Vancouver Energy
NPDES Engineering Report

EFSEC Application for Site Certification No. 2013-01
Docket No. EF131590

Appendix I
Geotechnical Investigation Reports

Geotechnical Investigation

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TABLE OF CONTENTS

Page

Figures

Tables

INTRODUCTION

At your request, GRI has completed a geotechnical investigation for the proposed petroleum-by-rail handling facility at the Port of Vancouver (Port) in Vancouver, Washington. The vicinity map, Figure 1, shows the general location of the project, which includes the northwest corner of Terminal 5 (T-5), Parcel 1A, Berths 13 and 14 in Terminal 4 (T-4), and pipeline areas that connect these three areas. This report addresses the upland portion of the facility. The supplemental investigation addressing the dock modifications and portions of the terminal along the riverbank is in progress.

The investigation was conducted to evaluate subsurface conditions at the site and provide our conclusions and recommendations for design and construction of the proposed facility. Our investigation has included a review of available geologic information, subsurface explorations, laboratory testing, and engineering analyses. This report describes the work accomplished and provides our conclusions and geotechnical recommendations for the design and construction of the proposed facility.

Preliminary design recommendations for Area 300 storage tanks were provided to you in our July 18, 2013, memorandum entitled, "Progress Memorandum and Preliminary Conclusions and Recommendations, Petroleum Tank Support and Performance, Tesoro Savage Petroleum Terminal, Port of Vancouver, USA." Preliminary design recommendations for Area 200 unloading and structure areas were provided in our July 25, 2013, memorandum entitled, "Progress Memorandum and Preliminary Conclusions and Recommendations, Area 200 – Unloading and Building Areas, Tesoro Savage Petroleum Terminal, Port of Vancouver, USA."

The following geotechnical information has been reviewed for this investigation:

> Dames and Moore, March 31, 1993, Geotechnical Investigation, Proposed T-Docks/Dolphins, Port of Vancouver, Washington; prepared for URS Consultants.
>
> GRI, May 18, 2011, Geotechnical Report, NW Gateway Avenue Rail Bridge and Access to Terminal 5, Port of Vancouver, USA; prepared for HDR Inc.
>
> GRI, August 23, 2007, Draft Geotechnical Investigation, Columbia Gateway Rail Improvements, Port of Vancouver, Washington; prepared for Jones & Stokes Environmental Specialist.
>
> GRI, December 20, 2006 (issued July 31, 2007), Geotechnical Investigation, Columbia Gateway Rail Expansion, Port of Vancouver, Washington; prepared for Jones & Stokes.

PROJECT DESCRIPTION

Overview

The Project Layout Plan, Figure 2, shows the proposed layout of the new petroleum-by-rail bulk handling facility. The facility will occupy portions of Terminals 4 and 5, and Parcel 1A at the Port. The project includes specific areas that have been designated Areas 200 through 600, as shown on Site Plans, Figures 3 through 5. Crude oil will be transported to the Port by unit railcar trains and unloaded in a railcar unloader located in Area 200. A boiler structure in Area 600, located adjacent to the west end of the unloader structure will heat the crude oil for transport through above- and below-ground pipes to up to six storage tanks located in Area 300. A boiler structure located near the storage tanks will heat the crude oil for transport from the storage tanks to ships at Berth 13 in Area 400. Additional site improvements will include a vapor combustion structure in Area 400 located near the dock to burn off excess vapor generated during transport of the crude oil through the pipeline. In addition, a modular office structure and two changing room structures (Area 200) will be located north of the unloader structure. The pipeline is referred to as Area 500 on Figure 2. The site improvements will be designed to meet the 2013 Washington State Building Code.

Areas 200 and 600 (Unloading, Office and West Boiler)

The site layout for the rail unloading area, west boiler, and administration and support structures is shown on Figure 3. The rail unloading area will allow the simultaneous unloading of tank cars on three rail lines and will be covered by a 90-ft-wide by 1,800-ft-long, relatively light, single-bay metal framed structure. The oil transport piping will be housed in embedded reinforced concrete trenches that are 5 to 7 ft deep and 9 ft wide. The rail unloading piping racks and piping trenches will be structurally independent of the unloader structure.

A small control room and fire pump and foam structure will be located on the south side of the unloading structure, together with transformer pads and pump pit. An office structure (48 by 70 ft) and two change rooms (48 by 70 ft and 36 by 70 ft) together with employee parking areas and six small holding tanks for rail car spill containment will be located north of the unloading area and north of the existing rails. Pedestrian bridges will span the rail lines and extend from the south side of the unloading area to near the planned office structure.

The west boiler is an approximate 6,600 sq ft, lightly loaded structure located west of the unloader structure.

Areas 300 (Storage Tanks)

The site layout for the product storage tanks, secondary containment berm, boiler structure, pump basin, control room/E-house and fire pump and foam structure is shown on the Area 300 Site Plan, Figure 4. Up to six product storage tanks will be constructed in Area 300. The tanks will be 240 ft in diameter, 48 ft high, and will be spaced 120 ft apart (wall to wall). The tanks will be of steel construction with a floating roof. The 50- by 60-ft boiler structure is located west of the tank farm. A small control room, fire pump and foam structure, transformer pads, and maintenance parking stalls are planned north of the boiler. A pump basin is planned adjacent to the south side of the boiler structure.

Area 400 (Marine Terminal)

The site layout for the marine terminal is shown on Figure 5. The area includes transfer pipeline, an E-house structure, fire pump and foam structure, dock transformer, vapor control unit, and maintenance parking planned near the riverbank near berths 13 and 14. The transfer pipeline is located within about 80 ft of the riverbank and will extend west from vapor control unit for approximately 1,050 ft before turning north. In addition, modifications to the existing docks and moorage dolphins are planned. Modifications to the docks and moorage dolphins and other Area 400 elements are not part of the scope of this geotechnical report and will be addressed in a supplemental report.

Area 500 (Transfer Pipelines)

The layout for the transfer pipelines is shown on Figure 2. The pipeline will consist of three, 24-in.-diameter steel pipelines extending from the rail unloading area to the product storage tanks. A 36-in.-

diameter steel pipeline with 6-in.-diameter return pipeline will run from the product storage tanks to the dock. Outside of the rail unloading area, the pipeline will typically be built above ground and rest on concrete cradles. Portions of the pipeline will be below ground where crossing beneath existing roadways and rail tracks. Based on conversations with the design team, underground portions of the 24- and 36-in.-diameter transfer pipelines will be housed inside 36-and 48-in.-diameter steel pipes, respectively.

SITE CONDITION AND BACKGROUND

Topography and Site Background

Areas 200 and 600. Existing topographic information indicates the ground surface in the area of planned improvements typically ranges from elevation +28 to +35 ft based on the National Geodetic Vertical Datum of 1929 (NGVD 29). All elevations in this report are reported in NGVD. The ground surface in the area of the planned unloader and boiler structure is typically mantled with crushed rock. The ground surface in the area of the office and changing room structure is mantled with sand fill or crushed rock. At the time of our investigation, the Port was placing and compacting additional sand fill in the area of the office and changing room structures. Up to eight rail tracks are located immediately north of the planned unloader structure. There are asphaltic-concrete (AC) paved roads around the planned boiler structure and north of the office and changing room structures.

T-5 was formerly occupied by an aluminum smelting facility owned and operated by Aluminum Company of America (Alcoa), Vancouver Aluminum Company (Vanalco), and Evergreen Aluminum at various periods beginning in 1940. Alcoa, in conjunction with Vanexco, also operated an aluminum rod and wire extrusion facility at T-5 until 1991. Soils in the area of the aluminum facility were contaminated with hazardous materials during the years of the aluminum facilities operations (ICF Jones and Stokes, 2009). As a result of the contamination, Alcoa and Evergreen Aluminum conducted a cleanup effort throughout T-5 that included construction of engineered landfills and caps that include the Vanexco concrete cap located adjacent to the north edge of the unloader structure. Soils in the Vanexco cap are impacted with polychlorinated biphenyls (PCBs). Based on current plans, the majority of the improvements are outside the area of the Vanexco cap except for a small portion of the unloader facility and the pedestrian bridge that provides access to the unloader rack from the facility offices.

Area 300. Existing topographic information indicates the ground surface of Area 300 typically ranges from elevation +27 to +30 ft and is mantled with crushed rock over the western half and sand fill over the eastern half. The northeast corner of the area contains a partially filled stormwater retention pond with the bottom at approximate elevation +12 ft and side slopes of about 2H:1V or flatter. It is our understanding the Port will complete filling of the remnant of the stormwater pond with compacted structural fill to match the surrounding site grades. A large scrap-metal stockpile is located in the southeast portion of the tanks site. The scrap metal piles are in the area of the southeast and middle south tank footprints, see Figure 4. Several stockpiles of concrete rubble were located on the western portion of the site and have subsequently been removed by the Port. Several rail lines are located adjacent to the south boundary of Area 300.

Area 400. Area 400 consists of two existing dock structures, identified as Berths 13 and 14, consisting of 250-ft-long pile-supported T-docks. Existing topographic information indicates the ground surface at the trestle abutments for the docks is relatively flat at about elevation +27 ft and is typically mantled with AC pavement, gravel, or grass. Two stormwater infiltration swales with the ground surface ranging from elevation +23 to +25 ft are located north of the paved areas and are mantled with grass and shrubs. A 2H:1V riprap protected slope extends from the trestle abutment down to the sandy beach at about elevation 17 ft. The dredge line at the face of the dock is at about elevation -38 ft. It is our understanding the berths will be dredged to elevation -41 ft.

Area 500. The pipeline alignment is relatively flat with the ground surface ranging from about elevation +22 to +32 ft. The ground is typically mantled with crushed rock with portions of the pipeline along AC-paved NW Gateway Avenue and NW Harborside Drive.

The pipeline alignment is adjacent to the N/N2 landfill east of the unloader structure, near the East Landfill south of the intersection of NW Gateway Avenue and NW Harborside Drive. The N/N2 landfill and East Landfill areas were impacted by contamination during the Alcoa operations and are considered part of the Department of Ecology restricted covenant.

Geology

Based on our understanding of the geology at the site, our experience with nearby sites, and the available exploration data, the project area is mantled by fill that is underlain by recent alluvial soils to depths of 50 to 90 ft below the existing ground surface. The manmade fills typically consist of fine to coarse granular soils, with silt, silty sand, and sandy silt. The alluvial soils typically consist of very soft to medium stiff silt with varying percentages of clay interbedded with layers of sandy silt and sand that are underlain by sand with a trace of silt. The recent alluvial soils are typically underlain by alluvial gravels that range from gravel in a matrix of sand to open-graded gravel. Recent geologic investigations near the Interstate 5 bridge about 3 miles upstream from the project site indicate the alluvial gravels on the Washington side of the Columbia River can be up to 100 ft thick near the project site.

Available geologic information indicates the alluvial gravels are underlain by the Troutdale Formation, a Pliocene-age unit of well-consolidated or cemented conglomerate and sandstone (Beeson, et al., 1991).

SUBSURFACE CONDITIONS

General

Subsurface materials and conditions at the site were investigated between June 5 and October 29, 2013, with 26 borings, designated B-1 through B-26, and six cone penetration test (CPT) probes, designated CPT-1 through CPT-6. The borings were advanced to depths of 21.5 to 104.2 ft, and the probes were advanced to depths of about 54 to 83 ft.

The locations of borings and probes performed for this investigation are shown on Figures 2 through 5. Referring to Figure 4, note that explorations have not been completed for the southeast and middle south tank footprints. Access to this area was not permitted due to the presence of a large stockpile of scrap metal. The field exploration and laboratory testing programs completed for this investigation are described in Appendix A. Logs of the borings and CPT probes are shown on Figures 1A through 32A. The terms used to describe the soils encountered in the borings and CPTs are defined in Tables 1A and 2A.

In addition to the borings and CPT probes made for this investigation, GRI also reviewed and utilized the logs of previous explorations made by GRI and others in the site vicinity for other projects. Overall, the results of the explorations recently completed for this investigation are in good agreement with previous work.

Soils

For the purpose of discussion, the materials disclosed by the explorations have been grouped into four units based on their physical characteristics, geologically significant features, and engineering properties. Listed as they were encountered from the ground surface downward, the units are:

1. **FILL**
2. **SILT**
3. **SAND**
4. **GRAVEL**

1. FILL. Fill was encountered at the ground surface in all explorations except B-26 and extends to depths ranging from about 5 to 25 ft (elevation +24.5 to +2 ft). The fill was thinnest near the northeast area of Parcel 1A (Area 300), where the Port plans to place additional fill, and near the northwest boundary of the unloader structure. The fill is thickest at the top of the riverbank in the dock area, where it is typically 20 to 25 ft thick.

The fill consists of layers of silt, sand, and gravel. Based on N-values and CPT tip resistances that vary widely across the site, the relative consistency of the sand and gravel fill varies from very loose to very dense, and the relative consistency of the silt fill ranges from very soft to hard. The fill in Area 300 is typically sand and gravel and medium dense to dense. The fill in Area 200 and 600 typically consists of layers of sand and gravel, is typically medium dense to dense, and grades to loose to medium dense below depths of 5 to 12.5 ft in several borings in the area. The fill in Area 400 consists primarily of sand and is typically loose to medium dense. The moisture content of sand fill ranges from 7 to 33% and increases with increasing silt content. The moisture content of silty fill ranges from 15 to 48%.

2. SILT. Silt was encountered in all explorations except borings B-10, B-23, and B-26. The silt was encountered beneath the fill in all borings and CPTs, except B-11, B-12, B-16, B-17, B-20, and B-25, where the silt was encountered beneath a layer of native sand that ranges from 2.5 to 15 ft thick. The silt extends to depths of 15 to 26 ft, and to the maximum depth explored in borings B-9, B-12 through B-14, and B-19. The silt ranges from 4-in.-thick interbedded layers to 19-ft-thick zone in Area 200 and 600; from 4 to 17 ft thick in Area 300; from 4 in. to 5 ft thick in Area 400; and from 12.5 to 17.5 ft thick in Area 500. Sand layers ranging from 3 to 4.5 ft thick are interbedded in the silt in Area 200. Based on N-values, Torvane shear strengths, and CPT side friction, the relative consistency of the silt ranges from very soft to stiff. Based on Atterberg limits testing, the liquid limit (LL) of the silt ranges from 27 to 76%, and the plasticity index (PI) ranges from 5 to 33%, indicating the soil has a low to high plasticity. The results of the Atterberg limits testing are shown on the Plasticity Charts, Figures 33A and 34A. The silt in Area 300 is typically very soft to medium stiff and has a medium to high plasticity with a PI greater than 16%. The silt in Area 200 and 600 is typically very soft to medium stiff and has a relatively low plasticity with a PI ranging from 5 to 15% and more typically 5 to 6%. The silt in Area 500 is very soft of medium stiff. The silt in Area 400 is medium stiff. An approximately 1-ft-thick layer of silt with varying gravel content was encountered above the gravel in the explorations in Area 400.

Consolidation test data for selected samples of silt obtained from borings B-4, B-5, B-7, B-8, B-11, and B-19 at depths of 10 to 32 ft indicate the silt is slightly preconsolidated and displays a relatively low compressibility in the preconsolidated range and a moderate compressibility in the normally consolidated range of stresses. Consolidation test results are shown on Figures 35A to 40A. Secondary compression testing was completed on selected samples and is shown on Figures 41A and 42A in the form of curves showing dial reading versus the log of time.

3. SAND. Sand was encountered at the ground surface in boring B-26 and beneath the silt in the remaining borings and CPTs and extends to the underlying gravel at depths ranging from 40 to 64 ft. Borings B-10, B-11, B-15, B-17, B-18, and B-20 were terminated in sand. The sand is fine to coarse grained and contains varying percentages of silt, ranging from a trace of silt to silty. A trace of gravel was present in some of the sand. The thickness of the sand ranges from 20 to 67 ft. Interbedded layers of silt ranging from 1 to 14 ft thick are present in the sand in Area 200 and 600. N-values recorded in the borings in Areas 200 and 600 indicate the relative density of the sand is typically loose from 10 to 20 ft and medium dense below a depth of about 30 ft. In Area 300, the sand is typically loose in the upper 5 to 10 ft of the sand layer and typically grades to medium dense below depths of 25 to 40 ft. In Area 400, the sand is typically loose to a depth of 35 ft and medium dense below a depth of 35 ft. In Area 500, the sand is typically loose in the upper 5 to 10 ft of the sand layer and grades to medium dense below a depth of 30 to 40 ft.

4. GRAVEL. Gravel was encountered beneath the sand in borings B-1 through B-8, B-16, B-21 through B-26, and probes CPT-1 through CPT-6. Gravel was encountered at about elevation -24 to -30 ft in Area 300, elevation -64.5 ft in Area 200, elevation -25.5 ft in Area 500, and elevations -43.5 to -60 ft in Area 400. The gravel is typically in a matrix of sand and silt and contains scattered cobbles and possible boulders. Layers of relatively open-graded gravels were noted in Area 300. Interbedded layers of sand ranging from 4 in. to 4.5 ft thick occur in the gravel in Area 300. Based on N-values, the relative density of the gravel ranges from medium dense to very dense.

Groundwater

Groundwater levels in the project area fluctuate in response to seasonal river levels, precipitation, and daily tidal fluctuations in the river. Shallow perched groundwater conditions can develop above the less-permeable silty deposits at the site and approach the ground surface during periods of prolonged or intense rainfall.

The Columbia River level is lowest in late summer and early fall. Historical low water in the last 20 years is about elevation +2.5 ft. The 100-year flood level is about elevation +28 ft. The ordinary high water level (OHW) is about elevation 17 ft. The higher river levels typically occur during storm events and the spring freshet, when snowmelt runoff causes high river flows.

Vibrating-wire piezometers were installed in borings B-4 and B-7 in Area 300. The piezometers are connected to data logger systems that automatically record the groundwater level. Installation details for the piezometers are described in Appendix A. The groundwater elevations between June 7 and July 10, and August 2 through 23, 2013, were recorded at up to 2-hour intervals and are shown on Figure 6. The piezometer data indicate the groundwater elevation fluctuated between elevation +4 and +10 ft over the recorded period. Hydrograph river levels recorded at a nearby station on the Columbia River during the same period were converted to NGVD elevations and are shown on the figure. Comparison of the recorded groundwater and river levels suggests the groundwater is typically near or slightly higher than the river elevation. In this regard, it should be anticipated the groundwater level at the site could rise to or very near the ground surface during flood events. The groundwater elevations shown on Figure 6 are based on survey data provided to GRI by McKay Sposito. The batteries in the data loggers recording piezometer data in borings B-4 and B-7 stopped working on June 19 and July 10, 2013, respectively, and were replaced on August 2, 2013. It should be anticipated the groundwater level in the project area will reflect the water levels in the Columbia River.

CONCLUSIONS AND RECOMMENDATIONS

General

The borings and CPT probes completed for this investigation indicate the site is mantled with 5 to 25 ft of silt, sand, and gravel fill that is underlain by alluvial silt and sand. Explorations for previous work at the port indicate a lesser thickness of fill is present along the proposed pipeline alignment between Gateway Avenue and the new Gateway Avenue bridge. Where there is less fill, the native silt soils are present near the ground surface. Boring B-26, completed on the beach near Berth 13 encountered sand at the ground surface. Dense gravel is present below depths of 40 to 95 ft. Groundwater levels at the site will fluctuate in response to precipitation and levels in the nearby Columbia River. Shallow perched groundwater conditions may develop in the fill and approach the ground surface during periods of prolonged precipitation.

Our studies indicate that during the design level earthquake, the loose to medium dense sands and layers of low-plasticity, soft to medium stiff silt and sandy silt that are present below the groundwater level in all areas of the site could liquefy to the top of the gravel layer. Liquefaction results in settlement, a reduction of soil strength, and significant lateral spreading deformations near the riverbank. Ground improvement, such as stone columns, vibro-compaction, jet-grouted columns, and soil mixing, can be designed to mitigate liquefaction-induced settlement and lateral spreading deformations. Ground improvement will likely be necessary in Area 400 and portions of Area 500 to limit lateral spreading along the riverbank. The section of the Area 500 pipeline adjacent to the riverbank is also close to the East Landfill. Geotechnical borings were not allowed in the landfill.

A compressible layer of silt is present in Area 300. The heavy product storage loads will induce significant consolidation (static) settlement beneath the tanks that will likely need to be mitigated with ground improvement.

Due to the static and seismic settlement considerations and lateral spread due to seismic loading, conventional spread footings may not be able to support some of the structures during the design seismic event. In this regard, portions of the improvements may be supported on driven steel piles or ground improvement to limit static and seismic deformations.

The following sections of this report provide our conclusions and recommendations for design and construction of the site improvements. Static settlements will be in addition to liquefaction-induced settlements following a strong earthquake.

Seismic Considerations

General. We understand seismic design of the project elements, except the dock structure, will be in accordance with the 2012 International Building Code (2012 IBC), which was recently adopted by the Washington State Building Code and incorporates recommendations from the ASCE 7-10, Minimum Design Loads for Building and Other Structures (ASCE 7-10). The 2012 IBC and ASCE 7-10 seismic hazard levels are based on a Risk-Targeted Maximum Considered Earthquake (MCE_R). The ground motion associated with the probabilistic MCE_R represents a targeted risk level of 1% in 50 years probability of collapse in the direction of maximum horizontal response. In general, these risk-targeted ground motions are developed by applying adjustment factors of directivity and risk coefficients to the 2% probability of exceedance in 50 years, or a 2,475-year return period hazard level ground motion developed from the 2008 U.S. Geologic Survey (USGS) probabilistic seismic hazard maps. The maximum horizontal direction spectral response accelerations were obtained from the USGS Seismic Design Maps for the coordinates of 45.65° N latitude and 122.71° W longitude. The S_S and S_1 values identified for the site are 0.94 and 0.41 g, respectively. These bedrock spectral ordinates are adjusted for Site Class with the short- and long-period site coefficients, F_a and F_v, based on subsurface conditions or with a site-specific response analysis. The design-level response spectrum is calculated as two-thirds of the Site Class-adjusted MCE_R-level spectrum.

Our analysis has identified a potential risk of liquefaction throughout the site. In accordance with ASCE 7-10, sites with subsurface conditions identified as vulnerable to failure or collapse, such as liquefied soils, shall be classified as Site Class F. For Site Class F sites, ASCE 7-10 section 20.3 requires completion of a site-specific ground motion analysis unless the structures have a fundamental period of vibration less than or equal to 0.5 second. The design response spectrum for sites with structures of fundamental period less than 0.5 second can be derived using the non-liquefied subsurface profile.

We expect the large storage tanks in Area 300 will have fundamental periods of vibration that exceed 0.5 second. Due to these anticipated longer periods, a site-specific seismic ground motion analysis is required due to the presence of liquefiable soils. For Areas 300 and 400, a site-specific ground motion analysis was completed with the aid of the computer software D-MOD, a non-linear seismic soil response software developed by GeoMotions, LLC. The D-MOD analyses are further discussed in the site-specific ground motion analysis in Appendix B. The site-specific seismic ground motion analysis completed for Area 400 was completed as part of a separate scope of work to assist with evaluating the existing dock.

The recommended spectra with no ground improvement are summarized below by Area. Depending on the final type and extent of ground improvement selected, the Site Class assumption should be confirmed during final design.

Areas 200, 500, and 600. For Areas 200, 500, and 600, we understand the structures have a fundamental period of vibration less than or equal to 0.5 second. Based on the site conditions and no ground improvement, we recommend using Site Class E, soft soil profile, and the corresponding code site coefficients to develop the design spectra for those areas. The subsurface conditions in Area 500 within about 300 ft of the riverbank are more sandy than the rest of the area and are similar to the Area 400 site conditions (i.e., more sand). Therefore, we recommend the design spectrum for the section of pipeline within 300 ft of the river be developed based on the recommendations for Area 400.

Area 300. For Area 300, the results of the site-specific modeling indicates the 2012 IBC code-based Site Class D spectrum provides an appropriate estimate of the spectral accelerations in Area 300 for short periods. For longer periods, a response spectrum consisting of the site-specific spectral response values and the spectral values corresponding to 80% of the Site Class E response spectrum is appropriate. The recommended design-level spectral acceleration for Area 300, based on the site-specific ground motion analysis with no ground improvement, is shown on Figure 12B.

Area 400. For Area 400, the results of the site-specific modeling indicate the 2012 IBC code-based Site Class D spectrum provides an appropriate estimate of the spectral accelerations at short periods, while a response spectrum encompassing the site-specific spectral values and 80% of Site Class E is considered to be appropriate at longer periods. The design-level spectral acceleration spectrum for Area 400 is shown on Figure 12B.

Liquefaction. Liquefaction is a process by which saturated, granular materials, such as sand, and to a somewhat lesser degree, non-plastic silts, temporarily lose strength during and immediately after a seismic event. Liquefaction occurs as seismic shear stresses propagate through a saturated soil and distort the soil structure causing loosely packed groups of particles to contract or collapse. If drainage is impeded and cannot occur quickly, the collapsing soil structure increases the porewater pressure between the soil grains. If the porewater pressure increases to a level approaching the weight of the overlying soil, the granular layer temporarily behaves as a viscous liquid rather than a solid. As strength is lost, there is an increased risk of settlement, lateral spread, and/or slope instability, particularly along waterfront areas. Liquefaction-induced settlement occurs as the elevated porewater pressures dissipate and the soil consolidates after the earthquake.

The potential for liquefaction is typically estimated using the simplified method which compares the cyclic shear stresses induced within a soil profile during an earthquake to the ability of the soils to resist these stresses. The stresses induced within the profile are estimated on the basis of earthquake magnitude and the accelerations within the profile. The ability of the soils to resist these stresses are based on their strength as characterized by SPT N-values and CPT cone tip resistances normalized for overburden pressures and corrected for factors, such as fines content. Bray and Sancio (2006) provide additional screening criteria regarding the liquefaction susceptibility of silty soils that are not addressed by the simplified method. According to Bray and Sancio, soils with a water content to liquid limit ratio greater than 0.85 and a plasticity index (PI) less than 12% are susceptible to liquefaction.

The potential for liquefaction at the site was evaluated with the simplified method based on two methodologies. The first methodology was utilized for all areas and is based on the simplified procedure by Youd, et al (2001). The analysis was completed with the aid of the computer software LiquefyPro, a seismically induced liquefaction and settlement analysis software developed by CivilTech Corporation. The Youd, et al., methodology utilizes the peak ground acceleration (PGA) to predict the cyclic shear stresses experienced by the soil. The second methodology is based on the simplified procedure by Idriss and Boulanger (2008) and was utilized for Area 300. The Idriss and Boulanger (2008) analysis was used to evaluate the liquefaction potential of soils based on cyclic shear stresses and increased pore pressures estimated by the site response analysis output from the computer program D-MOD. In accordance with ASCE 7-10 Section 11.8.3, the PGA used in liquefaction hazard evaluation is the Maximum Considered Earthquake Geometric Mean (MCE_G) PGA adjusted for site amplification and is determined either through site-specific ground motion analyses or is the mapped MCE_G PGA determined from ASCE 7-10, Figure 22-7. The mapped MCE_G is based on the 2008 USGS SDM and reflects a seismic hazard of 2% probability of exceedance in 50 years. The mapped bedrock MCE_G PGA for the site is 0.41 g.

It should be emphasized that the hazard level (MCE_G) used to estimate liquefaction hazards based on the new ASCE 7-10 document is approximately 50% larger than in previous code cycles. This new requirement results in much larger ground deformation estimates.

Based on the 2008 USGS interactive deaggregations, the subduction, subcrustal, and local crustal earthquakes all provide a significant contribution to the probabilistic seismic hazard at the site. We have considered magnitude M6.8 and M9.0 earthquakes, corresponding to a local crustal and subduction zone earthquake, respectively, with a PGA of 0.37 g and 0.45 g for Site Class E and D, respectively, for our liquefaction studies. These analyses indicate the crustal and subduction zone earthquake control the seismic hazard and contribute similar liquefaction hazards to the site. For the purpose of liquefaction studies, we have assumed a groundwater level at elevation 12 ft, corresponding to a seasonally averaged daily high river level.

The results of the liquefaction hazard analysis indicate the loose to medium dense sands and layers of low-plasticity, soft to medium stiff silt and sandy silt present below the groundwater level could liquefy to the top of the gravel layer at depths of up to 60 to 80 ft. Laboratory testing indicates the PI of the silt samples obtained from Area 300 ranges from 16 to 33%, which indicates the silt has a low risk of liquefaction based on the Bray and Sancio criteria. Our laboratory testing indicates the PI of the silt samples obtained from Areas 200 and 600 ranges from 5 to 15% and is typically 5 to 6% and has a moderate risk of liquefaction based on the Bray and Sancio criteria. For the purpose of estimating liquefaction-induced settlements, we have estimated the silt soils in Areas 200 and 600 are susceptible to liquefaction, and the silt soils in Area 300 are not susceptible to liquefaction. Based on these assumptions and on the new MCE_G ground motions, we estimate liquefaction-induced settlements will be on the order of 10 to 16 in. in Areas 200 and 600, 6 to 10 in. in Area 300, 3 to 15 in. in Area 500, and 12 to 24 in. in Area 400.

Lateral Spreading. Lateral spreading involves the horizontal displacement of large volumes of soil as a result of the liquefaction of underlying layers. The ground displacement occurs in response to the combination of gravitational forces and inertial forces generated by an earthquake acting upon the soil mass. Lateral spread can develop on shallow sloping ground or as a flow slide moving toward a moderately to steeply sloping free face, such as a river channel or lake bottom. Differential internal movements within the spreading mass usually create surface features, such as ground cracks or fissures, scarps, and grabens, in overlying unsaturated or non-liquefied soils. Lateral displacement may range from a few inches to many feet depending on soil conditions, the steepness of the slope, and the magnitude and epicentral distance of the earthquake. Associated differential vertical movements, or ground surface subsidence, may range up to about half of the total horizontal movement.

The method of analysis developed by Youd, et al. (2002), can be used to estimate lateral spread for both free-face and continuous slopes in free-field conditions. Calculations were completed assuming a subduction zone earthquake with a moment magnitude M9.0 at an epicentral distance of about 86 km and a crustal earthquake with a moment magnitude M6.8 at an epicentral distance of about 7 km to represent the MCE_G hazard level defined in ASCE 7-10. The 2008 USGS interactive deaggregations indicate these magnitude-distance pairs provide the highest contribution to the seismic hazard at the site for subduction zone and crustal earthquake scenarios, respectively. The analysis indicates a potential for tens of feet of horizontal deformation 100 ft from the top of the riverbank, 4 to 9 ft of movement 650 ft from the riverbank, and 0 ft of horizontal deformation 1,150 ft from the riverbank.

The methods used to estimate the seismically induced horizontal and vertical ground displacement at the site are largely based on empirical methods and, consequently, do not provide a precise estimate of the actual ground movement that may occur. Seismic events of a lesser magnitude, or of the same magnitude but occurring at a greater epicentral distance from the site, would be expected to produce lesser horizontal and vertical ground displacements. As discussed in the liquefaction section the hazard level required by the current code is much greater than previous codes. For estimating lateral spreading deformations, the greater hazard level results in lateral spreading displacement estimates that can be larger by an order of magnitude.

In summary, at the new MCE_G hazard level, it is estimated that lateral spreading can occur within about 1,150 ft of the riverbank. Lateral spreading causes horizontal displacement of structures and additional lateral structural loads on piles and walls if not mitigated. Based on the lateral spreading estimates provided above, the pipeline and other structures may not be able to accommodate the estimated horizontal movement or lateral spreading loads, if ground improvement is not completed, in particular near the riverbank.

Ground Improvement. A ground improvement program can be designed to improve the existing subsurface soils and reduce potential seismic-induced settlement and lateral spreading. We anticipate ground improvement, if used for Areas 200, 300, 500, and 600, would be designed by a specialty ground improvement contractor to meet specified performance criteria. Ground improvement design to reduce lateral spreading near the river could be designed by the project team or a contractor to limit seismic deformation to tolerable levels.

Lateral spreading is often mitigated by constructing a zone, or buttress, of improved soil along the riverbank that will not liquefy. The buttress needs to be of sufficient width and extend to adequate depth to maintain stability following ground shaking and minimize or prevent lateral displacement toward the river of the upland portion of the site behind the buttress.

Several ground improvement alternatives, including vibro-compaction, vibro-replacement (stone columns), displacement piles, soil mixing and jet grouting are feasible to mitigate seismically induced settlement and lateral spreading. Vibro-compaction is a ground improvement technique that densifies clean granular soils, such as clean sand, using a vibratory probe. The probe is vibrated and jetted into the ground until reaching the bottom of the improvement zone. The soils are densified by the vibratory process as the probed is removed. Stone columns are similar to vibro-compaction, except stone aggregate is added to the void created by the probe after reaching the bottom of the treatment zone. The aggregate is densified by lowering the probe into the aggregate in small lifts until reaching the ground surface, creating columns of compacted aggregate. Stone columns are typically used in sand that contains a significant portion of fine-grained soils (silt or clay) or in low-plasticity, fine-grained soils with risk of liquefaction.

Vibro-compaction is most effective in sands with fines contents (percentage passing the No. 200 sieve) of less than 15%, and stone columns are more effective for soils with a fines content greater than 15%. Due to the silt layers and relatively high fines content over most of the upland portion of the project site, ground improvement by vibro-replacement with stone columns is more appropriate than vibro-compaction over most of the site to limit liquefaction-induced settlement. However, subsurface conditions near the Area 400 riverbank are predominantly sand, which could be modified using vibro-compaction methods.

Soil mixing and jet grouting are ground improvement methods that mix cement into the in situ soils to create columns of soil with improved strength and stiffness. The soil mixing method mixes wet or dry cement by use of a mechanical paddle that is advanced similar to a drill. The diameter of the soil-mixed column is dependent on the diameter of the paddle tool. Jet grouting makes soil/cement columns by injecting cement grout through high-velocity grout jets. The jets erode the in situ soil and mix it with cement and sometimes air and water. Jet grouting can be used to construct improved soil/cement columns or overlapped to create continuous panels.

Other Seismic Considerations. In our opinion, the potential for earthquake-induced fault displacement and ground rupture at the site is low unless occurring on a previously unknown or unmapped fault, and the risk of tsunami at the site is absent. Due to the topography of the site, it is our opinion the risk of damage by seiche is low.

Areas 200 Unloading and Office Structures and Area 600 West Boiler

General. As previously mentioned, the site layout for the rail unloading area, administrative and support structures, and west boiler are shown on Figure 3. It is our understanding the unloader structure, boiler structure, trenches, office, changing rooms, control room, and fire pump and foam structure, and transformer pads will be lightly loaded. As discussed in the Seismic Considerations section of this report, we estimate 10 to 16 in. of liquefaction-induced settlement in Areas 200 and 600 during a design seismic event. It is reasonable to assume that differential settlement could be half of the total liquefaction-induced settlement over horizontal distances of about 50 ft.

Foundation Support. Spread footings for support of Area 200 and 600 units can be designed using the criteria summarized in the Spread Footings section of this report. Liquefaction-induced settlement of structures founded on spread footings is estimated to be the same as noted in the previous paragraph. Seismic settlement of structures can be reduced to less than 1 in. by using driven pipe pile foundations. Pile support for structures in Areas 200 and 600 can be designed using the criteria provided in Table 3 of the Driven Piles section of this report. LPILE criteria are provided in Table 4 of the Pile Lateral Load section of this report. Alternatively, a foundation system consisting of spread footings following ground improvement could likely be designed to limit seismic settlements to acceptable levels.

Unloading Trenches. The oil will be pumped from rail cars into transport pipelines supported in two embedded concrete trenches located adjacent to the rails for the length of the unloader structure. The unloader trench will have a reinforced concrete bottom and sidewalls and an open top. The floor will be established 5 to 7 ft below the ground surface.

For design of the unloader trench, it is prudent to assume the groundwater level could rise to the ground surface during periods of heavy rainfall, together with river flooding.

Design lateral earth pressures for retaining walls depend on the type of construction, i.e., the ability of the wall to yield. Possible conditions are 1) a wall that is laterally supported at its base and top and, therefore, is unable to yield (at rest condition), and 2) a conventional cantilevered retaining wall that yields (active condition) by tilting about its base.

Assuming groundwater at the ground surface, non-yielding walls should be designed using a lateral earth pressure based on an equivalent fluid having a unit weight of 90 pcf. Walls that are allowed to yield by tilting about their base can be designed using a lateral earth pressure based on an equivalent fluid having a unit weight of 80 pcf. These design lateral earth pressures assume the grade adjacent to the trenches is horizontal. We understand additional lateral loading on the embedded trench walls induced by nearby rails will be evaluated by the project designer.

Assuming the design-level earthquake and 100-year flood event will not occur concurrently, our analyses indicate the above design criteria result in larger lateral earth forces than a design-level earthquake with groundwater below the bottom of the trench. Our analyses indicate the critical lateral earth pressures for design of the vault walls are associated with groundwater at the ground surface, rather than the seismic loading conditions.

Resistance to buoyant forces, if necessary, is typically provided by increasing the weight/volume of concrete and/or by extending the base slab beyond the walls. The buoyant unit weight of backfill over the slab extension can be taken as 53 pcf.

The unloader trench excavation can be backfilled with excavated sand compacted to 95% of the maximum dry density as determined by ASTM D 698 (Standard Proctor). To avoid buildup of excessive lateral earth pressures, overcompaction of backfill within 5 ft of the walls should be avoided.

Based on review of the logs of borings along the unloader, the bottom of the excavation will expose sand and silt. We recommend placing a minimum 8-in. thickness of compacted crushed rock to prevent disturbance of the subgrade and provide a firm working surface. Areas of soft or unsuitable material exposed in the subgrade should be overexcavated and backfilled with compacted crushed rock. The crushed rock should be installed in a single lift and compacted by at least four passes with a vibratory roller. For the support of point loads on the trench bottom slab, we recommend using a modulus of subgrade reaction of 125 pci. We estimate the static settlement of the unloader trench will be less than 1 in. for a relatively uniform net bearing pressure of 1,500 psf.

As noted previously, liquefaction could induce an additional 10 to 16 in. of settlement in Area 200. It is reasonable to assume that differential settlement could be one-half of the total liquefaction-induced settlement over horizontal distances of about 50 ft.

Although the unloader trench is a stiff structure, we understand the estimated liquefaction-induced settlements are excessive. To mitigate seismic-induced settlements, the unloading trench can be supported on ground modified by ground improvement methods or on driven steel pipe piles. If used, piles can also provide uplift resistance to buoyancy.

Ground Improvement. We understand ground improvement is being considered as an alternative to limit liquefaction-induced settlements beneath the structures. If used, ground improvement would be designed by a specialty contractor to meet performance criteria developed by the design team. Vibro-compaction, vibro-replacement (stone columns), soil mixing, and jet grouting are typical ground improvement types that can be used to mitigate seismically induced settlement.

Area 300 Storage Tanks

General. The site layout for the storage tanks, boiler structure, pump basin, control room, and fire pump and foam structure is shown on Figure 4. As discussed in the Seismic Considerations section of this report, we estimate 6 to 10 in. of liquefaction induced settlement in Area 300 during a design seismic event. In addition, the storage area is underlain by compressible silt soils that will induce non-seismic consolidation settlement due to the weight of the storage tanks.

Steel Storage Tanks. The steel tanks will be 240 ft in diameter, 48 ft high, and have a floating roof. Based on conversations with the project structural designer, R&M Structural Engineering (R&M), the tanks can tolerate 8 in. of settlement at the center of the tanks. The perimeter of the tanks can tolerate up to 2 in. of total settlement and differential settlement of 1/2 in. over 32 ft.

Static Tank Settlement. Assuming a product unit weight of 52 pcf, we estimate the bearing pressure of the full tank filled with crude oil will be about 2,500 psf. Estimated total settlements at the center of tanks established at grade with no ground improvement under static conditions are summarized in Table 1 below. The range of settlements between tank locations is due primarily to the variable thickness of compressible silt soil in the tank area, which ranges from about 3 to 17 ft.

TABLE 1: TANK SETTLEMENT ESTIMATES

Location	Estimated Settlement, in. *
Northwest Tank	3 to 5.5
Middle North Tank	2 to 4.5
Northeast Tank	2 to 4.5
Southwest Tank	3.5 to 7
Middle South Tank	No Exploration Data
Southeast Tank	No Exploration Data

* Inner two-thirds of tank footprint

Settlements at the perimeter of the tanks are estimated to be about two-thirds of the settlement in the inner two-thirds of the tank. Subsurface explorations for the middle south and southeast tanks were not completed due to the presence of large stockpiles of scrap steel. Areas under the steel stockpiles will experience less consolidation settlement. Depending on the locations of the middle south and southeast tanks, the preloading by the stockpiles may result in significant differential settlement of the tanks. Additional field explorations and settlement analysis are planned for the middle south and southeast tanks when the area is accessible.

We anticipate 90% of the estimated settlement will occur over a timeframe of about 1 to 2 months. The estimated rate of settlement is based on available laboratory data and one-dimensional time-rate of consolidation theory, and compares well with settlement data from recent projects completed at the Port. The actual time to achieve the settlement may vary from the theoretical estimates depending on natural variations in the compressibility of the underlying soils, the time that may be required for the load to be applied, and variations in seasonal groundwater levels. The maximum amount of settlement will occur after a period where the groundwater levels are at their lowest, which typically corresponds to the seasonally lowest levels of the Columbia River in the late summer and early fall. The settlement estimates in Table 1 are based on a low groundwater level of elevation 2.5 ft.

In addition to the settlement discussed above, it is likely the tank loads will result in a small amount of secondary compression, which typically occurs over a long period of time. We estimate less than 1 in. of secondary compression over 20 years.

As an additional consideration, a portion of the northeast tank footprint is currently occupied by a remnant of a former stormwater facility. It appears that 17 ft or more of fill will be needed to match existing grades. Because the fill will induce settlement, we recommend installing the fill at least 3 months prior to final grading of the tank pad so the fill-induced settlement can occur prior to tank construction.

Ground Improvement. Ground improvement methods will likely be required to limit consolidation (static) and liquefaction-induced settlements beneath the tanks. Preliminary evaluation of combined ground improvement by vibro-replacement (stone columns) and soil mixing is being completed by a foundation specialty contractor to meet static and seismic deformation criteria. Ground improvement methods to improve drainage, such as wick drains, can also be designed to reduce the timeframe for static settlement of the tanks to occur.

Water Testing. We understand the tanks will be filled with water to test for leaks and allow the tanks to settle prior to attaching exterior piping to the tanks. We estimate the tanks completely filled with water will induce a bearing pressure of about 3,000 psf. Surcharging the tanks to 3,000 psf and allowing sufficient time for consolidation to occur will reduce post-construction settlement. Although water testing the tanks will not reduce liquefaction-induced settlements, it may reduce the amount of overall ground improvement required to meet the foundation performance criteria.

Based on conversations with R&M Engineering Consultants, filling the tanks in approximate one-quarter capacity or greater increments to allow re-leveling of the tank at each stage is being considered as an alternative to mitigate static settlement. Estimates of the total settlement for each loading increment are tabulated below.

TABLE 2: SETTLEMENT DUE TO TANK PRELOADING

	Estimated Total Settlement, in. *			
Location	**1/4 Full of Water**	**1/2 Full of Water**	**3/4 Full of Water**	**Full of Water**
Northwest Tank	< 1	1 to 2.5	2.5 to 5.5	4 to 7.5
Middle North Tank	< 1	1 to 2	2.5 to 4.5	3 to 5.5
Northeast Tank	<1	1 to 2	1 to 4.5	3 to 5.5

	Estimated Total Settlement, in. *			
Location	**1/4 Full of Water**	**1/2 Full of Water**	**3/4 Full of Water**	**Full of Water**
Southwest Tank	< 1	1.5 to 3.5	3.5 to 7	5 to 9.5
Middle South Tank	No Exploration Data			
Southeast Tank	No Exploration Data			

* Inner two-thirds of tank footprint

The estimated settlements tabulated above are total settlement for a given loading. Incremental settlement is the difference between one load increment and another. We anticipate 90% of the settlement at each stage will occur over a timeframe of about 1 to 2 months.

As an alternative to filling the tanks with water, the tank footprints may be preloaded or surcharged with a temporary fill to reduce post-construction settlement. We estimate 25- to 30-ft-thick temporary fill placed above finished floor elevation will be necessary to induce a load equivalent to a full tank of water, depending somewhat on the material used to construct the temporary fill. The top edge of the temporary preload fill should extend a minimum of 5 ft beyond the limits of the planned tanks. The sides of the preload fill can be sloped at about 1.5H:1V. We estimate 90% of the preload or surcharge settlement will occur over a timeframe of about 1 to 2 months.

For areas surcharged with a full tank of water or an equivalent stockpile of fill and with no ground improvement, we estimate the subsequent post-construction static settlement resulting from the 2,500-psf tank service bearing pressure will be reduced to about 0.5 to 2 in. The estimates of the magnitude and rate of settlement in this section assume no ground improvement. As noted previously, ground improvement will reduce the magnitude and timeframe of settlement and would be designed by a specialty contractor.

Settlement Monitoring. We recommend monitoring settlement during water testing or preloading. The monitoring data will serve as the basis for evaluating the rate of filling and settlement, when additional stages can be completed, and the duration of the water testing or preloading. In our opinion, settlement plates will provide the most direct and effective method of monitoring movement during and following preloading if a stockpile fill is used. Survey markers on the side of the tanks will be the most effective if preloading the tanks with water is used. For water testing, settlement in the central portion of each tank can be measured by installing electronic settlement transducers beneath the tanks. We recommend installing at least three settlement plates or transducers within the central portion of each tank. In addition, we recommend at least three additional vibrating-wire piezometers installed beneath the tank locations to allow collection of available groundwater/piezometric data. Piezometric data can be used to estimate the degree of consolidation completed. Depending on the rate of fill construction, the settlement plates or survey markers should be surveyed twice a week during preloading or water testing. A typical settlement plate detail is shown on Figure 7.

It is important to collect groundwater level data during the course of preloading the tank. As mentioned in the Groundwater section of this report, vibrating-wire piezometers were installed in borings B-4 and B-7. Care should be taken to protect these piezometers to allow accurate recording of groundwater levels at the site, important for monitoring settlement during the preload or water testing period.

The settlement monitoring program should be further developed during final design of the tanks once the foundation preparation and support details are known.

Driven Piles. As an alternative to ground improvement and surcharging the site, the tanks could be supported on driven steel piles. Pile support can be designed using the criteria provided in Table 3 of the Driven Piles section of this report. LPILE criteria for Area 300 are provided in Table 4 of the Pile Lateral Load section of this report.

Area 300 Lightly Loaded Structures Foundation Support

It is our understanding the boiler structure, pump basin, control room, and fire pump and foam structure will be lightly loaded. The ancillary structures in the tank area can be supported on spread footings; however, without ground improvement, the structures could settle up to about 10 in. due to liquefaction. Spread footings could be designed using the criteria summarized in the Spread Footings section of this report. Seismic settlements of structures can be reduced to less than 1 in. by using driven pipe pile foundations. Pile support for the structures in Area 300 can be designed using the criteria provided in Table 3 in the Driven Piles section of this report. LPILE criteria for Area 300 are provided in Table 4 in the Pile Lateral Load section of this report. Alternatively, a foundation system consisting of spread footings following ground improvement could likely be designed to limit seismic settlements to acceptable levels.

Area 400 Marine Terminal

General. Area 400 includes about 1,050 ft of transfer pipeline, a transformer pad, E-house structure, fire pump and foam structure, and vapor control unit. The layout is shown on Figure 5. Improvements planned for the existing dock structure and moorage dolphins are outside the scope of this report and will be addressed in a supplemental report. It is our understanding the transfer pipeline, structures, and transformer pad will be lightly loaded. As discussed in the Seismic Considerations section of this report, we estimate 12 to 24 in. of liquefaction-induced settlement in Area 400 and tens of feet of lateral spread within 100 ft of the riverbank slope. As shown on the Figure 5, approximately 1,050 ft of the transfer pipeline will be located within about 100 ft of the riverbank, including a section that extends into Area 500. Due to the potential for large lateral spreading deformations, it is our opinion that ground improvement will likely be required to mitigate the impact of large seismic lateral displacements on the proposed pipeline and structures located near the river.

We anticipate foundation support can be designed using the criteria provided in the Spread Footing or Pile Foundation sections of this report. However, foundation support for the Area 400 structures will depend on the ground improvement design, including the type of ground improvement and the performance criteria. Recommendations for support of structures in Area 400 should be evaluated concurrently with design of the ground improvement during final design of the facility.

Ground Improvement. Ground improvement, such as vibro-compaction or stone column methods, can be designed to reduce lateral spreading deformations and liquefaction-induced settlements within Area 400. Lateral spreading is typically mitigated by the construction of a zone, or buttress, of densified (improved) soil along the riverbank. We anticipate the ground improvement buttress for Area 400 will be constructed by a specialty contractor in accordance with plans and performance specifications developed by the design team.

The design of ground improvement (type, depth, width, and length) depends on the project performance criteria, such as allowable deformations and lateral loads on structures. The design will require detailed discussions with the project team to develop the performance criteria. For preliminary planning and cost estimating, we estimate a ground improvement buttress will be on the order of 90 ft deep, 100 ft wide, and extend about 100 ft beyond the upstream and downstream ends of the project improvements (about 1,200 ft along the river). The dimensions of the buttress could vary significantly based on the type of ground improvement used. It should be noted that portions of the transfer pipeline are located adjacent to the East Landfill Cap. Due to restrictions associated with the landfill, geotechnical borings were not completed within the landfill boundaries.

Area 500 Transfer Pipelines

General. Based on discussions with the design team, foundation support for the pipeline will consist of spread footings or driven pipe piles. In Area 500, for unimproved ground conditions, we estimate the liquefaction-induced settlement of spread footings will be in the range 3 to 15 in., and lateral spreading deformations may occur within about 1,100 ft of the river based on the MCE_G hazard level. As noted in the discussion above for Area 400, we anticipate a ground improvement buttress will be installed along the riverbank to mitigate lateral spreading deformations that could affect the pipeline. In our opinion, a ground improvement buttress would also mitigate lateral spreading of upland areas behind the improved zone, including the north-south run of pipeline that extends from the river north along Gateway Avenue.

Foundation Support. It is our understanding the pipeline will be lightly loaded. We estimate up to 15 in. of liquefaction-induced settlement of the pipeline if supported on spread footings. Spread footings can be designed using the criteria summarized in the Spread Footings section of this report. Seismic settlements of structures can be reduced to less than 1 in. by using driven pipe pile foundations. Pile support for the pipeline in Area 500 can be designed using the criteria provided in Table 3 of the Driven Piles section of this report. LPILE criteria for Area 500 are provided in Table 4 of the Pile Lateral Load section of this report. Alternatively, a foundation system consisting of spread footings following ground improvement could likely be designed to limit seismic settlements to acceptable levels. As previously noted, a portion of the Area 500 pipeline is located adjacent to the riverbank and may require ground improvement. Foundation support for that section should be developed in conjunction with the design of the ground improvement as discussed for Area 400.

Spread Footings (Areas 200, 300, 400, 500, and 600)

The lightly loaded structures in Areas 200, 300, 400, 500, and 600 can be supported on conventional spread footings established in medium dense sand fill, undisturbed silt, or new structural fill. However, as noted in the above report sections regarding specific project areas, relatively large seismic-related settlements are estimated for spread footings unless ground improvement is undertaken.

Based on the borings completed for this project and for past projects at the Port, most of the project area is mantled by at least 5 to 10 ft of medium dense sand fill. However, explorations for previous work at the Port indicate a lesser thickness of fill is present along the proposed pipeline alignment between Gateway Avenue and the new Gateway Avenue bridge. Where there is less fill, silt subgrade may be present at the bottom of the footings. Footings should be established in firm, undisturbed soil or compacted structural fill at a minimum depth of 18 in. below the lowest adjacent finished grade. The width of footings should not be less 24 in. All foundation subgrade should be observed by a qualified geotechnical engineer. Soft, loose, or unsuitable soils encountered at footing subgrade should be overexcavated and backfilled with granular structural fill. Excavations for all footings should be made using a smooth-edged bucket and evaluated by a geotechnical engineer. For spread footings established in silt or silty sand during wet weather conditions, we recommend placing a minimum 3 in. of crushed rock over the subgrade to prevent disturbance and softening by construction activities.

Settlement estimates for square and continuous/rectangular spread footings founded on silt subgrade in accordance with the above criteria are shown on Figure 8. The figure summarizes settlement as a function of column load, bearing pressure, and footing dimensions. As previously noted, we anticipate that most of the footings will be underlain by medium dense sand fill, and the settlement estimates for footings underlain by at least 2 ft of sand can be reduced by about 20%.

The bearing pressures apply to the total of dead load plus permanently and/or frequently applied live load and can be increased to 2,500 psf for the total of all loads; dead, live, and wind or seismic. For seismic conditions, the 2,500-psf seismic allowable bearing capacity includes a factor of safety of about 2 and assumes the footings are underlain by at least 2 ft of medium dense to dense sand subgrade. The minimum 2-ft thickness of granular fill should be verified during construction, and some overexcavation and backfilling with granular structural fill should be anticipated. The ultimate bearing capacity for seismic loading depends somewhat on the amount of foundation soil that is submerged and susceptible to soil strength reduction due to liquefaction of soils during the design earthquake. We have assumed a groundwater elevation of +12 ft during seismic loading.

Horizontal shear forces can be resisted partially or completely by frictional forces developed between the base of the spread footing foundations and the underlying soil and by soil passive resistance. The total frictional resistance between the footing and the soil is the normal force times the coefficient of friction between the soil and the base of the footing. We recommend using an ultimate value of 0.35 for the coefficient of friction; the normal force is the sum of the vertical forces (dead load plus real live load). If additional lateral resistance is required, passive earth pressures against embedded footings can be computed on the basis of an equivalent fluid having a unit weight of 250 pcf. This design passive earth pressure value assumes that backfill around footings will be placed as granular structural fill.

If ground improvement is used to mitigate seismic settlement of lightly loaded structures, the allowable bearing pressure could likely be increased accordingly depending upon the type and design of the ground improvement.

Driven Piles (Areas 200, 300, 400, 500, and 600)

As discussed in the above report sections for Areas 200 through 600, elements of the improvements that cannot tolerate the estimated static and/or liquefaction-induced settlements can be supported on driven steel piles. Although pile structural loads are not available at this time, we anticipate that with the exception of Area 300 the piles will be relatively lightly loaded. However, due to potential loss of support and downdrag loading as a result of liquefaction and seismic settlement, the piles will need to be driven to the underlying gravel to minimize the risk of pile settlement during the design earthquake. To develop sufficient end bearing capacity and minimize penetration into the gravel, we recommend driving the piles closed end with a flush-fitting end plate. Recommended ultimate capacities for potential pipe pile sizes driven into the gravel are provided for each Area in the table below for the static and seismic cases. As previously indicated, pile foundation recommendations for Area 400 should be developed after ground improvement has been designed for the Area.

TABLE 3: ULTIMATE PILE CAPACITIES

AREAS 200, 500, AND 600

	Ultimate Capacity, kips		
Pile Size	**Static**	**Seismic ***	**Seismic Downdrag Load, kips**
PP 12.75 x 0.500-in.	420	325	90
PP 16 x 0.500-in.	650	525	110

* Includes downdrag reduction

Assumed gravel elevation for pile design:

Area 200 and 600 = below elevation -60 ft
Area 500 = below elevation -34 ft

AREA 300

	Ultimate Capacity, kips		
Pile Size	**Static**	**Seismic ***	**Seismic Downdrag Load, kips**
PP 12.75 x 0.500-in.	420	375	60
PP 16 x 0.500-in.	650	500	80
PP 24 x 0.500-in.	1,000	910	120

* Includes downdrag reduction

Assumed gravel elevation for pile design:

Area 300 = below elevation -28 ft

The ultimate capacities in the above table are based on soil-support considerations and may be limited by structural properties. Based on soil support properties, a factor of safety (FS) of 2 is recommended for the static case, and a FS of 1.5 is recommended for the seismic case. The ultimate capacities assume piles will have a minimum center-to-center spacing of at least three diameters (3D). The seismic capacity includes a reduction due to liquefaction and downdrag loading. Estimated seismic downdrag loads should be included in the structural design of piles. However, downdrag loads do not have to be included in determining the allowable seismic capacity because the seismic capacity includes a reduction for downdrag. For piles embedded at least 5 ft into dense gravel, we estimate that static and seismic settlements will be limited to about the elastic shortening of the piles. We conservatively estimate that the piles may penetrate up to 10 to 15 ft into the gravel. Structural loads on the piles are not available at this time, and we acknowledge that other pile types or sizes may be used to support the structural loads. The use of driven grout piles as an alternative pile type is discussed in our July 18, 2013, progress memorandum for Area 300 referenced on page 1. Larger pile capacities, if needed, may be feasible. Steel pipe piles driven to practical refusal in the gravel with a sufficiently large pile-driving hammer can essentially develop the allowable structural capacity of the pile section.

The appropriate size of impact hammer to drive the piles into the gravel and develop sufficient end-bearing resistance will depend on the actual design pile capacities. The appropriate hammer size should be evaluated on a preliminary basis with a wave equation analyses using the computer program GRLWEAP published by Pile Dynamics, Inc. We recommend completing widely spaced Pile Dynamic Analyzer (PDA) testing during the initial pile installation to evaluate the appropriate terminal driving criteria. Restrike testing after a 24-hour waiting, or set-up, period can also be used to evaluate the ultimate pile capacity using the Modified Gates equation.

Pile Lateral Loads. For conditions of lateral loading, we anticipate the piles will be evaluated using the computer software L-Pile Plus developed by Ensoft, Inc. of Austin, Texas. For lateral load analysis, we have assumed the water table is at elevation +12 ft (NGVD) to correspond to seasonally averaged high water levels for the nearby Columbia River.

Recommended input parameters for the various soil units for L-Pile analysis are tabulated below for static and seismic conditions. The parameters for use in L-pile for liquefied soil conditions were calculated using the residual undrained shear strength evaluated using the relationship between clean-sand corrected N-values (SPT test) and residual strength described by Idriss and Boulanger (2007). Residual undrained shear strength and effective overburden pressure were then used to estimate a reduced soil friction for liquefied conditions and a corresponding initial modulus, k_i.

TABLE 4: SOIL PROPERTIES FOR L-PILE ANALYSIS

AREAS 200 AND 600

				Soil Properties				
Soil Unit	**Elevation, ft (NGVD 29)**	**L-Pile Soil Type**	**Condition**	**K, pci**	**γ', pci**	**ϕ'**	**c, psi**	**ε_{50}**
Fill	Above +24	Sand (Reese)	Static & Seismic	60	0.067	34	N/A	N/A
SILT	+12 to +24	Soft Clay	Static	N/A	0.06	N/A	2.1	0.02
		Soft Clay	Seismic	N/A	0.06	N/A	1.7	0.02
Submerged SILT(1)	-3 to +12	Soft Clay	Static	N/A	0.025	N/A	2.1	0.02
		Sand (API)	Seismic	10	0.025	4	N/A	N/A
Submerged SAND(1)	-60 to -3	Sand (Reese)	Static	60	0.030	35	N/A	N/A
		Sand (API)	Seismic	10	0.030	12	N/A	N/A
Submerged GRAVEL(1)	Below -60	Sand (Reese)	Static & Seismic	125	0.04	40	N/A	N/A

AREA 300

				Soil Properties				
Soil Unit	**Elevation, ft**	**L-Pile Soil Type**	**Condition**	**K, pci**	**γ', pci**	**ϕ'**	**c, psi**	**ε_{50}**
Fill	Above +12	Sand (Reese)	Static & Seismic	150	0.07	36	N/A	N/A
Submerged SILT(1)	+12 to -4	Soft Clay	Static	N/A	0.025	N/A	3.5	0.02
		Soft Clay	Seismic	N/A	0.025	N/A	2.8	0.02
Submerged SAND(1)	-4 to -30	Sand (Reese)	Static	60	0.030	35	N/A	N/A
		Sand (API)	Seismic	10	0.030	12	N/A	N/A

Submerged GRAVEL[(1)]	Below -30	Sand (Reese)	Static & Seismic	125	0.04	40	N/A	N/A

AREA 500 (NORTH OF HARBORSIDE DRIVE [3])

Soil Unit	Elevation, ft	L-Pile Soil Type	Condition	Soil Properties				
				K, pci	γ', pci	ϕ'	c, psi	ε_{50}
Fill	Above +24	Sand (Reese)	Static & Seismic	60	0.07	35	N/A	N/A
Sand and Silt	+24 to +12	Soft Clay	Static & Seismic	N/A	0.064	N/A	3.2	0.02
Submerged Sand and Silt	+12 to -16	Soft Clay	Static	N/A	0.028	N/A	3.2	0.02
		Soft Clay	Seismic	N/A	0.028	N/A	0.6	0.02
Submerged SAND	-16 to -34	Sand (Reese)	Static	60	0.030	35	N/A	N/A
		Sand (API)	Seismic	10	0.030	12	N/A	N/A
Submerged GRAVEL	Below -34	Sand (Reese)	Static & Seismic	125	0.04	40	N/A	N/A

AREA 500 (SOUTH OF HARBORSIDE DRIVE [3])

Soil Unit	Elevation, ft	L-Pile Soil Type	Condition	Soil Properties				
				K, pci	γ', pci	ϕ'	c, psi	ε_{50}
Fill	Above +21	Sand (Reese)	Static & Seismic[4]	60	0.07	35	N/A	N/A
Sand and Silt	+21 to +12	Sand (Reese)	Static & Seismic[4]	25	0.064	32	N/A	N/A
Submerged Sand and Silt	+12 to -7	Sand (Reese)	Static	20	0.028	32	N/A	N/A
		Sand (API)	Seismic[4]	10	0.028	6	N/A	N/A
Submerged SAND	-7 to -57	Sand (Reese)	Static	60	0.030	35	N/A	N/A
		Sand (API)	Seismic[3,5]	10	0.030	12	N/A	N/A
Submerged GRAVEL	Below -57	Sand (Reese)	Static & Seismic	125	0.04	40	N/A	N/A

Notes:

1) Submerged soils are below the groundwater level.
2) Groundwater table assumed at elevation +12 ft NGVD.
3) Harborside Drive is identified on Project Layout Plan, Figure 2.
4) Assumes no lateral spreading due to ground improvement in Area 400.

The soil properties provided in Table 4 will be affected by ground improvement. If piles are installed through areas where ground improvement is used to mitigate liquefaction, the static soil properties are appropriate for use in the seismic case to the depth of the ground improvement.

It should be noted that L-pile provides isolated single-pile capacities. Depending on the direction of the loading and layout of the piles, group effects may need to be considered. Group effects can be modeled in L-pile by applying an appropriate p-modifier in non-liquefiable soils. The p-modifier is a function of the center-to-center spacing and tabulated below.

TABLE 5: P-MODIFIERS FOR GROUP EFFECTS

Center-to-Center Pile/Shaft Spacing	P-Modifiers for Rows 1, 2, and 3+
3D	0.8, 0.4, 0.3
5D	1.0, 0.85, 0.7

For liquefied conditions the p-modifier is 1.0

If additional lateral resistance is required, passive earth pressures against embedded pile caps can be computed on the basis of an equivalent fluid having a unit weight of 325 pcf. This passive earth pressure would be applicable only if the backfill for the pile caps is placed as granular structural fill and above the groundwater level.

Site Preparation and Earthwork

Demolition of improvements within the limits of the new structures and pipelines should include removal of existing pavements; floor slabs; foundations and walls; underground utilities, and associated unsuitable backfill. Where fine-grained subgrade soils are present, we recommend using hydraulic excavators equipped with smooth cutting edges for site stripping and excavation. Excavations made during demolition to remove existing improvements should be backfilled with structural fill.

In previously unimproved areas, the ground surface within areas of mass grading or within the limits of proposed pathways or structures should be stripped of vegetation, surface organics, and loose surface soils. We estimate that stripping will generally be necessary to a depth of about 4 to 6 in. in the lightly vegetated areas. Strippings should be removed from the site or used in landscaped areas. Following stripping and prior to filling, the resulting subgrade should be evaluated by the geotechnical engineer for the presence of soft areas. If present, soft areas should be overexcavated and replaced with compacted structural fill as described below. During and following stripping and excavation, the contractor must use care to protect the subgrade from disturbance by construction traffic.

The borings, CPT probes, and existing geotechnical information indicate the site is typically surfaced with sand fill or crushed rock base course. These materials will generally provide a good working surface; however, the contractor will need to use care during wet conditions to avoid disturbing and loosening the subgrade. Sand subgrade should be moisture conditioned and compacted with a medium- to large-size vibratory roller to meet the compaction criteria of structural fill immediately prior to fill placement. Recommendations for structural fill are provided in the Structural Fill section below.

Due to the variable nature of the fill at the site, it should be anticipated that silty soils will be encountered near the ground surface in localized areas. Silty soil is fine grained and sensitive to moisture content. During wet conditions, silty soils are easily disturbed, rutted, and weakened by construction activities. If silty subgrade is encountered during site stripping, haul roads or work pads constructed of imported granular fill will be needed to provide access and protect areas of fine-grained subgrade from damage due to construction traffic during wet conditions. In our opinion, a 12-in.-thick granular work pad should be sufficient to prevent disturbance of the silt subgrade by lighter construction equipment and limited traffic by dump trucks. Haul roads and other high-density traffic areas will require at least 18 to 24 in. of crushed rock to prevent subgrade deterioration. Any subgrade soils that are disturbed by construction activity should be overexcavated to firm soil and backfilled with structural fill placed and compacted as recommended in the Structural Fill section of this report. Haul road requirements will be minimized if work is accomplished during the driest months of the year. The performance of haul roads can usually be improved by placing a geotextile fabric over the fine-grained subgrade soils prior to placing the rock.

Temporary cut and fill slopes should be 1H:1V or flatter. Permanent cut and fill slopes should be constructed at 2H:1V or flatter. Containment berms will be constructed around the tank farm in Area 300. The berms will likely be constructed of sand obtained within the project limits or imported materials. Sand can be placed as structural fill and maintain 2H:1V side slopes. However, the surface of the berm slopes may experience shallow sloughing due to wetting/drying and freeze/thaw cycles. Periodic maintenance may be required and can be minimized by initially overbuilding the structural fill and subsequently trimming back to the neat slope lines, or by constructing the berms with a flatter slope.

Structural Fill

On-site soils that are free of organics and other deleterious materials and debris are suitable for construction of compacted structural fill. As noted above, it should be anticipated that near-surface, silty soils will be encountered locally. Silty soils are sensitive to moisture content and can be placed and adequately compacted only during the dry, summer months. For construction during the wet, winter and spring months, fills should be constructed using granular materials that are relatively clean, i.e., less than about 7% passing the No. 200 sieve (washed analysis), such as on-site surficial sand fill material.

In general, approved on-site or imported, organic-free, fine-grained sand and silty soils used to construct structural fills within areas of mass filling, structures, and pathways should be placed in 9-in.-thick lifts (loose) and compacted to at least 95% of the maximum dry density as determined by ASTM D 698. Pieces of rock or concrete larger than about 6 in. should be removed from the fill prior to compaction. Fill placed in landscaped areas should be compacted to a minimum of about 90% of the maximum dry density as determined by ASTM D 698. The moisture content of structural fill soils at the time of compaction should be controlled to within 3% of optimum. Some moisture conditioning of fine-grained sand and silty soils may be required to achieve the recommended compaction criteria. All structural fills should extend a minimum horizontal distance of 5 and 2 ft beyond the limits of structures and pavement areas, respectively. Vibratory equipment is most effective for compacting the on-site sand and imported granular materials.

On-site or imported granular material used to construct structural fills or work pads during wet weather can consist of relatively clean granular material, such as sand, sand and gravel, or crushed rock with a maximum size of about 4 in. and with not more than about 7% passing the No. 200 sieve (washed analysis). The first lift of granular fill material placed over silt subgrade should be in the range of 12 to 18 in. thick (loose). Subsequent lifts should be placed 12 in. thick (loose). All lifts should be compacted to at least 95% of the maximum dry density as determined by ASTM D 698 using a medium-weight (48-in.-diameter drum), smooth, steel-wheeled, vibratory roller. Generally, a minimum of four passes with the roller are required to achieve compaction.

Backfill placed in utility trench excavations within the limits of the roadways, pavements, or structures should consist of sand, sand and gravel, or crushed rock with a maximum size of up to $1^1/_2$ in. and not more than 7% passing the No. 200 sieve (washed analysis). The granular backfill should be compacted to at least 95% of the maximum dry density as determined by ASTM D 698. Flooding or jetting the backfilled trenches with water to achieve the recommended compaction should not be permitted.

Utilities

In our opinion, there are three major considerations associated with design and construction of new utilities.

1) Provide stable excavation side slopes or support for trench sidewalls to minimize loss of ground.
2) Provide a safe working environment during construction.
3) Minimize post-construction settlement of the utilities and ground surface.

The method of excavation and design of trench support is the responsibility of the contractor and subject to applicable local, state, and federal safety regulation, including the current OSHA excavation and trench safety standards. The means, methods, and sequencing of construction operations and site safety are also the responsibility of the contractor. The information provided below is for the use of our client and should not be interpreted to mean that we are assuming responsibility for the contractor's actions or site safety.

According to current OSHA regulations, the majority of the sand, fine-grained soils, and gravelly materials encountered in the explorations may be classified as Type C. In our opinion, trenches less than 4 ft deep that do not encounter groundwater or sandy soils may be cut vertically and left unsupported during the normal construction sequence, i.e., assuming trenches are excavated and backfilled in the shortest possible sequence, and excavations are not allowed to remain open longer than 8 hours. Excavations more than 4 ft deep or through sandy soils should be laterally supported or alternatively provided with stable side slopes of 1H:1V or flatter. In our opinion, adequate lateral support may be provided by common methods, such as the use of a trench shield or hydraulic shoring systems.

Groundwater seepage, running soil conditions, and unstable trench sidewalls or soft trench subgrades, if encountered, will require dewatering of the excavation and trench sidewall support. The impact of these conditions can be minimized by completing trench excavation during the summer months when groundwater levels are lowest and by minimizing the depth of the trenches. All excavation sidewalls should be properly sloped or shored to conform to applicable local, state, or federal regulations. The design of dewatering systems is the responsibility of the contractor. However, we anticipate that groundwater inflow, if encountered, can be controlled by pumping from sumps.

Design Review and Construction Services

We welcome the opportunity to review and discuss construction plans and specifications for this project as they are being developed. In addition, GRI should be retained to review all geotechnical-related portions of the plans and specifications to evaluate whether they are in conformance with the recommendations provided in our report. Additionally, to observe compliance with the intent of our recommendations, design concepts, and the plans and specifications, we are of the opinion that all construction operations dealing with earthwork, ground improvement and pile installation should be observed by a GRI representative. Our construction-phase services will allow for timely design changes if site conditions are encountered that are different from those described in our report. If we do not have the opportunity to confirm our interpretations, assumptions, and analyses during construction, we cannot be responsible for the application of our recommendations to subsurface conditions that are different from those described in this report.

LIMITATIONS AND CONCLUDING REMARKS

This report has been prepared to aid the project team in the design of the project. The scope is limited to the specific project and location described herein, and our description of the project represents our understanding of the significant aspects of the project relevant to the design and construction of the earthwork and foundations. In the event that any changes in the design and location of the project elements as outlined in this report are planned, we should be given the opportunity to review the changes and to modify or reaffirm the conclusions and recommendations of this report in writing.

The conclusions and recommendations submitted in this report are based on the data obtained from the borings and probes made at the locations indicated on Figures 2 through 5 and from other sources of information discussed in this report. In the performance of subsurface investigations, specific information is obtained at specific locations at specific times. However, it is acknowledged that variations in soil conditions may exist between exploration locations. This report does not reflect any variations that may occur between these locations. The nature and extent of variation may not become evident until construction. If, during construction, subsurface conditions different from those encountered in the explorations are observed or encountered, we should be advised at once so that we can observe and review these conditions and reconsider our recommendations where necessary.

Submitted for GRI,

(12/20/13) *Expires 4/2014*

Dwight J. Hardin, PE	Matthew S. Shanahan, PE	Brian J. Bayne, PE
Principal	Associate	Project Engineer

This document has been submitted electronically.

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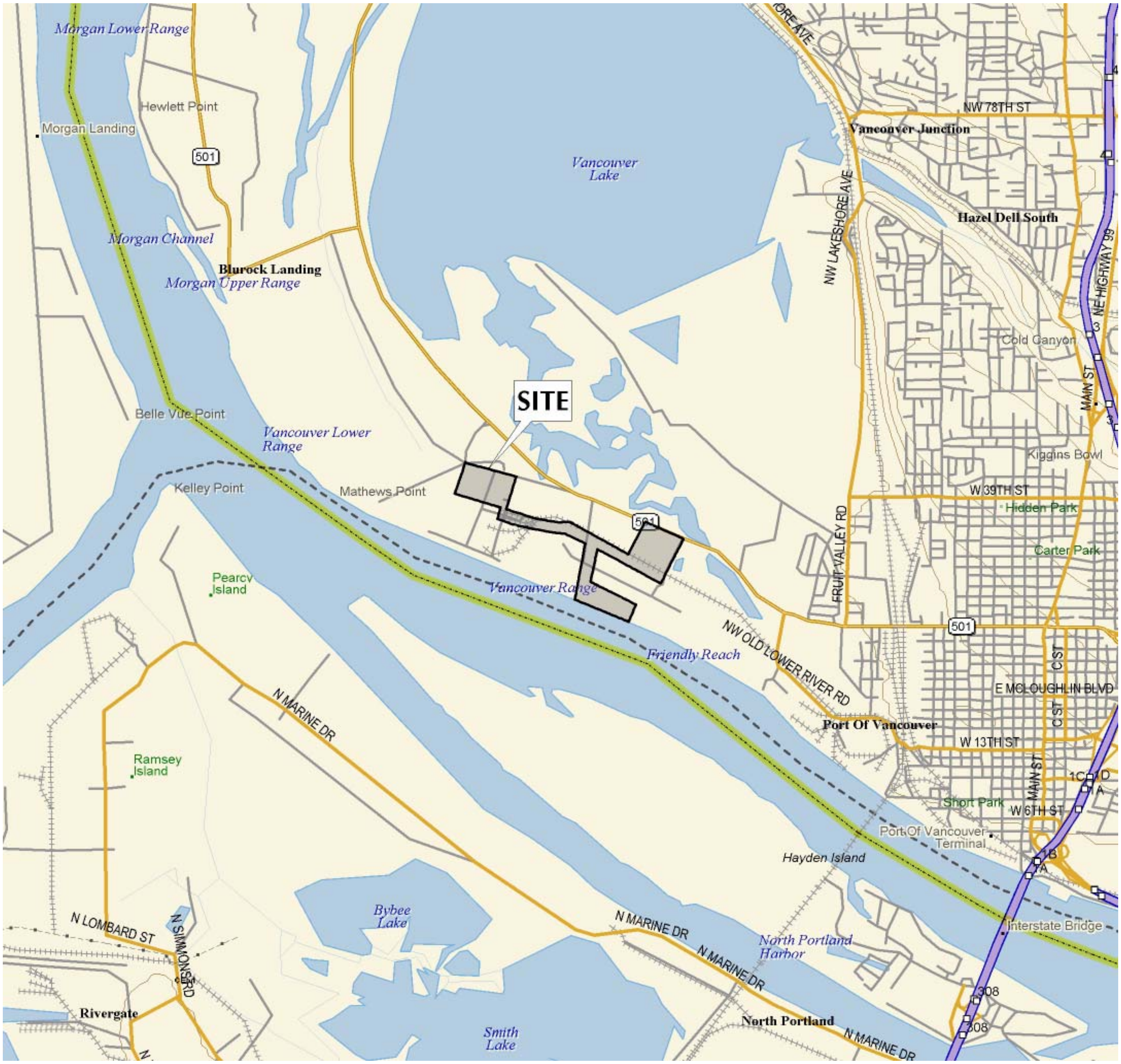

DELORME 3-D TOPOQUADS, WASHINGTON
VANCOUVER, WASH. (3cb, 3ca, & 3db) 2004

North

0 1 2 MILES

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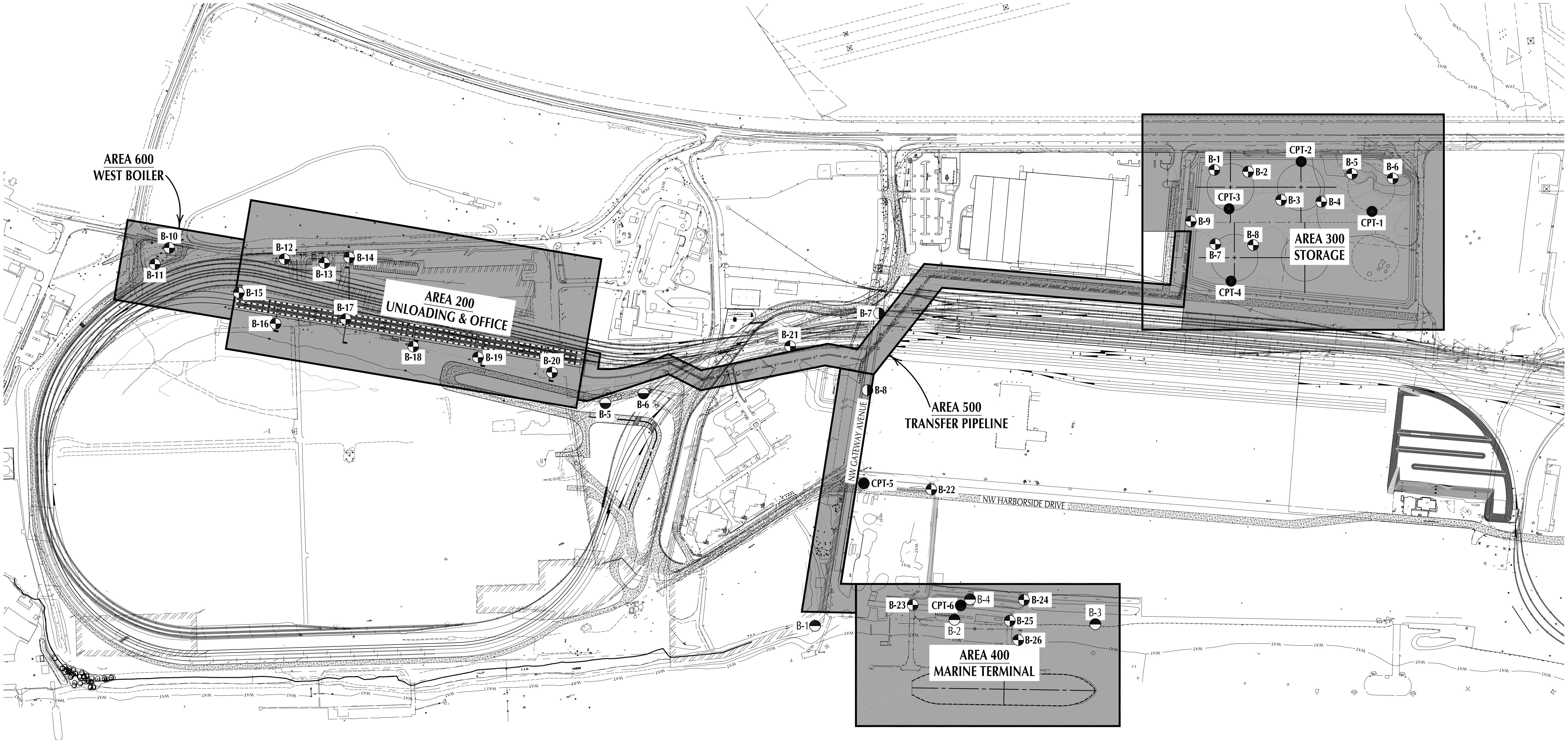
VICINITY MAP

DEC. 2013 JOB NO. W1114-T2 FIG. 1



BORING MADE BY GRI (JUNE 5 - OCTOBER 29, 2013)

CONE PENETRATION TEST MADE BY GRI (JUNE 6, - AUGUST 5, 2013)

BORING MADE BY GRI (JANUARY 18 - 19, 2011)

BORING MADE BY GRI (SEPTEMBER 6 - 11, 2006)

BORING MADE BY DAMES & MOORE (1993)

SITE PLAN FROM FILE BY MACKAY + SPOSITO

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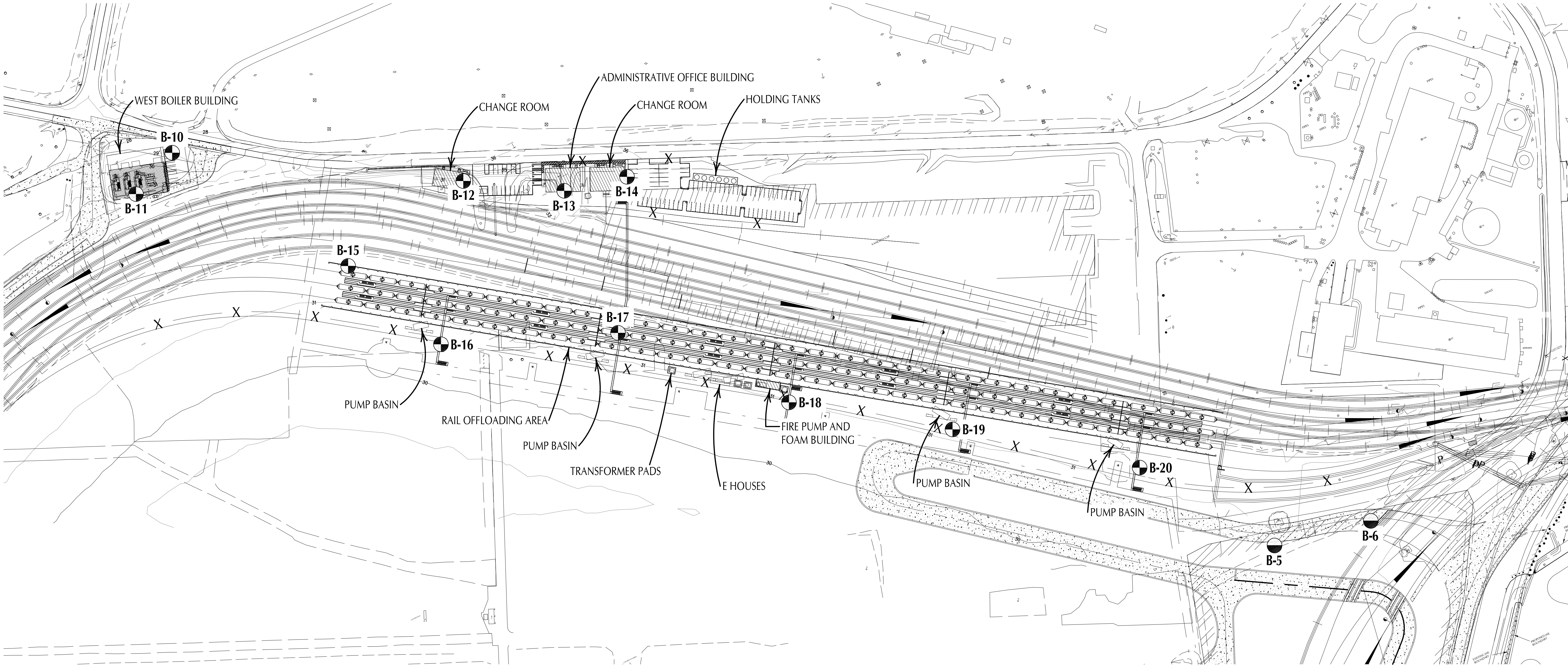
PROJECT LAYOUT PLAN

DEC. 2013 JOB NO. W1114-T2 FIG. 2



BORING MADE BY GRI
(JULY 1 - 9, 2013)

BORING MADE BY GRI
(JANUARY 18 - 19, 2011)

SITE PLAN FROM FILE BY MACKAY + SPOSITO

North

0 200 400 FT

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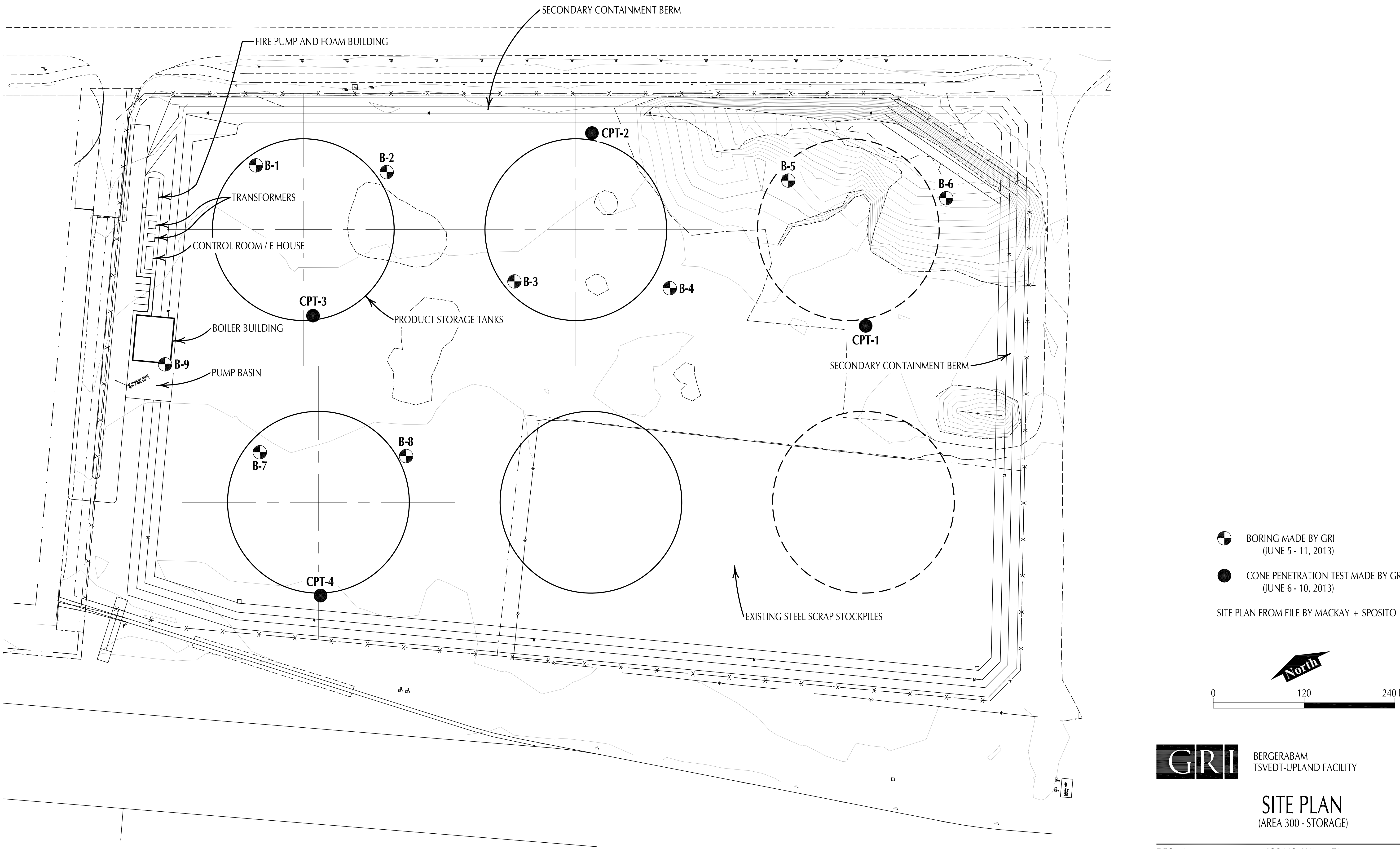
SITE PLAN

(AREAS 200 AND 600 - UNLOADING AND OFFICE / WEST BOILER)

DEC. 2013 JOB NO. W1114-T2 FIG. 3



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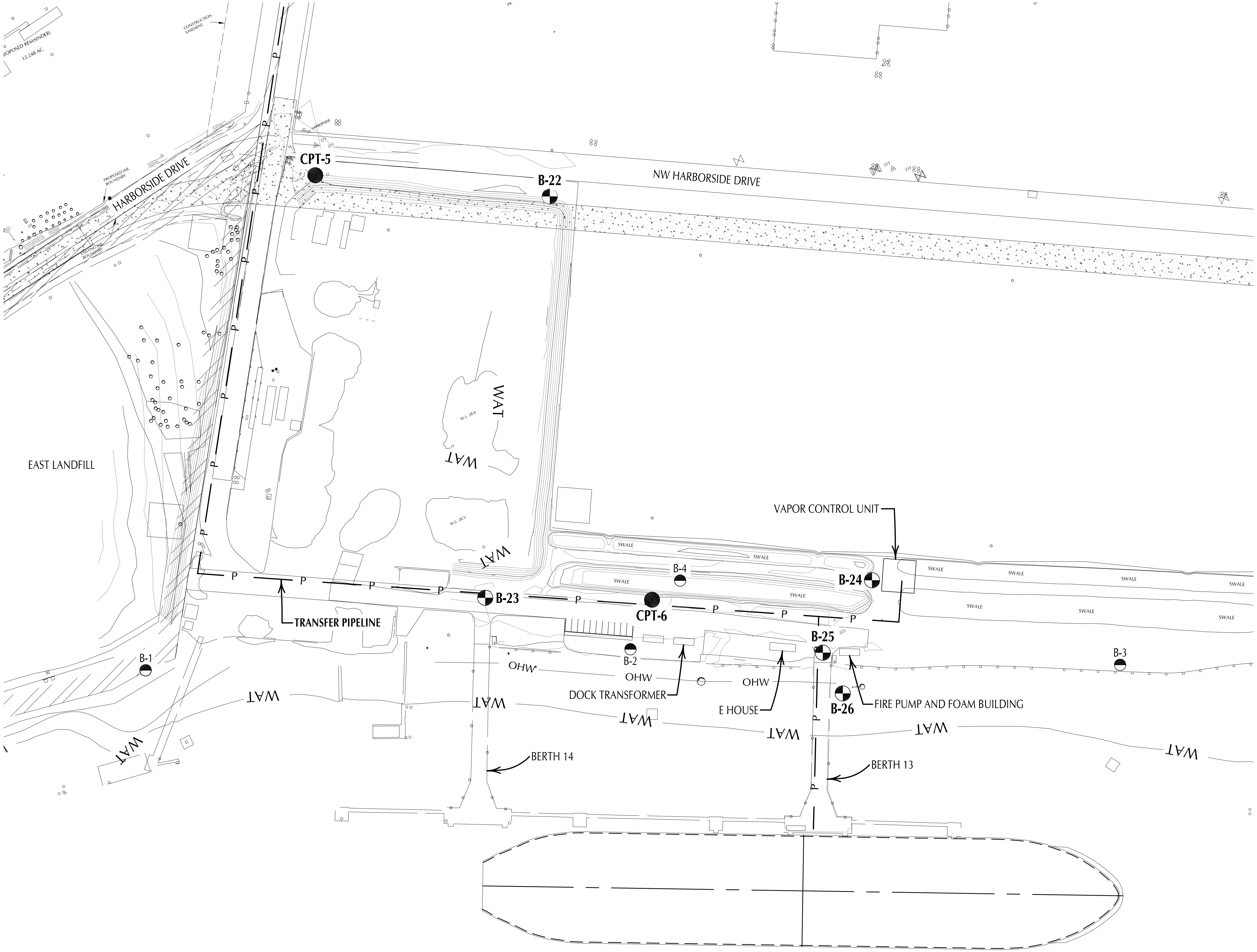
SITE PLAN

(AREA 300 - STORAGE)

DEC. 2013 | JOB NO. W1114-T2 | FIG. 4



BORING MADE BY GRI
(JULY 29 - OCTOBER 29, 2013)

CONE PENETRATION TEST MADE BY GRI
(JULY 29 - AUGUST 5, 2013)

BORING MADE BY DAMES & MOORE
(1993)

SITE PLAN FROM FILE BY MACKAY + SPOSITO

North

0 150 300 FT

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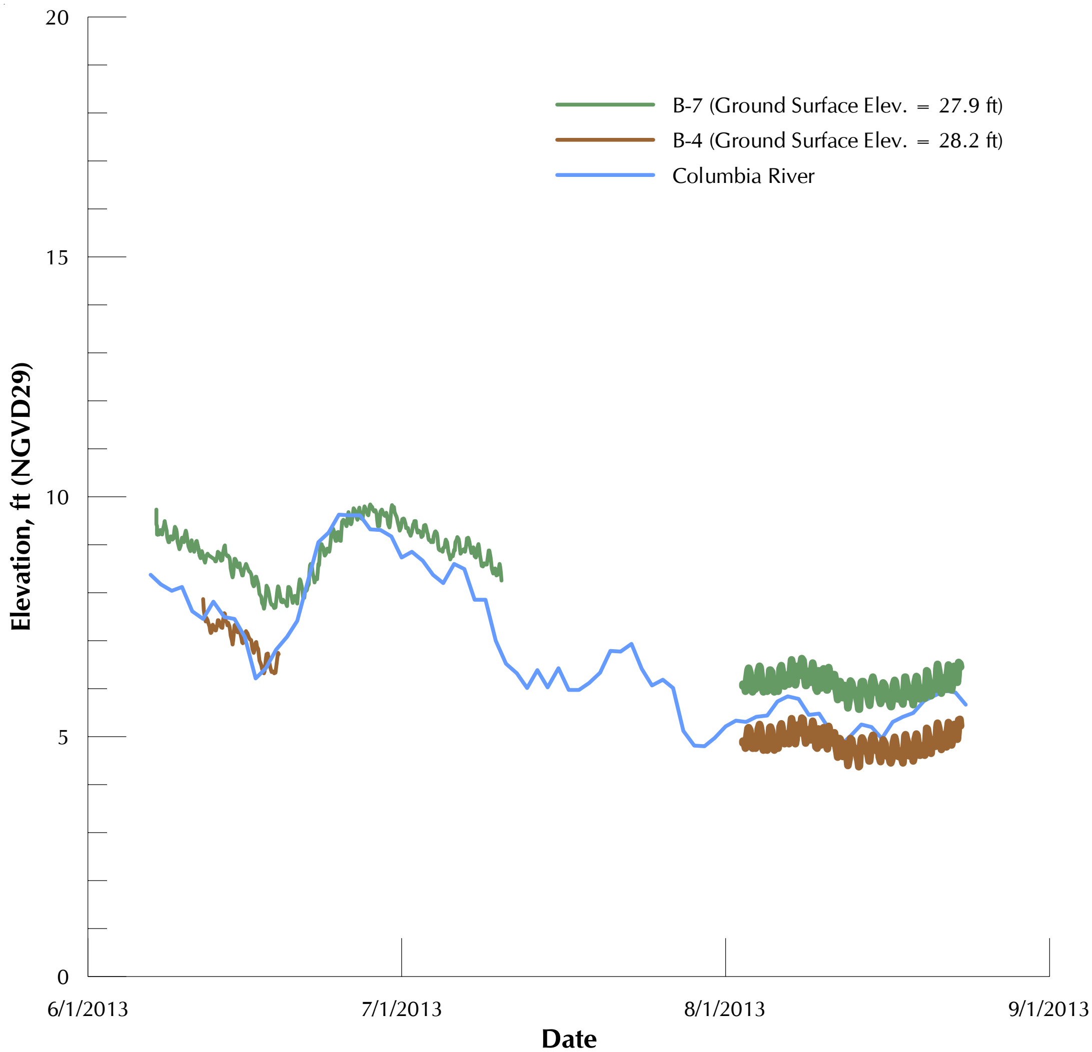
SITE PLAN
(AREA 400 - MARINE TERMINAL)

DEC. 2013 JOB NO. W1114-T2 FIG. 5



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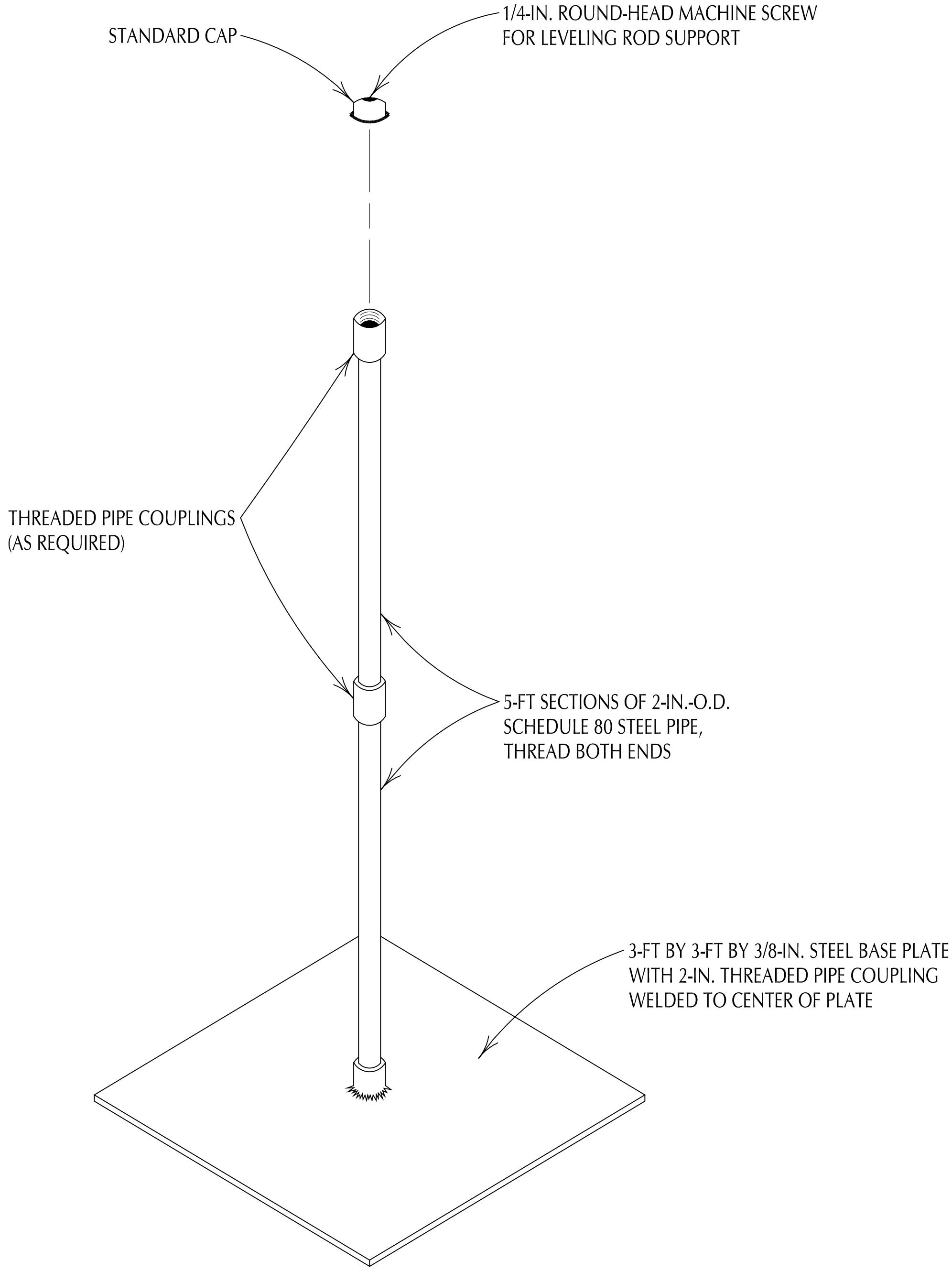
SUMMARY OF PIEZOMETERS

(AREA 300)

DEC. 2013 JOB NO. W1114-T2 FIG. 6



NOT TO SCALE

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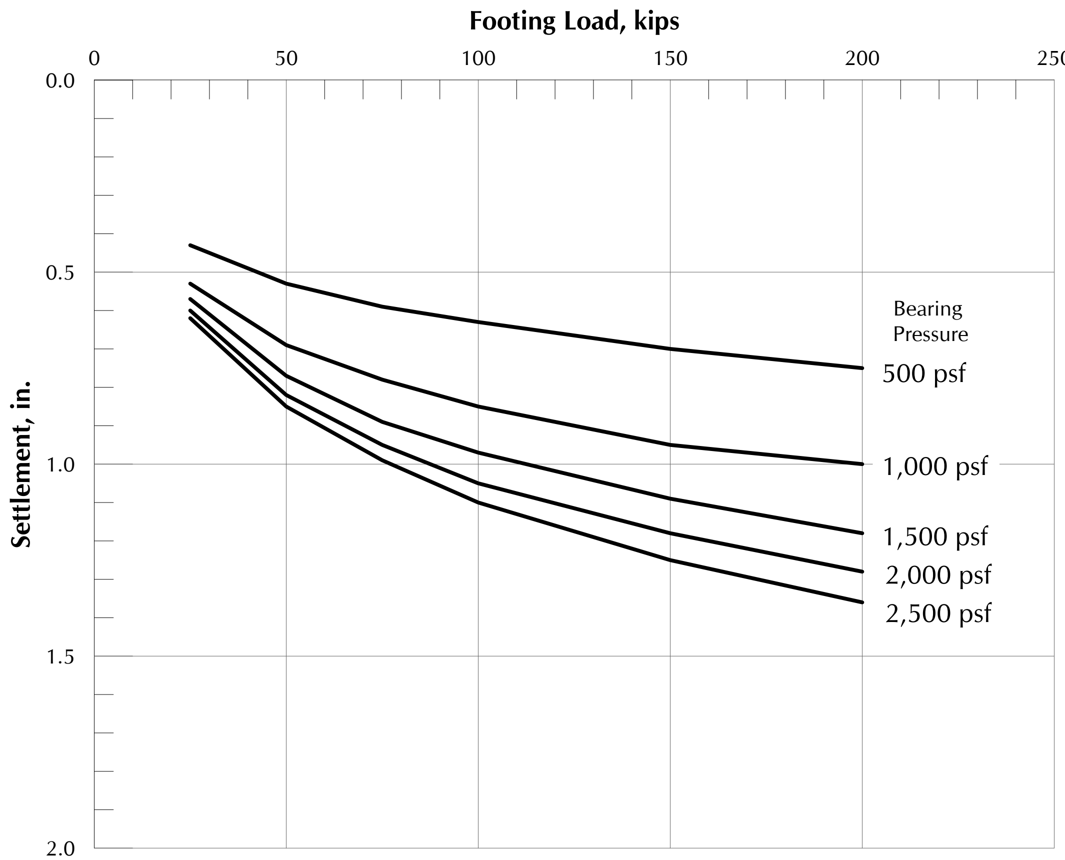
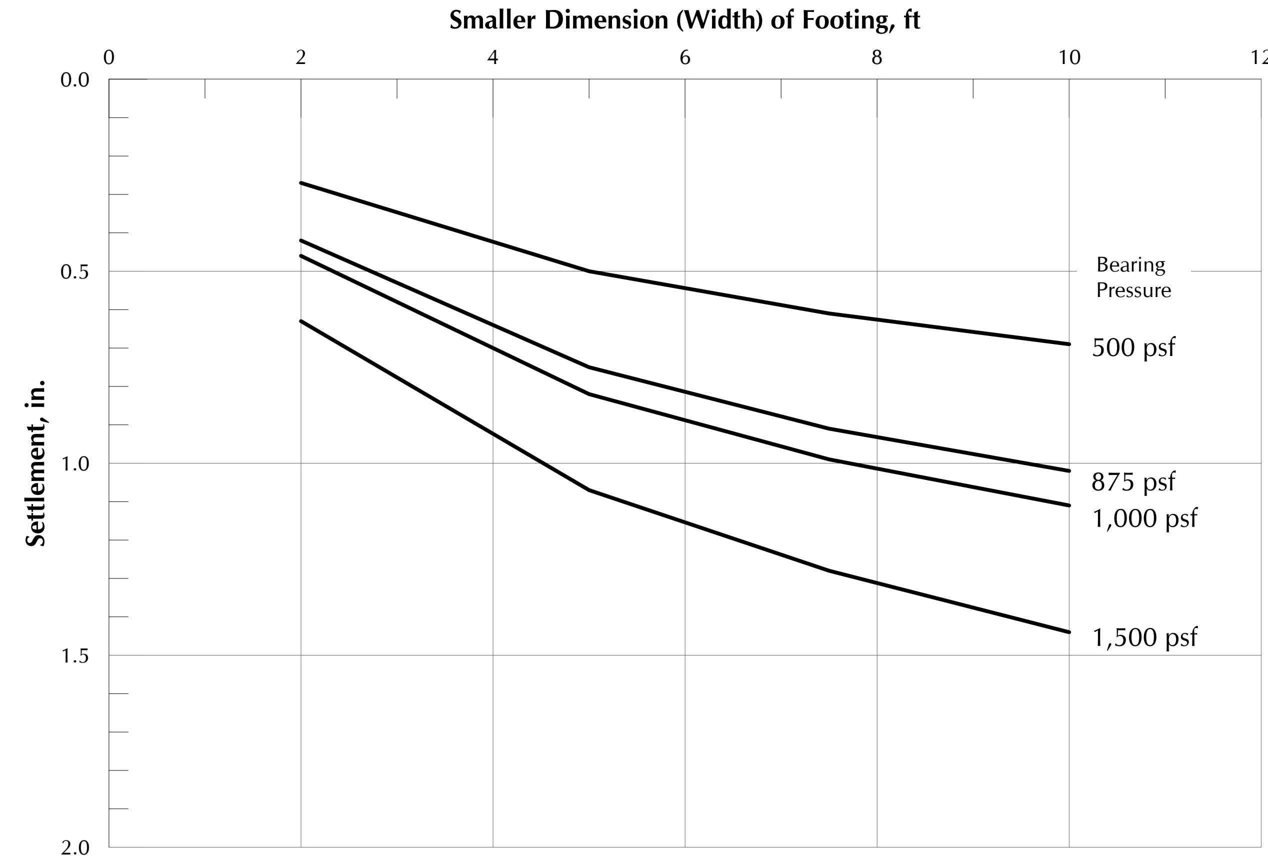
SETTLEMENT PLATE DETAIL

DEC. 2013 | JOB NO. W1114-T2 | FIG. 7

Square Footing Bearing Chart



Continuous/Rectangular Footing Bearing Chart



Settlement estimates are for footings established on silt subgrade. As noted in the text of this report, the majority of the footings will be founded in medium dense sand fill, and settlement estimates for footings underlain by at least 2 ft of sand can be reduced by 20%

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ESTIMATED FOOTING SETTLEMENT

APPENDIX A

Field Explorations, Instrumentation, and Laboratory Testing

APPENDIX A

FIELD EXPLORATIONS, INSTRUMENTATION, AND LABORATORY TESTING

FIELD EXPLORATIONS

General

Subsurface materials and conditions in the upland project area were investigated with 26 borings and six cone penetration test probes (CPTs). The approximate locations of the explorations are shown on Figures 2 through 5. An experienced geotechnical engineer from GRI directed the drilling and maintained a detailed log of the materials and conditions disclosed during the course of the work. The locations of the borings with respect to areas of the proposed facility are discussed below.

Borings

Disturbed and undisturbed samples were typically obtained from the borings at 2.5-ft intervals of depth in the upper 15 ft and at 5-ft intervals below this depth. Disturbed samples were obtained using a standard split-spoon sampler. At the time of sampling, the Standard Penetration Test was conducted. This test consists of driving a standard split-spoon sampler into the soil a distance of 18 in. using a 140-lb hammer dropped 30 in. The number of blows required to drive the sampler the last 12 in. is known as the standard penetration resistance, or N-value. The N-values provide a measure of the relative density of granular soils, such as sand or gravel, and the relative consistency, or stiffness, of cohesive soils, such as silt or clay. The split-spoon samples were carefully examined in the field and representative portions were saved in airtight jars. All samples were returned to our laboratory for further examination and physical testing.

Relatively undisturbed samples of fine-grained, cohesive soils were obtained by pushing 3-in.-O.D. Shelby tubes into the undisturbed soil a maximum distance of 24 in. using the drill rig. The soils exposed in the ends of the Shelby tubes were examined and classified in the field. After classification, the ends of the tubes were sealed with plastic end caps and tape to preserve the natural moisture content of the soils. All samples were returned to our laboratory for further examination and testing.

Logs of the borings are provided on Figures 1A through 26A. Each log presents a descriptive summary of the various types of materials encountered in the boring and notes the depth at which the materials and/or characteristics of the materials change. To the right of the descriptive summary, the depth to groundwater and the numbers and types of samples are indicated. Farther to the right, N-values are shown graphically, along with natural moisture contents and percent passing the No. 200 sieve. The terms used to describe the soils encountered in the borings are defined in Table 1A.

Details regarding the drilling in the various areas of the proposed facility are provided below.

Area 300. Borings in Area 300 were completed between June 5 and July 1, 2013, with nine borings, designated B-1 through B-9. The borings were advanced to depths of 50.9 to 82.0 ft with mud-rotary drilling methods using a truck-mounted CME drill rig provided and operated by Western States Soil Conservation of Hubbard, Oregon.

Areas 200 and 600. Borings in Areas 200 and 600 were completed between July 1 and 9, 2013, with 11 borings, designated B-10 through B-20. The borings were advanced to depths of 21.5 to 96.0 ft with mud-rotary drilling methods using a truck-mounted CME drill rig provided and operated by Western States Soil Conservation of Hubbard, Oregon.

Area 400. Borings in Area 400 were completed between July 29 and October 31, 2013, with four borings, designated B-23 through B-26. The borings were advanced to depths of 80 to 104.2 ft with mud-rotary drilling methods using a truck- or- track-mounted CME drill rig provided and operated by Cascade Drilling LP of Clackamas, Oregon.

Area 500. Borings in Area 500 were completed between July 31 and August 2, 2013, with two borings, designated B-21 and B-22. The borings were advanced to depths of 60.5 and 75.5 ft with mud-rotary drilling methods using a truck-mounted CME drill rig provided and operated by Cascade Drilling LP of Clackamas, Oregon.

Electric Cone Penetration Test (ECPT) Probes

Six CPT probes, designated CPT-1 through CPT-6, were advanced to practical refusal at depths of 54 to 84 ft below the ground surface using a truck-mounted Dutch Cone Unit provided and operated by Vandehey Exploration, Inc. of Banks, Oregon. Probes CPT-1 through CPT-4 were advanced to depths of about 54 to 56 ft in Area 300, CPT-5 was advanced to a depth of about 78 ft in Area 500, and CPT-6 was advanced to a depth of about 83 ft in Area 400.

The equipment is mounted on a truck and operated from within an enclosure on the back of the truck that houses the electrical equipment. The electrical cone probe has a cone and a sleeve that are similar to a mechanical probe, but the forces are measured electronically. In addition to the cone and sleeve transducers, a piezometer is fitted between the cone and the sleeve, which allows measurement porewater pressure and rate of dissipation as the probe is advanced. An accelerometer can also be fitted within the electrical probe. The accelerometer is used to measure the arrival times of shear waves produced at the ground surface as the exploration is advanced. Using these measurements, the shear wave velocity of the soils penetrated can be estimated. The shear wave velocities characterize the soils for the purpose of seismic studies. Shear wave measurements were made during advancement of probes CPT-1 and CPT-6. The terms used to describe the soils encountered in the CPT probes are defined in Table 2A. Logs of the CPT probes are provided on Figures 27A through 32A.

INSTRUMENTATION

Vibrating-Wire Piezometers

Geokon Model 4500 ALV low-pressure, vented vibrating-wire piezometers were installed in borings B-4 and B-7 at about elevation -22 and -17 ft, respectively. The piezometers are equipped with a Geokon Model 8002 (LC-2) single-channel data logger programmed to record data at 2-hr intervals. At the time of installation, the piezometers were saturated with water, taped to a 1-in.-O.D. PVC grout pipe in an inverted position to maintain saturation, and inserted into the open borehole to the desired depth. The borings were then filled with cement-bentonite grout to near the ground surface. The performance of each piezometer was verified before installation and immediately after insertion to design depth with a manual readout box. Each of the installations is equipped with a steel monument casing that was cement grouted into the borehole collar to protect the data logger and readout cables from vandalism and the elements. The data loggers are being downloaded periodically to evaluate the data. The piezometer data with the Columbia River hydrograph data are summarized graphically on Figure 6. The Columbia River hydrograph data are provided by the USGS station “14144700 Columbia River at Vancouver, WA” located about 3 miles upstream from the site.

LABORATORY TESTING

General

All samples obtained from the field were returned to our laboratory where the physical characteristics of the samples were noted, and the field classifications were modified where necessary. At the time of classification, the natural moisture content of each sample was measured. Additional testing included Torvane shear strength, Atterberg limits, washed sieve analysis, sieve analysis, dry unit weight determinations, and one-dimensional consolidation testing. The following sections describe the testing program in more detail.

Natural Moisture Content

Natural moisture content determinations were made in conformance with ASTM D 2216. The results are provided on Figures 1A through 26A.

Dry Unit Weight

The dry unit weight of 32 undisturbed samples was determined in the laboratory in accordance with ASTM D 2937 by cutting a cylindrical specimen of soil from a Shelby tube sample. The dimensions of the specimen were carefully measured, the volume calculated, and the specimen weighed. After oven-drying, the specimen was reweighed and the moisture content calculated. The dry unit weight was then computed. The dry unit weights are summarized below.

SUMMARY OF DRY UNIT WEIGHT DETERMINATIONS

Boring	Sample	Depth, ft	Natural Moisture Content, %	Dry Unit Weight, pcf	Soil Type
B-1	S-8	21.5	32	90	SILT; some clay, trace fine-grained sand, scattered gravel
B-2	S-8	21.5	34	80	SILT; trace to some clay and fine-grained sand
	S-11	31.5	26	91	Sandy SILT; fine grained sand, trace to some clay, trace organics
B-4	S-7	16.5	31	67	SILT; some clay, trace fine-grained sand and organics
	S-10	27.5	28	92	SAND; fine to medium grained, some silt
B-5	S-6	14.7	36	84	SILT: some clay, trace fine-grained sand
B-6	S-2	5	40	76	SILT; trace to some clay, trace fine-grained sand
	S-5	11.5	30	91	SAND; fine to medium grained, trace to some silt, trace subrounded gravel
B-7	S-12	31.5	21	87	Silty SAND; fine-grained
B-8	S-9	23.5	30	69	SILT; trace to some clay, trace fine-grained sand
B-9	S-7	20	38	82	SILT; some fine-grained sand, trace organics
B-11	S-4	10	28	89	Silty SAND; fine grained
B-12	S-6	15	33	88	Silty SAND; fine grained
	S-9	21.5	35	85	Sandy SILT: fine-grained sand
B-14	S-8	21.5	32	87	SILT; trace clay, fine-grained sand, and organics
B-15	S-4	10	30	84	Silty SAND; fine grained
	S-8	23	37	82	SILT; some fine-grained sand

Boring	Sample	Depth, ft	Natural Moisture Content, %	Dry Unit Weight, pcf	Soil Type
B-16	S-7	20	22	96	Silty SAND; fine grained
	S-11	33	28	88	Sandy SILT; fine-grained sand
B-17	S-7	18	24	92	SILT; some fine-grained sand
B-18	S-5	12.5	34	86	Silty SAND; fine grained
	S-8	23	21	95	SILT; trace fine-grained sand
B-19	S-5	12.5	18	102	Sandy SILT; fine-grained sand
	S-8	21.5	26	76	Sandy SILT; fine-grained sand
B-20	S-4	10	14	89	FILL: SAND; fine to medium grained, trace to some silt
	S-7	20	33	86	Silty SAND; fine grained
B-21	S-7	20	42	78	SILT; trace to some clay and fine grained sand
B-22	S-6	15	36	85	SILT; some sand
B-24	S-18	70	29	88	SAND; fine grained, trace silt
B-26	S-12	40	39	80	SAND; some silt, scattered wood debris
	S-14	45	34	81	SAND; some silt,
	S-16	50	31	90	SAND; some silt

Torvane Shear Strength

The approximate undrained shear strength of relatively undisturbed fine-grained soil samples was determined using a Torvane shear device. The Torvane is a hand-held apparatus with vanes that are inserted into the soil. The torque required to fail the soil in shear around the vanes is measured using a calibrated spring. The results of the Torvane shear tests are summarized on Figures 1A through 22A.

Atterberg Limits

Atterberg limits determinations were completed on nine representative soil samples in substantial conformance with ASTM D 4318. The test data are summarized on Figures 33A and 34A.

One-Dimensional Consolidation

Consolidation testing was performed in accordance with ASTM D 2435 to obtain data on the compressibility characteristics of six samples of relatively undisturbed fine-grained soil. Test results are summarized on Figures 35A through 40A in the form of a curve showing effective stress versus percent strain. The initial and final moisture content and unit weight of the sample are provided at the top of the figure.

Secondary compression was recorded in substantial conformance to ASTM D 2434 Test Method B during the one-dimensional consolidation tests. Compression was recorded at select compressive loads between 1 and 2 tsf for a minimum of 1,200 minutes following application of a compressive load increment. The results are summarized on Figures 41A and 42A in the form of curves showing dial reading versus the log of time.

Grain Size Analysis

Washed-Sieve Method. Washed sieve analyses were performed on representative soil samples to assist in their classification. The test is performed by taking a sample of known dry weight and washing it over a

No. 200 sieve. The material retained on the sieve is oven-dried and weighed, and the percentage of material passing the No. 200 sieve is calculated. The test results are shown on the Boring Logs, Figures 1A through 26A.

Dry Sieve Method. Sieve analyses were performed on five representative samples of sand in substantial conformance with ASTM D 6913. The test is performed by taking a sample of known dry weight and washing it over a No. 200 sieve. The material retained on the sieve is oven-dried and weighed, and the percentage of material passing the No. 200 sieve is calculated. The soil retained on the No. 200 sieve is then screened through a series of sieves of various sizes using a sieve shaker. The weight of each sieve is measured prior to and after the soil has been run through the shaker. The weight of the soil retained on each sieve is recorded and expressed as a percentage of the total sample weight. The test data are summarized on Figures 43A and 44A in the form of curves showing the percent of the total soil sample by weight finer versus sieve number or grain size in millimeters.

Table 1A

GUIDELINES FOR CLASSIFICATION OF SOIL

Description of Relative Density for Granular Soil

Relative Density	Standard Penetration Resistance (N-values) blows per foot
very loose	0 – 4
loose	4 – 10
medium dense	10 – 30
dense	30 – 50
very dense	over 50

Description of Consistency for Fine-Grained (Cohesive) Soils

Consistency	Standard Penetration Resistance (N-values) blows per foot	Torvane Undrained Shear Strength, tsf
very soft	2	less than 0.125
soft	2 - 4	0.125 - 0.25
medium stiff	4 - 8	0.25 - 0.50
stiff	8 - 15	0.50 - 1.0
very stiff	15 - 30	1.0 - 2.0
hard	over 30	over 2.0

Sandy silt materials which exhibit general properties of granular soils are given relative density description.

Grain-Size Classification

Boulders
12 - 36 in.

Cobbles
3 - 12 in.

Gravel
1/4 - 3/4 in. (fine)
3/4 - 3 in. (coarse)

Sand
No. 200 - No. 40 sieve (fine)
No. 40 - No. 10 sieve (medium)
No. 10 - No. 4 sieve (coarse)

Silt/Clay - pass No. 200 sieve

Modifier for Subclassification

Adjective	Percentage of Other Material In Total Sample
clean	0 - 2
trace	2 - 10
some	10 - 30
sandy, silty, clayey, etc.	30 - 50

GRI

Table 2A

SOIL CLASSIFICATION
BASED ON CONE PENETRATION TEST

Friction Ratio (Percent)	Soil Classification
0 to 2	Clean sand or slightly silty sand
2 to 5	Silty sand, clayey sand, or silt
> 5	Clayey silt, silty clay, or clay

COHESIVE SOILS

Sleeve Friction, tsf	Relative Consistency
< 0.12	Very Soft
0.12 to 0.25	Soft
0.25 to 0.50	Medium Stiff
0.50 to 1.00	Stiff
1.00 to 2.00	Very Stiff
> 2.00	Hard

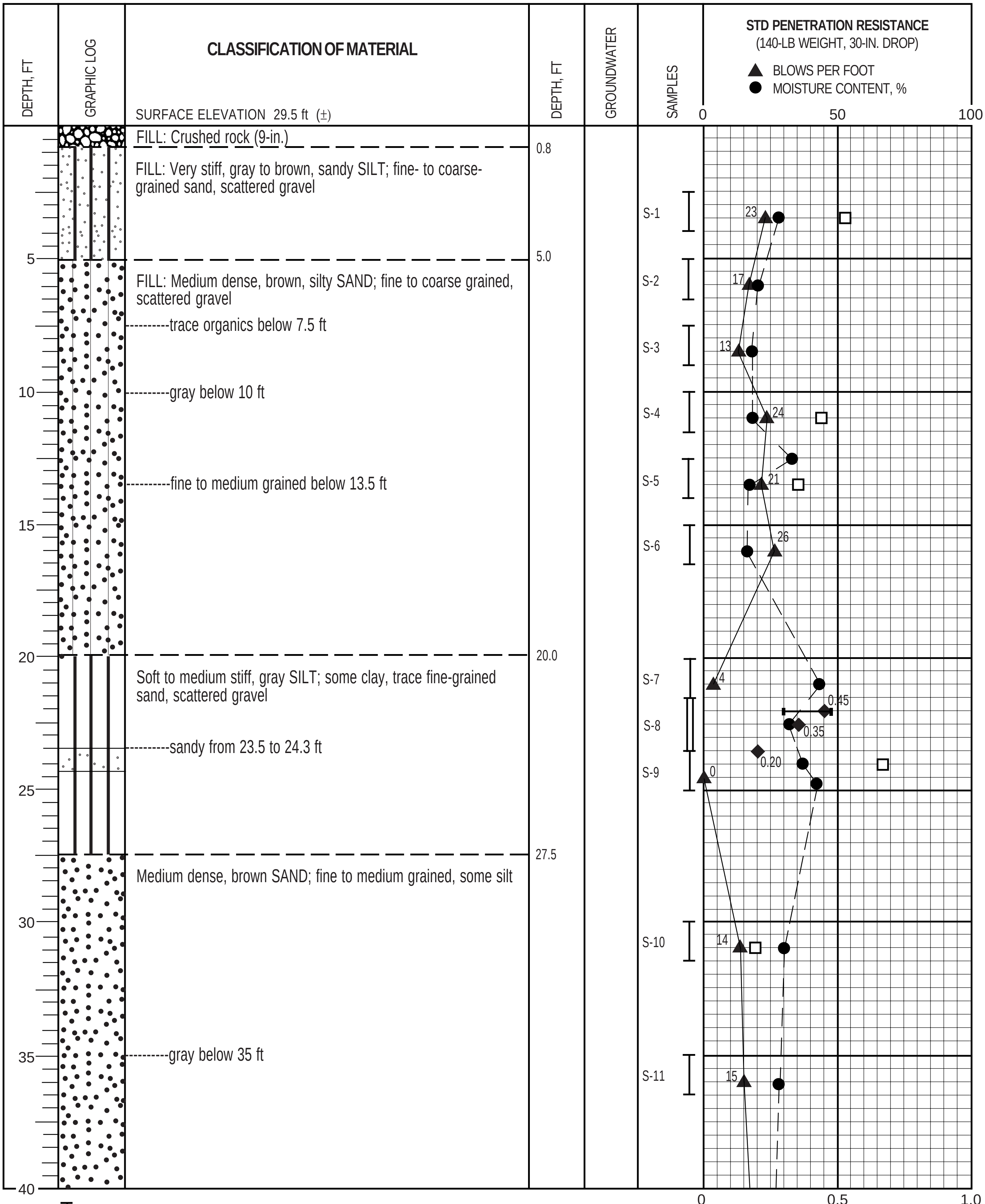
COHESIONLESS SOILS

	Soil Type*			
	ML, SM	SM, SP, SW	SP, SW, GW	SW, GP
Relative Density	Cone Penetration Resistance, tsf			
Very Loose	0 - 8	0 - 14	0 - 20	0 - 24
Loose	8 - 20	14 - 35	20 - 50	24 - 60
Med. Dense	20 - 60	35 - 105	50 - 150	60 - 180
Dense	60 - 100	105 - 175	150 - 250	180 - 300
Very Dense	> 100	> 175	> 250	> 300

* Unified Soil Classification System

1) Friction ratio is equal to sleeve friction (tsf) divided by cone penetration (tsf) expressed as a percent.

GRI



2-IN.-OD SPLIT-SPOON SAMPLER

3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

P VIBRATING-WIRE PIEZOMETER

SLOTTED PVC PIPE

Water Level (date)

TORVANE SHEAR STRENGTH, TSF

PERCENT PASSING NO. 200 SIEVE (WASHED)

* NO RECOVERY

Liquid Limit

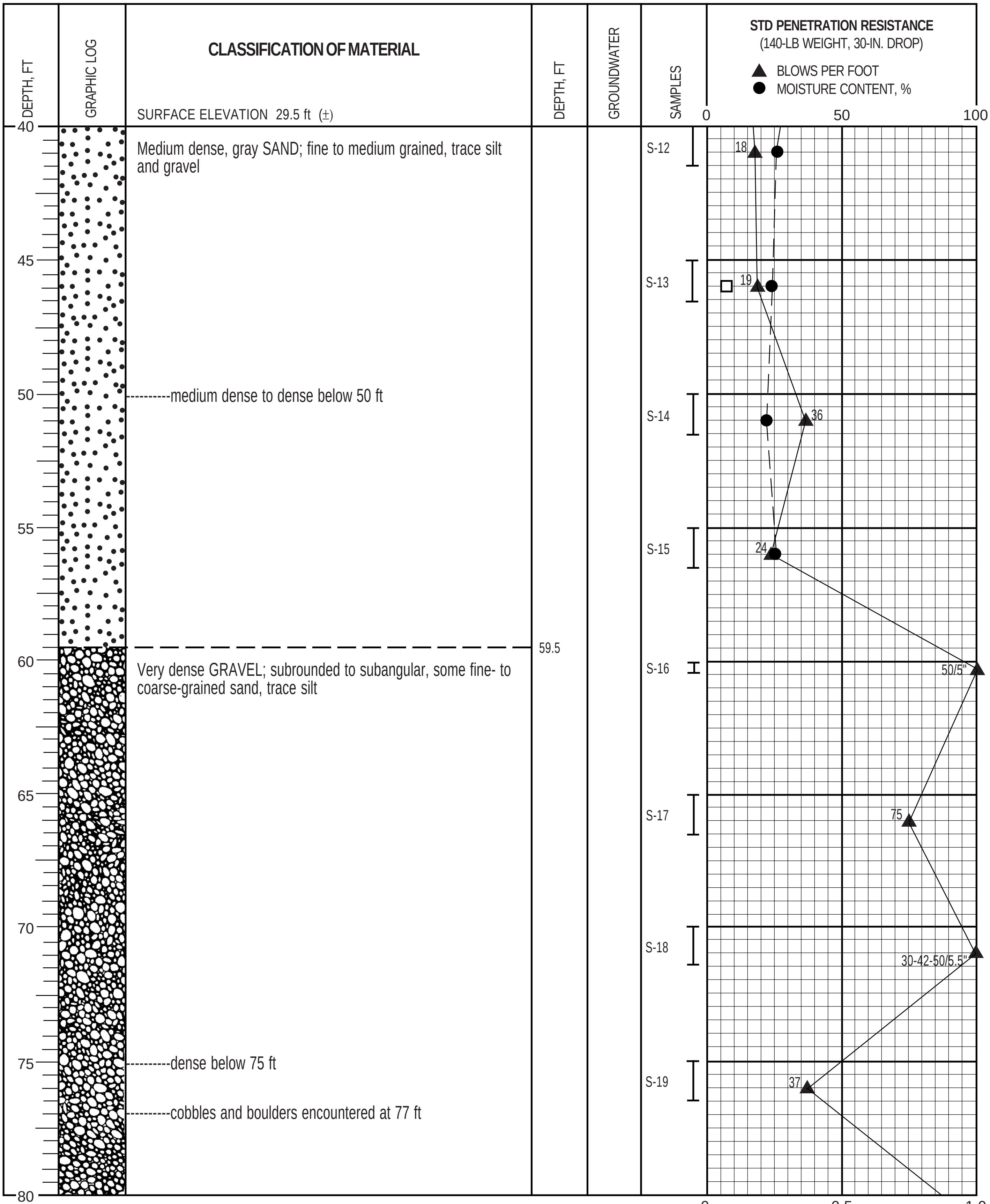
Moisture Content

Plastic Limit

GRI

BORING B-1

DEC. 2013 | JOB. NO. W1114 | FIG. 1A



2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
P VIBRATING-WIRE PIEZOMETER
SLOTTED PVC PIPE
Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF
□ PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-1 (cont.)

DEC. 2013 | JOB. NO. W1114 | FIG. 1A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL SURFACE ELEVATION 29.5 ft (±)	DEPTH, FT	GROUNDWATER	SAMPLES	STD PENETRATION RESISTANCE (140-LB WEIGHT, 30-IN. DROP) ▲ BLOWS PER FOOT ● MOISTURE CONTENT, % 0 50 100
80		Very dense GRAVEL; subrounded to subangular, some fine- to coarse-grained sand, trace silt	82.0		S-20 *	111
		(6/6/2013) Sidewall of boring caved to 35 ft while drilling at 70 ft; Circulation of drilling fluid lost at 74 ft (40 gallons)				
85						

0 0.5 1.0

(TONS PER FT ²)

2-IN.-OD SPLIT-SPOON SAMPLER

3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

P VIBRATING-WIRE PIEZOMETER

SLOTTED PVC PIPE

Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF

□ PERCENT PASSING NO. 200 SIEVE (WASHED)

* NO RECOVERY

Liquid Limit

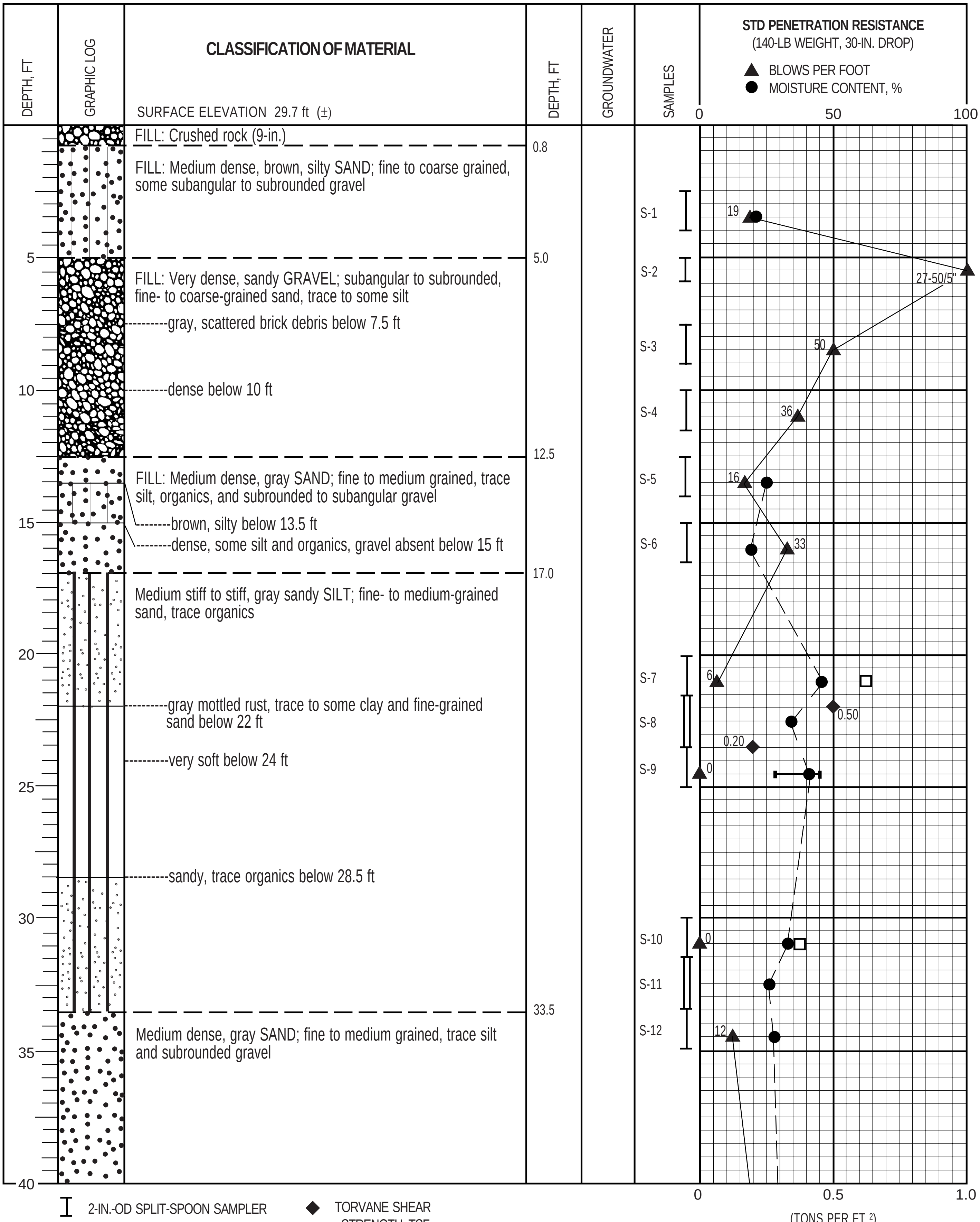
Moisture Content

Plastic Limit

GRI

BORING B-1 (cont.)

DEC. 2013 JOB. NO. W1114 FIG. 1A



2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
P VIBRATING-WIRE PIEZOMETER
SLOTTED PVC PIPE
Water Level (date)

TORVANE SHEAR STRENGTH, TSF
PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

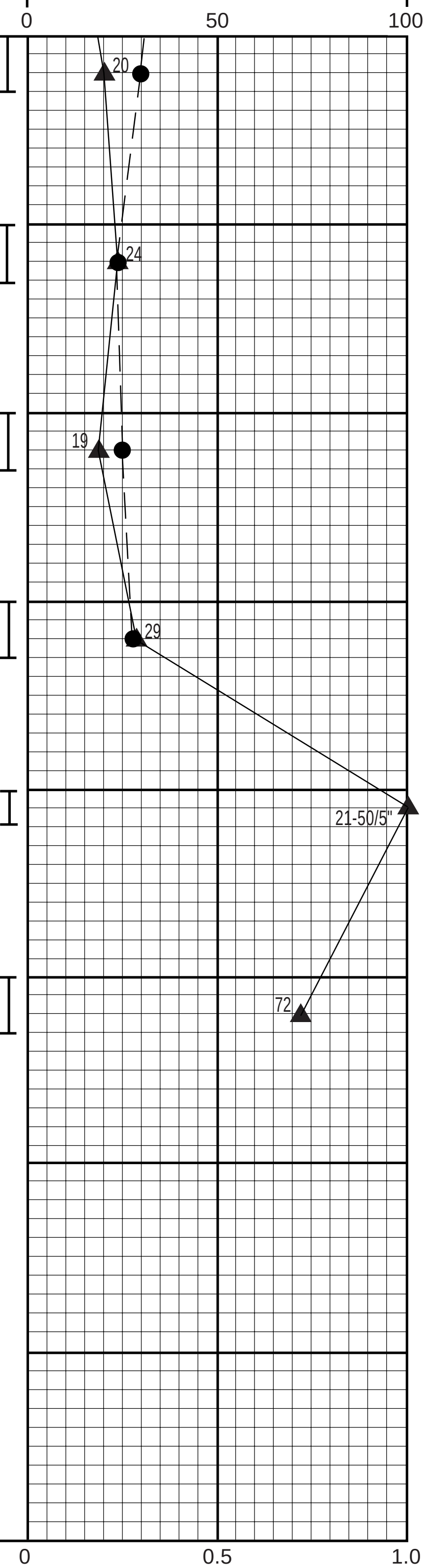
GRI

BORING B-2

DEC. 2013 | JOB. NO. W1114 | FIG. 2A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER	SAMPLES
		SURFACE ELEVATION 29.7 ft (±)			
40		Medium dense, gray SAND; fine to medium grained, trace silt			S-13
45		---------fine to coarse grained below 45 ft			S-14
50		---------fine to medium grained below 50 ft			S-15
55		---------trace subrounded gravel below 55 ft			S-16
		Very dense GRAVEL; subangular to subrounded, some fine- to coarse-grained sand, trace silt	58.5		
60					S-17 *
65					S-18
		(6/7/2013)	66.5		
70					

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)
▲ BLOWS PER FOOT
● MOISTURE CONTENT, %



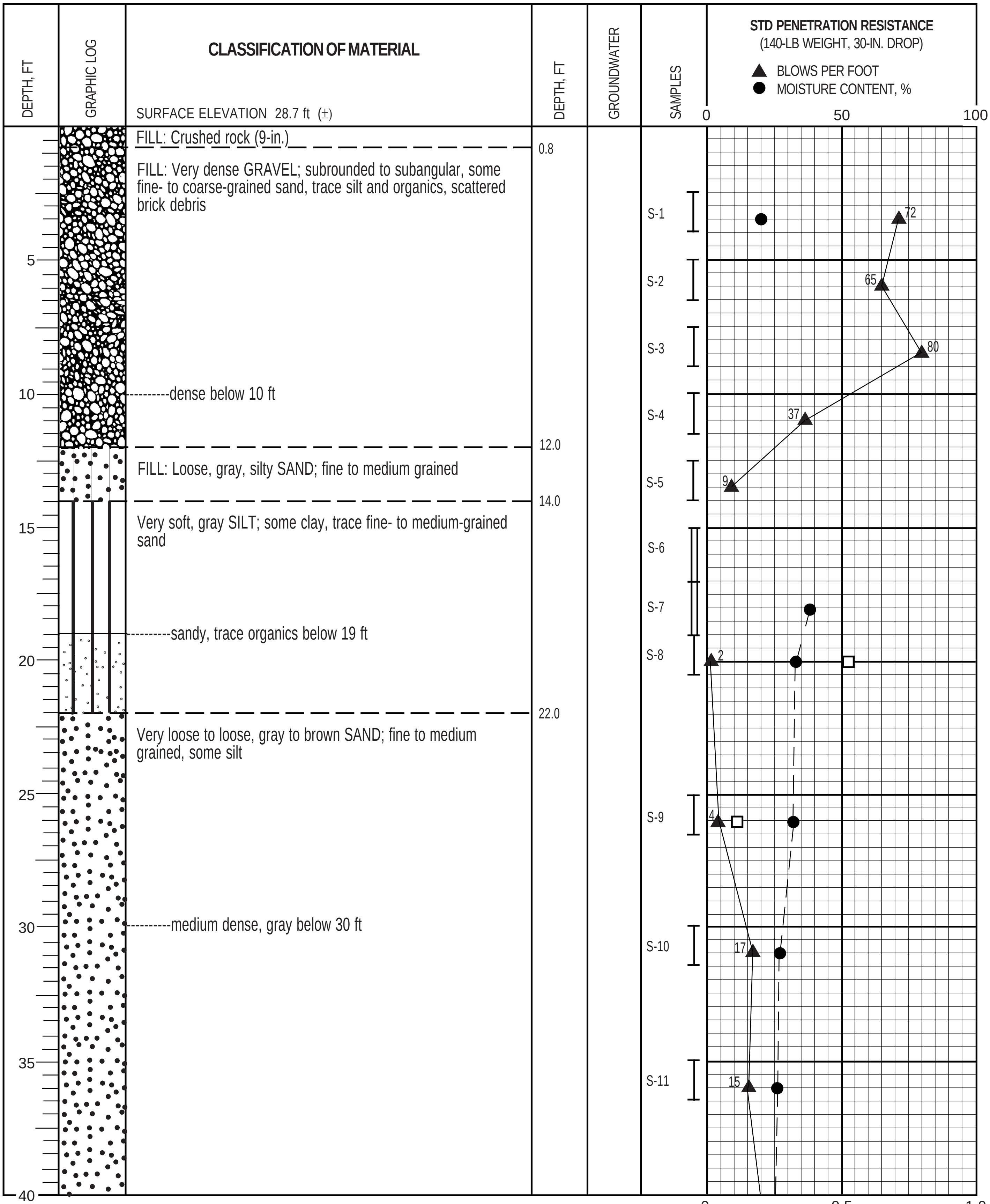
I 2-IN.-OD SPLIT-SPOON SAMPLER
II 3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
P VIBRATING-WIRE PIEZOMETER
SLOTTED PVC PIPE
Water Level (date)
◆ TORVANE SHEAR STRENGTH, TSF
□ PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-2 (cont.)

DEC. 2013 JOB. NO. W1114 FIG. 2A

SURFACE ELEVATION ----- ft (±)



2-IN.-OD SPLIT-SPOON SAMPLER

3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

P VIBRATING-WIRE PIEZOMETER

SLOTTED PVC PIPE

Water Level (date)

TORVANE SHEAR STRENGTH, TSF

PERCENT PASSING NO. 200 SIEVE (WASHED)

* NO RECOVERY

Liquid Limit

Moisture Content

Plastic Limit

GRI

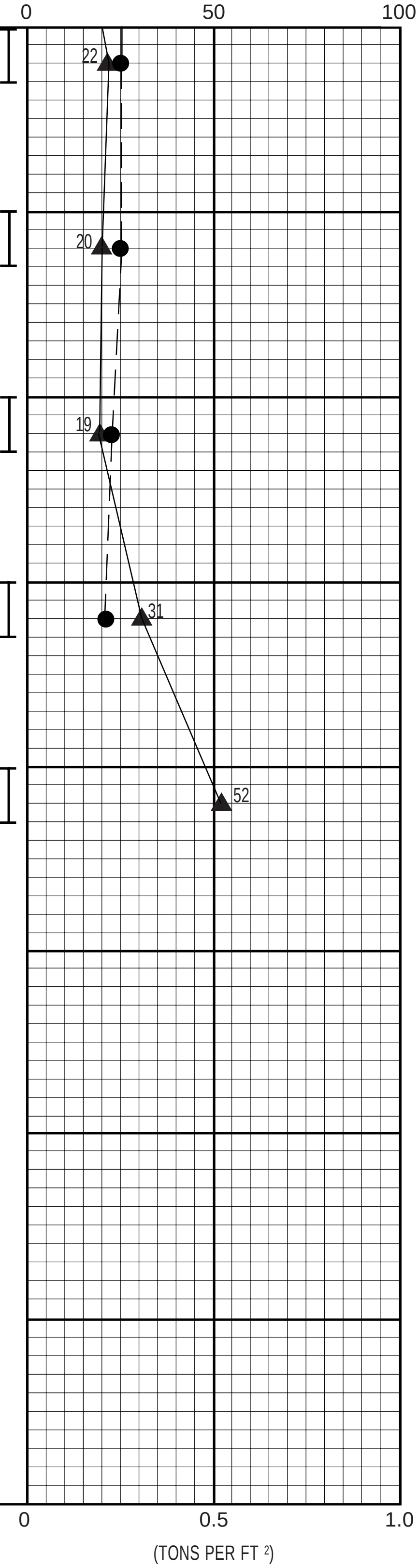
BORING B-3

DEC. 2013 | JOB. NO. W1114 | FIG. 3A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER	SAMPLES
		SURFACE ELEVATION 28.7 ft (±)			
40		Medium dense, gray SAND; fine to medium grained, trace silt			S-12
45					S-13
50					S-14
55		---------scattered subrounded gravel below 55 ft			S-15
		Very dense GRAVEL; subangular to subrounded, some fine- to coarse-grained sand, trace silt	57.0		
60					S-16
		(6/12/2013)	61.5		
65					
70					
75					
80					

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)

▲ BLOWS PER FOOT
● MOISTURE CONTENT, %



I 2-IN.-OD SPLIT-SPOON SAMPLER
II 3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
P VIBRATING-WIRE PIEZOMETER
SLOTTED PVC PIPE
▼ Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF
□ PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

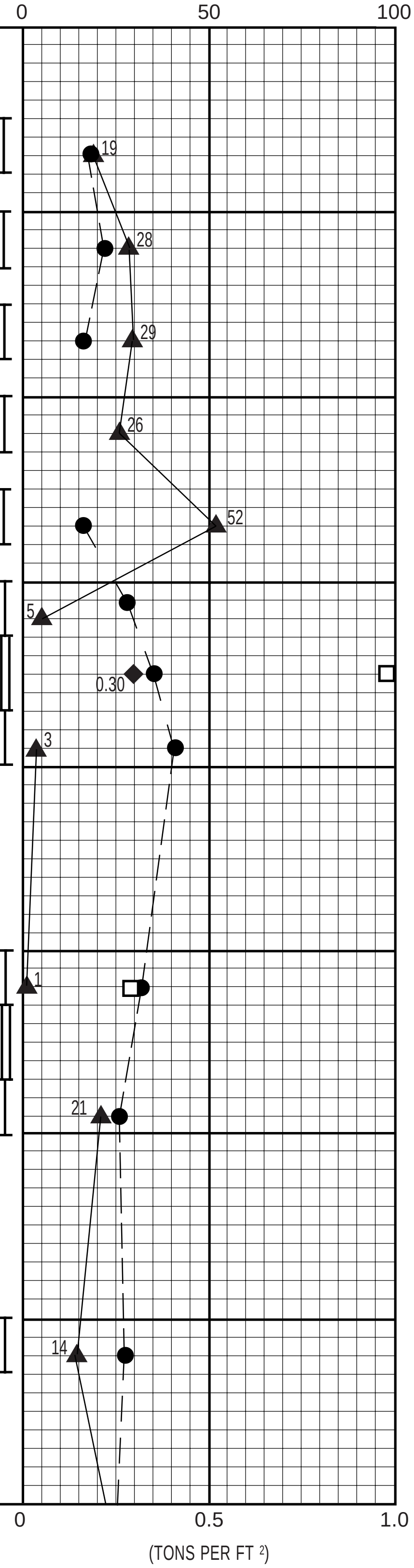
GRI

BORING B-3 (cont.)

DEC. 2013 JOB. NO. W1114 FIG. 3A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL SURFACE ELEVATION 28.2 ft (±)	DEPTH, FT	GROUNDWATER	SAMPLES
0–15.5		FILL: Medium dense, brown, silty SAND; fine to coarse grained, some subrounded to subangular gravel, scattered concrete and brick debris ---------very dense, gray, fine to medium grained, trace silt below 12.5 ft ---------loose, silty, trace organics below 15 ft	15.5		S-1, S-2, S-3, S-4 *, S-5, S-6
15.5–17.5		FILL: Medium stiff, gray, sandy SILT; fine-grained sand, trace organics	17.5		S-7
17.5–25.0		Medium stiff, gray SILT; some clay, trace fine-grained sand and organics --------soft below 19 ft	25.0	(6/12/13)	S-8
25.0–40		Very loose, brown SAND; fine to medium grained, some silt ---------medium dense, gray, trace silt, scattered gravel below 28.5 ft			S-9, S-10, S-11, S-12

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)
▲ BLOWS PER FOOT
● MOISTURE CONTENT, %



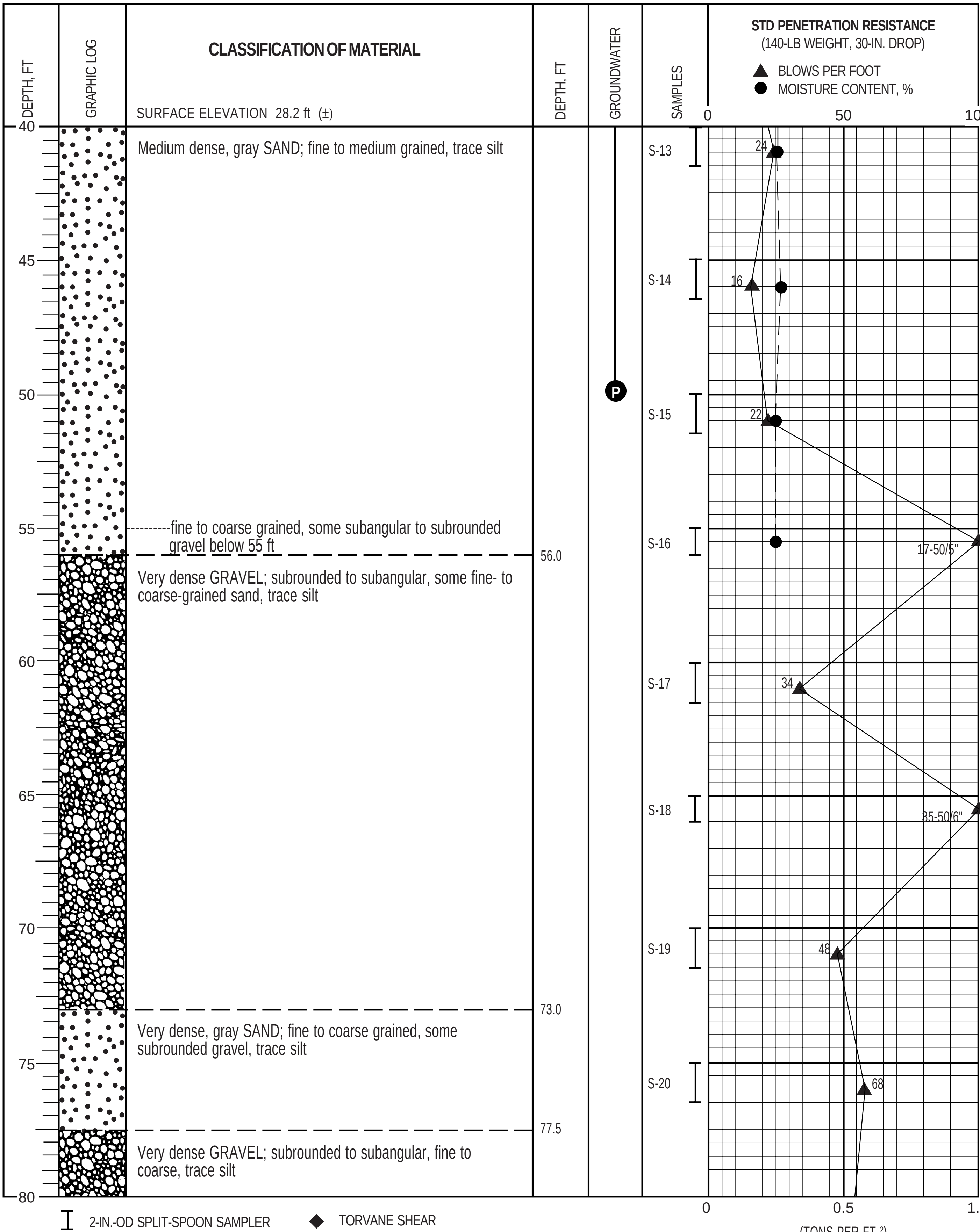
2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
P VIBRATING-WIRE PIEZOMETER
SLOTTED PVC PIPE
▼ Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF
□ PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-4

DEC. 2013 JOB. NO. W1114 FIG. 4A



2-IN.-OD SPLIT-SPOON SAMPLER

3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

P VIBRATING-WIRE PIEZOMETER

SLOTTED PVC PIPE

Water Level (date)

TORVANE SHEAR STRENGTH, TSF

PERCENT PASSING NO. 200 SIEVE (WASHED)

* NO RECOVERY

Liquid Limit

Moisture Content

Plastic Limit

GRI

BORING B-4 (cont.)

DEC. 2013 JOB. NO. W1114 FIG. 4A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL SURFACE ELEVATION 28.2 ft (±)	DEPTH, FT	GROUNDWATER	SAMPLES	STD PENETRATION RESISTANCE (140-LB WEIGHT, 30-IN. DROP) ▲ BLOWS PER FOOT ● MOISTURE CONTENT, % 0 50 100
80		Very dense GRAVEL; subrounded to subangular, some fine- to coarse-grained sand, trace silt	81.5		S-21	64
		(6/10/2013) Circulation of drilling fluid lost at 57 ft (40 gallons) and 62 ft (40 gallons)				
85						

0 0.5 1.0

(TONS PER FT²)

I 2-IN.-OD SPLIT-SPOON SAMPLER

II 3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

P VIBRATING-WIRE PIEZOMETER

SLOTTED PVC PIPE

▼ Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF

□ PERCENT PASSING NO. 200 SIEVE (WASHED)

* NO RECOVERY

Liquid Limit

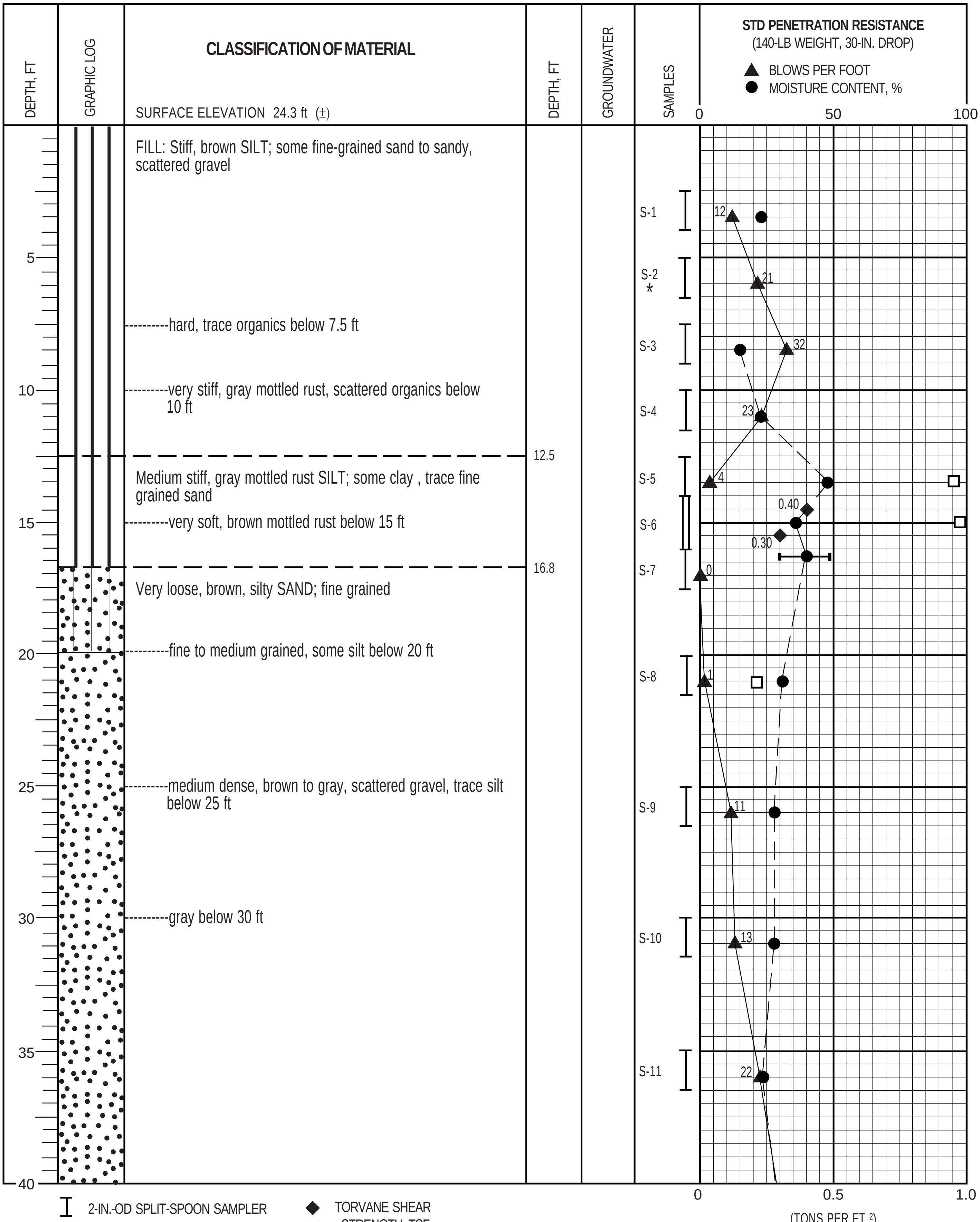
Moisture Content

Plastic Limit

GRI

BORING B-4 (cont.)

DEC. 2013 JOB. NO. W1114 FIG. 4A



2-IN.-OD SPLIT-SPOON SAMPLER

3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

P VIBRATING-WIRE PIEZOMETER

SLOTTED PVC PIPE

Water Level (date)

TORVANE SHEAR STRENGTH, TSF

PERCENT PASSING NO. 200 SIEVE (WASHED)

* NO RECOVERY

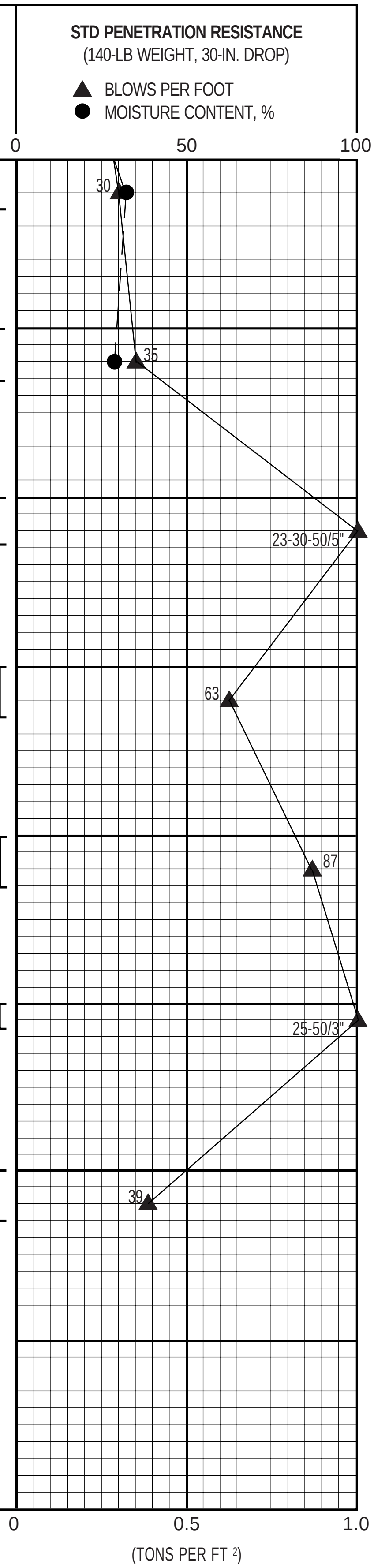
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-5

DEC. 2013 | JOB. NO. W1114 | FIG. 5A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER	SAMPLES
		SURFACE ELEVATION 24.3 ft (±)			
40		Medium dense to dense, gray SAND; fine to medium grained, trace silt			S-12
45					S-13
50		Very dense GRAVEL; subrounded to subangular, some silt and fine- to coarse-grained sand	50.0		S-14
55		---------trace silt below 55 ft			S-15
60					S-16
65					S-17
70		---------dense below 70 ft			S-18
		(6/5/2013)	71.5		
		Circulation of drilling fluid lost at 55 ft			
75					



2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
P VIBRATING-WIRE PIEZOMETER
SLOTTED PVC PIPE
Water Level (date)

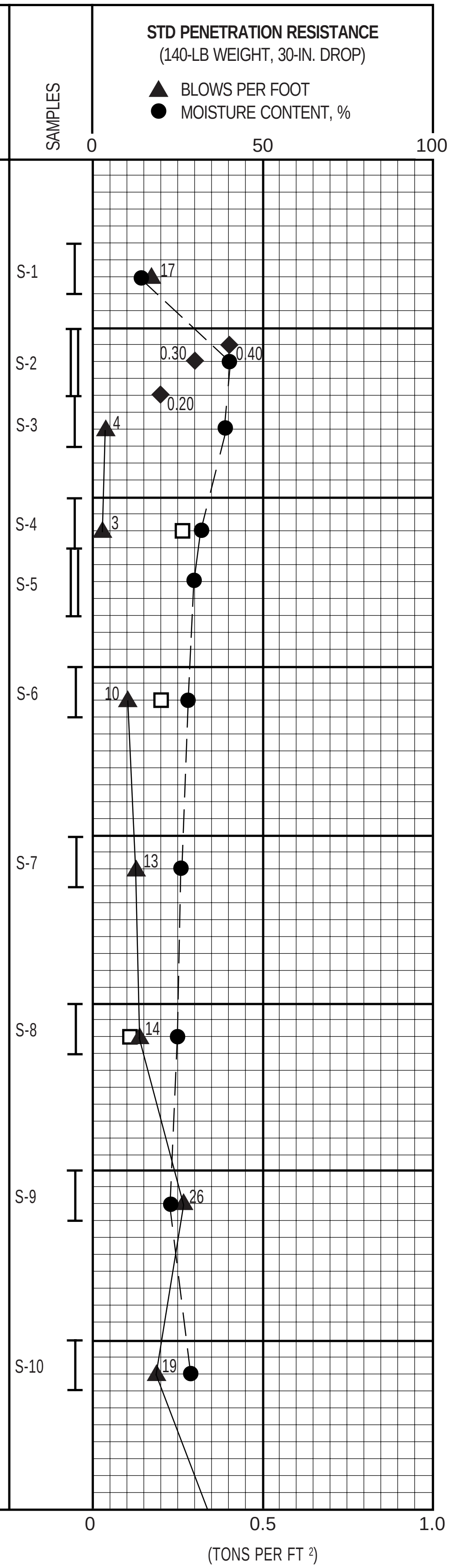
TORVANE SHEAR STRENGTH, TSF
PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-5 (cont.)

DEC. 2013 | JOB. NO. W1114 | FIG. 5A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER	SAMPLES
		SURFACE ELEVATION 15.3 ft (±)			
0–5.3		FILL: Medium dense, gray and brown, gravelly SAND; fine to coarse grained, subangular to angular gravel, some silt			S-1
5.3–10.0		Soft to medium stiff, gray mottled rust and dark gray SILT; trace to some clay, trace fine-grained sand	5.3		S-2, S-3
10.0–40		Very loose, brown SAND; fine to medium grained, trace to some silt, trace subrounded gravel	10.0		S-4, S-5, S-6, S-7, S-8, S-9, S-10
13		---------medium dense, gray, fine grained, trace to some silt below 13 ft			
20		---------trace subrounded gravel below 20 ft			
30		---------fine to coarse grained below 30 ft			
35		---------fine grained below 35 ft			



2-IN.-OD SPLIT-SPOON SAMPLER

3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

P VIBRATING-WIRE PIEZOMETER

SLOTTED PVC PIPE

Water Level (date)

TORVANE SHEAR STRENGTH, TSF

PERCENT PASSING NO. 200 SIEVE (WASHED)

* NO RECOVERY

Liquid Limit

Moisture Content

Plastic Limit

GRI

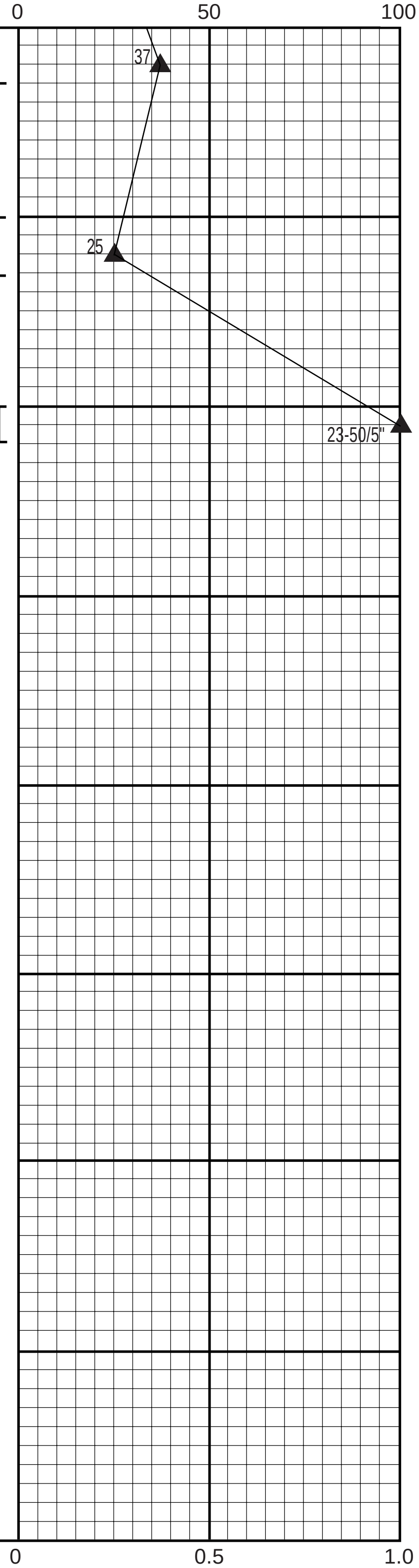
BORING B-6

DEC. 2013 | JOB. NO. W1114 | FIG. 6A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL SURFACE ELEVATION 15.3 ft (±)	DEPTH, FT	GROUNDWATER	SAMPLES
40		Dense GRAVEL; subrounded to subangular, some fine- to coarse-grained sand to sandy, trace silt			S-11
45		4- to 6-in. interbedded sand lenses from 45 to 48 ft			S-12
50			50.9		S-13
		(6/6/2013) Circulation of drilling fluid lost from 44.5 to 46 ft			
55					

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)

▲ BLOWS PER FOOT
● MOISTURE CONTENT, %



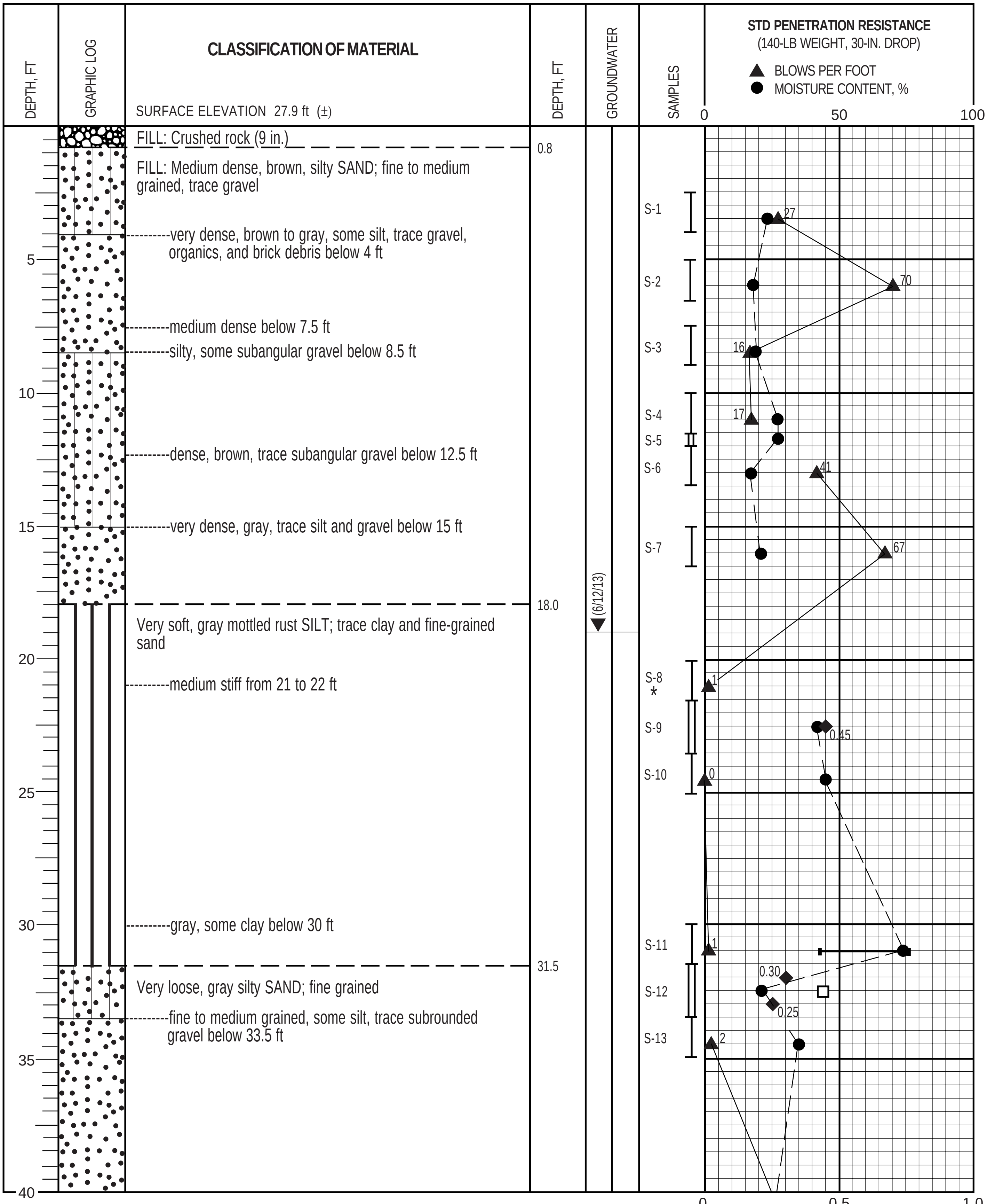
2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
P VIBRATING-WIRE PIEZOMETER
SLOTTED PVC PIPE
Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF
□ PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-6 (cont.)

DEC. 2013 JOB. NO. W1114 FIG. 6A



2-IN.-OD SPLIT-SPOON SAMPLER

3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

P VIBRATING-WIRE PIEZOMETER

SLOTTED PVC PIPE

Water Level (date)

TORVANE SHEAR STRENGTH, TSF

PERCENT PASSING NO. 200 SIEVE (WASHED)

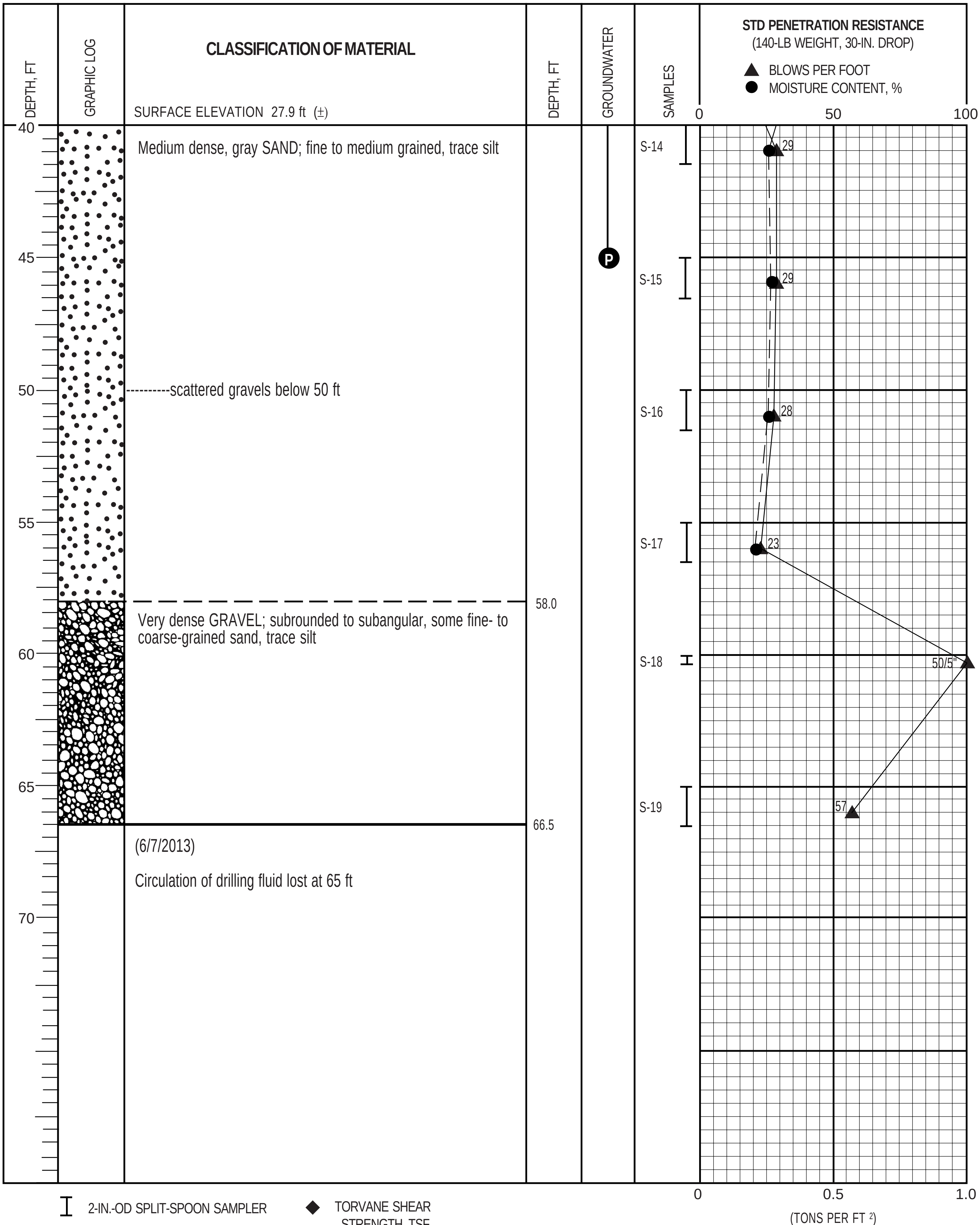
* NO RECOVERY

Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-7

DEC. 2013 JOB. NO. W1114 FIG. 7A



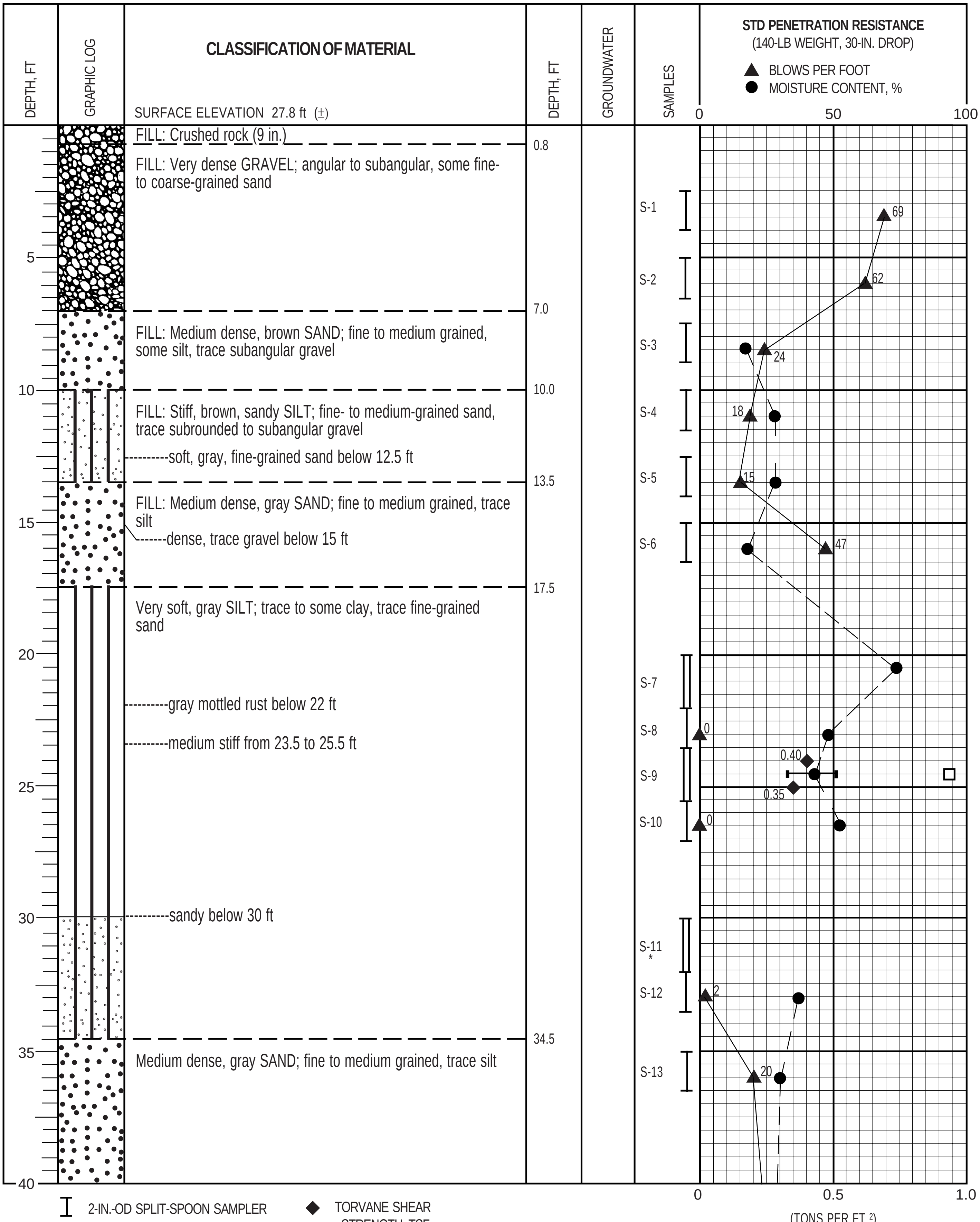
2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
P VIBRATING-WIRE PIEZOMETER
SLOTTED PVC PIPE
Water Level (date)

TORVANE SHEAR STRENGTH, TSF
PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-7 (cont.)

DEC. 2013 | JOB. NO. W1114 | FIG. 7A



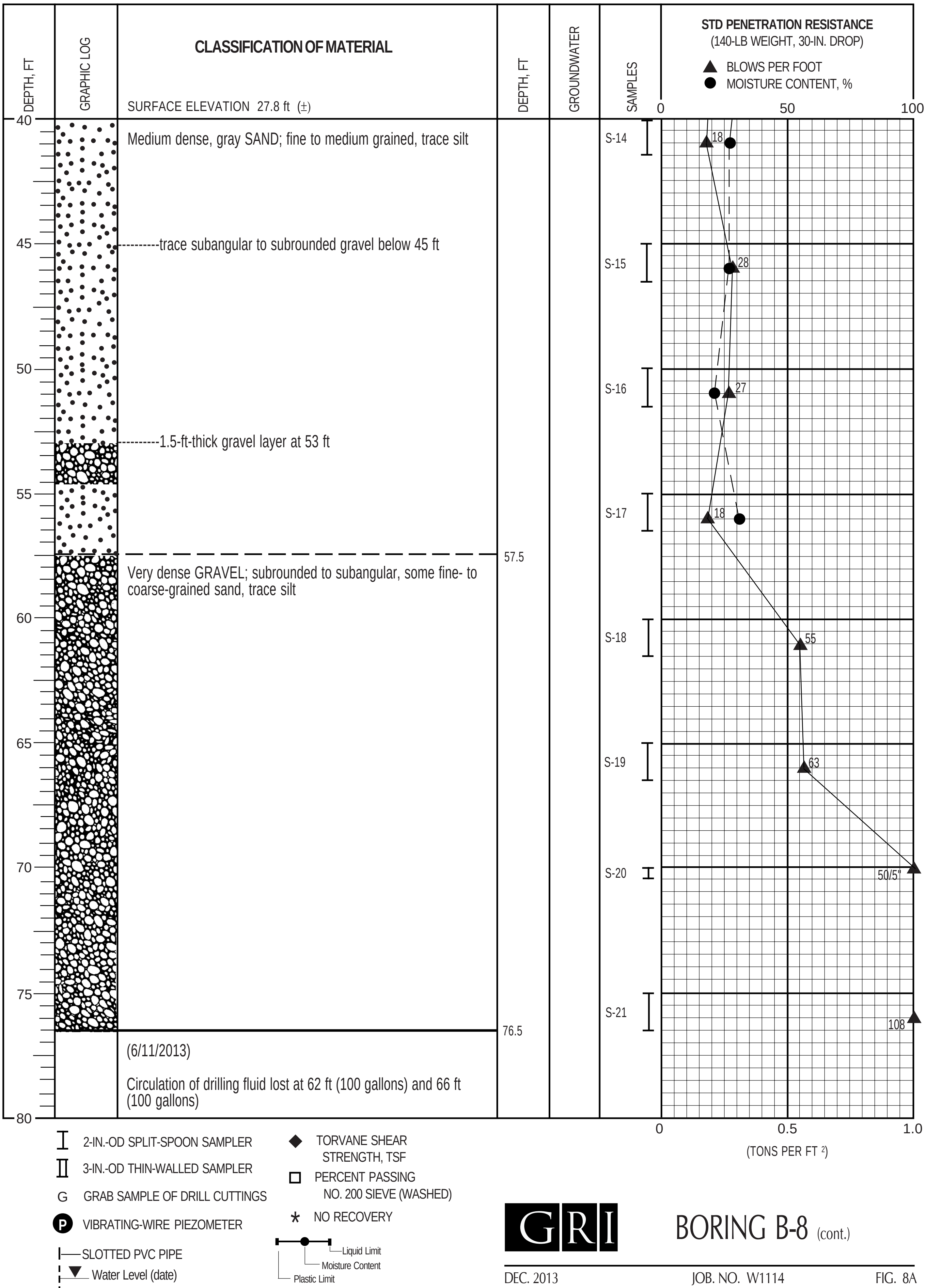
2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
P VIBRATING-WIRE PIEZOMETER
SLOTTED PVC PIPE
Water Level (date)

TORVANE SHEAR STRENGTH, TSF
PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-8

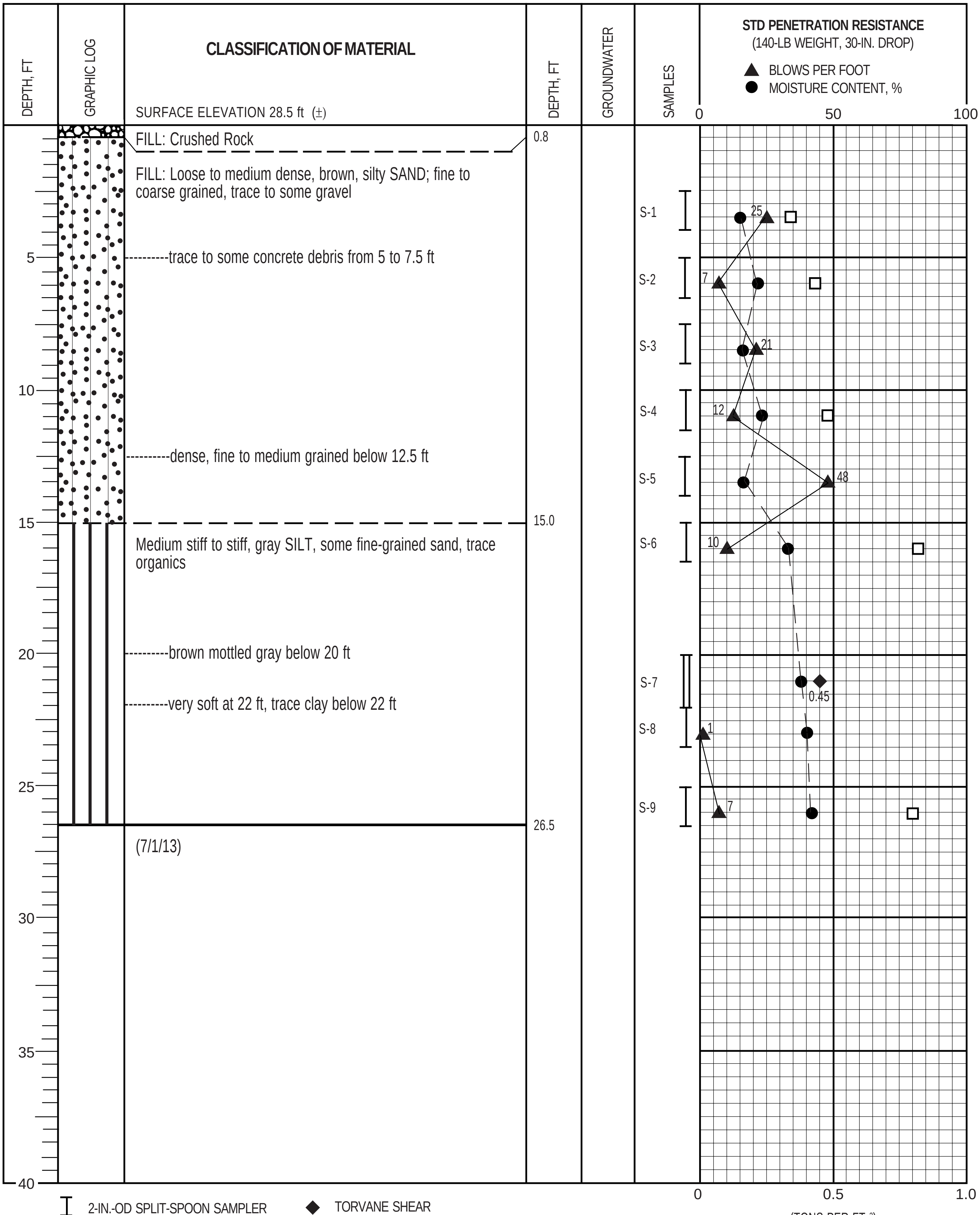
DEC. 2013 JOB. NO. W1114 FIG. 8A



GRI

BORING B-8 (cont.)

DEC. 2013 JOB. NO. W1114 FIG. 8A



2-IN.-OD SPLIT-SPOON SAMPLER

3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

P VIBRATING-WIRE PIEZOMETER

SLOTTED PVC PIPE

Water Level (date)

TORVANE SHEAR STRENGTH, TSF

PERCENT PASSING NO. 200 SIEVE (WASHED)

* NO RECOVERY

Liquid Limit

Moisture Content

Plastic Limit

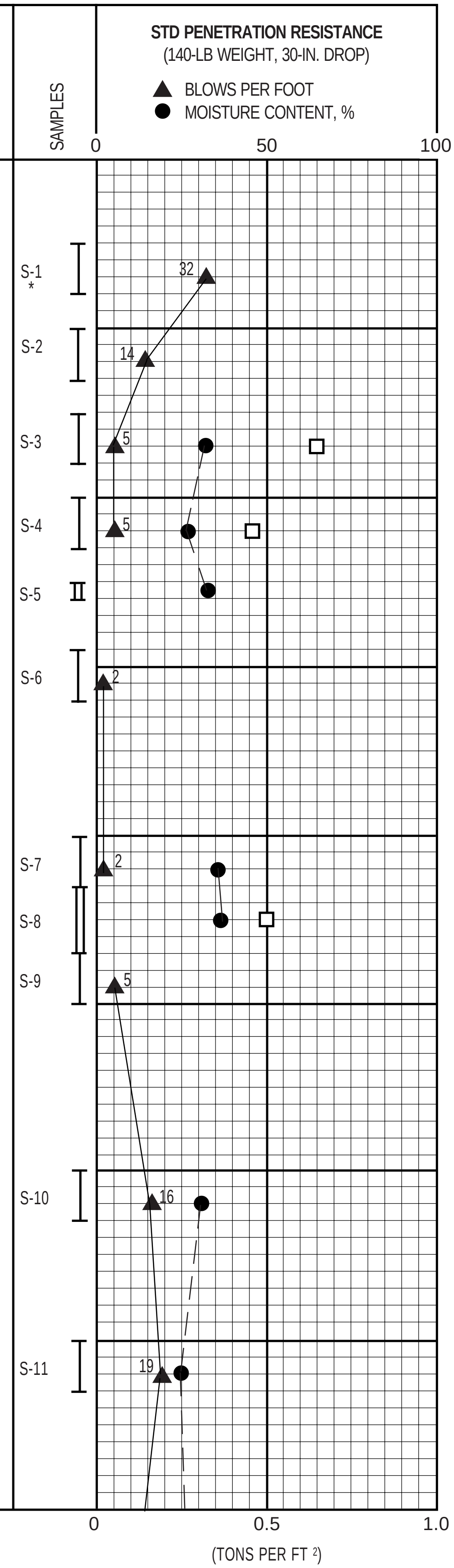
GRI

BORING B-9

DEC. 2013 JOB. NO. W1114 FIG. 9A

DEPTH, FT	CLASSIFICATION OF MATERIAL	DEPTH, FT
	SURFACE ELEVATION 29 ft (±)	
	FILL: Crushed Rock	1.7
	FILL: Medium dense to dense GRAVEL; subrounded to subangular, some fine- to coarse-grained sand	7.5
	FILL: Medium stiff, brown, sandy SILT; fine- to medium-grained sand, trace gravel, clay, and organics	10.0
	FILL: Loose, brown, silty SAND; fine grained, trace gravel and organics	12.5
	FILL: Medium stiff, brown SILT; some fine-grained sand	13.0
	FILL: Very loose to loose GRAVEL; subrounded to subangular, trace silt and fine-grained sand	20.0
	Very loose, brown mottled rust, silty SAND; fine-grained	
30	Medium dense, gray fine to coarse grained, trace silt below 30 ft	

GROUNDWATER



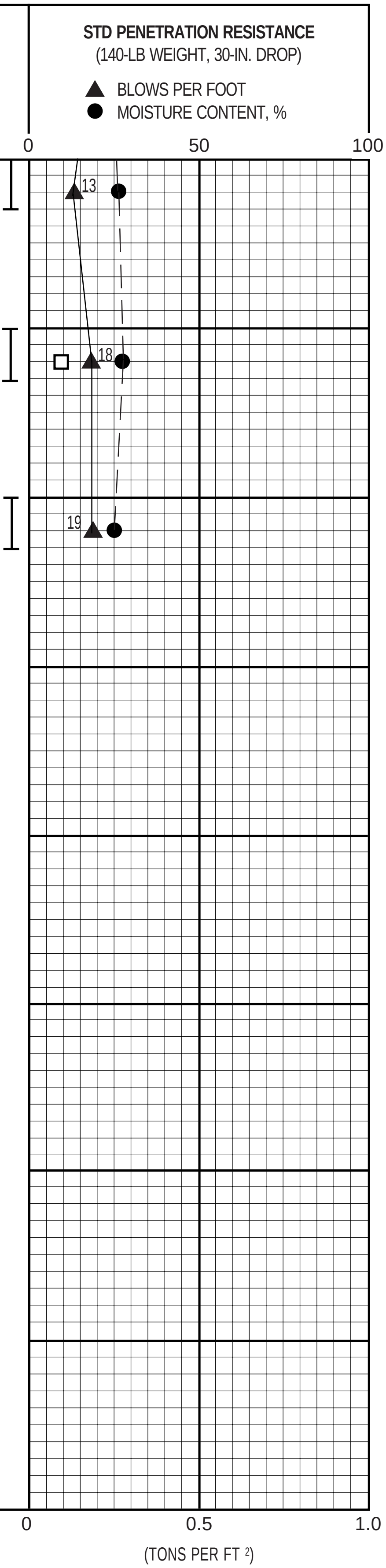
2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
P VIBRATING-WIRE PIEZOMETER
SLOTTED PVC PIPE
Water Level (date)
TORVANE SHEAR STRENGTH, TSF
PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-10

DEC. 2013 JOB. NO. W1114 FIG. 10A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL SURFACE ELEVATION 29 ft (±)	DEPTH, FT	GROUNDWATER	SAMPLES
40		Medium dense, gray SAND; fine to coarse grained, trace silt and gravel			S-12
45					S-13
50					S-14
		(7/1/2013)	51.5		
55					
60					
65					
70					



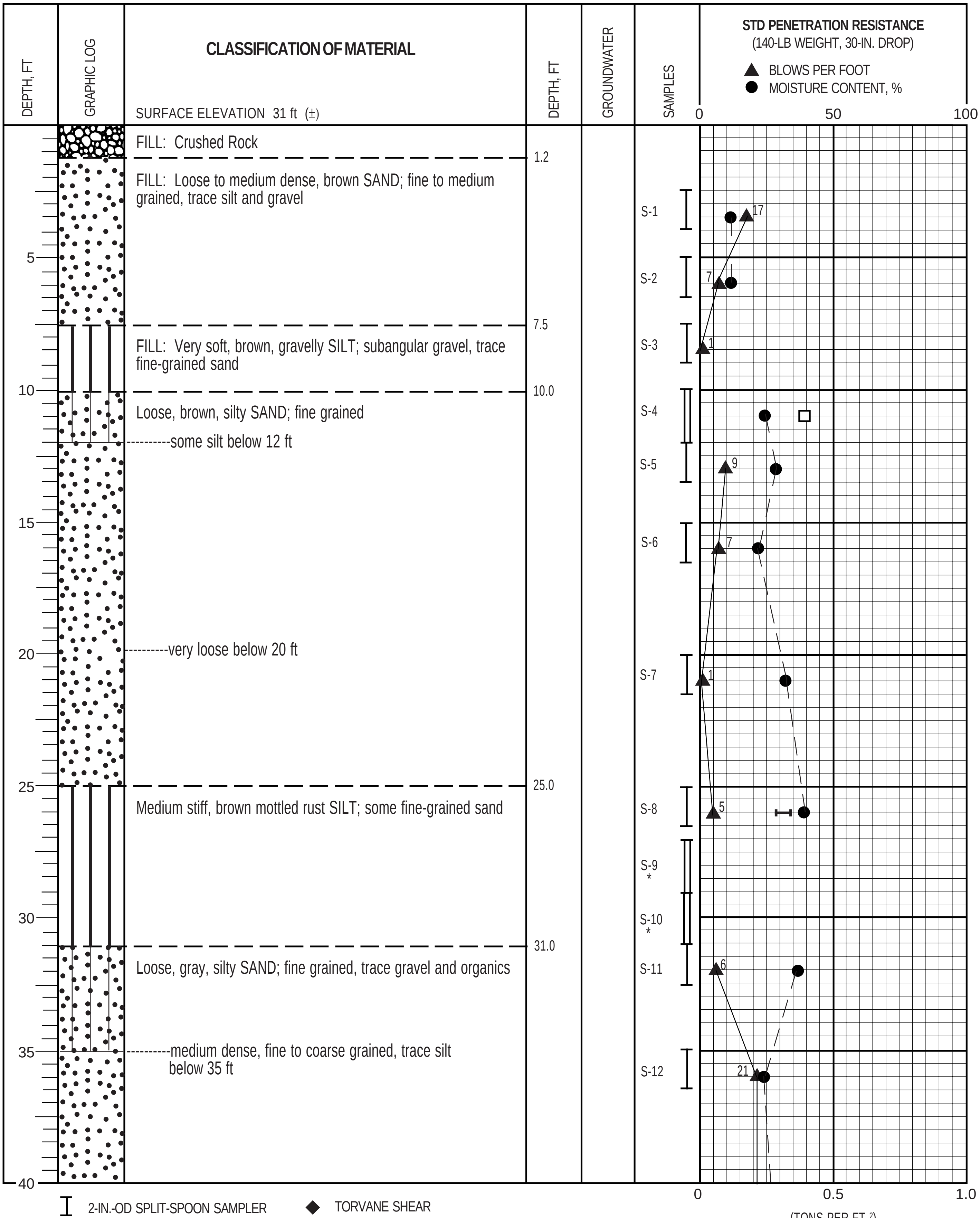
2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
P VIBRATING-WIRE PIEZOMETER
SLOTTED PVC PIPE
Water Level (date)

TORVANE SHEAR STRENGTH, TSF
PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-10 (cont.)

DEC. 2013 JOB. NO. W1114 FIG. 10A



2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
P VIBRATING-WIRE PIEZOMETER
SLOTTED PVC PIPE
Water Level (date)

TORVANE SHEAR STRENGTH, TSF
PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

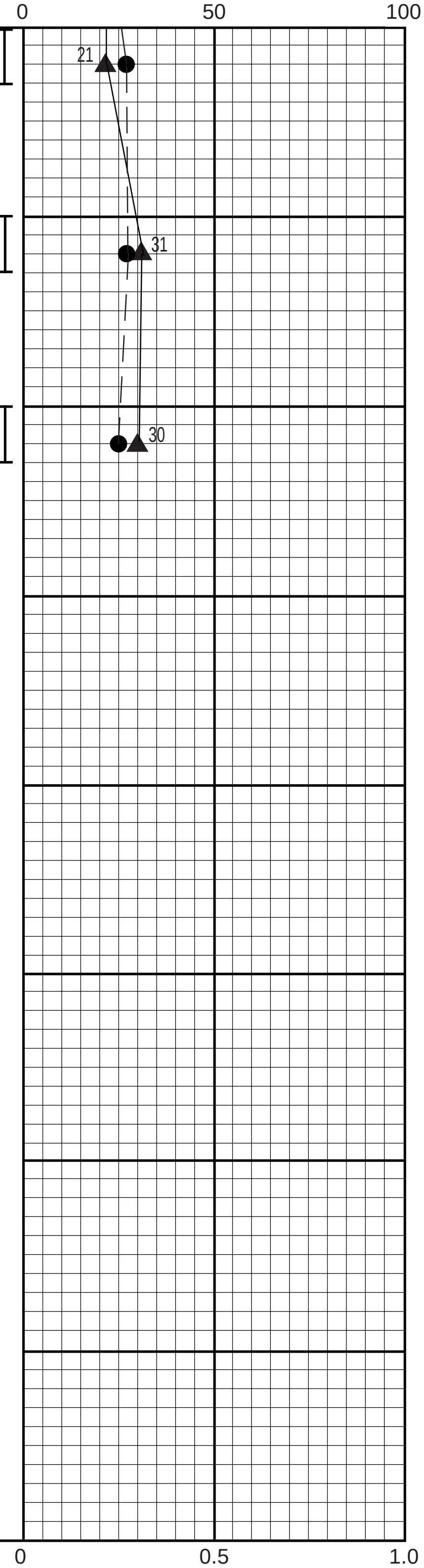
GRI

BORING B-11

DEC. 2013 JOB. NO. W1114 FIG. 11A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER	SAMPLES
		SURFACE ELEVATION 31 ft (±)			
40		Medium dense to dense, gray SAND; fine to medium grained, trace silt			S-13
45					S-14
50					S-15
		(7/1/2013)	51.5		
55					
60					
65					
70					
75					
80					

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)
▲ BLOWS PER FOOT
● MOISTURE CONTENT, %



(TONS PER FT ²)

I 2-IN.-OD SPLIT-SPOON SAMPLER
II 3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
P VIBRATING-WIRE PIEZOMETER
SLOTTED PVC PIPE
Water Level (date)

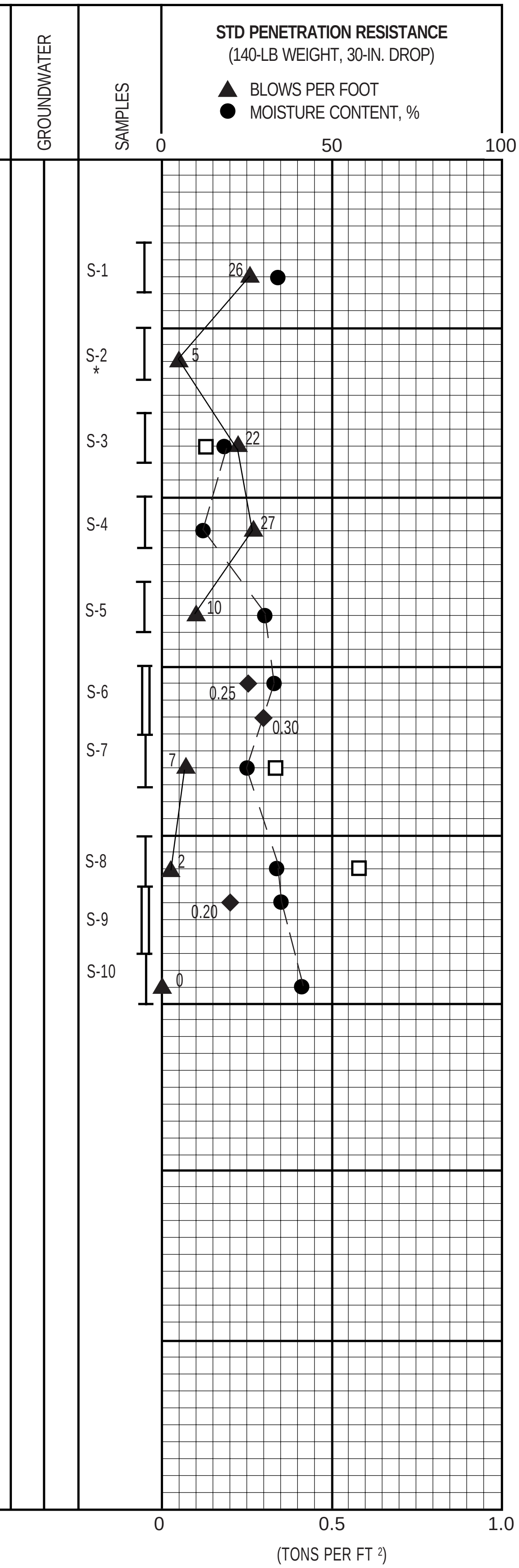
◆ TORVANE SHEAR STRENGTH, TSF
□ PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-11 (cont.)

DEC. 2013 JOB. NO. W1114 FIG. 11A

DEPTH, FT	CLASSIFICATION OF MATERIAL	DEPTH, FT
	SURFACE ELEVATION 31 ft (±)	
	FILL: Brown SAND	1.0
	FILL: Medium dense, brown, sandy GRAVEL; fine- to coarse-grained sand, some silt, trace organics	7.5
	FILL: Medium dense, brown SAND; fine to medium grained, trace to some silt	12.5
	FILL: Stiff, brown SILT; trace fine-grained sand ----------gray below 13 ft	15.0
	Loose, brown, silty SAND; fine grained	21.0
	Very soft to soft, brown, sandy SILT; fine-grained sand	25.0
	(7/2/2013)	



I 2-IN.-OD SPLIT-SPOON SAMPLER
II 3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
P VIBRATING-WIRE PIEZOMETER
SLOTTED PVC PIPE
Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF
□ PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

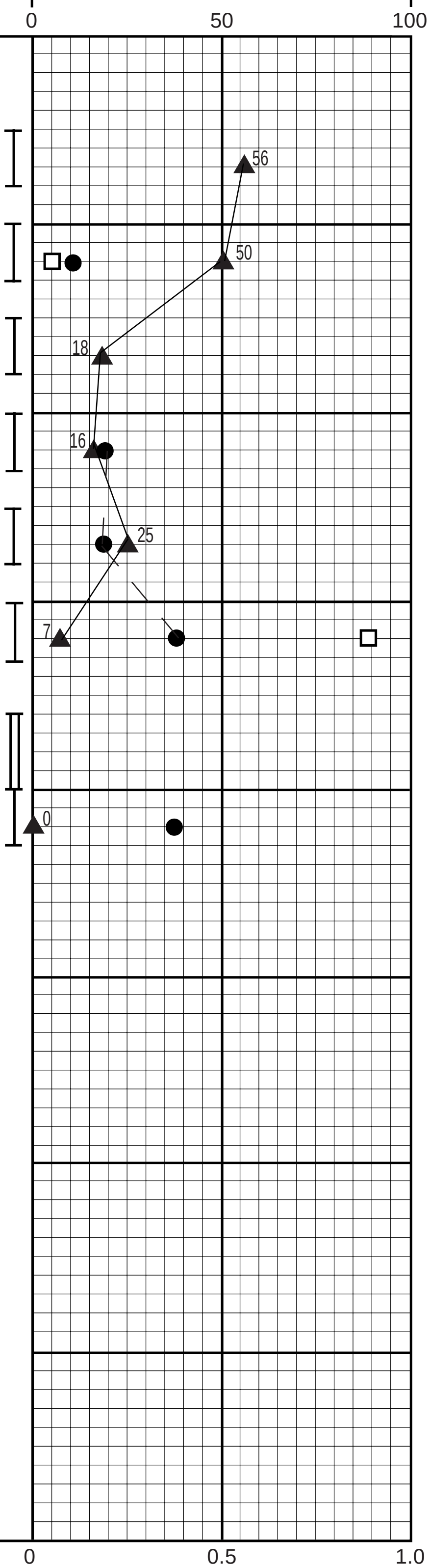
GRI

BORING B-12

DEC. 2013 JOB. NO. W1114 FIG. 12A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER	SAMPLES
		SURFACE ELEVATION 34 ft (±)			
		FILL: Very dense, sandy GRAVEL; fine- to coarse-grained sand, trace silt	4.5		S-1
5		FILL: Very dense, dark gray SAND; fine to medium grained, some gravel, trace silt, scattered asphaltic concrete debris	7.5		S-2
		FILL: Medium dense GRAVEL; trace silt and fine- to medium-grained sand	10.0		S-3
10		FILL: Medium dense, brown SAND; fine to medium grained, some gravel, trace silt ----------2-in.-thick layer of gray silt at 13.8 ft	15.0		S-4 S-5
15		Medium stiff, gray SILT; some fine-grained sand, trace gravel and organics ----------very soft, trace clay below 20 ft	21.5		S-6 S-7 * S-8
20		(7/2/13)			
25					
30					
35					
40					

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)
▲ BLOWS PER FOOT
● MOISTURE CONTENT, %



I 2-IN.-OD SPLIT-SPOON SAMPLER
II 3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
(P) VIBRATING-WIRE PIEZOMETER
SLOTTED PVC PIPE
▼ Water Level (date)
◆ TORVANE SHEAR STRENGTH, TSF
□ PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

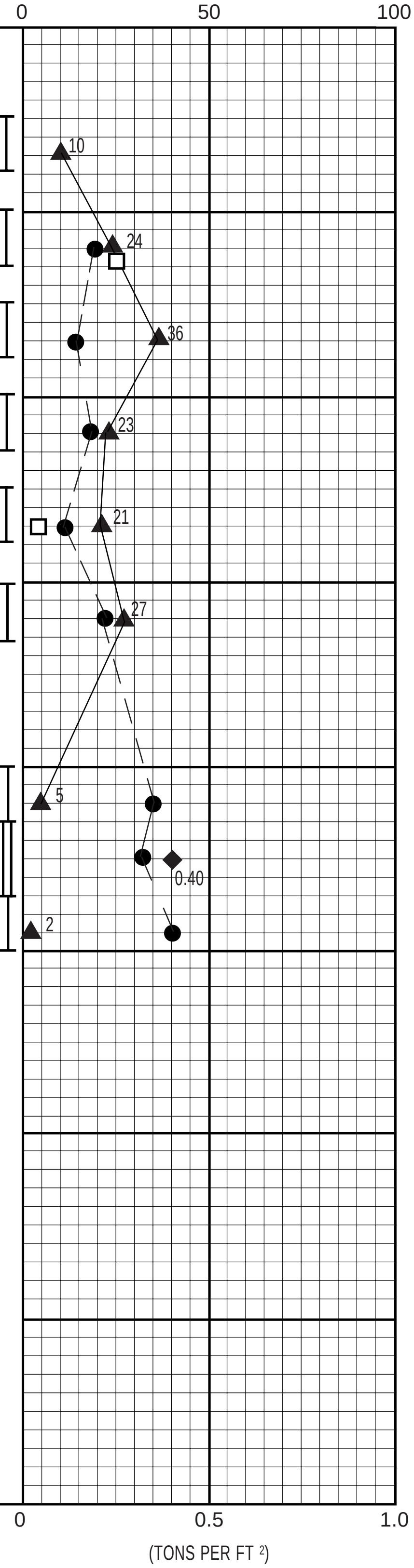
GRI

BORING B-13

DEC. 2013 JOB. NO. W1114 FIG. 13A

DEPTH, FT	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER	SAMPLES
	SURFACE ELEVATION 34 ft (±)			
0–1	FILL: Loose, brown SAND	1.0		
1–5	FILL: Medium dense, brown, sandy GRAVEL; fine- to coarse-grained and, trace silt and organics	5.0		S-1
5–20	FILL: Medium dense to dense, brown SAND; fine to medium grained, trace to some silt ---------trace gravel below 10 ft	20.0		S-2, S-3, S-4, S-5, S-6
20–25	Soft to medium stiff, brown and gray SILT; trace to some clay, trace fine-grained sand, and organics	25.0		S-7, S-8, S-9
	(7/2/13)			

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)
▲ BLOWS PER FOOT
● MOISTURE CONTENT, %



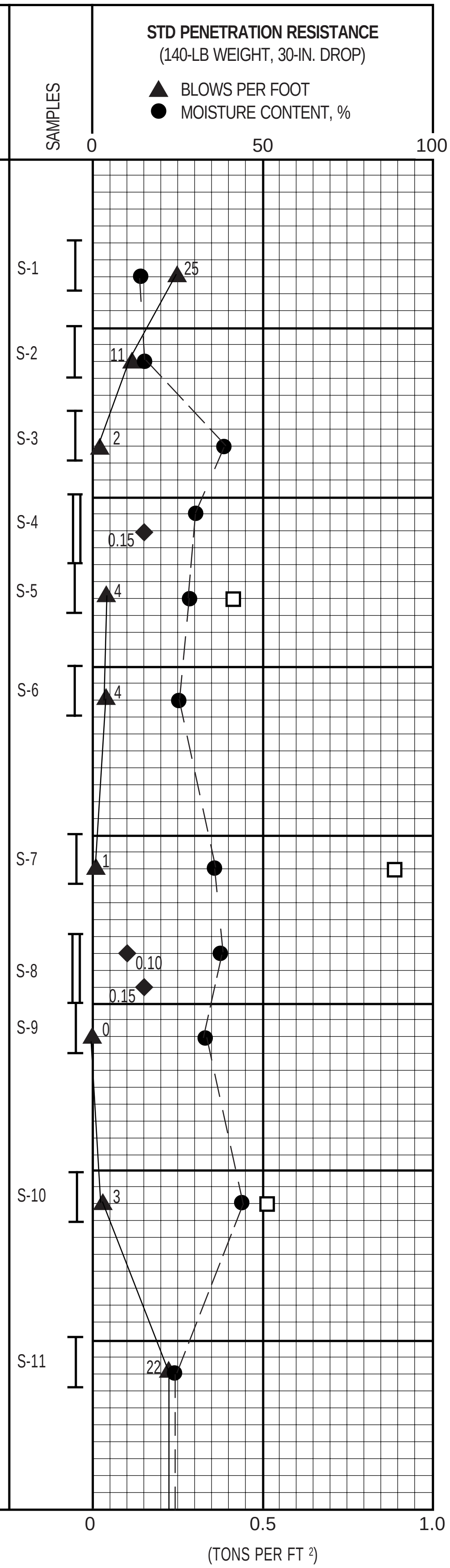
I 2-IN.-OD SPLIT-SPOON SAMPLER
II 3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
P VIBRATING-WIRE PIEZOMETER
SLOTTED PVC PIPE
Water Level (date)
◆ TORVANE SHEAR STRENGTH, TSF
□ PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-14

DEC. 2013 JOB. NO. W1114 FIG. 14A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER	SAMPLES
		SURFACE ELEVATION 31.5 ft (±)			
		FILL: Crushed Rock	0.8		
		FILL: Medium dense, brown SAND; fine to medium grained, trace to some gravel, trace silt	7.0		S-1, S-2
5					
		Very soft, brown SILT; trace to some clay, trace fine-grained sand and organics	10.0		S-3
10		Very loose, brown silty SAND; fine grained			S-4, S-5
15		---------trace to some silt below 15 ft	20.0		S-6
20		Very soft, brown mottled rust SILT; some fine-grained sand			S-7, S-8
25					S-9
30		---------soft, gray, sandy, trace organics below 30 ft	34.5		S-10
35		Medium dense, gray SAND; fine to coarse grained, trace silt			S-11
40					



2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
P VIBRATING-WIRE PIEZOMETER
SLOTTED PVC PIPE
Water Level (date)

TORVANE SHEAR STRENGTH, TSF
PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-15

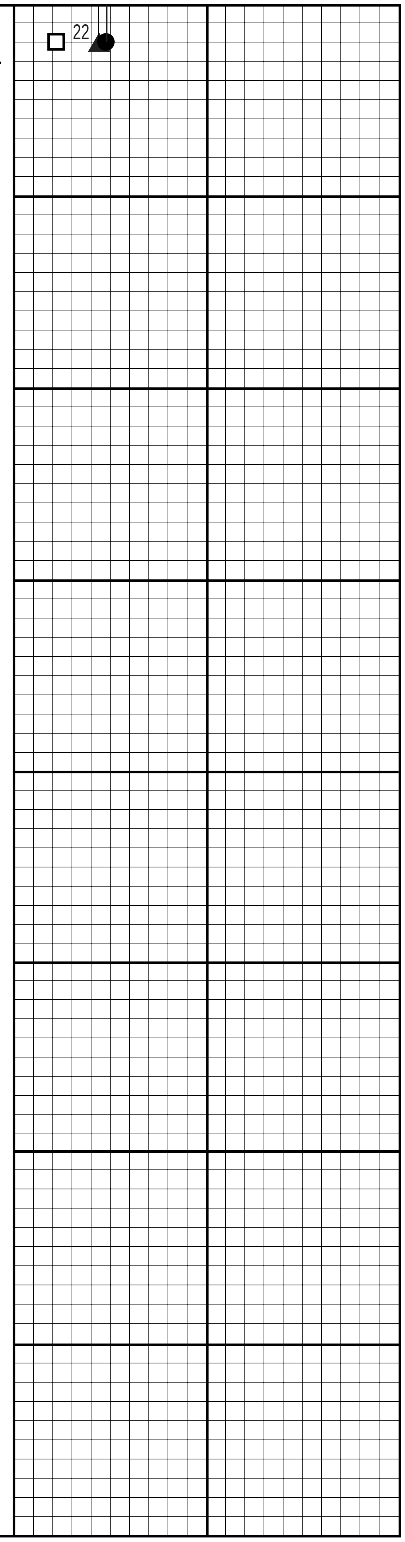
DEC. 2013 JOB. NO. W1114 FIG. 15A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL SURFACE ELEVATION 31.5 ft (±)	DEPTH, FT	GROUNDWATER	SAMPLES
40		Medium dense, gray SAND; fine to coarse grained, some silt, trace gravel	41.5		S-12
		(7/8/2013)			
45					
50					
55					
60					
65					
70					

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)

▲ BLOWS PER FOOT
● MOISTURE CONTENT, %

0 50 100



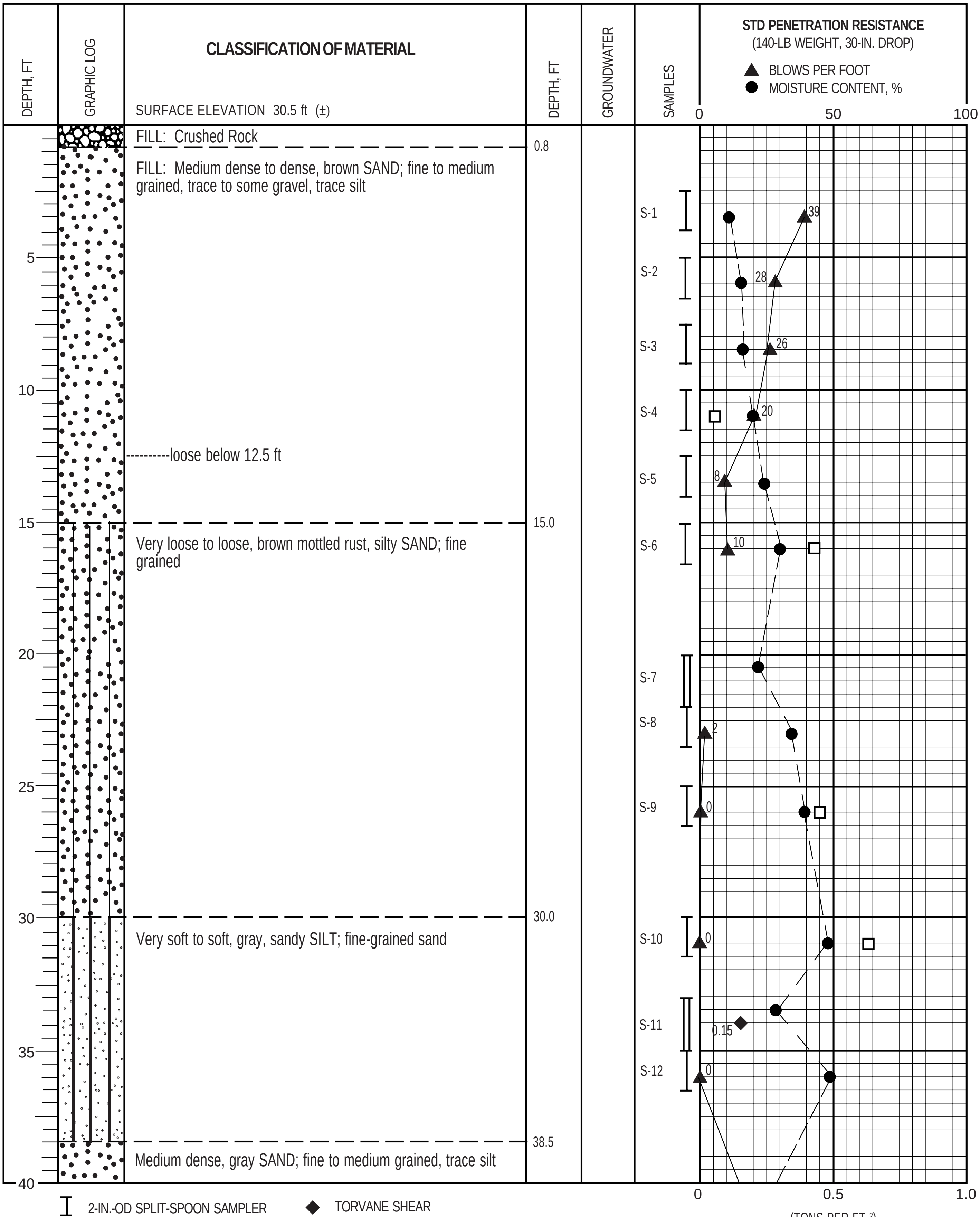
0 0.5 1.0
(TONS PER FT ^{2})

- 2-IN.-OD SPLIT-SPOON SAMPLER
- 3-IN.-OD THIN-WALLED SAMPLER
- G GRAB SAMPLE OF DRILL CUTTINGS
- P VIBRATING-WIRE PIEZOMETER
- SLOTTED PVC PIPE
- Water Level (date)
- ◆ TORVANE SHEAR STRENGTH, TSF
- □ PERCENT PASSING NO. 200 SIEVE (WASHED)
- * NO RECOVERY
- Liquid Limit
- Moisture Content
- Plastic Limit

GRI

BORING B-15 (cont.)

DEC. 2013 JOB. NO. W1114 FIG. 15A



2-IN.-OD SPLIT-SPOON SAMPLER

3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

P VIBRATING-WIRE PIEZOMETER

SLOTTED PVC PIPE

Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF

□ PERCENT PASSING NO. 200 SIEVE (WASHED)

* NO RECOVERY

Liquid Limit

Moisture Content

Plastic Limit

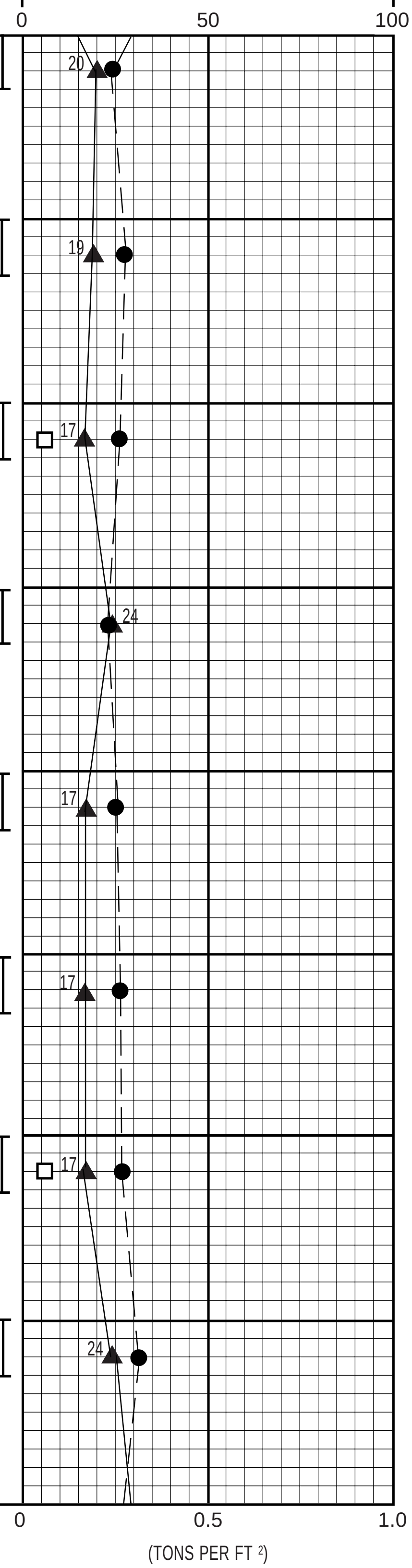
GRI

BORING B-16

DEC. 2013 JOB. NO. W1114 FIG. 16A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL SURFACE ELEVATION 30.5 ft (±)	DEPTH, FT	GROUNDWATER	SAMPLES
40		Medium dense, gray SAND; fine to coarse grained, trace silt			S-13
45					S-14
50					S-15
55					S-16
60					S-17
65					S-18
70					S-19
75					S-20
80					

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)
▲ BLOWS PER FOOT
● MOISTURE CONTENT, %



2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
P VIBRATING-WIRE PIEZOMETER
SLOTTED PVC PIPE
Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF
□ PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

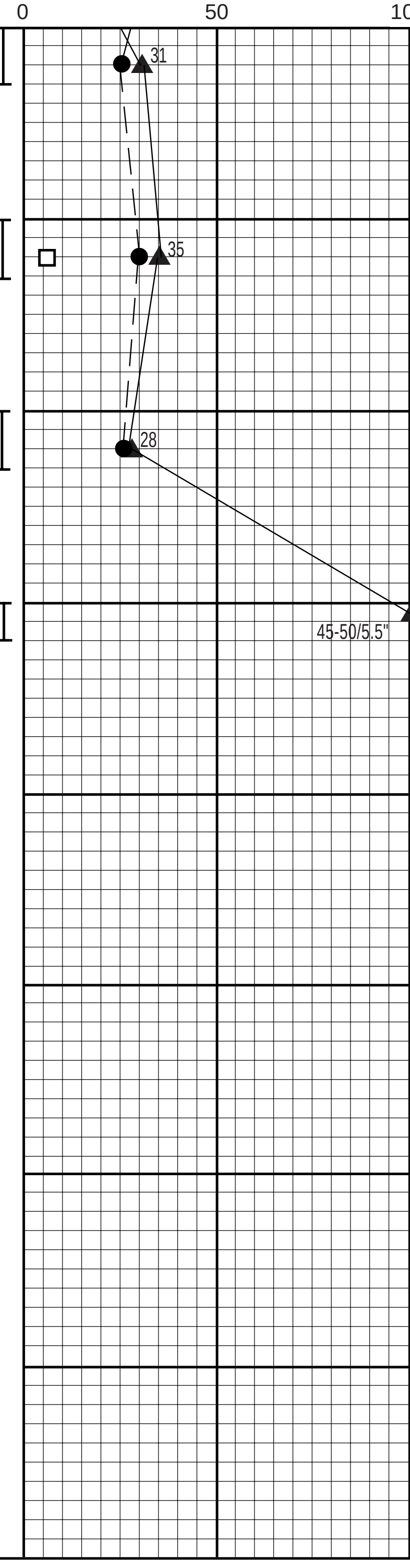
BORING B-16 (cont.)

DEC. 2013 JOB. NO. W1114 FIG. 16A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL SURFACE ELEVATION 30.5 ft (±)	DEPTH, FT	GROUNDWATER	SAMPLES
80		Medium dense to dense, gray SAND; fine to coarse grained, trace silt			S-21
85					S-22
90		----------trace gravel below 90 ft			S-23
95		Very dense GRAVEL; some fine- to coarse-grained sand, trace silt	95.0		S-24
		(7/3/2013)	96.0		
100					
105					
110					
120					
130					

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)

▲ BLOWS PER FOOT
● MOISTURE CONTENT, %



(TONS PER FT ^{2})

I 2-IN.-OD SPLIT-SPOON SAMPLER

II 3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

P VIBRATING-WIRE PIEZOMETER

SLOTTED PVC PIPE

Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF

□ PERCENT PASSING NO. 200 SIEVE (WASHED)

* NO RECOVERY

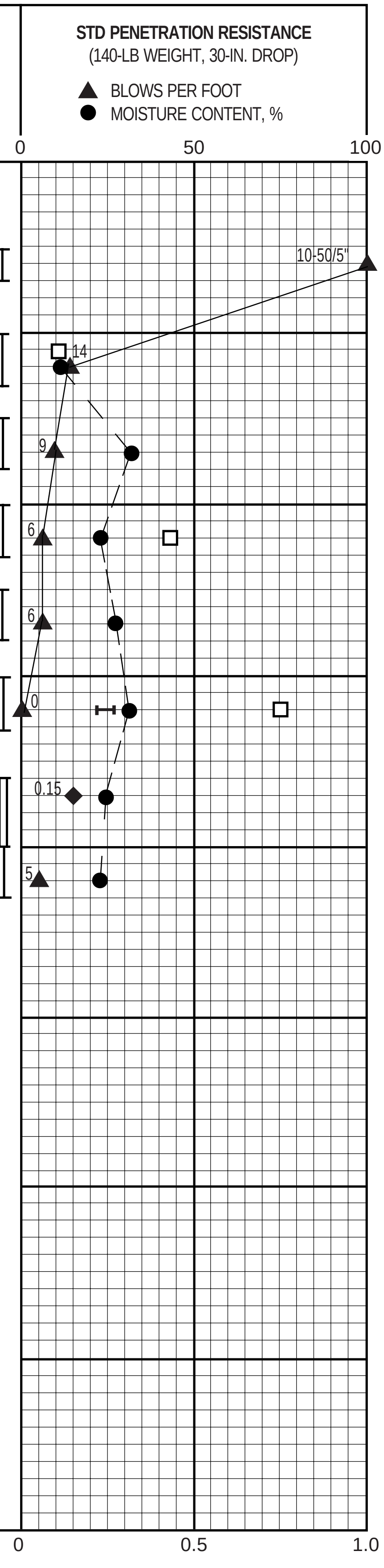
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-16 (cont.)

DEC. 2013 JOB. NO. W1114 FIG. 16A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER	SAMPLES
		SURFACE ELEVATION 31.5 ft (±)			
		FILL: Crushed Rock	0.8		
		FILL: Very dense GRAVEL; some fine- to coarse-grained sand, trace silt	5.0		S-1
5		FILL: Medium dense, brown SAND; fine to medium grained, some silt, trace gravel	7.5		S-2
		Stiff, gray to brown mottled rust SILT; some fine-grained sand, trace organics	10.0		S-3
10		Loose, brown, silty SAND; fine grained ---------trace to some silt below 12.5 ft	15.0		S-4 S-5
15		Very soft, brown mottled rust SILT; some fine-grained sand ----------soft to medium stiff below 18 ft	21.5		S-6 S-7
20					S-8
		(7/8/13)			
25					
30					
35					
40					



2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
P VIBRATING-WIRE PIEZOMETER
SLOTTED PVC PIPE
Water Level (date)

TORVANE SHEAR STRENGTH, TSF
PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

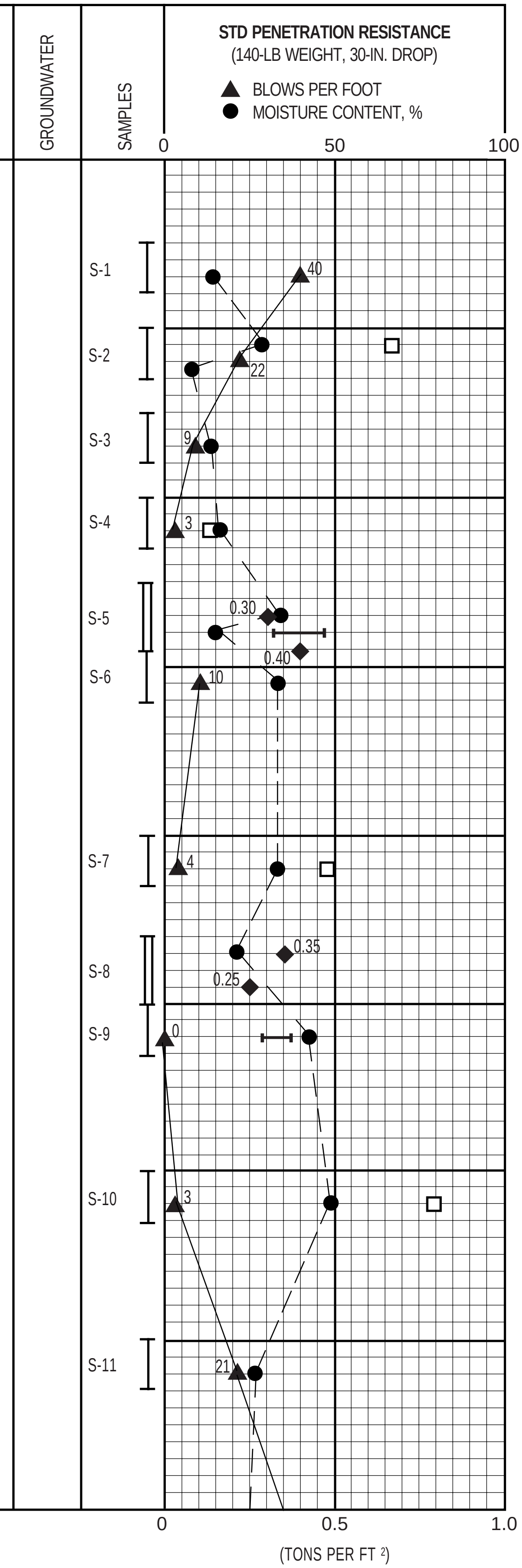
GRI

BORING B-17

DEC. 2013 | JOB. NO. W1114 | FIG. 17A

SURFACE ELEVATION ----- ft (±)

DEPTH, FT	CLASSIFICATION OF MATERIAL SURFACE ELEVATION 31 ft (±)	DEPTH, FT
0	FILL: Crushed Rock	0.8
	FILL: Dense, brown SAND; fine to coarse grained, trace gravel and silt	5.0
5	FILL: Stiff, gray mottled rust, sandy SILT; fine-grained sand	5.7
	FILL: Medium dense, brown SAND; fine to medium grained, trace silt ----------very loose to loose below 7.5 ft ----------trace gravel below 10 ft ----------silty, fine grained below 12 ft	14.0
15	Medium stiff to stiff, gray mottled rust SILT; trace to some clay and fine-grained sand	20.0
20	Very loose to loose, brown, silty SAND; fine grained	23.0
25	Medium stiff, brown mottled rust SILT; trace to some fine-grained sand ----------very soft to soft, gray mottled rust below 25 ft ----------gray, trace organics below 30 ft	34.0
35	Medium dense, gray SAND; fine to medium grained, some silt	
40		



2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
P VIBRATING-WIRE PIEZOMETER
SLOTTED PVC PIPE
Water Level (date)
TORVANE SHEAR STRENGTH, TSF
PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-18

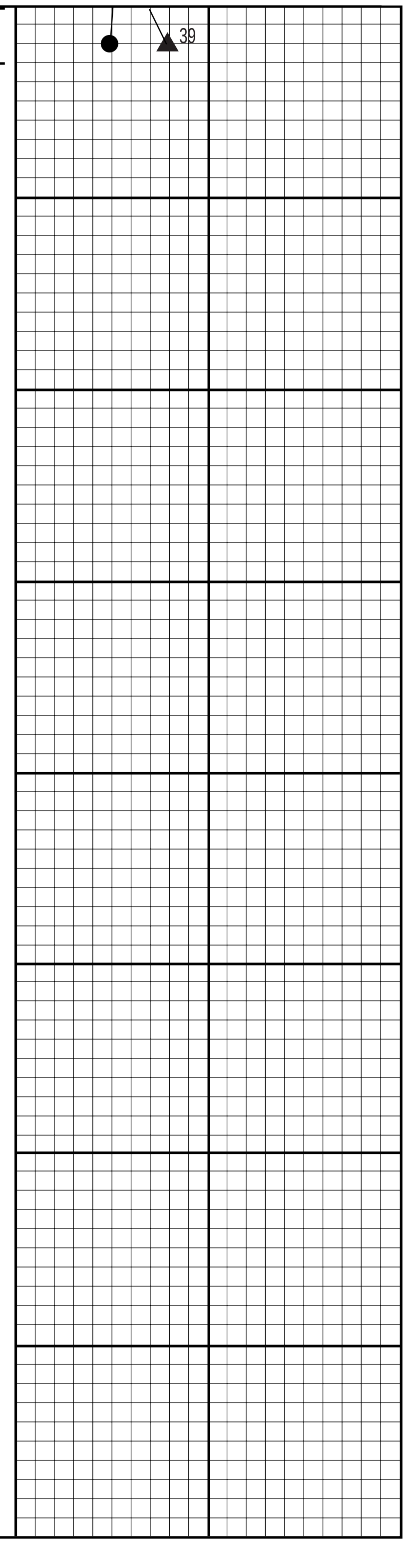
DEC. 2013 JOB. NO. W1114 FIG. 18A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL SURFACE ELEVATION 31 ft (±)	DEPTH, FT	GROUNDWATER	SAMPLES
40		Dense, brown SAND; fine to medium grained, trace silt	41.5		S-12
		(7/8/2013)			
45					
50					
55					
60					
65					
70					
75					
80					

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)

▲ BLOWS PER FOOT
● MOISTURE CONTENT, %

0 50 100



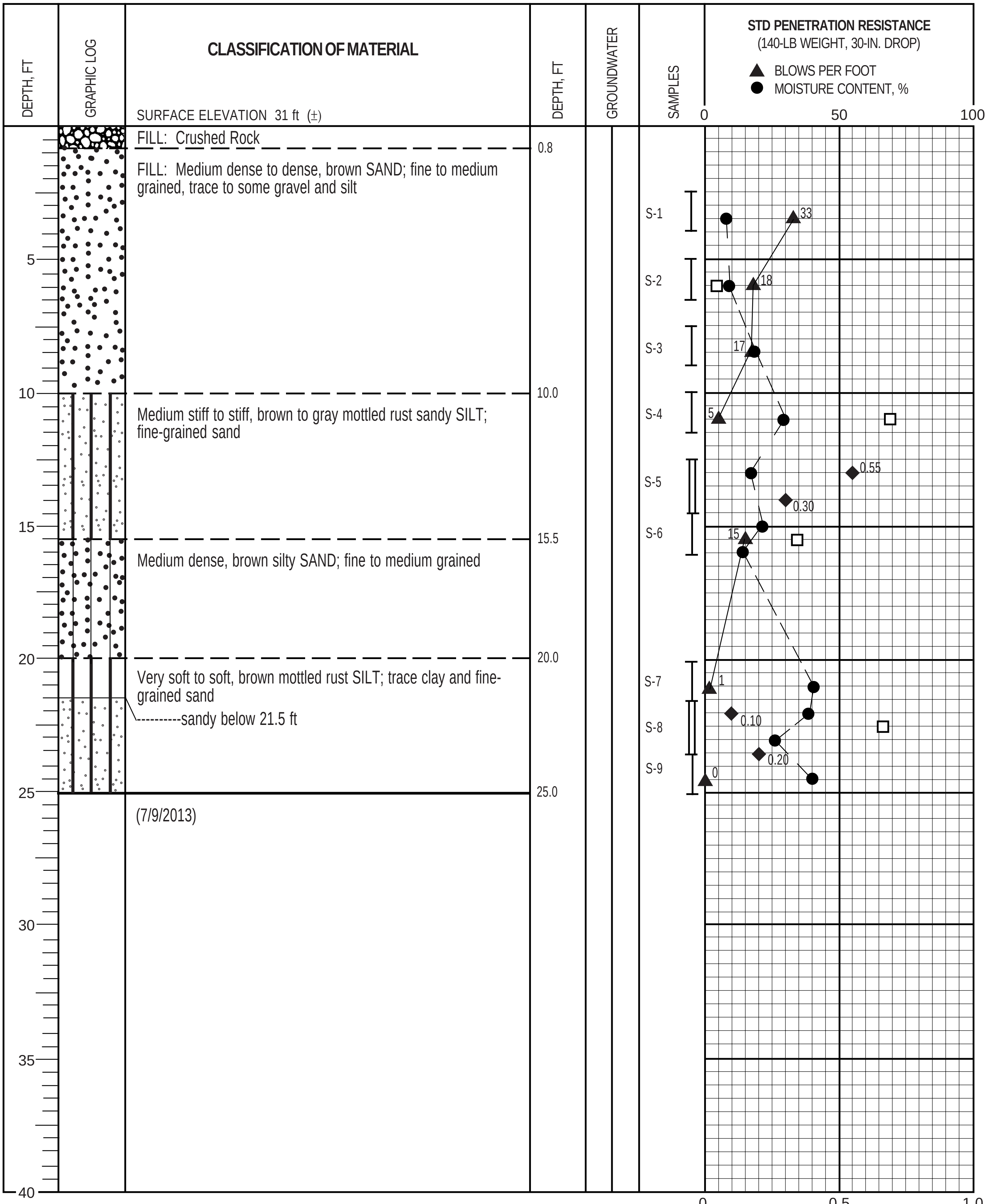
0 0.5 1.0
(TONS PER FT ^{2})

- 2-IN.-OD SPLIT-SPOON SAMPLER
- 3-IN.-OD THIN-WALLED SAMPLER
- G GRAB SAMPLE OF DRILL CUTTINGS
- P VIBRATING-WIRE PIEZOMETER
- SLOTTED PVC PIPE
- Water Level (date)
- ◆ TORVANE SHEAR STRENGTH, TSF
- □ PERCENT PASSING NO. 200 SIEVE (WASHED)
- * NO RECOVERY
- Liquid Limit, Moisture Content, Plastic Limit

GRI

BORING B-18 (cont.)

DEC. 2013 JOB. NO. W1114 FIG. 18A



2-IN.-OD SPLIT-SPOON SAMPLER

3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

P VIBRATING-WIRE PIEZOMETER

SLOTTED PVC PIPE

Water Level (date)

TORVANE SHEAR STRENGTH, TSF

PERCENT PASSING NO. 200 SIEVE (WASHED)

* NO RECOVERY

Liquid Limit
Moisture Content
Plastic Limit

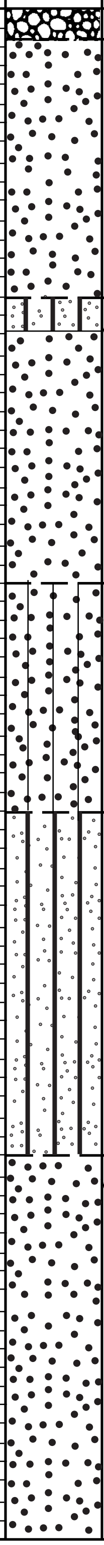
GRI

BORING B-19

DEC. 2013 | JOB. NO. W1114 | FIG. 19A

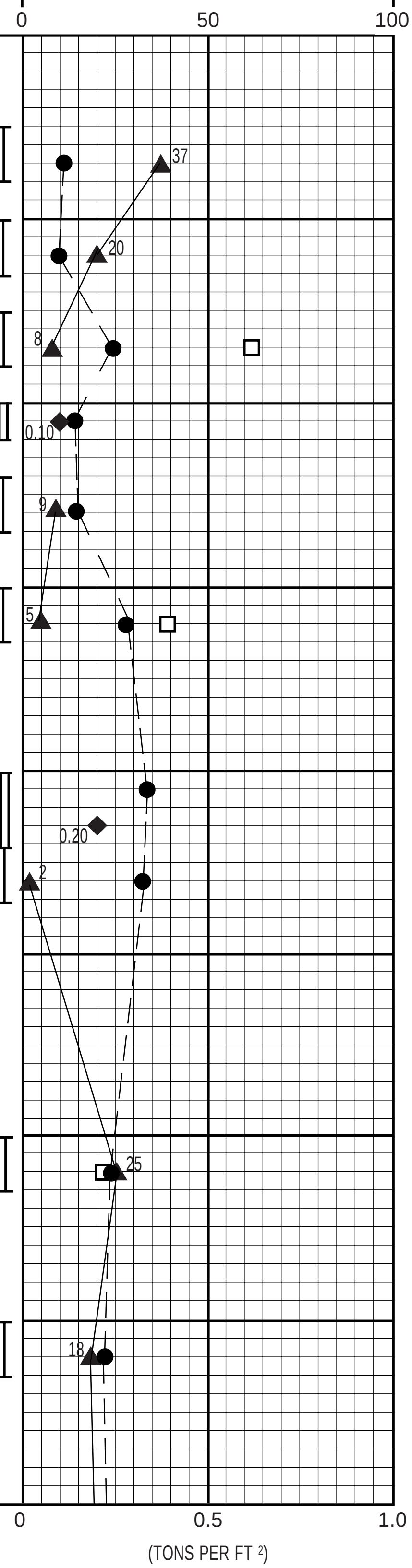
DEPTH, FT	CLASSIFICATION OF MATERIAL	DEPTH, FT	SAMPLES
	SURFACE ELEVATION 31 ft (±)		
	FILL: Crushed Rock	0.8	
	FILL: Medium dense to dense, brown SAND; fine to coarse grained, some gravel, trace silt		S-1
5			S-2
	FILL: Medium stiff, gray mottled rust, sandy SILT; fine-grained sand	7.5	S-3
10	FILL: Loose, brown SAND; fine to medium grained, trace to some silt	8.4	S-4, S-5
15	Loose, brown mottled rust, silty SAND; fine grained	15.0	S-6
20	Soft, brown, sandy SILT; fine-grained sand	21.0	S-7, S-8
25			
30	Medium dense, brown SAND; fine to medium grained, some silt ---------gray, trace silt below 31 ft	30.0	S-9
35			S-10
40			

GRAPHIC LOG



GROUNDWATER

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)
▲ BLOWS PER FOOT
● MOISTURE CONTENT, %



2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
P VIBRATING-WIRE PIEZOMETER
SLOTTED PVC PIPE
Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF
□ PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-20

DEC. 2013 JOB. NO. W1114 FIG. 20A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER	SAMPLES	STD PENETRATION RESISTANCE (140-LB WEIGHT, 30-IN. DROP) ▲ BLOWS PER FOOT ● MOISTURE CONTENT, %
		SURFACE ELEVATION 31 ft (±)				0 50 100
40		Medium dense, gray SAND; fine to medium grained, trace silt	41.5		S-11	20
		(7/9/2013)				
45						

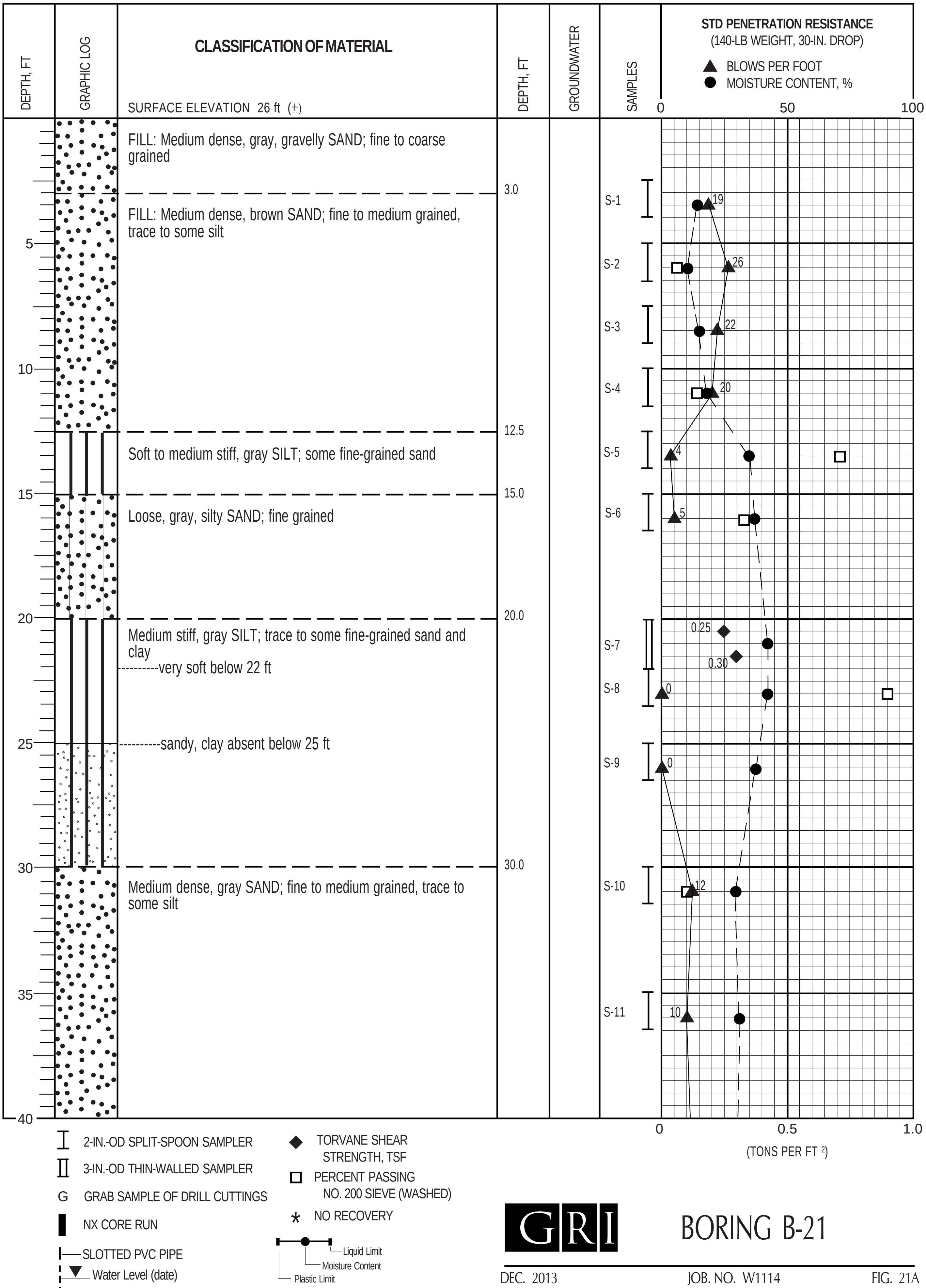
0 0.5 1.0
(TONS PER FT ²)

- I 2-IN.-OD SPLIT-SPOON SAMPLER
- II 3-IN.-OD THIN-WALLED SAMPLER
- G GRAB SAMPLE OF DRILL CUTTINGS
- P VIBRATING-WIRE PIEZOMETER
- SLOTTED PVC PIPE
- ▼ Water Level (date)
- ◆ TORVANE SHEAR STRENGTH, TSF
- □ PERCENT PASSING NO. 200 SIEVE (WASHED)
- * NO RECOVERY
- Liquid Limit / Moisture Content / Plastic Limit

GRI

BORING B-20 (cont.)

DEC. 2013 JOB. NO. W1114 FIG. 20A

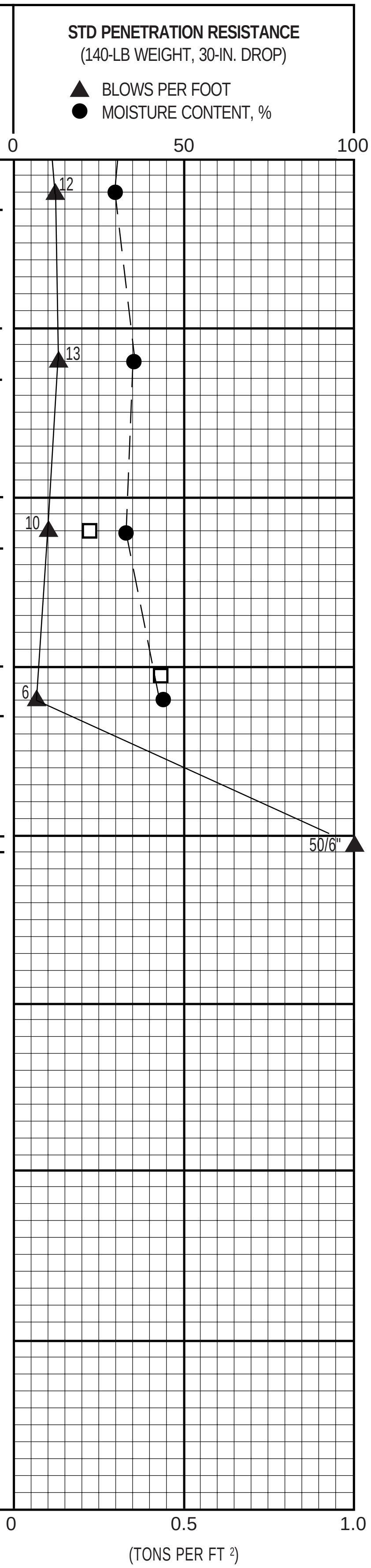


GRI

BORING B-21

DEC. 2013 JOB. NO. W1114 FIG. 21A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER	SAMPLES
		SURFACE ELEVATION 26 ft (±)			
40		Medium dense, gray SAND; fine to medium grained, trace to some silt, trace organics			S-12
45		---------organics absent below 45 ft			S-13
50					S-14
55		---------silty below 55 ft	55.5		S-15
		Medium stiff, gray SILT; trace clay, gravel, and fine- to coarse-grained sand	59.0		
60		Very dense GRAVEL	60.5		S-16
		(8/2/2013)			
65					



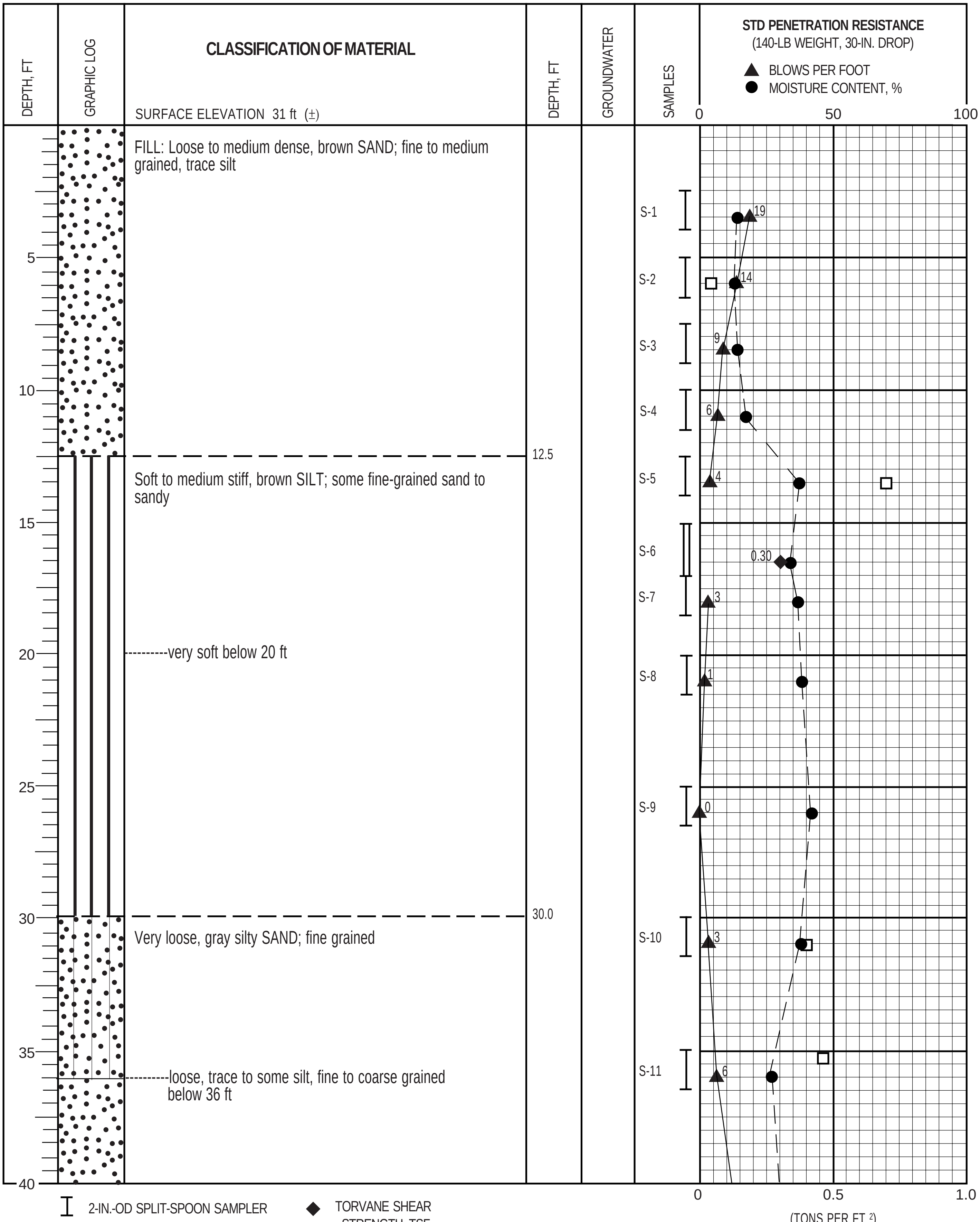
2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
NX CORE RUN
SLOTTED PVC PIPE
Water Level (date)

TORVANE SHEAR STRENGTH, TSF
PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-21 (cont.)

DEC. 2013 JOB. NO. W1114 FIG. 21A



2-IN.-OD SPLIT-SPOON SAMPLER

3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

NX CORE RUN

SLOTTED PVC PIPE

Water Level (date)

TORVANE SHEAR STRENGTH, TSF

PERCENT PASSING NO. 200 SIEVE (WASHED)

* NO RECOVERY

Liquid Limit

Moisture Content

Plastic Limit

GRI

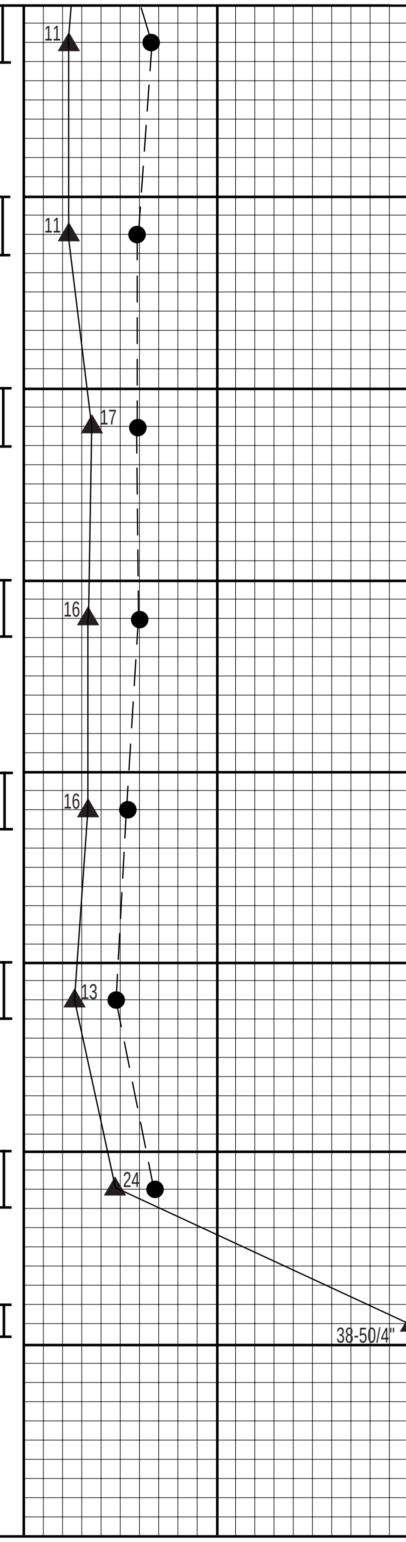
BORING B-22

DEC. 2013 JOB. NO. W1114 FIG. 22A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL SURFACE ELEVATION 31 ft (±)	DEPTH, FT	GROUNDWATER	SAMPLES
40		Medium dense SAND; fine to coarse grained, trace silt			S-12
45					S-13
50					S-14
55					S-15
60					S-16
65					S-17
70		-------trace subrounded gravel at 71.3 ft			S-18
		Stiff, gray, sandy SILT; fine grained	71.4		
		Very dense GRAVEL; some fine- to coarse-grained sand, trace silt	72.5		S-19
75		(8/1/2013)	75.5		
80					

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)

▲ BLOWS PER FOOT
● MOISTURE CONTENT, %



2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
NX CORE RUN
SLOTTED PVC PIPE
▼ Water Level (date)

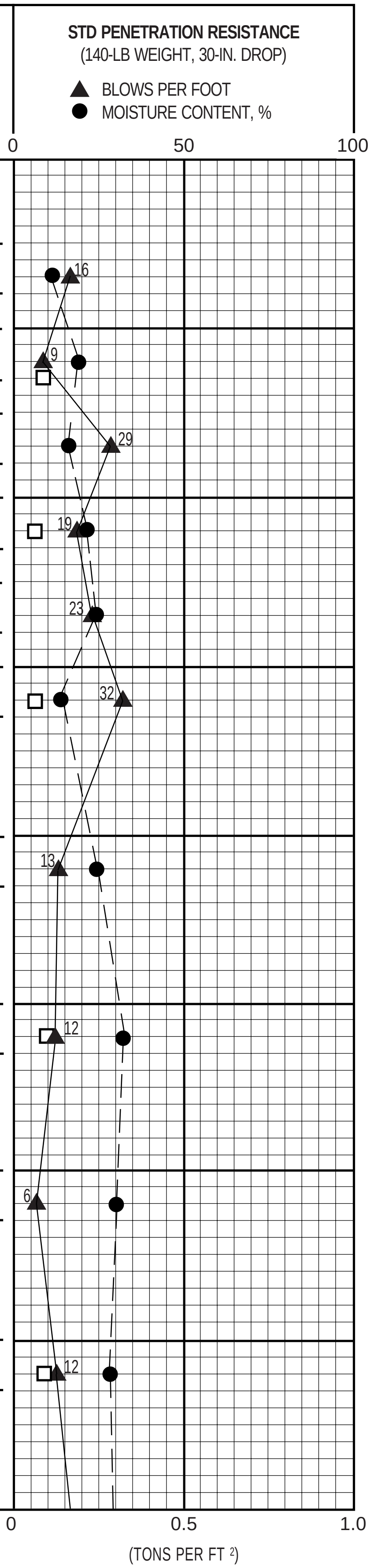
◆ TORVANE SHEAR STRENGTH, TSF
□ PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-22 (cont.)

DEC. 2013 | JOB. NO. W1114 | FIG. 22A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER	SAMPLES
		SURFACE ELEVATION 27 ft (±)			
		Asphaltic-concrete PAVEMENT (4 in.) over crushed rock BASE COURSE (10 in.)	1.2		
		FILL: Loose to medium dense, brown SAND; fine to medium grained, trace silt and gravel			S-1
5					S-2
		----------medium dense below 7.5 ft			S-3
10					S-4
		----------gray below 12.5 ft			
		----------scattered wood debris at 13.8 ft			S-5
15		----------4-in.-thick layer of sandy silt at 15 ft			S-6
20			20.0		
		Medium dense, gray SAND; fine to medium grained, trace silt, 1-in.-thick layer of silt at 20.1 ft			S-7
25		----------brown, trace to some silt below 25 ft			S-8
30		----------loose, trace silt below 30 ft			S-9
		----------medium dense, fine grained below 32 ft			
35					S-10
40					



2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
NX CORE RUN
SLOTTED PVC PIPE
Water Level (date)

TORVANE SHEAR STRENGTH, TSF
PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

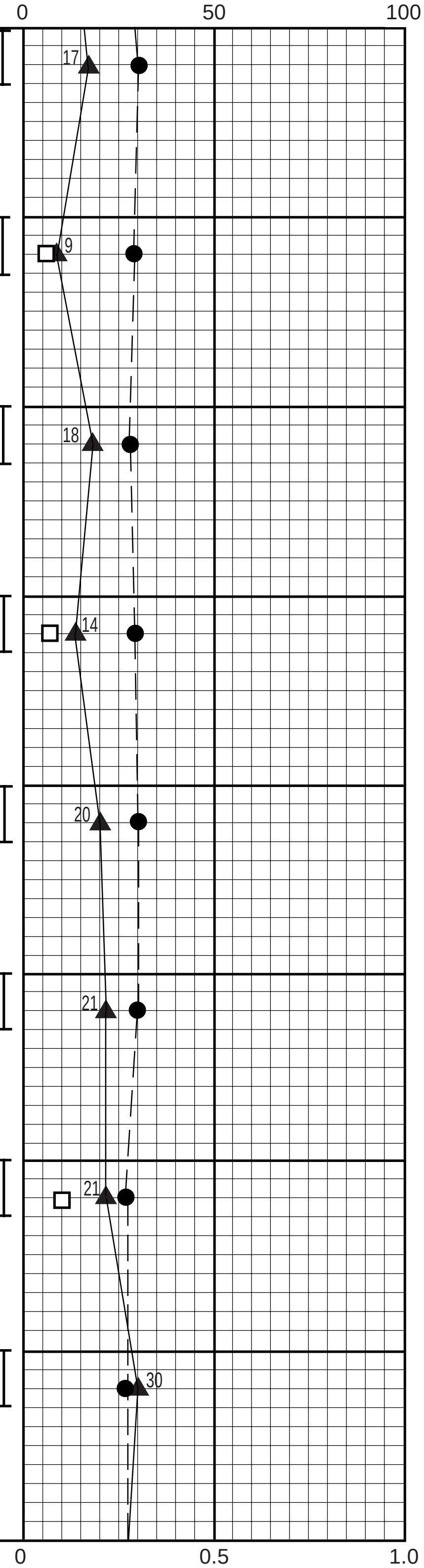
BORING B-23

DEC. 2013 | JOB. NO. W1114 | FIG. 23A

DEPTH, FT	CLASSIFICATION OF MATERIAL	SAMPLES	BLOWS PER FOOT
	SURFACE ELEVATION 27 ft (±)		
40	Medium dense, brown SAND; fine to medium grained, trace silt	S-11	17
45	---------loose, gray below 45 ft	S-12	9
50	---------medium dense below 50 ft	S-13	18
55		S-14	14
60		S-15	20
65		S-16	21
70	---------trace to some silt below 70 ft	S-17	21
75		S-18	30
80			

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)

▲ BLOWS PER FOOT
● MOISTURE CONTENT, %



I 2-IN.-OD SPLIT-SPOON SAMPLER
II 3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
▌ NX CORE RUN
|—SLOTTED PVC PIPE
▼ Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF
□ PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY

Liquid Limit
Moisture Content
Plastic Limit

GRI

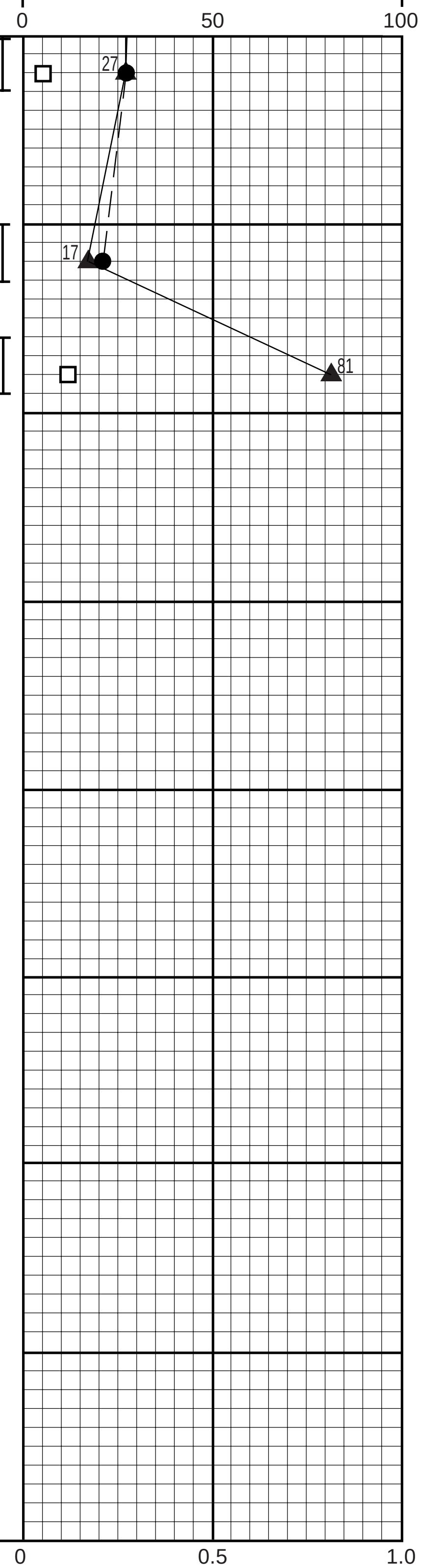
BORING B-23 (cont.)

DEC. 2013 JOB. NO. W1114 FIG. 23A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER	SAMPLES
		SURFACE ELEVATION 27 ft (±)			
80		Medium dense, gray SAND; fine grained, trace silt			S-19
85		---------fine to medium grained below 85 ft; scattered gravel below 85.5 ft			S-20
		Very dense GRAVEL; trace to some silt and fine- to coarse-grained sand	87.0		S-21
90		(7/29/2013)	89.5		
95					

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)

▲ BLOWS PER FOOT
● MOISTURE CONTENT, %



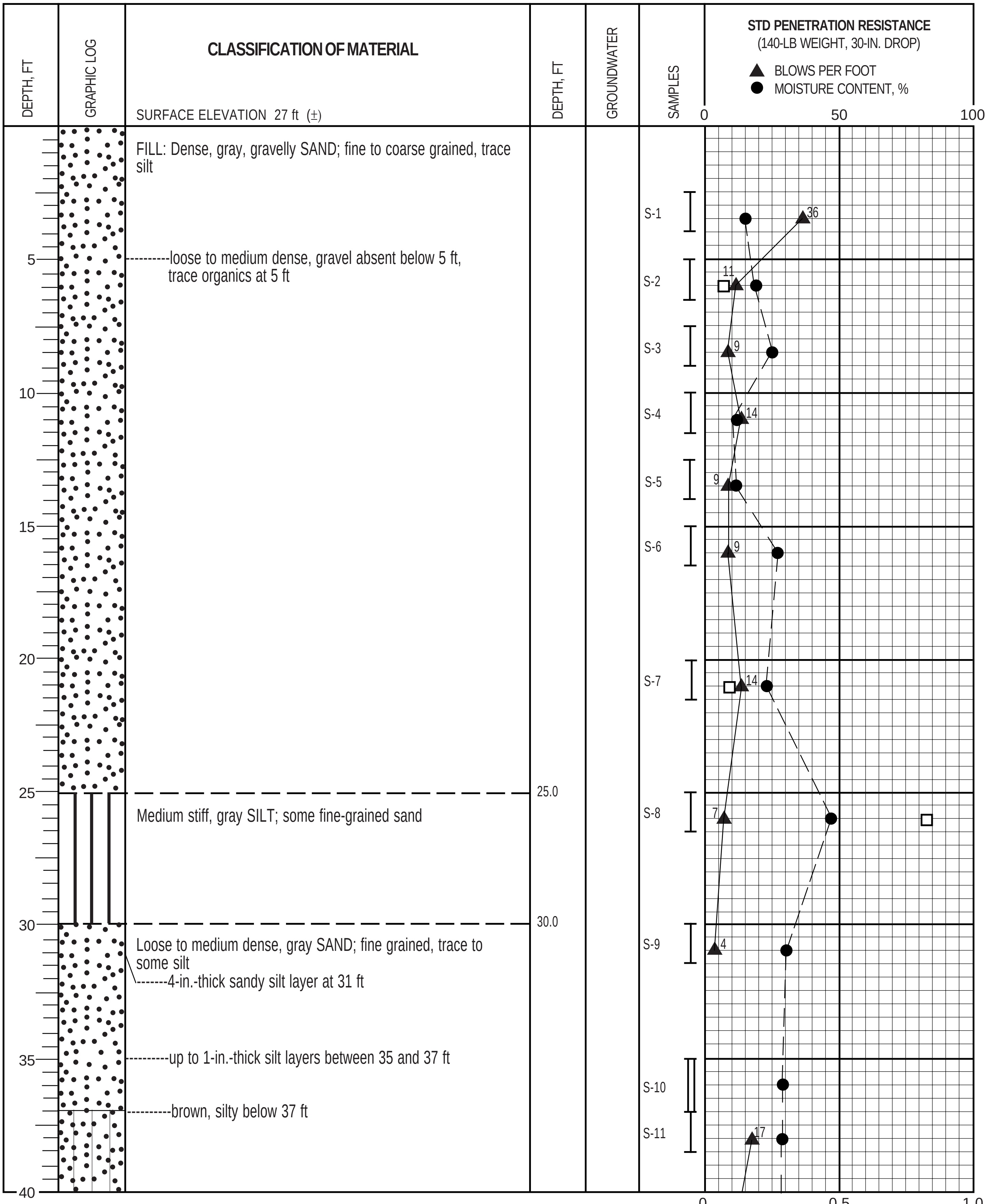
2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
NX CORE RUN
SLOTTED PVC PIPE
Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF
□ PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-23 (cont.)

DEC. 2013 JOB. NO. W1114 FIG. 23A



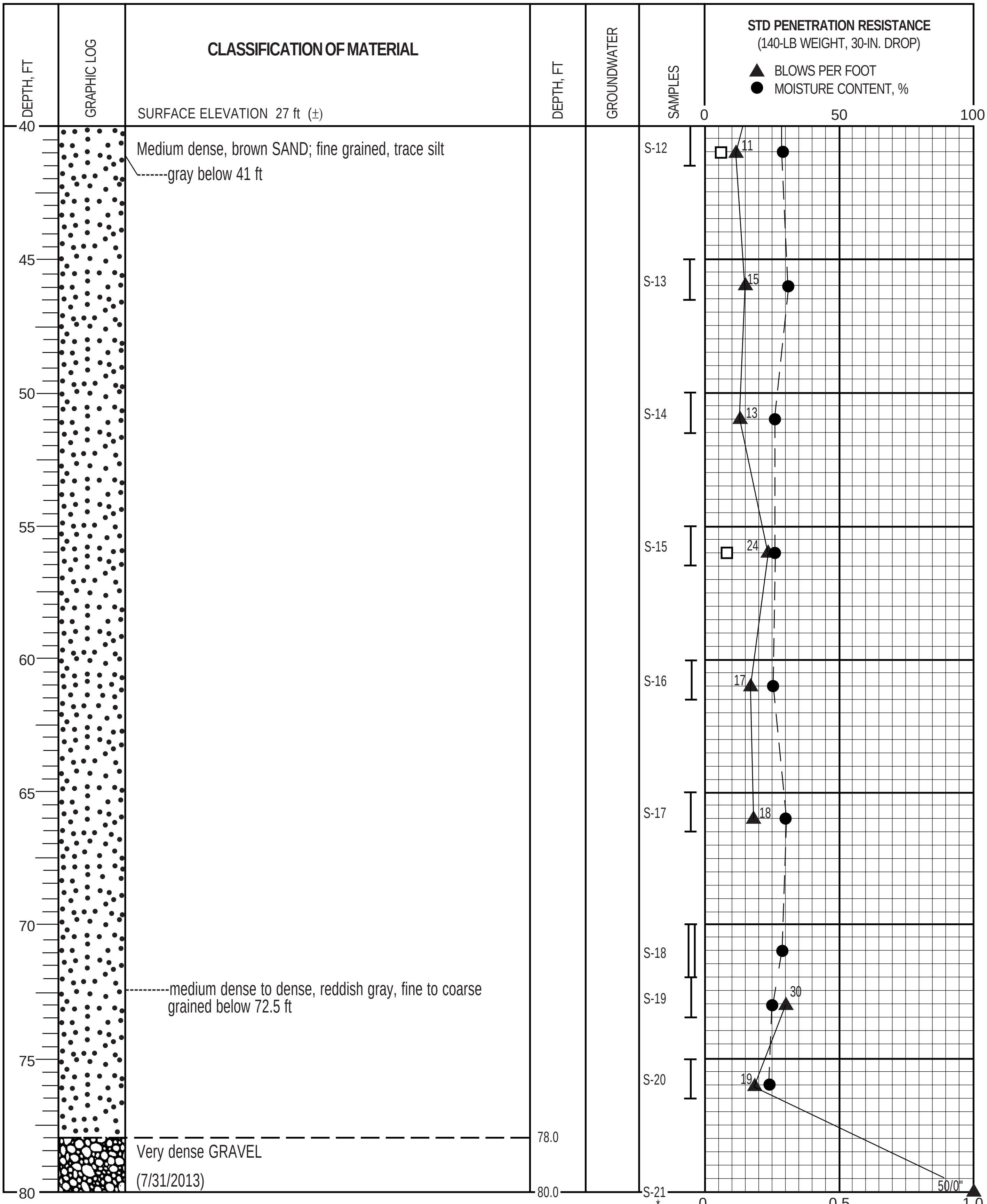
2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
NX CORE RUN
SLOTTED PVC PIPE
Water Level (date)

TORVANE SHEAR STRENGTH, TSF
PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-24

DEC. 2013 JOB. NO. W1114 FIG. 24A



2-IN.-OD SPLIT-SPOON SAMPLER

3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

NX CORE RUN

SLOTTED PVC PIPE

Water Level (date)

TORVANE SHEAR STRENGTH, TSF

PERCENT PASSING NO. 200 SIEVE (WASHED)

* NO RECOVERY

Liquid Limit

Moisture Content

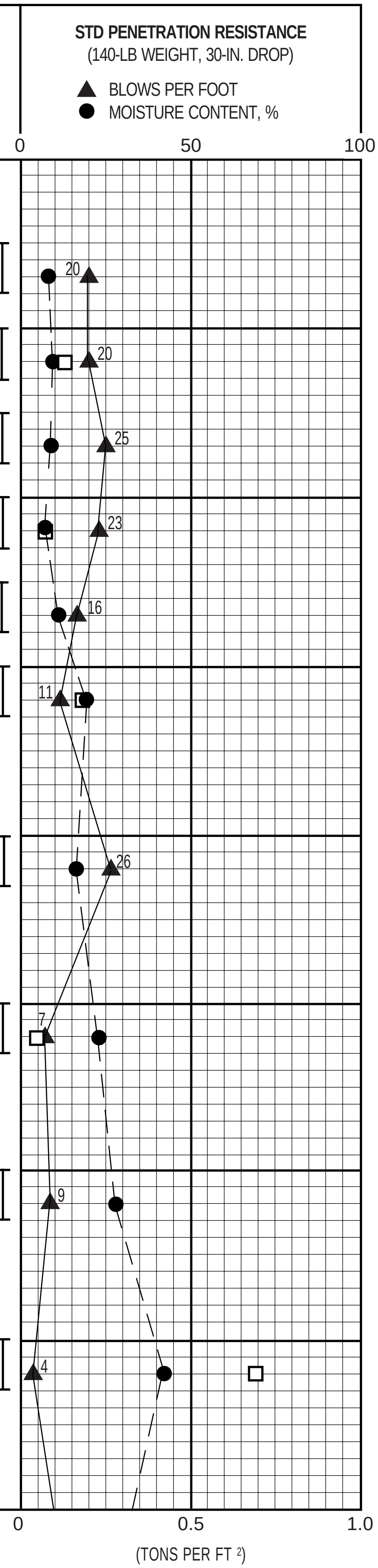
Plastic Limit

GRI

BORING B-24 (cont.)

DEC. 2013 JOB. NO. W1114 FIG. 24A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER	SAMPLES
		SURFACE ELEVATION 27 ft (±)			
		Asphaltic-concrete PAVEMENT (12 in.) over crushed rock BASE COURSE (24 in.)	3.0		
5		FILL: Medium dense, light brown SAND; fine grained, trace rounded gravel, trace silt -------gray and brown below 5 ft ---------1-in.-thick silt layer at 6.5 ft			S-1 S-2 S-3 S-4 S-5 S-6 S-7
25		Loose, dark gray SAND; fine grained, trace silt	25.0		S-8 S-9
35		Medium stiff, dark gray, sandy SILT; fine-grained sand	35.0		S-10
		Loose, dark gray SAND; fine to medium grained, trace to some silt	36.0		
40					



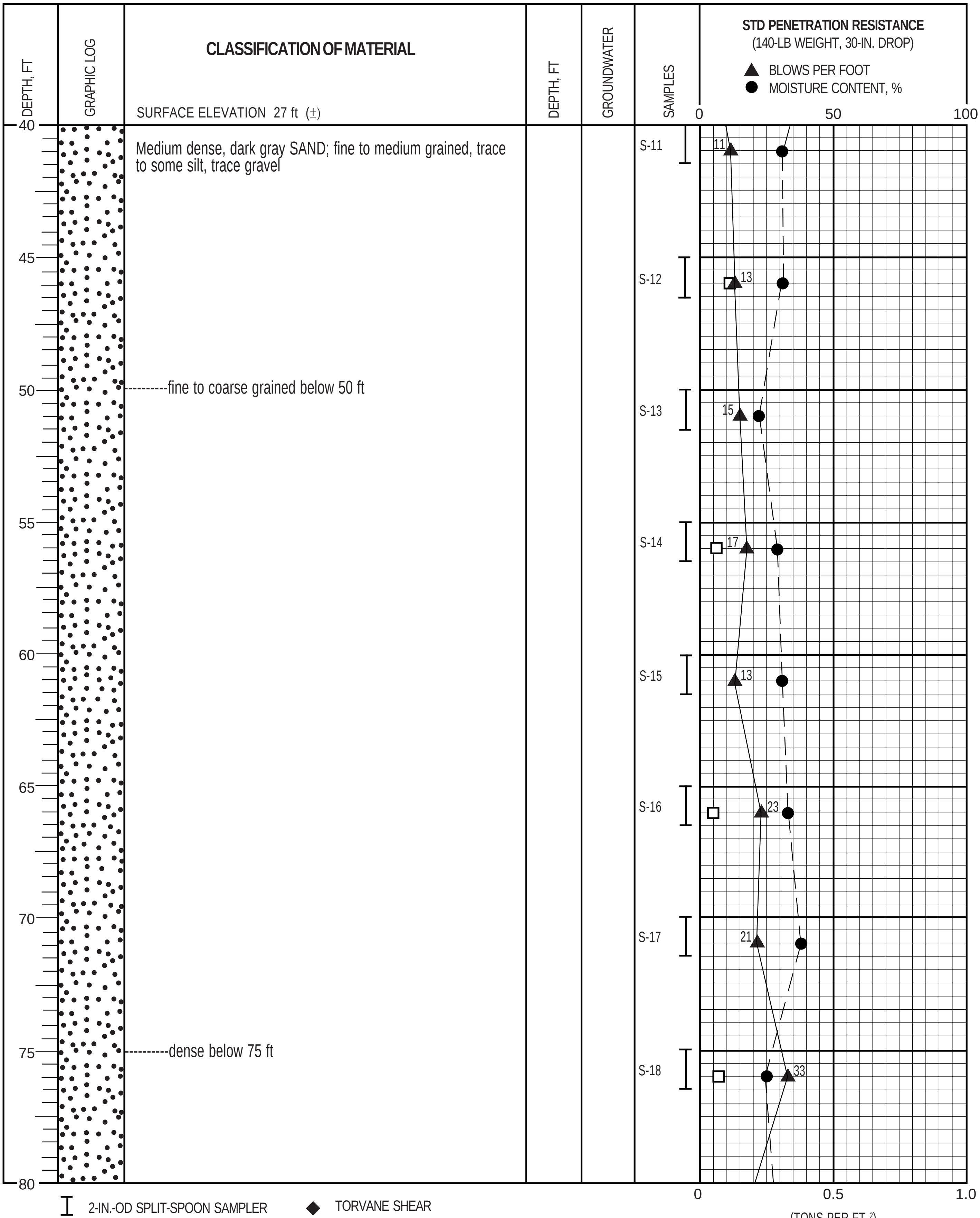
2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
NX CORE RUN
SLOTTED PVC PIPE
Water Level (date)

TORVANE SHEAR STRENGTH, TSF
PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-25

DEC. 2013 JOB. NO. W1114 FIG. 25A



2-IN.-OD SPLIT-SPOON SAMPLER

3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

NX CORE RUN

SLOTTED PVC PIPE

Water Level (date)

TORVANE SHEAR STRENGTH, TSF

PERCENT PASSING NO. 200 SIEVE (WASHED)

* NO RECOVERY

Liquid Limit

Moisture Content

Plastic Limit

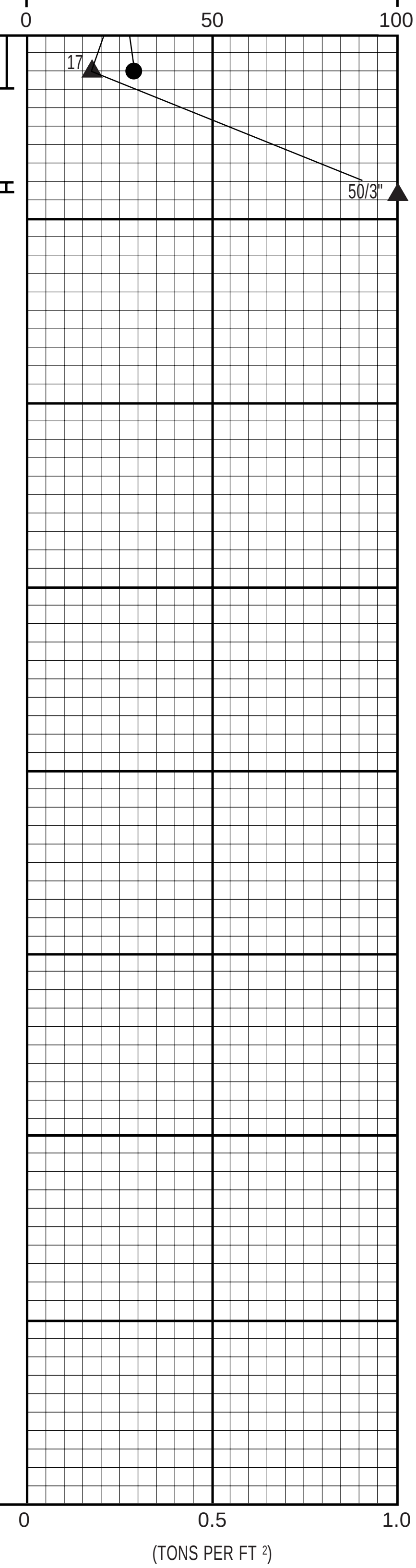
GRI

BORING B-25 (cont.)

DEC. 2013 JOB. NO. W1114 FIG. 25A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER	SAMPLES
		SURFACE ELEVATION 27 ft (±)			
80		Medium dense, dark gray SAND; fine to coarse grained, trace silt			S-19
		Very dense GRAVEL; scattered cobbles	82.0		
			84.3		S-20
85		(7/30/2013)			

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)
▲ BLOWS PER FOOT
● MOISTURE CONTENT, %



2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
NX CORE RUN
SLOTTED PVC PIPE
Water Level (date)
◆ TORVANE SHEAR STRENGTH, TSF
□ PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-25 (cont.)

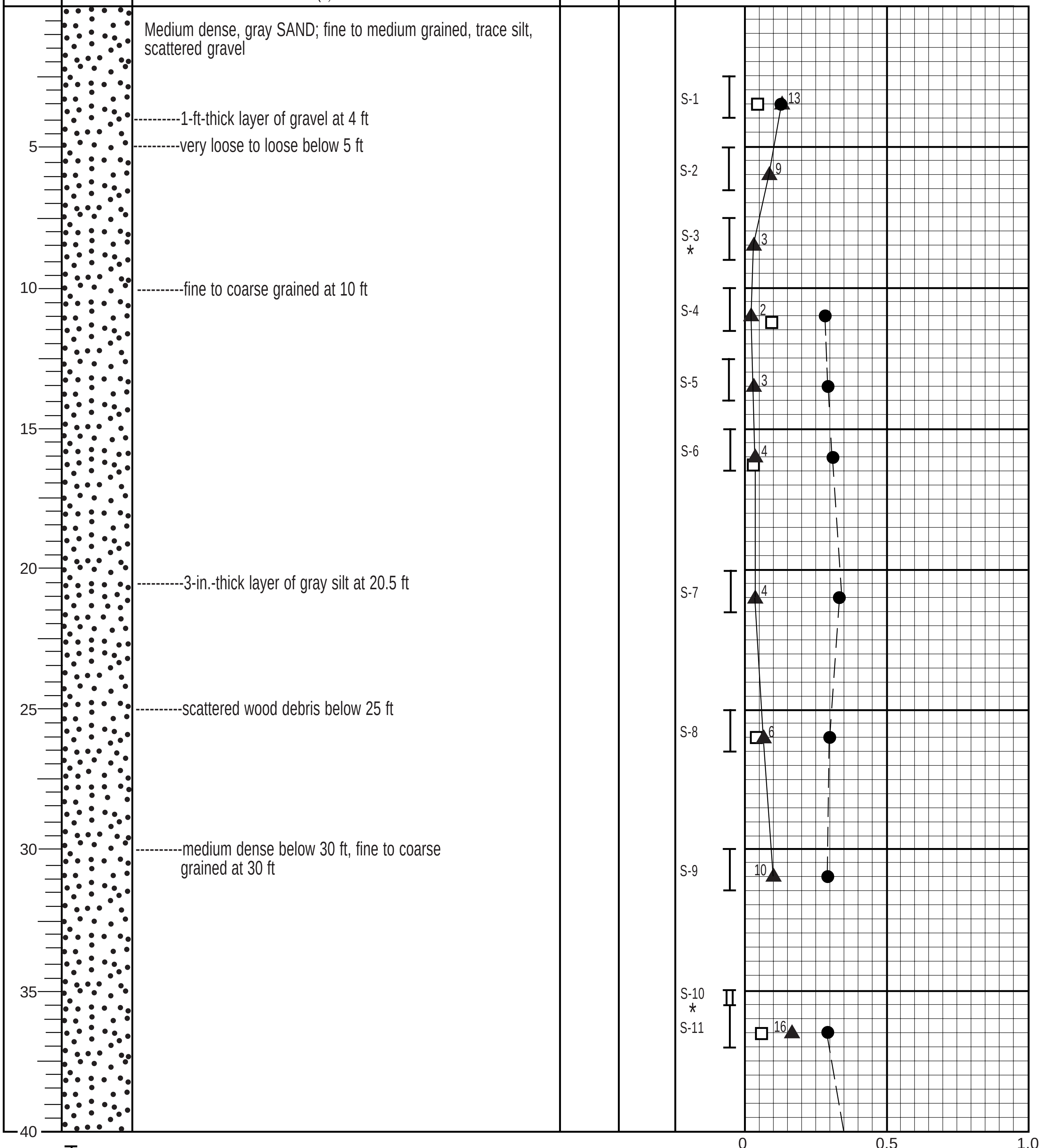
DEC. 2013 JOB. NO. W1114 FIG. 25A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER	SAMPLES	STD PENETRATION RESISTANCE (140-LB WEIGHT, 30-IN. DROP)

SURFACE ELEVATION 7 ft (±)

Medium dense, gray SAND; fine to medium grained, trace silt, scattered gravel

- ---------1-ft-thick layer of gravel at 4 ft
- ---------very loose to loose below 5 ft
- ---------fine to coarse grained at 10 ft
- ---------3-in.-thick layer of gray silt at 20.5 ft
- ---------scattered wood debris below 25 ft
- ---------medium dense below 30 ft, fine to coarse grained at 30 ft



2-IN.-OD SPLIT-SPOON SAMPLER

3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

NX CORE RUN

SLOTTED PVC PIPE

Water Level (date)

TORVANE SHEAR STRENGTH, TSF

PERCENT PASSING NO. 200 SIEVE (WASHED)

* NO RECOVERY

Liquid Limit

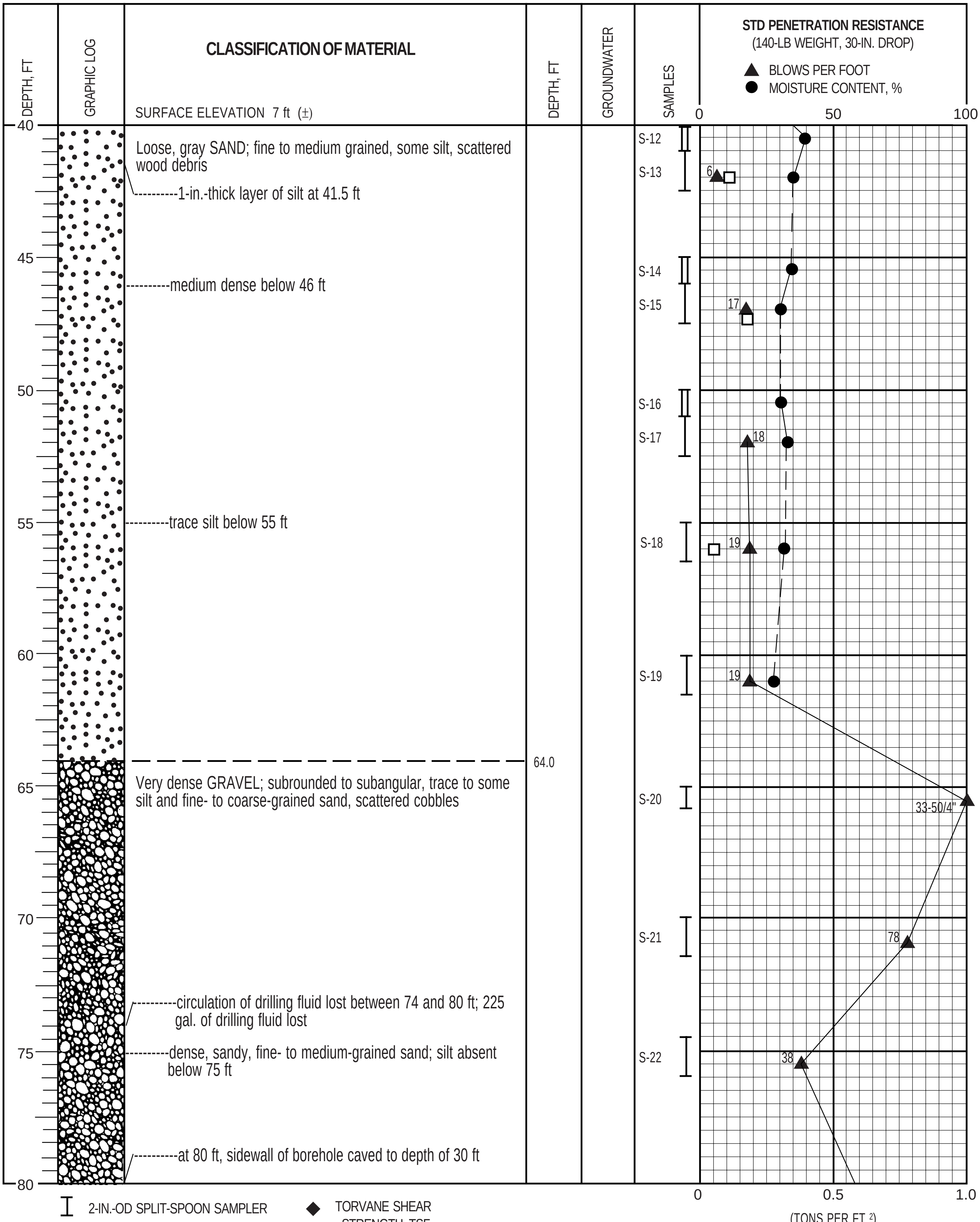
Moisture Content

Plastic Limit

GRI

BORING B-26

DEC. 2013 JOB. NO. W1114 FIG. 26A



2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
NX CORE RUN
SLOTTED PVC PIPE
Water Level (date)

TORVANE SHEAR STRENGTH, TSF
PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

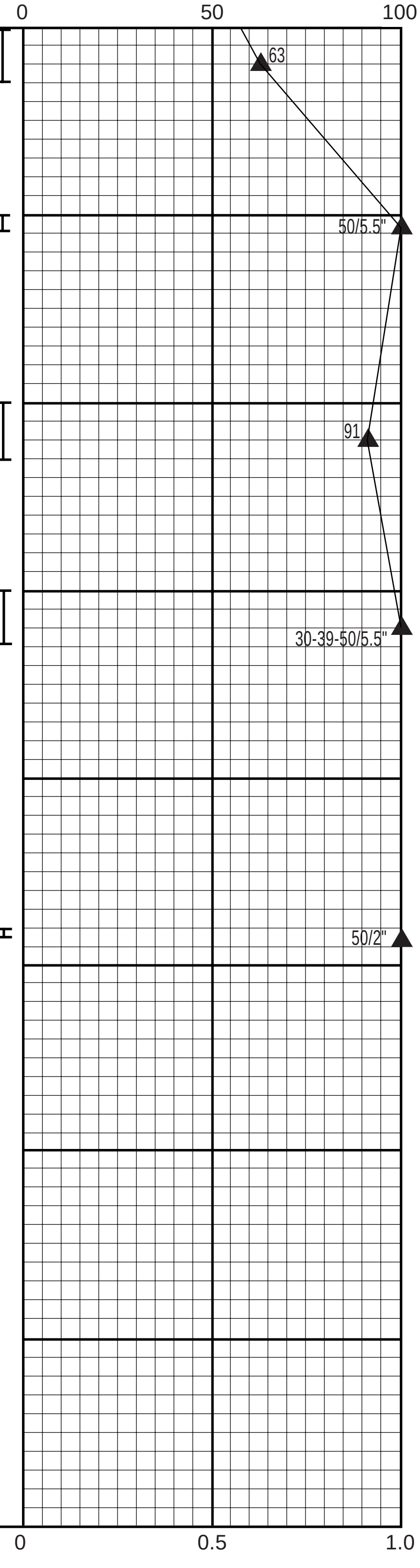
GRI

BORING B-26 (cont.)

DEC. 2013 JOB. NO. W1114 FIG. 26A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL SURFACE ELEVATION 7 ft (±)	DEPTH, FT	GROUNDWATER	SAMPLES
80		Very dense GRAVEL; subrounded to subangular, trace to some fine- to coarse-grained sand, trace silt, scattered cobbles			S-23
85					S-24
90					S-25
95					S-26
100					
105		(10/29/2013) 5.5-in.-diameter casing installed to 20 ft as part of drilling permit requirements	104.2		S-27

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)
▲ BLOWS PER FOOT
● MOISTURE CONTENT, %

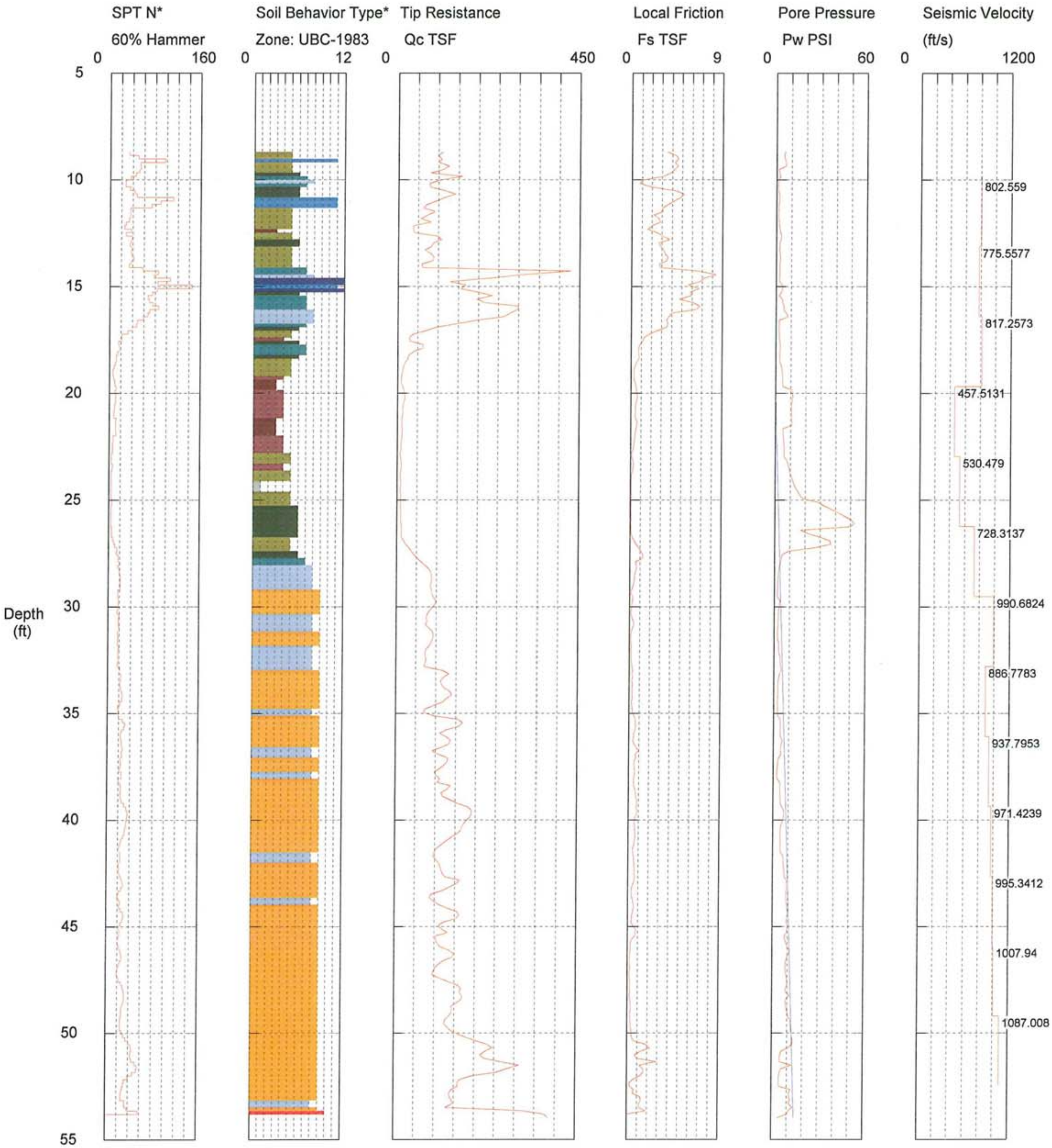


I 2-IN.-OD SPLIT-SPOON SAMPLER
II 3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
▌ NX CORE RUN
SLOTTED PVC PIPE
▼ Water Level (date)
◆ TORVANE SHEAR STRENGTH, TSF
□ PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-26 (cont.)

DEC. 2013 | JOB. NO. W1114 | FIG. 26A



Maximum Depth = 53.97 feet

Depth Increment = 0.164 feet

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

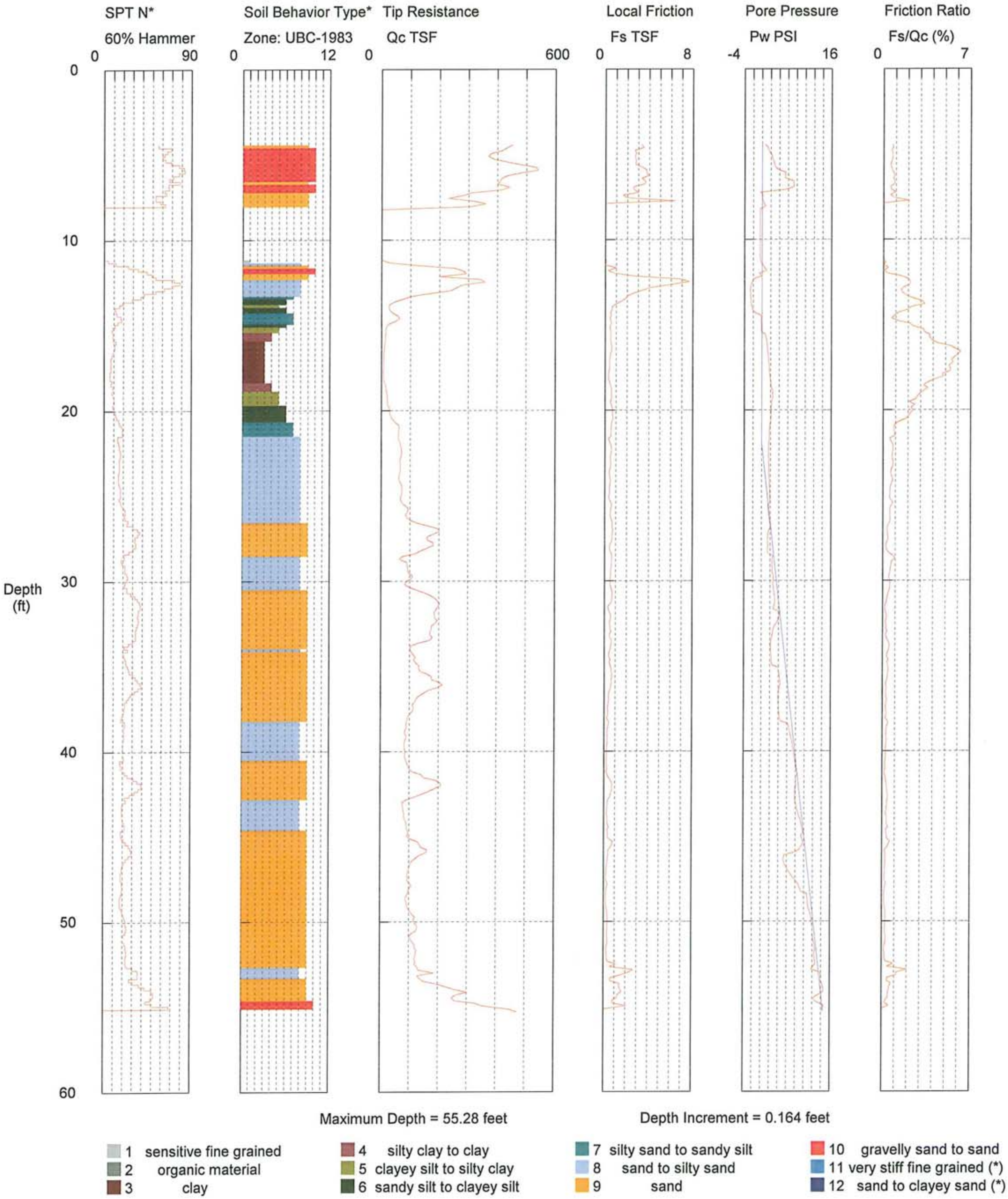
SURFACE ELEVATION = 28.5 FT

GRI

CONE PENETRATION TEST CPT-1

(WITH SEISMIC VELOCITY)

DEC. 2013 JOB NO. W1114 FIG. 27A

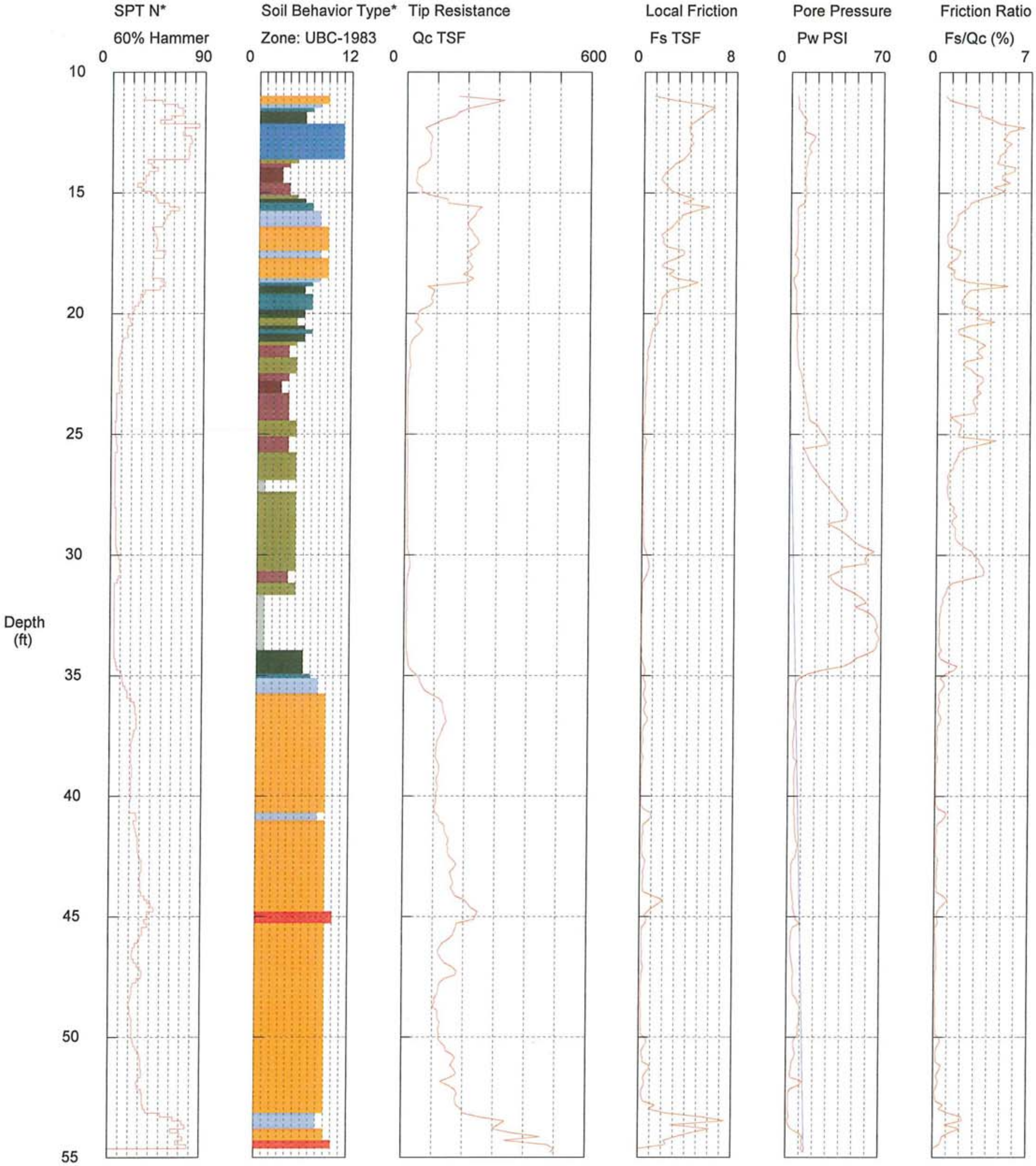


SURFACE ELEVATION = 29.9 FT

GRI

CONE PENETRATION TEST CPT-2

DEC. 2013 JOB NO. W1114 FIG. 28A



Maximum Depth = 54.79 feet

Depth Increment = 0.164 feet

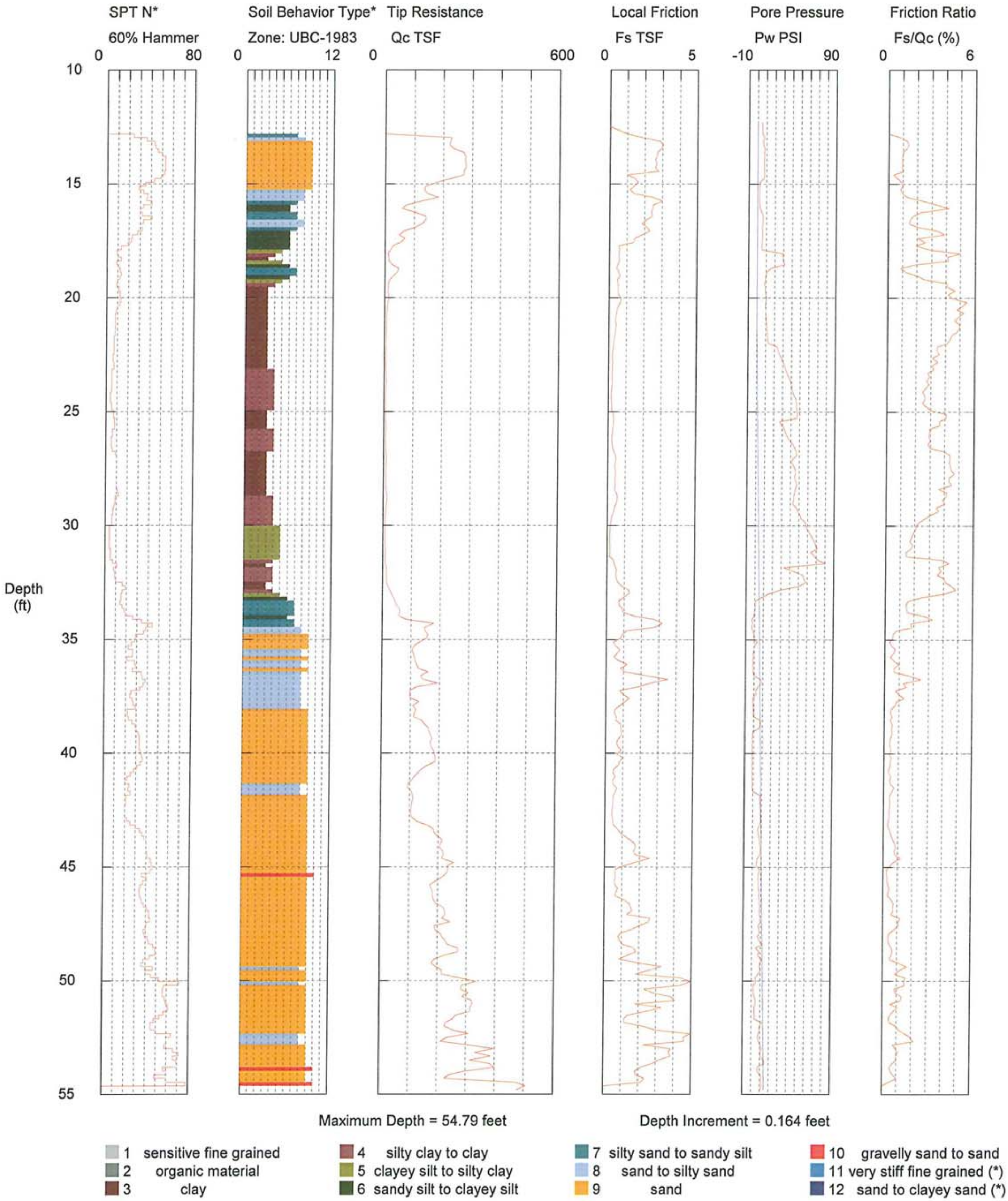
1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

SURFACE ELEVATION = 28.5 FT

GRI

CONE PENETRATION TEST CPT-3

DEC. 2013 JOB NO. W1114 FIG. 29A

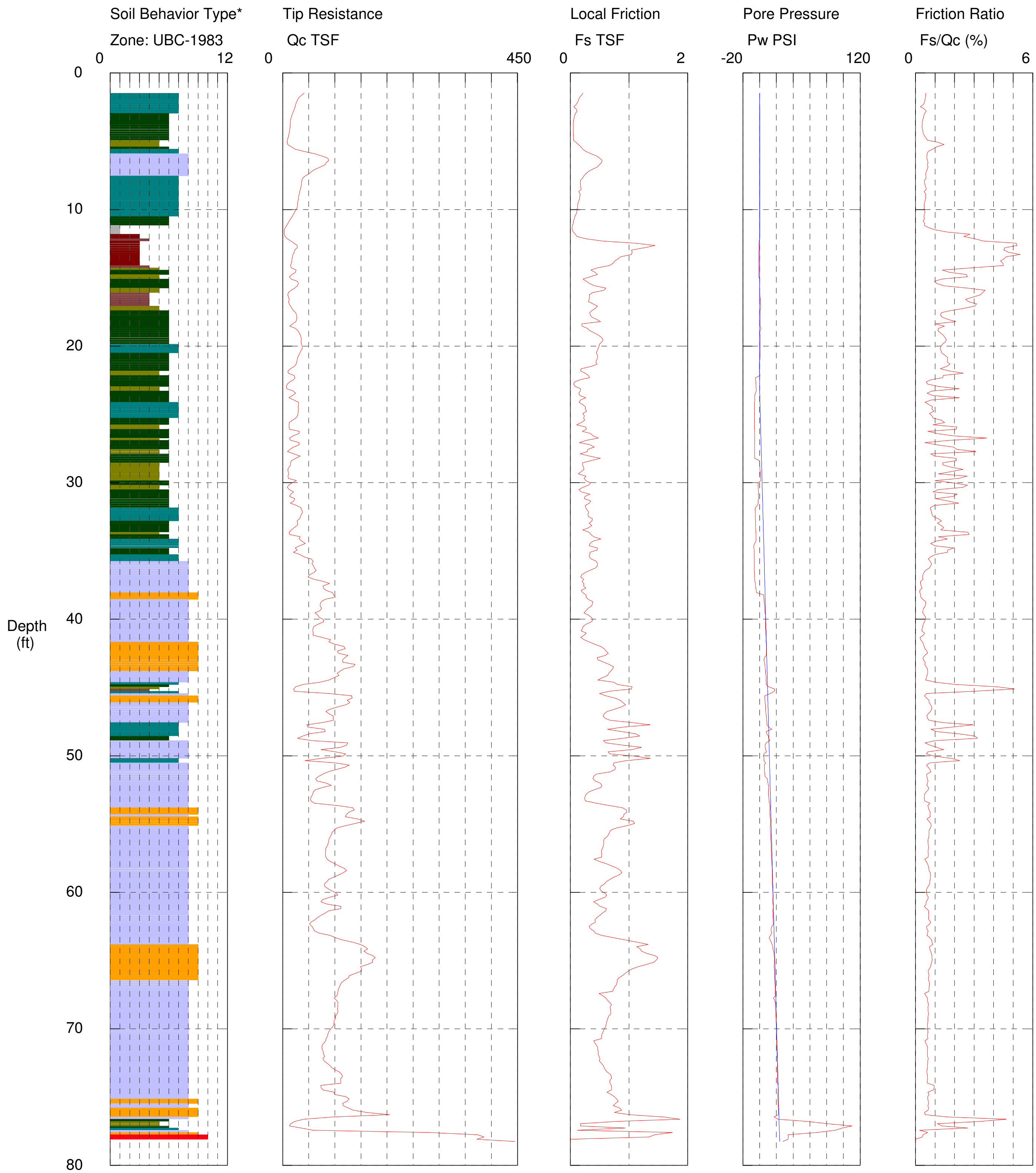


SURFACE ELEVATION = 26.8

GRI

CONE PENETRATION TEST CPT-4

DEC. 2013 JOB NO. W1114 FIG. 30A



Maximum Depth = 78.25 feet

Depth Increment = 0.164 feet

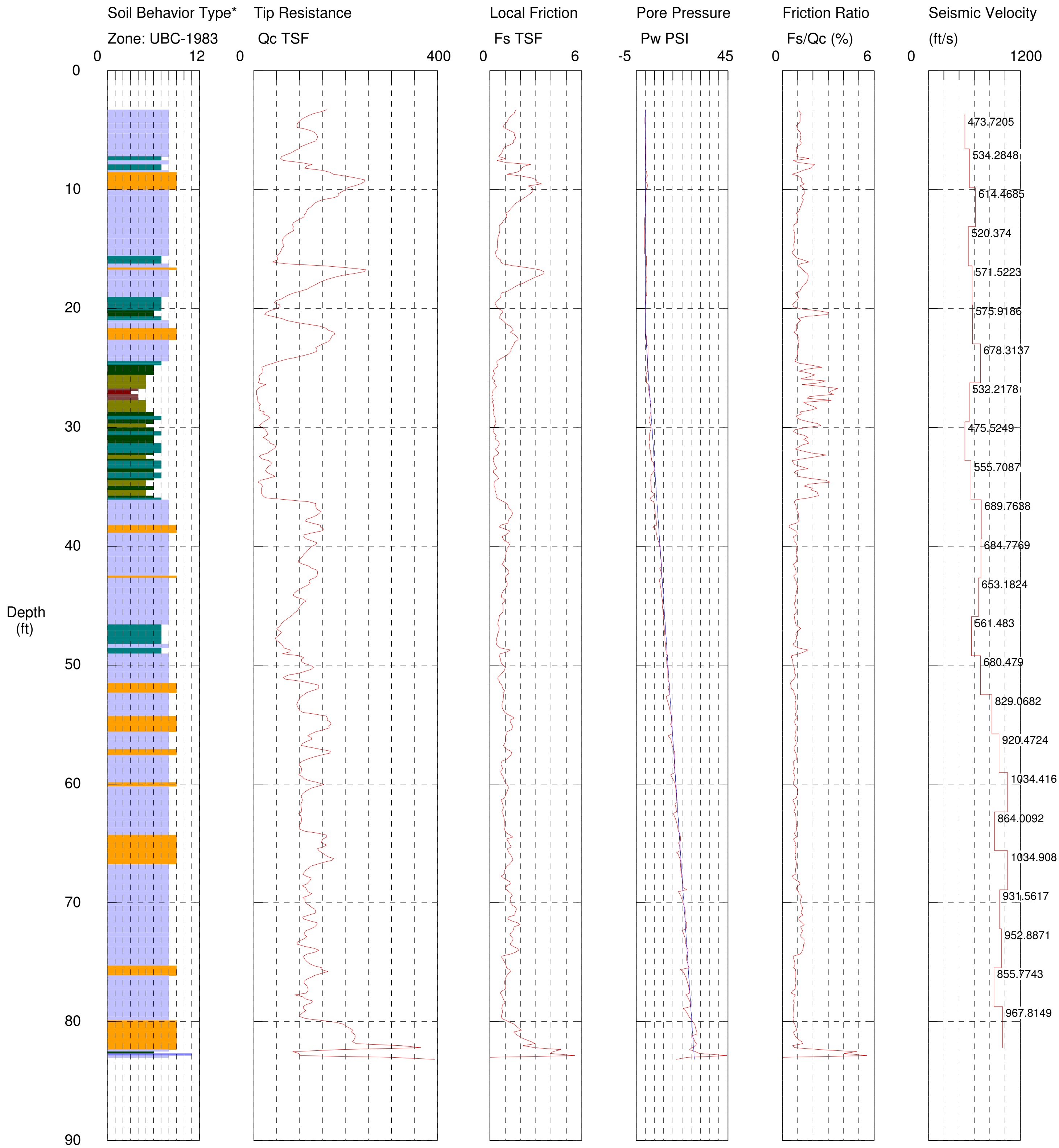
1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

SURFACE ELEVATION = 31 FT

GRI

CONE PENETRATION TEST CPT-5

DEC. 2013 JOB NO. W1114 FIG. 31A



Maximum Depth = 83.17 feet

Depth Increment = 0.164 feet

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

SURFACE ELEVATION = 27 FT

GRI

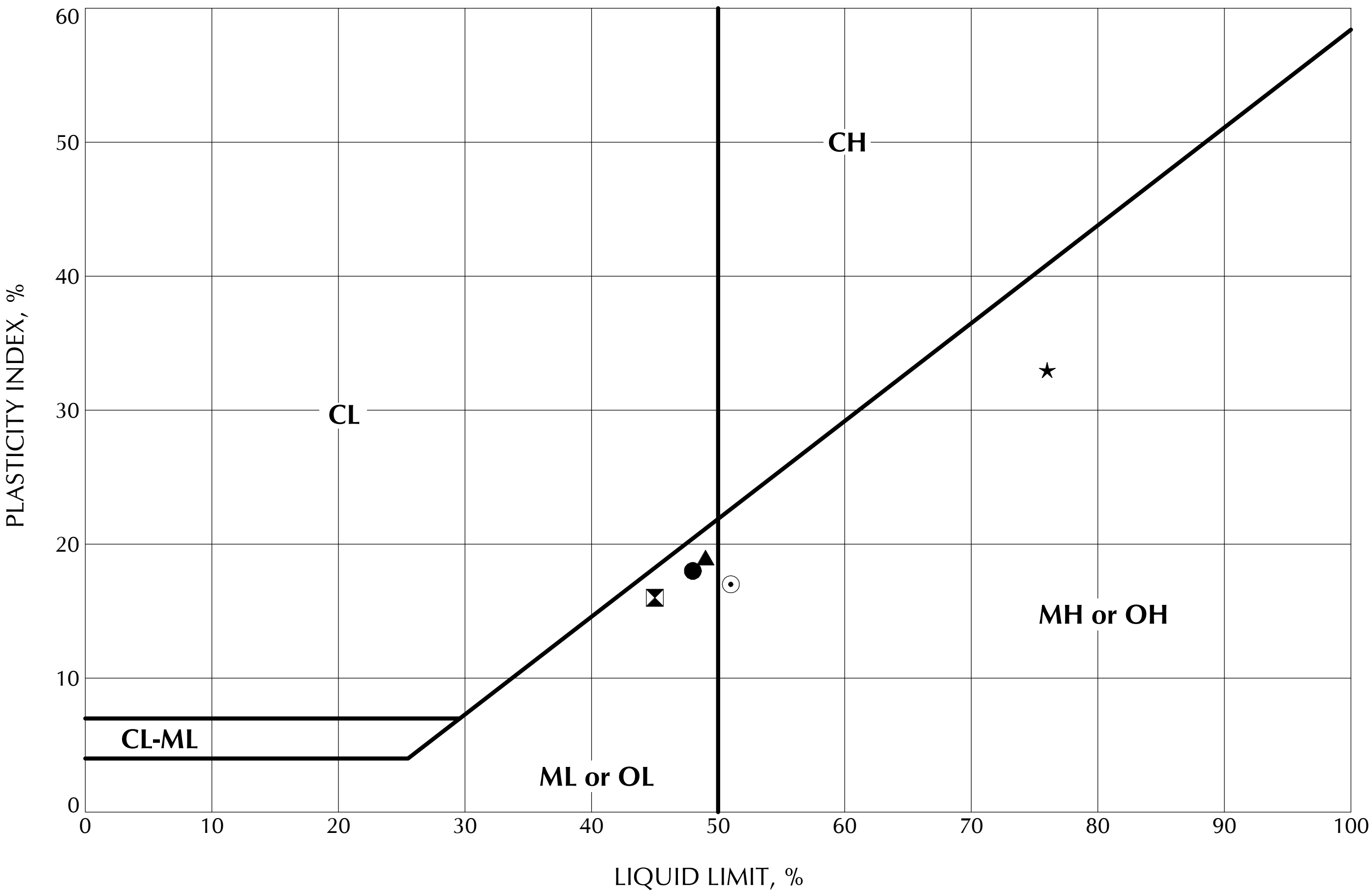
CONE PENETRATION TEST CPT-6

(WITH SEISMIC VELOCITY)

DEC. 2013 JOB NO. W1114 FIG. 32A

GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE-GRAINED SOIL GROUPS
OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
ML	INORGANIC CLAYEY SILTS TO VERY FINE SANDS OF SLIGHT PLASTICITY
CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY

GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE-GRAINED SOIL GROUPS
OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
MH	INORGANIC SILTS AND CLAYEY SILT
CH	INORGANIC CLAYS OF HIGH PLASTICITY



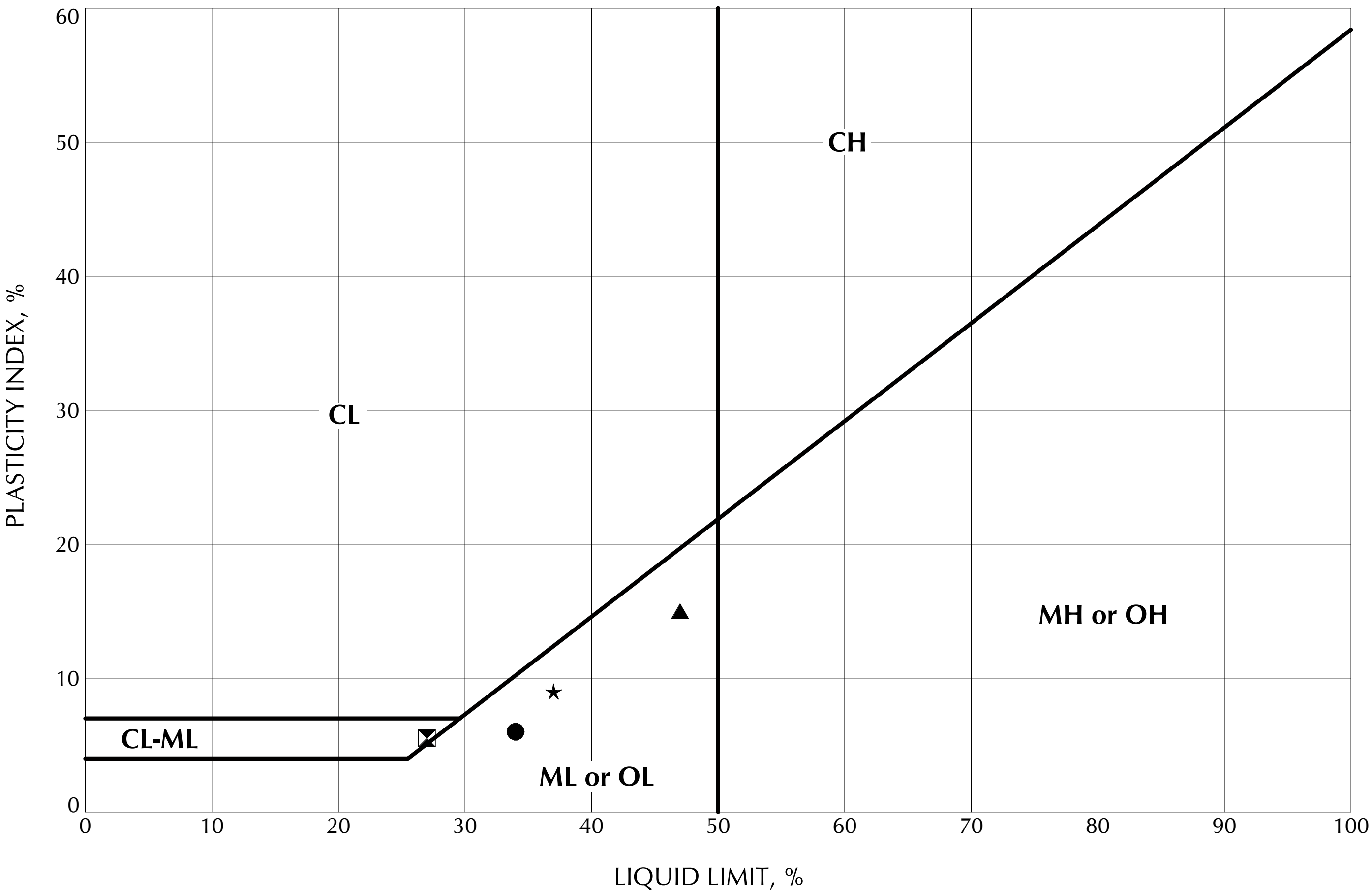
	Location	Sample	Depth, ft	Classification	LL	PL	PI	MC, %
●	B-01	S-8	21.8	SILT; some clay, trace sand	48	30	18	32
⧗	B-02	S-9	24.0	SILT; some clay, trace to some sand	45	29	16	41
▲	B-05	S-7	16.0	SILT; some clay	49	30	19	40
★	B-07	S-11	31.0	SILT; some clay, trace sand	76	43	33	74
⊙	B-08	S-9	25.3	SILT; trace to some clay, trace sand	51	34	17	43

ATTERBERG-PLASTICITY 5 PER PAGE W1114-LAB.GPJ GRI DATA TEMPLATE.GDT 12/10/13

PLASTICITY CHART

GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE-GRAINED SOIL GROUPS
OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
ML	INORGANIC CLAYEY SILTS TO VERY FINE SANDS OF SLIGHT PLASTICITY
CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY

GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE-GRAINED SOIL GROUPS
OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
MH	INORGANIC SILTS AND CLAYEY SILT
CH	INORGANIC CLAYS OF HIGH PLASTICITY



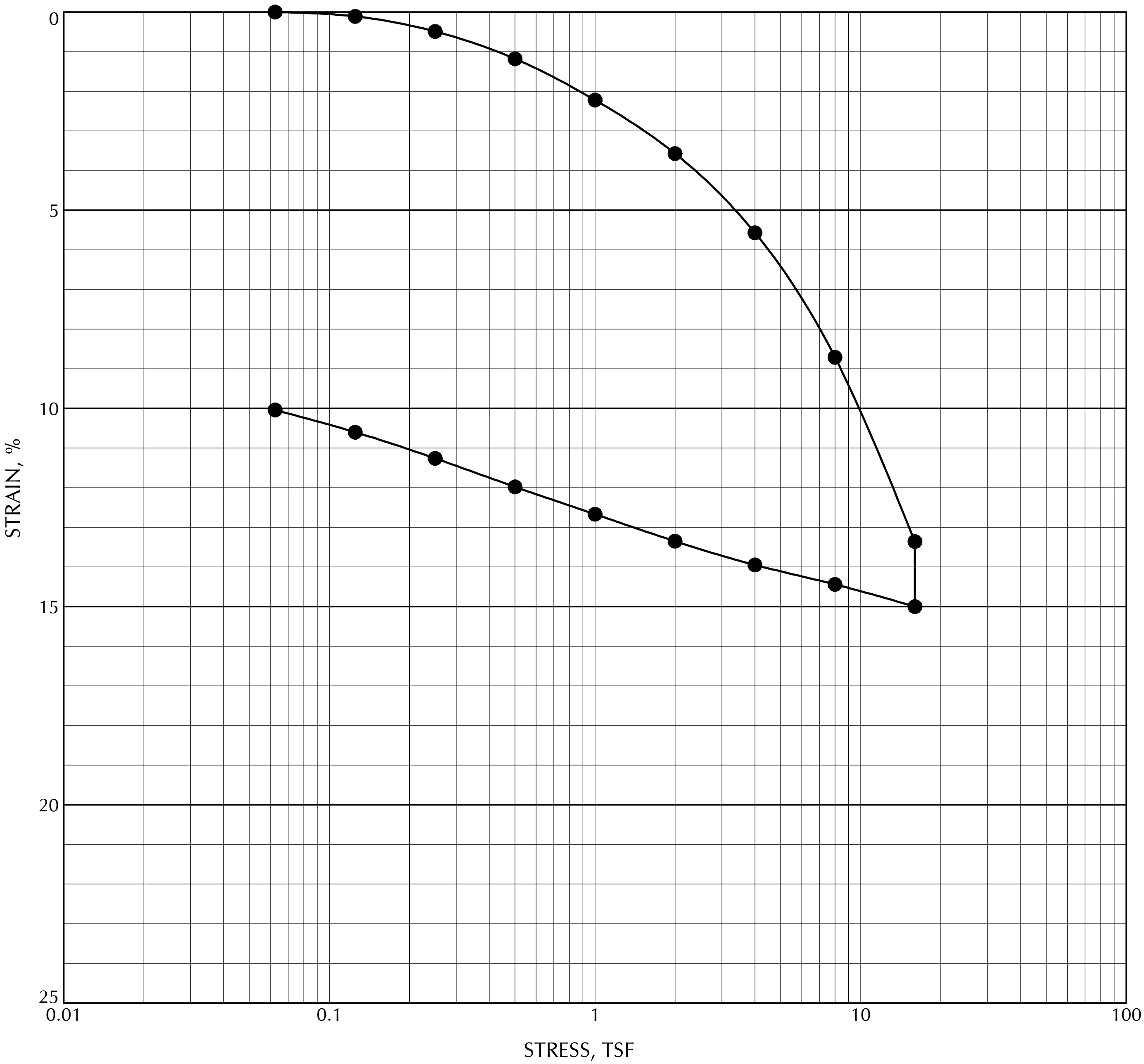
	Location	Sample	Depth, ft	Classification	LL	PL	PI	MC, %
●	B-11	S-8	25.0	SILT; some sand	34	28	6	39
⧗	B-17	S-6	15.0	SILT; some sand	27	22	5	31
▲	B-18	S-5	12.5	SILT; trace to some clay and sand	47	32	15	16
★	B-18	S-9	25.0	SILT; trace to some clay and sand	37	28	9	42
⊙								

GRI

PLASTICITY CHART

DEC. 2013 JOB NO. W1114 FIG. 34A

ATTERBERG-PLASTICITY 5 PER PAGE W1114-LAB.GPJ GRI DATA TEMPLATE.GDT 12/10/13

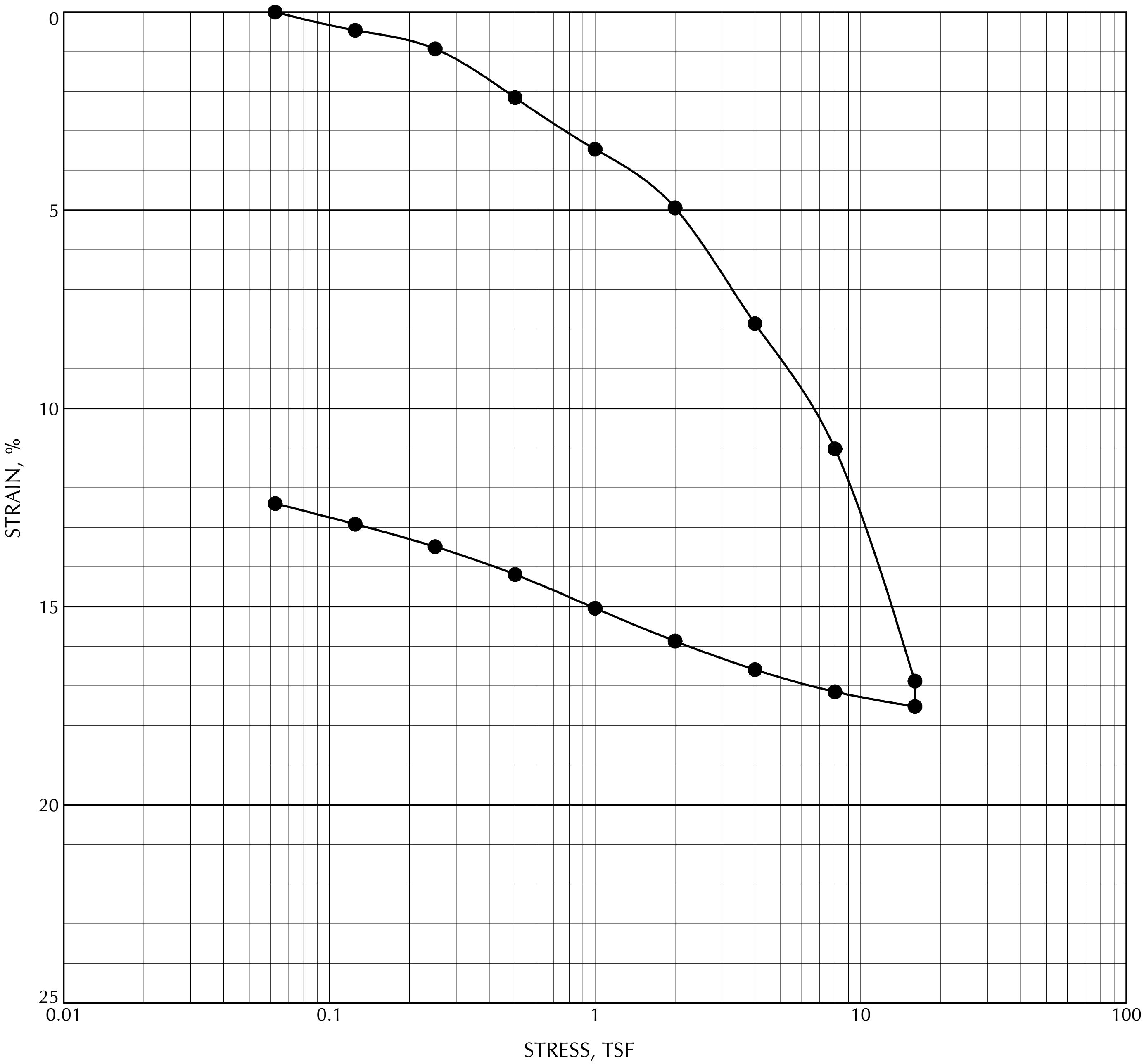


	Location	Sample	Depth, ft	Classification	Initial γ_d, pcf	Initial MC, %
●	B-04	S-7	18.1	SILT; some clay, trace sand and organics	92	30

GRI

CONSOLIDATION TEST

DEC. 2013 JOB NO. W1114 FIG. 35A

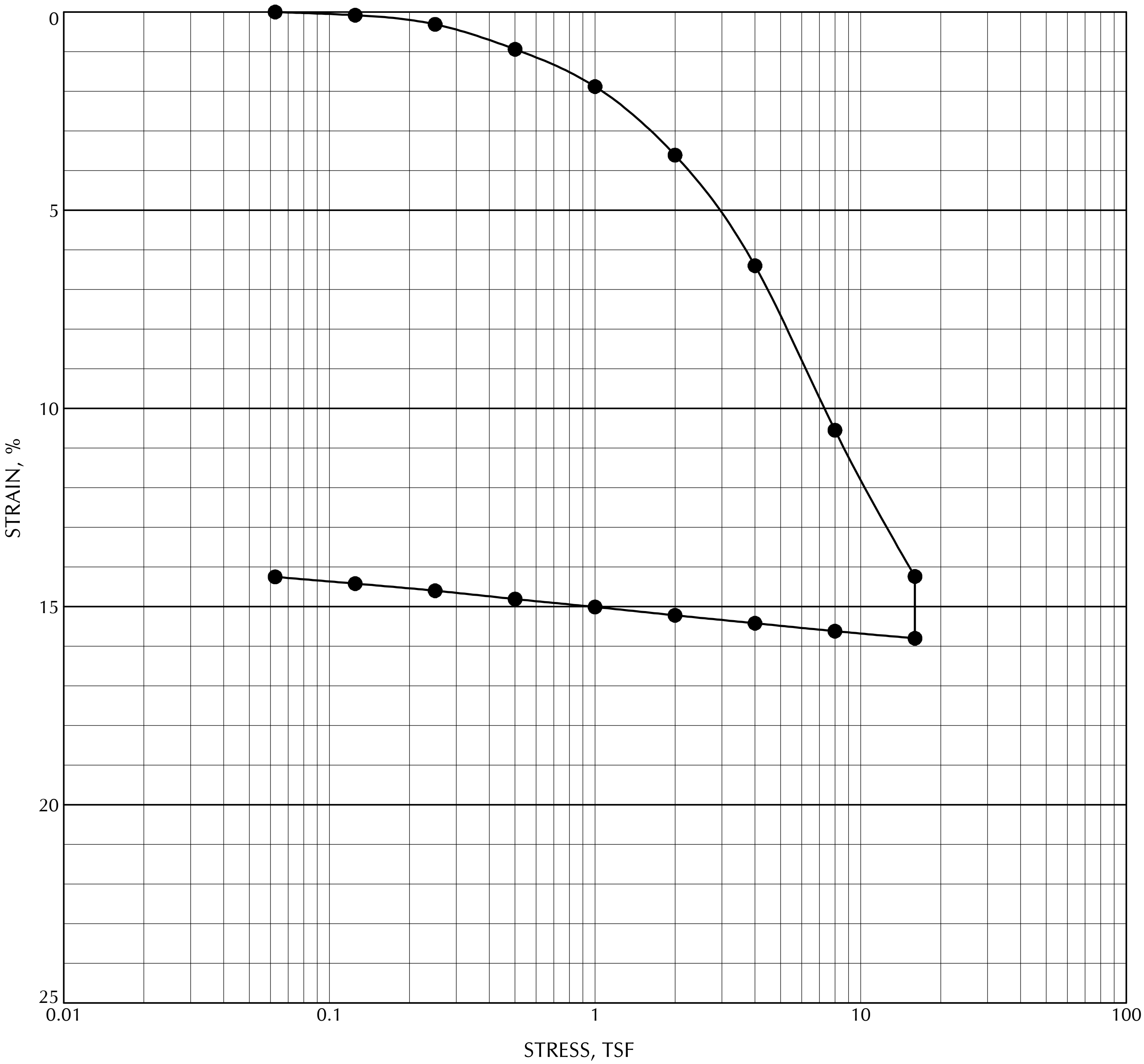


	Location	Sample	Depth, ft	Classification	Initial γ_d, pcf	Initial MC, %
●	B-05	S-6	14.7	SILT; some clay, trace sand	84	36

GRI

CONSOLIDATION TEST

DEC. 2013 JOB NO. W1114 FIG. 36A

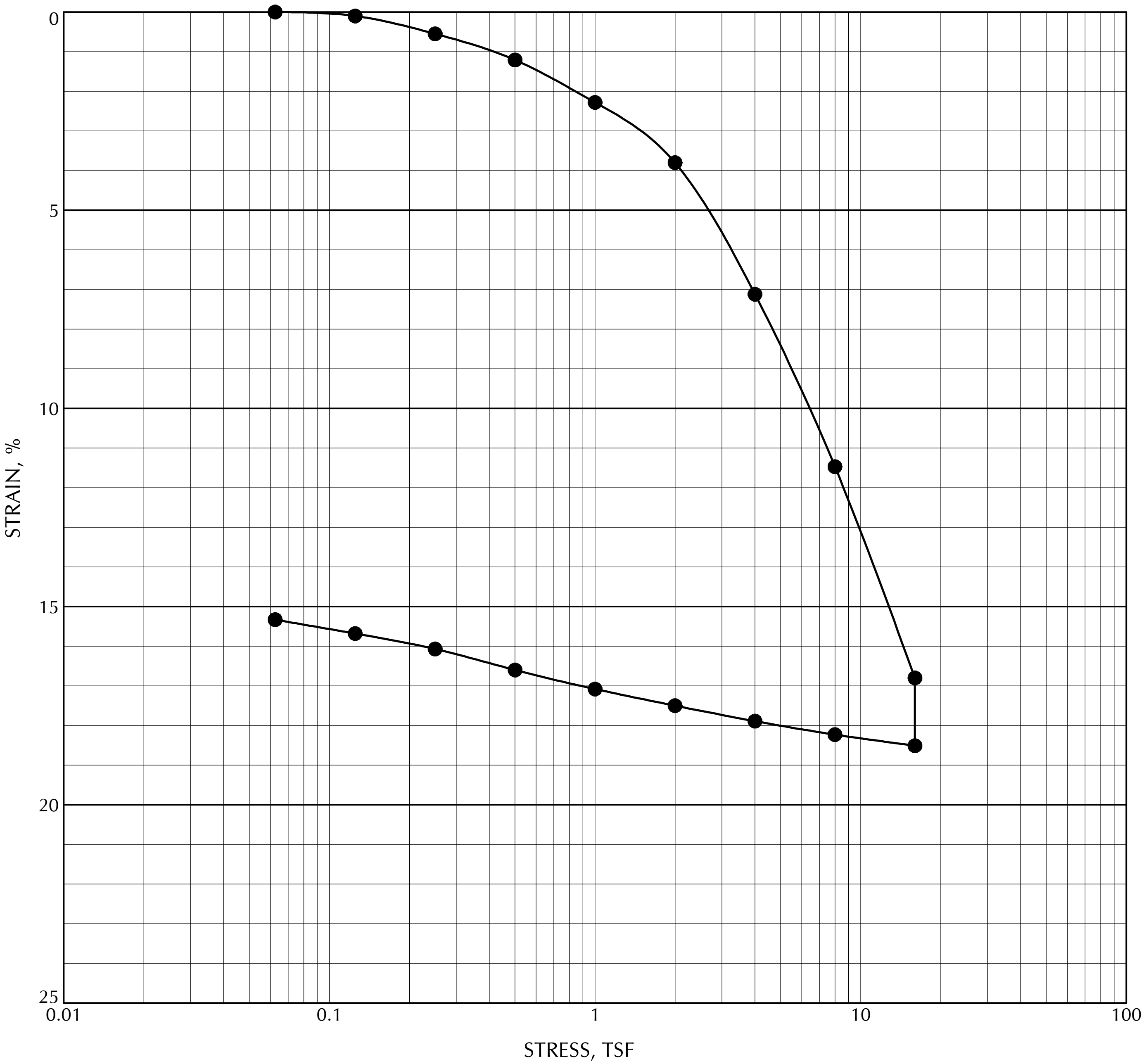


	Location	Sample	Depth, ft	Classification	Initial γ_d, pcf	Initial MC, %
●	B-07	S-12	32.0	Silty SAND	71	49

GRI

CONSOLIDATION TEST

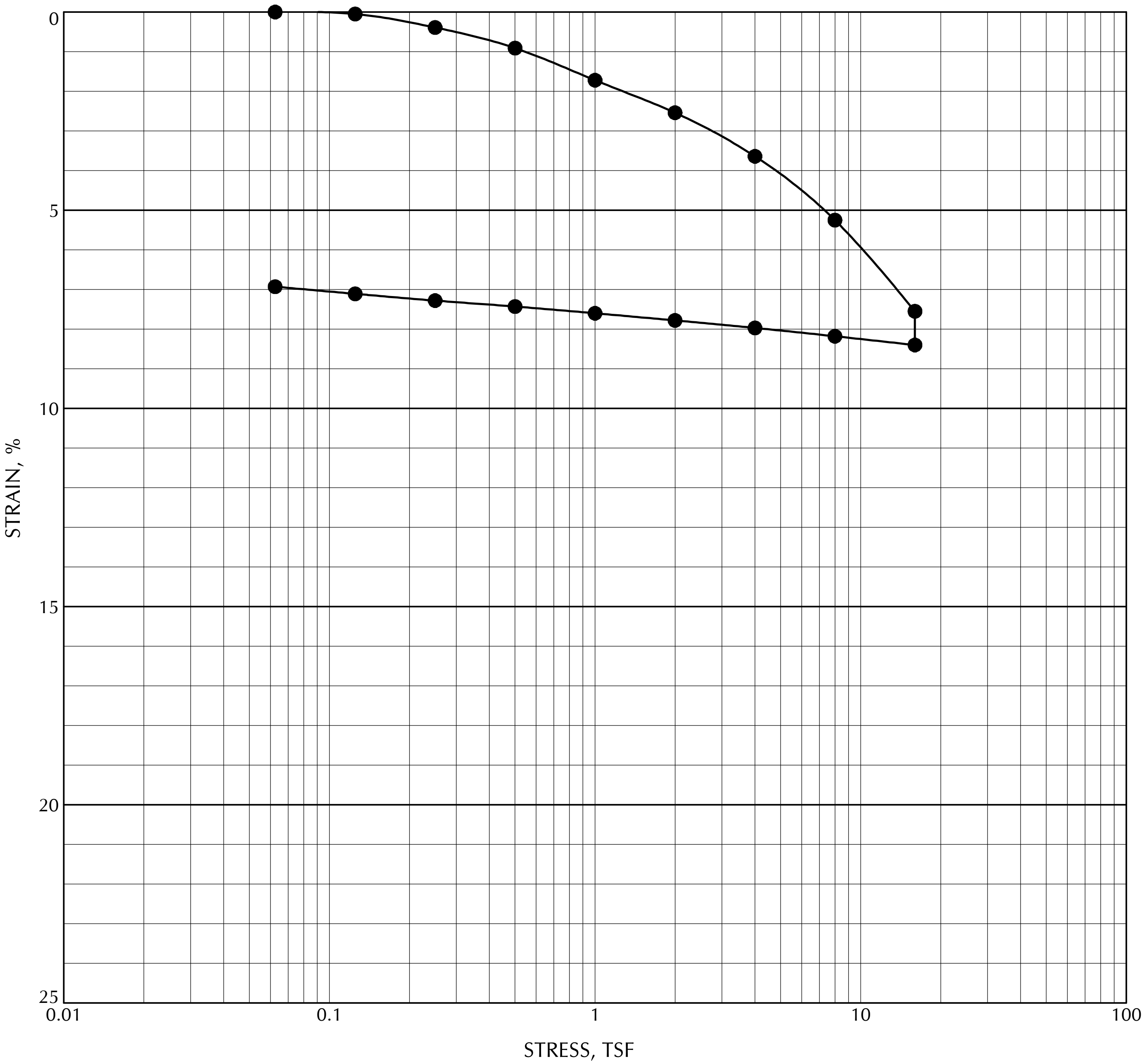
DEC. 2013 JOB NO. W1114 FIG. 37A



	Location	Sample	Depth, ft	Classification	Initial γ_d, pcf	Initial MC, %
●	B-08	S-9	25.3	SILT; trace to some clay, trace sand	80	41

GRI

CONSOLIDATION TEST

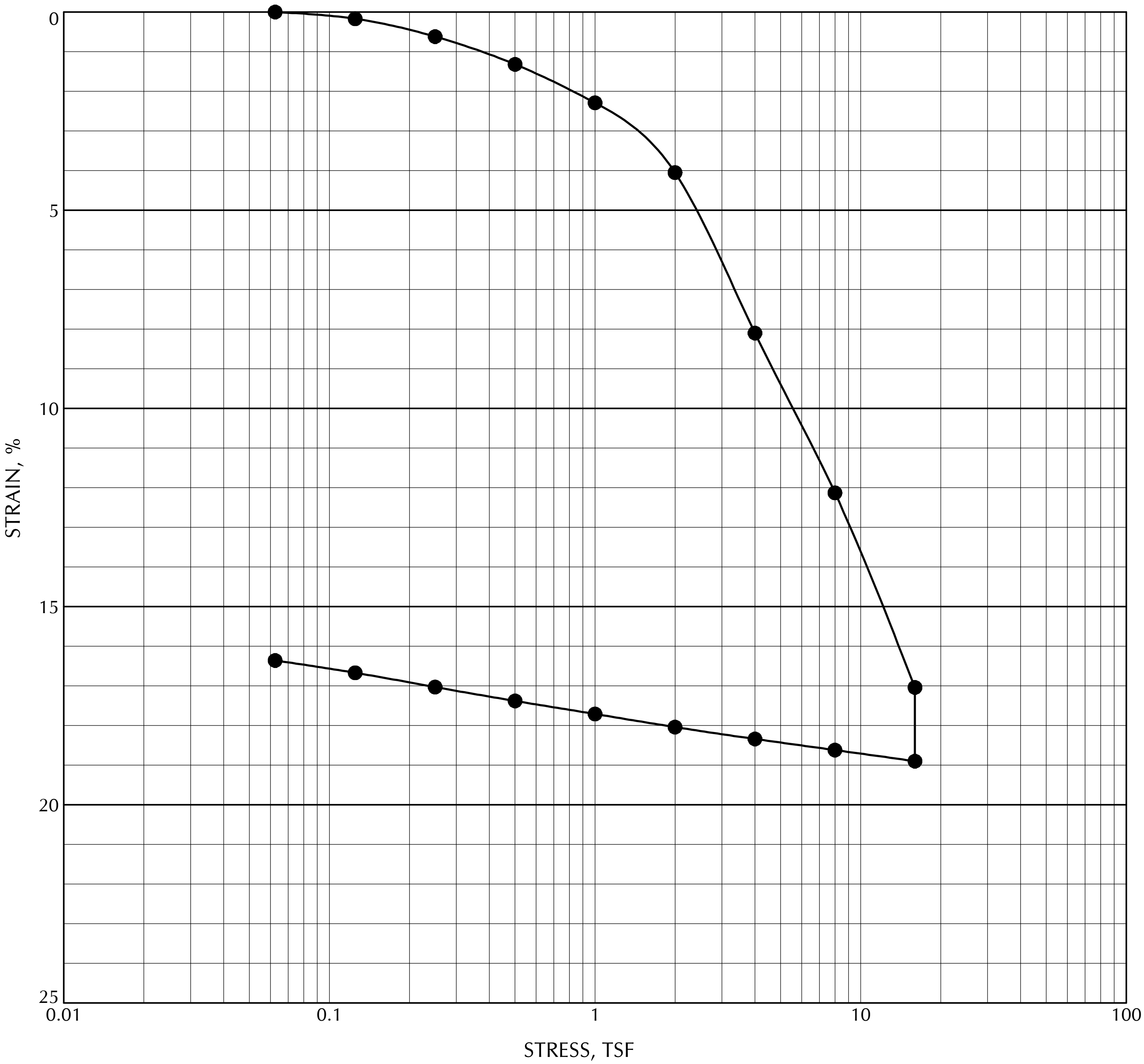


	Location	Sample	Depth, ft	Classification	Initial γ_d, pcf	Initial MC, %
●	B-11	S-4	10.0	Silty SAND	89	28

CONSOL STRAIN GRI-0 TO 25-1 PER PAGE W1114-LAB.GPJ GRI DATA TEMPLATE.GDT 12/10/13

GRI

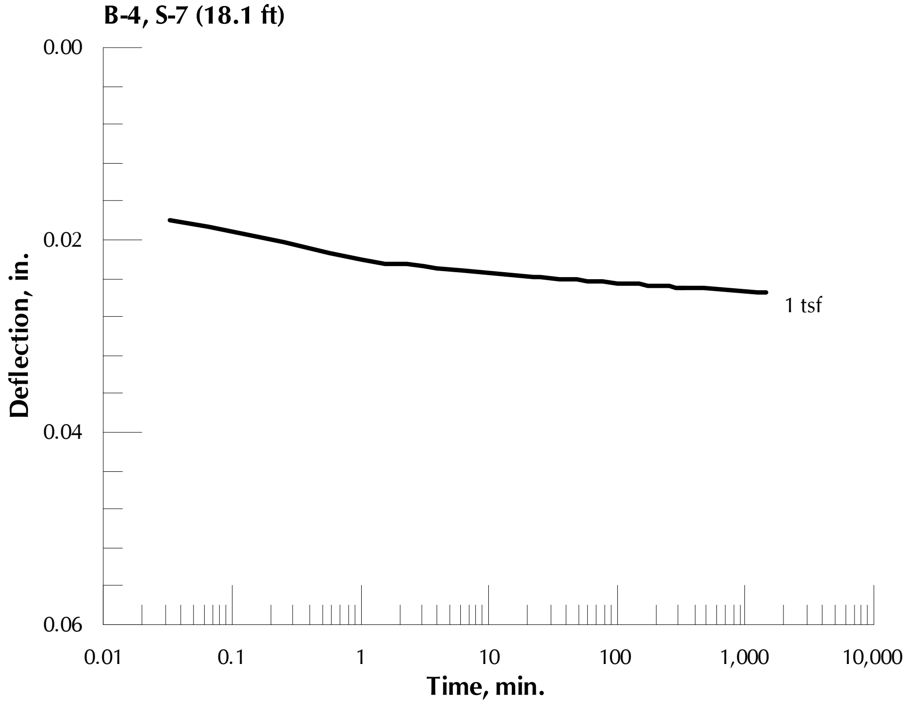
CONSOLIDATION TEST

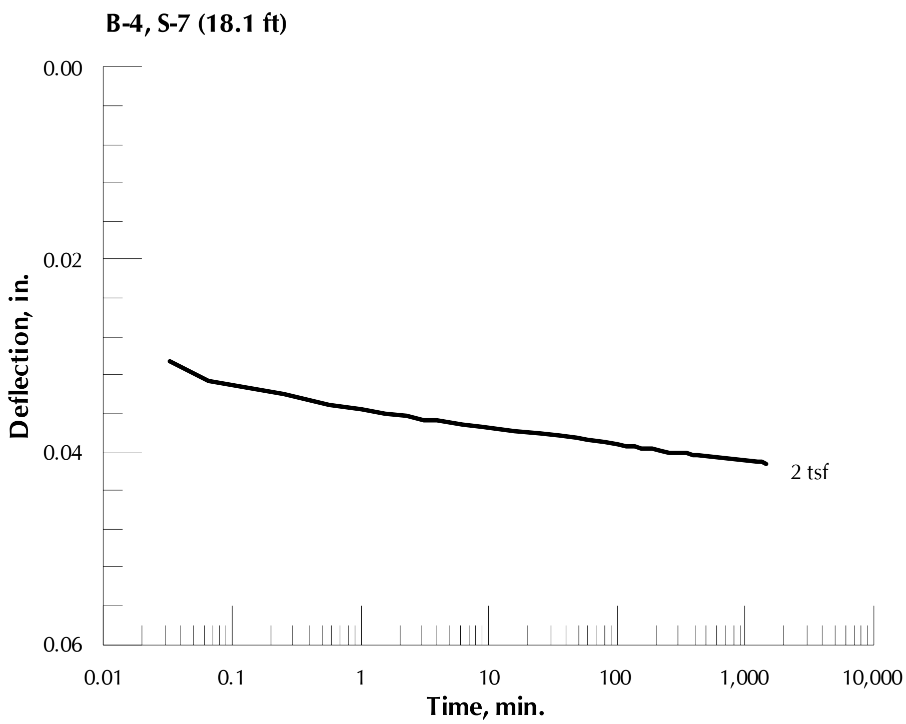


	Location	Sample	Depth, ft	Classification	Initial γ_d, pcf	Initial MC, %
●	B-19	S-8	21.5	Sandy SILT	80	41

GRI

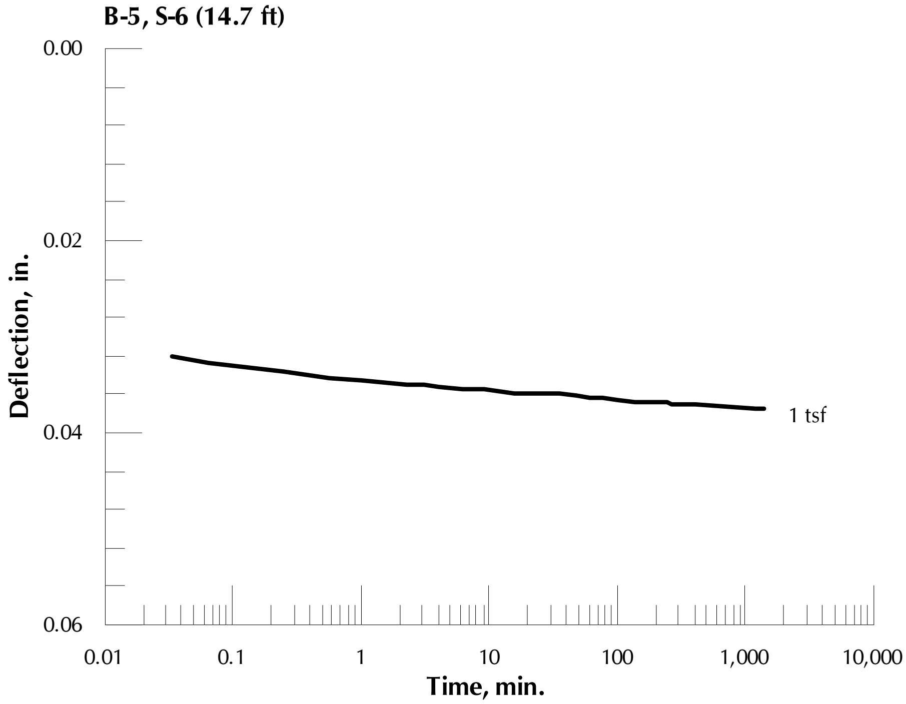
CONSOLIDATION TEST

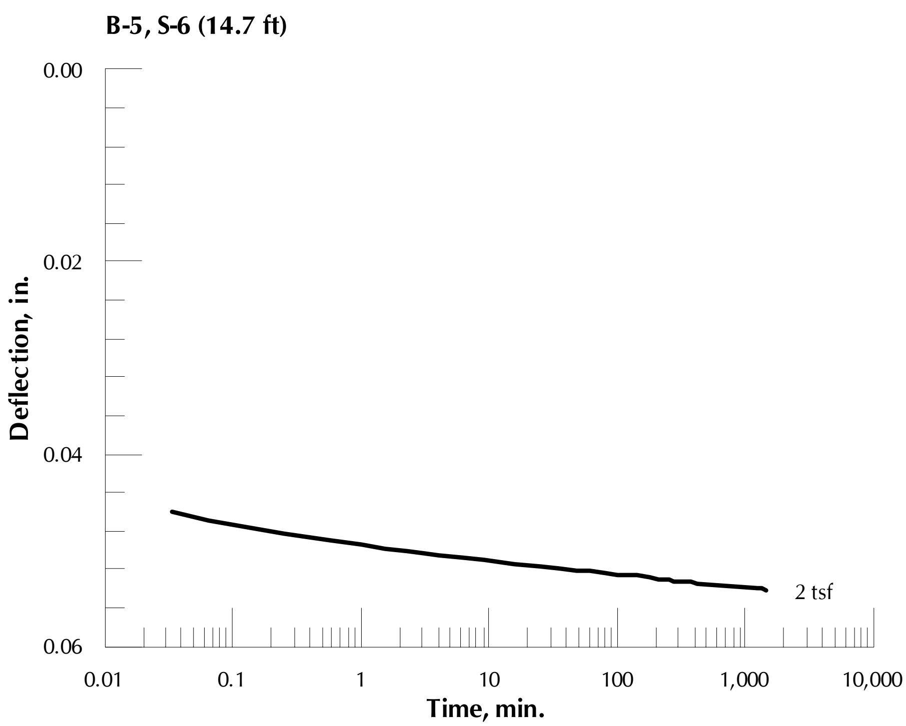




GRI

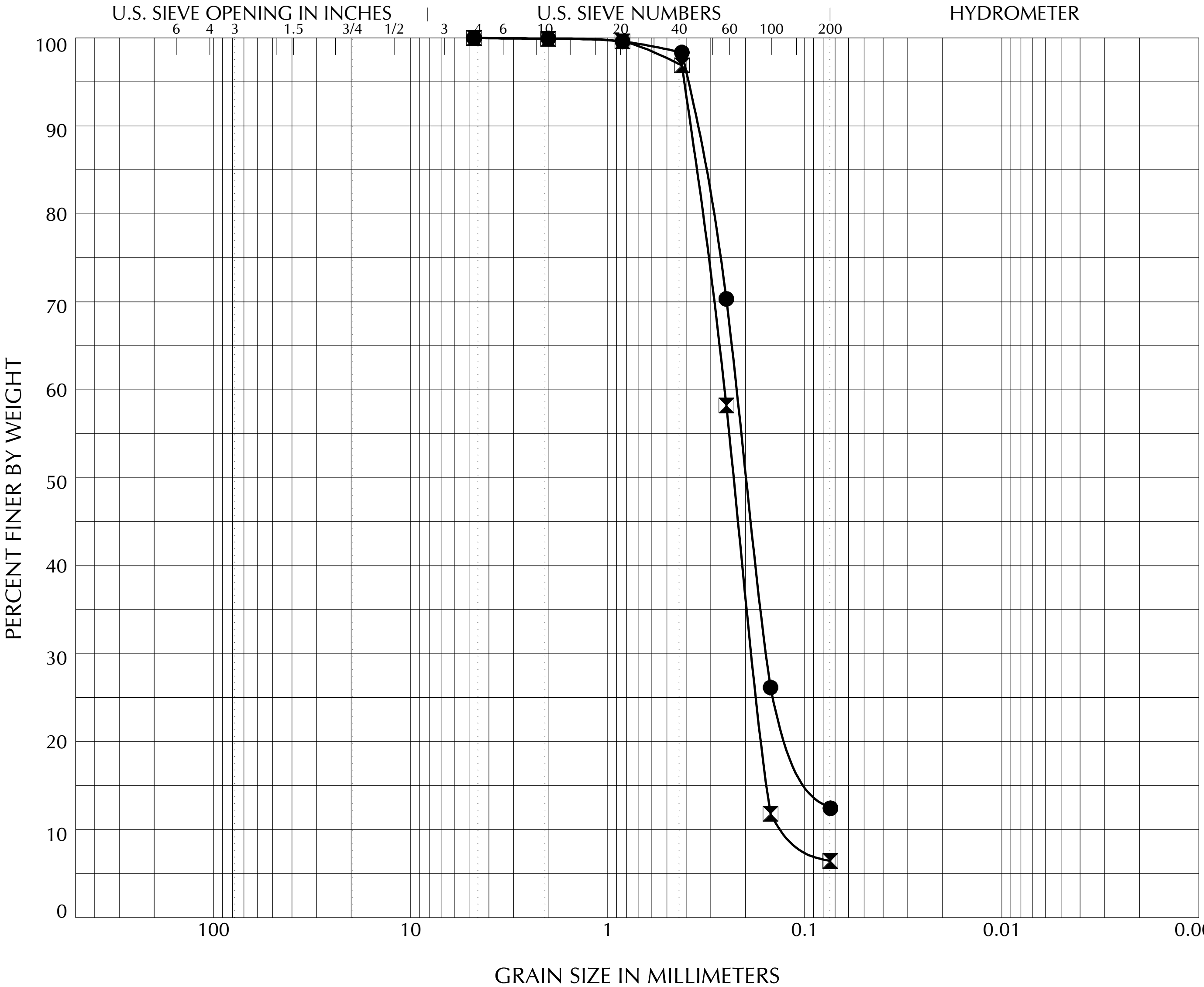
SECONDARY COMPRESSION





GRI

SECONDARY COMPRESSION



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

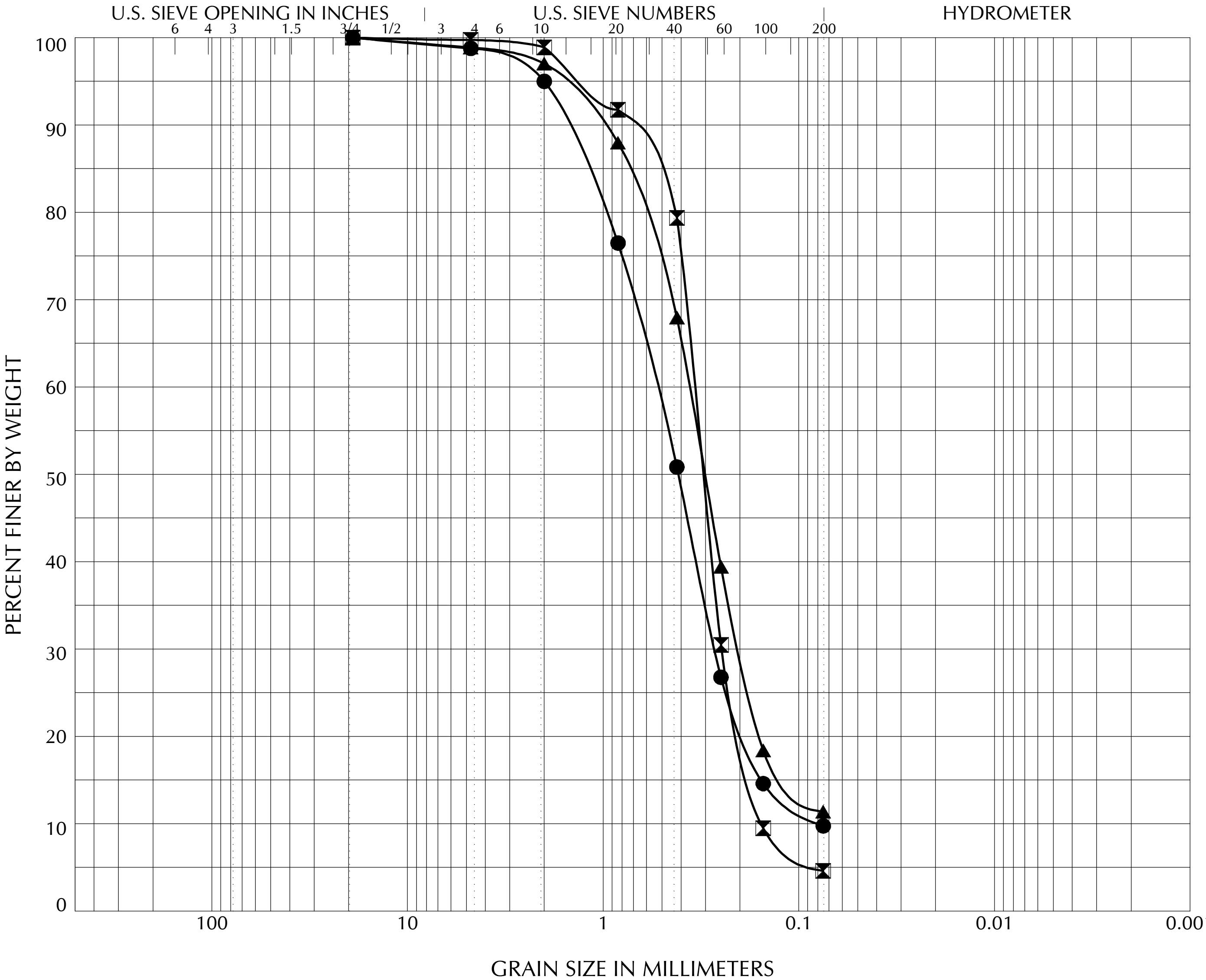
	Location	Sample	Depth, ft	Classification	Gravel, %	Sand, %	Fines, %
●	B-23	S-8	25.0	SAND; fine to medium grained, some silt	0.0	87.3	12.7
⧗	B-23	S-12	45.0	SAND; fine grained, trace silt	0.0	93.5	6.5

GRI

GRAIN SIZE DISTRIBUTION

DEC. 2013 JOB NO. W1114 FIG. 43A

GRAIN SIZE GRI-TWO PER PAGE W1114-LAB.GPJ GRI DATA TEMPLATE.GDT 12/10/13



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

	Location	Sample	Depth, ft	Classification	Gravel, %	Sand, %	Fines, %
●	B-26	S-4	10.0	Very loose, gray SAND; fine- to coarse-grained, trace silt	1.2	89.0	9.8
⧗	B-26	S-8	25.0	Loose, gray SAND; fine- to medium-grained, trace silt, scattered wood debris	0.3	95.2	4.6
▲	B-26	S-13	41.0	Loose, gray SAND; fine- to medium-grained, some silt, scattered wood debris	1.1	87.5	11.4

GRAIN SIZE GRI-THREE PER PAGE W1114-T3 LAB.GPJ GRI DATA TEMPLATE.GDT 12/10/13

GRI

GRAIN SIZE DISTRIBUTION

DEC. 2013 JOB NO. W1114-T3 FIG. 44A

APPENDIX B

Site-Specific Seismic Hazard Study

APPENDIX B

SITE-SPECIFIC SEISMIC HAZARD STUDY

General

GRI has completed a site-specific seismic hazard study for Areas 300 and 400 at the proposed Tesoro Savage Vancouver Energy Distribution Terminal - Upland Facility (TSVEDT) in Vancouver, Washington. The purpose of the study was to evaluate the potential seismic hazards associated with regional and local seismicity. The site-specific hazard study is intended to meet the requirements of the 2012 International Building Code (IBC), in compliance with the requirements of ASCE 7-10 Chapter 21. Our work was based on the potential for regional and local seismic activity, as described in the existing scientific literature, and on the subsurface conditions at the site, as disclosed by the geotechnical explorations completed for the project. Specifically, our work included the following tasks:

1) A detailed review of available literature, including published papers, maps, open-file reports, seismic histories and catalogs, and other sources of information regarding the tectonic setting, regional and local geology, and historical seismic activity that might have a significant effect on the site.

2) Compilation, examination, and evaluation of existing subsurface data gathered at and in the vicinity of the site, including classification and laboratory analyses of soil samples. This information was used to prepare a generalized subsurface profile for Areas 300 and 400 within the TSVEDT property.

3) Identification of the potential seismic sources appropriate for the site and characterization of those sources in terms of magnitude, distance and spectral response spectra.

4) Office studies, based on the generalized subsurface profile and the controlling seismic sources, resulting in conclusions and recommendations concerning:

    a) specific seismic events and characteristic earthquakes that might have a significant effect on Areas 300 and 400;

    b) the potential for seismic energy amplification in Areas 300 and 400; and

    c) site-specific acceleration response spectra for design of the proposed structures in Areas 300 and 400.

This appendix describes the work accomplished and summarizes our conclusions and recommendations.

Geologic Setting

General. On a regional scale, the site lies within the Willamette-Puget Sound lowland trough of the Cascadia convergent tectonic system (Blakely, et al., 2000). The lowland areas consist of broad north-south-trending basins in the underlying geologic structure between the Coast Range to the west and the

Cascade Mountains to the east. The lowland trough is characterized by alluvial plains with areas of buttes and terraces. The site lies approximately 95 km inland from the down-dip edge of the seismogenic extent of the Cascadia Subduction Zone (CSZ), an active convergent plate boundary along which remnants of the Farallon Plate (the Gorda, Juan de Fuca, and Explorer plates) are being subducted beneath the western edge of the North American continent. The subduction zone is a broad, eastward-dipping zone of contact between the upper portion of the subducting slabs and the over-riding North American Plate as shown on Figure 1B.

On a local scale, the site lies within the Portland Basin, a large, well-defined, northwest-trending structure characterized as a right-lateral pull-apart basin in the forearc of the CSZ. The Portland Basin is bounded by high-angle, northwest-trending, right-lateral strike-slip faults that are considered to be seismogenic; however, the relationship between specific earthquakes and individual faults in the area is not well understood since few of these faults are expressed clearly at the ground surface. A limited number of intrabasin faults have been mapped on the basis of stratigraphic offsets and geophysical evidence, and the site is located in close proximity to the inferred traces of the Portland Hills Fault and the East Bank Fault indicated on published geologic mapping (Personius, et al., 2003). The distribution of these crustal faults relative to the site is shown on the Regional Geologic Map and Local Fault Map, Figures 2B and 3B, respectively. The fault locations on the geologic map are inferred or approximate. Other faults may be present within the basin, but clear stratigraphic evidence regarding their location and extent is not presently available.

Because of the proximity of the site to the CSZ and its location within the Portland Basin, three seismic sources contribute to the potential for damaging earthquake motions at the site. Two of these sources are associated with tectonic activity related to the Cascadia Subduction Zone, the third is associated with movement on relatively shallow faults within and adjacent to the Portland Basin.

Site Soil and Geologic Conditions. The Area 300 and 400 portions of the project site are mantled by up to 25 ft of fill that is underlain by alluvial sand and silt deposited by the Columbia River. The alluvial deposits are underlain by gravel associated with late-Pleistocene catastrophic flood deposits that extend hundreds of feet below the ground surface in this portion of Vancouver. The catastrophic flood deposits consist of interbedded sands, silts, and gravels deposited by the repeated Missoula Flood events that occurred between 13,500 and 15,000 years ago. The flood deposits are underlain by interbedded sands and gravels of the Troutdale formation (Pliocene to Pleistocene) which are, in turn, underlain by the Columbia River Basalt bedrock (middle Miocene).

Seismicity

General. The geologic and seismologic information available for identifying the potential seismicity at the site is incomplete, and large uncertainties are associated with estimates of the probable magnitude, location, and frequency of occurrence of earthquakes that might affect the site. The available information indicates the potential seismic sources that may affect the site can be grouped into three independent categories: *subduction zone events* related to sudden slip between the upper surface of the Juan de Fuca plate and the lower surface of the North American plate, *subcrustal (intraslab) events* related to deformation and volume changes within the deeper portion of the subducted Juan de Fuca plate, and *local crustal events* associated with movement on shallow, local faults within and adjacent to the Portland Basin. Based on our review of currently available information, we have developed parameters for each of these potential seismic sources. The seismic sources are characterized by three important parameters: magnitude, distance to the subject site, and the peak horizontal bedrock accelerations produced by the controlling earthquake on the seismic source. The size of an earthquake is commonly defined by its moment magnitude M_W. Distance is measured using the closest horizontal distance to the surface projection of the rupture plane or the closest distance to the rupture plane, in kilometers. Peak horizontal bedrock accelerations are expressed in units of gravity (1 g = 32.2 ft/sec^2 = 981 cm/sec^2).

Subduction Zone Event. Written Japanese tsunami records provide evidence that a great CSZ earthquake occurred in January 1700. Geological studies show that great megathrust earthquakes have occurred repeatedly in the past 7,000 years (Atwater et al., 1995; Clague et al., 1997; Goldfinger, 2003; and Kelsey et al., 2005), and geodetic studies (Hyndman and Wang, 1995; Savage, et al., 2000) indicate rate of strain accumulation consistent with the assumption that the CSZ is locked beneath offshore northern California, Oregon, Washington, and southern British Columbia (Fluck, et al., 1997; Wang, et al., 2001). Numerous geological and geophysical studies suggest the CSZ may be segmented (Hughes and Carr, 1980; Weaver and Michaelson, 1985; Guffanti and Weaver, 1988; Goldfinger, 1994; Kelsey, et al., 1994; Mitchell, et al., 1994; Personius, 1995; Nelson and Personius, 1996; Witter, 1999), but the most recent studies suggest that for the last great earthquake in 1700, most of the subduction zone ruptured in a single M_w 9 earthquake (Satake, et al., 1996; Atwater and Hemphill-Haley, 1997; Clague, et al., 2000). Published estimates of the probable maximum size of subduction zone events range from moment magnitude M_W 8.3 to $>$9.0. Numerous detailed studies of coastal subsidence, tsunamis, and turbidites yield a wide range of recurrence intervals, but the most complete records ($>$4,000 years) indicate average intervals of 350 to 600 years between great earthquakes on the CSZ (Adams, 1990; Atwater and Hemphill-Haley, 1997; Witter, 1999; Clague, et al., 2000; Kelsey, et al., 2002; Kelsey, et al., 2005; Witter, et al., 2003). Tsunami inundation in buried marshes along the Washington and Oregon coast and stratigraphic evidence from the Cascadia margin support these recurrence intervals (Kelsey, et al., 2005; Goldfinger, 2003).

The USGS probabilistic analysis assumes four potential locations for the eastern edge of the earthquake rupture zone for the CSZ, as shown in Figure 4B. The 2008 USGS mapping effort indicates two rupture scenarios are assumed to represent these megathrust events: 1) M9±0.2 events that rupture the entire CSZ every 500 years and 2) M8.0 to 8.7 events with rupture zones that occur on segments of the CSZ and occur over the entire length of the CSZ during a period of about 500 years (Petersen, et al., 2008). The assumed distribution of earthquake magnitudes is shown on Figure 5B. This distribution assumes the larger M9.0 earthquakes likely occur more often than the smaller segmented ruptures. Therefore, for our deterministic analysis, we have chosen to represent the subduction zone event by a design earthquake of M_W 9.0 at a focal depth of 15 km and a rupture distance of 86 km. This corresponds to a sudden rupture of the entire length of the Juan de Fuca-North American plate interface with an assumed rupture zone along the coastline due west of Vancouver. Based on an average of the attenuation relationships published by Zhao (2006), Atkinson and Macias (2009), and Abrahamson (2012), a subduction zone earthquake with these parameters would result in an average peak bedrock acceleration of approximately 0.19 g at the project site.

Deaggregation of the 2008 USGS data suggests the Cascadia Subduction Zone contributes approximately 41% to the site seismic hazard.

Subcrustal Event. There is no historic earthquake record of subcrustal, intraslab earthquakes in Southwest Washington. Although both the Puget Sound and Northern California regions have experienced many of these earthquakes in historic times, Wong (2005) hypothesizes that due to subduction zone geometry, geophysical conditions and local geology, Southwest Washington/Oregon may not be subject to intraslab earthquakes. In the Puget Sound area, these moderate to large earthquakes are deep (40 to 60 km) and over 200 km from the deformation front of the subduction zone. Offshore, along the Northern California coast, the earthquakes are shallower (up to 40 km) and located along the deformation front. Estimates of the probable magnitude, distance, and frequency of subcrustal events in Southwest Washington are generally based on comparisons of the CSZ with active convergent plate margins in other parts of the world and on the historical seismic record for the region surrounding Puget Sound, where significant events known to have occurred within the subducting Juan de Fuca plate have been recorded. Published estimates of the probable maximum size of these events range from moment magnitude M_W 7.0 to 7.5. The 1949, 1965, and 2001 documented subcrustal earthquakes in the Puget Sound area correspond to M_W 7.1, 6.5, and 6.8, respectively. Published information regarding the location and geometry of the subducting zone indicates that a focal depth of 50 km is probable (Weaver and Shedlock, 1989). We have chosen to represent the subcrustal event by a characteristic earthquake of moment magnitude M_W 7.0 at a focal depth of 50 km and a rupture distance of 50 km. Based on the attenuation relationships published by Zhao (2006), and Atkinson and Boore (2003), and Abrahamson (2012), a subcrustal earthquake of this magnitude and distance would result in a peak horizontal bedrock acceleration of approximately 0.14 g at the site.

The results of the USGS deaggregation suggest a seismic hazard contribution of 22% from a subcrustal or intraslab earthquake.

Local Crustal Event. Sudden crustal movements along relatively shallow, local faults in the southwest Washington area, although rare, have been responsible for local crustal earthquakes. The precise relationship between specific earthquakes and individual faults is not well understood, since few of the faults in the area are expressed at the ground surface, and the foci of the observed earthquakes have not been located with precision. The history of local seismic activity is commonly used as a basis for determining the size and frequency to be expected of local crustal events. Although the historical record of local earthquakes is relatively short (the earliest reported seismic event in the area occurred in 1920), it can serve as a guide for estimating the potential for seismic activity in the area.

Based on fault mapping conducted by the U.S. Geological Survey (USGS, 2008), the Portland Hills Fault is the closest mapped crustal fault to the site that is considered active in the probabilistic hazard maps. The Portland Hills Fault is located approximately 7 km from the site and has a characteristic earthquake magnitude of M_W 7.0. A crustal earthquake of this magnitude and distance would result in a peak horizontal bedrock acceleration of approximately 0.33 g at the site based on an average of the NGA ground motion relations developed by the Pacific Earthquake Engineering Research (PEER) by Boore and Atkinson (2008), Campbell and Bozorgnia (2008), Idriss (2008) and Chiou and Youngs (2008).

Deaggregation of the 2008 USGS data suggests local crustal faults contribute approximately 35% to the site seismic hazard.

Other Seismic Hazards. Based on the presence of loose sands and soft silts below the water table at the site, there is a high risk of liquefaction and lateral spreading during a design-level earthquake. More detailed discussions regarding liquefaction and lateral spreading are provided in the Seismic Considerations section of the report. Although detailed tsunami modeling of the Columbia River due to a Cascadia Subduction Zone earthquake has not been completed, we anticipate the risk of upland damage by tsunami at the site is low due to the distance from the coast. River fluctuations may result from a tsunami generated by a CSZ earthquake. Due to the proximity of the Columbia River, there is a risk of seiche. Unless occurring on a previously unmapped fault, it is our opinion the risk of ground rupture at the site is very low.

Deterministic Earthquake Parameters

As discussed above, three distinctly different seismic sources affect seismicity in the project area. Deterministic evaluation of the earthquake sources using published attenuation relations provides estimates of peak bedrock accelerations and response spectra for each seismic source. These deterministic estimates are not associated with a relative hazard level or probability of occurrence like probabilistic estimates, but simply provide an estimate of the ground motion parameters for each seismic source at a given distance from the site. The basic parameters of each earthquake source are as follows:

TABLE 1B: DETERMINISTIC EARTHQUAKE PARAMETERS

Earthquake Source	Attenuation Relationships	Magnitude, M_w	Rupture Distance, km	Focal Depth, km	Median Peak Bedrock Acceleration, g	Average Median Peak Bedrock Acceleration, g
Subduction Zone	Zhao (2006)	9.0	86	15	0.19	
	Atkinson and Macias (2009)	9.0	86	15	0.17	0.19
	Abrahamson (2012)	9.0	86	15	0.23	
	Gregor, et al.(2002)					
Subcrustal	Zhao (2006)	7.0	50	50	0.15	
	Atkinson and Boore, (2003)	7.0	50	50	0.10	0.14
	Abrahamson (2012)	7.0	50	50	0.18	
Local Crustal	Campbell and Bozorgnia (2008)	7.0	7	NA	0.32	
	Chiou and Youngs (2008)	7.0	7	NA	0.36	0.33
	Boore and Atkinson (2008)	7.0	7	NA	0.27	
	Idriss (2008)	7.0	7	NA	0.38	

The values summarized in Table 1B represent the average of median peak bedrock accelerations for the characteristic earthquake on the controlling faults. IBC and its reference document, ASCE 7-10, require evaluating the 84th percentile (median plus one standard deviation) rock response spectrum in the maximum horizontal direction for developing the deterministic MCE_R level earthquake. The risk-targeted deterministic (MCE_R) bedrock spectra shown in Figure 6B represent a weighted average of the individual spectra produced by the attenuation relationships presented in Table 1B at the 84th percentile level.

These risk-targeted deterministic spectra were compared with the deterministic lower limit on MCE_R response spectrum, constructed per Figure 21.2-1 of ASCE 7-10, for selection of the MCE_R deterministic bedrock spectrum. Figure 6B shows that the individual fault deterministic response spectra are essentially at or lower than the deterministic lower limit on MCE_R response spectrum. Per Section 21.2.2 of ASCE 7-10, the deterministic spectrum shall be the greater of the fault deterministic spectrum or the lower limit deterministic spectrum of Figure 21.2-1. Therefore, the lower limit deterministic MCE_R response spectrum is selected to represent the bedrock deterministic (MCE_R) response spectrum.

Probabilistic Considerations

The probability of an earthquake of a specific magnitude occurring at a given location is commonly expressed by its return period, i.e., the average length of time between successive occurrences of an earthquake of that size or larger at that location. The return period of a design earthquake is calculated once a project design life and some measure of the acceptable risk that the design earthquake might occur or be exceeded are specified. These expected earthquake recurrences are expressed as a probability of exceedance during a given time period or design life. Historically, building codes have adopted an acceptable risk level by identifying ground acceleration values that meet or exceed a 10% probability of exceedance in 50 years, which corresponds to an earthquake with an expected recurrence interval of 475 years. Previous versions of the IBC developed response spectra based on ground motions associated with the Maximum Considered Earthquake (MCE), which is generally defined as a probabilistic earthquake with a 2% probability of exceedance in 50 years (return period of about 2,500 years), except where subject to deterministic limitations (Leyendecker, et al., 2000).

The current 2012 IBC develops response spectra using a Risk-Targeted Maximum Considered Earthquake (MCE_R), which is defined as the response spectrum that is expected to achieve a 1% probability of building collapse within a 50-year period. In addition, the spectral response values for the 2012 IBC are for the direction of maximum horizontal acceleration rather than the geometric mean horizontal acceleration used in previous codes. The design-level response spectrum is calculated as two-thirds of the MCE_R ground motions. The 2012 IBC changes to the site response spectra based on probability of building collapse and maximum directional accelerations result in a very slight increase in the code site response compared with the 2009 IBC. Although the MCE_R site response is similar to the previous code, it should be noted that seismic hazards, such as liquefaction and soil strength loss, are now evaluated using the MCE-level geometric mean (MCE_G) peak ground acceleration. Under previous codes, these seismic hazards were evaluated using the design-level peak ground acceleration. The design-level peak ground acceleration is two-thirds of the MCE_G peak ground accelerations, the same ratio as between the MCE_R and design response spectra.

The 2012 IBC design methodology uses two mapped spectral acceleration parameters, S_S and S_1, corresponding to periods of 0.2 and 1.0 second, to develop the Site Class B MCE_R response spectrum. The S_S and S_1 coefficients are 0.94 and 0.41 g, respectively, for the site located at the approximate latitude and longitude coordinates of 45.65°N and 122.71°W.

Target Bedrock Spectrum

Chapter 21 of ASCE 7-10, requires comparing the deterministic MCE_R response spectrum with the probabilistic MCE_R response spectrum to select the controlling spectrum. The probabilistic and deterministic MCE_R response spectra are shown in Figure 7B. The site-specific MCE_R bedrock response spectrum is taken as the lower of these two spectra per ASCE 7-10 Section 21.2.3. The probabilistic MCE_R spectrum is lower than the deterministic spectrum and, therefore, based on the above criterion, the probabilistic spectrum is defined to be the MCE_R bedrock spectrum. The risk-targeted probabilistic spectrum is also compared with the geometric mean probabilistic bedrock spectrum (i.e., defined by 2% probability of exceedance within a 50-year period) as shown on Figure 8B. Review of Figure 8B indicates the geometric mean bedrock spectrum is comparable with the MCE_R bedrock spectrum. The 2,475-year geometric mean bedrock spectrum was chosen as the target bedrock spectrum for the TSVEDT site to allow the use of one target bedrock spectrum for both structural analysis and liquefaction evaluation.

Estimated Site Response

The effect of a specific seismic event on the site is related to the type and thickness of soil overlying the bedrock at the site and the type and quantity of seismic energy delivered to the bedrock beneath the site by the earthquake. Site response analysis was completed to estimate this site-specific behavior in accordance with section 21.1 of ASCE 7-10. The site response analysis consisted of three components: 1) selection of target bedrock response spectrum, 2) numerical modeling to analyze the site-specific behavior of the soils using horizontal ground motion acceleration time histories scaled to the approximate level of the target bedrock response spectrum over the periods of interest, and 3) calculation of the ratio of the surface response spectra values to the bedrock response spectra values, at each spectral period, to develop a recommended ground surface response spectrum. The following paragraphs describe details of the site response modeling.

The target bedrock response spectrum for the site was developed for Site Class B, or rock site, conditions in accordance with the method outlined in the Target Bedrock Spectrum section of this report. A series of earthquake acceleration-time histories have been selected to estimate the earthquake motions in D-MOD 2000 (D-MOD), a non-linear site response program. From the available records, corrected free-field and basement/ground floor accelerograms were selected for input as bedrock time histories. Wherever possible, earthquakes of similar magnitude and duration to the characteristic earthquakes were selected. These records were checked for obvious errors, missing data points, and other anomalies and were transformed into a uniform data format. The selected strong-motion records are as follows:

Earthquake	Recording Station	Magnitude	Fault Distance, km	Peak Bedrock Acceleration, g
Loma Preita (1989)	San Jose - Santa Teresa Hills	6.9	14.7	0.28
Nisqually (2001)	Olympia, WSDOT Test Lab	6.8	18.3	0.22
Chile (2010)	Curico	8.8	65.1	0.47
Chile (2010)	Hualane	8.8	50	0.46
Japan (2011)	Kuroiso (TCG001)	9	102	0.42
Japan (2011)	Yamatsuri (FKS 014)	9	76	0.23
Japan (2011)	Hachinohe (AOM 012)	9	99	0.19

The time histories were scaled to reasonably match the bedrock target spectrum at periods of interest including the site fundamental period.

A generalized subsurface profile for the site was developed for use in D-MOD based on our subsurface explorations. To estimate shear wave velocities for the soil profiles for Areas 300 and 400, probes CPT-1 and CPT-6, respectively, were operated with an accelerometer fitted to the probe that allows measuring the arrival times of shear waves at the probe from impulses generated at the ground surface. Based on the arrival times, a shear wave velocity profile was generated for Areas 300 and 400. These assumed soil profiles are tabulated below.

AREA 300 SUBSURFACE PROFILE

Material	Thickness, ft	Total Unit Weight, pcf	Estimated Shear Wave Velocity, ft/sec
FILL (Dense silty SAND or GRAVEL)	6	120	800
FILL (Dense silty SAND or GRAVEL)	6	120	800
FILL (Dense silty SAND or GRAVEL)	6	120	800
Soft SILT	5	110	500
Soft SILT	4	110	500
Soft SILT	4	110	500
Soft SILT	4	110	500
SAND	5	120	950
SAND	5	100	950
SAND	5	100	950
SAND	5	110	950
Gravel with sand	8	125	1,300
Gravel with sand	7	125	1,325
Gravel with sand	7	125	1,350
Gravel with cobbles	8	130	1,425
Gravel with cobbles	8	130	1,500
Gravel with cobbles	8	130	1,550
Gravel with cobbles	8	130	1,625
Gravel with cobbles	8	130	1,700
Gravel with cobbles	12	130	1,775
Gravel with cobbles	12	130	1,850
Gravel with cobbles	12	130	1,925
Gravel with cobbles	12	130	1,975
Gravel with cobbles	12	130	2,050
Gravel with cobbles	12	130	2,125
Gravel with cobbles	12	130	2,200
Gravel with cobbles	12	130	2,250
Gravel with cobbles	12	130	2,325
Gravel with cobbles	12	130	2,400
Gravel with cobbles	12	130	2,475
Troutdale	N/A	140	2,500

AREA 400 SUBSURFACE PROFILE

Material	Thickness, ft	Total Unit Weight, pcf	Estimated Shear Wave Velocity, ft/sec
Medium dense SAND	3	110	473
Medium dense SAND	3	110	473
Medium dense SAND	3	110	534
Medium dense SAND	4	110	615
Medium dense SAND	3	110	520
Medium dense SAND	4	110	573

Material	Thickness, ft	Total Unit Weight, pcf	Estimated Shear Wave Velocity, ft/sec
Medium dense SAND	3	110	573
Medium dense SAND	3	110	678
Loose SAND	4	110	532
Loose SAND	3	110	475
Loose SAND	3	110	555
Medium dense SAND	4	110	690
Medium dense SAND	3	110	686
Medium dense SAND	3	110	653
Medium dense SAND	3	110	560
Medium dense SAND	4	110	680
Medium dense SAND	3	110	830
Medium dense SAND	3	110	920
Medium dense SAND	3	110	1,034
Medium dense SAND	4	110	865
Medium dense SAND	3	110	1,034
Medium dense SAND	3	110	950
Medium dense SAND	3	110	950
Medium dense SAND	4	110	860
Medium dense SAND	3	110	1,000
Very dense GRAVEL	5	125	1,200
Very dense GRAVEL	5	125	1,400
Very dense GRAVEL	5	125	1,450
Very dense GRAVEL	5	125	1,500
Very dense GRAVEL	10	125	1,550
Very dense GRAVEL	10	125	1,600
Very dense GRAVEL	10	125	1,650
Very dense GRAVEL	10	125	1,700
Very dense GRAVEL	10	125	1,750
Very dense GRAVEL	10	125	1,800
Very dense GRAVEL	10	125	1,850
Very dense GRAVEL	10	125	1,900
Very dense GRAVEL	10	125	1,950
Very dense GRAVEL	10	125	2,000
Very dense GRAVEL	10	125	2,100
Very dense GRAVEL	10	125	2,150
Very dense GRAVEL	10	125	2,200
Very dense GRAVEL	10	125	2,250
Very dense GRAVEL	10	125	2,350
Very dense GRAVEL	10	125	2,400
Troutdale	N/A	140	2,500

Using the generalized subsurface profiles for Areas 300 and 400, the peak bedrock accelerations estimated for the design event, and the strong-motion records listed in the preceding tables, pseudo acceleration response spectra were calculated with D-MOD. The spectra were produced for a ground surface elevation damped at 5% of critical damping. The ground surface spectra were compared to the input rock spectra to quantify amplification and/or attenuation through the soil column at the site. The ratio of ground surface to bedrock spectral accelerations, defined as the spectral amplification ratio (SAR), is shown on Figure 9B for Areas 300 and 400. To estimate ground surface site response throughout the range of spectral periods, the target response spectra is multiplied by the SAR to determine the ground surface response spectrum in accordance with Section 21.1.3 of ASCE 7-10. The results of the site-specific response modeling are shown on Figures 10B and 11B for Areas 300 and 400, respectively.

Figures 10B and 11B also include the code-based MCE_R hazard level spectrum, developed using site amplification factors based on the appropriate Site Class type. A discussion of the code-based site amplification factors and site class is provided in the following paragraph.

Area 300 is designated as Site Class D based on the average shear wave velocity (V_{S100}) in the upper 100 ft per Section 20.4 of ASCE 7-10. However, Area 300 would be designated as Site Class E based on the average standard penetration resistance for the upper 100 ft of the soil profile in accordance with Section 20.4 of ASCE 7-10. Area 400 is designated Site Class D, based on the average shear wave velocities and average standard penetration resistance in the upper 100 ft. Short- and long-period site coefficients, F_a and F_v, of 1.12 and 1.59, respectively, were used to develop the MCE_R Site Class D spectrum. The MCE_R Site Class E spectrum was developed using F_a and F_v of 0.97 and 2.40, respectively.

Sites that are underlain by soils subject to liquefaction are designated as Site Class F per Section 20.3.1 of ASCE 7-10 and are required to have a site response analysis performed to develop the ground surface response spectrum. Structures that have a fundamental period less than 0.5 second are exempted from this requirement. Ground surface response spectrum developed from site response analysis for Site Class F may not be less than 80% of Site Class E spectral acceleration values per Section 21.3 of ASCE 7-10. Both Site Class D and 80% of Site Class E spectra are shown on Figures 10B and 11B for comparison with the site-specific spectra. For Area 300, the site-specific response modeling resulted in a ground surface response spectrum with peak spectral acceleration values greater than the Site Class D peak spectral value. Per Section 21.4 of ASCE 7-10, the short-period spectral acceleration value is taken as 90% of the peak spectral acceleration that occurs at a period greater than 0.2 second. The site-specific peak spectral value, multiplied by 90%, is essentially the Site Class D peak spectral value. Therefore, the Site Class D curve is recommended for estimation of the spectral accelerations at short periods. The site-specific spectral response parameter at 1 second is selected as the greater of the spectral value at 1 second or two times the spectral value at 2 seconds. The site-specific response spectrum has a 1-second spectra value greater than twice the 2-second spectral value and is thus the 1-second spectral value. The peak horizontal portion of the Site Class D curve was extended to a period of 1 second to encompass the site-specific 1-second value. For periods in excess of 1 second, the site-specific response spectrum was used to the period where site-specific values are below 80% of Site Class E (approximately 1.5 seconds). At periods in excess of 1.5 seconds, 80% of Site Class E provides the spectral acceleration values that meet the requirement of ASCE 7-10.

For Area 400, the site-specific response modeling provided a ground surface response spectrum with peak spectral acceleration values lower than that of Site Class D. Site Class D peak spectral values were recommended for estimating short-period spectral accelerations to be consistent with Area 300. At longer periods, the Site Class D curve was modified to encompass the higher site-specific spectral values. At periods in excess of approximately 1.75 seconds, where the site-specific response spectrum falls below

80% if Site Class E, response spectral values corresponding to 80% of the Site Class E response spectrum is used to satisfy the requirement of ASCE 7-10.

Conclusions

The site specific response modeling for the TSVEDT site was completed using the 2,475-year geometric mean spectral accelerations as a target bedrock spectrum with spectral acceleration values of S_s and S_1, corresponding to periods of 0.2 and 1.0 second, equal to 0.96 and 0.37 g respectively.

For Area 300, the results of the site-specific modeling in accordance with ASCE 7-10 indicate the 2012 IBC code-based Site Class D spectrum provides an appropriate estimate of the spectral accelerations for short periods. For longer periods, a response spectrum consisting of the site-specific spectral response values and the spectral values corresponding to 80% of the Site Class E response spectrum is appropriate. The design-level spectral acceleration recommended for Area 300 is shown on Figure 12B.

For Area 400, the results of the site-specific modeling indicate the 2012 IBC code-based Site Class D spectrum provides an appropriate estimate of the spectral accelerations at short periods, while a response spectrum encompassing the site-specific spectral response values and 80% of the Site Class E response spectrum is appropriate. The design-level spectral acceleration spectrum for Area 400 is shown on Figure 12B.

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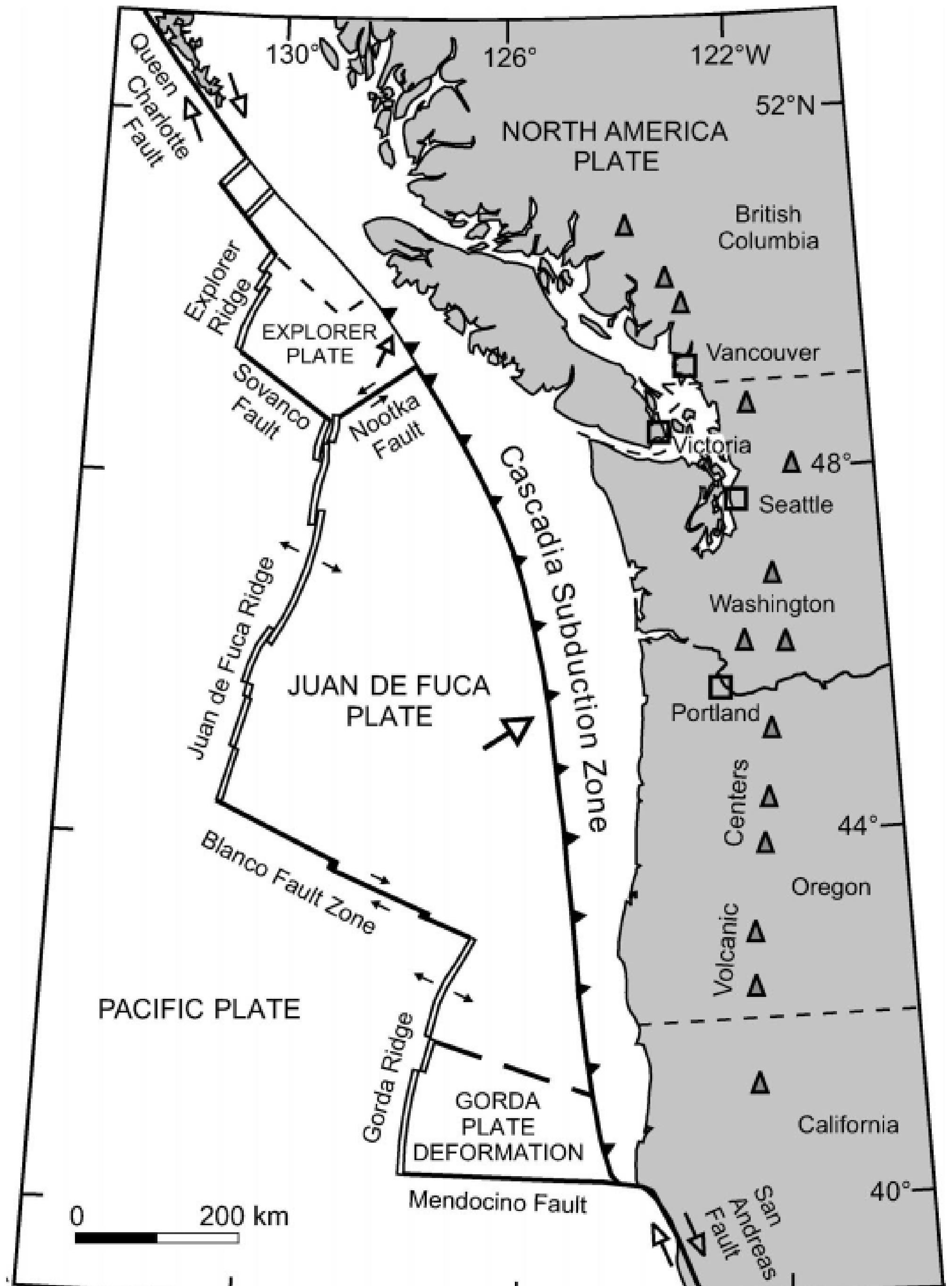
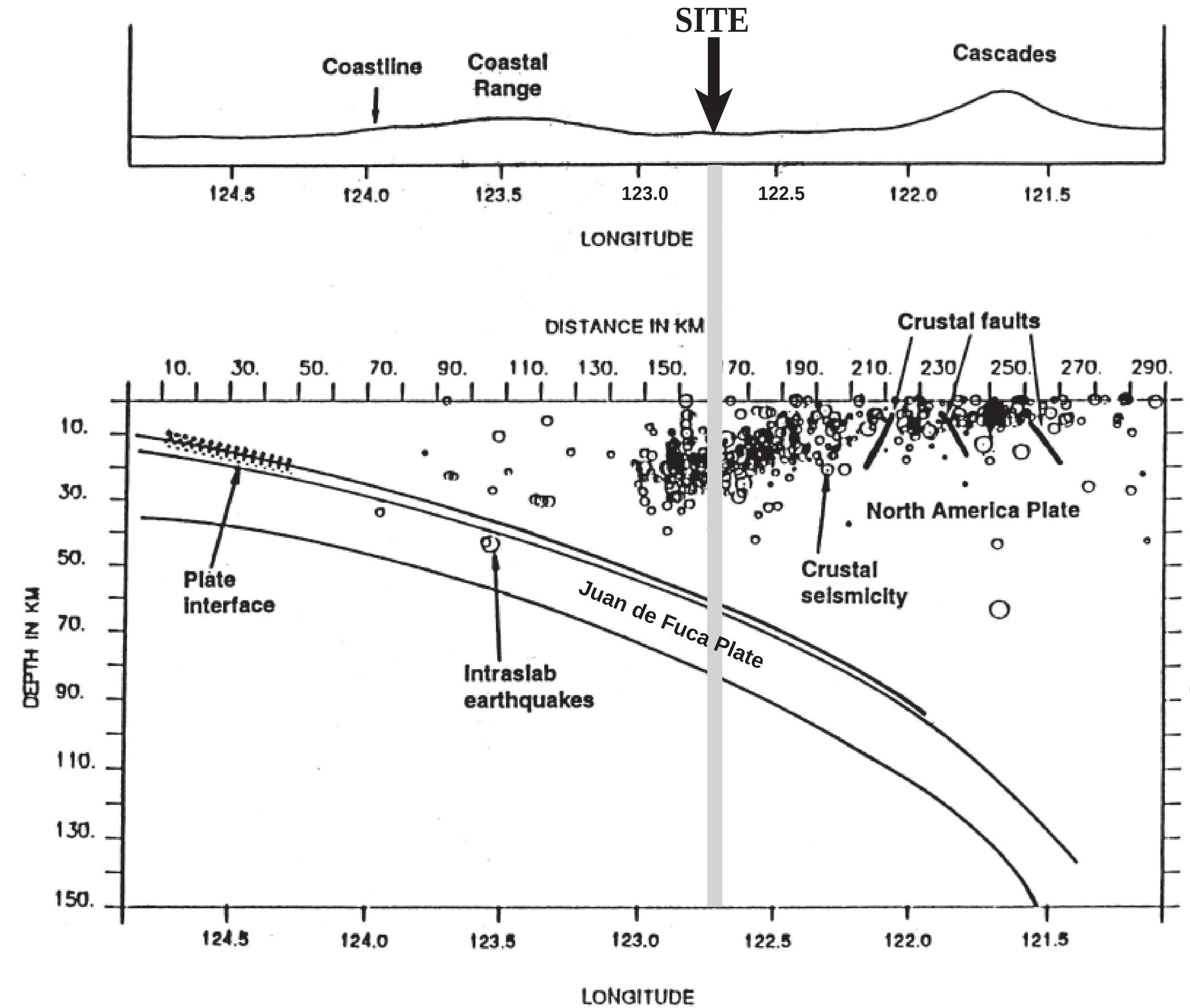


A) TECTONIC MAP OF PACIFIC NORTHWEST, SHOWING ORIENTATION AND EXTENT OF CASCADIA SUBDUCTION ZONE (MODIFIED FROM DRAGERT AND OTHERS, 1994)

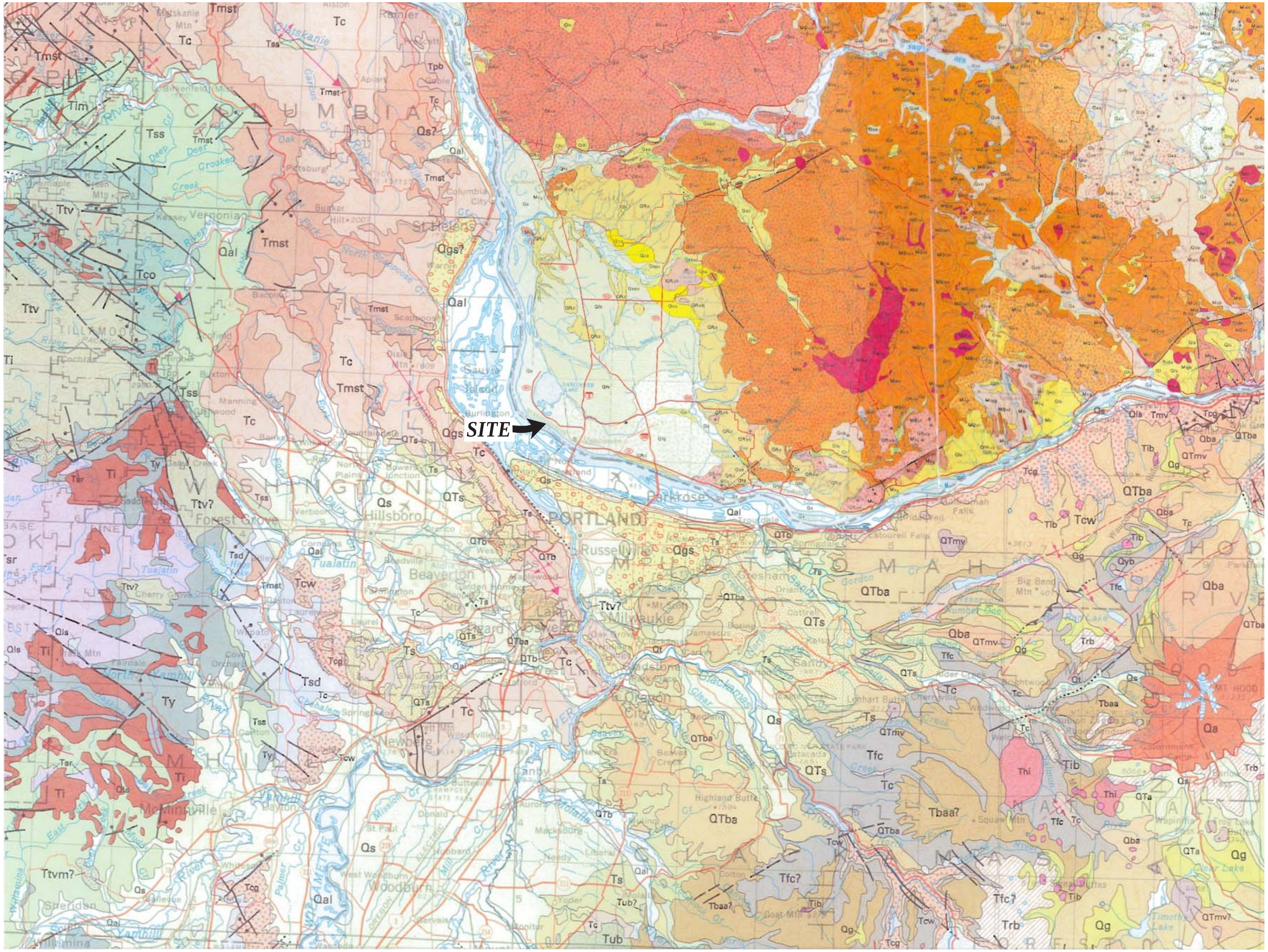


B) EAST-WEST CROSS-SECTION THROUGH WESTERN WASHINGTON AT THE LATITUDE OF VANCOUVER, SHOWING THE SEISMIC SOURCES CONSIDERED IN THE SITE-SPECIFIC SEISMIC HAZARD STUDY (MODIFIED FROM GEOMATRIX, 1995)

GRI

TECTONIC SETTING SUMMARY

DEC. 2013 | JOB NO. W1114 | FIG. 1B



Contact — Approximately located

Fault — Dashed where inferred; dotted where concealed; queried where doubtful; ball and bar on downthrown side

Thrust fault — Dashed where inferred; dotted where concealed; queried where doubtful; sawteeth on upper plate

7

Strike and dip of bed

FROM:

WALSH, T.J., KOROSEC, M.A., PHILLIPS, W.M., LOGAN, R.L., AND SCHASSE, H.W., 1987, GEOLOGIC MAP OF WASHINGTON-SOUTHWEST QUADRANT; 1:250,000: WASHINGTON DIVISION OF GEOLOGY AND EARTH RESOURCES, 6M-34

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North

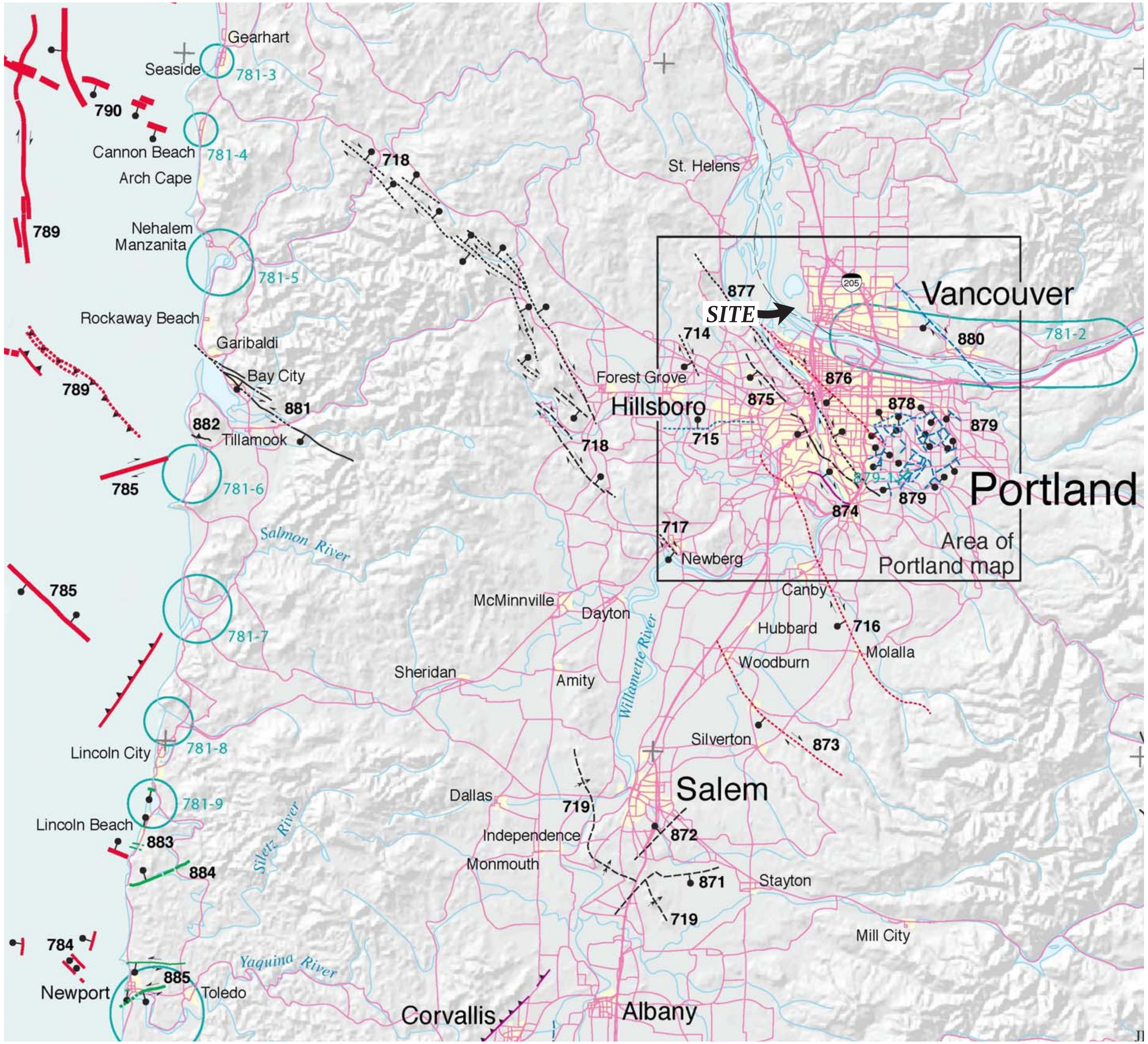
0 10 20 MILES

0 10 20 KILOMETERS

GRI

REGIONAL GEOLOGIC MAP

DEC. 2013 | JOB NO. W1114 | FIG. 2B



MAP EXPLANATION

TIME OF MOST RECENT SURFACE RUPTURE

- Holocene (<10,000 years) or post last glaciation (<15,000 years; 15 ka); no historic ruptures in Oregon to date
- Late Quaternary (<130,000; post penultimate glaciation)
- Late and middle Quaternary (<750,000 years; 750 ka)
- Quaternary, undifferentiated (<1,600,000 years; <1.6 Ma)
- Class B structure (age or origin uncertain)

SLIP RATE

- >5 mm/year
- 1.0-5.0 mm/year
- 0.2-1.0 mm/year
- <0.2 mm/year

TRACE

- Mostly continuous at map scale
- Mostly discontinuous at map scale
- Inferred or concealed

STRUCTURE TYPE AND RELATED FEATURES

- Normal or high-angle reverse fault
- Strike-slip fault
- Thrust fault
- Anticlinal fold
- Synclinal fold
- Monoclinal fold
- Plunge direction of fold
- Fault section marker

DETAILED STUDY SITES

- 731-2 Trench site
- 781-2 Subduction zone study site

CULTURAL AND GEOGRAPHIC FEATURES

- Divided highway
- Primary or secondary road
- Permanent river or stream
- Intermittent river or stream
- Permanent or intermittent lake

FAULT NUMBER	NAME OF STRUCTURE
876	EAST BANK FAULT
877	PORTLAND HILLS FAULT
880	LACAMAS LAKE FAULT
878	GRAND BUTTE FAULT

FROM: PERSONIUS, S.F., AND OTHERS, 2003, MAP OF QUATERNARY FAULTS AND FOLDS IN OREGON, USGS OPEN FILE REPORT OFR-03-095.

North

0 5 10 MILES

0 5 10 KILOMETERS

GRI

LOCAL FAULT MAP

DEC. 2013 JOB NO. W1114 FIG. 3B

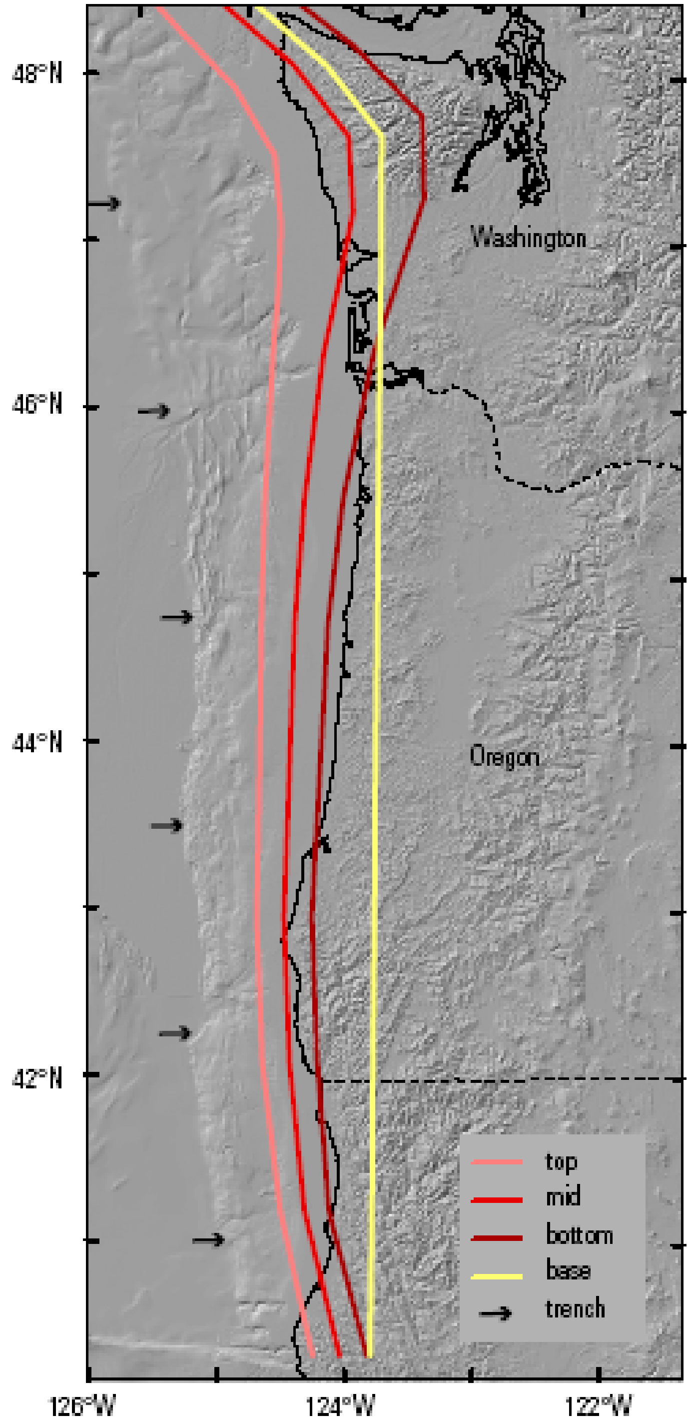


Figure 21. Location of the eastern edge of earthquake-rupture zones on the Cascadia subduction zone for the various models used in this study relative to the surficial expression of the trench: top, base of the elastic zone; mid, midpoint of the transition zone; bottom, base of the transition zones; base, base of the model that assumes ruptures extend to about 30-kilometers depth. Figure provided by Ray Weldon.

FROM: PETERSEN, MD, FRANKEL, AD, HARMSEN, SC, AND OTHERS, 2008, DOCUMENTATION FOR THE 2008 UPDATE OF THE UNITED STATES NATIONAL SEISMIC HAZARD MAPS: US GEOLOGICAL SURVEY, OPEN FILE REPORT 2008-1128

GRI

ASSUMED RUPTURE LOCATIONS

(CASCADIA SUBDUCTION ZONE)

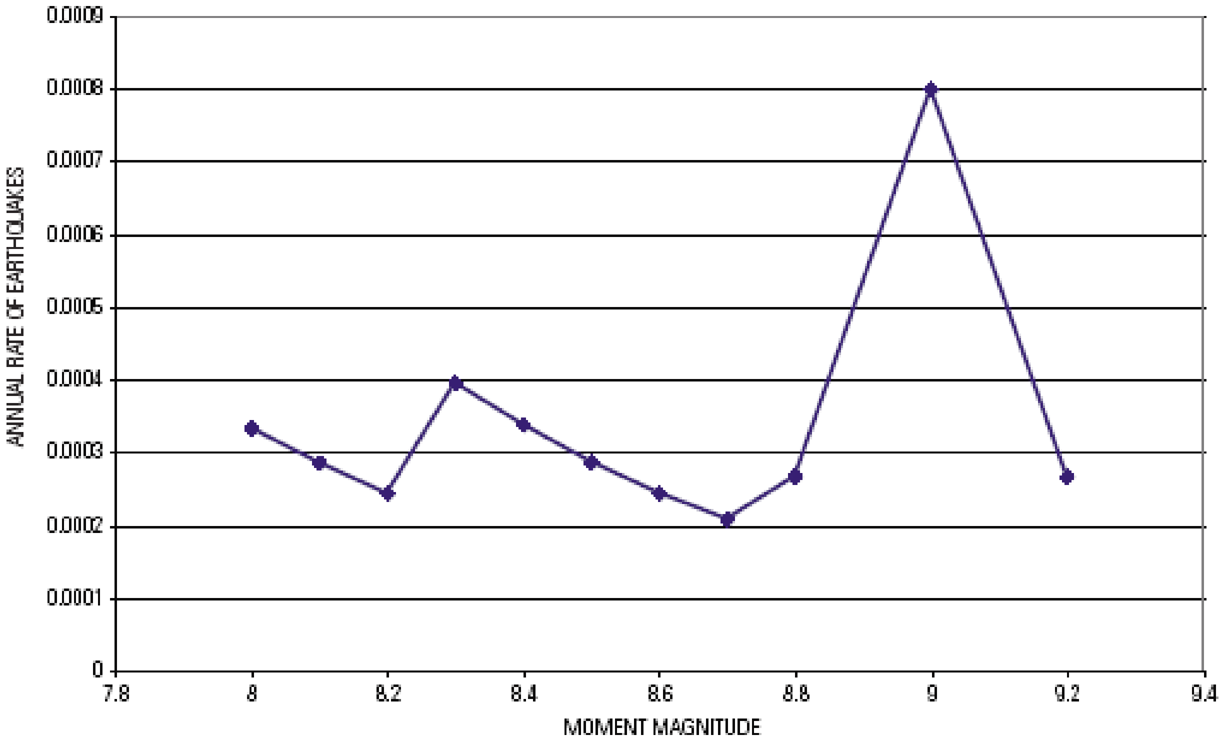


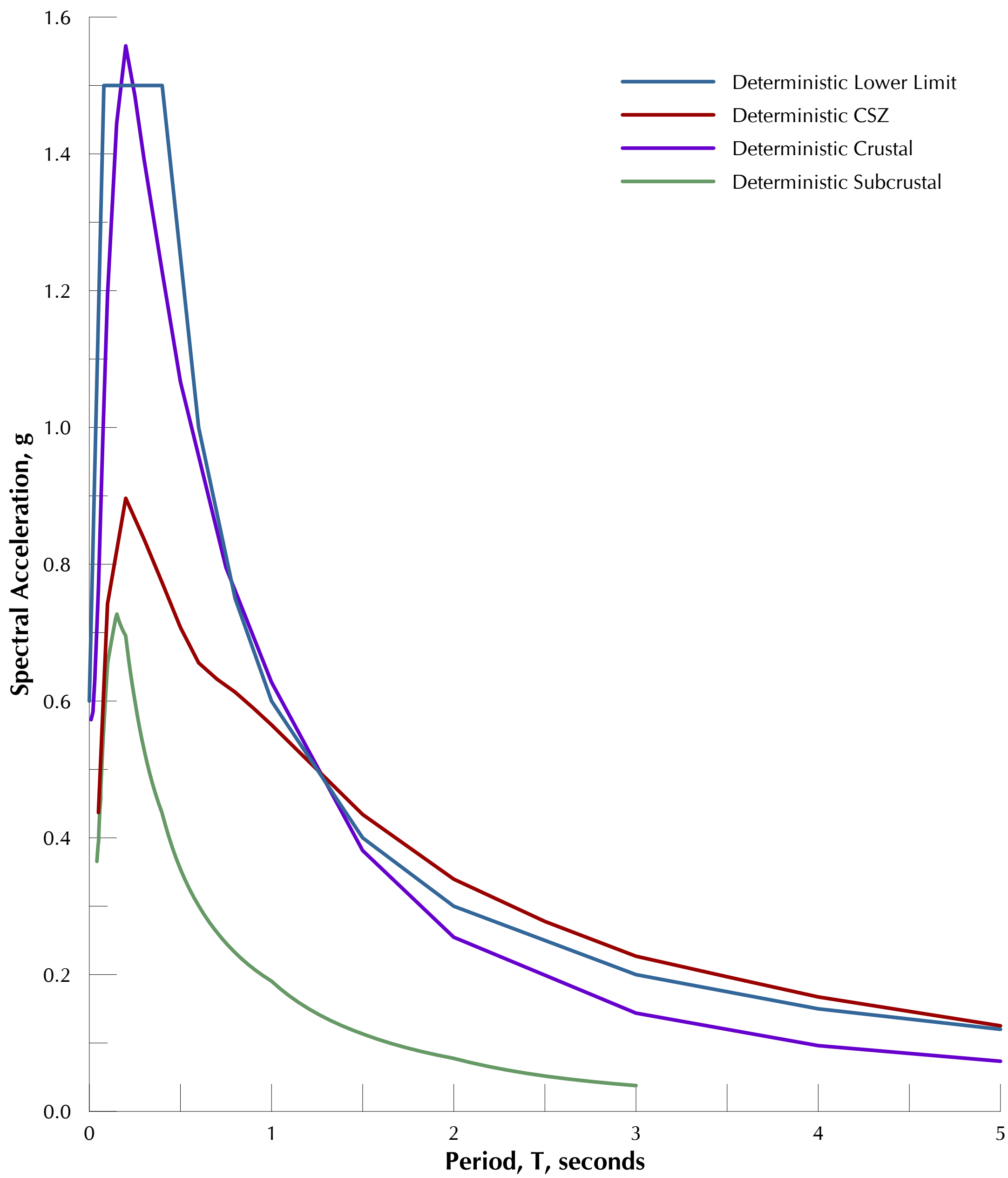
Figure 22. **Magnitude-frequency distribution of the Cascadia subduction zone.**

FROM: PETERSEN, M, FRANKEL, A, HARMSEN, S, AND OTHERS, 2008, DOCUMENTATION FOR THE 2008 UPDATE OF THE UNITED STATES NATIONAL SEISMIC HAZARD MAPS: US GEOLOGICAL SURVEY, OPEN FILE REPORT 2008-1128

GRI

ASSUMED
MAGNITUDE-FREQUENCY DISTRIBUTION
(CASCADIA SUBDUCTION ZONE)

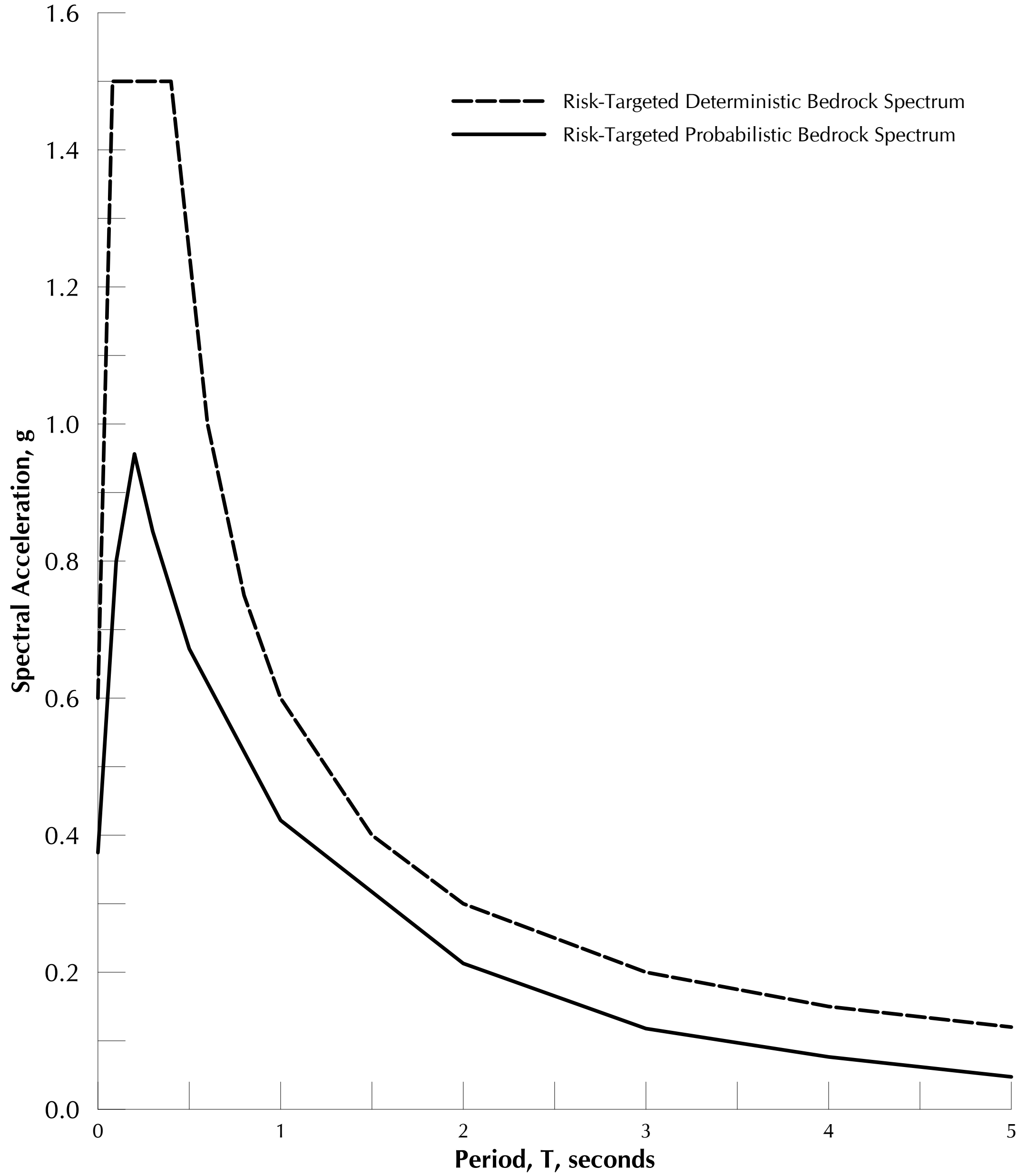
DEC. 2013 JOB NO. W1114 FIG. 5B



GRI

RISK-TARGETED DETERMINISTIC (MCE$_R$)
BEDROCK RESPONSE SPECTRA
(5% DAMPING)

DEC. 2013 | JOB NO. W1114-T2 | FIG. 6B

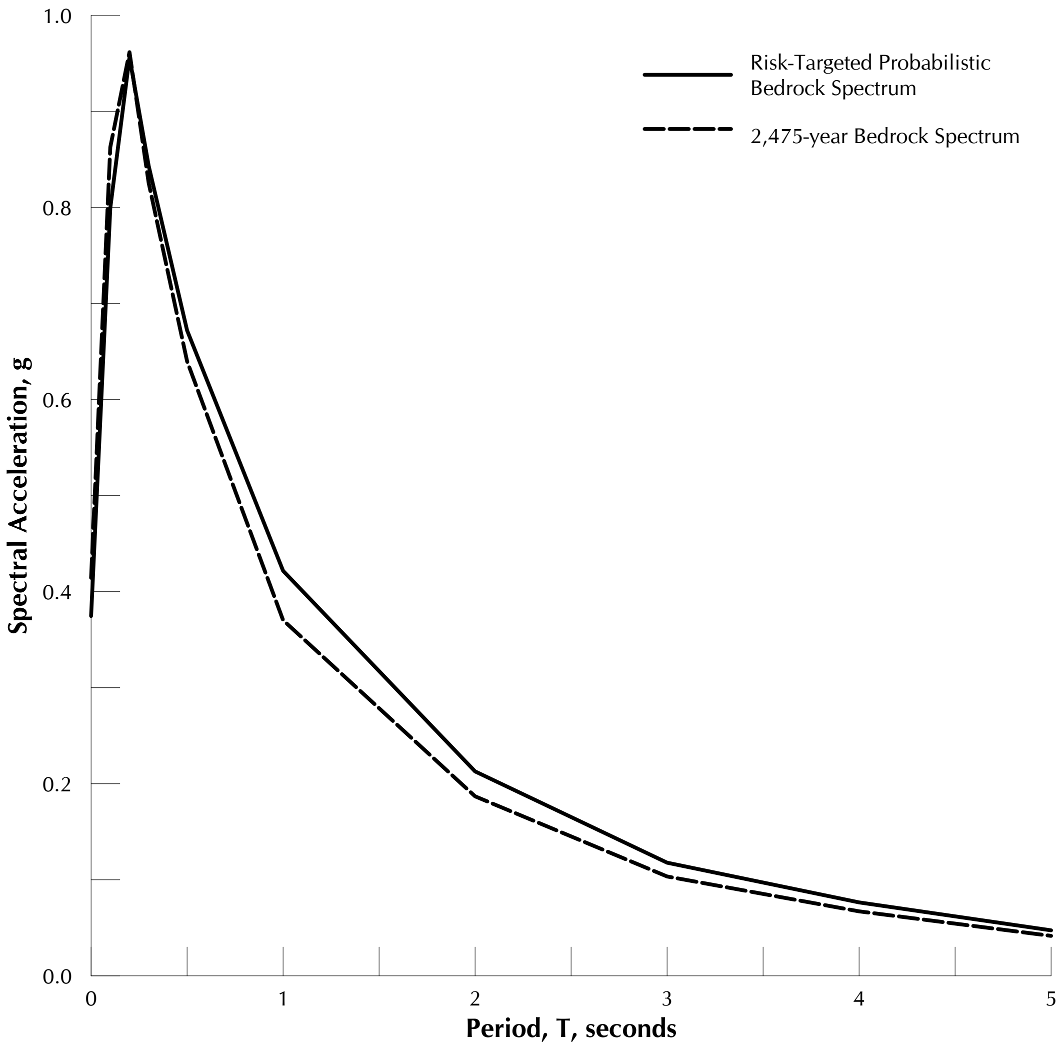


GRI

RISK-TARGETED DETERMINISTIC AND PROBABLILISTIC (MCE$_R$) BEDROCK RESPONSE SPECTRA

(5% DAMPING)

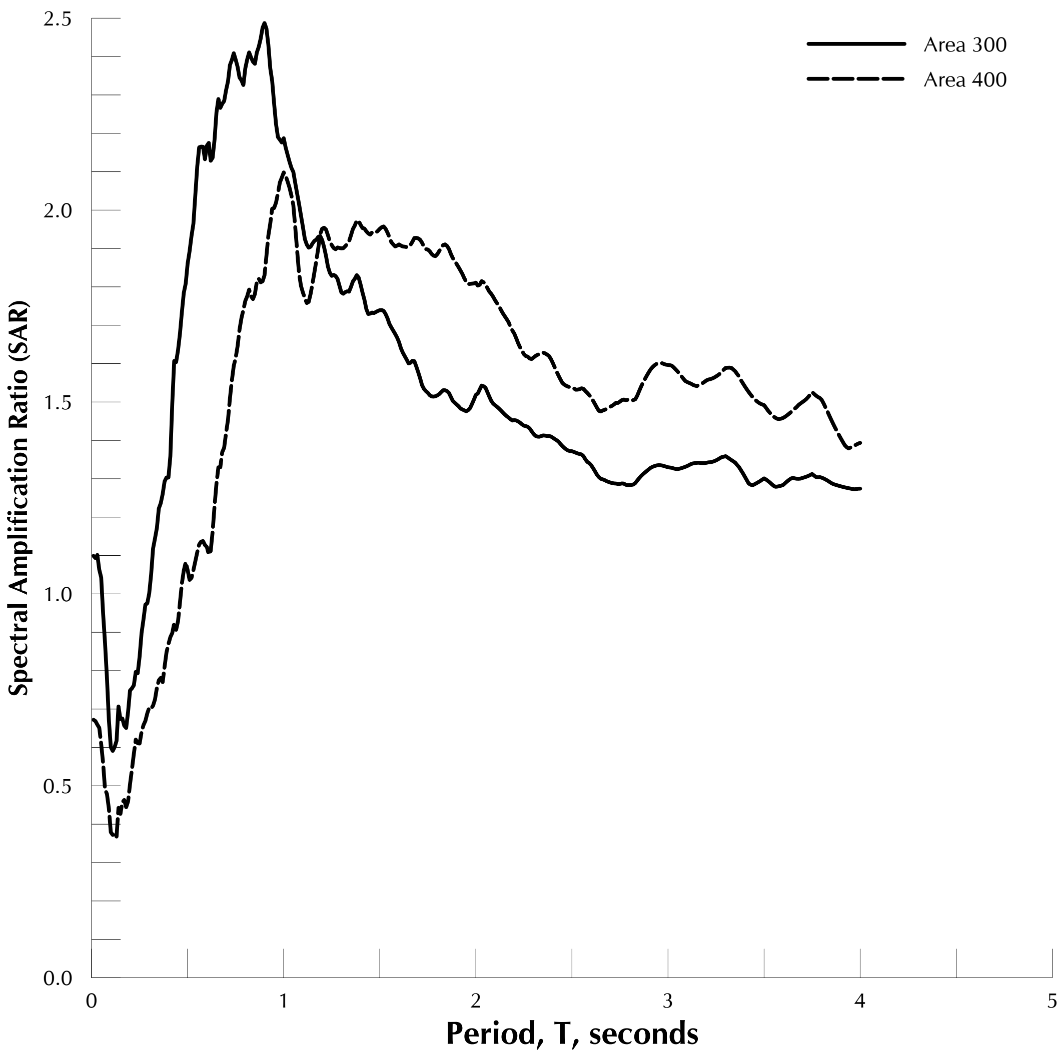
DEC. 2013 JOB NO. W1114-T2 FIG. 7B



GRI

COMPARISON OF RISK-TARGETED
PROBABILISTIC AND 2,475-YEAR HAZARD LEVEL
BEDROCK RESPONSE SPECTRA
(5% DAMPING)

DEC. 2013 JOB NO. W1114-T2 FIG. 8B

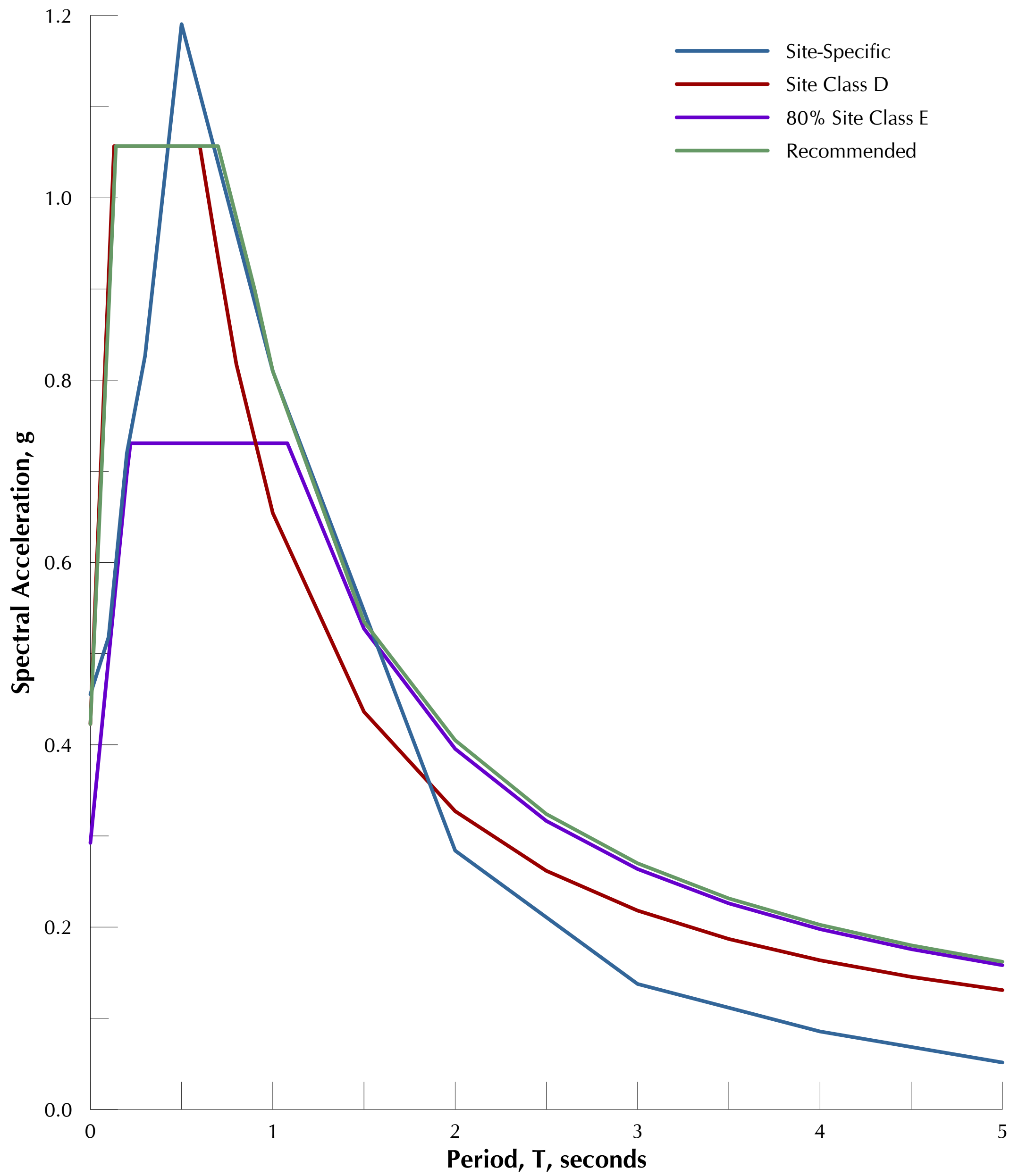


GRI

AREA 300 AND 400

SPECTRAL AMPLIFICATION RATIO (SAR)

DEC. 2013 JOB NO. W1114-T2 FIG. 9B



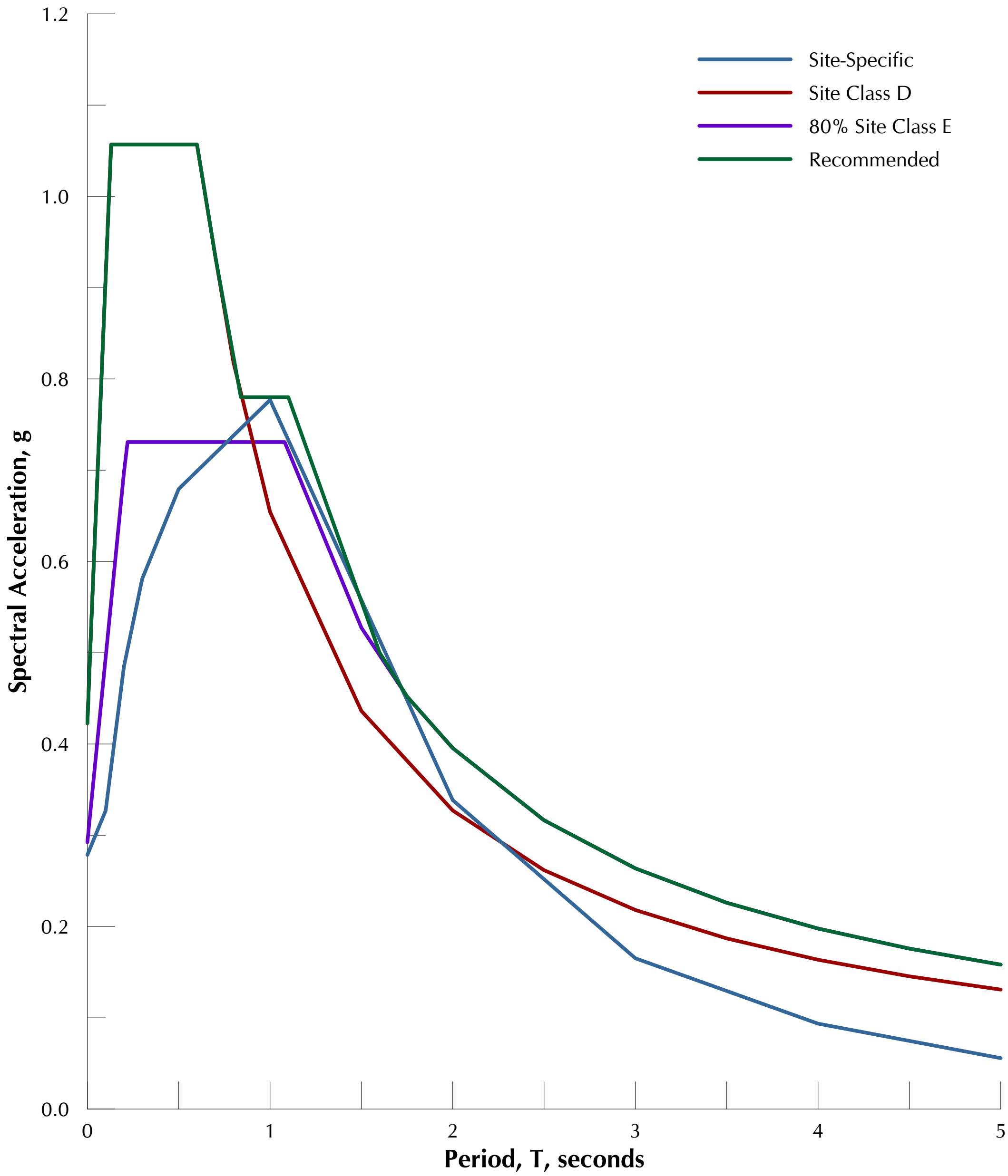
GRI

AREA 300

GROUND SURFACE DESIGN SPECTRA

(5% DAMPING)

DEC. 2013 JOB NO. W1114-T2 FIG. 10B

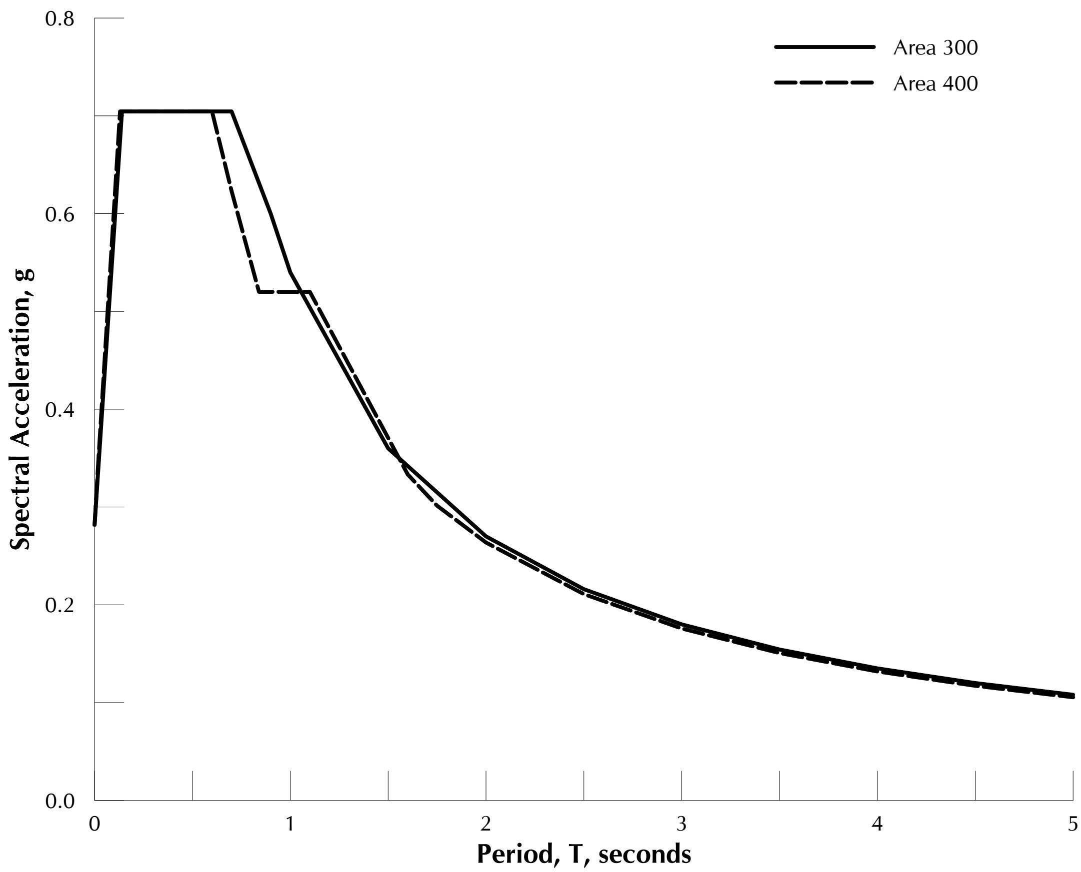


AREA 400

GROUND SURFACE RESPONSE SPECTRA

(5% DAMPING)

DEC. 2013 JOB NO. W1114-T2 FIG. 11B



GRI

AREA 300 AND 400

DESIGN RESPONSE SPECTRA

(5% DAMPING)

DEC. 2013 | JOB NO. W1114-T2 | FIG. 12B

Geotechnical Investigation

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Vancouver, WA 98660-2958

Tesoro Savage Vancouver Energy Distribution Terminal – Dock Facility
Port of Vancouver, USA

Prepared by

GRI
Washington & Oregon

September 5, 2014

TABLE OF CONTENTS

INTRODUCTION

At your request, GRI has completed a geotechnical investigation at the Port of Vancouver (Port) Berth 13 dock and trestle as part of the Tesoro Savage Vancouver Energy Distribution Terminal (TSVEDT). The Vicinity Map, Figure 1, shows the general location of the project. The investigation was conducted to evaluate subsurface conditions at the site and provide our conclusions and recommendations for design and construction of the proposed modifications. Our investigation has included a review of available geotechnical information, subsurface explorations, laboratory testing, and engineering analyses.

The following geotechnical information has been reviewed for this investigation:

> GRI, December 2013, Geotechnical Report, Tesoro Savage Vancouver Energy Distribution Terminal - Upland Facility, Port of Vancouver, USA; prepared for BergerABAM
>
> Dames and Moore, March 31, 1993, Geotechnical Investigation, Proposed T-Docks/Dolphins, Port of Vancouver, Washington; prepared for URS Consultants
>
> GRI, May 18, 2011, Geotechnical Report, NW Gateway Avenue Rail Bridge and Access to Terminal 5, Port of Vancouver, USA; prepared for HDR, Inc.
>
> Goble, Rauche, Likins and Associates, Inc., 1993, Dynamic Pile Measurements, October 11, 1993, T1 and T2 Dock, Port of Vancouver, Vancouver, Washington

PROJECT DESCRIPTION

Overview

Modifications to the existing Berth 13 dock and trestle are being designed to update the facility to current seismic design standards. Berth 13 consists of a T-shape dock and trestle, as shown on Figure 2. Proposed modifications will be primarily located between an area of proposed ground improvement on the riverbank and the Columbia River channel. We have assumed the area of ground improvement will include the trestle abutment. BergerABAM has indicated the channel depth in front of the dock is a minimum of elevation -43 ft Columbia River Datum (CRD). All elevations within this report reference NGVD29 datum, which is the project datum. Elevations can be converted to NGVD29 by adding approximately 1.78 ft to CRD.

Modifications include reconstruction of the existing deck; strengthening the existing dock, trestle, and mooring structures; and installation of ground anchors inside existing dock and dolphin piles. Landward of the Ordinary High Water Mark (OHWM), there will be two new dolphin pile structures located on the riverbank, new pile structures to support walkways that access existing dolphins, and new piles for the trestle abutment foundation. Additional improvements in the dock area (Area 400) are described in our above-referenced December 2013 report for the upland facility.

Seismic design of the dock modifications is intended to meet the requirements of the upcoming American Society of Civil Engineers (ASCE) standard, *Seismic Design of Pile-Supported Piers and Wharves*.

The proposed riverbank ground improvement is being designed by Hayward Baker, Inc., a ground improvement specialty contractor, to meet specified seismic performance criteria. The ground improvement design will affect the performance of the proposed Berth 13 modifications along the shore. We anticipate the impact of ground improvement on design of the dock elements will be further evaluated as the ground improvement design is developed.

SITE CONDITION AND BACKGROUND

Topography, Bathymetry, and Site Background

The existing site topography and bathymetry in the vicinity of Berth 13 is shown on Figure 2. The ground surface in the upland area located behind the trestle abutment is relatively flat at about elevation +27 ft and is typically surfaced with AC pavement, gravel, or grass. Two shallow stormwater infiltration swales with a bottom elevation of about +21 to +22 ft are located north of the paved areas and are mantled with grass and shrubs. The riverbank is protected with riprap and slopes down at about 2H:1V to a sandy beach at about elevation +17 ft. The sandy beach is relatively flat and slopes down at about 6H:1V to elevation +3 ft. Below elevation +3 ft, the slope increases to about 2.5H:1V. The existing mudline at the face of the dock is in the range of elevation -35 to -38 ft.

Geology

Based on our understanding of the geology at the site, our experience with nearby sites, and the available exploration data, the upland portion of the project area is mantled by fill that is underlain by recent alluvial soils. The fill typically consists of fine- to coarse-grained sand with silt, silty sand, sandy silt, and gravel. The alluvial soils beneath the fill and the mudline in the river typically consist of loose to medium dense sand and very soft to medium stiff silt. Gravel ranging from gravel in a matrix of sand to open-graded gravel with cobbles and possible boulders is present below elevations ranging from about -50 to -60 ft. Geologic investigations for the proposed Interstate 5 bridge replacement, about 3 miles upstream from the project site, indicate the alluvial gravels on the Washington side of the Columbia River can be up to 100 ft thick.

Available geologic information indicates the alluvial gravels are underlain by the Troutdale Formation, a Pliocene-age unit of well-consolidated or cemented conglomerate and sandstone (Beeson et al., 1991).

SUBSURFACE CONDITIONS

General

Subsurface materials and conditions at the dock site were investigated between July 29 and October 29, 2013, with four borings, designated B-23 through B-26, and one cone penetration test (CPT) probe, designated CPT-6. The borings were advanced to depths of 80 to 104 ft, and the probe was advanced to a depth of about 83 ft. The locations of the borings and CPT probe are shown on Figure 2.

The field exploration and laboratory testing programs completed for this investigation are described in Appendix A. Logs of the borings and CPT probe are shown on Figures 1A through 5A. The terms used to describe the soils encountered in the borings and CPTs are defined in Tables 1A and 2A.

In addition to the borings and CPT probe for this investigation, GRI also reviewed and utilized the logs of previous explorations made by GRI and others for other nearby projects. The soils encountered in the explorations for this investigation are consistent with previous investigations.

Soils

For the purpose of discussion, the materials disclosed by the explorations have been grouped into three units based on their physical characteristics, geologically significant features, and engineering properties. Listed as they were encountered from the ground surface downward, the units are:

1. **FILL**
2. **SAND**
3. **GRAVEL**

1. FILL. Fill was encountered at the ground surface in the upland explorations (Borings B-23 through B-25 and CPT-6) and extends to depths ranging from about 20 to 25 ft (about elevation +7 to +2 ft). Asphaltic-concrete pavement between 4 and 12 in.-thick over 10 to 24 in. of crushed rock base course was encountered at the ground surface in boring B-23 and B-25. Below the pavement and base course in boring B-23 and B-25, and at the ground surface of the other upland explorations, the fill consists of fine to coarse-grained sand with a trace of silt and scattered gravel and organic debris. Interbedded layers of silt and sandy silt up to about 4 in. thick were encountered in the fill. Based on N-values of 9 to 36 blows/ft and CPT tip resistances of about 50 to 240 tsf, the relative density of the fill ranges from loose to dense and is more typically medium dense. The moisture content of fill ranges from 7 to 27% and generally increases with increasing silt content.

2. SAND. Sand was encountered below the fill in the upland borings and at the ground surface in boring B-26. The sand extends to depths ranging from 64 to 87 ft (elevation -51 ft to -60 ft). The sand is fine to coarse grained and contains a trace to some silt. Interbedded layers of sandy silt and silt ranging from 1 in. to 5-ft-thick are present in the sand. The sand contains scattered a trace of gravel in all of the borings except boring B-24. Scattered wood debris was encountered below a depth of 25 ft in boring B-26. Based on N-values of 6 to 33 blows/ft and CPT tip resistances of about 50 to 150 tsf, the relative density of the sand typically ranges from loose to dense. More typically, the sand is medium dense with the exception of the sand in boring B-26 which has N-values of 2 to 10 blows/ft in the upper 40 ft indicating the upper 40 ft of sand is relatively loose. The moisture content of the sand ranges from 21 to 39%. The moisture content of silt layers ranges from 41 to 46%.

3. GRAVEL. Gravel was encountered beneath the sand in all four new borings. The gravel has a matrix of sand and silt and contains scattered cobbles and possible boulders. Loss of drilling fluid and caving of the borehole in Boring B-26 indicates that there are open-graded gravels. Based on N-values of 38 blows/ft and 50 blows for less than 6 in. of penetration, the relative density of the gravel ranges from dense to very dense.

Groundwater

Groundwater levels in the project area fluctuate in response to seasonal river levels, precipitation, and daily tidal fluctuations in the river. It should be anticipated the groundwater level in the project area will reflect the water levels in the Columbia River. Shallow perched groundwater conditions can develop above the less-permeable silty deposits at the site and approach the ground surface during periods of prolonged or intense rainfall.

The Columbia River level is lowest in late summer and early fall and is highest during winter storm events and the spring freshet, when snowmelt runoff causes high river flows. Historical low water in the last 20 years is about elevation +2.5 ft. The 100-year flood and OHWM is about elevation +27 and +17 ft, respectively.

CONCLUSIONS AND RECOMMENDATIONS

General

The borings and CPT probe completed for this investigation indicate the upland area is mantled with about 20 to 25 ft of sand fill that is underlain by alluvial sand and gravel that extend into the Columbia River. The top of a dense gravel unit ranges from about elevation -51 to -60 ft. Groundwater levels at the site will fluctuate in response to precipitation and levels in the nearby Columbia River. Shallow perched groundwater conditions may develop in the fill and approach the ground surface during periods of prolonged precipitation.

The primary geotechnical considerations for the Berth 13 modifications include axial and lateral capacity of proposed and existing piles and seismic hazard considerations. As discussed with BergerABAM, we understand new piles or foundation elements will be installed landward of the OHWM.

The loose to medium dense sand below the water table is liquefiable for the larger seismic hazard levels evaluated. Liquefaction of these soils will result in settlement, a reduction of soil strength, and significant lateral spreading near the riverbank. Lateral spreading is the horizontal displacement of large volumes of soil toward the river as a result of the liquefaction of underlying layers. Lateral ground movement will cause lateral loading and deformation of piles located within the zone of movement. Ground improvement is planned along the riverbank to mitigate the impacts of liquefaction-induced settlement and lateral spreading deformation on the TSVEDT facility improvements near the river. Although the ground improvement design is under development, we have assumed ground improvement will be completed upland of the trestle abutment to limit deformation of the abutment. Ground improvement will not be completed waterward of the OHWM; therefore, seismic-induced ground deformations will impact design of the riverside dock and trestle piles. Further evaluation of the dock modifications may be required as the ground improvement design is developed.

The following sections of this report provide our conclusions and recommendations for design and construction of the proposed modifications.

Seismic Considerations

General. The upcoming ASCE standard, *Seismic Design of Pile-Supported Piers and Wharves* (ASCE SSDPW), defines ground motions for three seismic hazard levels: the Operating Level Earthquake (OLE), the Contingency Level Earthquake (CLE), and the Design Earthquake (DE).

> OLE is defined by 50% probability of exceedance in 50 years, which corresponds to an earthquake with an expected recurrence interval of 72 years and represents a performance level with minimal structural damage.

CLE is defined by 10%probability of exceedance in 50 years, which corresponds to an earthquake with an expected recurrence interval of 475 years, and represents a performance level of controlled and repairable structural damage.

DE is defined per ASCE 7-05 which develops the response spectra based on ground motions associated with the Maximum Considered Earthquake (MCE), which is generally represented by a probabilistic earthquake with a 2% probability of exceedance in 50 years (return period of about 2,500 years), except where subject to deterministic limitations (Leyendecker et al., 2000). The design-level response spectrum that represents the DE is obtained by taking two-thirds of the MCE level ground motions.

The bedrock earthquake motions for each of the hazard levels were selected from the 2008 U.S. Geologic Survey (USGS) probabilistic seismic hazard maps for the coordinates of 45.65° N latitude and 122.71° W longitude. The code-based spectra are developed using two spectral response coefficients, S_S and S_1, corresponding to periods of 0.2 and 1.0 second. These bedrock spectral ordinates are adjusted for Site Class with the short- and long-period site coefficients, F_a and F_v, based on subsurface conditions or with a site-specific response analysis. A summary of the OLE, CLE, and DE hazard level S_S and S_1 coefficients for the dock are tabulated below.

SUMMARY OF SEISMIC DESIGN PARAMETERS

Hazard Level	S_S	S_1
Operating Level Earthquake (OLE)	0.11	0.03
Contingency Level Earthquake (CLE)	0.45	0.16
Design Earthquake (DE)	0.94	0.41

The site is generally designated as Site Class D based on the average shear wave velocity (V_s100) in the upper 100 ft per Section 20.4 of ASCE 7-05. Based on our evaluation, the seismic shaking from the OLE is insufficient to cause liquefaction, and the code-based Site Class D is recommended to estimate the ground response spectral acceleration. However, our analysis has identified a potential risk of liquefaction for the CLE and DE hazard levels. In accordance with ASCE SSDPW, sites with subsurface conditions identified as vulnerable to failure or collapse, such as liquefiable soils, shall be classified as Site Class F. For Site Class F sites, ASCE SSDPW Section 4.3.2 requires completion of a site-specific ground motion analysis for structures with a fundamental period of vibration greater than 0.5 second. BergerABAM has indicated the fundamental period of the dock is between 0.5 and 1.0 second. Due to these anticipated longer periods, a site-specific seismic ground motion analysis was completed for CLE and DE hazard levels at the dock and trestle area. The ground motion analysis was completed with the aid of the computer software D-MOD2000, a non-linear seismic soil response software developed by GeoMotions, LLC. The D-MOD2000 analyses are further discussed in Appendix B.

The site-specific response modeling results were compared with both Site Class D and E spectra due to the liquefaction considerations. The modeling indicates that 80% of code-based Site Class E spectral accelerations provide an appropriate estimate of the CLE hazard level in accordance with the ASCE SSDPW. The site-specific response modeling indicates the MCE hazard level spectral accelerations are greater than Site Class D spectral accelerations at periods less than 1.8 seconds. However, based on the research by Youd and Carter (2005), Site Class D is likely conservative for structures with a period less than 1 second. In this regard, the code-based Site Class D is recommended to estimate the spectral accelerations at short periods ($T < 0.89$ second). At periods between 0.89 and 1.8 seconds, we conservatively recommend the design spectrum include an increase above Site Class D and E to transition to longer periods and envelop the estimated site-specific ground surface response. At periods greater than approximately 1.8 seconds, the site-specific response spectrum is less than 80% of Site Class E, which is the minimum spectral amplification allowed by ASCE 7-05 for liquefied conditions. The DE is determined by taking two-thirds of the MCE. The results of our site-specific ground motion analysis, including plots of the spectral amplification ratio and recommended response spectra, are provided in Appendix B.

Liquefaction. Liquefaction is a process by which saturated, granular materials, such as sand, and non-plastic silts, temporarily lose strength during and immediately after a seismic event. Liquefaction occurs as seismic shear stresses propagate through a saturated soil and distort the soil structure causing loosely packed groups of particles to contract or collapse. If drainage is impeded and cannot occur quickly, the collapsing soil structure increases the pore water pressure between the soil grains. If the pore water pressure increases to a level approaching the weight of the overlying soil, the granular layer temporarily behaves as a viscous liquid rather than a solid. As strength is lost, there is an increased risk of settlement, lateral spread, and/or slope instability, particularly along waterfront areas. Liquefaction-induced settlement occurs as the elevated pore water pressures dissipate and the soil consolidates after the earthquake.

The potential for liquefaction at the site was evaluated with the simplified method based on two methodologies. The first methodology is based on the simplified procedure by Youd, et al. (2001). The analysis was completed with the aid of the computer software LiquefyPro, a seismically induced liquefaction and settlement analysis software developed by CivilTech Corporation. The second methodology is based on the simplified procedure by Idriss and Boulanger (2008). Both methodologies utilize the peak ground acceleration (PGA) adjusted for site amplification to estimate the cyclic shear stress ratio (CSR) experienced by the soil and in situ test data from the borings or CPTs to estimate the cyclic resistance ratio (CRR) of the soil. The factor of safety (FS) against liquefaction is estimated as the CRR divided by the CSR. The OLE PGA was based on the USGS 2008 interactive deaggregations. The CLE and DE PGAs were based on the results of site-specific ground motion analyses. The earthquake magnitudes chosen to represent the earthquake hazard levels for our liquefaction studies were based on the 2008 USGS interactive deaggregations for the OLE, CLE, and DE return intervals as well as the results of our site-specific ground motion analysis for the CLE and DE hazard level. The input values used for our liquefaction studies are tabulated below.

Hazard Level	PGA, g	Earthquake Magnitude, M
Operating Level Earthquake (OLE)	0.07	5.8
Contingency Level Earthquake (CLE)	0.26	8.4
Design Earthquake (DE)	0.28	9.0

For the purpose of liquefaction studies, we have conservatively assumed a groundwater level at elevation +12 ft, which corresponds to the seasonal high average daily river level. Based on our liquefaction studies, we estimate the risk of liquefaction for the OLE hazard level is low, i.e., the FS against liquefaction is greater than 1. The output from our liquefaction studies indicates the loose to medium dense sands below the groundwater level have a factor of safety against liquefaction less than 1 and could liquefy during the CLE and DE hazard level. A maximum free-field seismic settlement of about 24 in. was estimated for both the CLE and DE hazard levels based on the existing unimproved soil profile at boring B-26. The factor of safety against liquefaction for the CLE and DE hazard levels is summarized on Figures 3 and 4.

Lateral Spreading. Lateral spreading involves the horizontal displacement of large volumes of soil as a result of the liquefaction of underlying soil layers. Ground displacement occurs in response to the combination of gravitational forces and inertial forces generated by an earthquake acting upon the soil mass. Lateral spread can develop on shallow sloping ground or as a flow slide moving toward a moderately steep to steep free face, such as a river channel or lake bottom. Differential internal movement within the spreading mass usually creates surface features, such as ground cracks or fissures, scarps, and/or grabens, in overlying unsaturated or non-liquefied soils. Lateral displacement may range from a few inches to many feet depending on soil conditions, the steepness of the slope, and the magnitude and epicentral distance of the earthquake. Associated differential vertical movements, or ground surface subsidence, may range up to about half of the total horizontal movement.

The method of analysis summarized in Idriss and Boulanger (2008) were used to estimate lateral spreading deformations in free-field conditions. The methodology utilizes the same inputs as the simplified method for liquefaction hazard evaluation. Additionally, lateral spreading deformations were estimated using the methods of Youd et al. (2002). The basic inputs for Youd et al. (2002) include a characterization of the soil profile in terms of grain size, fines content (silt and clay), and Standard Penetration Test N-values; the overall geometry of the riverfront slope; and the magnitude and epicentral distance of the design-basis earthquakes. The risk of liquefaction at the OLE hazard level was estimated to be very low and therefore lateral spreading was not evaluated for the OLE. The range of estimates of lateral deformation for the CLE and DE hazard levels are tabulated below.

LATERAL SPREADING ESTIMATES
(without ground improvement)

Hazard Level	Earthquake Magnitude, M	Epicentral Distance, km	Estimated Range of Lateral Deformation, ft
Contingency Level Earthquake (CLE)	8.4	86	5 to 12
Design Earthquake (DE)	9.0	86	12 to 20

The methods used to estimate the seismically induced horizontal and vertical ground displacement at the site are largely based on empirical methods and, consequently, do not provide a precise estimate of the actual ground movement that may occur. Seismic events of a lesser magnitude, or of the same magnitude but occurring at a greater epicentral distance from the site, would be expected to produce lesser horizontal and vertical ground displacements.

Design Estimates for Lateral Displacement Forces. Earthquake-induced damage to waterfront structures at sites with liquefiable soils is well documented. Stresses induced on piles are typically generated from the inertial mass of the structure and lateral soil loading from both the lateral spreading liquefied soils and the non-liquefied crust of soil generally present above the groundwater table. Case histories have shown that the forces or displacements induced by the non-liquefied soil crust are generally significantly larger than the forces generated from the liquefied soils with reduced strengths. Design for the lateral spreading soils is typically completed by either applying estimates of soil displacements or forces to the structure. The displacement approach is commonly applied if the structure is somewhat flexible and can accommodate some deformation. The force approach is applied if the structure is more rigid and cannot accommodate the estimated movement. Based on discussions with BergerABAM, we understand the evaluation of the dock and trestle piles will likely be based on a force-based approach. Because the seismic lateral movements of the trestle abutment will be mitigated by ground improvement, we understand the abutment will be evaluated based on a displacement-based approach. The magnitude of displacement at the abutment will depend on the ground improvement design and should be evaluated as the design is developed. Preliminary earth pressures for the abutment are provided in the Trestle Abutment Earth Pressures section of this report.

A schematic cross section of the trestle alignment with the estimated lateral spreading failure surface is shown on Figure 5. The failure surface shown on Figure 5 assumes that sufficient ground improvement will be installed to limit lateral spreading at the trestle to the area below the OHWM at the CLE and DE hazard levels, which is discussed subsequently. Based on the soil profile and bathymetry, it should be assumed that lateral spreading will occur to a depth of about 35 ft below the mudline or ground surface, but no deeper than elevation -48 ft, for the CLE and DE hazard levels.

For the purpose of lateral spreading studies, we have assumed a groundwater level at elevation +7.5 ft that corresponds to the average level of the Columbia River. Based on our assumptions, the estimated soil movements will result in two different pressures acting on the trestle and dock piles: 1) unsaturated sand moving against the piles, and 2) saturated loose, liquefied sand moving against the piles. For the unsaturated sand above the ground water table, a lateral earth pressure based on an equivalent fluid unit weight of 400 pcf may be assumed to act against the trestle piles in the direction of the river for the CLE and DE hazard levels. Where the sand is saturated, a uniform rectangular pressure distribution of 250 psf may be assumed to act against the trestle piles in the direction of the river. In non-liquefied soils, passive lateral earth pressures tend to "arch" or develop a larger tributary area against piles. In this regard, we recommend assuming the lateral earth pressure against piles in the unsaturated sand material will act over an equivalent of two pile diameters. The lateral pressure against piles in the saturated liquefied sand material will act over one pile diameter. The recommended seismically induced lateral pressures are summarized on Figure 6. The extents of lateral spreading and the lateral spreading forces should be further evaluated as the ground improvement design is developed.

As noted in the ASCE SSDPW peak inertial forces do not necessarily occur at the same time as the peak soil (kinematic) loading on the structure. For this reason, consideration can be given to applying only a portion of the peak inertial loading at the same time as the kinematic loads. Design methods for evaluating combined inertial and kinematic loads are not well documented in the available literature, and ASCE SSDPW does not provide a recommended loading combination. However, another marine structures code, the Marine Oil Terminal Engineering and Maintenance Standards (MOTEMS), includes recommended load combinations for inertial and kinematic loading. In our opinion, it is reasonable to use inertial and kinematic load combinations similar to those provided in MOTEMS to evaluate the dock design.

Ground Improvement. Lateral spreading is often mitigated by constructing a zone, or buttress, of improved soil along the riverbank that will not liquefy. The buttress needs to be of sufficient width and extend to adequate depth to maintain stability following ground shaking and minimize or prevent lateral displacement toward the river of the upland portion of the site behind the buttress.

Due to the potential for large lateral spreading deformations, ground improvement will be designed and constructed to mitigate the impact of large seismic lateral displacements on the proposed transfer pipeline and structures located near the river.

Other Seismic Considerations. In our opinion, the potential for earthquake-induced fault displacement and ground rupture at the site is low unless occurring on a previously unknown or unmapped fault. Due to the topography of the site, it is our opinion the risk of damage by seiche is low. We are not aware of rigorous tsunami modeling for the Columbia River in available literature. However, based on the paper, "Tsunami Hydrodynamics in the Columbia River" (Yeh, 2012), the amplitude of potential tsunami waves at the Port is anticipated to be small at this distance from the Pacific Ocean.

Slope Grading and Protection

We understand the existing riprap and concrete debris slope protection will be left in place wherever possible. We anticipate this slope protection will have to be removed and replaced in the vicinity of the new piles to minimize the risk of obstructions during installation. Depending on the degree of disturbance and the subgrade conditions underlying the existing slope protection, a graded filter material may need to be placed prior to replacing the slope protection. GRI should observe the subgrade conditions during construction to recommend and appropriate filter gradation.

Pile Support

General. New 24-in.-diameter pipe piles (plumb and battered) are being considered for two proposed landside mooring dolphins, the reconstructed abutment, and walkway support piles. New driven piles will be located landward of the OHWM. Some strengthening of the existing 18-in.-diameter dock, trestle, and dolphin pipe piles is planned that includes installing grouted anchors or micropiles through the pipe piles into the gravel, and filling the piles with concrete. Based on discussions with the team, we understand the installation and testing of micropiles or grouted anchors will be through an assumed 10 1/2-in.-diameter hole in the plate at the tip of the existing pipe pile. Our recommendations regarding axial and lateral capacity of the new and existing piles are summarized below.

Proposed Dolphin Piles. The locations of two proposed landside mooring dolphins are shown on Figure 2. At the request of BergerABAM, GRI evaluated 24-in.-diameter pipe piles at the proposed dolphin locations. We anticipate the piles will be driven open-end into the underlying gravel unit. We estimate 24-in.-diameter pipe piles driven with sufficiently large hammers to adequate penetration resistance can develop an ultimate compression capacity of at least 750 kips. The explorations and our experience in the area indicate the sand content and the relative density of the underlying gravel unit tend to be highly variable. As a result, it is difficult to accurately predict the actual penetration of piles into the gravel to develop the estimated ultimate capacities. However, based on our experience, we anticipate the piles will develop the design capacity at a depth of about 15 ft into the gravel plus or minus 10 ft. Assuming penetration of 15 ft into the dense gravel, the resulting pile tips will be at about elevation -68 to -70 ft based on the available explorations. Due to the variation in subsurface conditions we recommend the piles are ordered at least 10 ft longer than the estimated tip elevation to limit the risk of splices.

Since a significant portion of the estimated ultimate compression pile capacity is derived from end-bearing resistance, the uplift capacity of the piles will be significantly less than the compressive capacity. For preliminary purposes, we estimate the 24-in.-diameter piles can develop an ultimate vertical uplift capacity of about 300 kips assuming the piles will penetrate a minimum of 15 ft into relatively dense gravel.

As discussed in the Liquefaction section of this report, the sand below the groundwater level is subject to liquefaction and/or seismically induced strength loss during the two larger hazard level events. Liquefaction-induced settlements will result in downdrag loads on the piles. In this regard, we recommend assuming a downward skin friction acting along the outside perimeter of the piles of 250 psf above elevation +12 ft and 150 psf from elevation +12 ft to the top of the gravel unit. The top of the gravel unit is estimated at about elevation -57 ft. For seismic conditions the ultimate axial capacities estimated above should be reduced to 280 kips in compression, and 170 kips uplift to account for seismically-induced loss of strength. For piles located within the zone of ground improvement, the depth of liquefaction and magnitude of the downdrag loads will depend on the ground improvement design. The effects of ground improvement on the potential for downdrag induced loads should be evaluated as the ground improvement design is developed.

Recommended factors of safety for allowable capacity are discussed in the Recommended Factor of Safety section of this report.

Trestle Abutment Piles. As currently planned, new 24-in.-diameter pipe piles are being considered at the reconstructed trestle abutment to resist seismic inertial loads from the dock and trestle structure. We have assumed that ground improvement will be designed to extend around the abutment piles to a sufficient depth to mitigate liquefaction and lateral spreading. In this regard the non-seismic axial capacity estimates presented in the previous section for dolphin piles can be used to preliminarily design abutment piles for the non-seismic and seismic loading conditions. The pile capacities at the abutment should be reevaluated as the ground improvement design is developed.

Existing Trestle, Dock, and Dolphin Piles. The existing piles are 18-in.-diameter pipe piles with $^3/_8$-in.-thick walls. The piles were reportedly fitted with a type of endplate that included a $10^1/_2$-in.-diameter center cut-out. The trestle piles are all plumb piles. The dock and dolphin piles are a combination of plumb and batter (5H:12V and 6H:12V) piles. The total embedded length of the piles typically ranged from about 34 to 100 ft depending primarily on the mudline elevation at the pile location. The piles at the dock typically have the least embedment and range from about 34 to 52 ft. The length of pile embedment into the gravel unit is typically in the range of 10 to 25 ft.

GRI evaluated the axial capacity of the existing piles based on available geotechnical data including pile driving logs and results of Pile Driving Analyzer (PDA) re-strike testing completed when the piles were installed. The PDA results are summarized in the above-referenced report by GRL (1993). The ultimate capacity computed from restrike PDA tests ranges from 435 to 550 kips. The PDA estimated skin friction resistance values of 210 kips and 225 kips for piles 91 and 96, respectively. Pile 91 and 96 were embedded 36 and 48 ft, and it is our opinion that the estimated ultimate skin friction is representative for the range of pile embedment at Berth 13.

Based on the wide variation in pile embedment depths we recommend an ultimate compression capacity of 435 kips. We estimate the pile tip provides about 200 kips of resistance and the skin friction on the outside of the pile provides the remaining resistance. Since a significant portion of the estimated ultimate compression pile capacity is derived from end-bearing resistance on the partially closed-end plate, the uplift capacity of the piles will be significantly less than the compressive capacity. To consider decreased skin friction values for tension piles relative to compression piles (FHWA, 2006), we recommend an ultimate uplift capacity of about 150 kips for the range of pile embedments at Berth 13.

As discussed in the Liquefaction section of this report, the sand below the groundwater level is subject to liquefaction and/or seismically induced strength loss. Liquefaction-induced settlements will result in downdrag loads on the piles. For the existing dock and trestle piles, we recommend assuming a downward skin friction of 100 psf acting along the outside perimeter of the piles from the mudline to the top of the gravel unit. The top of the gravel unit can be estimated at about elevation -57 ft for this purpose. For seismic conditions, the ultimate axial capacities estimated above should be reduced to 400 kips (compression) and 100 kips (uplift) to account for seismically induced loss of strength.

Recommended Factor of Safety. We recommend applying a factor of safety to the ultimate pile capacities provided above based on soil support properties. A minimum FS of 2 is recommended for typical non-seismic conditions. For seismic conditions we recommend a factor of safety of 1 in accordance with the ASCE SSDPW. For extreme moorage loading combinations such as flood and extreme wind conditions a FS of 1.5 is appropriate for dolphin piles.

Pile Installation. The new dolphin piles should be driven with a pile hammer of sufficient energy to develop the pile ultimate capacity and obtain adequate penetration into the gravel. During installation, we anticipate the top of the gravel can be identified by a noticeable increase in penetration resistance. Based on our experience on adjacent sites with similar soils, a vibratory hammer will likely not be effective in penetrating the gravel layer to the design embedment. In this regard, the contractor should assume final installation of steel pipe piles will require an air, steam, or diesel, impact hammers.

As discussed previously, our experience in the area indicates the driving resistance in the gravel tends to be highly variable. There is some risk that piles may encounter practical refusal before reaching the planned tip elevations.

We recommend an indicator-pile installation program as the initial step in the installation of production piles. The purpose of the program would be to evaluate the contractor's equipment and develop terminal pile driving resistance criteria. We recommend dynamic pile testing of two or three piles with the PDA during initial driving. As a guide, if 24-in.-diameter piles are being considered for the pile program, we recommend using a pile hammer with rated energy of at least 100,000 ft-lbs. The contractor should provide an impact hammer submittal and installation plan for the project team to review at least 2 weeks prior to pile driving.

We understand the pipe piles will likely have a 0.5-in. wall thickness. All piles should be fitted with commercially available tip protection that fit flush with the outside wall of the pile.

Lateral Pile Capacity. Lateral structural loads can be resisted by the piles in bending. The lateral load behavior of the piles can be analyzed using the computer program L-Pile 5.0 by Ensoft, Inc. We recommend using the input parameters summarized in the following tables to model the soils at the site.

SOIL PROPERTIES FOR L-PILE ANALYSIS (1)
Berth 13 Dock, Trestle, and Riverbank Dolphin Piles

Static, Frequently Applied Live Loads, and Operating Level Earthquake (72-year Return Interval)(2)

Soil Unit	Elevation, ft	L-Pile Soil Type	Condition	Soil Properties				
				K, pci	γ′, pci	ϕ′	c, psi	ε_{50}
Fill: SAND	Above +17	Sand (Reese)	Non-Liquefied	25	0.067	32	N/A	N/A
Submerged Fill: SAND	+17 to +2	Sand (Reese)	Non-Liquefied	20	0.03	32	N/A	N/A
Submerged SAND	+2 to -38	Sand (Reese)	Non-Liquefied	20	0.030	32	N/A	N/A
Submerged SAND	-38 to -57	Sand (Reese)	Non-Liquefied	60	0.030	35	N/A	N/A
Submerged GRAVEL	Below -57	Sand (Reese)	Non-Liquefied	125	0.040	40	N/A	N/A

Contingency Level and Design Level Earthquakes (475-year and 2/3 MCE Return Intervals, respectively) (3) (4)

Soil Unit	Elevation, ft	L-Pile Soil Type	Condition	Soil Properties				
				K, pci	γ′, pci	ϕ′	c, psi	ε_{50}
Fill: SAND	Above +7.5	Sand (Reese)	No Lateral Resistance	N/A	0.067	N/A	N/A	N/A
Submerged Fill: SAND	+7.5 to +2	Sand (Reese)	Liquefied	10	0.030	6	N/A	N/A
Submerged SAND	+2 to -38	Sand (Reese)	Liquefied	10	0.030	6	N/A	N/A
Submerged SAND	-38 to -57	Sand (Reese)	Liquefied	10	0.030	12	N/A	N/A
Submerged GRAVEL	Below -57	Sand (Reese)	Non-Liquefied	125	0.040	40	N/A	N/A

Notes:

1) Applicable for the river side of the trestle abutment.
2) Design groundwater level assumed at OHWM Columbia River Elevation +17 ft (NGVD).
3) Design groundwater level assumed at Average Columbia River Elevation +7.5 ft.
4) Lateral spreading occurs to depths/elevations shown on Figure 5.
5) Submerged soils are below the design groundwater level.

For preliminary planning we have assumed that ground improvement will be installed around the abutment and will be sufficient to mitigate liquefaction. In this regard, the lateral resistance of abutment piles can be estimated based on the static L-Pile parameters presented in the above table. Ground slope effects can be taken into consideration with the input of an appropriate slope angle.

It should be noted that L-Pile provides isolated, single-pile capacities. Depending on the direction of the loading and orientation of the piles, group effects should be considered for spacings less than eight (8) pile diameters. This reduction is often applied as a group efficiency or a p-multiplier. L-Pile uses a p-multiplier as a reduction of the k_h value for pile spacing less than eight pile diameters. The following table provides a summary of p-multipliers and reported group efficiencies in sand.

LATERAL PILE GROUP ANALYSIS

Center-to-Center Pile Spacing	Calculated p-multipliers for Rows 1, 2, and 3	Reported Group Efficiency
3d	0.80, 0.40, 0.30	0.75
5d	1.0, 0.85, 0.70	0.95
8d	1.0, 1.0, 1.0	1.0

Caution should be used when applying the reported group efficiencies to pile groups with significantly more than three rows of piles. Additional design methodology of laterally loaded pile groups is provided in the Federal Highway Administration (FHWA) publication entitled, "Design and Construction of Driven Pile Foundations."

Micropiles or Ground Anchors. Micropiles or ground anchors drilled through the tip of the existing piles are being considered to increase the existing pile capacity. We estimate micropiles or ground anchors in the gravel may develop ultimate capacities on the order of 350 kips. Larger capacities may be possible but could be limited by structural design considerations. The micropiles or ground anchors would be drilled and installed through the existing pipe piles and into the underlying gravel.

Drilling through the gravel at the tip of the existing pipe piles will alter the pile tip resistance. For preliminary planning purposes, we recommend considering no tip resistance for the existing pipe piles if micropiles or ground anchors are installed through the pile tip. Installation of micropiles or ground anchors will not reduce the skin friction of the existing pipe piles.

Grouted micropiles or ground anchors are typically designed by specialty contractors to meet specified performance criteria. As a general guideline, we recommend a minimum bonded length of 30 ft into the relatively dense gravel. Micropiles or ground anchors should be designed with a minimum factor of safety of 1.5 in compression and 2 in tension, based on soil support properties.

Micropile or ground anchor capacities should be evaluated by field testing which will be challenging because the existing piles are supporting the dock structure. The maximum test loads and the number of test anchors will depend on the total number and design of the micropiles or ground anchors. The testing program should be developed with the final plans and specifications. The unbonded and bonded lengths may need to be modified based on the actual gravel conditions encountered during drilling and the contractor's equipment and procedures. Micropiles and ground anchors should be provided with permanent corrosion protection.

Trestle Abutment Earth Pressures

Design lateral earth pressures for the trestle abutment depend on the type of construction, i.e., the ability of the abutment to yield. For static conditions, we anticipate the trestle abutment will be relatively rigid and can be designed to resist an at-rest lateral earth pressure computed on the basis of an equivalent fluid having a unit weight of 50 pcf. Additional lateral earth pressures due to surcharge loadings may be estimated using the guidelines presented on Figure 7.

Additional lateral loads due to seismic forces on retaining walls will be dependent on design of the ground improvement around the abutment and the seismic hazard level. Our studies indicate that liquefaction induced seismic deformations are not likely for the OLE hazard level. Based on this assumption, the abutment may be designed on the basis of static at-rest earth pressure plus an additional seismic load increment during the OLE seismic event. The additional load due to seismic forces can be evaluated based on a rectangular lateral earth pressure distribution with a uniform pressure equal to $H*\gamma$, where H is the height of the wall and γ is an equivalent unit weight of 2 pcf for the OLE.

We have assumed that ground improvement will be designed around the trestle abutment to mitigate liquefaction and lateral spreading loads and deformations at the CLE and DE hazard levels to meet a specified ground movement performance criteria. We understand that the final seismic performance criteria have not yet been selected. Seismic lateral ground deformations toward the river will cause lateral spreading loads on the abutment wall and foundations. Lateral spreading loads should be applied to the wall and piles until the wall has displaced the amount specified by the performance criteria. For preliminary planning, lateral spreading loads can be estimated based on an equivalent fluid unit weight of 400 pcf for the unsaturated sand above elevation 7.5 ft. Below elevation 7.5 ft where the sand is saturated and susceptible to liquefaction, a uniform rectangular pressure distribution of 250 psf may be assumed to act in the direction of the river. The depth of seismic deformation and lateral spreading loads will depend on the ground improvement design and the seismic design of the abutment should be re-evaluated once the ground improvement design is complete. We recommend assuming the lateral earth pressure against piles in the unsaturated sand material will act over an equivalent of two pile diameters due to arching. The lateral pressure against piles in the saturated liquefied sand material will act over one pile diameter. Once the seismic deformation has occurred, the earth pressures can be estimated based on the static at-rest earth pressures provided above.

Based on our discussion with the project team, we understand lateral loads at the trestle abutment may be resisted by battered piles or tieback anchors. Anchor capacities can be provided during final design if needed.

The above criteria assume drained conditions and that the abutment is backfilled with relatively clean, granular material, i.e., medium sand, sand and gravel, or well-graded gravel, with not more than 5% passing the No. 200 sieve (washed analysis). We recommend that this material be compacted to about 95% of the maximum dry density as determined by ASTM D 698. Heavy compaction equipment should not operate within 5 ft of the abutment.

Design Review and Construction Services

We welcome the opportunity to review and discuss construction plans and specifications for this project as they are being developed. In addition, GRI should be retained to review all geotechnical-related portions of the plans and specifications to evaluate whether they are in conformance with the recommendations provided in our report. Additionally, to observe compliance with the intent of our recommendations, design concepts, and the plans and specifications, we are of the opinion that all construction operations dealing with pile installation should be observed by a GRI representative. Our construction-phase services will allow for timely design changes if site conditions are encountered that are different from those described in this report. In our opinion, this is of particular importance during pile-driving operations. If we do not have the opportunity to confirm our interpretations, assumptions, and analyses during construction, we cannot be responsible for the application of our recommendations to subsurface conditions that are different from those described in this report.

LIMITATIONS

This report has been prepared to assist BergerABAM and the design team in the design of this project. The scope is limited to the specific project and location described herein. Our description of the project represents our understanding of the significant aspects of the project relevant to the design and construction of the trestle, platform, dolphins, and other supports. In the event that any changes in the design and location of the facilities, as outlined in this report, are planned, we should be given the opportunity to review the changes and to modify or reaffirm the conclusions and recommendations of this report in writing.

The analyses and recommendations submitted in this report are based on the data obtained from the borings made at the locations indicated on the Site Plan and from other sources of information discussed in this report. In the performance of subsurface investigations, specific information is obtained at specific locations at specific times. However, it is acknowledged that variations in soil conditions may exist between the boring locations, and groundwater levels fluctuate seasonally. This report does not reflect any variations that may occur between these explorations. The nature and extent of variations may not become evident until construction. If, during construction, subsurface conditions different from those encountered in the exploratory holes are observed or encountered, or appear to be present beneath or beyond foundations, we should be advised at once so that we can observe these conditions and reconsider our recommendations where necessary.

Submitted for GRI,

MATTHEW S. SHANAHAN
STATE OF WASHINGTON
44077
REGISTERED
PROFESSIONAL ENGINEER

Expires 4/2016

Matthew S. Shanahan, PE
Associate

Scott M. Schlechter, PE
Principal

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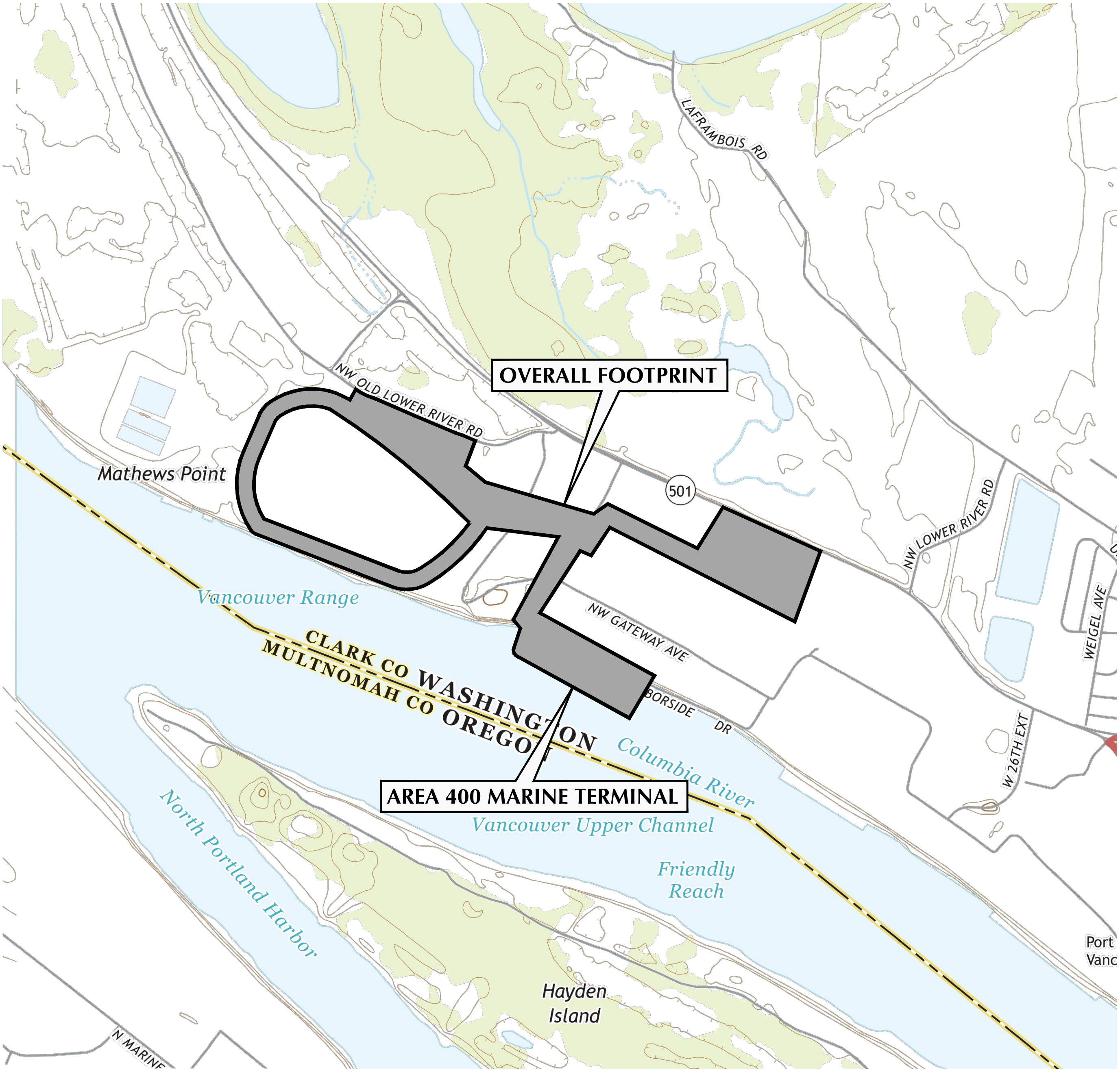

USGS TOPOGRAPHIC MAP
VANCOUVER, WASHINGTON (2011)

North

0 1/2 1 MILE

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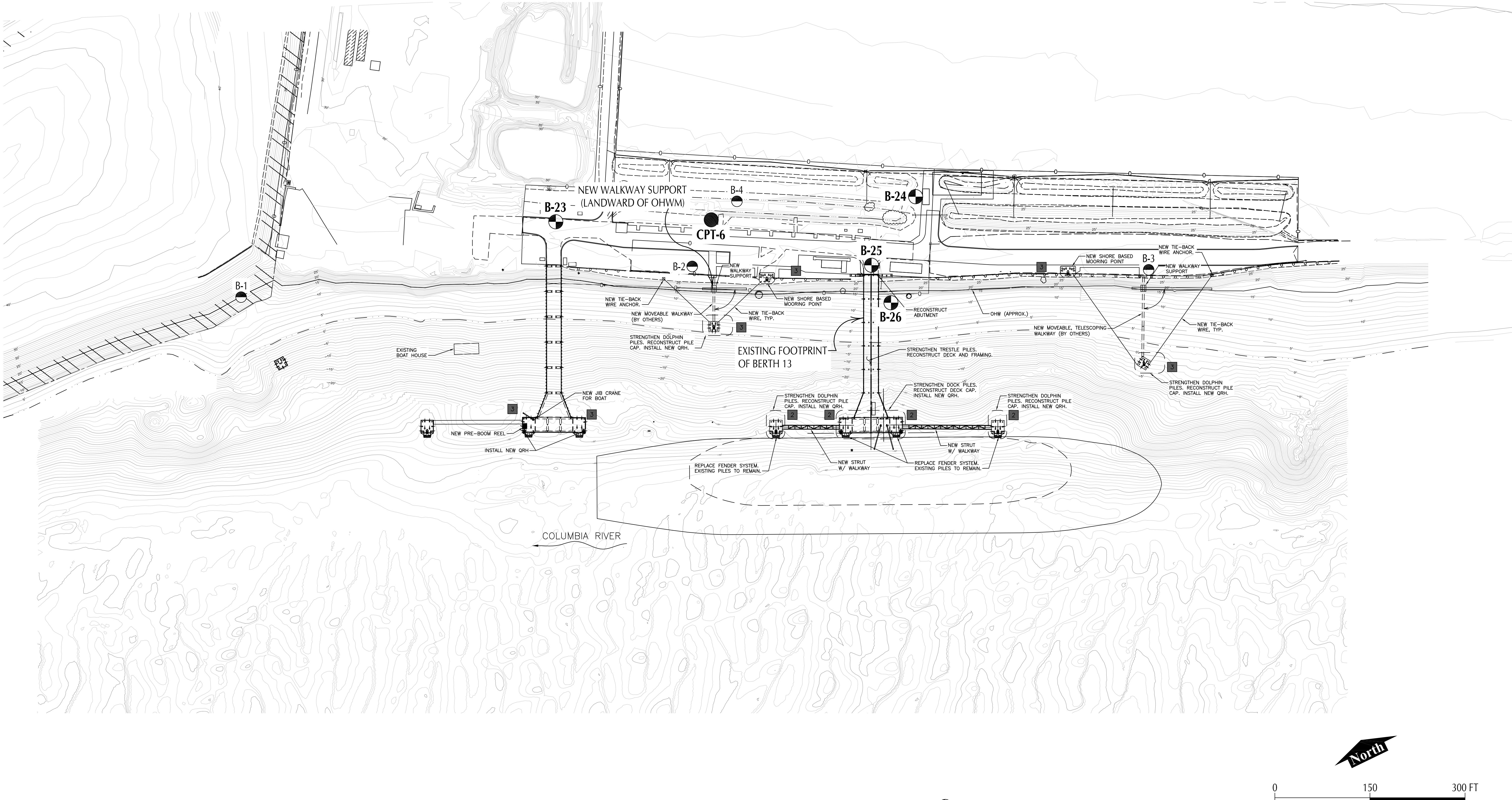
TSVEDT DOCK STRUCTURE

VICINITY MAP

SEP. 2014 JOB NO. W1114-T3 FIG. 1



- BORING MADE BY GRI (JULY 29 - OCTOBER 29, 2013)
- CONE PENETRATION TEST MADE BY GRI (JULY 29 - AUGUST 5, 2013)
- BORING MADE BY DAMES & MOORE (1993)

SITE PLAN FROM FILE BY MACKAY + SPOSITO

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SITE PLAN

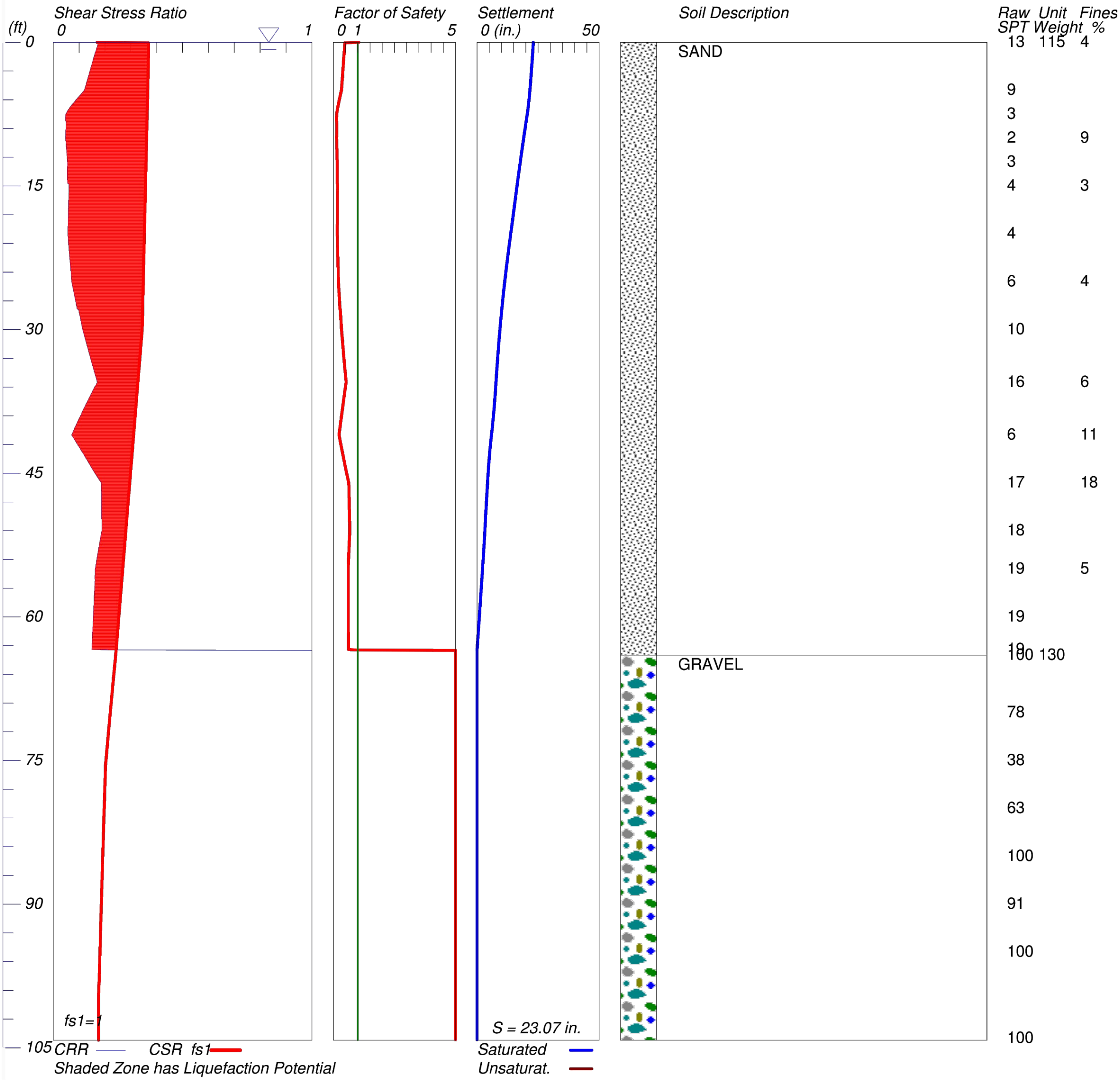
SEP. 2014 JOB NO. W1114-T3 FIG. 2

LIQUEFACTION ANALYSIS

Tesoro Savage Vancouver Energy Distribution Terminal

Hole No.=B-26 Water Depth=0 ft Surface Elev.=7

Magnitude=8.4
Acceleration=0.26g



Contingency Level Earthquake (CLE)

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LIQUEFACTION HAZARD ANALYSIS

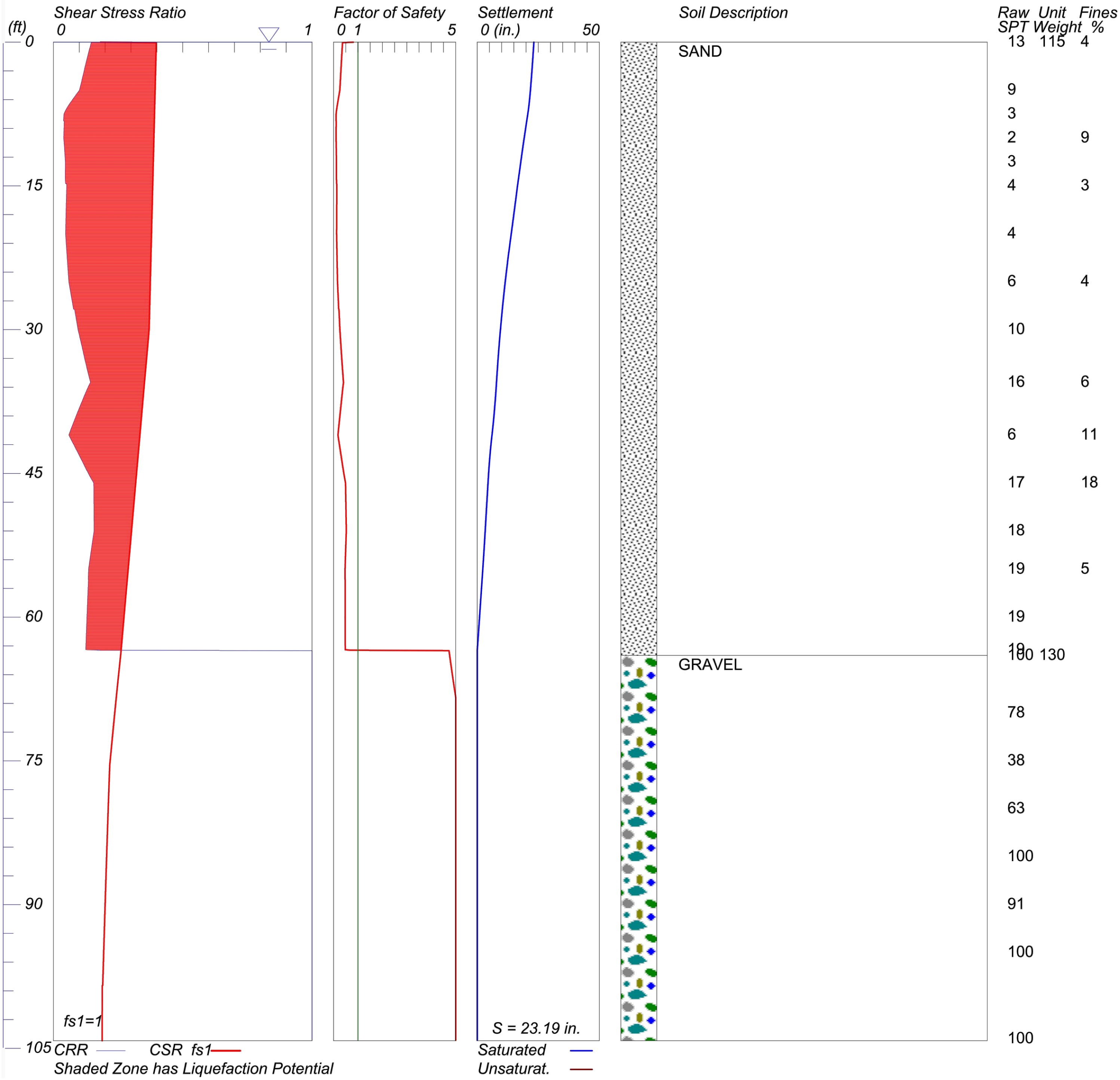
SEP. 2014 JOB NO. W1114-T3 FIG. 3

LIQUEFACTION ANALYSIS

Tesoro Savage Vancouver Energy Distribution Terminal

Hole No.=B-26 Water Depth=0 ft Surface Elev.=7

Magnitude=9.0
Acceleration=0.28g

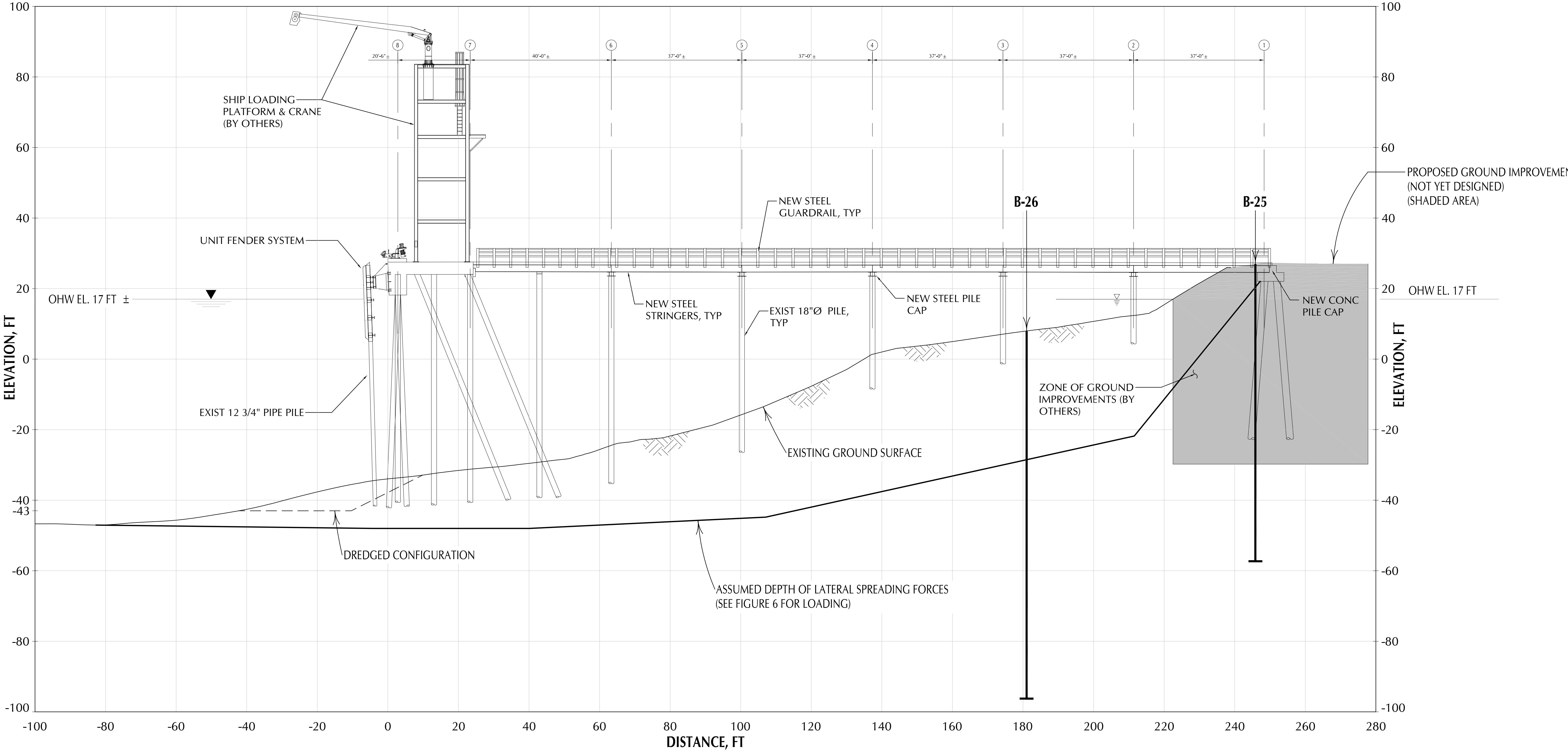


Design Earthquake (DE)

GRI BERGERABAM TSVEDT DOCK STRUCTURE

LIQUEFACTION HAZARD ANALYSIS

SEP. 2014 JOB NO. W1114-T3 FIG. 4



NOTES:

1) THE EXTENT OF LATERAL SPREAD SHOULD BE FURTHER EVALUATED AS THE GROUND IMPROVEMENT DESIGN IS DEVELOPED.

2) DATUM IS NGVD29.

3) EXISTING PILE TIPS ESTIMATED TO RANGE FROM ELEVATION -69 TO -93 FT BASED ON AVAILABLE PILE DRIVING RECORDS.

ELEVATION LOOKING WEST

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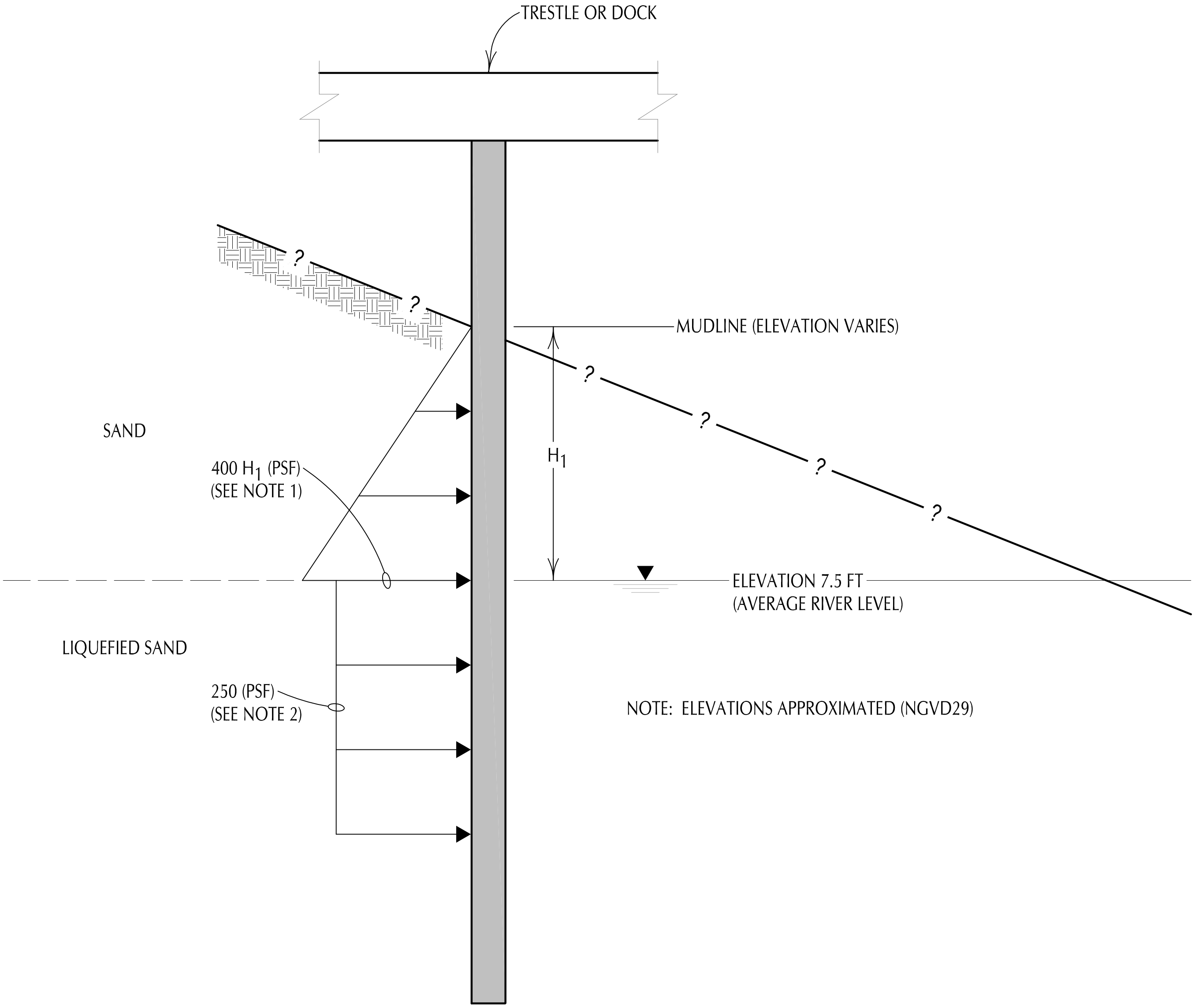
BERTH 13 TRESTLE AND DOCK PROFILE

SEP. 2014 JOB NO. W1114-T3 FIG. 5



NOTES:

1) ACTS OVER TWO PILE DIAMETERS.

2) ACTS OVER ONE PILE DIAMETER. LATERAL SPREADING FORCES ARE ASSUMED TO ACT ON THE PILES TO THE DEPTH SHOWN ON FIGURE 3.

3) FORCES ARE FOR CONTINGENCY LEVEL (475 YEAR RETURN PERIOD) AND DESIGN EARTHQUAKE (⅔ MCE) HAZARD LEVELS.

4) THE LATERAL SPREADING FORCES INDICATED ABOVE ARE FOR TRESTLE AND DOCK PILES ONLY. LATERAL SPREADING FORCES ON THE TRESTLE ABUTMENT WILL DEPEND ON THE GROUND IMPROVEMENT DESIGN WHICH HAS NOT BEEN COMPLETED AT THIS TIME.

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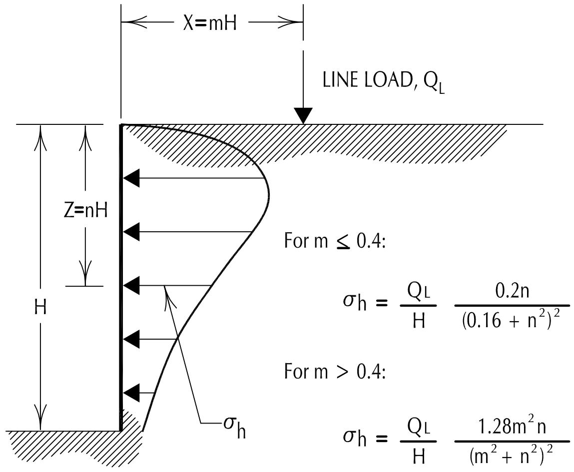
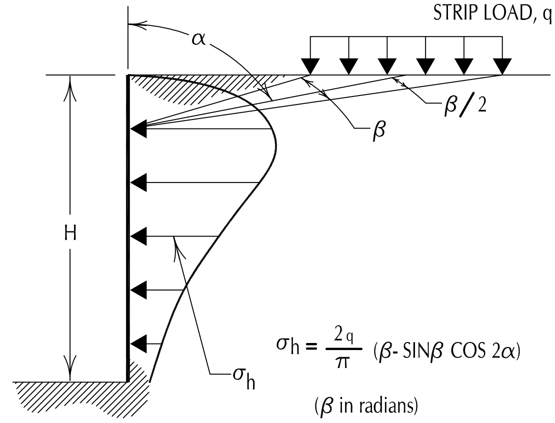
TSVEDT DOCK STRUCTURE

LATERAL SPREADING LOADS TRESTLE AND DOCK PILES

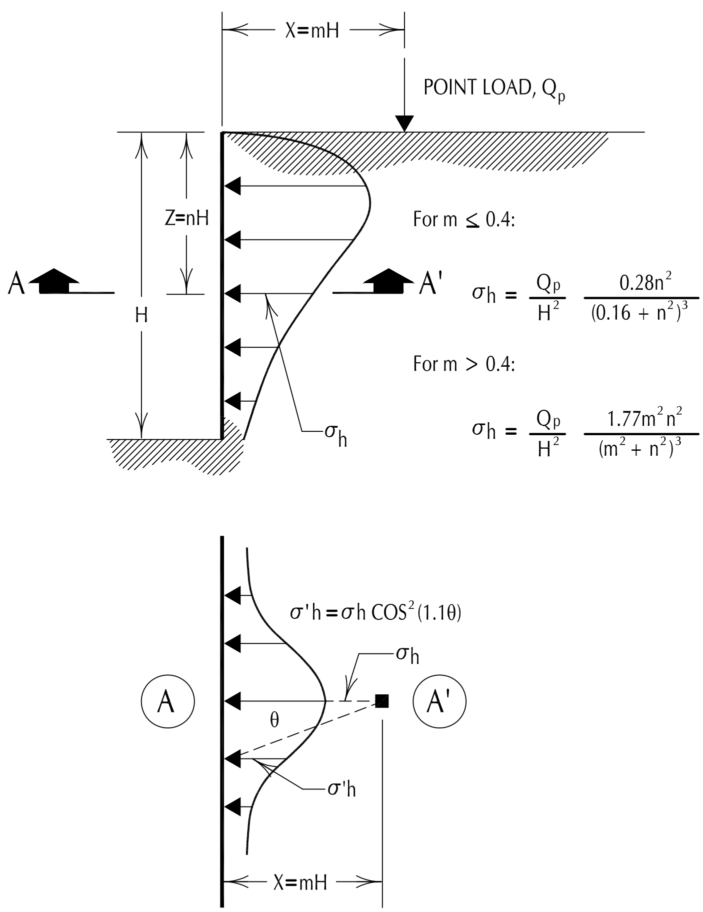
SEP. 2014 JOB NO. W1114-T3 FIG. 6



LINE LOAD PARALLEL TO WALL



STRIP LOAD PARALLEL TO WALL



DISTRIBUTION OF HORIZONTAL PRESSURES

VERTICAL POINT LOAD

NOTES:

1. THESE GUIDELINES APPLY TO RIGID WALLS WITH POISSON'S RATIO ASSUMED TO BE 0.5 FOR BACKFILL MATERIALS.

2. LATERAL PRESSURES FROM ANY COMBINATION OF ABOVE LOADS MAY BE DETERMINED BY THE PRINCIPLE OF SUPERPOSITION.

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SURCHARGE-INDUCED LATERAL PRESSURE

SEP. 2014 | JOB NO. W1114-T3 | FIG. 7

APPENDIX A

Field Explorations and Laboratory Testing

APPENDIX A

FIELD EXPLORATIONS
AND LABORATORY TESTING

FIELD EXPLORATIONS

General

Subsurface materials and conditions in the marine terminal project area were investigated between July 29 and October 29, 2013 with four borings and one electric cone penetration test (CPT) probe. The approximate locations of the explorations are shown on Figure 2. An experienced geotechnical engineer from GRI directed the drilling and maintained a detailed log of the materials and conditions disclosed during the course of the work. The locations of the borings with respect to areas of the proposed facility are discussed below.

Borings

The borings were advanced to depths of approximately 80 to 104 ft with mud-rotary drilling methods. Borings B-23 through B-25 were completed using a truck-mounted drill rig provided and operated by Western States Soil Conservation of Hubbard, Oregon. Boring B-26 was completed using a track-mounted drill rig provided and operated by Hardcore Drilling Inc. of Dundee, Oregon. Disturbed and undisturbed soil samples were typically obtained at 2.5-ft intervals of depth in the upper 15 ft and at 5-ft intervals below this depth. Disturbed samples were obtained using a standard split-spoon sampler. At the time of sampling, the Standard Penetration Test was conducted. This test consists of driving a standard split-spoon sampler into the soil a distance of 18 in. using a 140-lb hammer dropped 30 in. The number of blows required to drive the sampler the last 12 in. is known as the standard penetration resistance, or N-value. The N-values provide a measure of the relative density of granular soils, such as sand or gravel, and the relative consistency, or stiffness, of cohesive soils, such as silt or clay. The split-spoon samples were carefully examined in the field and representative portions were saved in airtight jars. All samples were returned to our laboratory for further examination and physical testing.

Relatively undisturbed samples were obtained by pushing 3-in.-O.D. Shelby tubes into the undisturbed soil a maximum distance of 24 in. using the drill rig. The soils exposed in the ends of the Shelby tubes were examined and classified in the field. After classification, the ends of the tubes were sealed with plastic end caps and tape to preserve the natural moisture content of the soils. All samples were returned to our laboratory for further examination and testing.

Logs of the borings are provided on Figures 1A through 4A. Each log presents a descriptive summary of the various types of materials encountered in the boring and notes the depth at which the materials and/or characteristics of the materials change. To the right of the descriptive summary, the numbers and types of samples are indicated. Farther to the right, N-values are shown graphically, along with natural moisture contents and percent passing the No. 200 sieve. The terms used to describe the soils encountered in the borings are defined in Table 1A.

Electric Cone Penetration Test (CPT) Probes

One CPT probe, designated CPT-6, was advanced to practical refusal at a depth of about 83 ft below the ground surface using a truck-mounted Dutch Cone unit provided and operated by Vandehey Exploration, Inc. of Banks, Oregon. The equipment is mounted on a truck and operated from within an enclosure on the back of the truck that houses the electrical equipment. The electrical cone probe has a cone and a sleeve that are similar to a mechanical probe, but the forces are measured electronically. In addition to the cone and sleeve transducers, a piezometer is fitted between the cone and the sleeve, which allows measurement of pore water pressure and rate of dissipation as the probe is advanced. An accelerometer can also be fitted within the electrical probe. The accelerometer is used to measure the arrival times of shear waves produced at the ground surface as the exploration is advanced. Using these measurements, the shear wave velocity of the soils penetrated can be estimated. The shear wave velocities characterize the soils for the purpose of seismic studies. Shear wave measurements were made during advancement of probe CPT-6. The terms used to describe the soils encountered in the CPT probes are defined in Table 2A. A log of the CPT probe is provided on Figure 5A.

LABORATORY TESTING

General

All samples obtained from the field were returned to our laboratory where the physical characteristics of the samples were noted, and the field classifications were modified where necessary. At the time of classification, the natural moisture content of each sample was measured. Additional testing included washed sieve analysis, sieve analysis, and dry unit weight determinations. The following sections describe the testing program in more detail.

Natural Moisture Content

Natural moisture content determinations were made in conformance with ASTM D 2216. The results are provided on Figures 1A through 4A.

Dry Unit Weight

The dry unit weight of four undisturbed samples was determined in the laboratory in accordance with ASTM D 2937 by cutting a cylindrical specimen of soil from a Shelby tube sample. The dimensions of the specimen were carefully measured, the volume calculated, and the specimen weighed. After oven-drying, the specimen was reweighed and the moisture content calculated. The dry unit weight was then computed. The dry unit weights are summarized below.

SUMMARY OF DRY UNIT WEIGHT DETERMINATIONS

Boring	Sample	Depth, ft	Natural Moisture Content, %	Dry Unit Weight, pcf	Soil Type
B-24	S-18	70	29	88	SAND; fine grained, trace silt
B-26	S-12	40	39	80	SAND; some silt, scattered wood debris
	S-14	45	34	81	SAND; some silt, scattered wood debris
	S-16	50	31	90	SAND; some silt, scattered wood debris

Grain Size Analysis

Washed-Sieve Method. Washed sieve analyses were performed on representative soil samples to assist in their classification. The test is performed by taking a sample of known dry weight and washing it over a No. 200 sieve. The material retained on the sieve is oven-dried and weighed, and the percentage of material passing the No. 200 sieve is calculated. The test results are shown on the Boring Logs, Figures 1A through 4A.

Dry Sieve Method. Sieve analyses were performed on five representative samples of sand in substantial conformance with ASTM D 6913. The test is performed by taking a sample of known dry weight and washing it over a No. 200 sieve. The material retained on the sieve is oven-dried and weighed, and the percentage of material passing the No. 200 sieve is calculated. The soil retained on the No. 200 sieve is then screened through a series of sieves of various sizes using a sieve shaker. The weight of each sieve is measured prior to and after the soil has been run through the shaker. The weight of the soil retained on each sieve is recorded and expressed as a percentage of the total sample weight. The test data are summarized on Figures 6A and 7A in the form of curves showing the percent of the total soil sample by weight finer versus sieve number or grain size in millimeters.

Table 1A: GUIDELINES FOR CLASSIFICATION OF SOIL

RELATIVE DENSITY FOR GRANULAR SOIL

Relative Density	Standard Penetration Resistance (N-values) blows per foot
very loose	0 - 4
loose	4 - 10
medium dense	10 - 30
dense	30 - 50
very dense	over 50

CONSISTENCY FOR FINE-GRAINED (COHESIVE) SOIL

Consistency	Standard Penetration Resistance (N-value) blows per foot	Torvane or Undrained Shear Strength, tsf
very soft	0 - 2	less than 0.125
soft	2 - 4	0.125 - 0.25
medium stiff	4 - 8	0.25 - 0.50
stiff	8 - 15	0.50 - 1.0
very stiff	15 - 30	1.0 - 2.0
hard	over 30	over 2.0

Sandy silt materials which exhibit general properties of granular soils are given relative density description.

Grain-Size Classification	Modifier for Subclassification	
Boulders >12 in.	**Adjective**	**Percentage of Other Material In Total Sample**
Cobbles 3 - 12 in.	clean	0 - 2
Gravel 1/4 - 3/4 in. (fine) 3/4 - 3 in. (coarse)	trace some	2 - 10 10 - 30
Sand No. 200 - No. 40 sieve (fine) No. 40 - No. 10 sieve (medium) No. 10 - No. 4 sieve (coarse)	sandy, silty, clayey, etc.	30 - 50
Silt/Clay - pass No. 200 sieve		

Table 2A

SOIL CLASSIFICATION
BASED ON CONE PENETRATION TEST

Friction Ratio (Percent)	Soil Classification
0 to 2	Clean sand or slightly silty sand
2 to 5	Silty sand, clayey sand, or silt
> 5	Clayey silt, silty clay, or clay

COHESIVE SOILS

Sleeve Friction, tsf	Relative Consistency
<0.12	Very Soft
0.12 to 0.25	Soft
0.25 to 0.50	Medium Stiff
0.50 to 1.00	Stiff
1.00 to 2.00	Very Stiff
>2.00	Hard

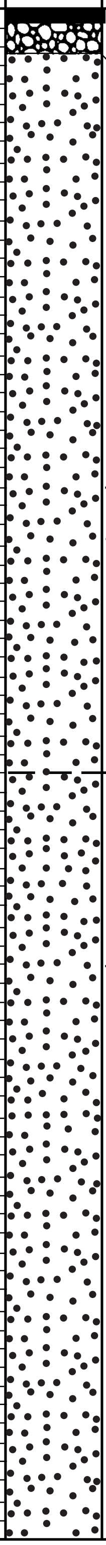
COHESIONLESS SOILS

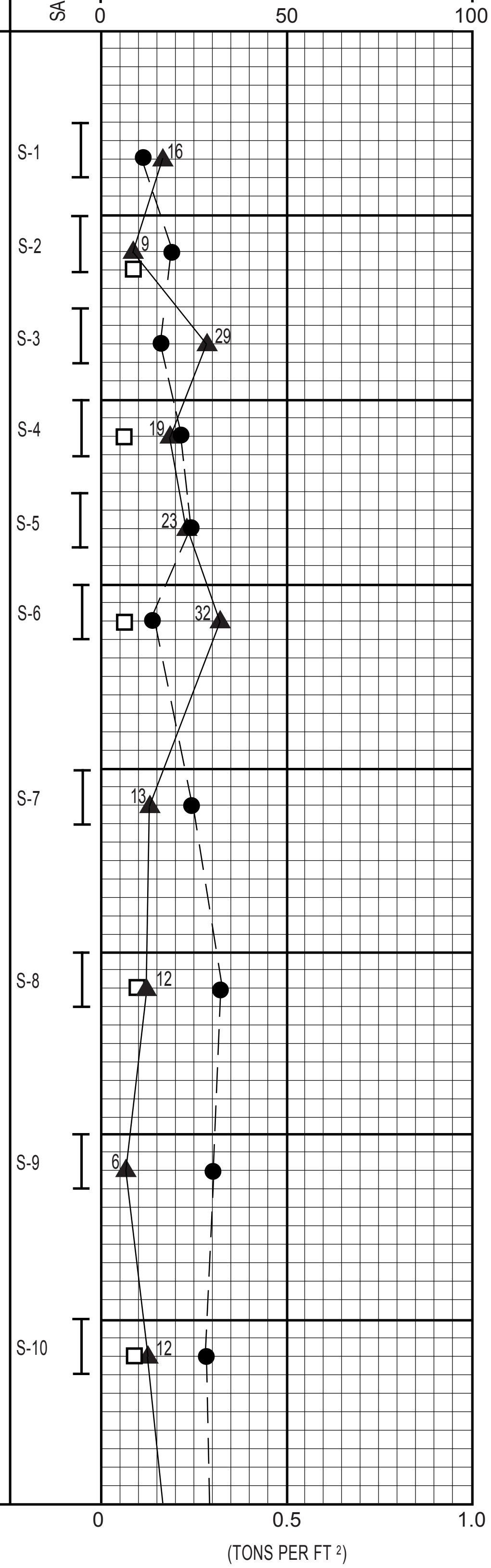
	Soil Type*			
	ML, SM	SM, SP, SW	SP, SW, GW	SW, GP
Relative Density	**Cone Penetration Resistance, tsf**			
Very Loose	0 - 8	0 - 14	0 - 20	0 - 24
Loose	8 - 20	14 - 35	20 - 50	24 - 60
Med. Dense	20 - 60	35 - 105	50 - 150	60 - 180
Dense	60 - 100	105 - 175	150 - 250	180 - 300
Very Dense	> 100	> 175	> 250	> 300

* Unified Soil Classification System

1) Friction ratio is equal to sleeve friction (tsf) divided by cone penetration (tsf) expressed as a percent.

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER	SAMPLES	STD PENETRATION RESISTANCE (140-LB WEIGHT, 30-IN. DROP) ▲ BLOWS PER FOOT ● MOISTURE CONTENT, %
		SURFACE ELEVATION 27 ft (±)				
0		Asphaltic-concrete PAVEMENT (4 in.) over crushed rock BASE COURSE (10 in.)	1.2			
		FILL: Loose to medium dense, brown SAND; fine to medium grained, trace silt and gravel				
7.5		----------medium dense below 7.5 ft				
12.5		----------gray below 12.5 ft				
13.8		----------scattered wood debris at 13.8 ft				
15		----------4-in.-thick layer of sandy silt at 15 ft				
20		Medium dense, gray SAND; fine to medium grained, trace silt, 1-in.-thick layer of silt at 20.1 ft	20.0			
25		----------brown, trace to some silt below 25 ft				
30		----------loose, trace silt below 30 ft				
32		----------medium dense, fine grained below 32 ft				
40						





I 2-IN.-OD SPLIT-SPOON SAMPLER

II 3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

▌ NX CORE RUN

|— SLOTTED PVC PIPE

▼ Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF

□ PERCENT PASSING NO. 200 SIEVE (WASHED)

* NO RECOVERY

Liquid Limit

Moisture Content

Plastic Limit

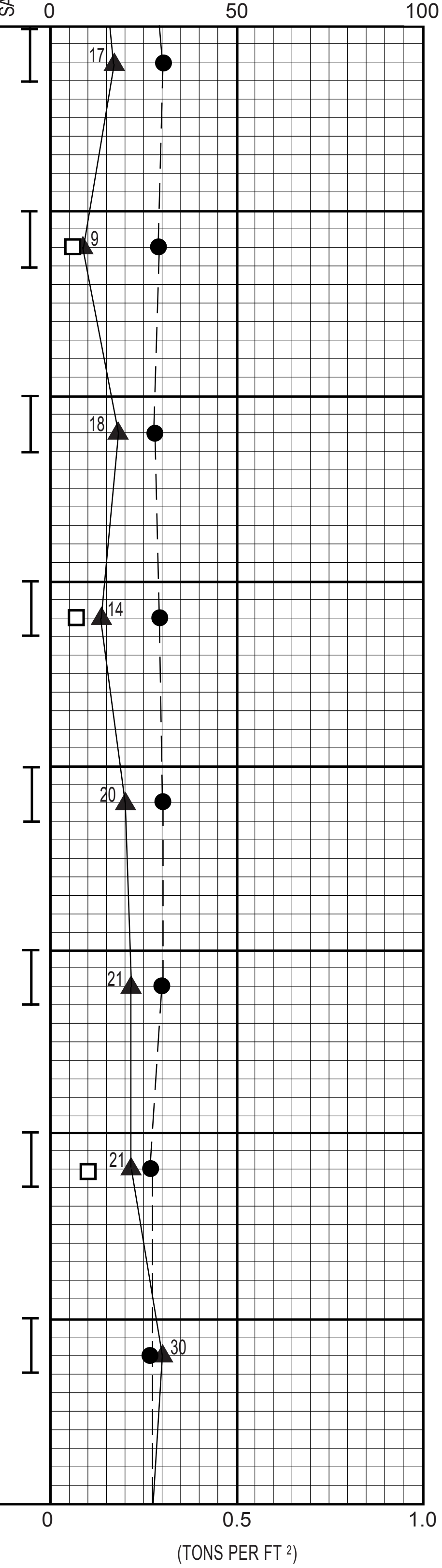
GRI

BORING B-23

SEP. 2014 JOB NO. W1114 FIG. 1A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER	SAMPLES
		SURFACE ELEVATION 27 ft (±)			
40		Medium dense, brown SAND; fine to medium grained, trace silt			S-11
45		----------loose, gray below 45 ft			S-12
50		----------medium dense below 50 ft			S-13
55					S-14
60					S-15
65					S-16
70		----------trace to some silt below 70 ft			S-17
75					S-18
80					

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)
▲ BLOWS PER FOOT
● MOISTURE CONTENT, %



I 2-IN.-OD SPLIT-SPOON SAMPLER
II 3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
▌ NX CORE RUN
|—SLOTTED PVC PIPE
▼ Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF
□ PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

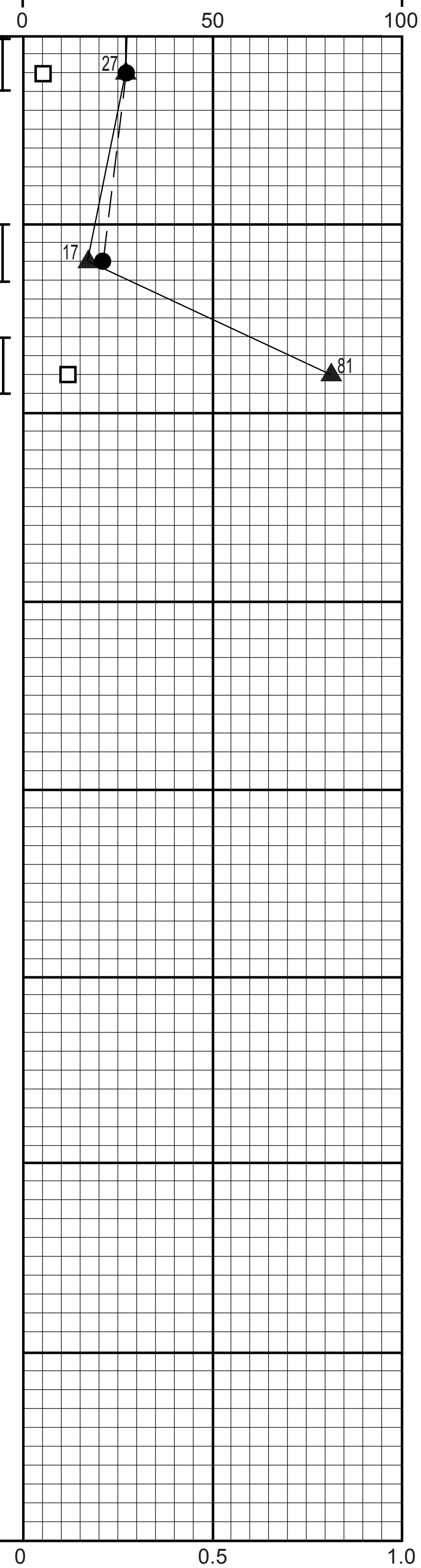
BORING B-23 (cont.)

SEP. 2014 | JOB NO. W1114 | FIG. 1A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER	SAMPLES
		SURFACE ELEVATION 27 ft (±)			
80		Medium dense, gray SAND; fine grained, trace silt			S-19
85		----------fine to medium grained below 85 ft; scattered gravel below 85.5 ft			S-20
		Very dense GRAVEL; trace to some silt and fine- to coarse-grained sand	87.0		S-21
90		(7/29/2013)	89.5		
95					

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)

▲ BLOWS PER FOOT
● MOISTURE CONTENT, %



(TONS PER FT ^{2})

I 2-IN.-OD SPLIT-SPOON SAMPLER
II 3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
▌ NX CORE RUN
|—SLOTTED PVC PIPE
▼ Water Level (date)

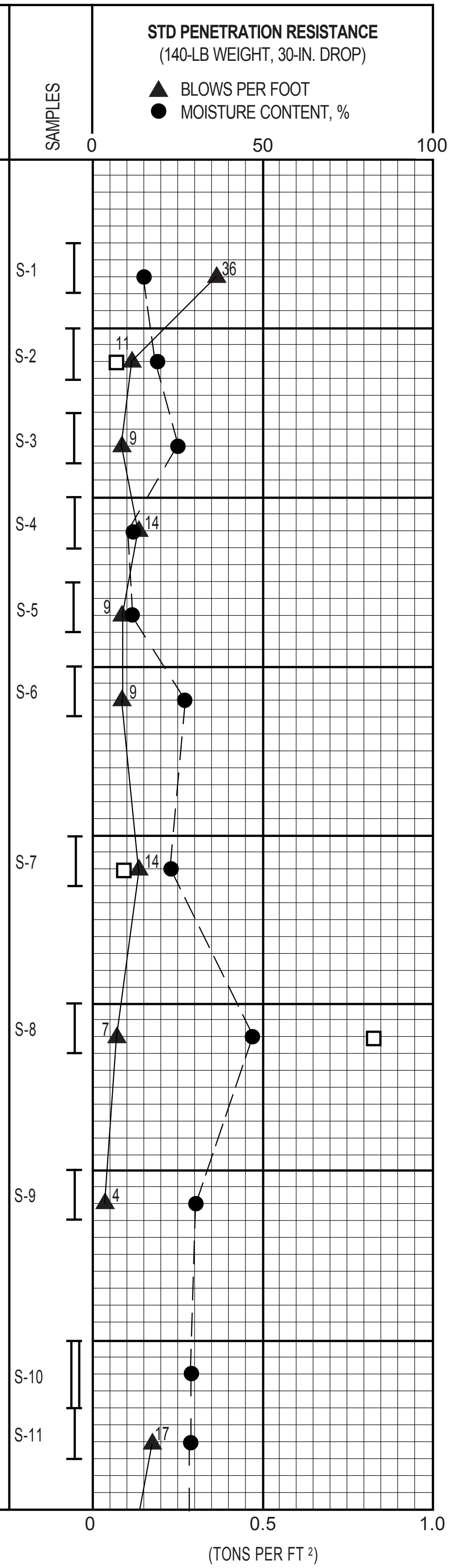
◆ TORVANE SHEAR STRENGTH, TSF
□ PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-23 (cont.)

SEP. 2014 JOB NO. W1114 FIG. 1A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER
		SURFACE ELEVATION 27 ft (±)		
0–25		FILL: Dense, gray, gravelly SAND; fine to coarse grained, trace silt		
5		---------loose to medium dense, gravel absent below 5 ft, trace organics at 5 ft		
25–30		Medium stiff, gray SILT; some fine-grained sand	25.0	
30–40		Loose to medium dense, gray SAND; fine grained, trace to some silt	30.0	
31		-------4-in.-thick sandy silt layer at 31 ft		
35		---------up to 1-in.-thick silt layers between 35 and 37 ft		
37		---------brown, silty below 37 ft		



I 2-IN.-OD SPLIT-SPOON SAMPLER

II 3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

▌ NX CORE RUN

|—SLOTTED PVC PIPE

▼ Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF

□ PERCENT PASSING NO. 200 SIEVE (WASHED)

* NO RECOVERY

Liquid Limit

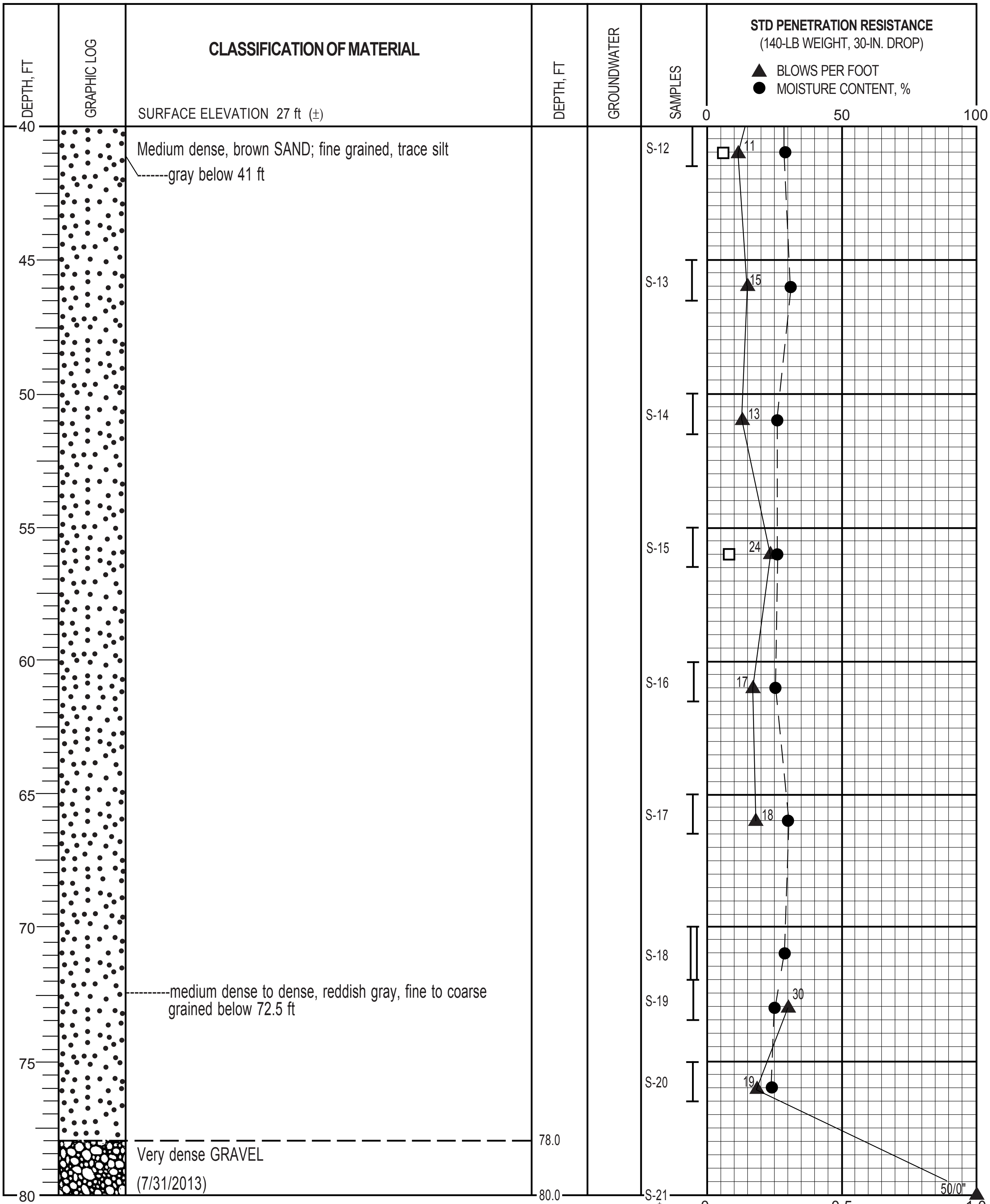
Moisture Content

Plastic Limit

GRI

BORING B-24

SEP. 2014 JOB NO. W1114 FIG. 2A



2-IN.-OD SPLIT-SPOON SAMPLER

3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

NX CORE RUN

SLOTTED PVC PIPE

Water Level (date)

TORVANE SHEAR STRENGTH, TSF

PERCENT PASSING NO. 200 SIEVE (WASHED)

* NO RECOVERY

Liquid Limit

Moisture Content

Plastic Limit

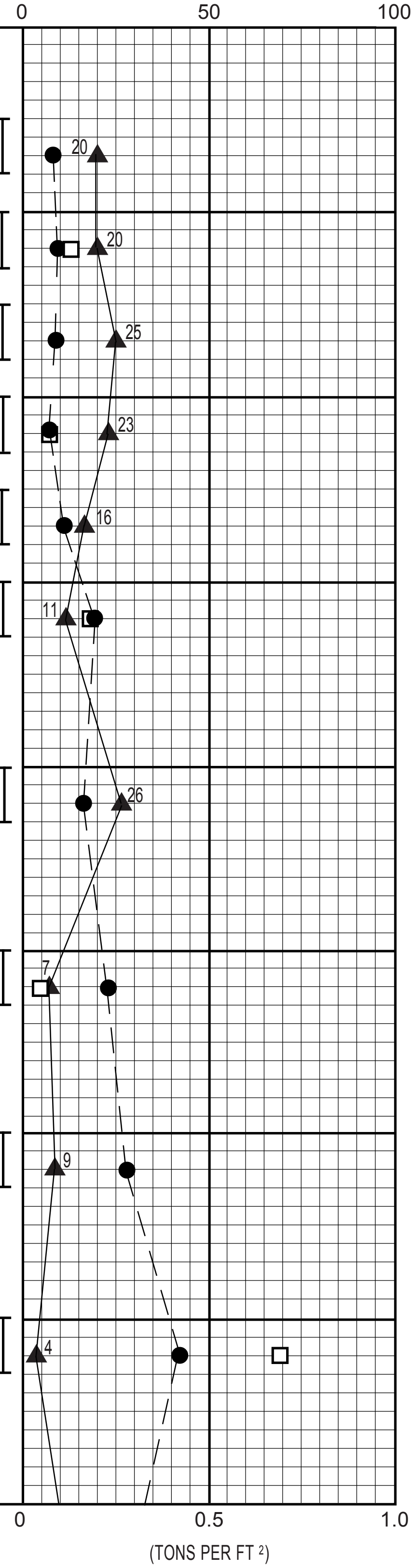
GRI

BORING B-24 (cont.)

SEP. 2014 JOB NO. W1114 FIG. 2A

DEPTH, FT	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER	SAMPLES
	SURFACE ELEVATION 27 ft (±)			
0–3	Asphaltic-concrete PAVEMENT (12 in.) over crushed rock BASE COURSE (24 in.)	3.0		
3–25	FILL: Medium dense, light brown SAND; fine grained, trace rounded gravel, trace silt -------gray and brown below 5 ft ---------1-in.-thick silt layer at 6.5 ft	25.0		S-1, S-2, S-3, S-4, S-5, S-6, S-7
25–35	Loose, dark gray SAND; fine grained, trace silt	35.0		S-8, S-9
35–36	Medium stiff, dark gray, sandy SILT; fine-grained sand	36.0		S-10
36–40	Loose, dark gray SAND; fine to medium grained, trace to some silt			

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)
▲ BLOWS PER FOOT
● MOISTURE CONTENT, %

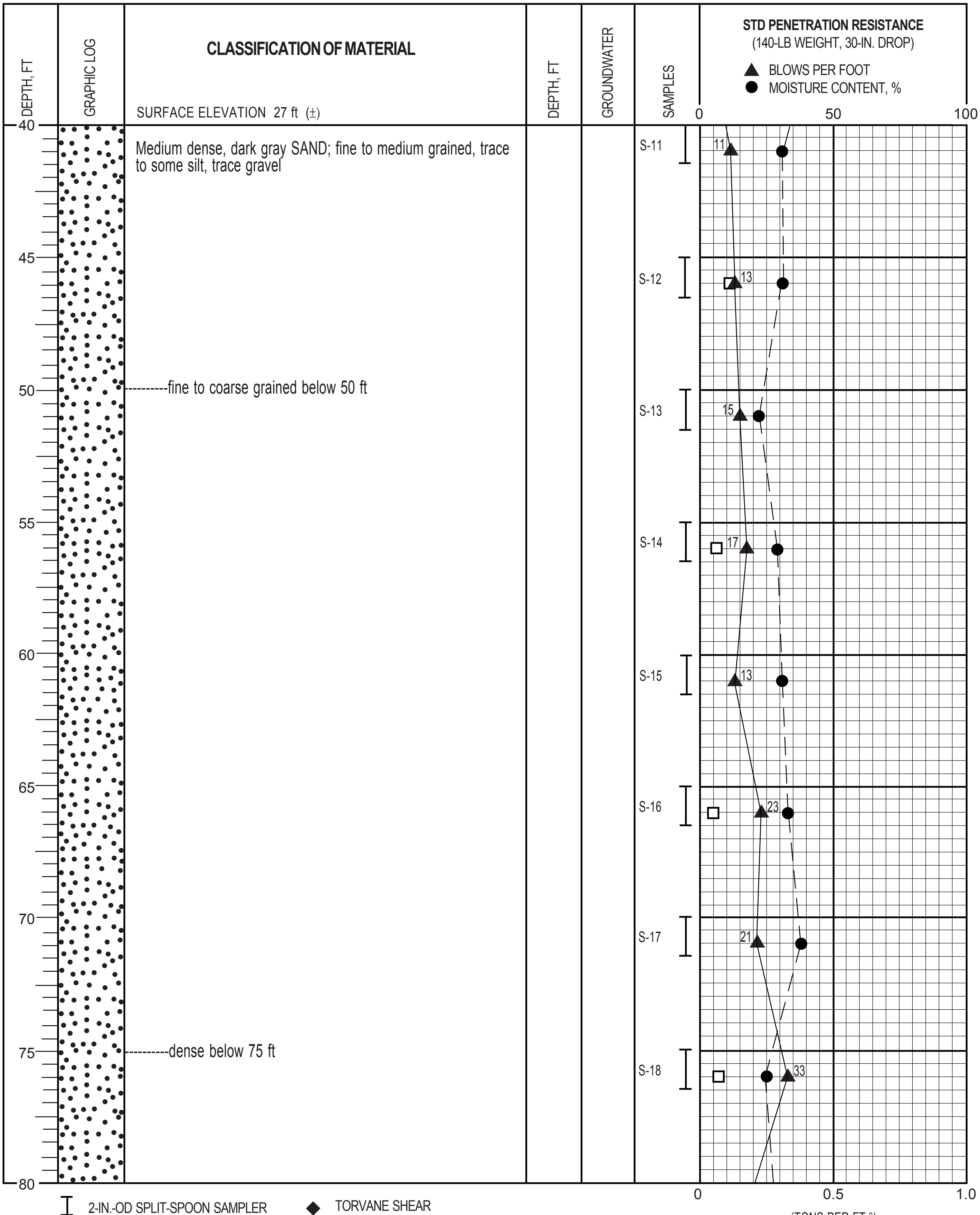


- 2-IN.-OD SPLIT-SPOON SAMPLER
- 3-IN.-OD THIN-WALLED SAMPLER
- G GRAB SAMPLE OF DRILL CUTTINGS
- NX CORE RUN
- SLOTTED PVC PIPE
- Water Level (date)
- ◆ TORVANE SHEAR STRENGTH, TSF
- □ PERCENT PASSING NO. 200 SIEVE (WASHED)
- * NO RECOVERY
- Liquid Limit / Moisture Content / Plastic Limit

GRI

BORING B-25

SEP. 2014 JOB NO. W1114 FIG. 3A



I 2-IN.-OD SPLIT-SPOON SAMPLER

3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

NX CORE RUN

SLOTTED PVC PIPE

Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF

□ PERCENT PASSING NO. 200 SIEVE (WASHED)

* NO RECOVERY

Liquid Limit
Moisture Content
Plastic Limit

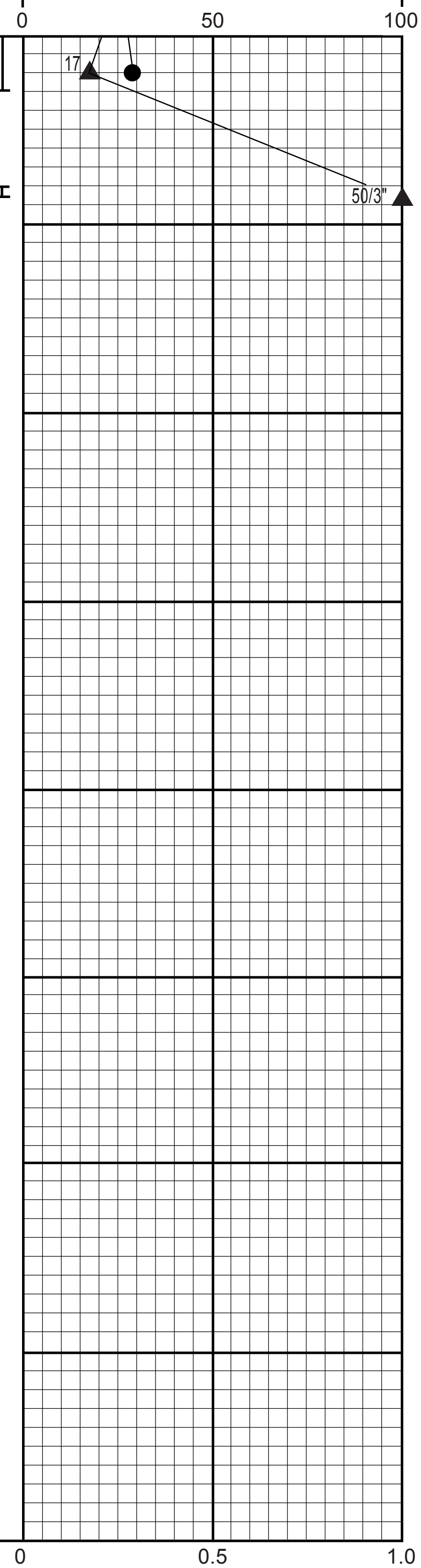
GRI

BORING B-25 (cont.)

SEP. 2014 JOB NO. W1114 FIG. 3A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER	SAMPLES
		SURFACE ELEVATION 27 ft (±)			
80		Medium dense, dark gray SAND; fine to coarse grained, trace silt	82.0		S-19
		Very dense GRAVEL; scattered cobbles	84.3		S-20
85		(7/30/2013)			

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)
▲ BLOWS PER FOOT
● MOISTURE CONTENT, %



I 2-IN.-OD SPLIT-SPOON SAMPLER
II 3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
▌ NX CORE RUN
SLOTTED PVC PIPE
▼ Water Level (date)

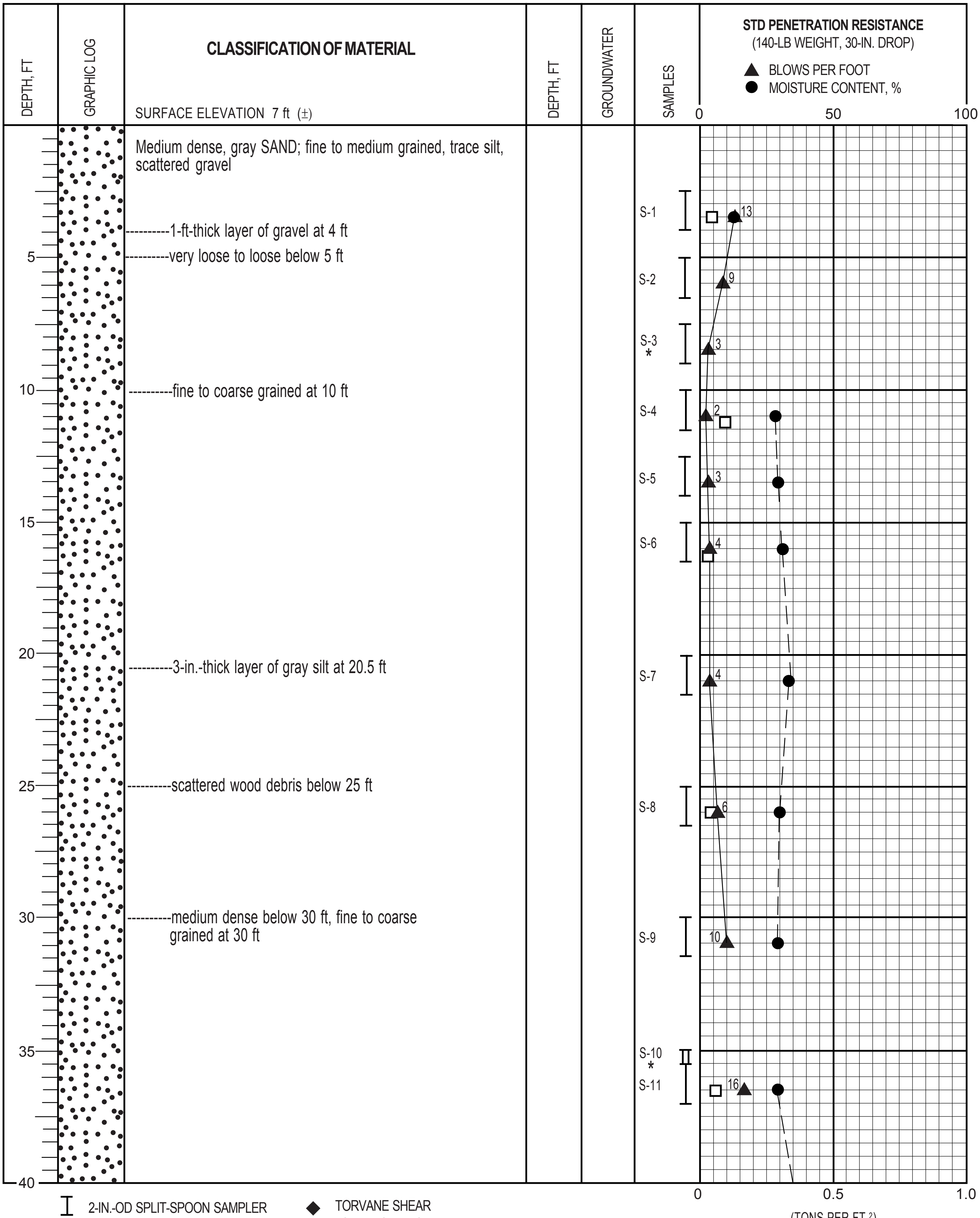
◆ TORVANE SHEAR STRENGTH, TSF
□ PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY

Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-25 (cont.)

SEP. 2014 JOB NO. W1114 FIG. 3A



I 2-IN.-OD SPLIT-SPOON SAMPLER

II 3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

▌ NX CORE RUN

|—SLOTTED PVC PIPE

▼ Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF

□ PERCENT PASSING NO. 200 SIEVE (WASHED)

* NO RECOVERY

Liquid Limit

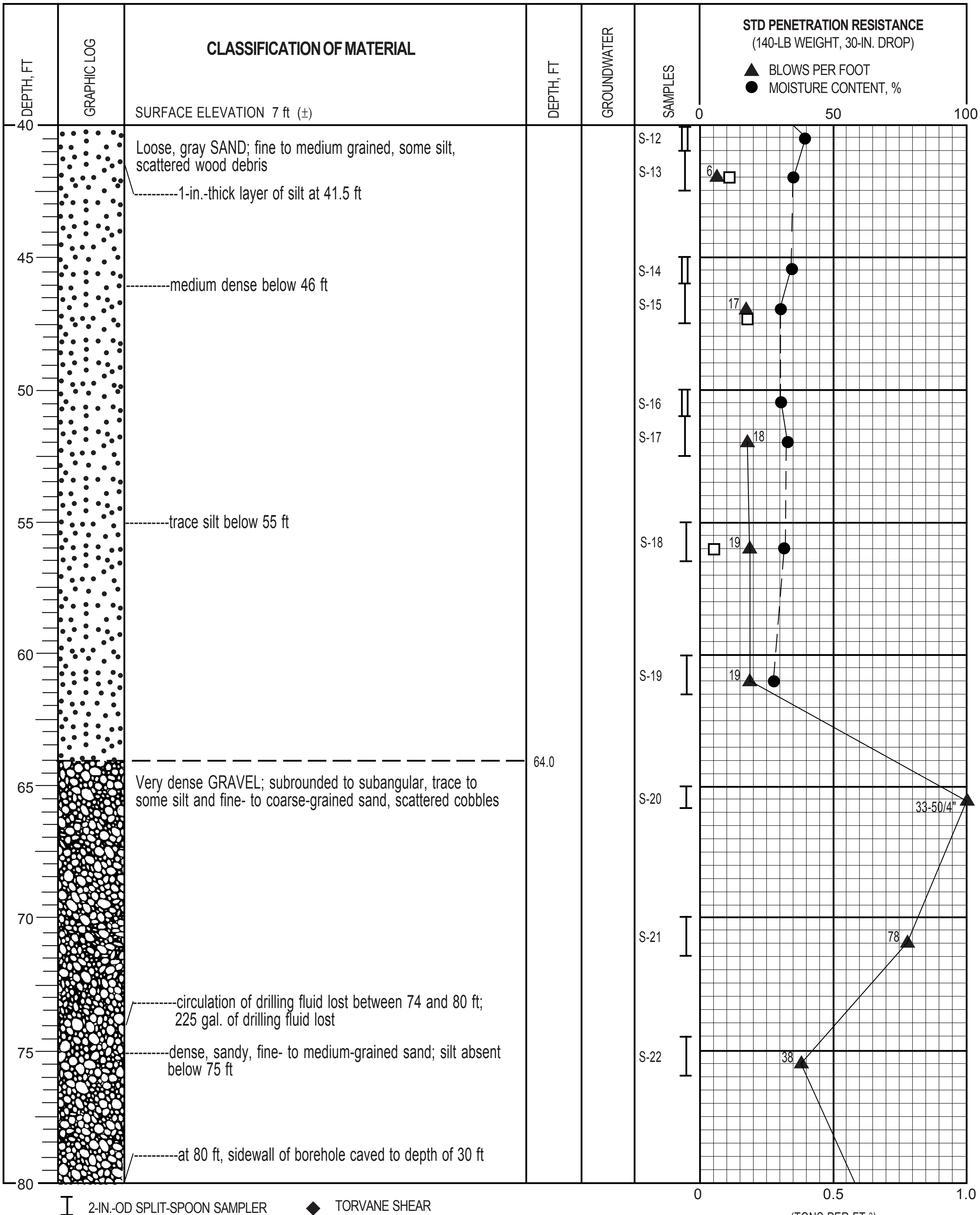
Moisture Content

Plastic Limit

GRI

BORING B-26

SEP. 2014 | JOB NO. W1114 | FIG. 4A



2-IN.-OD SPLIT-SPOON SAMPLER

3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

NX CORE RUN

SLOTTED PVC PIPE

Water Level (date)

TORVANE SHEAR STRENGTH, TSF

PERCENT PASSING NO. 200 SIEVE (WASHED)

* NO RECOVERY

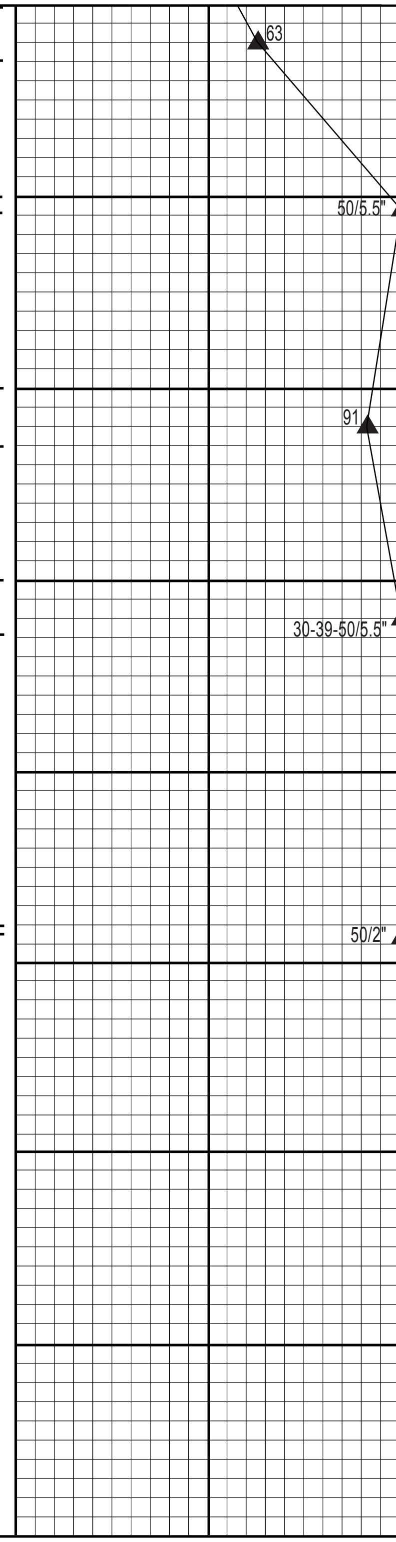
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-26 (cont.)

SEP. 2014 JOB NO. W1114 FIG. 4A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER	SAMPLES	STD PENETRATION RESISTANCE (140-LB WEIGHT, 30-IN. DROP) ▲ BLOWS PER FOOT ● MOISTURE CONTENT, %
		SURFACE ELEVATION 7 ft (±)				0 50 100
80		Very dense GRAVEL; subrounded to subangular, trace to some fine- to coarse-grained sand, trace silt, scattered cobbles			S-23	63
85					S-24	50/5.5"
90					S-25	91
95					S-26	30-39-50/5.5"
100						
			104.2		S-27	50/2"
105		(10/29/2013) 5.5-in.-diameter casing installed to 20 ft as part of drilling permit requirements				



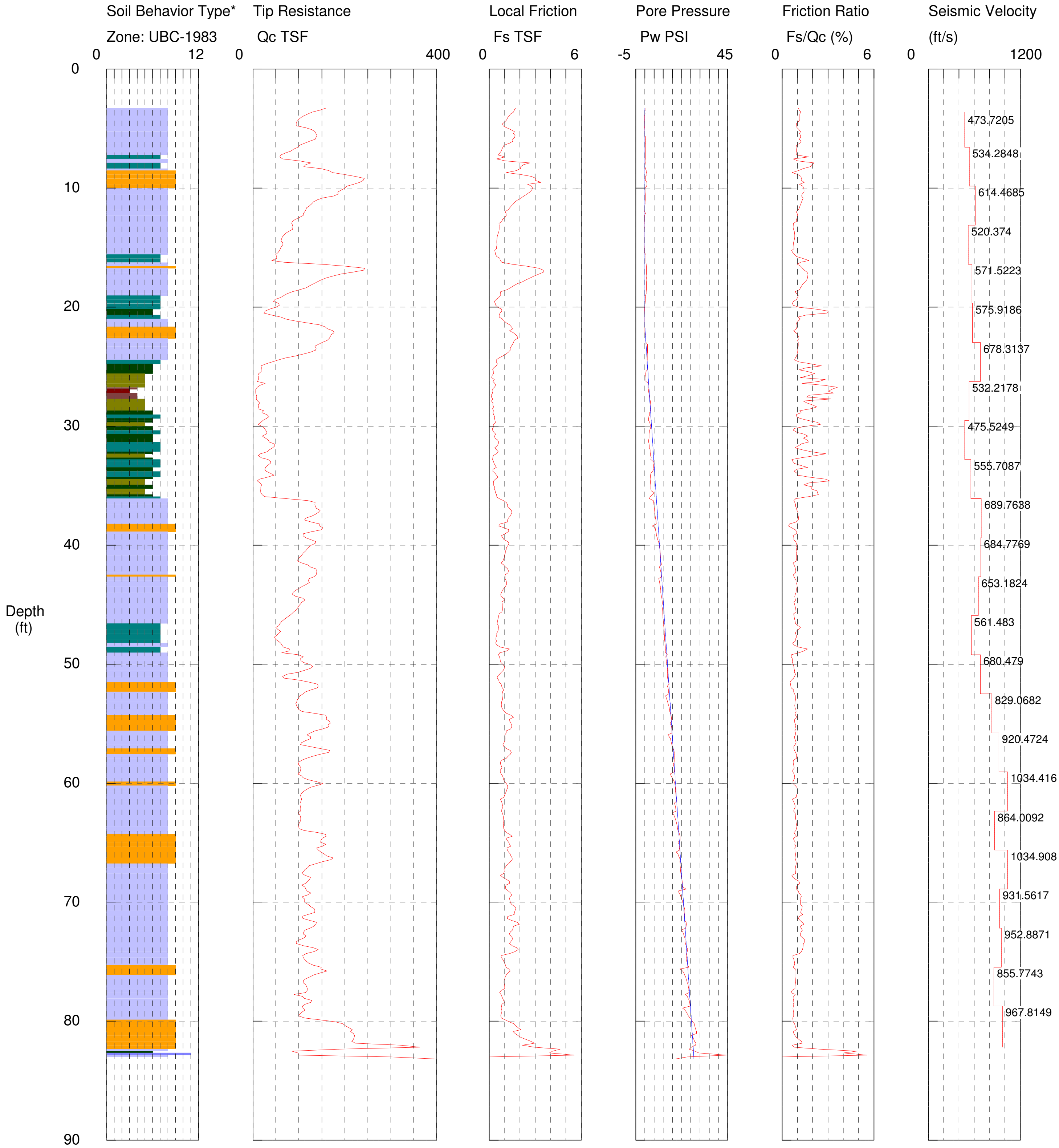
I 2-IN.-OD SPLIT-SPOON SAMPLER
II 3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
▌ NX CORE RUN
SLOTTED PVC PIPE
▼ Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF
□ PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-26 (cont.)

SEP. 2014 JOB NO. W1114 FIG. 4A



Maximum Depth = 83.17 feet

Depth Increment = 0.164 feet

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

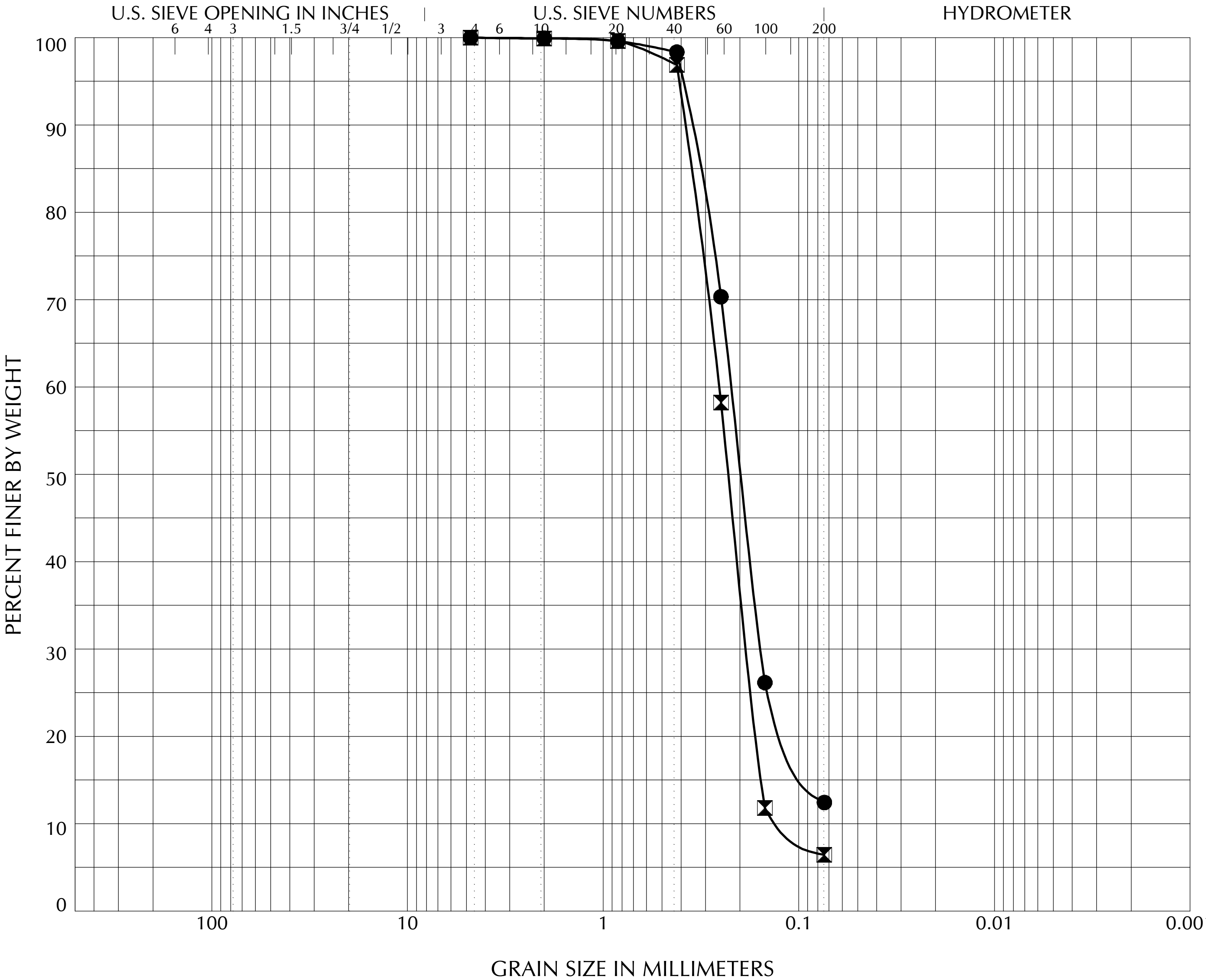
SURFACE ELEVATION = 27 FT

GRI

CONE PENETRATION TEST CPT-6

(WITH SEISMIC VELOCITY)

SEP. 2014 JOB NO. W1114 FIG. 5A



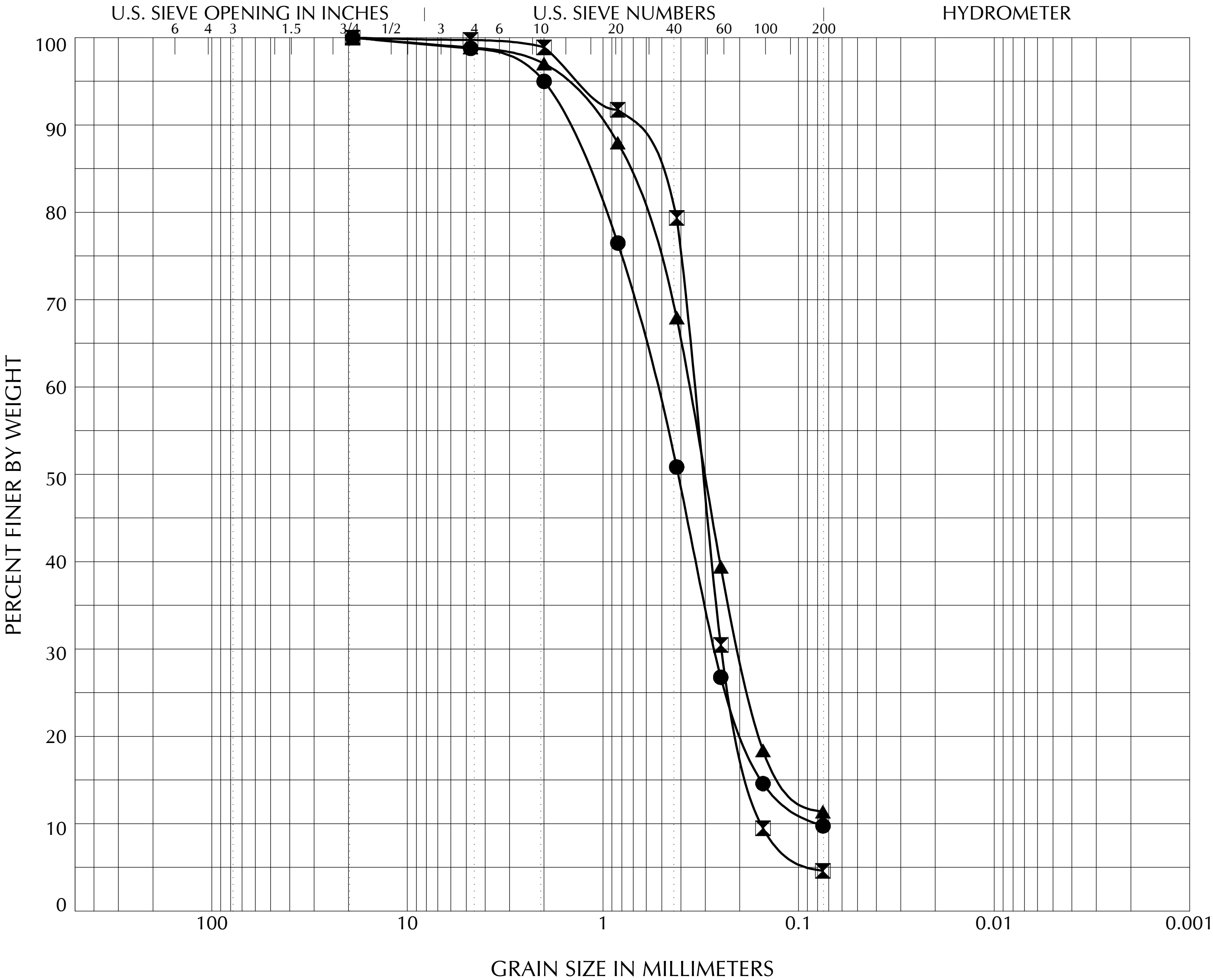
<table>
<tr><td rowspan="2">COBBLES</td><td colspan="2">GRAVEL</td><td colspan="3">SAND</td><td rowspan="2">SILT OR CLAY</td></tr>
<tr><td>Coarse</td><td>Fine</td><td>Coarse</td><td>Medium</td><td>Fine</td></tr>
</table>

	Location	Sample	Depth, ft	Classification	Gravel, %	Sand, %	Fines, %
●	B-23	S-8	25.0	SAND; fine to medium grained, some silt	0.0	87.3	12.7
⧗	B-23	S-12	45.0	SAND; fine grained, trace silt	0.0	93.5	6.5

GRI

GRAIN SIZE DISTRIBUTION

SEP. 2014 JOB NO. W1114 FIG. 6A



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

	Location	Sample	Depth, ft	Classification	Gravel, %	Sand, %	Fines, %
●	B-26	S-4	10.0	Gray SAND; fine to coarse grained, trace silt	1.2	89.0	9.8
⧗	B-26	S-8	25.0	Gray SAND; fine to medium grained, trace silt, scattered wood debris	0.3	95.2	4.6
▲	B-26	S-13	41.0	Gray SAND; fine to medium grained, some silt, scattered wood debris	1.1	87.5	11.4

GRI

GRAIN SIZE DISTRIBUTION

SEP. 2014 JOB NO. W1114-T3 FIG. 7A

GRAIN SIZE GRI-THREE PER PAGE W1114-T3 LAB.GPJ GRI DATA TEMPLATE.GDT 6/4/14

APPENDIX B

Site-Specific Seismic Hazard Study

APPENDIX B

SITE-SPECIFIC SEISMIC HAZARD STUDY

General

GRI has completed a site-specific seismic hazard study for the docks at the proposed Tesoro Savage Vancouver Energy Distribution Terminal - Upland Facility (TSVEDT) in Vancouver, Washington. The purpose of the study was to evaluate the potential seismic hazards associated with regional and local seismicity. The site-specific hazard study is intended to meet the requirements of the upcoming American Society of Civil Engineers (ASCE) *Seismic Design of Pile-Supported Piers and Wharves* in compliance with the requirements of ASCE 7-05 Chapter 21. Our work was based on the potential for regional and local seismic activity, as described in the existing scientific literature, and on the subsurface conditions at the site, as disclosed by the geotechnical explorations completed for the project. Specifically, our work included the following tasks:

1) A detailed review of available literature, including published papers, maps, open-file reports, seismic histories and catalogs, and other sources of information regarding the tectonic setting, regional and local geology, and historical seismic activity that might have a significant effect on the site.

2) Compilation, examination, and evaluation of existing subsurface data gathered at and in the vicinity of the site, including classification and laboratory analyses of soil samples. This information was used to prepare a generalized subsurface profile for the docks on the TSVEDT property.

3) Identification of potential seismic sources appropriate for the site and characterization of those sources in terms of magnitude, distance, and acceleration response spectra.

4) Office studies, based on the generalized subsurface profile and the controlling seismic sources, resulting in conclusions and recommendations concerning:

    a) specific seismic events and characteristic earthquakes that might have a significant effect on the Docks;
    b) the potential for seismic energy amplification at the Docks; and
    c) site-specific acceleration response spectra for design of the Docks

This appendix describes the work accomplished and summarizes our conclusions and recommendations.

Geologic Setting

General. On a regional scale, the site lies within the Willamette-Puget Sound lowland trough of the Cascadia convergent tectonic system (Blakely et al., 2000). The lowland areas consist of broad north-south-trending basins in the underlying geologic structure between the Coast Range to the west and the Cascade Mountains to the east. The lowland trough is characterized by alluvial plains with areas of buttes and terraces. The site lies approximately 95 km inland from the down-dip edge of the seismogenic extent of the Cascadia Subduction Zone (CSZ), an active convergent plate boundary along which remnants of the Farallon Plate (the Gorda, Juan de Fuca, and Explorer plates) are being subducted beneath the western edge of the North American continent. The subduction zone is a broad, eastward-dipping zone of contact between the upper portion of the subducting slabs and the over-riding North American Plate as shown on Figure 1B.

On a local scale, the site lies within the Portland Basin, a large, well-defined, northwest-trending structure characterized as a right-lateral pull-apart basin in the forearc of the CSZ. The Portland Basin is bounded by high-angle, northwest-trending, right-lateral strike-slip faults that are considered to be seismogenic; however, the relationship between specific earthquakes and individual faults in the area is not well understood since few of these faults are expressed clearly at the ground surface. A limited number of intrabasin faults have been mapped on the basis of stratigraphic offsets and geophysical evidence, and the site is located in close proximity to the inferred traces of the Portland Hills Fault and the East Bank Fault indicated on published geologic mapping (Personius et al., 2003). The distribution of these crustal faults relative to the site is shown on the Regional Geologic Map and Local Fault Map, Figures 2B and 3B, respectively. The locations of faults on the geologic map are inferred or approximate. Other faults may be present within the basin, but clear stratigraphic evidence regarding their location and extent is not presently available.

Because of the proximity of the site to the CSZ and its location within the Portland Basin, three seismic sources contribute to the potential for damaging earthquake motions at the site. Two of these sources are associated with tectonic activity related to the Cascadia Subduction Zone, the third is associated with movement on relatively shallow faults within and adjacent to the Portland Basin.

Site Soil and Geologic Conditions. The dock site is covered by alluvial sand and silt deposited by the Columbia River. The alluvium extends to variable depths, ranging from 16 ft at the T-section of the docks to 64 ft at pier 3 of the trestle. The sand and silt deposits are underlain by gravel associated with late Pleistocene catastrophic flood materials deposited by repeated Missoula Flood events that occurred between 13,500 and 15,000 years ago. Geologic investigations for the proposed Interstate 5 bridge replacement, about 3 miles upstream from the project site, indicate the gravel on the Washington side of the Columbia River can be up to 100 ft thick. The flood deposits are underlain by well-consolidated or cemented conglomerate and sandstone units of the Troutdale Formation (Pliocene), which are, in turn, underlain by the Sandy River Mudstone bedrock (Miocene to Pliocene; Beeson et al., 1991; Trimble, 1963).

Seismicity

General. The geologic and seismologic information available for identifying the potential seismicity at the site is incomplete, and large uncertainties are associated with estimates of the probable magnitude, location, and frequency of occurrence of earthquakes that might affect the site. The available information indicates the potential seismic sources that may affect the site can be grouped into three independent categories: *subduction zone events* related to sudden slip between the upper surface of the Juan de Fuca plate and the lower surface of the North American plate, *subcrustal (intraslab) events* related to deformation and volume changes within the deeper portion of the subducted Juan de Fuca plate, and *local crustal events* associated with movement on shallow, local faults within and adjacent to the Portland Basin. Based on our review of currently available information, we have developed parameters for each of these potential seismic sources. The seismic sources are characterized by three important parameters: magnitude, distance to the subject site, and the peak horizontal bedrock accelerations produced by the controlling earthquake on the seismic source. The size of an earthquake is commonly defined by its moment magnitude M_W. Distance is measured using the closest horizontal distance to the surface projection of the rupture plane or the closest distance to the rupture plane, in kilometers. Peak horizontal bedrock accelerations are expressed in units of gravity (1 g = 32.2 ft/sec^2 = 981 cm/sec^2).

Subduction Zone Event. Written Japanese tsunami records provide evidence that a great CSZ earthquake occurred in January 1700. Geological studies show that great megathrust earthquakes have occurred repeatedly in the past 7,000 years (Atwater et al., 1995; Clague, 1997; Goldfinger, 2003; and Kelsey et al., 2005), and geodetic studies (Hyndman and Wang, 1995; Savage et al., 2000) indicate rate of strain accumulation consistent with the assumption that the CSZ is locked beneath offshore northern California, Oregon, Washington, and southern British Columbia (Fluck et al., 1997; Wang et al., 2001). Numerous geological and geophysical studies suggest the CSZ may be segmented (Hughes and Carr, 1980; Weaver and Michaelson, 1985; Guffanti and Weaver, 1988; Goldfinger, 1994; Kelsey and Bockheim, 1994; Mitchell et al., 1994; Personius, 1995; Nelson and Personius, 1996; Witter, 1999), but the most recent studies suggest that for the last great earthquake in 1700, most of the subduction zone ruptured in a single M_W 9 earthquake (Satake et al., 1996; Atwater and Hemphill-Haley, 1997; Clague et al., 2000). Published estimates of the probable maximum size of subduction zone events range from moment magnitude M_W 8.3 to >9.0. Numerous detailed studies of coastal subsidence, tsunamis, and turbidites yield a wide range of recurrence intervals, but the most complete records (>4,000 years) indicate average intervals of 350 to 600 years between great earthquakes on the CSZ (Adams, 1990; Atwater and Hemphill-Haley, 1997; Witter, 1999; Clague et al., 2000; Kelsey et al., 2002; Kelsey et al., 2005; Witter et al., 2003). Tsunami inundation in buried marshes along the Washington and Oregon coast and stratigraphic evidence from the Cascadia margin support these recurrence intervals (Kelsey et al., 2005; Goldfinger, 2003).

The USGS probabilistic analysis assumes four potential locations for the eastern edge of the earthquake rupture zone for the CSZ, as shown in Figure 4B. The 2008 USGS mapping effort indicates two rupture scenarios are assumed to represent these megathrust events: 1) M_W 9.0±0.2 events that rupture the entire CSZ every 500 years and 2) M_W 8.0 to 8.7 events with rupture zones that occur on segments of the CSZ and occur over the entire length of the CSZ during a period of about 500 years (Petersen et al., 2008). The assumed distribution of earthquake magnitudes is shown on Figure 5B. This distribution assumes the larger M_W 9.0 earthquakes likely occur more often than the smaller segmented ruptures. Therefore, for our deterministic analysis, we have chosen to represent the subduction zone event by a design earthquake of M_W 9.0 at a focal depth of 15 km and a rupture distance of 86 km. This corresponds to a sudden rupture of the entire length of the Juan de Fuca-North American plate interface with an assumed rupture zone along the coastline due west of Vancouver. Based on an average of the attenuation relationships published by Zhao (2006), Atkinson and Macias (2009), and Abrahamson, et al. (2012), a subduction zone earthquake with these parameters would result in an average peak bedrock acceleration of approximately 0.19 g at the project site.

Subcrustal Event. There is no historic earthquake record of subcrustal, intraslab earthquakes in Southwest Washington. Although both the Puget Sound and Northern California regions have experienced many of these earthquakes in historic times, Wong (2005) hypothesizes that due to subduction zone geometry, geophysical conditions and local geology, Southwest Washington/Oregon may not be subject to intraslab earthquakes. In the Puget Sound area, these moderate to large earthquakes are deep (40 to 60 km) and over 200 km from the deformation front of the subduction zone. Offshore, along the Northern California coast, the earthquakes are shallower (up to 40 km) and located along the deformation front. Estimates of the probable magnitude, distance, and frequency of subcrustal events in Southwest Washington are generally based on comparisons of the CSZ with active convergent plate margins in other parts of the world and on the historical seismic record for the region surrounding Puget Sound, where significant events known to have occurred within the subducting Juan de Fuca plate have been recorded. Published estimates of the probable maximum size of these events range from moment magnitude M_W 7.0 to 7.5. The 1949, 1965, and 2001 documented subcrustal earthquakes in the Puget Sound area correspond to M_W 7.1, 6.5, and 6.8, respectively. Published information regarding the location and geometry of the subducting zone indicates that a focal depth of 50 km is probable (Weaver and Shedlock, 1989). We have chosen to represent the subcrustal event by a characteristic earthquake of moment magnitude M_W 7.0 at a focal depth of 50 km and a rupture distance of 50 km. Based on the attenuation relationships published by Zhao (2006), and Atkinson and Boore (2003), and Abrahamson, et al. (2012), a subcrustal earthquake of this magnitude and distance would result in a peak horizontal bedrock acceleration of approximately 0.14g at the site.

Local Crustal Event. Sudden crustal movements along relatively shallow, local faults in the southwest Washington area, although rare, have been responsible for local crustal earthquakes. The precise relationship between specific earthquakes and individual faults is not well understood, since few of the faults in the area are expressed at the ground surface, and the foci of the observed earthquakes have not been located with precision. The history of local seismic activity is commonly used as a basis for determining the size and frequency to be expected of local crustal events. Although the historical record of local earthquakes is relatively short (the earliest reported seismic event in the area occurred in 1920), it can serve as a guide for estimating the potential for seismic activity in the area.

Based on fault mapping conducted by the U.S. Geological Survey (USGS, 2008), the Portland Hills Fault is the closest mapped crustal fault to the site that is considered active in the probabilistic hazard maps. The Portland Hills Fault is located approximately 7 km from the site and has a characteristic earthquake magnitude of M_W 7.0. A crustal earthquake of this magnitude and distance would result in a peak horizontal bedrock acceleration of approximately 0.33g at the site based on an average of the NGA ground motion relations developed by the Pacific Earthquake Engineering Research (PEER) by Boore and Atkinson (2008), Campbell and Bozorgnia (2008), Idriss (2008) and Chiou and Youngs (2008).

Other Seismic Hazards. Based on the presence of loose sands and soft silts below the water table at the site, there is a high risk of liquefaction and lateral spreading during a design-level earthquake. More detailed discussions regarding liquefaction and lateral spreading are provided in the Seismic Considerations section of the report. Although detailed tsunami modeling of the Columbia River due to a Cascadia Subduction Zone earthquake has not been completed, we anticipate the risk of upland damage by tsunami at the site is low due to the distance from the coast. River fluctuations may result from a tsunami generated by a CSZ earthquake. Due to the proximity of the Columbia River, there is a risk of seiche. Unless occurring on a previously unmapped fault, it is our opinion the risk of ground rupture at the site is very low.

Deterministic Earthquake Parameters

As discussed above, three distinctly different seismic sources affect seismicity in the project area. Deterministic evaluation of the earthquake sources using published attenuation relations provides estimates of peak bedrock accelerations and response spectra for each seismic source. These deterministic estimates are not associated with a relative hazard level or probability of occurrence like probabilistic estimates, but simply provide an estimate of the ground motion parameters for each seismic source at a given distance from the site. The basic parameters of each earthquake source are as follows:

TABLE 1B: DETERMINISTIC EARTHQUAKE PARAMETERS

Earthquake Source	Attenuation Relationships	Magnitude, M_w	Rupture Distance, km	Focal Depth, km	Median Peak Bedrock Acceleration, g	Average Median Peak Bedrock Acceleration, g
Subduction Zone	Zhao (2006)	9.0	86	15	0.19	
	Atkinson and Macias (2009)	9.0	86	15	0.17	0.19
	Abrahamson (2012)	9.0	86	15	0.23	
Subcrustal	Zhao (2006)	7.0	50	50	0.15	
	Atkinson and Boore, (2003)	7.0	50	50	0.10	0.14
	Abrahamson (2012)	7.0	50	50	0.18	
Local Crustal	Campbell and Bozorgnia (2008)	7.0	7	NA	0.32	
	Chiou and Youngs (2008)	7.0	7	NA	0.36	0.33
	Boore and Atkinson (2008)	7.0	7	NA	0.27	
	Idriss (2008)	7.0	7	NA	0.38	

The values summarized in Table 1B represent the average of median peak bedrock accelerations for the characteristic earthquake on the controlling faults. The upcoming ASCE *Seismic Design of Pile-Supported Piers and Wharves* references ASCE 7-05 which requires an evaluation of 150 percent of the largest median spectral response acceleration taking into account the characteristic earthquakes on all known active faults within the region. Figure 6b compares 150% of these median deterministic spectral values with the deterministic lower limit on MCE response spectrum, in accordance with Figure 21.2-1 of ASCE 7-05 to develop the deterministic Maximum Considered Earthquake (MCE) bedrock spectra. Figure 6B shows that the individual fault deterministic response spectra are lower than the deterministic lower limit on MCE response spectrum. Per Section 21.2.2 of ASCE 7-05, the deterministic spectrum shall be the greater of the 150% deterministic spectrum and the deterministic lower limit spectrum of Figure 21.2-1. Therefore, the deterministic lower limit response spectrum is selected to represent the bedrock deterministic MCE response spectrum.

Probabilistic Considerations

The probability of an earthquake of a specific magnitude occurring at a given location is commonly expressed by its return period, i.e., the average length of time between successive occurrences of an earthquake of that size or larger at that location. The return period of a design earthquake is calculated once a project design life and some measure of the acceptable risk that the design earthquake might occur or be exceeded are specified. These expected earthquake recurrences are expressed as a probability of exceedance during a given time period or design life. The ASCE standard *Seismic Design of Pile-Supported Piers and Wharves* defines three seismic hazard levels: the Operating Level Earthquake (OLE), the Contingency Level Earthquake (CLE), and the Design Earthquake (DE). The OLE is defined by 50% probability of exceedance in 50 years, which corresponds to an earthquake with an expected recurrence interval of 72 years, and represents a performance level with minimal structural damage. The CLE is defined by 10% probability of exceedance in 50 years, which corresponds to an earthquake with an expected recurrence interval of 475 years, and represents a performance level of controlled and repairable structural damage. The DE is defined per ASCE 7-05 which develops the response spectra based on ground motions associated with the MCE. The MCE is represented by a probabilistic earthquake with a 2% probability of exceedance in 50 years (return period of 2,475years), except where subject to deterministic limitations (Leyendecker et al., 2000). The Design Earthquake (DE) response spectrum is obtained by taking two-thirds of the MCE level ground motions.

For the Dock site, located at the approximate latitude and longitude coordinates of 45.65°N and 122.71°W, the spectral acceleration values corresponding to the 72, 475 and 2,475 years of return periods were obtained for Site Class B from the 2008 USGS hazard curves and uniform-hazard maps. These spectral accelerations for the three hazard levels are plotted on Figure 7B, and the values are presented on Table 2B.

TABLE 2B: RESPONSE SPECTRA FOR DIFFERENT PROBABILISTIC HAZARD LEVELS

	Spectral Acceleration, g		
Period, seconds	**72 Years**	**475 Years**	**2,475 Years**
0	0.05	0.20	0.41
0.1	0.09	0.39	0.86
0.2	0.11	0.45	0.96
0.3	0.10	0.38	0.83
0.5	0.07	0.29	0.64
1	0.03	0.16	0.37
2	0.01	0.07	0.19
3	0.01	0.04	0.10
4	0.00	0.02	0.07
5	0.00	0.02	0.04

Deaggregation of the 2008 USGS data suggests that the Cascadia Subduction Zone, subcrustal events, and local crustal faults all contribute to the seismic hazard at the site.

Development of Target Bedrock Spectra

The site-specific analysis requires developing a bedrock target spectrum prior to selecting and scaling input acceleration time histories. The bedrock target spectra are developed for all three seismic hazard levels, which are previously discussed in the probabilistic consideration section. The target spectra for the OLE and CLE conditions can be directly represented by the site–specific probabilistic uniform hazard curves which correspond to the ground motion with 50% probability of exceedance in 50 years and 10% probability of exceedance in 50 years, respectively. The target bedrock spectrum for the DE is developed as per the requirements of ASCE 7-05. According to Chapter 21 of ASCE 7-05, the controlling target spectrum is developed by comparing the deterministic and probabilistic MCE response spectra, and taking the lower of the two spectra to represent the site-specific MCE bedrock response spectrum. The comparison of the MCE probabilistic and deterministic response spectra are shown in Figure 8B for the dock site. The probabilistic MCE spectrum is lower than the deterministic spectrum and, therefore, based on the above criterion, the probabilistic spectrum is defined to be the MCE bedrock spectrum.

Site Response Modeling

The effect of a specific seismic event on the site is related to the type and thickness of soil overlying the bedrock at the site and the type and quantity of seismic energy delivered to the bedrock beneath the site by the earthquake. Site response analysis was completed to estimate this site-specific behavior in accordance with ASCE standard for Pile-Supported Piers and Wharves. The site response analysis consisted of three components: 1) selection of target bedrock response spectrum, 2) numerical modeling to analyze the site-specific behavior of the soils using horizontal ground motion acceleration time histories scaled to the approximate level of the target bedrock response spectrum over the periods of interest, and 3) calculation of the ratio of the surface response spectra values to the bedrock response spectra values, at each spectral period, to develop a recommended ground surface response spectrum.

The site response modeling was completed using the D-MOD2000 program by GeoMotions, LLC. D-MOD2000 is a one-dimensional non-linear, time–domain site response modeling program capable of capturing the nonlinear- hysteretic soil behavior during cyclic seismic loading and unloading. The program computes the dynamic response of a layered soil profile to vertically propagating shear waves using total stress or effective stress analyses. The effective stress option provides a means to evaluate the influence of excess pore pressure development and cyclic degradation of soil strength/stiffness (i.e., pore water pressure generation, and pore water pressure dissipation and redistribution) on the dynamic response of the soil profile. D-MOD2000 uses the hyperbolic modified Kodner and Zelasko (MKZ) model to characterize the nonlinear stress-strain soil behavior. The MKZ parameters are generally obtained by fitting the hyperbolic model to published empirical curves.

Within the D-MOD2000 program, the user creates a discretized soil profile and inputs a variety of soil modeling parameters derived from field and laboratory testing and established correlations in the geotechnical literature. A suite of scaled earthquake records are input into the program and propagated up through the soil column to the ground surface. From the modeled ground surface response for a particular soil profile, a Spectral Acceleration Ratio (SAR) can be determined for each earthquake record as the ratio of ground surface to bedrock spectral acceleration ($SA_{surface}/SA_{bedrock}$) at selected periods.

Input Parameters

For the Dock, D-MOD2000 based total and effective stress analyses were performed for evaluating the seismic response and performance of the soil underlying the site. First, a generalized subsurface profile for the site was developed based on our subsurface explorations. Two shear wave velocity profiles at the T-section of the Dock and Pier 3 of the Trestle were estimated based on boring B-26 drilled at Pier 3 and CPT-6 measurements located in the upland area. The ratio of the effective overburden stresses to the one-fourth power, $(\sigma'_{v,\ Dock}/\ \sigma'_{v,\ CPT\text{-}6})^{0.25}$, was used as an adjustment factor to derive the shear wave velocity profiles from CPT-6 measurements. The assumed soil profiles at the T-section of the Dock and Pier 3 of the Trestle are tabulated below in Tables 3B and 4B, respectively.

TABLE 3B: SUBSURFACE PROFILE AT T-SECTION OF DOCK

Material	Thickness, ft	Total Unit Weight, pcf	Estimated Shear Wave Velocity, ft/sec
SAND	3	110	400
SAND	3	110	450
SAND	3	110	500
SAND	4	110	500
SAND	3	110	600
Very dense GRAVEL	5	125	850
Very dense GRAVEL	5	125	950
Very dense GRAVEL	5	125	1,100
Very dense GRAVEL	5	125	1,150
Very dense GRAVEL	10	125	1,200
Very dense GRAVEL	10	125	1,250
Very dense GRAVEL	10	125	1,300
Very dense GRAVEL	10	125	1,350
Very dense GRAVEL	10	125	1,400
Very dense GRAVEL	10	125	1,450
Very dense GRAVEL	10	125	1,500
Very dense GRAVEL	10	125	1,550
Very dense GRAVEL	10	125	1,600
Very dense GRAVEL	10	125	1,650
Very dense GRAVEL	10	125	1,700
Very dense GRAVEL	10	125	1,750
Very dense GRAVEL	10	125	1,800
Very dense GRAVEL	10	125	1,900
Very dense GRAVEL	10	125	2,100
Very dense GRAVEL	10	125	2,200
Troutdale	10	130	2,500

TABLE 4B SUBSURFACE PROFILE AT PIER 3 OF TRESTLE

Material	Thickness, ft	Total Unit Weight, pcf	Estimated Shear Wave Velocity, ft/sec
Loose SAND	3	110	300
Loose SAND	3	110	400
Loose SAND	4	110	350
Loose SAND	3	110	350
Loose SAND	3	110	400
Medium dense SAND	4	110	500
Medium dense SAND	3	110	550
Medium dense SAND	3	110	500
Medium dense SAND	3	110	450
Medium dense SAND	4	110	550
Medium dense SAND	3	110	700

Material	Thickness, ft	Total Unit Weight, pcf	Estimated Shear Wave Velocity, ft/sec
Medium dense SAND	3	110	750
Medium dense SAND	3	110	850
Medium dense SAND	4	110	750
Medium dense SAND	3	110	900
Medium dense SAND	3	110	800
Medium dense SAND	3	110	850
Medium dense SAND	4	110	750
Medium dense SAND	3	110	900
Very dense GRAVEL	5	125	1,100
Very dense GRAVEL	5	125	1,200
Very dense GRAVEL	5	125	1,300
Very dense GRAVEL	5	125	1,350
Very dense GRAVEL	10	125	1,400
Very dense GRAVEL	10	125	1,450
Very dense GRAVEL	10	125	1,500
Very dense GRAVEL	10	125	1,550
Very dense GRAVEL	10	125	1,600
Very dense GRAVEL	10	125	1,650
Very dense GRAVEL	10	125	1,700
Very dense GRAVEL	10	125	1,750
Very dense GRAVEL	10	125	1,800
Very dense GRAVEL	10	125	1,850
Very dense GRAVEL	10	125	1,900
Very dense GRAVEL	10	125	2,000
Very dense GRAVEL	10	125	2,100
Very dense GRAVEL	10	125	2,200
Very dense GRAVEL	10	125	2,300
Very dense GRAVEL	10	125	2,400
Troutdale	10	130	2,500

The material properties and boundary conditions were specified in D-MOD2000 for the Dock site. The unit weights for each of the soil profile were estimated based on the laboratory unit weight test results. The sand and gravel layers encountered throughout the soil profile were assigned the depth- dependent EPRI (1993) sand and rock modulus reduction and damping curves, which accounts for the effects of confining pressure. The representative pore-pressure generation parameters for the sand layers were selected based on grain size distribution curve matching procedures with D-MOD2000. The grain size distribution curve for the sand layer was compared to the liquefiable sands and silts curves within D-MOD2000 database, and the Santa Monica Beach (SMB) sand was selected to approximate the pore-pressure generation parameters. The half-space boundary condition at the base of the model was represented by a visco-elastic boundary with a unit weight of 130 pounds per cubic foot (pcf) and a shear wave velocity of 2,500 feet per second.

Ground Motion Selection and Scaling

The target bedrock response spectrum for the site was developed for Site Class B, or rock site, conditions in accordance with the method outlined in the Target Bedrock Spectrum section of this report for OLE, CLE and MCE hazard levels. A series of earthquake acceleration-time histories have been selected and scaled to the target bedrock spectrum. From the available records, corrected free-field and basement/ground floor accelerograms were selected for input as bedrock time histories. Wherever possible, earthquakes of similar magnitude and duration to the characteristic earthquakes were selected. These records were checked for obvious errors, missing data points, and other anomalies and were transformed into a uniform data format. The selected strong-motion records are tabulated below in Table 5B.

TABLE 5B SELECTED STRONG-MOTION RECORDS

Earthquake	Recording Station	Magnitude	Rupture Distance, km	Peak Bedrock Acceleration, g
Loma Prieta (1989)	San Jose - Santa Teresa Hills	6.9	14.7	0.28
Nisqually (2001)	Olympia, WSDOT Test Lab	6.8	18.3	0.22
Chile (2010)	Curico	8.8	65.1	0.47
Chile (2010)	Hualane	8.8	50	0.46
Japan (2011)	Kuroiso (TCG001)	9	102	0.42
Japan (2011)	Yamatsuri (FKS 014)	9	76	0.23
Japan (2011)	Hachinohe (AOM 012)	9	99	0.19

The selected acceleration time histories were then scaled to reasonably match the bedrock target spectrum at periods of interest including the site fundamental period for the OLE, CLE and MCE hazard levels. The scaling process involves selecting a single factor and multiplying the acceleration time history by this factor so that its pseudo acceleration response spectrum more closely matches the target spectrum at the period of interest.

Site Response Results

Using the generalized subsurface profiles (i.e., at the T-section of the Dock and Pier 3 of the Trestle), the target spectra developed at the bedrock, and the strong ground motion records listed in the preceding tables, pseudo acceleration response spectra were computed for the Dock site with the D-MOD2000 nonlinear model. The ground surface response spectra were developed at 5% of critical damping. The ground surface spectra were compared to the input rock spectra to quantify amplification and/or attenuation through the soil column at the site. The ratio of ground surface to bedrock spectral accelerations, defined as the spectral amplification ratio (SAR), is shown on Figure 9B and Figure 10B for the CLE and MCE conditions, respectively. To estimate ground surface site response throughout the range of spectral periods, the target response spectra is multiplied by the SAR to determine the ground surface response spectrum. The results of the site-specific response modeling are shown on Figures 11B and 12B for the CLE and MCE hazard levels, respectively. Figures 11B and Figure 12B also include the code-based spectrum (i.e., for example Site Class D and Site Class E), developed using site amplification factors based on the appropriate Site Class type. The code-based spectrum is typically derived based on the 0.2 and 1 second spectral accelerations values at the bedrock and site coefficients (i.e., F_a and F_v) provided in Table 11.4-1 and Table 11.4-2 of ASCE 7-05. The code-based site coefficients and the spectral values corresponding to 0.2 and 1 second periods are provided on Table 6B for the OLE, CLE and MCE hazard levels. The two spectral values are obtained from the map of spectral acceleration parameters provided in Chapter 22 of ACSE 7-05 for the MCE condition while the spectral values corresponding to 72- and 475-year return periods are obtained from USGS data for the OLE and CLE conditions, respectively.

TABLE 6B: SITE COEFFICIENTS AND SPECTRAL VALUES

Hazard Level, Ss and S1 Values	Site Class	F_a	F_v
OLE $S_s = 0.11$ g, $S_1 = 0.03$ g	D	1.6	2.4
	E	2.5	3.5
CLE $S_s = 0.45$ g, $S_1 = 0.16$ g	D	1.44	2.17
	E	1.87	3.33
MCE $S_s = 0.94$ g, $S_1 = 0.41$ g	D	1.12	1.59
	E	0.97	2.40

The site is generally designated as Site Class D based on the average shear wave velocity (V_{S100}) in the upper 100 ft per Section 20.4 of ASCE 7-05. For the OLE hazard level the degree of ground shaking is insufficient to cause liquefaction of the soil. Therefore, the code-based Site Class D depicted on Figure 13B is recommended for the OLE condition to estimate the ground response spectral acceleration.

For the CLE and MCE hazard levels, our analyses indicate the soil may liquefy and the site is designated as Site Class F in accordance with section 20.3.1 of ASCE 7-05. A site response analyses is required for Site Class F designations. Structures with a fundamental period less than 0.5 second are exempted from this requirement, and the response spectrum may be developed based on the appropriate Site Class for non-liquefied site conditions. Research by Youd and Carter (2005) indicates that code–based response spectra for non-liquefied conditions may be conservatively extended to represent the response spectra for structures with fundamental periods less than 1.0 sec for liquefied conditions. Per Section 21.3 of ASCE 7-05, the ground surface response spectra developed from site response analysis for Site Class F may not be less than 80% of the Site Class response spectrum values assuming no liquefaction.

The calculated and recommended site specific ground surface response spectra at the T-section of the dock and at Pier 3 of the trestle are shown on Figures 11B and 12B for the CLE and MCE, respectively. The calculated values presented on the figures are based on total stress analysis. However, effective stress analyses were also completed for comparison. As anticipated, the effective stress analyses resulted in a "softening" of the soil response and generally lower response values than the total stress analyses. The trends associated with the effective stress analyses were considered in selecting the recommended spectra. The recommended response spectra are also intended to conservatively envelope the estimated response for both the T-section of the dock and the Pier 3 location of the trestle. It should be noted that Figures 11B and 12B also include a spectra based on 80% of Site Class E in addition to site class D spectral values due to liquefaction and code considerations for liquefiable profiles.

CLE Hazard Level. The estimated CLE ground surface response spectra at periods greater than 0.5 second are less than or equal to 80% of the Site Class E response spectrum which is the minimum allowed by the code for periods in this range of interest. At periods less than 0.5 second, the code allows the use of the non-liquefied site class D spectrum. We have recommended a slight increase from the site class D spectrum at short periods to match 80% of the site class E spectrum to account for the shorter period amplification observed in the total stress analyses of the T-section of the dock. Considering the range of analyses and uncertainties, we have recommended a single CLE hazard level response spectrum at all periods for both the dock and trestle structure. This spectrum based on 80% of the Site Class E response spectrum is shown on Figure 11B.

DE Hazard Level. The estimated MCE ground surface response spectra from the total stress analysis typically exceed either the Site Class D or 80% of Site Class E spectra at periods less than 1.8 seconds. However, total stress parameters do not account for reductions in the response due to softening of the liquefied soils observed in our effective stress analyses, particularly at short periods. At the MCE hazard level, the site class D spectrum is more conservative than the site class E spectrum and we have recommended the site class D spectrum in accordance with the recommendations in ASCE7-05. At periods between 0.89 and 1.8 seconds we conservatively recommend the design spectrum include an increase above Site Class D and E to transition to longer periods and envelop the estimated site specific ground surface response. At periods greater than approximately 1.8 seconds, the site-specific response spectrum is less than 80% of Site Class E, which is the minimum spectral amplification allowed by ASCE 7-05 for liquefied conditions. Plots of our recommended response spectra are shown on the Figure 12B for the MCE. The design earthquake (DE) response spectrum is determined by taking two-thirds of the MCE response spectrum and is shown on Figure 14B.

Conclusions

The site-specific response modeling for the Dock site was completed using total and effective stress parameters based on the generalized subsurface profiles developed at the T-section of the Dock and Pier 3 of the Trestle. The site response modeling was performed for three seismic hazard levels (i.e., OLE, CLE and DE) to meet the requirements of ASCE standard *Seismic Design of Pile-Supported Piers and Wharves*.

For the OLE hazard level, we recommend the code-based Site Class D response spectrum as shown on Figure 13B. For the CLE hazard level, the recommended spectrum is based on 80% of the code based Site Class E response spectrum. For the DE hazard level, the code-based Site Class D response spectrum is recommended for periods less than 0.89 second. At longer periods the recommended response spectrum envelopes the site-specific spectral response values and 80% of the Site Class E response spectrum. The recommended spectra for CLE and DE are shown on Figures 11B and 14B, respectively.

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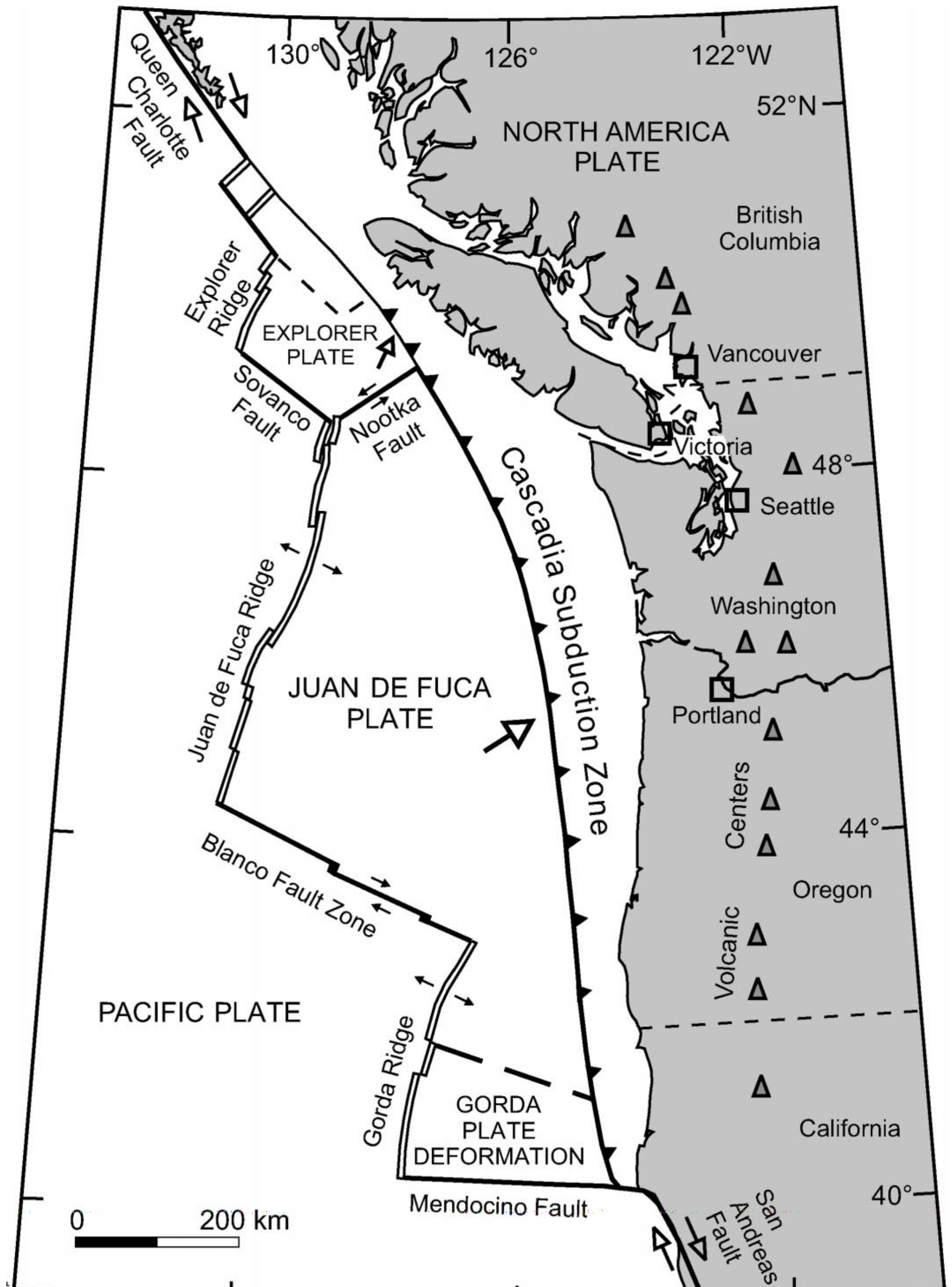
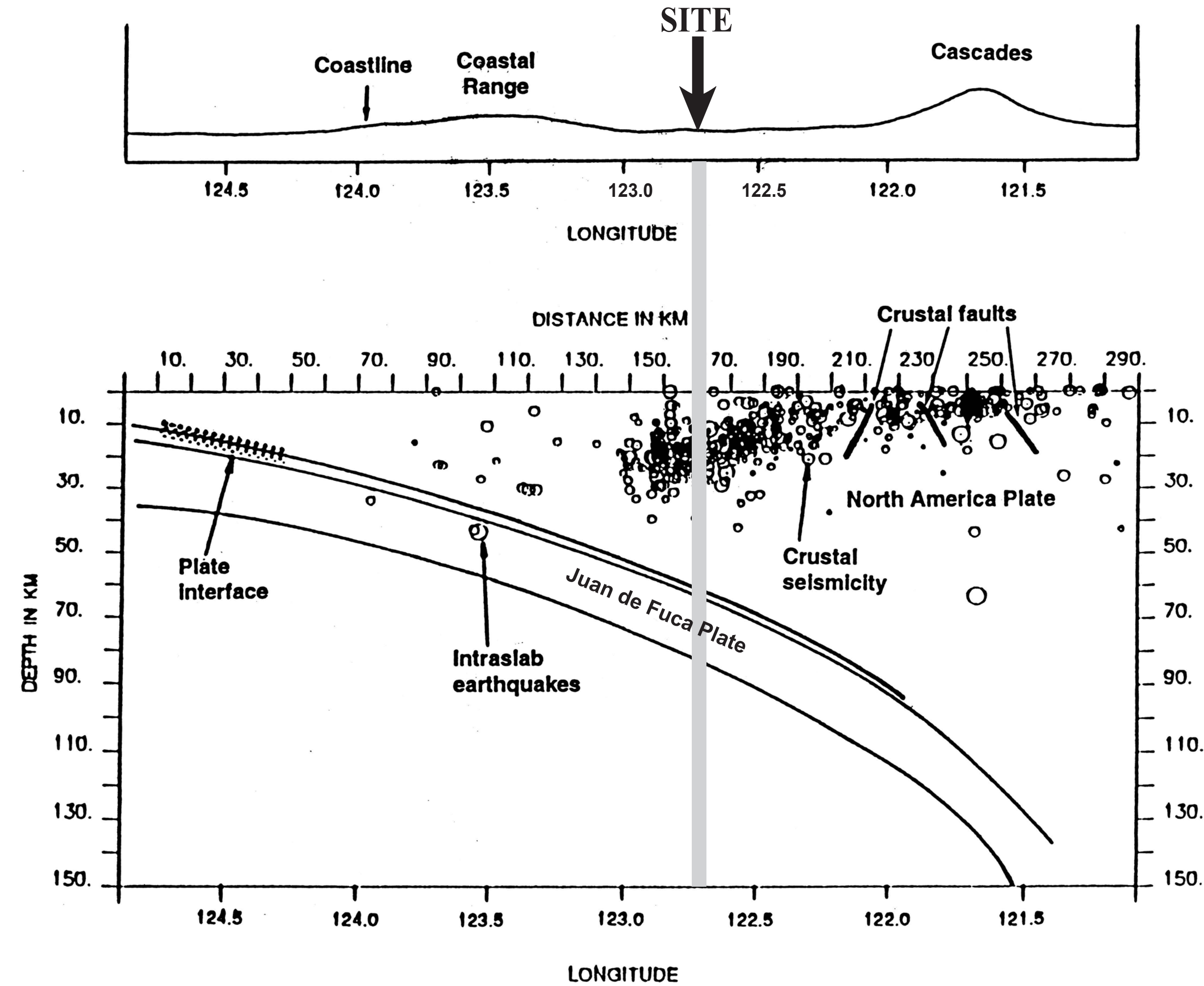


A) TECTONIC MAP OF PACIFIC NORTHWEST, SHOWING ORIENTATION AND EXTENT OF CASCADIA SUBDUCTION ZONE (MODIFIED FROM DRAGERT AND OTHERS, 1994)

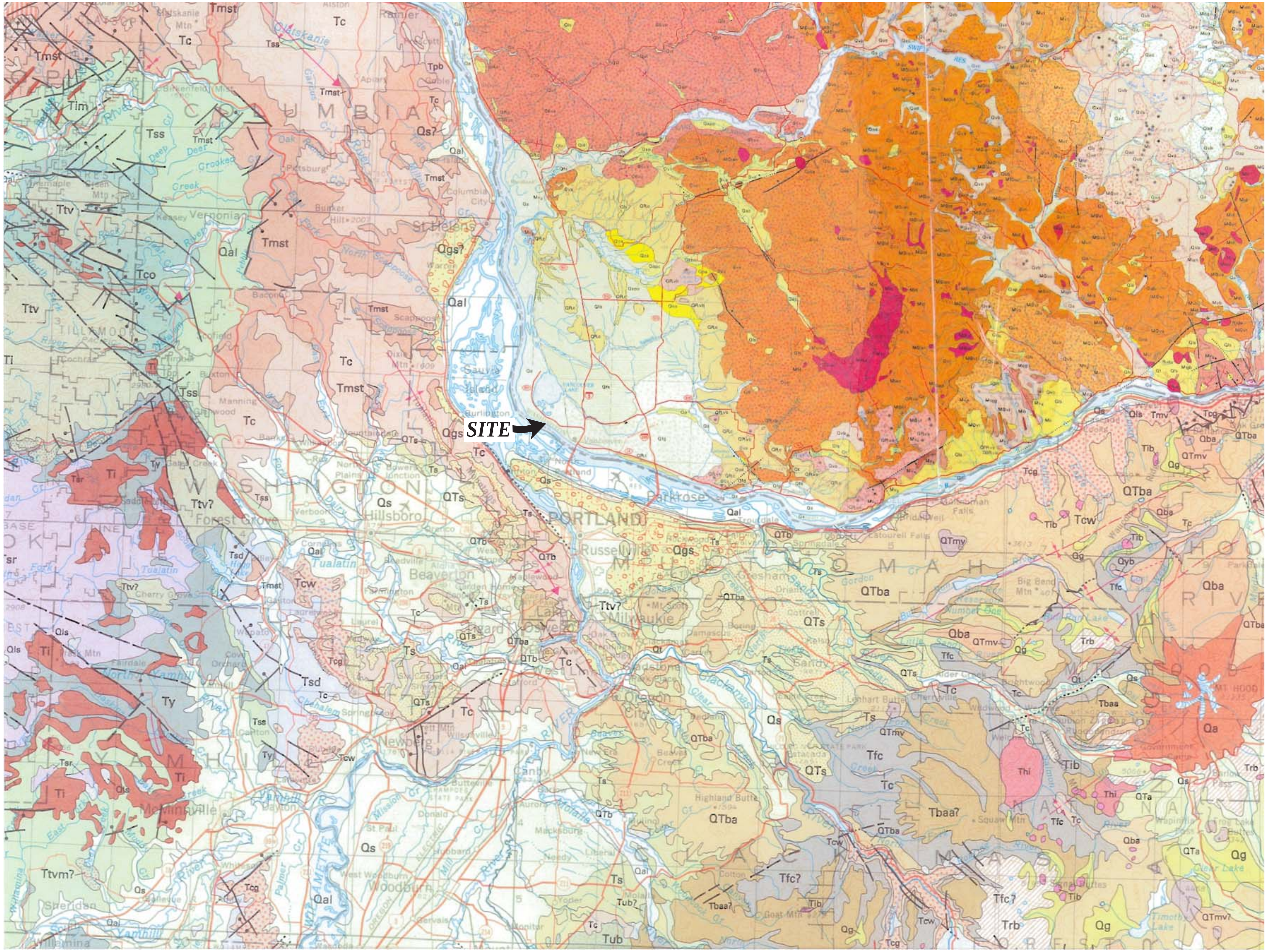


B) EAST-WEST CROSS-SECTION THROUGH WESTERN OREGON AT THE LATITUDE OF PORTLAND, SHOWING THE SEISMIC SOURCES CONSIDERED IN THE SITE-SPECIFIC SEISMIC HAZARD STUDY (MODIFIED FROM GEOMATRIX, 1995)

GRI

TECTONIC SETTING SUMMARY

SEP. 2014 JOB NO. W1114-T3 FIG. 1B



Contact — Approximately located

Fault — Dashed where inferred; dotted where concealed; queried where doubtful; ball and bar on downthrown side

Thrust fault — Dashed where inferred; dotted where concealed; queried where doubtful; sawteeth on upper plate

7 Strike and dip of bed

FROM:

WALSH, T.J., KOROSEC, M.A., PHILLIPS, W.M., LOGAN, R.L., AND SCHASSE, H.W., 1987, GEOLOGIC MAP OF WASHINGTON-SOUTHWEST QUADRANT; 1:250,000: WASHINGTON DIVISION OF GEOLOGY AND EARTH RESOURCES, GM-34

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North

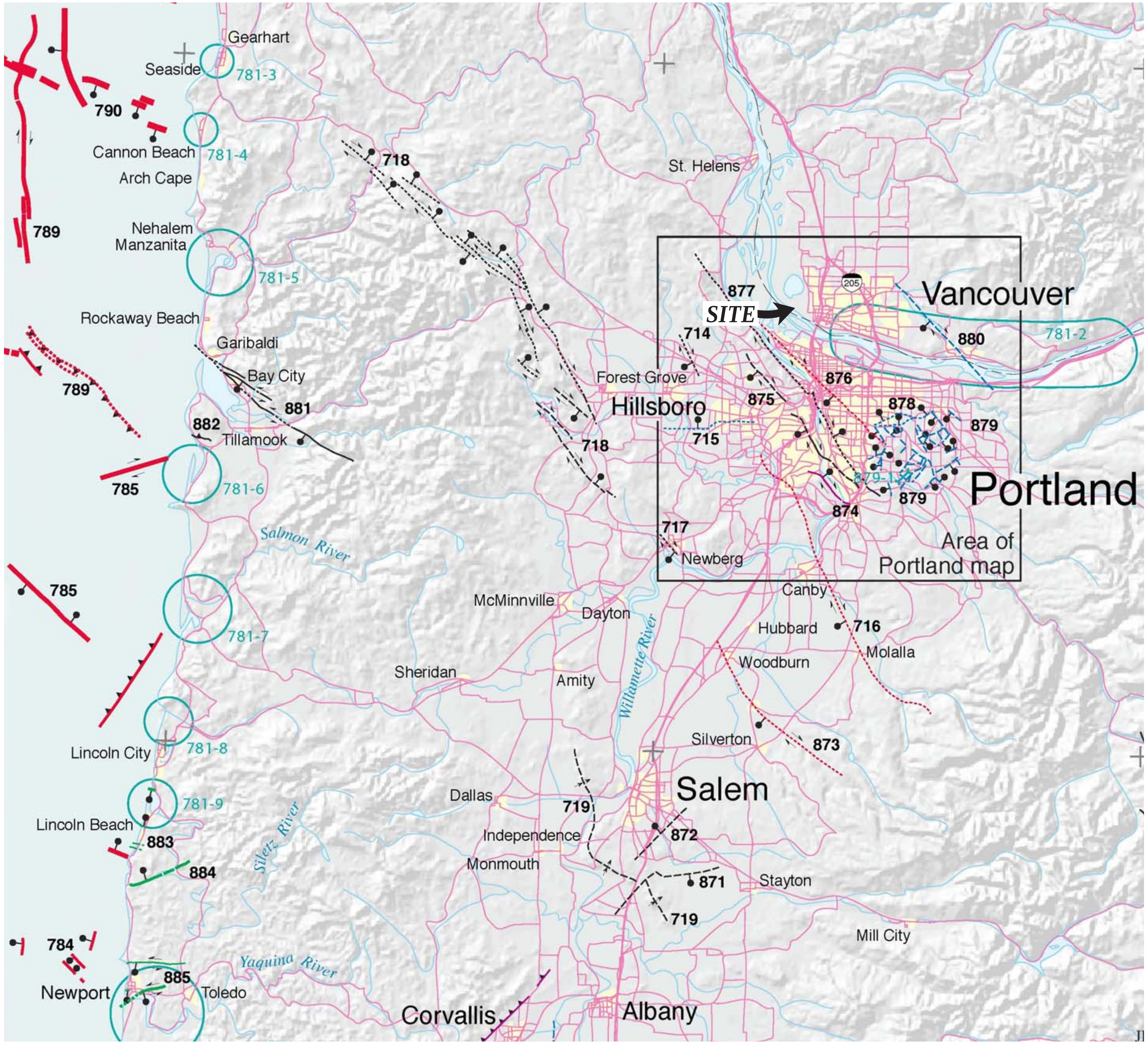
0 10 20 MILES

0 10 20 KILOMETERS

GRI

REGIONAL GEOLOGIC MAP

SEP. 2014 JOB NO. W1114-T3 FIG. 2B



MAP EXPLANATION

TIME OF MOST RECENT SURFACE RUPTURE
- Holocene (<10,000 years) or post last glaciation (<15,000 years; 15 ka); no historic ruptures in Oregon to date
- Late Quaternary (<130,000; post penultimate glaciation)
- Late and middle Quaternary (<750,000 years; 750 ka)
- Quaternary, undifferentiated (<1,600,000 years; <1.6 Ma)
- Class B structure (age or origin uncertain)

SLIP RATE
- >5 mm/year
- 1.0-5.0 mm/year
- 0.2-1.0 mm/year
- <0.2 mm/year

TRACE
- Mostly continuous at map scale
- Mostly discontinuous at map scale
- Inferred or concealed

STRUCTURE TYPE AND RELATED FEATURES
- Normal or high-angle reverse fault
- Strike-slip fault
- Thrust fault
- Anticlinal fold
- Synclinal fold
- Monoclinal fold
- Plunge direction of fold
- Fault section marker

DETAILED STUDY SITES
- 731-2 Trench site
- 781-2 Subduction zone study site

CULTURAL AND GEOGRAPHIC FEATURES
- Divided highway
- Primary or secondary road
- Permanent river or stream
- Intermittent river or stream
- Permanent or intermittent lake

FAULT NUMBER	NAME OF STRUCTURE
876	EAST BANK FAULT
877	PORTLAND HILLS FAULT
880	LACAMAS LAKE FAULT
878	GRAND BUTTE FAULT

FROM: PERSONIUS, S.F., AND OTHERS, 2003, MAP OF QUATERNARY FAULTS AND FOLDS IN OREGON, USGS OPEN FILE REPORT OFR-03-095.

North

0 5 10 MILES

0 5 10 KILOMETERS

GRI

LOCAL FAULT MAP

SEP. 2014 JOB NO. W1114-T3 FIG. 3B

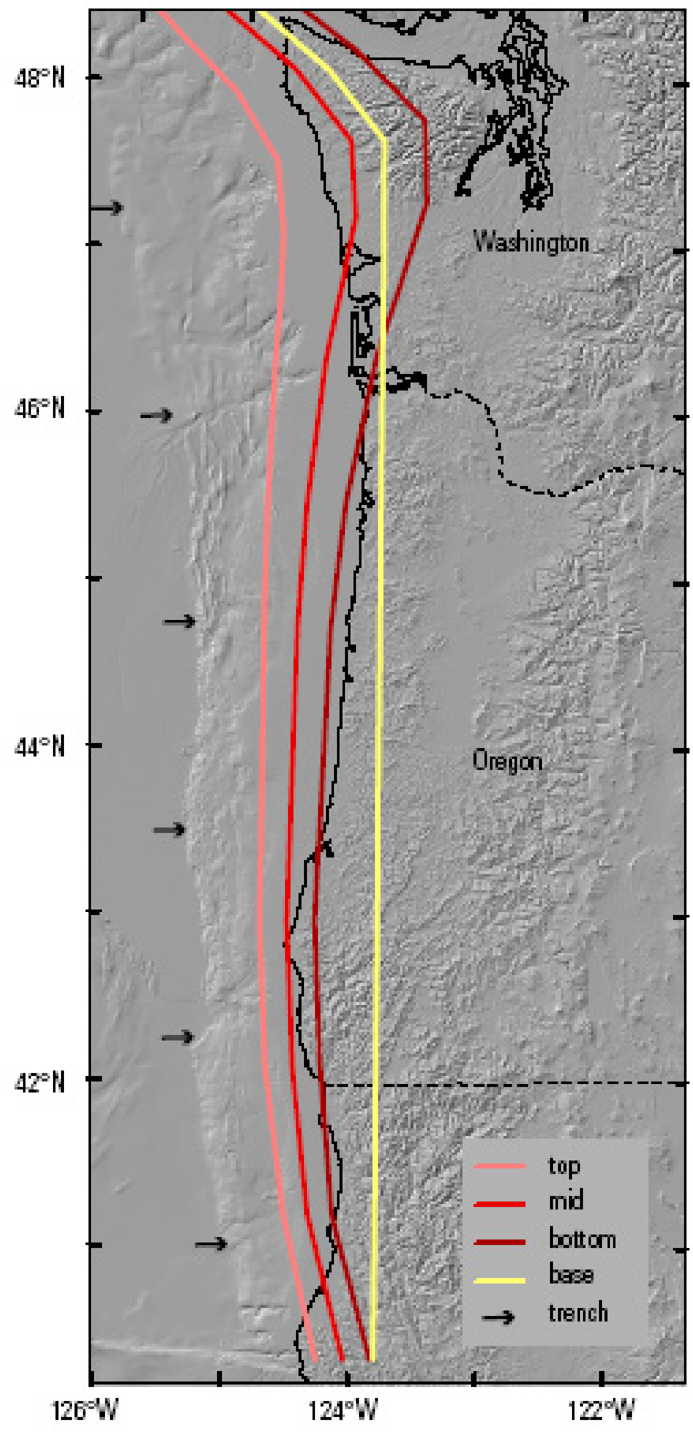


Figure 21. Location of the eastern edge of earthquake-rupture zones on the Cascadia subduction zone for the various models used in this study relative to the surficial expression of the trench: top, base of the elastic zone; mid, midpoint of the transition zone; bottom, base of the transition zones; base, base of the model that assumes ruptures extend to about 30-kilometers depth. Figure provided by Ray Weldon.

FROM: PETERSEN, MD, FRANKEL, AD, HARMSEN, SC, AND OTHERS, 2008, DOCUMENTATION FOR THE 2008 UPDATE OF THE UNITED STATES NATIONAL SEISMIC HAZARD MAPS: US GEOLOGICAL SURVEY, OPEN FILE REPORT 2008-1128

GRI

ASSUMED RUPTURE LOCATIONS
(CASCADIA SUBDUCTION ZONE)

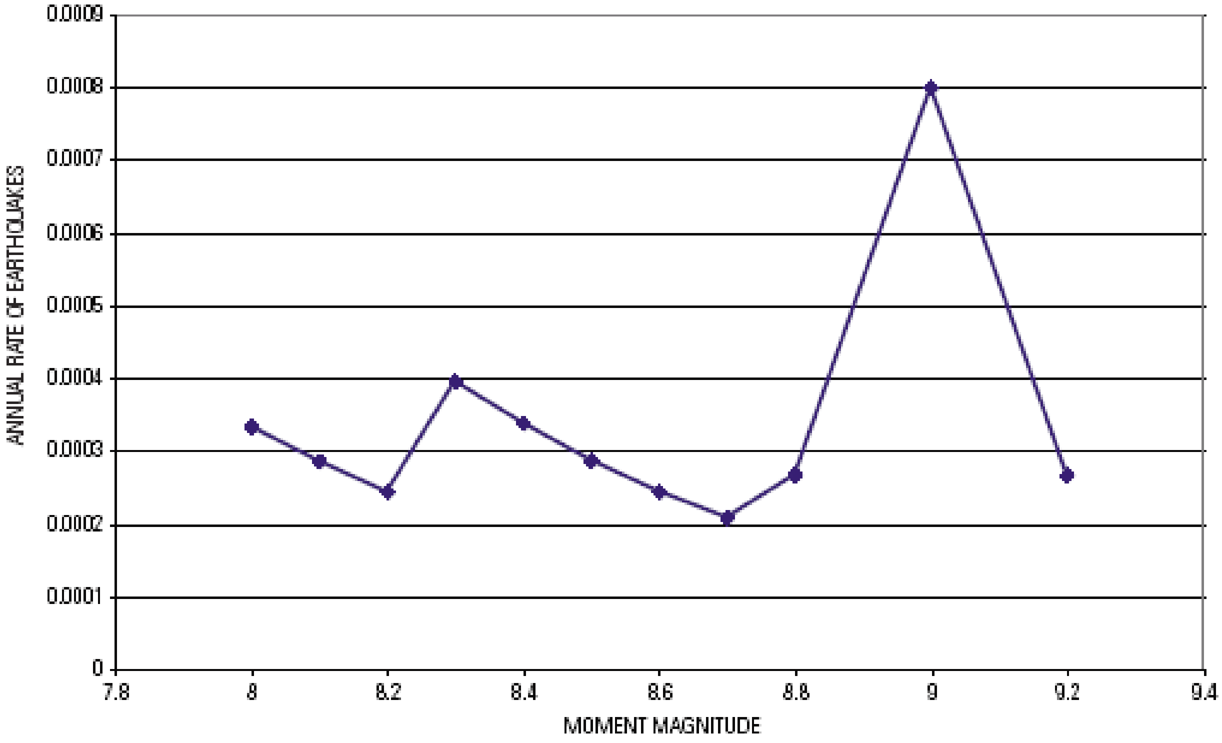


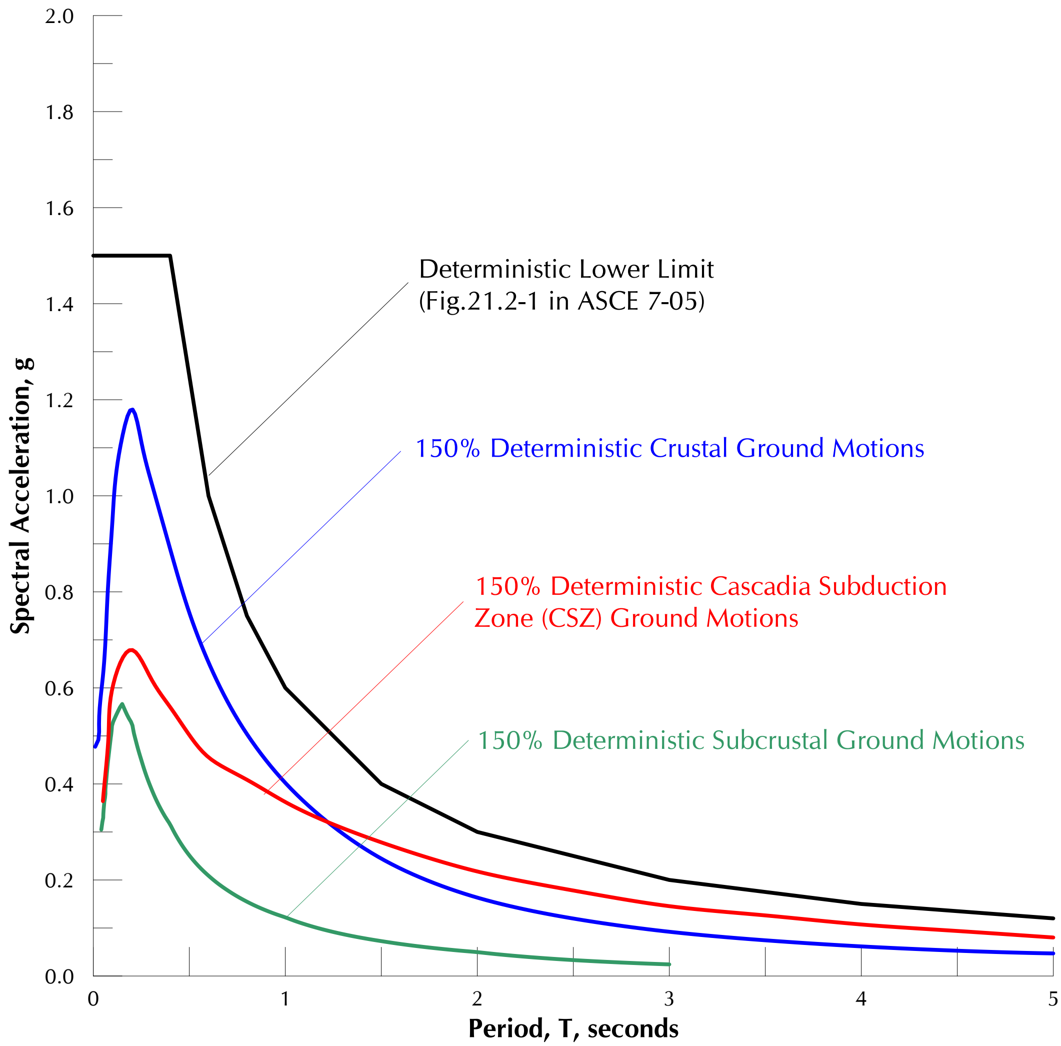
Figure 22. Magnitude-frequency distribution of the Cascadia subduction zone.

FROM: PETERSEN, M, FRANKEL, A, HARMSEN, S, AND OTHERS, 2008, DOCUMENTATION FOR THE 2008 UPDATE OF THE UNITED STATES NATIONAL SEISMIC HAZARD MAPS: US GEOLOGICAL SURVEY, OPEN FILE REPORT 2008-1128

GRI

ASSUMED
MAGNITUDE-FREQUENCY DISTRIBUTION
(CASCADIA SUBDUCTION ZONE)

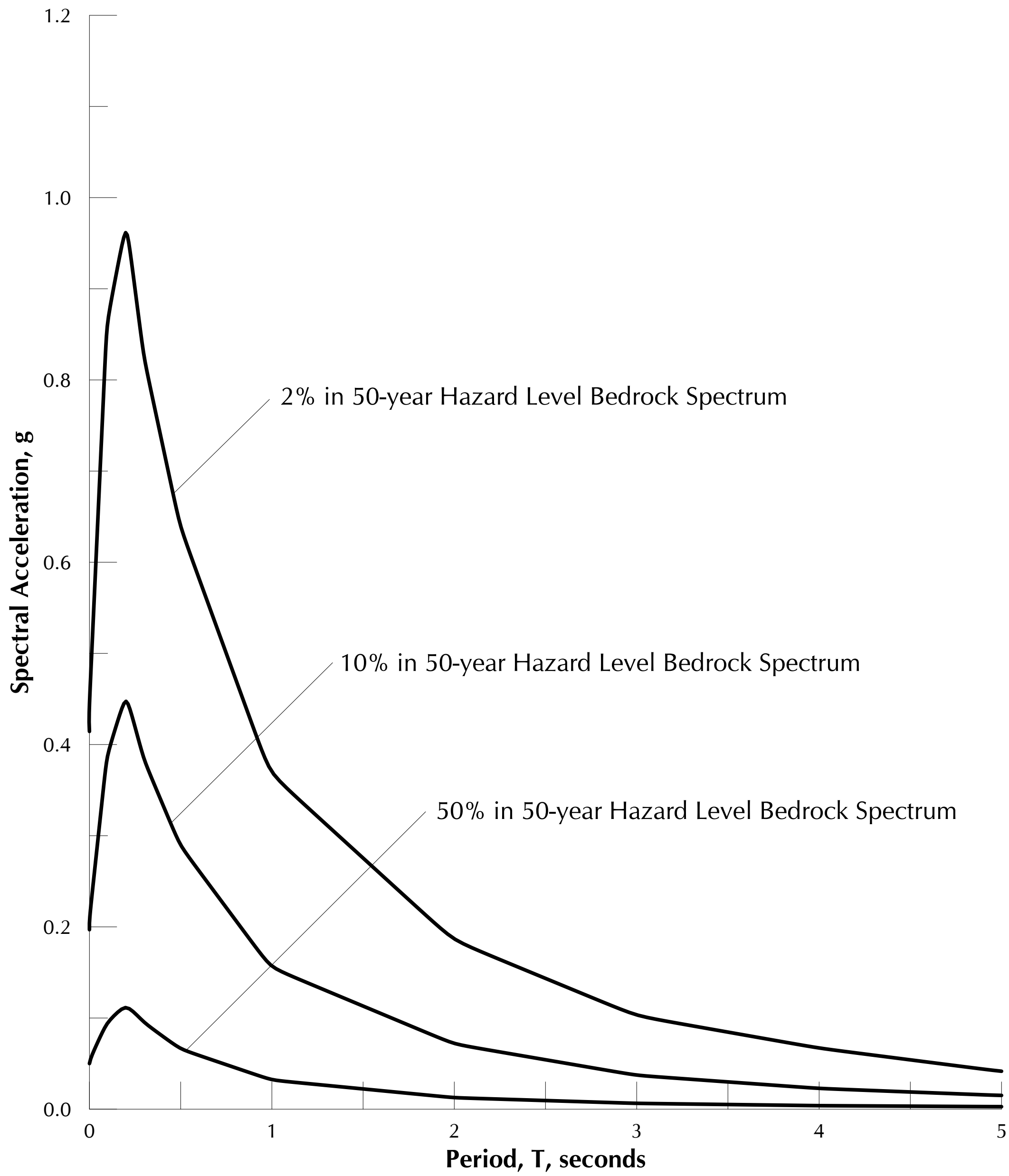
SEP. 2014 | JOB NO. W1114-T3 | FIG. 5B



GRI

DETERMINISTIC MCE
BEDROCK RESPONSE SPECTRA COMPARISON
(5% DAMPING)

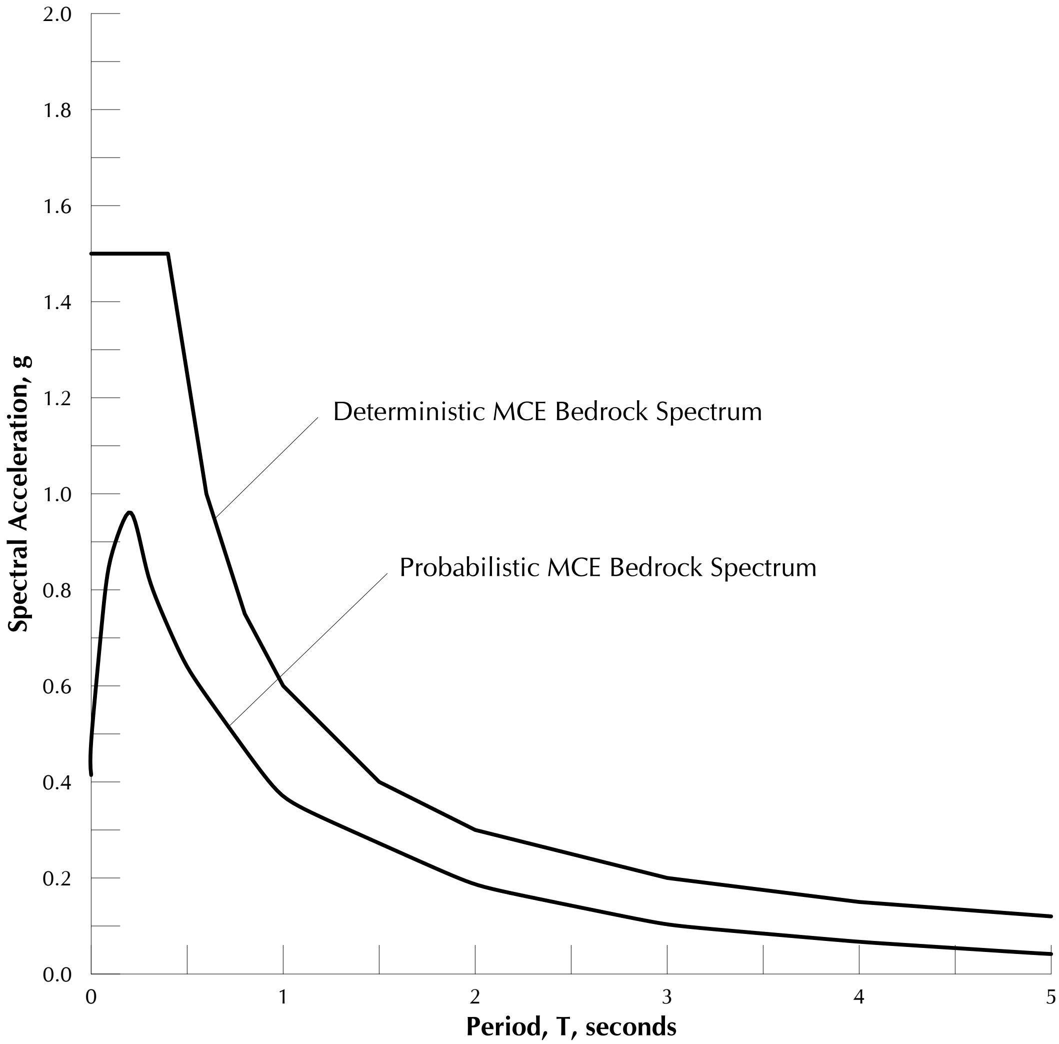
SEP. 2014 | JOB NO. W1114-T3 | FIG. 6B



GRI

BEDROCK RESPONSE SPECTRA FOR
DIFFERENT PROBABILISTIC HAZARD LEVELS
(5% DAMPING)

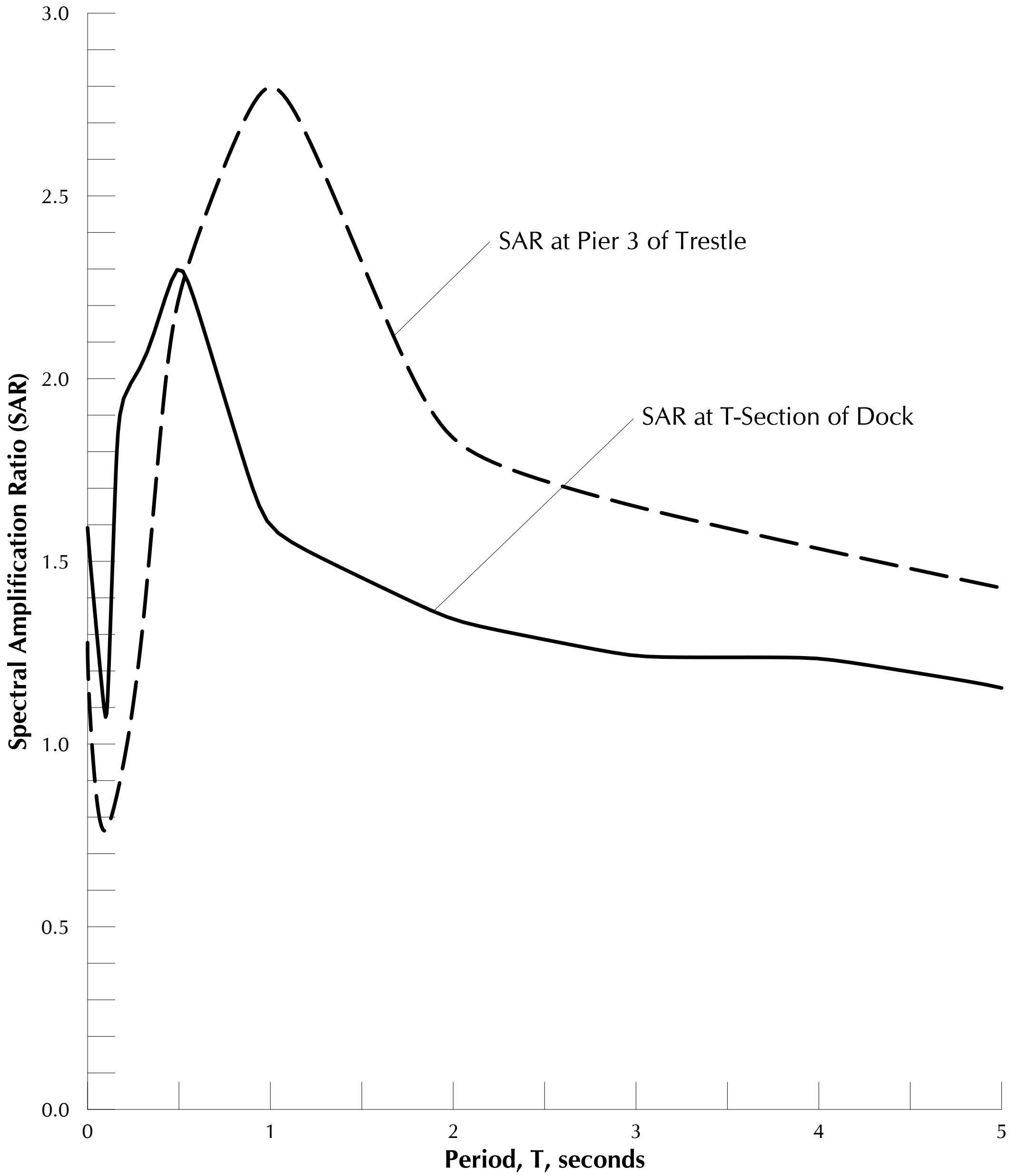
SEP. 2014 | JOB NO. W1114-T3 | FIG. 7B



GRI

DETERMINISTIC AND PROBABILISTIC MCE
BEDROCK RESPONSE SPECTRA COMPARISON
(5% DAMPING)

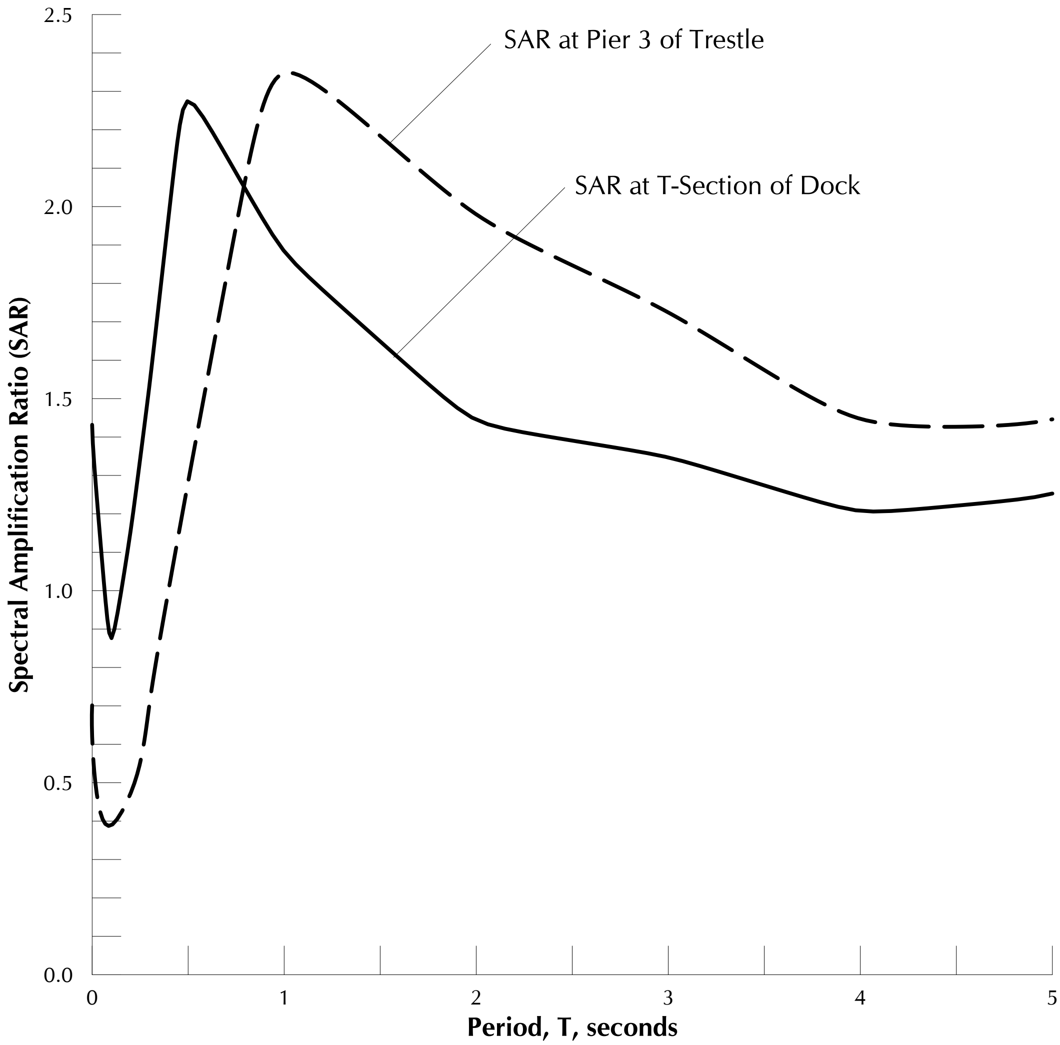
SEP. 2014 JOB NO. W1114-T3 FIG. 8B



GRI

SPECTRAL AMPLIFICATION RATIO (SAR)
FOR CLE (475-YEAR) HAZARD LEVEL

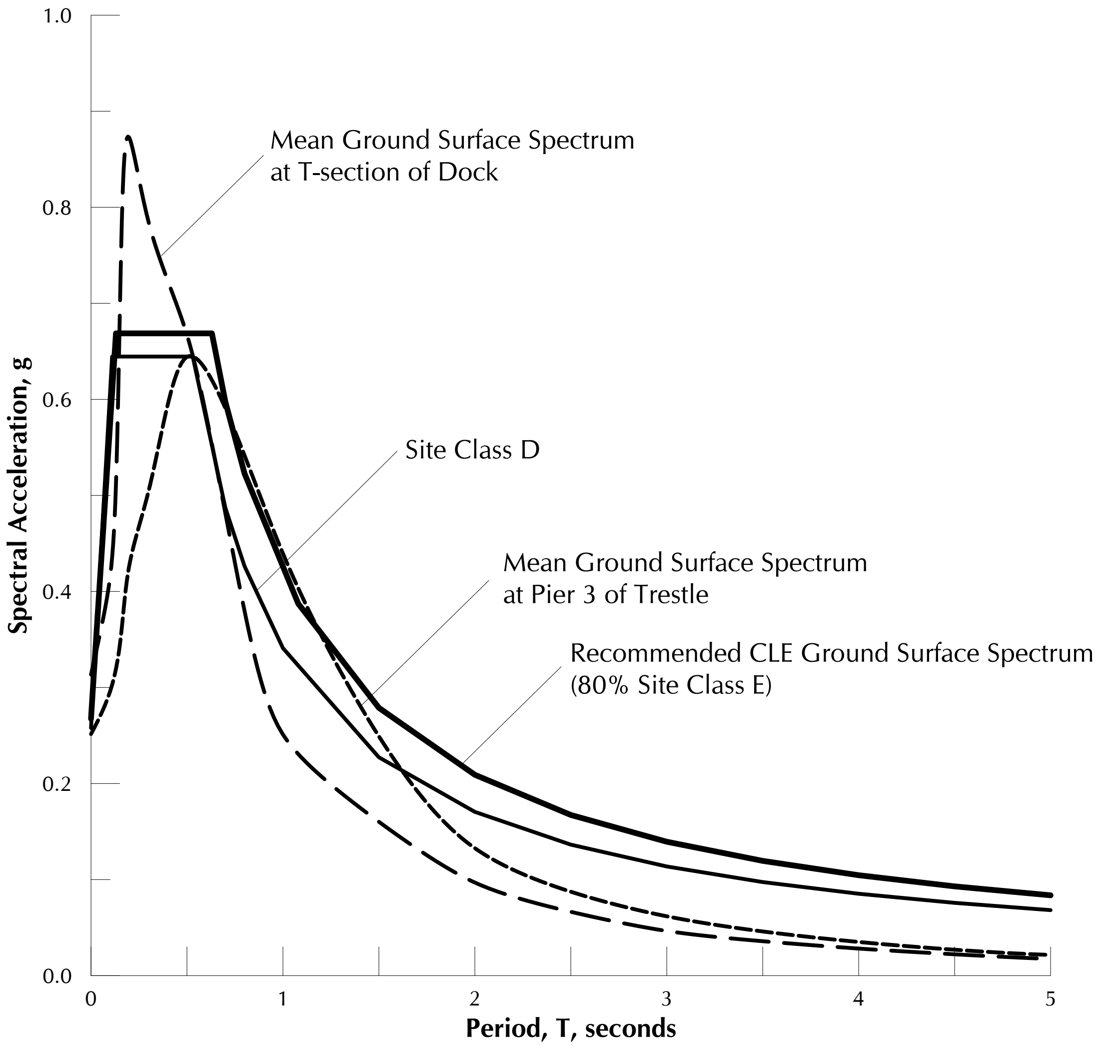
SEP. 2014 | JOB NO. W1114-T3 | FIG. 9B



GRI

SPECTRAL AMPLIFICATION RATIO (SAR)
FOR MCE (2,475-YEAR) HAZARD LEVEL

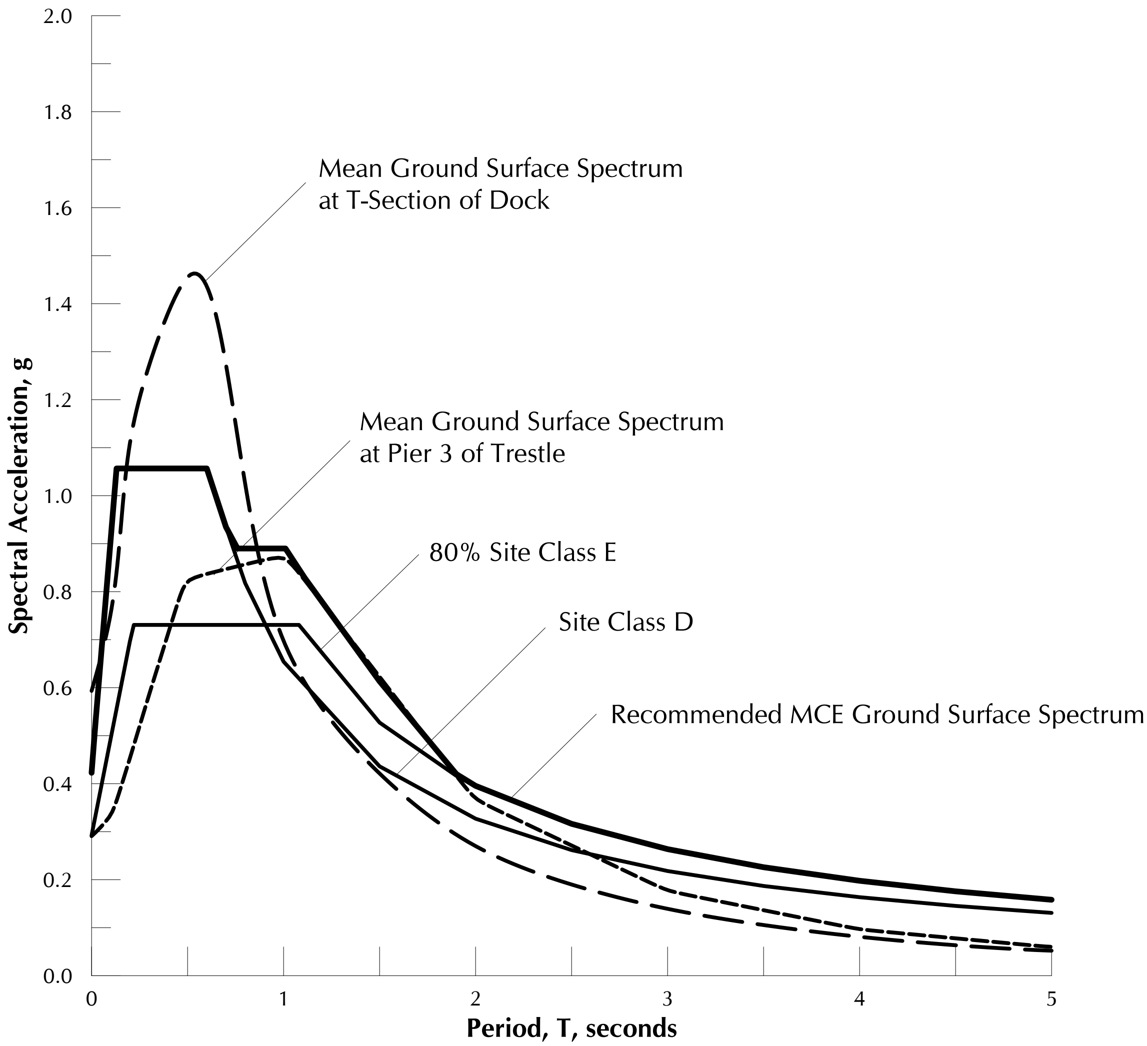
SEP. 2014 | JOB NO. W1114-T3 | FIG. 10B



GRI

CLE GROUND SURFACE
RESPONSE SPECTRA
(5% DAMPING)

SEP. 2014 JOB NO. W1114-T3 FIG. 11B



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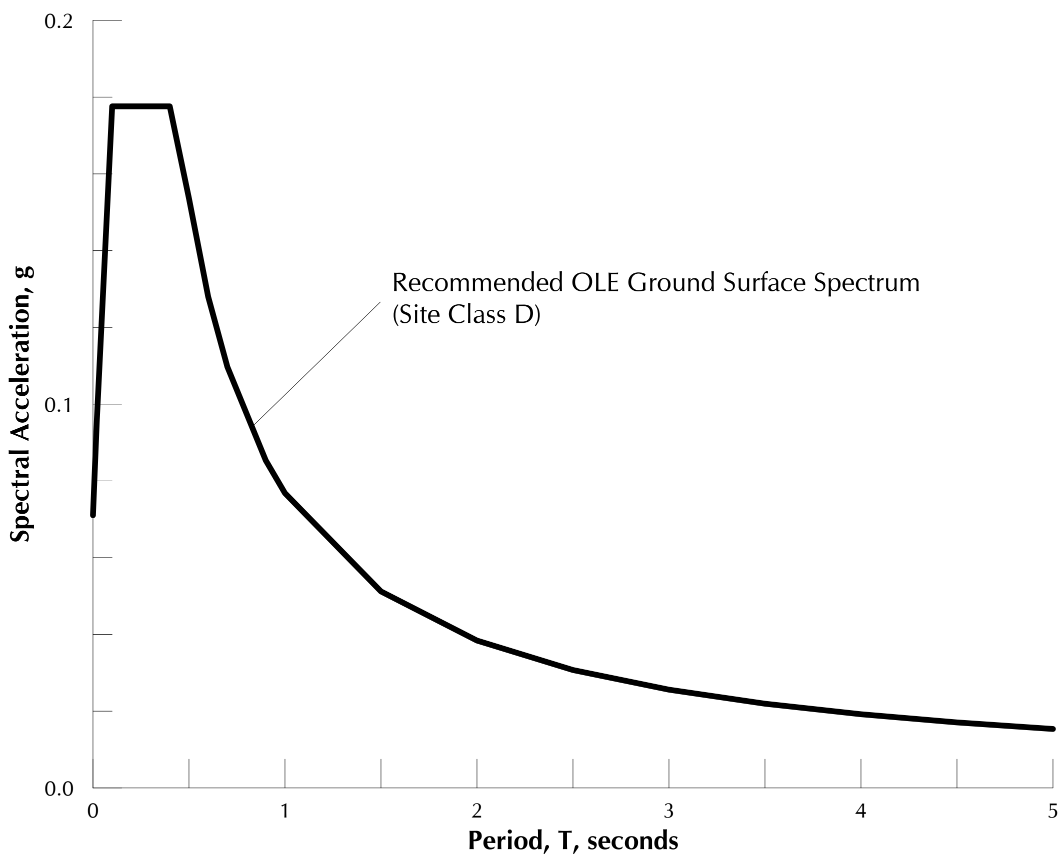
MCE GROUND SURFACE
RESPONSE SPECTRA
(5% DAMPING)

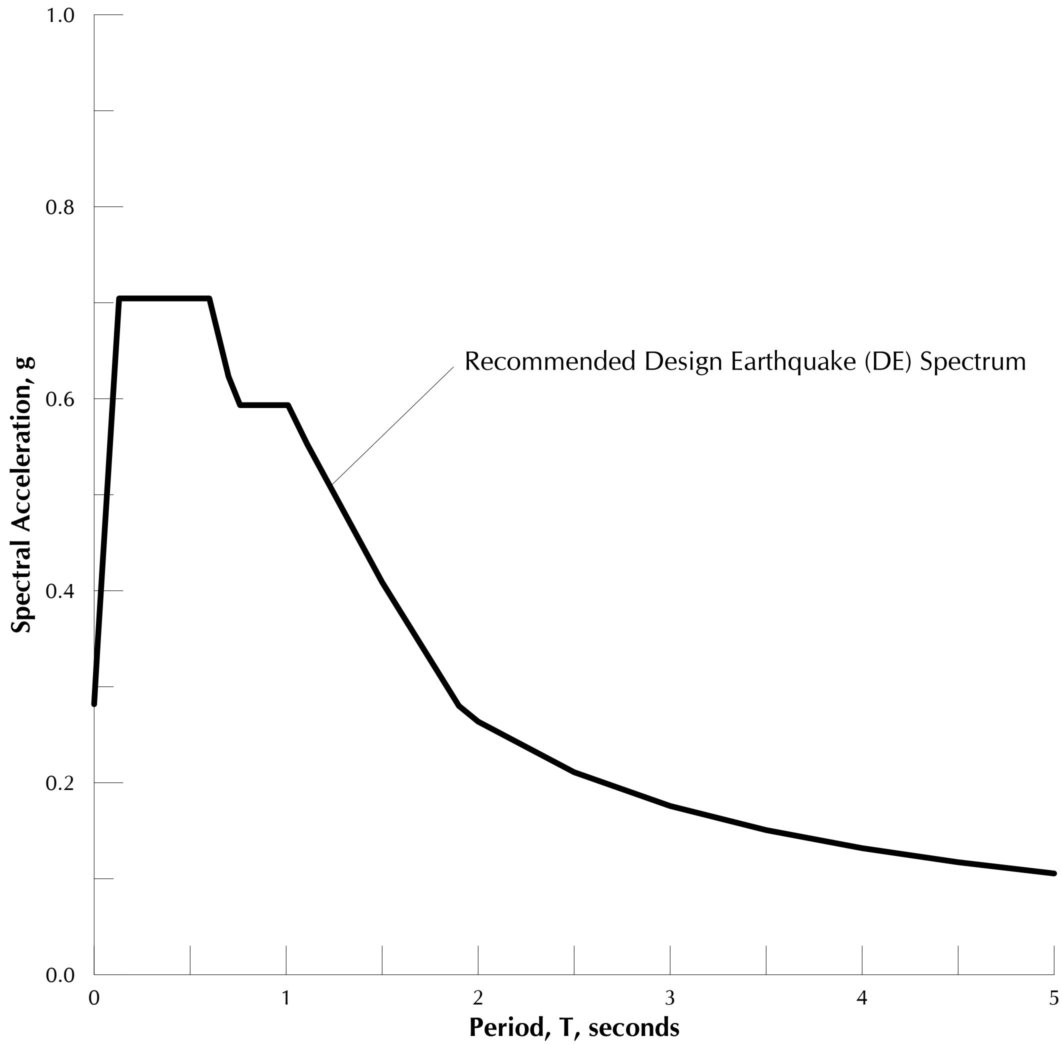
SEP. 2014 | JOB NO. W1114-T3 | FIG. 12B



GRI

OLE GROUND SURFACE
RESPONSE SPECTRUM
(5% DAMPING)

SEP. 2014 JOB NO. W1114-T3 FIG. 13B



GRI

DE (2/3 MCE) GROUND SURFACE
RESPONSE SPECTRUM
(5% DAMPING)

SEP. 2014 JOB NO. W1114-T3 FIG. 14B