Geotechnical Report

Hawk Street Extension
Community of Neskowin
Tillamook County, Oregon

Prepared for:

OBEC Consulting Engineers
Eugene, Oregon

May 16, 2017

Professional
Geotechnical
Services

Foundation Engineering, Inc.
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Austin Bloom
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920 Country Club Road
Eugene, Oregon 97401

May 16, 2017

Hawk Street Extension
Geotechnical Report
Community of Neskowin
Tillamook County, Oregon

Project 2151044

Dear Mr. Bloom:

We have completed the requested geotechnical investigation for the proposed Hawk Street Extension project in Neskowin, Oregon. This report includes a description of our work, a discussion of the site conditions, a summary of field and laboratory testing, and a discussion of engineering analyses. Recommendations are included for site preparation, embankment construction, and bridge foundation design.

This report was prepared in substantial conformance with the Oregon Department of Transportation (ODOT) Geotechnical Design Manual (GDM) (2015). Our construction recommendations refer to sections in the Oregon Standard Specifications for Construction (2015).

It has been a pleasure assisting you with this phase of your project. Please do not hesitate to contact us if you have any questions or require further assistance.

Sincerely,

FOUNDATION ENGINEERING, INC.

Matt Mason

Matt Mason, P.E.
Project Engineer

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GEOTECHNICAL REPORT

HAWK STREET EXTENSION
COMMUNITY OF NESKOWIN
TILLAMOOK COUNTY, OREGON

1.0 INTRODUCTION

1.1 Project Description

Most of the project information contained in this report is from the Preliminary Report prepared by OBEC Consulting Engineers dated August 5, 2016, and from the Advanced Plan Set dated April 3, 2017.

Salem Avenue currently provides the primary access to the Community of Neskowin. However, Salem Avenue is prone to flooding. Therefore, the community of Neskowin and Tillamook County plan to construct secondary access road. The most feasible option is to extend Hawk Street, from the Amity Avenue intersection in Neskowin to the Oregon State Parks Wayside. The project location is shown on the Vicinity Map, Figure 1A (Appendix A).

The Hawk Street Extension project includes raising the unimproved street enough to minimize overtopping from the 100 year flood. The 100-year flood elevation is El. 15.03. Therefore, the street grade will be raised from its current elevation of ±El. 11 to 12, to a minimum of El. 14.9.

A 32-foot long, single-span bridge over Butte Creek and a 32-foot long, single-span bridge over Hawk Creek are proposed. The crossing at Tributary 1 will be improved with the addition of a 12-foot wide by 10-foot high, 30-foot long box culvert.

A 90-foot long, 5-foot high retaining wall is required at the south end of the street to keep the new embankment fill with the right of way.

Tillamook County is the project owner. West Consultants completed the hydraulics study and OBEC Consulting Engineers (OBEC) is the roadway and structural designer. Foundation Engineering, Inc. was retained by OBEC as the geotechnical consultant. Our original statement of work is provided as part of Task Order No. 7 (Tillamook County Contract #4572) between OBEC and Foundation Engineering, Inc. dated May 12, 2015. An amendment to add additional geotechnical work as outlined in a memorandum dated October 27, 2016, was added and subsequently authorized by Task Order No. 7.01 dated January 26, 2017.

1.2 Purpose and Scope

The purpose of the investigation was to develop geotechnical design and construction recommendations for various project components that include foundations for the new bridges, subgrade preparation for the new culverts, an MSE retaining wall, embankment construction, and new pavements. The scope of the geotechnical work included exploratory drilling, laboratory testing, engineering analysis, and preparation of this report.

1.3 Literature Search and Site Observations

We reviewed geologic maps, water well logs and the existing site conditions prior to the subsurface investigation. In addition, the boring log from the adjacent Salem Avenue bridge, located immediately south of the project, was also reviewed. The information was used to estimate the subsurface conditions and proposed drilling depths, and to provide a general overview of the site geology.

2.0 LOCAL GEOLOGY AND FAULTING

2.1 Local Geology

The project site is in a low-lying area between Neskowin to the west and Highway 101 to the east. Geologic maps show the low-lying project area as alluvium with dune sand underlying the City of Neskowin. Basalt of Cascade Head comprises the hills to the south and east (Snavely et al., 1990). The mapping is generally consistent with the site observations and subsurface conditions encountered in our explorations.

Stabilized dunes form small parallel ridges in the developed area of Neskowin. These coastal sand dunes have partially blocked Hawk Creek and Butte Creek resulting in swampy, shallow standing water immediately east of the proposed alignment. Alluvial sediments in this environment are expected to consist of very soft clay and silt with organics, peat, and interbedded sand lenses.

A 1987 geotechnical boring completed by Foundation Engineering for the Salem Avenue Bridge found soft silt, loose sand and very soft organic silt to 40 feet, followed by decomposed rock to extending to 54 feet. The boring log is included in Appendix B for reference.

2.2 Local Faults

A review of mapped faults was completed to evaluate the seismic setting and seismic sources. The review indicates numerous, primarily northeast-trending faults are present in this region landward of the Pacific Ocean. However, none of these faults show evidence of movement within the last ±1.6 million years (Geomatrix Consultants, 1995; Personius et al., 2003; USGS, 2006). The closest, potentially active crustal faults are the Cascadia fold and thrust faults and unnamed offshore faults with a mapped surface expression within ±3 to 5 miles to the west of the project site (Geomatrix Consultants, 1995; USGS, 2006). The earthquake hazard from the crustal faults account for less than 1% of the seismic hazard.

The project site is ±60 miles east of the surface expression of the Cascadia Subduction Zone (CSZ). The CSZ is a converging, oblique plate boundary, where the Juan de Fuca plate is being subducted beneath the western edge of the North

American continent. The CSZ extends from central Vancouver Island in British Columbia, Canada, through Washington and Oregon to Northern California. Available information indicates the subduction zone is capable of generating earthquakes within the descending Juan de Fuca plate (intraplate), along the inclined interface between the two plates (interface), or within the overriding North American Plate (crustal). The Puget Sound in Washington has experienced three intraplate events since 1949. However, no significant subduction zone earthquakes have occurred in Oregon during historic times. Crustal earthquakes dominate Oregon's seismic history.

The USGS 2002 interactive deaggregation indicates the primary seismic source affecting the site is the CSZ (USGS, 2002). Additional fault information can be found in the literature (Personius et al., 2003; USGS, 2006).

3.0 SUBSURFACE EXPLORATION AND CONDITIONS

3.1 Exploration

Five exploratory boreholes (BH-1 to BH-5) were drilled at the site on June 1 and 2, 2015. The borings were drilled on the existing gravel and asphaltic concrete road surface using a truck-mounted, CME 75 drill rig with mud-rotary drilling techniques. The borehole locations are shown on Figure 2A (Appendix A).

Disturbed samples were obtained in the borings in conjunction with the Standard Penetration Test (SPT), typically at ±2.5-foot intervals, to a depth of ±20 feet and at 5-foot intervals thereafter. Relatively undisturbed samples were obtained at various depths within the upper 20 feet of each boring by pushing thin-walled Shelby tubes.

The borings were continually logged during drilling. The final logs (Appendix B) were prepared based on a review of the field logs, laboratory test results, and an examination of the soil samples in our office.

3.2 Subsurface Conditions

A general discussion of the subsurface conditions is presented below. A more detailed description of the soil conditions encountered in each boring is summarized on the appended logs. The logs are also shown on the appended Foundation Data Sheets (Appendix A).

The boring locations and elevations are based on survey data provided by OBEC.

BH-1 (Tributary 1). BH-1 was drilled north of the existing culvert that will be replaced with a 12-foot wide by 10-foot high, 30-foot long box culvert at Sta. 11+83.54. This crossing is called Tributary 1. The paved surface at the boring location is at ±El. 11.8. The pavement section consisted of ±3 inches of asphaltic concrete (AC) underlain by ±15 inches of crushed rock.

Very loose sand (beach sand) extends below the crushed rock to 5 feet. Flood plain alluvium consisting of alternating layers of very soft clayey silt and very loose silty sand extend below the beach sand to ±30 feet (±El. -18.2). SPT N values recorded in the silty sand were 2 and 4, and SPT N values recorded in the clayey silt ranged from 0 to 3. The flood plain alluvium changes to medium dense sandy gravel below ±30 feet and extends to ±36.5 feet (±El. -24.7), the limits of the exploration. N values of 19 and 15 were recorded in the sandy gravel.

BH-2 and BH-3 (Embankment Construction). BH-2 and BH-3 were drilled on the existing gravel road surface to evaluate the subsurface conditions for raising the existing road grade. The elevation of the crushed rock surface at BH-2 is ±El. 13.0 and at BH-3 is El. 12.5. The crushed rock extends to 2 feet in BH-2 and 3 feet in BH-3.

Flood plain alluvium consisting of alternating layers of very soft clayey silt and very loose silty sand extend below the crushed gravel to ±11.5 feet in both borings. An SPT N value of 0 was recorded in the silty sand and N values of 0 to 5 were recorded in the clayey silt.

BH-4 (Hawk Creek Crossing). BH-4 was drilled south of Hawk Creek. The boring was drilled to evaluate the subsurface conditions for a new single-span bridge that will replace a 60-inch diameter culvert. The paved surface at the boring location is at ±El. 11.2. The pavement section consisted of ±5 inches of AC underlain by ±10 inches of crushed rock.

Very loose sand (beach sand) extends below the crushed rock to 3 feet. Flood plain alluvium consisting of alternating layers of very soft clayey silt and very loose silty sand extend below the beach sand to ±37.5 feet (±El. -26.3). SPT N values recorded in the silty sand ranged from 2 to 5 and the N values recorded in the clayey silt ranged from 0 to 2. The flood plain alluvium changes to medium dense gravel with some silt and sand below ±37.5 feet and extends to ±50 feet (±El. -38.8). Three N values of 22 were recorded in the gravel. Medium dense sand (flood plain alluvium) extends below the gravel to 66 feet and is followed by very stiff clayey silt (residual soil) to 71.5 feet (±El. -60.3), the limits of the exploration. SPT N values recorded in the sand were 21 and 29, and an N value of 30 was recorded in the residual soil.

BH-5 (Butte Creek Crossing). BH-5 was drilled south of Butte Creek to evaluate the subsurface conditions for a new single-span bridge that will replace two, 60-inch diameter culverts. The paved surface at the boring location is at ±El. 13.0. The pavement section consisted of ±2 inches of AC underlain by ±28 inches of crushed rock.

Very loose to loose silty sand with some gravel (fill) extends below the crushed rock to 6.5 feet. Flood plain alluvium consisting of alternating layers of very loose to loose silty sand and very soft to soft clayey silt with scattered organics extend below the fill to ±34.0 feet (±El. -21.0). SPT N values recorded in the silty sand ranged from 1 to 7 and the N values recorded in the clayey silt ranged from 0 to 2. The flood plain alluvium changes to dense sand below ±34.0 feet and extends to ±36.5 feet (±El. -23.5), the limits of the exploration. An N value of 34 was recorded in the sand.

3.3 Ground Water Conditions

Mud-rotary drilling precluded a measurement of the ground water level in the borings at the time of drilling. However, the ground water level along the proposed Hawk Street Extension will be at or near the ground surface throughout much of the year and will correspond to the water elevation in the adjacent creeks and areas of standing water.

4.0 FIELD AND LABORATORY TESTING

4.1 Laboratory Testing

The laboratory testing included natural water contents, Atterberg limits and percent fines tests on the flood plain alluvium to help classify the materials and estimate their engineering properties. The test results are summarized in Table 1C (Appendix C).

Two, one-dimensional consolidation tests were run on soil specimens obtained from Shelby tube samples SH-2-2 and SH-4-3 to evaluate the compressibility of the alluvial soils. The results for SH-2-2 indicate a compression index (C_{ce}) of 0.11 and a recompression index (C_{re}) of 0.016. The results for SH-4-3 indicate a C_{ce} of 0.20 and a C_{re} of 0.015. The estimated preconsolidation pressures are coincident with normally consolidated to slightly overconsolidated soils. The consolidation test results are shown on Figures 1C and 2C (Appendix C).

4.2 pH and Resistivity Testing

pH testing was performed on samples from BH-1 and BH-4. The test results, summarized in Table 2C (Appendix C), indicate a soil pH range of 6.1 to 6.5.

We performed in-situ resistivity testing (ASTM G 57) near BH-1 and BH-4. The resistivity tests were completed using a Nilsson 400, 4-pin, soil resistance meter. The 4-pin resistance meter provides an estimate of the average resistivity of a soil profile extending to a depth equal to the spacing between the pins. The tests were performed with the pins spaced at ±4, 6 and 8 feet. The recorded resistivities are summarized in Table 3C (Appendix C).

5.0 HYDRAULICS AND SCOUR

WEST Consultants, Inc. (WEST) completed a hydraulics and scour assessment study and summarized their findings in a report dated December 13, 2016, and in a revised reported dated March 23, 2017 (WEST 2017). The following is a brief summary of the pertinent information related to the foundation design. Refer to WEST (2017) for additional details.

No structure overtopping is predicted at the two crossing locations for the 25-year flood event. However, overtopping of 0.1 to 0.3 feet is predicted during the 100-year flood event, and overtopping of 0.5 to 0.8 feet is predicted during the 500-year flood event.

No contraction scour is predicted, since the proposed, single-span bridges will provide a clear span over each creek. The channel thalweg is not expected to shift over time, but it is recommended the bridge foundations be placed a minimum of 6 feet below the channel thalweg. This recommendation is covered by the proposed foundation type.

ODOT Class 50 riprap is recommended for abutment protection at Hawk Creek and Butte Creek. Class 50 riprap is also recommended for protection of the western (downstream) face of the Hawk Street embankment to reduce the potential for erosion during a roadway overtopping flood event. The riprap should be placed along the entire face of the embankment for all locations where the roadway crest elevation is less than 15.25 feet.

6.0 SEISMIC DESIGN

6.1 Bedrock Acceleration and Site Response

The recommended seismic design maps included in the AASHTO LRFD Bridge Design Specifications (2014) are based on USGS National Seismic Hazard Maps (2002). The maps provide peak ground acceleration (PGA), short-period spectral acceleration (S_s), and long-period spectral acceleration (S_1) values. The spectral acceleration coefficients on rock are summarized in Figure 3A (Appendix A) for the 1,000-year return event.

Following the AASHTO General Procedure, the bedrock values were scaled to the ground surface using F_{pga}, F_a, and F_v values appropriate for the Site Class. The Site Class accounts for the average soil and/or rock conditions within 100 feet of the ground surface. A Site Class E is appropriate for this site due to the loose sand and soft silt. The selected F_{pga}, F_a, and F_v values and General Procedure Response Spectra are also shown on Figure 3A (Appendix A).

6.2 Liquefaction and Lateral Spread

Liquefiable soils typically consist of saturated, loose sand and non-plastic silt. Drilling along the road alignment encountered alternating layers of very loose to loose sand and silty sand, and very soft to soft, medium to high plasticity clayey silt. These very loose to loose and very soft to soft flood plain deposits extend to depths of ±30 to 34 feet. Based on the low N-values in the sand and silty sand, the material is prone to liquefaction during the design Cascadia Subduction Zone (CSZ) event. In addition, the very soft to soft clayey silt will be susceptible to seismic strain-softening.

The roadway alignment will experience liquefaction-induced settlement following the design CSZ earthquake. Slope instability and lateral spread of the road embankment (including the MSE wall) may result from liquefaction and strain softening of the site soils. Considering the widespread presence of potentially liquefiable soils in the vicinity of the project and the limited use of the road, we understand liquefaction and lateral spread mitigation is not considered cost-effective and will not be completed as part of this project. Therefore, analysis and design for the mitigation of liquefaction and lateral spread was not included in the scope for this project.

7.0 FOUNDATION ANALYSIS AND DESIGN RECOMMENDATIONS – BUTTE CREEK AND HAWK CREEK CROSSINGS

7.1 Discussion of Foundation Options

The proposed Hawk Creek and Butte Creek crossing structures will be 32-foot long, single-span bridges with an out-to-out width of 16 feet and 20 feet, respectively. Foundation options considered for these crossings included spread footings and deep foundations (i.e., driven piles or drilled shafts). The soils encountered in the upper ±30 feet are too compressible to support the abutments on conventional spread footings without excessive settlement, and some of the soils are also prone to liquefaction. Therefore, the subsurface conditions require a deep foundation (i.e., driven piles or drilled shafts) to support the new structures. Based on discussions with the design team and our experience with the Salem Avenue bridge to the south, PP12.75x0.5, closed-ended piles were selected as the preferred foundation option.

7.2 Downdrag

AASHTO (2014) and the ODOT GDM (2015) require consideration for downdrag where post construction settlement results in at least ± ½ inch of soil settlement around the pile following installation.

The entire road alignment and bridge approaches will be raised ±4.5 feet above the existing grade. Consequently, our analysis indicates that post-construction settlement will be sufficient to mobilize downdrag on the abutments piles. In addition to the static settlement, should an earthquake occur during the life span of the structures, seismically-induced liquefaction settlement would also contribute to the total settlement required to mobilize downdrag.

Settlement calculations indicated post construction settlement at the pile locations would extend to ±El. -12 at each crossing. Therefore, the side resistance of the piles above these depths were used to calculate the downdrag load. The estimated, unfactored downdrag load is 22 kips per pile at each crossing.

Liquefaction induced settlement may also induce downdrag loading. However, the liquefied soil is unlikely to adhere to the piles, resulting in large downdrag loads. Furthermore, the liquefaction is considered an extreme event condition and the potential liquefaction induced downdrag will not exceed the nominal pile resistance.

7.3 Foundation Loads

The Service (unfactored) and Strength (factored) loads provided by OBEC are summarized in Table 1. The loads are for individual piles using 5 piles per abutment for the Butte Creek crossing and 4 piles per abutment for the Hawk Street crossing.

Table 1. Design Maximum Foundation Loads Per Pile

Crossing	Service Load (kips)	Strength I Load (kips)
Butte Creek	65	100
Hawk Creek	60	90

7.4 Driven Pile Analysis and Design

The pile analysis was completed using the AASHTO (2014) Load Resistance Factor Design (LRFD) approach with Interim Revisions (2015 and 2016), as appropriate. The profiles of BH-4 and BH-5 were used to establish the soil profile for the Butte Creek Crossing. The new crossing at Butte Creek was originally planned as a culvert. Therefore, BH-5 only extended to 36.5 feet (El. -23.5). BH-4 was drilled to 71.5 feet and was used to represent the deeper soil profile for the Butte Creek Crossing foundation design. The profile of BH-4 was used to establish the soil profile for the Hawk Creek Crossing.

7.4.1 *Pile Type and Material Specifications.* Recommendations presented herein assume PP12.75x0.5 (ASTM A252, Grade 3; F_y of 45 ksi) steel pipe piles will be used to support the abutments. The pipes should be driven closed-ended to medium dense to dense sand and gravel to develop the required axial resistance. The recommended pile properties are summarized in Table 2.

Table 2. Recommended Pile Section and Pile Properties

Pile Section	PP12.75x0.5
Steel Grade	ASTM A252, Grade 3
Yield Stress (F_y)	45 ksi
Area Steel (A_s)	19.2 in^2
Nominal Structural Resistance (P_n)	864 kips
End Condition	Closed-ended

7.4.2 *Nominal and Factored Axial Resistances.* The estimated nominal and factored axial resistances versus elevation for the PP12.75x0.5 section for the profiles at Hawk Creek and Butte Creek are plotted on Figure 4A and Figure 5A (Appendix A).

The axial resistance at each location was estimated using the AASHTO Load Factor Resistance Design (LRFD) method. The analysis used an LRFD resistance (γ_p) factor of 0.4, as recommended by AASHTO (2014) for use when pile resistance is confirmed using the FHWA Gates equation.

The required nominal driving resistance (R_{ndr}) was established based on the Strength I axial load per pile (Table 1) and the estimated downdrag load of 22 kips/pile. AASHTO (2014) Section 10.7.3.7 recommends calculating R_{ndr} using the following equation:

$$R_{ndr} = (\sum \gamma_i Q_i)/\phi_{dyn} + \gamma_p DD/\phi_{dyn} + R_{sdd}$$

Where: $\sum \gamma_i Q_i$ is the factored load per pile excluding downdrag; ϕ_{dyn} is the resistance factor for driving (0.4); γ_p is the load factor for downdrag (1.4); and R_{sdd} is the skin friction that must be overcome during driving through the downdrag zone, which is equal to DD.

Use of this equation resulted in required driving resistances that were determined to be excessive, since the equation included 3.5 times the estimated downdrag load in addition to the driving resistance required to support the bridge foundation loads. We brought this to the attention of ODOT's geotechnical engineers and in consultation with ODOT, the load and resistance factors for the downdrag component were taken as 1.0. Therefore, the equation is reduced to:

$$R_{ndr} = (\sum \gamma_i Q_i)/\phi_{dyn} + DD + R_{sdd}$$

The required nominal driving resistances for each bridge are summarized in Table 3.

Table 3. Required Nominal Driving Resistance per Pile

Crossing	Nominal Driving Resistance/Pile (kips)
Butte Creek	294
Hawk Creek	269

7.4.3 *<u>Minimum and Estimated Pile Tip Elevations and Pile Lengths</u>*. The estimated pile tip elevations were based on the calculated factored resistances and the factored loads provided by OBEC plus consideration for the downdrag load. We calculated the closed-ended PP12.75x0.5 piles will develop the required axial resistance in the medium dense gravel. The minimum tip elevation was chosen based on at least 5 feet of penetration into the gravel. The estimated pile tip elevation corresponds to the elevation where the calculated resistance equals the nominal axial resistance in Table 4.

The bottom of cap elevations were provided by OBEC. The pile cut-off elevations for each bent are based on the bottom of cap elevation plus 1.5 feet for embedment into the pile cap. The finished pile lengths, also summarized in Table 4, are based on the cut off elevation and estimated tip elevation rounded up to the nearest 1-foot interval.

Table 4. Minimum/Estimated Tip Elevations and Pile Lengths

Crossing	Nominal Axial Resistance (kips)	Est. Cut-Off Elevation (ft)	Min. Tip Elevation (ft)	Estimated Tip Elevation (ft)	Recommended Pile Length (ft)
Butte Creek	294	11.5	-27.0	-36.0	48
Hawk Creek	269	13.5	-29.0	-36.0	50

Note: The Nominal Axial Resistance includes the downdrag load.

7.4.4. *Nominal and Factored Uplift Resistances*. The nominal uplift resistances were calculated based on the estimated skin resistance mobilized in the materials above the minimum tip elevations. The nominal uplift resistances include the estimated downdrag load. For Butte Creek, the nominal uplift resistance was calculated to be 60 kips. For Hawk Creek, the nominal uplift resistance was calculated to be 44 kips. AASHTO (2014) recommends a resistance factor of 0.8 for extreme event uplift and 0.2 for long-term uplift of piles.

7.4.5. *Pile Settlement*. The pile tips will be driven to medium dense gravel with low compressibility. Therefore, pile settlement is expected to be limited to the elastic compression of the section caused by the working load and is expected to be less than ± ¼ inch.

7.4.6. *Tip Condition*. A steel plate welded to the tip of the pile is recommended for the closed-ended pile.

7.5 Pile Driving

Pile driving is covered in the Construction Recommendations, Section 11.2.

7.6 Abutments and Wing Walls

The proposed abutments and wing walls will have a maximum height of ±4.75 feet. We assume that Granular Wall Backfill (Section 00510.12) will be used in the zone behind the walls. A friction angle of 34 degrees and a unit weight of 125 pcf were used for the wall backfill. Drained conditions were also assumed.

A lateral deflection of at least ±0.001*H (where H is the height of the wall) is required for the walls to mobilize an active earth pressure condition within the granular wall backfill. For a 5-foot tall wall, the deflection is 0.06 inches. Typically, abutment walls deflect to mobilize active earth conditions. However, integral abutment walls or wing wall-to-abutment wall corners may not be free to deflect. Therefore, we calculated earth pressures for both active and at-rest conditions.

For restrained abutment walls, an at-rest earth pressure coefficient (k_o) of 0.44 was calculated. The nominal lateral earth pressure on restrained walls may be estimated using an equivalent fluid density of 55 pcf. For walls free to rotate, an active earth pressure coefficient (k_a) of 0.28 was calculated. The nominal lateral earth pressure on unrestrained walls may be estimated using an equivalent fluid density of 35 pcf.

AASHTO (2014) recommends calculating the traffic loads applied to the top of the abutment walls using an equivalent soil surcharge. For an abutment height of less than 5 feet, a minimum surcharge height of 4.0 feet is recommended. A unit weight of 125 pcf and a surcharge height of 4.0 feet results in a surcharge of 500 psf. An at-rest pressure coefficient of 0.44 results in an additional uniform lateral pressure of 220 psf. The active pressure coefficient of 0.28 results in an additional nominal uniform lateral pressure of 140 psf.

For the wing walls or walls parallel to traffic, an equivalent soil surcharge of 2 feet is recommended. A unit weight of 125 pcf and a surcharge height of 2 feet results in a nominal uniform surcharge pressure of 250 psf. An at-rest pressure coefficient of 0.44 results in an additional nominal uniform lateral pressure of 110 psf on the wing walls. An active pressure coefficient of 0.28 results in an additional nominal uniform lateral pressure of 70 psf on the wing walls.

The GDM (ODOT, 2015) requires walls that affect the performance or structural integrity of the bridge be designed for a peak horizontal acceleration corresponding to a 1,000-year return period. For design, we used a horizontal acceleration (k_h), equal to 0.5 times the ground surface acceleration (A_s) of 0.36g. The A_s is based on the PGA (on rock) of 0.39g and an AASHTO site factor (F_{pga}) of 0.93 for an AASHTO Site Class E soil profile.

The Mononobe-Okabe analysis was used to calculate a seismic active earth pressure coefficient (k_{ae}). For the analyses, the peak horizontal ground acceleration (k_h) and corresponding seismic lateral earth pressure coefficient (k_{ae}) depend upon the allowable lateral deflection of the wall during an earthquake. The allowable seismic wall displacement was assumed to be up to ±1 to 2 inches.

For the 4.75-foot high abutment wall, a resulting horizontal seismic force of 160 lb/ft. was calculated for the 1,000-year seismic event. Therefore, the seismic force on the wall may be modeled using an additional uniform pressure of 34 psf.

A summary of the calculated abutment and wing wall lateral earth pressures is provided in Table 5.

Table 5. Lateral Earth Parameters for Abutment and Wing Wall Design

Parameter	Source	Value	γ_p
At Rest Earth Pressure Coefficient, k_o	$1-\sin(\phi)$	0.44	
Active Earth Pressure Coefficient, k_a	$\tan^2(45 - \phi/2)$	0.28	
At-Rest Equivalent Fluid Density	$k_o*\gamma_{backfill}$	55 pcf	1.35
Active Equivalent Fluid Density	$k_a*\gamma_{backfill}$	35 pcf	1.50
Traffic Load Surcharge for Abutment Walls (At Rest)	500 psf*k_o	220 psf	1.35/1.75
Traffic Load Surcharge for Abutment Walls (Active)	500 psf*k_a	140 psf	1.35/1.75
Traffic Load Surcharge for Wing Walls (At Rest)	250 psf*k_o	110 psf	1.35/1.75
Traffic Load Surcharge for Wing Walls (Active)	250 psf*k_a	70 psf	1.35/1.75
Seismic Pressure for 4.75-foot high wall (1,000-year) (assumes up to 2 inches of displacement)	Mononobe-Okabe	34 psf	1.00

The appropriate load factors (γ_p) provided in AASHTO (2014) Table 3.4.1-2 should be applied to the preceding nominal pressures to estimate the factored lateral earth loads. Selection of the appropriate load factors are dependent on the load case being analyzed. AASHTO (2014) recommends a load factor of 1.35 for at-rest earth loads and 1.5 for active earth loads. For the traffic load surcharge, a load factor of 1.75 is recommended for Strength I and 1.35 for Strength II and V.

8.0 APPROACHES AND EMBANKMENTS

8.1 Discussion

New embankment up to ±6 feet thick is planned along the Hawk Street Extension. Since the alignment is underlain by a relatively thick sequence of soft/loose alluvial soil, total and differential settlement will occur during and following embankment construction. The settlement will be greatest near the bridge approach at Hawk Creek where the fill thickness is ±6 feet, and less at box culvert at Tributary 1 and towards the end of the project limits where the fill tapers to match the existing grades.

8.2 Embankment Settlement

Settlement along two sections of the alignment was estimated using the computer program Settle^{3D}. The embankment height from Sta. “H” 11+00 to Sta. “H” 13+40 and from Sta. “H” 17+20 to Sta. “H” 20+85 was estimated based on the Advanced Plan set provided by OBEC. These sections were selected to estimate minimum and maximum settlement values that should be anticipated along the entire alignment. The settlement model is based on the laboratory testing data discussed in Section 4.

Settlement was calculated at stages ranging from 15 to 1,800 days. However, there is negligible primary consolidation following 360 days. A summary of the settlement values at various locations along the alignment is provide in Table 6.

Table 6. Hawk Creek Extension Settlement

Alignment Section	"H" Station	Fill Thickness	Stage/Est. Settlement (in)
11+00 to 13+40	11+80 CL	@Box Culvert	30 days/0.5 60 days/1.0 120 days/1.3 360 days/1.5
	12+40 CL	3.5 ft	30 days/3.0 60 days/4.0 120 days/4.5 360 days/5.0
	13+00 RT	MSE Wall	30 days/1.5 60 days/2.0 120 days/2.5 360 days/3.0
17+20 to 20+85	17+80	4.0 ft	30 days/3.5 60 days/4.5 120 days/5.5 360 days/6.0
	18+80	@Hawk Creek Abutment 6.0 ft	30 days/5.0 60 days/6.5 120 days/7.5 360 days/8.0

Notes:
1. Settlement values reported to 0.1 for illustrative purposes only, as this level of precision is impractical.
2. CL is centerline of alignment.
3. RT is right side of roadway.

To manage post-construction settlement, we recommend constructing the new roadway fill first and then letting it sit for 360 days to allow for consolidation of the underlying alluvium. Completing the finish grading and paving prior to 360 days will result in settlement along the alignment that may distress the AC surface. A settlement profile for each stage along the two sections of alignment are shown in Figure 6A and Figure 7A (Appendix A).

9.3 Embankment Slope Stability

Evaluation of the long-term, static slope stability was not completed for the relatively low embankment. Although the subgrade soils are soft, they will consolidate and stiffen/densify over time, thus decreasing the risk of future instability.

The risk of ground failure and embankment instability during a seismic event is high due to liquefaction that is anticipated along the alignment. However, liquefaction mitigation is beyond the project scope.

9.4 MSE Retaining Wall Design

An 90-foot long MSE wall is planned to retain the embankment fill along the east side of Hawk Street. The MSE wall will extend from Sta. "H" 12+30 to Sta. "H" 13+20 Rt. The top-of-wall (TOW) elevation of El. 13.60 and a bottom-of-wall (BOW) elevation of El. 8.60, for a wall height of 5.0 feet. The east edge of the road will be built over the MSE wall. A minimum 18-inch thick cover is indicated in the plans. The cover will extend at a 2(H):1(V) back slope from the top of wall to the back edge of the guardrail.

A minimum reinforcement length of 8 feet is planned, which is equal to 1.6H (where H is the height of the wall). The wall will have a minimum embedment depth of 2 feet. The base of the wall will be underlain by a minimum of 2 feet of Stone Embankment Material to help mitigate soft soil conditions to provide a stable base for the wall.

The MSE wall will be designed using a proprietary system with internal stability analysis and design provided by the manufacturer. Therefore, our work is limited to providing parameters for the MSE wall design, and external stability checks including; bearing capacity, sliding resistance and overturning resistance, and global stability of the retained fill and slope.

The soil profile of BH-1 was used to complete the external stability checks for the MSE wall.

9.4.1. <u>LRFD Design Parameters</u>. External stability analyses were completed using the AASHTO (2014) LRFD approach. Table 7 summarizes the load factors based on AASHTO (2014) Table 3.4.1-1 and 3.4.1-2.

Table 7. Load Factors for External Stability

Condition	Strength I-a (Sliding and Eccentricity)	Strength I-b (Bearing Resistance)	Extreme Event I (Sliding and Eccentricity)	Extreme Event I (Bearing Resistance)
Horizontal Active Earth Pressure, EH	1.5	1.5	1.5	1.5
Vertical Earth Pressure, EV	1.0	1.35	1.0	1.35
Live Load Surcharge, LL	1.75	1.75	γ_{EQ}	γ_{EQ}
Dead Load (Embankment) Surcharge, ES	1.5	1.5	1.5	1.5
Earthquake Load, EQ	--	--	1.0	1.0

Note: γ_{EQ} is project dependent and is typically less than 1.0.

The MSE external design resistance factors (φ) from AASHTO (2014) Table 11.5.6-1 are summarized in Table 8.

Table 8. Resistance Factors for External Stability

Condition	Strength	Extreme Event
Sliding Resistance	1.0	1.0
Bearing Resistance	0.65	1.0

9.4.2. *Lateral Earth Pressures and Seismic Loading*. Lateral earth pressures for the MSE wall design were calculated based on the design practices recommended in the ODOT GDM (2015), FHWA (2009) and AASHTO (2014). Calculations include the effects of traffic surcharge parallel to the walls, dead load active earth pressures from roadway base fill, and seismic considerations, including inertial seismic forces.

Static Loading. We anticipate the MSE wall will deflect sufficiently to mobilize active conditions. Therefore, active earth pressures were assumed. The wall geometry was used along with the assumed internal friction angle (ϕ=34°) of the retained soil to calculate and active Earth Pressure Coefficient (k_a) of 0.28 based on Coulomb analysis. Applying a unit weight of 125 pcf for the retained soil, the active earth pressure may be calculated using an equivalent fluid density of 35 pcf.

Seismic Loading. The ODOT GDM (2016) requires walls be designed for a peak horizontal acceleration corresponding to a 1,000-year return period. The USGS 2002 map indicates a peak bedrock acceleration of 0.39g, for the 1,000-year design earthquake. An AASHTO F_{pga} value of 0.93 for Site Class E was used to calculate a peak seismic ground acceleration coefficient (A_S) of 0.36g at the surface.

The total seismic earth pressure coefficient (k_{ae}) was calculated using the Mononobe-Okabe (M-O) analysis method. For the M-O analysis, the vertical acceleration coefficient (k_v) was assumed to be zero.

For external stability, a reduced horizontal acceleration coefficient (k_{h_d}) was calculated to account for ±3 inches of potential wall displacement (d). The maximum horizontal coefficient (k_h) was then calculated for the MSE walls accounting for inertial wall forces.

For internal stability, the seismic forces should be calculated using the maximum acceleration developed within the wall ($A_{m(int)}$) without reduction for displacement. The recommended parameters for static and seismic design are summarized in Table 9.

Table 9. Lateral Earth and Seismic Parameters for MSE Wall Design

Parameter	Equation	Value
Active Earth Pressure Coefficient, k_a	$\tan^2(45 - \Phi/2)$	0.28
Active Earth Equivalent Fluid Density	$k_a * \gamma_{backfill}$	35 pcf
Ground Acceleration, A_S	PGA* F_{pga}	0.36g
Max. Acceleration (Internal Stability), $k_h = A_{m(int)}$	$(1.45 - A_S)A_S$	0.39g
Max. Acceleration, Reduced for Displacement*, k_{h_d}	$0.74A_S(A_S/d)^{0.25}$	0.16g
Max. Horizontal Acceleration (External Stability), k_h	$(1.45 - k_{h\_d})k_{h\_d}$	0.20g
Seismic Earth Pressure Coefficient, k_{ae}	M-O	0.41
Seismic Thrust Coefficient* Δk_{ae}	$k_{ae} - k_a$	0.13

Traffic and Embankment Surcharge. A vertical traffic surcharge pressure of 250 psf was estimated for the wall using a soil surcharge height of 2 feet based on AASHTO (2014) Table 3.11.6.4-1. A factored, uniform surcharge pressure of 438 psf was calculated using a load factor (γ_{LL}) of 1.75. This corresponds to a factored, uniform lateral earth pressure of 123 psf calculated using a k_a value of 0.28.

A vertical embankment surcharge pressure will act on top of the wall from the weight of the roadway base and pavement. We assumed a nominal surcharge pressure of 200 psf based on the anticipated fill thickness. This will be conservative near the front of the wall since the embankment slopes down to meet the top of the wall. A factored, uniform surcharge pressure of ±300 psf was calculated using a load factor (γ_{ES}) of 1.5. A k_a of 0.28 applied to the factored surcharge pressure corresponds to a uniform, lateral earth pressure of 84 psf.

9.4.3. *Soil Parameters*. MSE Granular Backfill will be used in the reinforced zone. Recommended strength parameters for the reinforced zone and the retained backfill soils are provided in Table 10.

Table 10. Recommended Soil Parameters for MSE Wall Design

Material	Moist Unit Weight γ_m (pcf)	Friction Angle ϕ (degrees)	Cohesion C (psf)
Reinforced Soil - MSE Granular Backfill	130	34	0
Retained Soil – Compacted Embankment Fill	125	34	0

To remove soft, compressible soil directly beneath the MSE wall, a minimum of 2 feet will be overexcavated and replaced with Stone Embankment Material. The Stone Embankment Material will be underlain by loose silty sand. The foundation soils beneath the wall should be confirmed at the time of construction and additional recommendations should be provided, if necessary, for any additional overexcavation.

The wall is relatively short and will have limited reinforced length. As a result, the influence depth (below the bottom of the wall) will also be limited and primarily influenced by the properties of the Stone Embankment Material. Therefore, for bearing and sliding resistance, we assumed a composite profile based on properties for Stone Embankment and the underlying sand. The assumed composite material properties are summarized in Table 11. For slope stability analysis, we assumed the soil properties for the individual soil layers, as summarized in Table 12.

Table 11. Recommended Soil Parameters for Bearing and Sliding Analysis

Material	**Moist Unit Weight γ_m (pcf)**	**Friction Angle ϕ (degrees)**	**Cohesion (psf)**
Stone Embankment over Loose Silty Sand	115	34	0

Note: Ground water is assumed to be at the base of the wall.

Table 12. Recommended Soil Parameters for Slope Stability Analysis

Material	**Moist Unit Weight γ_m (pcf)**	**Friction Angle ϕ (degrees)**	**Cohesion (psf)**
Stone Embankment (directly beneath the MSE wall to 2 ft)	115	34	0
Soft Clayey Silt (extending to a depth of 2 ft, except below the MSE wall)	90	0	200
Loose Silty Sand (extending below the Clayey Silt or Stone Embankment)	105	32	0

Note: Ground water is assumed to be at the base of the wall.

9.4.4. *<u>Nominal and Factored Bearing Resistance</u>*. The nominal bearing resistance (q_n) for the foundation soils was calculated using the strength parameters presented in Table 11 for the composite Stone Embankment Material and lose silty sand. The nominal bearing resistance was calculated using the bearing capacity equation and tables in FHWA NHI-10-024:

$$q_n = cN_c + 0.5(L')\gamma N_\gamma$$

Where q_n is in units of lb/ft^2, c is the foundation soil cohesion, N_c and N_γ are unitless bearing capacity coefficients, L' is the effective foundation width accounting for eccentricity (L' = L-2e), and γ is the effective unit weight of the foundation soil. The eccentricity varies depending on wall height and loading conditions.

For bearing resistance calculations, the effective (i.e., buoyant) unit weight of the foundation soil (γ) was used. Using the recommended soil parameters, the nominal bearing resistance can be calculated as:

$$q_n = 0 + 1{,}088(L') \ \text{(psf)}$$

The factored bearing resistance is the nominal bearing resistance multiplied by a resistance factor (φ) of 0.65.

9.4.5. Sliding Resistance. MSE wall sliding resistance is a function of the weight of the reinforced fill and the friction developed between the materials at the base of the wall. The frictional resistance is estimated using the lessor of the sliding resistance developed within the foundation soil ($c_f + \tan\phi_f$) or within the reinforced fill ($\tan\phi_r$). For our analysis, ϕ_f and ϕ_r equal 34 degrees, so sliding resistance may be calculated based on either material.

Depending on the type of reinforcement, sliding resistance may also depend on the soil-reinforcement interface. It is assumed the sliding resistance at the soil-reinforcement interface will be checked by the wall designer for the final wall configurations.

9.4.6. External Stability. External stability calculations (bearing resistance, eccentricity/overturning resistance and sliding resistance) were completed based on the design equations in FHWA NHI-10-024 using the soil parameters recommended herein. A wall height of 5.0 feet was used for our calculations. The calculations indicate a minimum reinforced length of 8 feet is sufficient for design.

Table 13 summarizes the results of the analyses. The results indicate acceptable Capacity to Demand Ratio (CDR) greater than 1.0 for bearing resistance and sliding and an e/L value less than 0.25 for overturning evaluation.

Table 13. MSE Wall External Stability Calculations (Static)

Wall Height, H (feet)	Assumed Reinforced Length, L (feet)	Factored Bearing Resistance (lb/ft^2)	Bearing CDR	Eccentricity Ratio (e/L)	Sliding CDR
5.0	8.0	5,236	3.08	0.09	2.09

Note: H is the total height of the wall.

The external stability calculations were also performed for seismic conditions using the seismic acceleration parameters discussed above and the LRFD extreme event load and resistance factors. Results of the seismic analyses are summarized in Table 14. For each case, the results indicated acceptable CDR values greater than 1.0 for bearing resistance and sliding. An e/L value of ±0.30 was calculated for overturning evaluation.

Table 14. MSE Wall External Stability Calculations (Seismic)

Wall Height, H (feet)	Assumed Reinforced Length, L (feet)	Factored Bearing Resistance (lb/ft^2)	Bearing CDR	Eccentricity Ratio (e/L)	Sliding CDR
5.0	8.0	7,881	4.54	0.11	1.67

Note: H is the total height of the wall.

9.5.6 <u>*Wall Settlement*</u>. Settlement along the new alignment is discussed in Section 8.2. Settlement at the MSE wall location is estimated to be ±3 inches. Since a flexible wall system with no rigid facing is proposed, this amount of settlement is considered tolerable.

9.5.7 <u>*Global Stability*</u>. Global stability analyses were completed for the MSE walls using the computer program Slide 5.0. The geometry was established using the wall plans and a minimum wall reinforcement length of 8 feet. Potential failure planes were assumed to extend behind and below (but not through) the reinforced zone.

The profile was based on the subsurface conditions and wall geometry indicated herein. The MSE wall backfill and embankment fill were modeled using the ϕ values and moist unit weights indicated in Table 10. The underlying soils were modeled using the parameters from Table 12.

A horizontal ground acceleration (k_h) of 0.20g was used for the seismic global stability analysis, consistent with the k_h value in Table 9. The results of the analyses are summarized in Table 15.

A minimum factor of safety of 1.5 is required for static design to coincide with a resistance factor of 0.65. A minimum factor of safety of 1.0 is required for seismic design. The results of the analyses indicate factors of safety meeting these minimum values.

The seismic factors of safety and CDR values to not account for cyclic softening or liquefaction from seismic loading.

Table 15. Global Stability Results

Wall Height, H (feet)	Assumed Reinforced Length, L (feet)	Factor of Safety (Static)	Factor of Safety (Seismic)
5.0	8.0	1.5	1.0

10.0 PAVEMENTS

A formal pavement analysis and design was not completed for the Hawk Street Extension project, since the minimum section proposed by Tillamook County is adequate for the low volume road. In addition, since the alignment grade will be raised by several feet, the subbase material will consist predominately of Stone Embankment.

We understand the proposed pavement section will consist of 8 inches of asphaltic concrete (AC) over 9 inches of aggregate base. It is our opinion this section is adequate for the low volume road. Roadway section details are provided in the plans.

11.0 CONSTRUCTION RECOMMENDATIONS

11.1 Specifications

All specifications contained herein refer to ODOT's Oregon Standard Specifications for Construction (2015). It is assumed these specifications will be referred to for general or specific items not addressed in this report.

11.2 Driven Piles

The specifications for piles and pile driving should follow the requirements of ODOT's Section 00520.

11.2.1. Driving Criteria and Driveability Analysis. The FHWA Gates Equation was used to calculate a range of hammer energies required to drive the piles to a nominal axial resistance of up to 294 kips, with a final driving resistance from 2 to 10 blows/inch (bpi). Our calculations suggest a hammer field energy range of 12,700 foot-pounds to 30,000 foot-pounds would be required. However, ODOT recommends a minimum hammer field energy of 13,000 foot-pounds for piles driven to a nominal axial resistance between 180 and 300 kips. Therefore, we recommend a hammer energy range of 13,000 to 30,000 foot-pounds. The final driving criterion should be established by the design team after the contractor has selected a pile hammer.

A monitoring program is recommended during construction to confirm that all pile driving criteria are followed. We anticipate a construction inspector will log each pile for driving resistance and hammer efficiency. Driving should be halted if the pile meets practical refusal (defined herein as a driving resistance exceeding 10 bpi for 3 consecutive inches).

11.2.2. Potential Obstructions. We did not observe potential obstructions in the subsurface exploration. However, the exploration did encounter woody debris in the alluvium and large woody debris is common in the local alluvial environment. Therefore, piles may encounter obstructions that result in misalignment of the pile or refusal above the minimum tip elevation. If the pile cannot be advanced, the pile may need to be extracted and drilling may be required to remove the obstruction. Jetting is not recommended.

11.2.3. *Set Period and Redriving*. The piles will be driven into a medium dense gravel layer. Driving beyond this layer will result in piles ±10 to 20 feet longer than estimated. Therefore, if the piles drive below the estimated tip elevation without attaining the required driving resistance, the contractor should stop driving and allow the piles to set for a period of 48 hours before performing a restrike to test the pile resistance after set. Therefore, the Special Provision .42(d) for a set period of 48 hours should be included if the piles are driven to El. -36 at both crossings without achieving the required nominal axial resistance.

11.3 Subgrade Preparation and Embankment Construction

The following construction recommendations are based on the requirements of Section 00330, and are provided for the widening of existing alignment. We have assumed the earthwork will be completed during dry weather. If project scheduling requires fill placement during winter months, additional subgrade excavation may be required to remove softened materials near the surface.

11.3.1. *Seasonal Issues*. The soils outside the existing alignment will be moisture-sensitive and will become soft, weak, and unworkable when exposed to excessive moisture. Therefore, we recommend the construction of new embankments be completed during dry weather to reduce subgrade disturbance.

11.3.2. *Subgrade Preparation*. The existing ground should be properly stripped prior to constructing the new embankment. Clearing and grubbing should be conducted in accordance with Section 00320.40. A grubbing depth of ±4 to 6 inches should be anticipated.

The subgrade beneath the new roadway embankment should not be compacted prior to backfilling. However, the subgrade should be observed by a member of the design team prior to constructing the embankment to identify areas requiring Subgrade Stabilization (Section 00331), and Unsuitable Subgrade Material (Section 00330.41 (a-8c).

11.3.3. *Embankment Fill*. Embankment construction and widening should be completed in accordance with Section 00330.42. The embankment fill should consist of clean, angular, granular fill meeting the requirements of Stone Embankment Material (Section 00330.16). Finished embankment slopes constructed with Stone Embankment Material mays be constructed at 1.5(H):1(V), or flatter.

11.3.4. *Wing Wall Backfill*. Placement and compaction of imported fill behind the cast-in-place wing walls should be completed using light, vibratory equipment within a distance equal to one-half of the wall height. Granular Wall Backfill (00510.12) should be used behind the walls.

11.4 Excavations/Shoring/Dewatering

11.4.1. *MSE Wall*. Excavations up to 2 feet deep will be required for construction of the MSE retaining wall. Excavations extending below ±1 to 2 feet are likely to encounter wet soils, or the subgrade may be below the water table. Therefore, open-graded, Stone Embankment material is recommended for the bedding material.

11.4.2. *Box Culvert at Tributary 1*. An excavation extending ±8 feet below the bottom of the existing tributary will be required. Therefore, shoring of the existing channel and dewatering will be required. It is the responsibility of the contractor to provide a plan for excavation shoring and dewatering prior to construction.

11.5 Box Culvert at Tributary 1

Provide excavation shoring and dewatering as required for excavation, grading and placement of the bedding material. The excavation should extend a minimum of 12 inches below the bottom of the culvert base to provide room for the bedding material. The bedding material should consist of compacted, Granular Structure backfill meeting the requirement of Section 00510.13.

12.0 LIMITATIONS

12.1 Construction Observation/Testing

We recommend a member of the design team be present to observe the pile installation. Embankment and retaining wall construction, and construction of the box culvert should be continuously observed to confirm the subgrade conditions, fill placement and compaction procedures. Any geotechnical engineering judgment in the field should be provided by a representative of Foundation Engineering.

12.2 Variation of Subsurface Conditions, Use of Report and Warranty

The analysis, conclusions and recommendations contained herein are based on the assumption that the subsurface profiles encountered in the borings are representative of the overall site conditions. The above recommendations assume we will have the opportunity to review final drawings and be present during construction to confirm the assumed foundation and subgrade conditions. No changes in the enclosed recommendations should be made without our approval. We will assume no responsibility or liability for any engineering judgment, inspection or testing performed by others.

This report was prepared for the exclusive use of the OBEC Consulting Engineers, Tillamook County, and their design consultants for the Hawk Street Extension project in Tillamook County, Oregon. Information contained herein should not be used for other sites or for unanticipated construction without our written consent. This report is intended for planning and design purposes. Contractors using this information to estimate construction quantities or costs do so at their own risk. Our services do not include any survey or assessment of potential surface contamination or contamination of the soil or ground water by hazardous or toxic materials. We assume those services, if needed, have been completed by others.

Our work was done in accordance with generally accepted soil and foundation engineering practices. No other warranty, expressed or implied, is made.

13.0 REFERENCES

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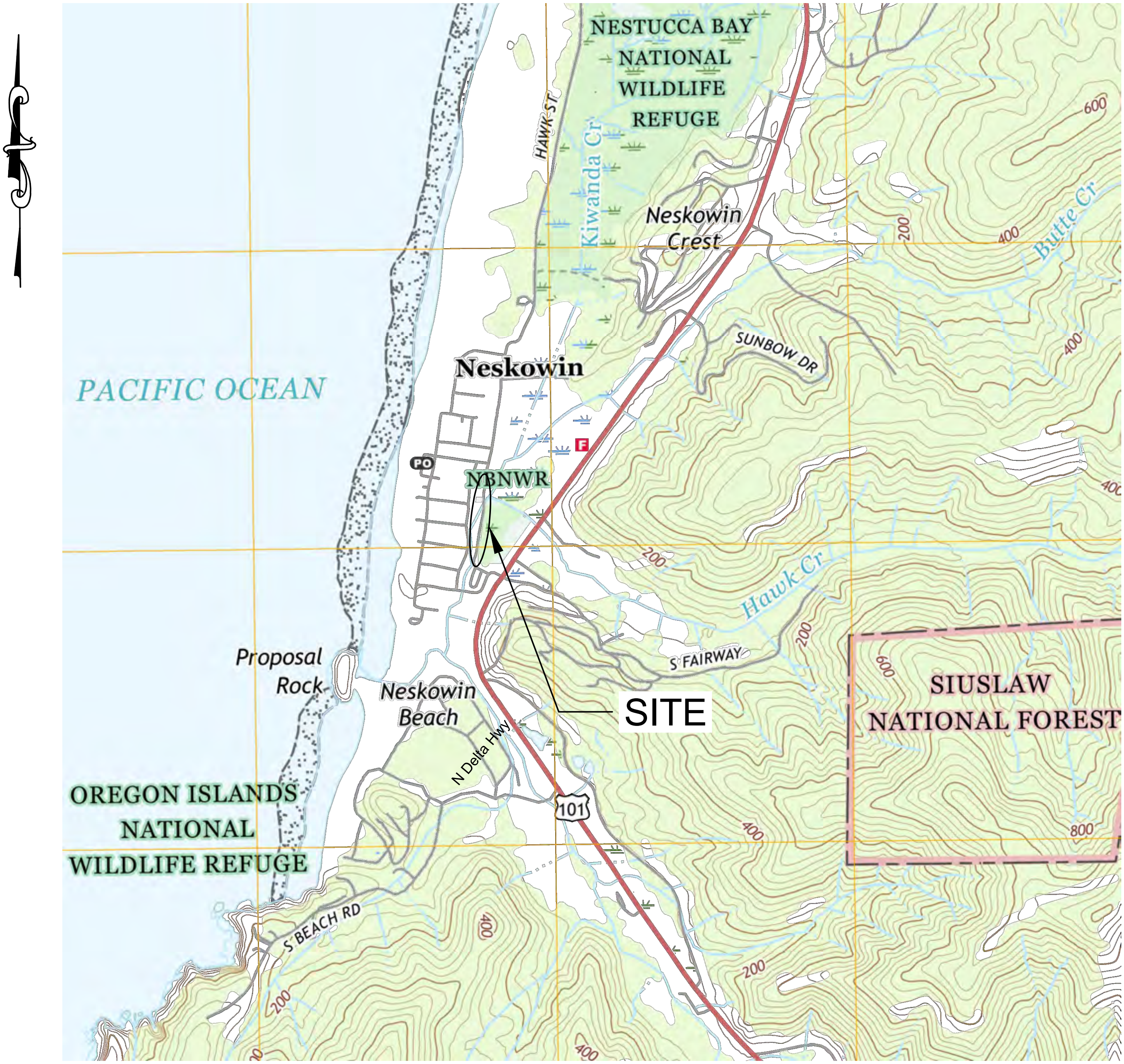
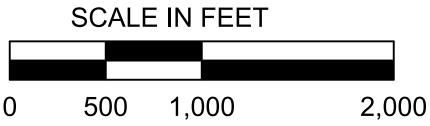
Appendix A

Figures and Foundation Data Sheets

Professional
Geotechnical
Services

Foundation Engineering, Inc.





DATE APRIL 2016
DWN. mdm
APPR.
REVIS.
PROJECT NO.
21510443

FOUNDATION ENGINEERING INC.
PROFESSIONAL GEOTECHNICAL SERVICES
820 NW CORNELL AVENUE
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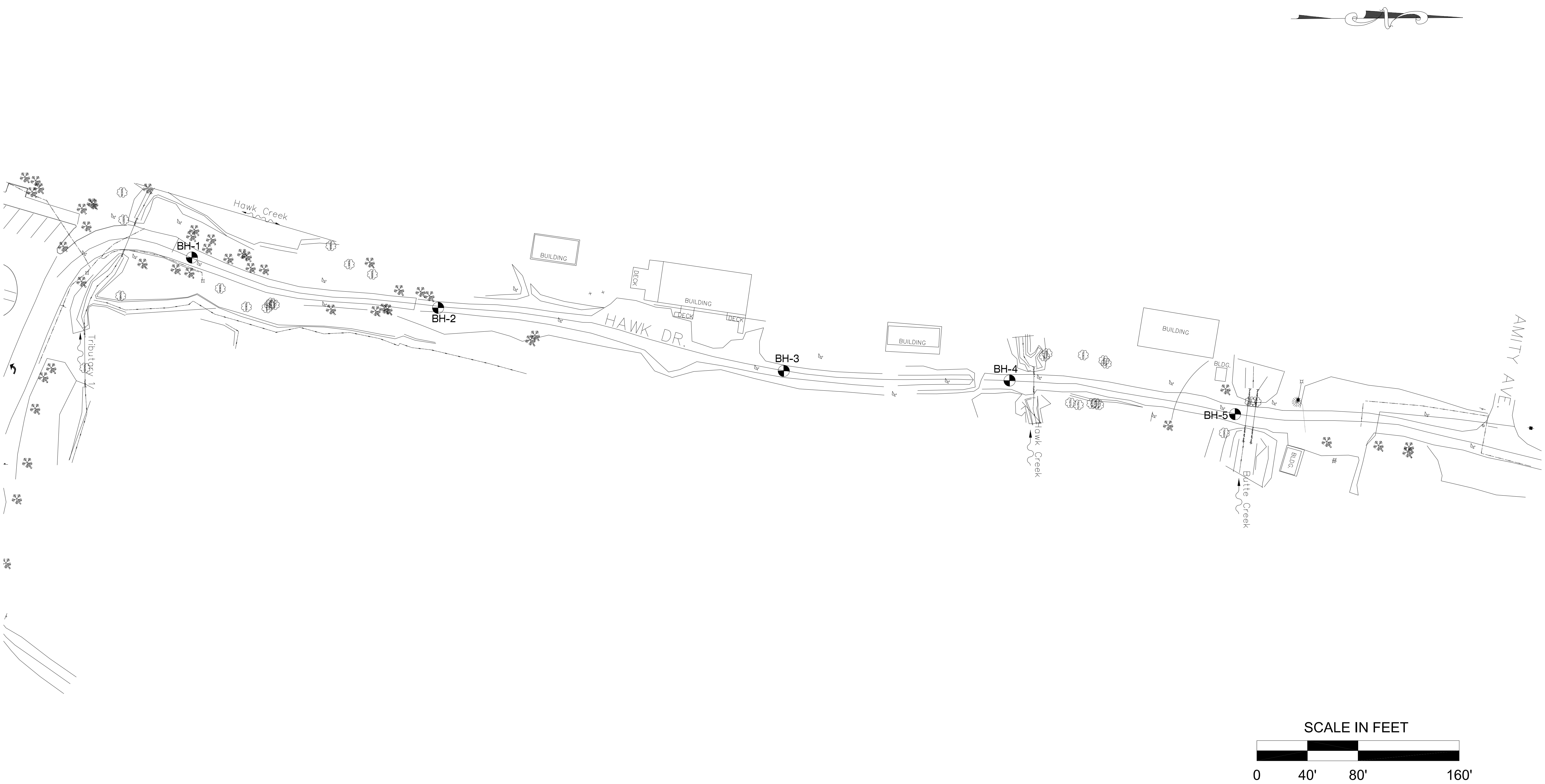
BUS. (541) 757-7645 FAX (541) 757-7650

VICINITY MAP

HAWK STREET EXTENSION
NESKOWIN, OREGON

FIGURE NO.
1A

FILE NAME: Fig1A



NOTES:
1. BOREHOLE LOCATIONS WERE SURVEYED AFTER DRILLING.
2. SEE REPORT FOR A DISCUSSION OF SUBSURFACE CONDITIONS.
3. BASE MAP WAS PROVIDED BY OBEC CONSULTING ENGINEERS.

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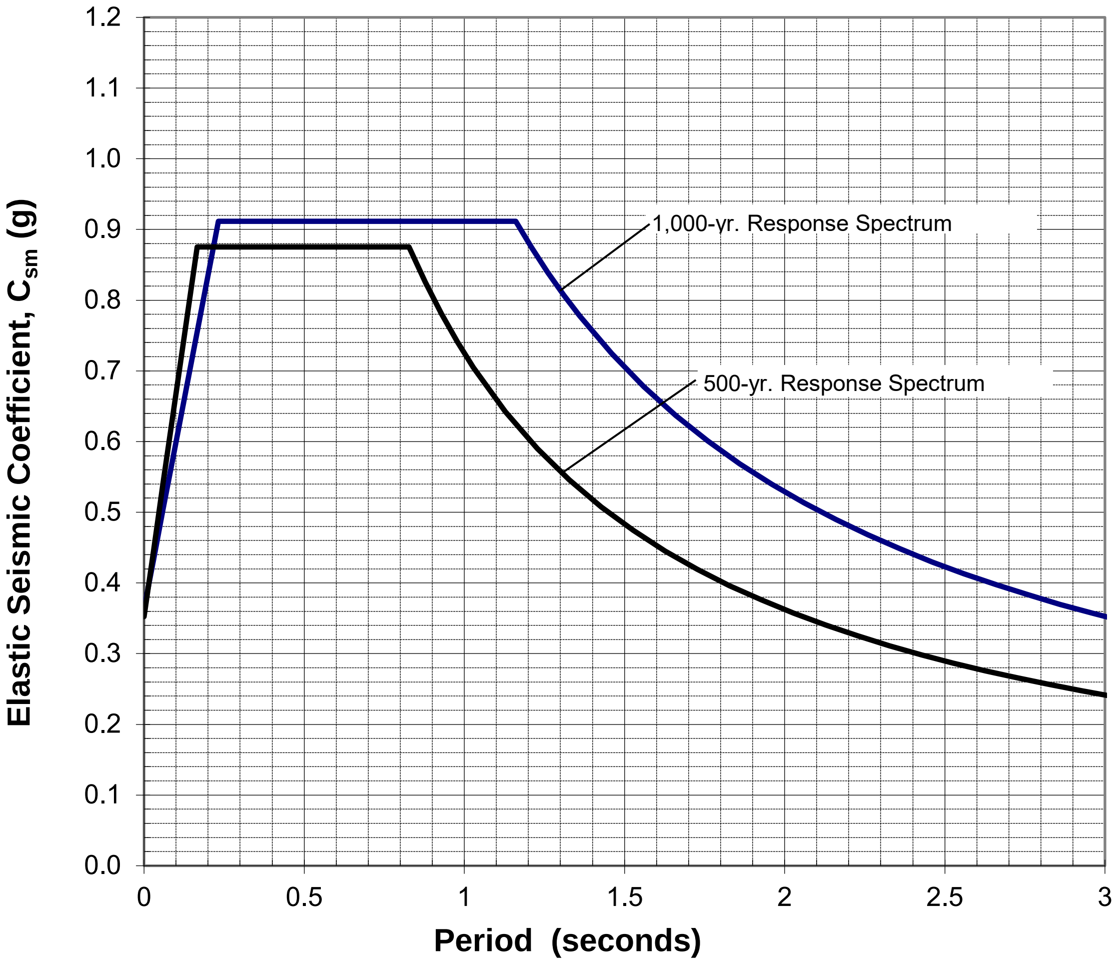
DATE JUNE 2015
DWN. MDM
APPR.
REVIS.
PROJECT NO.
2151044

BORING LOCATIONS

HAWK STREET EXTENSION
NESKOWIN, OREGON

FIGURE NO.
2A

FILE NAME:FIG 2A



Notes:

1. The Design Response Spectrum is based on AASHTO 2014 Section 3.10.3 using the following parameters:

	Site Class= E		Damping = 5%			
1,000-yr.	PGA =	0.39	F_{pga} =	0.93	A_s =	0.36
	S_S =	0.93	F_a =	0.98	S_{DS} =	0.91
	S_1 =	0.44	F_v =	2.40	S_{D1} =	1.06
500-yr.	PGA =	0.23	F_{pga} =	1.55	A_s =	0.35
	S_S =	0.53	F_a =	1.64	S_{DS} =	0.88
	S_1 =	0.24	F_v =	3.04	S_{D1} =	0.72

2. PGA, S_S, and S_1 values are based on USGS 2002 maps and procedures included in AASHTO 2012. The 1,000-yr. values assume 7% probability of exceedence in 75 years. The 500-yr. values assume 10% probability of exceedence in 50 years.

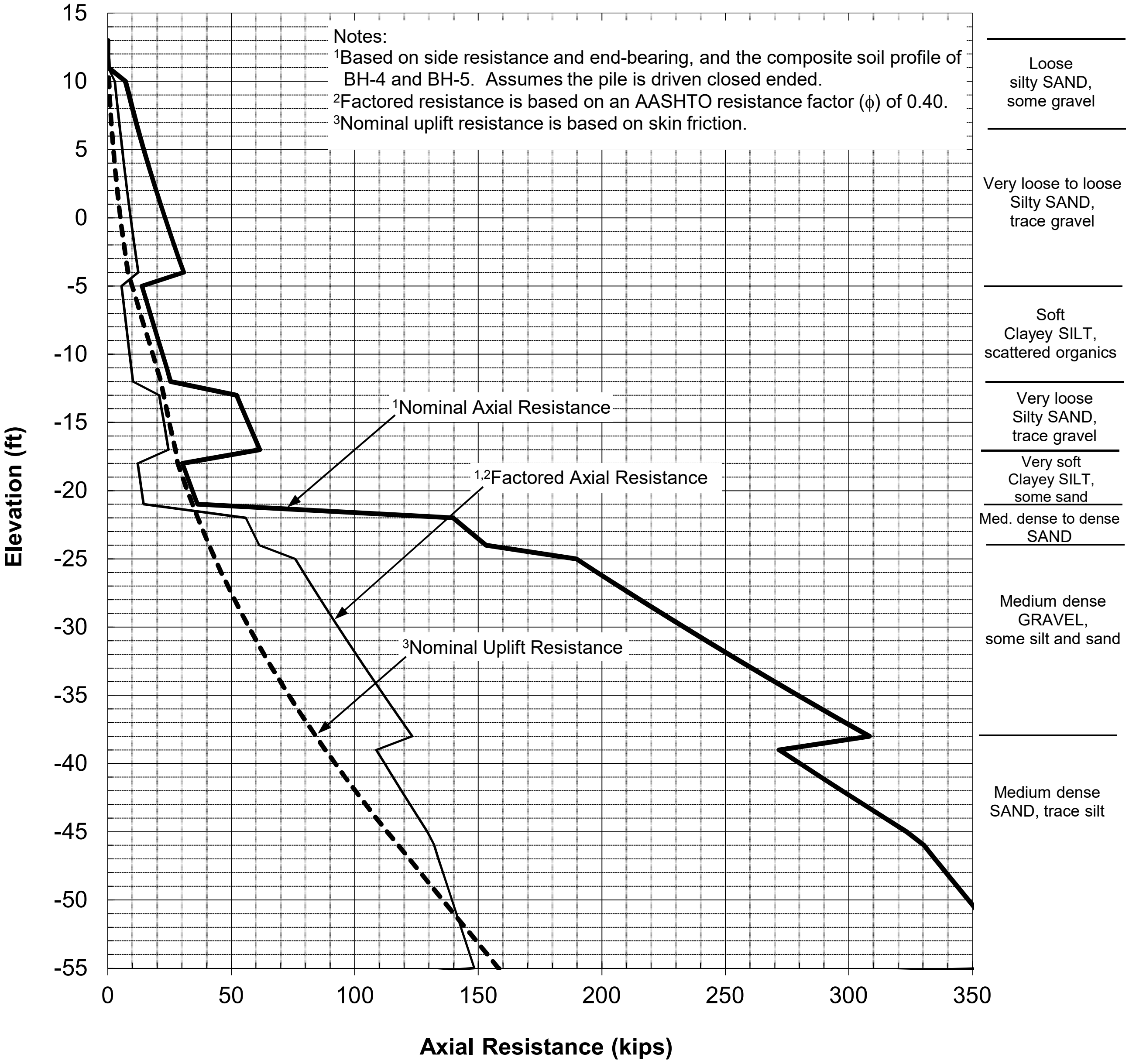
3. F_{pga}, F_a, and F_v were established based on AASHTO 2012, Tables 3.10.3.2.1-1, 3.10.3.2.1-2 and 3.10.3.2.1-3 using the selected PGA, S_s, and S_1 values, respectively.

4. Site location is: Latitude 45.1054, Longitude -123.9817.

FIGURE 3A.

AASHTO 2014 GENERAL PROCEDURE RESPONSE SPECTRA

Hawk Street Extension
Tillamook County, Oregon
FEI Project No. 2151044



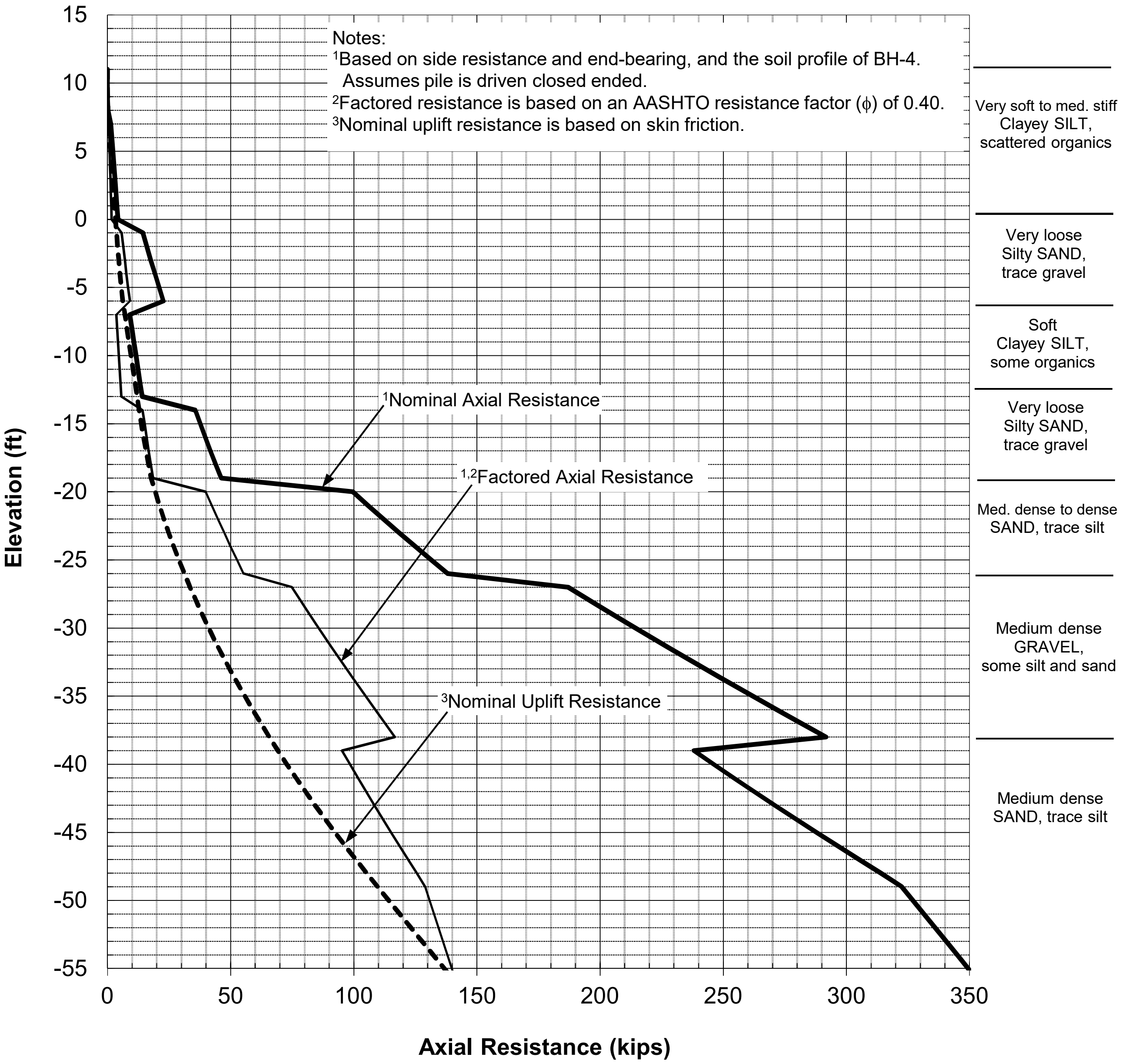
Notes:

[1] Based on side resistance and end-bearing, and the composite soil profile of BH-4 and BH-5. Assumes the pile is driven closed ended.

[2] Factored resistance is based on an AASHTO resistance factor (ϕ) of 0.40.

[3] Nominal uplift resistance is based on skin friction.

FIGURE 4A
AXIAL RESISTANCE vs. ELEVATION
PP12.75x0.5 CLOSED-ENDED PILE
BUTTE CREEK CROSSING
Hawk Street Extension - Community of Neskowin
Tillamook County, Oregon
Project 2151044

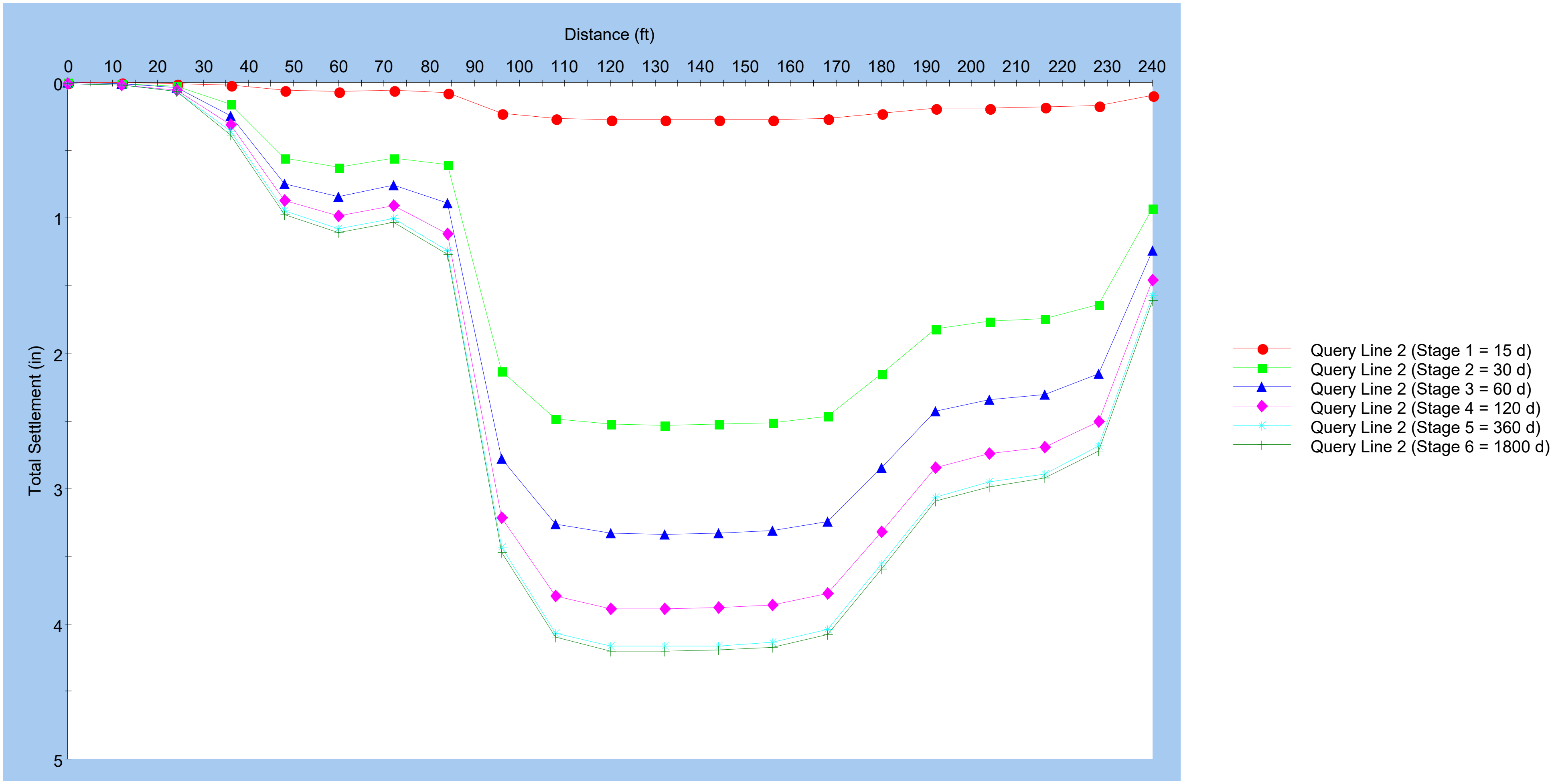


Notes:
[1] Based on side resistance and end-bearing, and the soil profile of BH-4. Assumes pile is driven closed ended.
[2] Factored resistance is based on an AASHTO resistance factor (ϕ) of 0.40.
[3] Nominal uplift resistance is based on skin friction.

FIGURE 5A
AXIAL RESISTANCE vs. ELEVATION
PP12.75x0.5 CLOSED-ENDED PILE
HAWK CREEK CROSSING
Hawk Street Extension - Community of Neskowin
Tillamook County, Oregon
Project 2151044

Data Type:

Distance vs. Total Settlement

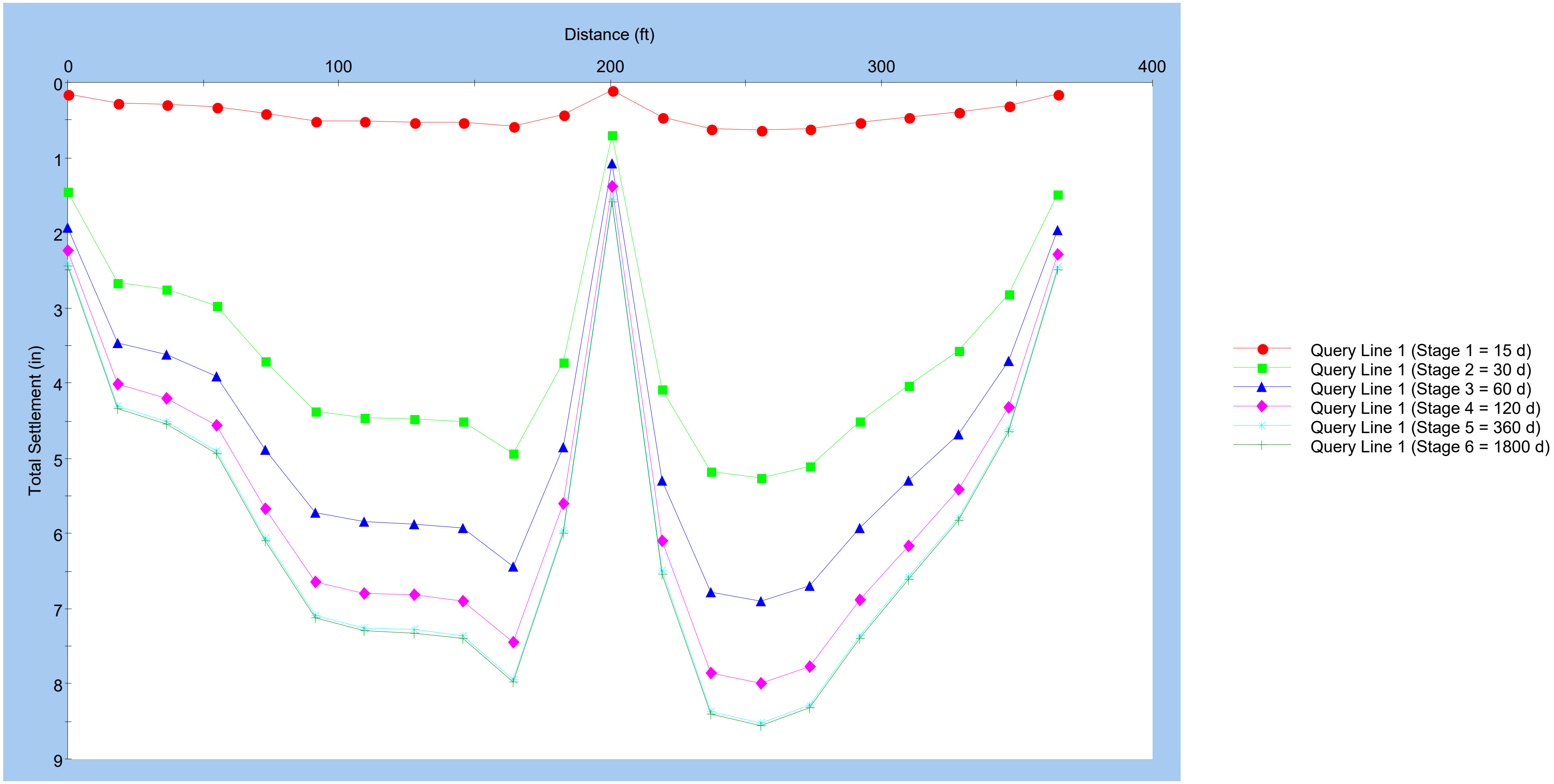


Total Settlement at Depth = 0 ft
Reference Stage: None

SETTLE3D 2.019	*Project* Hawk Street Extension		
	Analysis Description Figure 6A. Centerline Settlement (Sta. 11+00 to 13+40)		
	Drawn By mdm	*Company* Foundation Engineering, Inc.	
	Date 4-14-17	*File Name* Settlement (11+00 to 13+40) (4-14-17).s3z	

Data Type:

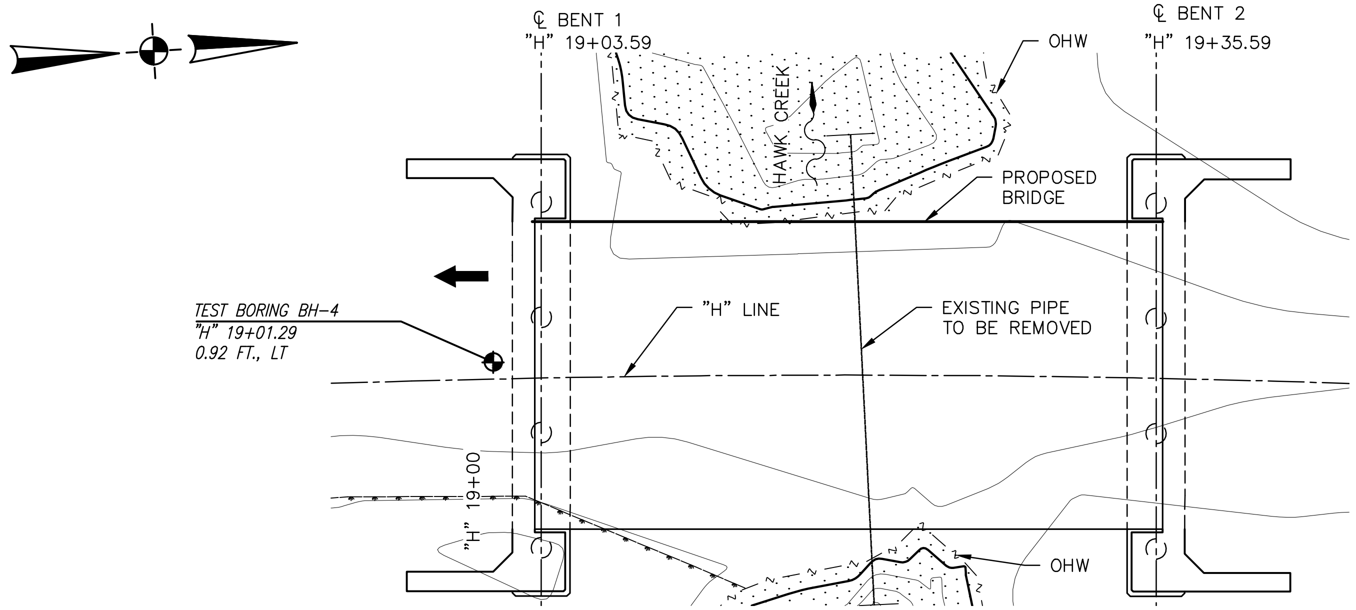
Distance vs. Total Settlement



Total Settlement at Depth = 0 ft
Reference Stage: None

SETTLE3D 2.019

Project	Hawk Street Extension
Analysis Description	Figure 7A. Centerline Settlement (Sta. 17+20 to 20+85)
Drawn By	mdm
Company	Foundation Engineering, Inc.
Date	4-14-17
File Name	Settlement (17+20 to 20+85) (4-14-17).s3z

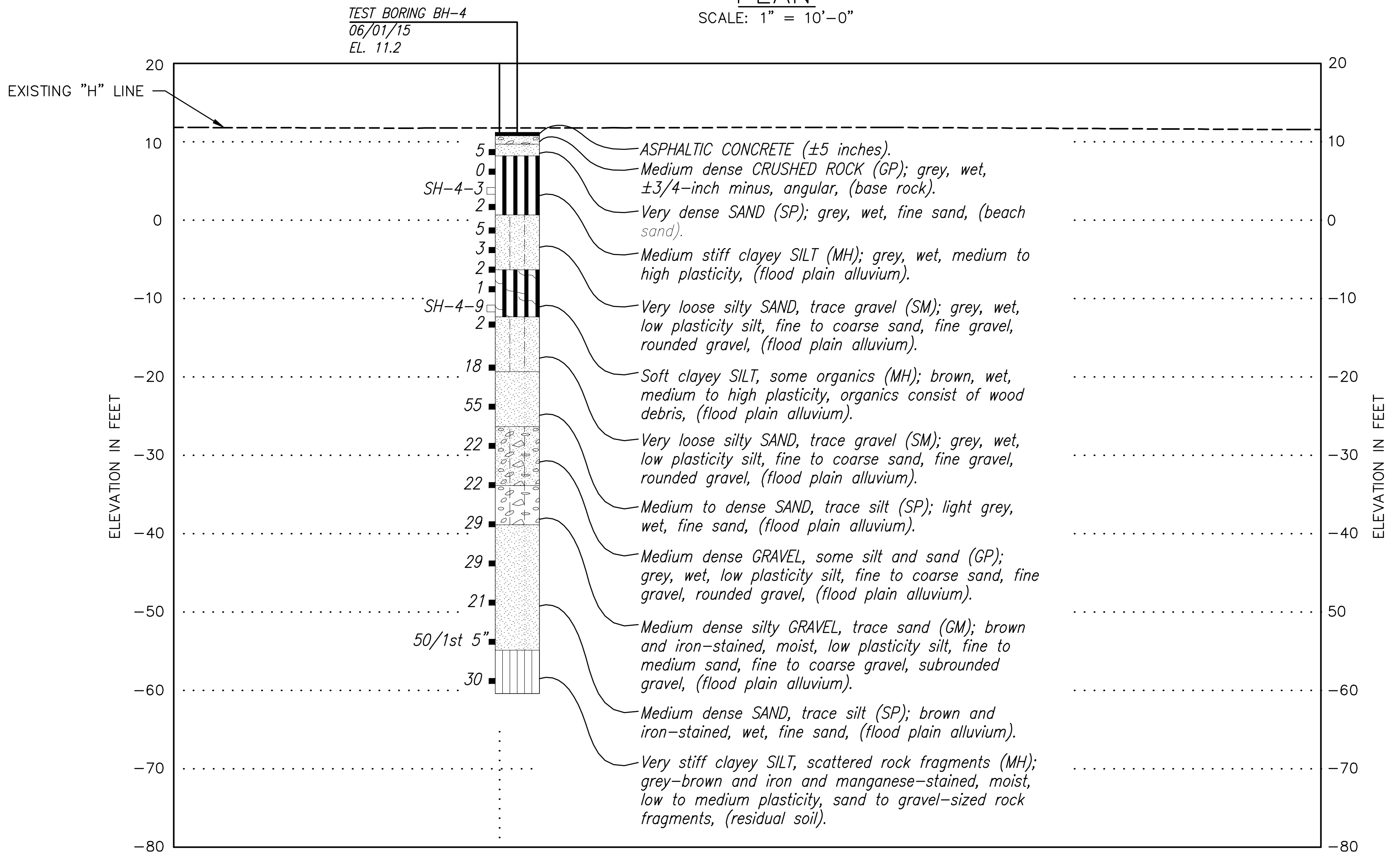


PLAN

SCALE: 1" = 10'–0"

LEGEND OF MATERIALS

- ■ *ASPHALTIC CONCRETE*
- ▨ *CRUSHED ROCK*
- ▨ *SAND*
- ▥ *Clayey SILT*
- ▨ *Silty SAND*
- ▥ *Clayey SILT, some organics*
- ▨ *GRAVEL, some silt and sand*
- ▨ *Silty GRAVEL*
- ▥ *Clayey SILT, scattered rock fragments*



ASPHALTIC CONCRETE (±5 inches).

Medium dense CRUSHED ROCK (GP); grey, wet, ±3/4–inch minus, angular, (base rock).

Very dense SAND (SP); grey, wet, fine sand, (beach sand).

Medium stiff clayey SILT (MH); grey, wet, medium to high plasticity, (flood plain alluvium).

Very loose silty SAND, trace gravel (SM); grey, wet, low plasticity silt, fine to coarse sand, fine gravel, rounded gravel, (flood plain alluvium).

Soft clayey SILT, some organics (MH); brown, wet, medium to high plasticity, organics consist of wood debris, (flood plain alluvium).

Very loose silty SAND, trace gravel (SM); grey, wet, low plasticity silt, fine to coarse sand, fine gravel, rounded gravel, (flood plain alluvium).

Medium to dense SAND, trace silt (SP); light grey, wet, fine sand, (flood plain alluvium).

Medium dense GRAVEL, some silt and sand (GP); grey, wet, low plasticity silt, fine to coarse sand, fine gravel, rounded gravel, (flood plain alluvium).

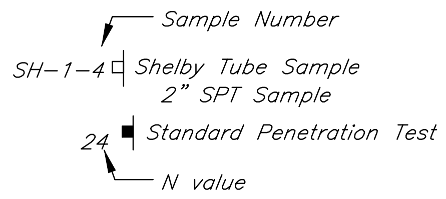
Medium dense silty GRAVEL, trace sand (GM); brown and iron–stained, moist, low plasticity silt, fine to medium sand, fine to coarse gravel, subrounded gravel, (flood plain alluvium).

Medium dense SAND, trace silt (SP); brown and iron–stained, wet, fine sand, (flood plain alluvium).

Very stiff clayey SILT, scattered rock fragments (MH); grey–brown and iron and manganese–stained, moist, low to medium plasticity, sand to gravel–sized rock fragments, (residual soil).

ELEVATION

HORIZONTAL SCALE: 1" = 10'–0"
VERTICAL SCALE: 1" = 20'–0"



Foundation data shown on this drawing may be a consolidation of information and/or revision in terminology from the Soils and Geological Exploration Logs. The Soils and Exploration Logs used in compiling this drawing are available upon request for review at the office of Foundation Engineering, Inc., Corvallis, OR.

ADVANCE
MM-DD-YYYY

FOUNDATION ENGINEERING, INC.
PROFESSIONAL GEOTECHNICAL SERVICES
820 N.W. CORNELL AVENUE
CORVALLIS, OREGON 97330
BUS. (541) 757–7645 FAX (541) 757–7650

△	DATE	REVISION	BY

ACCOMPANIED BY DRAWINGS:
SEE WH001

SCALE WARNING
If scale bar does not measure one inch, then drawing is not to scale

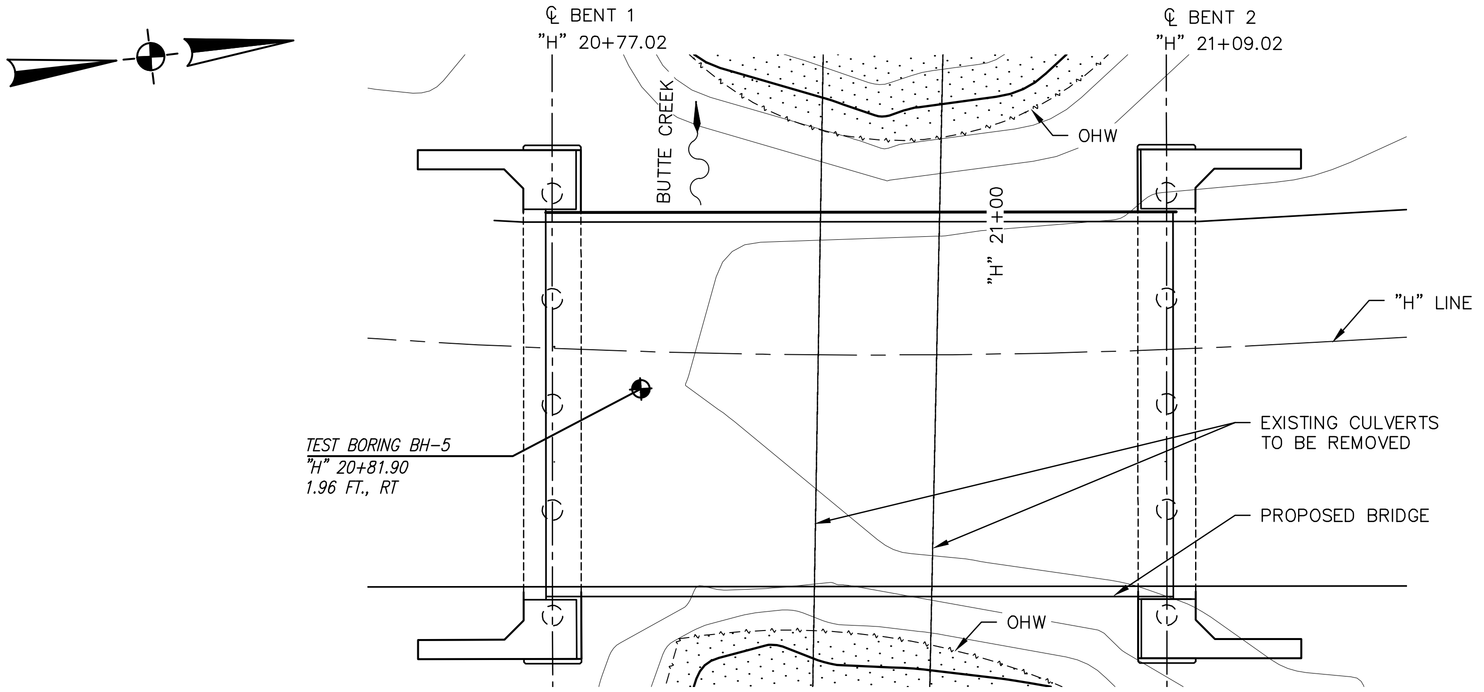
HAWK CREEK CROSSING
HAWK STREET EXTENSION
TILLAMOOK COUNTY

FOUNDATION DATA

OBEC CONSULTING ENGINEERS www.obec.com
CORPORATE OFFICE:
920 COUNTRY CLUB ROAD, SUITE 100B EUGENE, OREGON 97401-6089
REGIONAL OFFICES:
LAKE OSWEGO; SALEM; MEDFORD, OREGON ; VANCOUVER, WASHINGTON

DESIGNER: ERIC J. VAVRA, EIT
REVIEWER: DOUGLAS A. KIRKPATRICK, PE
CHECKER: XIQIN LONG, PE
DRAFTER: OBEC CAD

DATE	STRUCTURE NO.	CALC. BOOK	SHEET	DRAWING NO.
MARCH 2017	22664	0000	3 OF 10	WH003

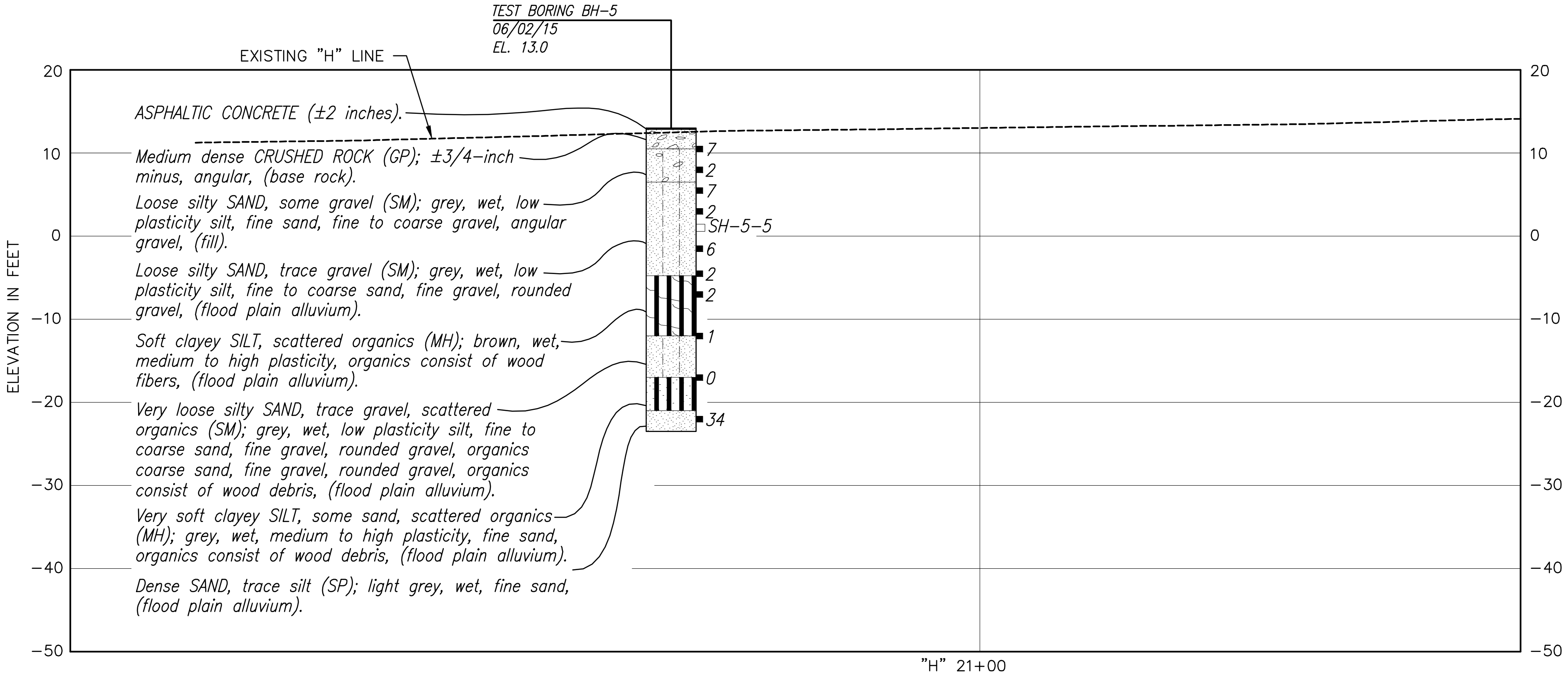


PLAN

SCALE: 1" = 10'-0"

LEGEND OF MATERIALS

- ■ *ASPHALTIC CONCRETE*
- ▨ *CRUSHED ROCK*
- ▨ *Silty SAND, some gravel*
- ▨ *Silty SAND*
- ▥ *Clayey SILT, scattered organics*
- ▥ *Clayey SILT, some sand*



ASPHALTIC CONCRETE (±2 inches).

Medium dense CRUSHED ROCK (GP); ±3/4-inch minus, angular, (base rock).

Loose silty SAND, some gravel (SM); grey, wet, low plasticity silt, fine sand, fine to coarse gravel, angular gravel, (fill).

Loose silty SAND, trace gravel (SM); grey, wet, low plasticity silt, fine to coarse sand, fine gravel, rounded gravel, (flood plain alluvium).

Soft clayey SILT, scattered organics (MH); brown, wet, medium to high plasticity, organics consist of wood fibers, (flood plain alluvium).

Very loose silty SAND, trace gravel, scattered organics (SM); grey, wet, low plasticity silt, fine to coarse sand, fine gravel, rounded gravel, organics coarse sand, fine gravel, rounded gravel, organics consist of wood debris, (flood plain alluvium).

Very soft clayey SILT, some sand, scattered organics (MH); grey, wet, medium to high plasticity, fine sand, organics consist of wood debris, (flood plain alluvium).

Dense SAND, trace silt (SP); light grey, wet, fine sand, (flood plain alluvium).

ELEVATION

HORIZONTAL SCALE: 1" = 10'-0"
VERTICAL SCALE: 1" = 20'-0"

Sample Number
SH-1-4 ☐ *Shelby Tube Sample*
2" SPT Sample
24 ■ *Standard Penetration Test*
N value

Foundation data shown on this drawing may be a consolidation of information and/or revision in terminology from the Soils and Geological Exploration Logs. The Soils and Exploration Logs used in compiling this drawing are available upon request for review at the office of Foundation Engineering, Inc., Corvallis, OR.

ADVANCE
MM-DD-YYYY

△	DATE	REVISION	BY

ACCOMPANIED BY DRAWINGS:
SEE WBOO1

SCALE WARNING
If scale bar does not measure one inch, then drawing is not to scale

FOUNDATION ENGINEERING, INC.
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BUTTE CREEK CROSSING
HAWK STREET EXTENSION
TILLAMOOK COUNTY

FOUNDATION DATA

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920 COUNTRY CLUB ROAD, SUITE 100B EUGENE, OREGON 97401-6089
REGIONAL OFFICES:
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DESIGNER: ERIC J. VAVRA, EIT	REVIEWER: DOUGLAS A. KIRKPATRICK, PE
CHECKER: XIQIN LONG, PE	DRAFTER: OBEC CAD

DATE	STRUCTURE NO.	CALC. BOOK	SHEET	DRAWING NO.
MARCH 2017	22663	0000	3 OF 10	WB003

Appendix B

Boring Logs

Professional
Geotechnical
Services

Foundation Engineering, Inc.

DISTINCTION BETWEEN FIELD LOGS AND FINAL LOGS

A field log is prepared for each boring or test pit by our field representative. The log contains information concerning sampling depths and the presence of various materials such as gravel, cobbles, and fill, and observations of ground water. It also contains our interpretation of the soil conditions between samples. The final logs presented in this report represent our interpretation of the contents of the field logs and the results of the sample examinations and laboratory test results. Our recommendations are based on the contents of the final logs and the information contained therein and not on the field logs.

VARIATION IN SOILS BETWEEN TEST PITS AND BORINGS

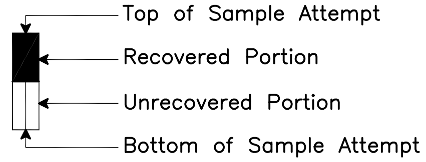
The final log and related information depict subsurface conditions only at the specific location and on the date indicated. Those using the information contained herein should be aware that soil conditions at other locations or on other dates may differ. Actual foundation or subgrade conditions should be confirmed by us during construction.

TRANSITION BETWEEN SOIL OR ROCK TYPES

The lines designating the interface between soil, fill or rock on the final logs and on subsurface profiles presented in the report are determined by interpolation and are therefore approximate. The transition between the materials may be abrupt or gradual. Only at boring or test pit locations should profiles be considered as reasonably accurate and then only to the degree implied by the notes thereon.

SAMPLE OR TEST SYMBOLS

SH–3–4
Sample Number
Boring or Test Pit Number
Sample Type



S – Grab Sample
SS – Standard Penetration Test Sample (split–spoon)
SH – Thin–walled Shelby Tube Sample
C – Pavement Core Sample
CS – Rock Core Sample

▲ Standard Penetration Test Resistance equals the number of blows a 140 lb. weight falling 30 in. is required to drive a standard split–spoon sampler 1 ft. Practical refusal is equal to 50 or more blows per 6 in. of sampler penetration.

● Water Content (%).

UNIFIED SOIL CLASSIFICATION SYMBOLS

G – Gravel
S – Sand
M – Silt
C – Clay
Pt – Peat

W – Well Graded
P – Poorly Graded
L – Low Plasticity
H – High Plasticity
O – Organic

FIELD SHEAR STRENGTH TEST

Shear strength measurements on test pit side walls, blocks of soil or Shelby tube samples are typically made with Torvane or Field Vane shear devices.

TYPICAL SOIL/ROCK SYMBOLS

▩ Concrete
▢ Organics
▨ Clay

░ Sand
▦ Gravel
▥ Silt

▧ Basalt
░ Sandstone
▤ Siltstone

WATER TABLE

▼ Water Table Location

(1/31/16) Date of Measurement

FOUNDATION ENGINEERING INC.
PROFESSIONAL GEOTECHNICAL SERVICES

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CORVALLIS, OR 97330
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7857 SW CIRRUS DRIVE, BUILDING 24
BEAVERTON, OR 97008
BUS. (503) 641–1541

SYMBOL KEY

BORING AND TEST PIT LOGS

Explanation of Common Terms Used in Soil Descriptions

Field Identification	Cohesive Soils			Granular Soils	
	SPT*	S_u^{**} (tsf)	Term	SPT*	Term
Easily penetrated several inches by fist.	0 – 2	< 0.125	Very Soft	0 – 4	Very Loose
Easily penetrated several inches by thumb.	2 – 4	0.125–0.25	Soft	4 – 10	Loose
Can be penetrated several inches by thumb with moderate effort.	4 – 8	0.25 – 0.50	Medium Stiff	10 – 30	Medium Dense
Readily indented by thumb but penetrated only with great effort.	8 – 15	0.50 – 1.0	Stiff	30 – 50	Dense
Readily indented by thumbnail.	15 – 30	1.0 – 2.0	Very Stiff	> 50	Very Dense
Indented with difficulty by thumbnail.	>30	> 2.0	Hard		

* SPT N–value in blows per foot (bpf)
** Undrained shear strength

Term	Soil Moisture Field Description
Dry	Absence of moisture. Dusty. Dry to the touch.
Damp	Soil has moisture. Cohesive soils are below plastic limit and usually moldable.
Moist	Grains appear darkened, but no visible water. Silt/clay will clump. Sand will bulk. Soils are often at or near plastic limit.
Wet	Visible water on larger grain surfaces. Sand and cohesionless silt exhibit dilatancy. Cohesive soil can be readily remolded. Soil leaves wetness on the hand when squeezed. Soil is wetter than the optimum moisture content and above the plastic limit.

Term	PI	Plasticity Field Test
Non–plastic	0 – 3	Cannot be rolled into a thread at any moisture.
Low Plasticity	3 – 15	Can be rolled into a thread with some difficulty.
Medium Plasticity	15 – 30	Easily rolled into thread.
High Plasticity	> 30	Easily rolled and re–rolled into thread.

Term	Soil Structure Criteria
Stratified	Alternating layers at least ¼ inch thick.
Laminated	Alternating layers less than ¼ inch thick.
Fissured	Contains shears and partings along planes of weakness.
Slickensided	Partings appear glossy or striated.
Blocky	Breaks into small lumps that resist further breakdown.
Lensed	Contains pockets of different soils.

Term	Soil Cementation Criteria
Weak	Breaks under light finger pressure.
Moderate	Breaks under hard finger pressure.
Strong	Will not break with finger pressure.

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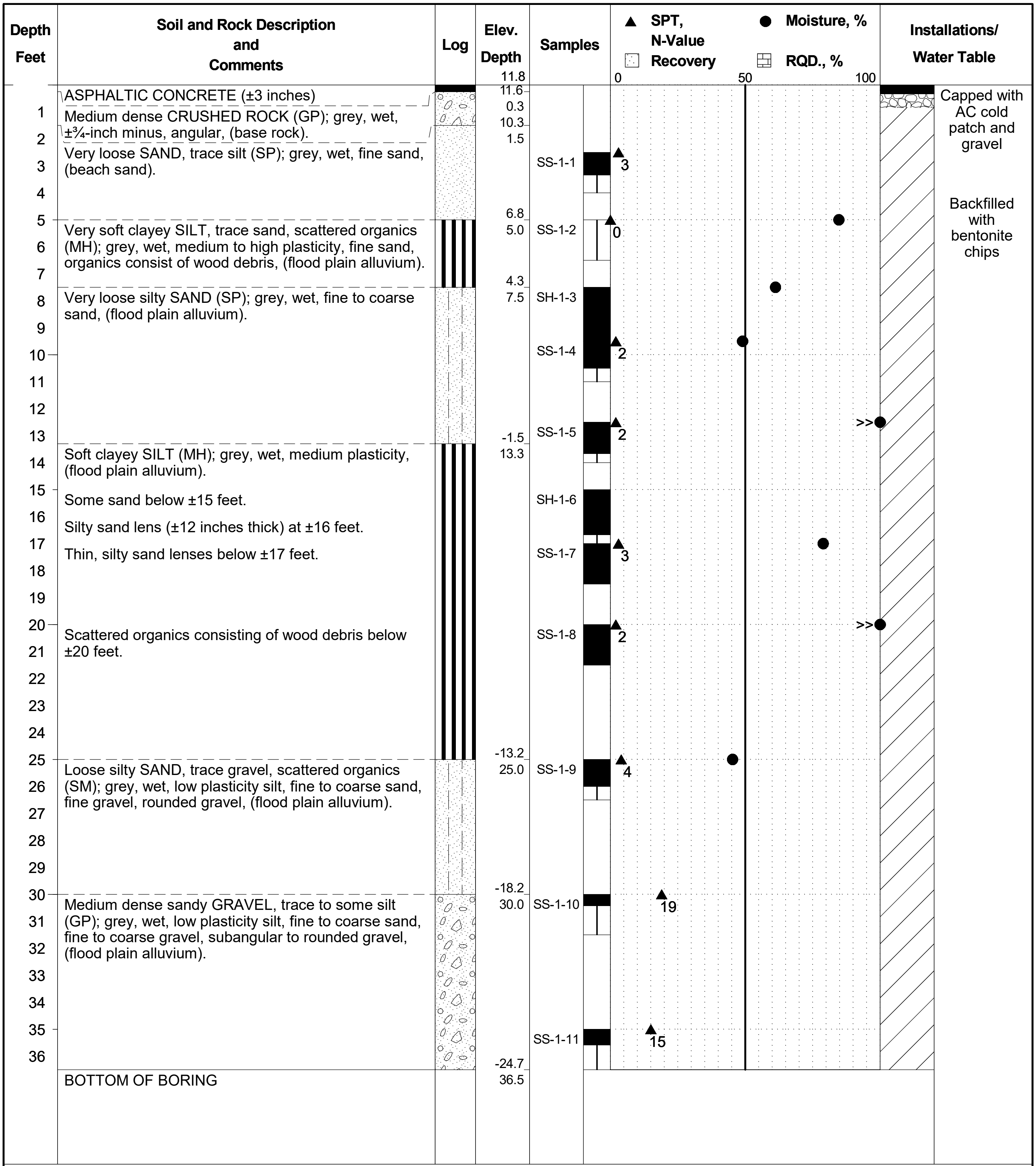
COMMON TERMS

SOIL DESCRIPTIONS



Depth Feet	Soil and Rock Description and Comments	Elev. Depth	Samples	SPT, N-Value
		11.8		
	ASPHALTIC CONCRETE (±3 inches)	11.6 0.3		
1	Medium dense CRUSHED ROCK (GP); grey, wet, ±¾-inch minus, angular, (base rock).	10.3 1.5		
2–4	Very loose SAND, trace silt (SP); grey, wet, fine sand, (beach sand).	6.8 5.0	SS-1-1	3
5–7	Very soft clayey SILT, trace sand, scattered organics (MH); grey, wet, medium to high plasticity, fine sand, organics consist of wood debris, (flood plain alluvium).	4.3 7.5	SS-1-2	0
8–13	Very loose silty SAND (SP); grey, wet, fine to coarse sand, (flood plain alluvium).	-1.5 13.3	SH-1-3 SS-1-4 SS-1-5	 2 2
14–24	Soft clayey SILT (MH); grey, wet, medium plasticity, (flood plain alluvium). Some sand below ±15 feet. Silty sand lens (±12 inches thick) at ±16 feet. Thin, silty sand lenses below ±17 feet. Scattered organics consisting of wood debris below ±20 feet.	-13.2 25.0	SH-1-6 SS-1-7 SS-1-8	 3 2
25–29	Loose silty SAND, trace gravel, scattered organics (SM); grey, wet, low plasticity silt, fine to coarse sand, fine gravel, rounded gravel, (flood plain alluvium).	-18.2 30.0	SS-1-9	4
30–36	Medium dense sandy GRAVEL, trace to some silt (GP); grey, wet, low plasticity silt, fine to coarse sand, fine to coarse gravel, subangular to rounded gravel, (flood plain alluvium).	-24.7 36.5	SS-1-10 SS-1-11	19 15
	BOTTOM OF BORING			

Installations/ Water Table

Capped with AC cold patch and gravel

Backfilled with bentonite chips

Project No.: 2151044

Surface Elevation: 11.8 feet (Approx.)

Date of Boring: June 2, 2015

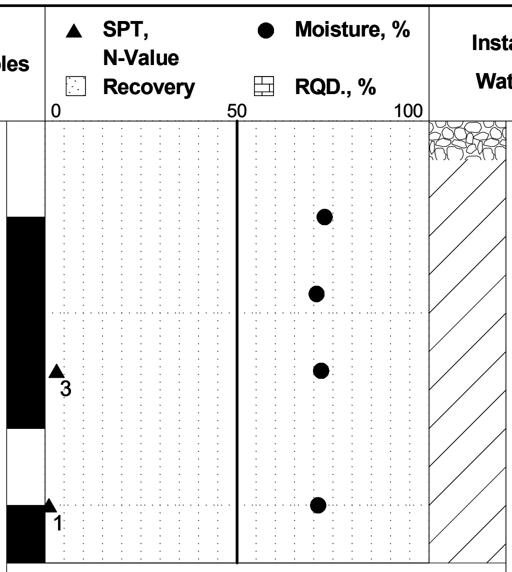
Boring Log: BH-1

Walton Way - Neskowin

Neskowin, Oregon

Foundation Engineering, Inc.

Depth Feet	Soil and Rock Description and Comments	Elev. Depth	Samples	Installations/ Water Table
1 2	Medium dense CRUSHED ROCK (GP); grey, wet, ±¾-inch minus, angular, (base rock).	12.8 0.0		Capped with gravel
3 4 5	Very loose to loose silty SAND (SM); grey, wet, low plasticity silt, fine sand, (beach sand). Scattered clayey silt lenses below ±4 feet. Field vane on SH-2-1: S_u = ±0.1 tsf at ±4.3 feet.	10.8 2.0	SH-2-1 SH-2-2	Backfilled with bentonite chips
6 7 8 9 10	Very soft to soft clayey SILT, trace sand (MH); grey, wet, medium to high plasticity, fine sand, (flood plain alluvium). Field vane on SH-2-2: S_u = ±0.1 tsf at ±6 feet. Some sand and scattered organics consisting of wood debris and wood fibers below ±6.5 feet.	7.3 5.5	SS-2-3 SS-2-4	
11	Very loose silty SAND, scattered organics (SM); grey, wet, low plasticity silt, fine to coarse sand, organics consist of wood debris, (flood plain alluvium).	1.8 11.0		
	BOTTOM OF BORING	1.3 11.5		



Project No.: 2151044

Surface Elevation: 12.8 feet (Approx.)

Date of Boring: June 2, 2015

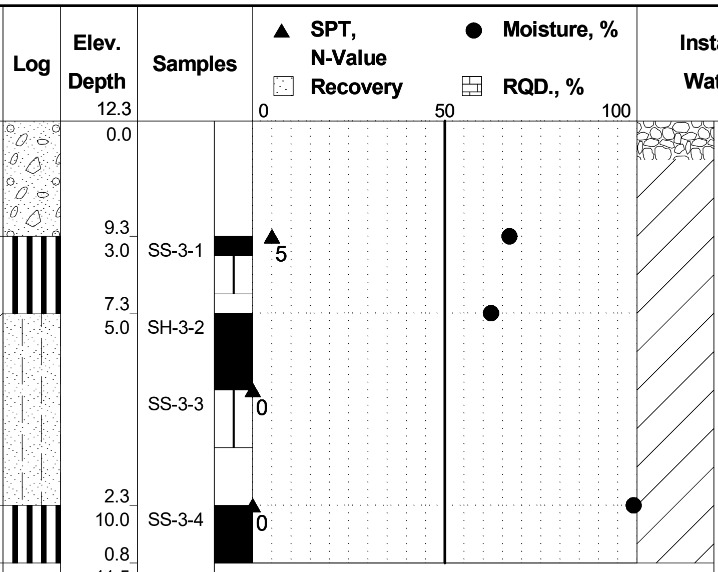
Foundation Engineering, Inc.

Boring Log: BH-2

Walton Way - Neskowin

Neskowin, Oregon

Depth Feet	Soil and Rock Description and Comments	Elev. Depth	Samples	▲ SPT, N-Value	Installations/ Water Table
		12.3			
1 2	Medium dense CRUSHED ROCK (GP); grey-brown, moist, ±1-inch minus, angular, (base rock).	0.0			Capped with gravel
3 4 5	Medium stiff clayey SILT, scattered sand lenses and organics (MH); grey, wet, medium to high plasticity, fine sand, organics consist of wood fibers, (flood plain alluvium).	9.3 3.0	SS-3-1	5	Backfilled with bentonite chips
6 7 8 9	Very loose silty SAND (SM); grey, wet, low plasticity silt, fine sand, (flood plain alluvium). Fine to coarse sand with trace silt below ±6 feet	7.3 5.0	SH-3-2 SS-3-3	0	
10 11	Very soft clayey SILT, trace sand, scattered organics (MH); grey, wet, medium to high plasticity, fine sand, organics consist of wood debris, (flood plain alluvium).	2.3 10.0	SS-3-4	0	
	BOTTOM OF BORING	0.8 11.5			



Project No.: 2151044

Surface Elevation: 12.3 feet (Approx.)

Date of Boring: June 1, 2015

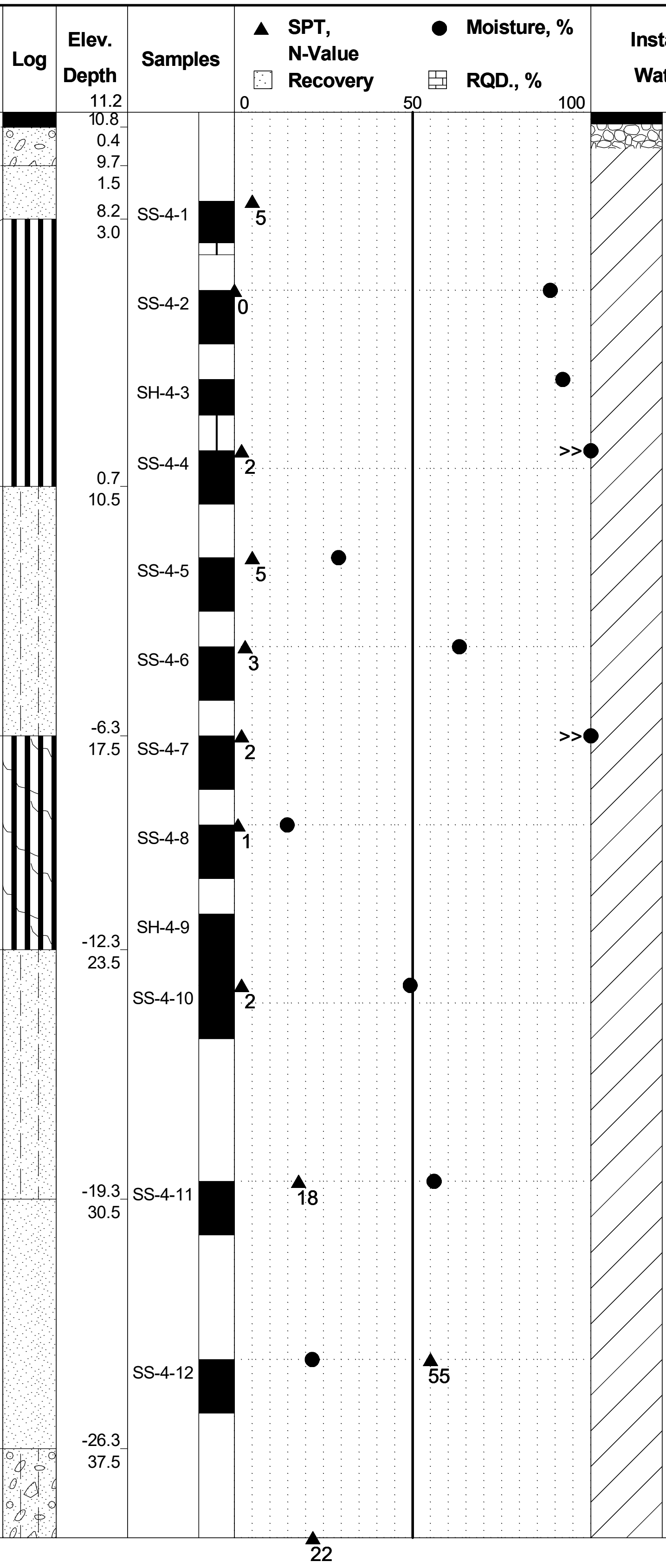
Foundation Engineering, Inc.

Boring Log: BH-3

Walton Way - Neskowin

Neskowin, Oregon

Depth Feet	Soil and Rock Description and Comments	Elev. Depth	Samples	SPT, N-Value	Installations/ Water Table
		11.2			
	ASPHALTIC CONCRETE (±5 inches).	10.8 0.4			Capped with AC cold patch and gravel Backfilled with bentonite chips (±1 to 3 feet)
1	Medium dense CRUSHED ROCK (GP); grey, wet, ±¾-inch minus, angular, (base rock).	9.7 1.5			
2					
3	Very dense SAND (SP); grey, wet, fine sand, (beach sand).	8.2 3.0	SS-4-1	5	
4	Medium stiff clayey SILT (MH); grey, wet, medium to high plasticity, (flood plain alluvium).				
5	Very soft with scattered organics and sand lenses below ±5 feet.		SS-4-2	0	
6					
7					
8	Field vane on SH-4-3: S_u = ±0.12 tsf at ±8 feet.		SH-4-3		Bentonite grout (±3 to 71.5 feet)
9					
10		0.7 10.5	SS-4-4	2	
11	Very loose silty SAND, trace gravel (SM); grey, wet, low plasticity silt, fine to coarse sand, fine gravel, rounded gravel, (flood plain alluvium).				
12					
13	Sandy silt lens (±12 inches thick) at ±12.5 feet.		SS-4-5	5	
14					
15			SS-4-6	3	
16	Clayey silt lens (±9 inches thick) at ±15.3 feet.				
17		-6.3 17.5			
18	Soft clayey SILT, some organics (MH); brown, wet, medium to high plasticity, organics consist of wood debris, (flood plain alluvium).		SS-4-7	2	
19					
20	Trace, fine to coarse sand below ±20 feet.		SS-4-8	1	
21					
22					
23	Field vane on SH-4-9: S_u = ±0.25 tsf at ±22.5 feet.	-12.3 23.5	SH-4-9		
24	Very loose silty SAND, trace gravel (SM); grey, wet, low plasticity silt, fine to coarse sand, fine gravel, rounded gravel, (flood plain alluvium).				
25			SS-4-10	2	
26					
27					
28					
29					
30	Scattered organics with no gravel below ±30 feet.	-19.3 30.5	SS-4-11	18	
31	Medium to dense SAND, trace silt (SP); light grey, wet, fine sand, (flood plain alluvium).				
32					
33					
34					
35	Very dense below ±35 feet.		SS-4-12	55	
36					
37		-26.3 37.5			
38	Medium dense GRAVEL, some silt and sand (GP); grey, wet, low plasticity silt, fine to coarse sand, fine gravel, rounded gravel, (flood plain alluvium).				
39					
				22	



Project No.: 2151044

Surface Elevation: 11.2 feet (Approx.)

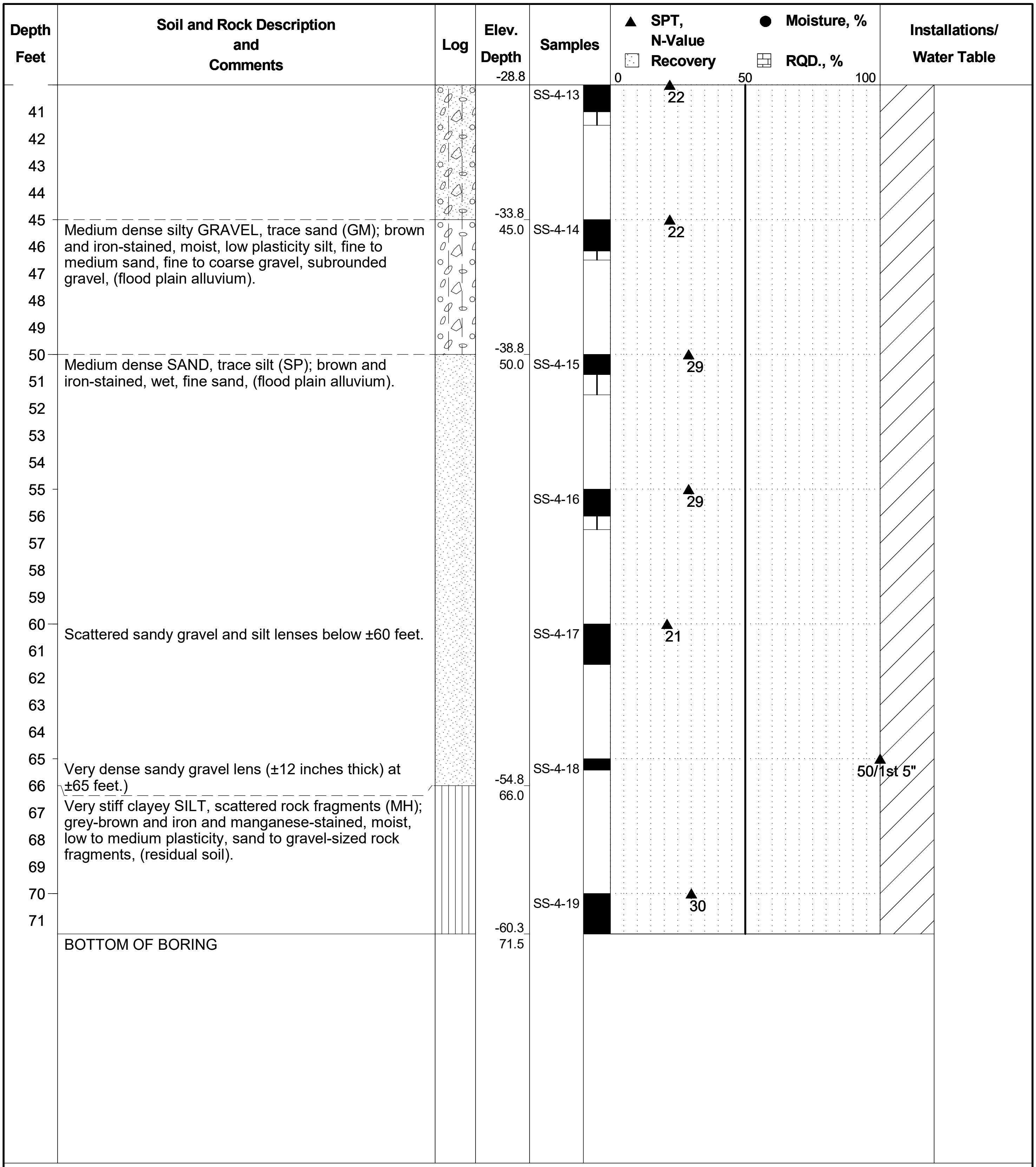
Date of Boring: June 1, 2015

Foundation Engineering, Inc.

Boring Log: BH-4

Walton Way - Neskowin

Neskowin, Oregon



Depth Feet	Soil and Rock Description and Comments	Elev. Depth	Samples	SPT, N-Value
40		-28.8	SS-4-13	22
45	Medium dense silty GRAVEL, trace sand (GM); brown and iron-stained, moist, low plasticity silt, fine to medium sand, fine to coarse gravel, subrounded gravel, (flood plain alluvium).	-33.8 45.0	SS-4-14	22
50	Medium dense SAND, trace silt (SP); brown and iron-stained, wet, fine sand, (flood plain alluvium).	-38.8 50.0	SS-4-15	29
55			SS-4-16	29
60	Scattered sandy gravel and silt lenses below ±60 feet.		SS-4-17	21
65	Very dense sandy gravel lens (±12 inches thick) at ±65 feet.)		SS-4-18	50/1st 5"
66	Very stiff clayey SILT, scattered rock fragments (MH); grey-brown and iron and manganese-stained, moist, low to medium plasticity, sand to gravel-sized rock fragments, (residual soil).	-54.8 66.0		
70			SS-4-19	30
71.5	BOTTOM OF BORING	-60.3 71.5		

Project No.: 2151044

Surface Elevation: 11.2 feet (Approx.)

Date of Boring: June 1, 2015

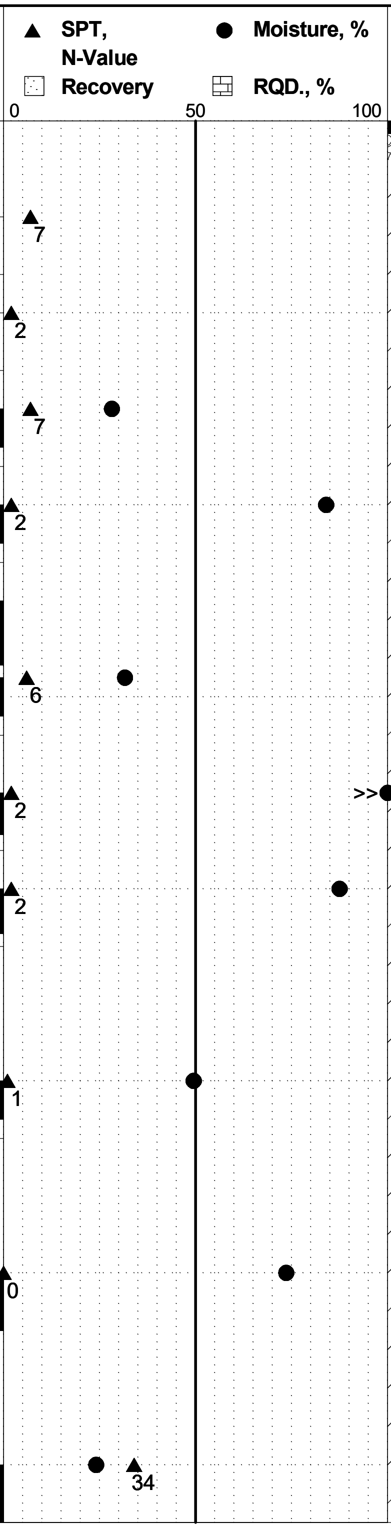
Foundation Engineering, Inc.

Boring Log: BH-4

Walton Way - Neskowin

Neskowin, Oregon

Depth Feet	Soil and Rock Description and Comments	Elev. Depth	Samples	SPT, N-Value	Installations/ Water Table
0	ASPHALTIC CONCRETE (±2 inches).	13			Capped with AC cold patch and gravel
1–2	Medium dense CRUSHED ROCK (GP); ±¾-inch minus, angular, (base rock).	12.8 0.2			
3–4	Loose silty SAND, some gravel (SM); grey, wet, low plasticity silt, fine sand, fine to coarse gravel, angular gravel, (fill).	10.5 2.5	SS-5-1	7	
5–6	Very loose below ±5 feet.		SS-5-2	2	Backfilled with bentonite chips
7–9	Loose silty SAND, trace gravel (SM); grey, wet, low plasticity silt, fine to coarse sand, fine gravel, rounded gravel, (flood plain alluvium).	6.5 6.5	SS-5-3	7	
10–12	Very loose with scattered organics consisting of wood debris and clayey silt lenses at ±10 feet.		SS-5-4	2	
13			SH-5-5		
14–17	Some silt below ±13.5 feet.		SS-5-6	6	
18–19	Soft clayey SILT, scattered organics (MH); brown, wet, medium to high plasticity, organics consist of wood fibers, (flood plain alluvium).	-4.8 17.8	SS-5-7	2	
20–24	Trace fine sand below ±20 feet.		SS-5-8	2	
25–29	Very loose silty SAND, trace gravel, scattered organics (SM); grey, wet, low plasticity silt, fine to coarse sand, fine gravel, rounded gravel, organics consist of wood debris, (flood plain alluvium).	-12.0 25.0	SS-5-9	1	
30–33	Very soft clayey SILT, some sand, scattered organics (MH); grey, wet, medium to high plasticity, fine sand, organics consist of wood debris, (flood plain alluvium).	-17.0 30.0	SS-5-10	0	
34–36	Dense SAND, trace silt (SP); light grey, wet, fine sand, (flood plain alluvium).	-21.0 34.0	SS-5-11	34	
	BOTTOM OF BORING	-23.5 36.5			



Project No.: 2151044

Surface Elevation: 13.0 feet (Approx.)

Date of Boring: June 2, 2015

Foundation Engineering, Inc.

Boring Log: BH-5

Walton Way - Neskowin

Neskowin, Oregon

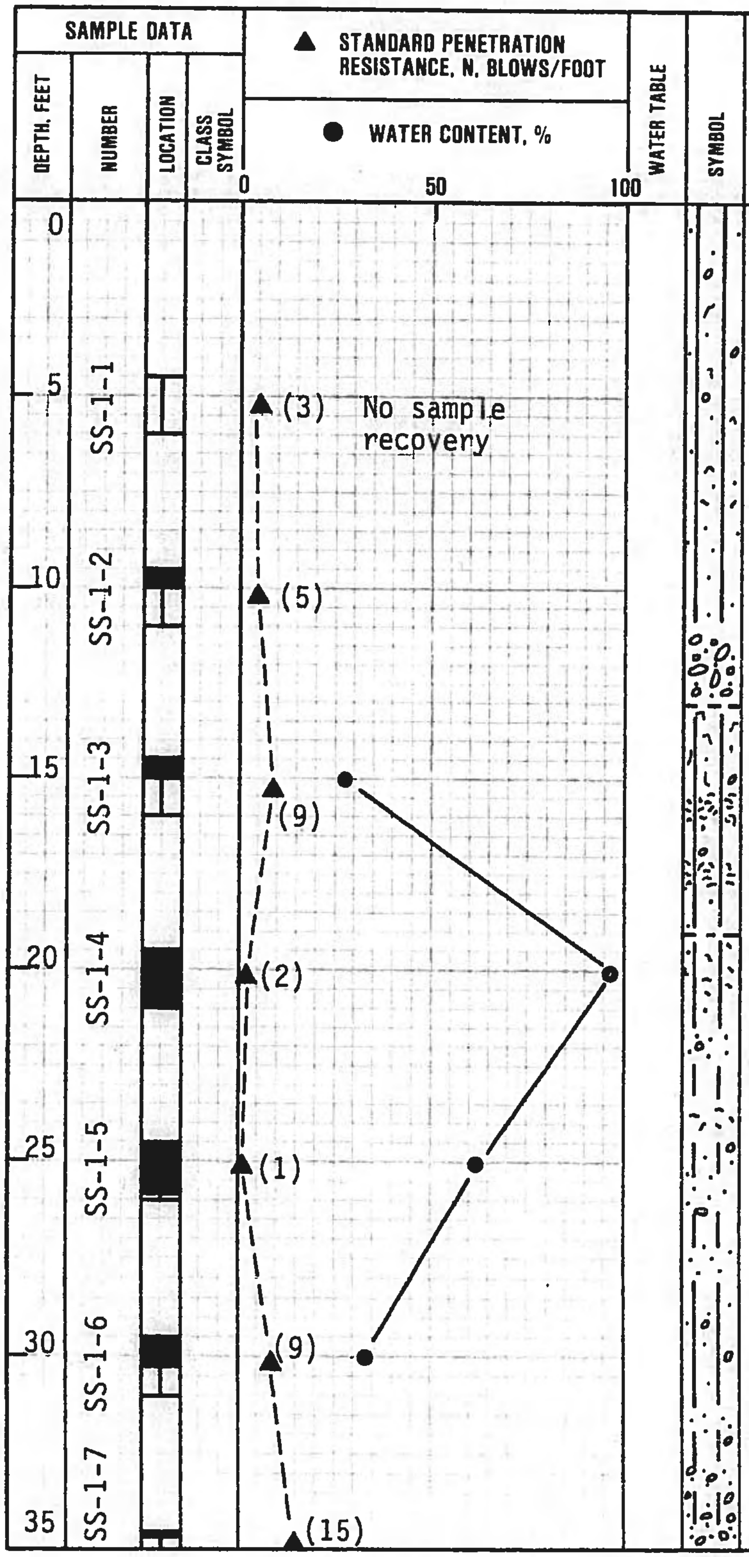
BORING LOG

PROJECT NAME Salem Avenue Bridge Neskowin, Oregon

PROJECT NUMBER P-340 **BORING NUMBER** BH-1

DATE OF BORING 3/18 to 3/19/87

GROUND ELEVATION AT BORING WHEN DRILLED El. 9 (approx.)



SOIL AND ROCK DESCRIPTION AND COMMENTS

Dark grey to blue-grey, loose or soft silt and sand with scattered pebbles and gravels. Relatively few organics.

Very easy, very smooth drilling indicates little gravel or rock.

Significantly more gravels noted between 11 and 13 ft.

Dark brown, saturated, soft, highly organic silt with traces of sand and scattered pebbles and fine gravel. Layers or seams of peat and fine organics noted within the silt.

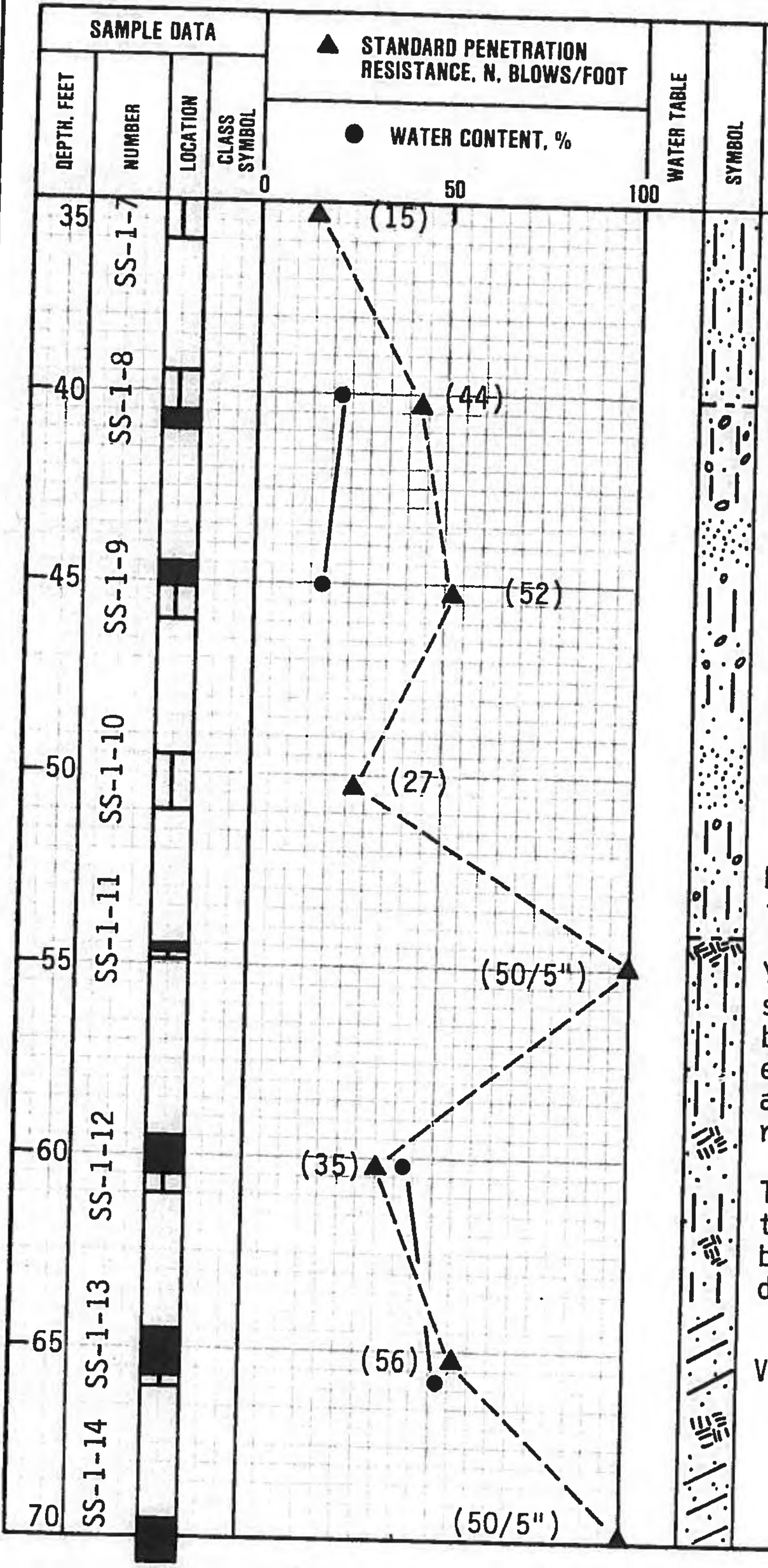
Grey, moist, organic silty sand to sandy silt with scattered pebbles. The soil contains less organic matter than the overlying silt.

Smooth, easy drilling indicates the presence of only scattered gravels.

Increasing amounts of fine gravel.

BORING LOG

PROJECT NAME Salem Avenue Bridge Neskowin, Oregon
PROJECT NUMBER P-340 **BORING NUMBER** BH-1
DATE OF BORING 3/18 to 3/19/87
GROUND ELEVATION AT BORING WHEN DRILLED El. 9 (approx.)



SOIL AND ROCK DESCRIPTION AND COMMENTS

Silty sand to sandy silt (same as above).

Yellow-brown to tan, medium to dense, variable mixture of slightly silty sand, gravel and fragments of deeply weathered rock.

The above formation alternates with layers of tan, moist, clean to slightly silty fine sand. The smooth drilling probably represents these sand layers.

Hard drilling below about 54 ft. indicates bedrock surface (?)

Yellow-brown to tan, iron-stained, deeply weathered basalt bedrock. The bedrock has altered to a silty sand or sandy clay and contains fragments of indurated, less weathered rock.

The consistency (hardness) of the formation varies considerbly with depth due to varying degrees of weathering.

Very hard drilling.

Appendix C

Laboratory Test Results

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Table 1C. Natural Water Contents, Percent Fines & Atterberg Limits

Sample Number	Sample Depth (ft)	Natural Water Content (percent)	LL	PL	PI	Percent Fines (%)	USCS Classification
SS-1-2	5.0 - 6.5	84.7					
SH-1-3	7.5 – 9.5	61.2					
SS-1-4	9.5 – 11.0	49.0				14.8	
SS-1-5	12.5 – 14.0	100.8					
SH-1-6	15.0 – 17.0	100.3					
SS-1-7	17.0 – 18.5	78.9					
SS-1-8	20.0 – 21.5	147.9					
SS-1-9	25.0 – 26.5	45.3				14.9	
SH-2-1	2.5 – 4.5	72.8					
SH-2-2	4.5 – 6.5	70.7	73	49	24		MH
SS-2-3	6.5 – 8.0	71.9					
SS-2-4	10.0 – 11.5	71.1					
SS-3-1	3.0 – 4.5	66.8					
SH-3-2	5.0 – 7.0	62.0				39.2	
SS-3-4	10.0 – 11.5	99.1					
SS-4-2	5.0 – 6.5	88.6					
SH-4-3	7.5 – 9.5	92.1	98	57	41		MH
SS-4-4	9.5 – 11.0	101.6					
SS-4-5	12.5 – 14.0	29.2					
SS-4-6	15.0 – 16.5	63.1					
SS-4-7	17.5 – 19.0	160.8					

Table 1C. Natural Water Contents, Percent Fines & Atterberg Limits

Sample Number	Sample Depth (ft)	Natural Water Content (percent)	LL	PL	PI	Percent Fines (%)	USCS Classification
SS-4-8	20.0 – 21.5	14.8					
SS-4-10	24.5 – 26.0	49.3				22.5	
SS-4-11	30.0 – 31.5	56.0					
SS-4-12	35.0 – 36.5	21.8					
SS-5-3	7.5 – 9.0	28.2				12.7	
SS-5-4	10.0 – 11.5	84.0					
SS-5-6	14.5 – 16.0	31.6				10.2	
SS-5-7	17.5 – 19.0	109.6					
SS-5-8	20.0 – 21.5	87.5					
SS-5-9	25.0 – 26.5	49.5					
SS-5-10	30.0 – 31.5	73.6					
SS-5-11	35.0 – 36.5	24.1					

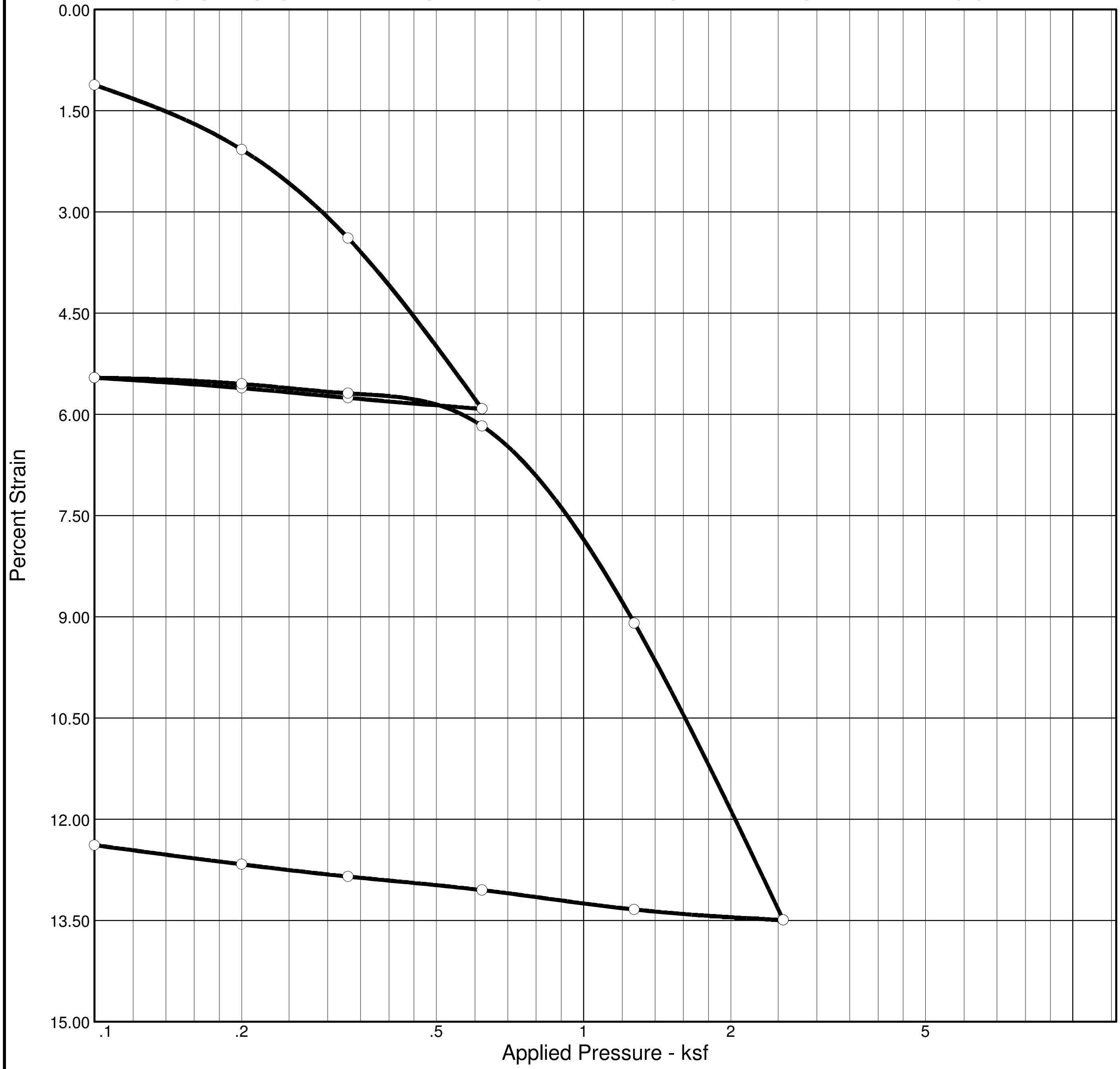
Table 2C. pH Test Results (ASTM G51)

Sample Number	Sample Depth (ft)	Sample Description	pH
SS-1-2	5.0 – 6.5	Very soft clayey silt, scattered organics	6.2
SS-1-5	12.5 – 14.0	Soft clayey silt	6.1
SS-4-2	5.0 – 6.5	Very soft clayey silt, scattered organics	6.3
SS-4-7	17.5 – 19.0	Soft clayey silt, some organics	6.5

Table 3C. Summary of Resistivity Testing (ASTM G57)

Location	Pin Spacing (ft.)	Resistivity (Ω-cm)
Near BH-1	4	38,300
	6	24,129
	8	18,384
Near BH-4	6	4,366
	8	3,370

CONSOLIDATION TEST REPORT - ASTM D2435



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_r	Swell Press. (ksf)	Swell %	e_o
Sat.	Moist.											
90.2 %	61.1 %	59.1			2.65		0.59	0.41	0.02			1.797

MATERIAL DESCRIPTION	USCS	AASHTO
Clayey SILT; grey, wet, medium to high plasticity	MH	

Project No. 2156001-610 **Client:** Foundation Engineering, Inc.; Project #2151044

Project: Hawk Street Extension

Source: 5746 **Sample No.:** SH-2-2 **Elev./Depth:** 4.5 - 5.5'

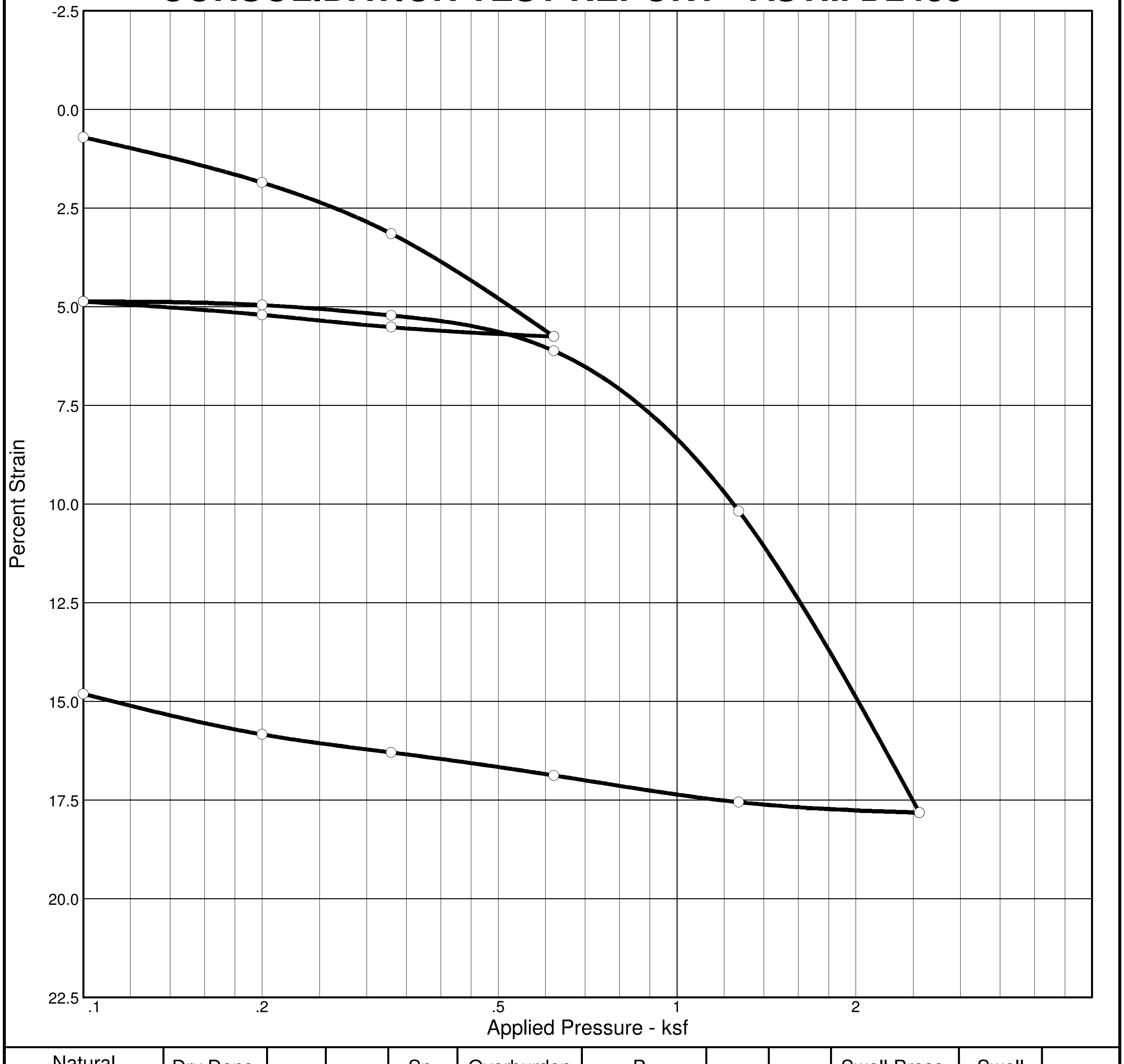
Remarks:

CONSOLIDATION TEST REPORT - ASTM D2435

FEI Testing & Inspection, Inc.
Corvallis, OR

Figure 1C

CONSOLIDATION TEST REPORT - ASTM D2435



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_r	Swell Press. (ksf)	Swell %	e_o
Sat.	Moist.											
88.4 %	87.0 %	45.8			2.65		1.22	0.93	0.06			2.609

MATERIAL DESCRIPTION	USCS	AASHTO
Clayey SILT; grey, wet, medium to high plasticity	MH	

Project No. 2156001-610 **Client:** Foundation Engineering, Inc.; Project #2151044

Project: Hawk Street Extension

Source: 5746 **Sample No.:** SH-4-3 **Elev./Depth:** 7.5 - 9.5'

Remarks:

CONSOLIDATION TEST REPORT - ASTM D2435

FEI Testing & Inspection, Inc.
Corvallis, OR

Figure 2C