for a CSZ event is expected to be in the range of Moment Magnitude (MW) 8 to 9 with a possible reoccurrence interval of 500 to 600 years (Barnett and others, 2004). Intraslab events have occurred on a frequent basis in the Puget Sound, but there is no strong historical evidence for such events in Oregon. Known and suspected crustal faults in the region have been characterized by the United States Geological Survey (USGS) and the Oregon Department of Geology and Mineral Industries (DOGAMI).

According to the USGS Quaternary Fault and Fold Database of the United States (Personius, S.F., 2002), the nearest mapped Quaternary fault is the Oatfield fault approximately 3 kilometers (Km) to the east northeast of the site. Several additional faults with evidence of movement during the Quaternary Period, listed below in Table 2, have been mapped within an approximate 20-kilometer radius of the project site. Each of the faults in Table 2 is defined as a "Class A" Fault by the USGS. Class A faults are those for which there is demonstrable evidence of tectonic movement during the Quaternary Period that are known or presumed to be associated with large-magnitude earthquakes.

TABLE 2: QUATERNARY FAULTS WITHIN A 20-KILOMETER RADIUS OF THE LAKE OSWEGO – TIGARD WATER TREATMENT PLANT SITE

Name	Distance and Direction from Site	Most Recent Deformation*	Slip Rate
Oatfield Fault	3 Km Northeast	<1.6 Ma	<0.2 mm/yr
Portland Hills Fault	4 Km Northeast	<1.6 Ma	<0.2 mm/yr
Canby-Molalla Fault	7 Km West	<15 Ka	<0.2 mm/yr
Damascus-Tickle Creek Faults	8 Km Northeast	<750 Ka	<0.2 mm/yr
East Bank Fault	11 Km Northeast	<15 Ka	<0.2 mm/yr
Beaverton Fault Zone	18 Km Northwest	<750 Ka	<0.2 mm/yr

*Ka= "Kilo-annum," or thousand years; Ma= "Mega-annum," or million years

3.3 Seismic Site Classification

The site is underlain by approximately 25 feet of soft to stiff silt and sandy silt above the water table, and approximately 25 to 30 feet of very loose to medium-dense silty sand below the water table overlying dense to very dense gravel. Based on our simplified empirical liquefaction analyses and the effective stress numerical modeling, the saturated silty sand is susceptible to liquefaction during the design earthquake event. Thus, in accordance with the OSSC, the site should generally be classified as Site Class F due to the liquefiable materials.

In OSSC 2010 and ASCE 7-05, no site coefficient values are specified for Site Class F, and the building codes require a site-specific ground motion evaluation to establish the spectral response acceleration parameters. However, the codes allow to use Site Class E values for Site Class F structures with design period less than 0.5 seconds (we understand that this is the case for the proposed structures). Therefore, we used Site Class E values to estimate the "code-based" spectral response accelerations at the ground surface for comparison to the site-specific site response.

3.4 "Code-Based" Seismic Site Response

As stated previously, the OSSC 2010 code specifies the use of an earthquake event having a 2-percent probability of exceedance in 50 years (an approximate return period of 2,475 years).

This earthquake is defined as the Maximum Considered Earthquake (MCE) for use in structural design. The design spectral accelerations were obtained from the 2002 U.S. Geological Survey (USGS) National Seismic Hazard Mapping Program probabilistic seismic hazard analyses (PSHA), (Frankel et al., 2002). The location of the ground motions for the evaluation is based on the following geographical information:

- Latitude = 45.386
- Longitude = -122.632

The seismically induced acceleration values at the rock interface, and the coefficient used to estimate ground surface response adjusted for Site Class E, for the MCE at the site are presented in Table 3:

TABLE 3: USGS CODE BASED MCE AND DESIGN SEISMIC PARAMETERS

Seismic Parameters	Value
MCE Peak Bedrock Acceleration (PBA)	0.40g
MCE Bedrock Spectral Acceleration, 0.2 second period (S_S)	0.94g
MCE Bedrock Spectral Acceleration, 1.0 second period (S_1)	0.33g
Short-Period Site Factor, F_a	0.97
Long-Period Site Factor, F_v	2.68
Soil MCE Peak Ground Acceleration (MCE PGA)	0.36
Soil MCE Spectral Acceleration, 0.2 second period, Site Class E (S_{MS})	0.91
Soil Design Spectral Acceleration, 1.0 second period, Site Class E (S_{M1})	0.88
Soil Design Peak Ground Acceleration (Design PGA)	0.24
Soil Design Spectral Acceleration, 0.2 second period, Site Class E (S_{DS})	0.61
Soil Design Spectral Acceleration, 1.0 second period, Site Class E (S_{D1})	0.59

3.5 Site-Specific Site Response

The site-specific ground motion evaluation was performed in accordance with the procedures specified in ASCE 7-05, using recorded or simulated horizontal ground motion acceleration time histories. The site-specific analysis procedures are detailed below.

3.5.1 Earthquake Source Hazard Contribution

Within the present understanding of the regional tectonic framework and historical seismicity, three broad earthquake sources have been identified. These three types of earthquakes and their maximum plausible earthquakes, as determined by Wells and others (2000), are as follows:

- **Subduction Zone Interface Earthquakes:** Originate along the Cascadia Subduction Zone (CSZ), which is located 25 miles beneath the coastline. Paleoseismic evidence and historic tsunami studies indicate that the most recent subduction zone thrust fault event occurred in the year 1700, probably ruptured the full length of the CSZ, and may have reached a Magnitude 9.
- **Deep-focus, Intraplate Earthquakes:** Originate from within the subducting Juan de Fuca oceanic plate as a result of the downward bending and contortion of the plate. These earthquakes typically occur 28 to 38 miles beneath the surface. Such events could be as large as Moment Magnitude 7.5. Examples of this type of earthquake include the 1949 Magnitude 7.1 Olympia earthquake, the 1965 Magnitude 6.5 earthquake between Tacoma and Seattle, and the 2001 Nisqually (slightly north of Olympia) earthquake at Magnitude 6.8. Intraslab events have occurred frequently in the Puget Sound, but are historically rare in Oregon.
- **Shallow-focus Crustal Earthquakes:** Typically are located within the upper 12 miles of the continental crust and could be generated by contortion of the overriding North American plate beneath the project area. The largest known crustal earthquake in the Pacific Northwest is the 1872 North Cascades quake at Magnitude 7.4. Other examples include the 1993 Magnitude 5.6 Scotts Mill earthquake and 1993 Magnitude 6 Klamath Falls earthquake.

The contribution of earthquake hazards from various seismogenic sources was analyzed using the Probabilistic Seismic Hazard Deaggregation results at the WTP (Latitude = 45.386 and Longitude = -122.632) from the USGS website. In the analysis, the MCE is considered to have a 2-percent probability of exceedance in 50 years (return period of 2,475 years) as inferred in the 2010 OSSC. Table 4 shows the relative hazard contributions from the CSZ and shallow crustal seismogenic sources, which constitute the primary earthquake hazards at the site.

TABLE 4: EARTHQUAKE HAZARD CONTRIBUTION

Return period (years)	Exceedance Probability	Spectral Acceleration Period	CSZ Megathrust EQ	Shallow Crustal EQ
2475	2 %	PGA	14 %	86 %
		0.2 sec	16 %	84 %
		1 sec	47 %	53 %

As shown in Table 4, the local shallow earthquake contributes the highest seismic hazard at the site for the 2-percent probability of exceedance in 50 years MCE event.

3.5.2 Time History Selection

We searched publically-available ground motion databases for previously recorded earthquake motions that generally matched the design PGA and shape of the code-based response spectrum. Eight acceleration time histories, five shallow crustal events, and three subduction zone events were selected and scaled to the bedrock PGA from the USGS PSHA using a single-scale factor. The selected input ground motion records and their scaling factors are presented in Table 5.

TABLE 5: EARTHQUAKE TIME HISTORY SELECTION

Record Name	Type	Earthquake	Date	Station	Comp.	Mag	Dist (km)	Duration (sec)	Scale Factor	Length (sec)
A-KRE090.AT3	Crustal	Whittier Narrows	Oct. 1, 1987	Pasadena Kresge Lab	090°	5.99	17.3(1)	3.39	4.05	38.895
A-KRE360.AT3	Crustal	Whittier Narrows	Oct. 1, 1987	Pasadena Kresge Lab	360°	5.99	17.3(1)	6.18	5.29	38.895
GIL067.AT3	Crustal	Loma Prieta	Oct. 18, 1989	Gilroy Gavilan Coll.	067°	6.93	10(1)	5.00	0.99	39.955
L04111.AT3	Crustal	San Fernando	Feb. 9, 1971	Lake Hughes #4	111°	6.61	25.1(1)	12.71	2.27	36.890
SIL090.AT3	Crustal	N. Palm Springs	Jul. 8, 1986	Silent Vall Poppet F	090°	6.06	17(1)	6.99	3.80	24.000
apa090c.acc	Sub.	Michoacan, Mexico	Sept. 19, 1985	Apatzingan	090°	8.1	100(3)	53.04	4.75	88.365
frn090c.acc	Sub.	Central Chile	Mar. 3, 1985	San Fernando, Chile	E-W	7.8	180(4)	26.42	0.98	81.220
hua090c.acc	Sub.	Central Chile	Mar. 3, 1985	Hualane, Chile	E-W	7.8	205(4)	33.69	2.62	78.240

Notes

1) Info found on the Peer Database and the PEER NGA Flat File

2) Info found on the COSMOS website http://db.cosmos-eq.org

3) No info on the station can be found; therefore distance measured from epicenter of earthquake to city.

4) Distance measure from station location to earthquake epicenter using Google Earth. Station information found on the COSMOS website.

3.5.3 Site Response Analysis

The scaled time histories were then used to conduct a non-linear 1-D soil column response analysis to determine a design PGA (at the ground surface) and site-specific response spectra that is based on the nonlinear effective stress site-specific site response of the project location. This analysis was completed using the fully non-linear code, D-MOD2000. D-MOD2000 computes the dynamic response of a layered soil profile to vertically propagating shear waves using a non-linear stress-strain model.

Along with the response spectra development, excess pore water generation and dissipation is explicitly modeled for the soils below the water table. Pore water pressure (PWP) development with depth was calculated and expressed as the ratio of the pore water pressure to the initial vertical effective stress. As the pore water pressure approaches the initial vertical effective stress and the corresponding pore water pressure ratio (r_u) approaches 1.0, the soil is considered to be liquefied. By calculating the r_u depth profile, the potential depth of liquefaction of the site soils can be estimated when subjected to the expected ground motions.

3.5.4 Soil Model

The soil model for the analyses was developed based on measured and estimated site-specific soil properties using both empirical and theoretical parameters. The primary source of information is the shear wave velocity measurements, but other important information — including standard penetration testing, visual classification and laboratory index testing of soils — was used to develop the model.

3.6 Site-Specific Evaluation Results

The PGA determined using the nonlinear effective stress model is 0.20g, indicating that ground motions are not expected to amplify appreciably through the soil that overlies the gravel base layer. The site-specific analysis represents an approximately 20-percent reduction from the PGA determined using code-based site response. The average input ground motion spectrum, individual time history ground surface response, and the average ground surface response spectrum is shown in Figure 6, Predicted Ground Surface Response Spectrum.

Figure 7, Average Effective Stress Spectra Amplification Ratio, provides the spectral amplification ratio, i.e. ratio of input spectral acceleration to ground surface acceleration, for the project site. The site-specific response spectrum is determined by multiplying the average input time history response by the average SAR at each period.

Based on the ASCE 7-05, the site-specific 5 percent damped response spectra can be used to design site structures if the spectra calculated are greater than or equal to 80 percent of the code-based response spectra for Site Class E. The recommended site specific spectra, as well as the appropriate code-based spectra, are presented on Figure 8, Average Ground Surface Site Response Spectra. Table 6 summarizes the site-specific and the code-based spectrum response acceleration parameters.

TABLE 6: SITE-SPECIFIC AND CODE-BASED SEISMIC PARAMETERS

Seismic Parameters	Site-Specific	Code-Based
Design PGA	0.20	0.24
S_{DS}	0.62	0.61
S_{D1}	0.40	0.59

3.7 Seismic Site Hazards

3.7.1 Fault Rupture Hazard

According to a review of the available literature, there are no currently mapped faults within the specific project area; also, it appears that all currently mapped faults are 3 Km or farther from the WTP. Based upon all of this information, we would classify the risk of fault surface rupture to be low. However, it should be noted that western Oregon is considered to be seismically active, and undetected faults may be present in the region, concealed by the overlying geology and vegetation canopy.

3.7.2 Liquefaction and Lateral Spread Hazard

Liquefaction involves the substantial loss of shear strength in saturated soil, usually taking place within a soil layer exhibiting a uniform granular characteristic, such as sands or silty sands, loose condition, and low confining pressure when subjected to impact by seismic or cyclic loading. Considering the subsurface conditions and the area seismicity, the site is considered to have a relatively high risk potential for soil liquefaction. The liquefaction-susceptible soils are the saturated very loose to medium-dense silty sand and very soft to medium-stiff silt in the Fine-Grained Flood Deposits, located below the groundwater levels.

To estimate the total liquefaction settlements at the ground surface, we conducted analyses using conventional simplified methods at each deep boring and CPT location. The analysis results are presented in Appendix D and are summarized in Table 7.

TABLE 7: ESTIMATED TOTAL SEISMIC LIQUEFACTION SETTLEMENT

Borings/CPT's	Total Liquefaction Settlement at ground surface (in) (rounded to nearest 0.5 in)
B-1	6.5
B-2	6
B-3	5
B-4	7
B-5	6.5
CPT-1	8
CPT-2	7.5
CPT-3	8.5
CPT-4	7
CPT-5	7
CPT-6	7.5
CPT-7	6

In the analyses, a CSZ type of earthquake with a Magnitude of 9.0 was selected to account for the high level of shaking required by the 2-percent in 50 year probability design earthquake, and a design PGA of 0.2g from the site-specific evaluation was used. It should be noted that we conducted the seismic liquefaction potential and settlement analyses based on the conventional simplified procedures and methods (Seed-Idriss and Robertson-Wride methods for liquefaction potential, and Tokimatsu-Seed and Zhang methods for liquefaction settlement). These procedures do not account for the yielding of soil and pore pressure redistribution effects at high levels of shaking induced by the design earthquake, and may over-predict the potential liquefaction settlement. Therefore, the predicted total liquefaction settlement may be somewhat conservative.

The total liquefaction settlement results in Table 7 are used to assess the differential liquefaction settlements for the proposed structures. In our assessment, we considered the effect of the relatively thick surficial non-liquefiable crust (about 25 feet thick above the liquefaction zone), and concluded that this site has a very low potential for large surface ruptures and sand boils. Based on this, we estimated the liquefaction differential settlements of the light, on-grade shallow structures by comparing the total liquefaction settlement estimates from borings/CPTs around or near the building corners. However, for the water-holding deep structure (Clearwell/FWPS), which is embedded into liquefaction zone (no crust), we anticipate relatively large potential differential settlement approaching the total liquefaction settlement. For the water-holding intermediate and shallow facilities (Ballasted Flocculation, Ozone Contactors, new Filters, WW Equalization Basin and WW Clarifier), because of the reduced crust thickness (approximately 10 to 12 feet thick) and heavy load, we anticipate that the liquefaction differential settlement may be on the order of 50 percent of total liquefaction settlement.

Detailed differential liquefaction settlement assessment results of the proposed structures are presented Table 8 (located at the end of the text, before figures), along with the static settlements (to be discussed in Section 4.2) and overall settlements.

Because the existing WTP is located approximately 1,000 feet from the Willamette River and that the ground at the WTP is relatively flat, the lateral spreading hazard is considered low.

3.7.3 Increase of Lateral Earth Pressures

Typically, seismically-induced lateral earth pressure under non-liquefiable conditions can be modeled using the Mononobe-Okabe method. However, for the deep structure extending below the groundwater table into the soil liquefaction zone (i.e. Clearwell/FWPS), the liquefied soil may need to be treated as a heavy viscous fluid, which could result in a higher lateral earth pressure load than the non-liquefiable condition. This aspect is discussed in more detail in the design section (Subsection 5.1.4) of this report. For the other embedded structures (Ballasted Floculation, Filters, WW Clarifiers, and Gravity Thickener), the seasonal groundwater table and the liquefaction zone are located below the embedded portions of the structures; therefore, the risk of increased lateral earth pressure is not an issue for these intermediate buried structures.

3.7.4 Potential Flotation Effect

Further, because of the dramatic increase in the pore water pressure associated with soil liquefaction, the Clearwell/FWPS will be subject to buoyancy forces which may generate a risk for flotation if there is not sufficient uplift resistance. The project structural engineer can evaluate this risk by treating the liquefied soil as a heavy viscous fluid with a unit weight of 100 pcf. In this case, the depth of the fluid can be taken as from the seasonal high groundwater level to the base of the structure. For the other structures, similar to the risk previously discussed, the flotation risk caused by soil liquefaction is not an issue.

3.7.5 Other Hazards

Due to the flat ground and the large distance/high elevation relative to the Willamette River, seismic slope stability, tsunamis, and seiche are not potential seismic hazards at this site. Ground motion amplification has been analyzed in the form of ground motion response spectrum and discussed previously in the site-specific ground motion analysis section.

4.0 KEY GEOTECHNICAL ISSUES AND MITIGATIONS

4.1 Identification of Key Geotechnical Issues

Based on the results of our field explorations, laboratory testing, and engineering analyses, we have identified the following key geotechnical issues for the development of the proposed new structures:

1. Excessive total and differential settlement from seismic soil liquefaction and static compression.
2. Seismic lateral earth pressure increase and potential flotation effect on the deep embedded structure (Clearwell/FWPS) caused by soil liquefaction.
3. Complex shoring arrangement to protect the existing and new structures during operation, due to the WTP Expansion Project being phased construction.

4.2 Structural Categories

For clarity, our discussions and mitigation strategies for the key geotechnical issues are grouped into five categories for the proposed site developments: (1) Water-Holding Deep Structure, (2) Water-Holding Intermediate and Shallow-Depth Structures, (3) On-Grade Buildings and Shallow/Intermediate Depth Ancillary Structures, (4) remaining existing structures, and (5) pipelines. The following is the list of structures and buildings in each category:

1. Water-Holding Deep Structure
   - Clearwell/FWPS
2. Water-Holding Intermediate and Shallow-Depth Structures
   - Ballasted Flocculation
   - Filters
   - WW Equalization Basin
   - WW Clarifiers
   - Ozone Contactors
3. On-Grade Buildings and Shallow/Intermediate Depth Ancillary Structures
   - Administration/Operations Building
   - Electrical Building
   - Chemical Building
   - Dewatering Building
   - Solids Thickener Tanks and Pump Station
4. Remaining Existing Structures
   - Existing Operations Building/Clearwell
   - Existing Backwash Lagoons #3 and #4
5. Pipelines and other Shallow Buried Utilities

- On-site Raw Water and Finished Water Conduits
- Yard Piping
- Utility Duct Banks

Geotechnical issue discussions and mitigation strategies for Categories 1 and 2 are presented in Sections 4.3.1.1 and 4.3.1.2. Categories 3, 4 and 5 are discussed in Sections 4.3.1.3 and 4.3.1.4.

4.3 Discussion of Key Geotechnical Issues and Mitigation

4.3.1 Issue No. 1: Differential Settlement and Mitigation

4.3.1.1 Category 1 Water-Holding Deep Structure

Based on the information provided by the MWH project structural engineer, we understand that the design criteria for both static and seismic differential settlements is ¼-inch per 40 feet for the deep water-holding Clearwell/FWPS.

As discussed in the previous section, the liquefaction differential settlement for the Clearwell/FWPS is estimated to be on the order of 5 inches in the short dimension of the structure (see Table 8). For the static settlements, due to the deep excavation and essentially no-net-loading increase, the total and differential static settlements are expected to be negligible. Therefore, the seismic differential settlement will govern, and dividing by the short dimension of the structure, we estimate 1.3 inches per 40 feet differential settlement for the Clearwell/FWPS (see Table 8) which exceeds the required structural differential settlement criteria.

As defined in Section 1803.5.12 of the OSSC 2010, three general design philosophies can be applied to mitigate the liquefaction hazard, which include: 1) ground improvement (e.g. stone columns and soil mixing), 2) foundation selection (e.g. shallow and deep foundation types), and 3) structural system selection (e.g. structural framing systems) to accommodate the liquefaction settlement impacts. At the current design stage, we understand that without mitigation, the Clearwell/FWPS cannot tolerate the anticipated differential settlement. Therefore, the mitigation strategy is to focus on selection of appropriate ground improvement and/or selection of appropriate foundation type and depth to mitigate the liquefaction differential settlement issue.

Table 9 presents an overall list of typical seismic liquefaction hazard mitigation techniques of ground improvement methods and foundation types; however, many of these techniques are not appropriate for the Clearwell/FWPS.

TABLE 9. SEISMIC LIQUEFACTION HAZARD MITIGATION TECHNIQUES

Ground Improvement Options	Foundation Type Options
Geo-Piers Vibro-Compaction Dynamic Compaction Compaction Grouting Vibro-Replacement (Stone-Columns) Soil Mixing Jet Grouting Earthquake Drains (EQD) Stone-Columns with EQD	Thick Mat Foundation Micro-Piles Driven Steel Pipe Piles Driven Steel H Piles Drilled Shafts Auger-Cast-Piles

Among the various ground improvement and deep foundation options, our preliminary short-list of the technically feasible options include ***Soil Mixing*** and ***Auger-Cast Piles*** which will satisfy the current preliminary design differential settlement criteria. If the preliminary differential settlement criteria can be relaxed somewhat, a third mitigation option of ***Stone Columns Combined with Earthquake Drains*** can be considered. Table 10 presents the advantages, disadvantages and the ranking of these technically feasible and possibly feasible options.

TABLE 10: DISCUSSIONS ON FEASIBLE LIQUEFACTION SETTLEMENT MITIGATION OPTIONS (CLEARWELL)

Options and Ranks	Advantages	Disadvantages	Rough Cost[1)]
Option 1: Auger-Cast-Piles	• deep foundation elements provide vertical and uplift supports to the structure • can achieve the design differential settlement criteria • very fast installation • widely used deep-foundation system in the region allowing completive bidding • low mobilization/demobilization costs for phased construction • potentially can be part of the shoring system for deep excavation adjacent to structures further reducing costs noise and vibration is not an issue	• would require slightly thicker slab/mat for structural connection to the structure • more expensive than stone columns with earthquake drains	Approximately $800,000
Option 2: Soil Mixing Columns	• provide reinforcement and containment to the soil matrix, reducing shear strain and liquefaction settlement • proven performance and reliability for silty soil liquefaction mitigation • can achieve the design differential settlement criteria • potentially can act as shoring system	• most expensive among feasible mitigation options • longer setup time; therefore, may require long construction period • require a thick (3 to 4 ft) reinforced crushed rock mat on top (between columns and base of the structure); therefore, increase the excavation depth • require specialty contractor outside of this	Approximately $1,800,000

Options and Ranks	Advantages	Disadvantages	Rough Cost[1)]
Option 2 (continuous)	for deep excavation adjacent to structures • noise and vibration is not an issue	region • with phased construction, mobilization/demobilization costs will be significant • need favorable soils for mixing	
Possible Option 3: Stone Columns with Earthquake Drains[2)]	• provide densification and drainage to the soil matrix preventing soil liquefaction • common ground improvement method • least expensive among feasible mitigation options	• limited densification effect on soils with fine content more than 15%; not effective in significantly reducing liquefaction settlement • best scenario can only reduce the differential liquefaction settlement to about ½ to 1-inch every 40 ft • in order to use, must require a relaxed differential settlement criteria • treatment area will need to be extended outside the footprint of the structure (typically 16 ft outside perimeter); therefore, increase the excavation volume • require a thick (3 to 4 ft) reinforced crushed rock mat on top (between columns and base of the structure); therefore , increase the excavation depth • noise and vibration during installation • require specialty contractor outside of this region • with phased construction, mobilization/demobilization costs will be significant	Part 1: Approximately $600,000 for stone columns, earthquake drains reinforced crushed rock mat Part 2: Approx. $200,000 for potential additional excavation for outside treatment (min 16 ft outside perimeter)

1) The rough cost estimates are only related to the ground improvement and foundation elements installation, and do not include the excavation, working/drainage mat, and groundwater control costs of Clearwell.

2) Only feasible if differential settlement criteria can be relaxed (increased allowable settlement)

Stone columns with earthquake drains, although is the least expensive option, will need a relaxed differential settlement criteria of ½ to 1-inch to be a viable solution as the liquefaction mitigation method for the Clearwell.

4.3.1.2 Category 2 Water-Holding Intermediate and Shallow-Depth Structures

We understand that the same differential settlement design criteria of the Clearwell/FWPS (1/4-inch per 40 feet) applies to the water-holding, intermediate and shallow structures. As shown in Table 8, the liquefaction differential settlements for the Ballasted Flocculation, Ozone Contactors, new Filters, WW Equalization Basin and WW Clarifier are estimated to be on the order of 2 to 4 inches in the short dimensions of the structures.

For the static settlement, we estimated less than 1 inch of total settlement and ½-inch of differential (also along the short dimensions of the structures) for most of these structures, except for the Ozone Contactors. This is due to the relatively high net loading pressure at the foundation level of the Ozone Contactors, which could generate over 1-inch of total settlement (mainly in the form of elastic settlement) and over ½-inch of differential settlement.

Combining the seismic and static differential settlement and divided by the short dimensions of the structures, we estimate 1.7 to 2.8 inches per 40 feet differential settlement for these main treatment water holding structures (see Table 8). These estimated seismic and static differential settlements, either considered separately or additive, exceed the required structural differential settlement criteria; therefore, we assume that without mitigation, the structure systems themselves cannot tolerate the anticipated differential settlements.

Similar to the Clearwell/FWPS, the considered mitigation strategies and options for these main treatment water-holding facilities include ground improvements and deep foundations as listed and discussed in Tables 9 and 10. However, the stone columns with earthquake drain option is assumed not feasible , due to the close proximity to the adjacent existing and newly constructed structures (constructed at different phases), and the vibration and associated potential settlement during and after installation. Therefore, the only two viable options are the auger-cast piles and soil mixing. Based on the discussions and preliminary ranking of these options, auger-cast piles appear to be the preferred mitigation method. Final refinements to this preliminary recommendation will be completed during detailed design phase.

4.3.1.3 Category 3 On-Grade Buildings and Shallow/Intermediate Depth Ancillary Structures

For the proposed on-grade buildings and shallow/intermediate depth ancillary structures, we estimate about 1.5 to 3.5 inches of combined differential settlement (seismic and static) in the short dimensions of the structures.

At the current near completion of predesign and beginning early design stage, the differential settlement criteria for these structures is not yet finalized, and the foundation design strategies and options have not been fully developed. In addition to the ground improvement and deep foundation options, using more rigid foundation systems (i.e. mat foundation) and appropriate structural systems to accommodate the liquefaction settlement impacts (defined in OSSC 2010) may be feasible. This issue will be evaluated in the upcoming detailed design phase.

As mentioned previously, the predicted liquefaction settlements may be conservative due to the conventional analysis procedure and method used. If it’s desired to refine the liquefaction potential and settlement analysis to evaluate the feasibility of other foundation options during detailed design, a more refined, high level analytical approach can be considered. Typically, this high level analysis is accomplished using advanced numerical modeling technique (i.e. FLAC analysis), which uses more recent hysteretic effective stress procedures and is a more reliable liquefaction assessment. Often, this type of advanced numerical modeling analysis yields less liquefaction settlement; however, these results are not guaranteed as they depend on the site subsurface conditions, earthquake shaking level, structural loading and other factors.

Coupled with potential less stringent differential settlement criteria, the numerical analysis results could lead to less mitigation effort for these on-grade structures and the ancillary treatment structures. The need for additional analysis, if any, will be determined during the detailed design phase.

4.3.1.4 Category 4 and 5 Facilities

Similar to the Category 3 structures, the differential settlement criteria for the existing structures to remain and the new and existing pipelines are not yet finalized. Therefore, the potential needs for foundation mitigation/design strategies and options have not been fully developed. These will be addressed later in the upcoming detailed design phase.

Specifically, for the existing Operations Building, the future use of the structure has not been determined;the critical level for operation and seismic mitigation will be decided during detailed design. If mitigation is needed for the existing Operations Building, micro-piles will be feasible option since the mitigation will require working in tight space inside the building.

4.3.2 Issue No. 2: Seismic Lateral Earth Pressure Increase and Potential Flotation Effect due to Liquefaction

This issue is only related to the deep embedded Clearwell/FWPS extending into the liquefiable zone (below the groundwater table). The lateral earth pressure increase will depend on the backfill materials used for the Clearwell excavation and is more significant for the selected native backfill option than the imported crushed rock backfill option. As discussed previously, the flotation potential can be assessed by treating the liquefied soil as a heavy viscous fluid with a unit weight of 100 pcf. More discussion about this issue is presented in Subsections 5.1.3 and 5.1.4.

4.3.3 Issue No. 3: Phase Construction Complex Shoring Requirements

For this third geotechnical issue, we understand that the project will be constructed in three phases to ensure no disruption to the existing WTP operations. During each phase, structural excavation and shoring protection will have to be carefully selected with a type that protects the existing and newly constructed structures/facilities and keeps them fully functioning.

The first phase involves the demolition of the existing Lime building, demolishing the Lagoon No. 1 & 2, and construction of the Ballasted Flocculation, part of the Chemical Building, Clearwell/FWPS, and the Electrical building. Due to the close vicinity of the Ballasted Flocculation excavation to the existing facilities (filters to the north, sedimentation basins to the west, and utility trenches nearby), shoring protection is needed to the north and west sides of the excavation. Considering the deep groundwater and the low plasticity nature of the Fine-Grained soil unit, a cantilevered, closely-spaced tangent pile wall or soldier pile and lagging wall can be used as the shoring system. These piles can potentially be used as permanent foundation elements of the adjacent structures to be constructed in later phases. Additionally, the partial Chemical Building construction will be constructed simultaneously with the Ballasted Flocculation excavation, which will a require cantilevered, closely-spaced tangent pile wall or soldier pile and lagging wall as the excavation shoring and foundation support system. For the Clearwell/FWPS and Electrical Building, we assume that the Clearwell/FWPS will be constructed and backfilled before the Electrical Building. Also, the Electrical Building may be moved to west outside the limits of the Clearwell excavation. In this case, the Clearwell excavation can be conducted with a temporary cut slope ranging from 1 horizontal to 1 vertical (1H:1V) to 1.5H:1V. For any of these cut slopes, the slope surfaces should be protected with a crushed-rock slope protection layer. The groundwater seepage into the excavation can be controlled by installing excavation drainage collection systems at the bottom of the excavations and sumping the collected water for treatment, then to an approved discharge location. More detailed discussion and recommendations are presented in Section 6.2. We also recommend to first construct and backfill the Clearwell/FWPS, then to construct the on-grade Electrical Building to save shoring on the west side.

The second phase involves the demolition of the existing sedimentation basins and construction of the new Filters, WW Equalization Basin, WW Clarifier and new Administration Building. Due to the similar depth of new Filters to the newly constructed Ballasted Flocculation and the existing Clearwell (below the existing Operations Building), the only shoring requirement will be at the east side of the excavation against the existing filters, though all existing structures will need to be protected. Similar to Phase 1 construction, we recommend a cantilevered, closely-spaced tangent pile wall or soldier pile and lagging wall as the excavation shoring system and potential future foundation support system. We also recommend to first construct and backfill the Filter, WW Equalization Basin and WW Clarifier excavation, and then to construct the on-grade Administration building to save shoring on the north side, if possible.

The third phase involves the demolition of the existing filters and construction of the new Ozone Contactors, the rest of the Chemical Building, Dewatering Building, and the Gravity Thickeners. These are shallow (7 feet +/- of bury) or on-grade structures, and shoring systems are not anticipated.

5.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

5.1 Water-Holding Deep Structure (Clearwell/FWPS)

The proposed new Clearwell/FWPS will be located on the Mapleton Property south of the existing WTP. The new Clearwell/FWPS will be a rectangular-shaped structure, with a footprint of approximately 150 feet by 195 feet, and a depth of approximately 35 feet below existing ground surface (bgs). The deeper portion of the structure is for the FWPS located at the northern 30 feet of the structure. In general, the Clearwell will be a fully buried structure, except for the FWPS, which will have a one-story above-grade portion to house the pumping equipments.

5.1.1 Foundation System

As discussed in Section 4.3, to mitigate the excessive liquefaction differential settlement, we preliminarily recommend that the Clearwell/FWPS be supported by auger-cast piles; final refinements to this recommendation will occur during the upcoming detailed design phase. An auger-cast pile is constructed by drilling down to the prescribed bearing stratum with a hollow-stem, continuous-flight auger. The auger is left in place to support the walls of the borehole. A high-strength grout mix is then pumped through the hollow stem under pressure while the auger is slowly withdrawn from the hole. Care is required to coordinate the rate of grout placement with the rate of auger withdrawal to prevent the sides of the hole from sloughing in and necking, thereby reducing the pile cross section area. Immediately after grout placement, a rebar cage is lowered into the grouted shaft. This type of pile requires installation by an experienced and competent foundation contractor, as well as full-time construction observation and QA/QC documentation under the supervision of an experienced geotechnical engineer to assure satisfactory installation.

We recommend 18-inch or 24-inch-diameter auger-cast piles to optimize the pile length and provide a pile diameter conducive to installation of the reinforcing cage through the in-place grout. Due to the high risk potential of liquefaction under the considered earthquake event, the seismic loading requires that axial compressive, uplift, and lateral bearing resistances for the proposed structure will have to be derived from the non-liquefiable Older Sand and Gravel Alluvium. Therefore, we recommend that the pile should be extended to a tip elevation of El. 60 feet for 18-inch-diameter piles or El. 65 feet for 24-inch-diameter piles. Assuming a base mat thickness of 2 feet, these recommended tip elevations correspond to approximate pile lengths of 37 and 42 feet, depending on pile diameter. Table 11 shows the allowable axial compressive and uplift load capacities of the pile.

TABLE 11: ALLOWABLE LOAD CAPACITIES FOR CLEARWELL/FWPS AUGER-CAST PILES

Pile Type and Diameter (inches)	Allowable Axial Compressive Capacity (kips)	Allowable Uplift Capacity (kips)
18”-diameter Auger-Cast Pile (Tip EL 60 feet)	190	150
24”-diameter Auger-Cast Pile (Tip EL 65 feet)	220	180

The allowable compressive and uplift capacities have a factor of safety (FS) of 3.0 under static loading condition and a FS of 2.0 under seismic loading condition. The seismic downdrag load, which is mainly due to the settlement of the non-liquefiable crust during the seismic event, is not considered for the Clearwell/FWPS because the piles are below the non-liquefiable crust.

We expect that the static and seismic compressive loads on the piles will be resisted through a combination of end-bearing and skin friction, but largely through skin friction in the Older Sand and Gravel Alluvium below 55 feet. Minor pile settlement will result from the proposed structural loads. Based upon our experience and engineering analyses, we anticipate that the maximum static or seismic total and differential settlements for the auger-cast pile should be less than ½-inch and ¼-inch (over the short dimension of the structure), respectively.

The lateral capacities of the recommended auger-cast piles were calculated using the computer program LPILE. Lateral loads imposed by seismic forces are resisted primarily by the stiffness of the soil adjacent to the pile shafts. The lateral capacity of a pile depends on its length, stiffness in the direction of loading, proximity to other piles, and degree of fixity at the head of the piles (at bottom of pile cap), as well as the engineering properties in the soil, especially within the upper portion of the pile. We assumed that the lateral capacity of the piles would be controlled by the seismic loading. The analysis results are presented in Table 12 for pile free-headed and fixed-headed conditions with 1-inch pile head deflection.

TABLE 12: LATERAL LOAD INFORMATION FOR AUGER-CAST PILES (1-INCH DEFLECTION) FOR CLEARWELL/FWPS

Pile Type and Diameter (inches)	Loading Condition	Unfactored Lateral Resistance (kips)	Maximum Bending Moment (in-kips)	Depth of Maximum Moment (feet)	Depth to Points of Fixity (feet)
18”-diameter Auger-Cast Pile	Free	18	850	9	30
	Fixed	32	2200	0	30
24”-diameter Auger-Cast Pile	Free	26	1700	12	32
	Fixed	50	4400	0	35

Note: Maximum moment depth zero means maximum moment is at top (head) of pile. The point of fixity is defined as near zero pile lateral deflection.

The lateral resistance values presented above are unfactored. The structural design engineer should apply an appropriate factor of safety. The horizontal deflection criteria in the analysis is 1 inch at the pile head.

The above-mentioned values for compressive, uplift, and lateral capacity refer to single piles unaffected by group interactions. To reduce or eliminate group effects, we recommend that the pile spacing never be less than three pile diameters center-to-center. If piles are at least three diameters apart, group effects can be neglected for compressive and uplift. However, for lateral loads, group effects reduce the lateral load capacity of the pile at a pile spacing of less than five diameters. If the pile spacing is less than five times the pile diameters, the following pile group reduction factors should be applied to the above unfactored pile lateral capacities.

TABLE 13: REDUCTION FACTORS FOR IN-LINE LATERALLY LOADED PILES

Pile Spacing	In-line Load Reduction Factor
5 pile widths	1.0
3 pile widths	0.75

Note: Widths are measured center-to-center of the piles.

We anticipate that the lateral load resistance of the piles will be limited by the cross-sectional ratio of the reinforcing steel cage to grout; therefore, the actual pile lateral load capacity used may be different from the values provided above. Additionally, the project structural engineer should verify that the piles have sufficient internal strength to accommodate the lateral loads, and determine the depth of the reinforcing cage. We assume a steel cage with close spiral will be used as reinforcement and a single bar will be installed full-depth of the pile. These preliminary recommendations will be further refined during detailed design.

5.1.2 Seismic Performance

Due to the support from auger-cast pile foundation, the structure will not be affected by seismic soil liquefaction and should essentially have no seismic total and differential settlement.

5.1.3 Uplift Protection

If the new Clearwell/FWPS will be affected by the liquefaction-induced flotation issue, or it is to be protected from uplift forces during periods of high groundwater level when the basin is partially or entirely empty, mitigation strategies may include increasing structure weight, using pile tension capacity, or extending the foundation outside the structure to mobilize the soil weight above (vertical projection) of the extended foundation.

For the option of utilizing additional soil weight above the extended foundation, 130 pcf for crushed rock backfill and 120 pcf for selected native backfill can be used for soils above the design groundwater level. For soils below the groundwater level, buoyant values of 68 pcf for crushed rock and 58 pcf for selected native materials can be used.

For the option of using a pile foundation, tension capacities of the auger-cast pile foundation provided in Table 11 can be used in the design. We understand that by using the structural weight, additional soil weights above the foundation and tension capacities of auger-cast piles combination, foundation underdrains for these structures will not be needed.

5.1.4 Lateral Earth Pressures on Embedded Walls

The lateral earth pressures on the embedded walls were evaluated as equivalent fluid pressures. In the analysis, we assume that the embedded walls will be designed as non-yielding walls under static loading conditions and will have a level backfill surface. Further, we assume two cases for the planned wall backfill material:

- Case 1 – Lateral earth pressure from imported crushed rock backfill, and
- Case 2 – Lateral earth pressure from the select native sandy silt to silt soils.

5.1.4.1 Case 1: Earth Pressure Distribution (Crushed Rock Backfill)

Case 1 is for the open-cut excavation with a safe side slope (as discussed in Section 6.5) and backfilled with crushed rock. In this case, the behavior and properties of the imported crushed rock backfill will govern the determination of the lateral earth pressure on the embedded walls. Table 14 presents the recommended lateral earth pressure values, as equivalent fluid pressures, for the crushed rock backfill.

TABLE 14: LATERAL EARTH PRESSURES FOR IMPORTED CRUSHED ROCK BACKFILL (CASE 1)

Groundwater Design Conditions	Static At-rest Pressure (psf)	Static Active Pressure (psf)	Surcharge At-Rest Pressure (psf)	Seismic Pressure (psf)	Hydrostatic Pressure (psf)
Above Water Level	$55H_s$	$35H_s$	0.4q	15H	
Below Water Level	$55H_s+25H_w$	$35H_s+15H_w$	0.4q	15H	$62H_w$

In Table 11, H is defined as the total height of the buried wall. H_s is defined as the portion of the buried wall height above the project design groundwater level. H_w is defined as the groundwater height above the bottom of the buried wall, and q is the surcharge load with q in units of pounds per square foot.

For the static lateral earth pressures, we recommend at-rest earth pressure be used in the design for the non-yielding wall. For the seismic loading condition, the seismic pressure was analyzed using Mononobe-Okabe method with the full design PGA of 0.20g (from the site-specific evaluation). This pressure can be applied in an inverted triangular distribution, and is additive to the static soil and water pressures. The resultant seismic load acts at a point above the bottom of the structure that is about 0.6 times the height of the wall.

The distribution and resultant of the backfill, groundwater, and seismic loading are shown in Figure 9.

5.1.4.2 Case 2: Earth Pressure Distribution (Select Native Backfill)

Case 2 is considered for the scenarios using on-site select sandy silt to silt soils as backfill materials, and thin crushed rock backfill with thickness on the order of 3 to 5 feet. This condition can exist where shoring walls minimize backfill, and in these conditions the strength of temporary shoring left in place is typically ignored. The lateral earth pressure on the embedded wall will essentially be governed by the behavior of the silty soils, especially during earthquake conditions and post-liquefaction conditions as discussed below. Table 15 presents the recommended lateral earth pressure values, as equivalent fluid pressures, for this backfill scenario. Variables in this table are the same as defined in Section 5.1.4.1.

TABLE 15: LATERAL EARTH PRESSURES FOR SELECT NATIVE SOIL BACKFILL (CASE 2)

Groundwater Design Conditions	Static At-rest Pressure (psf)	Static Active Pressure (psf)	Surcharge At-Rest Pressure (psf)	Seismic Pressure (psf)	Hydrostatic Pressure (psf)
Above Water Level	$60H_s$	$40H_s$	0.4q	$20H_s$	
Below Water Level	$60H_s+30H_w$	$40H_s+20H_w$	0.4q	$120H_s+100H_w$	$62H_w$*

* Only used as a component in the static condition.

For Case 2, the static lateral earth pressure components can be incorporated into the design following a similar approach as in Case 1. However, for the designed seismic loading condition in which the liquefaction of the selected native soil backfill is anticipated, we recommend a different design approach for the liquefied soil. In this approach, the liquefied soil is treated as a heavy viscous fluid exerting a hydrostatic pressure on the wall, and the unsaturated soil above the liquefied soil is treated as a surcharge that increases the fluid pressure within the underlying liquid soil. Therefore, the seismic lateral earth pressure on the walls will consist of four components: the static at-rest earth pressure above the groundwater level, static surcharge pressure above the ground surface (if any), the seismic earth pressure above the groundwater level, and seismic pressure of the liquefied soil below the groundwater level (including the pressure increase from the overlying non-liquefiable soil layer). The distribution and resultants of these lateral pressures are shown on Figure 10.

5.1.5 Lateral Load Resistance

Lateral resistances can be provided by passive resistance around the embedded structure and base, and by pile lateral capacity. The pile lateral capacities are presented in the previous section (Subsection 5.2.1). For passive resistance under static loading condition, a partial passive equivalent fluid pressure of 250H is recommended for Case 1 backfill conditions. A pressure of 200H should be used for Case 2 backfill conditions. Partial passive pressure is also recommended, as the large amounts of wall movement that would be necessary to mobilize full passive resistance will likely be considered unacceptable for structural design.

5.1.6 Construction Considerations

We anticipate that the excavation for the Clearwell/FWPS will be approximately 35 feet deep to accommodate the thickness of the foundation slab and the excavation drainage/working pad below. With a properly installed slope drainage protection layer, we recommend that the temporary excavation slope can be cut at slopes ranging from 1.5H to 1H : 1V slope or flatter. For the groundwater control, we recommend a 2-feet-thick crushed rock excavation drainage/working pad with perimeter drainage pipe/trench at the bottom of the excavation, and continuous pumping from engineered sumps located along the perimeter pipe and inside the crushed rock layer. Detailed recommendations for the slope drainage protection layer and excavation drainage/working pad are presented in Sections 6.1 and 6.2.

For the auger-cast pile foundation, the piles should be installed within a tolerance of 3 inches of the locations shown on the plans. The completed piles should be plumb to within 2 percent from vertical. We also recommend that the pile construction specification and construction procedures should follow most recent edition of "Augered Cast-in-Place Piles Manual," developed by Deep Foundation Institute (DFI). Further, we recommend the full-time inspection of the pile foundation installation by a qualified geotechnical field representative of Shannon & Wilson.

5.2 Water-Holding Intermediate and Shallow-Depth Structures

As stated in Section 4.2 the following structures are in the category of Water-Holding Intermediate and Shallow-Depth Structures: Ozone Contactors, Ballasted Flocculation, Filters, WW Equalization Basin, and WW Clarifiers. The new Ozone Contactors will be located immediately south of the existing Operations building, at the location of the current filter structure location (to be demolished). The base of this structure will be approximately 5 feet bgs and the top of the structure approximately 14 feet above the ground surface. The footprint will be approximately 70 feet by 75 feet.

The new Ballasted Flocculation structure will be located south of the Ozone Contactors. The Ballasted Flocculation is a half-buried structure. The base of the building will be at approximately 13 feet bgs with the top of the building approximately 13 feet above the ground surface. The footprint will be approximately 70 feet by 95 feet.

The new Filters will be located immediately west of the Ozone Contactors and the Ballasted Flocculation structure, and will be at the area currently occupied by the existing sedimentation basins (to be demolished). Similar to the Ballasted Flocculation structure, it will also be a partially buried structure with a base depth at about 13 feet bgs and a top height of 12 feet above the ground surface. The footprint will be approximately 105 feet by 130 feet.

The WW Equalization Basin and the WW Clarifier will be located west of the proposed Filters. These structures are also buried structures. The base of the WW Equalization is 22 feet bgs, and the WW Clarifier is 13 feet bgs. The footprints will be approximately 55 feet by 90 feet for the WW Equalization Basin, and 30 feet by 35 feet for the WW Clarifier.

5.2.1 Foundation System

Similar to the Clearwell/FWPS and as discussed in Section 4.3, auger-cast piles are preliminarily recommended to mitigate the excessive liquefaction differential settlement for these structures; refinements to these recommendations will occur during detailed design. These piles should extend either 10 feet (for the 24-inch-diameter pile) or 15 feet (for the 18-inch-diameter pile) into the Older Sand and Gravel Alluvium. The recommended tip elevation for these piles is El. 60 feet for 18-inch-diameter piles or El. 65 feet for 24-inch-diameter piles. Assuming a base mat thickness of 2 feet, the approximate pile lengths for each structure in this category are shown in Table 16.

TABLE 16: APPROXIMATE PILE LENGTHS FOR AUGER-CAST PILES FOR THE WATER-HOLDING INTERMEDIATE AND SHALLOW-DEPTH STRUCTURES

Structure	Approximate Pile Length for 18" diameter Auger Cast Pile (feet)	Approximate Pile Length for 24" diameter Auger Cast Pile (feet)
Ozone Contactors	63	58
Ballasted Flocculation	57	52
Filters	57	52
WW Equalization Basin	52	47
WW Clarifier	52	47

Table 17 shows the allowable axial compressive and uplift load capacities of the pile for the water-holding intermediate and shallow structures, and Table 18 shows the LPILE lateral loading results for these structures.

TABLE 17: ALLOWABLE LOAD CAPACITIES FOR INTERMEDIATE- AND SHALLOW-DEPTH WATER-HOLDING STRUCTURES' AUGER-CAST PILES

Pile Type and Diameter (inches)	Allowable Axial Compressive Capacity (kips)	Allowable Uplift Capacity (kips)
18" diameter Auger-Cast Pile (Tip EL 60 feet)	140	150
24" diameter Auger-Cast Pile (Tip EL 65 feet)	180	180

The discussion and assumptions made about axial and lateral capacities and pile settlement are presented in Section 5.1.1. A condition different to the pile capacities in Section 5.1.1 is that downdrag load under the seismic case is considered for this section, due to the top of these piles being in the non-liquefiable crust. The estimated downdrag loads for 18 inch and 24 inch diameter piles is 60 and 80 kips; respectively. These downdrag loads were treated as reduction to the pile compressive capacities. In Table 17, the allowable compressive capacities have a FS of 1.5 under seismic loading condition after the downdrag load reduction, and have a FS of 4.5 under static loading condition without the downdrag load reduction. The project structural engineer should add the downdrag load in the pile structural evaluation to verify that the piles have sufficient internal strength to accommodate this additional load.

The lateral resistance values are presented in Table 18 – note that these values are unfactored. The structural design engineer should apply an appropriate factor of safety. The discussions about group effects (axial and lateral) and reinforcing steel recommendations are the same as presented in Section 5.1.1.

TABLE 18: LATERAL LOAD INFORMATION FOR AUGER-CAST PILES (1-INCH DEFLECTION) FOR INTERMEDIATE-DEPTH WATER-HOLDING STRUCTURES

Pile Type and Diameter (inches)	Loading Condition	Unfactored Lateral Resistance (kips)	Maximum Bending Moment (in-kips)	Depth of Maximum Moment (feet)	Depth to Points of Fixity (feet)
18-inch diameter Auger Cast Pile	Free	35	1400	7	30
	Fixed	65	3500	0	32
24-inch diameter Auger Cast Pile	Free	55	2600	8	35
	Fixed	95	6400	0	40

Note: Maximum moment depth zero means maximum moment is at top (head) of pile. The point of fixity is defined as near zero pile lateral deflection.

5.2.2 Lateral Earth Pressures on Embedded Walls

These structures should be designed as non-yielding walls by utilizing at-rest earth pressures in the design, and we have assumed two cases for the planned wall backfill material:

- Case 1 **–** Lateral earth pressure from imported crushed rock backfill, and
- Case 2 **–** Lateral earth pressure from the select native sandy silt to silt soils.

5.2.2.1 Case 1: Earth Pressure Distribution (Crushed Rock Backfill)

Due to the similarity of surface and subsurface condition between the Clearwell/FWPS and the Water-Holding Intermediate structures, lateral earth pressures presented in Section 5.1.4.1 are recommended.

5.2.2.2 Case 2: Earth Pressure Distribution (Select Native Backfill)

Case 2 is similar to the recommendations presented in Section 5.1.4.2, but due to the seasonal groundwater level being below the bottom of these foundations, there is no increase of pressures due to the liquefaction. Table 19 presents the recommended lateral earth pressure values, as equivalent fluid pressures, for this backfill scenario. Variables in this table are the same as defined in Section 5.1.4.1.

TABLE 19: LATERAL EARTH PRESSURES FOR SELECT NATIVE SOIL BACKFILL (CASE 2)

Ground Water Design Conditions	Static At-rest Pressure (psf)	Static Active Pressure (psf)	Surcharge At-Rest Pressure (psf)	Seismic Pressure (psf)	Hydrostatic Pressure (psf)
Above Water Level	$60H_s$	$40H_s$	0.4q	$20H_s$	
Below Water Level	$60H_s+30H_w$	$40H_s+20H_w$	0.4q	$20H_s$	$62H_w$

For Case 2, the static lateral earth pressure components can be incorporated into the design following the approach in Section 5.1.4.1.

5.2.3 Seismic Performance, Uplift Protection, and Lateral Load Resistance

Due to the similarity of surface and subsurface conditions between the Clearwell/FWPS and the Water Holding Intermediate/Shallow structures, the seismic performance presented in Section 5.1.2, uplift protection in Section 5.1.3, and lateral load resistance values presented in Sections 5.1.5 can be used to design the Water-Holding Intermediate/Shallow Structures.

5.2.4 Construction Considerations

We anticipate that the excavations for these intermediate and shallow water-holding structures will range from 7 feet (Ozone Contactors) to 25 feet (WW Equalization Basin), but with majority on the order of 15 feet deep to accommodate the thickness of the foundation slab and the excavation working pad below. We understand that open-cut excavation is the preferred construction method; however, due to the space restriction and phased construction (see Section 4.2.3), shoring systems will be needed at portions of the excavation to protect the adjacent existing and newly constructed structures. For the shoring systems, we recommend a closely-spaced tangent pile wall or soldier pile and lagging wall, which can also be used as future foundation support system.

For the open-cut excavation, with properly installed slope protection system and excavation working pad, a cut slope of 1.5H to 1H:1V or flatter can be used. More discussion and recommendations about the slope protection and excavation working pad are presented in Sections 6.1 and 6.2.

5.3 On-Grade Buildings and Shallow/Intermediate Depth Ancillary Structures

As stated previously, the following buildings are in this category: Administration/Operations, Electrical/Generator/Maintenance, Chemical, Dewatering Buildings, and Solids Thickener Tanks and Pump Station.

As the only two-story building in the plant expansion, the new Administration/Operations building (Admin Building) will be constructed to the west of the existing Operations Building. The new Admin Building will be a rectangular, on-grade structure, with a footprint of approximately 45 feet by 100 feet.

Also planned on the Mapleton Property, the new Electrical/Generator/Maintenance Building (Electrical Building) will be located just west of the Clearwell/FWPS. The building will be an on-grade, one-story building, with a footprint of approximately 55 feet by 100 feet.

Also in the central portion of the WTP, the new Chemical building will be located just east of the Ozone Contactors and the Ballasted Flocculation structure. This building will be an on-grade, one-story structure, with a footprint of approximately 60 feet by 155 feet.

At the east portion of the existing WTP, the new Dewatering building and the Gravity Thickeners and Pump Station will be located at the existing Backwash Lagoons #1 and 2 areas. The Dewatering building is an on-grade, one-story building with an approximate footprint of 70 feet by 70 feet. The Gravity Thickeners are two cylindrical tanks that have a diameter of approximately 40 feet and will be partially buried to a depth of approximately 13 feet bgs.

5.3.1 Foundation System Discussions

As discussed in Section 4.3, at the current early design stage, the differential settlement criteria for these new structures has yet to be finalized. Therefore, the foundation design strategies and detailed foundation recommendations have not been fully developed. These items will be addressed in the upcoming detailed design phase.

5.4 Remaining Existing Structures

For the existing Operations Building, the future use of the structure has not been determined, and the critical level for operation and seismic mitigation has not been decided. Additionally, the tolerable differential settlement criteria are not yet finalized, and the foundation mitigation/design strategies and options have not been fully developed. These will be addressed in the upcoming design stages.

5.5 Pipeline Design Parameters

5.5.1 Modulus of Soil Reaction for Flexible Pipe

The modulus of soil reaction, E', for flexible pipeline design characterizes the stiffness of the backfill placed around buried flexible pipelines. E' is an empirical parameter (Spangler's Iowa formula) that is dependent on the deflection and the pressure developed at the springline of the pipe. Variables also depend on the depth of the pipe, the type and density of the backfill, and the thickness of compacted pipe zone backfill between the pipe and the trench wall. An E' value of 1,500 psi is recommended for a pipe zone consisting of compacted crushed rock. If non-crushed granular materials, such as select native materials of silty sand and sandy silt, are used in the pipe zone, and if the pipe diameter is greater than 4 feet, typically the native soils in the trench wall may control deflection. In this case, we recommend an E' value of 1,000 psi.

5.5.2 Pipeline Thrust Resistance

For the proposed pipelines, thrust force will be developed at the angle points of the pipelines. Depending on the required resistance, the thrust force may be resisted by: (a) restrained joints along the pipe, (b) frictional forces between pipe and surrounding backfill, and (c) by soil lateral bearing pressure using a thrust block.

The frictional resistance will be determined by the shearing strength between the pipe surface and the backfill material, which we assume will be either well-graded crushed rock (1½-inch or ¾-inch minus), or if CLSM is used, between the CLSM and the native soil. Recommended coefficients of friction for these two scenarios are 0.35 for the steel pipe and crushed rock fill and 0.5 for the CLSM and native soils.

For lateral resistance using thrust block, an allowable lateral bearing capacity of 1,500 psf can be used for the thrust block design.

5.5.3 Utility Duct Bank Recommendations

We understand that on-site utilities (i.e. electric lines) will be installed in utility duct banks. We assume the duct banks will be shallow concrete box structures. Providing the subgrade preparation/acceptance and fill placement recommendations discussed in Sections 6.1.2 and 6.1.4 are incorporated into the design and construction, duct bank foundations can be designed for an allowable soil bearing pressure of 1,500 psf, based on dead load plus design live load. For short-term transient loads, this bearing pressure can be increased by one-third.

6.0 SITE EARTHWORKS RECOMMENDATIONS

6.1 General Earthwork

6.1.1 General

Construction of the proposed new facilities for this WTP Expansion project will involve a range of geotechnical-related considerations that may affect the construction sequence and approach. This section provides an assessment of some issues we have noted and recommendations for design team consideration. We assume that our comments and recommendations are provided as an initial assessment of the issues and that further evaluations will occur during final preparation of construction technical specifications and drawing details. It should be noted that the Contractor's construction approach, including means, methods, and sequencing of construction elements, as well as the responsibility for site safety, remain with the Contractor. By providing our opinions on the construction issues, Shannon & Wilson does not assume responsibility for design and construction issues that belong to the Contractor.

6.1.2 Site Preparation

All areas to be excavated, filled, or intended to perform as a subgrade should be stripped. Prior to stripping and excavation, utilities should be located and rerouted as necessary, and any abandoned pipes or utility conduits should be removed or stabilized in a manner that does not adversely affect performance of new facilities.

Due to the moisture-sensitive nature of the silty soil on-site, all stripping and excavations should be performed using a smooth-bladed tracked excavator working from areas where material has yet to be removed. Stripping and excavation should remove surficial organic soil (sod and topsoil), trees/roots, asphalt pavement and base rock, and any loose/soft materials as determined by a qualified geotechnical engineering representative. Subgrade areas should be cleanly cut to firm, undisturbed soil.

6.1.2.1 Subgrade Verification and Acceptance

Typically, on-site relative compaction of the subgrades should be based on the Modified Proctor test method. However, because much of the exposed subgrade will contain high-moisture, fine-grained silty soils, the use of the Proctor test method to establish level of compaction may not be appropriate. In this case, proof-rolling with approved equipment and number of passes, as discussed below, should be considered as an alternate method of performing subgrade proof-testing.

Prior to placement of structural fill, roadway fill, and base course, the subgrade should be proof-rolled with a self-propelled compaction equipment weighing at least 8 tons (dead weight). The approved equipment should make a sufficient number of passes to obtain complete coverage of the subgrade. Any areas that pump, weave, appear soft, have deflection of more than ¼-inch, or are judged to be problematic should be removed by overexcavation and backfilled with imported crushed rock or select native materials and compacted to structural fill standards discussed below. The actual amount of soft or disturbed material to be excavated should be determined in the field and observed and approved by a geotechnical engineering representative. The specifications should include a unit cost bid item for any overexcavation and subgrade stabilization with additional thickness of backfilled materials. If significant time passes between completion of subgrade preparation and commencement of other construction activities, or if significant traffic has been routed across the site, the site should be similarly proofrolled before placement of fill, base rock, or paving. A geotechnical engineering representative should observe all the subgrades prior to placing geotextile (fabric), fill, or foundation materials.

6.1.3 Segregation and Stockpiling Materials

In the areas of the proposed intermediate and deep excavations, the excavated materials will generally include top soil, pavement or gravel sections, concrete foundation debris, and fill or native clayey silt, sandy silt and silty sand materials. We recommend segregating and appropriately stockpiling the sandy silt, silty sand, and gravels (including appropriately crushed concrete) for future use as the “select native soil” (select earth) backfill. The select native soil materials should be free of deleterious materials such as organic soils, woody debris, and rocks with a diameter greater than 6 inches. Unsuitable material for backfill and other engineering purposes includes topsoil, wet native silt/clayey silt, demolished asphalt pavement materials, and existing fill soils containing deleterious construction debris of wood/organic pieces, PVC, etc. (leftover from the demolition of the previous structures). These materials will likely need to be hauled to an offsite disposal area.

6.1.4 Fill Materials and Placement

We understand that different fill materials will be utilized for the construction of this project. Shannon & Wilson anticipates that all fill materials and their specific locations and placement criteria will be fully described in the construction plan and specifications. The following sections describe general fill criteria that are subject to modification under specific design recommendations and the construction plans and specifications.

6.1.4.1 Compaction Standard

We recommend that the compaction standard for this site should be the Modified Proctor, either AASHTO T-180 or ASTM D1557. Therefore, compaction requirements will reference to this Standard.

6.1.4.2 General Structural Fill Materials and Compaction

Generally, we recommend that imported crushed rock be used beneath any structure, pipeline and pavement, behind buried walls that were designed to withstand Case 1 lateral earth pressures, using drained backfill, or under settlement sensitive areas. We recommend that the crushed-rock materials used beneath structures and pavement (not requiring high permeability, such as the foundation drainage/working pads), should be clean, fractured on at least two faces, well-graded, 1½-inch minus material with less than 5 percent by weight passing the No. 200 wet sieve.

In areas where the final backfill surface can tolerate settlement, and behind walls that are designed to withstand undrained lateral earth, water pressure loading (no drainage), and Case 2 lateral earth pressures, select native soils may be used as backfill. Moisture conditioning may be required before the on-site soils are suitable for placement as select native fill.

Generally, the structural fill should be compacted to a minimum 92 percent according to ASTM D1557. The structural fill materials should be compacted within the range of +-2 percent the optimum moisture content value. If water must be added, it should be uniformly applied and thoroughly mixed into the soil by disking or scarifying. Each lift of compacted engineered fill should be tested by a qualified representative of a qualified testing agency prior to placement of subsequent lifts.

The fill should extend horizontally outward beyond the exterior perimeter of the building a distance equal to the height of the fill or 5 feet, whichever is greater, prior to sloping. Also, fill should extend horizontally outward from the exterior perimeter of the pavement a distance equal to the height of the fill or 3 feet; whichever is greater, prior to sloping.

6.1.4.3 Excavation Drainage/Working Pad Placement and Compaction

For the intermediate and deep structures, we recommend over-excavation of at least 2 feet below the bottoms of the structures and backfill with clean crushed rock to form excavation drainage/working pads. The pad should consist of a layer of non-woven geotextile filter fabric (such as Mirafi 140N) installed directly on the prepared subgrade, and clean crushed rock (open-graded ¼ to 1½-inch gradation) placed on top of the geotextile, and potential perimeter drain pipes for the excavation extending below groundwater level (see recommendation in Section 6.2).

Excavation to subgrade level and placement of filter fabric and crushed rock shall be done in stages so that the exposed subgrade is covered as soon as possible after exposure. The filter fabric shall be placed on the subgrade before placement of the crushed rock. If the subgrade becomes disturbed during excavation, the disturbed areas should be overexcavated, and the filter fabric shall be placed on the final subgrade surface. The overlap at the edges and ends of fabric rolls shall be a minimum of 2 feet.

Above the filter fabric, the crushed rock should be placed and compacted in two lifts. The first lift should be 16 inches thick in order to support spreading and compaction equipment and not overstress the subgrade soil. Compaction of the first lift shall be conducted with a roller in the static mode only to a minimum 90 percent compaction, according to ASTM D1557. The second lift shall be placed with a maximum 10-inch uncompacted thickness. Compaction of the second lift shall be in static mode for at least two passes and then in vibratory mode, and the top 8 inches compacted to at least 92 percent compaction according to ASTM D1557. Each lift of compacted fill should be tested by a qualified representative of a qualified testing agency prior to placement of subsequent lifts.

6.1.4.4 Embedded Wall Backfill Placement and Compaction

Considerations should be incorporated into the design of the intermediate and deep buried structures to withstand or relieve the hydrostatic pressure on the embedded walls. On-site "select native soil," discussed in the previous section, may be suitable for use as structural backfill in non-settlement sensitive areas if the soils can be properly moisture-conditioned and adequately compacted. Due to their generally fine-grained nature, these on-site soils will likely be useable only in dry weather conditions, and only if the stockpiles have been properly sloped and covered with plastic membranes to protect against moisture increase. If excavated on-site soil cannot be used, we recommend using imported granular fill material; however we recommend not using sand or sandy materials.

Backfilling methods and compaction equipment near the embedded walls should be controlled to eliminate over-compaction and equipment surcharging against the walls. We recommend that backfill materials within a 5-foot zone behind the embedded wall should be compacted to 90 percent of the modified proctor maximum dry density (ASTM D 1557). Beyond this zone, the backfill should be compacted to not less than 92 percent of the modified Proctor maximum dry density. The select native soil materials should be placed in maximum loose lifts of 8 inches and compacted within the range of +-2 percent of the optimum moisture content value. Each lift of compacted backfill should be tested by a representative of a qualified testing agency.

For select native soil backfill, even after proper compaction, some settlement will likely occur over time. Therefore, it is recommended that the site development within the backfill areas be delayed for at least a year to allow the potential settlement to occur and re-grading to be done.

Structure backfill material used in settlement-sensitive areas or when placed during wet weather should consist of free-draining imported crushed rock. The imported crushed rock should be maximum 1½-inch particle size and contain less than 5 percent passing the No. 200 sieve. Unless otherwise noted, crushed rock structural backfill should be compacted to maximum 92 percent of ASTM D 1557. Care should be taken to not overcompact, which would induce much higher lateral earth pressure values than the values recommended in Section 5.1. The crushed rock structural fill should be placed in maximum lifts of 10 inches of loose material. Each lift of compacted engineered fill should be tested by a qualified representative of a qualified testing agency.

6.2 Excavation and Groundwater Control

All excavations should be completed in accordance with applicable OSHA and state regulations. While we have described certain approaches for excavations in the foregoing discussions, the contractor should be responsible for selecting the excavation and groundwater control methods, monitoring the cut slopes and the trench excavations for safety, and providing shoring, as required, to protect personnel and adjacent improvements.

6.2.1 Open Excavation and Groundwater Control

We understand that open excavation methods will be used for the excavation and construction of the Clearwell/FWPS. We also understand that open excavation is the preferred construction approach for the rest of the on-site excavations (Ballasted Flocculation, Filters, WW Equalization Basin and WW Clarifier).

For the Clearwell/FWPS excavation, with the base of the structure about 35 feet below the existing grade, the excavation subgrade will be about 10 feet below the groundwater level.

Based on soil conditions and our past experiences, we anticipate that groundwater control can be accomplished with a properly installed slope drainage protection layer, a clean crushed-rock layer (excavation drainage layer) at the excavation bottom, perimeter drainage collection ditches with perforated pipe at the edges of the crushed-rock layer, and continuous pumping from engineered sumps as the excavation proceeds below the groundwater (interim sumping) and at the final subgrade elevation at locations along the perimeter pipe and inside the crushed-rock layer. The crushed-rock layer will also provide support for the construction equipments of auger-cast pile installation; therefore, to act as a working pad. The construction for this excavation drainage/working pad discussed in details in the previous section (Subsection 6.1.4.3).

The cut slope drainage protection layer should intersect the perched water zones, and as a minimum, start from at least 3 feet above the groundwater level and continue down along the slope to the bottom crushed rock layer. This layer is needed in order to prevent the cut slope from severe erosion during adverse weather conditions and to convey any seepage and perched water to the crushed-rock drainage layer. The material for the slope protection layer should consist of 18 inches (thickness perpendicular to the slope) of clean, free drained, crushed rock, similar to the foundation working/drainage pad at the bottom of the excavation. Between the protection layer and the native cut slope, geotextile should not be placed because it would create a low-friction layer, likely causing instability. Above this crushed rock protection layer, plastic sheeting and an acceptable system to secure the plastic should be used as cover for the remaining cut slope to prevent surface erosion and the drying and wetting of the slope.

The perimeter trench drains at the edges of the excavation should be constructed using the same type of crushed rock and non-woven geotextile, with a perforated drain pipe to collect the groundwater, perched water, and seepage from the slope and at the bottom. The trench drains should be placed at least 6 inches below the bottom of the crushed rock mat and outside the foundation pressure zone, which is the area within a 0.5H:1V pressure distribution boundary line extending down from the edge of the foundation.

With the slope protection and groundwater control measures discussed above, the cut slope for the open excavation should be constructed at a slope no steeper than 1H:1V. If localized material of low strength is encountered, slope will need to be flattened.

For the intermediate deep structures, the anticipated excavation depths will be in the order of 15 to 24 feet. These depths are generally above the anticipated groundwater levels; therefore, the crushed rock slope protection system and the perimeter trench drains may not be needed. However, the 2-foot thick drainage/working pad should still be used, as described in Section 6.1.4.3, to support auger-cast pile installation. Other slope protection features for above the groundwater discussed above should be implemented to protect the cut slope. Drainage control of incidental perched groundwater/surface water seepages would still require the placement of crushed rock drainage protection at localized areas. With these requirements, a 1.5 to 1H:1V cut slope or flatter can be used for the excavation of this structures.

6.2.2 Temporary Shoring

Temporary shoring and other measures (i.e. underpinning) necessary to protect excavations and existing facilities should be the responsibility of the contractor. The following paragraphs mainly serve as constructability and feasibility discussion.

As discussed in Section 4.3.3, due to the phased construction approach, multiple shoring systems will likely be implemented to protect the existing and newly constructed structures/facilities and keep them fully functioning.

Considering the utilization of deep foundations (auger-cast piles) on-site, the temporary shoring system can consist of closely-spaced tangent piles and soldier piles with laggings.

In addition, some other shoring systems may be needed for the excavation of new pipelines/conduits or vault structures near existing sensitive facilities. For these smaller excavations, we recommend a positively restrained shoring system (i.e. sliderails), which can provide lateral restraint and pressure to the excavation sidewalls to maintain the stability and movement.

6.3 Pipeline Installation

We recommend that pipeline trenches be backfilled, in the pipe zone, with imported crushed rock material, which allows for the bedding material to be worked under the curvature of the pipe and compaction in wet weather. We believe that on-site excavated materials are not suitable for bedding or pipe zone backfill. The bedding material for the piping should consist of well-graded granular material such as ¾-inch minus crushed aggregate. The recommended minimum thickness of granular bedding below the invert of the pipes is 6 inches.

It should be necessary to stabilize and provide drainage to the base of pipeline trenches if groundwater or perched water is present and soil at subgrade elevation is the fine-grained materials. For these conditions, we recommend overexcavation below the bedding material, and placing a 12-inch-thick layer of 1½ -inch minus crushed rock, underlain with a layer of non-woven geotextile, directly on top of the subgrade. The crushed rock should contain no more than 2 percent fines (material passing the standard U.S. No. 200 Sieve). The crushed rock should be installed in one lift and compacted with an excavator until well keyed-in. Pipe bedding should be placed above this layer.

After installing the pipe on the bedding, imported crushed rock material should be used for the pipe zone, which typically extends at least 12 inches above the top of the pipe, or as set out by the City standards. Pipe zone compaction should be 90 percent of ASTM D698. Above the pipe zone, the trench should be backfilled with structural fill.

Trench backfill placed above the pipe zone to within 2 feet below subgrade elevation should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D 698. If the area above trench is to be paved, trench backfill should be crushed rock and placed in 8-inch loose lifts to the pavement subgrade elevation and compacted to at least 95 percent of the maximum dry density in the top 3 feet of the backfill.

7.0 LIMITATIONS

The preliminary analysis, conclusions, and recommendations contained in this report are based on site conditions as they currently exist. We have assumed that the explorations are representative of the subsurface conditions at the site of the proposed improvements and that subsurface conditions everywhere are not significantly different from those disclosed by the explorations. Within the limitations of the scope, schedule and budget, the preliminary analyses, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted professional geotechnical engineering principles and practice in this area at the time this report was prepared. We make no warranty, either express or implied. Our conclusions and recommendations are based on our understanding of the project as described in this report and the site conditions as interpreted from the explorations.

If, later in the final design phase, new or additional subsurface information indicates that conditions different from those encountered in the field explorations are or appear to be present, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary. If there is substantial lapse of time between the submission of this and the final design report, or if conditions have changed because of natural forces or construction operations at or adjacent to the site, we recommend that this report be reviewed to determine the applicability of the conclusions and recommendations concerning the changed conditions and/or the time lapse.

This preliminary report was prepared for the exclusive use of MWH Americas, Inc. in support of the Land Use Application process; final refinements to the analysis and recommendations will be presented in the final Geotechnical Engineering Report to be included as part if the Building Permit Process. It is a preliminary finding report, interpretive in content, and should not be made available to prospective bidders, contractors and/or subcontractor as a base for bidding. This report is not a warranty of subsurface conditions, such as those interpreted from the exploration logs and presented in the discussions of the subsurface conditions included in this report.

The scope of our geotechnical services did not include any environmental assessment or evaluation regarding the presence or absence of hazardous or toxic materials in the soil, surface water, groundwater, or air, on or below the site, or for evaluation of disposal of contaminated soils or groundwater, should any be encountered, except as noted in this report.

SHANNON & WILSON, INC.

REGISTERED PROFESSIONAL
ENGINEER
78866PE
OREGON
12-31-07
YUXIN LANG

EXP 12-31-2012

Yuxin (Wolfe) Lang, PE, GE
Associate Geotechnical Engineer

Jerry L. Jacksha, PE, GE
Senior Associate Geotechnical Engineer
Principal-in-Charge

YWL:JLJ/rrb/amn

8.0 REFERENCES

American Society of Civil Engineers, 2006, Minimum design loads for buildings and other structures: Reston, Va. American Society of Civil Engineers, ASCE Standard ASCE/SEI 7-05.

Barnett, E.A., Weaver, C.S., Meagher, K.L., Wang, Z., Madin, I.P., Wang, M., Haugerud, R.A., Wells, R.E., Ballantyne, D.B., Darienzo, M., and the Southern Willamette Valley Working Group, 2004, Lifelines and Earthquake Hazards along the Interstate 5 Urban Corridor: Cottage Grove to Woodburn, Oregon: U.S. Geological Survey Open-File Report 2004-1052.

Beeson, M.H., and Tolan, T.L., and Madin, I.P., 1989, Geologic Map of the Lake Oswego Quadrangle, Clackamas, Multnomah, and Washington Counties, Oregon: Oregon Department of Geology and Mineral Industries, Geological Map Series GMS-59, scale 1:24,000.

Cetin, K. O.; Seed, R. B.; Der Kiureghian, Armen; and others, 2004, Standard penetration test-based probabilistic and deterministic assessment of seismic soil liquefaction potential: Journal of Geotechnical and Geoenvironmental Engineering, v. 130, no. 12, p. 1314-1340.

CH2M Hill, Inc., August 1975, Geotechnical Investigation for City of Lake Oswego Water Treatment Plant.

City of West Linn, Natural Hazards Mitigation Plan, Prepared for City of West Linn, in cooperation with Clackamas County.

Consortium of Organizations for Strong-Motion Observation Systems, 2007, Earthquakes Database: Available: http://db.cosmos-eq.org/scripts/earthquakes.plx

Frankel et al., 2002, Documentation for the 2002 update of the national seismic hazard maps, USGS Open-File Report 02-420.

GeoMotions LLC., September 2011, D-MOD2000, version 8.9.0: http://www.geomotions.com

International Code Council, May 2009, Internal Building Code.

Oregon Department of Consumer and Business Services & The International Code Council, February 2010, 2010 Oregon Structural Specialty Code.

Pacific Earthquake Engineering Research Center, 2005, NGA Database: Available: http://peer.berkeley.edu/nga/flatfile.html.

Personius, S.F., compiler, 2002, Fault number 715, Beaverton fault zone, in Quaternary fault and fold database of the United States: U.S. Geological Survey website, http://earthquakes.usgs.gov/regional/qfaults, accessed 11/04/2011 11:53 AM.

Personius, S.F., compiler, 2002, Fault number 716, Canby-Molalla fault, in Quaternary fault and fold database of the United States: U.S. Geological Survey website, http://earthquakes.usgs.gov/regional/qfaults, accessed 11/04/2011 11:54 AM.

Personius, S.F., compiler, 2002, Fault number 875, Oatfield fault, in Quaternary fault and fold database of the United States: U.S. Geological Survey website, http://earthquakes.usgs.gov/regional/qfaults, accessed 11/04/2011 11:55 AM.

Personius, S.F., compiler, 2002, Fault number 876, East Bank fault, in Quaternary fault and fold database of the United States: U.S. Geological Survey website, http://earthquakes.usgs.gov/regional/qfaults, accessed 11/04/2011 11:56 AM.

Personius, S.F., compiler, 2002, Fault number 877, Portland Hills fault, in Quaternary fault and fold database of the United States: U.S. Geological Survey website, http://earthquakes.usgs.gov/regional/qfaults, accessed 11/04/2011 11:55 AM.

Personius, S.F., compiler, 2002, Fault number 878, Grant Butte fault, in Quaternary fault and fold database of the United States: U.S. Geological Survey website, http://earthquakes.usgs.gov/regional/qfaults, accessed 11/04/2011 11:56 AM.

Robertson, P. K. and Wride, C. E., 1997, Cyclic liquefaction and its evaluation based on SPT and CPT, in Youd, T. L., and Idriss, I. M., eds., Proceeding of the NCEER workshop on evaluation of liquefaction resistance of soils: Buffalo, N. Y., National Center for Earthquake Engineering Research, Technical Report no. NCEER-97-0022.

Tokimatsu, Kohji and Seed, H. B., 1987, Evaluation of settlements in sands due to earthquake shaking: Journal of Geotechnical Engineering, v. 113, no. 8, p. 861-878.

United States Geological Survey, 2002 Interactive Deaggregations: https://geohazards.usgs.gov /deaggint/2002/

Wells, R.E., Blakely, R.J., Simpson, R.W., Weaver, C.S., Haugerud, R., and Wheeler, K., 2000, Tectonic plate motions, crustal blocks, and shallow earthquakes in Cascadia, http://geomaps.wr.usgs.gov/pacnw/rescasp1.html, accessed 8/23/2008, the information provided on this page was originally presented as a poster at the Great Cascadia Earthquake Tri-centennial Open House, January 26, 2000, Burke Museum, University of Washington, Seattle WA.

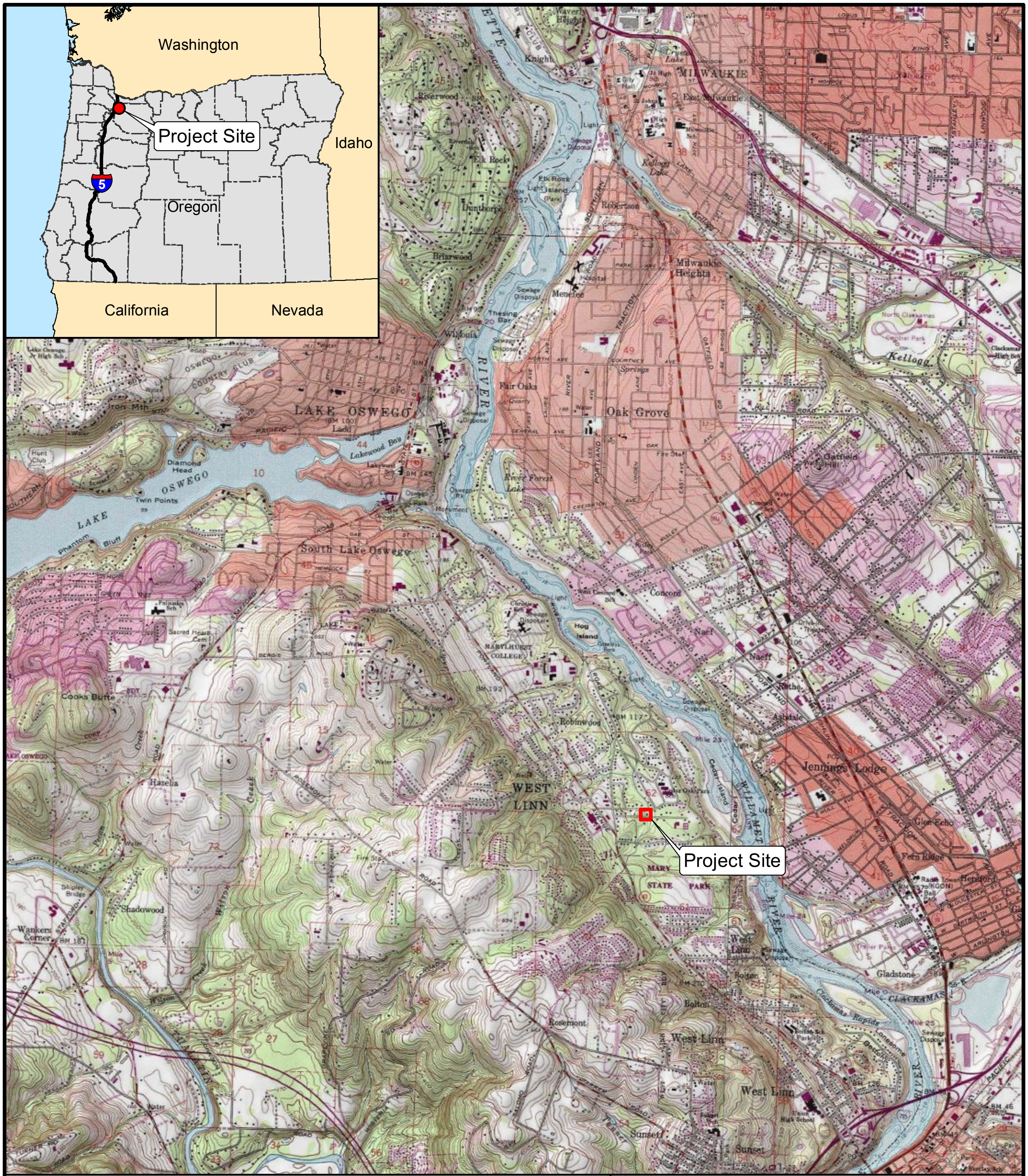
Zhang, G.; Robertson, P. K.; and Brachman, R. W. I., 2002, Estimating liquefaction-induced ground settlements from CPT for level ground: Canadian Geotechnical Journal, v. 39, no. 5, p. 1168-1180.

Table 8: Estimated Static and Seismic Total and Differential Settlement

Category	Structures[1]	Nearby Borings/CPT's	Estimated Static Settlement			Estimated Seismic Settlement			Static + Seismic Settlement		
			Total Settlement (in)	Differential Settlement (in)	Differential Settlement across Short Dimension (in/40 ft or in/30 ft)	Total Liquefaction Settlement below foundation (in)	Differential Liquefaction Settlement (in)	Differential Settlement across Short Dimension (in/40 ft or in/30 ft)	Total Settlement (in)	Differential (Static + Seismic) Settlement (in)	Differential Settlement across Short Dimension (in/40 ft or in/30 ft)
Water Holding Deep Structure	**Clearwell/FWPS[2]**	B-4; B-5; CPT-6 and CPT-7	--	--	--	5 to 6	5[4]	1.3/40 ft	5 to 6	5	1.3/40 ft
Water Holding Intermediate/Shallow Structures	**Ballasted Floc.**	B-3; CPT-3, CPT-5	0.7	0.3	0.4/40 ft	5 to 9	4[5]	2.4/40 ft	6.7 to 8.7	4.3	2.8/40 ft
	Filters	B-1, B-2; CPT-1, CPT-2, CPT-3	0.2	0.1	0.1/40 ft	6 to 9	4[5]	1.6/40 ft	7.25 to 9.2	4.1	1.7/40 ft
	WW EQ Basin[2]	B-2, CPT-1	--	--	--	6.0 to 8	3[5]	2.2/40 ft	6.0 to 7	3	2.2/40 ft
	WW Clarifiers[2]	B-2, CPT-2	--	--	--	6.0 to 8	3[5]	3.3/40 ft	6.0 to 7	3	3.3/40 ft
	Ozone Contactors	B-1, B-3, CPT-4	1.5	0.7	0.8/40 ft	5 to 7	2[6]	1.2/40 ft	7 to 9	2.7	2.0/40 ft
On Grade Buildings and Shallow and Intermediate Ancillary Structures	**Electrical Bldg.**	B-4, CPT-3, CPT-7	1[3]	0.5	0.3//30 ft	7 to 9	2[6]	1.1/30 ft	8 to 10	2.5	1.4/30 ft
	Admin Bldg.	B-1, CPT-1	1[3]	0.5	0.3/30 ft	7 to 8	1[6]	0.7/30 ft	8 to 9	1.5	1.0/30 ft
	Chemical Bldg	B-3, CPT-5, CPT-4	1[3]	0.5	0.2/30 ft	5 to 7	2[6]	1.0/30 ft	7 to 9	2.5	1.2/30 ft
	Dewatering Bldg	All Borings and CPTs	1[3]	0.5	0.2/30 ft	5 to 9	3[6,7]	1.3/30 ft	7 to 10	3.5	1.5/30 ft
	Solids Thickener Tanks/PS	All Borings and CPTs	1[3]	0.5	0.3/30 ft	5 to 9	3[6,7]	2.0/30 ft	7 to 10	3.5	2.3/30 ft

Notes

1. See site exploration plan for Borings/CPTs locations and proposed structure locations.
2. Essentially no net load increase at the base levels; therefore, the static settlement will be minimal
3. The total settlement assumes that the loose fill has been over excavated and replaced with compacted crushed rock.
4. The base of the Clearwell is about 35' deep and is into the upper portion of the liquefaction zone. Large liquefaction differential settlement potentially equal to the total liquefaction settlement may occur.
5. The intermediate depth structures bases will only be approximately 10' to 15' above the liquefaction zone. Differential settlements are taken as 50% of the total liquefaction settlement.
6. Differential liquefaction settlement was estimated from the difference between total settlements at the Borings/CPTs near the corners of the structures.
7. Total and differential liquefaction settlements for these buildings were taken as the average of the total and differential settlements for the nearby deep borings.



DRAFT

N
W E
S

0 4,000 8,000
Feet

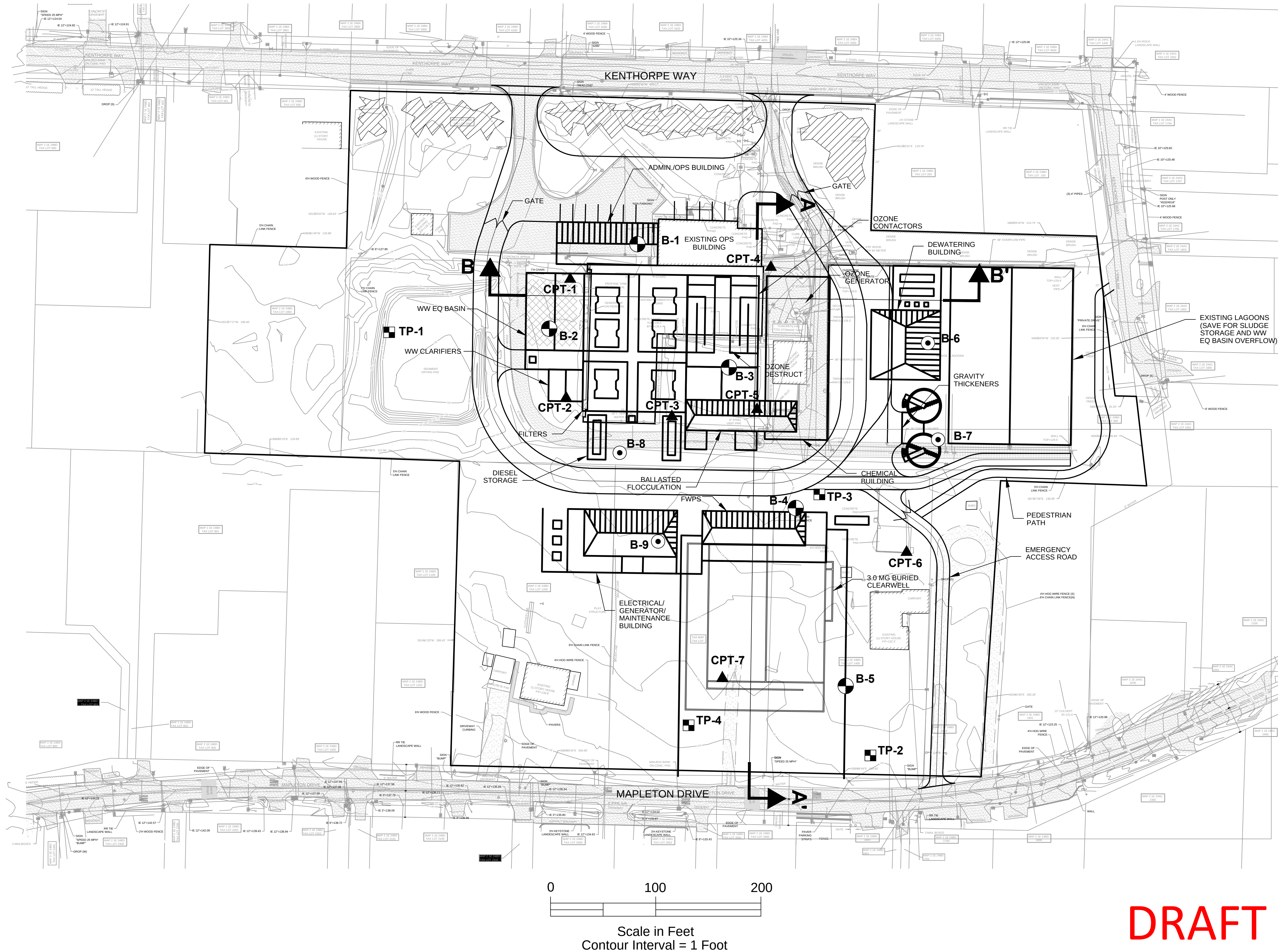
Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

VICINITY MAP

January 2012 24-1-03658-001

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 1



DRAFT

LEGEND

- **B-1** Deep Boring (60') Designation and Approximate Location
- **B-6** Shallow Boring (20' to 30') Designation and Approximate Location
- **CPT-1** CPT Designation and Approximate Location
- **TP-1** Test Pit Designation and Approximate Location
- **A** Generalized Subsurface Profile (See Figures 3 and 4)

NOTE

Figure adapted from client file, *"45817-RWTP.dwg"*, received 9-22-11.

Lake Oswego & Tigard
Water Treatment Plant Expansion
West Linn, Oregon

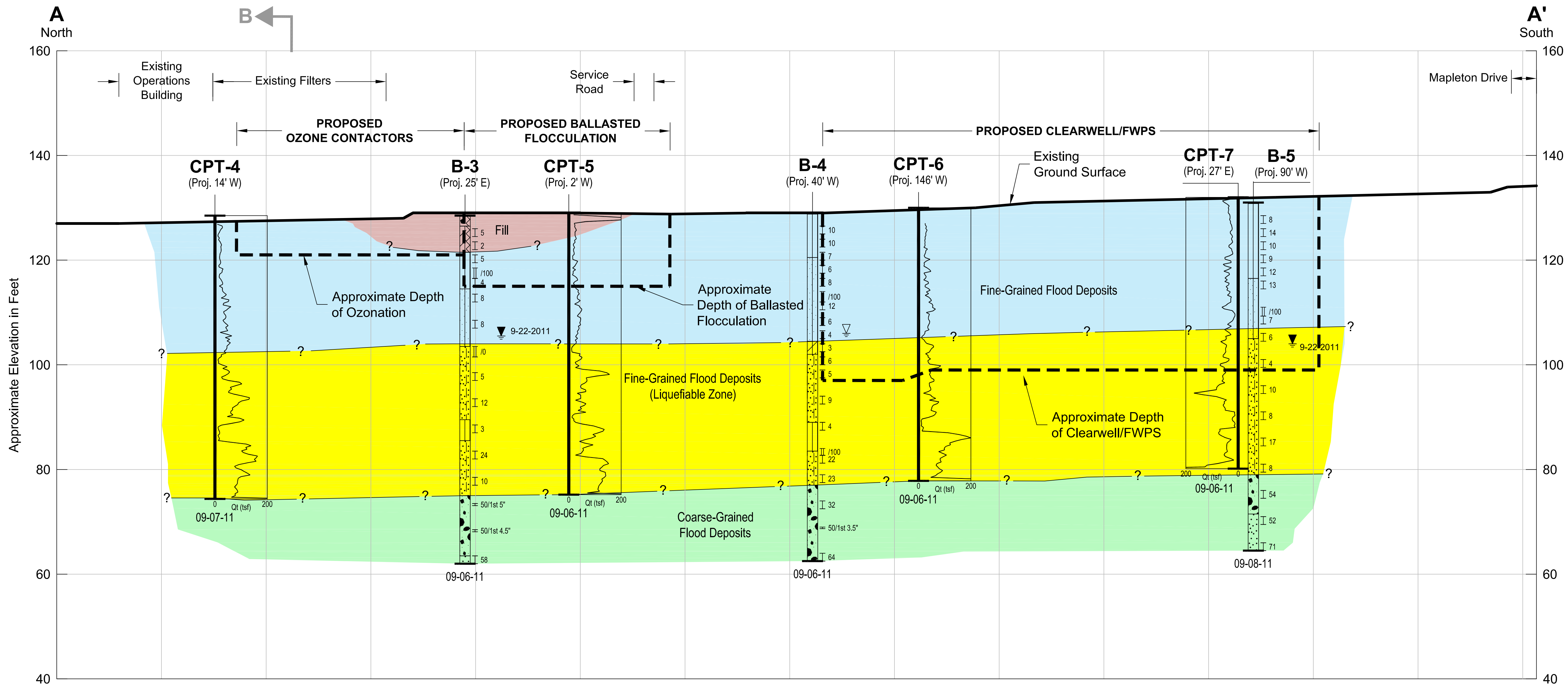
SITE AND EXPLORATION PLAN

January 2012 — 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 2

Filename: I:\WP\Projects\24-1 Portland\3600\3658 Lake Oswego Water Treatment Plant\Graphics\CAD\From Seattle Drafting\24-1-03658-245 Site Plan.dwg Date: 12-27-2011 Login: rrb



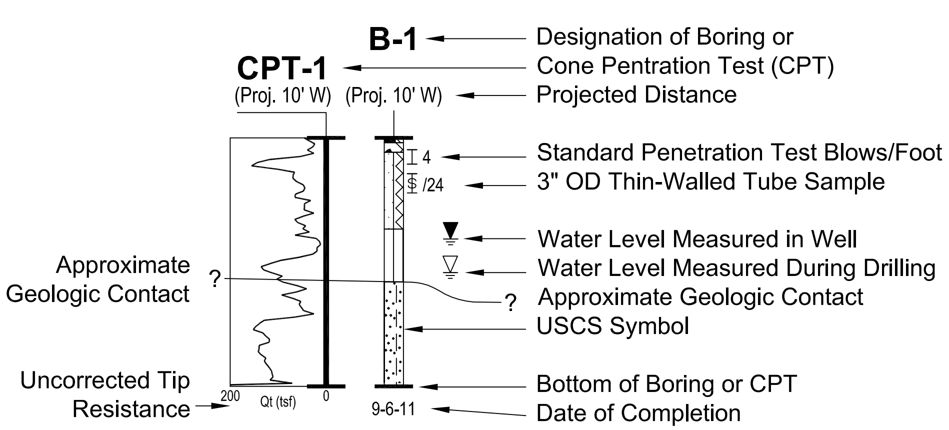
0 20 40
Vertical Scale in Feet

0 40 80
Horizontal Scale in Feet

Vertical = Horizontal

DRAFT

LEGEND



SUBSURFACE MATERIAL LEGEND

Artificial Fill: Sandy silty gravel and clayey silt to sandy silt.

Fine-Grained Flood Deposits: Clayey silt, sandy silt, and silty sand to sand.

Fine-Grained Flood Deposits: Clayey silt, sandy silt, and silty sand to sand. (Liquefiable zone below Groundwater)

Older Sand & Gravel Alluvium: Sandy gravel to gravelly sand, and sand.

NOTE

1. This subsurface profile is generalized from materials observed in soil borings. Variations may exist between profile and actual conditions.
2. The upper boundary of the liquefiable zone is the estimated seasonal high groundwater table at approximately 25 feet below ground surface. The groundwater was measured between 23 to 28 feet below the ground surface.

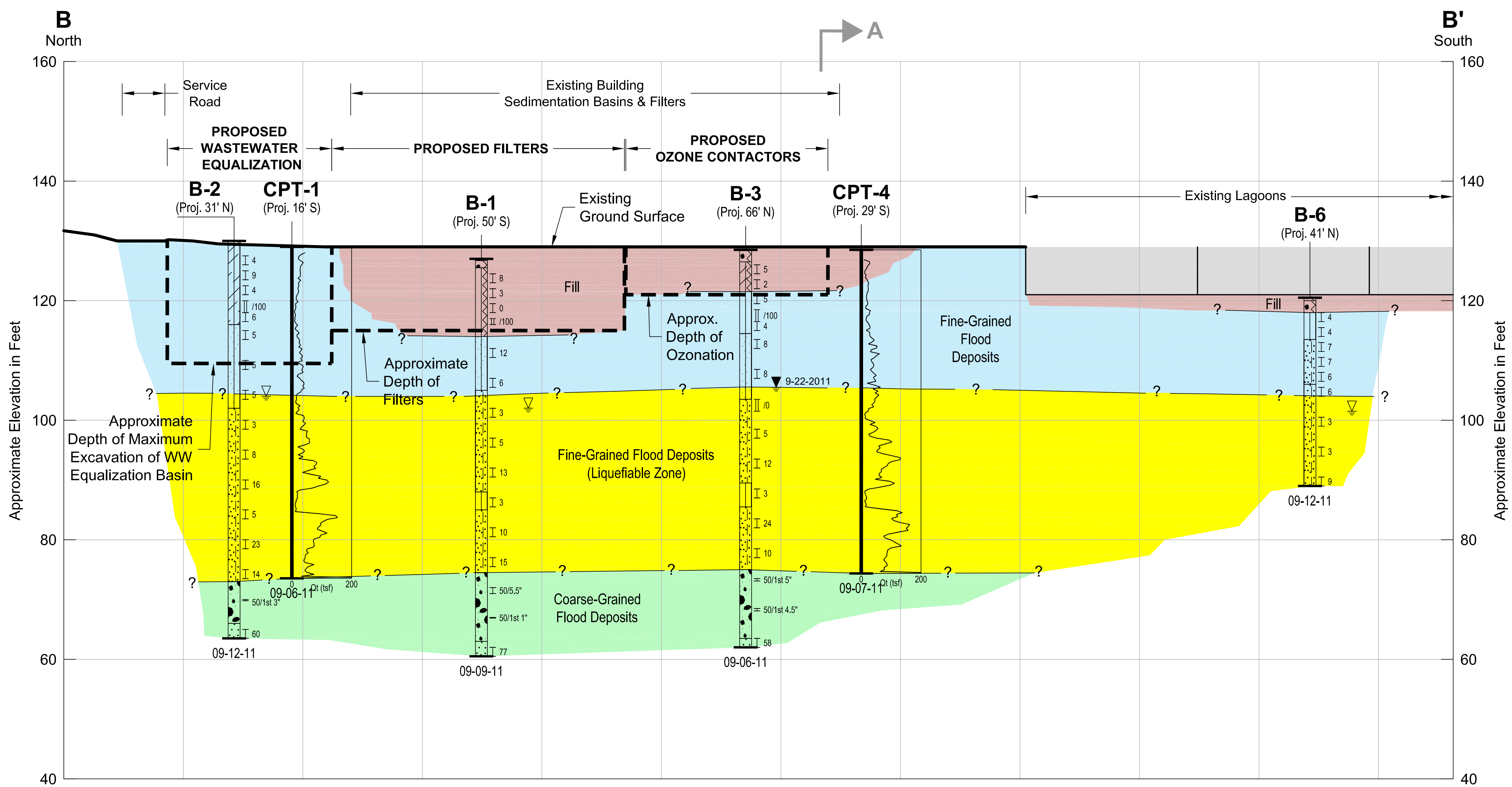
Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

GENERALIZED SUBSURFACE PROIFLE A-A'

January 2012 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 3



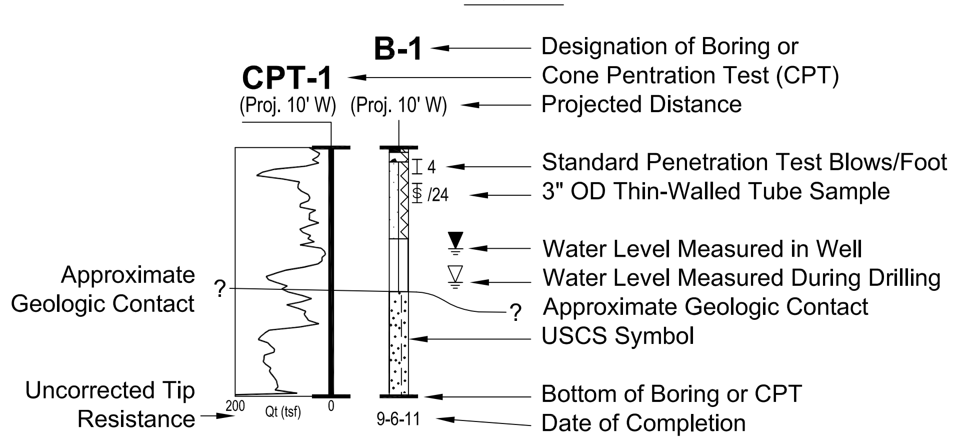
0 20 40 — Vertical Scale in Feet

0 40 80 — Horizontal Scale in Feet

Vertical = Horizontal

DRAFT

LEGEND



SUBSURFACE MATERIAL LEGEND

Artificial Fill: Sandy silty gravel and clayey silt to sandy silt.

Fine-Grained Flood Deposits: Clayey silt, sandy silt, and silty sand to sand.

Fine-Grained Flood Deposits: Clayey silt, sandy silt, and silty sand to sand. (Liquefiable zone below Groundwater)

Older Sand & Gravel Alluvium: Sandy gravel to gravelly sand, and sand.

NOTE

1. This subsurface profile is generalized from materials observed in soil borings. Variations may exist between profile and actual conditions.
2. The upper boundary of the liquefiable zone is the estimated seasonal high groundwater table at approximately 25 feet below ground surface. The groundwater was measured between 23 to 28 feet below the ground surface.

Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

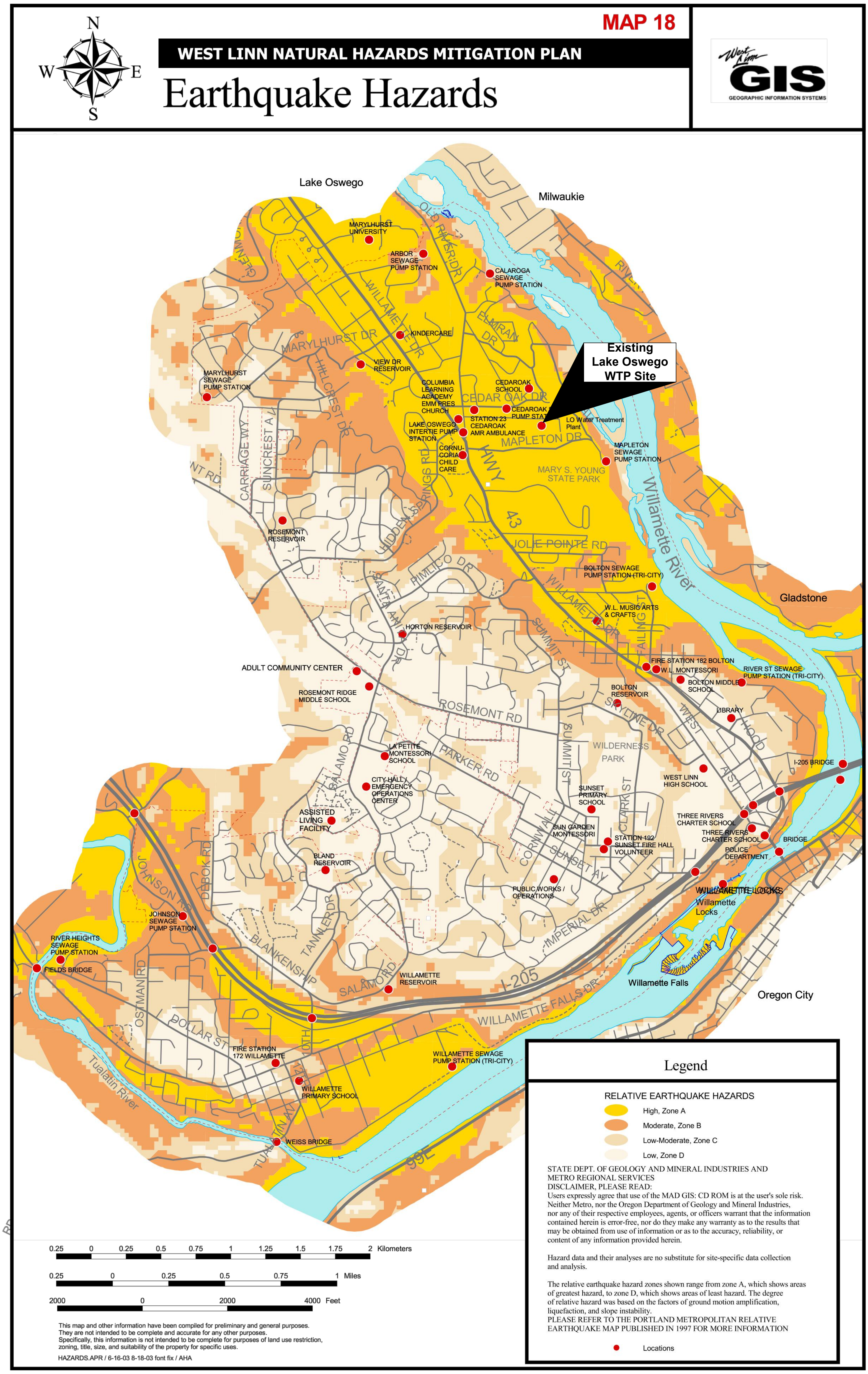
GENERALIZED SUBSURFACE PROFILE B-B'

January 2012 | 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 4

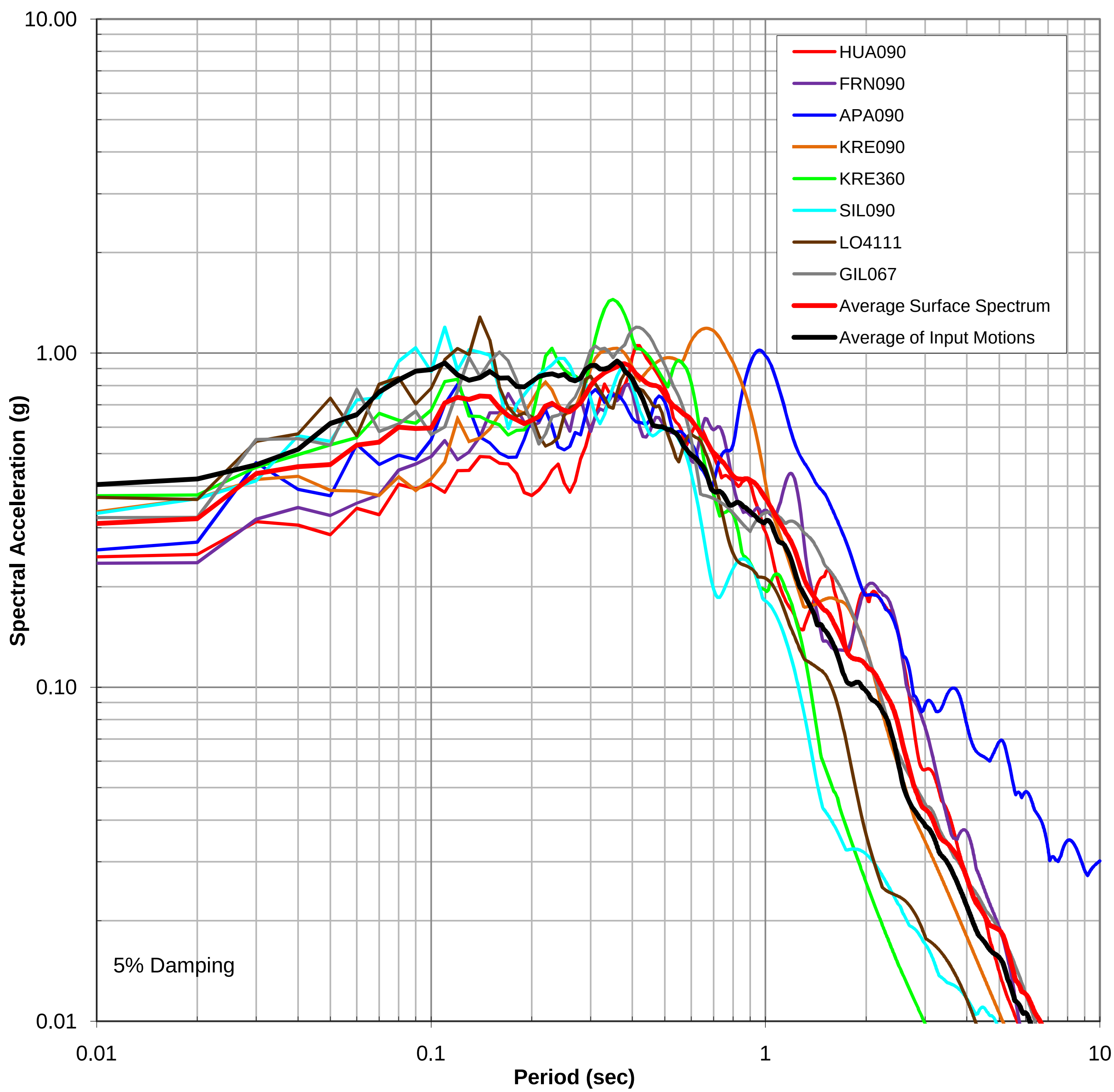
Filename: I:\WIP\Projects\24-1 Portland\3600\3658 Lake Oswego Water Treatment Plant\Graphics\CAD\From Seattle Drafting\Figure 8 Hazard Map.dwg Date: 12-28-2011 Login: rrb



Note:
Map is from City of West Linn Natural Hazards Mitigation Plan.

DRAFT

Lake Oswego & Tigard Water Treatment Plant Expansion Project West Linn, Oregon	
CITY OF WEST LINN EARTHQUAKE HAZARD MAP	
January 2012	24-1-03658-245
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. 5**



DRAFT

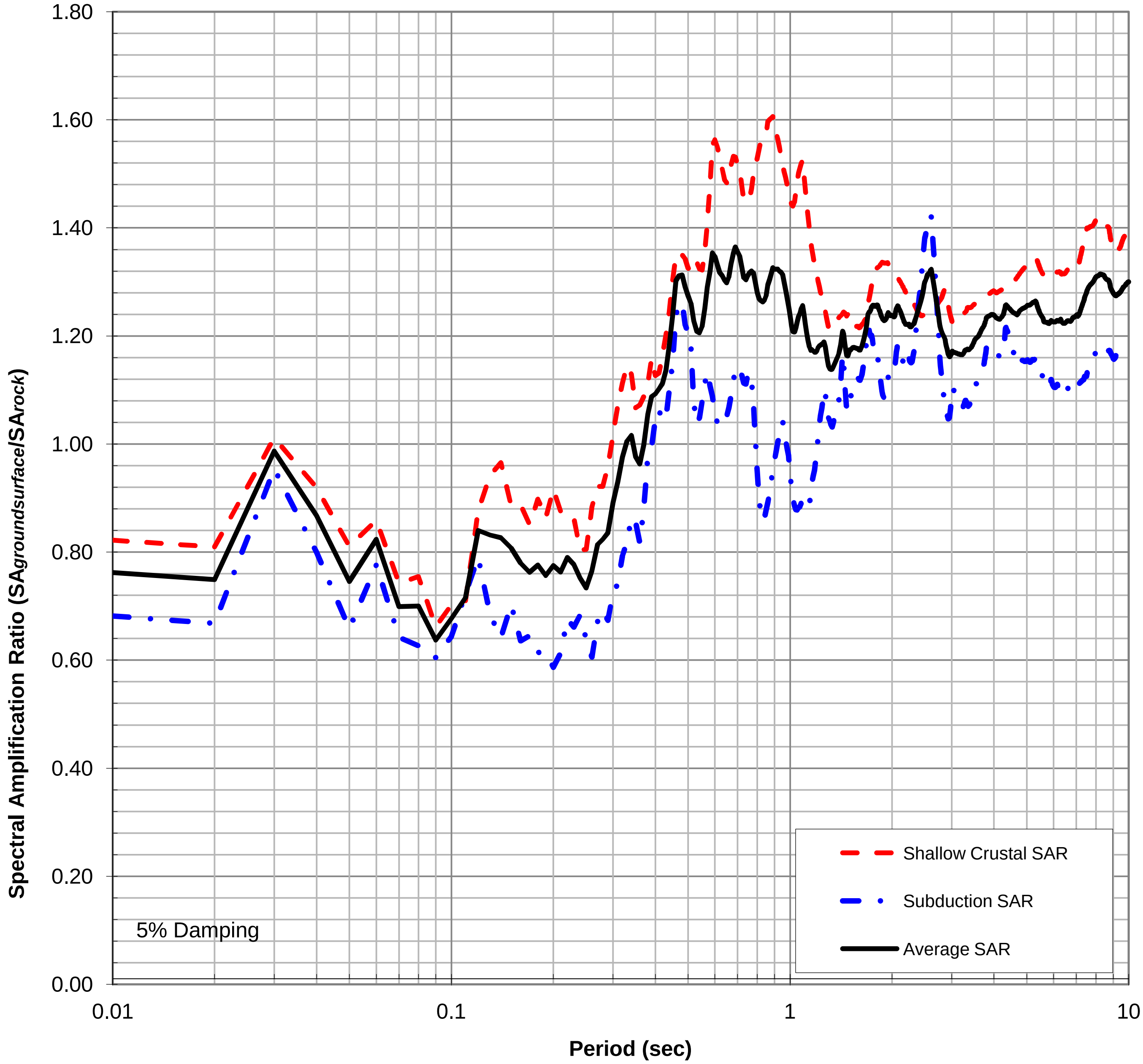
Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

PREDICTED GROUND SURFACE RESPONSE SPECTRUM

January 2012 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 6



DRAFT

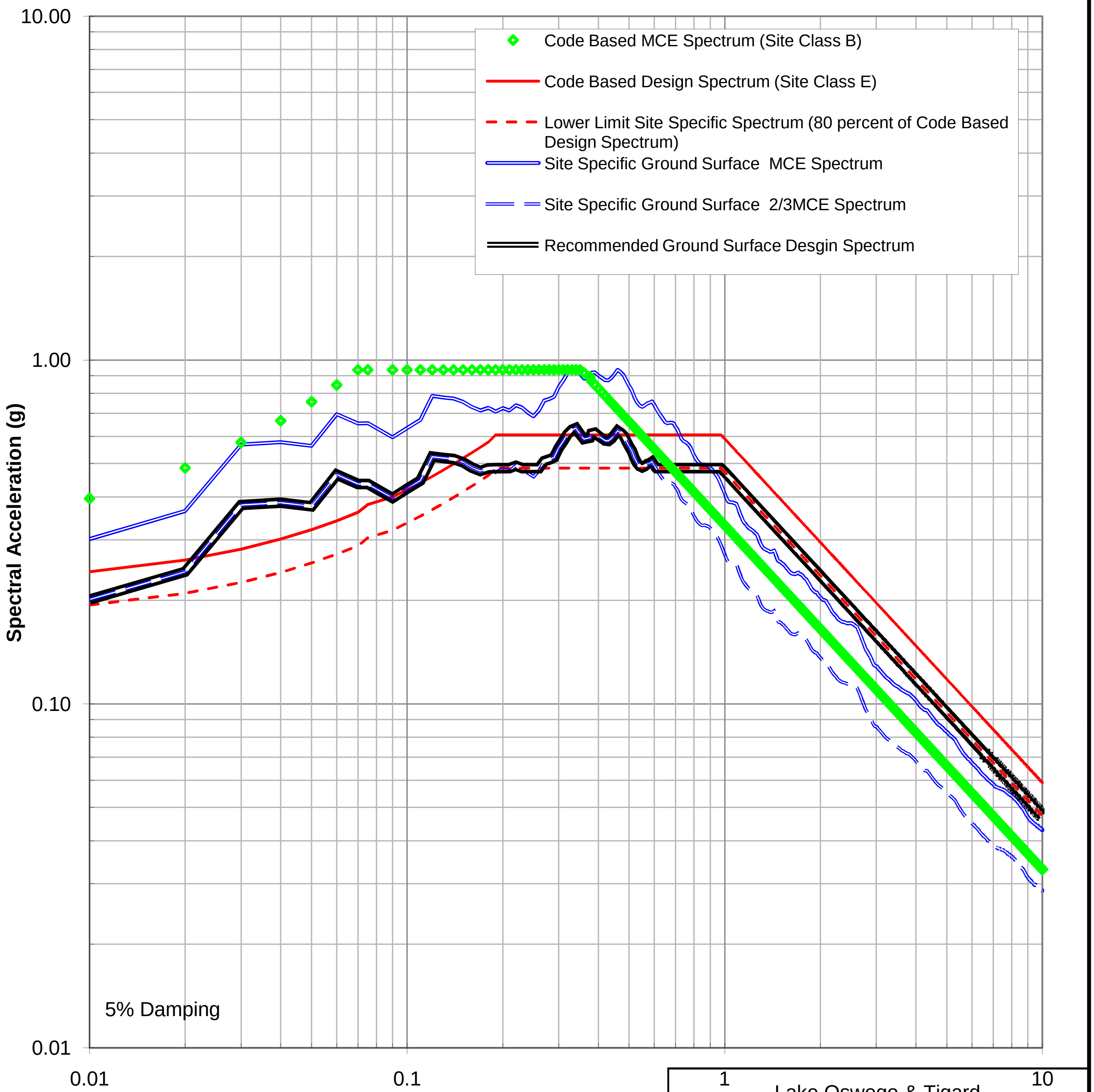
Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

AVERAGE EFFECTIVE STRESS SPECTRA AMPLIFICATION RATIO

January 2012 | 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 7



Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

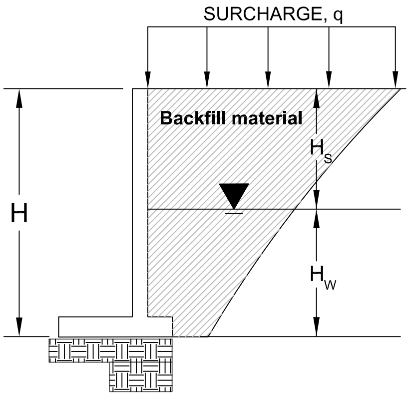
AVERAGE GROUND SURFACE SITE RESPONSE SPECTRA

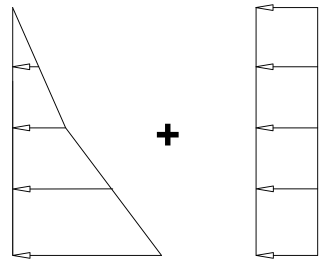
January 2012 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 8

PART A: STATIC CONDITION
TOTAL STATIC LATERAL EQUIVALENT FLUID PRESSURES

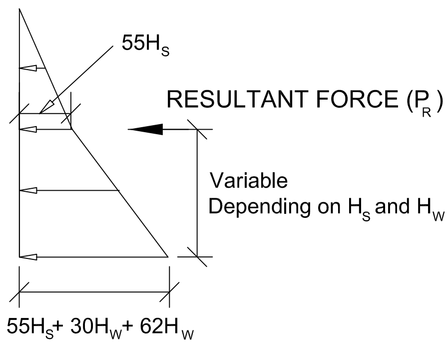




SOIL BACKFILL AND HYDROSTATIC COMPONENT

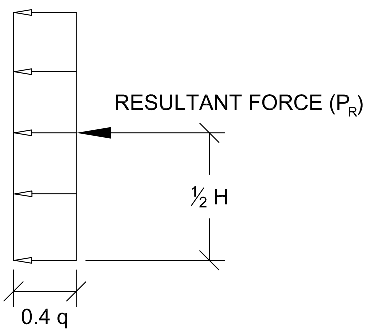
SURCHARGE COMPONENT

BACKFILL AND GROUNDWATER COMPONENT



P_R = [PRESSURE VALUE] x $\frac{H}{2}$

SURCHARGE COMPONENT



P_R = [PRESSURE VALUE] x H

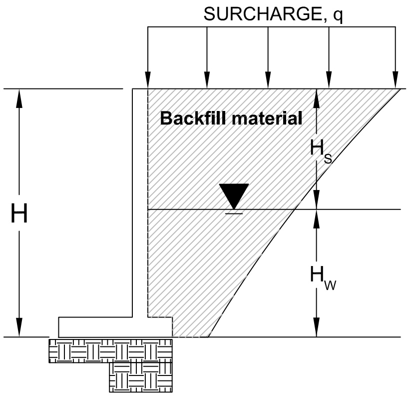
Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

LATERAL EARTH PRESSURE
CASE 1: IMPORTED CRUSHED
ROCK BACKFILL

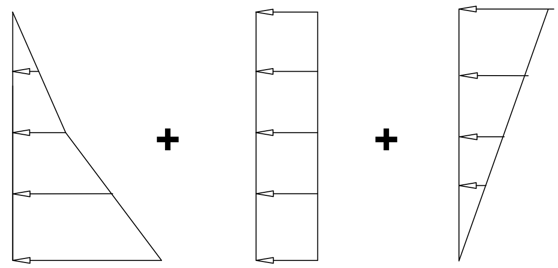
January 2012 | 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 9
Sheet 1 of 2



PART B: SEISMIC CONDITION
TOTAL SEISMIC LATERAL EQUIVALENT FLUID PRESSURES

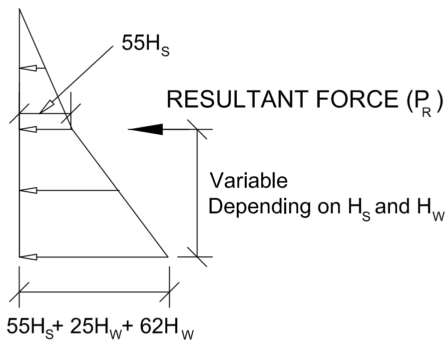


SOIL BACKFILL AND HYDROSTATIC COMPONENT

SURCHARGE COMPONENT

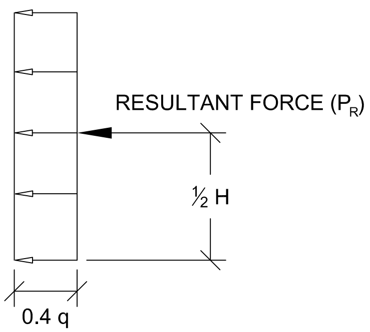
SEISMIC SOIL BACKFILL COMPONENT

BACKFILL AND GROUNDWATER COMPONENT



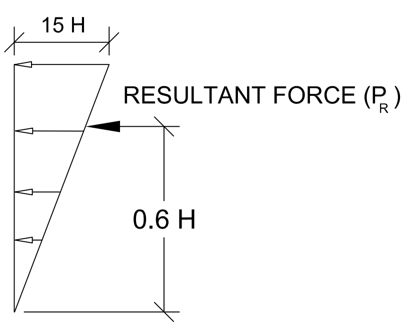
P_R = [PRESSURE VALUE] x $\frac{H}{2}$

SURCHARGE COMPONENT



P_R = [PRESSURE VALUE] x H

SEISMIC BACKFILL COMPONENT



P_R = [PRESSURE VALUE] x $\frac{H}{2}$

DRAFT

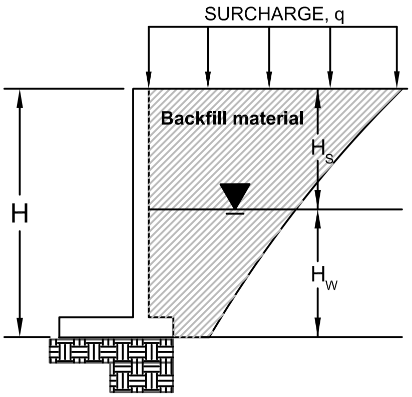
Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

LATERAL EARTH PRESSURE CASE 1: IMPORTED CRUSHED ROCK BACKFILL

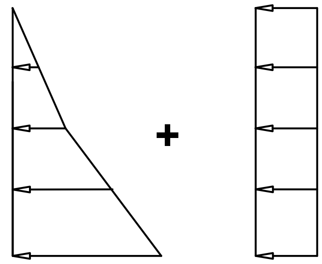
January 2012 | 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 9
Sheet 2 of 2



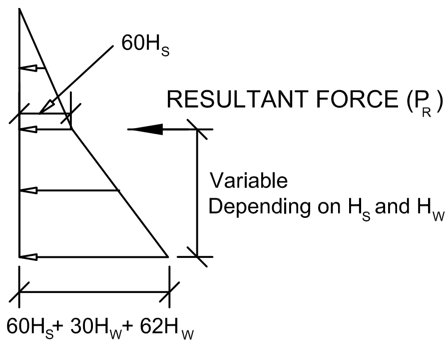
PART A: STATIC CONDITION
TOTAL STATIC LATERAL EQUIVALENT FLUID PRESSURES



SOIL BACKFILL AND HYDROSTATIC COMPONENT

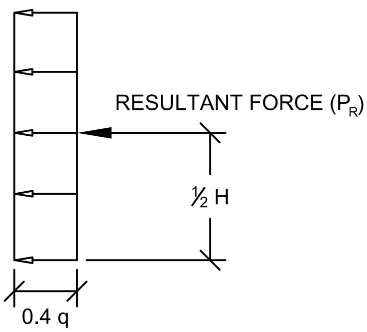
SURCHARGE COMPONENT

BACKFILL AND GROUNDWATER COMPONENT



P_R = [PRESSURE VALUE] x $\frac{H}{2}$

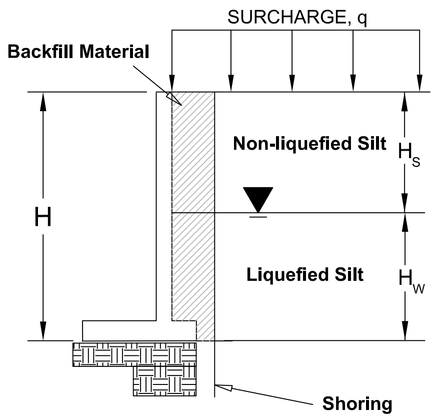
SURCHARGE COMPONENT



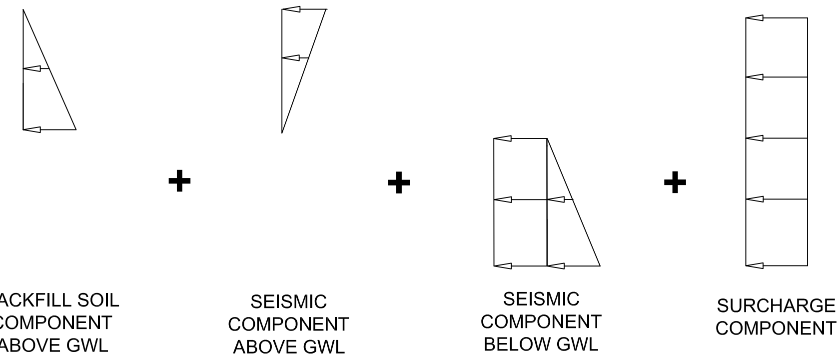
P_R = [PRESSURE VALUE] x H

DRAFT

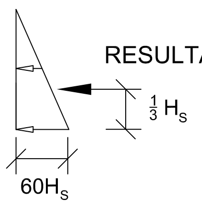
Lake Oswego & Tigard Water Treatment Plant Expansion Project West Linn, Oregon	
LATERAL EARTH PRESSURE CASE 2: SELECT NATIVE SOIL BACKFILL	
January 2012	24-1-03658-245
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. 10** **Sheet 1 of 2**



PART B: SEISMIC CONDITION
TOTAL SEISMIC LATERAL EQUIVALENT FLUID PRESSURES

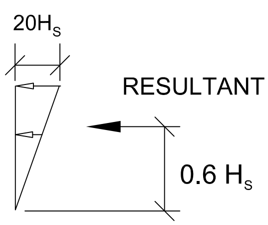


SOIL AND GROUNDWATER COMPONENT



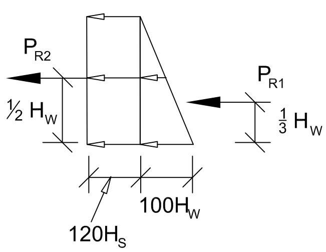
P_R = [PRESSURE VALUE] x $\frac{H}{2}$

SEISMIC COMPONENT ABOVE GWL



P_R= [PRESSURE VALUE] x $\frac{H}{2}$

SEISMIC COMPONENT BELOW GWL

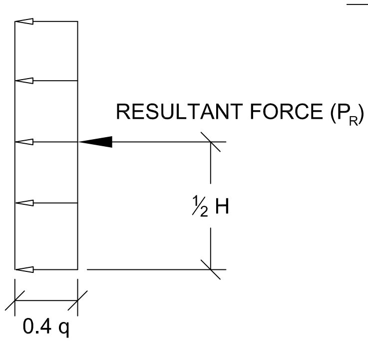


P_{R1} = [PRESSURE VALUE] x $\frac{H}{2}$

P_{R2} = [PRESSURE VALUE] x H

RESULTANT FORCE (P_R) = P_{R1}+ P_{R2}

SURCHARGE COMPONENT



P_R = [PRESSURE VALUE] x H

DRAFT

Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

LATERAL EARTH PRESSURE
CASE 2: SELECT NATIVE
SOIL BACKFILL

January 2012 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 10
Sheet 2 of 2

APPENDIX A

FIELD EXPLORATION PROGRAM

Appendix Contents:

Figure A1:	Soil Classification and Log Key
Figures A2 through A10:	Logs of Borings
Figures A11 through A14:	Logs of Test Pits
Figures A15 through A18:	Test Pit Photographs
Attachment:	Cone Penetration Test Results

Note: Information in this appendix is contained in this report for reference only. For a complete description of the data collected, including laboratory test results, see the Geotechnical Data Report.

Shannon & Wilson, Inc. (S&W), uses a soil classification system modified from the Unified Soil Classification System (USCS). Elements of the USCS and other definitions are provided on this and the following page. Soil descriptions are based on visual-manual procedures (ASTM D 2488-93) unless noted otherwise.

S&W CLASSIFICATION OF SOIL CONSTITUENTS

Major constituents compose more than 50 percent, by weight, of the soil. Major consituents are capitalized (i.e., SAND).

Modifying (secondary) constituents precede the major constituents (i.e., silty SAND) and compose 15 to 45 percent, by weight, for fine-grained soils and 30 to 45 percent, by weight, for coarse-grained soils.

Minor constituents follow major and modifying constituents (i.e., silty SAND with gravel) and compose 10 percent, by weight, for fine-grained soils and 10 to 25 percent, by weight for coarse-grained soils.

Trace constituents follow all other constituents and are labeled "trace" (i.e., silty SAND with trace gravel). Trace constituents comprise 5 percent, by weight of coarse-grained soils and 5 to 10 percent, by weight of fine-grained soils.

Percentages are based on estimating amounts to the nearest 5 percent.

MOISTURE CONTENT DEFINITIONS

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

ABBREVIATIONS

ATD	At Time of Drilling
Elev.	Elevation
ft	feet
FeO	Iron Oxide
MgO	Magnesium Oxide
HSA	Hollow Stem Auger
ID	Inside Diameter
in	inches
lbs	pounds
Mon.	Monument cover
N	Blows for last two 6-inch increments
NA	Not applicable or not available
NP	Nonplastic
OD	Outside diameter
OVA	Organic vapor analyzer
PID	Photo-ionization detector
ppm	parts per million
PVC	Polyvinyl Chloride
SS	Split spoon sampler
SPT	Standard penetration test
USC	Unified soil classification
q_u	Unconfined Compressive Strength

GRAIN SIZE DEFINITION

DESCRIPTION	SIEVE NUMBER AND/OR SIZE
FINES	< #200 (0.08 mm)
SAND*	
- Fine	#200 to #40 (0.08 to 0.4 mm)
- Medium	#40 to #10 (0.4 to 2 mm)
- Coarse	#10 to #4 (2 to 5 mm)
GRAVEL*	
- Fine	#4 to 3/4 inch (5 to 19 mm)
- Coarse	3/4 to 3 inches (19 to 76 mm)
COBBLES	3 to 12 inches (76 to 305 mm)
BOULDERS	> 12 inches (305 mm)

* Unless otherwise noted, sand and gravel, when present, range from fine to coarse in grain size.

RELATIVE DENSITY / CONSISTENCY

COARSE-GRAINED SOILS		FINE-GRAINED SOILS	
N, SPT, BLOWS/FT.	RELATIVE DENSITY	N, SPT, BLOWS/FT.	RELATIVE CONSISTENCY
0 - 4	Very loose	Under 2	Very soft
4 - 10	Loose	2 - 4	Soft
10 - 30	Medium dense	4 - 8	Medium stiff
30 - 50	Dense	8 - 15	Stiff
Over 50	Very dense	15 - 30	Very stiff
		Over 30	Hard

WELL AND OTHER SYMBOLS

Bent. Cement Grout	Surface Cement Seal
Bentonite Grout	Asphalt or Cap
Bentonite Chips	Slough
Silica Sand	Bedrock
PVC Screen	Fill
Vibrating Wire	

PLASTICITY

PLASTICITY ADJECTIVE	PLASTICTY INDEX (PI) RANGE
Nonplastic	0 - 4
Low Plasticity	>4 - 10
Medium Plasticity	>10 - 20
High Plasticity	>20 - 40
Very High Plasticity	>40

DRAFT

Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

SOIL CLASSIFICATION AND LOG KEY

January 2012 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A1
Sheet 1 of 2

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)
(From ASTM D 2488)

MAJOR DIVISIONS			GROUP/GRAPHIC SYMBOL	TYPICAL DESCRIPTION
COARSE-GRAINED SOIL *(more than 50% retained on No. 200 sieve)*	Gravel *(more than 50% of coarse fraction retained on No. 4 sieve)*	Clean Gravel *(less than 5% fines)*	GW	Well-graded gravel, gravel, gravel/sand mixtures, little or no fines.
			GP	Poorly graded gravel, gravel-sand mixtures, little or no fines
		Gravel with Fines *(more than 10% fines)*	GM	Silty gravel, gravel-sand-silt mixtures
			GC	Clayey gravel, gravel-sand-clay mixtures
	Sand *(50% or more of coarse fraction passes the No. 4 sieve)*	Clean Sand *(less than 5% fines)*	SW	Well-graded sand, gravelly sand, little or no fines
			SP	Poorly graded sand, gravelly sand, little or no fines
		Sand with Fines *(more than 10% fines)*	SM	Silty sand, sand-silt mixtures
			SC	Clayey sand, sand-clay mixtures
FINE-GRAINED SOIL *(50% or more passes the No. 200 sieve)*	Silt and Clay *(liquid limit less than 50)*	Inorganic	ML	Inorganic silt of low to medium plasticity, rock flour, sandy silt, gravelly silt, or clayey silt with slight plasticity
			CL	Inorganic clay of low to medium plasticity, gravelly clay, sandy clay, silty clay
		Organic	OL	Organic silt and organic silty clay of low plasticity
	Silt and Clay *(liquid limit 50 or more)*	Inorganic	MH	Inorganic silt, micaceous or diatomaceous fine sand or silty soils, elastic silt
			CH	Inorganic clay or medium to high plasticity
		Organic	OH	Organic clay of medium to high plasticity, organic silt
HIGHLY-ORGANIC SOIL	Primarily organic matter, dark in color, and organic odor		PT	Peat, humus, swamp soils with high organic content (see ASTM D 4427)

NOTE: No. 4 size = 5 mm; No. 200 size = 0.075 mm

NOTES

1. Solid lines on the logs indicate contacts between major units. Dashed lines indicate contacts between different material types within the same unit. Dotted lines indicate subtle or uncertain contacts within a unit. The contacts shown are an interpretation of the condition encountered and actual contacts may be more gradational than shown.

2. Dual symbols (symbols separated by a hyphen, i.e., SP-SM, SAND with silt) are used for coarse-grained soils with 10 percent fines or when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart.

3. Borderline symbols (symbols separated by a slash, i.e., CL/ML and GW/SW) indicate that the soil may fall into one of two possible basic groups.

DRAFT

Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

SOIL CLASSIFICATION AND LOG KEY

January 2012 | 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A1
Sheet 2 of 2

POST_07_BORING_CLASS2 24-1-03658-001.GPJ SWNEW.GDT 12/27/11

Total Depth:	66.5 ft.	Northing:	~	Drilling Method:	Mud Rotary	Hole Diam.:	5 in.
Top Elevation:	127	Easting:	~	Drilling Company:	Subsurface Technologies	Rod Type:	NWJ
Vert. Datum:		Station:	~	Drill Rig Equipment:	Mobil B-53	Hammer Type:	Automatic
Horiz. Datum:		Offset:	~	Other Comments:			

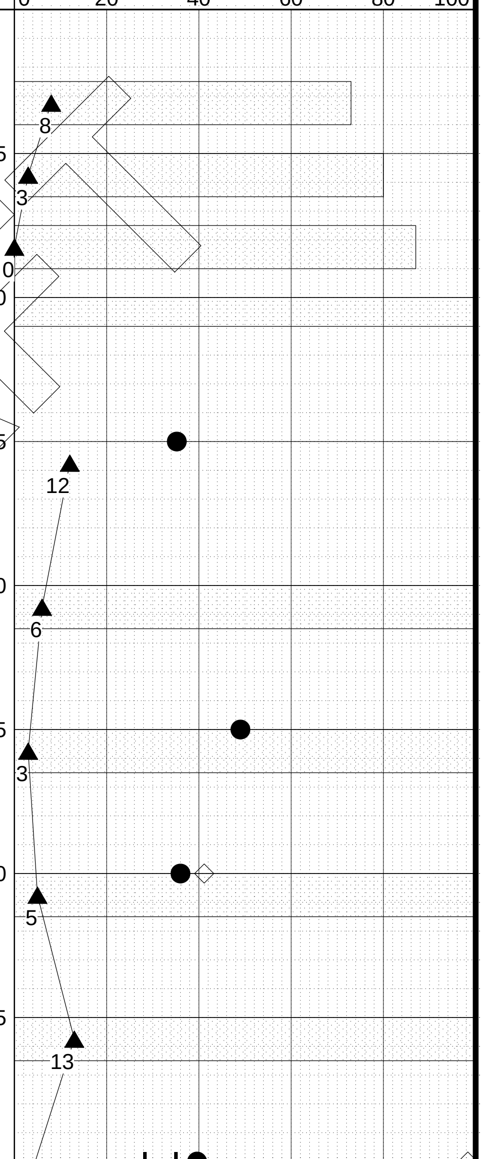
SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

SOIL DESCRIPTION	Elev. Depth (ft.)	Samples	Ground Water
ASPHALT CONCRETE	*126.7* 0.3		
BASE AGGREGATE	*125.5* 1.5		
Medium stiff grading to very soft gray and brown in upper to brown in lower SILT with sand grading to sandy SILT; moist to wet; low plasticity; fine to coarse sand, trace gravel and occasional organics in upper 4.0 feet. (ML) **ARTIFICIAL FILL** Blocky texture with scattered fragments of stiff CLAY at 7.5 feet.	*114.0* 13.0	S-1, S-2, S-3, S-4	
Medium dense to loose brown sandy SILT to silty SAND; moist to wet; nonplastic fines; fine to medium sand; stratified with occasional layers of SAND, trace silt; micaceous. (ML/SM) **FINE-GRAINED FLOOD DEPOSITS**	*105.0* 22.0	S-5 *, S-6	
Loose to medium dense brown silty SAND; wet; fine to medium sand; stratified; micaceous. (SM)	*88.0* 39.0	S-7, S-8, S-9	During Drilling

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches



CONTINUED NEXT SHEET

LEGEND

* Sample Not Recovered

Standard Penetration Test

3" O.D. Shelby Tube

Groundwater Level

Recovery (%)

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├──┤ Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

LOG OF BORING B-1

January 2012 — 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A2
Sheet 1 of 2

REV 1

Total Depth:	66.5 ft.	Northing:	~	Drilling Method:	Mud Rotary
Top Elevation:	127	Easting:	~	Drilling Company:	Subsurface Technologies
Vert. Datum:		Station:	~	Drill Rig Equipment:	Mobil B-53
Horiz. Datum:		Offset:	~	Other Comments:	

Hole Diam.:	5 in.
Rod Type:	NWJ
Hammer Type:	Automatic

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

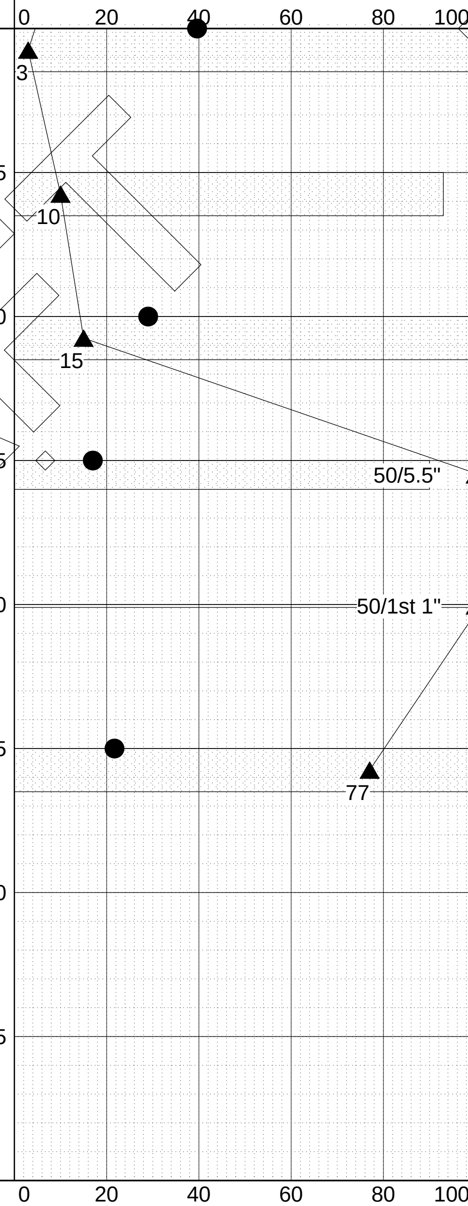
Soil Description	Elev. / Depth (ft.)	Samples
Soft brown-gray SILT, trace sand; moist to wet; low plasticity; fine sand; stratified; slight iron oxidation and staining. (ML)	*85.0* / 42.0	S-10
Loose to medium dense gray silty SAND; wet; fine to medium sand; stratified; micaceous. (SM) **FINE-GRAINED FLOOD DEPOSITS**	*74.5* / 52.5	S-11, S-12
Very dense dark gray sandy GRAVEL to gravelly SAND, trace silt; moist; fine to coarse sand; rounded to subangular gravel; slight iron-oxide staining; moderately consolidated. (GP-SP) **OLDER SAND & GRAVEL ALLUVIUM** Hard drilling below 53 feet, grinding action, possibly due to cobbles?	*63.0* / 64.0	S-13, S-14
Very dense black SAND, trace silt and gravel; moist; fine to coarse sand; fine subangular gravel; moderately consolidated. (SP)	*60.5* / 66.5	S-15

Completed - September 9, 2011

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches

Depth, ft.: 45, 50, 55, 60, 65, 70, 75



Recovery (%)

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├──┤ Liquid Limit

LEGEND

* Sample Not Recovered

Standard Penetration Test

3" O.D. Shelby Tube

Groundwater Level

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

LOG OF BORING B-1

January 2012 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A2
Sheet 2 of 2

REV 1

MASTER_LOG_E 24-1-03658-001.GPJ SHAN_WIL.GDT 12/27/11 Log: CKS Rev: Typ: MAS/RRB

Total Depth:	*66.5 ft.*	Northing:	~	Drilling Method:	*Mud Rotary*	Hole Diam.:	*5 in.*
Top Elevation:	*130*	Easting:	~	Drilling Company:	*Subsurface Technologies*	Rod Type:	*NWJ*
Vert. Datum:		Station:	~	Drill Rig Equipment:	*Mobil B-53*	Hammer Type:	*Automatic*
Horiz. Datum:		Offset:	~	Other Comments:			

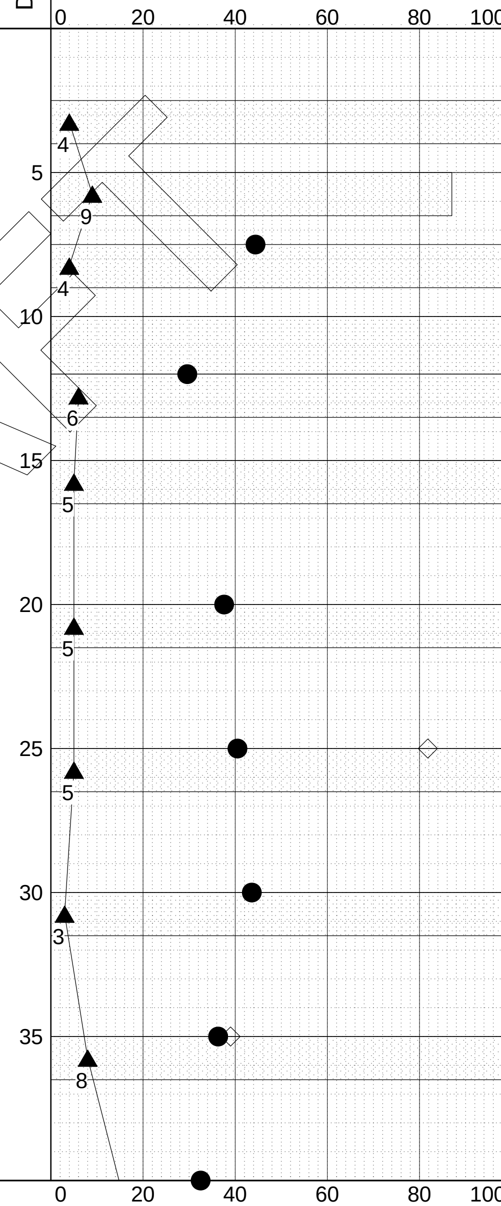
SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

SOIL DESCRIPTION	Elev. Depth (ft.)	Samples	Ground Water
Soft to stiff gray with orange-brown mottling clayey SILT with sand to sandy SILT: moist; low plasticity; fine sand; slight iron oxidation and red-orange staining and mottling; occasional layers of silty SAND or clayey SILT; micaceous. (ML) **FINE-GRAINED FLOOD DEPOSITS**		S-1, S-2, S-3, S-4, S-5	
Loose brown sandy SILT; moist; non to low plasticity fines; fine to medium sand; stratified; scattered layers of sandy SILT or SAND, trace silt; micaceous. (ML/SM) Grades to wet at 25.0 feet.	*116.0* 14.0	S-6, S-7, S-8	During Drilling
Very loose to medium dense brown silty SAND; wet; fine to medium sand; stratified; occasional to scattered layers of SILT, fine sand or silty SAND. (SM) Slight iron oxidation and red-brown staining from 40.0 to 52.0 feet.	*102.0* 28.0	S-9, S-10	

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: *140 lbs / 30 inches*



CONTINUED NEXT SHEET

LEGEND

- * Sample Not Recovered
- Standard Penetration Test
- 3" O.D. Shelby Tube
- Groundwater Level
- Recovery (%)
- ◇ % Fines (<0.075mm)
- ● % Water Content
- Plastic Limit ├──┤ Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

LOG OF BORING B-2

January 2012 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A3
Sheet 1 of 2

REV 1

MASTER_LOG_E 24-1-03658-001.GPJ SHAN_WIL.GDT 12/27/11 Log: CKS Rev: Typ: MAS/RRB

Total Depth: 66.5 ft.	Northing: ~	Drilling Method:	Mud Rotary	Hole Diam.:	5 in.
Top Elevation: 130	Easting: ~	Drilling Company:	Subsurface Technologies	Rod Type:	NWJ
Vert. Datum:	Station: ~	Drill Rig Equipment:	Mobil B-53	Hammer Type:	Automatic
Horiz. Datum:	Offset: ~	Other Comments:			

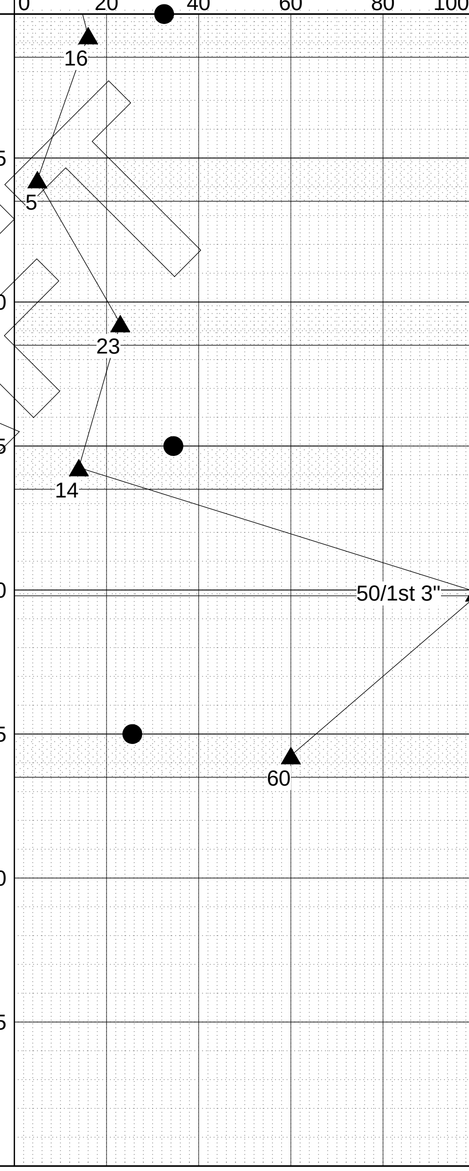
SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Soil Description	Elev. Depth (ft.)	Samples	Depth, ft.
Continued: Very loose to medium dense brown silty SAND; wet; fine to medium sand; stratified; occasional to scattered layers of SILT, fine sand or silty SAND. (SM) SILT layer from 45.5 to 46.1 feet. **FINE-GRAINED FLOOD DEPOSITS**		S-11, S-12, S-13, S-14	45, 50, 55
Very dense brown and dark gray sandy GRAVEL, trace silt; moist; fine to coarse sand; subrounded to subangular gravel; slight iron oxidation and red-brown staining; moderately consilidated. (GP) **OLDER SAND & GRAVEL ALLUVIUM** Hard drilling below 57 feet, grinding action, possibly due to cobbles?	*73.0* 57.0	S-15	60
Very dense black SAND, trace silt; moist; fine to medium sand; moderately consolidated. (SP)	*66.0* 64.0	S-16	65
Completed - September 12, 2011	*63.5* 66.5		70, 75

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches



Recovery (%)
◇ % Fines (<0.075mm)
● % Water Content
Plastic Limit ├──┤ Liquid Limit

LEGEND

* Sample Not Recovered
Standard Penetration Test
3" O.D. Shelby Tube
Groundwater Level

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

LOG OF BORING B-2

January 2012 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A3
Sheet 2 of 2

REV 1

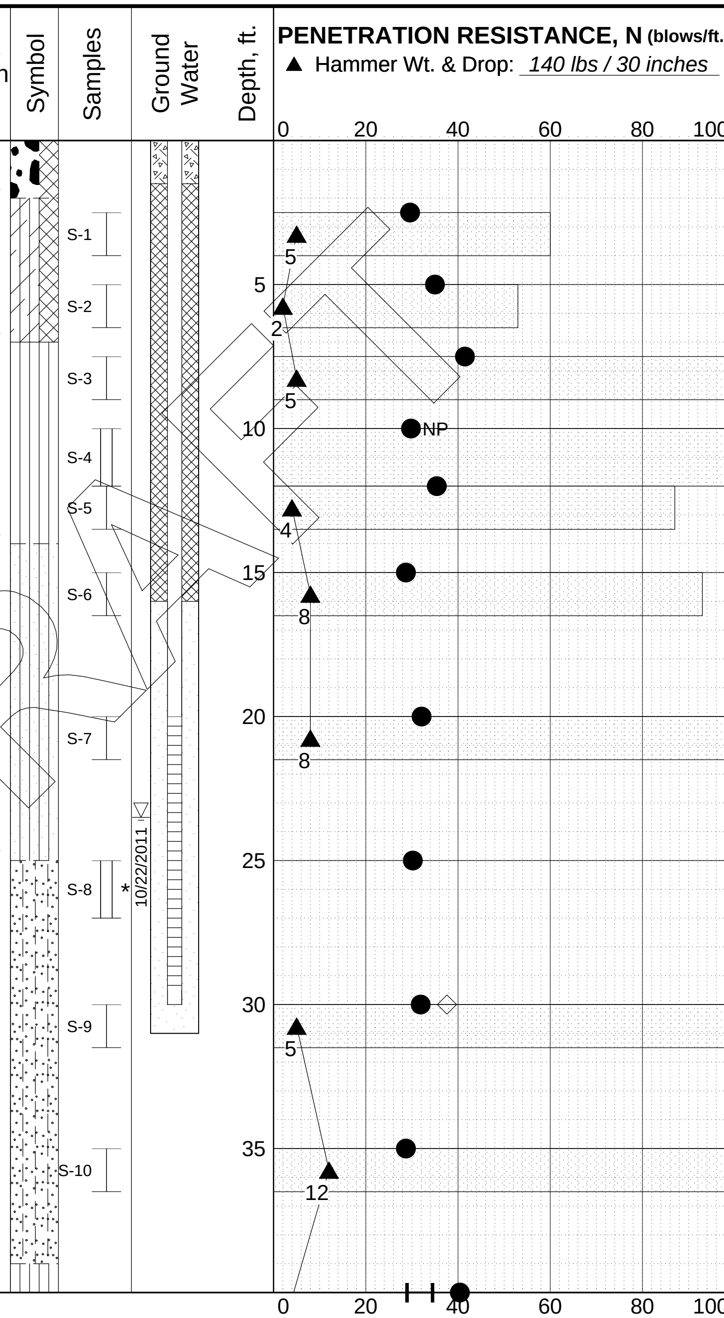
Total Depth:	*66.5 ft.*	Northing:	~	Drilling Method:	*Mud Rotary*
Top Elevation:	*128.5*	Easting:	~	Drilling Company:	*Subsurface Technologies*
Vert. Datum:		Station:	~	Drill Rig Equipment:	*Mobil B-53*
Horiz. Datum:		Offset:	~	Other Comments:	

Hole Diam.: *5 in.*
Rod Type: *NWJ*
Hammer Type: *Automatic*

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Soil Description	Elev. / Depth (ft.)
BASE AGGREGATE	*126.5* / 2.0
Medium stiff to soft dark gray clayey SILT with sand; moist; low plasticity; fine to coarse sand; scattered organics and rootlets; micaceous. (ML) **ARTIFICIAL FILL**	*121.5* / 7.0
Medium stiff to soft brown SILT, trace sand grading to sandy SILT; moist; nonplastic to low plasticity; fine sand; stratified with occasional layers of silty SAND; micaceous. (ML) **FINE-GRAINED FLOOD DEPOSITS**	*114.5* / 14.0
Loose brown sandy SILT to silty SAND; moist; nonplastic fines; fine to medium sand; stratified; light iron oxidation and orange-brown staining; micaceous. (ML/SM)	*103.5* / 25.0
Loose to medium dense brown silty SAND; wet; fine to medium sand; faintly stratified; micaceous. (SM)	*89.5* / 39.0



CONTINUED NEXT SHEET

LEGEND

* Sample Not Recovered
Standard Penetration Test
3" O.D. Shelby Tube

Recovery (%)
◇ % Fines (<0.075mm)
● % Water Content
Plastic Limit ├──┤ Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

LOG OF BORING B-3

January 2012 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A4
Sheet 1 of 2

REV 1

Log: CKS Rev: Typ: MAS/RRB

MASTER_LOG_E 24-1-03658-001.GPJ SHAN_WIL.GDT 12/27/11

Total Depth:	*66.5 ft.*	Northing:	~	Drilling Method:	*Mud Rotary*	Hole Diam.:	*5 in.*
Top Elevation:	*128.5*	Easting:	~	Drilling Company:	*Subsurface Technologies*	Rod Type:	*NWJ*
Vert. Datum:		Station:	~	Drill Rig Equipment:	*Mobil B-53*	Hammer Type:	*Automatic*
Horiz. Datum:		Offset:	~	Other Comments:			

SOIL DESCRIPTION

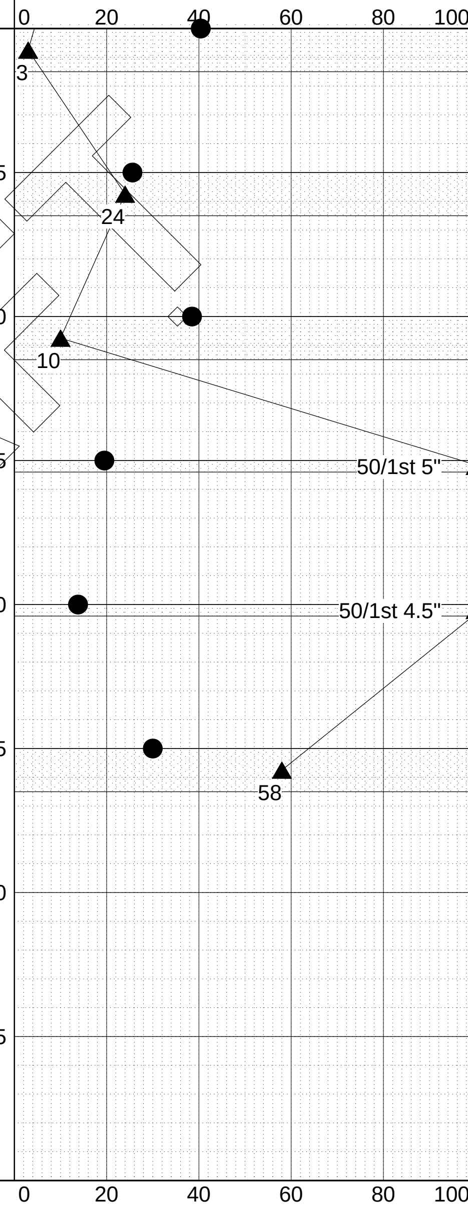
Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

SOIL DESCRIPTION	Elev. / Depth (ft.)	Samples	Depth, ft.
Soft gray-brown with orange-brown mottling SILT, trace sand; wet; low plasticity; fine sand; stratified with occasional sandy SILT layers; micaceous. (ML)	*85.5* / 43.0	S-11	
Medium dense to loose brown silty SAND; moist to wet; fine to medium sand; stratified; occasional layers ofscattered organic debris; micaceous. (SM) Grades to gray at 46.0 feet. **FINE-GRAINED FLOOD DEPOSITS**	*75.0* / 53.5	S-12, S-13	45, 50
Very dense dark gray and green-black sandy GRAVEL, trace silt; moist; fine to coarse sand; subrounded to subangular gravel; moderately consolidated. (GP) **OLDER SAND & GRAVEL ALLUVIUM** Hard drilling below 54 feet, grinding action, possibly due to cobbles?	*63.5* / 65.0	S-14, S-15	55, 60, 65
Very dense dark gray and black SAND, trace gravel; moist; fine to coarse sand; subrounded gravel. (SP)	*62.0* / 66.5	S-16	

Completed - September 7, 2011

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: *140 lbs / 30 inches*



LEGEND

* Sample Not Recovered

Standard Penetration Test

3" O.D. Shelby Tube

Recovery (%)

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├──┤ Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

LOG OF BORING B-3

January 2012 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A4
Sheet 2 of 2

REV 1

Total Depth:	66.5 ft.	Northing:	~	Drilling Method:	Mud Rotary
Top Elevation:	129	Easting:	~	Drilling Company:	Subsurface Technologies
Vert. Datum:		Station:	~	Drill Rig Equipment:	Mobil B-53
Horiz. Datum:		Offset:	~	Other Comments:	

Hole Diam.: 5 in.
Rod Type: NWJ
Hammer Type: Automatic

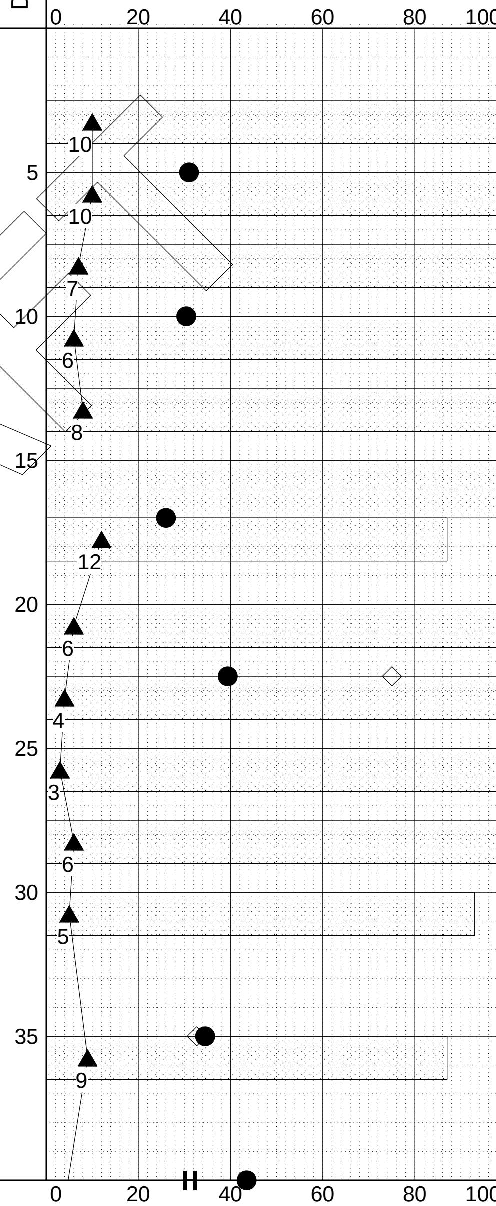
SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Soil Description	Elev. / Depth (ft.)	Samples	Ground Water
SOD	128.8 / 0.2		
Stiff to medium stiff gray with orange mottling grading to brown SILT; moist; low plasticity; fine sand; slight iron oxidation and orange-brown staining; micaceous. (ML) **FINE-GRAINED FLOOD DEPOSITS**		S-1, S-2	
Loose to medium dense brown with orange-brown mottling sandy SILT to silty SAND; moist; nonplastic fines; fine sand; stratified; slight iron oxidation and staining; micaceous. (ML/SM) Grades to wet at 23.0 feet.	120.5 / 8.5	S-3, S-4, S-5, S-6, S-7, S-8, S-9	During Drilling
Soft gray-brown clayey SILT, trace sand; wet; nonplastic to low plasticity; fine sand; stratified; occasional layers of silty SAND. (ML)	104.5 / 24.5	S-10	
Loose brown silty SAND; wet; fine to medium sand; stratified; occasional layers of iron oxidation and staining; micaceous. (SM)	102.0 / 27.0	S-11, S-12, S-13	

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches



CONTINUED NEXT SHEET

LEGEND

* Sample Not Recovered
Standard Penetration Test
3" O.D. Shelby Tube
Groundwater Level

Recovery (%)
% Fines (<0.075mm)
% Water Content
Plastic Limit — Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

LOG OF BORING B-4

January 2012 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A5
Sheet 1 of 2

REV 1

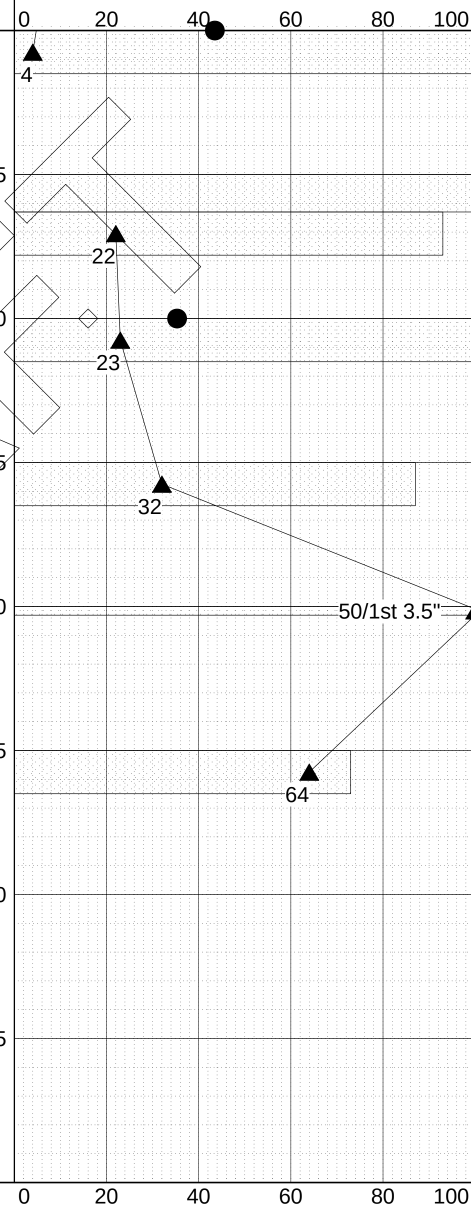
MASTER LOG E 24-1-03658-001.GPJ SHAN WIL.GDT 12/27/11 Log: CKS Rev: Typ: MAS/RRB

Total Depth:	*66.5 ft.*	Northing:	~	Drilling Method:	*Mud Rotary*	Hole Diam.:	*5 in.*
Top Elevation:	*129*	Easting:	~	Drilling Company:	*Subsurface Technologies*	Rod Type:	*NWJ*
Vert. Datum:		Station:	~	Drill Rig Equipment:	*Mobil B-53*	Hammer Type:	*Automatic*
Horiz. Datum:		Offset:	~	Other Comments:			

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	*Elev.* Depth (ft.)	Symbol	Samples	Ground Water	Depth, ft.
Soft brown SILT, trace sand grading to silty SAND; wet; low plasticity; fine sand; stratified with occasional SAND, trace silt layers. (ML/SM) **FINE-GRAINED FLOOD DEPOSITS**	*89.0* 40.0		S-14		
Medium dense gray silty SAND; moist to wet; fine to medium sand; stratified; occasional layers of fine sand; micaceous. (SM)	*83.5* 45.5		S-15 S-16 S-17		45 50
Very dense green-black to dark gray sandy GRAVEL, trace silt; moist, fine to coarse sand; subrounded to subangular gravel; moderately consolidated; occasional highly weathered gravel clasts in upper. (GP) **OLDER SAND & GRAVEL ALLUVIUM** Hard drilling below 52 feet, grinding action, possibly due to cobbles? Gravelly SAND layer from 64.5 to 66.0 feet	*77.0* 52.0		S-18 S-19 S-20		55 60 65
Completed - September 6, 2011	*62.5* 66.5				70 75

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: *140 lbs / 30 inches*



Recovery (%)
◇ % Fines (<0.075mm)
● % Water Content
Plastic Limit ├──┤ Liquid Limit

LEGEND

* Sample Not Recovered
Standard Penetration Test
3" O.D. Shelby Tube
Groundwater Level

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

LOG OF BORING B-4

January 2012 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A5
Sheet 2 of 2

REV 1

MASTER LOG E 24-1-03658-001.GPJ SHAN WIL.GDT 12/27/11
Log: CKS Rev: Typ: MAS/RRB

Total Depth:	*66.5 ft.*	Northing:	~	Drilling Method:	*Mud Rotary*
Top Elevation:	*131*	Easting:	~	Drilling Company:	*Subsurface Technologies*
Vert. Datum:		Station:	~	Drill Rig Equipment:	*Mobil B-53*
Horiz. Datum:		Offset:	~	Other Comments:	

Hole Diam.: *5 in.*
Rod Type: *NWJ*
Hammer Type: *Automatic*

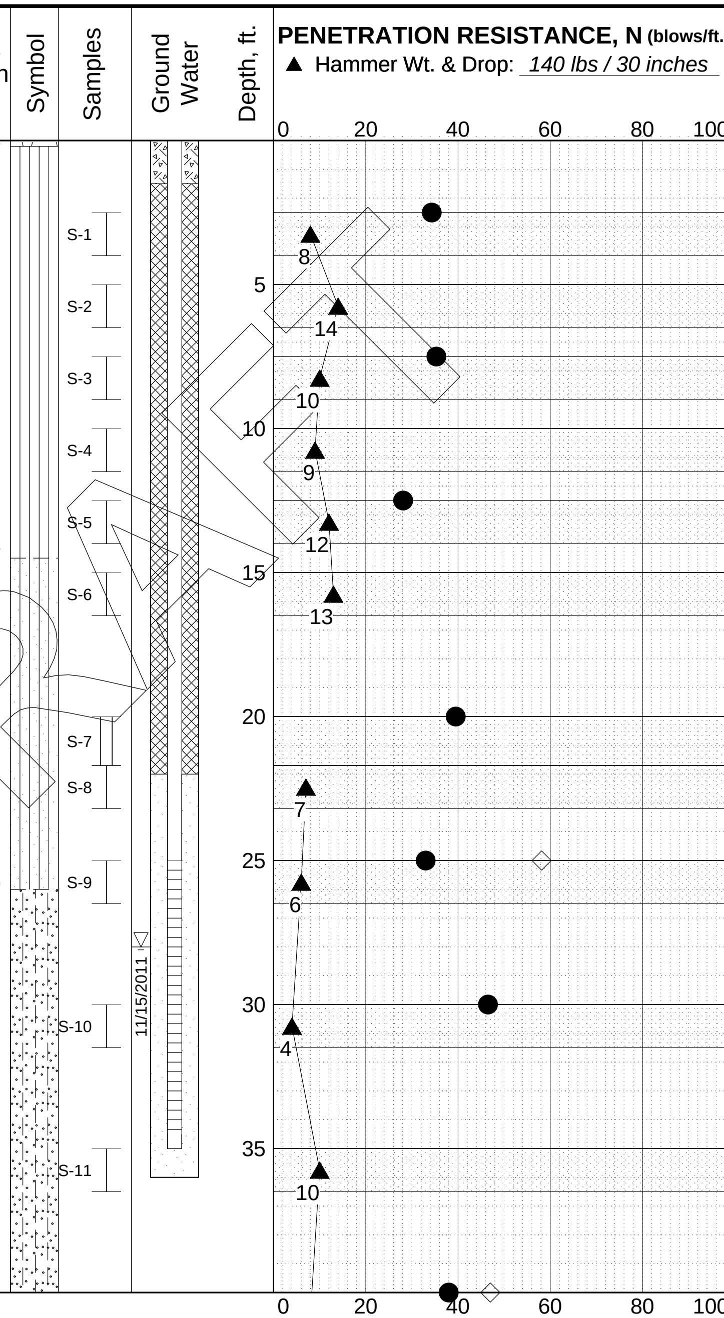
SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Soil Description	Elev. / Depth (ft.)
SOD	*130.8* / 0.2
Medium stiff to stiff gray and brown with orange mottling SILT with sand to sandy SILT; moist; low plasticity; fine sand; stratified; slight to moderate iron-oxidation and orange staining; micaceous. (ML) **FINE-GRAINED FLOOD DEPOSITS**	*116.5* / 14.5
Medium dense to loose brown sandy SILT; moist; nonplastic to low plasticity fines; fine sand; stratified with occasional to scattered layers of SILT or SAND, trace silt; micaceous. (ML/SM) Grades to wet at 25.0 feet.	*105.0* / 26.0
Very loose to medium dense brown silty SAND, trace silt; wet; fine to medium sand; stratified with occasional to scattered layers of sandy SILT; micaceous. (SM)	

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: *140 lbs / 30 inches*



CONTINUED NEXT SHEET

LEGEND

* Sample Not Recovered
Standard Penetration Test
3" O.D. Shelby Tube

Recovery (%)
◇ % Fines (<0.075mm)
● % Water Content
Plastic Limit ├──┤ Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

LOG OF BORING B-5

January 2012 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A6
Sheet 1 of 2

REV 1

Typ: MAS/RRB Rev: Log: CKS

MASTER_LOG_E 24-1-03658-001.GPJ SHAN_WIL.GDT 12/27/11

Total Depth:	66.5 ft.	Northing:	~	Drilling Method:	Mud Rotary	Hole Diam.:	5 in.
Top Elevation:	131	Easting:	~	Drilling Company:	Subsurface Technologies	Rod Type:	NWJ
Vert. Datum:		Station:	~	Drill Rig Equipment:	Mobil B-53	Hammer Type:	Automatic
Horiz. Datum:		Offset:	~	Other Comments:			

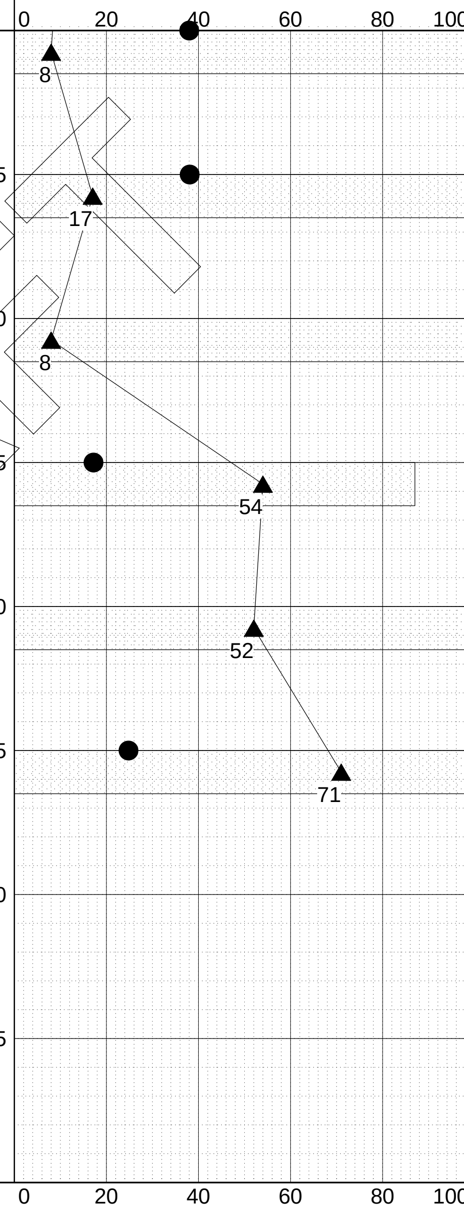
SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Soil Description	Elev. Depth (ft.)	Symbol	Samples	Ground Water	Depth, ft.
Continued: Very loose to medium dense brown silty SAND, trace silt; wet; fine to medium sand; stratified with occasional to scattered layers of sandy SILT; micaceous. (SM)			S-12, S-13, S-14		45, 50
Very dense red-brown and brown sandy GRAVEL, trace silt; moist; fine to coarse sand; subrounded to subangular gravel; slight iron oxidation. (GP) **OLDER SAND & GRAVEL ALLUVIUM** Hard drilling below 52 feet, grinding action, possibly due to cobbles?	*79.0* 52.0		S-15		55
Very dense dark brown-gray to black SAND, trace silt; moist; fine to medium sand; moderately consolidated. (SP) Gravelly SAND layer from 63.5 to 65.0 feet.	*71.5* 59.5		S-16, S-17		60, 65
Completed - September 8, 2011	*64.5* 66.5				70, 75

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches



LEGEND

* Sample Not Recovered

Standard Penetration Test

3" O.D. Shelby Tube

Recovery (%)

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ⊢—⊣ Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

LOG OF BORING B-5

January 2012 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A6
Sheet 2 of 2

REV 1

Total Depth:	*31.5 ft.*	Northing:	~	Drilling Method:	*Mud Rotary*
Top Elevation:	*120.5*	Easting:	~	Drilling Company:	*Subsurface Technologies*
Vert. Datum:		Station:	~	Drill Rig Equipment:	*Mobil B-53*
Horiz. Datum:		Offset:	~	Other Comments:	

Hole Diam.: *5 in.* | Rod Type: *NWJ* | Hammer Type: *Automatic*

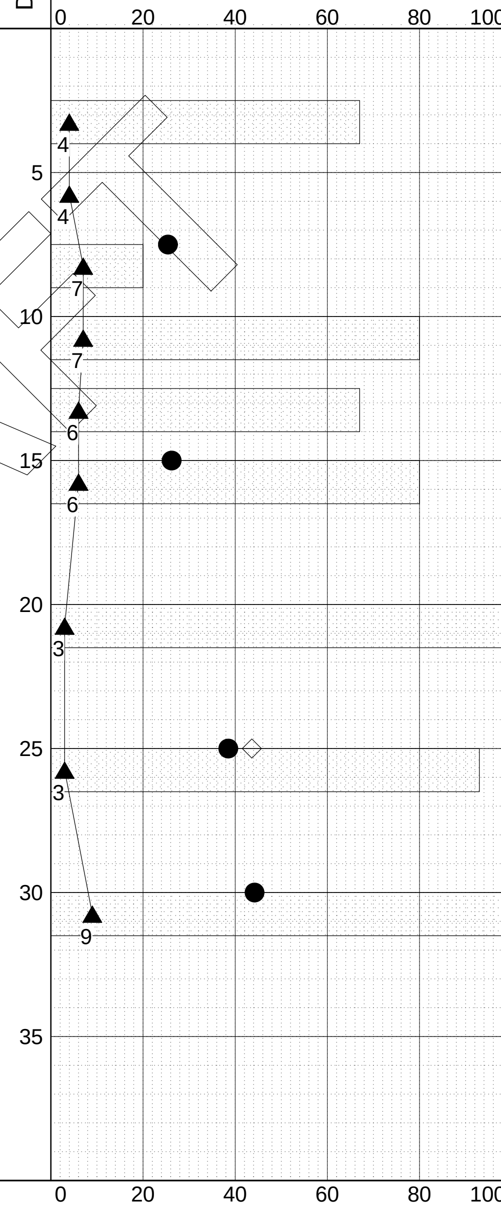
SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Soil Description	Elev. Depth (ft.)	Samples	Ground Water
PORTLAND CEMENT CONCRETE	*120.0* 0.5		
BASE AGGREGATE	*118.0* 2.5		
ARTIFICIAL FILL			
Soft brown SILT, trace sand; moist; nonplastic to low plasticity; fine sand; micaceous. (ML)	*113.5* 7.0	S-1, S-2 *	
Loose brown silty SAND; moist; nonplastic; fine sand; micaceous; stratified with occasional layers of SAND, trace silt or SILT. (SM)	*106.0* 14.5	S-3, S-4, S-5	
FINE-GRAINED FLOOD DEPOSITS			
Loose brown silty SAND; moist; nonplastic to low plasticity fines; fine to medium sand; stratified; micaceous. (SM)		S-6	
Grades to wet at 19.0 feet.			During Drilling
Numerous SILT layers from 20.0 to 22.0 feet.		S-7	
Slight iron oxidation and red-brown staining from 20.0 to 31.5 feet.	*89.0* 31.5	S-8, S-9	
Completed - September 12, 2011			

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: *140 lbs / 30 inches*



LEGEND

* Sample Not Recovered

Standard Penetration Test

Groundwater Level

Recovery (%)

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├──┤ Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

LOG OF BORING B-6

January 2012 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A7

REV 1

MASTER LOG E 24-1-03658-001.GPJ SHAN WIL.GDT 12/27/11

Log: CKS Rev: Typ: MAS/RRB

Total Depth:	*31.5 ft.*	Northing:	~	Drilling Method:	*Hollow Stem Auger*
Top Elevation:	*127.5*	Easting:	~	Drilling Company:	*Subsurface Technologies*
Vert. Datum:		Station:	~	Drill Rig Equipment:	*Mobil B-53*
Horiz. Datum:		Offset:	~	Other Comments:	

Hole Diam.:	*5 in.*
Rod Type:	*NWJ*
Hammer Type:	*Automatic*

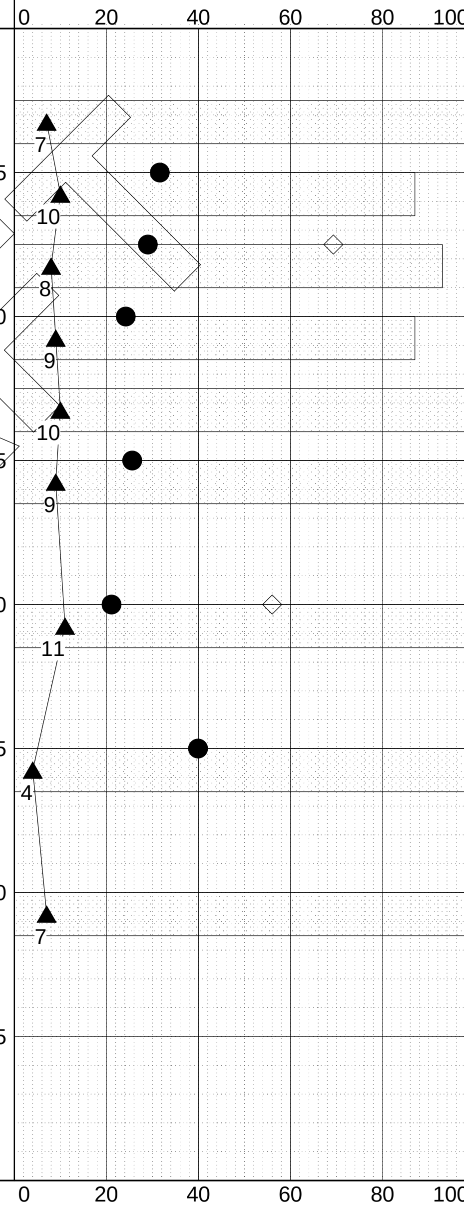
SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

SOIL DESCRIPTION	Elev. Depth (ft.)	Samples	Ground Water
PORTLAND CEMENT CONCRETE	*126.7* 0.8		
BASE AGGREGATE	*126.0* 1.5		
ARTIFICIAL FILL			
Medium stiff to stiff brown and gray with orange mottling SILT, trace sand grading to sandy SILT; moist; low plasticity fines; fine sand; stratified; occasional layers of silty SAND; micaceous. (ML) **FINE-GRAINED FLOOD DEPOSITS**	*115.5* 12.0	S-1, S-2, S-3, S-4	
Loose to medium dense brown silty SAND; moist; nonplastic to low plasticity fines; fine sand; stratified; micaceous. (SM) Slight iron oxidation and red-brown staining at 20.0 feet. Grades to wet at 25.0 feet.	*102.0* 25.5	S-5, S-6, S-7	During Drilling
Soft brown clayey SILT, trace sand; wet; low to medium plasticity; fine sand. (ML)	*101.2* 26.3	S-8	
Loose brown silty SAND; wet; fine to medium sand; stratified; micaceous. (SM)	*96.0* 31.5	S-9	
Completed - September 13, 2011			

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: *140 lbs / 30 inches*



LEGEND

* Sample Not Recovered

Standard Penetration Test

Groundwater Level

Recovery (%)

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit —— Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

LOG OF BORING B-7

January 2012 — 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A8

REV 1

Total Depth:	31.5 ft.	Northing:	~	Drilling Method:	Mud Rotary
Top Elevation:	131	Easting:	~	Drilling Company:	Subsurface Technologies
Vert. Datum:		Station:	~	Drill Rig Equipment:	Mobil B-53
Horiz. Datum:		Offset:	~	Other Comments:	

Hole Diam.:	5 in.
Rod Type:	NWJ
Hammer Type:	Automatic

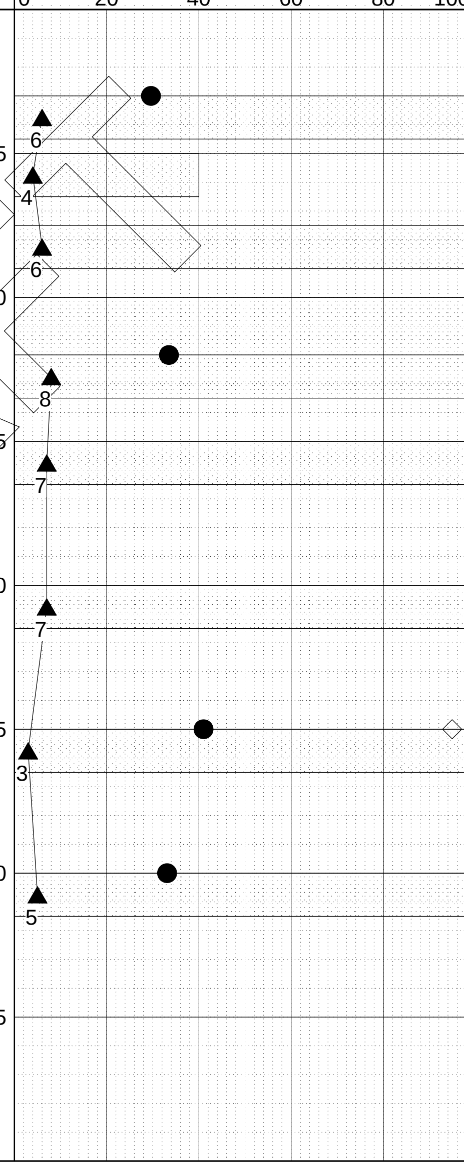
SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

SOIL DESCRIPTION	Elev. Depth (ft.)	Samples	Ground Water	Depth, ft.
ASPHALT	130.7 / 0.3			
BASE AGGREGATE	129.0 / 2.0			
Medium stiff to soft gray-brown and gray with orange-brown mottling clayey SILT: with sand grading to sandy SILT; moist; low to medium plasticity grading to low plasticity; fine sand; trace gravel and coarse sand in upper 5.0 feet; occasional scattered wood debris. (ML) **ARTIFICIAL FILL**	124.0 / 7.0	S-1, S-2, S-3		5
Medium stiff gray-brown with orange-brown mottling sandy SILT; moist; low plasticity; fine sand; micaceous. (ML) Grades to silty SAND to sandy SILT at 12.0 feet.	117.0 / 14.0	S-4, S-5		10
Loose brown sandy SILT to silty SAND; moist; nonplastic fines; fine sand; stratified; occasional layers of SAND, trace silt; micaceous. (ML/SM) **FINE-GRAINED FLOOD DEPOSITS** Interbedded silty SAND and SILT, trace sand at 25.0 feet.	104.0 / 27.0	S-6, S-7, S-8	During Drilling	15, 20, 25
Loose brown silty SAND; wet; fine to medium sand; stratified; micaceous. (SP) Slight iron oxidation and red-brown staining from 30.0 to 30.2 feet.	99.5 / 31.5	S-9		30
Completed - September 8, 2011				35

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches



LEGEND

- * Sample Not Recovered
- Standard Penetration Test
- 3" O.D. Shelby Tube
- Groundwater Level

- Recovery (%)
- ◇ % Fines (<0.075mm)
- ● % Water Content
- Plastic Limit ├──┤ Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

LOG OF BORING B-8

January 2012 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A9

REV 1

MASTER_LOG_E 24-1-03658-001.GPJ SHAN_WIL.GDT 12/27/11 Log: CKS Rev: Typ: MAS/RRB

Total Depth:	*31.5 ft.*	Northing:	~	Drilling Method:	*Mud Rotary*
Top Elevation:	*130*	Easting:	~	Drilling Company:	*Subsurface Technologies*
Vert. Datum:		Station:	~	Drill Rig Equipment:	*Mobil B-53*
Horiz. Datum:		Offset:	~	Other Comments:	

Hole Diam.:	*5 in.*
Rod Type:	*NWJ*
Hammer Type:	*Automatic*

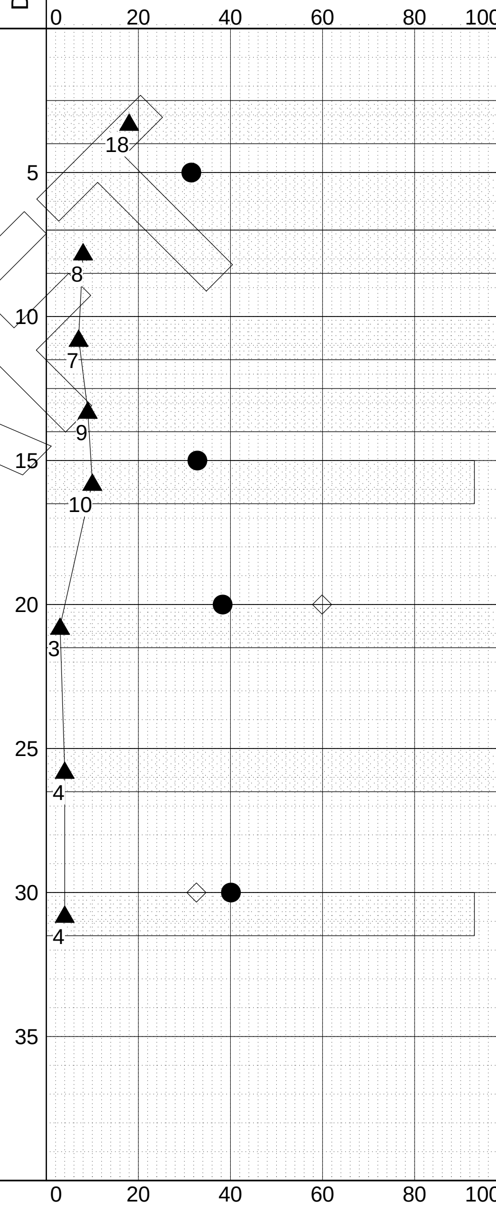
SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Soil Description	Elev. / Depth (ft.)	Samples	Ground Water
SOD	*129.8* / 0.2		
Very stiff to medium stiff gray and brown with orange-brown mottling SILT with sand grading to sandy SILT; moist; low plasticity; fine sand; slight iron oxidation and orange-brown staining; micaceous. (ML) **FINE-GRAINED FLOOD DEPOSITS**		S-1, S-2, S-3, S-4	
Loose to very loose brown sandy SILT to silty SAND; moist; nonplastic to low plasticity fines; fine sand; stratified with occasional to scattered layers of SAND, trace silt; slight iron oxidation and staining; micaceous. (ML/SM) Grades to wet at 20.0 feet.	*118.0* / 12.0	S-5, S-6, S-7	
Very loose brown silty SAND; wet; fine to medium sand; stratified; micaceous. (SM)	*107.0* / 23.0	S-8, S-9	During Drilling
Completed - September 9, 2011	*98.5* / 31.5		

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: *140 lbs / 30 inches*



Recovery (%)

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├──┤ Liquid Limit

LEGEND

* Sample Not Recovered

Standard Penetration Test

3" O.D. Shelby Tube

Groundwater Level

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

LOG OF BORING B-9

January 2012 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A10

REV 1

Elev. Depth	CLASSIFICATION OF MATERIAL	Ground Water	Samples No.	Samples Type	Samples wc %	Depth	Remarks
135.0	Medium stiff gray and brown sandy SILT, trace gravel; moist; nonplastic to low plasticity; fine sand occasional oranics and wood debris; micaceous. (ML) **ARTIFICIAL FILL**						
			S-1	B		5	
129.0 6.0	Soft to medium stiff dark brown clayey SILT, trace sand; moist; low plasticity; fine sand; numerous organics and wood debris (logs and branches); slight iron oxidation and staining. (ML)						
127.5 7.5	Medium dense gray sandy SILT; moist; nonplastic fines; fine sand; micaceous; weakly cemented. (ML)						
			S-2	B	32.6	9	
124.0 11.0	Medium dense bornw sandy SILT to silty SAND; moist; nonplastic fines; fine sand; slight iron oxidation and red-orange mottling; micaceous. (ML/SM) **FINE-GRAINED FLOOD DEPOSITS**						
			S-3	B		15	
119.5 15.5	Completed - September 13, 2011 No caving or sloughing. No seepage observed.						

LEGEND

B Bag Sample

NOTE:
Lines between units are approximate and transitions may be gradual.

Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

LOG OF TEST PIT TP-1

January 2012 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A11

TPLG 24-1-03658-001.GPJ 12/27/11

Elev. Depth	CLASSIFICATION OF MATERIAL	Ground Water	Samples No.	Samples Type	Samples wc %	Depth	Remarks
132.0	Medium dense gray and brown with orange mottling SILT with to trace sand; moist; low plasticity; fine sand; occasional layers of silty SAND or clayey SILT; slight iron oxidation and red-orange staining; micaceous. (ML) **FINE-GRAINED FLOOD DEPOSITS**		S-1	B		5	
126.0 6.0	Soft to loose gray and brown with orange-brown mottling sandy SILT; moist; stratified with scattered layers of silty SAND; and SAND, trace silt; micaceous. (ML)		S-2	B		10	
			S-3	B	28.9	15	
116.5 15.5	Completed - September 13, 2011 No caving or sloughing. No seepage observed.						

LEGEND

B Bag Sample

NOTE:
Lines between units are approximate and transitions may be gradual.

Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

LOG OF TEST PIT TP-2

January 2012 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A12

Elev. / Depth	CLASSIFICATION OF MATERIAL	Ground Water	Samples No.	Type	wc %	Depth	Remarks
129.0	Medium dense brown sandy silty GRAVEL with COBBLES; dry to moist; nonplastic fines; fine to medium sand; rounded to subrounded gravel and cobbles. (GM) **ARTIFICIAL FILL**						
127.5 / 1.5	Medium stiff gray and brown with orange-brown mottling SILT, trace sand; moist; nonplastic to low plasticity; fine sand; occasional layers of clayey SILT or silty SAND; micaceous. (ML) **FINE-GRAINED FLOOD DEPOSITS**		S-1	B	29.5	5.5	
			S-2	B	35.5	10.5	
115.0 / 14.0	Medium dense brown sandy SILT; moist; nonplastic; fine sand; stratified; occasional to scattered layers of SAND, trace silt; micaceous. (SM)		S-3	B		16.0	
112.5 / 16.5	Completed - September 13, 2011 No caving or sloughing. No seepage observed.						

LEGEND

B Bag Sample

NOTE:
Lines between units are approximate and transitions may be gradual.

Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

LOG OF TEST PIT TP-3

January 2012 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A13

TPLG 24-1-03658-001.GPJ 12/27/11

Elev. Depth	CLASSIFICATION OF MATERIAL	Ground Water	Samples No.	Type	wc %	Depth	Remarks
133.0	Medium stiff brown clayey SILT; trace sand; dry to moist; low plasticity fines; fine sand. (ML) **FINE-GRAINED FLOOD DEPOSITS**						
130.5 2.5	Soft to medium stiff gray and brown with orange-brown mottling SILT, trace sand grading to sandy SILT; moist; nonplastic to low plasticity; fine sand; slight iron oxidation and orange staining; micaceous. (ML)		S-1	Bag	29.5		
124.5 8.5	Medium dense brown SILT with sand; moist; nonplastic; fine sand; stratified with scattered layers of sandy SILT or SAND, trace silt; slight iron oxidation and orange staining; micaceous. (ML/SM)		S-2	Bag			
			S-3	Bag			
118.5 14.5	Completed - September 13, 2011 No caving or sloughing. No seepage observed.						

LEGEND

Bag Sample

NOTE:
Lines between units are approximate and transitions may be gradual.

Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

LOG OF TEST PIT TP-4

January 2012 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A14

Photograph 1: Test pit TP-1 excavated depth 0 to 15.5'

Photograph 2: Test pit TP-1 excavated material

DRAFT

Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

TEST PIT TP-1
PHOTOGRAPHS 1 and 2

January 2012 24-1-03658-245

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

A15

Photograph 3: Test pit TP-2
excavated depth 0 to 15.5'

Photograph 4: Test pit TP-2
excavated material

DRAFT

Lake Oswego & Tigard Water Treatment Plant Expansion Project West Linn, Oregon	
TEST PIT TP-2 **PHOTOGRAPHS 3 and 4**	
January 2012	24-1-03658-245
SHANNON & WILSON, INC. GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS	**A16**

Photograph 5: Test pit TP-3
excavated depth 0 to 16.5'

Photograph 6: Test pit TP-3
excavated material

DRAFT

Lake Oswego & Tigard Water Treatment Plant Expansion Project West Linn, Oregon	
TEST PIT TP-3 PHOTOGRAPHS 5 and 6	
January 2012	24-1-03658-245
SHANNON & WILSON, INC. GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS	**A17**

Photograph 7: Test pit TP-4
excavated depth 0 to 14.5'

Photograph 8: Test pit TP-4
excavated material

DRAFT

Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

TEST PIT TP-4
PHOTOGRAPHS 7 and 8

January 2012 24-1-03658-245

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

A18

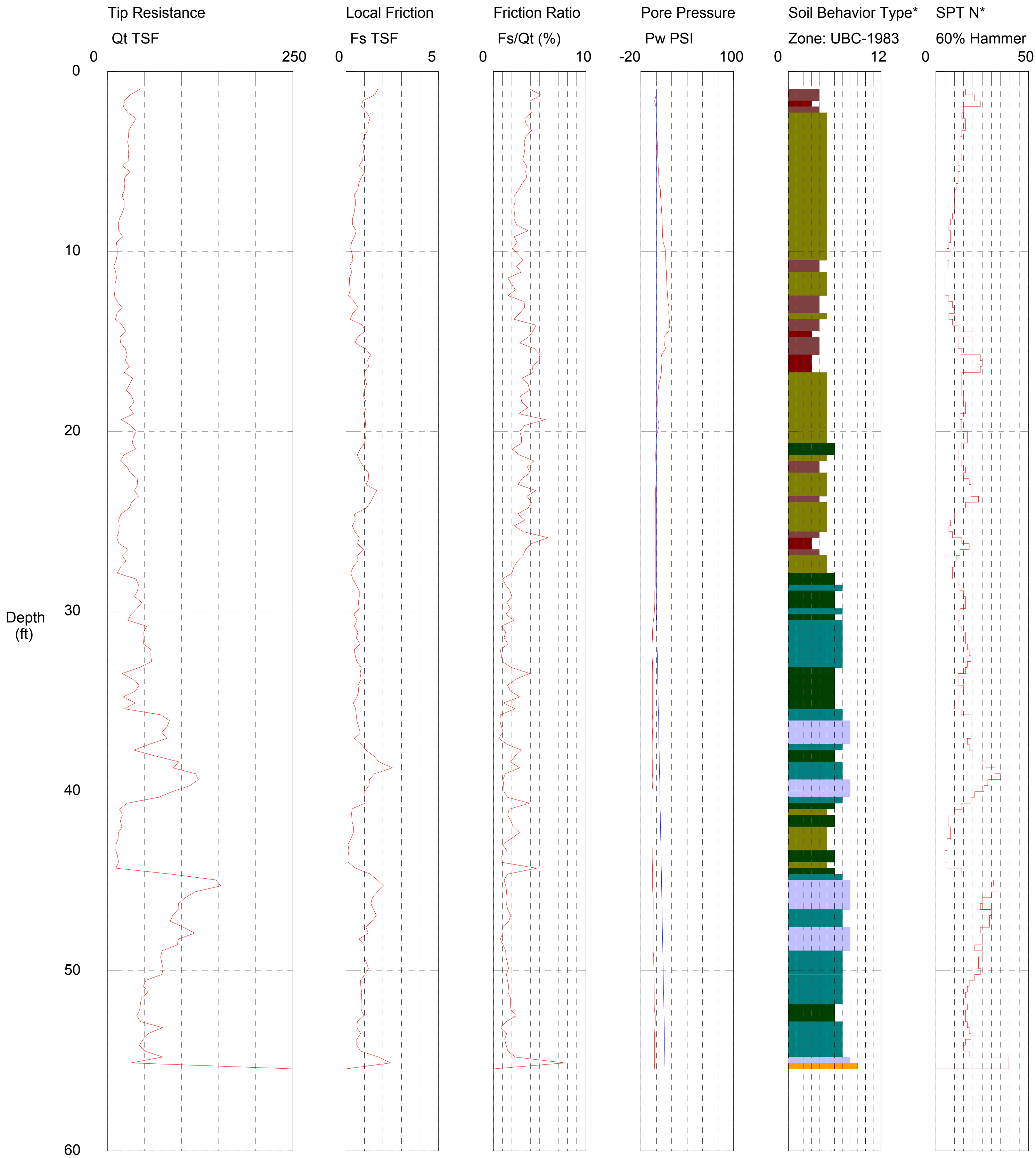
ATTACHMENT

SUBSURFACE TECHNOLOGIES CONE PENETRATION TEST RESULTS (CPT-1 THROUGH CPT-7)

Subsurface Technologies

Operator: SAM
Sounding: CPT-1
Cone Used: DDG1170

CPT Date/Time: 9/6/2011 12:37:30 PM
Location: LAKE OSWEGO WWTP
Job Number: 24-1-03658



Maximum Depth = 55.45 feet

Depth Increment = 0.328 feet

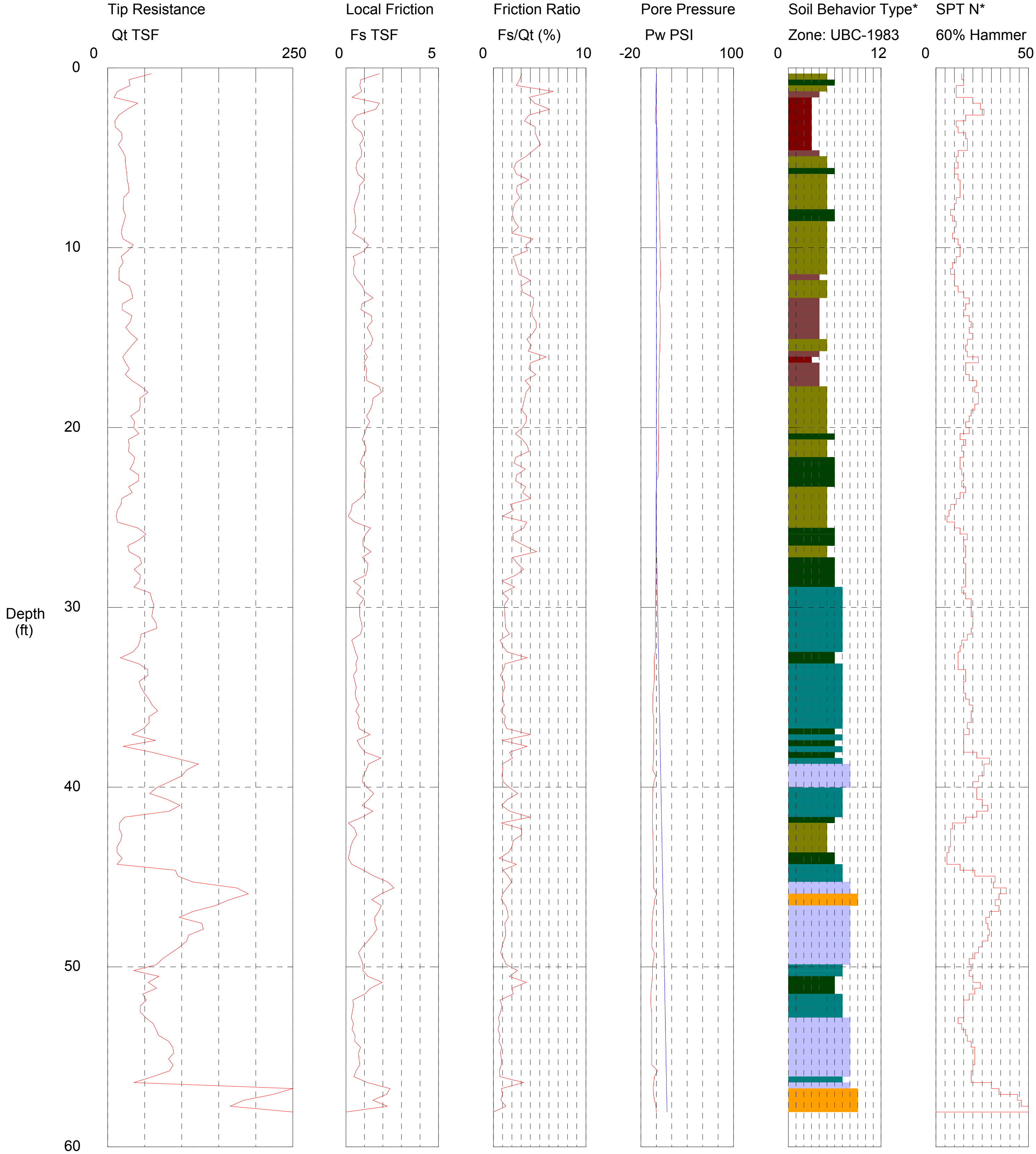
1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

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

Subsurface Technologies

Operator: SAM
Sounding: CPT-2
Cone Used: DDG1170

CPT Date/Time: 9/6/2011 1:32:04 PM
Location: LAKE OSWEGO WWTP
Job Number: 24-1-03658



Maximum Depth = 58.07 feet

Depth Increment = 0.328 feet

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

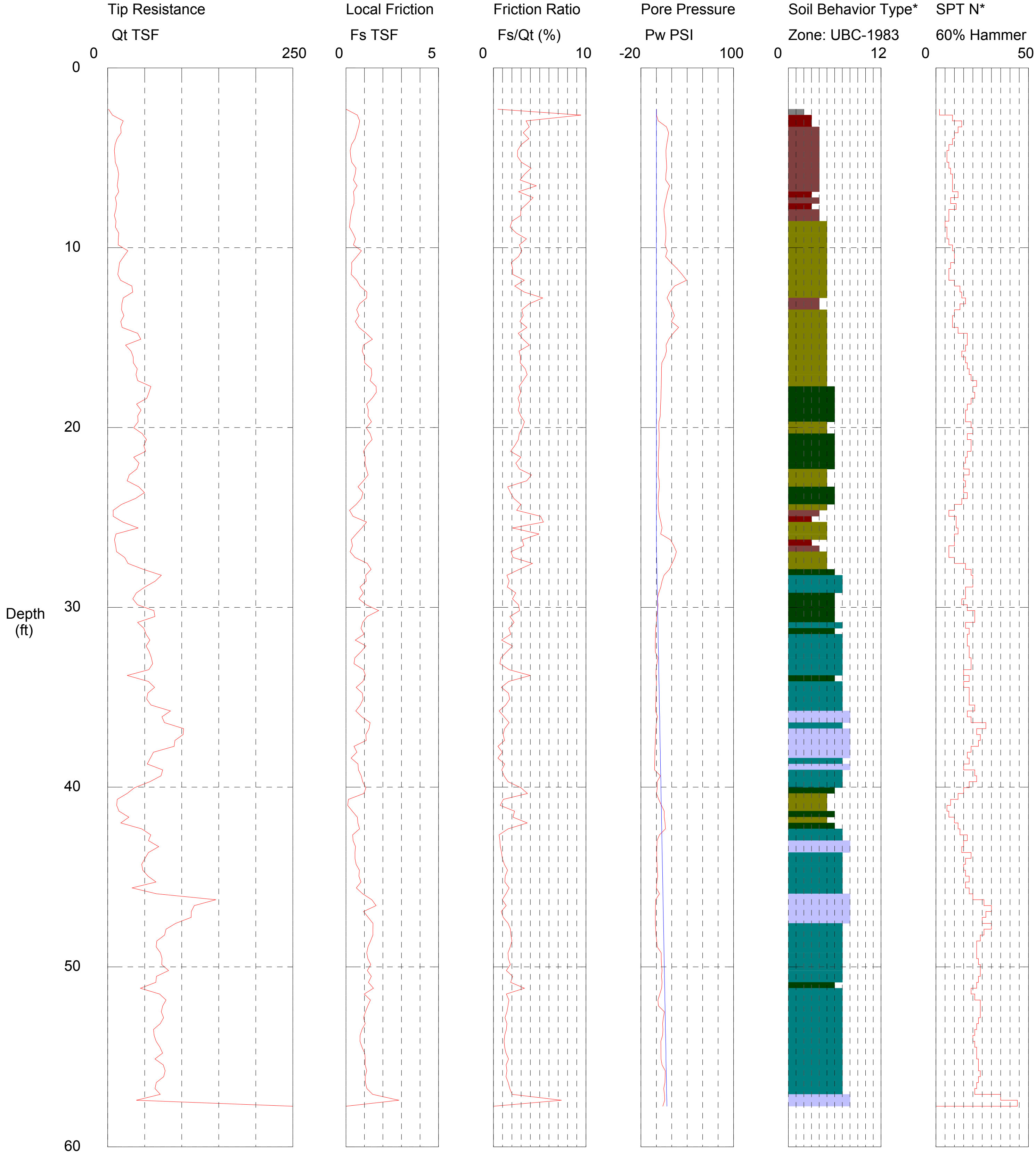
WWTP ò□

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

Subsurface Technologies

Operator: SAM
Sounding: CPT-3
Cone Used: DDG1170

CPT Date/Time: 9/7/2011 9:48:56 AM
Location: LAKE OSWEGO WWTP
Job Number: 24-1-03658



Maximum Depth = 57.74 feet

Depth Increment = 0.328 feet

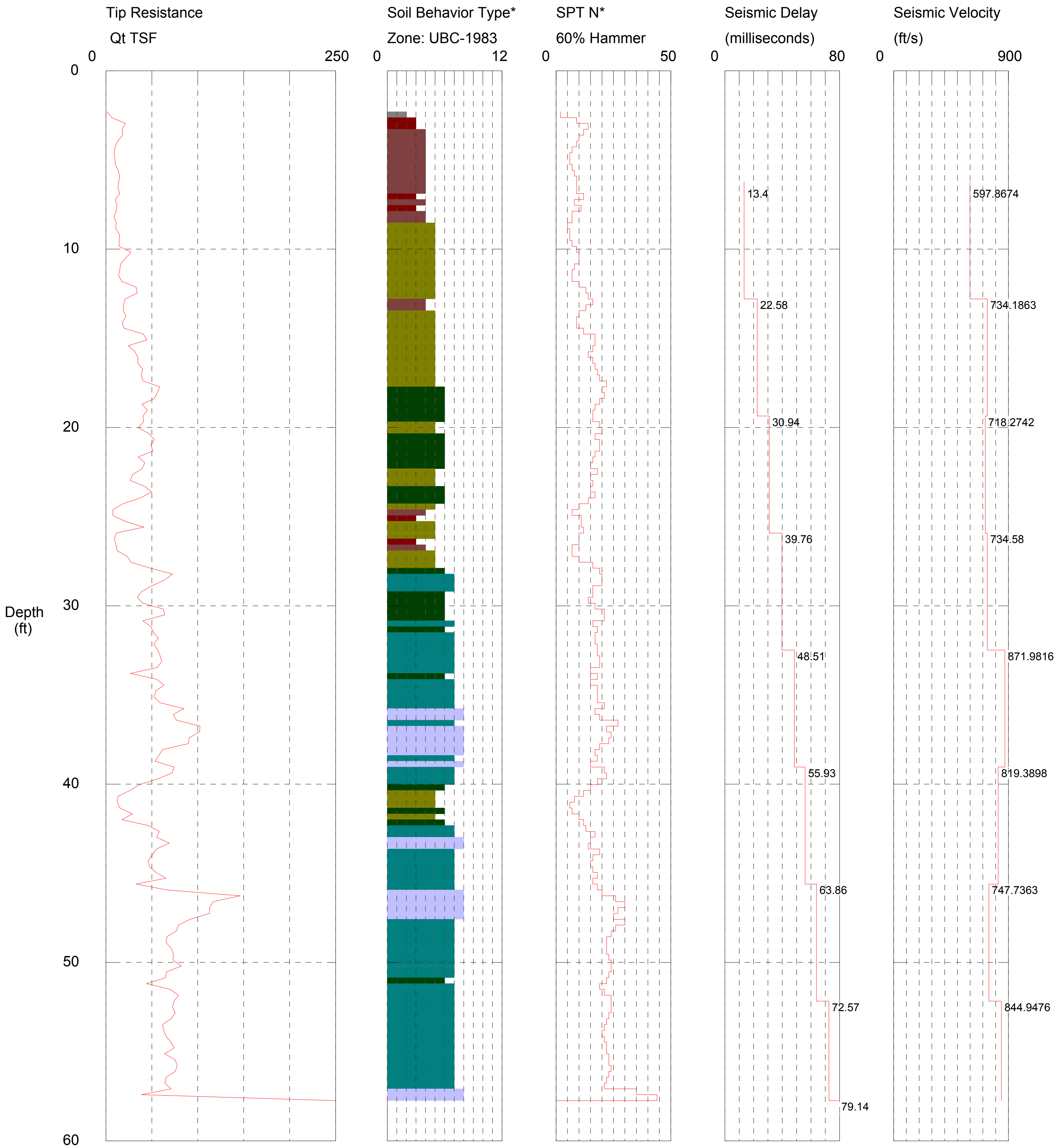
1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

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

Subsurface Technologies

Operator: SAM
Sounding: CPT-3
Cone Used: DDG1170

CPT Date/Time: 9/7/2011 9:48:56 AM
Location: LAKE OSWEGO WWTP
Job Number: 24-1-03658



Maximum Depth = 57.74 feet

Depth Increment = 0.328 feet

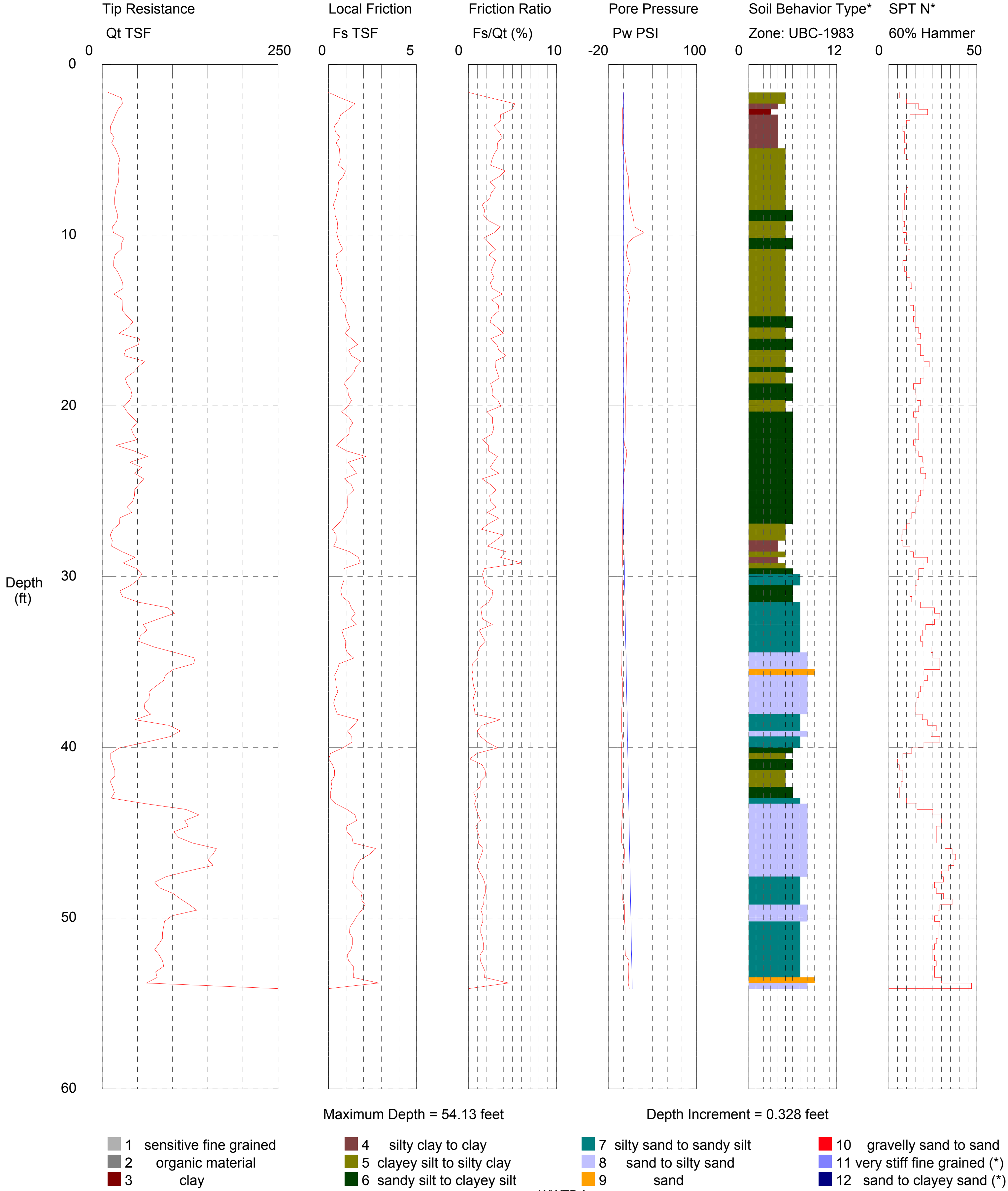
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

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

Subsurface Technologies

Operator: SAM
Sounding: CPT-4
Cone Used: DDG1170

CPT Date/Time: 9/7/2011 11:24:24 AM
Location: LAKE OSWEGO WWTP
Job Number: 24-1-03658



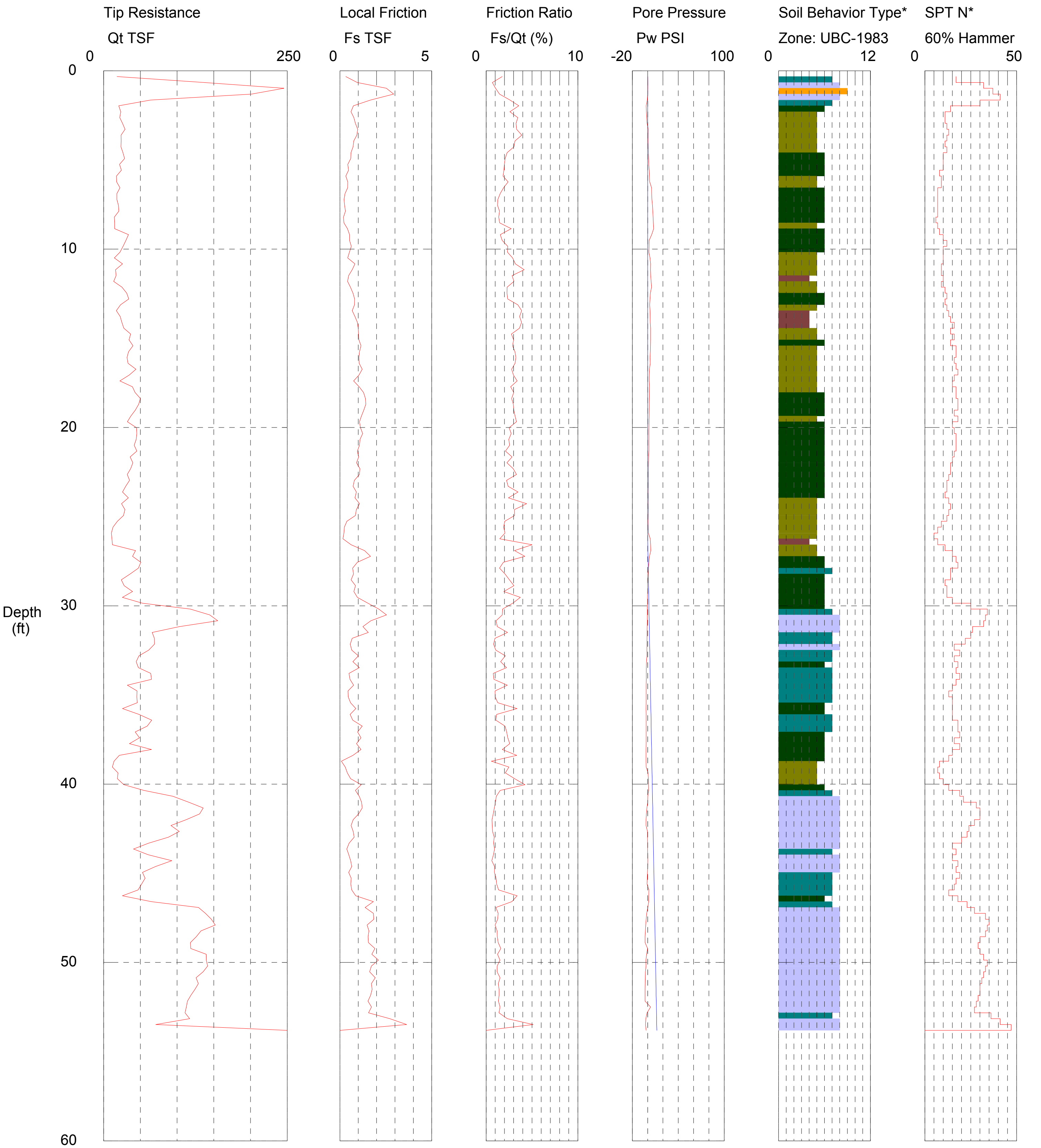
WWTP ò

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

Subsurface Technologies

Operator: SAM
Sounding: CPT-5
Cone Used: DDG1170

CPT Date/Time: 9/6/2011 2:24:36 PM
Location: LAKE OSWEGO WWTP
Job Number: 24-1-03658



Maximum Depth = 53.81 feet

Depth Increment = 0.328 feet

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

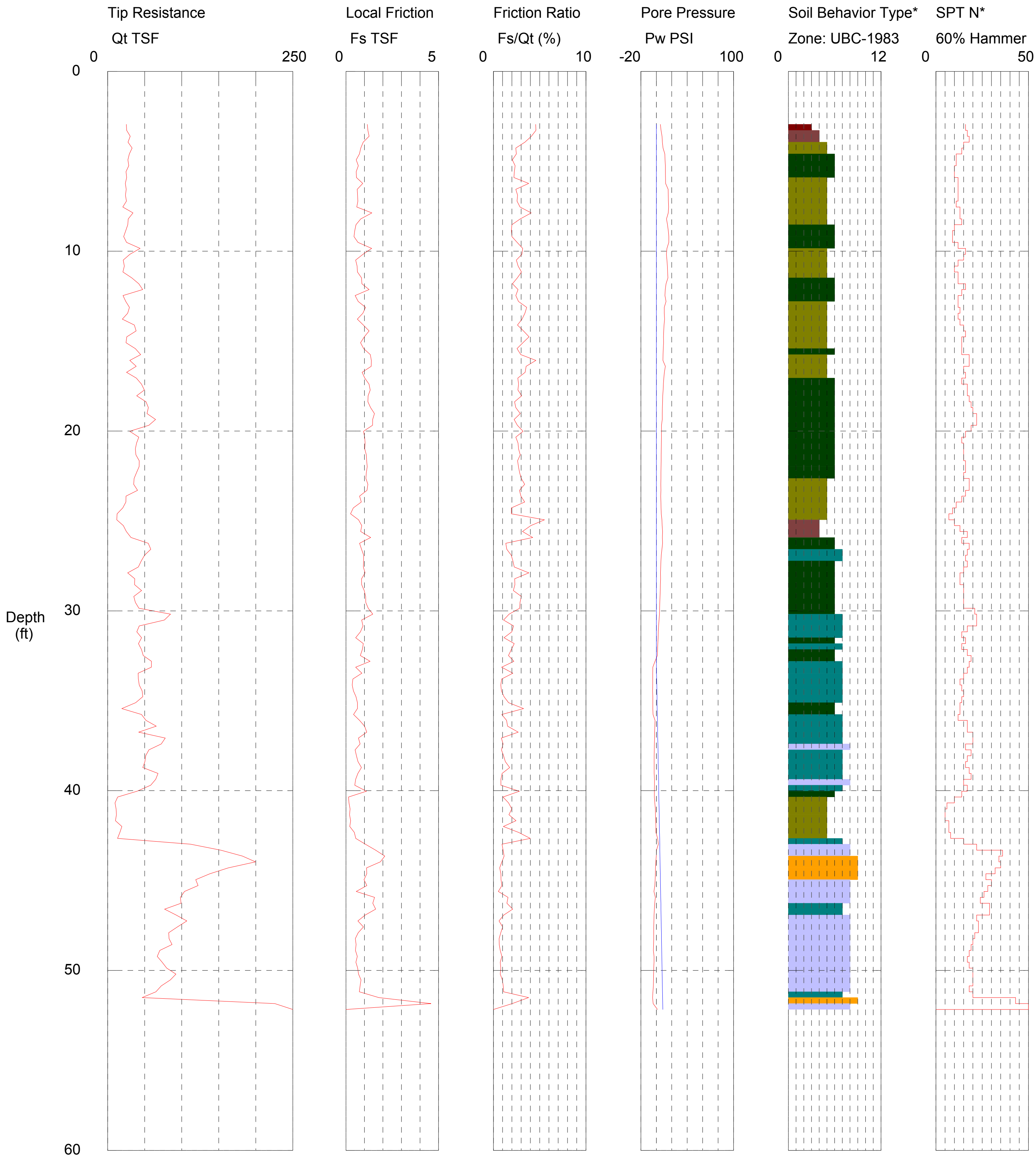
WWTP ò

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

Subsurface Technologies

Operator: SAM
Sounding: CPT-6
Cone Used: DDG1170

CPT Date/Time: 9/6/2011 9:56:27 AM
Location: LAKE OSWEGO WWTP
Job Number: 24-1-03658



Maximum Depth = 52.17 feet

Depth Increment = 0.328 feet

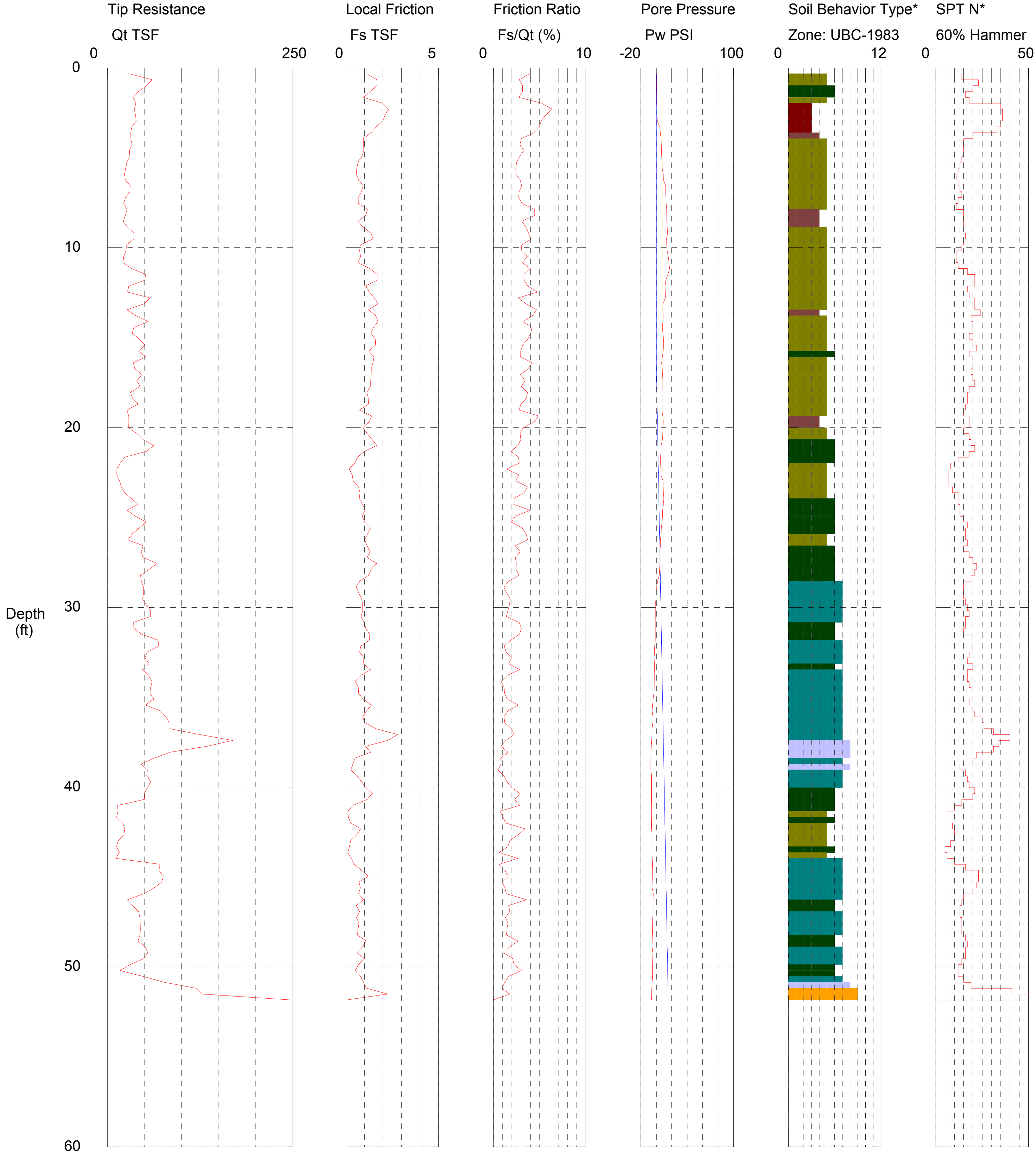
1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

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

Subsurface Technologies

Operator: SAM
Sounding: CPT-7
Cone Used: DDG1170

CPT Date/Time: 9/6/2011 11:06:14 AM
Location: LAKE OSWEGO WWTP
Job Number: 24-1-03658



Maximum Depth = 51.84 feet

Depth Increment = 0.328 feet

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

WWTP ò

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

APPENDIX B

LABORATORY TESTING PROGRAM

Appendix Contents:

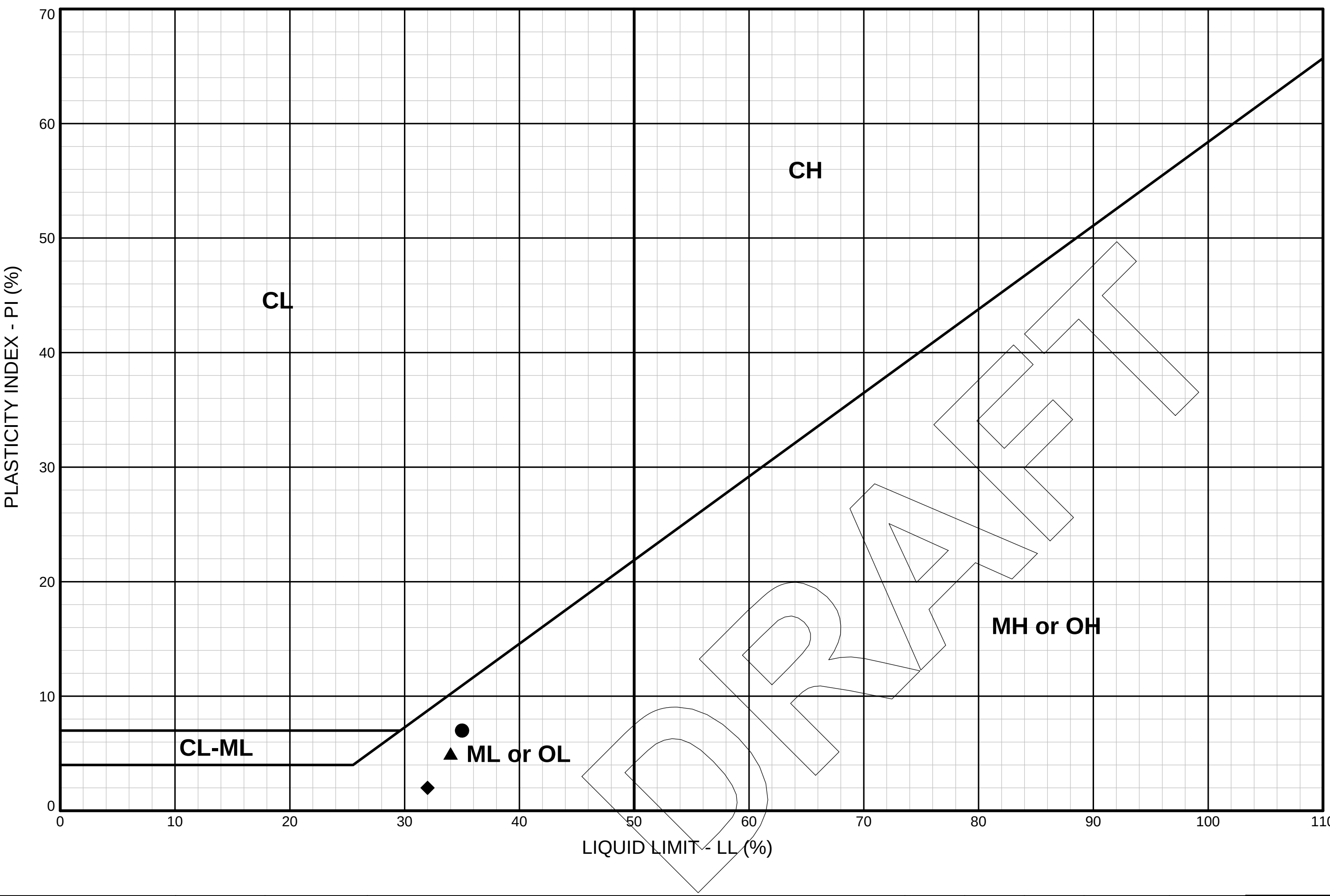
Table B1 Laboratory Test Summary

Boring	Top Depth (feet)	Sample Number	Sample Type	Blow Count (field)	USCS	Sample Description	Water Content (%)	Dry Unit Weight (pcf)	Gravel (%)	Sand (%)	Fines (%)	<2mic (%)	Liquid Limit (%)	Plastic Limit (%)	Other
B-1	2.5	S-1	SPT	8											
B-1	5	S-2	SPT	3											
B-1	7.5	S-3	SPT	0											
B-1	10	S-4	TW												
B-1	15	S-5	SPT	12			35.2								
B-1	20	S-6	SPT	6											
B-1	25	S-7	SPT	3			49								
B-1	30	S-8	SPT	5	SM	Silty SAND	36		0	58.9	41.1				
B-1	35	S-9	SPT	13											
B-1	40	S-10	SPT	3	ML	SILT, low plasticity	39.6				98.3		35	28.3	
B-1	45	S-11	SPT	10											
B-1	50	S-12	SPT	15			29								
B-1	55	S-13	SPT	50/5.5"	SP	SAND with gravel, trace silt	17		24.7	68.6	6.7				
B-1	60	S-14	SPT	50/1st 1"											
B-1	65	S-15	SPT	77			21.7								
B-2	2.5	S-1	SPT	4											
B-2	5	S-2	SPT	9											
B-2	7.5	S-3	SPT	4			44.4								
B-2	10	S-4	TW												
B-2	12	S-5	SPT	6			29.6								
B-2	15	S-6	SPT	5											
B-2	20	S-7	SPT	5			37.6								
B-2	25	S-8	SPT	5	ML	SILT with sand	40.5				81.8				
B-2	30	S-9	SPT	3			43.6								
B-2	35	S-10	SPT	8	SM	Silty SAND	36.3		0	61	39				
B-2	40	S-11	SPT	16			32.5								
B-2	45	S-12	SPT	5											
B-2	50	S-13	SPT	23											
B-2	55	S-14	SPT	14			34.5								
B-2	60	S-15	SPT	50/1st 3"											
B-2	65	S-16	SPT	60			25.6								
B-3	2.5	S-1	SPT	5			29.6								
B-3	5	S-2	SPT	2			35								
B-3	7.5	S-3	SPT	5			41.5								
B-3	10	S-4	TW		ML	SILT, nonplastic	29.8						NP	NP	Consolidation Test
B-3	12	S-5	SPT	4			35.4								
B-3	15	S-6	SPT	8			28.7								
B-3	20	S-7	SPT	8			32.1								
B-3	25	S-8	TW				30.2								

Boring	Sample Top Depth	Sample Number	Sample Type	N-value	USCS	Sample Description	Water Content (%)	Dry Unit Weight (pcf)	Gravel (%)	Sand (%)	Fines (%)	<2mic (%)	Liquid Limit (%)	Plastic Limit (%)	Other
B-3	30	S-9	SPT	5	SM	Silty SAND	31.9		0	62.4	37.6				
B-3	35	S-10	SPT	12			28.7								
B-3	40	S-11	SPT	3	ML	SILT, low plasticity	40.4						34.4	28.9	
B-3	45	S-12	SPT	24			25.6								
B-3	50	S-13	SPT	10	SM	Silty SAND	38.5		0	64.6	35.4	3.9			
B-3	55	S-14	SPT	50/1st 5"			19.5								
B-3	60	S-15	SPT	50/1st 4.5"			13.8								
B-3	65	S-16	SPT	58			30								
B-4	2.5	S-1	SPT	10											
B-4	5	S-2	SPT	10			31								
B-4	7.5	S-3	SPT	7											
B-4	10	S-4	SPT	6			30.4								
B-4	12.5	S-5	SPT	8											
B-4	15	S-6	TW												
B-4	17	S-7	SPT	12			26								
B-4	20	S-8	SPT	6											
B-4	22.5	S-9	SPT	4	ML	Sandy SILT	39.4		0	25	75				
B-4	25	S-10	SPT	3											
B-4	27.5	S-11	SPT	6											
B-4	30	S-12	SPT	5											
B-4	35	S-13	SPT	9	SM	Silty SAND	34.5		0	67.3	32.7	2.7			
B-4	40	S-14	SPT	4	ML	SILT, nonplastic	43.5						32.3	30.1	
B-4	45	S-15	TW												
B-4	46.3	S-16	SPT	22											
B-4	50	S-17	SPT	23	SM	SAND with silt	35.3				16				
B-4	55	S-18	SPT	32											
B-4	60	S-19	SPT	50/1st 3.5"											
B-4	65	S-20	SPT	64											
B-5	2.5	S-1	SPT	8			34.3								
B-5	5	S-2	SPT	14											
B-5	7.5	S-3	SPT	10			35.3								
B-5	10	S-4	SPT	9											
B-5	12.5	S-5	SPT	12			28.1								
B-5	15	S-6	SPT	13											
B-5	20	S-7	TW				39.5								
B-5	21.7	S-8	SPT	7											
B-5	25	S-9	SPT	6	ML	Sandy SILT	33		0	41.9	58.1				
B-5	30	S-10	SPT	4			46.5								
B-5	35	S-11	SPT	10											
B-5	40	S-12	SPT	8	SM	Silty SAND	38				47				
B-5	45	S-13	SPT	17			38.1								

Boring	Sample Top Depth	Sample Number	Sample Type	N-value	USCS	Sample Description	Water Content (%)	Dry Unit Weight (pcf)	Gravel (%)	Sand (%)	Fines (%)	<2mic (%)	Liquid Limit (%)	Plastic Limit (%)	Other
B-5	50	S-14	SPT	8											
B-5	55	S-15	SPT	54			17.2								
B-5	60	S-16	SPT	52											
B-5	65	S-17	SPT	71			24.8								
B-6	2.5	S-1	SPT	4											
B-6	5	S-2	SPT	4											
B-6	7.5	S-3	SPT	7			25.4								
B-6	10	S-4	SPT	7											
B-6	12.5	S-5	SPT	6											
B-6	15	S-6	SPT	6			26.2								
B-6	20	S-7	SPT	3											
B-6	25	S-8	SPT	3	SM	Silty SAND	38.5		0	56.4	43.6				
B-6	30	S-9	SPT	9			44.2								
B-7	2.5	S-1	SPT	7											
B-7	5	S-2	SPT	10			31.6								
B-7	7.5	S-3	SPT	8	ML	Sandy SILT	29				69.3				
B-7	10	S-4	SPT	9			24.2								
B-7	12.5	S-5	SPT	10											
B-7	15	S-6	SPT	9			25.6								
B-7	20	S-7	SPT	11	ML	Sandy SILT	21.1				56				
B-7	25	S-8	SPT	4			39.9								
B-7	30	S-9	SPT	7											
B-8	3	S-1	SPT	6			29.6								
B-8	5	S-2	SPT	4											
B-8	7.5	S-3	SPT	6											
B-8	10	S-4	TW												
B-8	12	S-5	SPT	8			33.5								
B-8	15	S-6	SPT	7											
B-8	20	S-7	SPT	7											
B-8	25	S-8	SPT	3	ML	SILT, trace sand	41		0	5.1	94.9				
B-8	30	S-9	SPT	5			33.1								
B-9	2.5	S-1	SPT	18											
B-9	5	S-2	TW				31.5								
B-9	7	S-3	SPT	8											
B-9	10	S-4	SPT	7											
B-9	12.5	S-5	SPT	9											
B-9	15	S-6	SPT	10			32.8								
B-9	20	S-7	SPT	3	ML	Sandy SILT	38.3				59.9				
B-9	25	S-8	SPT	4											
B-9	30	S-9	SPT	4	SM	Silty SAND	40.1				32.6				
TP-1	5	S-1	BAG												

Boring	Sample Top Depth	Sample Number	Sample Type	N-value	USCS	Sample Description	Water Content (%)	Dry Unit Weight (pcf)	Gravel (%)	Sand (%)	Fines (%)	<2mic (%)	Liquid Limit (%)	Plastic Limit (%)	Other
TP-1	9	S-2	BAG		ML	Sandy SILT	32.6		0	38.8	61.2	11			
TP-1	15	S-3	BAG												
TP-2	5	S-1	BAG												
TP-2	9.5	S-2	BAG												
TP-2	15	S-3	BAG		ML	Sandy SILT	28.9		0	32.8	67.2	12			
TP-3	5.5	S-1	BAG		ML	SILT with clay	29.5								Standard Proctor
TP-3	10.5	S-2	BAG		ML	SILT, trace sand	35.5		0	9.4	90.6	3.2			
TP-3	16	S-3	BAG												
TP-4	5.5	S-1	BAG		ML	SILT, trace sand	29.5		0	9.2	90.8	10.9			
TP-4	9.5	S-2	BAG												
TP-4	14	S-3	BAG												



LEGEND

CL: Low plasticity inorganic clays; sandy and silty clays

CH: High plasticity inorganic clays

ML or OL: Inorganic and organic silts and clayey silts of low plasticity

MH or OH: Inorganic and organic silts and clayey silts of high plasticity

CL-ML: Silty clays and clayey silts

NOTE

Atterberg limits tests performed in general accordance with ASTM D4318 unless otherwise noted in the report.

BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	LL %	PL %	PI %	NAT. W.C. %	FINES %
● B-1, S-10	40.0	ML	SILT, low plasticity	35	28	7	40	98
B-3, S-4	10.0	ML	SILT, nonplastic	NP	NP	NP	30	
▲ B-3, S-11	40.0	ML	SILT, low plasticity	34	29	5	40	
◆ B-4, S-14	40.0	ML	SILT, nonplastic	32	30	2	44	

Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

ATTERBERG LIMITS RESULTS

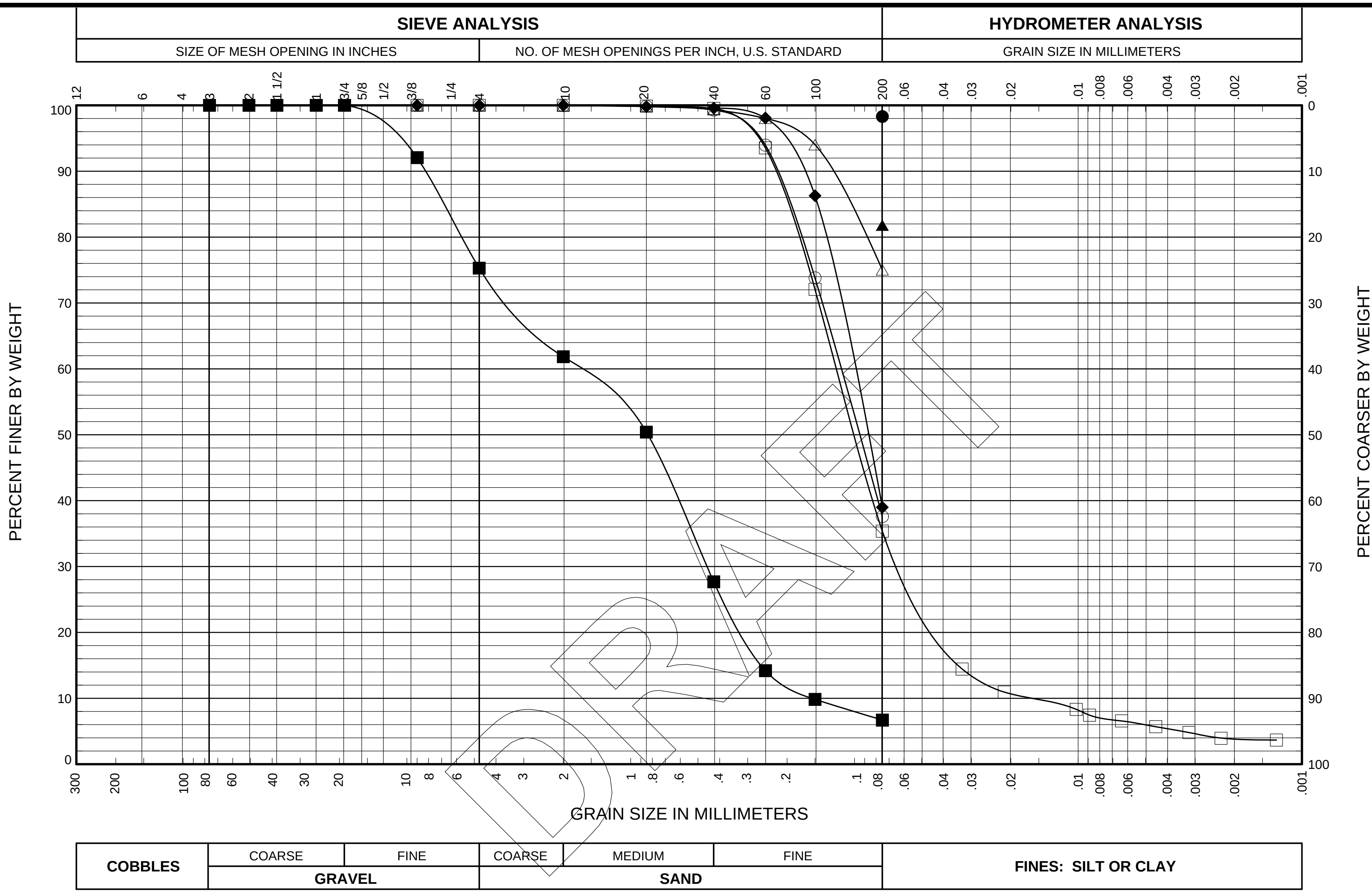
December 2011 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B1

ATT MAIN 24-1-03658-001.GPJ SHAN WIL.GDT 12/13/11

FIG. B1



BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	GRAVEL %	SAND %	FINES %	NAT. W.C. %	DRY DENSITY PCF
● B-1, S-10	40.0	ML	SILT, low plasticity	-	-	98	40	
■ B-1, S-13	55.0	SP	SAND with gravel, trace silt	25	69	7	17	
▲ B-2, S-8	25.0	ML	SILT with sand	-	-	82	41	
◆ B-2, S-10	35.0	SM	Silty SAND	0	61	39	36	
○ B-3, S-9	30.0	SM	Silty SAND	0	62	38	32	
□ B-3, S-13	50.0	SM	Silty SAND	0	65	35	39	
△ B-4, S-9	22.5	ML	Sandy SILT	0	25	75	39	

Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

GRAIN SIZE DISTRIBUTION

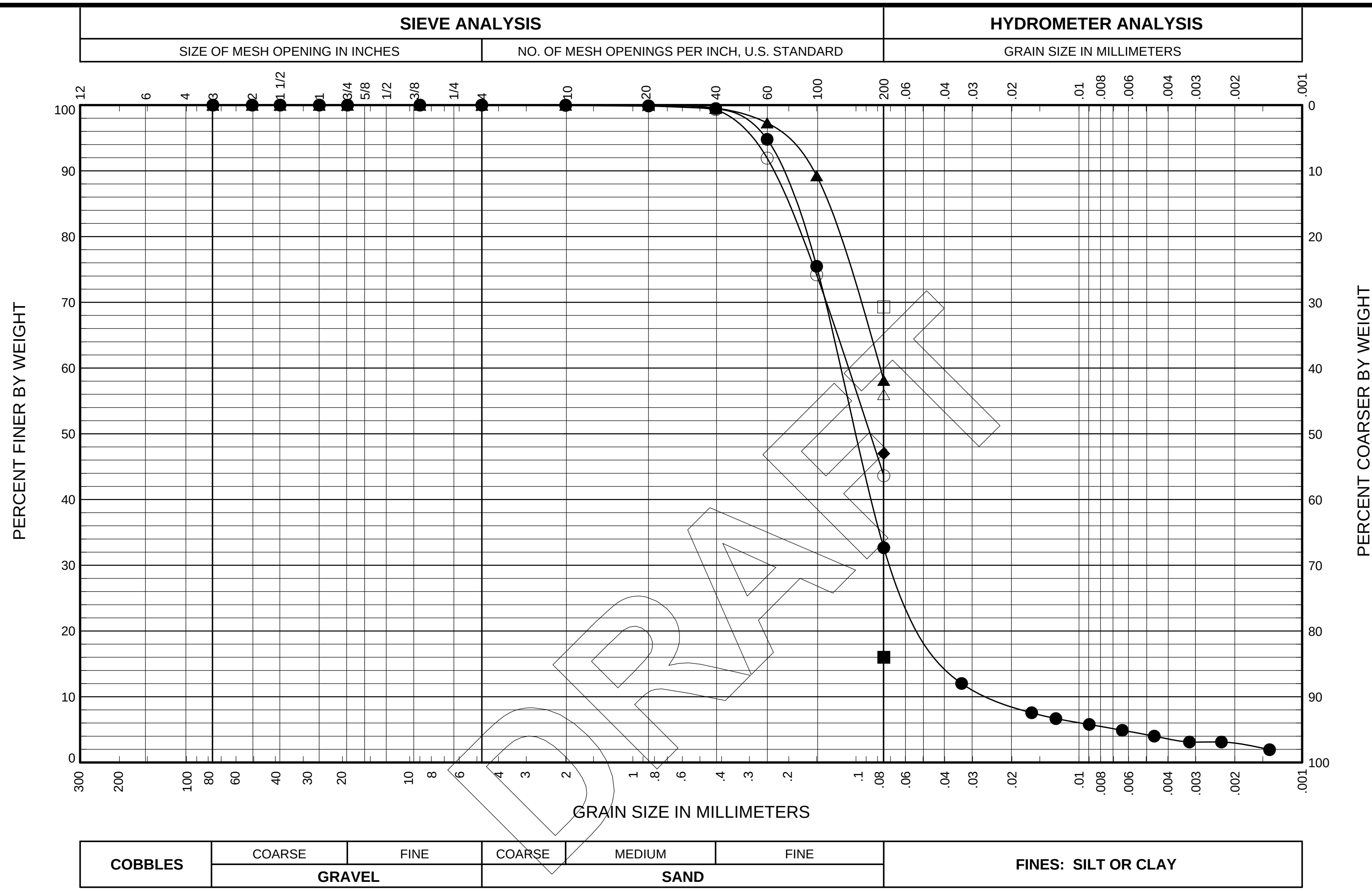
December 2011 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B2
Sheet 1 of 3

FIG. B2

GSA MAIN 24-1-03658-001.GPJ SHAN WIL.GDT 12/13/11



BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	GRAVEL %	SAND %	FINES %	NAT. W.C. %	DRY DENSITY PCF
● B-4, S-13	35.0	SM	Silty SAND	0	67	33	35	
■ B-4, S-17	50.0	SM	SAND with silt	-	-	16	35	
▲ B-5, S-9	25.0	ML	Sandy SILT	0	42	58	33	
◆ B-5, S-12	40.0	SM	Silty SAND	-	-	47	38	
○ B-6, S-8	25.0	SM	Silty SAND	0	56	44	39	
□ B-7, S-3	7.5	ML	Sandy SILT	-	-	69	29	
△ B-7, S-7	20.0	ML	Sandy SILT	-	-	56	21	

Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

GRAIN SIZE DISTRIBUTION

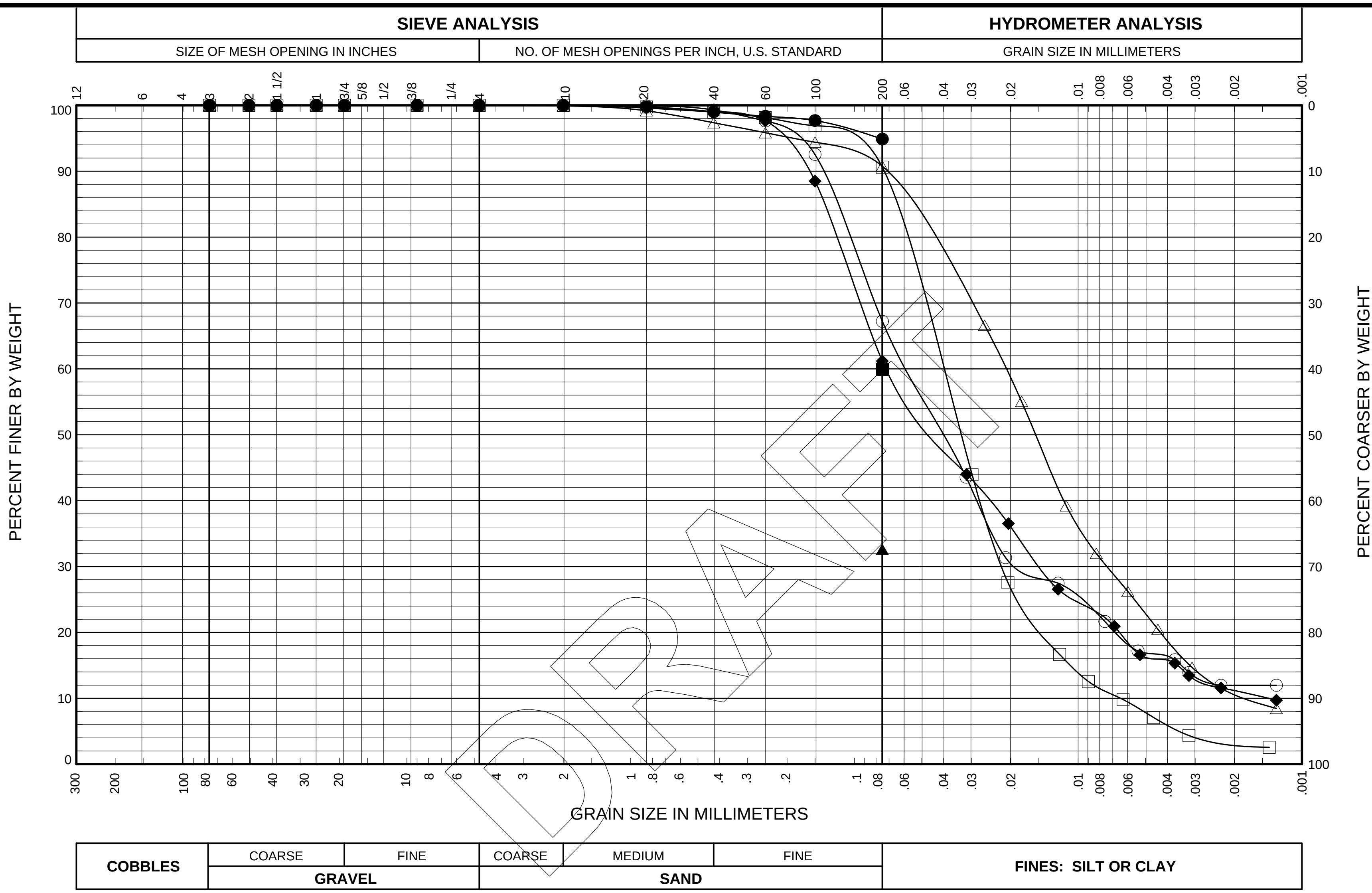
December 2011 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B2
Sheet 2 of 3

GSA MAIN 24-1-03658-001.GPJ SHAN WIL.GDT 12/13/11

FIG. B2



BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	GRAVEL %	SAND %	FINES %	NAT. W.C. %	DRY DENSITY PCF
● B-8, S-8	25.0	ML	SILT, trace sand	0	5	95	41	
■ B-9, S-7	20.0	ML	Sandy SILT	-	-	60	38	
▲ B-9, S-9	30.0	SM	Silty SAND	-	-	33	40	
◆ TP-1, S-2	9.0	ML	Sandy SILT	0	39	61	33	
○ TP-2, S-3	15.0	ML	Sandy SILT	0	33	67	29	
□ TP-3, S-2	10.5	ML	SILT, trace sand	0	9	91	36	
△ TP-4, S-1	5.5	ML	SILT, trace sand	0	9	91	30	

Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

GRAIN SIZE DISTRIBUTION

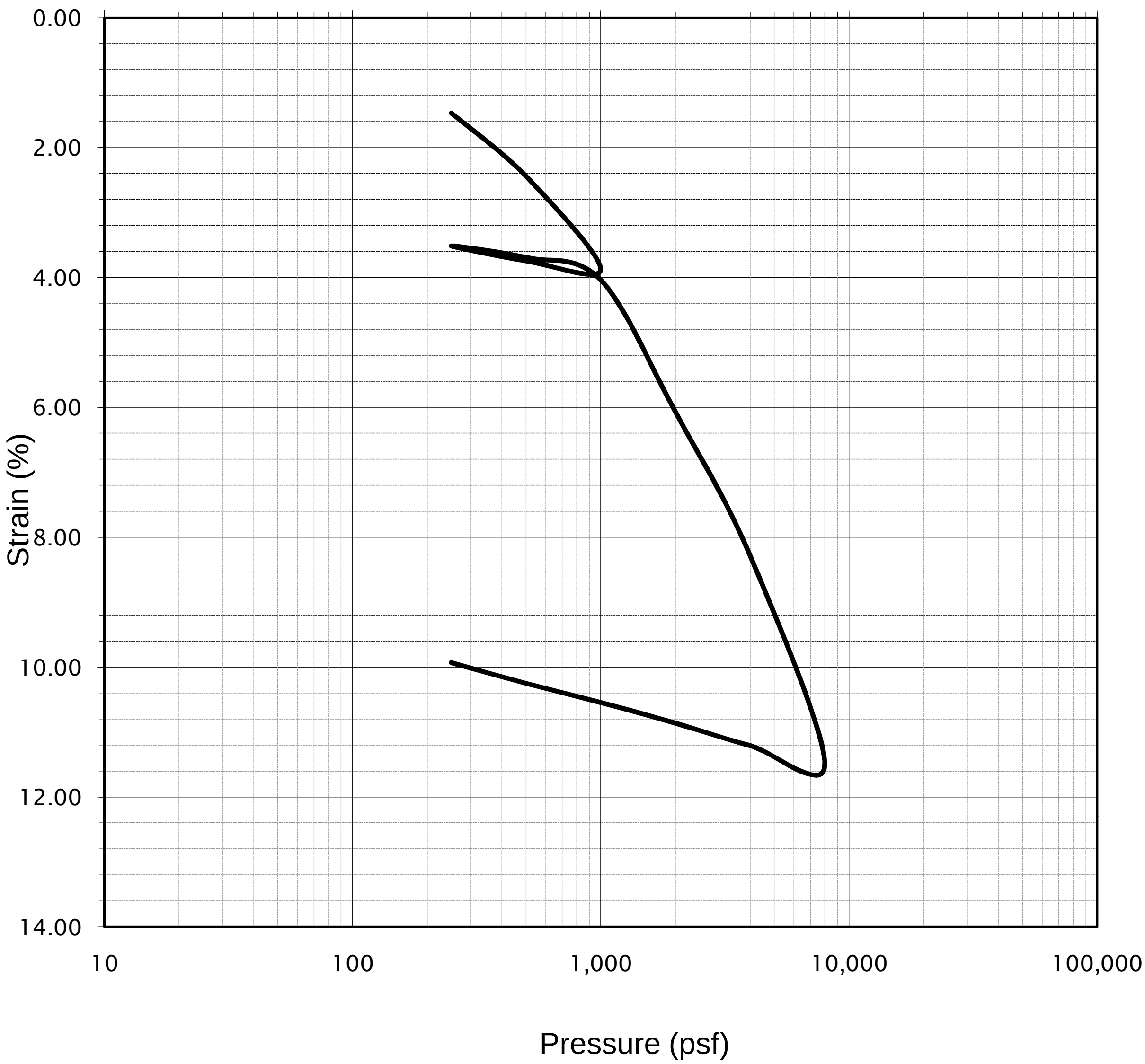
December 2011 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B2
Sheet 3 of 3

FIG. B2

GSA MAIN 24-1-03658-001.GPJ SHAN WIL.GDT 12/13/11



Boring & Sample No.	Depth (feet)	U.S.C.S Symbol	Sample Description	Natural W.C. (%)	Wet Unit Wt (pcf)	Dry Unit Wt (pcf)
B-3; S-4	10	SM	SILT	33.1	106.7	80

DRAFT

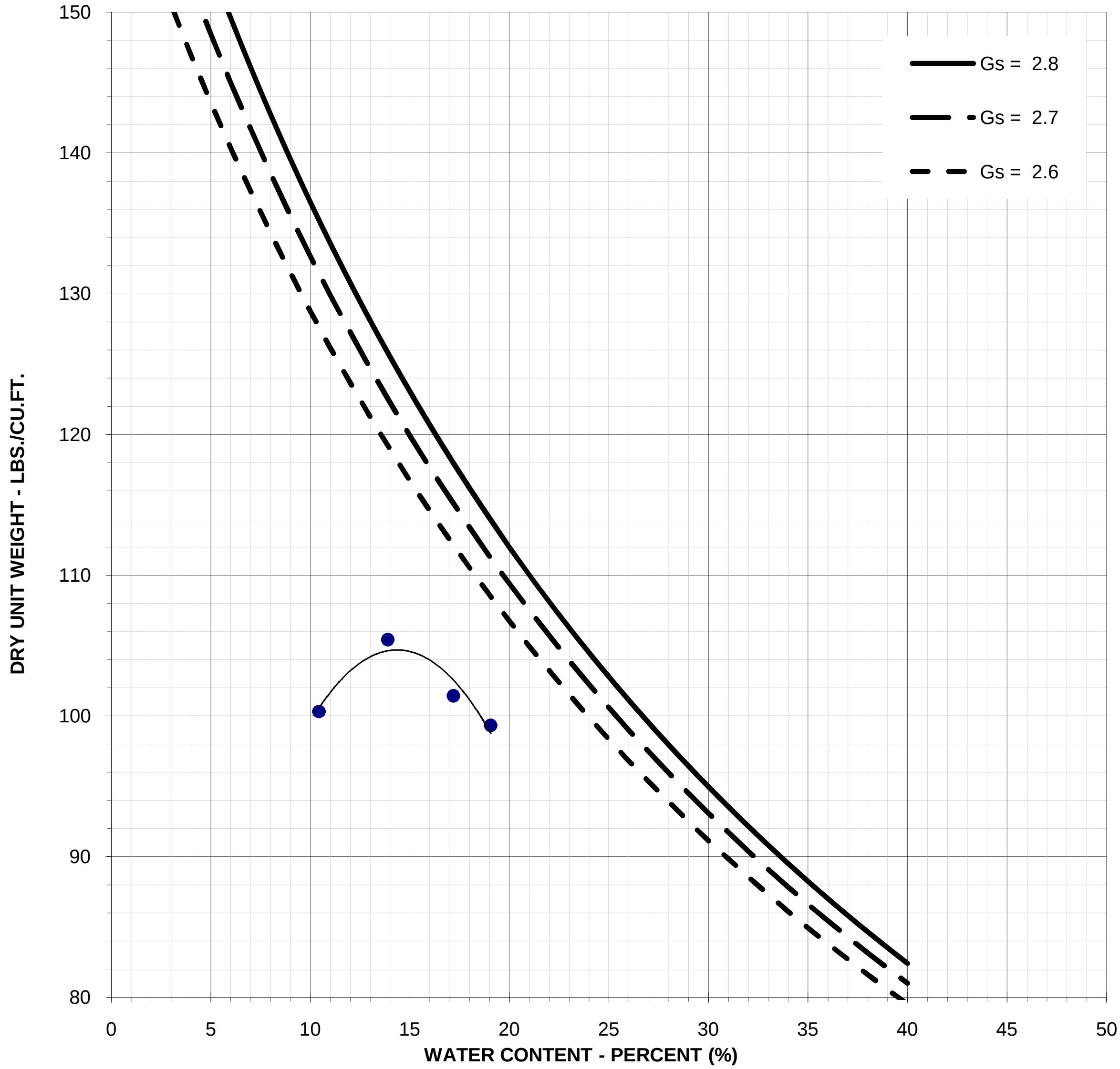
Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

1D CONSOLIDATION
LABORATORY TESTING RESULTS
BORING: B-3; SAMPLE: S-4

December 2011 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B3



Boring & Sample No.	Depth (feet)	U.S.C.S Symbol	Sample Description	Natural W.C. (%)	W (%) Optimum	Max Dry Density (pcf)
TP-3; S-1	Bulk	ML	SILT with clay (ML)	29.5	14.0	106

DRAFT

Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

STANDARD COMPACTION TEST

December 2011 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B4

APPENDIX C

PREVIOUS GEOTECHNICAL EXPLORATION INFORMATION BY OTHERS

Appendix Contents:

CH2M Hill 1975 geotechnical exploration information (Boring/CPT location and information) provided to Shannon & Wilson by the City of Lake Oswego

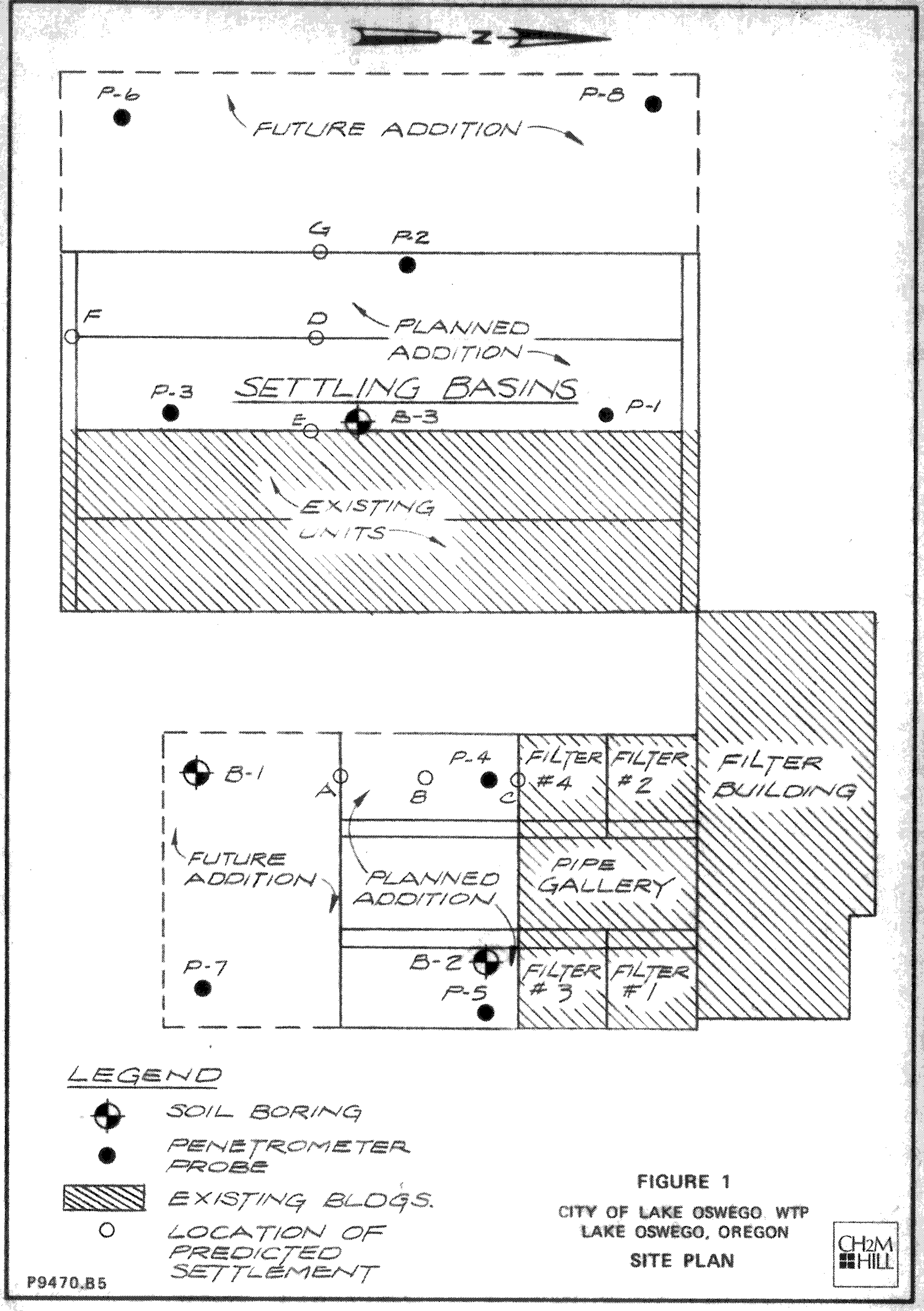
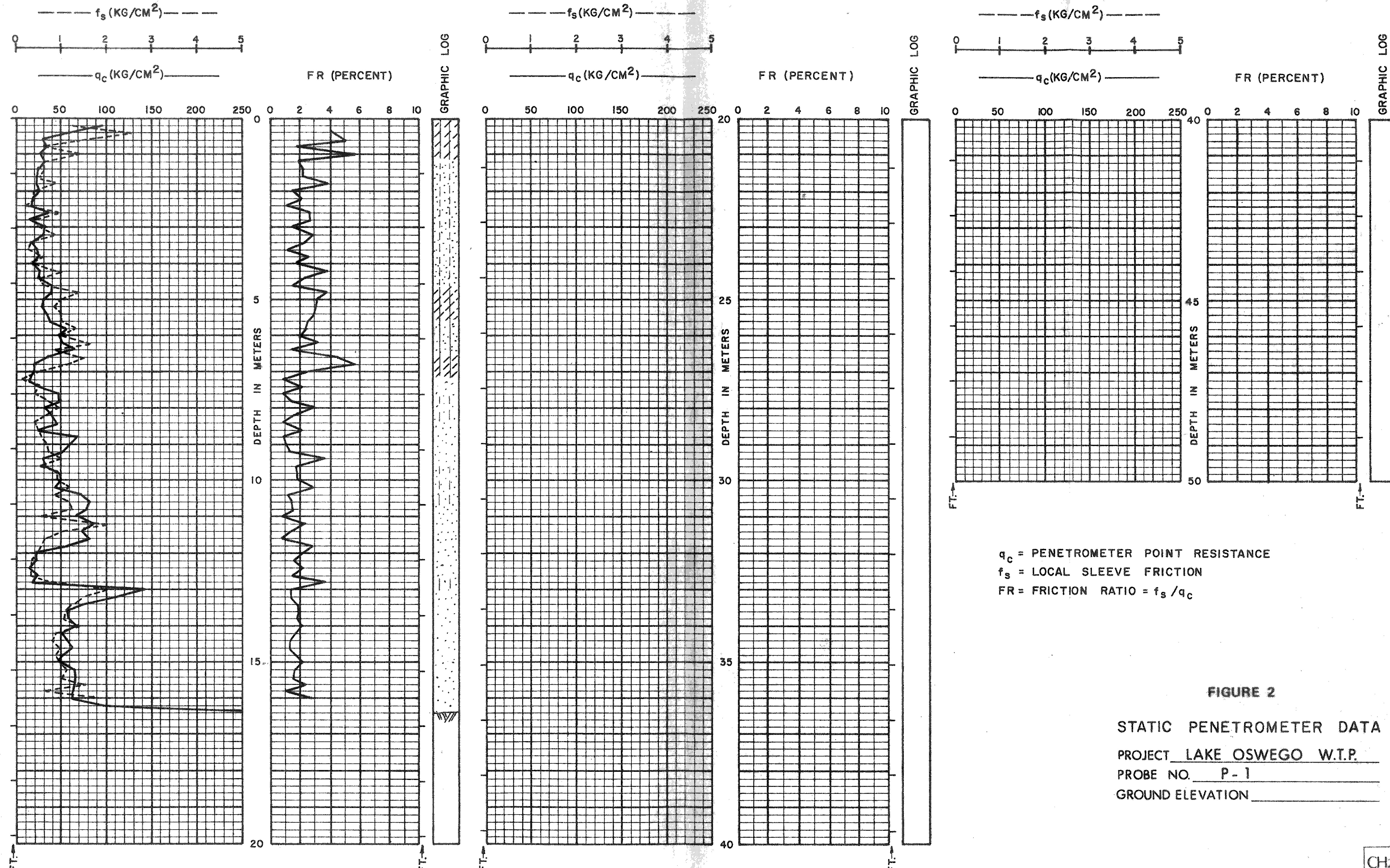


FIGURE 1

CITY OF LAKE OSWEGO WTP
LAKE OSWEGO, OREGON

SITE PLAN

CH2M HILL



q_c = PENETROMETER POINT RESISTANCE
f_s = LOCAL SLEEVE FRICTION
FR = FRICTION RATIO = f_s / q_c

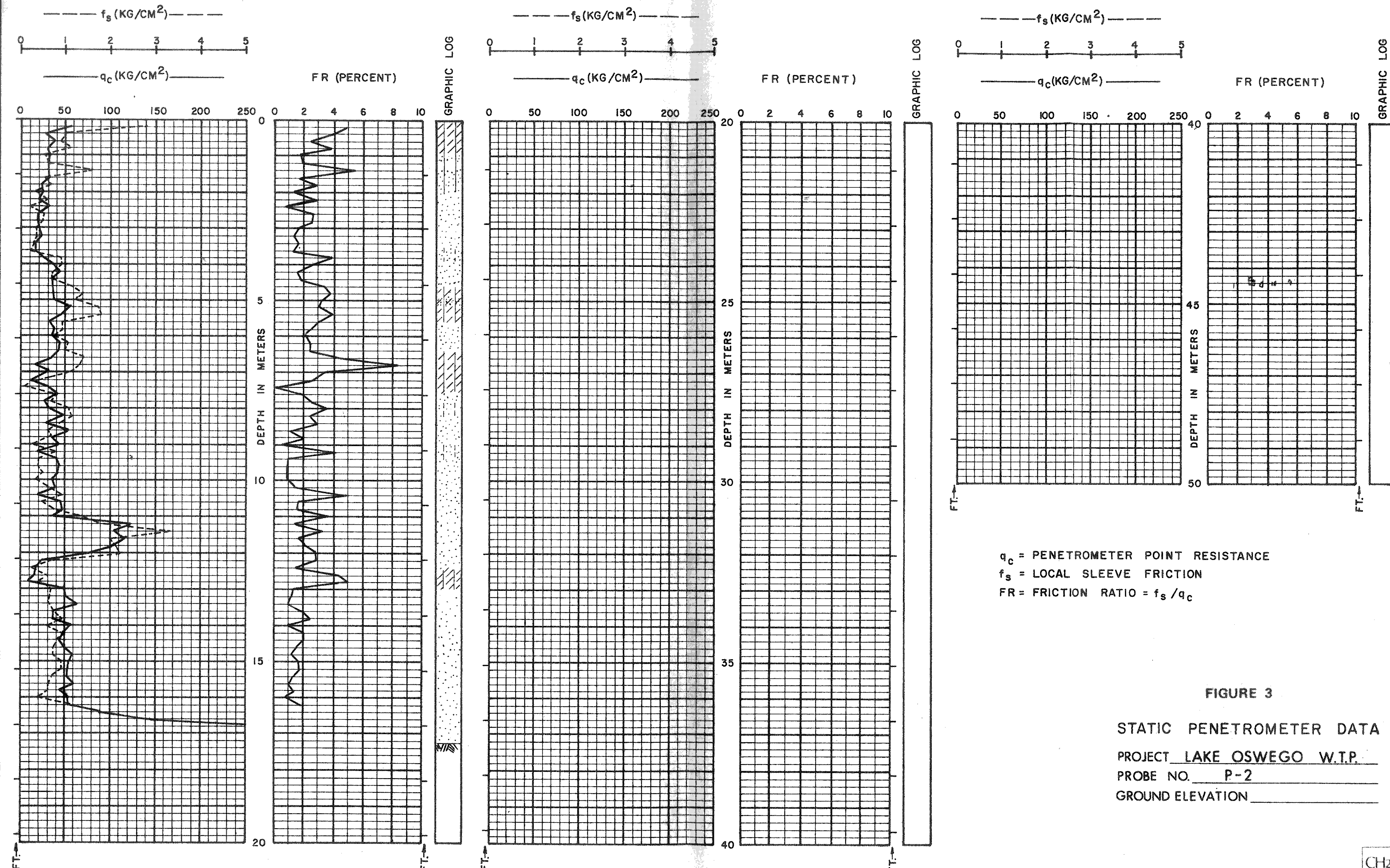
FIGURE 2

STATIC PENETROMETER DATA

PROJECT LAKE OSWEGO W.T.P.
PROBE NO. P - 1
GROUND ELEVATION

P 9470.B5

CH2M HILL



q_c = PENETROMETER POINT RESISTANCE
f_s = LOCAL SLEEVE FRICTION
FR = FRICTION RATIO = f_s / q_c

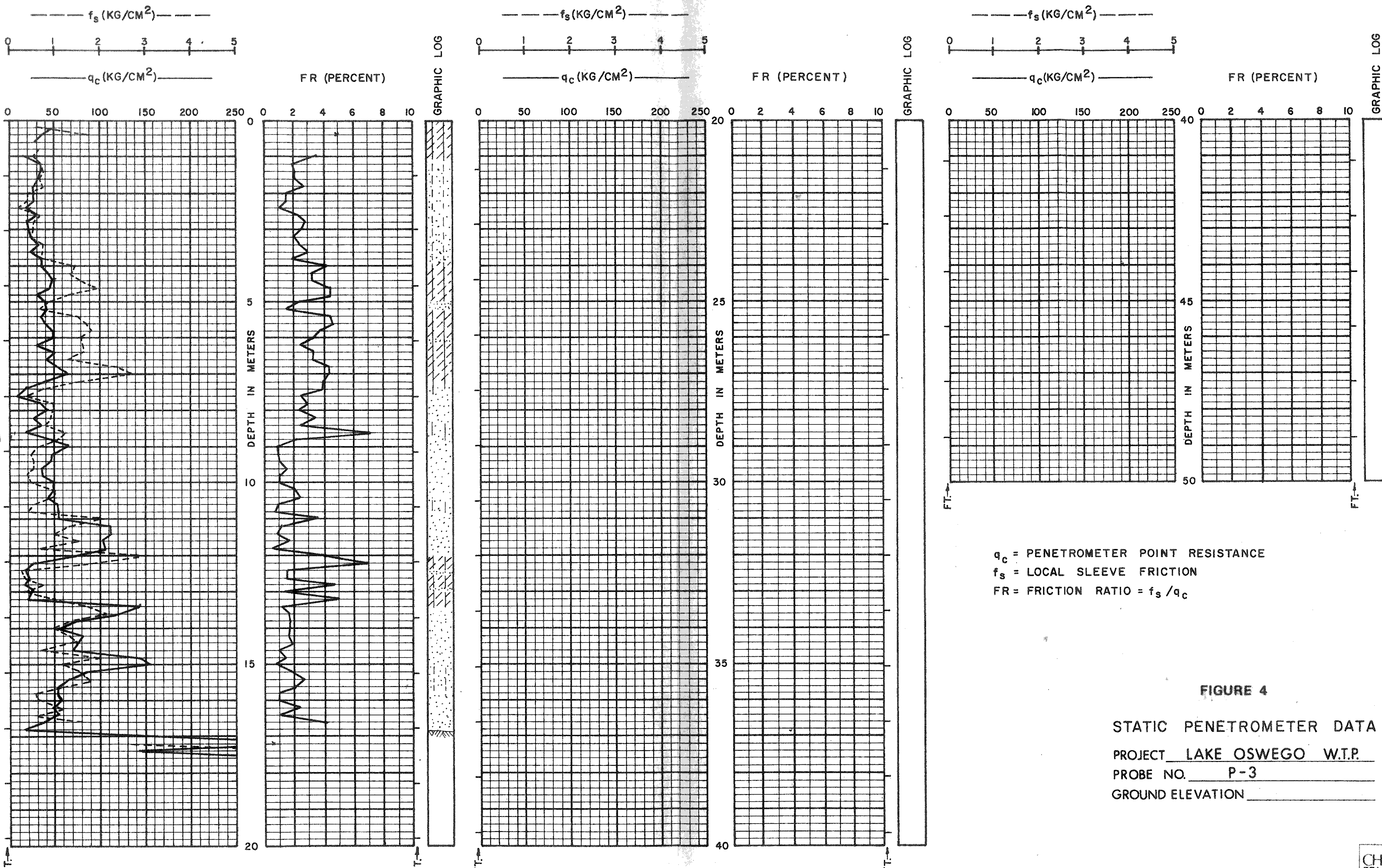
FIGURE 3

STATIC PENETROMETER DATA

PROJECT LAKE OSWEGO W.T.P.
PROBE NO. P-2
GROUND ELEVATION ______

CH2M HILL

P 9470.B5



q_c = PENETROMETER POINT RESISTANCE

f_s = LOCAL SLEEVE FRICTION

FR = FRICTION RATIO = f_s / q_c

FIGURE 4

STATIC PENETROMETER DATA

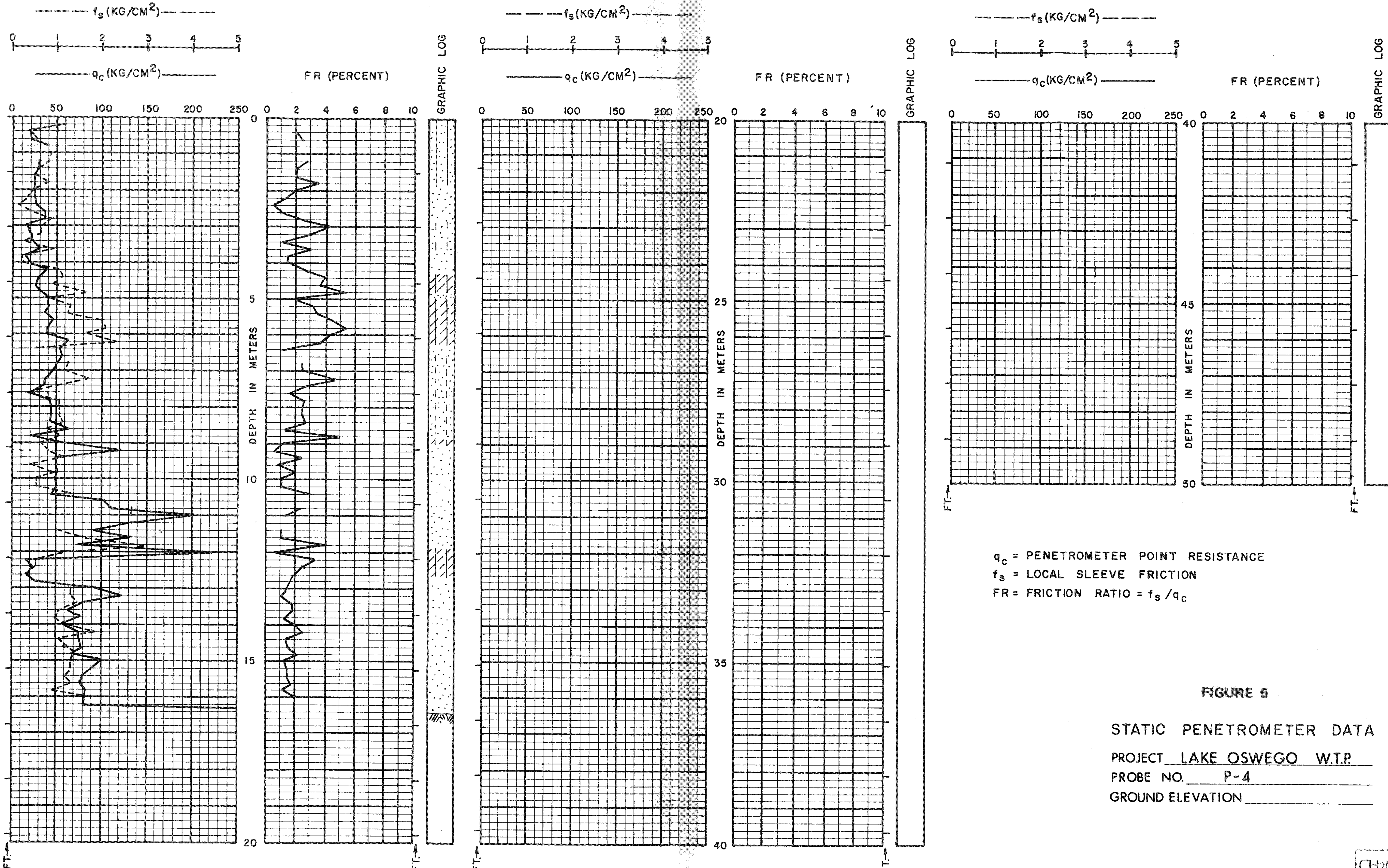
PROJECT LAKE OSWEGO W.T.P.

PROBE NO. P-3

GROUND ELEVATION

P 9470.B5

CH2M HILL



q_c = PENETROMETER POINT RESISTANCE

f_s = LOCAL SLEEVE FRICTION

FR = FRICTION RATIO = f_s / q_c

FIGURE 5

STATIC PENETROMETER DATA

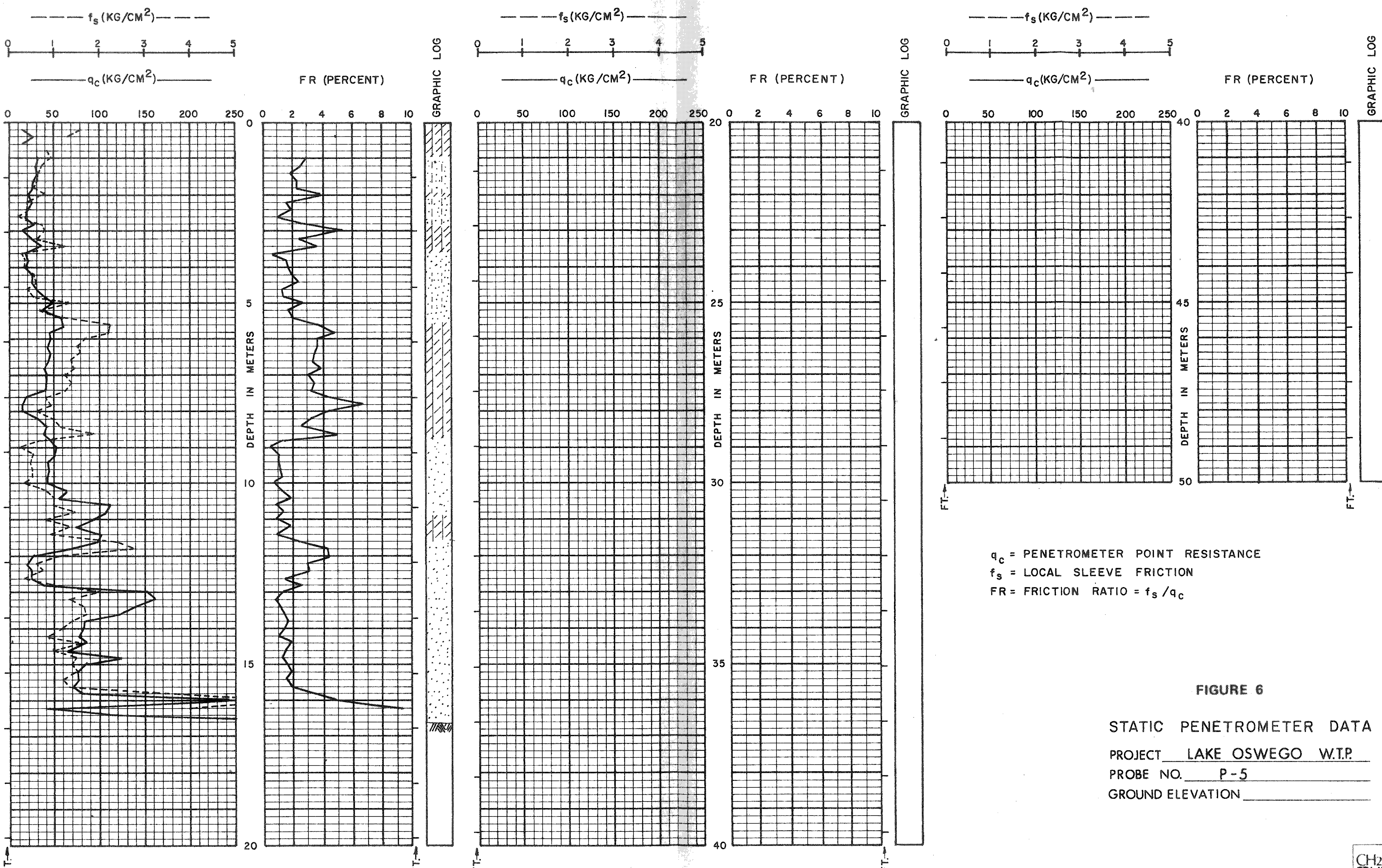
PROJECT LAKE OSWEGO W.T.P.

PROBE NO. P-4

GROUND ELEVATION

CH2M HILL

P9470.B5



q_c = PENETROMETER POINT RESISTANCE
f_s = LOCAL SLEEVE FRICTION
FR = FRICTION RATIO = f_s / q_c

FIGURE 6

STATIC PENETROMETER DATA

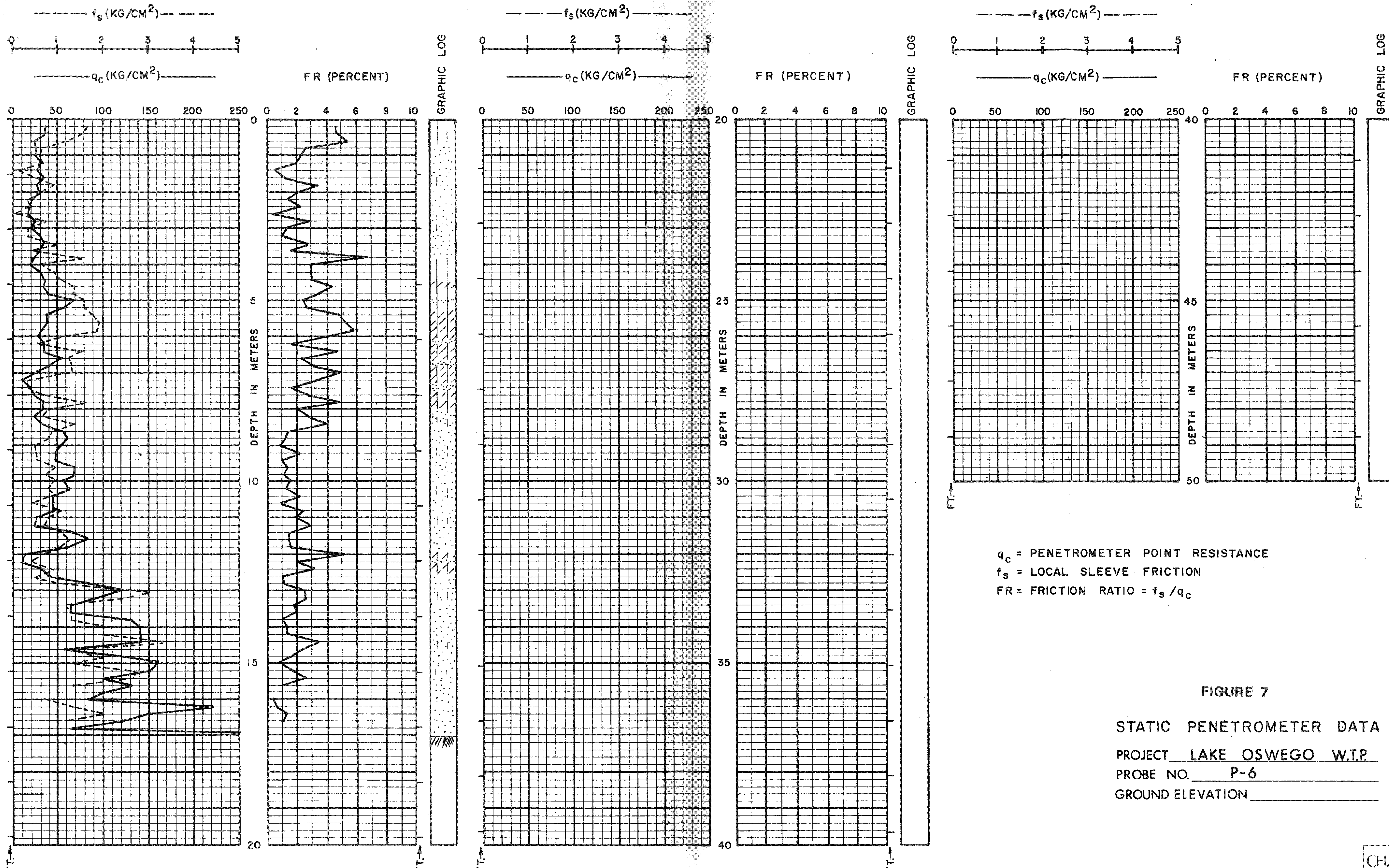
PROJECT LAKE OSWEGO W.T.P.

PROBE NO. P-5

GROUND ELEVATION

CH2M HILL

P9470.B5



q_c = PENETROMETER POINT RESISTANCE
f_s = LOCAL SLEEVE FRICTION
FR = FRICTION RATIO = f_s / q_c

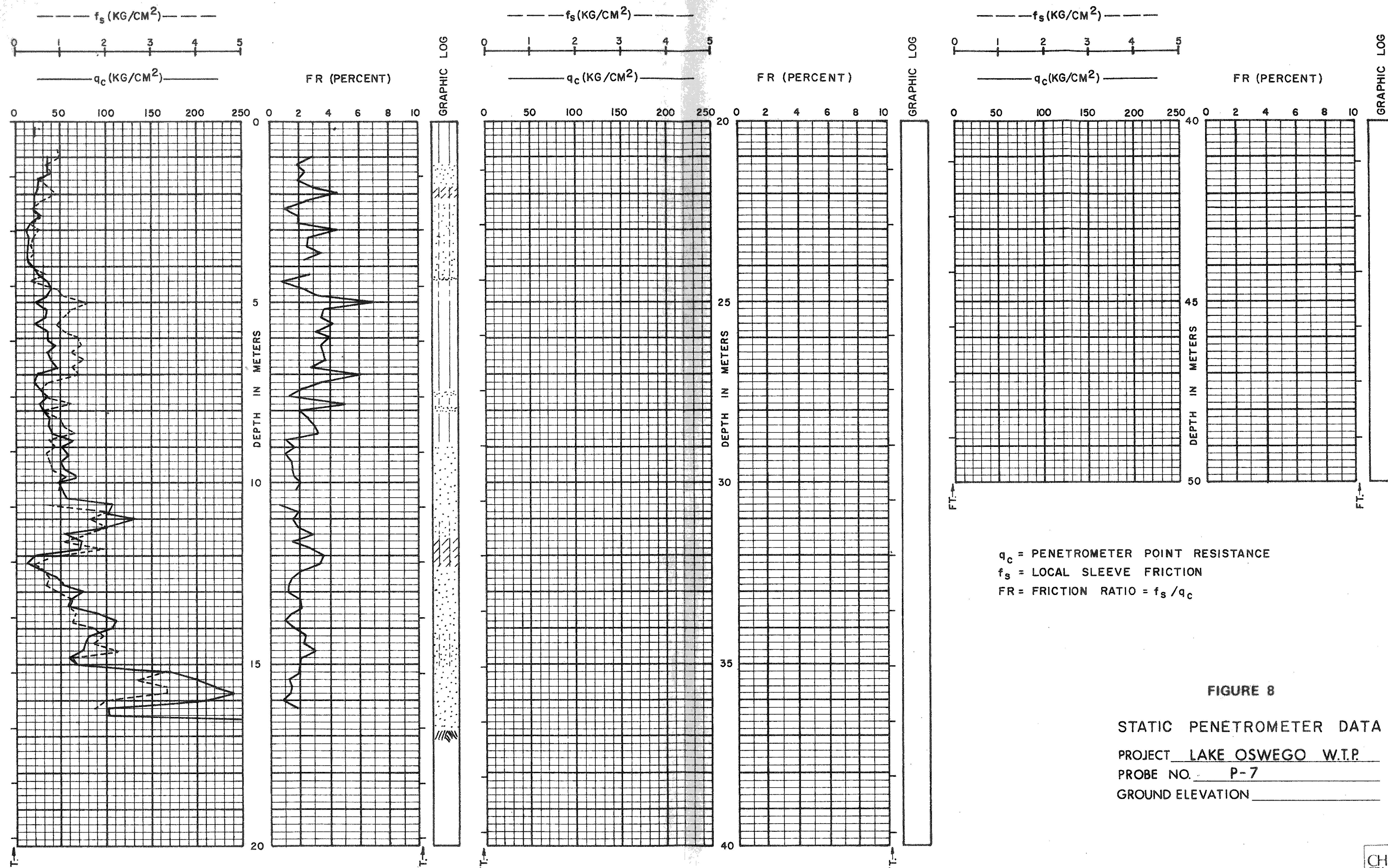
FIGURE 7

STATIC PENETROMETER DATA

PROJECT LAKE OSWEGO W.T.P.
PROBE NO. P-6
GROUND ELEVATION

P 9470.B5

CH2M HILL



q_c = PENETROMETER POINT RESISTANCE
f_s = LOCAL SLEEVE FRICTION
FR = FRICTION RATIO = f_s / q_c

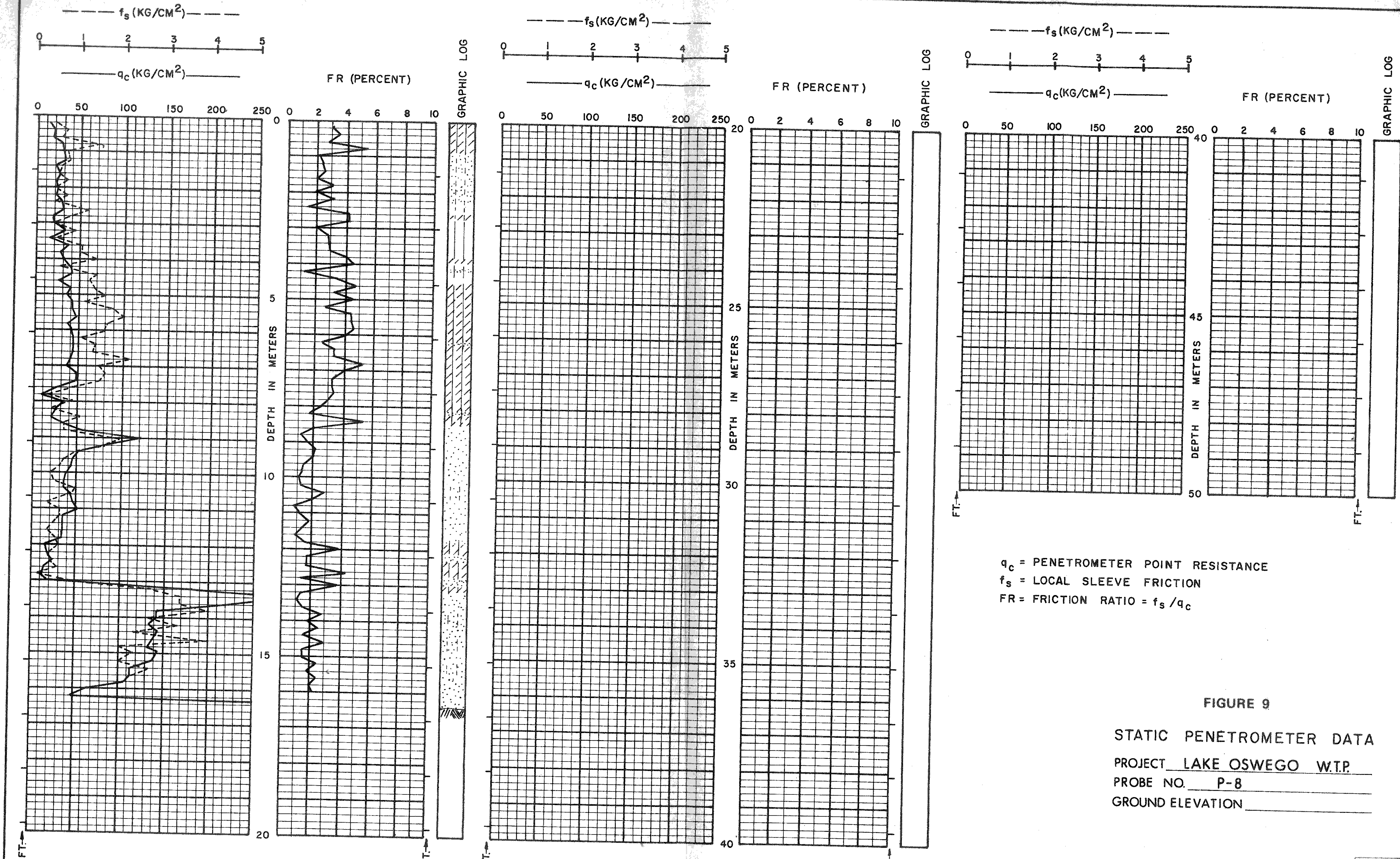
FIGURE 8

STATIC PENETROMETER DATA

PROJECT LAKE OSWEGO W.T.P.
PROBE NO. P-7
GROUND ELEVATION ______

P9470.B5

CH2M HILL



q_c = PENETROMETER POINT RESISTANCE
f_s = LOCAL SLEEVE FRICTION
FR = FRICTION RATIO = f_s / q_c

FIGURE 9

STATIC PENETROMETER DATA

PROJECT LAKE OSWEGO W.T.P.
PROBE NO. P-8
GROUND ELEVATION ______

P9470.B5

CH2M HILL

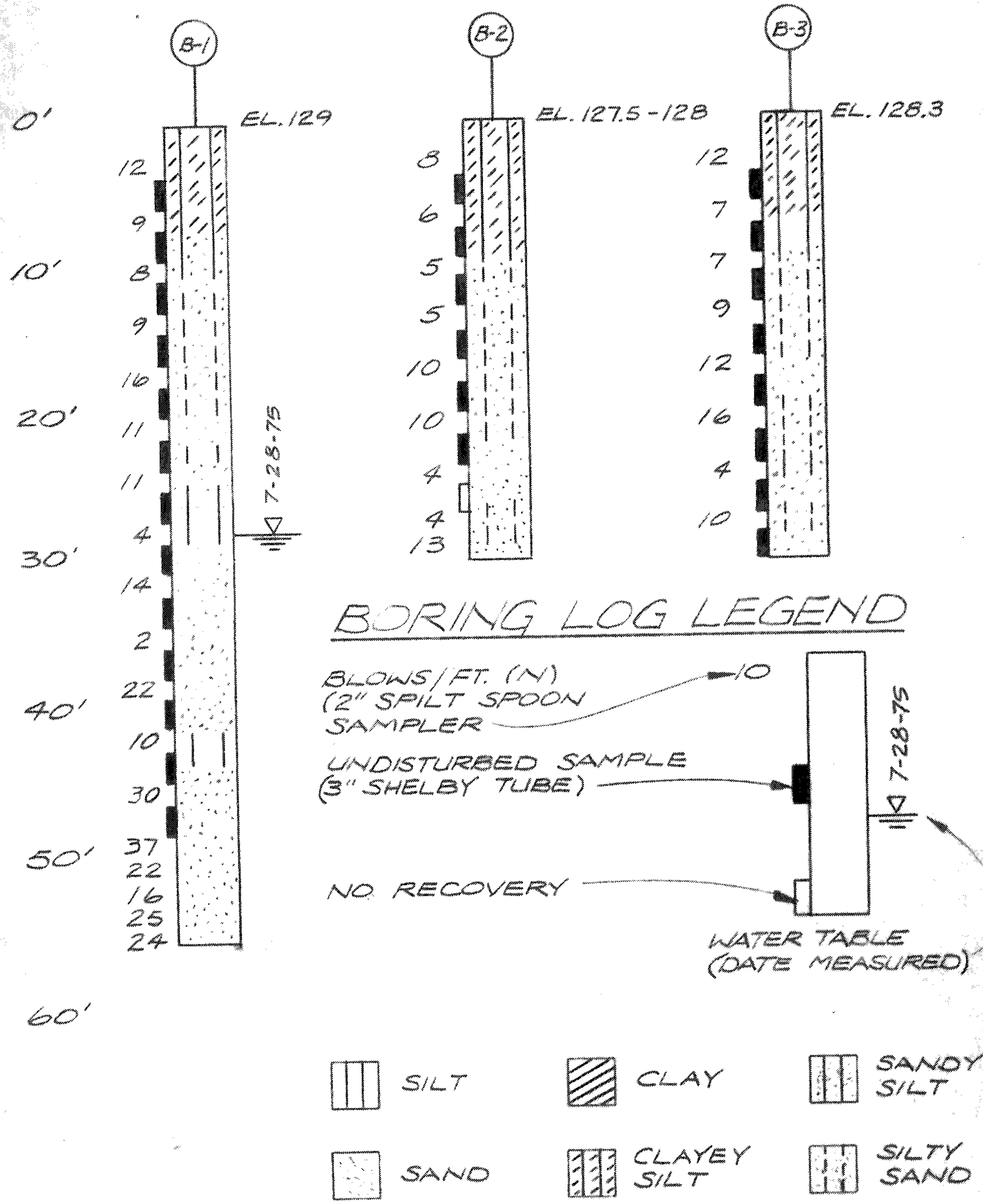


FIGURE 10

CITY OF LAKE OSWEGO, WTP
LAKE OSWEGO, OREGON

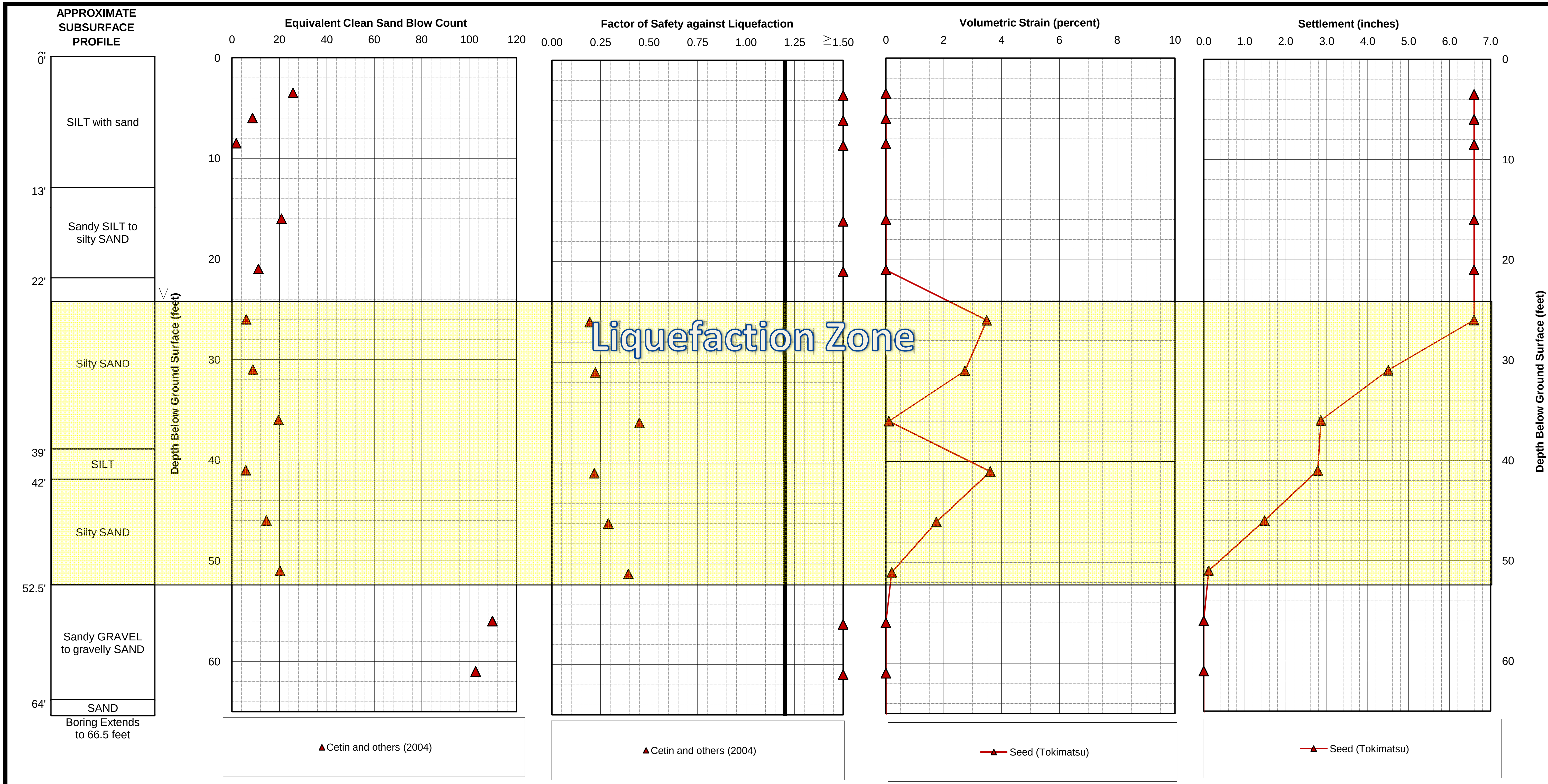
LOG OF SOIL BORINGS

CH2M HILL

P9470.B5

APPENDIX D

LIQUEFACTION POTENTIAL AND SETTLEMENT ANALYSIS RESULTS



DRAFT

Lake Oswego & Tiard
Water Treatment Plant Expansion Project
West Linn, Oregon

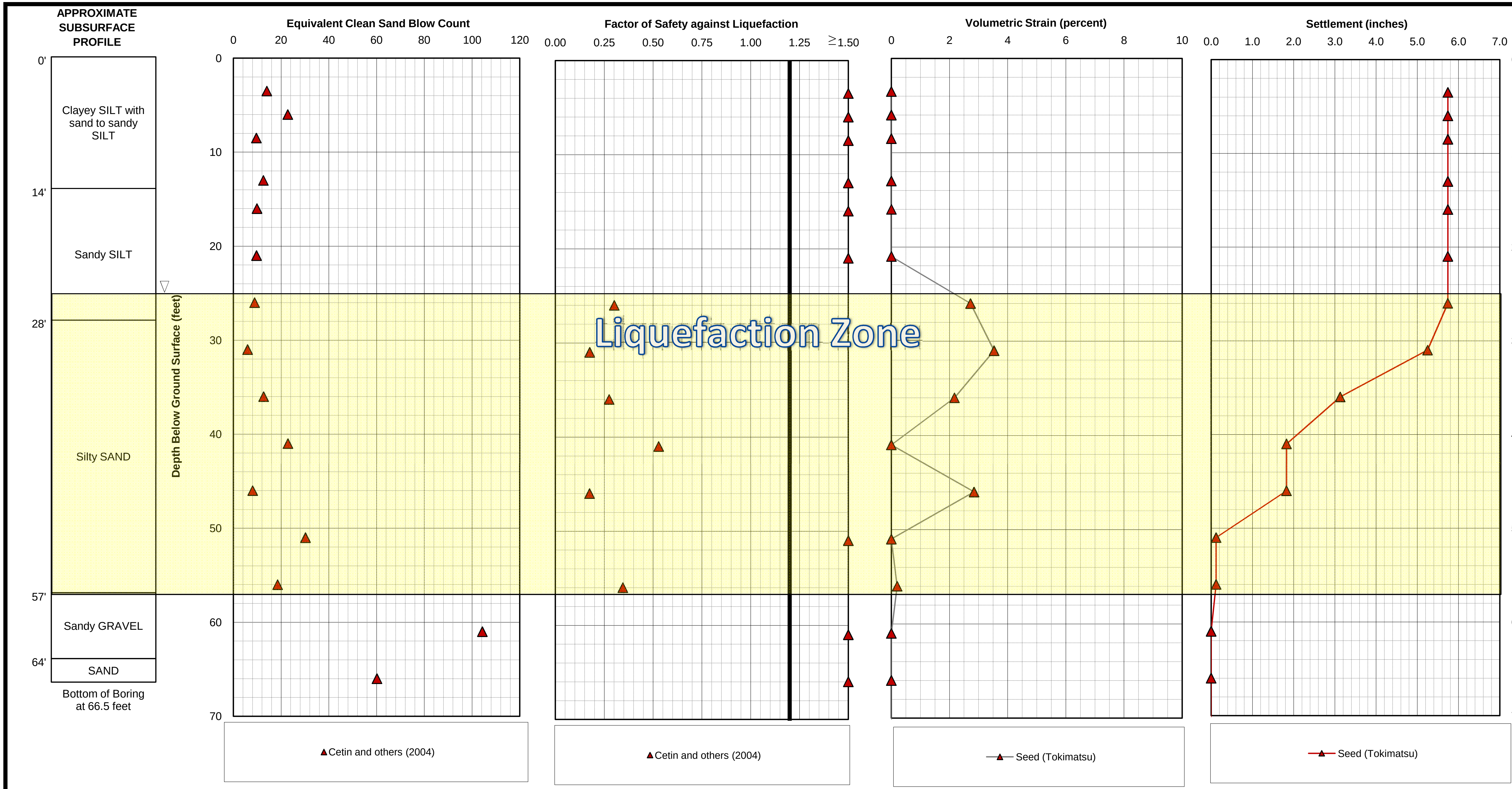
SEISMIC SETTLEMENT
BORING B-1
M = 9.0, PGA = 0.20

January 2012 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. D1

SPT_Liquefaction_(B-2).xlsm Printed: 12/28/2011 11:00 AM



DRAFT

Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

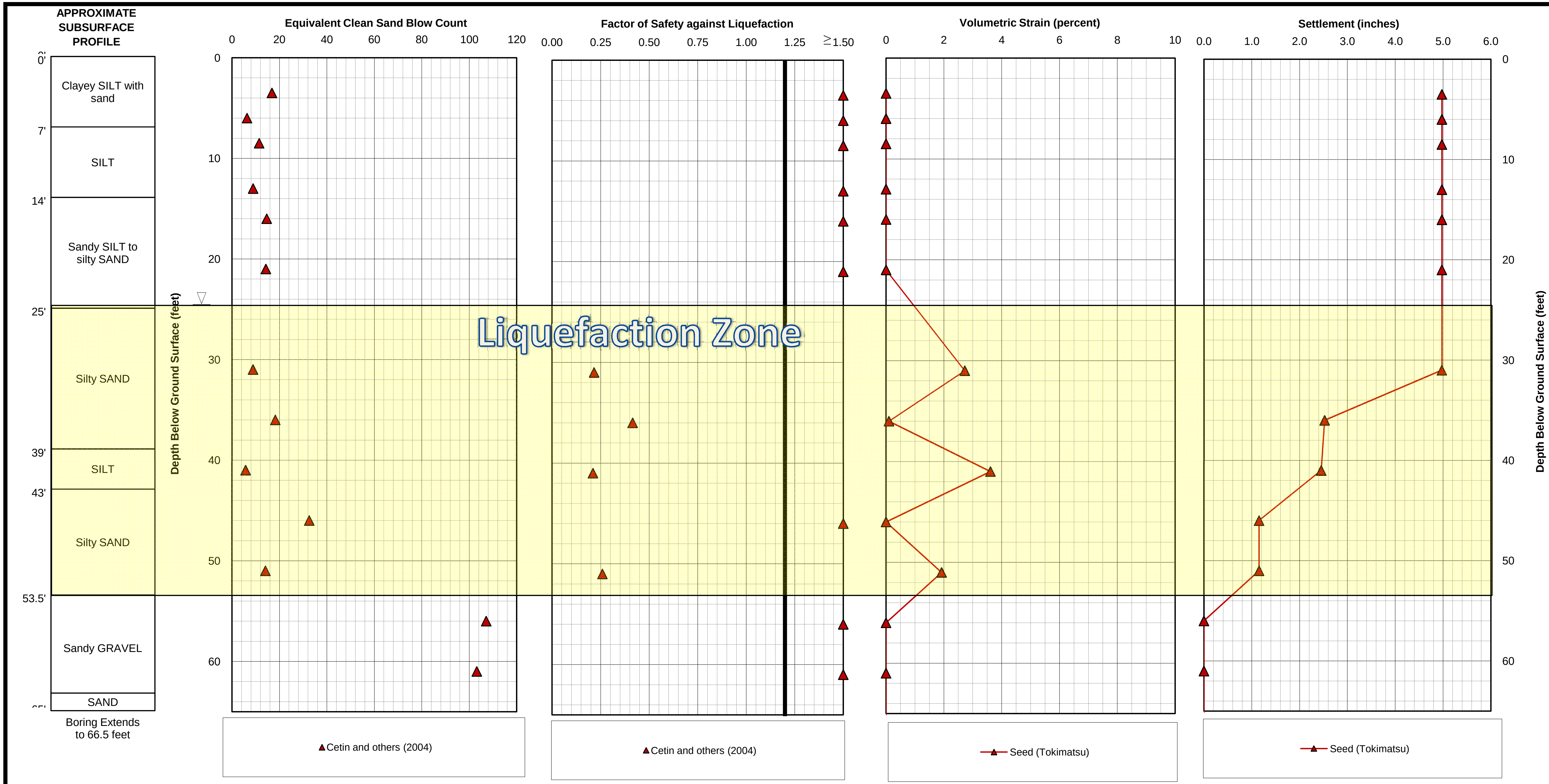
SEISMIC SETTLEMENT
BORING B-2
M = 9.0, PGA = 0.20

January 2012 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. D2

SPT_Liquefaction_(B-3).xlsm Printed: 12/28/2011 10:45 AM



DRAFT

Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

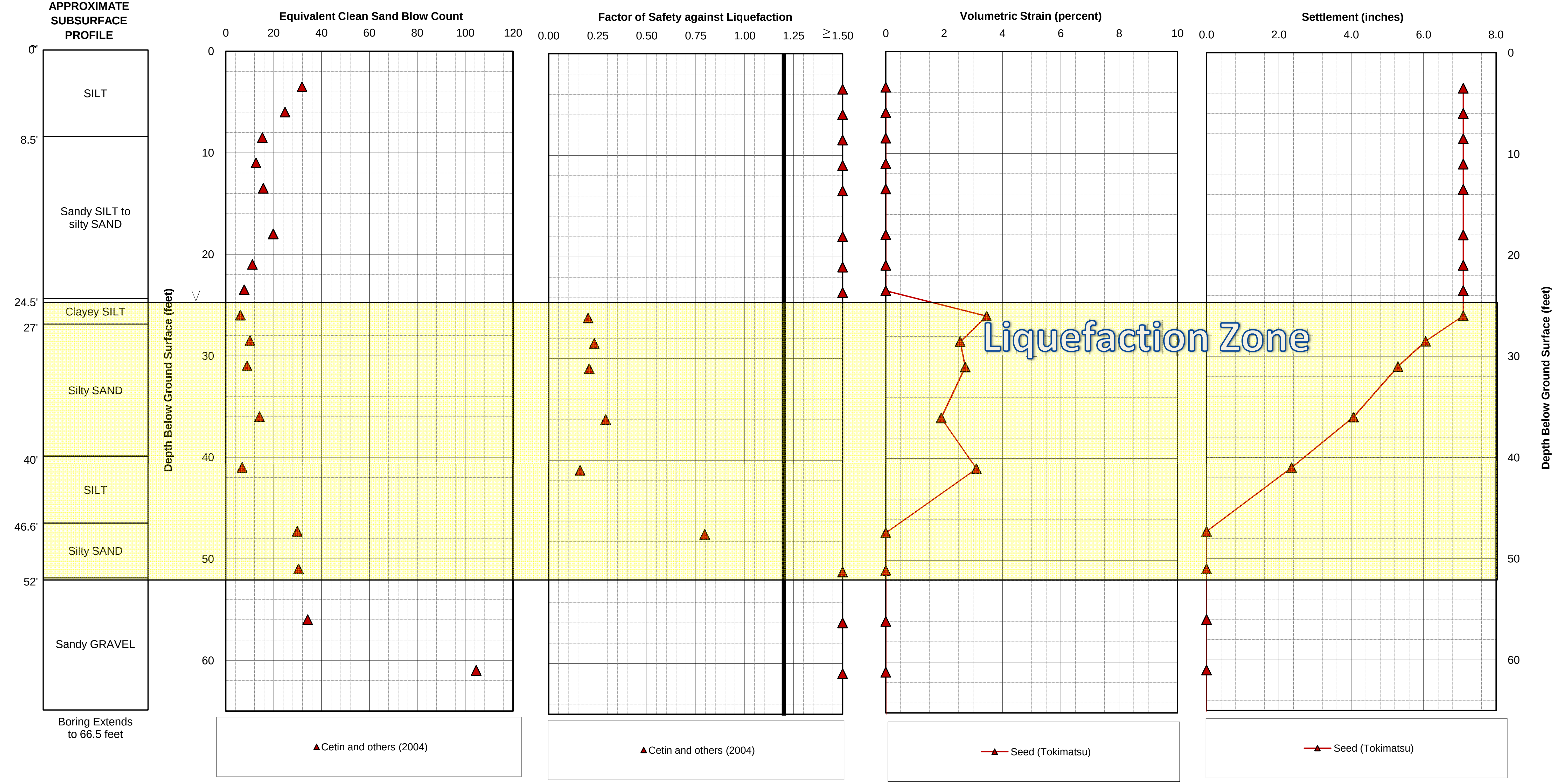
SEISMIC SETTLEMENT
BORING B-3
M = 9.0, PGA = 0.20

January 2012 | 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. D3

SPT_Liquefaction_(B-4).xlsm Printed: 12/28/2011 10:52 AM



DRAFT

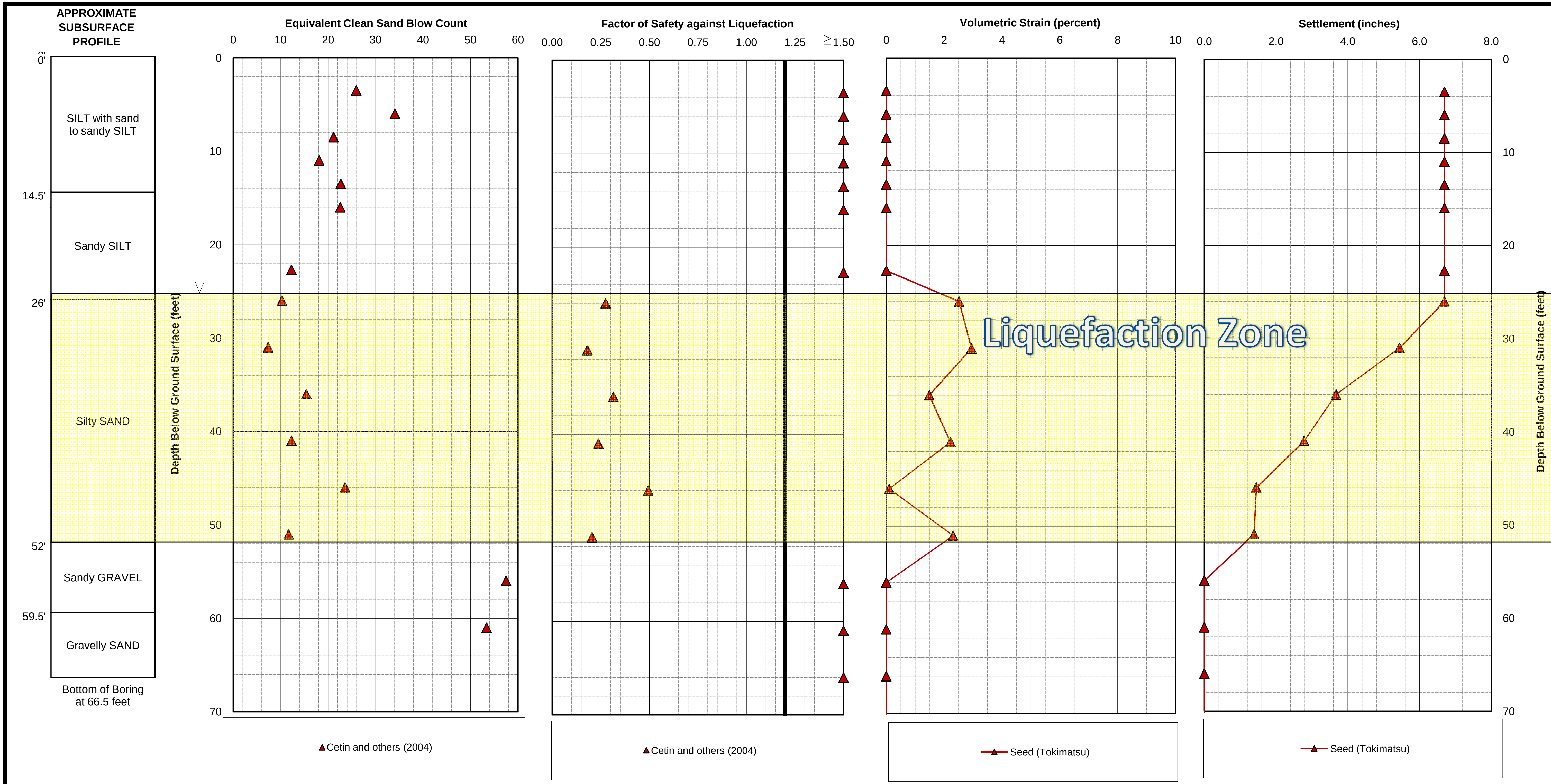
Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

SEISMIC SETTLEMENT
BORING B-4
M = 9.0, PGA = 0.20

January 2012 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. D4



DRAFT

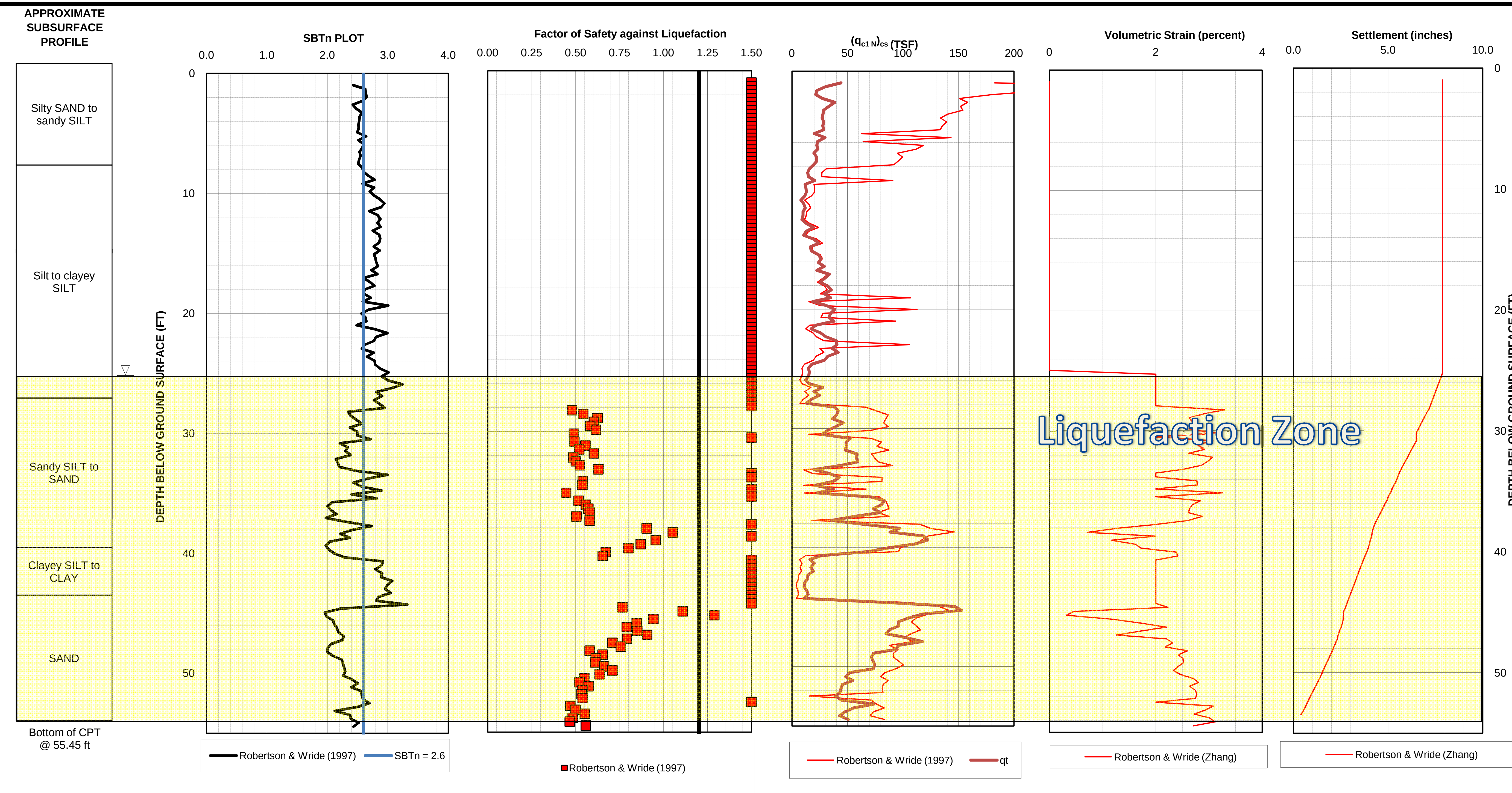
Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

SEISMIC SETTLEMENT
BORING B-5
M = 9.0, PGA = 0.20

January 2012 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. D5



DRAFT

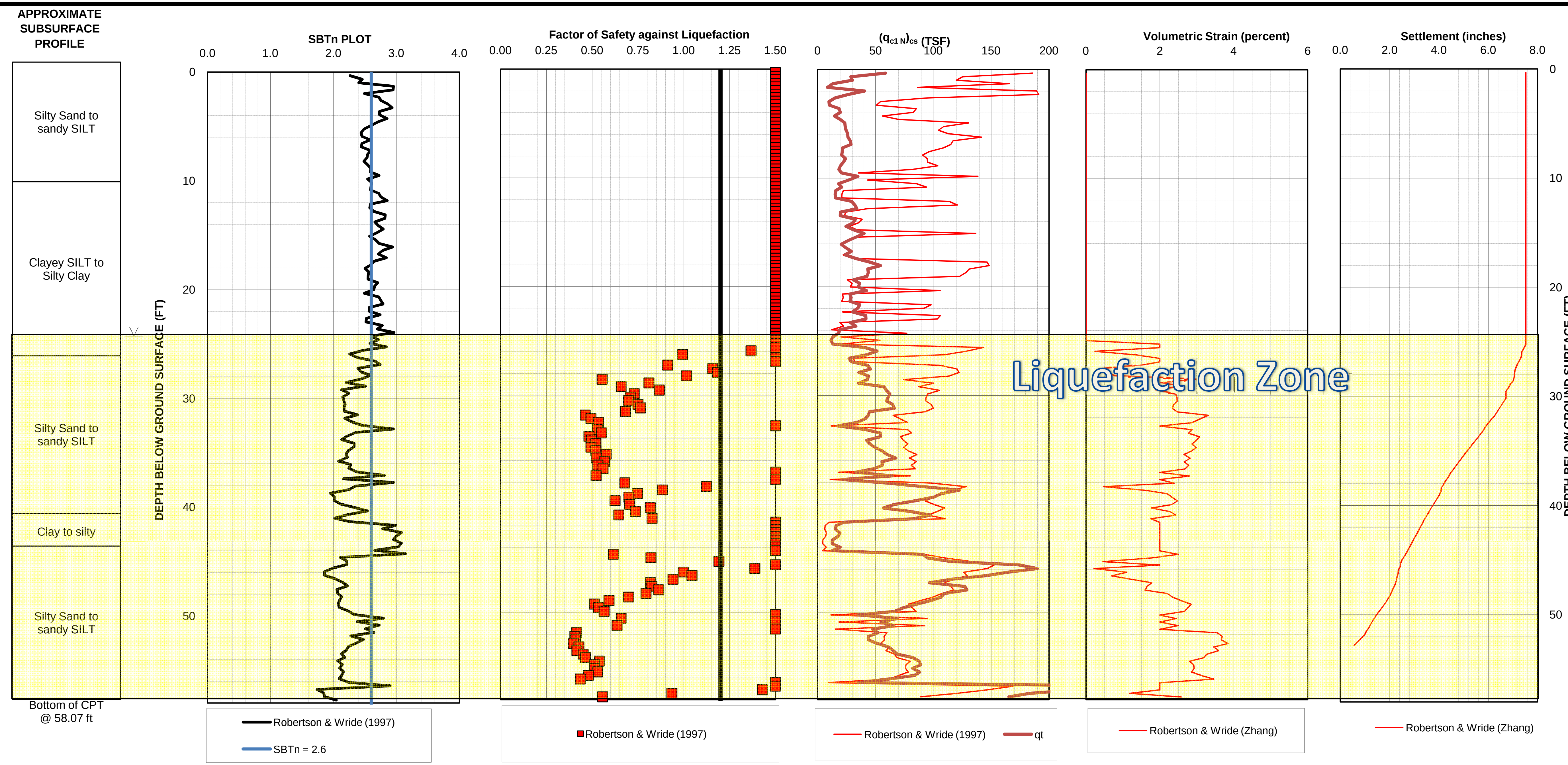
Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

SEISMIC SETTLEMENT
CPT-1
M = 9.0, PGA = 0.2

January 2012 | 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. D6



DRAFT

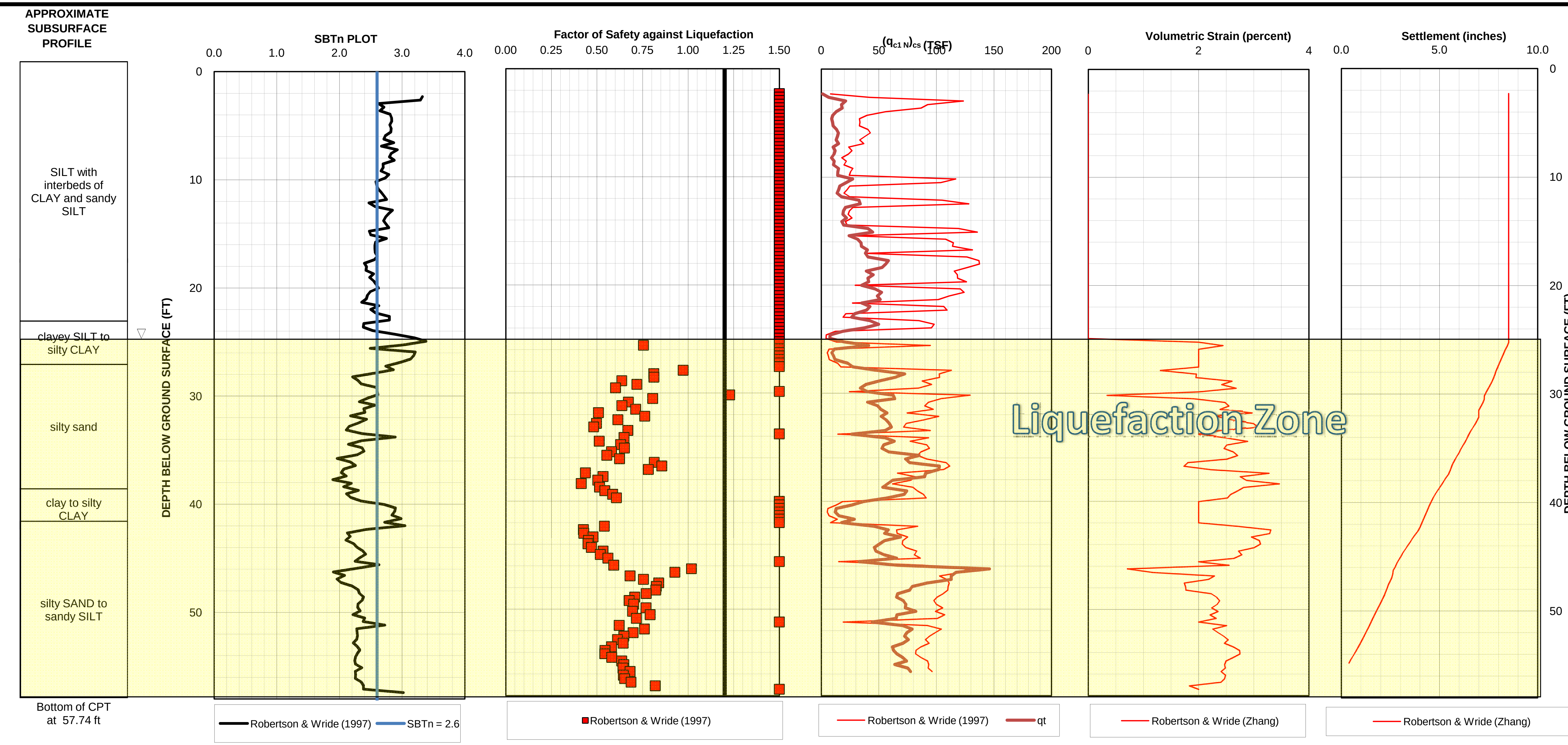
Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

SEISMIC SETTLEMENT
CPT-2
M = 9.0, PGA = 0.2

January 2012 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. D7



DRAFT

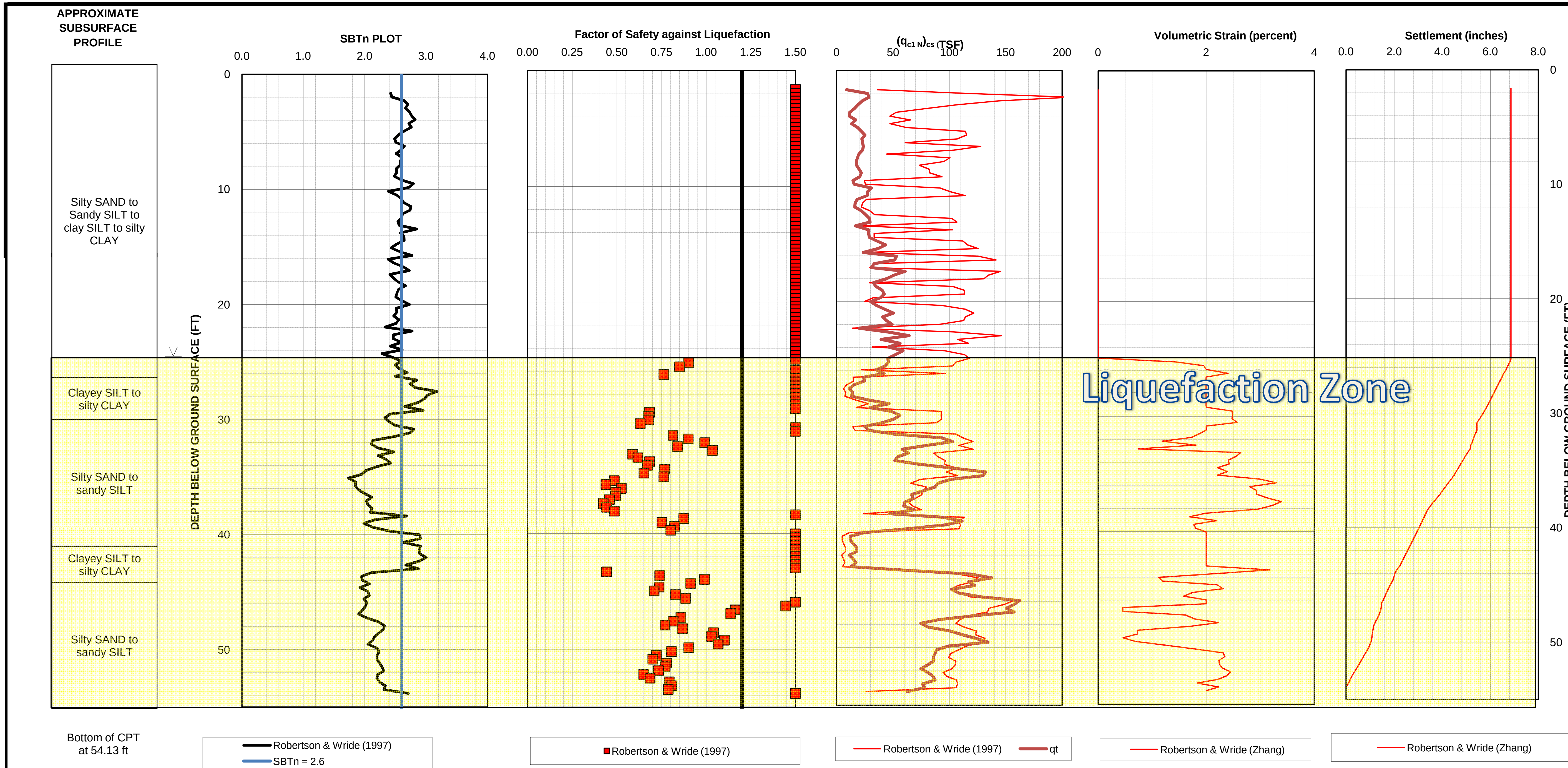
Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

SEISMIC SETTLEMENT
CPT-3
M = 9.0, PGA = 0.2

January 2012 | 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. D8



DRAFT

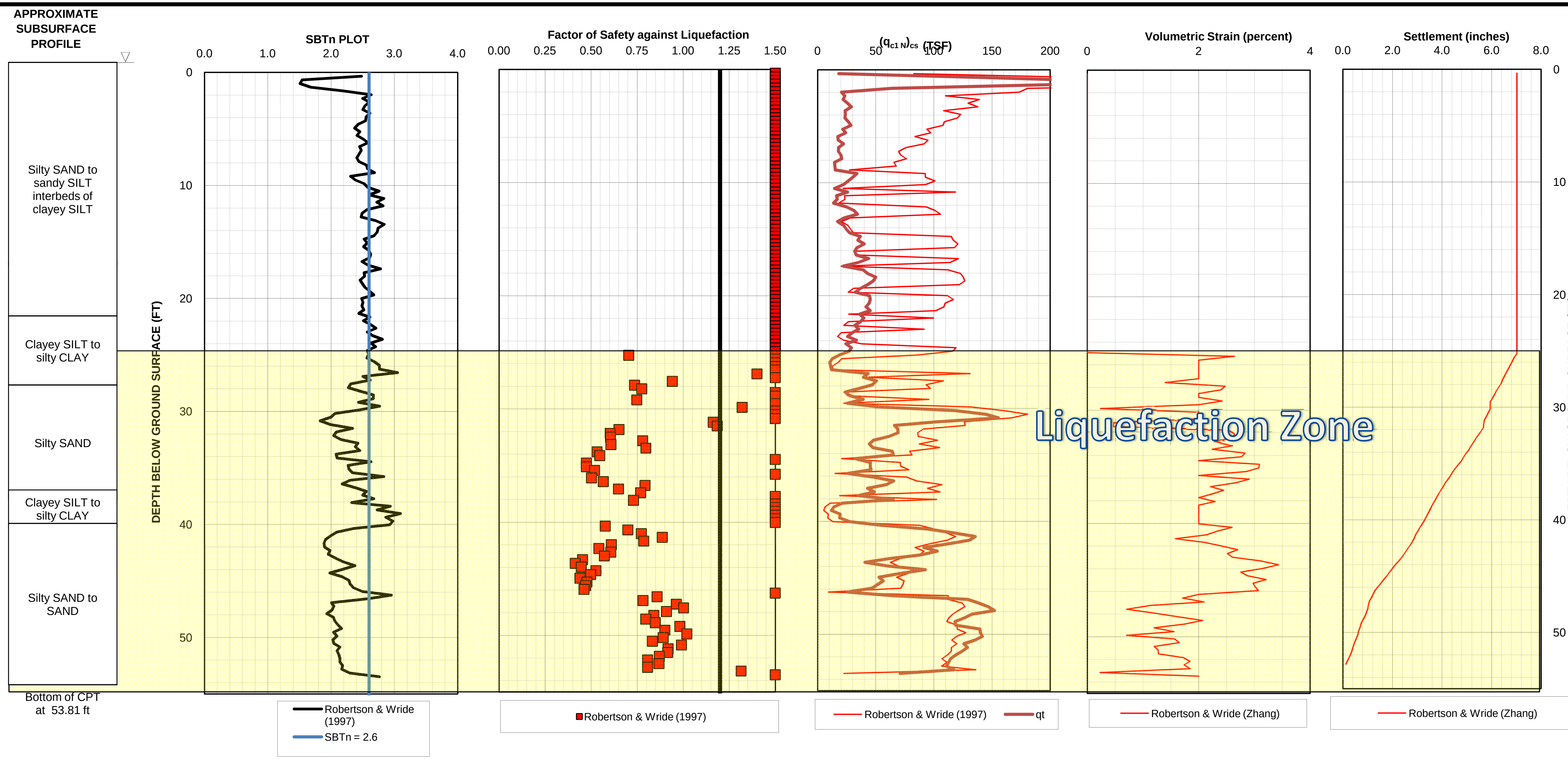
Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

SEISMIC SETTLEMENT
CPT-4
M = 9.0, PGA = 0.24

January 2012 | 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. D9



DRAFT

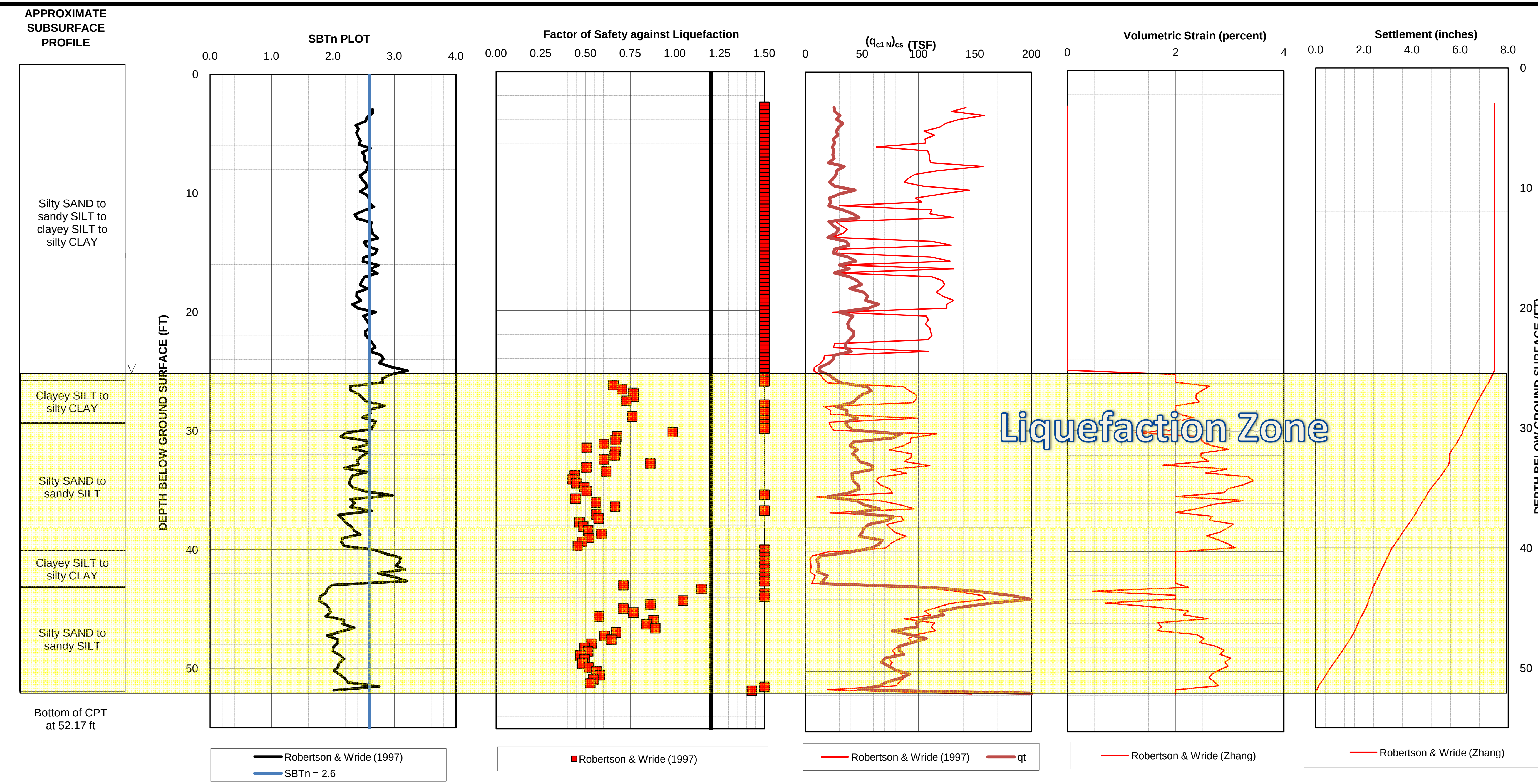
Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

SEISMIC SETTLEMENT
CPT-5
M = 9.0, PGA = 0.2

January 2012 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. D10



DRAFT

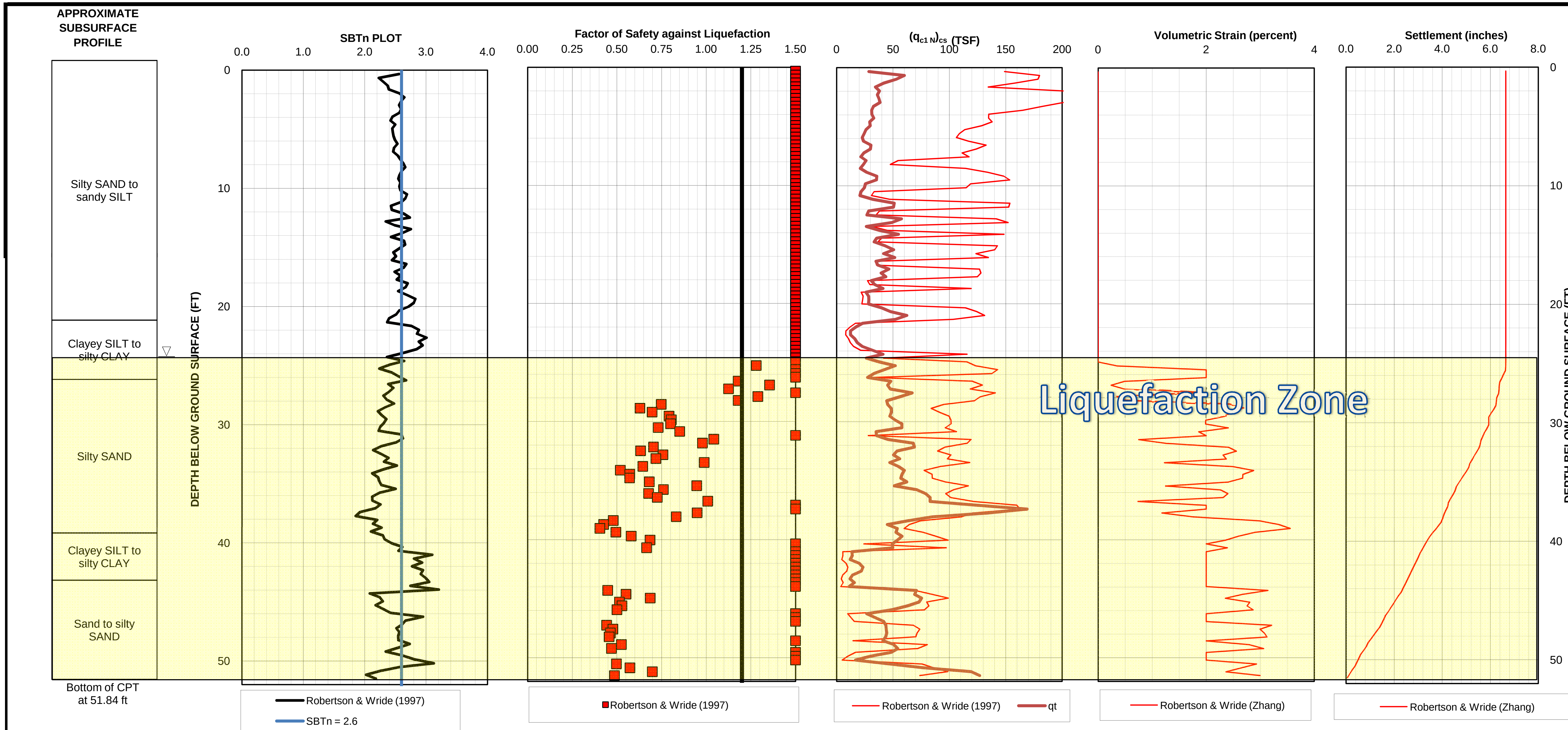
Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

SEISMIC SETTLEMENT
CPT-6
M = 9.0, PGA = 0.2

January 2012 | 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. D11



DRAFT

Lake Oswego & Tigard
Water Treatment Plant Expansion Project
West Linn, Oregon

SEISMIC SETTLEMENT
CPT-7
M = 9.0, PGA = 0.2

January 2012 24-1-03658-245

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. D12